
Geotechnical Engineering: *Earth Retaining Structures*

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Credit: 6 PDH

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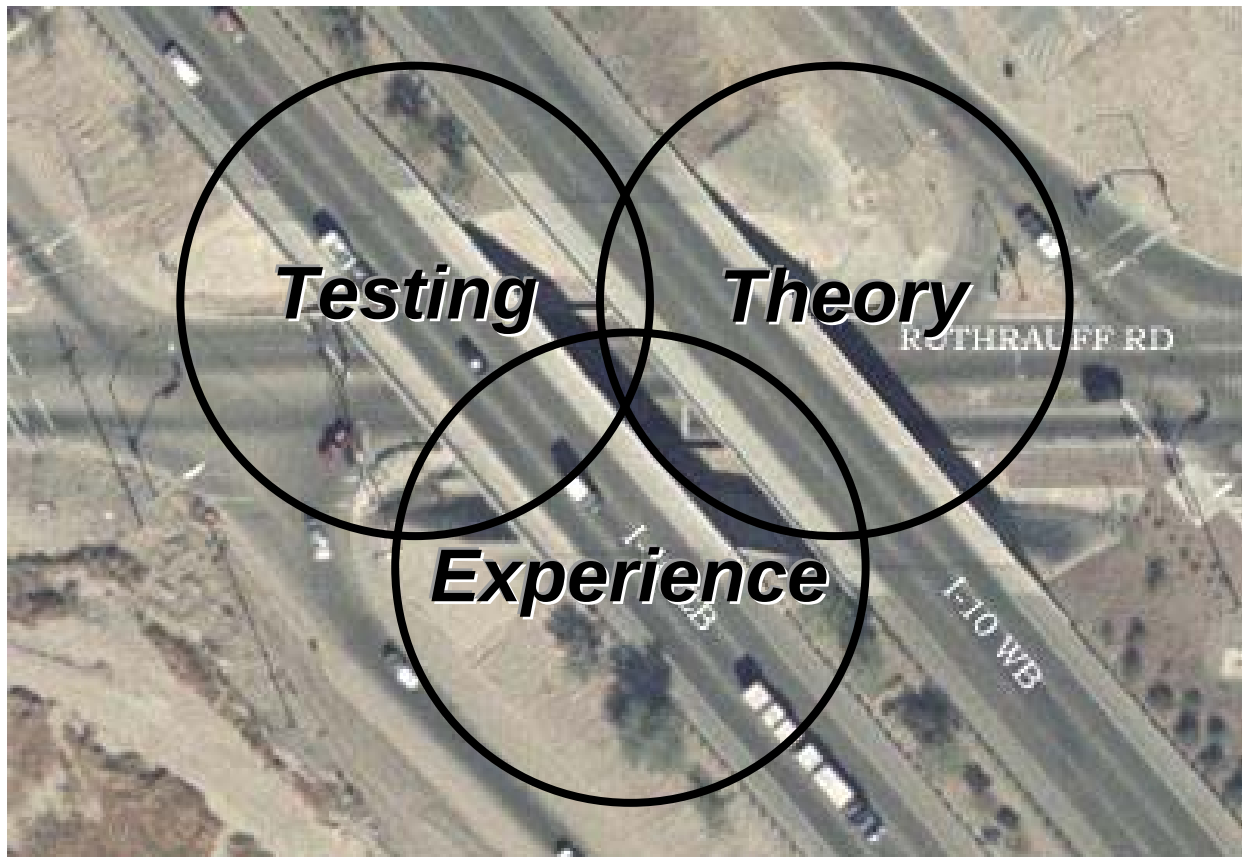
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SOILS AND FOUNDATIONS

Reference Manual – Volume II



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16. Abstract The Reference Manual for Soils and Foundations course is intended for design and construction professionals involved with the selection, design and construction of geotechnical features for surface transportation facilities. The manual is geared towards practitioners who routinely deal with soils and foundations issues but who may have little theoretical background in soil mechanics or foundation engineering. The manual's content follows a project-oriented approach where the geotechnical aspects of a project are traced from preparation of the boring request through design computation of settlement, allowable footing pressure, etc., to the construction of approach embankments and foundations. Appendix A includes an example bridge project where such an approach is demonstrated. Recommendations are presented on how to layout borings efficiently, how to minimize approach embankment settlement, how to design the most cost-effective pier and abutment foundations, and how to transmit design information properly through plans, specifications, and/or contact with the project engineer so that the project can be constructed efficiently. The objective of this manual is to present recommended methods for the safe, cost-effective design and construction of geotechnical features. Coordination between geotechnical specialists and project team members at all phases of a project is stressed. Readers are encouraged to develop an appreciation of geotechnical activities in all project phases that influence or are influenced by their work.			
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CHAPTER 10.0

EARTH RETAINING STRUCTURES

Earth retaining structures or systems are used to hold back earth and maintain a difference in the elevation of the ground surface as shown in Figure 10-1. The retaining wall is designed to withstand the forces exerted by the retained ground or “backfill” and other externally applied loads, and to transmit these forces safely to a foundation and/or to a portion of the restraining elements, if any, located beyond the failure surface.

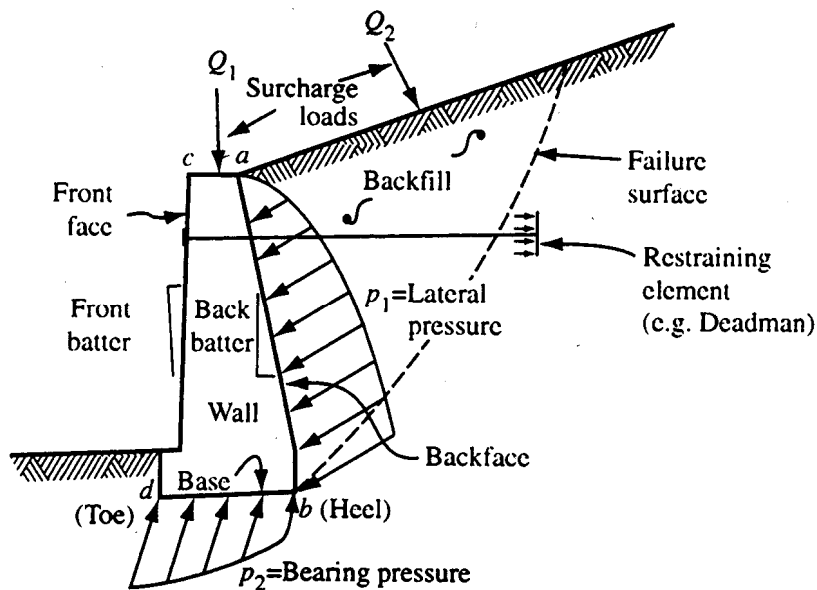


Figure 10-1. Schematic of a retaining wall and common terminology.

In general, the cost of constructing a retaining wall is usually high compared with the cost of forming a new slope. Therefore, the need for a retaining wall should be assessed carefully during preliminary design and an effort should be made to keep the retained height as low as possible.

In highway construction, retaining walls are used along cuts or fills where space is inadequate for construction of cut slopes or embankment slopes. Bridge abutments and foundation walls, which must support earth fills, are also designed as retaining walls.

Typical applications for earth retaining structures in highway construction include:

- new or widened highways in developed areas;
- new or widened highways at mountain or steep slopes;
- grade separation;
- bridge abutments, wing walls and approach embankments;

- culvert walls;
- tunnel portals and approaches;
- flood walls, bulkheads and waterfront structures;
- cofferdams for construction of bridge foundations;
- stabilization of new or existing slopes and protection against rockfalls; and
- groundwater cut-off barriers for excavations or depressed roadways.

Figure 10-2 provides schematic illustrations of several retaining wall systems traditionally used in highway applications. A great number of wall systems have been developed in the past two decades by specialty contractors who have been promoting either a special product or a specialized method of construction, or both. Due to the rapid development of these diversified systems and their many benefits, the design engineer is now faced with the difficult task of having to select the best possible system; design the structure; and ensure its proper construction.

An important breakthrough in the design of earth retaining structures (ERS) that occurred in this era was the recognition that the earth pressure acting on a wall is a function of the type of wall and the amount and distribution of wall movement. Classical earth pressure theories, which were developed by Coulomb (1776) and Rankine (1857), were formalized for use by Caquot and Kerisel (1948) and others. Sophisticated analyses of soil-structure interaction and wall/soil movements began in the 1960s with the development of finite difference and finite element analytical procedures. The simultaneous advancement of geotechnical instrumentation equipment and monitoring procedures made the “observational method” of design (Peck, 1969) popular and cost effective.

Since 1970 there has been a dramatic growth in the number of methods and products for retaining soil. O’Rourke and Jones (1990) describe two trends in particular that have emerged since 1970. First, there has been an increasing use of reinforcing elements, either by incremental burial to create reinforced soils (MSE walls), or by systematic in situ installation to reinforce natural soils or even existing fills (soil nailing); see Figure 10-2b. Mechanically stabilized earth and soil nailing have changed the ways we construct fill or cut walls, respectively, by providing economically attractive alternatives to traditional designs and construction methods. Second, there has been an increasing use of polymeric products to reinforce the soil and control drainage. Rapid developments in polymer manufacturing have supplied a wide array of geosynthetic materials. The use of these products in construction has encouraged a multitude of different earth retention schemes.

The rapid development of these new trends and the increased awareness of the impact of construction on the environment, have led to the emergence of the concept of “earth walls.”

In this concept, the soil supports itself or is incorporated into the structure and assumes a major structural or load carrying function. With this concept, structural member requirements of the system are reduced, or eliminated altogether. Examples of recently developed earth walls include the soil-reinforcement systems discussed above, as well as systems involving chemical treatment of the in-situ soil such as jet grouting or deep soil mixing.

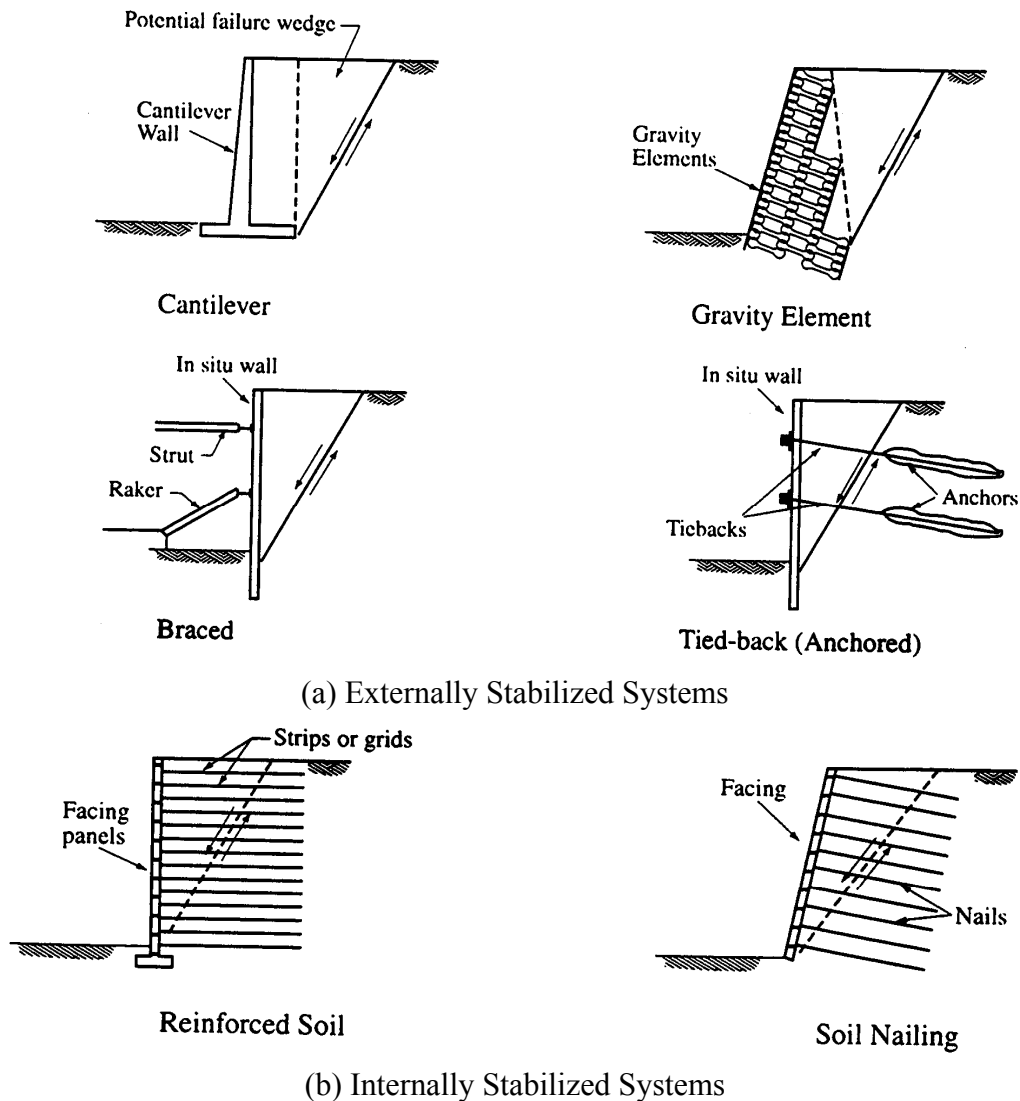


Figure 10-2. Variety of retaining walls (after O'Rourke and Jones, 1990)

10.01 Primary References:

The two primary references for earth retaining structures are:

FHWA (2005b). *Earth Retaining Structures - DRAFT*. Report No. FHWA-SA-05-046, Authors: Tanyu, B.F., Sabatini, P.J. and Berg, R.R., Federal Highway Administration, U.S. Department of Transportation.

AASHTO (2004 with 2006 Interims). *AASHTO LRFD Bridge Design Specifications*, 3rd Edition, American Association of State Highway and Transportation Officials, Washington, D.C.

10.1 CLASSIFICATION OF EARTH RETAINING STRUCTURES

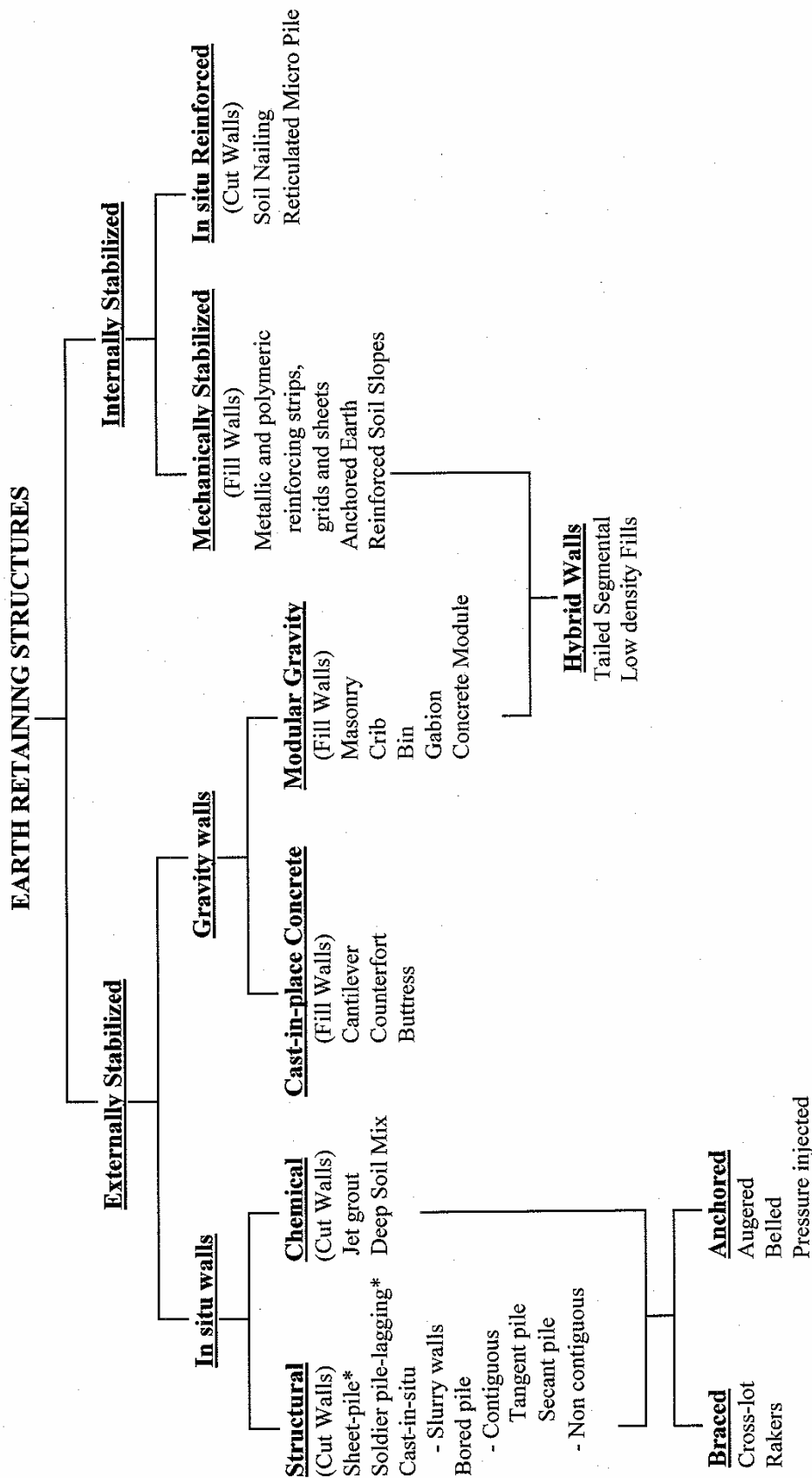
Earth retaining systems may be classified according to:

- load support mechanism, i.e., externally or internally stabilized walls;
- construction method, i.e., fill or cut walls; and
- system rigidity, i.e., rigid or flexible walls.

Every retaining wall can now be classified by using these three factors. For example, a sheet-pile wall would be classified as an **externally-stabilized cut** wall that is relatively **flexible**. A mechanically stabilized earth (MSE) wall is an **internally stabilized fill** wall that is relatively **flexible**. Further description of these classifications is provided subsequently.

10.1.1 Classification by Load Support Mechanism

The stability component of walls can be organized according to two principal categories: externally and internally stabilized systems (O'Rourke and Jones, 1990) as shown in Figure 10-3. An externally stabilized system uses an external structural wall against which stabilizing forces are mobilized. An internally stabilized system involves reinforcements installed within the retained soil mass and extending beyond the potential failure plane. Hybrid systems combine elements of both internally and externally supported walls.



* can also be used in fill conditions

Figure 10-3. Classification of earth retaining systems (after O'Rourke and Jones, 1990).

Virtually all traditional types of walls may be regarded as externally stabilized systems (Refer to Figure 10-2a). Gravity walls, in the form of cantilever structures or gravity elements (e.g., bins, cribs and gabions), support the soil and, through their weight and stiffness, resist sliding, overturning, and shear. Bracing systems, such as cross-lot struts and rakers, provide temporary support for in situ structural and chemically stabilized walls. Ground anchors provide support through their pullout capacity in stable soils outside of the zone of potential failure.

It is in the area of internally stabilized systems that relatively new concepts have been introduced (Refer to Figure 10-2b). Shear transfer to mobilize the tensile capacity of closely spaced reinforcing elements embedded in the retained soil mass has enabled retaining structures to be constructed without an external structural wall element. The shear transfer mechanism allows a composite system of reinforcing elements and soil to serve as the primary structural entity. A facing is required on an internally stabilized system, however, its purpose is to prevent raveling and deterioration rather than to provide primary structural support.

10.1.2 Classification by Construction Method

Earth retaining structures (ERS) can also be classified according to the method required for their construction, i.e., fill construction or cut construction. Fill wall construction refers to a wall system in which the wall is constructed from the base of the wall up to the top, i.e., “bottom-up” construction. Cut wall construction refers to a wall system in which the wall is constructed from the top of the wall down to the base concurrent with excavation operations, i.e., “top-down” construction. The classification of each wall system according to its construction method is also presented in Figure 10-3.

It is important to recognize that the “cut” and “fill” designations refer to how the wall is constructed, not necessarily the nature of the earthwork associated with the project. For example, a prefabricated modular gravity wall, which may be used to retain earth for a major highway cut, is considered a fill wall because its construction is not complete until the backfill has been placed from the “bottom-up” after the excavation for the cut has reached its final grade.

10.1.3 Classification by System Rigidity

The rigidity or flexibility of a wall system is fundamental to the understanding of the development of earth pressures, discussed in Section 10.2. In simple terms, a wall is considered to be rigid if it moves as a unit in rigid body rotation and/or translation and does not experience bending deformations. Most gravity walls can be considered rigid walls. Flexible walls are those that undergo bending deformations in addition to rigid body motion. Such deformations result in a redistribution of lateral pressures from the more flexible to the stiffer portions of the system. Virtually all wall systems, except gravity walls, may be considered to be flexible.

10.1.4 Temporary and Permanent Wall Applications

Permanent wall systems are generally considered to have a service life of 75 to 100 years. However, the ERS listed in Figure 10-3 are technically feasible for both temporary and permanent applications. In most cases, however, certain systems may not be cost-effective for temporary applications. Temporary walls generally have less restrictive requirements on material durability, design factors of safety, performance, and overall appearance than do permanent walls. Also, walls that can be constructed rapidly are often used for temporary applications. For example, MSE walls with segmental, precast facings are not typically used for temporary applications since the cost of the facing components and the select backfill may be more than 50 percent of the total cost of the wall.

The service life of temporary earth support systems is based on the time required to support the ground while the permanent systems are installed. This document has adopted the AASHTO guidance which considers temporary systems to be those that are removed upon completion of the permanent systems. The time period for temporary systems is commonly stated to be 18 to 36 months, but may be shorter or longer based on actual project conditions.

Temporary systems may be divided into “support of excavation” (SOE) temporary systems and “critical” temporary systems. In general the owner will determine which temporary systems are to be designated as critical. That decision is often based on the owner’s need to restrict lateral movement of the support system to minimize ground movements behind the support system. In general, specific components or design features for temporary systems may be designed to the same or similar criteria as used for permanent systems. Conversely, SOE systems are commonly designed to less restrictive criteria than permanent systems. The owner commonly assigns the responsibility for design and performance of SOE systems to the contractor. The design of SOE systems is often based more on system stability than on minimizing ground movements.

10.1.5 Wall Selection Considerations

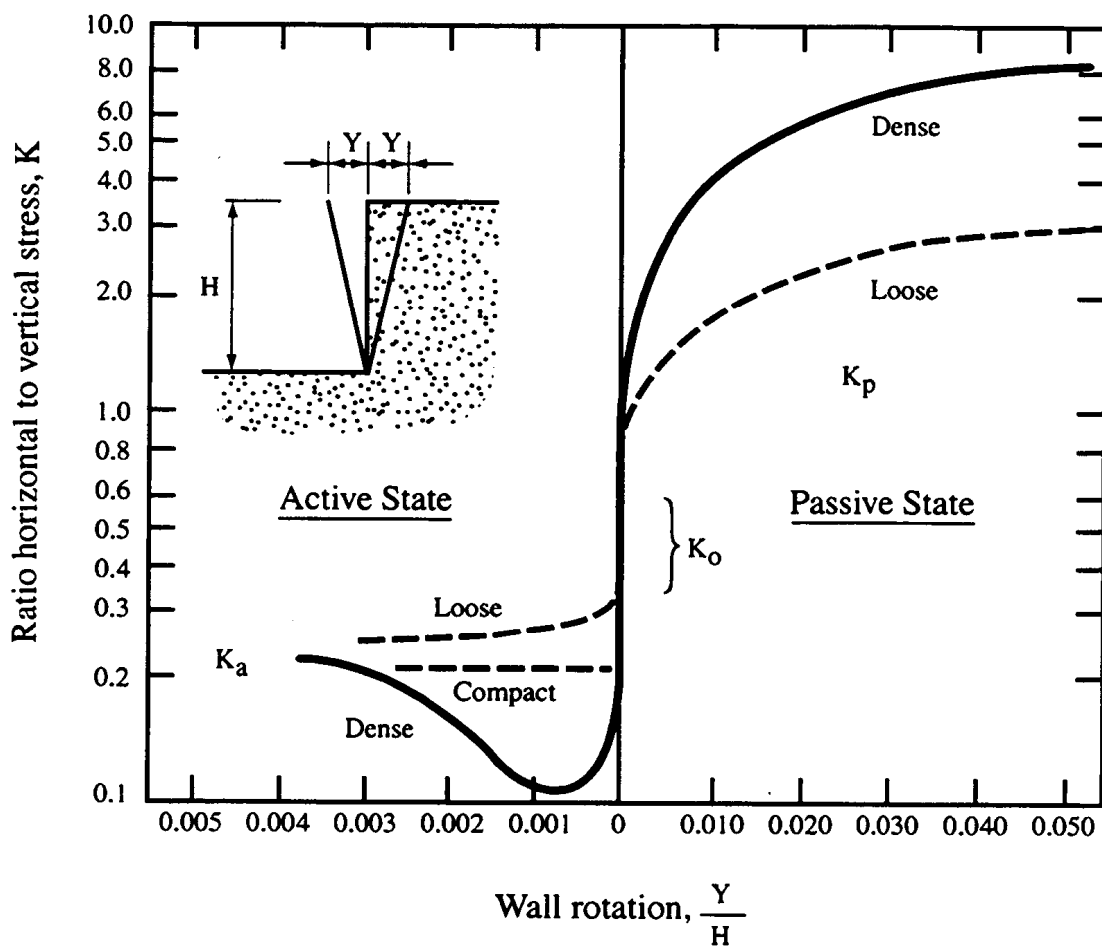
Given the wide variety of retaining walls as shown in Figure 10-3, it is important to select a wall that is most economical for the application being considered. The wall selection process should include consideration of various factors such as (1) ground type, (2) groundwater, (3) construction considerations, (4) speed of construction, (5) right of way, (6) aesthetics, (7) environmental concerns, (8) durability and maintenance, (9) tradition and (10) local contracting practices. A detailed discussion of these wall selection factors is outside the scope of this manual. The reader is referred to FHWA (2005b) where a systematic wall selection process considering these factors is described.

10.2 LATERAL EARTH PRESSURES

Some of the basic concepts of lateral earth and water pressures were discussed in Chapter 2. It is recommended that the reader should review Section 2.9 before proceeding further in this Chapter. Here the principles of lateral earth pressure are explained on the basis of deformation. A total lateral pressure diagram consistent with the assumed deformations is developed for use in assessing the forces acting on the wall from the backfill or retained ground. This section focuses primarily on theoretical earth pressure diagrams, which are most commonly used in the design of rigid gravity structures, nongravity cantilevered walls, MSE walls, and anchored walls with stiff structural facings such as diaphragm walls.

A wall system is designed to resist lateral earth pressures and water pressures that develop behind the wall. Earth pressures develop primarily as a result of loads induced by the weight of the backfill and/or retained in-situ soil, earthquake ground motions, and various surcharge loads. For purposes of earth retaining system design, three different types of lateral earth pressure are usually considered: (1) at-rest earth pressure; (2) active earth pressure; and (3) passive earth pressure. These conditions are shown in Figure 10-4 relative to lateral deformation of the walls. The conditions are defined as follows:

- At-rest earth pressure is defined as the lateral earth pressure that exists in level ground for a condition of no lateral deformation.
- Active earth pressure is developed as the wall moves away from the backfill or the retained soil. This movement results in a decrease in lateral pressure relative to the at-rest condition. A relatively small amount of lateral movement is necessary to reach the active condition.



Magnitude of Wall Rotation to Reach Failure

Soil type and condition	Rotation, Y/H	
	Active	Passive
Dense cohesionless	0.001	0.02
Loose cohesionless	0.004	0.06
Stiff cohesive	0.010	0.02
Soft cohesive	0.020	0.04

Figure 10-4. Effect of wall movement on wall pressures (after Canadian Geotechnical Society, 1992).

- Passive earth pressure is developed as the wall moves towards the backfill or the retained soil. This movement results in an increase in lateral pressure relative to the at-rest condition. The movements required to reach the passive condition are approximately ten times greater than those required to develop active earth pressure.

Each of these earth pressure conditions can be expressed in general form by:

$$p_h = Kp_o \quad 10-1$$

where p_h is the lateral earth pressure at a given depth behind the wall, p_o , is the vertical stress at the same depth, and K is the earth pressure coefficient that has a value related to the at-rest condition (K_o), active conditions of movement, (K_a), or passive conditions of movement, (K_p).

As shown in Figure 10-4, the magnitudes of these earth pressure coefficients follow the relationship of $K_p > K_o > K_a$. The relationship between the magnitude of retaining wall movement, in this case rotation, Y/H , into or away from the retained material about its toe, and the horizontal pressure exerted by the soil is presented in Figure 10-4, with angular movement along the x axis and the mobilized coefficient of lateral earth pressure on the y axis. Figure 10-4 can also be used to estimate the state of stress for walls with uniform horizontal translation equal to Y . As illustrated in this figure, significantly larger lateral displacements are required to mobilize the passive resistance than those required to develop active pressures. The maximum values of K_a and K_p correspond to fully mobilized pressures that represent active and passive failure conditions, respectively.

When the estimated wall movement is less than the value required to fully mobilize active or passive pressure, the earth pressure coefficient can be adjusted proportionally based on the graphical relationship presented in Figure 10-4.

10.2.1 At-Rest Lateral Earth Pressure

The at-rest earth pressure represents the lateral effective stress that exists in a natural soil in its undisturbed state. For cut walls constructed in near normally consolidated soils, the at-rest earth pressure coefficient, K_o , can be approximated by the equation (Jaky, 1944):

$$K_o = 1 - \sin \phi' \quad 10-2$$

where ϕ' is the effective (drained) friction angle of the soil. The magnitude of the at-rest earth pressure coefficient is primarily a function of soil shear strength and degree of

overconsolidation, which, as indicated in Chapter 7, may result from natural geologic processes for retained natural ground or from compaction effects for backfill soils.

In overconsolidated soils, K_o can be estimated as (Schmidt, 1966):

$$K_o = (1 - \sin \phi')(\text{OCR})^\Omega \quad 10-3$$

where Ω is a dimensionless coefficient, which, for most soils, can be taken as $\sin \phi'$ (Mayne and Kulhawy, 1982) and OCR is the overconsolidation ratio.

Usually, Equations 10-2 and 10-3 for the at-rest earth pressure coefficient are sufficiently accurate for normally to lightly overconsolidated soils provided the overconsolidation ratio has been evaluated from laboratory consolidation testing. For moderately to heavily overconsolidated clays, or where a more accurate assessment is required, laboratory triaxial tests on undisturbed samples and in-situ testing such as pressuremeter testing may be used.

For normally consolidated clay, K_o is typically in the range of 0.55 to 0.65; for sands, the typical range is 0.4 to 0.5. For lightly overconsolidated clays ($\text{OCR} \leq 4$), K_o may reach a value up to 1; for heavily overconsolidated clays ($\text{OCR} > 4$), K_o values may be greater than 2 (Brooker and Ireland, 1965). For heavily overconsolidated soils, values for K_o can be very large. A relatively stiff wall would be required to resist the large forces resulting from the lateral earth pressures in this case. For walls constructed in such soils, consideration should be given to performing pressuremeter tests, which provide a direct measure of lateral pressures in the ground.

In the context of wall designs consisting of steel soldier beams or sheet-pile wall elements, design earth pressures based on at-rest conditions are not typically used since at-rest earth pressures imply that the wall system undergoes no lateral deformation. This condition may be appropriate for heavily preloaded, stiff wall systems, but designing to a requirement of zero wall movement for flexible wall systems is not practical.

10.2.2 Active and Passive Lateral Earth Pressures

As discussed in Chapter 2, in stability analyses active and passive earth pressures are developed as a result of soil displacement within a failure zones developed behind the wall (active) or in front of the wall (passive) assuming that the wall displaces outward. For the purpose of illustration Figure 10-5 shows the two conditions with respect to wall movement relative to the backfill only. In one case the wall moves away from the backfill (active case) in the other case the wall moves into the backfill (passive case) As shown in the figure, the failure zone for both cases is typically bounded by the back face of the wall and a failure surface through the retained soil mass along which the soil has attained limiting equilibrium. In addition to the effect of lateral movements on the values of K_a and K_p shown in Figure 10-4, the magnitude of the active and passive earth pressure coefficients are functions of the soil shear strength, the backfill geometry, i.e., horizontal backfill surface or sloping ground surface above the wall, the orientation of the surface where the wall contacts the backfill or retained soil, i.e., vertical or inclined, and the friction and cohesive forces that develop on this surface as the wall moves relative to the retained ground.

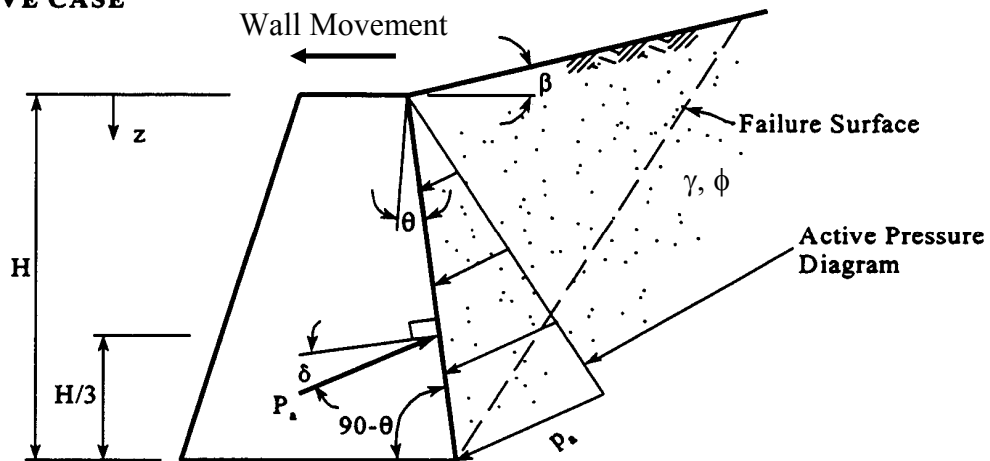
Active and passive earth pressure coefficients based on a plane wedge theory, which considers the effect of wall friction, sloping backfill and sloping wall face, was first proposed by Coulomb (1776) and are shown in Figure 10-5. The pressures calculated by using these coefficients are commonly known as the Coulomb earth pressures. Since Coulomb's method is based on limit equilibrium of a wedge of soil, only the magnitude and direction of the earth pressure is found. Pressure distributions and the location of the resultant are assumed to be triangular.

For simple cases involving vertical walls retaining homogeneous soil with a level ground surface, without friction between the soil and the wall face, and without the presence of groundwater, the formulas for computing the earth pressure coefficients can be simplified considerably by substituting, $\delta = \theta = \beta = 0$ in Coulomb's equations, as shown in Figure 10-5. For such simplified cases, K_a and K_p can be expressed by Equations 10-4 and 10-5, respectively:

$$K_a = \frac{1 - \sin \phi'}{1 + \sin \phi'} = \tan^2 (45 - \phi'/2) \quad 10-4$$

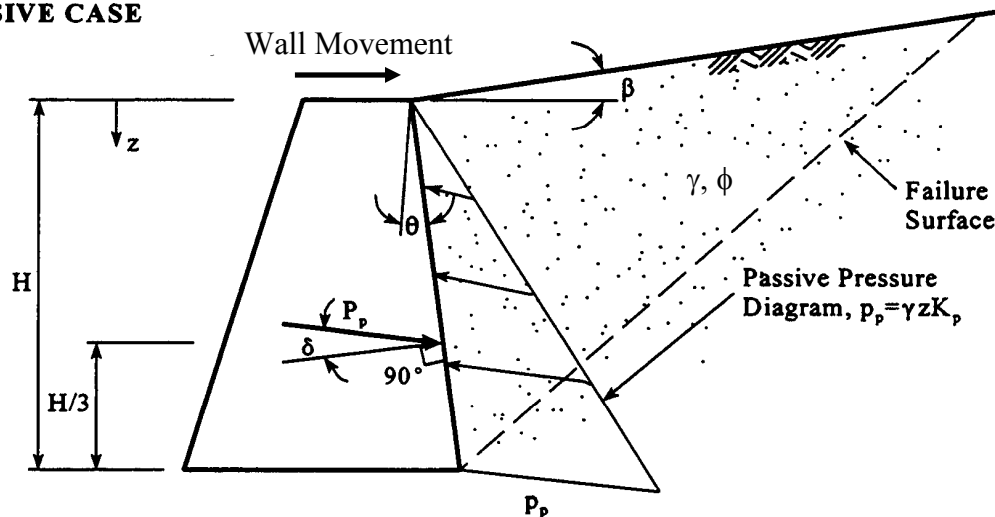
$$K_p = \frac{1 + \sin \phi'}{1 - \sin \phi'} = \tan^2 (45 + \phi'/2) \quad 10-5$$

ACTIVE CASE



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

PASSIVE CASE



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

Figure 10-5. Coulomb coefficients K_a and K_p for sloping wall with wall friction and sloping cohesionless backfill (after NAVFAC, 1986b).

These simplified equations were also derived independently by Rankine (1857). Hence, the earth pressures computed by using these equations are commonly known as the Rankine earth pressures.

For a cohesionless soil with a groundwater table, the effective lateral earth pressure acting on the wall at any depth, z , below the surface is a function of the pore water pressure u as follows,

$$p_a' = K_a (\gamma z - u) \quad 10-6$$

$$p_p' = K_p (\gamma z - u) \quad 10-7$$

10.2.3 Effect of Cohesion on Lateral Earth Pressures

For a cohesive soil defined by effective stress strength parameters ϕ' and c' , the active and passive earth pressure coefficients are:

$$K_a = \tan^2 (45 - \phi' / 2) - \frac{2c'}{p_o'} \tan^2 (45 - \phi' / 2) \quad 10-8$$

$$K_p = \tan^2 (45 + \phi' / 2) + \frac{2c'}{p_o'} \tan^2 (45 + \phi' / 2) \quad 10-9$$

Figure 10-6(a) presents active and passive pressure distributions for cohesionless soils ($c' = 0$) while Figure 10-6(b) shows similar pressure distributions for c' - ϕ' soils.

For a c' - ϕ' soil with a groundwater table, the effective lateral earth pressure acting on the wall at any depth, z , below the surface is,

$$p_a' = K_a (\gamma z - u) - 2c' \sqrt{K_a} \quad 10-10$$

$$p_p' = K_p (\gamma z - u) + 2c' \sqrt{K_p} \quad 10-11$$

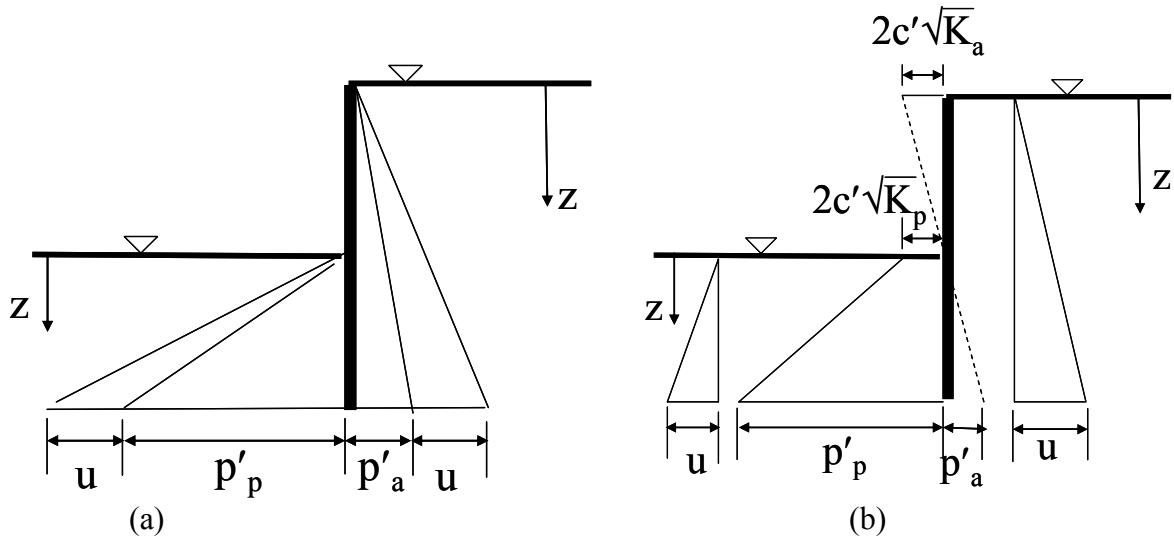


Figure 10-6. (a) Wall pressures for a cohesionless soil, and (b) Wall pressures for soil with a cohesion intercept – with groundwater in both cases (after Padfield and Mair, 1984)

Theoretically, in soils with cohesion, the active earth pressure behind the wall becomes negative from the ground surface to a critical depth z where γz is less than $2c'\sqrt{K_a}$. This critical depth is referred to as the “tension crack.” The active earth pressure acting against the wall within the depth of the tension crack is assumed to be zero. Unless positive drainage measures are provided, water infiltration into the tension crack may result in hydrostatic pressure on the retaining structure.

Use of values of c' for the retained soil, greater than say, 100 psf (5 kPa), results in a significant depth of theoretical negative active earth pressure. Therefore, it is important either to:

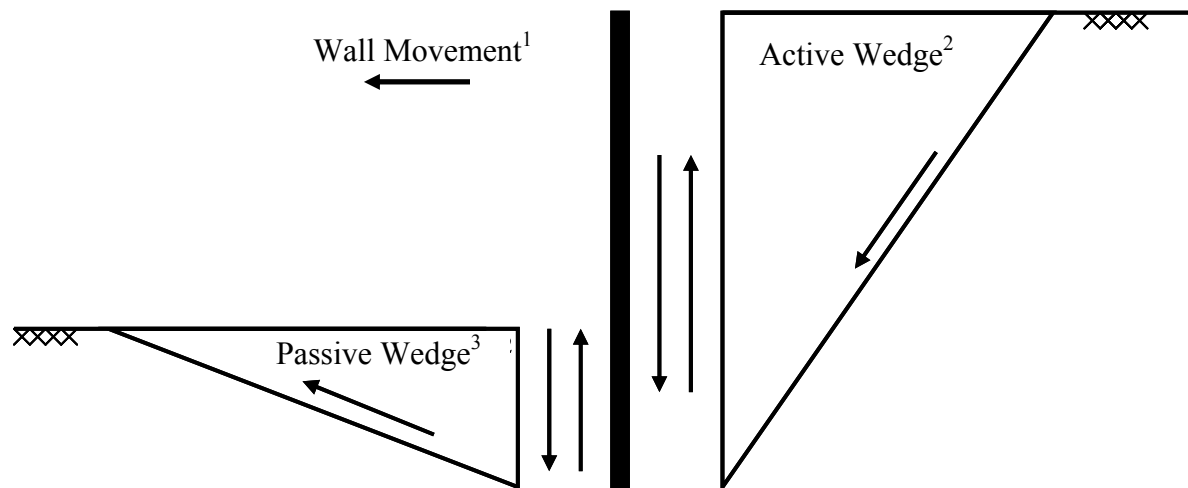
- reduce c' towards the surface, which may be realistic for many clays in view of weathering;
- or
- assume that the effective pressure on the wall at any depth should not be less than 30z psf where z = depth in ft ($5z$ kN/m² (z = depth in m)).

In all cases where water is present in the soil, full hydrostatic pressure is added to the lateral earth pressure computed by Equations 10-8 to 10-11 to obtain the total lateral pressure that will be experienced by a retaining wall.

10.2.4 Effect of Wall Friction and Wall Adhesion on Lateral Earth Pressures

In practice, walls are not smooth. As indicated previously, wall friction and wall adhesion modify the stress distribution near a wall. Therefore, wall friction, δ , and wall adhesion, c_w , should both be considered as proportions of ϕ' , and c' or s_u , respectively. For a rigid wall moving away from the retained soil, the frictional forces exerted by the wall on the soil are in the sense shown in Figure 10-7. The active wedge moves down with respect to the wall, while the passive wedge moves upwards.

An important exception to this mechanism is when the wall acts as a significant load-bearing element, when large vertical loads are applied to the top of the wall, or when an inclined ground anchor is stressed to an appreciable load and the vertical component of the load acts downward. In such cases, the wall has to move down relative to the soil on both sides of the wall in order to mobilize the required skin friction to support the load. Therefore, the friction acts to increase the pressures on both the active and passive sides, because it acts on the soil wedges in a downward direction. This effect, however, is neglected because limiting or failure conditions are considered in calculation of overall stability and the directions in which the frictional forces act should be taken as shown in Figure 10-7.



Note: (1) Assume wall moves as a rigid body to the left.
(2) Active wedge moves downward relative to wall
(3) Passive wedge moves upward relative to wall.

Figure 10-7. Wall friction on soil wedges (after Padfield and Mair, 1984)

Wall friction, δ , and wall adhesion, c_w , have an important effect on soil pressures. Equations 10-10 and 10-11 can be written to account for those effects in a more general as follows:

$$p_a' = K_a(\gamma z - u) - K_{ac}c' \quad 10-12$$

$$p_p' = K_p(\gamma z - u) + K_{pc}c' \quad 10-13$$

where K_a and K_p depend on δ and K_{ac} and K_{pc} depend on δ and c_w , and p_a' and p_p' are the components of effective pressure normal to the wall. Where c' is incorporated into the soil strength characterization, approximate values of K_{ac} and K_{pc} should be calculated from the following expressions:

$$K_{ac} = 2 \sqrt{K_a (1 + c_w / c')} \quad 10-12a$$

$$K_{pc} = 2 \sqrt{K_p (1 + c_w / c')} \quad 10-13a$$

Different values of δ are given by several sources. As shown in Table 10-1, values of δ depend on soil type and the wall material. The maximum wall friction suggested for design is:

$$\text{Active: } \delta = 2/3 \phi'$$

$$\text{Passive: } \delta = 1/2 \phi'$$

Where a cohesion intercept is used as part of the characterization of strength in terms of effective stress, a maximum wall adhesion of $c_w = 0.5c'$ could be used, but in view of the inevitable remolding of the clay close to the wall by any construction process, it is recommended that no wall adhesion be allowed in the design.

The values of wall friction provided above and in Table 10-1 are maximum values for design. These values can be adopted in most cases, but the design engineer should consider any circumstances where the values might be affected by the relative movement of the soil and the wall. For example, on the active side, reduced values should be used if there is a tendency for the wall to move downwards, e.g., for load-bearing walls or walls supported by prestressed ground anchors. For walls retaining soft cohesive soils or granular soils that will be subjected to significant vibration, e.g., walls near railway tracks or machine foundations, δ should be assumed to be zero in the design.

Table 10-1
Wall friction and adhesion for dissimilar materials (after NAVFAC, 1986b)

Interface Materials	Friction Factor, $\tan \delta$	Friction angle, δ degrees
Mass concrete on the following foundation materials:		
Clean sound rock	0.70	35
Clean gravel, gravel sand mixtures, coarse sand	0.55 to 0.60	29 to 31
Clean fine to medium sand, silty medium to coarse sand, silty or clayey gravel	0.45 to 0.55	24 to 29
Clean fine sand, silty or clayey fine to medium sand	0.35 to 0.45	19 to 24
Fine sandy silt, nonplastic silt	0.30 to 0.35	17 to 19
Very stiff and hard residual or preconsolidated clay	0.40 to 0.50	22 to 26
Medium stiff and stiff clay and silty clay (Masonry on foundation materials has same friction factor)	0.30 to 0.35	17 to 19
Steel sheet piles against the following soils:		
Clean gravel, gravel-sand mixtures, well-graded rock fill with spalls	0.40	22
Clean sand, silty sand-gravel mixtures, single size hard rock fill	0.30	17
Silty sand, gravel or sand mixed with silt or clay	0.25	14
Fine sandy silt, nonplastic silt	0.20	11
Formed concrete or concrete sheet piling against the following soils:		
Clean gravel, gravel-sand mixture, well-graded rock fill with spalls	0.40 to 0.50	22 to 26
Clean sand, silty sand-gravel mixture, single size hard rock fill	0.30 to 0.40	17 to 22
Silty sand, gravel or sand mixed with silt or clay	0.30	17
Fine sandy silt, nonplastic silt	0.25	14
Various structural materials:		
Masonry on masonry, igneous and metamorphic rocks:		
Dressed soft rock on dressed soft rock	0.70	35
Dressed hard rock on dressed soft rock	0.65	33
Dressed hard rock on dressed hard rock	0.55	29
Masonry on wood (cross grain)	0.50	26
Steel on steel at sheet pile interlocks	0.30	17
Interface Materials (Cohesion)	Adhesion c_u (kPa)	
Very soft cohesive soil (0 - 12 kPa)	0 - 12	
Soft cohesive soil (12 - 24 kPa)	12 - 24	
Medium stiff cohesive soil (24 - 48 kPa)	24 - 36	
Stiff cohesive soil (48 - 96 kPa)	36 - 45	
Very stiff cohesive soil (96 - 192 kPa)	45 - 62	

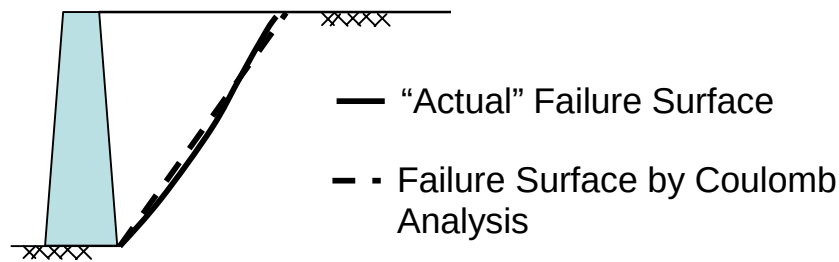
The effect of wall friction on the Rankine and Coulomb methods of earth pressure computation is as follows:

1. The Rankine method cannot take account of wall friction. Accordingly, K_a is overestimated slightly and K_p is under-estimated, thereby making the Rankine method conservative for most applications.
2. The Coulomb theory can take account of wall friction, but the results are unreliable for passive earth pressures for wall friction angle values greater than $\phi'/3$ because the failure surface is assumed to be a plane. The failure wedges assumed in the Coulomb analysis take the form of straight lines as shown in Figure 10-8. This may be contrasted with the curved shapes of failure surface observed in model tests. The curvature results from the disturbing influence of wall friction on the stress field near the wall. The error in the Coulomb solutions results in K_a being underestimated slightly and K_p being overestimated very significantly for large values of ϕ' .

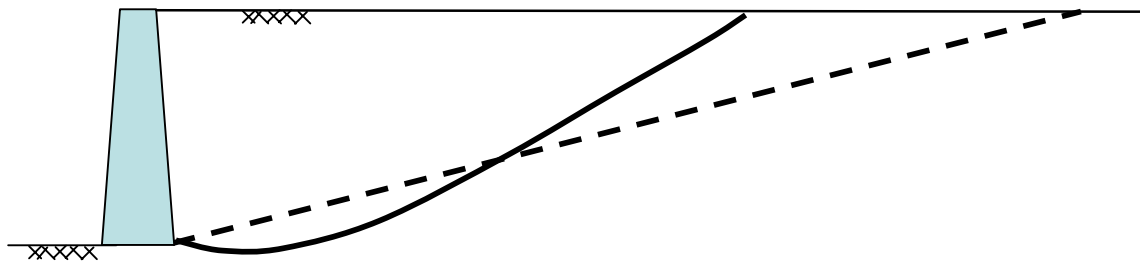
If the angle of wall friction δ is small, the failure surface is almost linear. For large values of δ , the failure surface is curved and can be approximated by a log-spiral. The deviation of the curved surface from a planar surface is minor for the active case but significant for the passive case as shown in Figure 10-8. For most applications, the effect of wall friction on active earth pressures is relatively small and is often neglected.

For the passive case, however, large values of δ cause downward tangential shear forces to act on the passive wedge of soil adjacent to the wall, increasing its resistance to upward movement. This increased resistance to upward movement causes a curved failure surface to occur in the soil, as shown in Figure 10-8b. The soil fails on this curved surface of least resistance and not on the Coulomb plane, which would require greater lateral driving force. Hence, passive pressures computed on the basis of the plane wedge theory are always greater than those calculated on the basis of a log-spiral failure surface and may be on the unsafe side since passive earth pressure forces are generally resisting forces in stability analyses.

Based on the above discussions, it is recommended that the log-spiral theory be used for the determination of the passive earth pressure coefficients. Charts for two common wall configurations, sloping wall with level backfill and vertical wall with sloping backfill based on the log-spiral theory are presented in Figures 10-9 and 10-10 (Caquot and Kerisel, 1948; NAVFAC, 1986b). For walls that have a sloping backface and sloping backfill, the passive earth pressure coefficient can be calculated as indicated in Figure 10-5 by using $\delta = \phi'/3$.



(a) Active Case ($\phi' = 30^\circ$, $\delta = 30^\circ$)



(b) Passive Case ($\phi' = 30^\circ$, $\delta = 30^\circ$)

Figure 10-8. Comparison of plane and log-spiral failure surfaces (a) Active case and (b) Passive case (after Sokolovski, 1954) – Note: Depiction of gravity wall is for illustration purpose only.

For the active case, the resultant load predicted by using coefficients based on the plane wedge theory is within 10 percent of that obtained with the more exact log-spiral theory. Hence, for the active case, Coulomb’s theory can be used to calculate the earth pressure coefficient (Refer to Figure 10-5).

For some wall types, such as cantilever retaining walls and an MSE walls, the “interface” where the earth pressures are computed is within the retained soils along a vertical plane passing through the heel of the base slab. In such cases, there is soil-to-soil contact and the resultant may be oriented at the angle of mobilized friction. The angle of mobilized friction depends on the factor of safety used for the angle of internal friction. For these cases, it is generally conservative to assume that the earth pressure is parallel to the slope of the backfill.

REDUCTION FACTOR (R) OF K_p FOR VARIOUS RATIOS OF $-\delta/\phi$									
ϕ	δ/ϕ	-0.7	-0.6	-0.5	-0.4	-0.3	-0.2	-0.1	0.0
10		.978	.962	.946	.929	.912	.898	.881	.864
15		.961	.934	.907	.881	.854	.830	.803	.775
20		.939	.901	.862	.824	.787	.752	.716	.678
25		.912	.860	.808	.759	.711	.666	.620	.574
30		.878	.811	.746	.686	.627	.574	.520	.467
35		.836	.752	.674	.603	.536	.475	.417	.362
40		.783	.682	.592	.512	.439	.375	.316	.262
45		.718	.600	.500	.414	.339	.276	.221	.174

Note: Curves shown are for $\delta/\phi = -1$

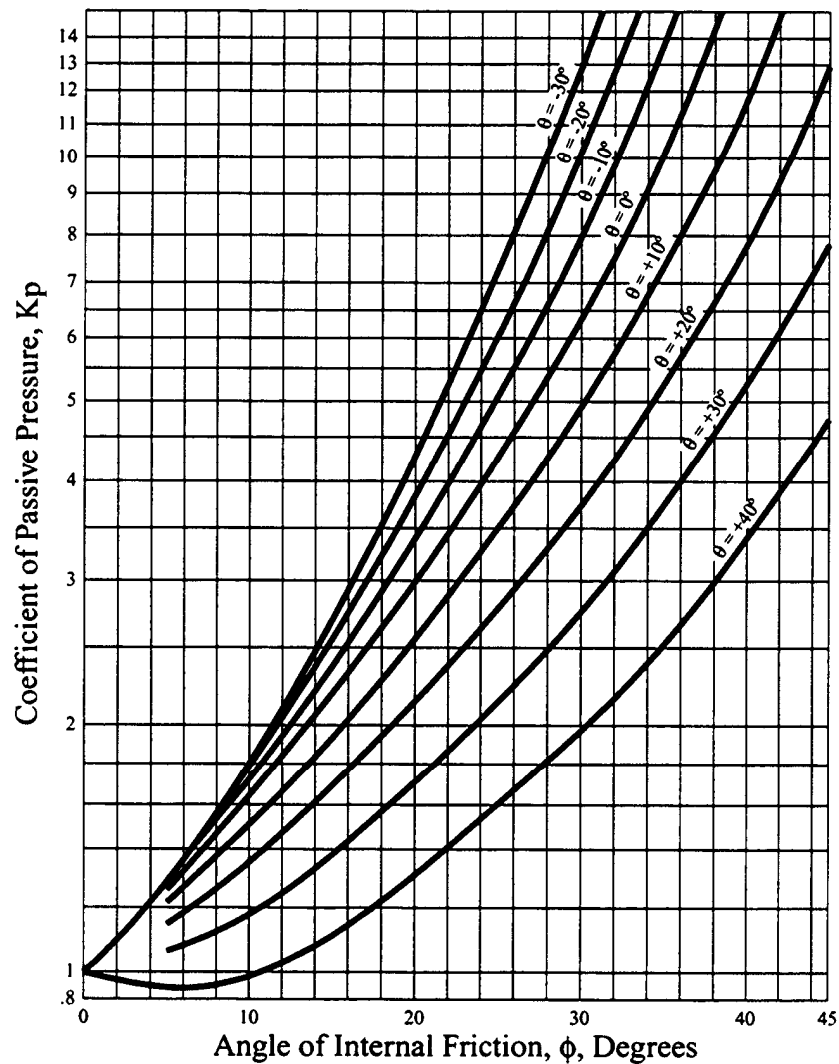
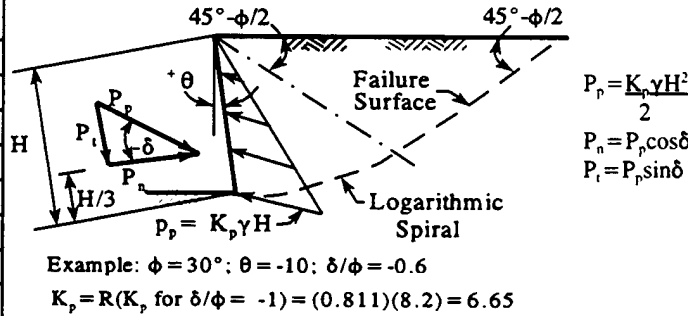
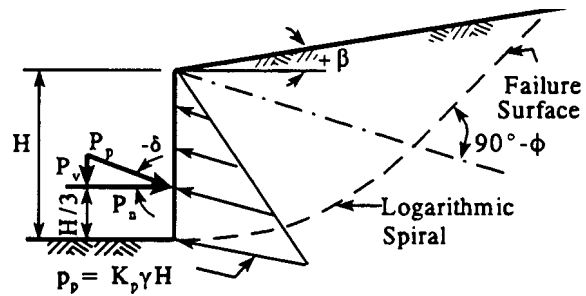


Figure 10-9. Passive coefficients for sloping wall with wall friction and horizontal backfill (Caquot and Kerisel, 1948; NAVFAC, 1986b).

REDUCTION FACTOR (R) OF K_p FOR VARIOUS RATIOS OF $-\delta/\phi$									
$\phi \backslash \delta/\phi$	-0.7	-0.6	-0.5	-0.4	-0.3	-0.2	-0.1	0.0	
10	.978	.962	.946	.929	.912	.898	.881	.864	
15	.961	.934	.907	.881	.854	.830	.803	.775	
20	.939	.901	.862	.824	.787	.752	.716	.678	
25	.912	.860	.808	.759	.711	.666	.620	.574	
30	.878	.811	.746	.686	.627	.574	.520	.467	
35	.836	.752	.674	.603	.536	.475	.417	.362	
40	.783	.682	.592	.512	.439	.375	.316	.262	
45	.718	.600	.500	.414	.339	.276	.221	.174	

Note: Curves shown are for $\delta/\phi = -1$



$$P_p = K_p \gamma H^2/2; P_a = P_p \cos \delta; P_v = P_p \sin \delta$$

Example: $\phi = 25^\circ; \beta/\phi = -0.2; \delta/\phi = -0.3$

$$K_p = R(K_p \text{ for } \delta/\phi = -1) = (0.711)(3.62) = 2.58$$

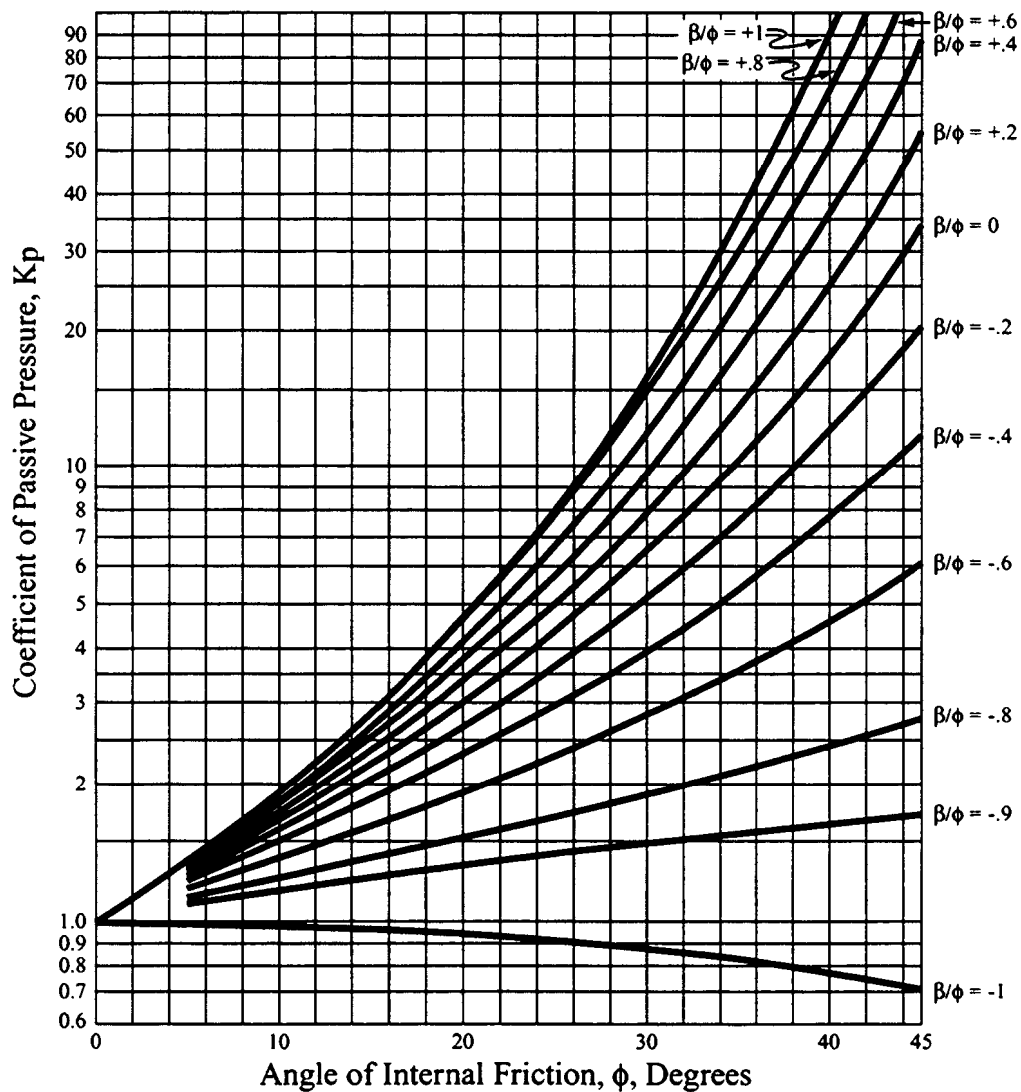
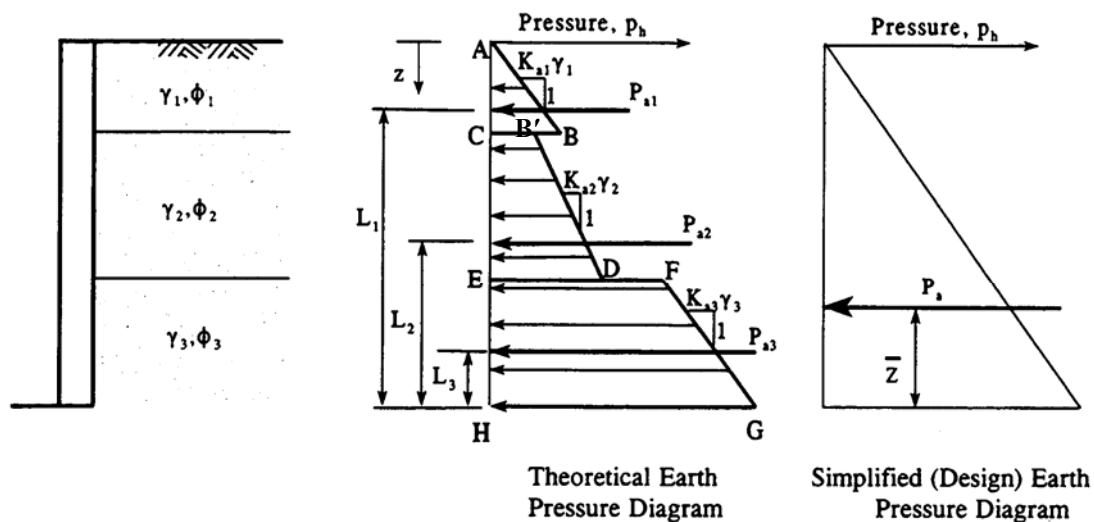


Figure 10-10: Passive coefficients for vertical wall with wall friction and sloping backfill (Caquot and Kerisel, 1948; NAVFAC, 1986b).

10.2.5 Theoretical Lateral Earth Pressures in Stratified Soils

For stratified or non-homogeneous soils, the theoretical earth pressures are assumed to be distributed as shown in Figure 10-11 where the discontinuities in the earth pressure diagram occur at the boundary between soil strata having different unit weights and shear strength parameters. Unless the computed earth pressures vary widely with depth, the total applied lateral force determined from the computed pressure diagram may be redistributed to a corresponding simplified equivalent triangular pressure diagram as indicated in Figure 10-11.

For complex cases such as layered soils, irregular backfill, irregular surcharges, wall friction, and sloping groundwater level, pressures can be determined by graphical solutions. Among the many graphical solutions are Culmann's method (1866) and the Trial Wedge method. These procedures can be found in Bowles (1996) or NAVFAC (1986b). The Trial Wedge method has the advantage of including cohesion as a soil parameter in the analysis.



The lateral force is equal to the area of the pressure diagram. Thus,

$$P_{a1} = \text{Area ABC} \quad P_{a2} = \text{Area CB'DE} \quad P_{a3} = \text{Area EFGH}$$

Resultant (total) active force per unit length, $P_a = P_{a1} + P_{a2} + P_{a3}$

$$\text{Location of resultant from base of wall, } \bar{z} = \frac{P_{a1}L_1 + P_{a2}L_2 + P_{a3}L_3}{P_a}$$

- Use buoyant unit weight for soils below water table.
- Add water pressure as appropriate to obtain total lateral pressure.
- The simplified distribution may not be justified for all soil conditions. Use judgment to determine validity of such simplified distributions.

Figure 10-11. Pressure distribution for stratified soils.

10.2.6 Semi Empirical Lateral Earth Pressure Diagrams

The earth pressure distributions discussed in the previous sections are strictly applicable to rigid wall systems, i.e., walls that translate and/or rotate as a unit and do not experience bending deformations. Most gravity walls can be considered rigid walls.

If a wall system undergoes bending deformations in addition to rigid body motion then such a wall system is considered flexible. Virtually all wall systems, except gravity walls, may be considered to be flexible. The bending deformations result in a redistribution of the lateral pressures from the more flexible to the stiffer portions of the system. Thus, in these walls the final distribution and magnitude of the lateral earth pressure may be considerably different from those used for rigid walls. For example, soldier-pile and lagging walls with multiple levels of support are usually designed by using empirical earth pressure distributions based on observed data. The shape of these empirical earth pressure distributions may vary from rectangular to trapezoidal. The magnitude of the pressures may also vary depending on the soil type.

Other factors that may influence the development of earth pressures are the type of construction, e.g., “bottom-up” or “top-down,” the wall support mechanism, e.g., tie-backs, struts, rakers, soil nails, reinforcing elements, single or multiple levels of support, etc., the geometry of the retained soil, e.g., silo pressure, the superimposed or surcharge loads, e.g., strip, line, concentrated, or equipment loads, and the type of analysis, e.g., static or seismic. In addition, for cases of soil reinforced by inclusions such as MSE walls or soil-nailed walls, different types of earth pressure distributions are used to evaluate the internal and external stability of the wall system. The empirical earth pressure distributions are generally related to the basic earth pressure coefficients K_a , K_p and K_o , which, as indicated previously, are a function of the shear strength of the soil.

10.2.7 Lateral Earth Pressures in Cohesive Backfills

Most DOTs involved in the design and procurement of fill wall systems, such as MSE walls, have well-defined backfill material requirements. In general, specifications for wall backfill require high-quality, granular, relatively free-draining backfills. However, in some cases a poorer quality on-site backfill material may be used, especially for temporary systems. These poorer quality backfills are generally more fine-grained and not free-draining. Methods to calculate earth pressures in clayey soils were described previously. In this section cautions are provided regarding the use of fine-grained cohesive backfill soils.

Lateral pressures can be caused by the volume expansion of ice in fine-grained soils such as fine sand, silt and clay. Lateral pressures due to volume expansion of the retained soil may achieve relatively high values that are difficult to predict. Since structures are usually not designed to withstand frost-generated stresses, provisions should be made so that frost-related stresses will not develop behind the structure or be kept to a minimum. The use of one or more of the following measures may be necessary:

- Isolate the backfill from underground sources of water either by providing a permeable drainage system or an impervious barrier;
- Use pervious backfill and provide weep holes in the structure;
- Provide an impervious soil layer near the ground surface, and grade the ground behind the wall to drain surface water away from the wall.

Expansive clays can cause very high lateral pressures on the back of a retaining structure and should therefore be avoided whenever possible. In cases where expansive clays are present behind a wall, swelling pressures should be evaluated based on laboratory tests so that the wall can be designed properly to withstand these swelling pressures, which can be significant. Alternatively, one of the following measures can be taken:

- A granular filter material can be provided between the clay backfill and the back of the wall. This material will drain the groundwater away from the expansive soil and, at the same time act as a buffer zone between the expansive soil and the structure.
- The expansive soil can be treated with lime to reduce or even eliminate its swelling potential, if the soil does not contain gypsum. Expansive soils that contain gypsum should not be treated with lime because the combination of the minerals in expansive soils with gypsum and water may lead to the formation of ettringite, which has a much higher swelling potential than the untreated expansive soils.

The following is noted by Duncan, *et al.* (1990) concerning the use of clayey soils as backfill for fill wall applications:

- Clayey backfills generally have lower drained shear strength than cohesionless soils. Low drained shear strength results in: (1) larger lateral earth pressures against the back of the wall; (2) lower frictional resistance along the reinforcement for MSE

walls that employ frictional reinforcement; and (3) lower bearing value for MSE walls that employ passive reinforcement.

- Clayey backfills are more plastic and contain more fines than cohesionless soils. Higher plasticity results in: (1) poor drainage and the potential for the development of water pressures behind the wall; (2) the potential for freezing of retained water and development of ice pressures on the back of the wall; and (3) greater potential for corrosion of metallic reinforcements for MSE walls.
- Clayey backfills have the potential to undergo creep deformations that can lead to higher earth pressures and greater wall face deformations than will occur with soils that do not exhibit significant creep potential. Earth pressures used for design of gravity walls employing clayey backfills should be based on past performance and field experience, as wall design methods do not consider the effects of creep.

Despite these problems, silts and clays may be used as backfill soils provided suitable design procedures are employed, including conservative estimates of lateral earth pressures, and construction control measures are incorporated into the contract documents. When silts and clays are used as backfills, walls may need to be designed for pressures between active and at-rest conditions. For soils that are deemed to have high swell potential, an earth-pressure coefficient as great as 1.0 may be used for design (Canadian Geotechnical Society, 1992). In all cases, water pressures and appropriate surcharge loads also need to be added to these earth pressures.

In general, any permanent fill wall system that incorporates silty or clayey backfills must have an appropriately designed subsurface and surface drainage system to minimize pore pressure build-up and soil saturation. Such wall systems should also include periodic measurements of wall face movements.

10.3 LATERAL PRESSURES DUE TO WATER

In retaining wall design, it is general practice to provide drainage paths, commonly known as “weep holes,” through the earth retaining structure, or use other methods to drain groundwater that may otherwise collect behind the structure. The purpose of these drainage features is to prevent the development of water pressure on the structure. Occasionally, however, it may not be feasible or desirable to drain the water from behind the structure. For example, maintenance of existing ground water levels may be desirable to safeguard against potential settlement of adjacent structures or to prevent contaminated groundwater from entering the excavation. In such instances, the earth retaining structure must be designed for both lateral earth pressure and water pressure.

Computation of active lateral earth pressures for the case of a uniform backfill and static groundwater is illustrated in Figure 10-12. In this case, the water pressure represents a hydrostatic condition since there is no seepage or flow of water through the soil. The lateral earth pressure below the water level is based on the effective vertical stress, p'_o , times the active lateral earth pressure coefficient. The lateral pressure due to the water is added to the active lateral earth pressure to obtain the total lateral pressure on the wall. By analogy to lateral earth pressure coefficients, the lateral water pressure coefficient = 1.0. The lateral pressure computations should consider the greatest unbalanced water head anticipated to act on the wall, since this generally results in the largest total lateral load.

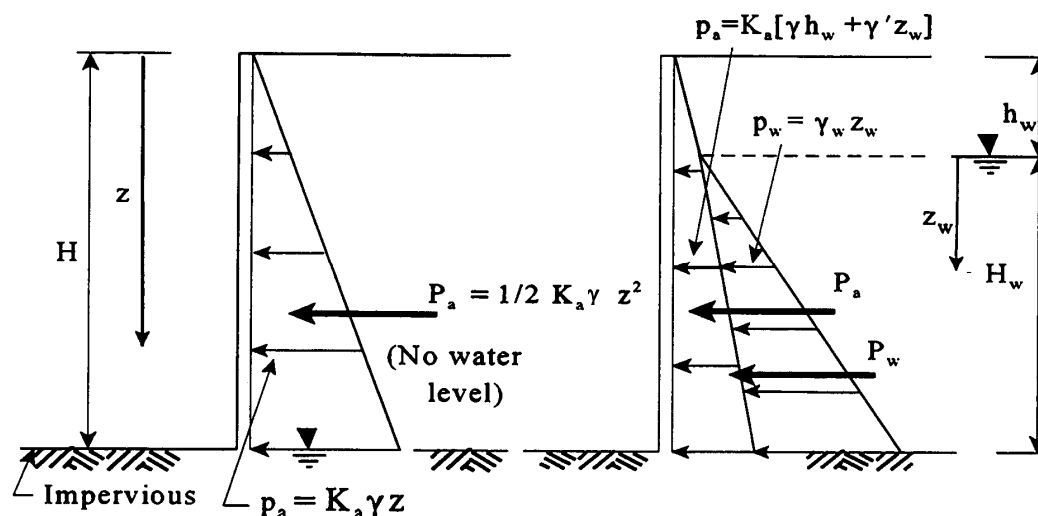
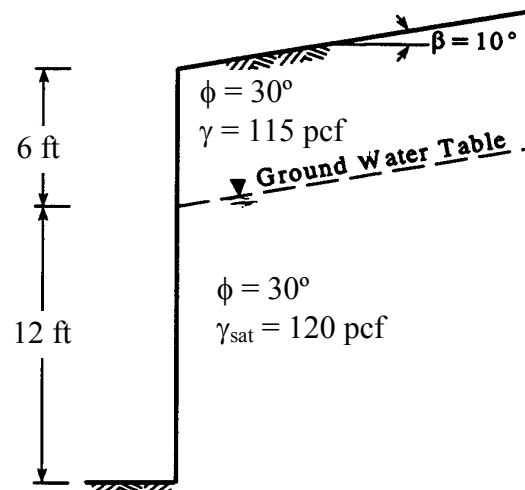


Figure 10-12. Computation of lateral pressures for static groundwater case.

For cases where seepage may occur through or beneath the earth retaining structure, the resulting seepage gradients will result in an increase or reduction in the water pressure depending on the direction of the seepage path. For such cases, flow net procedures can be used to compute the lateral pressure distribution due to water.

The concepts of lateral earth pressures and lateral pressures due to water are illustrated in Example 10-1.

Example 10-1: For the wall configuration shown below, construct the lateral pressure diagram. Assume the face of the wall to be smooth ($\delta = 0$, $c_w = 0$).



Solution:

Use the Coulomb method (Figure 10-5) for $\phi = 30^\circ$, $\beta = 10^\circ$, $\theta = 0$, and $\delta = 0$:

$$K_a = 0.374$$

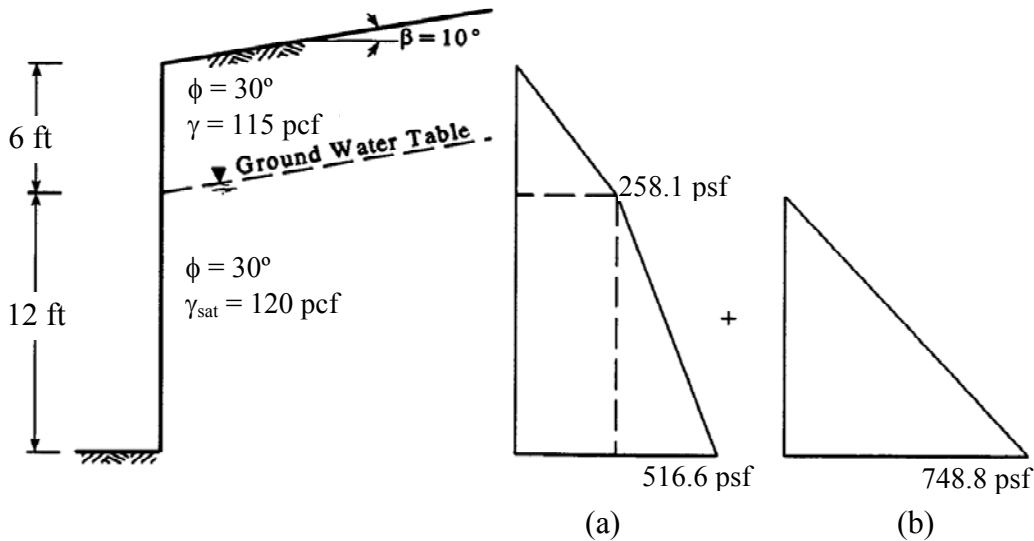
The pressures at various depths can then be calculated as shown in a tabular format as follows. Based on the values in the table, the lateral pressure diagrams due to earth and water can be constructed as shown below. The total lateral pressure diagram is the sum of the two lateral pressure diagrams shown in the figure accompanying this example.

Effective Lateral Earth Pressures, p'_a

z, ft	p_o , psf	$p_a = K_a p_o$, psf
0	0	0
6	(115 pcf) (6 ft) = 690.0 psf	$0.374(690.0 \text{ psf}) = 258.1 \text{ psf}$
18	$690 \text{ psf} + (120 \text{ pcf} - 62.4 \text{ pcf})(12 \text{ ft}) = 1381.2 \text{ psf}$	$0.374(1381.2 \text{ psf}) = 516.6 \text{ psf}$

Hydrostatic Pressure, $u = K_w u_w$

z , ft	z_w , ft	$u_w = z_w \gamma_w$, psf	Lateral water pressures, u psf
0	0	0	= 0
6	0	0	= 0
18	12	12 ft (62.4 pcf) = 748.8 psf	1.0(748.8 psf) = 748.8 psf



(a) Lateral effective earth pressure diagram and (b) Lateral water pressure diagram.

10.4 LATERAL PRESSURE FROM SURCHARGE LOADS

10.4.1 General

Surcharge loads on the backfill surface near an earth retaining structure also cause lateral pressures on the structure. Typical surcharge loadings may result from railroads, highways, sign/light structures, electric/telecommunications towers, buildings, construction equipment, and material stockpiles.

The loading cases of particular interest in the determination of lateral pressures are:

- uniform surcharge;
- point loads;
- line loads parallel to the wall; and
- strip loads parallel to the wall.

Figure 10-13 shows examples of retaining walls with surcharge loads.



(a)



(b)

Figure 10-13: (a) Retaining wall with uniform surcharge load and (b) Retaining wall with line loads (railway tracks) and point loads (catenary support structure).

10.4.2 Uniform Surcharge Loads

Surcharge loads are vertical loads applied at the ground surface, which are assumed to result in a uniform increase in lateral pressure over the entire height of the wall. The increase in lateral pressure for a uniform surcharge loading can be written as:

$$\Delta p_h = Kq_s \quad 10-14$$

where: Δp_h is the increase in lateral earth pressure due to the vertical surcharge load, q_s , applied at the ground surface, and K is an appropriate earth pressure coefficient. Examples of surcharge loads for highway wall system applications include: (1) dead load surcharges such as that resulting from the weight of a bridge approach slab or concrete pavement; (2) live load surcharges such as that due to traffic loadings; and (3) surcharges due to equipment or material storage during construction of the wall system.

When traffic is expected to come to within a distance from the wall face equivalent to one-half the wall height, the wall should be designed for a live load surcharge. For temporary walls that are not considered critical, actual surcharge loads may be evaluated and considered in the design instead of this prescriptive value. Both temporary and permanent wall designs should account for unusual surcharges such as large material stockpiles. Calculated lateral pressures resulting from these surcharges should be added explicitly to the design lateral earth pressure diagram. Surcharge loads from existing buildings need to be considered if they are within a horizontal distance from the wall equal to the wall height.

10.4.3 Point, Line, and Strip Loads

Point loads, line loads, and strip loads are vertical surface loadings that are applied over limited areas as compared to surcharge loads. As a result, the increase in lateral earth pressure used for wall system design is not constant with depth as is the case for uniform surcharge loadings. These loadings are typically calculated by using equations based on elasticity theory for lateral stress distribution with depth (Figure 10-14). Examples of such loads include heavy cranes (temporary) or walls (permanent). Lateral pressures resulting from these surcharges should be added explicitly to other lateral pressures.

A numerical problem solved by use of Figure 10-14 is presented in Example 10-2.

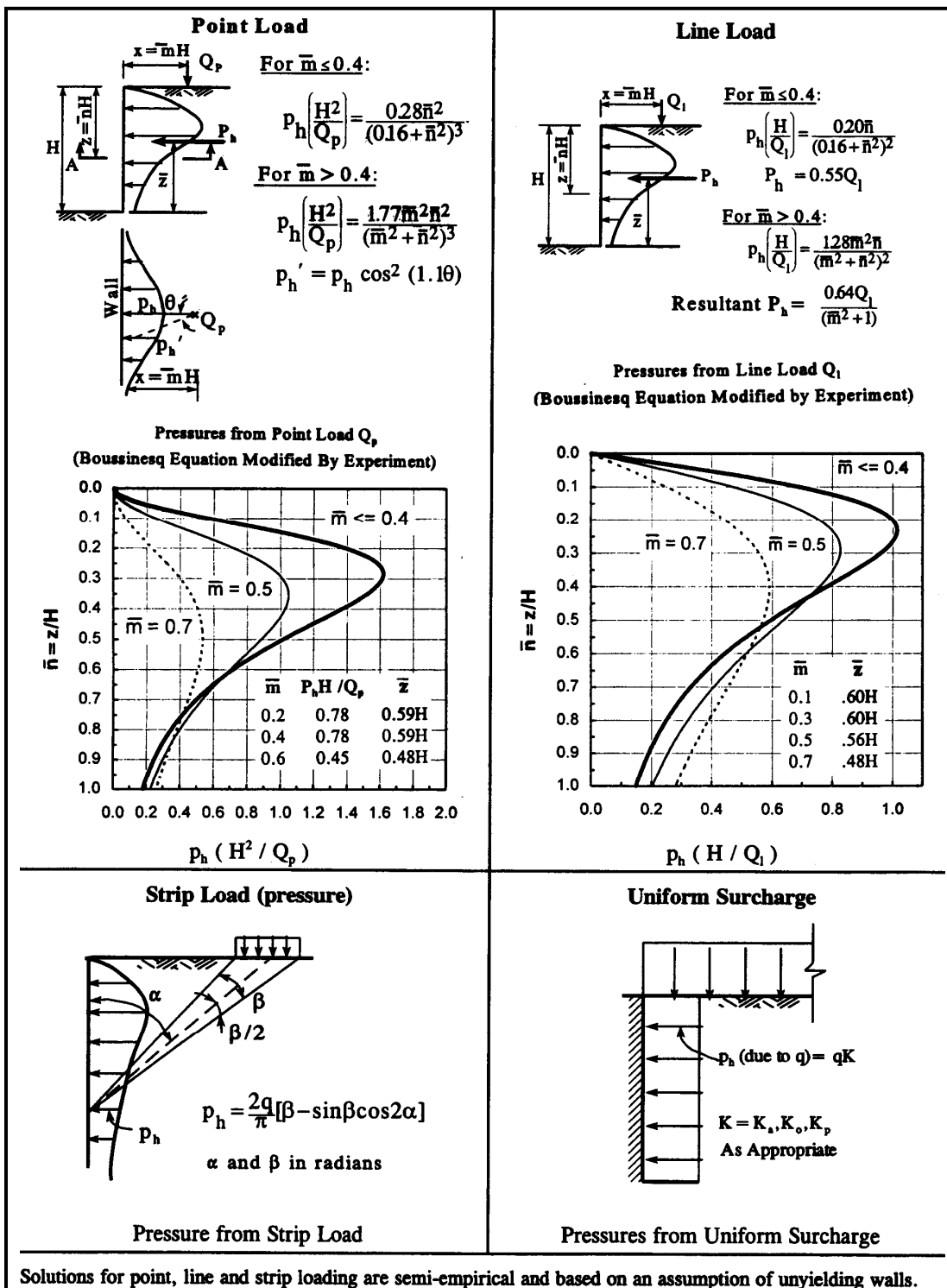
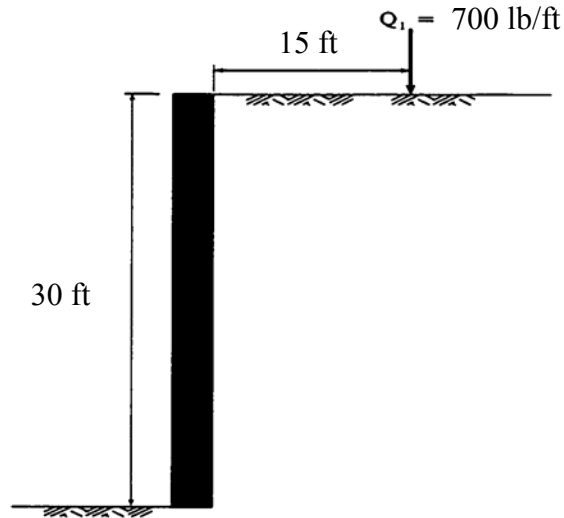


Figure 10-14. Lateral pressure due to surcharge loadings (after USS Steel, 1975)

Example 10-2: Construct the lateral pressure diagram due to a line load of 700 lb/ft located 15 ft behind the top of a 30 ft high unyielding wall shown below.



Geometry of the Example Problem 10-2

Solution:

The procedure to calculate the lateral pressures due to a line load is given in Figure 10-14. From this figure the lateral pressure can be found as follows:

$$\bar{m} = \frac{15 \text{ ft}}{30 \text{ ft}} = 0.5 > 0.4$$

For $\bar{m} > 0.4$, the lateral pressure is given by:

$$P_h = 1.28 \left(\frac{Q_1}{H} \right) \left[\frac{\bar{m}^2 \bar{n}}{(\bar{m}^2 + \bar{n}^2)^2} \right]$$

For $\bar{m} = 0.5$, $Q_1 = 700 \text{ lb/ft}$ and $H = 30 \text{ ft}$, the lateral pressure is given by:

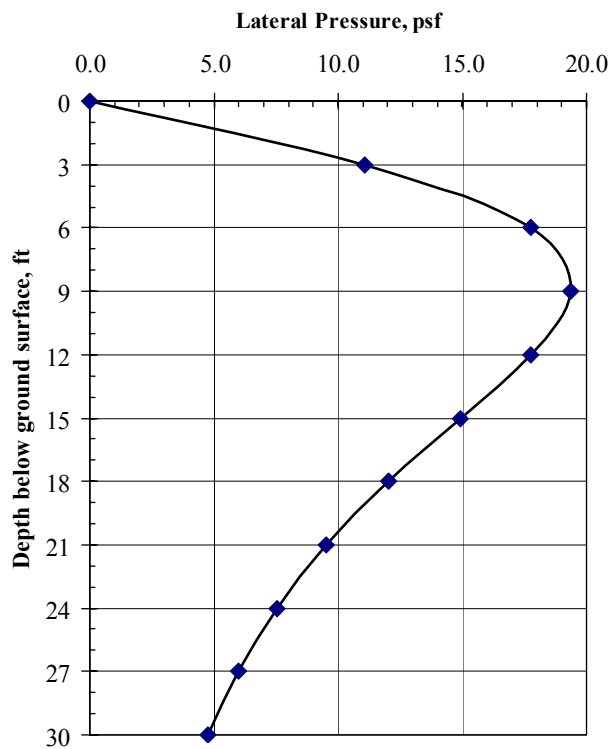
$$P_h = 1.28 \left(\frac{700 \text{ lb/ft}}{30 \text{ ft}} \right) \left[\frac{0.5^2 \bar{n}}{(0.5^2 + \bar{n}^2)^2} \right] \rightarrow P_h = 29.9 \left[\frac{0.25 \bar{n}}{(0.25 + \bar{n}^2)^2} \right]$$

Lateral pressures computed at various depths by using the above formula and the chart for line loads in Figure 10-14 are tabulated below.

Computation of Lateral Earth Pressures Due To Line Load

$\bar{n} = z / H$	Depth below top of wall (ft)	P_h (psf)
0	0	0.00
0.1	3	11.0
0.2	6	17.8
0.3	9	19.4
0.4	12	17.8
0.5	15	14.9
0.6	18	12.0
0.7	21	9.5
0.8	24	7.5
0.9	27	6.0
1.0	30	4.8

The information in the table is used to construct the curve of depth vs. lateral pressure shown below.



10.5 WALL DESIGN

There are many different types of walls as shown in Figure 10-3. All walls have to be evaluated for stability with respect to different modes of deformation. There are four basic modes of instability from a geotechnical viewpoint. These are (a) sliding, (b) limiting eccentricity or overturning, (c) bearing capacity, and (d) global stability. The four modes of instability are shown in Figure 10-15. Since these modes of instability assume that the wall is intact, the evaluation of these modes is commonly referred to as the “external stability” analysis. All four modes may or may not be applicable to all wall types. Furthermore, depending on the wall type and its load support mechanism (refer to Section 10.1), there may be additional instability modes, such as pullout, tension breakage, bending and shear. The evaluation of these additional modes of instability are commonly referred to as “internal stability” analyses.

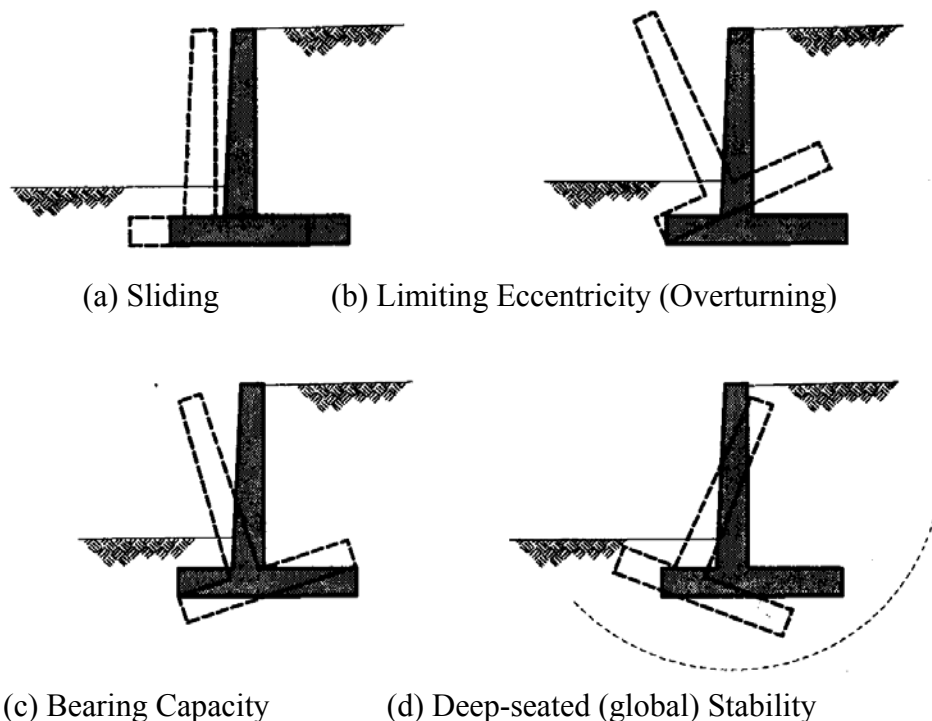


Figure 10-15. Potential failure mechanisms for rigid gravity and semi-gravity walls.

The external stability analysis is best illustrated by using the concept of gravity and semi-gravity walls. Table 10-2 summarizes the major design steps for cast-in-place concrete (CIP) gravity and semi-gravity walls.

Table 10-2
Design steps for gravity and semi-gravity walls

Step 1.	Establish project requirements including all geometry, external loading conditions such as (temporary, permanent, and seismic, performance criteria, and construction constraints.
Step 2.	Evaluate site subsurface conditions and relevant properties of in situ soil and rock and wall backfill.
Step 3.	Evaluate soil and rock parameters for design and establish factors of safety.
Step 4.	Select initial base dimension of wall for evaluation of external stability.
Step 5.	Select lateral earth pressure distribution. Add appropriate water, surcharge, and seismic pressures and develop total lateral pressure diagram for design.
Step 6.	Evaluate bearing capacity.
Step 7.	Evaluate limiting eccentricity (overturning) and sliding.
Step 8.	Check overall stability and revise wall design if necessary.
Step 9.	Estimate maximum lateral wall movement, tilt, and wall settlement. Revise design if necessary.
Step 10.	Design wall drainage systems.

10.5.1 Steps 1, 2, and 3 – Establish Project Requirements, Subsurface Conditions, Design Parameters

It is assumed that Steps 1, 2 and 3 are completed and a CIP wall has been deemed appropriate. Soil and/or rock parameters for design have been established. In general, the required parameters for in situ soil and rock are the same as those required for a spread footing, in particular, foundation shear strength for bearing resistance and compression parameters of the foundation materials to allow for computations of wall settlement. For gravity walls that require deep foundation support, the soil/rock parameters are the same as those required for the design of a driven pile or drilled shaft foundation.

The drainage and shear strength characteristics of the wall backfill soil are assessed as part of Step 3. Guidelines for wall backfill material gradation and drainage behind gravity retaining walls can be found in the AASHTO (2002). Whenever possible, the backfill material should be free draining, nonexpansive, and noncorrosive. All backfill material should be free of organic material. The backfill gradation should follow the guidelines presented in Table 10-3.

Table 10-3
Suggested gradation for backfill for cantilever
semi-gravity and gravity retaining walls

Sieve Size	Percent Passing
3 in. (76.2 mm)	100
No. 4 (4.75 mm)	35 – 100
No. 30 (0.6 mm)	20 – 100
No. 200 (0.075 mm)	0 – 15

10.5.2 Step 4 – Select Base Dimension Based on Wall Height

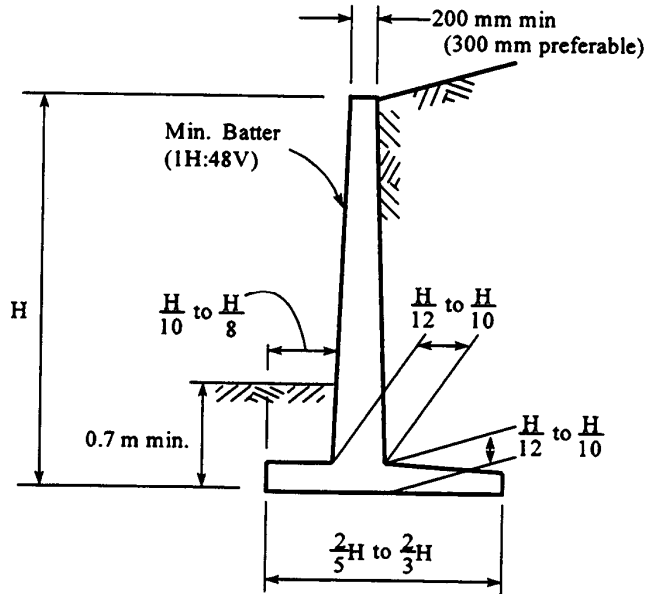
Figure 10-16 shows typical dimensions for a semi-gravity cantilever retaining wall and for a counterfort wall. These dimensions were developed based on a range of backfill properties, geometries, and stable foundation soils and can be used for preliminary design. However, the final external stability calculations should be performed based on the geometry requirements and specific conditions of the project, e.g., limited right-of-way. Similar guidelines exist for other wall types and can be found in FHWA (2005b).

10.5.3 Step 5 – Select Lateral Earth Pressure Distribution

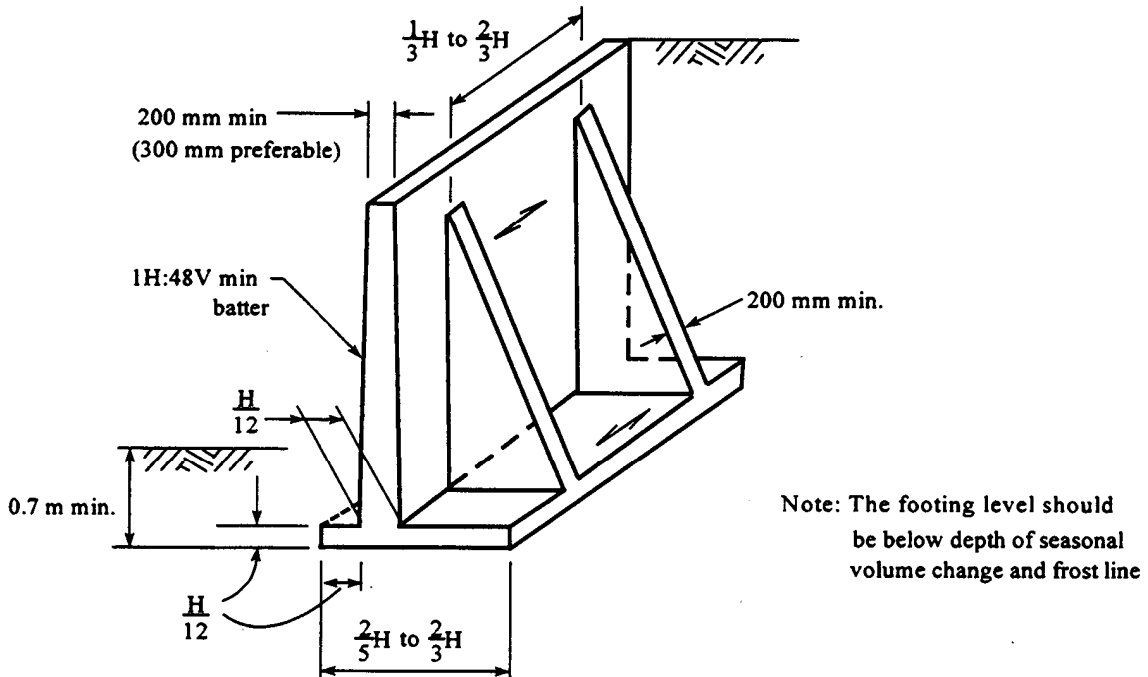
Lateral earth pressures for design of CIP walls are determined by using the procedures presented previously. Generally, Coulomb theory is used to compute earth pressures either directly on the back face of the wall, as is the case with a gravity wall, or on a vertical plane passing through the heel of the base slab, as is the case with a semi-gravity wall. Both of these concepts are illustrated in Figure 10-17.

The procedures described in Figure 10-17 are used to calculate the earth pressure loading for the wall subject to the following considerations:

- Use at-rest earth pressures for walls where rotation and displacement are restrained, e.g., rigid gravity retaining walls resting on rock or batter piles, unyielding walls such as culverts, tunnels and rigid abutment U-walls such as the CIP abutment with integral wingwalls shown in Figure 10-18.



(a)



(b)

Figure 10-16. Typical dimensions (a) Cantilever wall, (b) Counterfort wall (Teng, 1962).

[1 m = 3.28 ft; 25.4 mm = 1 in]

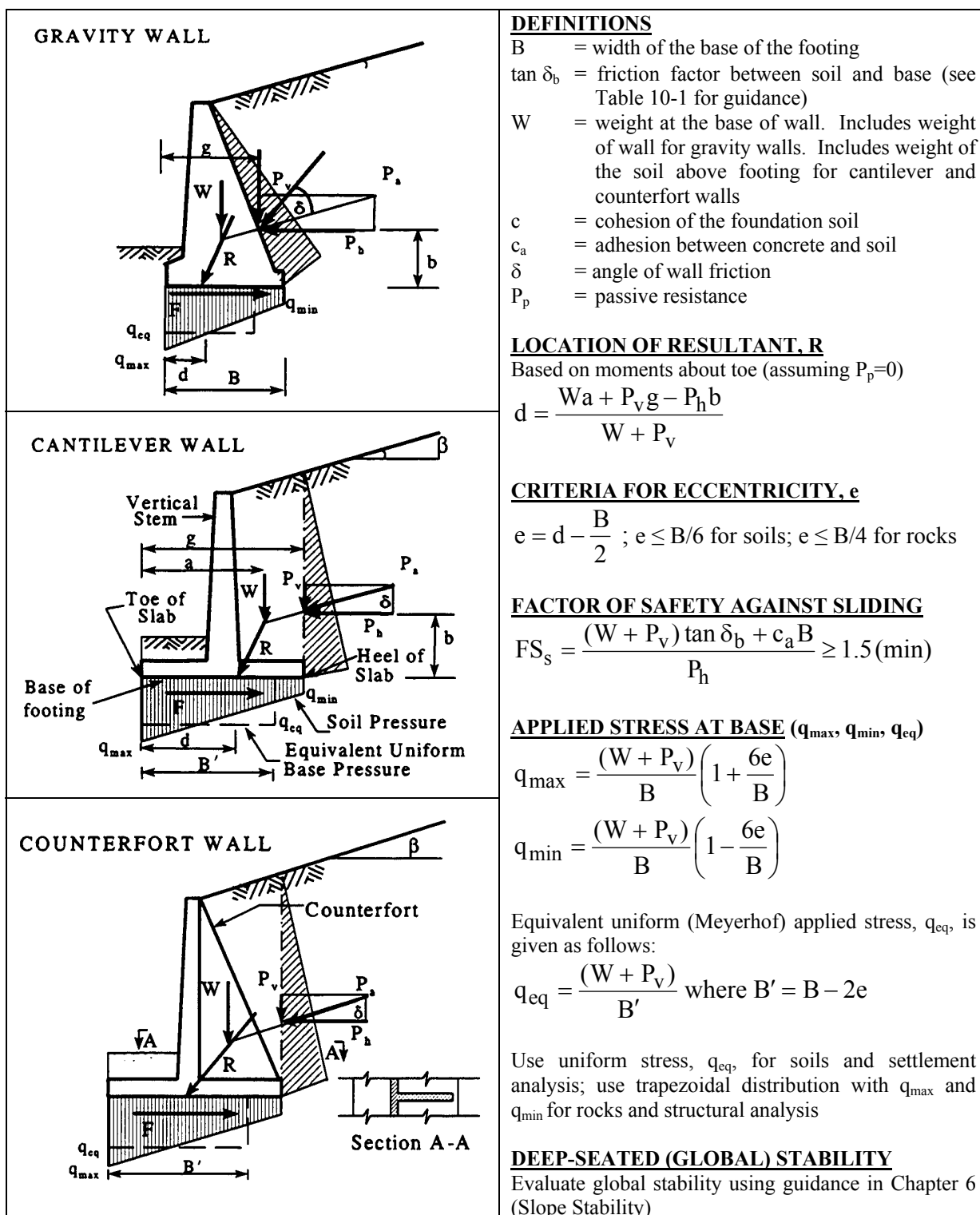


Figure 10-17. Design criteria for cast-in-place (CIP) Concrete retaining walls (after NAVFAC, 1986b).



Figure 10-18. CIP abutment with integral wingwalls

- Use the average of the at-rest and active earth pressures for CIP semi-gravity walls that are founded on rock or restrained from lateral movements, e.g., by the use of batter piles, and are less than 15 ft (5 m) in height.
- Use active earth pressures for CIP semi-gravity walls founded on rock or restrained from lateral movements that are greater than 15 ft (5 m) in height.
- Use the procedures described previously to compute pressure due to water and lateral earth pressures due to compaction and/or surcharges. Add these pressures to lateral earth pressure due to retained soil.
- Passive resistance in front of the wall should **not** be used in the analyses unless the wall extends well below the depth of frost penetration, scour or other types of disturbance such as a utility trench excavation in front of the wall. Development of the passive earth pressure in the soil in front of the wall requires a relatively large rotation or outward displacement of the wall; accordingly, the passive earth pressure

is neglected for walls with deep foundations and for other cases where the wall is restrained from rotation or displacement.

Figure 10-17 shows general loading diagrams for rigid gravity and semi-gravity walls. Loadings due to earth pressures behind the wall and for resultant vertical pressures at the base of the wall are shown.

If adequate drainage measures are provided, the hydrostatic pressure due to groundwater behind the wall generally need not be considered. However, hydrostatic pressure must be considered for portions of the wall below the level of the weep holes unless a deeper drainage system is provided behind the base of the wall. The wall must be designed for the full hydrostatic pressure when it is necessary to maintain the groundwater level behind the wall.

In addition to the lateral earth pressure, the wall must be designed for lateral pressure due to surcharge loads (see Section 10.4). For stability analyses of CIP gravity walls, the surcharge loads are generally assumed to be applied starting directly behind the top of the wall, unless specific conditions dictate otherwise. For CIP semi-gravity walls, the surcharge loads are generally assumed to be located behind the heel of the wall, and conservatively neglected within the width of the base slab since they contribute to overturning and sliding resistance. However, the surcharge loads within the width of the base slab are considered for the structural design of the wall stem.

10.5.4 Step 6 – Evaluate Bearing Capacity

10.5.4.1 Shallow Foundations

The computed vertical pressure at the base of the wall footing must be checked against the ultimate bearing capacity of the soil. The generalized distribution of the bearing pressure at the wall base is illustrated in Figure 10-17. Note that the bearing pressure at the toe is greater than that at the heel. The magnitude and distribution of these pressures are computed by using the applied loads shown in Figure 10-17. The equivalent uniform bearing pressure, q_{eq} , should be used for evaluating the factor of safety against bearing capacity failure. The procedures for determining the allowable bearing capacity of the foundation soils can be found in Chapter 8 (Spread Foundations) of this manual. Generally, a minimum factor of safety against bearing capacity failure of 3.0 is required for the spread footing foundation.

10.5.4.2 Deep Foundations

CIP walls founded on a deep foundation may be subject to potentially damaging ground and structural displacements at sites underlain by cohesive soils. Such damage may occur if the weight of the backfill material exceeds the bearing capacity of the cohesive subsoils causing plastic displacement of the ground beneath the retaining structure and heave of the ground surface in front of the wall. When the cohesive soil layer is located at or below the base of the wall, the factor of safety against this type of bearing capacity failure can be approximated by the following equation (Peck, *et al.*, 1974):

$$FS = \frac{5c}{(\gamma H + q)} \quad 10-15$$

where H is the height of the fill, γ is the unit weight of fill, c is the shear strength of the cohesive soil and q is the uniform surcharge load.

The computed factor of safety should not be less than 2.0 for the embankment loading. Below this value progressive lateral movements of the retaining structure are likely to occur (Peck, *et al.*, 1974). As the factor of safety decreases, the rate of movement will increase until failure occurs at a factor of safety of unity. For CIP walls founded on vertical piles or drilled shafts, this progressive ground movement would be reflected by an outward displacement of the wall. CIP walls founded on battered piles typically experience an outward displacement of the wall base and a backward tilt of the wall face (Figure 10-19).

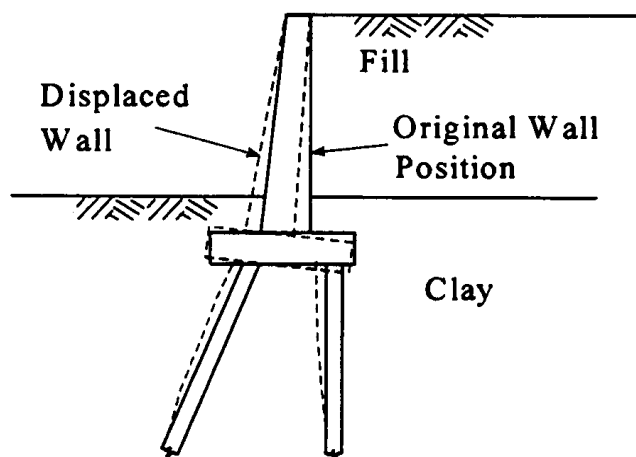


Figure 10-19. Typical Movement of pile-supported cast-in-place (CIP) wall with soft foundation.

10.5.5 Step 7 – Evaluate Overturning and Sliding

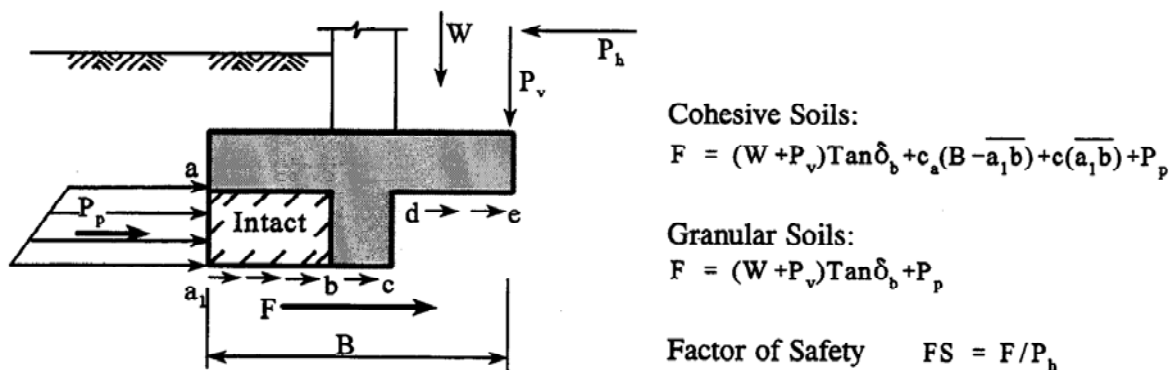
Figure 10-17 presents criteria for the design of CIP walls against sliding and eccentricity. The base dimensions of a CIP wall are determined by satisfying the following criteria:

- Sliding: $FS \geq 1.5$

Sliding resistance along the base of the wall is evaluated by using the same procedures as for spread footing design (Refer to Chapter 8.0). Note that any passive resistance provided by soil at the toe of the wall by embedment is ignored due to the potential for the soil to be removed through natural or manmade processes during the service life of the structure. Also, the live load surcharge is not considered as a stabilizing force over the heel of the wall when sliding resistance is being checked.

If adequate sliding resistance cannot be achieved, design modifications may include: (1) increasing the width of the wall base; (2) using an inclined wall base or battering the wall to decrease the horizontal load; (3) incorporating deep foundation support; (4) constructing a shear key; and (5) embedding the wall base to a sufficient depth so that passive resistance can be relied upon.

If the wall is supported by rock, granular soils or stiff clay, a key may be installed below the foundation to provide additional resistance to sliding. The method for calculating the contribution of the key to sliding resistance is shown in Figure 10-20.



Note: See Figure 10-17 for list of symbol definitions.

Figure 10-20. Resistance against sliding from keyed foundation.

- Eccentricity, e , at base: $\leq B/6$ in soil
 $\leq B/4$ in rock

The eccentricity criterion essentially requires that the safety factor of the wall against overturning is approximately of 2.0 for soils and 1.5 for rocks. If the eccentricity is not within the required limits then it implies inadequate resistance to overturning and consideration should be given to either increasing the width of the wall base or providing a deep foundation.

10.5.6 Step 8 – Evaluate Global Stability

Where retaining walls are underlain by inadequate foundation materials, the overall stability of the soil mass must be checked with respect to the most critical failure surface. As shown in Figure 10-21, both circular and non-circular slip surfaces must be considered. A minimum factor of safety of 1.5 is desirable. If global stability is found to be a problem, deep foundations or the use of lightweight backfill may be considered. Alternatively, measures can be taken to improve the shear strength of the weak soil stratum. Other wall types, such as an anchored soldier pile and lagging wall or tangent or secant pile wall, should also be considered in this case.

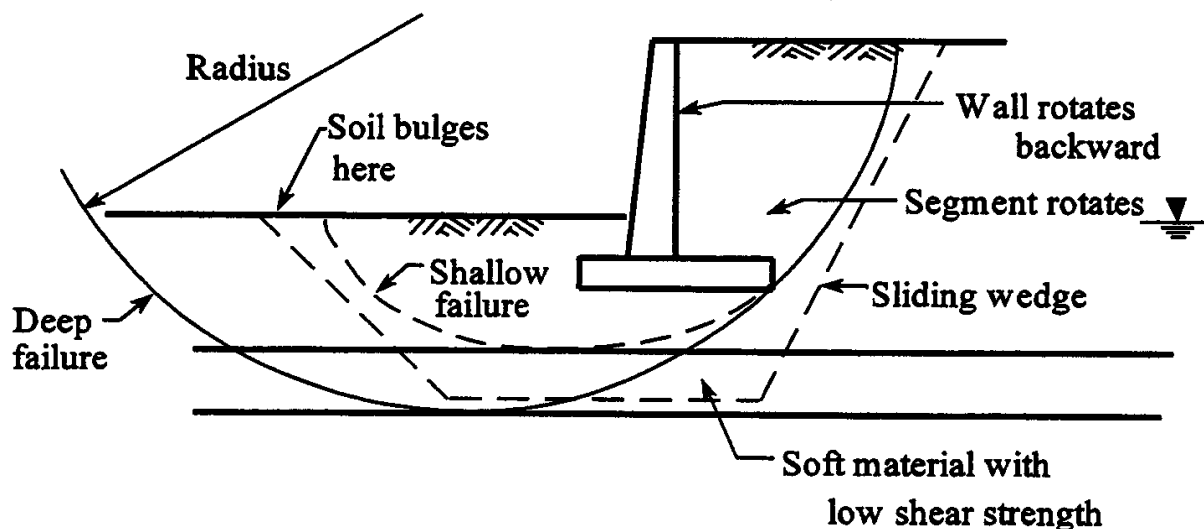


Figure 10-21. Typical modes of global stability (after Bowles, 1996)

10.5.7 Step 9 – Evaluate Settlement and Tilt

Foundation settlement can be computed by the methods discussed in Chapter 8 (Spread Foundations). CIP walls can generally accommodate a differential settlement of up to about 1/500 measured as the ratio of differential settlement of two points along the wall to the horizontal distance between the points. In general, tolerable total settlements of CIP walls are limited to 1 inch as a means to control differential settlement. If the computed settlement and tilt exceed acceptable limits, the wall dimensions can be modified to shift the resultant force closer to the center of the base and thereby reduce the load eccentricity and differential settlement. In some cases, use of lightweight backfill material may solve the problem. The use of deep foundations can also be considered.

Unless CIP walls are provided with a deep foundation, a small amount of wall tilting should be anticipated. It is therefore advisable to provide the face of the wall with a small inward batter to compensate for the forward tilting. Otherwise, a small amount of forward tilting may give the illusion that the wall is unstable.

In cases where the foundation materials are stiffer or firmer at the toe of the base than at the heel, the resulting settlement may cause the wall to rotate backwards towards the retained soil. Such wall movements could substantially increase the lateral pressures on the wall since the wall is now pushing against the soil i.e., generating a passive pressure condition. Such wall movements can be avoided by reportioning the wall, supporting the wall on a deep foundation, or treating the foundation soils.

10.5.8 Step 10 – Design Wall Drainage Systems

Water can have detrimental effects on earth retaining structures. Subsurface water and surface water can cause damage during and/or after construction of the wall. Control of water is a key component of the design of earth retaining structures.

A subsurface drainage system serves to prevent the accumulation of destabilizing hydrostatic pressures, which may develop as a result of groundwater seepage and/or infiltration of surface water. Subsurface drainage is addressed in Section 10.5.8.1. There may be several soil zones behind an earth retaining structure. Groundwater flow from one zone to another, and then to a drain and outlet feature, should be unimpeded. If impeded, water will backup at the interface of the two adjacent zones thereby increasing hydrostatic pressures and decreasing the stability of the wall structure. Soil filtration and permeability requirements must be met between the two adjacent zones of different soils to prevent impeded flow. Soil

and geotextile filter design and water collection components are discussed in Section 10.5.8.2.

Surface water runoff can destabilize a structure under construction by inundating the backfill. Surface water can also destabilize a completed structure by erosion or by infiltrating into the backfill. Design for surface water runoff is discussed in Section 10.5.8.3.

In most cases, and especially for fill walls, it is preferable to provide backfill drainage rather than design the wall for the large hydrostatic water pressure resulting from a saturated backfill. Saturation of the backfill may result from either a high static water table, from direct and/or indirect rainfall infiltrations, or from other wetting conditions, e.g., ruptured water lines, etc.

10.5.8.1 Subsurface Drainage

Potential sources of subsurface water are surface water infiltration and groundwater as illustrated in Figure 10-22. Groundwater present at an elevation above the base of the wall may have flowed into the backfill from an excavation backcut. Ground water may also be present beneath the bottom of the wall. A groundwater surface beneath a wall may rise into the structure, depending on the hydrogeology of the site. Surface water may infiltrate into the wall backfill from above, or from the front face of the wall for the case of flowing water in front of the structure (after Collin, *et al.*, 2002).

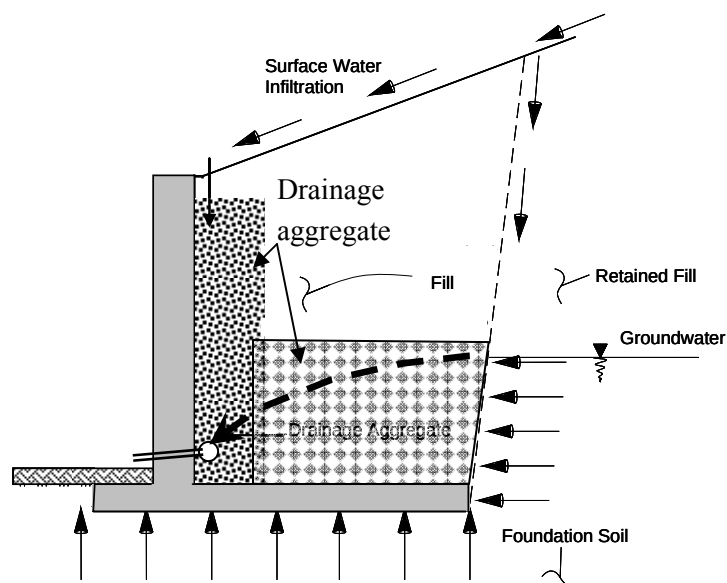


Figure 10-22. Potential sources of subsurface water.

Drainage system design depends on wall type, backfill and/or retained soil type, and groundwater conditions. Drainage system components such as granular soils, prefabricated drainage elements and filters, are usually sized and selected based on local experience, site geometry, and estimated flows, although detailed design is only occasionally performed. Drainage systems may be omitted if the wall is designed to resist full water pressure.

Drainage measures for fill wall systems, such as CIP walls, and cut wall systems typically consist of the use of a free-draining material at the back face of the wall, with “weep holes” and/or longitudinal collector drains along the back face as shown in Figure 10-23. The collector drains may be perforated pipes or gravel drains. This minimum amount of drainage should be sufficient if the wall backfill is relatively free-draining and allows the entire backfill to serve as a drain. It may be costly to fully backfill with free-draining or relatively free-draining material for some project applications therefore, it may be necessary to construct other types of drainage systems.

Fill wall drains may be placed (1) immediately behind the concrete facing or wall stem; (2) between wall backfill and embankment fill; (3) along a backcut; and (4) as a blanket drain beneath the wall. Examples of drains behind a wall stem are shown in Figure 10-24. The drainage system shown in the figure primarily serves to collect surface water that has infiltrated immediately behind the wall and transport it to an outlet. The system may also serve to drain the wall backfill, if the backfill soil is relatively free-draining.

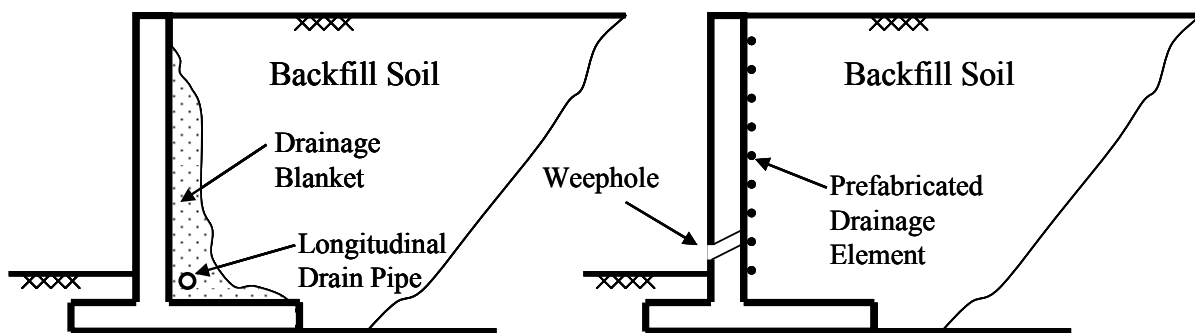


Figure 10-23. Typical retaining wall drainage alternatives.

A drain behind the wall backfill should be used when the backfill is not relatively free-draining. Such a drain may be located as noted in (2) or (3) above, and as illustrated in Figure 10-24. A granular blanket drain with collection pipes and outlets should be used beneath fill wall structures where a high or seasonally high groundwater table exists.

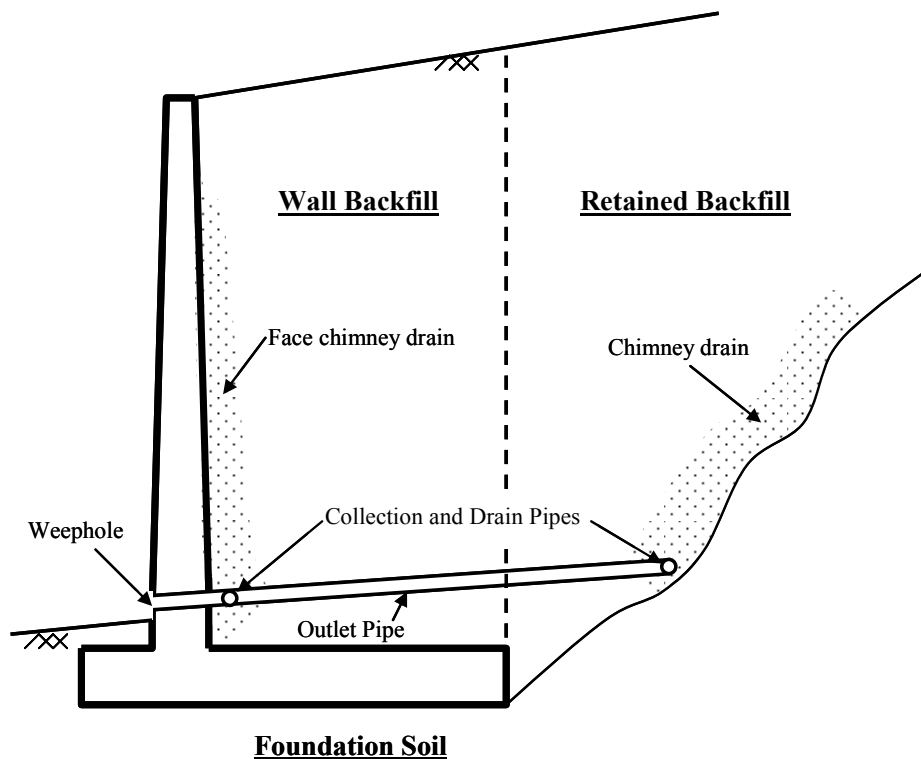


Figure 10-24. Drains behind backfill in cantilever wall in a cut situation.

10.5.8.2 Drainage System Components

Drainage systems for fill walls may include:

- column(s) or zone(s) of free-draining gravel or coarse sand to collect water seepage from the backfill;
- perforated pipe(s) to collect water in the granular column(s) or zone(s);
- conveyance piping;
- outlet(s); and
- filter(s) between backfill soil(s) and granular column(s) or zone(s).

Longitudinal pipes transport collected water to outlet pipes that discharge at appropriate points in front of and/or below the wall. Outlets may be via weep holes through the wall facing that discharge in front of the structure to grade; via conveyance piping to storm sewers as is common in urban applications, or via conveyance piping to a slope beneath the wall structure. Weep holes generally consist of 1½ - 3 in (40 - 75 mm) diameter holes that extend through the wall facing and are closely spaced horizontally along the wall, typically less than 10 ft (3 m) apart. If weep holes are used with a counterfort wall, at least one weep hole

should be located between counterforts. A screen and/or filter are used to prevent soil piping through a weep hole.

The collection and conveyance pipes need to be large enough and sufficiently sloped to effectively drain water by gravity flow from behind the wall while maintaining sufficient pipe flow velocity to prevent sediment buildup in the pipe. Use of 3 to 4 in (75 to 100 mm) diameter pipes is typical and practical. The diameter is usually much greater than that required for theoretical flow capacity. Procedures for the design of pipe perforations, such as holes or slots, is provided in Section 5.2 of Cedergren (1989). Pipe outlets to slope areas beneath wall structures should be detailed similar to pavement drain outlets. If the outlet is to a grass area, it should have a concrete apron, a vertical post marking its location (for maintenance), and a screen to prevent animal ingress.

Filters are required for water flowing between zones of different soils. A filter must prevent piping of the retained soil while providing sufficient permeability for unimpeded flow. The filter may be a soil or a geotextile. A geotextile is not required if the two adjacent soils meet certain soil filtration criteria. An open-graded aggregate will generally not allow the development of a soil filter at its interface with the backfill soil. In this case a geotextile filter will be required.

Geocomposite drains may be used in lieu of clean gravel or coarse sand and a geotextile. A geocomposite, or prefabricated, drain consists of a geotextile filter and a water collection and conveyance core. The cores convey the water and are generally made of plastic waffles, three-dimensional meshes or mats, extruded and fluted plastic sheets, or nets. A wide variety of geocomposites are readily available. However, the filtration and flow properties, detailing requirements, and installation recommendations vary and may be poorly defined for some products.

The flow capacity of geocomposite drains can be determined by using the procedures described in ASTM D 4716. Long-term compressive stresses and eccentric loadings on the geocomposite core should be considered during design and selection. The geotextile of the geocomposite should be designed to meet filter and permeability requirements.

Installation details, such as joining adjacent sections of the geocomposite and connections to outlets, are usually product-specific. Product-specific variances should be considered and addressed in the design, specification, detailing and construction phases of a project. Post installation examination of the drainage core/path with a camera scope should be considered for critical applications.

10.5.8.3 Surface Water Runoff

Surface drainage is an important aspect of ensuring wall performance and must be addressed during design. Appropriate measures to prevent surface water from infiltrating into the wall backfill should be included in the design of all earth retaining structures.

During construction of a fill wall, the backfill surface should be graded away from the wall face at the end of each day of construction to prevent water from ponding behind the wall and saturating the soil. Surface water running onto a partially completed backfill can carry fine-grained soils into the backfill work area and locally contaminate a free-draining granular backfill with fines. If a fine-grained soil is being utilized for the backfill, saturation can cause movements of the partially constructed wall facing.

Finish grading at the top of a wall structure should provide positive drainage away from the wall, when possible, to prevent or minimize infiltration of surface water into the backfill. If the area above the wall is paved, a curb and gutter is typically used to direct the flow away from the wall. Concrete-, asphalt- or vegetation-lined drainage swales may be used where a vegetated finished grade slopes to the wall. Water runoff over the top of a wall where the backfill slopes towards it can lead to erosion and undercutting of the wall and can cause staining of the wall face as soil is carried with the water. Construction of a collection swale close to the wall will help to prevent runoff from going over the top of the wall. Runoff flow will concentrate at grading low points behind the face. Ponding of runoff behind the wall leads to undesirable infiltration of water into the backfill.

Collection and conveyance swales should prevent overtopping of the wall for the design storm event. Extreme events (e.g., heavy rainfalls of short duration) have been known to cause substantial damage to earth retaining structures due to erosion and undercutting, flooding, and/or increased hydrostatic pressures both during and after construction. This is particularly true for sites where surface drainage flows toward the wall structure and where finer-grained backfills are used.

Site drainage features are designed for an assumed or prescribed design storm event, such as, the 25 year storm event. However, extreme events can occur that result in short duration flows, e.g., 1 to 3 hours, that significantly exceed the design capacity of the stormwater management system. When such events occur, site flooding can cause overtopping of the wall, erosion and undercutting, and an increase in hydrostatic forces within and behind the reinforced soil mass.

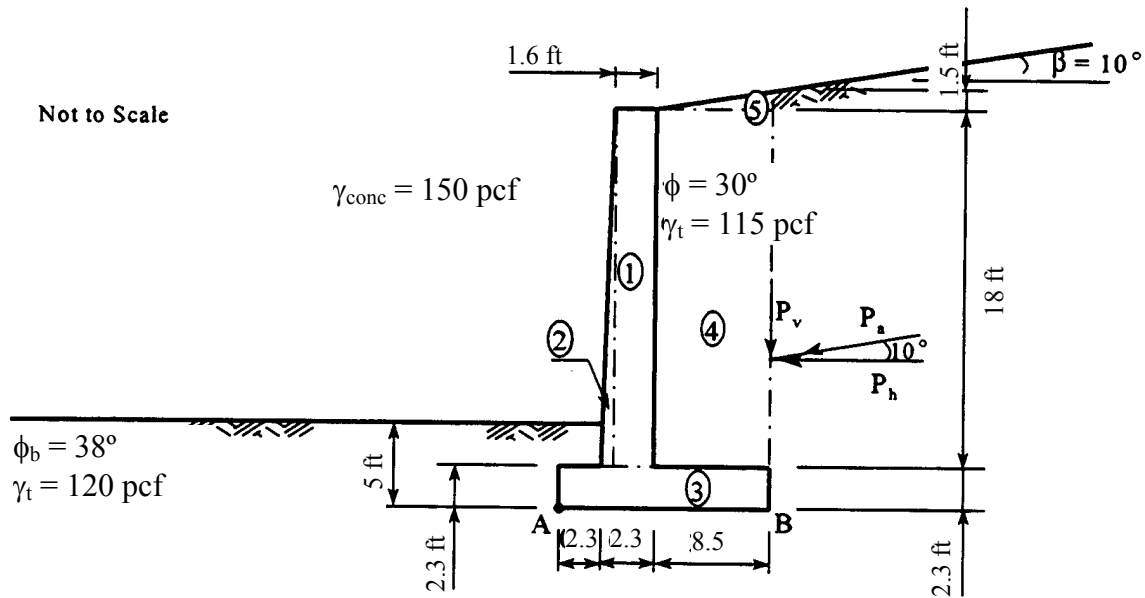
If surface water flows toward an earth retaining structure, the water is likely to be picked up in a gutter or other collection feature. Such features are often sized based upon the design storm event. The site layout and wall structure should include features for handling flows greater than the design event as is typically done in the design of an overflow spillway for a dam. The wall designer should address potential excess flows and coordinate work with other project designers. Consideration should be given to incorporating details of overflow features, such as a spillway, into the wall design for sites where surface water flows towards the wall structure.

10.6 EXTERNAL STABILITY ANALYSIS OF A CIP CANTILEVER WALL

The following example problem is used to illustrate the procedure for performing an external stability analysis of a CIP cantilever retaining wall.

Example 10-3.

Analyze the CIP cantilever wall shown below for factors of safety against sliding, overturning and bearing capacity failure. The backfill and foundation soils consist of clean, fine to medium sand, and the groundwater table is well below the base of the wall.



Geometry and parameters for example problem.

Solution

Step 1: Determine the total height of soil exerting pressure.

$$H = \text{thickness of base slab} + \text{height of stem} + (\text{width of heel slab}) \tan (\text{backslope angle})$$

$$\begin{aligned} H &= 2.3 \text{ ft} + 18 \text{ ft} + 8.5 \text{ ft} (\tan 10^\circ) \\ &= 21.8 \text{ ft} \end{aligned}$$

Step 2: Compute the coefficient of active earth pressure by using the equation of K_a in Figure 10-5 for a vertical backface ($\theta=0$).

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

where:

ϕ = internal friction angle of soil = 30°

β = angle of backfill slope = 10°

δ = angle of wall friction = $\beta = 10^\circ$

For the example problem:

$$K_a = \frac{\cos^2 30^\circ}{\cos 10^\circ \left[1 + \sqrt{\frac{\sin(30^\circ + 10^\circ) \sin(30^\circ - 10^\circ)}{\cos 10^\circ \cos(-10^\circ)}} \right]^2}$$

$$K_a = 0.35$$

Step 3. Compute the magnitude of the resultant of active pressure, P_a , per foot of wall into the plane of the paper.

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

$$= \frac{1}{2} (0.35)(115 \text{ pcf})(21.8 \text{ ft})^2 = 9,564.2 \text{ lb/ft}$$

Step 4. Resolve P_a into horizontal and vertical components:

$$\begin{aligned} P_h &= P_a \cos \beta & P_v &= P_a \sin \beta \\ &= (9,564.2 \text{ lb/ft}) \cos 10^\circ & &= (9,564.2 \text{ lb/ft}) \sin 10^\circ \\ &= 9,418.9 \text{ lb/ft} & &= 1,660.8 \text{ lb/ft} \end{aligned}$$

Moment arm of P_h about point A = $(2.3 \text{ ft} + 18 \text{ ft} + 1.5 \text{ ft})/3 = 21.8/3 = 7.27 \text{ ft} = b$

Moment arm of P_v about point A = $2.3 \text{ ft} + 2.3 \text{ ft} + 8.5 \text{ ft} = 13.1 \text{ ft} = g$

Step 5: Determine weights and sum moments about the toe of the wall (point A).

The weights of various areas and the moments due to the weights shown in the geometry of the example problem are set out in the following table. The unit weight of concrete is assumed to be 150 pcf and the weight of the soil above the footing toe is neglected.

Area	Weight, lb/ft	Moment arm about A, ft	Moment about A, lb.ft/ft
1	(1.6 ft) (18 ft) (150 pcf) = 4,320	2.3 ft+0.7 ft+(1.6/2) ft = 3.80	(4,320 lb) (3.80 ft) = 16,416.0
2	(0.5) (0.7 ft) (18 ft) (150 pcf) = 945	2.3 ft+ (2/3) (0.7) ft = 2.77	(945 lb) (2.77 ft) = 2,617.7
3	(13.1 ft) (2.3 ft) (150 pcf) = 4,519.5	13.1/2 ft = 6.55	(4,519.5 lb) (6.55 ft) = 29,602.7
4	(8.5 ft) (18 ft) (115 pcf) = 17,595	2.3 ft+ 2.3 ft+(8.5/2) ft = 8.85	(17,595 lb) (8.85 ft) = 155,715.8
5	(0.5) (8.5 ft) (1.5 ft) (115 pcf) = 733.1	2.3 ft+2.3 ft+(2/3)(8.5) ft = 10.27	(733.1lb) (10.27 ft) = 7,528.9
Total	W = 28,112.6		M_w = 211,881.1

Step 6: Check factor of safety against sliding; neglect passive resistance of embedment depth soil (Refer to Figure 10-20)

$$FS_s = \frac{(W + P_V) \tan \delta_b}{P_h}$$

where:

W = weight of concrete and soil on the base of the wall footing AB

δ_b = friction angle between concrete base and foundation soil

Use $\delta_b = (3/4) \phi_b = (3/4) (38^\circ) = 28.5^\circ$, for friction angle between concrete and clean, fine to medium sand (see NAVFAC, 1986b). This value of δ_b is within the range of values listed in Table 10-1 for clean fine to medium sand.

$$FS_s = \frac{(28,112.6 \text{ lb/ft} + 1,660.8 \text{ lb/ft}) \tan 28.5^\circ}{9,418.9 \text{ lb/ft}} = \frac{16,165.6 \text{ lb/ft}}{9,418.9 \text{ lb/ft}} = 1.72 \quad \text{O.K.}$$

Step 7: Check the limiting eccentricity and factor of safety against bearing failure.

(1) Compute the location of resultant at distance d from point A.

$$d = \frac{\sum M_R - \sum M_0}{\sum V}$$

$$d = \frac{M_W - P_h b + P_V g}{W + P_V}$$

$$d = \frac{21,1881.1 \text{ lb.ft/ft} + (1,660.8 \text{ lb/ft})(13.1 \text{ ft}) - (9,418.9 \text{ lb/ft})(7.27 \text{ ft})}{28,112.6 \text{ lb/ft} + 1,660.8 \text{ lb/ft}}$$

where: $W + P_V = \sum V$

$$d = \frac{16,5162.2 \text{ lb.ft/ft}}{29,773.4 \text{ lb/ft}} = 5.55 \text{ ft}$$

(2) Compute the eccentricity of the load about the center of base.

$$e = \frac{B}{2} - d = \frac{13.1 \text{ ft}}{2} - 5.55 \text{ ft} = 1.0 \text{ ft}$$

$$e = 1.0 \text{ ft} < \frac{B}{6} = \frac{13.1 \text{ ft}}{6} = 2.18 \text{ ft} \quad \text{O.K.}$$

(3) Compute the maximum and minimum pressures under the wall footing.

$$\begin{aligned} q_{\max, \min} &= \frac{\sum V}{B} \left(1 \pm \frac{6e}{B} \right) \\ &= \frac{29,773.4 \text{ lb/ft}}{13.1 \text{ ft}} \left(1 \pm \frac{6(1.0 \text{ ft})}{13.1 \text{ ft}} \right) \\ &= 2,272.7 \text{ psf (1.46 or 0.54)} \end{aligned}$$

$$\begin{aligned} \text{i.e., } q_{\max} &= 3,318.1 \text{ psf} \\ q_{\min} &= 1,227.3 \text{ psf} \end{aligned}$$

(4) Estimate ultimate bearing capacity.

Use the procedures presented in Chapter 8 (Shallow Foundations). Assume that for a footing with eccentric and inclined loading the ultimate bearing capacity computed by the geotechnical specialist is:

$$q_{\text{ult}} = 20,000 \text{ psf}$$

(5) Check factor of safety against bearing capacity failure.

$$FS_{bc} = \frac{q_{\text{ult}}}{q_{\max}} = \frac{20,000 \text{ psf}}{3,318.1 \text{ psf}} = 6.03 > 3.0 \quad \text{O.K.}$$

SUMMARY

Factor of safety against sliding	FS_s	$= 1.72$
Eccentricity	e	$= 1.0 \text{ ft} < B/6$
Factor of safety against bearing failure	FS_{bc}	$= 6.03$

In addition, the factor of safety against global failure and wall settlement including tilting and lateral squeeze should be evaluated to complete the analysis.

10.7 CONSTRUCTION INSPECTION

FHWA (2005b) discusses construction considerations for many of the walls presented in Figure 10-3. Construction considerations for CIP walls only are presented in this manual. In general, the construction inspection requirements for CIP walls are similar to those for other concrete structures. In some cases, state agencies may have inspector checklists for this type of construction. Table 10-4 provides a summary of typical construction inspection requirements for CIP retaining walls.

Table 10-4
Inspector responsibilities for a typical CIP gravity and semi-gravity wall project

CONTRACTOR SET UP
Review plans and specifications
Review the contractor's schedule
Review test results and certifications for preapproved materials, e.g., cement, coarse and fine aggregate.
Confirm that the contractor's stockpile and staging area are consistent with locations shown on the plans
Discuss anticipated ground conditions and potential problems with the contractor
Review the contractor's survey results against the plans
EXCAVATION
Verify that excavation slopes and/or structural excavation support is consistent with the plans
Confirm that limits of any required excavations are within right-of-way limits shown on plans
Confirm that all unsuitable materials, e.g., sod, snow, frost, topsoil, soft/muddy soil are removed to the limits and depths shown on the plans and that the excavation is backfilled with granular material and properly compacted
Confirm that leveling and proof-rolling of the foundation area is consistent with requirements of the specifications
Confirm that the contractor's excavation operations do not result in significant water ponding
Confirm that existing drainage features, utilities, and other features are protected
Identify areas not shown on the plans where unsuitable material exists and notify the engineer

FOOTING
Approve condition of footing foundation soil/rock before concrete is poured
Confirm reinforcement strength, size, and type consistent with the specifications
Confirm the consistency of the contractor's outline of the footing (footing size and bottom of footing depth) with the plans
Confirm the location and spacing of reinforcing steel consistent with the plans
Confirm water/cement ratio and concrete mix design consistent with the specifications
Record concrete volumes poured for the footing
Confirm appropriate concrete curing times and methods as provided in the specifications
Confirm that concrete is not placed on ice, snow, or otherwise unsuitable ground
Confirm that concrete is being placed in continuous horizontal layers and that the time between successive layers is consistent with the specifications
STEM
Confirm the placement of weep hole inserts (number, elevation, and specific locations) with the plans if weep holes are used,
Confirm that concrete is poured in section lengths consistent with the specifications
Record concrete volumes used to form the stem
Confirm that all wall face depressions, air pockets, gaps, rough spots, etc. are repaired
Confirm that storage of reinforcing bars is consistent with the specifications, e.g.-use of platform or supports.
Perform preliminary check of condition of epoxy-coated reinforcing bars
Confirm that forms are clean and appropriately braced during concrete pour operations
Confirm that all reinforcing bars are held securely in place and are being rigidly supported at the face of forms and in the bottom of wall footings
Confirm that construction joints are being made only at locations shown on the plans or otherwise at locations approved by the engineer
DRAINAGE SYSTEMS AND BACKFILL
Confirm that installation of the drainage system is consistent with the specifications and plans
Confirm that the backfill material being used is approved by the engineer
Confirm that placement of the backfill is performed in lifts consistent with the specifications
Confirm that minimum concrete strength is achieved before backfill is placed and compacted against back of wall
Confirm that the backfill placement method used by the contractor does not cause damage to prefabricated drainage material or drain pipes
Confirm that earth cover over drainage pipes is sufficient to prevent damage from heavy equipment. The minimum cover based on ground pressure from equipment should be provided in the specifications.
Perform required backfill density tests at the frequencies specified, especially for areas that are compacted with lightweight equipment, e.g., areas just behind the wall.
Check that the drainage backfill just behind weep holes is the correct gradation and that it is properly installed
POST INSTALLATION
Verify pay quantities

Note: Throughout the project, check submittals for completeness before transmitting them to the engineer.