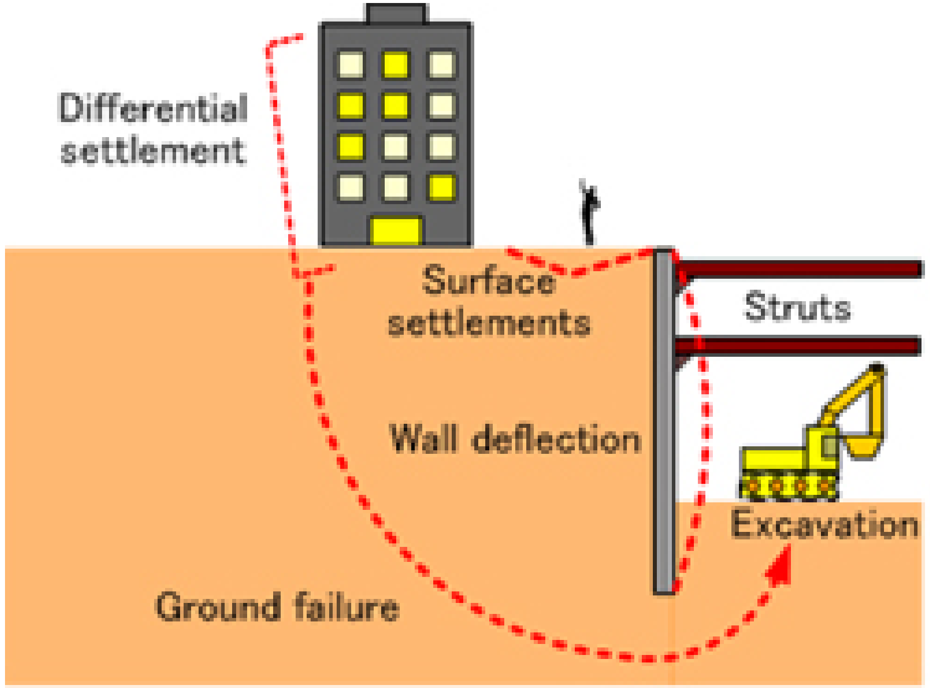
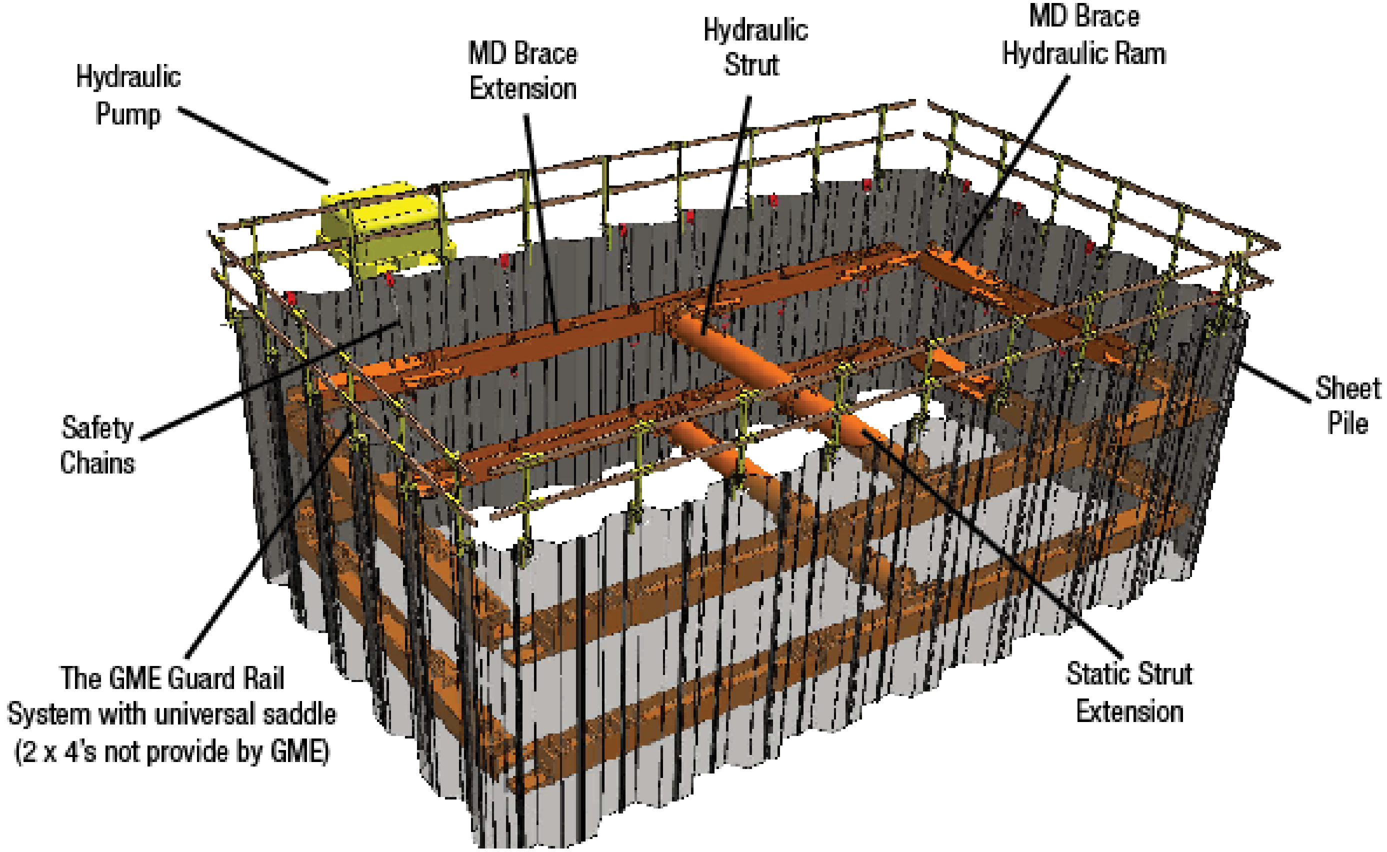
BRACED CUTS
GROUND ANCHORS

5560B



Braced excavation near existing building


Hydraulic Pump
MD Brace Extension
Hydraulic Strut
MD Brace Hydraulic Ram
Sheet Pile
Safety Chains
The GME Guard Rail System with universal saddle (2 x 4's not provide by GME)
Static Strut Extension

What is a braced cut?

Sometimes construction work requires ground excavations with vertical or near-vertical faces—for example, basements of buildings in developed areas or underground transportation facilities at shallow depths below the ground surface(a cut-and-cover type of construction).

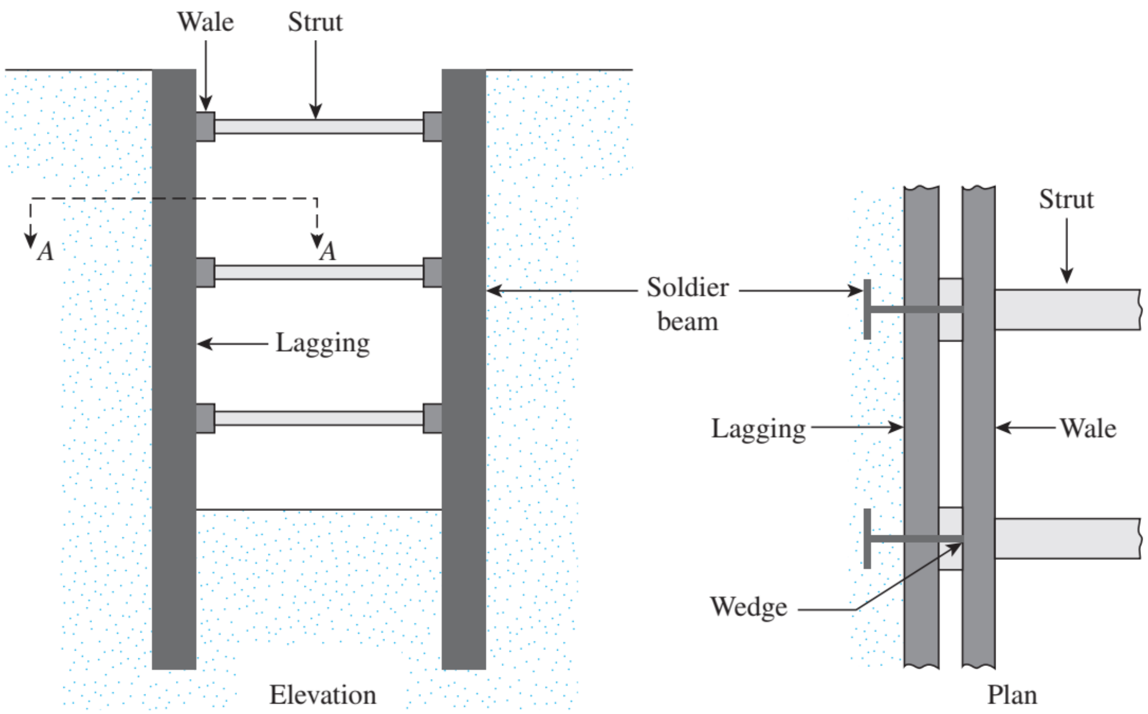
The vertical faces of the cuts need to be protected by temporary bracing systems to avoid failure that may be accompanied by considerable settlement or by bearing capacity failure of nearby foundations.

Two types of braced cut commonly used in construction work.

1) Soldier beam

2) Interlocking sheet piles

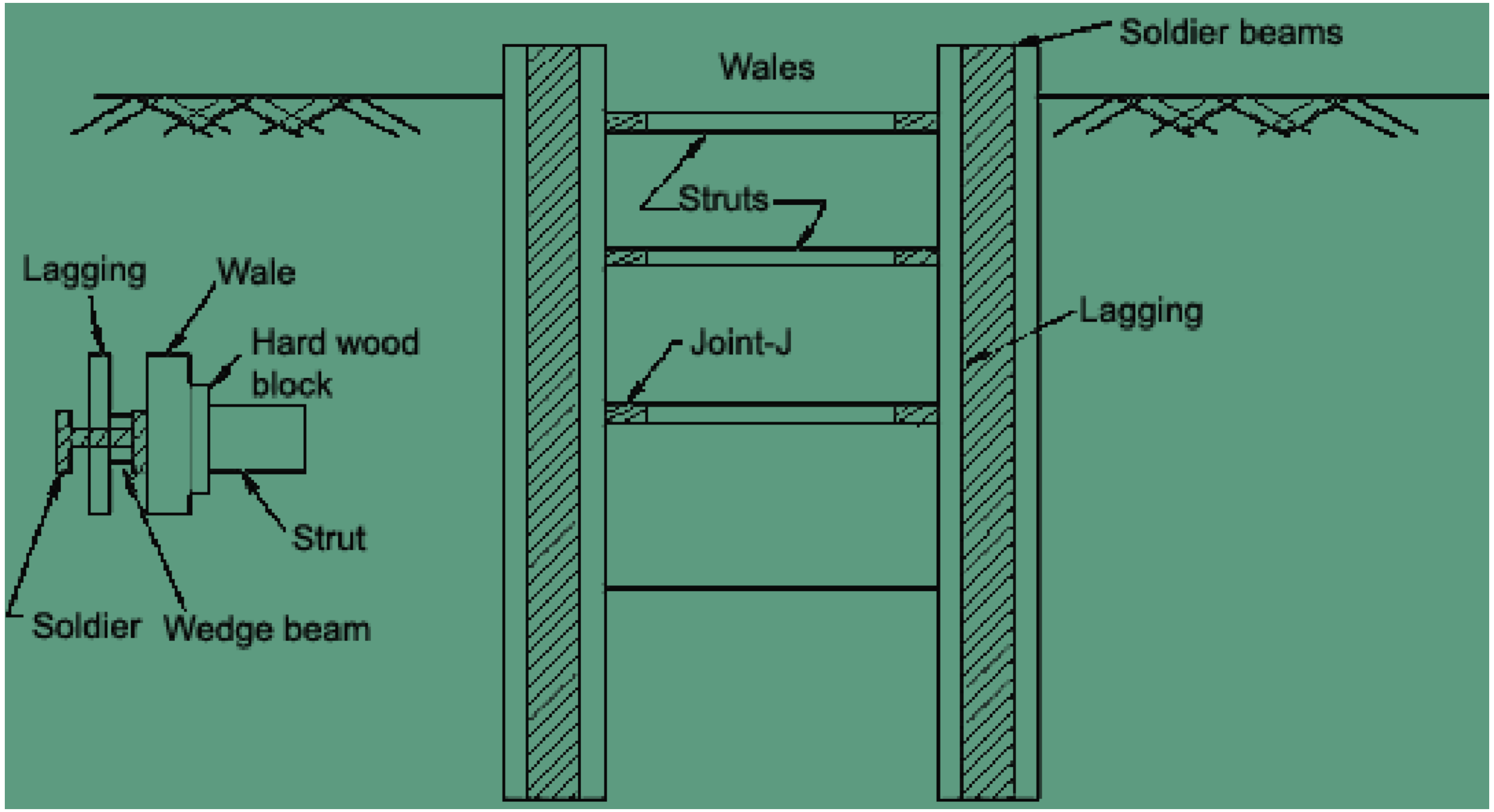
Soldier Beams



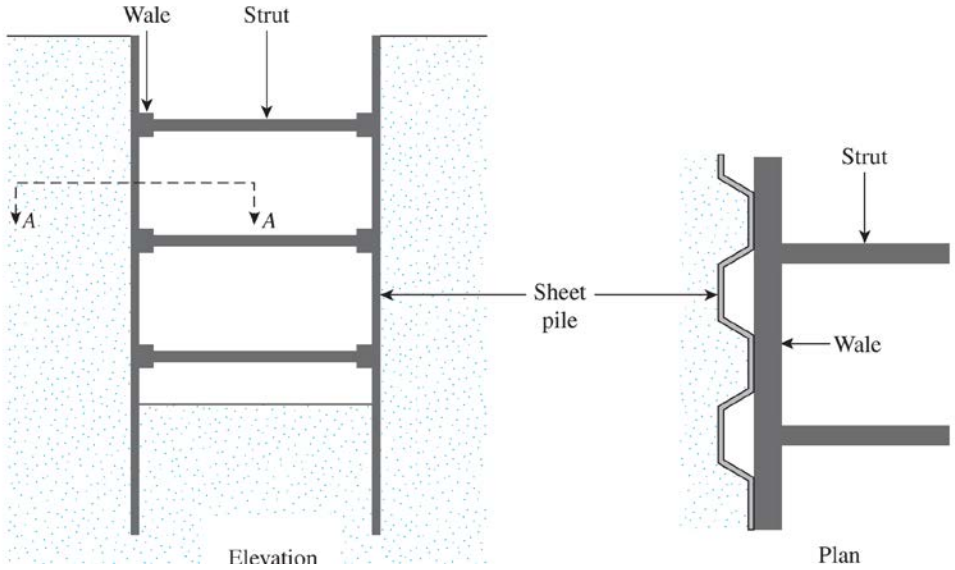
The soldier beam is driven into the ground before excavation and is a vertical steel or timber beam.

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 compression members.

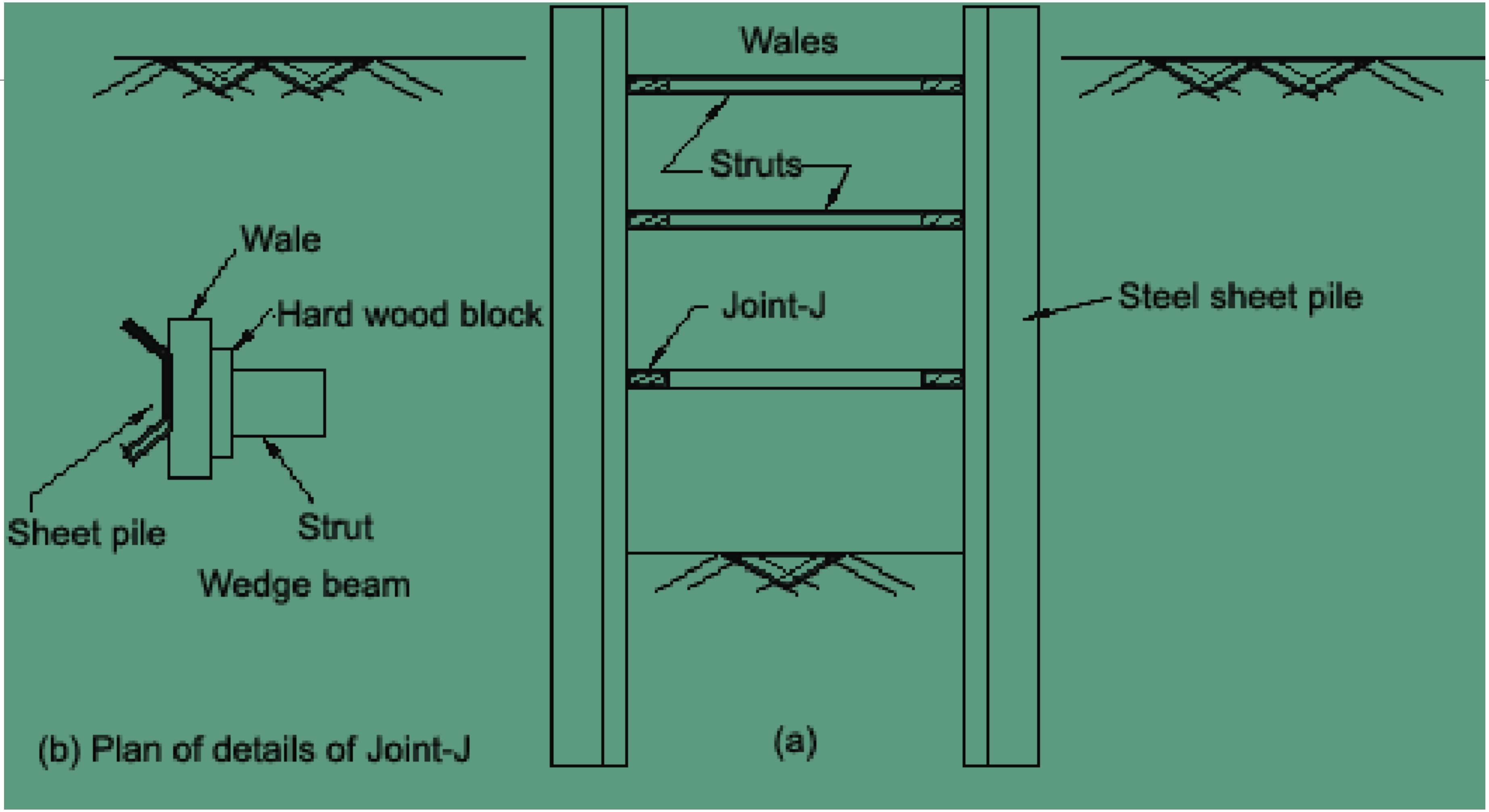

Soldier beams
Wales
Struts
Lagging
Wale
Hard wood block
Joint-J
Lagging
Strut
Soldier
Wedge beam

Interlocking sheet piles



Interlocking sheet piles are driven into the soil before excavation.

Wales and struts are inserted immediately after excavation reaches the appropriate depth.


Wales
Struts
Wale
Hard wood block
Joint-J
Steel sheet pile
Sheet pile
Strut
Wedge beam
(b) Plan of details of Joint-J
(a)

Design of Braced Excavation

To design braced excavations (i.e., 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.

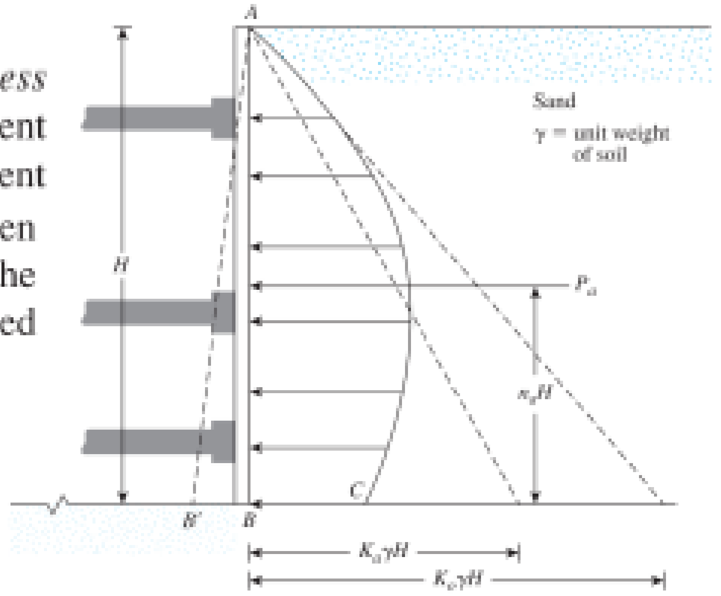
The total active force per unit length of the wall (Pa) can be calculated by using the general wedge theory.

However, that analysis will not provide the relationships required for estimating the variation of lateral pressure with depth, which is a function of several factors, such as the type of soil, the experience of the construction crew, the 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.

Braced Cut Analysis Based on General Wedge Theory

Figure 15.4 shows a braced cut of height H. Let us assume that AB is a *frictionless* wall retaining a granular soil. During the excavation process followed by the placement of struts, the upper portion of the soil mass next to the cut does not undergo sufficient lateral deformation. However, as the depth of excavation increases, the time lag between the excavation and placement of struts increases, also resulting in a gradual increase in the lateral deformation of wall AB. Ideally, at the end of excavation, wall AB will be deformed to the shape AB'.

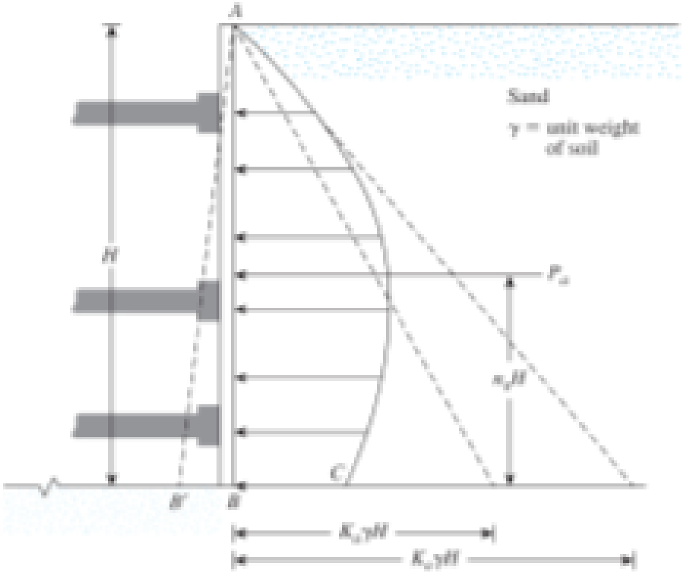


Note: K_o = earth pressure coefficient at-rest
K_a = Rankine active earth pressure coefficient

Braced Cut Analysis Based on General Wedge Theory

The lateral earth-pressure distribution along wall *AB* will be of the nature shown in Figure 15.4. It is important to note the following:

- The wall *AB* rotates about *A* (i.e., rotation about the top).
- At *A*, the lateral earth pressure will be close to the at-rest earth pressure (practically no lateral deformation of the wall).
- At *B*, the lateral earth pressure may be less than the Rankine active earth pressure. (The deformation of the wall is large, and the soil may be in a state well past the plastic equilibrium.)
- Hence, the lateral earth-pressure diagram will approximate to the form *ACB*, as shown in Figure 15.4.

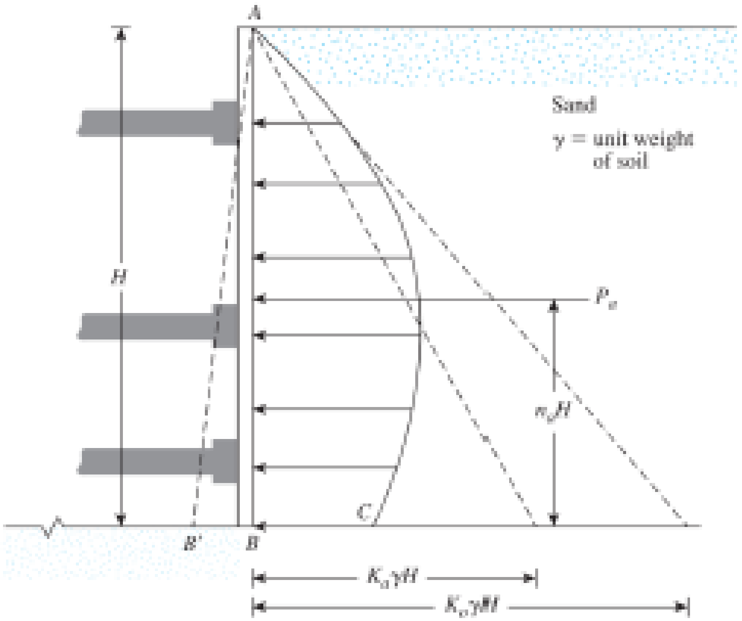


Note: K_o = earth pressure coefficient at-rest
K_a = Rankine active earth pressure coefficient

Figure 15.4

Braced Cut Analysis Based on General Wedge Theory

With this type of pressure distribution, the point of application of the resultant active thrust, P_a, will be at a height $n_a H$ measured from the bottom of the wall. The magnitude of n_a will be greater than 1/3.



Note: K_o = earth pressure coefficient at-rest
K_a = Rankine active earth pressure coefficient

Pressure Envelope for Braced-Cut Design

The lateral earth pressure in a braced cut is dependent on the type of soil, construction method, and type of equipment used. The lateral earth pressure changes from place to place.

Each strut should also be designed for the maximum load to which it may be subjected. Therefore, the braced cuts should be designed using apparent-pressure diagrams that are envelopes of all the pressure diagrams determined from measured strut loads in the field.

Pressure Envelope for Braced-Cut Design

Using the procedure just described for strut loads observed from the Berlin subway cut, Munich subway cut, and New York subway cut, Peck (1969) provided the envelope of apparent-lateral-pressure diagrams for design of cuts in *sand*. This envelope is illustrated in Figure 15.7, in which

$$\sigma_a = 0.65\gamma H K_a$$

where

γ = unit weight
H = height of the cut
K_a = Rankine active pressure coefficient = $\tan^2(45 - \phi'/2)$
ϕ' = effective friction angle of sand

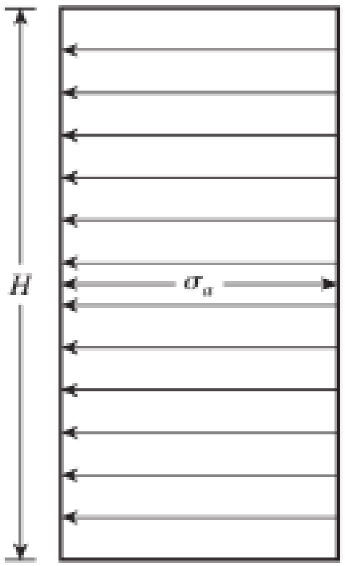


Figure 15.7 Peck's (1969) apparent-pressure envelope for cuts in sand

Pressure Envelope for Braced-Cut Design

Cuts in Clay

In a similar manner, Peck (1969) also provided the envelopes of apparent-lateral-pressure diagrams for cuts in *soft to medium clay* and in *stiff clay*. The pressure envelope for soft to medium clay is shown in Figure 15.8 and is applicable to the condition

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

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

The pressure, σ_a, is the larger of

$$\sigma_a = \gamma H\left[1 - \left(\frac{4c}{\gamma H}\right)\right]$$

and

$$\sigma_a = 0.3\gamma H$$

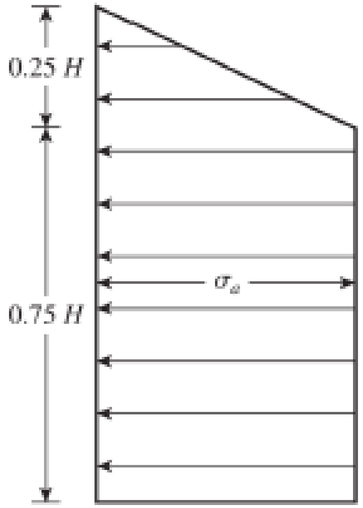


Figure 15.8 Peck's (1969) apparent-pressure envelope for cuts in soft to medium clay

Pressure Envelope for Braced-Cut Design

Cuts in Clay

The pressure envelope for cuts in stiff clay is shown in Figure 15.9, in which

$$\sigma_a = 0.2\gamma H \text{ to } 0.4\gamma H \qquad \text{(with an average of } 0.3\gamma H\text{)}$$

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

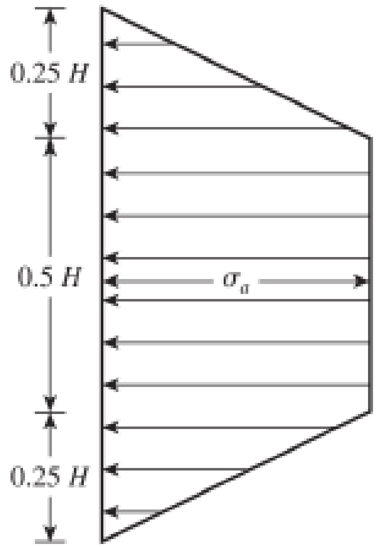


Figure 15.9 Peck's (1969) apparent-pressure envelope for cuts in stiff clay

Pressure Envelope for Braced-Cut Design

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

1. They apply to excavations having depths greater than about 6 m ($\approx$20 ft).
2. They are based on the assumption that the water table is below the bottom of the cut.
3. Sand is assumed to be drained with zero pore water pressure.
4. Clay is assumed to be undrained and pore water pressure is not considered.

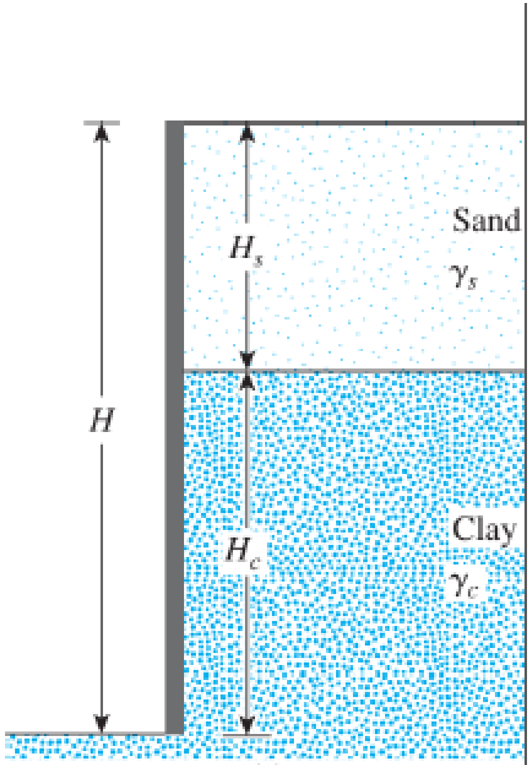
Pressure Envelope for Cuts in Layered Soil

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 ($\phi = 0$) should be determined according to the formula (see Figure 15.10a).

$$c_{av} = \frac{1}{2H}\left[\gamma_s K_s H_s^2 \tan\phi'_s + (H - H_s)n' q_u\right] \qquad (15.8)$$

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 = effective 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)



(a)

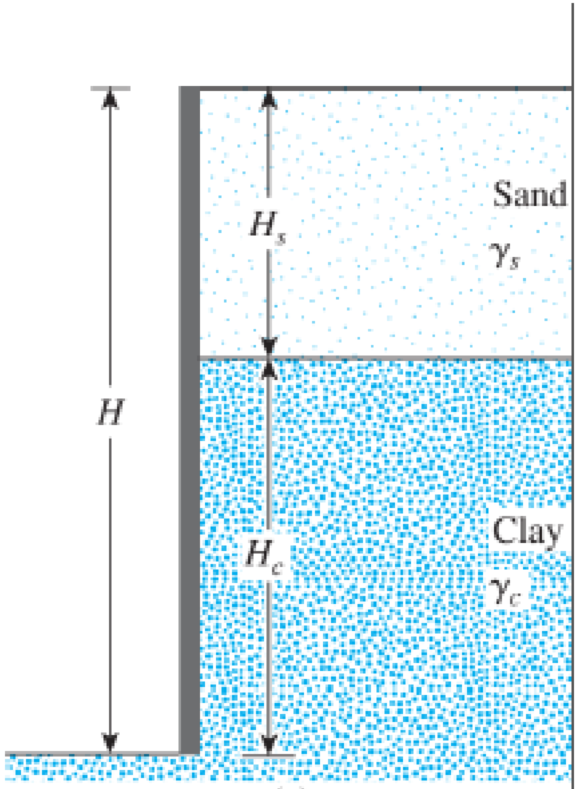
Pressure Envelope for Cuts in Layered Soil

The average unit weight of the layers may be expressed as

$$\gamma_a = \frac{1}{H}[\gamma_s H_s + (H - H_s)\gamma_c] \tag{15.9}$$

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.



(a)

Pressure Envelope for Cuts in Layered Soil

Similarly, when several clay layers are encountered in the cut (Figure 15.10b), the average undrained cohesion becomes

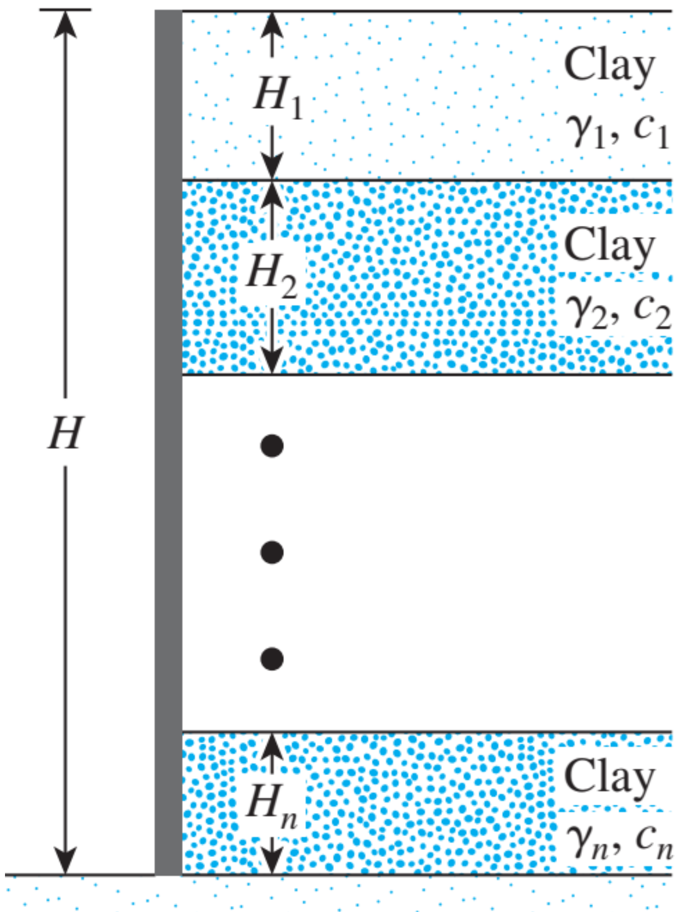
$$c_{av} = \frac{1}{H}(c_1H_1 + c_2H_2 + \cdots + c_nH_n) \qquad (15.10)$$

where

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

The average unit weight is now

$$\gamma_a = \frac{1}{H}(\gamma_1H_1 + \gamma_2H_2 + \gamma_3H_3 + \cdots + \gamma_nH_n) \qquad (15.11)$$



(b)

Design of Various Components of a Braced Cut

Struts

In construction work, struts should have a minimum vertical spacing of about 2.75 m (9 ft) or more. Struts are horizontal columns subject to bending. The load-carrying capacity of columns depends on their *slenderness ratio*, which 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. (12.8),

$$\sigma'_a = \gamma z K_a - 2c'\sqrt{K_a}$$

where K_a = coefficient of Rankine active pressure.

For determining the depth of tensile crack,

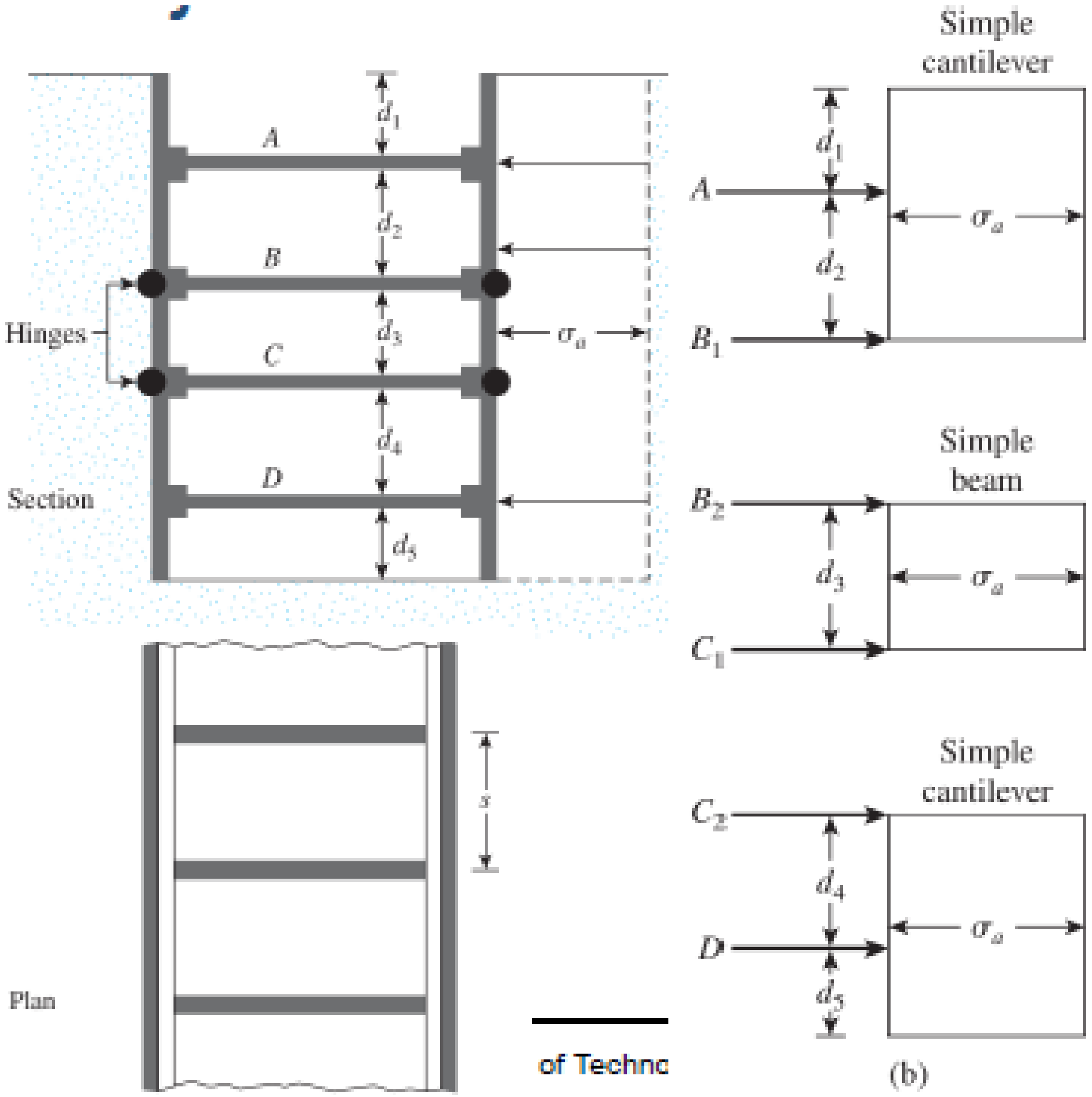
$$\sigma'_a = 0 = \gamma z_c K_a - 2c'\sqrt{K_a}$$

or

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

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

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



Simple cantilever
A
B
C
D
Hinges
Section
Plan
of Techno
Simple beam
Simple cantilever
(b)

Figure 15.11

Struts

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 methodology (see Figure 15.11):

Step 1. Draw the pressure envelope for the braced cut. (See Figures 15.7, 15.8, and 15.9.) Also, show the proposed strut levels. Figure 15.11a 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 15.11a, 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.)

Step 2. Determine the reactions for the two simple cantilever beams (top and bottom) and all the simple beams between. In Figure 15.11b, these reactions are A, B_1, B_2, C_1, C_2, and D.

Struts

Step 3. The strut loads in the figure may be calculated via the formulas

$$P_A = (A)(s)$$
$$P_B = (B_1 + B_2)(s) \qquad (15.12)$$
$$P_C = (C_1 + C_2)(s)$$

and

$$P_D = (D)(s)$$

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 the unit: force/unit length of the braced cut)

s = horizontal spacing of the struts (see plan in Figure 15.11a)

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

Design of Sheet Piles

Sheet Piles

The following steps are involved in designing the sheet piles:

Step 1. For each of the sections shown in Figure 15.11b, determine the maximum bending moment.

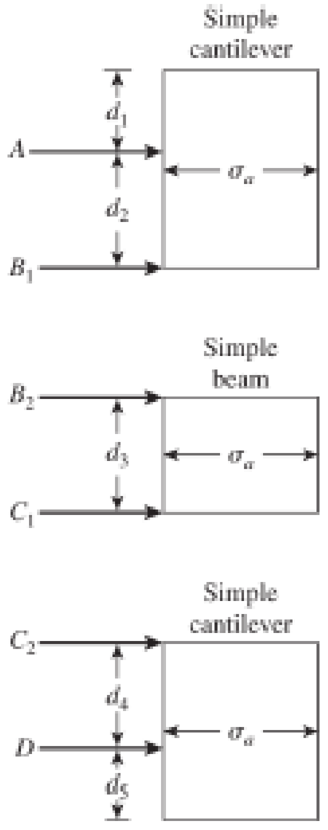
Step 2. Determine the maximum value of the maximum bending moments (M_{max}) obtained in Step 1. Note that the unit of this moment will be, for example, kN-m/m (lb-ft/ft) length of the wall.

Step 3. Obtain the required section modulus of the sheet piles, namely,

$$S = \frac{M_{max}}{\sigma_{all}} \tag{15.13}$$

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

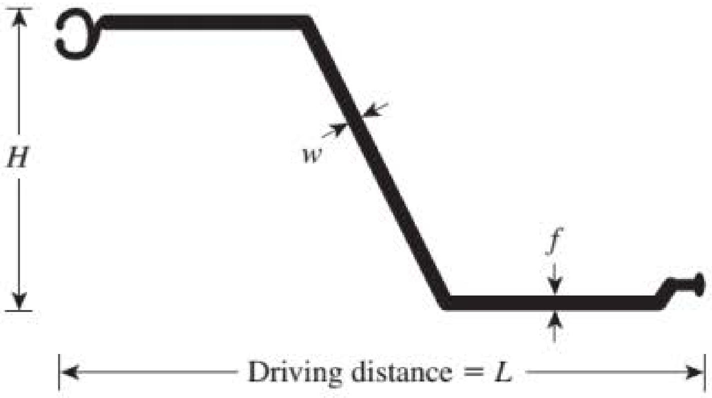
Step 4. Choose a sheet pile having a section modulus greater than or equal to the required section modulus from a table such as Table 14.1.



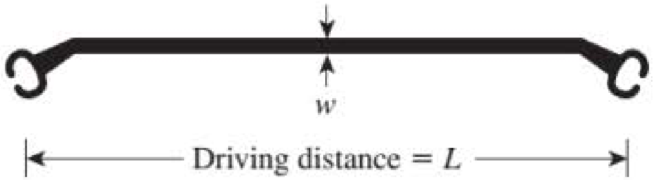
(b)

Table 14.1 Properties of Some Commercially Available Sheet-Pile Sections (Based on Hammer and Steel, Inc., Hazelwood, Missouri, USA)

Section designation	*H* mm (in.)	*L* mm (in.)	*f* mm (in.)	*w* mm (in.)	Section modulus m^3/m of wall ($in.^3$/ft of wall)	Moment of inertia m^4/m of wall ($in.^4$/ft of wall)
PZC-12	318.0	708.2	8.51	8.51	120.42×10^{-5}	192.06×10^{-6}
	(12.52)	(27.88)	(0.335)	(0.335)	(22.4)	(140.6)
PZC-13	319.0	708.2	9.53	9.53	130.1×10^{-5}	207.63×10^{-6}
	(12.56)	(27.88)	(0.375)	(0.375)	(24.2)	(152.0)
PZC-14	320.0	708.2	10.67	10.67	139.78×10^{-5}	225.12×10^{-6}
	(12.6)	(27.88)	(0.420)	(0.420)	(26.0)	(164.8)
PZC-17	386.3	635.0	8.51	8.51	166.67×10^{-5}	322.38×10^{-6}
	(15.21)	(25.00)	(0.335)	(0.335)	(31.0)	(236.6)
PS-27.5	—	500	—	10.16	10.21×10^{-5}	4.1×10^{-6}
	—	(19.69)	—	(0.4)	(1.9)	(3.0)
PS-31	—	500	—	12.7	10.21×10^{-5}	4.1×10^{-6}
	—	(19.69)	—	(0.5)	(1.9)	(3.0)



PZC and PZ section



PS section

Design of Wales

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 15.11a, the maximum moments for the wales (assuming that they are pinned at the struts) are,

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

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

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

Design of Wales

and

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

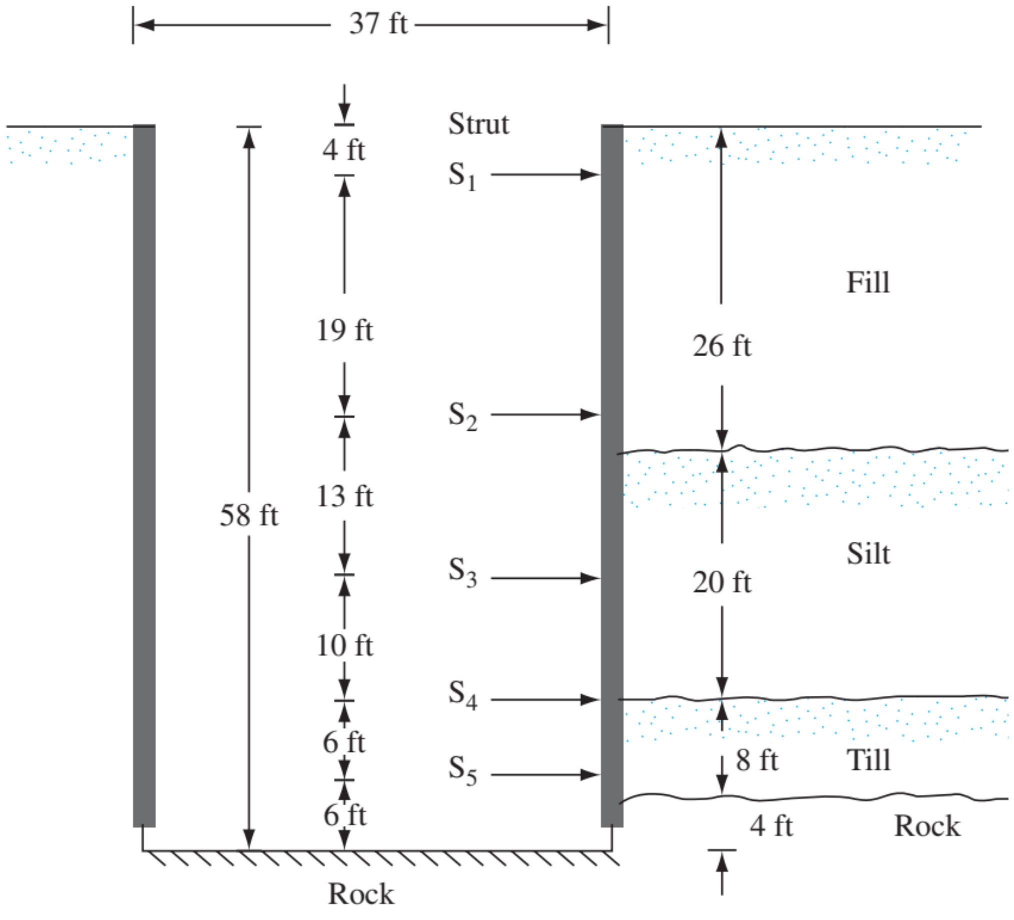
where A, B_1, B_2, C_1, C_2, and D are the reactions under the struts per unit length of the wall (see Step 2 of strut design).

Now determine the section modulus of the wales:

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

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

Example



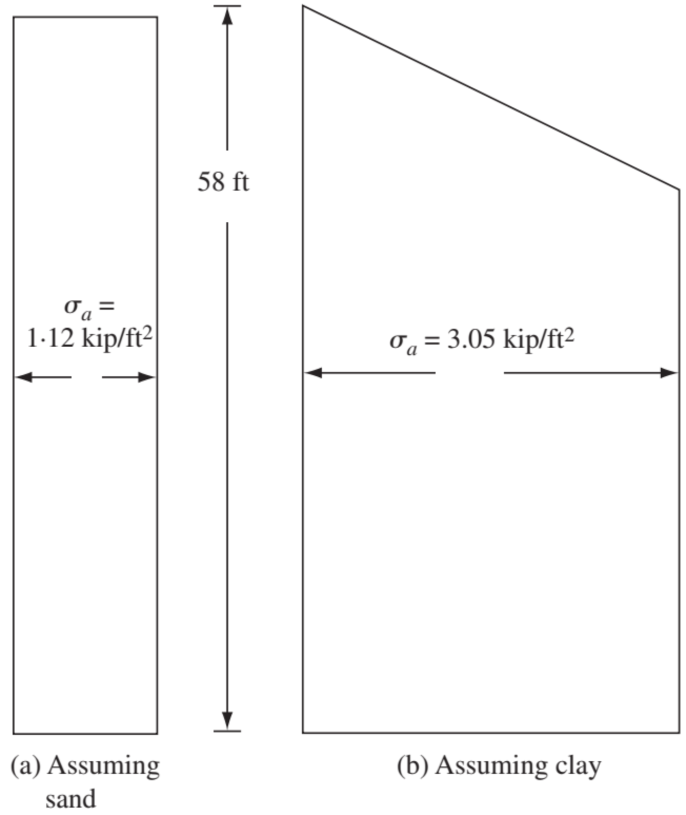


Figure 15.17 Pressure envelopes: (a) assuming sand; (b) assuming clay

Example

Table 15.2 Computed and Measured Strut Loads at Test Section B

Strut number	Computed load (kip): Envelope based on sand	Computed load (kip): Envelope based on clay	Measured strut load (kip)
S-1	182	230	70.4
S-2	215	580	215
S-3	154	420	304
S-4	108	292	230
S-5	75	219	274

1. In most cases the measured strut loads differed widely from those predicted. This result is due primarily to the uncertainties involved in the assumption of the soil parameters.
2. The actual design strut loads were substantially higher than those measured.

Bottom Heave 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 long braced excavations against bottom heave. The failure surface for such a case in a homogeneous soil is shown in Figure 15.20. In the figure, the following notations are used: B = width of the cut, H = depth of the cut, T = thickness of the clay below the base of excavation, and q = uniform surcharge adjacent to the excavation.

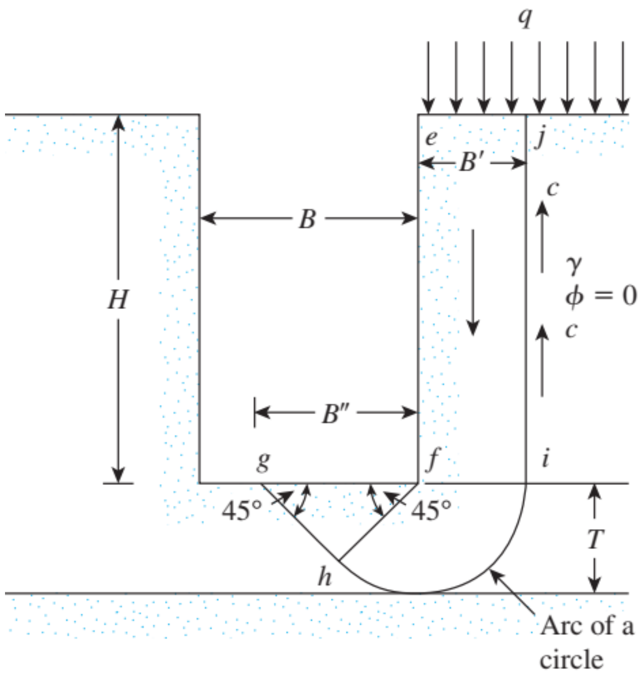
The ultimate bearing capacity at the base of a soil column with a width of B' can be given as

$$q_{\text{ult}} = cN_c \tag{15.16}$$

where $N_c = 5.7$ (for a perfectly rough foundation).

The vertical load per unit area along fi is

$$q = \gamma H + q - \frac{cH}{B'} \tag{15.17}$$



Bottom Heave of a Cut in Clay

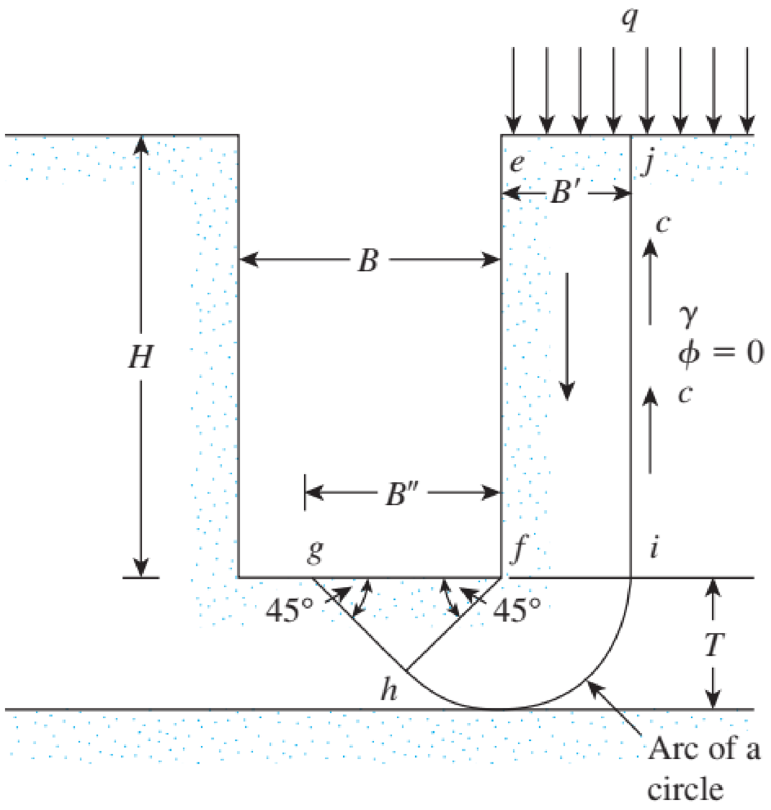
Hence, the factor of safety against bottom heave is

$$\text{FS} = \frac{q_{\text{ult}}}{q} = \frac{cN_c}{\gamma H + q - \dfrac{cH}{B'}} = \frac{cN_c}{\left(\gamma + \dfrac{q}{H} - \dfrac{c}{B'}\right)H} \tag{15.18}$$

For excavations of limited length L, the factor of safety can be modified to

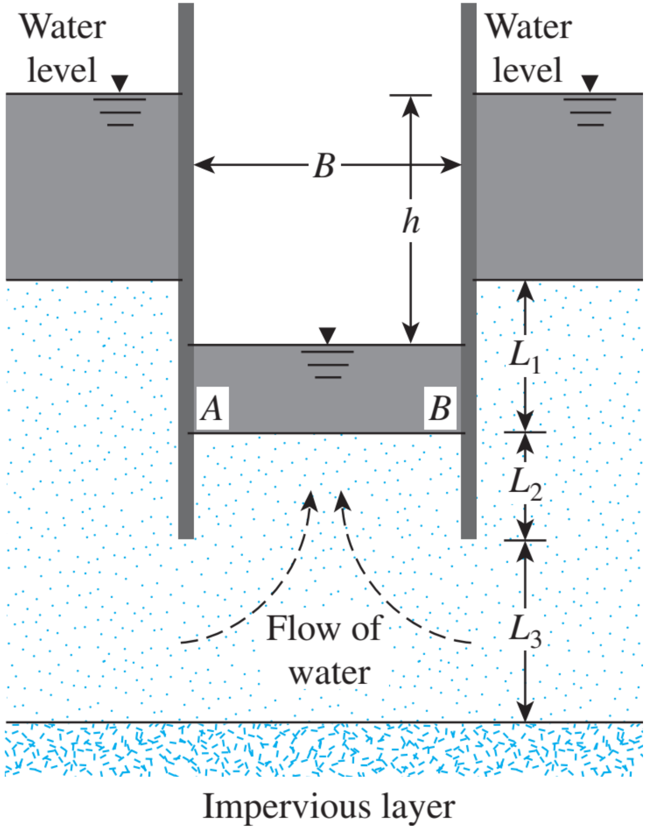
$$\text{FS} = \frac{cN_c\left(1 + 0.2\dfrac{B'}{L}\right)}{\left(\gamma + \dfrac{q}{H} - \dfrac{c}{B'}\right)H} \tag{15.19}$$

where $B' = T$ or $B/\sqrt{2}$ (whichever is smaller).



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 (see Figure 15.23), the factor of safety against piping should be checked. [*Piping* is another term for failure by heave, as defined in Section 2.12; see Eq. (2.50).] 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 15.24 shows such a flow net, for which the maximum



Stability of the Bottom of a Cut in Sand

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

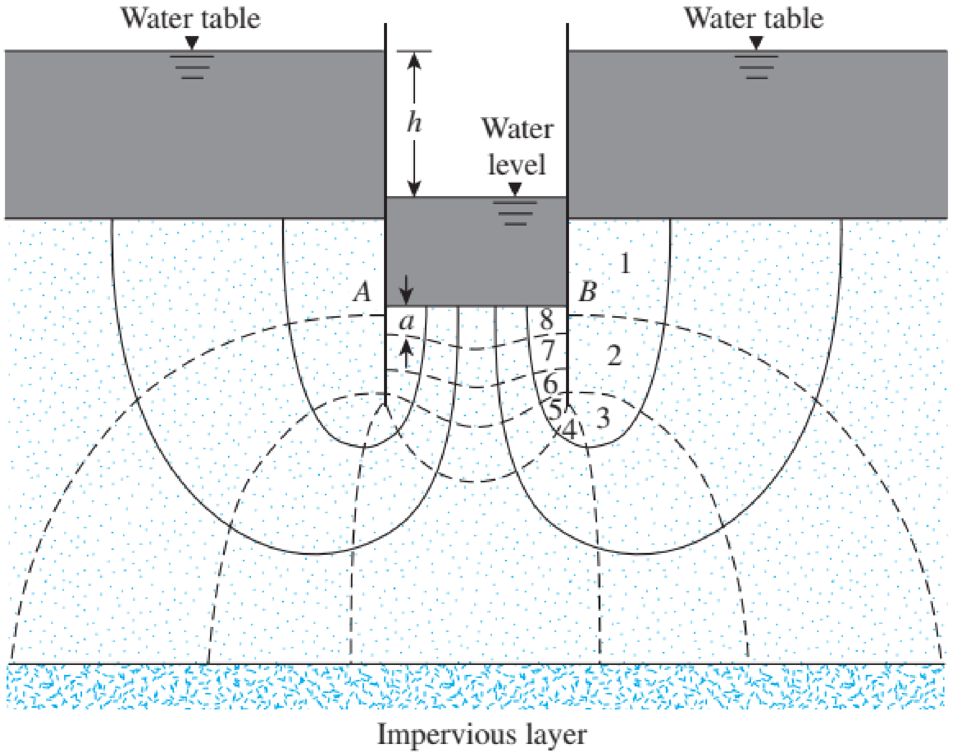
where

a = length of the flow element at A (or B)
N_d = number of drops (*Note:* in Figure 15.24, $N_d = 8$; see also Section 2.11)

The factor of safety against piping may be expressed as

$$\text{FS} = \frac{i_{\text{cr}}}{i_{\max(\text{exit})}}$$

where i_{cr} = critical hydraulic gradient.



Stability of the Bottom of a Cut in Sand

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

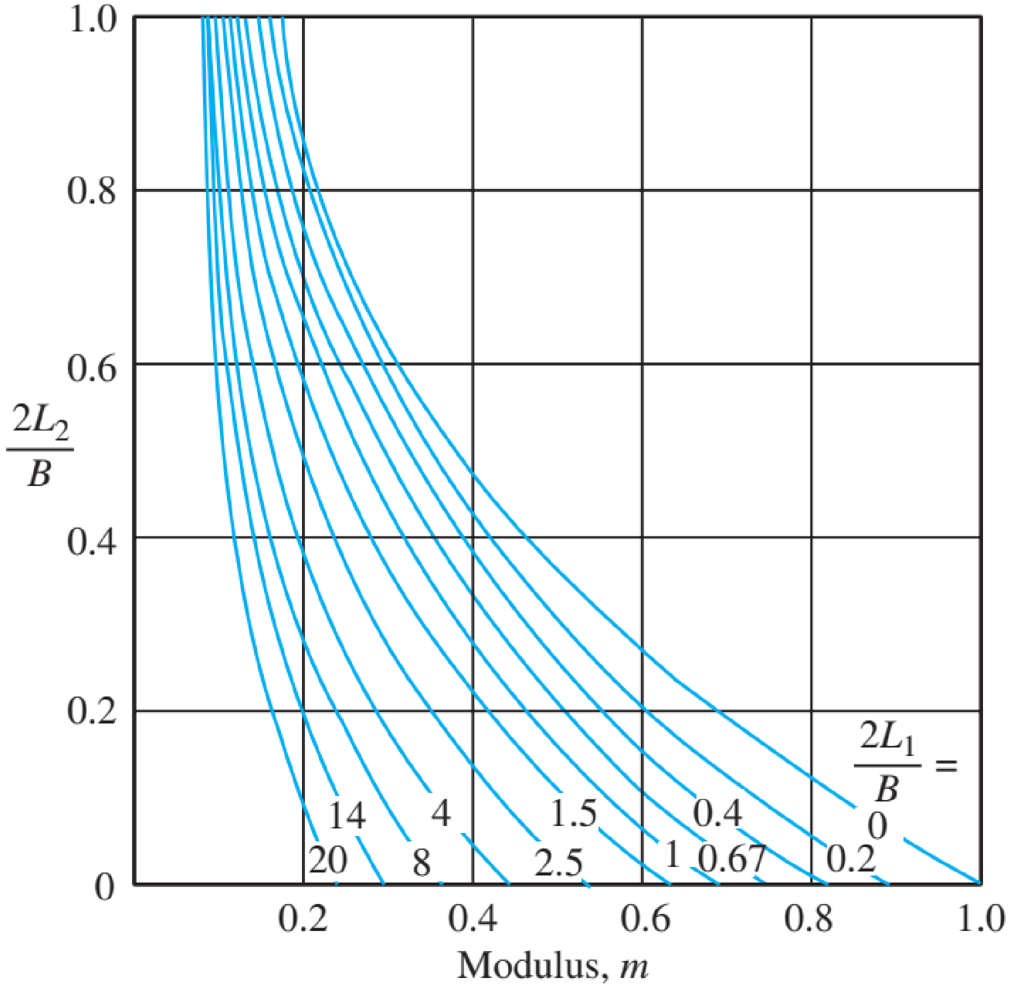
$$i_{cr} = \frac{G_s - 1}{e + 1}$$

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.

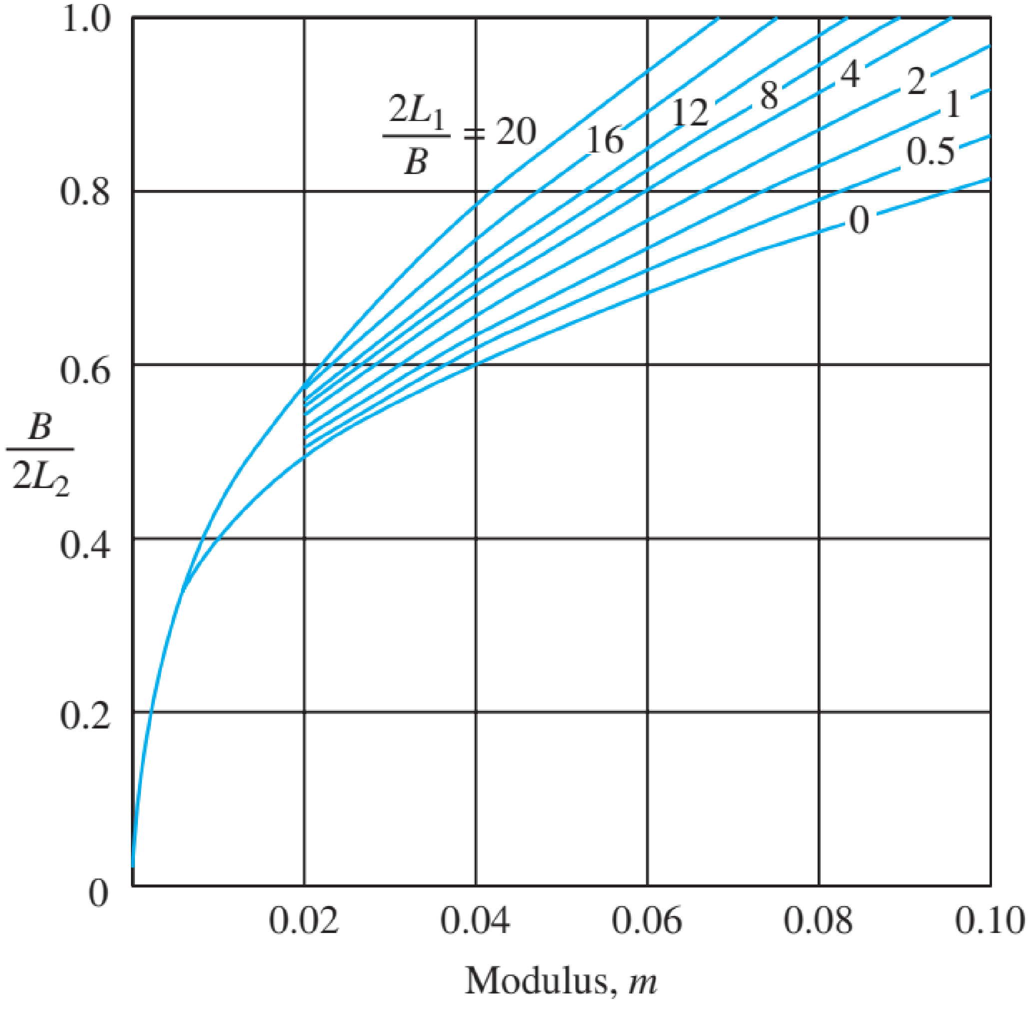
The maximum exit gradient for sheeted excavations in sands with $L_3 = \infty$ can also be evaluated theoretically (Harr, 1962). (Only the results of these mathematical derivations will be presented here. For further details, see the original work.) To calculate the maximum exit gradient, examine Figures 15.25 and 15.26 and perform the following steps:

1. Determine the modulus, m, from Figure 15.25 by obtaining $2L_2/B$ (or $B/2L_2$) and $2L_1/B$.
2. With the known modulus and $2L_1/B$, examine Figure 15.26 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. (15.24).

Stability of the Bottom of a Cut in Sand

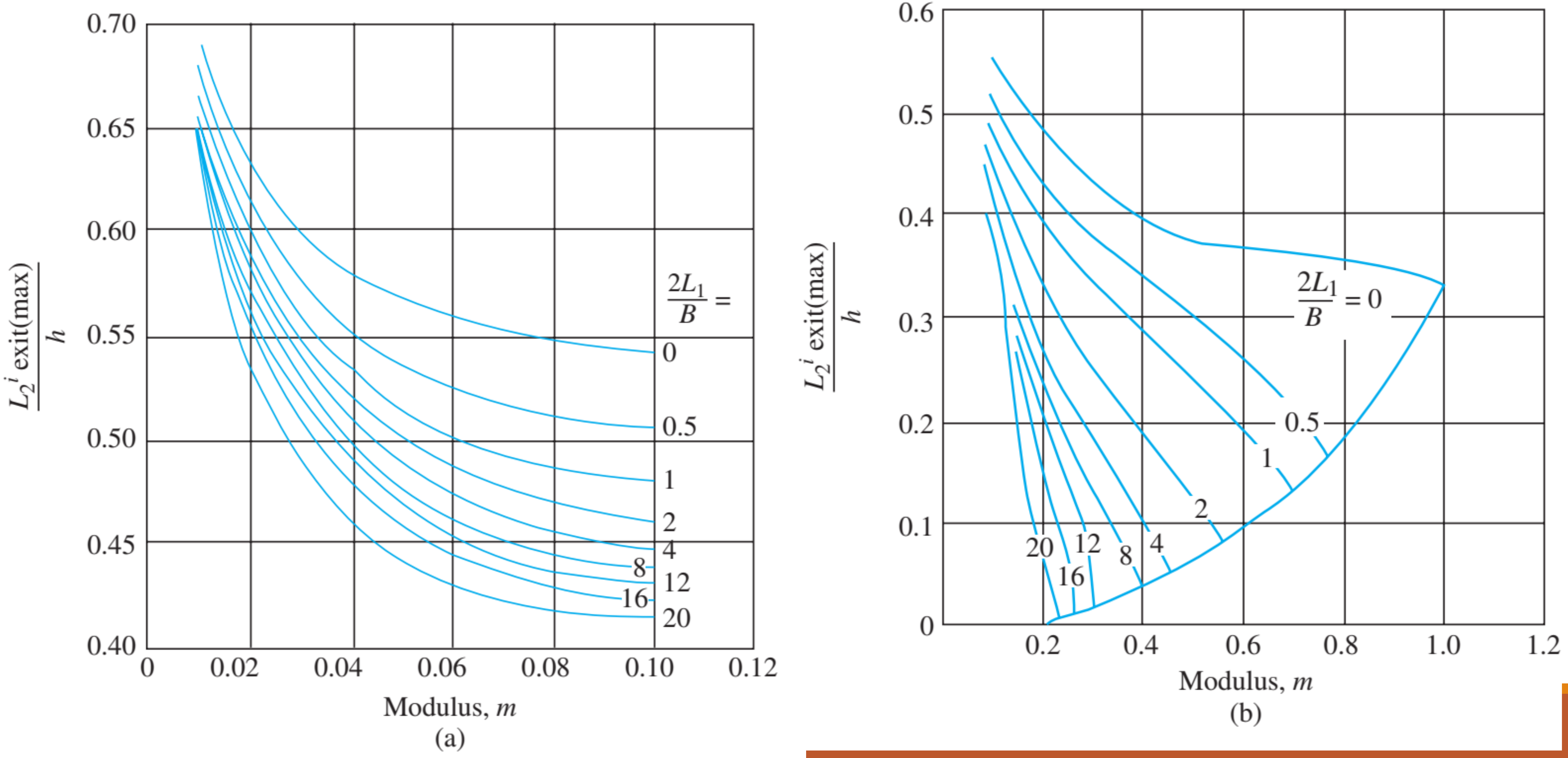


(a)



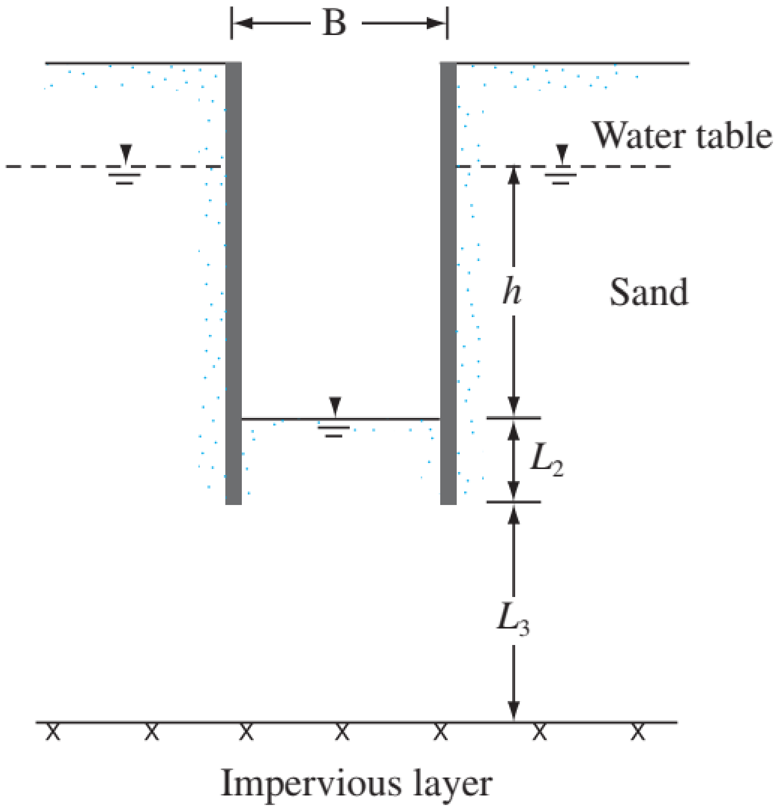
(b)

Stability of the Bottom of a Cut in Sand



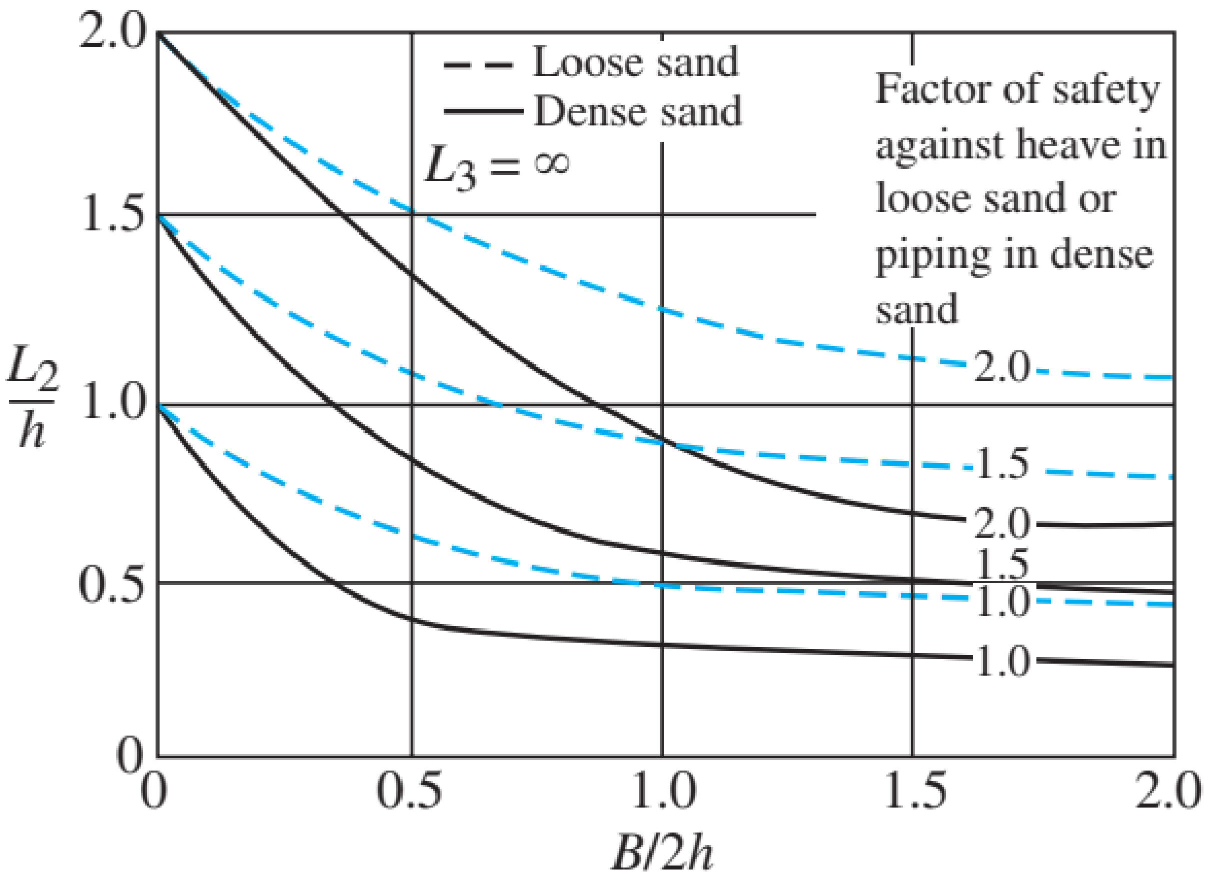
Stability of the Bottom of a Cut in Sand

Marsland (1958) presented the results of model tests conducted to study the influence of seepage on the stability of sheeted excavations in sand. The results were summarized by the U.S. Department of the Navy (1971) in NAVFAC DM-7 and are given in Figure 15.27a, b, and c. Note that Figure 15.27b 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. By contrast, Figure 15.27c represents the case in which an impervious layer lies at a limited depth below the bottom of the excavation.

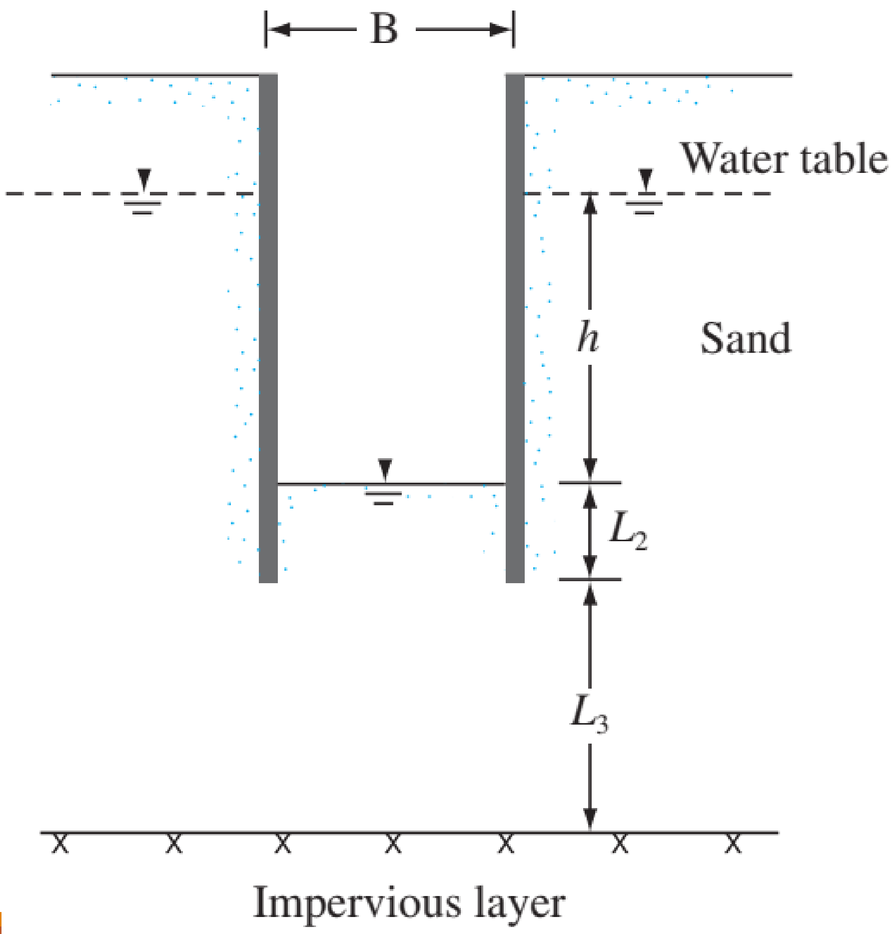


(a)

Stability of the Bottom of a Cut in Sand

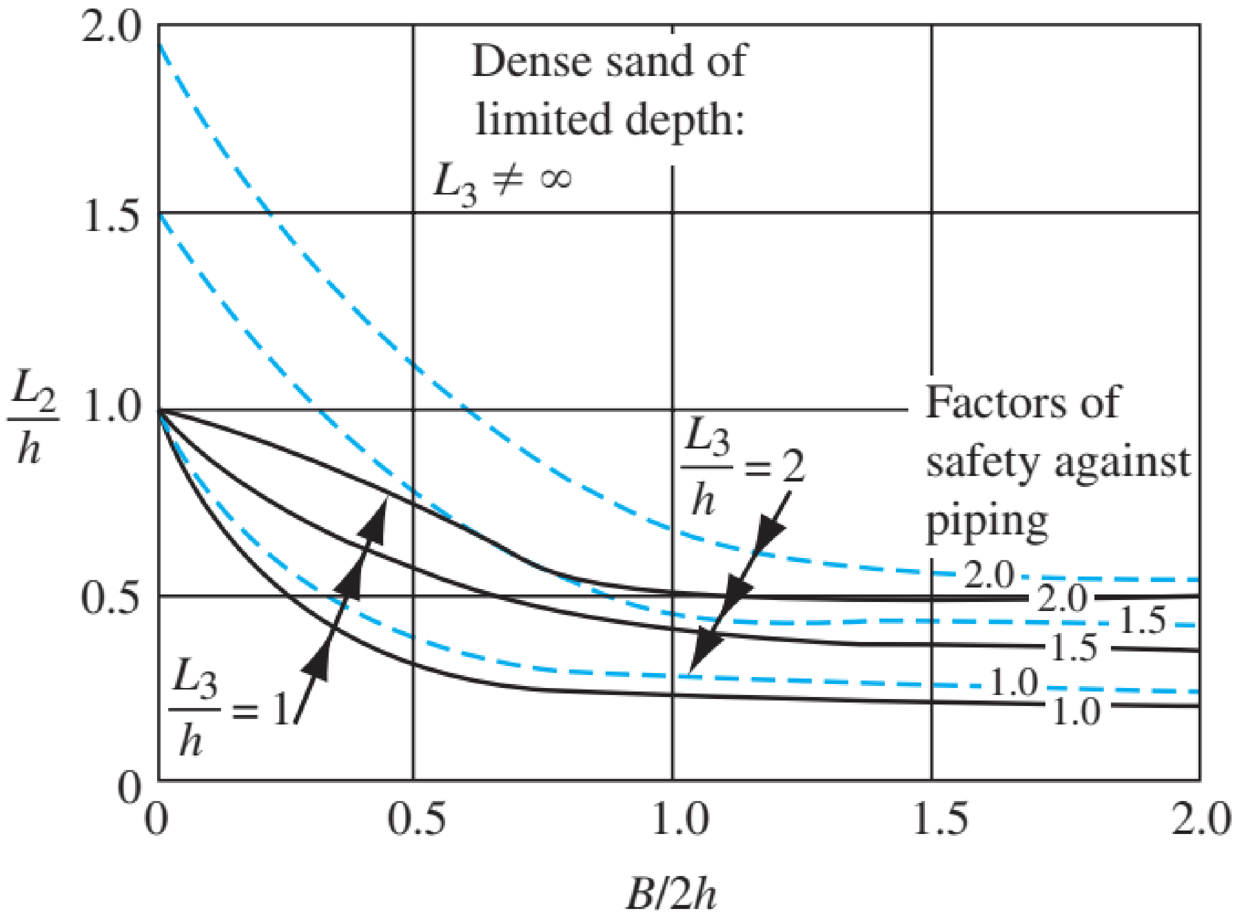


(b)

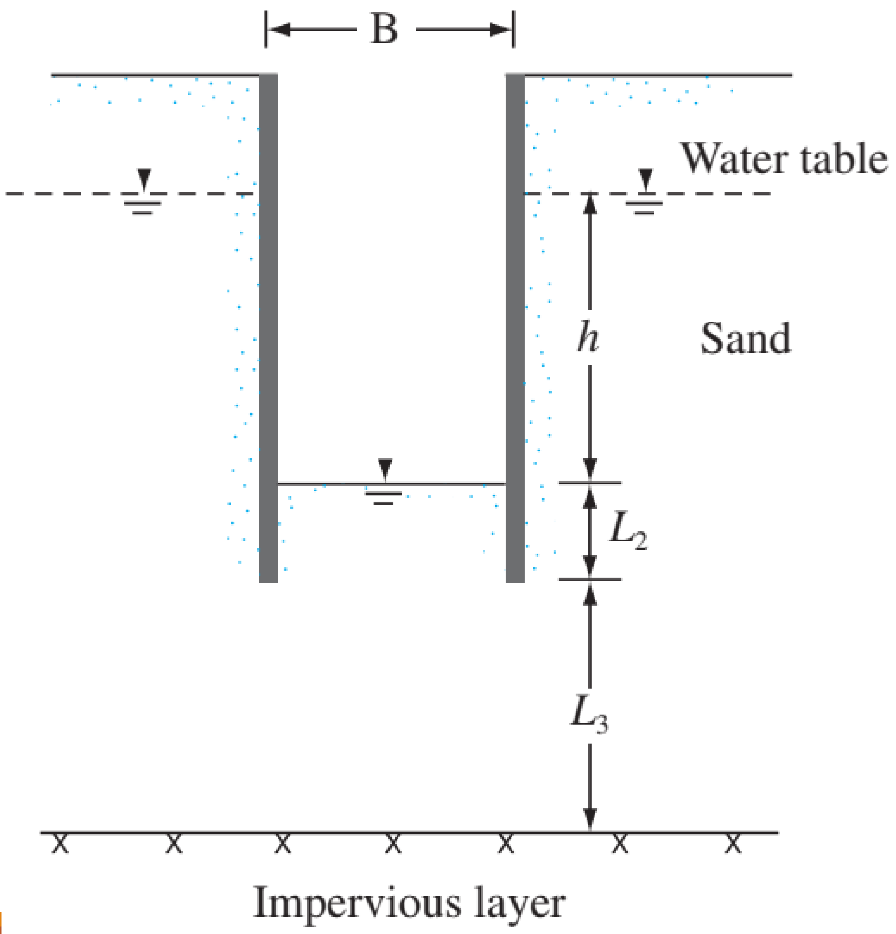


(a)

Stability of the Bottom of a Cut in Sand

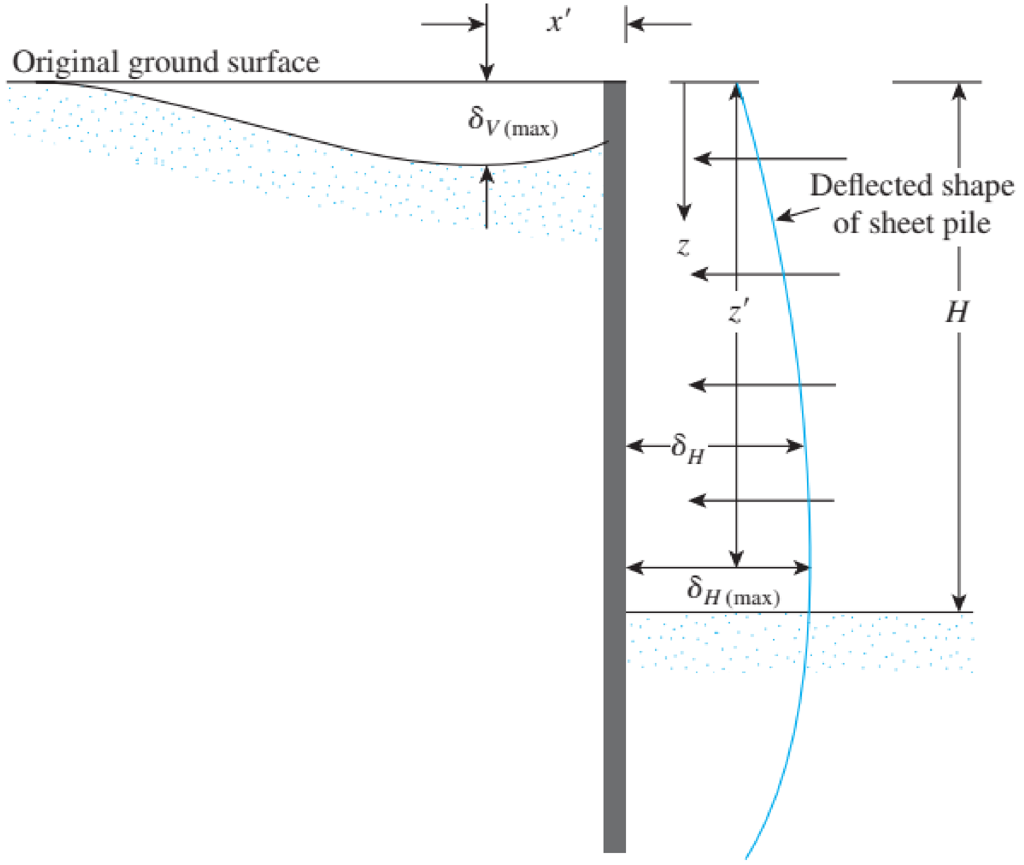


(c)



(a)

Lateral Yielding of Sheet Piles and Ground Settlement



Lateral Yielding of Sheet Piles and Ground Settlement

Moormann (2004) analyzed about 153 case histories dealing mainly with the excavation in soft clay (that is, undrained shear strength, $c \leqslant 75\ \text{kN/m}^2$). Following is a summary of his analysis relating to $\delta_{V(\max)}$, x', $\delta_{H(\max)}$, and z' (see Figure 15.28).

- Maximum Vertical Movement [$\delta_{V(\max)}$]

  $\delta_{V(\max)}/H \approx 0.1$ to 10.1% with an average of 1.07% (soft clay)
  $\delta_{V(\max)}/H \approx 0$ to 0.9% with an average of 0.18% (stiff clay)
  $\delta_{V(\max)}/H \approx 0$ to 2.43% with an average of 0.33% (non-cohesive soils)

- Location of $\delta_{V(\max)}$, that is x' (Figure 15.28)

  For 70% of all case histories considered, $x' \leqslant 0.5H$.
  However, in soft clays, x' may be as much as $2H$.

- Maximum Horizontal Deflection of Sheet Piles, $\delta_{H(\max)}$

  For 40% of excavation in soft clay, $0.5\% \leqslant \delta_{H(\max)}/H \leqslant 1\%$.
  The average value of $\delta_{H(\max)}/H$ is about 0.87%.

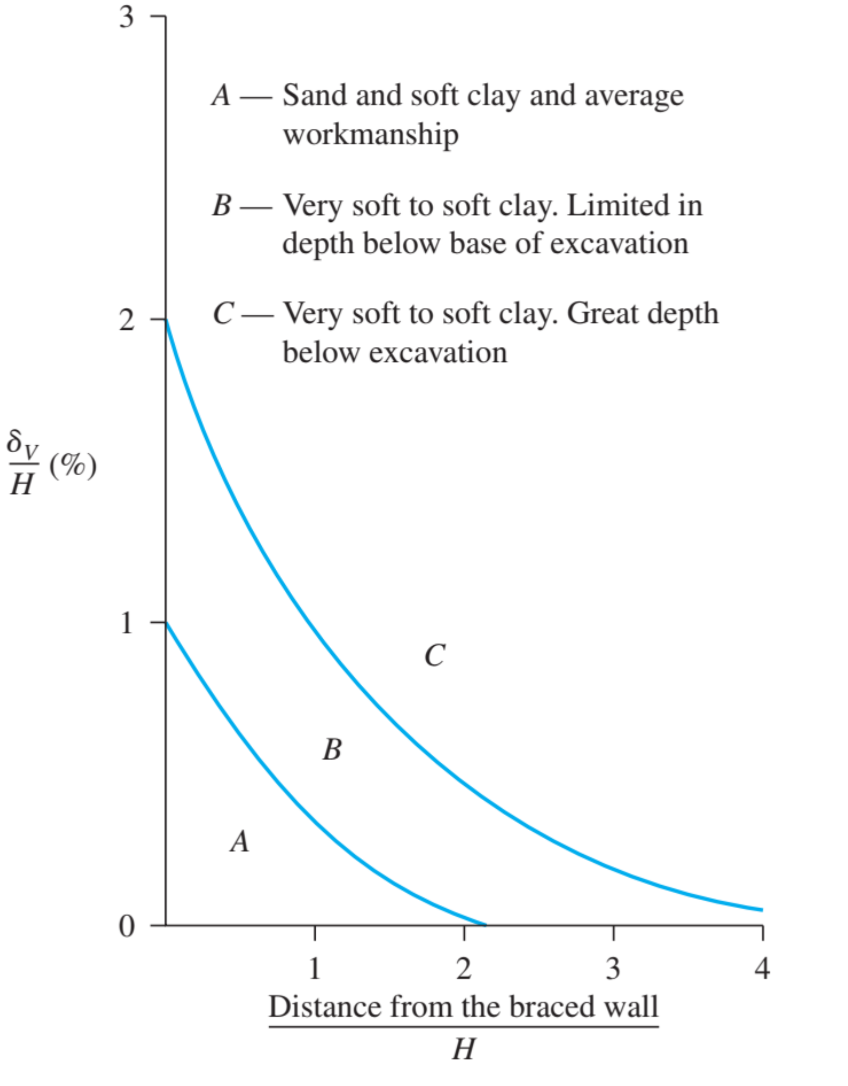
Lateral Yielding of Sheet Piles and Ground Settlement

In stiff clays, the average value of $\delta_{H(\max)}/H$ is about 0.25%.
In non-cohesive soils, $\delta_{H(\max)}/H$ is about 0.27% of the average.

- Location of $\delta_{H(\max)}$, that is z' (Figure 15.28)

For deep excavation of soft and stiff cohesive soils, z'/H is about 0.5 to 1.0.

Lateral Yielding of Sheet Piles and Ground Settlement



Ground Anchors

A prestressed grouted ground anchor is a structural element installed in soil or rock that is used to transmit an applied tensile load into the ground. Anchors are installed in grout filled drill holes.

Ground anchors consisting of cables or rods connected to a bearing plate are often used for

- the stabilization of steep slopes
- slopes consisting of softer soils,
- enhancement of embankment or foundation soil capacity,
- prevent excessive erosion and landslides.

The use of steel ground anchors is often constrained by overall durability in placement (due to weight), and the difficulty in maintaining tension levels in the anchor.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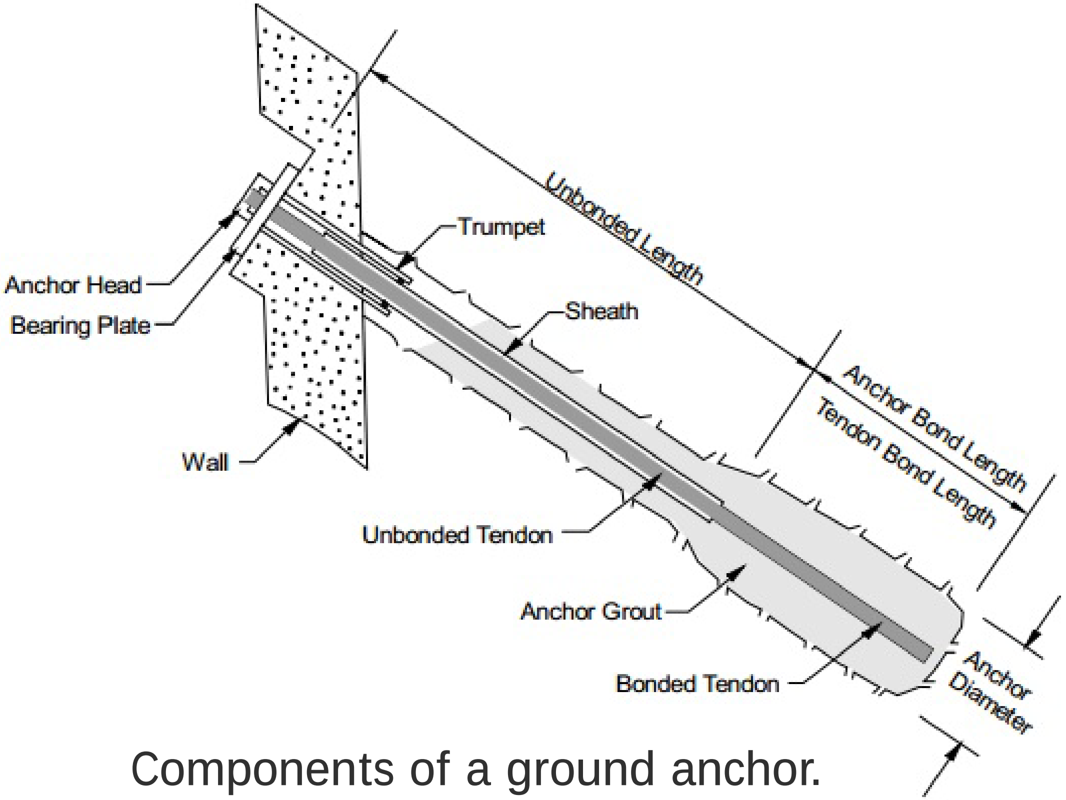
Ground Anchors

The basic components of a grouted ground anchor include the:

1. anchorage;
2. free stressing (unbonded) length;
3. bond length.

The anchorage is the combined system of anchor head, bearing plate, and trumpet that is capable of transmitting the prestressing force from the prestressing steel (bar or strand) to the ground surface or the supported structure.



Components of a ground anchor.

Ground Anchors

Ground anchors can be used for temporary or permanent applications.

- Temporary ground anchors are used for shoring during construction. Their service life is for the duration of the construction project, usually, two to five years.
- Permanent ground anchors are required for the life of the permanent structure.

The anchor bonded length shall be located beyond the limits of the potential active zone. A minimum distance between the front of the bonded zone of the anchor and the limits of the potential active zone behind the wall of 5 feet or H/5 is needed to ensure that no load from the bonded zone of the ground anchor is transferred to the retained soil mass by the grout column. Determination of the anchor un-bonded length, inclination from horizontal and overburden cover shall be based on:

- The location of the limits of the potential active zone behind the wall,
- The minimum length required to ensure minimal loss of anchor pre-stress due to creep of soil and rock, but not less than 15 feet,
- The depth to adequate anchoring strata,
- The method of anchor installation and grouting,
- The seismic performance of the wall and anchors.

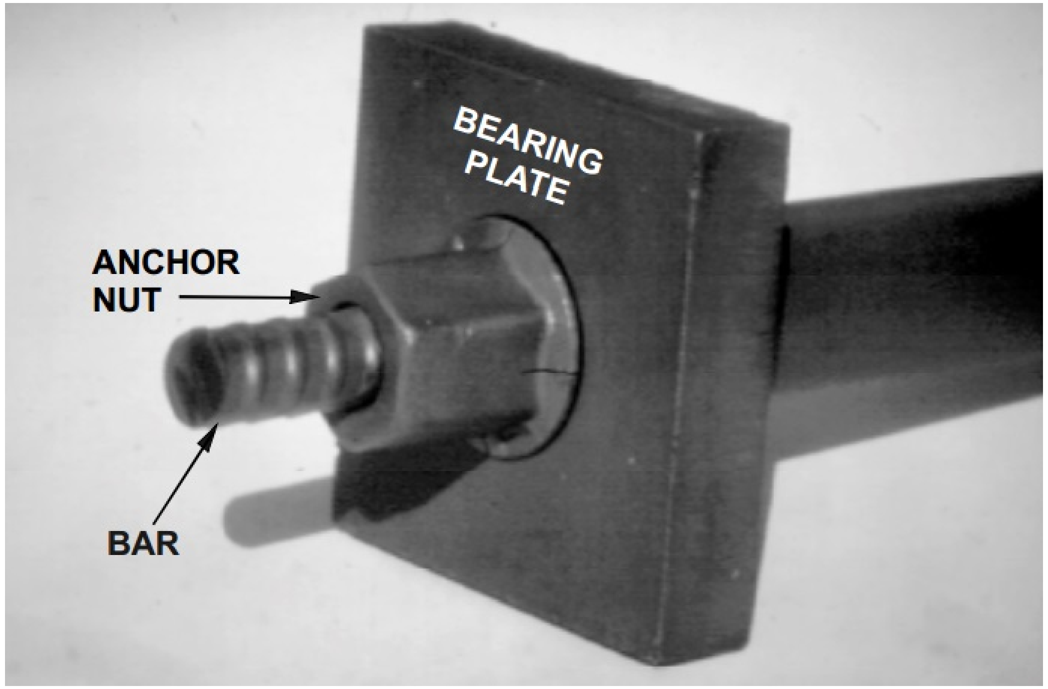
Ground Anchors

The minimum spacing between ground anchors should be the larger of three times the diameter of the hole within the bonded length, or 5 feet, to avoid group effects of the anchors. If tighter spacing is required to develop the required anchor design force, the angle of inclination can be varied on alternating anchors.

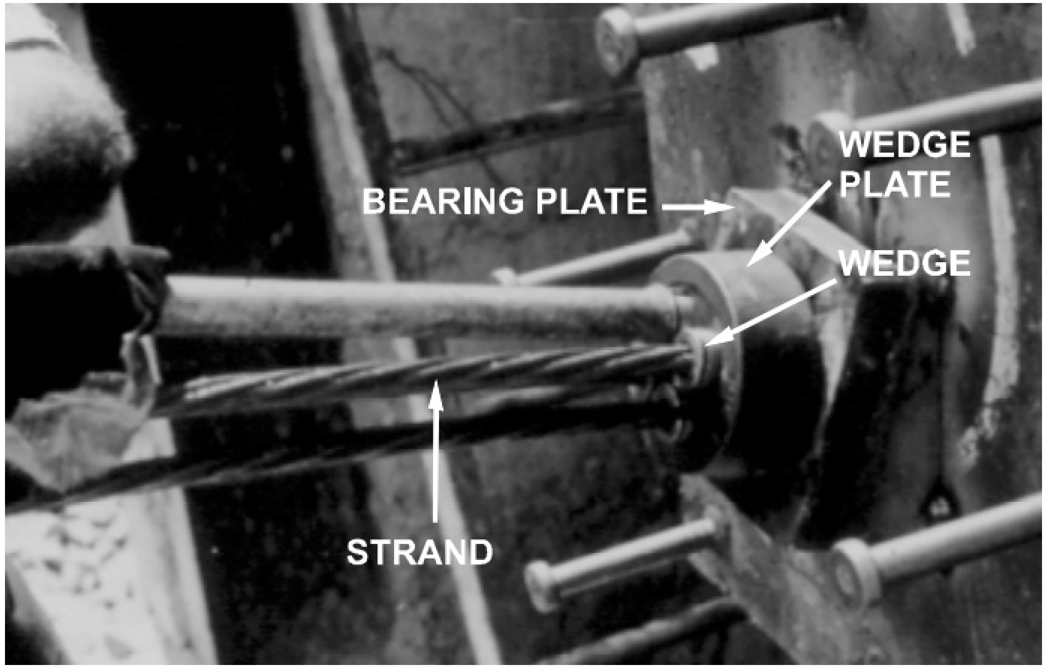
Based upon past experience, the majority of ground anchors are small diameter, straight shaft gravity-grouted anchors with the following typical characteristics:

- Design Load between 260 and 1000 kN
- Total Anchor Length between 10 to 20 m
- Ground Anchor Inclination between 10 to 30 degrees

Ground Anchors – Details

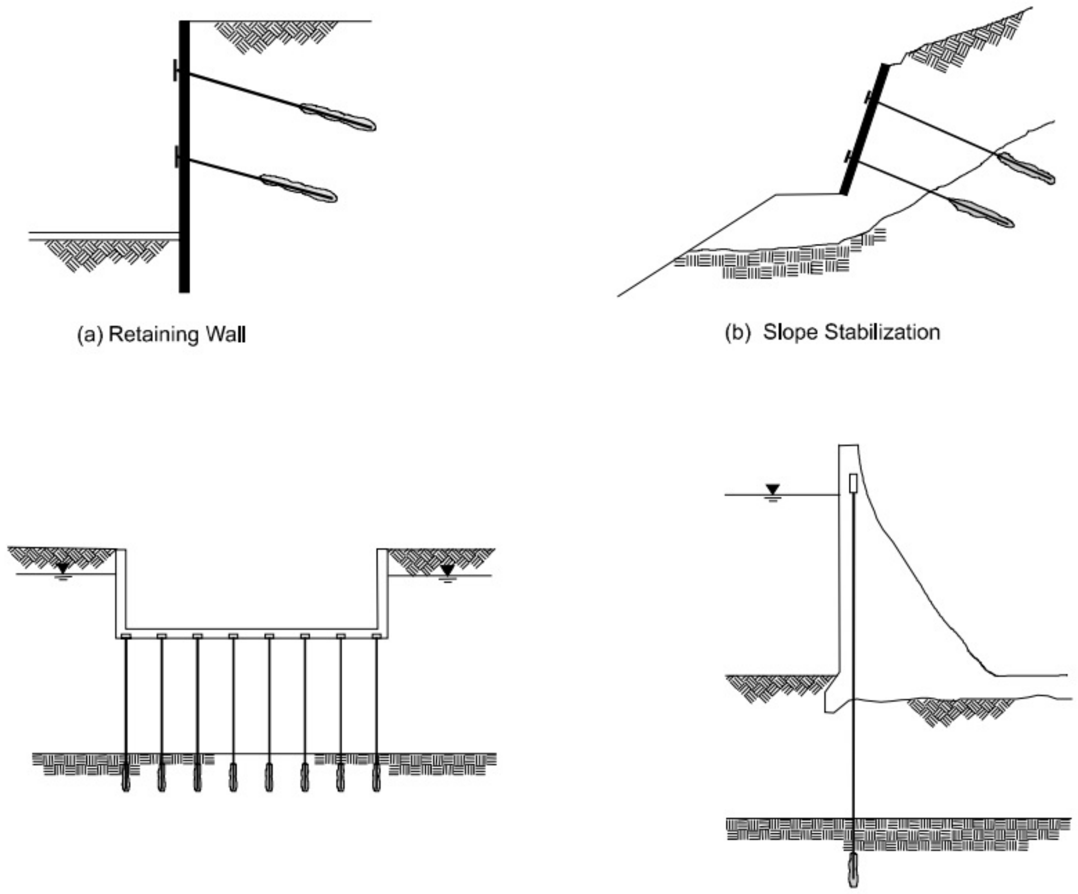


Anchorage components for a bar tendon



Anchorage components for a strand tendon

Applications of Ground Anchors



(a) Retaining Wall
(b) Slope Stabilization
(c) Uplift Slab
(d) Concrete Dam Stabilization

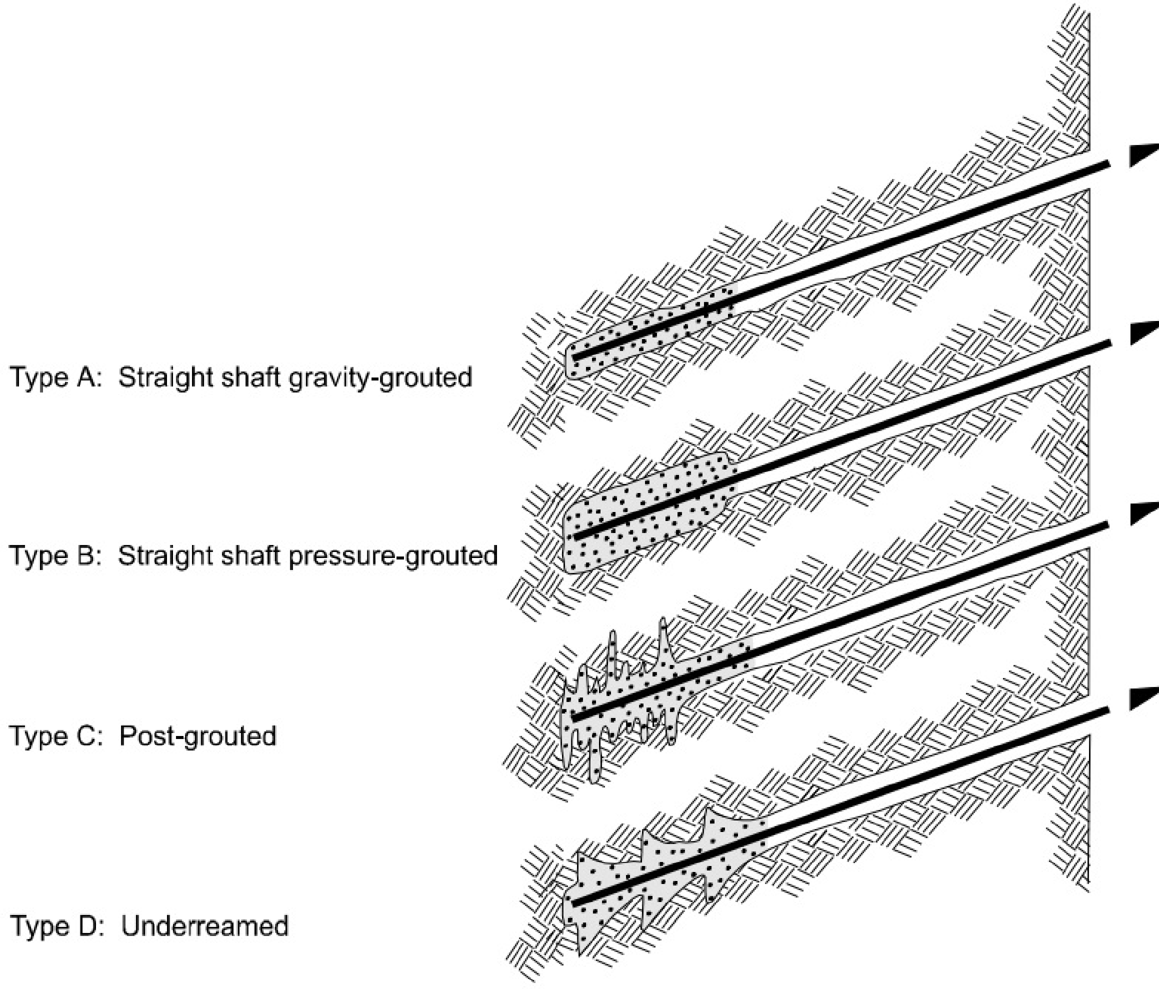
Types of Ground Anchors

A- Straight shaft gravity-grouted ground anchors:
typically installed in rock and very stiff to hard cohesive soil deposits using either rotary drilling or hollow-stem auger methods. Tremie (gravity displacement) methods are used to grout the anchor in a straight shaft borehole.

B- Straight shaft pressure-grouted ground anchors:
most suitable for coarse granular soils and weak fissured rock. Also used in fine grained cohesionless soils. Grout is injected into the bond zone under pressures greater than 0.35 MPa.

C- Post-grouted ground anchors:
use delayed multiple grout injections to enlarge the grout body of straight shafted gravity grouted ground anchors.

D- Underreamed anchors:
consist of tremie grouted boreholes that include a series of enlargement bells or underreams. This type of anchor may be used in firm to hard cohesive deposits.



Ground Anchors

Advantages

Execute excavations neatly to create large construction plan without using props in order to make mechanized excavation.

Keep excavation walls sustainable, make very deep excavations without depending on the basement structure.

Anchors combine with soft retaining walls to redistribute the internal forces of wall structure, so this can reduce the size, depth of steel bars in retaining walls.

Disadvantages

It is necessary to use specified equipment, experienced professional engineers.

It is difficult to apply anchors in weak soil and to implement anchors with great depth.

Anchor execution would affect the land of surrounding construction works, which must be accepted by their owners.

Anchor Load Testing

A unique aspect of ground anchors, as compared to other structural systems, is that every ground anchor that is to be part of a completed structure is load tested to verify its load capacity and load-deformation behavior before being put into service. The acceptance or rejection of ground anchors is determined based on the results of:

1- Performance tests;

Performance tests involve incremental loading and unloading of a production anchor. The performance test is used to verify anchor capacity, establish load-deformation behavior, identify causes of anchor movement, and to verify that the actual unbonded length is equal to or greater than that assumed in the anchor design.

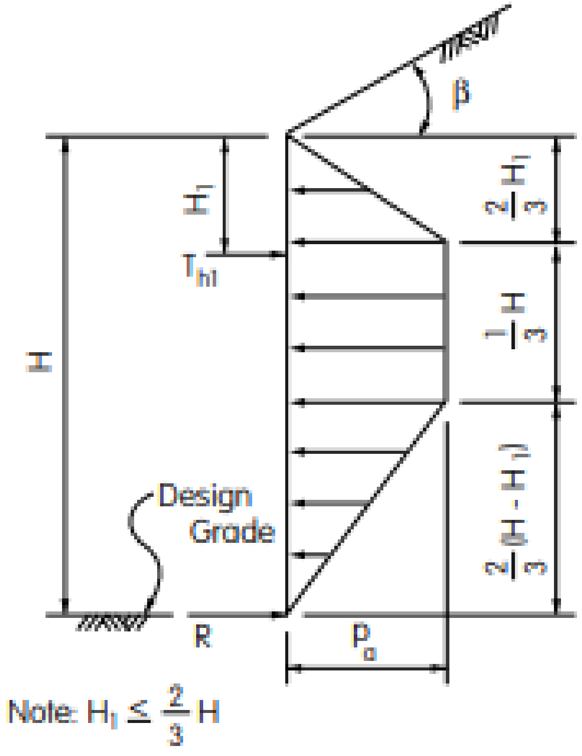
2- Proof tests;

The proof test involves a single load cycle and a load hold at the test load. The magnitude of the applied load is measured using the jack pressure gauge.

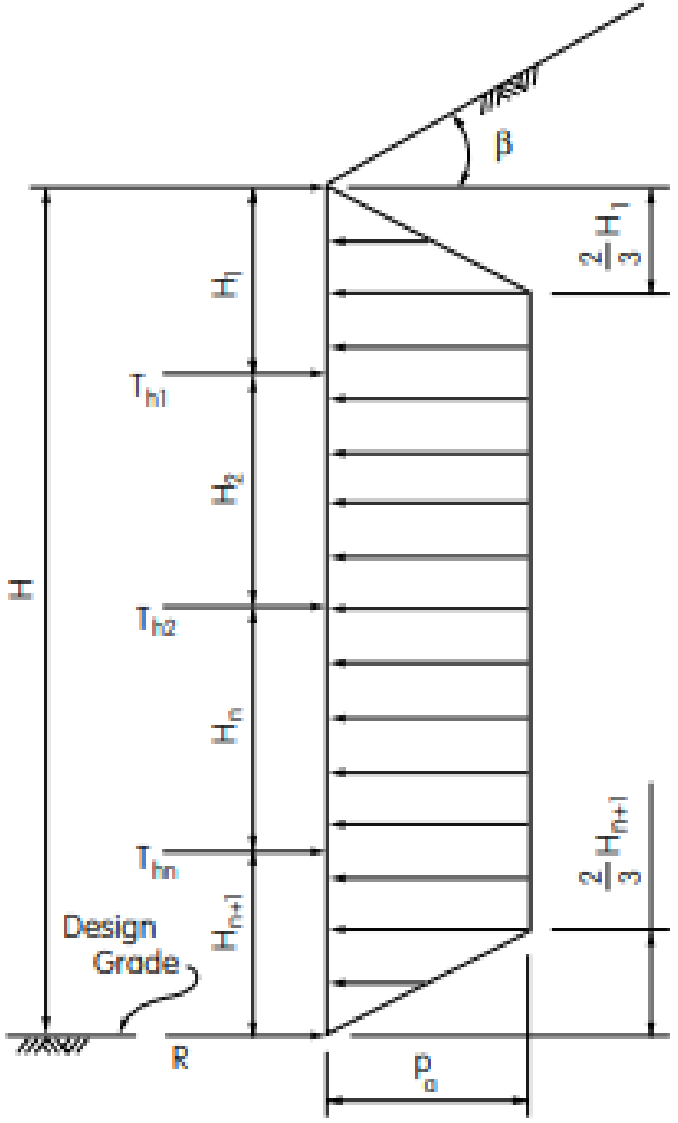
3- Extended creep tests.

An extended creep test is a long duration test (e.g., approximately 8 hours) that is used to evaluate creep deformations of anchors.

The results of these tests are compared to specified acceptance criteria to evaluate whether the ground anchor can be put into service. The acceptance criteria are based on allowable creep and elastic movements of the anchor during load testing.



a) Wall with a single level of anchors



b) Wall with multiple levels of anchors

Design steps for retaining walls with ground anchors

Step 1 Establish project requirements including all geometry, external loading conditions (temporary and/ or permanent, seismic, etc.), performance criteria, and construction constraints. Consult with Geotechnical Services for the requirements.

Step 2 Evaluate site subsurface conditions and relevant properties of the in situ soil or rock; and any specifications controlled fill materials including all materials strength parameters, ground water levels, etc. This step is to be performed by Geotechnical Services.

Step 3 Evaluate material engineering properties, establish design load and resistance factors, and select level of corrosion protection. Consult with Geotechnical Services for soil and rock engineering properties and design issues.

Step 4 Consult with Geotechnical Services to select the lateral earth pressure distribution acting on back of wall for final wall height. Add appropriate water, surcharge, and seismic pressures to evaluate total lateral pressure. Check stability at intermediate steps during contruction. Geotechnical numerical analysis may be required to simulate staged construction. Consult Geotechnical Services for the task, should it be required.

Step 5 Space the anchors vertically and horizontally based upon wall type and wall height. Calculate individual anchor loads. Revise anchor spacing and geometry if necessary.

Step 6 Determine required anchor inclination and horizontal angle based on right-of-way limitations, location of appropriate anchoring strata, and location of underground structures.

Design steps for retaining walls with ground anchors

Step 7	Resolve each horizontal anchor load into a vertical force component and a force along the anchor.
Step 8	Structure Design checks the internal stability and Geotechnical Services checks the external stability of anchored system. Revise ground anchor geometry if necessary.
Step 9	When adjacent structures are sensitive to movements Structure Design and Geotechical Services shall jointly decide the appropriate level and method of analysis required. Revise design if necessary. For the estimate of lateral wall movements and ground surface settlements, geotechnical numerical analysis is most likely required. Consult with Geotechnical Services for the task, should it be required.
Step 10	Structure Design analyzes lateral capacity of pile section below excavation sub-grade. Geotechnical Services analyzes vertical capacity. Revise pile section if necessary.
Step 11	Design connection details, concrete facing, lagging, walers, drainage systems, etc. Consult with Geotechnical Services for the design of additional drainage needs.
Step 12	Design the wall facing architectural treatment as required by the Architect.

(A) Single Level of Anchors

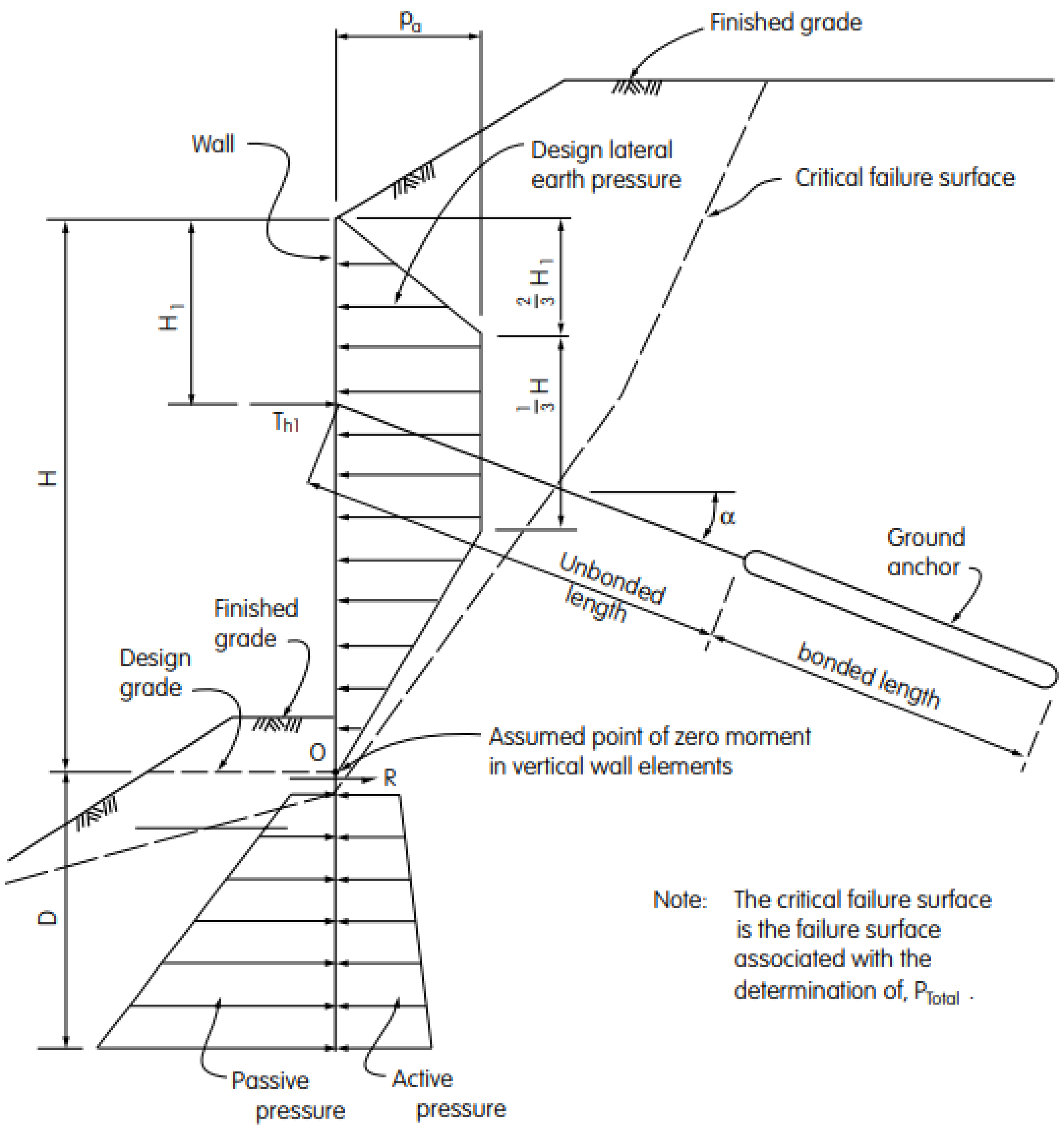


Figure 5-12.2 Anchored Wall with Single Level of Ground Anchors, Critical Failure Surface Near Bottom of Wall, $H_1 \leq = {}^1/_2\, H$

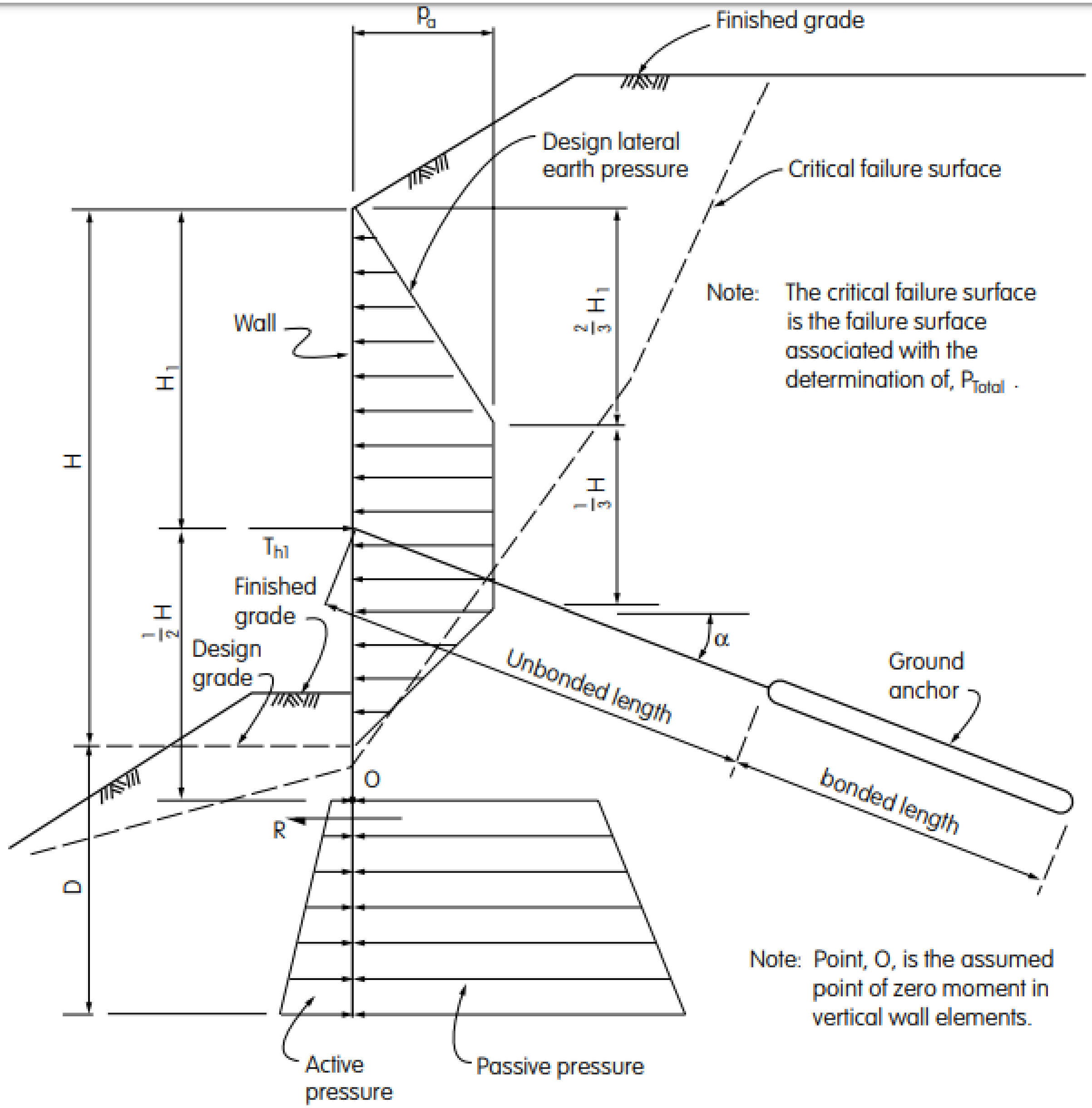


Figure 5-12.3 Anchored Wall with Single Level of Ground Anchors, Critical Failure Surface near Bottom of Wall, ${}^1/_2\, H < H_1 < {}^2/_3 H$

(B) Multiple Levels of Anchors

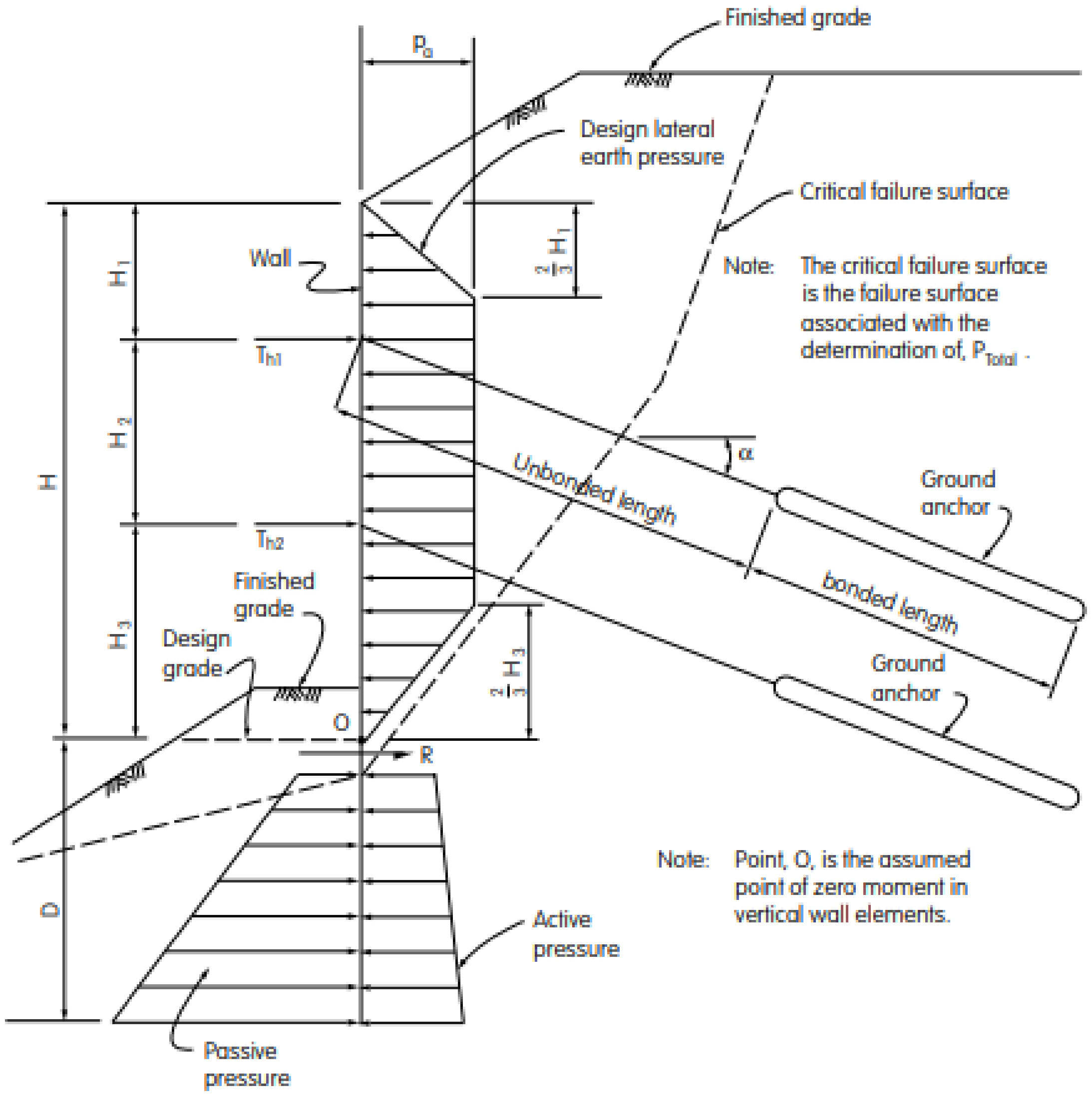


Figure 5-12.5 Anchored Wall with Multiple Levels of Ground Anchors and Critical Failure Surfaces Near Bottom of Wall

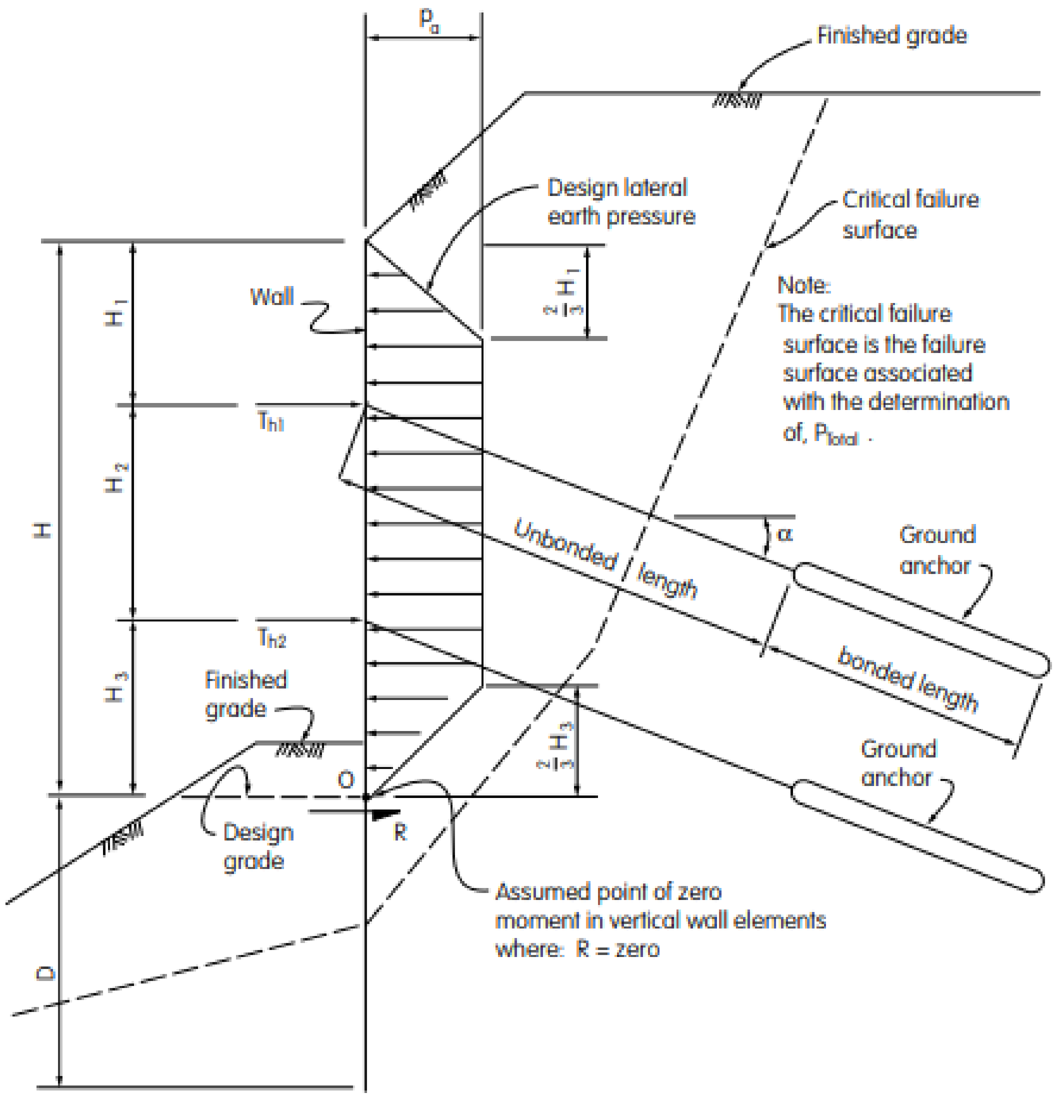
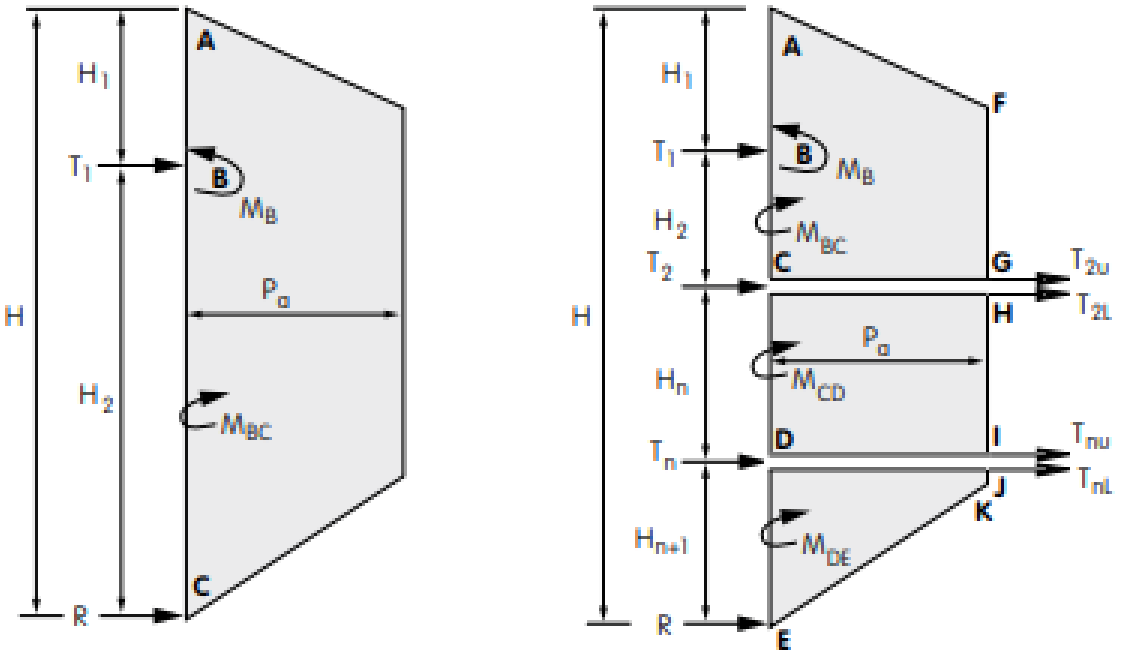


Figure 5-12.6 Anchored Wall with Multiple Levels of Ground Anchors and Critical Failure Surface a Significant Distance below the Bottom of Wall



$M_B = \Sigma M_B$

M_{BC} = Maximum moment between B and C; located at point where shear = 0

(a) Walls with one level of ground anchors

$M_B = \Sigma M_B$

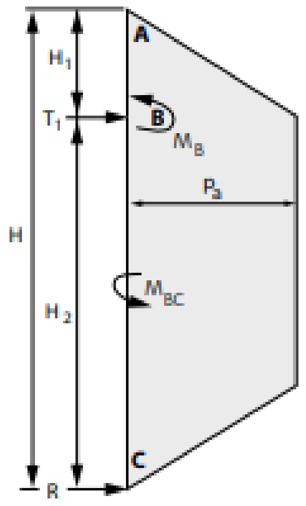
$M_C = M_D = M_E = 0$

M_{BC} = Maximum moment between B and C; located at point where shear = 0

$M_{BC} = M_{CD}$: Calculate as for M_{BC}

(b) Walls with multiple levels of ground anchors

Figure 5-12.7 Calculation of Anchor Loads Using the Hinge Method (After Figure 38, Sabatini, et al, 1999)



$$M_B = \frac{13}{54} H_1^2 \, p_a$$

$$T_1 = \frac{(23H^2 - 10\,HH_1)}{54\,(H - H_1)} \, p_a$$

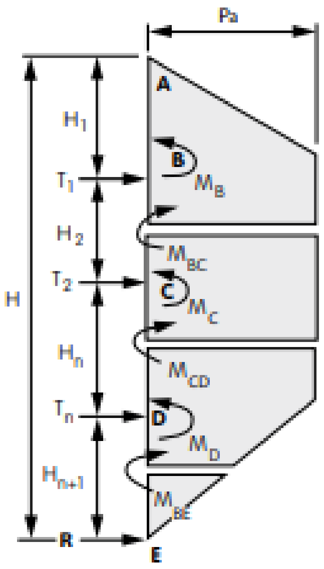
$$R = \frac{2}{3} H p - T_1$$

Solve for point of zero shear

$$x = \frac{1}{9} \sqrt{(26H^2 - 52\,HH_1)}$$

$$M_{BC} = Rx - \frac{p_a x^3}{4\,(H - H_1)}$$

(a) Walls with single level of ground anchors



$$M_B = \frac{13}{54} H_1^2 \, p_a$$

$$T_1 = \left(\frac{2}{3} H_1 + \frac{H_2}{2}\right) p_a$$

$$T_2 = \left(\frac{H_2}{2} + \frac{H_n}{2}\right) p_a$$

$$T_n = \left(\frac{H_n}{2} + \frac{23H_{n+1}}{48}\right) p_a$$

$$R = \left(\frac{3}{16} H_{n+1}\right) p_a$$

Maximum moment between B = $pL^2/10$
where L is the larger of H_2, H_n, H_{n+1}

(b) Walls with multiple level of ground anchors

Figure 5-12.8 Calculation of Anchor Loads for Multi-Level Wall Using the Tributary Area Method (After Figure 39, Sabatini, et al, 1999)