8.19 ULTIMATE RESISTANCE OF TIE BACKS

According to Figure 8.48, the ultimate resistance offered by a tie back in sand is

$$P_u = \pi d l \bar{\sigma}'_v K \tan \phi \tag{8.104}$$

where P_u = ultimate resistance
ϕ = angle of friction of soil
$\bar{\sigma}'_v$ = average effective vertical stress ($=\gamma z$ in dry sand)
K = earth pressure coefficient

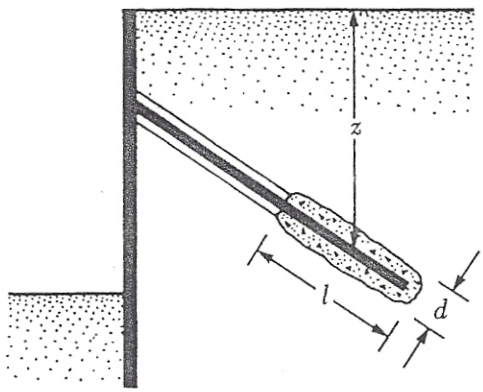
The magnitude of K can be taken to be equal to the earth pressure coefficient at rest (K_O) if the concrete grout is placed under pressure (Littlejohn, 1970). The lower limit of K can be taken to be equal to the Rankine active earth pressure coefficient.

In clays, the ultimate resistance of tie backs may be approximated as

$$P_u = \pi d l c_a \tag{8.105}$$

where c_a = adhesion

The value of c_a may be approximated as $\frac{2}{3}c_u$ (where c_u = undrained cohesion). A factor of safety of 1.5–2 may be used over the ultimate resistance to obtain the allowable resistance offered by each tie back.



▼ FIGURE 8.48 Parameters for defining the ultimate resistance of tie backs

BRACED CUTS

8.20 BRACED CUTS—GENERAL

Sometimes construction work requires ground excavations with vertical or near vertical faces — for example, basements of buildings in developed areas or under ground transportation facilities at shallow depths below the ground surface (cut and-cover type of construction). The vertical faces of the cuts need to be protected by temporary bracing systems to avoid failure that may be accompanied by considerable settlement or by bearing capacity failure of nearby foundations.

Figure 8.49 shows two types of braced cut commonly used in construction work. One type uses the *soldier beam* (Figure 8.49a), which is driven into the ground before excavation and is a vertical steel or timber beam. *Laggings,* which are horizontal timber planks, are placed between soldier beams as the excavation proceeds. When the excavation reaches the desired depth, *wales* and *struts* (horizontal steel beams) are installed. The struts are horizontal compression members. Figure 8.49b shows another type of braced excavation. In this case, interlocking *sheet piles* are driven into the soil before excavation. Wales and struts are inserted immediately after excavation reaches the appropriate depth.

To design braced excavations (that is, to select wales, struts, sheet piles, and soldier beams), an engineer must estimate the lateral earth pressure to which the braced cuts will be subjected.

The theoretical aspects of the lateral earth pressure on a braced cut were discussed in Section 6.8. The total active force per unit length of the wall (P_a) was calculated using the general wedge theory. However, that analysis does not provide the relationships for estimation of the variation of lateral pressure with depth, which is a function of several factors such as the type of soil, experience of the construction crew, type of construction equipment used, and so forth. For that reason, empirical pressure envelopes developed from field observations are used for the design of braced cuts. This procedure is discussed in the following section.

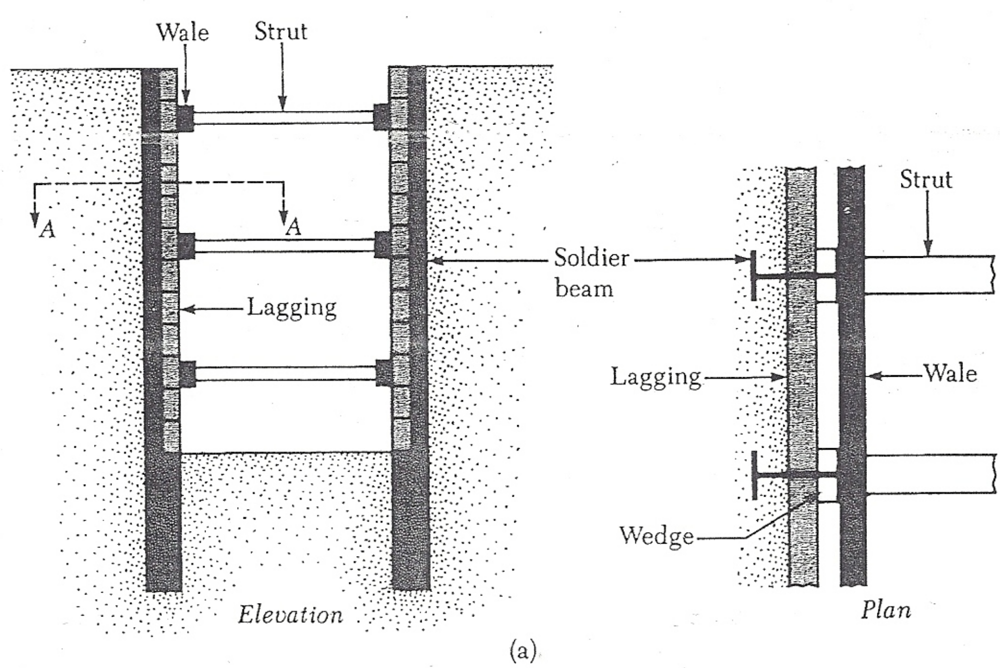
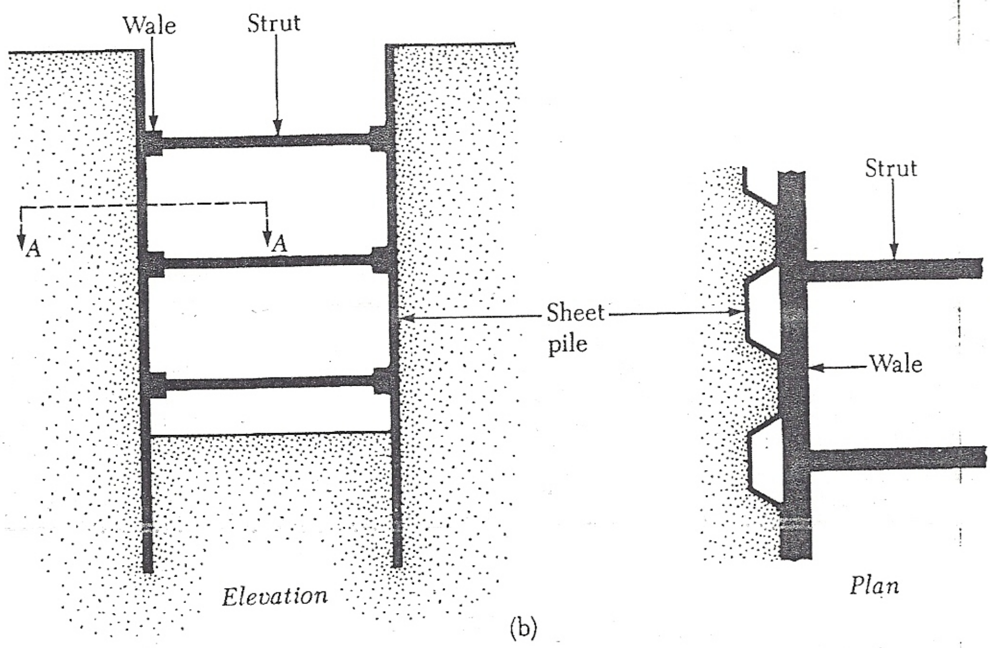


FIGURE 8.49 Types of braced cut: (a) use of soldier beams; (b) use of sheet piles



▼ FIGURE 8.49 (*Continued*)

8.21 PRESSURE ENVELOPE FOR BRACED-CUT DESIGN

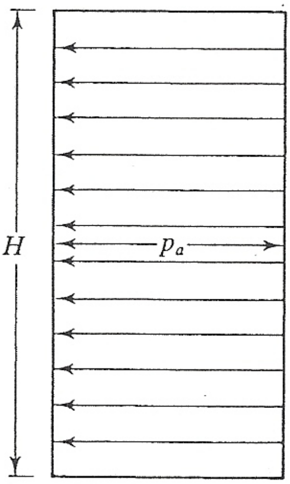
After observation of several braced cuts, Peck (1969) suggested using *design pressure envelopes* for braced cuts in sand and clay. Figures 8.50, 8.51, and 8.52 show Peck's pressure envelopes, to which the following guidelines apply.

Cuts in Sand

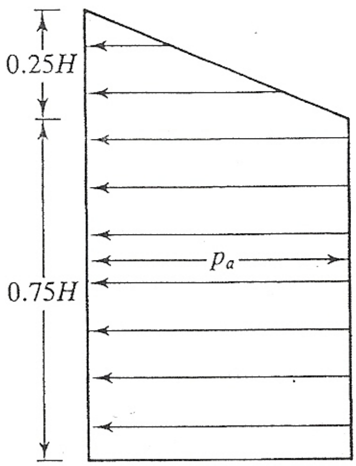
Figure 8.50 shows the pressure envelope for cuts in sand. This pressure, p_a, may be expressed as

$$\boxed{p_a = 0.65\gamma H K_a} \tag{8.106}$$

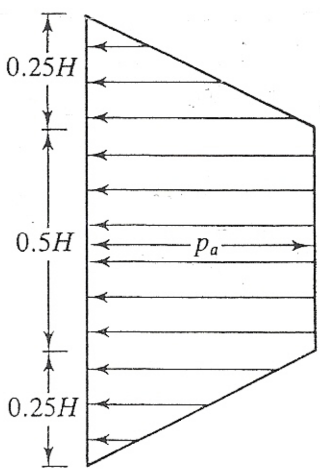
where γ = unit weight
H = height of the cut
K_a = Rankine active pressure coefficient = $\tan^2 (45 - \phi/2)$



▼ FIGURE 8.50 Peck's (1969) apparent pressure envelope for cuts in sand



▼ FIGURE 8.51 Peck's (1969) apparent pressure envelope for cuts in soft to medium clay



▼ FIGURE 8.52 Peck's (1969) apparent pressure envelope for cuts in stiff clay

Cuts in Soft and Medium Clay

The pressure envelope for soft to medium clay is shown in Figure 8.51. It is applicable for the condition

$$\frac{\gamma H}{c} > 4$$

where c = undrained cohesion ($\phi = 0$)

The pressure, p_a, is the larger of

$$\boxed{\begin{aligned} &p_a = \gamma H\left[1 - \left(\frac{4c}{\gamma H}\right)\right] \\ &\text{or} \\ &p = 0.3\gamma H \end{aligned}} \tag{8.107}$$

where γ = unit weight of clay

Cuts in Stiff Clay

The pressure envelope shown in Figure 8.52, in which

$$\boxed{p_a = 0.2\gamma H \text{ to } 0.4\gamma H \qquad \text{(with an average of } 0.3\gamma H)} \tag{8.108}$$

is applicable to the condition $\gamma H/c \le 4$.

Limitations of the Pressure Envelopes

When using the pressure envelopes just described, keep the following points in mind:

1. The pressure envelopes are sometimes referred to as *apparent pressure envelopes*. However, the actual pressure distribution is a function of the construction sequence and the relative flexibility of the wall.
2. They apply to excavations having depths greater than about 20 ft (≈6 m)
3. They are based on the assumption that the water table is below the botton of the cut.
4. Sand is assumed to be drained with zero pore water pressure.
5. Clay is assumed to be undrained and pore water pressure is not considered

Cuts in Layered Soil

Sometimes, layers of both sand and clay are encountered when a braced cut is being constructed. In this case, Peck (1943) proposed that an equivalent value of cohesion ($\phi = 0$ concept) should be determined in the following manner (refer to Figure 8.53a):

$$\boxed{c_{av} = \frac{1}{2H}[\gamma_s K_s H_s^2 \tan \phi_s + (H - H_s)n' q_u]} \tag{8.109}$$

where H = total height of the cut
γ_s = unit weight of sand
H_s = height of the sand layer
K_s = a lateral earth pressure coefficient for the sand layer (≈ 1)
ϕ_s = angle of friction of sand
q_u = unconfined compression strength of clay
n' = a coefficient of progressive failure (ranging from 0.5 to 1.0; average value 0.75)

The average unit weight, γ_a, of the layers may be expressed as

$$\boxed{\gamma_a = \frac{1}{H}[\gamma_s H_s + (H - H_s)\gamma_c]} \tag{8.110}$$

where γ_c = saturated unit weight of clay layer

Once the average values of cohesion and unit weight are determined, the pressure envelopes in clay can be used to design the cuts.

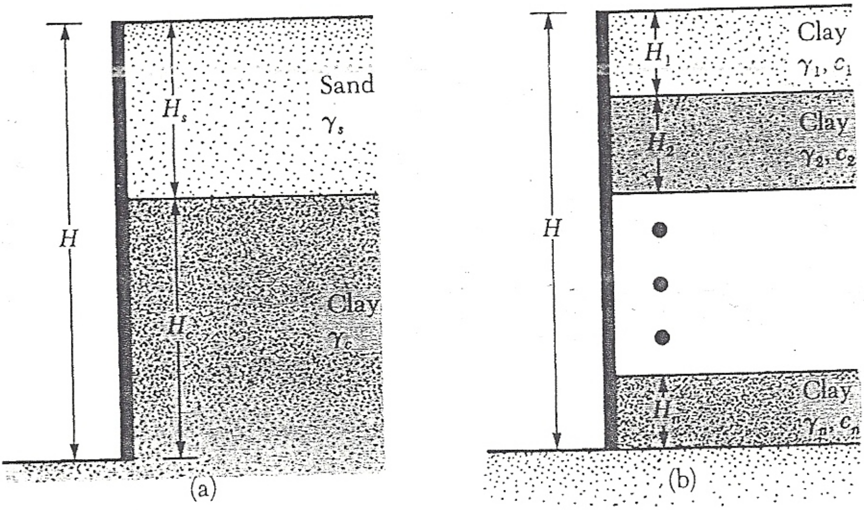


FIGURE 8.53 Layered soils in braced cuts

Similarly, when several clay layers are encountered in the cut (Figure 8.53b), the average undrained cohesion becomes

$$c_{av} = \frac{1}{H}(c_1H_1 + c_2H_2 + \cdots + c_nH_n) \tag{8.111}$$

where $c_1, c_2, \ldots, c_n$ = undrained cohesion in layers 1, 2, ..., n
$H_1, H_2, \ldots, H_n$ = thickness of layers 1, 2, ..., n

The average unit weight, γ_a, is

$$\gamma_a = \frac{1}{H}(\gamma_1H_1 + \gamma_2H_2 + \gamma_3H_3 + \cdots + \gamma_nH_n) \tag{8.112}$$

8.22 DESIGN OF VARIOUS COMPONENTS OF A BRACED CUT

Struts

In construction work, struts should have a minimum vertical spacing of about 9 ft (2.75 m) or more. The struts are actually horizontal columns subject to bending. The load-carrying capacity of columns depends on the *slenderness ratio*. The slenderness ratio can be reduced by providing vertical and horizontal supports at intermediate points. For wide cuts, splicing the struts may be necessary. For braced cuts in clayey soils, the depth of the first strut below the ground surface should be less than the depth of tensile crack, z_c. From Eq. (6.13),

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

where K_a = coefficient of Rankine active pressure

For determining the depth of tensile crack,

$$\sigma_a = 0 = \gamma z_c K_a - 2c\sqrt{K_a}$$

or

$$z_c = \frac{2c}{\sqrt{K_a}\gamma}$$

With $\phi = 0$, $K_a = \tan^2(45 - \phi/2) = 1$, so

$$z_c = \frac{2c}{\gamma}$$

A simplified conservative procedure may be used to determine the strut loads. Although this procedure will vary, depending on the engineers involved in the project, the following is a step-by-step outline of the general procedure (refer to Figure 8.54).

1. Draw the pressure envelope for the braced cut (see Figures 8.50, 8.51, and 8.52). Also show the proposed strut levels. Figure 8.54a shows a pressure envelope for a sandy soil; however, it could also be for a clay. The strut levels are marked A, B, C, and D. The sheet piles (or soldier beams) are assumed to be hinged at the strut levels, except for the top and bottom ones. In Figure 8.54a, the hinges are at the level of struts B and C. (Many designers also assume the sheet piles, or soldier beams, to be hinged at all strut levels, except for the top.)

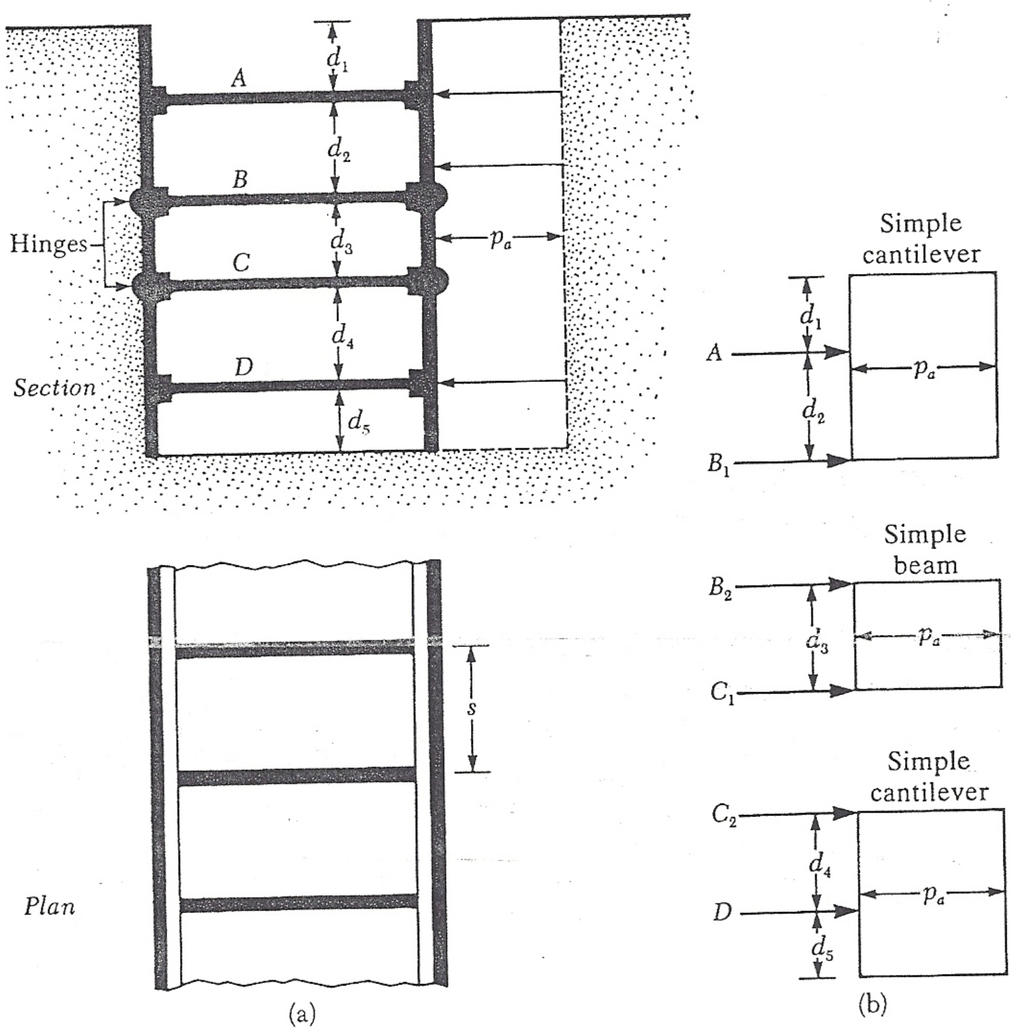


FIGURE 8.54 Determination of strut loads; (a) section and plan of the cut; (b) method for determining strut loads

2. Determine the reactions for the two simple cantilever beams (top and bottom) and all the simple beams between. In Figure 8.54b, these reactions are A, B_1, B_2, C_1, C_2, and D.
3. The strut loads in Figure 8.54 may be calculated as follows:

$$
\begin{aligned}
P_A &= (A)(s) \\
P_B &= (B_1 + B_2)(s) \\
P_C &= (C_1 + C_2)(s) \\
P_D &= (D)(s)
\end{aligned}
\tag{8.113}
$$

where P_A, P_B, P_C, P_D = loads to be taken by the individual struts at levels A, B, C, and D, respectively

A, B_1, B_2, C_1, C_2, D = reactions calculated in Step 2 (note unit: force/unit length of the braced cut)

s = horizontal spacing of the struts (see plan in Figure 8.54a)

4. Knowing the strut loads at each level and the intermediate bracing conditions allows selection of the proper sections from the steel construction manual.

Sheet Piles

The following steps are involved in designing the sheet piles:

1. For each of the sections shown in Figure 8.54b, determine the maximum bending moment.
2. Determine the maximum value of the maximum bending moments (M_{max}) obtained in Step 1. Note that the unit of this moment will be, for example, lb-ft/ft (kN · m/m) length of the wall.
3. Obtain the required section modulus of the sheet piles:

$$S = \frac{M_{max}}{\sigma_{all}} \tag{8.114}$$

where σ_{all} = allowable flexural stress of the sheet pile material

4. Choose a sheet pile having a section modulus greater than or equal to the required section modulus from a table such as Table C.1 (Appendix C).

Wales

Wales may be treated as continuous horizontal members if they are spliced properly Conservatively, they may also be treated as though they are pinned at the struts

For the section shown in Figure 8.54a, the maximum moments for the wales (assuming that they are pinned at the struts) are

$$\text{At level } A, \quad M_{max} = \frac{(A)(s^2)}{8}$$

$$\text{At level } B, \quad M_{max} = \frac{(B_1 + B_2)s^2}{8}$$

$$\text{At level } C, \quad M_{max} = \frac{(C_1 + C_2)s^2}{8}$$

$$\text{At level } D, \quad M_{max} = \frac{(D)(s^2)}{8}$$

where A, B_1, B_2, C_1, C_2, and D are the reactions under the struts per unit length of the wall (Step 2 of strut design)

Determine the section modulus of the wales:

$$S = \frac{M_{max}}{\sigma_{all}}$$

The wales are sometimes fastened to the sheet piles at points that satisfy the lateral support requirements.

BOTTOM HEAVING OF A CUT IN CLAY

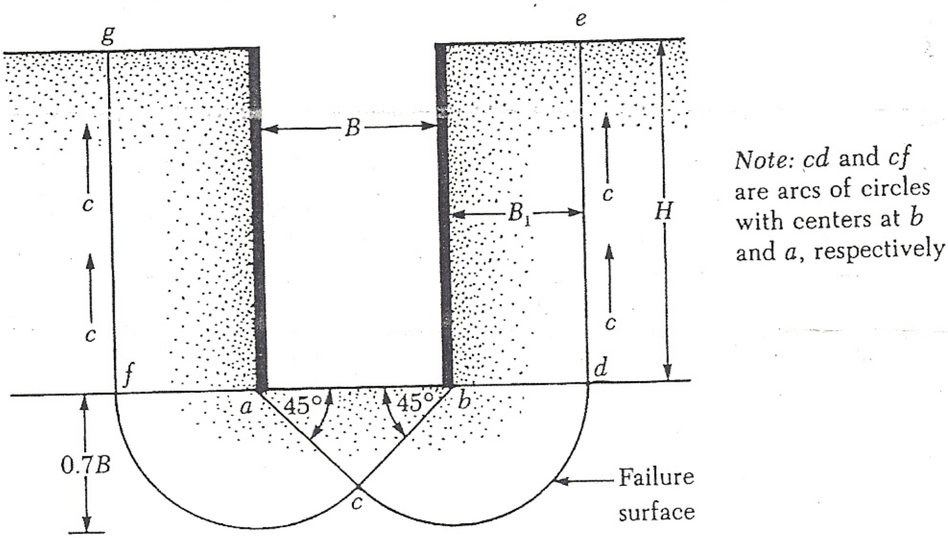
Braced cuts in clay may become unstable as a result of heaving of the bottom of the excavation. Terzaghi (1943) analyzed the factor of safety of braced excavations against bottom heave. The failure surface for such a case is shown in Figure 8.59. The vertical load (per unit length of the cut) at the bottom of the cut along line *bd* and *af* is

$$Q = \gamma H B_1 - cH \tag{8.115}$$

where $B_1 = 0.7B$
c = cohesion ($\phi = 0$ concept)

This load Q may be treated as a load per unit length on a continuous foundation at the level of *bd* (and *af*) and having a width of $B_1 = 0.7B$. Based on Terzaghi's bearing capacity theory, the net ultimate load-carrying capacity per unit length of this foundation (Chapter 3) is

$$Q_u = cN_cB_1 = 5.7cB_1$$



▼ FIGURE 8.59 Factor of safety against bottom heave

Hence from Eq. (8.115), the factor of safety against bottom heave is

$$\boxed{FS = \frac{Q_u}{Q} = \frac{5.7cB_1}{\gamma HB_1 - cH} = \frac{1}{H}\left(\frac{5.7c}{\gamma - \dfrac{c}{0.7B}}\right)} \tag{8.116}$$

This factor of safety is based on the assumption that the clay layer is homogeneous, at least to a depth of $0.7B$ below the bottom of the cut. However, a *hard layer of rock or rocklike material at a depth of $D < 0.7B$* will modify the failure surface to some extent. In such a case, the factor of safety becomes

$$\boxed{FS = \frac{1}{H}\left(\frac{5.7c}{\gamma - c/D}\right)} \tag{8.117}$$

Bjerrum and Eide (1956) also studied the problem of bottom heave for braced cuts in clay. For the factor of safety, they proposed:

$$\boxed{FS = \frac{cN_c}{\gamma H}} \tag{8.118}$$

The bearing capacity factor, N_c, varies with the ratios H/B and L/B (where L = length of the cut). For infinitely long cuts ($B/L = 0$), $N_c = 5.14$ at $H/B = 0$ and increases to $N_c = 7.6$ at $H/B = 4$. Beyond that — that is, for $H/B > 4$ — the value of N_c remains constant. For cuts square in plan ($B/L = 1$), $N_c = 6.3$ at $H/B = 0$, and $N_c = 9$ for $H/B \geq 4$. In general, for any H/B,

$$N_{c(\text{rectangle})} = N_{c(\text{square})}\left(0.84 + 0.16\frac{B}{L}\right) \tag{8.119}$$

Figure 8.60 shows the variation of the value of N_c for $L/B = 1$, 2, 3, and ∞.

When Eqs. (8.118) and (8.119) are combined, the factor of safety against heave becomes

$$\boxed{FS = \frac{cN_{c(\text{square})}\left(0.84 + 0.16\dfrac{B}{L}\right)}{\gamma H}} \tag{8.120}$$

Equation (8.120) and the variation of the bearing capacity factor, N_c, as shown in Figure 8.60 are based on the assumptions that the clay layer below the bottom of the cut is homogeneous and that the magnitude of the undrained cohesion in

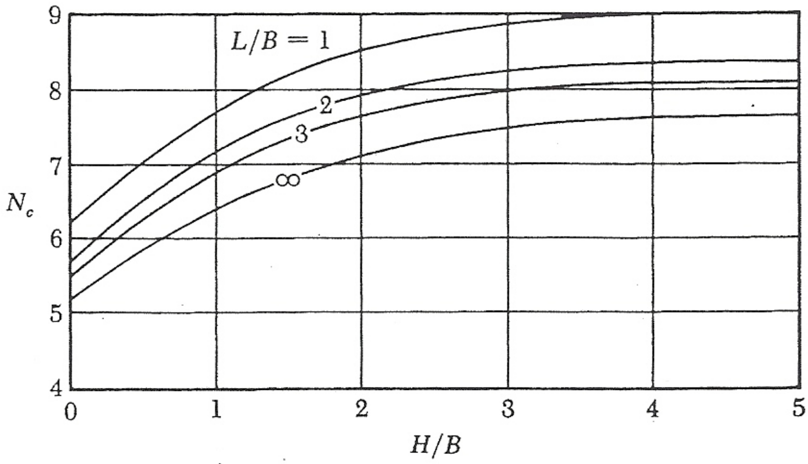


FIGURE 8.60 Variation of N_c with L/B and H/B [based on Bjerrum and Eide's equation, Eq. (8.119)]

the soil that contains the failure surface is equal to c (Figure 8.61). However, if a stronger clay layer is encountered at a shallow depth, as shown in Figure 8.62a, the failure surface below the cut will be controlled by the undrained cohesions c_1 and c_2. For this type of condition, the factor of safety is

$$\boxed{FS = \frac{c_1[N'_{c(\text{strip})} F_d] F_s}{\gamma H}} \tag{8.121}$$

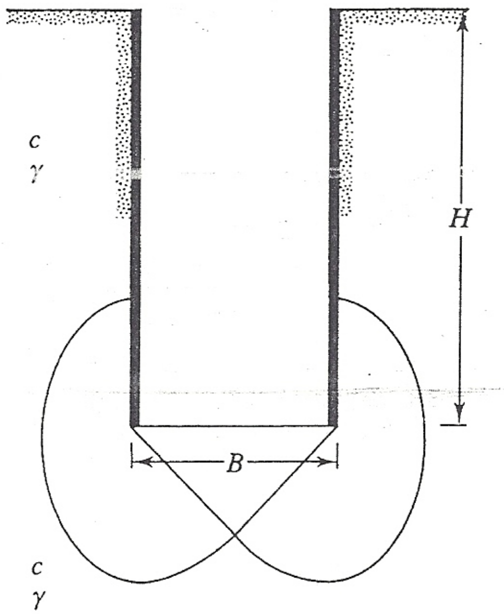
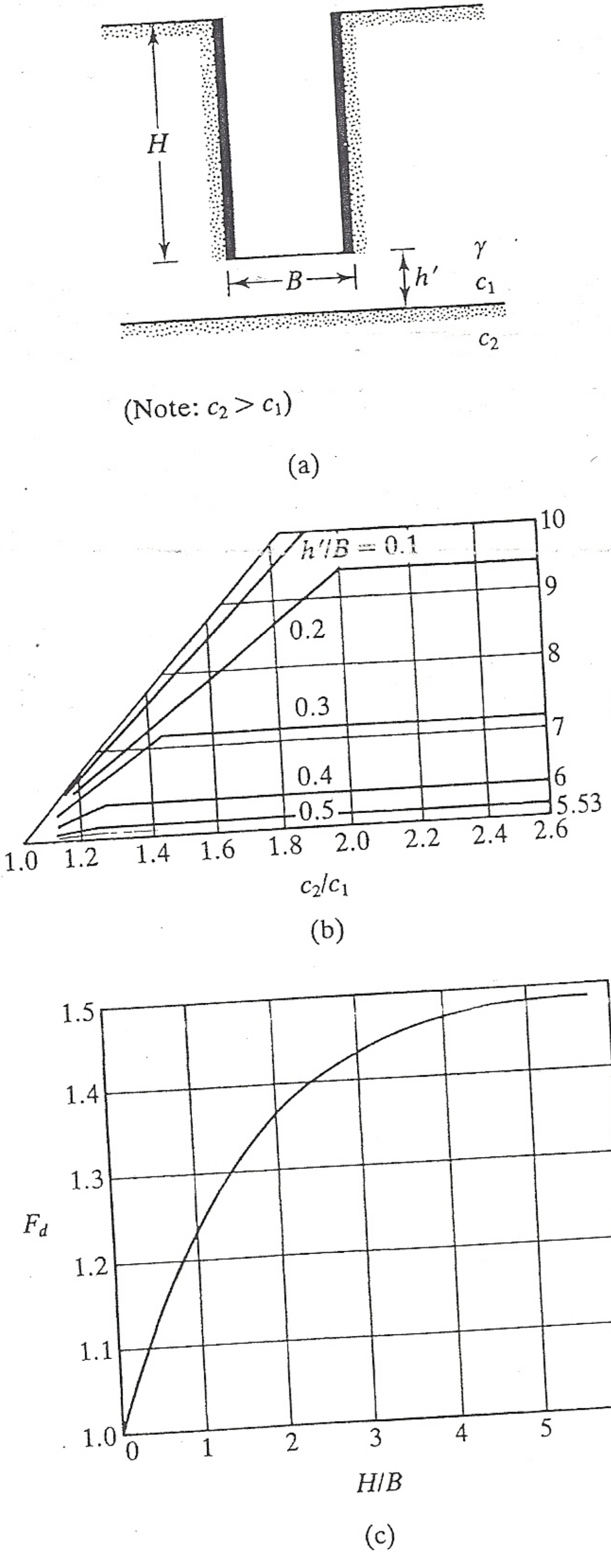


FIGURE 8.61 Derivation of Eq. (8.120)



▼ FIGURE 8.62 (a) Layered clay below the bottom of the cut; (b) variation of $N'_{c(\text{strip})}$ with c_2/c_1 and h'/B (redrawn after Reddy and Srinivasan, 1967); and (c) variation of F_d with H/B

where $N'_{c(\text{strip})}$ = bearing capacity factor of an infinitely long cut $(B/L = 0)$, which is a function of h'/B and c_2/c_1

F_d = depth factor, which is a function of H/B

F_s = shape factor

The variation of $N'_{c(\text{strip})}$ is shown in Figure 8.62b, and the variation of F_d as a function of H/B is given in Figure 8.62c. The shape factor, F_s, is

$$\boxed{F_s = 1 + 0.2\frac{B}{L}} \tag{8.122}$$

In most cases, a factor of safety of about 1.5 is generally recommended. If FS becomes less than about 1.5, the sheet pile is driven deeper (Figure 8.63). Usually the depth, d, is kept less than or equal to $B/2$. In that case, the force, P, per unit length of the buried sheet pile (aa' and bb') may be expressed as follows (U.S. Department of the Navy, 1971):

$$\boxed{P = 0.7(\gamma HB - 1.4cH - \pi cB) \qquad \text{for } d > 0.47B} \tag{8.123}$$

and

$$\boxed{P = 1.5d\left(\gamma H - \frac{1.4cH}{B} - \pi c\right) \qquad \text{for } d < 0.47B} \tag{8.124}$$

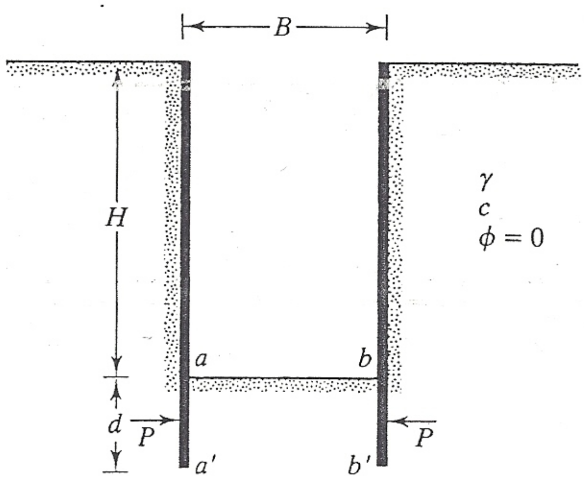


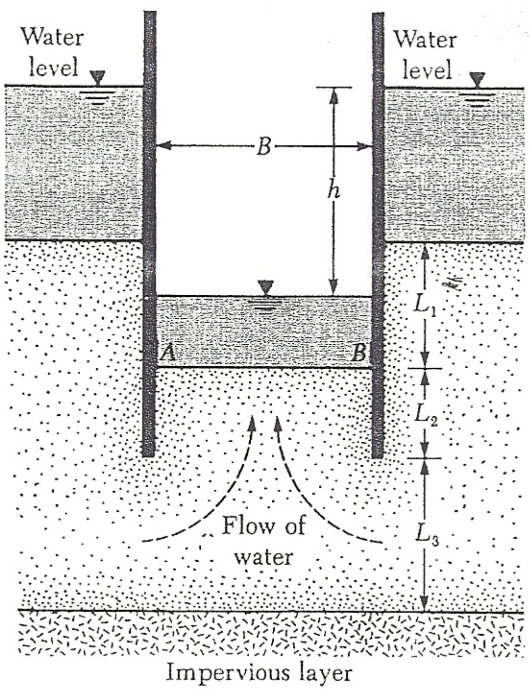
FIGURE 8.63 Force on the buried length of sheet pile

8.24 STABILITY OF THE BOTTOM OF A CUT IN SAND

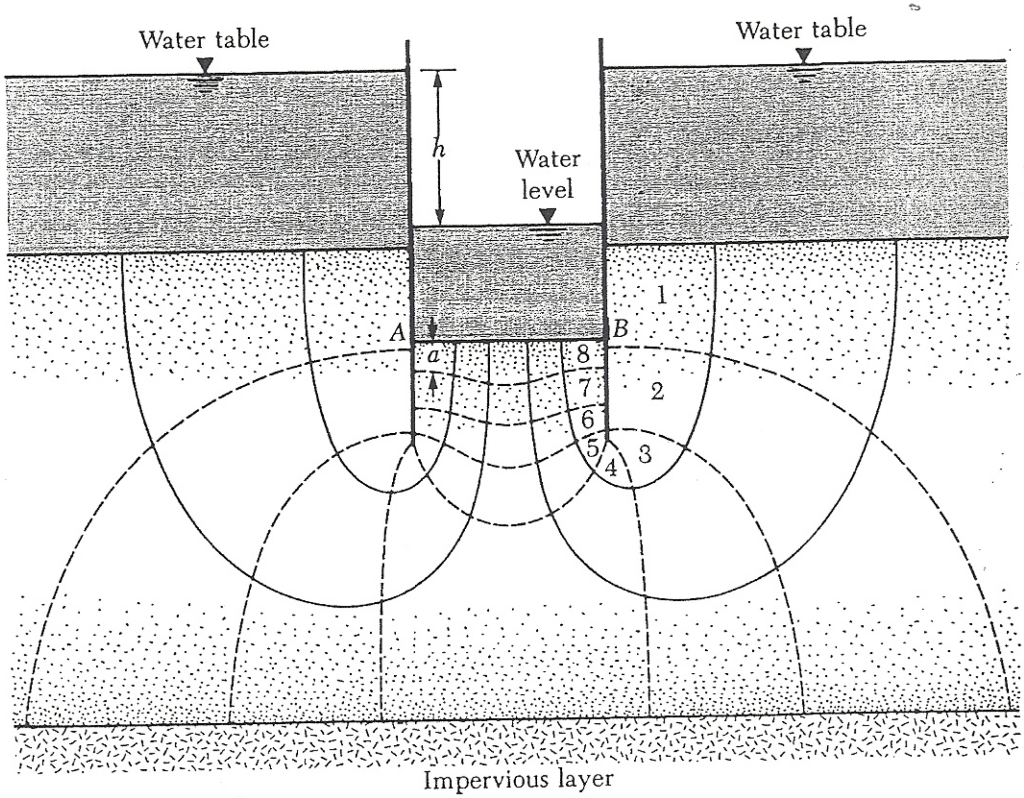
The bottom of a cut in sand is generally stable. When the water table is encountered, the bottom of the cut is stable as long as the water level inside the excavation is higher than the groundwater level. In case dewatering is needed (Figure 8.64), the factor of safety against piping should be checked. [*Piping* is another term for failure by heave, as defined in Section 1.11; see Eq. (1.51)]. Piping may occur when a high hydraulic gradient is created by water flowing into the excavation. To check the factor of safety, draw flow nets and determine the maximum exit gradient [$i_{max(exit)}$] that will occur at points A and B. Figure 8.65 shows such a flow net, for which the maximum exit gradient is

$$i_{max(exit)} = \frac{\frac{h}{N_d}}{a} = \frac{h}{N_d a} \tag{8.125}$$

where a = length of the flow element at A (or B)
N_d = number of drops (*note:* in Figure 8.65, $N_d = 8$ — also see Section 1.9)



▼ FIGURE 8.64



▼ FIGURE 8.65 Determining the factor of safety against piping by drawing flow net

The factor of safety against piping may be expressed as

$$\boxed{FS = \frac{i_{cr}}{i_{max(exit)}}} \tag{8.126}$$

where i_{cr} = critical hydraulic gradient

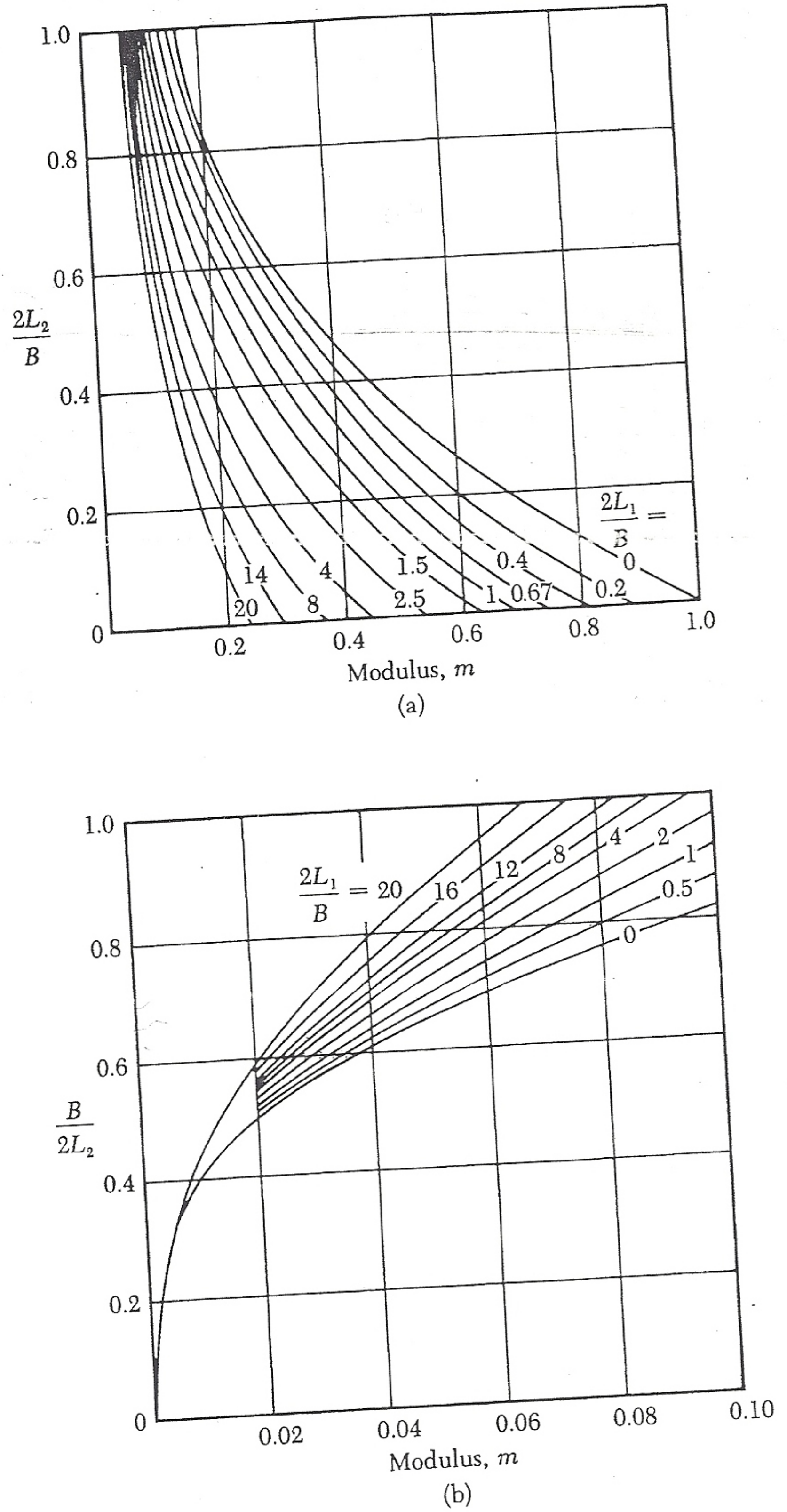
The relationship for i_{cr} was given in Chapter 1 as

$$i_{cr} = \frac{G_s - 1}{e + 1}$$

The magnitude of i_{cr} varies between 0.9 and 1.1 in most soils, with an average of about 1. A factor of safety of about 1.5 is desirable.

The maximum exit gradient for sheeted excavations in sands with $L_3 = \infty$ can also be evaluated theoretically (Harr, 1962). (Only the results of these mathematical derivations will be presented here. For further details, refer to the original work.) To calculate the maximum exit gradient, refer to Figures 8.66 and 8.67 and perform the following steps.

1. Determine the modulus, m, from Figure 8.66 by obtaining $2L_2/B$ (or $B/2L_2$) and $2L_1/B$.



▼ FIGURE 8.66 Variation of modulus (from *Groundwater and Seepage*, by M. E. Harr. Copyright © 1962 by McGraw-Hill. Used with permission.)

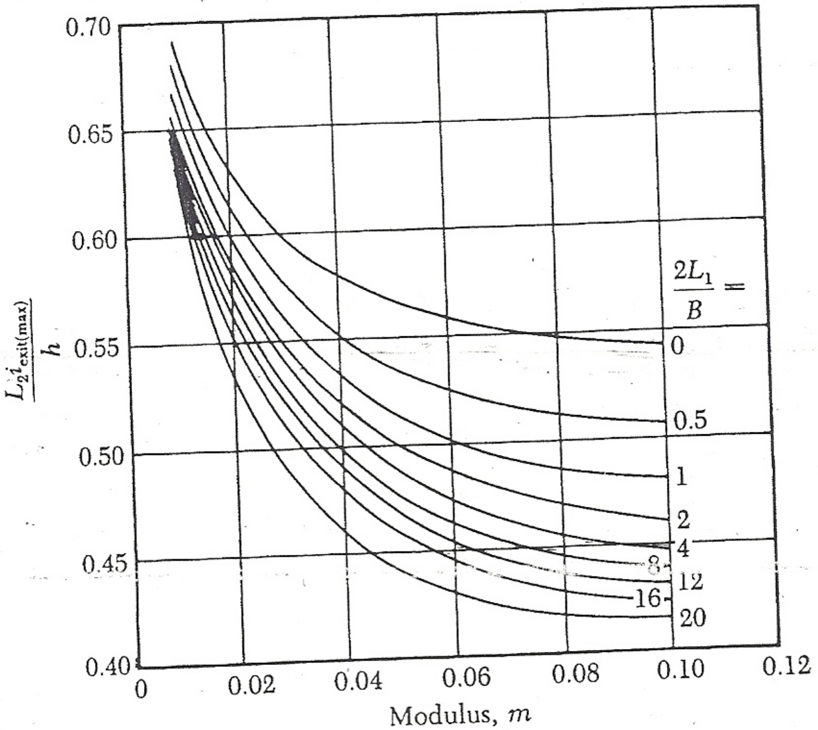
2. With the known modulus and $2L_1/B$, refer to Figure 8.67 and determine $L_2 i_{exit(max)}/h$. Because L_2 and h will be known, $i_{exit(max)}$ can be calculated.
3. The factor of safety against piping can be evaluated by using Eq. (8.126).

Marsland (1958) presented the results of model tests conducted to study the influence of seepage on the stability of sheeted excavations in sand. They were summarized by the U.S. Department of the Navy (1971) in *NAVFAC DM-7* and are given in Figure 8.68a, b, and c. Note that Figure 8.68b is for the case of determining the sheet pile penetration (L_2) needed for the required factor of safety against piping when the sand layer extends to a great depth below the excavation. However, Figure 8.68c represents the case in which an impervious layer lies at depth $L_2 + L_3$ below the bottom of the excavation.

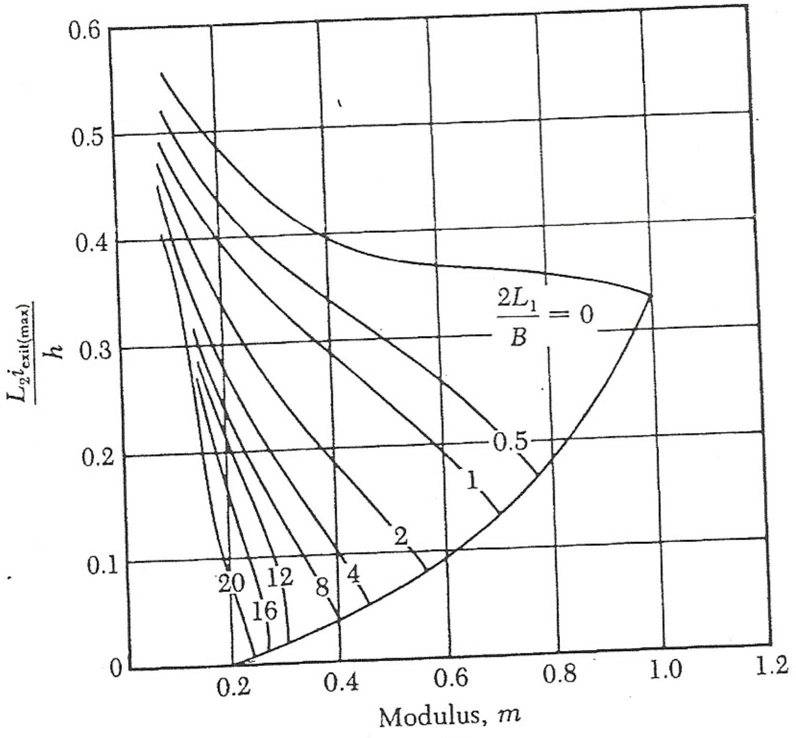
8.25

LATERAL YIELDING OF SHEET PILES AND GROUND SETTLEMENT

In braced cuts, some lateral movement of sheet pile walls may be expected (Figure 8.69, p. 845). The amount of lateral yield depends on several factors, the most important of which is the elapsed time between excavation and placement of wales and struts. Mana and Clough (1981) analyzed the field records of several braced cuts in clay from the San Francisco, Oslo (Norway), Boston, Chicago, and Bowline Point (New York) areas. Under ordinary construction conditions, they found that the maximum lateral wall yield, $\delta_{H(\max)}$, has a definite relationship with the factor of



(a)



(b)

▼ FIGURE 8.67 Variation of maximum exit gradient with modulus (from *Groundwater and Seepage*, by M. E. Harr. Copyright © 1962 by McGraw-Hill. Used with permission.)

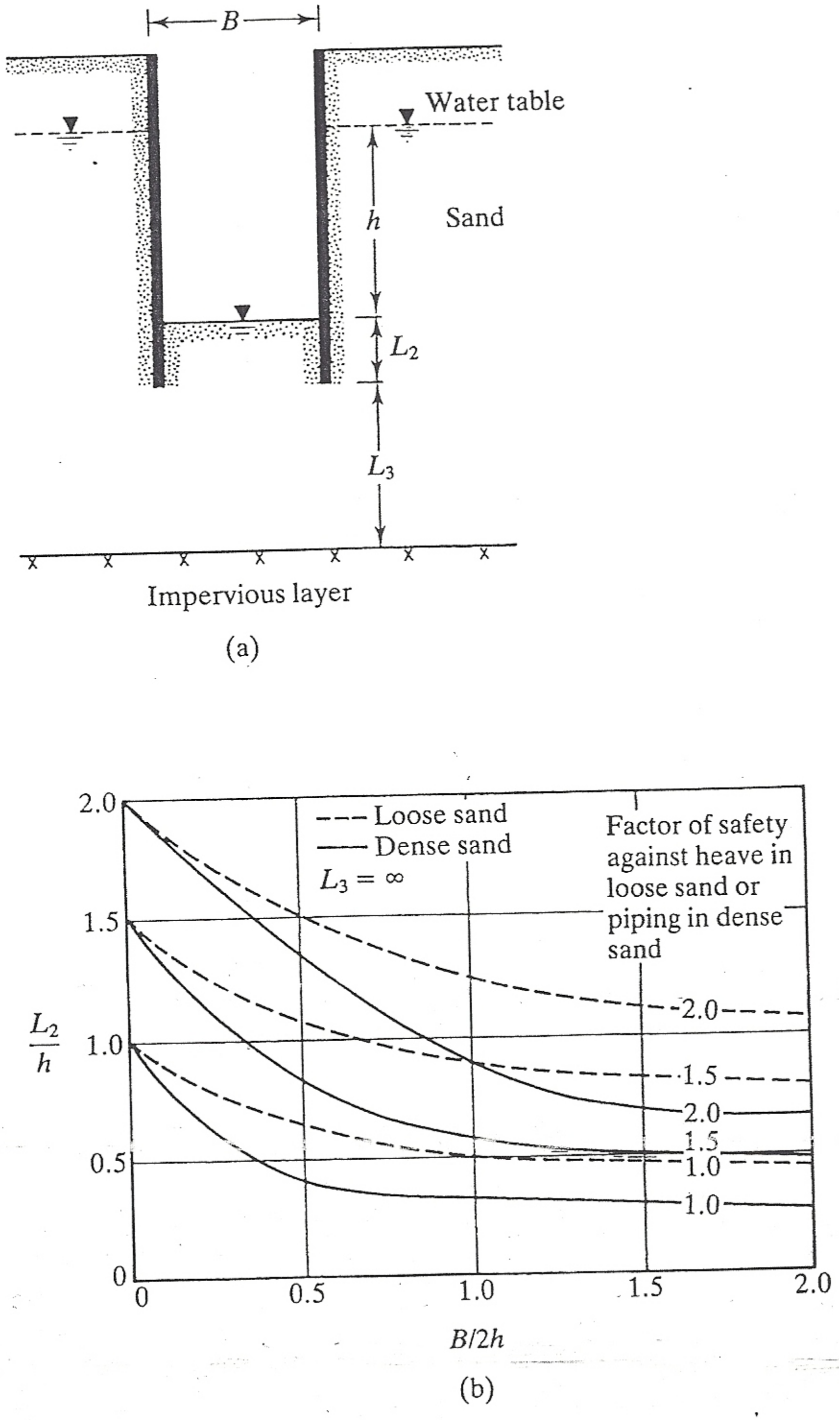
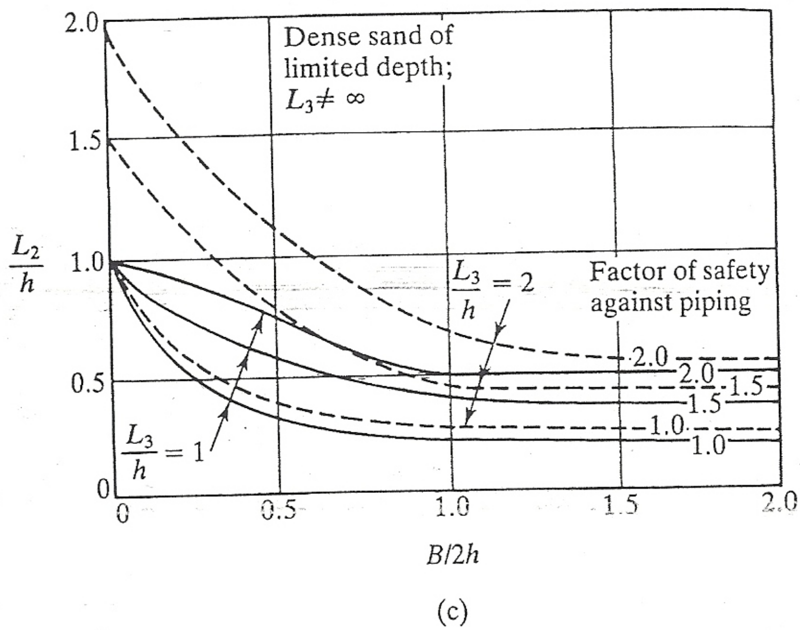
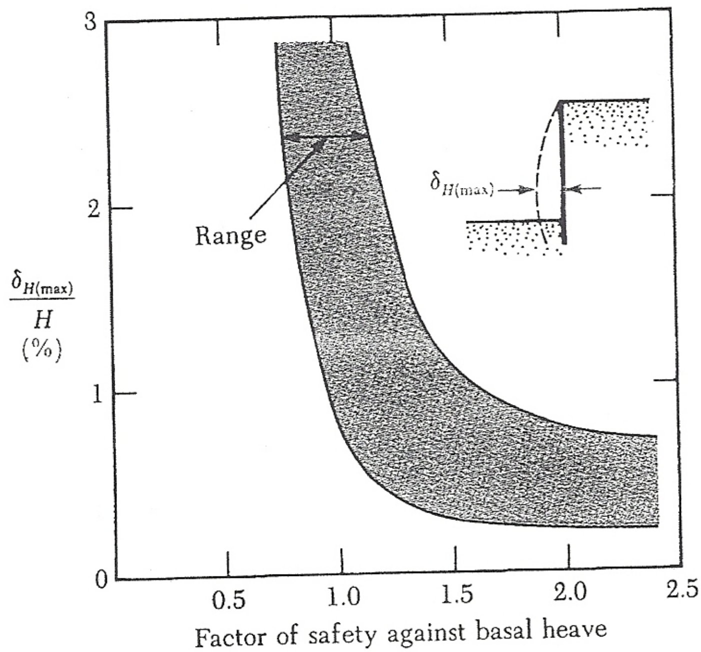


FIGURE 8.68 Influence of seepage on the stability of sheeted excavation (after U.S. Department of the Navy, 1971)



▼ FIGURE 8.68 (Continued)



▼ FIGURE 8.69 Range of variation of $\delta_{H(max)}/H$ with FS against basal heave from field observation (redrawn after Mana and Clough, 1981) safety against heave, as shown in Figure 8.69. Note that the factor of safety against heave plotted in Figure 8.69 was calculated by using Eqs. (8.116) and (8.117).

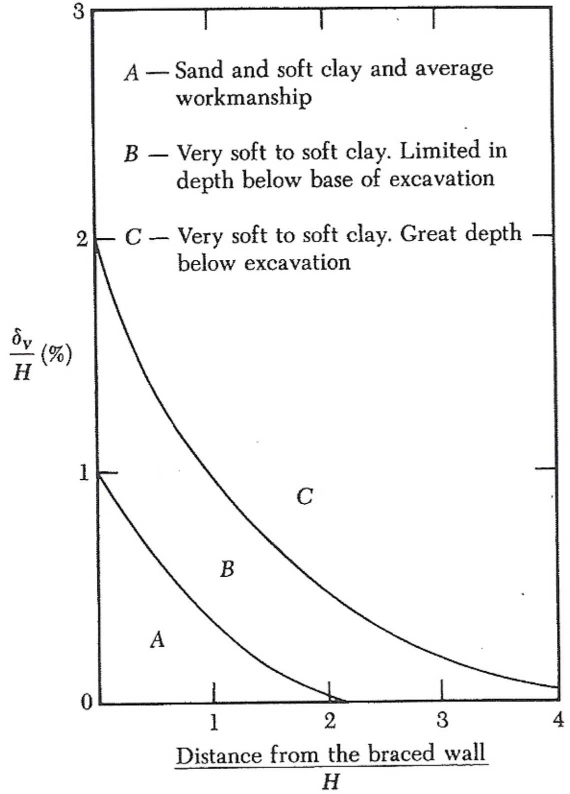
As discussed before, in several instances the sheet piles (or the soldier piles, as the case may be) are driven to a certain depth below the bottom of the excavation. The reason is to reduce the lateral yielding of the walls during the last stages of excavation. Lateral yielding of the walls will cause the ground surface surrounding the cut to settle. The degree of lateral yielding, however, depends mostly on the soil type below the bottom of the cut. If clay below the cut extends to a great depth and $\gamma H/c$ is less than about 6, extension of the sheet piles or soldier piles below the bottom of the cut will help considerably in reducing the lateral yield of the walls.

However, under similar circumstances, if $\gamma H/c$ is about 8, the extension of sheet piles into the clay below the cut does not help greatly. In such circumstances, we may expect a great degree of wall yielding that may result in the total collapse of the bracing systems. If a hard soil layer lies below a clay layer at the bottom of the cut, the piles should be embedded in the stiffer layer. This action will greatly reduce lateral yield.

The lateral yielding of walls will generally induce ground settlement, δ_V, around a braced cut, which is generally referred to as *ground loss*. Based on several field observations, Peck (1969) provided curves for predicting ground settlement in various types of soil (see Figure 8.70). The magnitude of ground loss varies extensively; however, Figure 8.70 may be used as a general guide.

Based on the field data obtained from various cuts in the areas of San Francisco, Oslo, and Chicago, Mana and Clough (1981) provided a correlation between the maximum lateral yield of sheet piles, $\delta_{H(\max)}$, and the maximum ground settlement, $\delta_{V(\max)}$. It is shown in Figure 8.71. Note that

$$\delta_{V(\max)} \approx 0.5\delta_{H(\max)} \quad \text{to} \quad 1.0\delta_{H(\max)} \tag{8.127}$$



▼ **FIGURE 8.70** Variation of ground settlement with distance (after Peck, 1969)