22

Retaining Structures

Jonathan D. Bray
University of California at Berkeley

22.1 Introduction

Civil engineering projects often require the construction of systems that retain earth materials. An excavation support system for a cut-and-cover trench for utilities installation is an example of a temporary retaining structure. A reinforced concrete retaining wall utilized in a highway project to accommodate a change in elevation over a limited distance is an example of a permanent retaining structure. Numerous earth retention systems have been developed over the years and a few systems are shown in Fig. 22.1. The design of retaining structures requires an evaluation of the loads likely to act on the system during its design life and the strength, load-deformation, and volume-change response of the materials to the imposed loads. Lateral pressures develop on retaining structures as a result of the adjacent earth mass, surcharge, water, and equipment. The development of lateral earth pressures and the transfer of these pressures to the retaining system are inherently governed by soil-structure interaction considerations. Hence, the analytical procedure should consider the relative rigidity/flexibility of the earth retention system. In this chapter, retaining structures will be broadly classified as either *rigid* or *flexible*. Before applicable design procedures are discussed, **lateral earth pressure** concepts will be reviewed.

22.2 Lateral Earth Pressures

The at-rest lateral earth pressure within a level earth mass of large areal extent can be estimated using the following relationship:

$$\sigma_h' = K_0 \sigma_v' \tag{22.1}$$

where σ_h' is the horizontal effective pressure, σ_v' is the vertical effective pressure, and K_0 is the **lateral earth pressure coefficient at rest**. Note that this relationship is valid in terms of effective stress only. The parameter K_0 is difficult to evaluate. Based on Jaky [1944], K_0 for normally consolidated soils, $K_{0,nc}$, can be approximated as

$$K_{0,nc} \approx 1 - \sin\phi' \tag{22.2}$$

where ϕ' is the effective friction angle. This correlation appears to be more reasonable for clays and less reasonable for sands, but it is widely used in practice [see data presented in Mayne and Kulhawy, 1982].

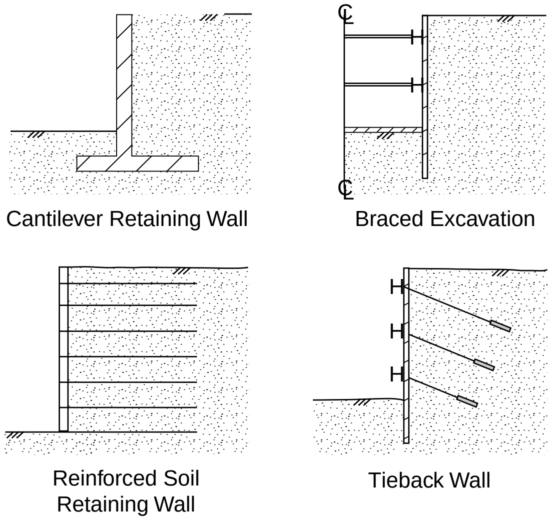


FIGURE 22.1 Examples of earth retention systems.

A strong case could be made for just using $K_0 \approx 0.4$ for most normally consolidated sands. Schmidt's [1966] relationship as modified by Mayne and Kulhawy [1982] can be used to estimate the coefficient of lateral earth pressure at rest during unloading, $K_{0,u}$, as

$$K_{0,u} \approx K_{0,nc}(\text{OCR})^{\sin\phi'} \tag{22.3}$$

where OCR is the **overconsolidation ratio.** The at-rest lateral earth pressure coefficient during reloading is not known precisely, but it can be approximated as halfway between $K_{0,u}$ and $K_{0,nc}$ [Clough and Duncan, 1991]. K_0 may also be estimated on a site-specific basis using *in situ* test devices, such as the pressure meter or dilatometer [see Kulhawy and Mayne, 1990].

If a perfectly rigid vertical wall was wished into place (i.e., no lateral deformation occurred), then the at-rest *in situ* pressure distribution would be preserved. In homogeneous, dry soil with a constant K_0 and unit weight, γ, the horizontal effective stress would increase linearly with depth, z, proportional to the linearly increasing vertical effective stress (i.e., $\sigma_h' = K_0 \gamma z$). This would result in a triangular pressure distribution. With the presence of a level water table, the total lateral pressure would consist of two components: horizontal effective earth pressure and water pressure ($u = \gamma_w \cdot z_w$, where u = pore water pressure, γ_w = unit weight of water, and z_w = depth below water table). Layered soil profiles can be easily handled by calculating the vertical effective stress at the depth of interest and multiplying this value by the parameter K_0 that best represents the soil layer at this depth. The compaction of earth fill behind a rigid wall that does not move will lock in higher lateral earth pressures than the at-rest condition [see Duncan and Seed, 1986].

If the rigid vertical retaining wall discussed above translates outward the lateral earth pressure on the back of the wall reduces, since the adjacent earth mass mobilizes strength as it deforms to follow the outward wall movement. The minimum active lateral earth pressure is reached when the soil has mobilized its maximum shear strength. Conversely, if the rigid wall translates inward, the lateral earth pressure on the back of the wall increases as the adjacent earth mass mobilizes strength and resists the inward wall translation. At the limiting state, the maximum passive lateral earth pressure is attained. Hence, a range of possible magnitudes for the lateral earth pressure on the back of the wall exist, depending on the direction and magnitude of wall movement (see Fig. 22.2). The **minimum active earth pressure** defined by $p_a' = K_a \sigma_v'$ and the **maximum passive earth pressure** defined by $p_p' = K_p \sigma_v'$ represent only the limiting states of the possible lateral pressure range. Note that p_a' is attained at relatively low wall

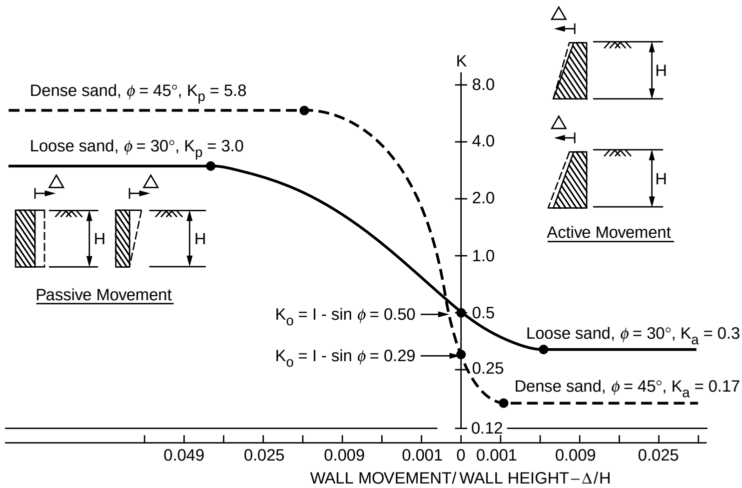


FIGURE 22.2 Relationship between wall movement and earth pressure. (After Clough, G. W., and Duncan, J. M. 1991. Earth Pressures. In *Foundation Engineering Handbook,* ed. H-Y. Fang, pp. 224–235. Van Nostrand Reinhold, New York.)

displacements, and that an order of magnitude more wall displacement is required to reach p_a'. Consequently, engineers often design retaining structures for the full active state when the earth retaining wall can move outward, but only for a fraction of the full passive state when the retaining wall moves inward.

22.3 Earth Pressure Theories

Rankine Theory

If the vertical wall is frictionless and the retained earth materials are level, homogeneous, isotropic, and characterized by the Mohr–Coulomb strength criterion (i.e., $s = c' + \sigma_n' \tan \phi'$), the limiting states of stress can be estimated using Rankine [1857] earth pressure theory. The minimum active earth pressure is

$$p_a' \;=\; K_a \sigma_v' - 2c' \sqrt{K_a} \tag{22.4}$$

where $K_a = \tan^2(45 - \phi'/2)$. Note that the vertical effective stress can include the effects of any applied surcharge as well as the gravity load of the earth materials. Typically, only the destabilizing effects of the active earth pressure (i.e., $p_a' > 0$) are included in developing design pressures, and a tension crack filled with water is conservatively assumed to exist down to the depth at which $p_a' = 0$. Layered soil profiles can be easily handled by calculating p_a' at the top and bottom of each soil layer and realizing that the lateral earth pressure varies linearly between these points. With freely draining cohesionless materials, effective strength parameters should be used and the earth and water pressure distributions should be computed separately. In a short-term undrained analysis of cohesive soils, total strength parameters may be used, but now the earth and water pressures will be calculated together since the total strength parameters include pore water effects.

The Rankine maximum passive earth pressure is

$$p_p' \;=\; K_p \sigma_v' + 2c' \sqrt{K_p} \tag{22.5}$$

where $K_p = \tan^2(45 + \phi'/2)$. Rankine theory underestimates the actual maximum passive earth pressure, so engineers often use the full Rankine passive earth pressure in design.

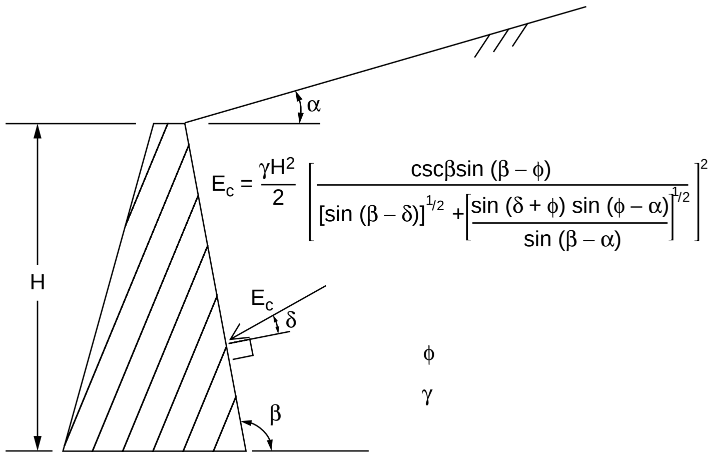


FIGURE 22.3 Closed-form solution for Coulomb minimum active earth force.

Coulomb Theory

Coulomb [1776] solved the lateral earth pressure problem assuming a homogeneous, isotropic material, rough wall, planar failure surface, and Mohr–Coulomb strength criterion. A wide range of earth pressure problems can be solved using the force polygon technique implied in Coulomb's method. The closed-form solution for the minimum active earth force for a general case including dry, cohesionless material, inclined rough wall, and sloping backfill is presented in Fig. 22.3. Coulomb theory can overestimate the actual maximum passive earth pressure acting on a rough wall with the wall friction angle, δ, greater than half of ϕ', so its use should be avoided. The use of log-spiral failure surfaces, as opposed to planar failure surfaces, provides good estimates of minimum active and maximum passive earth pressures, and graphical solution charts are available (see Fig. 22.4).

Example 22.1 — Lateral Earth Pressure

Calculate the resultant active earth pressure by Rankine theory for the case shown in Fig. 22.5(a).

Solution.

Sand ($\phi' = 30°$, $c' = 0$): $K_a = \tan^2(45° - \phi'/2) = \tan^2(45° - 30°/2) = 0.333$; $p_a' = K_a\gamma' z = K_a\rho' gz$

Clay ($\phi = 0°$, $c = 24$ kPa): $K_a = \tan^2(45 - 0°/2) = 1.0$; $p_a = \rho_t gz - 2c$

At $z = 3$ m, $p_a' = K_a\rho' gz = (0.333)\,(2.0\text{ Mg/m}^3)\,(9.81\text{ m/s}^2)\,(3\text{ m}) = 19.6$ kPa

At $z = 6^-$ m, $p_a' = 19.6\text{ kPa} + (0.333)\,(2.0 - 1.0\text{ Mg/m}^3)\,(9.81\text{ m/s}^2)(3\text{ m}) = 29.4$ kPa

At $z = 6^-$ m, $u = \rho_w g z_w = (1\text{ Mg/m}^3)\,(9.81\text{ m/s}^2)\,(3\text{ m}) = 29.4$ kPa

At $z = 6^+$ m, $p_a = \rho_t gz - 2c = (2\text{ Mg/m}^3)\,(9.81\text{ m/s}^2)\,(6\text{ m}) - 2(24\text{ kPa}) = 69.7$ kPa

At $z = 12^-$ m, $p_a = 69.7\text{ kPa} + (1.8\text{ Mg/m}^3)\,(9.81\text{ m/s}^2)\,(6\text{ m}) = 175.6$ kPa

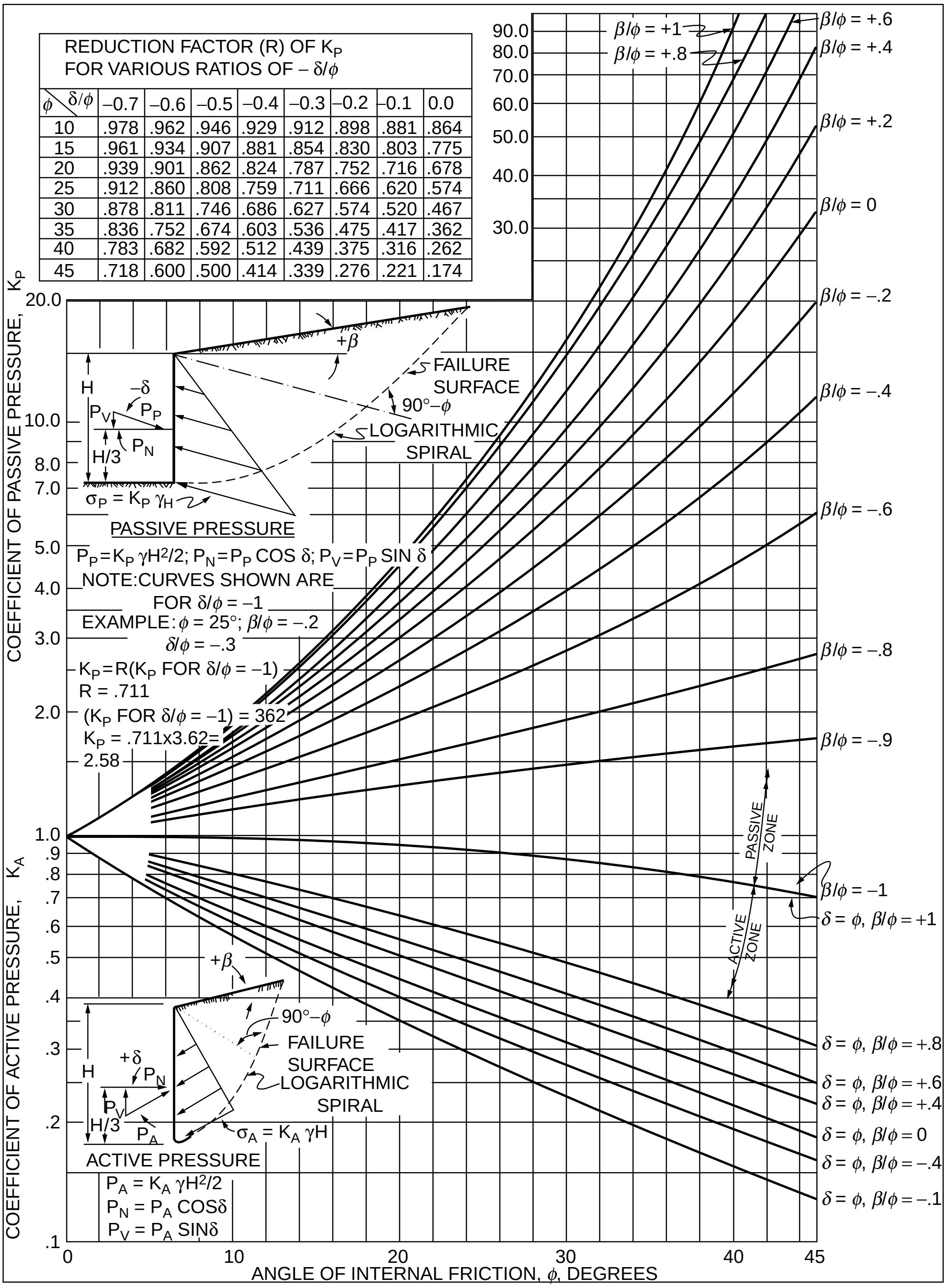
$$P_{ae} = \tfrac{1}{2}(19.6\text{ kN/m}^2)\,(3\text{ m}) + \tfrac{1}{2}(19.6 + 29.4\text{ kN/m}^2)\,(3\text{ m}) + \tfrac{1}{2}(69.7 + 175.6\text{ kN/m}^2)\,(6\text{ m})$$
$$= 839\text{ kN/m}$$

$$P_w = \tfrac{1}{2}(29.4\text{ kN/m}^2)\,(3\text{ m}) = 44\text{ kN/m}$$

See Fig. 22.5(b) for pressure diagram.

22.4 Rigid Retaining Walls

Design lateral active earth pressures for low retaining walls (i.e., height <6 m) are often estimated using conservative design charts [see Department of the Navy, 1982] or using equivalent fluid pressures. The equivalent fluid unit weight, γ_{eq}, equals the product of the minimum active earth pressure coefficient and the backfill material's unit weight (i.e., $\gamma_{eq} = K_a \cdot \gamma$), and



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

FIGURE 22.4 Minimum active and maximum passive lateral earth pressure coefficients developed from log-spiral solution techniques. (After Department of the Navy. 1982. *Foundations and Earth Structures: Design Manual 7.2.* NAVFAC DM-7.2, May.)

$$p_a = \gamma_{eq} \cdot z \tag{22.6}$$

Conservative estimates of γ_{eq} for a variety of backfill materials are listed in Table 22.1. All earth retention structures should be designed to sustain potential surcharge loadings, and typically a minimum surcharge

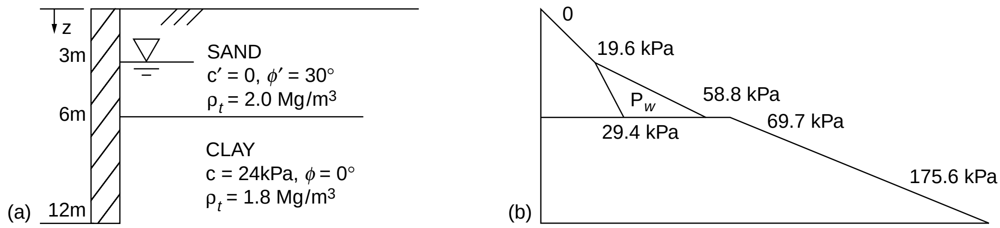


FIGURE 22.5 Example 22.1.

TABLE 22.1 Equivalent Fluid Unit Weights (kN/m3) for Design of Low Retaining Walls

	Level Backfill		2H:IV Backfill
Soil	At-Rest	Active	Active
Clean sand or gravel	7.5	5	6
Silty sand	8.5	6	8
Clayey sand	9.5	7	9
Sandy clay	11	10	11
Fat clay	13	12	14

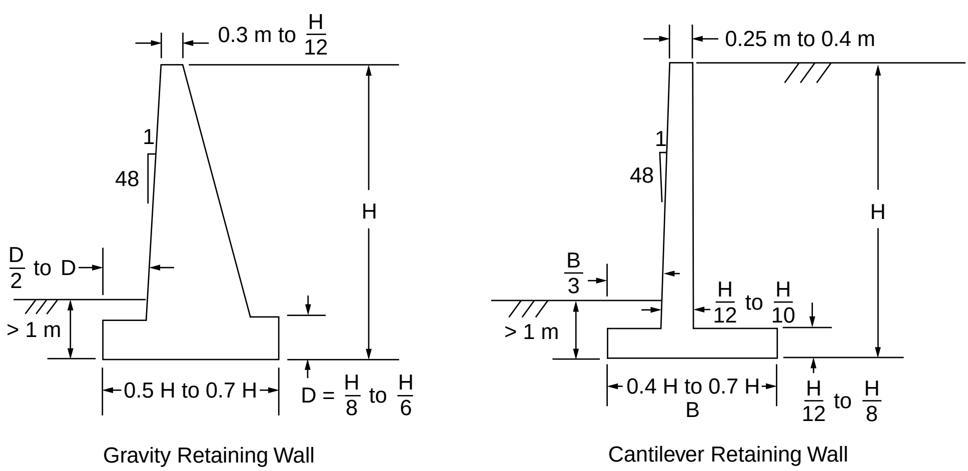


FIGURE 22.6 Tentative gravity and cantilever wall dimensions.

load equivalent to an additional 0.6-m thickness of backfill is specified. Retaining walls should be constructed with free-draining backfill materials and with effective drainage systems, because if water can pond behind the wall the additional water pressure will dramatically increase the load on the wall. If ponding cannot be precluded, the wall should be designed to resist the higher total pressures.

The general design procedure for gravity and cantilever retaining walls follows:

1. Characterize project site and subsurface conditions. Pay particular attention to groundwater and surface water, site geology, availability of free-draining backfill soils, and potentially weak seams.
2. Select tentative wall dimensions (see Fig. 22.6).
3. Estimate the forces acting on the retaining wall (i.e., active earth pressure, weight, surcharge, and resultant).
4. Check overturning stability; the resultant force should act within the middle third of the base of the wall.

5. Check bearing capacity; maximum earth pressure on the wall base should be less than the allowable earth pressure regarding bearing capacity or permissible settlement.
6. Check sliding; horizontal frictional resisting force on the base of the wall should be at least 1.5 times the horizontal driving force.
7. Check for excessive settlement from deeper soil deposits.
8. Check the overall stability of the earth mass that contains the retaining structure.
9. Apply load factors and compute reactions, shears, and moments in the wall.
10. Compute the ultimate strength of the structural components.
11. Check adequacy of structural components against applied factored forces and moments.

Design procedures differ for retaining systems built of reinforced soil [see Mitchell and Villet, 1987].

Example 22.2 — Cantilever Retaining Wall

Check the adequacy of the cantilever retaining wall shown in Fig. 22.7 regarding overturning, bearing capacity, and sliding. The allowable bearing pressure is 360 kPa.

Solution. (a) The overall dimensions of the wall appear to be appropriate (see Fig. 22.6).

(b) Estimate the forces acting on wall:

$$W_1 = \rho_c g A = (2.4\ \text{Mg/m}^3)(9.81\ \text{m/s}^2)(0.25\ \text{m} \cdot 8\ \text{m}) = 47.1\ \text{kN/m}$$
$$W_2 = (2.4\ \text{Mg/m}^3)(9.81\ \text{m/s}^2)(0.5 \cdot 0.45\ \text{m} \cdot 8\ \text{m}) = 42.4\ \text{kN/m}$$
$$W_3 = (2.4\ \text{Mg/m}^3)(9.81\ \text{m/s}^2)(0.6\ \text{m} \cdot 4.5\ \text{m}) = 63.6\ \text{kN/m}$$
$$W_4 = \rho_t g A \approx (1.9\ \text{Mg/m}^3)(9.81\ \text{m/s}^2)(8.5\ \text{m} \cdot 2.8\ \text{m}) = 444\ \text{kN/m}$$
$$W_5 = (1.9\ \text{Mg/m}^3)\ (9.81\ \text{m/s}^2)\ (0.9\ \text{m} \cdot 1.0\ \text{m}) = 16.8\ \text{kN/m}$$
$$W_T = \sum W_i = 614\ \text{kN/m}$$

Add 0.6 m of soil behind the wall to account for surcharge; hence, $H = 0.6\ \text{m} + 8\ \text{m} + 1\ \text{m} + 0.6\ \text{m} = 10.2\ \text{m}$. Conservatively assume $P_p \approx 0$. Use the log-spiral solution for the sloping backfill to estimate K_a (see Fig. 22.4).

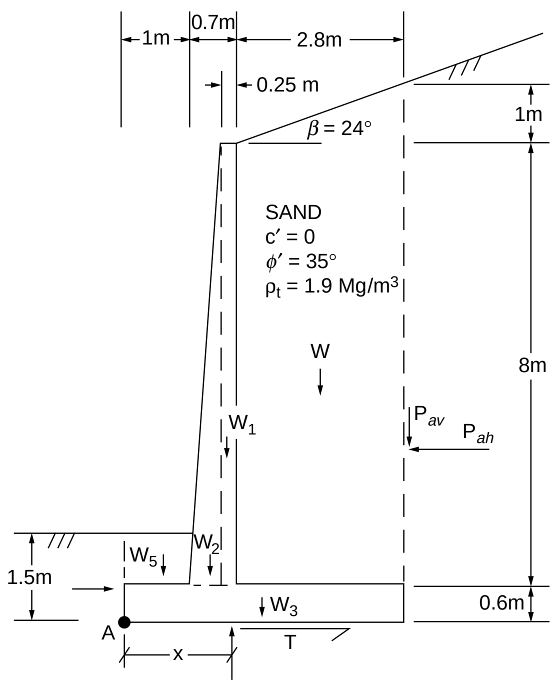


FIGURE 22.7 Example 22.2.

$\beta/\phi = 24°/35° \cong 0.7$ and $\phi' = 35°$ with $\delta = \phi' \rightarrow K_a = 0.38$
$P_a \;= K_a \gamma H^2/2 = (0.38)(1.9\text{ Mg/m}^3)(9.81\text{ m/s}^2)(10.2\text{ m})^2/2 = 368\text{ kN/m}$
$P_{ah} = P_a \cos\delta = (368\text{ kN/m})\cos 35° = 301\text{ kN/m}$
$P_{av} = P_a \sin\delta = (368\text{ kN/m})\sin 35° = 211\text{ kN/m}$
$N \;= W_T + P_{av} = 614\text{ kN/m} + 211\text{ kN/m} = 825\text{ kN/m}$
$T \;= N\tan\delta_b = (825\text{ kN/m})\tan 35° = 578\text{ kN/m}$

Location of resultant, N:

$$\begin{aligned}\sum M_A &= 0 = (47.1\text{ kN/m})(1.575\text{ m}) + (42.4\text{ kN/m})(1.3\text{ m})\\ &\quad + (63.6\text{ kN/m})(2.25\text{ m}) + (444\text{ kN/m})(3.1\text{ m})\\ &\quad + (16.8\text{ kN/m})(0.5\text{ m}) + (211\text{ kN/m})(4.5\text{ m})\\ &\quad - (301\text{ kN/m})(3.2\text{ m}) - (825\text{ kN/m})(x)\\ x &= 2.0\text{ m}\end{aligned}$$

(c) Check overturning:

$$B/3 = 4.5\text{ m}/3 = 1.5\text{ m} \qquad 2B/3 = 2\cdot 1.5\text{ m} = 3\text{ m}$$

$$1.5\text{ m} < 2.0\text{ m} < 3\text{ m} \quad \text{OK, since } N \text{ acts within middle third of base}$$

(d) Check bearing capacity:

$$e = \left|\frac{B}{2} - x\right| = \left|\frac{4.5\text{ m}}{2} - 2.0\text{ m}\right| = 0.25\text{ m}$$

$$P_{\max} = \frac{N}{B}\left(1 + \frac{6e}{B}\right) = \frac{825\text{ kN/m}}{4.5\text{ m}}\left(1 + \frac{6\cdot 0.25\text{ m}}{4.5\text{ m}}\right) = 245\text{ kPa}$$

$$P_{\max} = 245\text{ kPa} < q_a = 360\text{ kPa} \quad \text{OK}$$

(e) Check sliding:

$$\text{FS} = \frac{T}{P_{ah}} = \frac{578\text{ kN/m}}{301\text{ kN/m}} = 1.9 > 1.5 \quad \text{OK}$$

22.5 Flexible Retaining Structures

Flexible retaining structures include systems used in braced excavations, tie-back cuts, and anchored bulkheads. In this section, braced excavation systems will be discussed. Braced excavation support systems include walls, which may be steel sheetpiles, soldier piles with wood lagging, slurry placed tremie concrete, or secant/tangent piles; and supports, which may be cross-lot struts, rakers, diagonal bracing, tiebacks, or the earth itself in cantilever walls. Active earth pressure theories cannot be used directly to develop estimates of the lateral earth pressure acting on flexible retention structures. The pattern of wall movements during the excavation process does not satisfy Rankine-type assumptions of rigid wall translation or rigid wall rotation about its toe. With respect to the active Rankine state, the movement at the top of the wall is less and the movement at the base of the wall is more. Terzaghi [1943] showed that the resultant force on the flexible retaining structure is about 10% greater than the active Rankine resultant force and that the resultant force is located nearer to midheight of the wall rather than at its lower-third point. Theory is inadequate, since much depends on construction procedures, soil-structure interaction, and stress transfer. Moreover, more conservatism is desirable to guard against a progressive failure of the

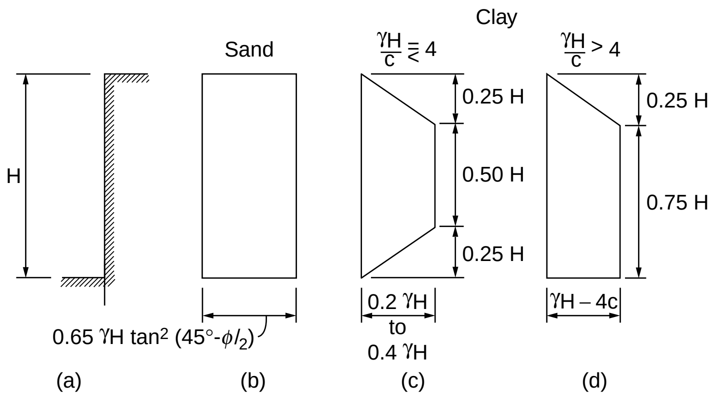


FIGURE 22.8 Lateral pressure distribution for computation of strut loads in braced excavation systems. (After Terzaghi, K., and Peck, R. 1967. *Soil Mechanics in Engineering Practice*. John Wiley & Sons, New York. Copyright © 1967 John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.)

support system. Consequently, **apparent lateral earth pressure** diagrams, which envelop the maximum strut loads measured for excavation systems (in specific subsurface conditions), are used.

Terzaghi and Peck [1967] have developed the apparent pressure diagrams shown in Fig. 22.8 for sand and clay sites [see Tschebotarioff, 1951 for other diagrams]. Note that for sand, the ratio of the resultant force due to the apparent pressure distribution shown in Fig. 22.8 to that due to active Rankine earth pressures is 1.3. The corresponding ratio for clay is about 1.7. Individual strut loads are computed based on the associated tributary area of the apparent pressure diagram. This is merely a reversal of the procedure used to develop the apparent pressure diagrams. The apparent pressure diagrams are based on field measurements of maximum strut loads, so normal surcharge loads are already included. Some engineers increase the strength of the upper struts by 15% to guard against surcharge overload.

The design wall and wale moments are typically calculated using the assumption that the wall and wale are simply supported between adjacent wales and struts, respectively. If the wall or wale is continuous over at least three supports, then the moment formula for a continuous beam can be used to calculate moments. Since the design of the wall and wale do not require the level of conservatism needed to guard against progressive failure of the struts, only two-thirds of the magnitude of the apparent pressure diagram is used in the computation. Hence, the maximum wall or wale moment, M_{max}, can be calculated by the following formula:

$$M_{max} = \frac{\frac{2}{3} \cdot \mathrm{AP} \cdot l_{max}^2}{(8 \text{ or } 10)} \tag{22.7}$$

where AP = apparent distributed load, l_{max} = maximum span length, and the denominator is 8 for the simply supported condition or 10 for the continuous beam condition. Excavations in deep soft to medium stiff clays may present situations in which the maximum wall moment occurs prior to installing the bottom strut, so wall moments should not just be computed for the final bracing configuration. Lastly, structural details are critically important in braced excavation systems. Typically, stiffeners are added to the wales at strut locations, and long internal struts are braced at points along their length.

The overall stability of the excavation must also be evaluated. Some potential failure mechanisms that should be investigated include **base heave**, bottom blowout, and piping. Calculating the factor of safety against base heave in deep clay deposits is especially important because a low safety factor indicates marginal stability and the potential for excessive movements (see Fig. 22.9). The engineer's primary concern in urban areas or where sensitive structures are near the excavation is often limiting ground movements.

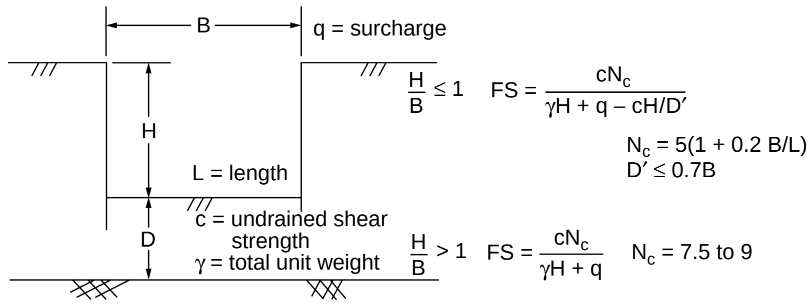


FIGURE 22.9 Factor of safety against base heave.

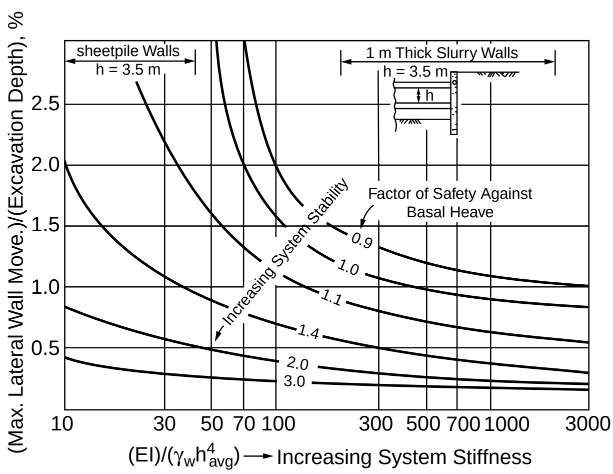


FIGURE 22.10 Design curves to obtain maximum lateral wall movement for soft to medium stiff clays. (After Clough, G.W., Smith, E.M., and Sweeny, B.P. 1989. Movement Control of Excavation Support Systems by Iterative Design. *Proc. ASCE Foundation Engineering: Current Principles and Practices,* 2: 869–884. Reproduced by permission of ASCE.)

Clough and O'Rourke [1990] present empirical data and analytical results that assist engineers in estimating excavation-induced ground movements. The maximum lateral wall movement, $\Delta_{h,m}$, in well-constructed excavations in stiff clays, residual soils, and sands is typically 0.1% to 0.5% of the height of the excavation, H, with most of the data suggesting $\Delta_{h,m} \approx 0.2\%H$. The stiffness of the support system is not critically important in those soil deposits. Conversely, the support system stiffness is important in controlling movements with excavations in soft to medium stiff clays. In soft to medium stiff clays, the maximum lateral wall movement can range from 0.3%H to over 3%H depending on the factor of safety against base heave and the support system stiffness (see Fig. 22.10). When the factor of safety against base heave is less than 1.5, special care should be exercised in controlling the excavation procedures to minimize movements. Preloading struts, not allowing overexcavation, and employing good construction details (e.g., steel shims) have proven useful in minimizing ground movements. Vertical movements of the ground surface, Δ_v, surrounding the excavation are largely a function of the lateral wall movements, and $\Delta_{v,m} \approx \Delta_{h,m}$. However, vertical ground movements can be much higher if excavation dewatering induces consolidation settlement in underlying clay deposits. Driving sheetpiles in loose sands can also induce significant ground settlement [see Clough and O'Rourke, 1990].

The design of tieback walls used in temporary excavations is similar to that described previously for braced excavations, but there are a number of significant differences [see Juran and Elias, 1991]. Tieback systems often permit less movement because they use higher preloads, positive connections, smaller support spacing, and less overexcavation. The tieback anchor itself, however, is more flexible than an internal strut, and its capacity depends greatly on the bond developed between the soil and grouted anchorage. Tieback systems consequently require good soil conditions and the absence of obstructions in the surrounding ground. Typically, the tieback system is checked against anchor pullout by proof testing each anchor to 100 to 150% of the design load, and the required lengths of the anchors are determined by ensuring their anchorage zones are located behind potential failure surfaces. Because preloading anchors to roughly 80% of their design load maintains a nearly at-rest stress state in the ground, lateral earth pressures are often assumed to be near at-rest pressures. Numerous other support systems have been developed and, depending on the availability of specialized contractors, these systems may be advantageous. For example, in good soils, soil nailing with a reinforced shotcrete wall has proved to be effective and cost-efficient.

Example 22.3 — Braced Excavation

For the braced excavation shown in Fig. 22.11(a), develop estimates of the strut loads, maximum wale moment, maximum wall moment, and maximum excavation-induced ground movements.

Solution. (a) Apparent pressures:

$$\text{Stability number } N = \frac{\gamma H}{c} = \frac{(2\ \text{Mg/m}^3)(9.81\ \text{m/s}^2)(8\ \text{m})}{35\ \text{kPa}} = 4.5$$

Since N is only slightly larger than 4, both Terzaghi and Peck [1967] clay apparent pressure diagrams should be calculated. The one with the largest resultant should be used (i.e., the left one in which R = 283 kN/m).

(b) Strut loads:

$$S1 = 1.15[\tfrac{1}{2}(47.1\ \text{kPa})(2\ \text{m}) + (47.1\ \text{kPa})(1.5\ \text{m})]6\ \text{m} = 810\ \text{kN}$$

$$S2 = [(47.1\ \text{kPa})(2.5\ \text{m}) + \tfrac{1}{2}(47.1\ \text{kPa} + 35.4\ \text{kPa})(0.5\ \text{m})]6\ \text{m} = 830\ \text{kN}$$

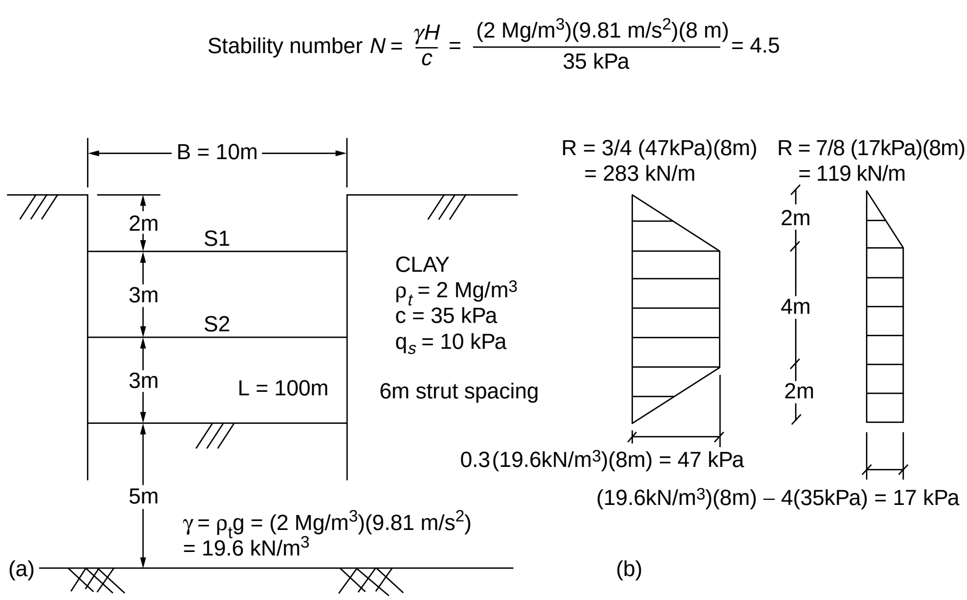


FIGURE 22.11 Example 22.3.

(c) Maximum wale moment:

$$M_{\max} = \frac{\left(\frac{2}{3}\,\text{AP}\right)l_{\max}^2}{10} = \frac{\left(\frac{2}{3}\cdot\frac{830\text{ kN}}{6\text{ m}}\right)(6\text{ m})^2}{10} = 330\text{ kN-m}$$

(d) Maximum wall moment:

$$M_{\max} = \frac{\left(\frac{2}{3}\,\text{AP}\right)l_{\max}^2}{8} = \frac{\left(\frac{2}{3}\cdot 47.1\text{ kPa}\right)(3\text{ m})^2}{8} = 35\text{ kN-m/m}$$

(e) Estimate ground movements:

$$N_c = 5(1 + 0.2B/L) = 5(1 + (0.2)(10\text{ m})/100\text{ m}) = 5.1$$

$$D' = 5\text{ m} \leq 0.7(10\text{ m}) = 7\text{ m}$$

$$\text{FS}_{BH} = \frac{cN_C}{\gamma H + q - cH/D'}$$

$$= \frac{(35\text{ kPa})(5.1)}{(2\text{ Mg/m}^3)(9.81\text{ m/s}^2)(8\text{ m}) + 10\text{ kPa} - (35\text{ kPa})(8\text{ m})/5\text{ m}} = 1.6$$

Using Fig. 22.10, for a typical sheetpile wall with $\text{FS}_{BH} \approx 1.6$, $\Delta_{h,m} \approx 1.0\%H$, or $\Delta_{h,m} \approx 0.01(8\text{ m}) = 0.08$ m = 8 cm.

In urban areas, maximum wall movements should normally be kept less than 5 cm. A heavy sheetpile wall (e.g., PZ 40) or a relatively stiff concrete slurry wall could be used to increase the system stiffness and reduce ground movements.

22.6 Summary

The analytical techniques presented in this section for retaining structures are based on simple models that have been empirically calibrated. Much depends on the method of construction of these systems and the quality of the workmanship involved. Hence, sound engineering judgment should be exercised and local experience in similar ground conditions is invaluable. Much can be gained by implementing an integrated approach that uniquely considers the project's subsurface conditions, site constraints, and excavation procedures [see Bray et al., 1993]. Finite element programs [e.g., SOILSTRUCT, Filz et al., 1990], which capture the unique soil-structure response of each excavation system, can provide salient insights and assist in identifying critical aspects of a particular project. The monitoring of field instrumentation (e.g., inclinometers) during excavation allows the engineer to verify the reasonableness of the analysis and to employ the observational method to optimize the design during construction.

Defining Terms

Apparent lateral earth pressure — Lateral earth pressure acting on tributary area of flexible retaining wall that is necessary to develop measured strut loads.

Base heave — Upward movement of base of excavation and associated inward movement of retaining wall due to bearing capacity-type instability of base soil.

Lateral earth pressure coefficient — Horizontal effective stress divided by vertical effective stress at a point.

Lateral earth pressure coefficient at rest — Lateral earth pressure coefficient when the lateral strain in the soil is zero. Realized for case of 1-D vertical compression (e.g., level ground).

Maximum passive earth pressure coefficient — Maximum value of the lateral earth pressure coefficient. Realized when soil compresses laterally and its full strength is mobilized.

Minimum active earth pressure coefficient — Minimum value of the lateral earth pressure coefficient. Realized when soil expands laterally and its full strength is mobilized.

Overconsolidation ratio — Maximum vertical effective stress in the past divided by the current vertical effective stress.

References

Barker, R. M., Duncan, J. M., Rojiani, V. B., Ooi, P. S. K., Tan, C. K., and Kim, S. G. 1991. *Manuals for the Design of Bridge Foundations*. National Cooperative Highway Research Program Report 343, TRB, Washington, D.C., December.

Bray, J. D., Deschamps, R. J., Parkison, R. S., and Augello, A. J. 1993. Braced excavation at the NIPSCO Bailly Station power plant. In *Proc. 3rd Int. Conf. Case Histories Geotech. Eng*., pp. 765–774. St. Louis, MO.

Clough, G. W., and Duncan, J. M. 1991. Earth pressures. In *Foundation Engineering Handbook,* ed. H-Y. Fang, pp. 224–235. Van Nostrand Reinhold, New York.

Clough, G. W., and O'Rourke, T. D. 1990. Construction induced movements of *in situ* walls. In *Proc. ASCE Design Performance Earth Retaining Struct.*, eds. P. C. Lambe and L. A. Hansen, pp. 439–470. Ithaca, NY, June 18–21.

Clough, G. W., Smith, E. M., and Sweeny, B. P. 1989. Movement control of excavation support systems by iterative design. *Proc. ASCE Found. Eng.: Curr. Principles Pract.*, 2:869–884.

Coulomb, C. A. 1776. Essai sur une application des règles des maximis et minimis à quelques problèmes be statique relatifs à l'architecture. *Mèm. Acad. Roy. des Sciences,* Paris. 3:38.

Department of the Navy. 1982. *Foundations and Earth Structures: Design Manual 7.2.* NAVFAC DM-7.2, May.

Duncan, J. M., and Seed, R. B. 1986. Compaction-induced earth pressure under K_0-conditions. *J. Geotech. Eng., ASCE*. 112(1):1–22.

Filz, G., Clough, G. W., and Duncan, J. M. 1990. Draft user's manual for program SOILSTRUCT (isotropic) plane strain with beam element. *Geotech. Eng. Rep*. Virginia Tech, Blacksburg, March.

Jaky, J. 1944. The coefficient of earth pressure at-rest. *J. Soc. Hungarian Architects Engineers*. Budapest, Hungary, pp. 355–358.

Juran, I., and Elias, V. 1991. Ground anchors and soil nails in retaining structures. In *Foundation Engineering Handbook,* ed. H-Y. Fang, pp. 868–905. Van Nostrand Reinhold, New York.

Kulhawy, F. H., and Mayne, P. W. 1990. Manual on Estimating Soil Properties for Foundation Design. *EL-6800, Research Project 1493-6 Final Report*. Electric Power Research Institute, Palo Alto, CA.

Lambe, P. C., and Hansen, L. A., eds. 1990. *Design and Performance of Earth Retaining Structures*. ASCE Geotechnical Special Publication No. 25, New York.

Mayne, P. W., and Kulhawy, F. H. 1982. K_0-OCR relationships in soil. *J. Geotech. Eng., ASCE*. 108(GT6):851–872.

Mitchell, J. K., and Villet, W. C. B. 1987. Reinforcement of Earth Slopes and Embankments. *National Cooperative Highway Research Program Report 290,* TRB, Washington, D.C., June.

Peck, R. B., Hanson, W. E., and Thornburn, T. H. 1974. *Foundation Engineering*. John Wiley & Sons, New York.

Rankine, W. J. M. 1857. On the Stability of Loose Earth. *Philos. Trans. R. Soc.,* London, 147(1):9–27.

Rowe, P. W. 1957. Limit design of flexible walls. *Proc. Midland Soil Mech. Found. Eng. Soc*. 1:29–40.

Schmidt, B. 1966. Discussion of "Earth pressures at-rest related to stress history," *Can. Geotech. J*. 3(4):239–242.

Terzaghi, K. 1943. *Theoretical Soil Mechanics*. John Wiley & Sons, New York.
Terzaghi, K., and Peck, R. 1967. *Soil Mechanics in Engineering Practice*. John Wiley & Sons, New York.
Tschebotarioff, G.P. 1951. *Soil Mechanics, Foundations and Earth Structures*. McGraw-Hill, New York.

Further Information

Foundation Engineering by Peck et al. [1974] provides a good overview of earth pressure theories and retaining structures, with illustrated design examples.

Retaining walls and abutments are discussed in Barker et al. [1991], and reinforcement of earth slopes and embankments are discussed by Mitchell and Villet [1987].

The design of anchored bulkheads is presented in Department of the Navy [1982].

State-of-the-art papers on the design and performance of earth retaining structures are presented in Lambe and Hansen [1990].