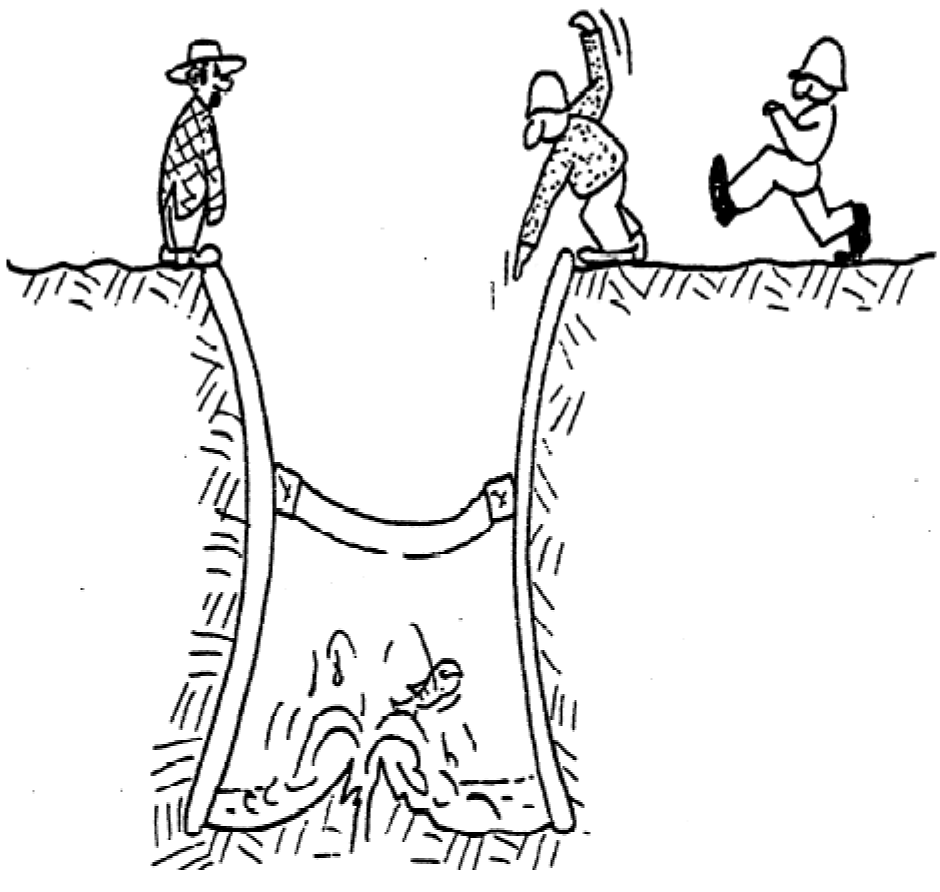
CHAPTER 9

CONSTRUCTION AND SPECIAL CONSIDERATIONS



9.0 *SPECIAL CONDITIONS*

The best shoring system in the world is of little value if the soil being supported does not act as contemplated by the designer. Adverse soil properties and changing conditions need to be considered.

Anchors placed within a soil failure wedge will exhibit little holding value when soil movement in the active zone occurs. The same reasoning holds true for the anchors or piles in soils that decrease bonding or shear resistance due to changes in plasticity or cohesion. Additional information regarding anchors may be found in the USS Steel Sheet Piling Design Manual.

When cohesive soils tend to expand or are pushed upward in an excavation, additional forces are exerted on the shoring system, which may induce lateral movement of the shoring system. Soil rising in an excavation indicates that somewhere else soil is settling. Water rising in an excavation can lead to quick conditions, while water moving horizontally can transport soil particles leaving unwanted voids at possibly critical locations.

A very important issue to consider that is present in most types of shoring systems is the potential for a sudden failure sudden failure due to slippage of the soil around the shoring system along a surface offering the least amount of resistance.

Sample situations of the above are included on the following pages.

9.1 *ANCHOR BLOCK*

The lateral support for sheet pile and/or soldier pile walls can be provided by tie rods that extend to a concrete anchor block (deadman) or a continuous wall as shown in Figure 9-1. Tie rod spacing is a function of wall height, wall backfill properties and the soil-foundation properties below the dredge line, structural properties of the wall and the anchor block.

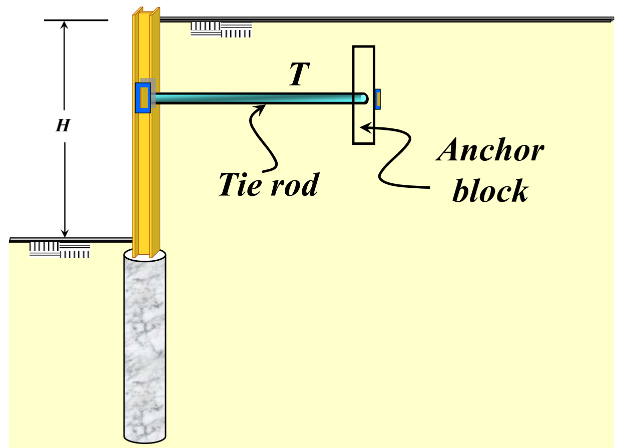


Figure 9-1. Anchor Block and Tie Rod

The size, shape, depth and location of an anchor block affect the resistance capacity developed by that anchor. Figure 9-2 explains how the distance from the wall affects capacity

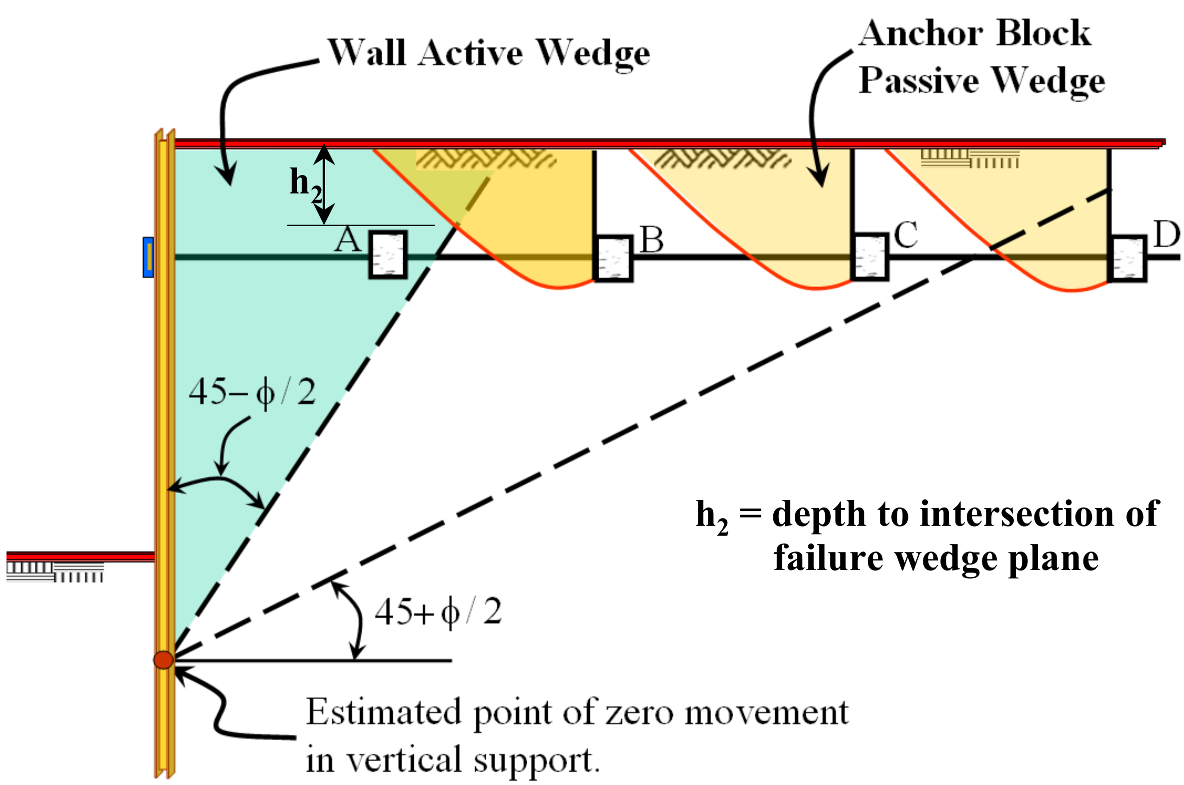


Figure 9-2. Anchor block

Anchor block A is located inside active wedge and offers no resistance.

Anchor block B resistance is reduced due to overlap of the active wedge (wall) and the passive wedge (anchor).

Anchor reduction: (Granular soils)

$$\Delta P_p = \frac{\gamma h_2^2 \left(K_p - K_a\right)}{2} \qquad \text{Eq. 9-1}$$

ΔP_P is transferred to the wall.

Anchor block C develops full capacity but increases pressure on wall.

Anchor block D develops full capacity and has no effect on bulkhead.

Anchor blocks should be placed against firm natural soil and should not be allowed to settle.

A safety factor of 2 is recommended for all anchors and anchor blocks.

The following criteria are for anchors or anchor blocks located entirely in the passive zone as indicated by Anchor D.

9.1.1 Anchor Block in Cohesionless Soil

Case A – Anchor block extends to the ground surface

The forces acting on an anchor are near the ground surface is shown in Figure 9-3.

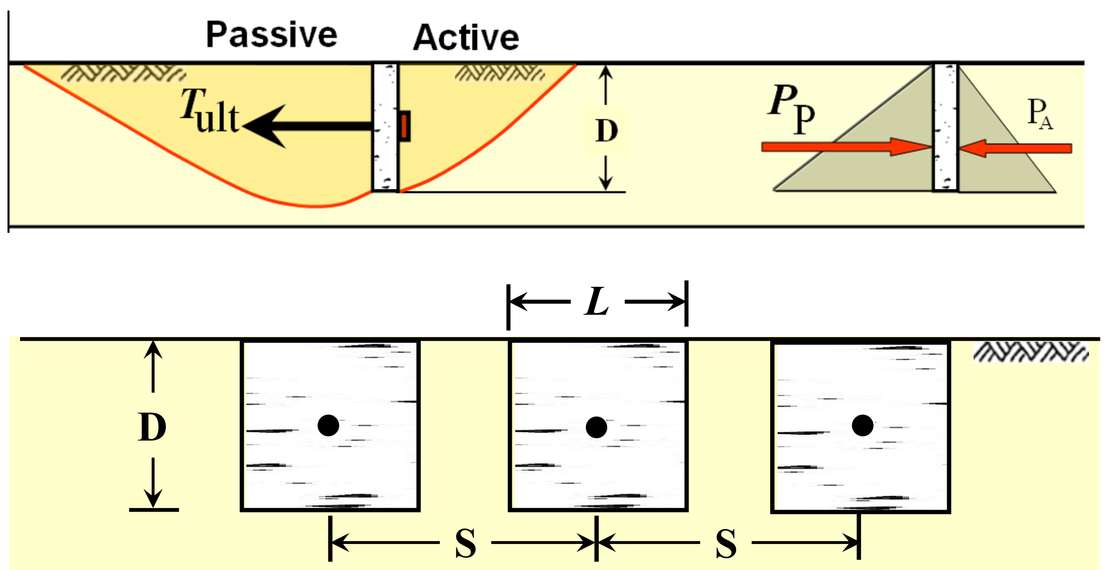


Figure 9-3. Anchor block in cohesionless soil near ground surface

The capacity of an anchor block also depends on whether it is continuous or isolated. An anchor block is considered continuous when its length greatly exceeds it height. The conventional earth pressure theories using two-dimensional conditions corresponding to a long wall can be used to calculate the resistance force against the anchor block movement.

The basic equation for a continuous anchor block is shown below:

$$T_{ult} = L(P_p - P_a) \qquad \text{Eq. 9-2}$$

Where:

$$P_a = K_a \gamma \frac{D^2}{2} \qquad \text{Eq. 9-3}$$

$$P_p = K_p \gamma \frac{D^2}{2} \qquad \text{Eq. 9-4}$$

Substituting Eq. 9-3 and Eq. 9-4 into Eq. 9-2 then:

$$T_{ult} = \gamma D^2 \Delta K \frac{L}{2} \qquad \text{Eq. 9-5}$$

$$\Delta K = (K_p - K_a) \qquad \text{Eq. 9-6}$$

L = Length of the anchor block.

In case of isolated and short anchor blocks a large passive pressure may develop because of three-dimensional effects due to a wider passive zone in front of the anchor block as shown below.

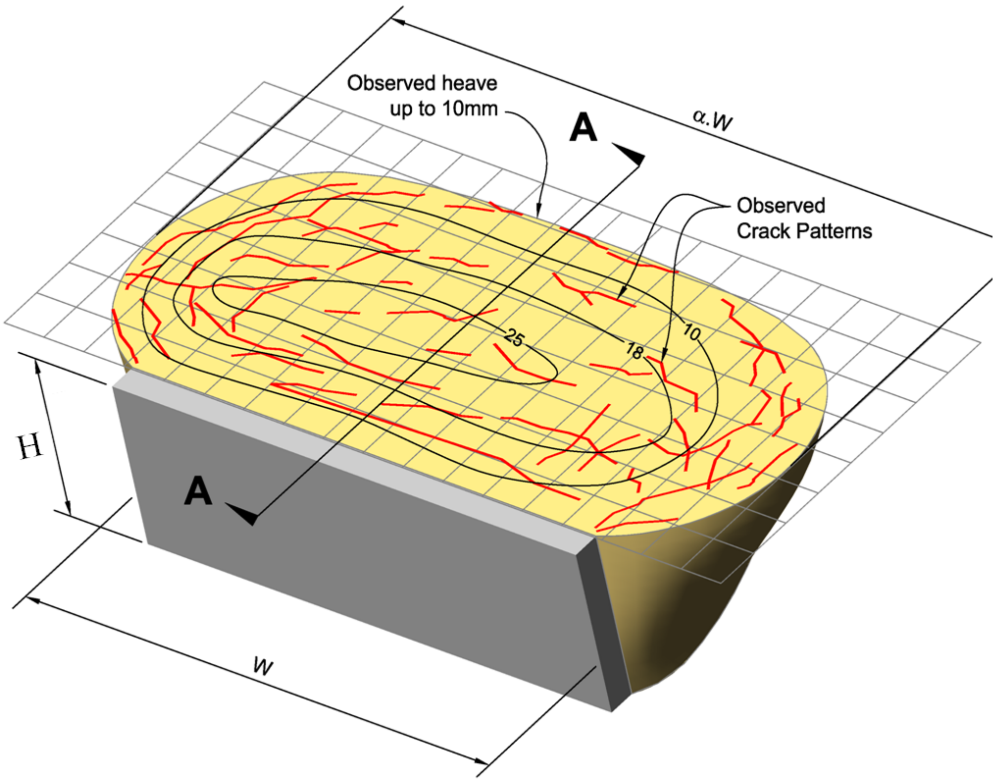


Figure 9-4. Anchor block in 3D (Shamsabadi, A., Nordal, S. (2006))

Figure 9-5. Section A-A (Shamsabadi, A., *et al*., 2007).

The ratio between three-dimensional and two-dimensional soil resistance varies with the soil friction angle and the depth below the ground surface. Ovesen's theory can be used to estimate the magnitude of the three-dimensional effects as shown below.

$$T_{ult} = R\left[\gamma D^2 \Delta K \frac{L}{2}\right] \qquad \text{Eq. 9-7}$$

Where,

$$R = 1 + \Delta K^{2/3}\left[1.1E^4 + \frac{1.6B}{1 + 5\frac{L}{D}} + \frac{0.4\Delta K E^3 B^2}{1 + 0.05\frac{L}{D}}\right] \qquad \text{Eq. 9-8}$$

$$B = 1 - \frac{L}{S} \qquad \text{Eq. 9-9}$$

$$E = 1 - \frac{H}{d + H} \qquad \text{Eq. 9-10}$$

Case B – Anchor block does not extend to the ground surface.

The forces acting on an anchor, which is not near the ground surface is shown in Figure 9-6.

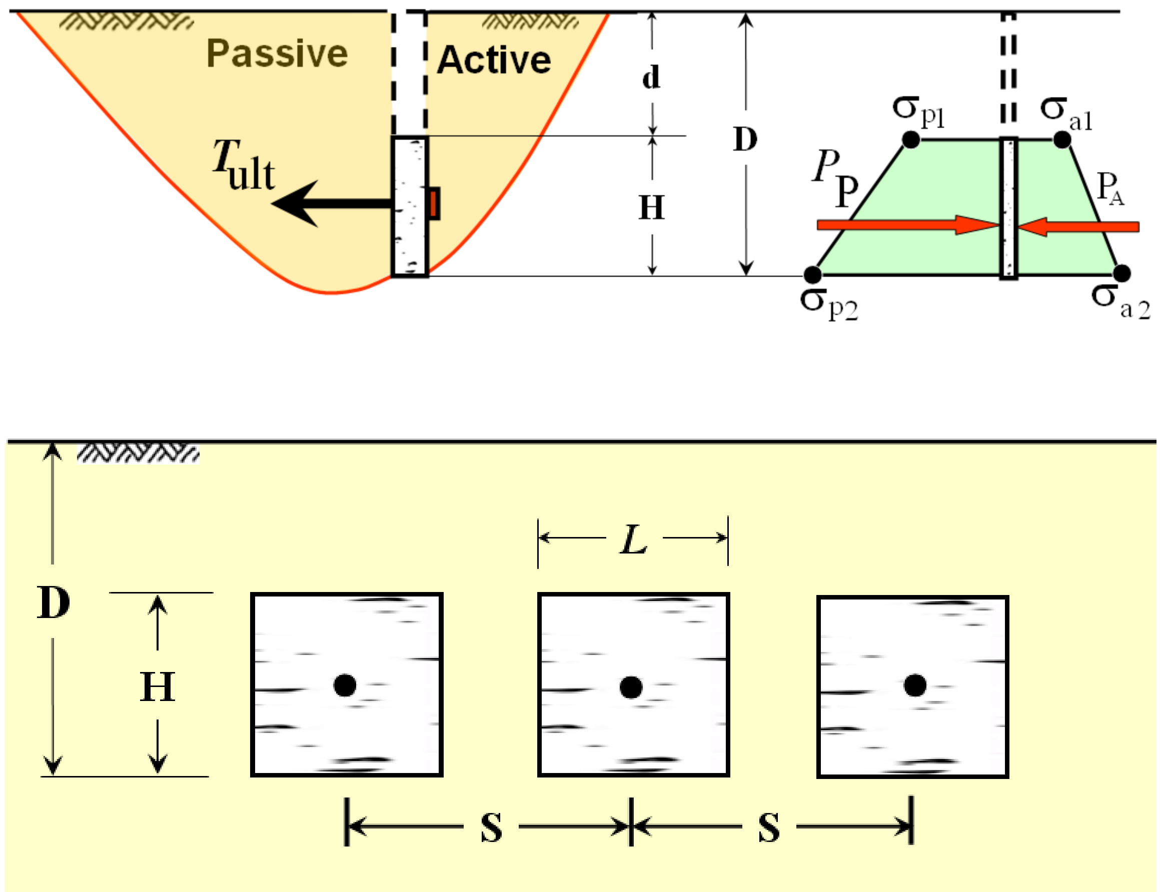


Figure 9-6. Anchor block not near the ground surface

The basic equation to calculate the capacity of a continuous anchor block with length L, not extended near the ground surface is shown Eq. 9-2.

$$T_{ult} = L(P_P - P_A)$$

Where the PA and PP are the areas of active and passive earth pressure developed in the front and back of the anchor block as shown in Figure 9-6 and Eq. 9-13 and Eq. 9-16.

$$\sigma_{a1} = \gamma d k_a \qquad \text{Eq. 9-11}$$

$$\sigma_{a2} = \gamma D k_a \qquad \text{Eq. 9-12}$$

$$P_A = \left[\frac{\sigma_{a1} + \sigma_{a2}}{2}\right] H \qquad \text{Eq. 9-13}$$

$$\sigma_{p1} = \gamma d k_p \qquad \text{Eq. 9-14}$$

$$\sigma_{p2} = \gamma D k_p \qquad \text{Eq. 9-15}$$

$$P_P = \left[\frac{\sigma_{p1} + \sigma_{p2}}{2}\right] H \qquad \text{Eq. 9-16}$$

L = Length of the anchor block

In case of isolated and short anchor blocks the Ovesen's three-dimensional factor (R) shall be estimated using Eq. 9-17.

$$R = 1 + \Delta K^{2/3}\left[1.1E^4 + \frac{1.6B}{1 + 5\frac{L}{D}} + \frac{0.4\Delta K E^3 B^2}{1 + 0.05\frac{L}{D}}\right] \qquad \text{Eq. 9-17}$$

$$B = 1 - \frac{L}{S}$$

$$E = 1 - \frac{H}{d + H}$$

9.1.2 Anchor Block in Cohesionless Soil where 1.5 ≤ D/H ≤ 5.5

The chart shown in Figure 9-7 is based on sand of medium density, ($\phi = 32.5$). For other values of ϕ, a linear correlation may he made from ($\phi/32.5$). The chart is valid for ratios of depth to height of anchor (*D/H*) between 1.5 and 5. 5.

For square anchor blocks the value from the chart ($K_p^{'}$) is larger than the value for continuous anchor blocks (K_p). This is because the failure surface is larger than the actual dimensions of the anchor block. In testing it is determined to be approximately twice the width.

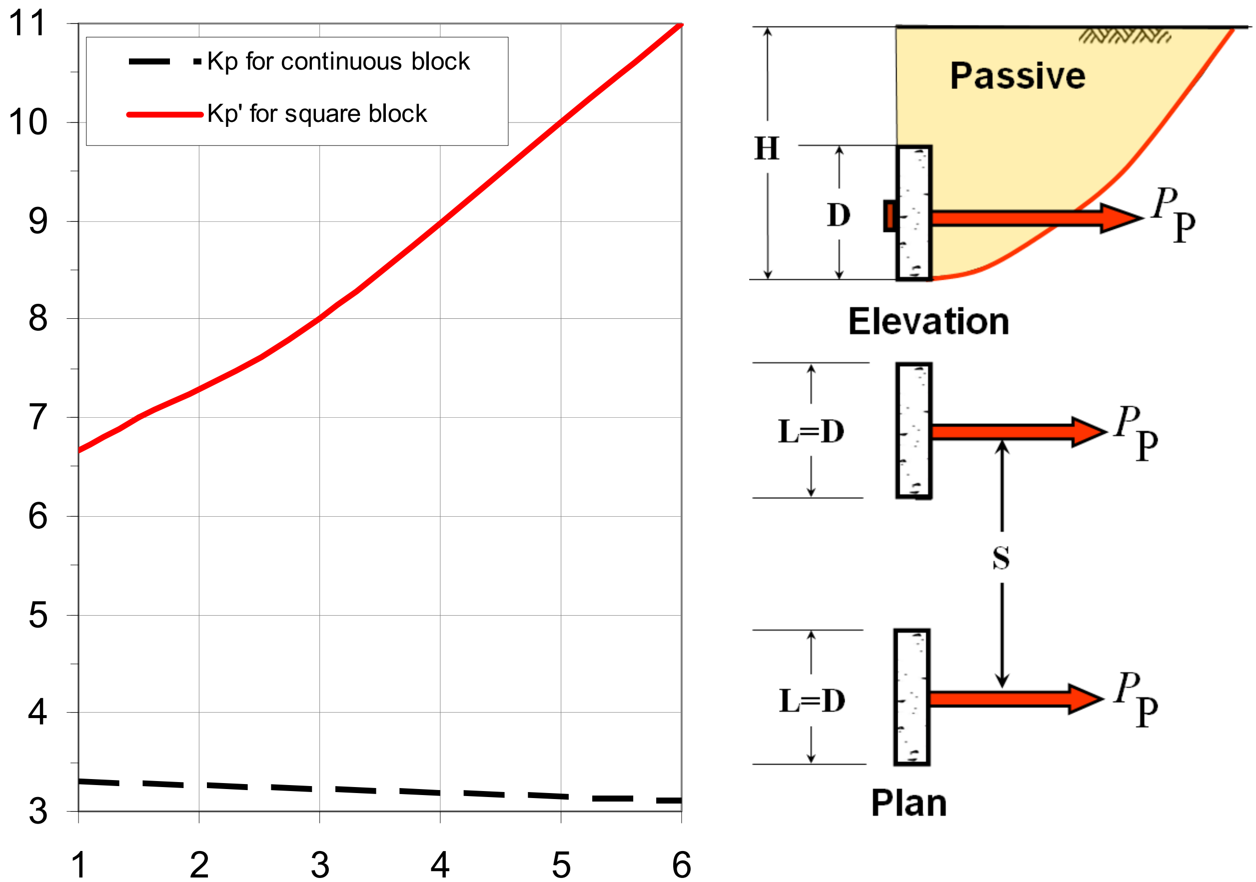


Figure 9-7. Anchor block in cohesionless soil 1.5 ≤ D/H ≤ 5.5

For continuous anchor blocks:

Use Ovesen's equations to estimate the magnitude of the three-dimensional factor (R) as shown above.

For square (or short) anchor blocks where $D = L$.

$$P_{ult} = \frac{\gamma H^2 K_p^{'} L}{2} \qquad \text{Eq. 9-18}$$

It is recommended that a minimum factor of safety of 2 be used.

9.1.3 Anchor Block in Cohesive Soil near the Ground Surface D ≤ H/2

The forces acting on an anchor are shown in the Figure 9-8. For this case, D ≤ H/2 (Figure 9-6) where H is the height of the block, it is assumed that the anchor extends to the ground surface. Capacity of the anchor depends upon whether it is considered continuous or short.

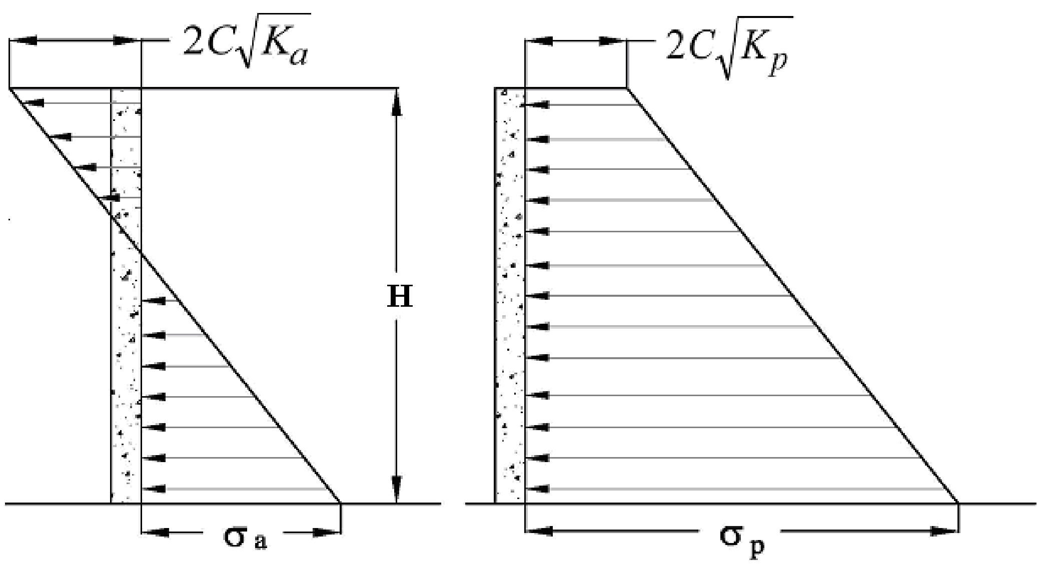


Figure 9-8. Anchor block in cohesive soil near ground D ≤ H/2

Where:

$$\sigma_p = \gamma D K_p + 2C\sqrt{K_p}\,. \qquad \text{Eq. 9-19}$$

$$\sigma_a = \gamma D K_a - 2C\sqrt{K_a}\,. \qquad \text{Eq. 9-20}$$

The pressure diagram for cohesive soils assumes short load duration. For duration of a period of years it is likely that creep will change the pressure diagram. Therefore, conservative assumptions should be used in the analysis, such as $c = 0$ and $\phi = 27°$.

The basic equation is:

$$T_{ult} = L(P_p - P_a) \qquad \text{Eq. 9-21}$$

Where L = Length of anchor block.

For continuous anchor blocks:

$$P_p = \frac{\gamma D^2 K_p}{2} + 2CD\sqrt{K_p} \qquad \text{Eq. 9-22}$$

$$P_a = \frac{\left(\gamma D K_a - 2C\sqrt{K_a}\right)\left(D - \frac{2C}{\gamma}\right)}{2} \qquad \text{Eq. 9-23}$$

It is recommended that the tension zone be neglected.

For short anchor blocks where $D = L$:

$$T_{ult} = L(P_p - P_a) + 2CD^2 \qquad \text{Eq. 9-24}$$

9.1.4 Anchor Blocks in Cohesive Soil where D ≥ H/2

The chart shown in Figure 9-9 was developed through testing for anchor blocks other than near the surface. The chart relates a dimensionless coefficient (*R*) to the ratios of depth to height of an anchor (*D/H*) to determine the capacity of the anchor block. The chart applies to continuous anchors only.

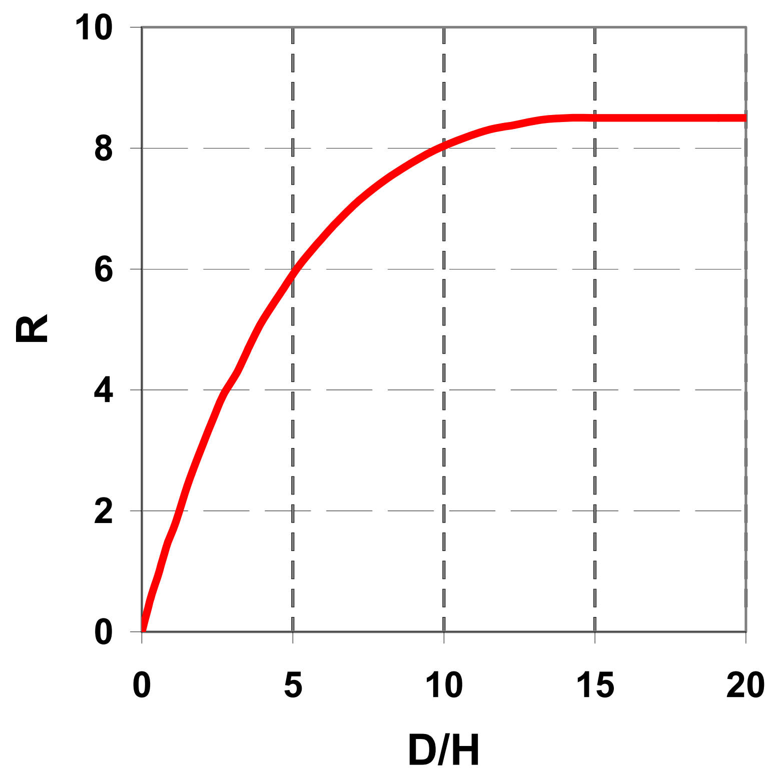


Figure 9-9. Anchor block in cohesive soil D ≥ H/2

The above graph is from Strength of Deadmen Anchors in Clay, Thomas R. Mackenzie, Master's Thesis Princeton University, Princeton, New Jersey, 1955.

$P_{ult} = RCHL$ with a maximum value of $R = 8.5$.

It is recommended that a minimum factor of safety of 2 be used.

9.1.4.1 Example 9-1 Problem – Anchor Blocks

Given:

Check the adequacy of contractor's proposed shoring system shown in Figure 9-10. The 2x2 anchor blocks are to be buried 3' below the ground surface. The required tie load on the wall is 11,000 lbs.

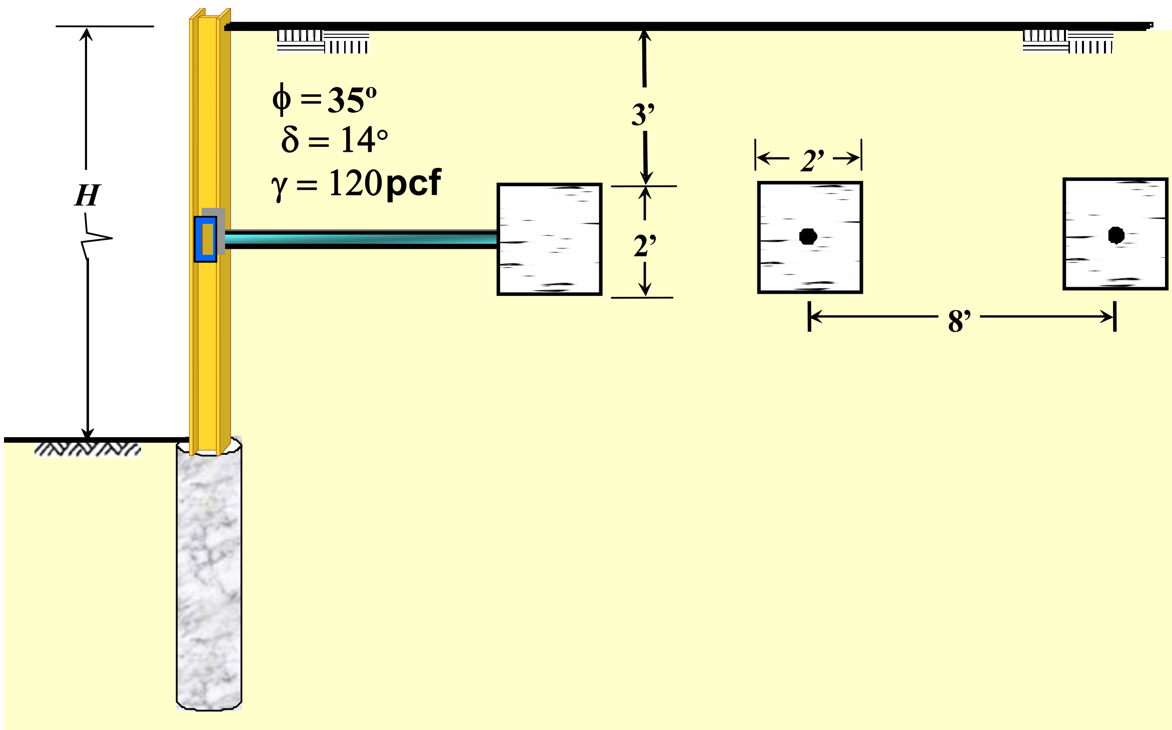


Figure 9-10. Anchor blockExample 9-1

Solution:

***Step 1:* Calculate active and passive earth pressure in the front and back of the anchor block shown in Figure 9-11.**

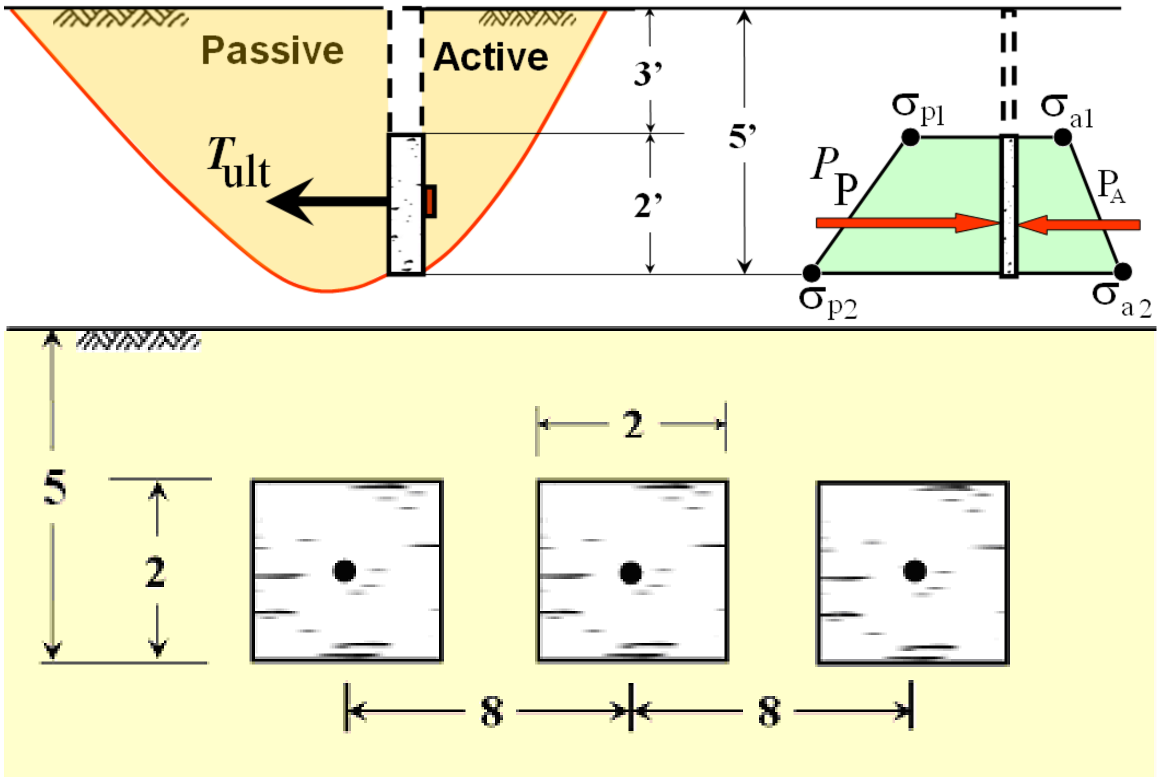


Figure 9-11. Anchor block Example 9-1 solution

$$K_p = 6.27$$
$$K_a = 0.27$$

$$\sigma_{a1} = \gamma d k_a * \cos(\delta) = 120 * 3 * 0.27 * \cos(14°) = 94.31$$
$$\sigma_{a2} = \gamma H k_a * \cos(\delta) = 120 * 5 * 0.27 * \cos(14°) = 157.19$$

$$P_A = \left[\frac{94.31 + 157.19}{2}\right] 2 = 251.50$$

$$\sigma_{p1} = \gamma d k_p * \cos(\delta) = 120 * 3 * 6.27 * \cos(14°) = 2{,}190.15$$
$$\sigma_{p2} = \gamma H k_p * \cos(\delta) = 120 * 5 * 6.27 * \cos(14°) = 3{,}650.25$$

$$P_P = \left[\frac{2{,}190.15 + 3{,}650.25}{2}\right] 2 = 5{,}840.40$$

Step 2: **Use Ovesen's theory to estimate the magnitude of the three-dimensional effects R, using Eq. 9-17.**

$$R = 1 + \Delta K^{2/3}\left[1.1E^4 + \frac{1.6B}{1+5\frac{L}{H}} + \frac{0.4\Delta K E^3 B^2}{1+0.05\frac{L}{H}}\right]$$

$$\Delta K_{horz} = (K_p - K_a)\cos(\delta) = \left(6.27 - 0.27\right)\cos(14°) = 5.82$$

$$B = 1 - \left(\frac{L}{S}\right)^2 = 1 - \left(\frac{2}{8}\right)^2 = 0.94$$

$$E = 1 - \frac{H}{d+H} = 1 - \frac{2}{3+2} = 0.60$$

$$R = 1 + 5.82^{2/3}\left[1.1 * 0.6^4 + \frac{1.6 * 0.94}{1+5\frac{2}{2}} + \frac{0.4 * 5.82 * 0.60^3 * 0.94^2}{1+0.05\frac{2}{2}}\right] = 3.46$$

$$R = 3.46 > 2.0 \quad Use \quad R = 2$$

Step 3: **Calculate ultimate anchor block capacity**, T_{ult}.

$$T_{ult} = R * (P_p - P_a) * L = 2 * (5{,}840.40 - 251.50) * 2 = 22{,}355.6 \text{ lb/ft}$$

Where L is the length of the anchor block.

$$FS = \frac{T_{ult}}{T} = \frac{22{,}355.6}{11{,}000.0} = 2.03$$

9.2 *HEAVE*

The condition of heave can occur in soft plastic clays when the depth of the excavation is sufficient to cause the surrounding clay soil to displace vertically with a corresponding upward movement of the material in the bottom of the excavation.

The possibility of heave and slip circle failure in soft clays, and in the underlying clay layers, should be checked when the Stability Number (N_O) exceeds 6.

Stability Number, $N_O = \gamma H/C$ Eq. 9-25

Where:

γ = Unit weight of the soil in pcf

H = Height of the excavation in ft

C = Cohesion of soil in psf

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 is shown in Figure 9-12. The vertical load per unit length of cut at the bottom of the cut along line *dc* is:

$$Q = W + (0.7B)q - S \qquad \text{Eq. 9-26}$$

Where:

Q = Vertical load per unit length.

W = Weight of soil column in pounds = γH.

B = Width of open excavation in feet.

q = Surcharge loading in psf.

S = Resistance of soil due to cohesion over depth of excavation H (cH) in plf

c = Cohesion of soil in psf.

H = Height of excavation in feet.

The load Q may be treated as a load per unit length on a continuous foundation at the level of dc and having a width of $0.7B$. Based on Terzaghi's bearing capacity theory, the net ultimate load-carrying capacity per unit length is:

$$Q_U = cN_C(0.7B) \qquad \text{Eq. 9-27}$$

Where:

Q_U = Ultimate load carry capacity per unit length.
c = Cohesion of soil in psf.
N_c = Bearing capacity factor from Figure 9-13.
B = Width of open excavation in feet.

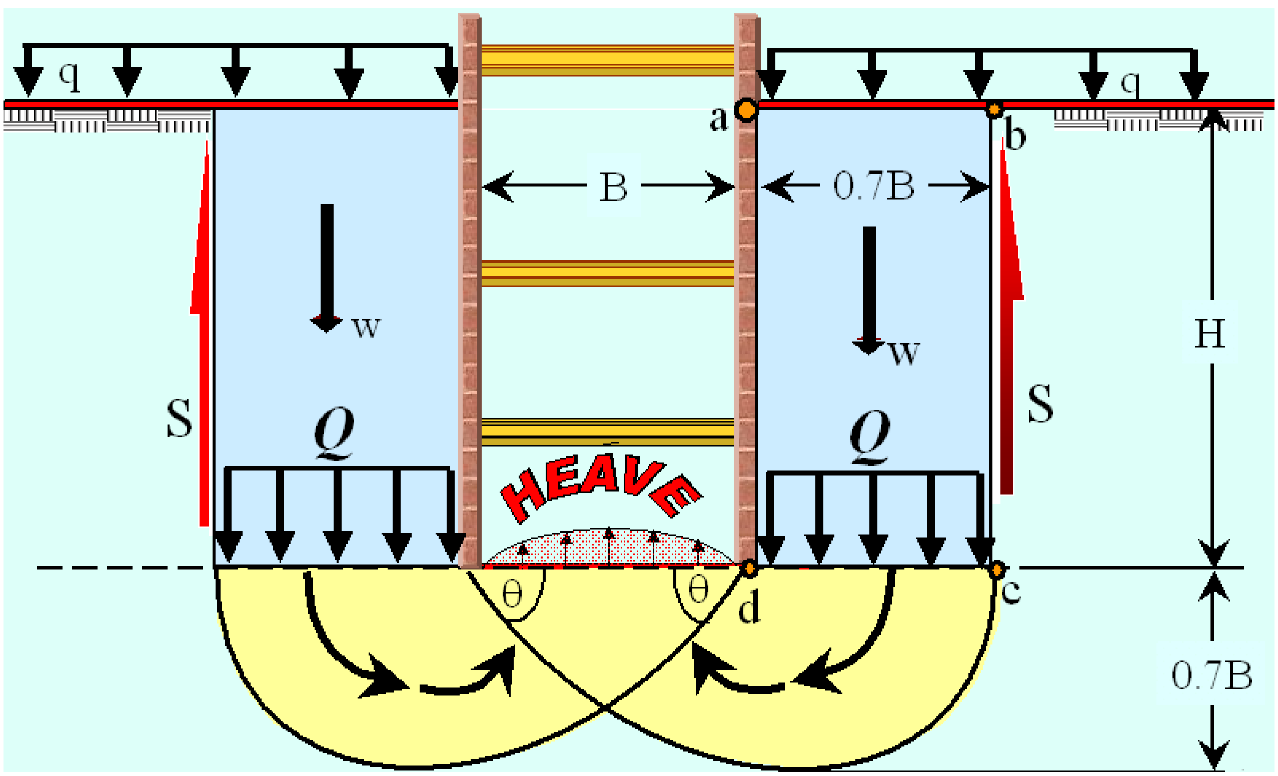
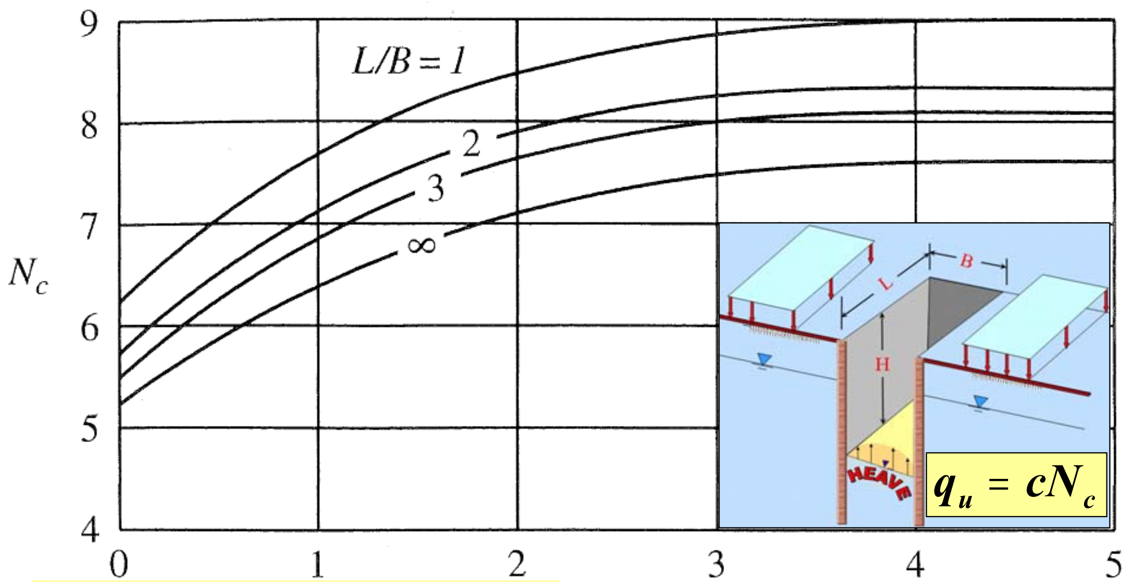


Figure 9-12. Bottom Heave

It is recommended that a minimum safety factor of 1.5 should be used.

If the analysis indicates that heave is probable, modifications to the shoring system may be needed. The sheeting may be extended below the bottom of the excavation into a more stable layer, or for a distance of one-half the width of the excavation (typically valid for only excavations where H>B). Another possible solution when in submerged condition or when in clay could be to over-excavate and construct a counterweight to the heaving force.

NOTE - Strutting a wall near its bottom will not prevent heave but such strutting may prevent the wall from rotating into the excavation.



$$N_c = \left[0.84 + 0.16\frac{B}{L}\right] * N_{c\ square}$$

Figure 9-13. Bearing Capacity Factor

9.2.1 Factor of Safety Against Heave

The factor of safety against bottom heave as shown in Figure 9-14 is:

$$FS = \frac{F_{RS}}{F_{DR}} \geq 1.5 \qquad \text{Eq. 9-28}$$

Where:

F_{RS} = Resisting Force = Q_U from Eq. 9-27.

F_{DR} = Driving Force = Q from Eq. 9-26

This factor of safety is based on the assumption that the clay layer is homogeneous, at least to a depth of 0.7*B* below the bottom of the excavation. However, if a hard layer of rock or rocklike material at a depth of $D<0.7B$ is present, then the failure surface will need to be modified to some extent. Note that *B'* shown in Figure 9-14 below is equal to 0.7*B*.

The bearing capacity factor, *Nc*, shown in Figure 9-13, varies with the ratios of *H/B* and *L/B*. In general, for *H/B*:

$$N_{c(rectangle)} = N_{c(square)}\left(0.84 + 0.16\frac{B}{L}\right) \qquad \text{Eq. 9-29}$$

Where:

$N_{c(square)}$ = Bearing capacity factor based on $L/B=1$.

B = Width of excavation in feet

L = Length of excavation in feet

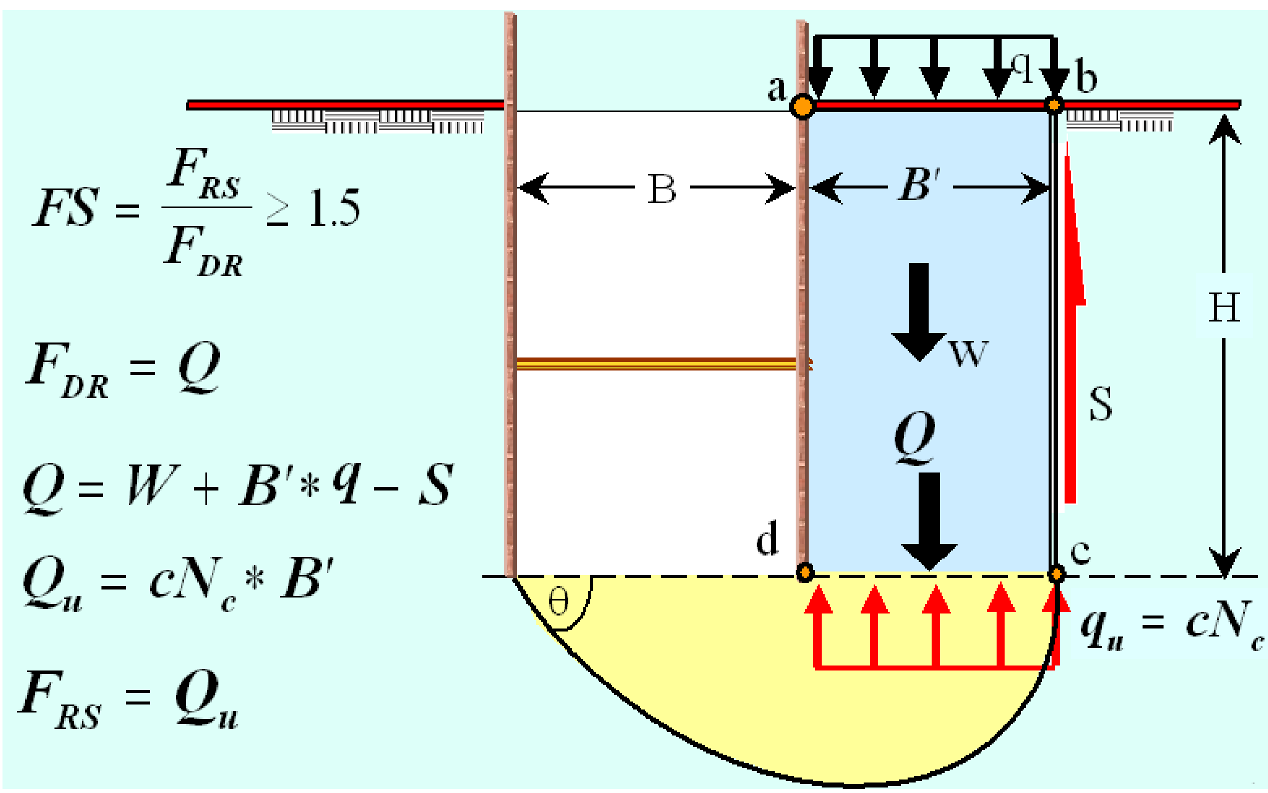


Figure 9-14. Factor of safety

9.2.1.1 Example 9-2 Problem – Heave Factor of Safety

Given: $H = 30'$, $B = 15'$, $L = 45'$

$q = 300$ psf, $c = 500$ psf, $\gamma = 120$ pcf

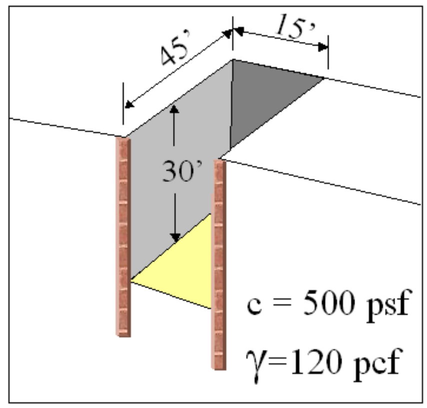
Solution:

Note in the following example *B' = 0.7B = 0.7(15) = 10.5'*.

$N_{c(square)}$ as determined from Figure 9-13 for *H/B= 2* is *8.5*.

$$H/B = 2.0 \qquad L/B = 3$$

$$q_u = cN_c$$



$$N_c = \left[0.84 + 0.16\frac{B}{L}\right] * N_{c\ square}$$

$$N_c = \left[0.84 + 0.16\frac{15}{45}\right] * 8.5 = 7.60$$

$$q_u = 0.50 * 7.60 = 3.80\ ksf$$

$$F_{DR} = W + B' * q - S$$

$$W = (10.5 * 30)0.120 = 37.8$$

$$B' * q = 10.5 * 0.30 = 3.15$$

$$S = 0.5 * 30 = 15$$

$$F_{DR} = 37.8 + 3.15 - 15 \approx 26.0$$

$$F_{RE} = q_u B' = 3.8 * 10.5 \approx 40.0$$

$$FS = \frac{40.0}{26.0} = 1.54 \geq 1.5$$

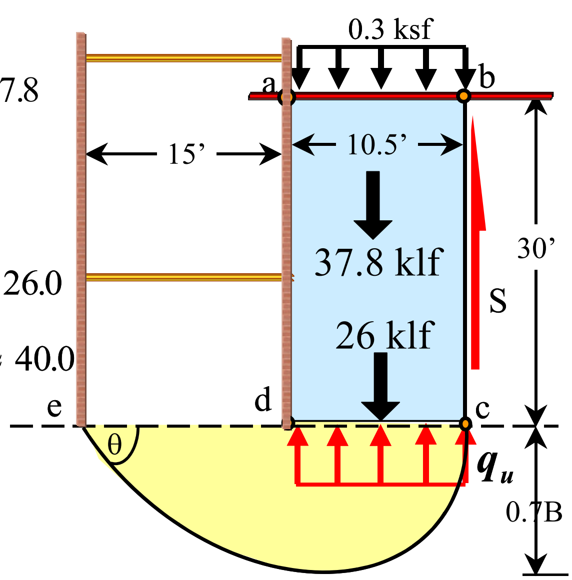


Figure 9-15. Heave example problem

9.3 *PIPING*

For excavations in pervious materials (sands), the condition of piping can occur when an unbalanced hydrostatic head exists. This causes large upward flows of water through the soil and into the bottom of the excavation. This movement of water into the excavation will transport material and will cause settlement of the soil adjacent to the excavation, if the piping is allowed to continue. This is also known as a sand boil or a quick condition. The passive resistance of embedded members will be reduced in this condition.

To correct this problem, either equalize the unbalanced hydraulic head by allowing the excavation to fill with water or lower the water table outside the excavation by dewatering. On Caltrans projects, one of the common methods used to protect or mitigate against piping is the use of a seal course. Refer to BCM 130-17.0 for additional information regarding seal course construction.

If the embedded length of the shoring system member is long enough, the condition of piping should not develop. Charts giving lengths of sheet pile embedment, which will result in an adequate factor of safety against piping, are shown on page 65 of the USS Steel Sheet Piling Design Manual. These charts are of particular interest and a good resource for cofferdams.

9.3.1 Hydraulic Forces on Cofferdams and Other Structures

Moving water imposes not only normal forces acting on the normal projection of the cofferdam but also substantial forces in the form of eddies can act along the sides of sheet piles as shown in the figure below. The drag force (D) in equation form (after Ratay) is:

$$D = (A)(C_d)(\rho)\frac{V^2}{2g} \qquad \text{Eq. 9-30}$$

Where:

ρ = Water density in lbs/ft³

A = Projected area of the obstruction normal to the current in ft²

C_d = Coefficient of drag, dimensionless

V = Velocity of the current in ft/sec

g = Acceleration due to gravity ft/sec²

In English units $\rho \approx 2g$ so that:

$$D = (A)(C_d)(V^2) \qquad \text{Eq. 9-31}$$

Where:

A = Projected area of the obstruction normal to the current in ft^2

V = Velocity of the current in ft/sec

C_d = Coefficient of drag, lbs sec^2/ft^4 (Note: C_d is not dimensionless in the above equation for D to be in lbs.)

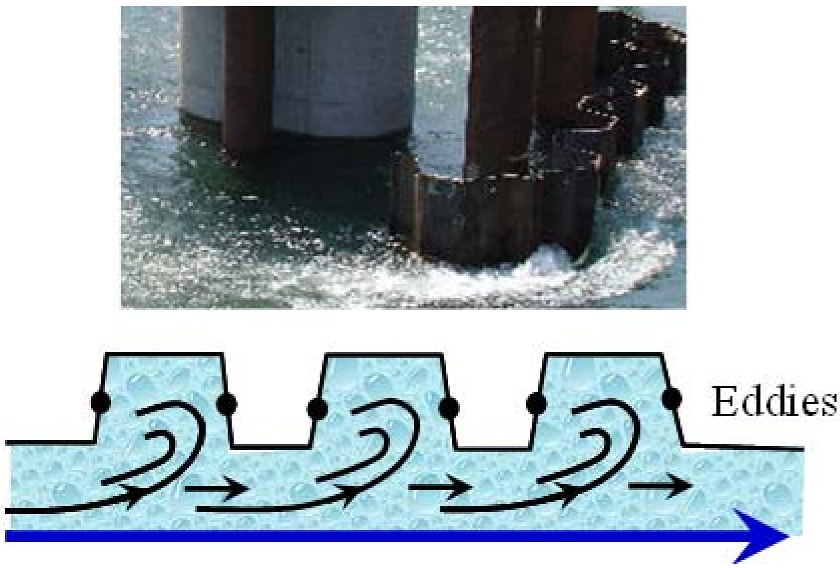


Figure 9-16. Hydraulic forces on cofferdams

Considering the roughness along the sides of the obstructions (as for a sheetpile cofferdam) the practical value for $C_d = 2.0$.

$$D = 2AV^2$$

Which may be considered to be applied in the same manner as a wind rectangular load on the loaded height of the obstruction.

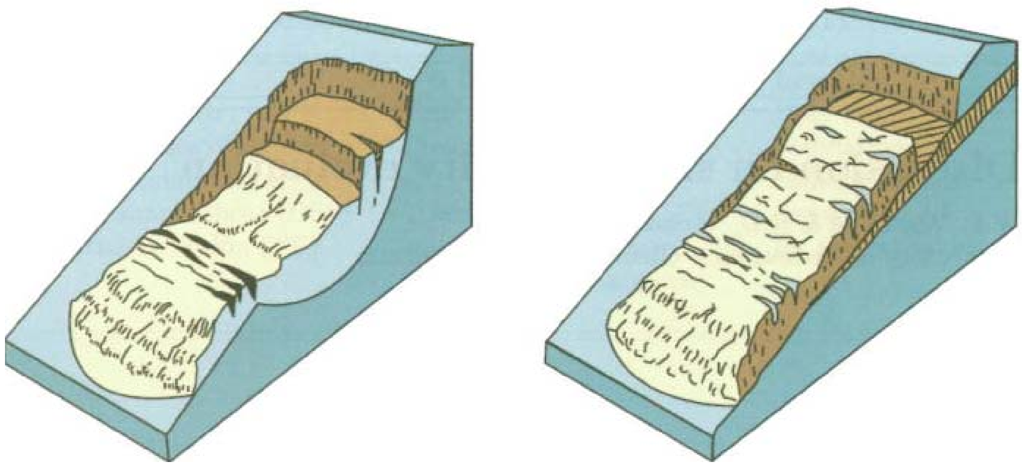
Example: Determine the drag force on a six foot diameter corrugated metal pipe placed vertically in water of average depth of 6 feet flowing at 4 feet per second.

Projected Area $= 6(6) = 36\ \text{ft}^2$.

$D = 2(36)(4)^2 = 1{,}152$ lbs.

9.4 *SLOPE STABILITY*

When the ground surface is not horizontal at the construction site, a component of gravity may cause the soil to move in the direction of the slope. Slopes fail in different ways. Figure 9-17 shows some of the most common patterns of slope failure in soil. The slope failure of rocks is out of the scope of this Manual.



a-Rotational Slide b-Translational Slide

Figure 9-17. Common pattern of soil slope failure (USGS)

A slope stability analysis can be very complex and is most properly within the realm of geotechnical engineering. In many cases, construction engineers and geotechnical engineers are expected to perform a slope stability analysis to check the safety of an excavated slope. There are various computer programs for slope stability analyses, using conventional limited equilibrium method or the strength reduction method based on finite element analysis.

The fundamental assumption of the limit-equilibrium method is that failure occurs when a mass of a soil slides along a slip surface as shown in Figure 9-17. The popularity of limit-equilibrium methods is primarily due to their relative simplicity, and the many years of experience analyzing slope failures.

Construction equipment, stockpiles and other surcharges, which may cause excavation instabilities, should be considered when performing a slope stability analysis. The slope stability analysis involves the following:

- Obtain surface geometry, stratigraphy and subsurface information
- Determine soil shear strengths
- Determine soil-structure-interaction such as presence of sheet piles, soldier piles tieback, soil nails and so forth
- Determine surcharge loads
- Perform slope stability analysis to calculate the minimum factor of safety against failure for various stage constructions

The stability of an excavated slope is expressed in terms of the lowest factor of safety, FS, found utilizing multiple potential failure surfaces. Circular solutions to slope stability have been developed primarily due of the ease this geometry lends to the computational procedure. The most critical failure surface will be dependent on site geology and other factors mentioned above. However, the most critical failure surface is not necessarily circular as shown in Figure 9-17. Non-circular failure surfaces can be caused by adversely dipping bedding planes, zones of weak soil or unfavorable ground water conditions.

9.4.1 Rotational Slides

Slope stability analysis of slopes with circular failure surfaces can be explained using method of slices as shown in Figure 9-18 in which AB is an arc of a circle representing a trial failure surface. The soil above the trial surface is divided into number of slices. The forces acting on a typical slice i are shown in Figure 9-18 b, c and d. The ordinary method of slices, which is the simplest method, does not consider interslice forces acting on the side of the slices. The Simplified Bishop's Method of Slices accounts only for the horizontal interslice forces while more refined methods such as Spencer's solution, accounts for both vertical and horizontal interslice forces acting on each side of the slice.

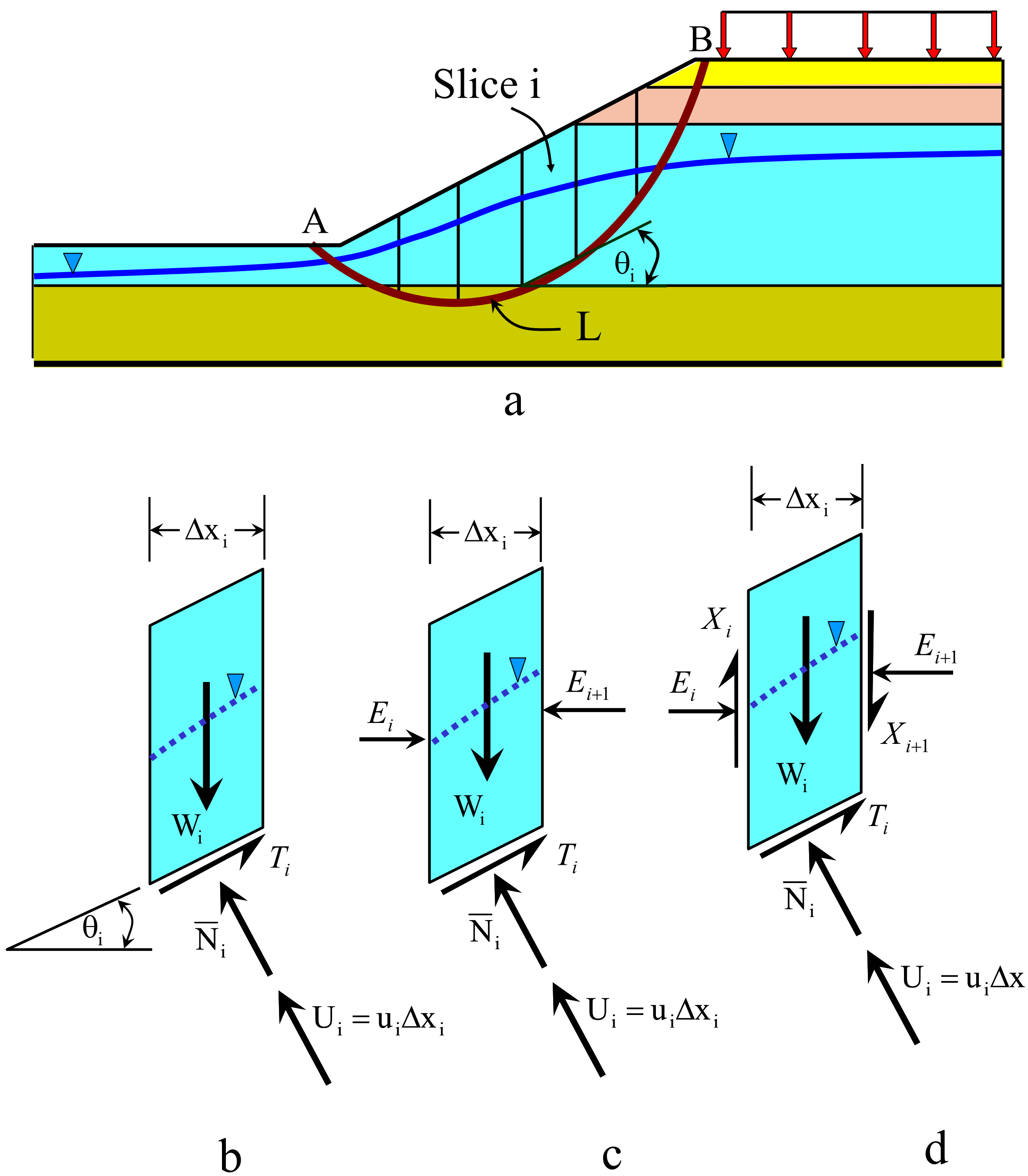


Figure 9-18. Method of slices and forces acting on a slice

Variations of this method used for investigating the factor of safety for potential stability failure include:

'Fellenius Method of Slices'

'Simplified Bishop Method of Slices'

'Spencer and Janbu Method of Slices'

Also known as 'Ordinary Method of Slices' or 'Swedish Circle', the Fellenius Method was published in 1936. The Simplified Bishop Method (1955) also uses the method of slices to find the factor of safety for the soil mass. The failure is assumed to occur by rotation of a mass of soil on a circular slip surface centered on a common point as shown in Figure 9-18.

The basic equation for each of these methods is:

$$F = \frac{\overline{C}L + \tan\overline{\phi}\sum_{i=1}^{i=n}\overline{N}_i}{\sum_{i=1}^{i=n} W_i \sin\theta_i}$$

<u>Nomenclature</u>

F = Factor of safety

F_a = Assumed factor of safety

i = Represents the current slice

$\overline{\phi}$ = Friction angle based on effective stresses

$\overline{C}$ = Cohesion intercept based on effective stresses

W_i = Weight of the slice

$\overline{N}_i$ = Effective normal force

θ_i = Angle from the horizontal of a tangent at the center of the slice along the slip surface

T_i = Tensile force

u_i = Pore-water pressure force on a slice

U_i = Resultant neutral (pore-water pressure) force

Δl_i = Length of the failure arc cut by the slice

L = Length of the entire failure arc

For major excavations in side slopes, slope stability failure for the entire system should be investigated.

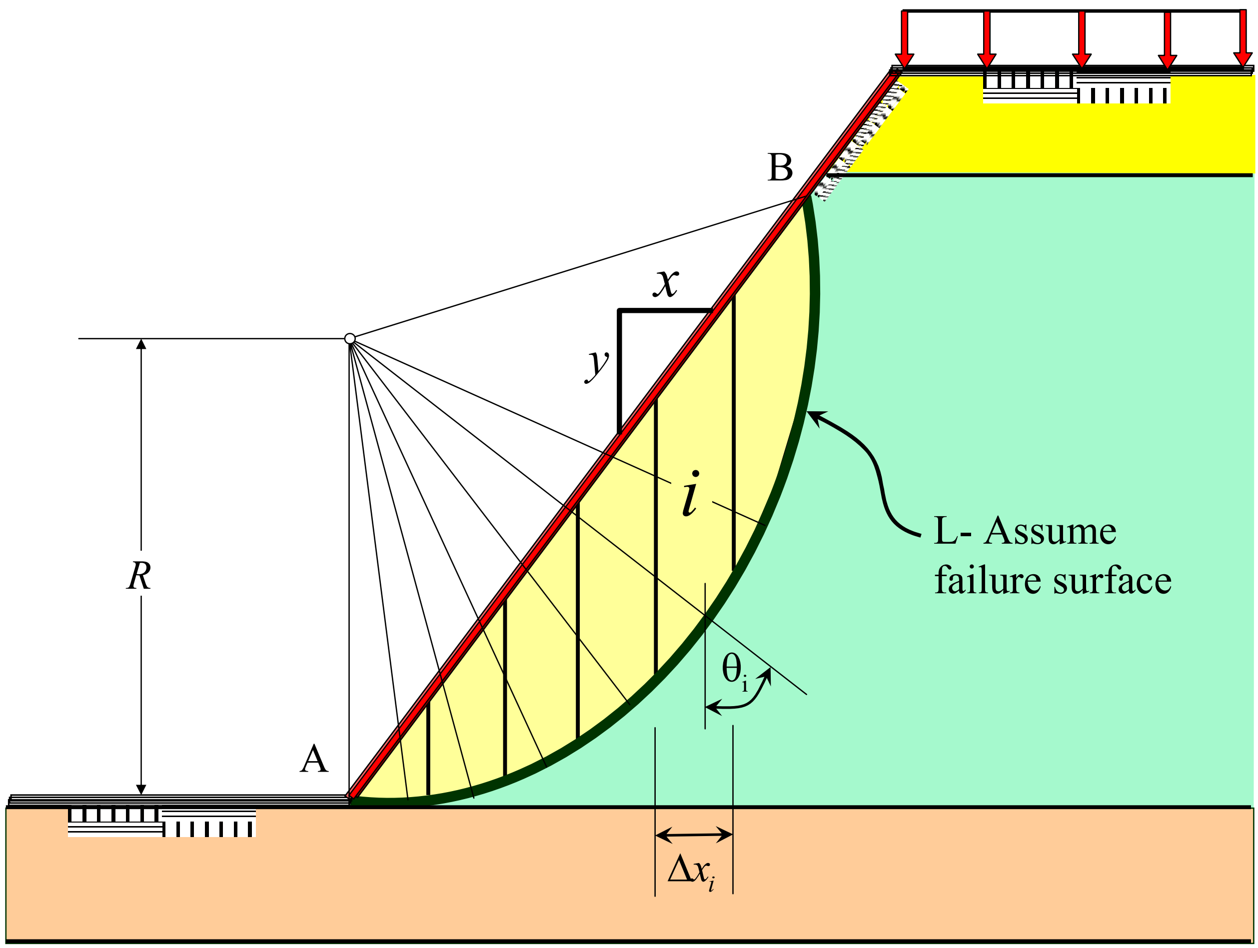


Figure 9-19. A Trial Surface, for Fellenius and Bishop method of slices

9.4.2 Fellenius Method

This method assumes that for any slice, the forces acting upon its sides have a resultant of zero in the direction normal to the failure arc. This method errs on the safe side, but is widely used in practice because of its early origins and simplicity.

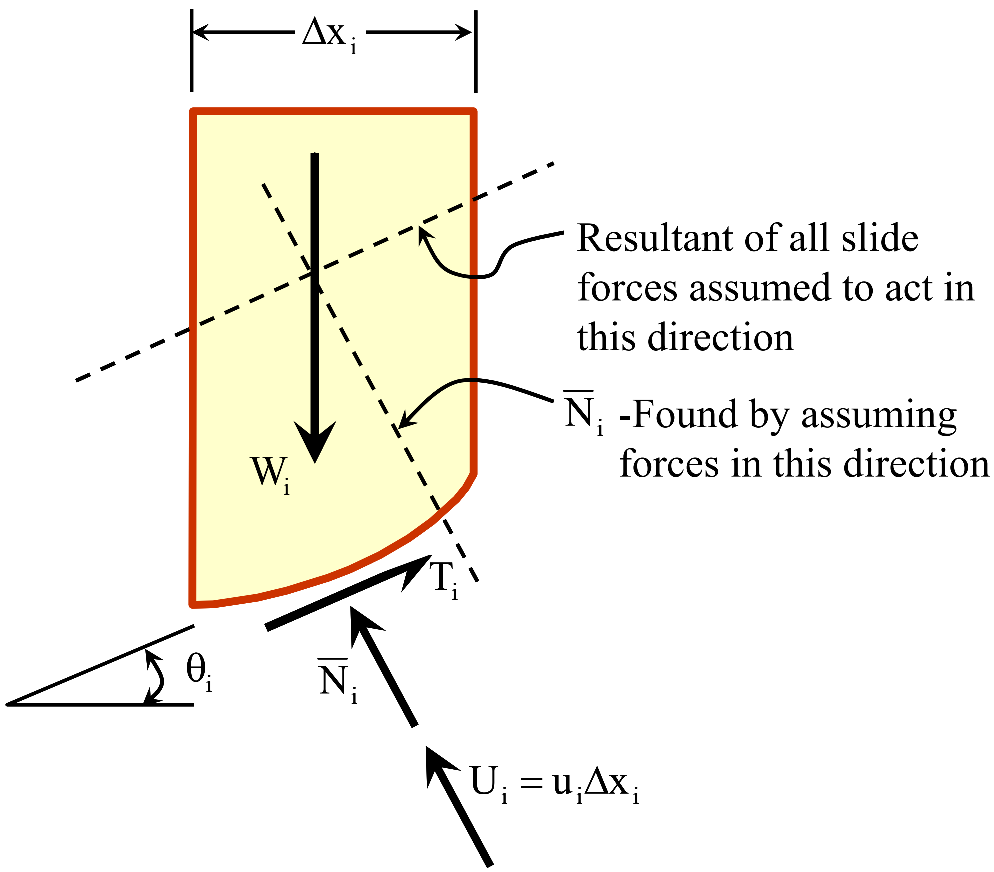


Figure 9-20. Slice i, Fellenius Method

$$\overline{N}_i = W_i \cos\theta_i - u_i \Delta_i l_i$$

The basic equation becomes:

$$F = \frac{\overline{C}L + \tan\overline{\phi}\sum_{i=1}^{i=n}\left(W_i \cos\theta_i - u_i \Delta_i\right)}{\sum_{i=1}^{i=n} W_i \sin\theta_i}$$

The procedure is to investigate many possible failure planes, with different centers and radii, to zero in on the most critical.

9.4.2.1 Example 9-3 Problem – Fellenius Method

Given: γ = 115 pcf $\qquad \bar{\phi}$ = 30° $\qquad \bar{C}$ = 200 psf $\qquad$ No Groundwater

Solution:

The trial failure mass is divided into 6 slices with equal width as shown in Figure 9-21. Each slice makes an angle θ with respect to horizontal as shown in Table 9-1.

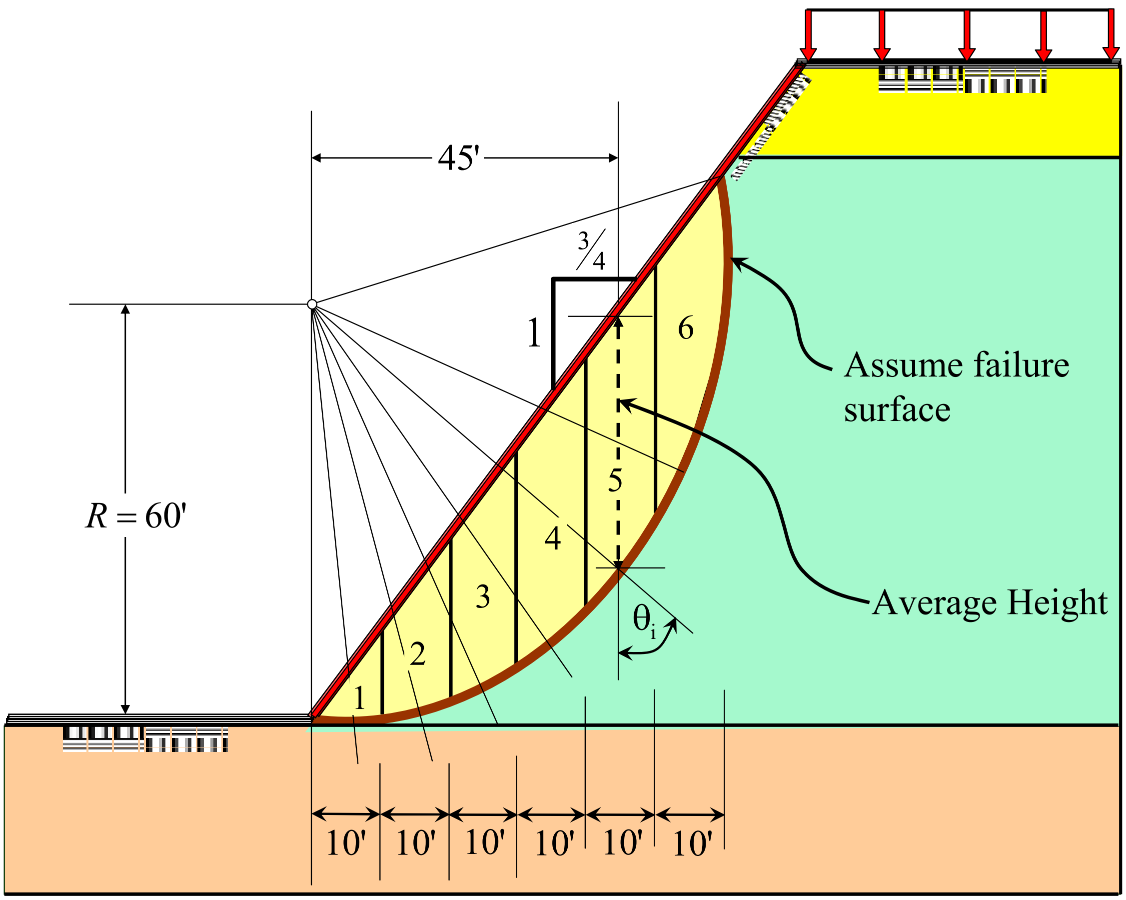


Figure 9-21. Example of Fellenius and Bishop method of slices

Table 9-1. Fellenius Table of Slices

Angles θ_i (°)	Average Height (ft)	Slice Weights (kips)
$\theta_1 = \sin^{-1}(6.7/60) = 6.41°$	12.4	$W_1 = (1/2)(12.4)(10)(0.115) = 7.13$
$\theta_2 = \sin^{-1}(15.0/60) = 14.47°$	17.8	$W_2 = (17.8)(10)(0.115) = 20.47$
$\theta_3 = \sin^{-1}(25.0/60) = 24.62°$	27.6	$W_3 = (27.6)(10)(0.115) = 31.74$
$\theta_4 = \sin^{-1}(35.0/60) = 35.69°$	34.7	$W_4 = (34.7)(10)(0.115) = 39.91$
$\theta_5 = \sin^{-1}(45.0/60) = 48.59°$	40.0	$W_5 = (40.0)(10)(0.115) = 46.00$
$\theta_6 = \sin^{-1}(55.0/60) = 66.44°$	35.8	$W_6 = (35.8)(10)(0.115) = 41.17$

Slice	θ_i (°)	W_i (kips)	$W_i \sin\theta_i$	$W_i \cos\theta_i$	$\overline{N}_i$
1	6.41	7.13	0.80	7.09	7.09
2	14.47	20.47	5.11	19.82	19.82
3	24.62	31.74	13.22	28.85	28.85
4	35.69	39.91	23.28	32.41	32.41
5	48.59	46.00	34.50	30.43	30.43
6	66.44	41.17	37.74	16.46	16.46
			Σ = 114.66		Σ = 135.06

$U_i = 0$ L = 112 ft (by geometry)

$$F = \frac{(0.2)(112) + (0.577)(135.06)}{114.66} = 0.87 < 1$$

This is the value for one trial failure plane. Additional trials are necessary to determine the critical one that gives the minimum factor of safety. The slope for this sample problem is deemed to be unstable since the computed safety factor determined by this single calculation is less than one.

9.4.3 Bishop Method

This method assumes that the forces acting on the sides of any slice have a zero resultant in the vertical direction.

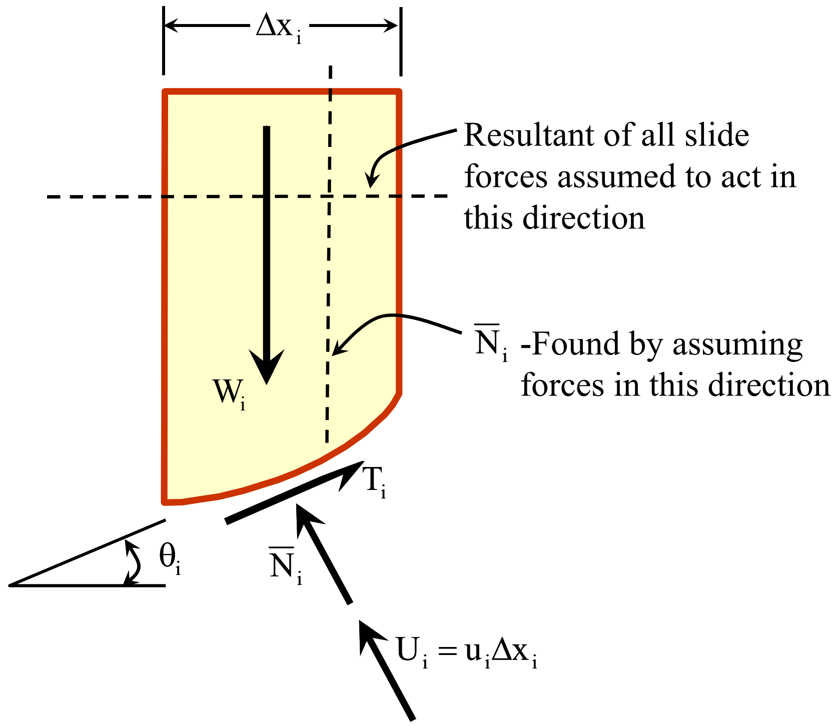


Figure 9-22. Slice i, Bishop Method

$$\mathrm{N_i} = \frac{\mathrm{W_i} - u_i \Delta x_i - \dfrac{\overline{C}\Delta x_i \tan\theta_i}{F_a}}{\cos\theta_i \left\{ 1 + \dfrac{\mathrm{Tan}\,\theta_i \tan\overline{\phi}}{F_a} \right\}}$$

The basic equation becomes:

$$\mathrm{F} = \frac{\sum_{\mathrm{i=n}}^{\mathrm{i=n}} \left(\dfrac{\overline{\mathrm{C}}\Delta \mathrm{x_i} + (W_i - u_i \Delta x_i)\tan\overline{\phi}}{M_i} \right)}{\sum_{\mathrm{i=n}}^{\mathrm{i=n}} \mathrm{W_i} \sin\theta_i}$$

Where: $\mathrm{M_i} = \cos\theta_i \left\{ 1 + \dfrac{\mathrm{Tan}\,\theta_i \tan\overline{\phi}}{F_a} \right\}$

For the Bishop Method, the Factor of Safety (F_a) must be assumed and a trial and error solution is required. The assumed "F_a's" converge on the Factor of Safety for that trial failure plane. Good agreement between the assumed "F_a" and the calculated "F" indicated the selection of center and radius was good.

9.4.3.1 Example 9-4 Problem – Bishop Method

Given: γ = 115 pcf $\qquad \bar{\phi} = 30°$ $\qquad \bar{C}$ = 200 psf $\qquad$ No Groundwater

Solution:

Column	A	B	C	D	E	F	G
Slice	θ_i	W_i	$\bar{C}\,\Delta x_i$	$W_i \tan\bar{\phi}$	$\cos\theta_i$	$\tan\theta_i \tan\bar{\phi}$	C + D
1	6.41	7.13	2	4.12	0.99	0.06	6.12
2	14.47	20.47	2	11.82	0.97	0.15	13.82
3	24.62	31.74	2	18.33	0.91	0.26	20.33
4	35.69	39.91	2	23.04	0.81	0.41	25.04
5	48.59	46.00	2	26.56	0.66	0.65	28.56
6	66.44	41.17	2	23.77	0.40	1.32	25.77

Column	Ha	Hb	Ia	Ib	J
Slice	M_i F_a = 1.5	 F_a = 0.8	G/Ha F_a = 1.5	G/Hb F_a = 0.8	$W_i \sin\theta_i$
1	1.04	1.07	5.94	5.72	0.80
2	1.06	1.15	12.92	12.02	5.11
3	1.07	1.21	19.00	16.94	13.22
4	1.04	1.23	24.31	20.36	23.28
5	0.95	1.20	30.06	23.80	34.50
6	0.75	1.06	34.36	24.31	37.74
			Σ = 126.59	Σ = 103.15	Σ = 114.65

For F_a = 1.5: $\qquad F = \dfrac{126.59}{114.65} = 1.104$

For F_a = 0.8: $\qquad F = \dfrac{103.15}{114.65} = 0.900$

The factor of safety for this trial converges to ≈ 0.9

Again, this is the value for one trial failure plane. Additional trials are necessary to determine the critical one that gives the minimum factor of safety.

If ground water were present, pore pressure would need to be considered. The values are most typically field measured.

9.4.4 Translational Slide

For excavations into stratified deposits where the strata are dipping toward the excavation or there is a definite plane of weakness near the base of the slope, the slope may fail along a plane parallel to the weak strata as shown in Figure 9-23.

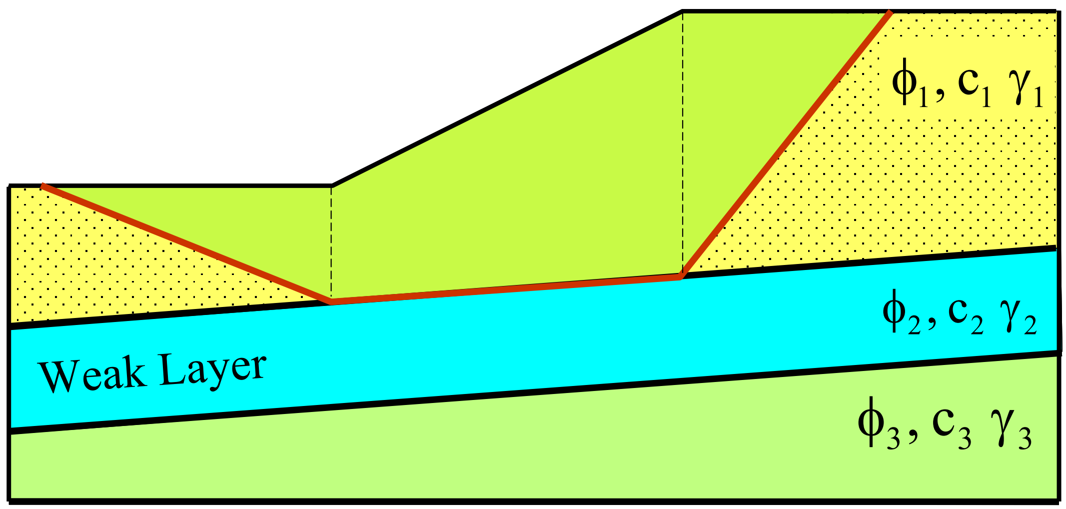
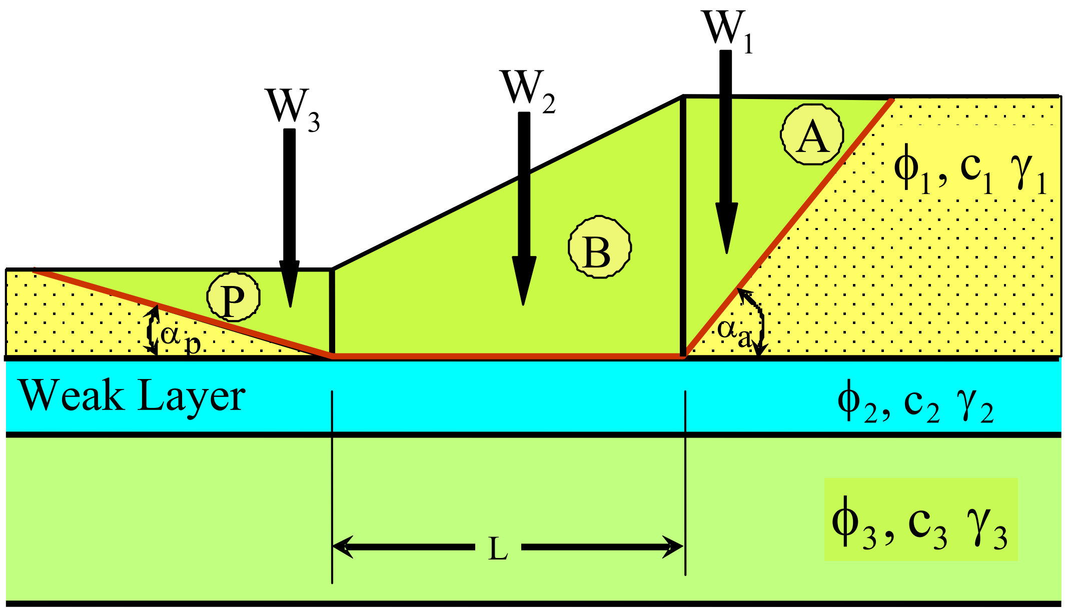


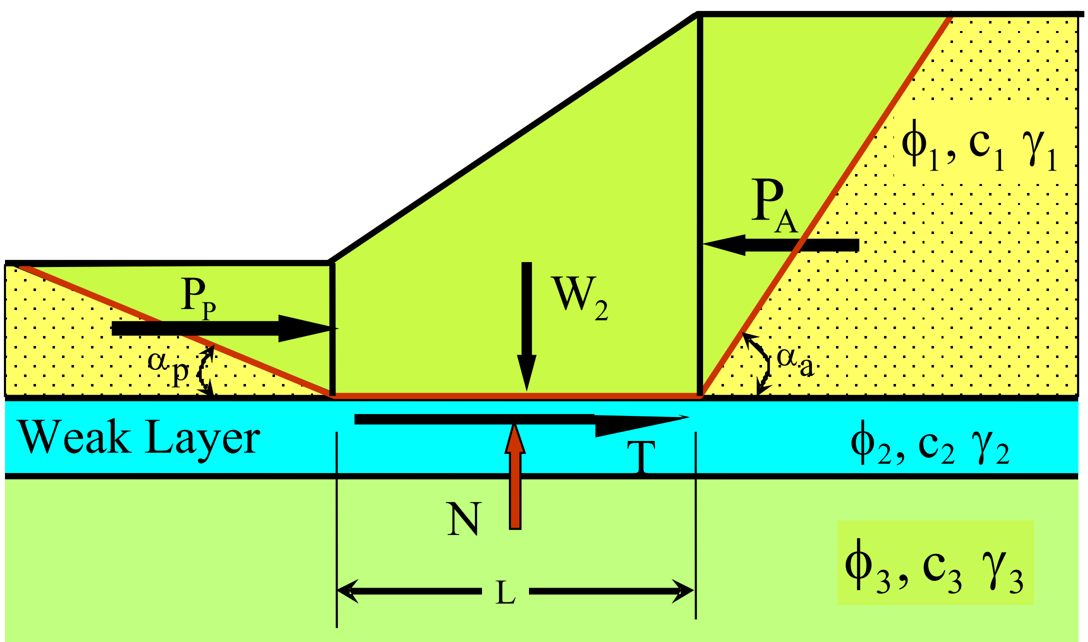
Figure 9-23. Mechanism of Translational Slide

The movement of the soil mass within the failure surface is translational rather than rotational. Methods of analysis that consider blocks or wedges sliding along plane surfaces shall be used to analyze slopes with a specific plane of weakness.

Figure 9-24 shows a sliding mass consisting of a tri-planar surface. The force equilibrium of the blocks or wedges is more sensitive to shear forces than moment equilibrium as shown in Figure 9-24. The potential failure mass consists of an upper or active Block A, a central or neutral Block B and a lower or passive Block P. The active earth pressure from Block A tends to initiate translational movement. This movement is opposed by the passive resistance to sliding of Block P and by shearing resistance along the base of central Block B. The critical failure surface can be located using an iterative process as explained previously.



a



b

Figure 9-24. Mechanism of Translational Slide

The Factor of Safety of the slope against translational sliding is established by the ratio of resisting to driving forces. The resisting force is a function of passive pressure at the toe of the slope and the shearing resistance along the base of Block B. The driving force is the active earth pressure due to thrust of Block A. Thus the Factor of Safety can be expressed as follows:

$$\mathrm{FS} = \frac{T + W_2 * \tan\phi_2 + P_p}{P_a}$$

In which:

$$T = c_2 * L + W_2 * \tan\phi_2$$

Where:

T = tangential resistance force at the base of Block B

c = unit cohesion along base of the Block B

W_2 = weight of section of Block B

L = length of base of Block B

P_p = passive pressure on Block B $\quad \mathrm{P_p} = \mathrm{W_3}\ \tan(\alpha_P + \phi_1)$

P_a = resultant active pressure on Block B $\quad \mathrm{P_a} = \mathrm{W_1}\ \tan(\alpha_a - \phi_1)$

W_1 = weight of section of Block A

W_3 = weight of section of Block C

α_p = failure plane angle with horizontal for passive pressure

α_a = failure plane angle with horizontal for active pressure

ϕ_1 = internal friction angle of soil for Block A

ϕ_2 = internal friction angle of soil for Block C

FS = Factor of Safety

9.4.4.1 Example 9-5 Problem – Translational Slide

Calculate the Factor of Safety for a translational slide for a given failure surface shown below.

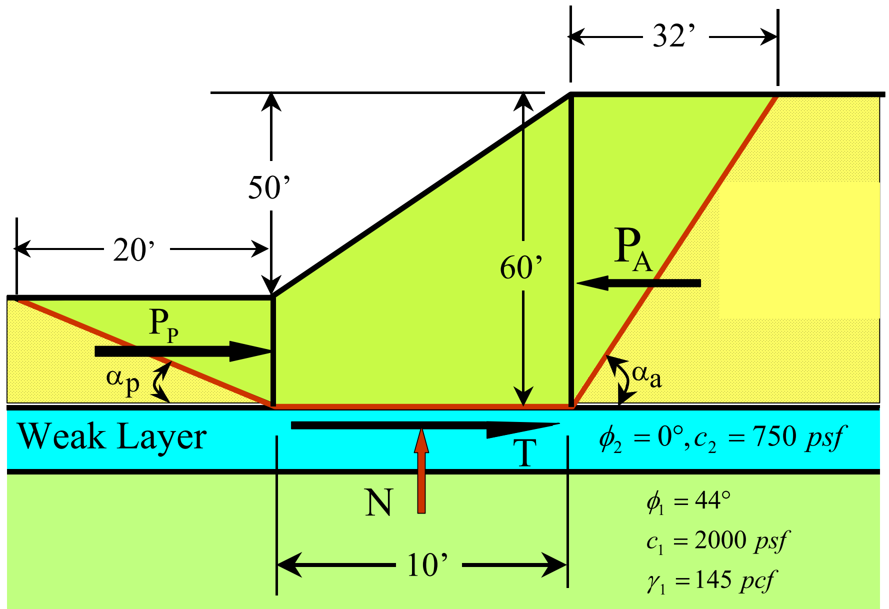


Figure 9-25. Example of a Translational Slide

Solution:

By geometry: $\alpha_a = 62°\quad \alpha_p = 26.6°$

$$W_1 = \frac{(32')(60')}{2}\left(\frac{120}{1000}\right) = 115.2\,\text{k/ft}$$

$$W_2 = \frac{(10'+60')(10')}{2}\left(\frac{120}{1000}\right) = 42.0\,\text{k/ft}$$

$$W_3 = \frac{(20')(10')}{2}\left(\frac{120}{1000}\right) = 12.0\,\text{k/ft}$$

$$T = W_2 Tan(\phi_2) + c_2 L = (42)(Tan(0°)) + \frac{(750)(10')}{1000} = 75.0\,\text{k/ft}$$

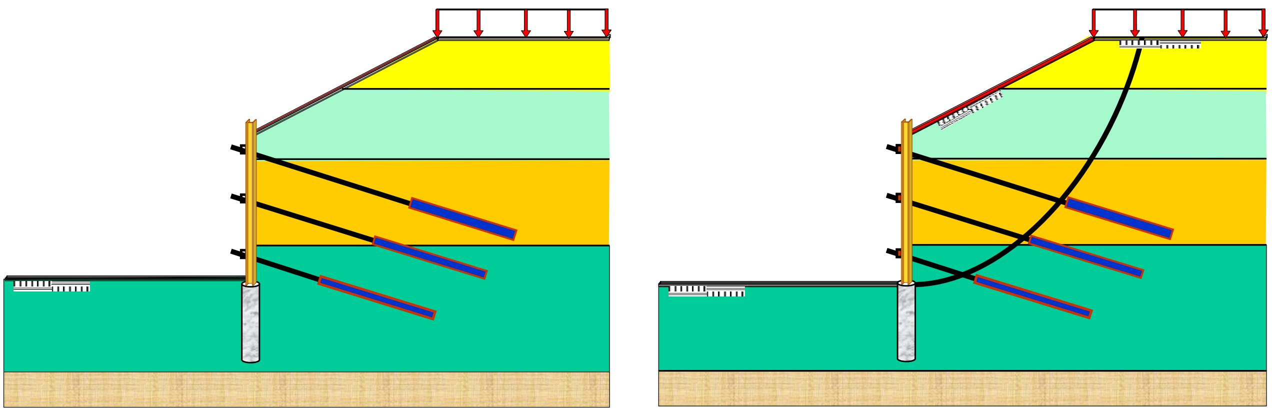
$$P_a = (115.2)(Tan(62° - 34°)) = 61.3\,\text{k/ft}$$

$$P_p = (12.0)(Tan(26.6° + 34°)) = 20.8\,\text{k/ft}$$

$$FS = \frac{75 + 20.8}{61.3} = 1.56$$

9.4.5 Stability Analysis of Shoring Systems

Deep-seated stability failure should be investigated for major shoring systems such as tieback walls. The slip surface passes behind the anchors and underneath the base tip of the vertical structural members as shown in Figure 9-26a. A minimum factor of safety of 1.25 is required for the deep-seated stability failure. Local system failure should also be investigated for major tieback systems as shown in Figure 9-26b. The trial surface shall extend to the depth of the excavation to calculate the minimum factor of safety of 1.25. The un-bonded length shall extend beyond the failure surface.



a. Deep seated b. Local system

Figure 9-26. Stability failure modes

9.4.6 The Last Word on Stability

The previous slope stability discussion serves to demonstrate the complexity of stability analysis. Soil failure analysis should not be limited to circular arc solutions. There are various computer programs for slope stability analysis using non-circular shapes.

When it appears that shoring or a cut slope presents a possibility of some form of slip failure, a stability analysis should be requested from the contractor. Submittals relative to the soils data and analysis should be from a recognized soils lab or from a qualified Geotechnical Engineer or Geologist. In addition, Geotechnical Services in Sacramento has the capability of performing computer aided stability analysis to verify the submitted analysis.

9.5 *CONSTRUCTION CONSIDERATIONS*

9.5.1 Construction

The integrity of a shoring system, like any other structure, is dependent upon the adequacy of the design, the quality of the materials used and the quality of the workmanship. Frequent and thorough inspection of the excavation and the shoring system during all stages of construction must be performed by qualified personnel. An awareness of the changing conditions is essential. The following is a list of potential / common considerations:

1. Check to ensure the contractor has a current excavation permit from Cal-OSHA. The permits are valid for January 1 to December 31, and must be renewed each year.
2. Prior to the beginning of excavation work, become familiar with all aspects of the approved plans, the location of the work, assumptions made, available soils data, ground water conditions, surcharge loads expected, sequence of operations, location of utilities and underground obstructions, and any other factors that may restrict the work at the site.
3. Since the primary function of the shoring is the protection of the workers, adjacent property and the public, it is essential that the inspector be knowledgeable in the minimum safety requirements.
4. Check all soil being excavated to confirm that it is consistent with the log of test borings and/or with what is contemplated in the excavation plan.
5. Check for changes in the groundwater conditions.
6. As the excavation progresses, be alert for indicators of distress such as tension cracks forming, potential failure of structural members or subsidence of soil near the excavation
7. If the excavation is sloped back without shoring, the need for inspection remains. Sloughing and cave-ins can occur. As always, verify that the slope configurations are per the approved plan.
8. Review all the materials for quality, integrity and/or strength grade specified. Also check members for bending, buckling and crushing.
9. For shored excavations, check the shoring members for size and spacing as shown on the approved plans. The sequence of operations shown on the plans must be followed.

Check for full bearing at the ends of jacks and struts and make sure they are secure and will not fall out under impact loads. Also check members for bending, buckling and crushing.

10. Manufactured products, such as hydraulic struts, jacks and shields, should be installed and used according to the manufacturer's recommendations.
11. If a tieback system is used, the tiebacks should be installed per the approved plan and preloaded to avoid overloading individual ties.
12. When cables are used in conjunction with anchors, they should not be wrapped around sharp corners. Thimbles should be used and cable clamps installed properly.
13. Surcharge loads need to be monitored to verify that such loads do not exceed the design assumptions for the system.
14. Weather conditions may have an adverse affect on excavations and some materials, especially clays, may fail due to change in moisture content. Some situations may benefit by protecting the slopes with sheeting or other stabilizing material.
15. Good workmanship makes an excavation safer and easier to inspect. Trouble spots are easier to detect when the excavation is uniform and straight.
16. Vibrations from dynamic loadings such as vibratory equipment, pile driving or blasting operations require special attention.
17. Utility owners should be notified prior to commencement of work if their facilities are within 5 times the excavation depth.

    Underground Service Alert:

    811 or 1-800-227-2600

Northern California (USA)	www.usanorth.org
Southern California (USA)	www.digalert.org
Statewide	www.call811.com

18. Encourage the use of benchmarks to monitor ground movement in the vicinity of the shoring system (within a distance of 10 times the shoring depth) before, during and after excavation. The benchmarks should be monitored for horizontal and vertical displacement. In general, ground settlement accompanies shoring deflection.

19. Egress provisions such as ladders, ramps, stairways, or other means shall be provided in excavations over 4 feet in depth so that no more than 25 feet of lateral travel is required to exit trench excavations.
20. Adequate protection from hazardous atmospheres must be provided. Air monitoring and other confined space regulations must be followed, including documentation.
21. Employees shall be protected from the hazards of accumulating water, loose or falling debris, or potentially unstable structures.
22. Daily inspections, inspections after storms, and those as otherwise required for hazardous conditions are to be made by a competent person. Inspections are to be conducted before the start of the work and as needed throughout the shift. The competent person will need to check for potential cave-ins, indications of failure of the system, and for hazardous atmospheres. When the competent person finds a hazardous situation he shall have the authority to remove the endangered employees from the area until the necessary precautions have been taken to ensure their safety.
23. Adequate barrier physical protection is to be provided at all excavations. All wells, pits, shafts, etc. shall be barricaded or covered. Upon completion of exploration and similar operations, temporary shafts, etc. shall be backfilled.

9.5.2 Encroachment Permit Projects

An Encroachment Permit is required for projects performed by others within State highway right-of-way or adjacent to State highways including those done under a Cooperative Agreement such as a Capital Improvement Project. The contractor, builder or owner must apply for and be issued an Encroachment Permit by the District Permits Engineer.

If the scope of work requires excavation and shoring, plans for this work must accompany the Permit application. The Plan must be reviewed and approved by the Permits Engineer prior to a Permit being issued. The Department has an obligation with respect to trenching and shoring work. Be informed of legal responsibilities and requirements. (Refer to CHAPTER 1)

Many of the encroachment permit projects are quite simple however; some might require complex shoring systems. The District Permits Engineer, on receipt of an application for an encroachment permit, will decide if technical assistance is necessary to review the Plan. The Plan may be routed to OSM, OSD or OSC.

The Plan must conform to all applicable requirements as outlined in CHAPTER 2 of this Manual. It must also conform to the requirements set forth in the Permit application. The review process is similar to the process for a typical State contract except that all correspondence regarding approval or rejection of the Plan must be routed through the Permits Engineer.

Note that consultants who prepare shoring plans for Encroachment Permit projects do not necessarily use the recommended allowable stresses given in this Manual. In making a review, keep this in mind. Acceptance should be based on nothing less than that required for a State project, with due consideration being given to the background of the contractor, the work to be done, and the degree of risk involved. Remember, geotechnical engineering is not an exact or precise science.

In order for the State to review and approve a contractor's excavation plan or proposed shoring system, a detailed plan of the work to be done must be submitted. As a minimum the shoring plan shall contain the following information:

Encroachment Permit No. (Contractor)

Contractor: (Name, Address, phone)

Owner: For whom the work is being done. Include Contract no or designation

Owner Encroachment Permit No.:

Location: Road, street, highway stationing, etc. indicating the scope or extent of the project.

Purpose: A description of what the trench or excavation is for (sewer line, retaining wall, etc).

Soil Profile: A description of the soil including the basis of identification such as surface observation, test borings, observation of adjacent work in same type of material, reference to a soils investigation report, etc.

Surcharge Loadings:

Any loads, including normal construction loads, that are adjacent to the excavation or trench should be identified and shown on the plans with all pertinent dimensions; examples are highways, railroads, existing structures, etc. The lateral pressures due to these loads will then be added to the basic soil pressures. The minimum surcharge is to be used where not exceeded by above loading considerations.

Excavation/Trenching & Shoring Plan:

The Plan for simple trench work can be in the form of a letter covering the items required. For more complex systems, a complete description of the shoring system including all members, materials, spacing, etc, is required. The Plan may be in the form of a drawing or referenced to the applicable portions of the Construction Safety Orders. In accordance with California Labor Code (CA law), if a shoring system varies from Title 8 of the Safety Orders, then the shoring plans must be prepared and signed by an engineer who is registered as a Civil Engineer in the State of California.

Manufactured Data:

Catalogs or engineering data for a product should be identified in the plan as supporting data. All specific items or applicable conditions must be outlined on the submittal.

Construction Permit:

Any plan or information submitted should confirm that a permit has been secured from Cal/OSHA to perform the excavation work. This is not an approval of the shoring system by Cal/OSHA.

Inspection: The contractor’s plan must designate who the competent person on site will be.

The State Department of Transportation will review a Contractor's shoring plan in accordance with applicable State Specifications and the Construction Safety Orders. Deviations from Cal/OSHA or different approaches will be considered, providing adequate supporting data such as calculations, soils investigations, manufacturer's engineering data and references are submitted. The Caltrans Trenching & Shoring Manual is one of the resources available to assist the Engineer during the shoring plan review process.

The inspection of the fieldwork is the responsibility of the District Permits Engineer and his staff. However, there will be occasions where the complexity of the excavation and/or shoring requires assistance from OSC. For major Encroachment Permit projects the District may request that OSC assign an Engineer as a representative of the District Permits Engineer. Remember that the administrative or control procedure is different from typical State construction contracts. The OSC person assisting the Permits Engineer is a representative of the Permits Engineer not the Resident Engineer. Major corrections must be routed through the Permits Engineer. If there are difficulties with compliance, the Permits Engineer has the authority to withdraw the Encroachment Permit, which would have the effect of stopping the work. Close communication between OSC and the Permits Engineer is very important during all phases of the Encroachment Permit project.

In addition to verifying that the excavation and/or shoring work is in conformance with the approved Plan, a portion of the field review or monitoring will be to verify that the contractor and/or owner have all of the proper permits to do the work.

For more information regarding the Encroachment Permit process, contact your local District Permits Engineer or click on the link below.
http://www.dot.ca.gov/hq/traffops/developserv/permits/encroachment_permits_manual/index.html

9.5.3 Tieback Systems

9.5.3.1 Construction Sequence

The construction sequence for an anchored sheet pile or soldier pile system must be considered when making an engineering analysis. Different loads are imposed on the system before and after the completion of a level of tieback anchors. An analysis should be included for each stage of construction and an analysis may be needed for each stage of anchor removal during backfilling operations

9.5.3.2 Tieback Anchor Systems

There are many variations or configurations of tieback anchor systems. The tension element of a tieback may be either prestressing strands or bars using either single or multiple elements. Tiebacks may be anchored against wales, piles, or anchor blocks, which are placed directly on the soil. The example problems in this chapter illustrate the use of tiebacks with several different types of shoring systems.

Figure 9-27 illustrates a typical temporary tieback anchor. In this diagram, a bar tendon system is shown; strand systems are similar.

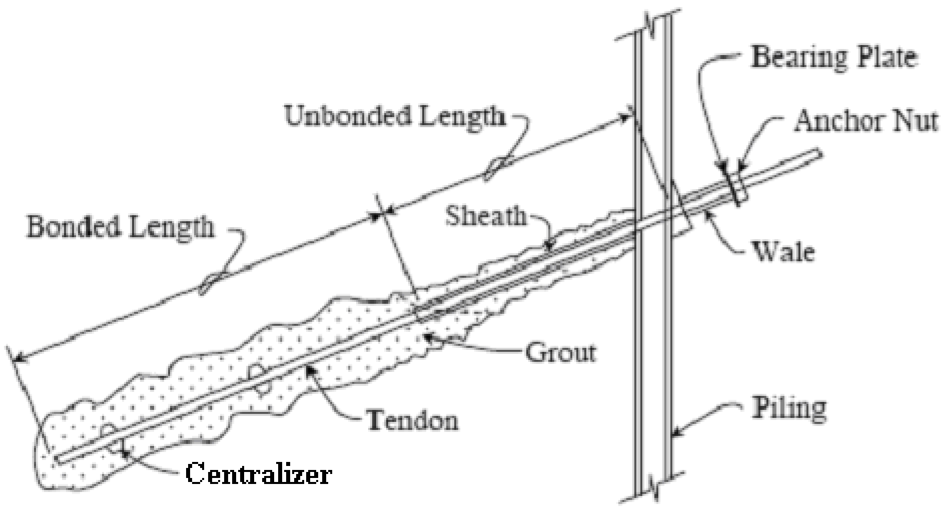


Figure 9-27. Typical temporary tieback

The more common components, criteria, and materials used in conjunction with tieback shoring systems are listed below:

Piling	Sheet piling and soldier piles. See CHAPTER 5 for common materials and allowable stresses.
Wale	These components transfer the resultant of the earth pressure from the piling to the tieback anchor. A design overstress of 33% is permitted for wales when proof testing the tieback anchor. Anchors for temporary work, are often anchored directly against the soldier piling through holes or slots made in the flanges, eliminating the need for wales. Bearing stiffeners and flange cover plates are generally added to the pile section to compensate for the loss of section. A structural analysis of this cut section should always be required.
Tendon	Tieback tendons are generally the same high strength bars or strands used in prestressing structural concrete. The anchorage of the tieback tendons at the shoring members consists of

bearing plates and anchor nuts for bar tendons and bearing plates, anchor head and strand wedges for strand tendons. The details of the anchorage must accommodate the inclination of the tieback relative to the face of the shoring members. Items that may be used to accomplish this are shims or wedge plates placed between the bearing plate and soldier pile or between the wale and sheet piling or soldier piles. Also, for bar tendons spherical anchor nuts with special bearing washers plus wedge washers if needed or specially machined anchor plates may be used.

The tendon should be centered within the drilled hole within its bonded length. This is accomplished by the use of centralizers (spacers) adequately spaced to prevent the tendon from contacting the sides of the drilled hole or by installation with the use of a hollow stem auger.

Stress — Allowable tensile stress values are-based on a percentage of the minimum tensile strength (F_{pu}) of the tendons as indicated below:

Bars: F_{pu} = 150 to 160 ksi
Strand: F_{pu} = 270 ksi
(Check manufacturers data for actual ultimate strength)

Allowable tensile stresses:

At design load $F_t \leq 0.6\ F_{pu}$
At proof load $F_t \leq 0.8\ F_{pu}$
(Both conditions must be checked.)

Grout — A flowable portland cement mixture of grout or concrete which encapsulates the tendon and fills the drilled hole within the bonded length. Generally, a neat cement grout is used in drilled holes of diameters up to 8 inches. A sand-cement mixture is used for hole diameters greater than 8 inches. An aggregate concrete mix is commonly used in very large holes. Type I or II cement, is commonly recommended for tiebacks. Type III cement may be used when high early strength is desired. Grout, with very few exceptions, should always be injected at the bottom of the drilled hole.

This method ensures complete grouting and will displace any water that has accumulated in the hole.

9.5.3.3 Tieback Anchor

There are several different types of tieback anchors. Their capacity depends on a number of interrelated factors:

- Location - amount of overburden above the tieback
- Drilling method and drilled hole configuration
- Strength and type of the soil
- Relative density of the soil
- Grouting method
- Tendon type, size, and shape

Typical shapes of drilled holes for tieback anchors are depicted in Figure 9-28.

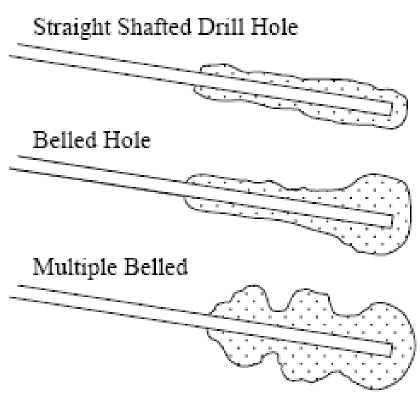


Figure 9-28. Anchor shapes

This is the simplest type and the one encountered most often.

In this case the resistance is a combination of perimeter bond and bearing against the soil.

Similar to above, this type of anchor is referred to as under-reamed. It is used in stiff cohesive soil. The soil must be stiff enough to prevent collapse of the under-reams or drill hole in the anchor length.

The presence of water either introduced during drilling or existing ground water can cause significant reduction in anchor capacity when using a rotary drilling method in some cohesive soils (generally the softer clays).

High pressure grouting of 150 psi or greater in granular soils can result in significantly greater tieback capacity than by tremie or low pressure grouting methods. High pressure grouting is seldom used for temporary tieback systems.

Re-grouting of tieback anchors has been used successfully to increase the capacity of an anchor. This method involves the placing of high-pressure grout in a previously formed anchor. Re-grouting breaks up the previously placed anchor grout and disperses new grout into the anchor zone; compressing the soil and forming an enlarged bulb of grout thereby increasing the anchor capacity. Re-grouting is done through a separate grout tube installed with the anchor tendon. The separate grout tube will generally have sealed ports uniformly spaced along its length, which open under pressure allowing the grout to exit into the previously formed anchor.

Due to the many factors involved, the determination of anchor capacity can vary quite widely. Proof tests or performance tests of the tiebacks are needed to confirm the anchor capacity. A Federal publication, the FHWA/RD-82/047 report on tiebacks, provides considerable information for estimating tieback capacities for the various types of tieback anchors. Also see "Supplemental Tieback Information" in Appendix E.

Bond capacity is the tieback’s resistance to pull out, which is developed by the interaction of the anchor grout (or concrete) surface with the soil along the bonded length.

Determining or estimating the bond (resisting) capacity is a prime element in the design of a tieback anchor.

Some shoring designs may include a Soils Laboratory report, which will contain recommended value for the bond capacity to be used for tieback anchor design. The appropriateness of the value of the bond capacity will only be proven during tieback testing.

For most of the temporary shoring work normally encountered, the tieback anchors will be straight shafted with low-pressure grout placement. For these conditions the following criteria can generally be used for estimating the tieback anchor capacity.

The Engineer is only required to check the unbonded length of the tieback. The determination of the bonded length L_b and capacity of the tieback is solely the responsibility of the contractor. The minimum distance between the front of the bonded zone and the active failure surface behind the wall shall not be less H/5. In no case shall the minimum distance be less than 5 ft. The unbonded length shall not be less than 15 ft.

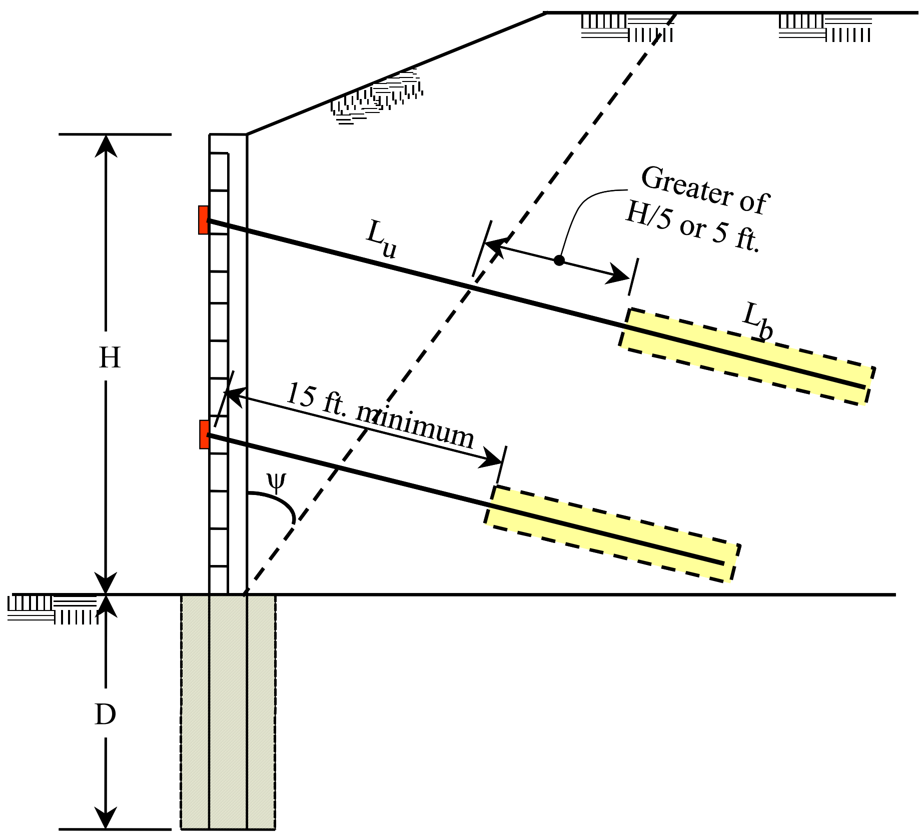


Figure 9-29. Bond Length

The ultimate capacity of the tieback is defined as follows:

$$P_{ult} = \pi d L_b S_b$$

Where:

d = Diameter of drilled hole
L_b = Bonded length of the tieback
L_u = Unbonded length of the tieback
S_b = Bond strength
ψ = Angle between assumed failure plane and vertical

The bond strength for tiebacks depends on a number of interrelated factors:

- Location - amount of effective overburden pressure above the tieback
- Drilling method and drilled hole configuration
- Strength properties, type and relative density of the soil
- Grouting method and pressure

Therefore, bond strength must be included in the geotechnical report that is submitted by the Contractor just as any other soil property. The Geotechnical Services of the Division of Engineering Services (DES) is available for consultation for concerns or other information regarding bond strength.

9.5.3.4 Forces on the Vertical Members

Tiebacks are generally inclined; therefore the vertical component of the tieback force must be resisted by the vertical member through skin friction on the embedded length of the piling in contact with the soil and by end bearing. Problems with tieback walls have occurred because of excessive downward wall movement.

The vertical capacity of the shoring system should be checked when the initial review of the soil parameters indicates a problem may develop. Situations that can lead to problems with the vertical capacity are shoring embedded in loose granular material or soft clays. Vertical capacity should also be checked when tieback angles are steeper than the standard 15 degrees or when there are multiple rows of tiebacks. The Engineer is reminded to contact Caltrans Geotechnical Services for assistance when performing a check of the vertical capacity of the shoring elements.

9.5.3.5 Testing Tieback Anchors

The Contractor is responsible for providing a reasonable test method for verifying the capacity of the tieback anchors after installation. Anchors are tested to ensure that they can sustain the design load over time without excessive movement. The need to test anchors is more important when the system will support, or be adjacent to existing structures, and when the system will be in place for an extended period of time.

The number of tiebacks tested; the duration of the test, and the allowable movement, or load loss, specified in the contractor's test methods should take into account the degree of risk to the adjacent surroundings. High-risk situations would be cases where settlement or other damage would be experienced by adjacent facilities. See Table 9-2 for a list of minimum recommended criteria for testing temporary tieback anchors.

Table 9-2. Tieback Proof Test Criteria

Test Load	Load Hold Duration	% of tiebacks to be load tested
Cohesionless Soils Normal Risk 1.2 to 1.3 Design Load	10 minutes	10% for each soil type encountered
High Risk 1.3 Design Load	10 minutes	20% to 100%
Cohesive Soils Normal Risk 1.2 to 1.3 Design Load	30 minutes	10%
High Risk 1.3 Design Load	60 minutes	30% to 100%
Use 100% when in soft clay or when ground water is encountered. Use load hold of 60 minutes for 10% and load hold of 10 minutes for remaining 90% of tiebacks		

Generally the shoring plans should include tieback load testing criteria which should minimally consist of proof load test values, frequency of testing (number of anchors to be tested), test load duration, and allowable movement or loss of load permissible during the testing time frame and the anticipated life of the shoring system. The shoring plans should also include the remedial measures that are to be taken when, or if, test anchors fail to meet the specified criteria.

Pressure gages or load cells used for determining test loads should have been recently calibrated by a certified lab, they should be clean and not abused, and they should be in good working order. The calibration dates should be determined and recorded.

Tiebacks that do not satisfy the testing criteria may still have some value. Often an auxiliary tieback may make up for the reduced value of adjacent tiebacks; or additional reduced value tiebacks may be installed to supplement the initial low value tiebacks.

9.5.3.6 Proof Testing

Applying a sustained proof load to a tieback anchor and measuring anchor movement over a specified period of time normally accomplish proof testing of tiebacks anchors. Proof testing may begin after the grout has achieved the desired strength. A specified number of the tieback anchors will be proof tested by the method specified on the Contractor's approved plans (see Table 9-2).

Generally, the unbonded length of a tieback is left ungrouted prior to and during testing (see Figure 9-30). This ensures that only the bonded length is carrying the proof load during testing. It is not desirable to have loads transferred to the soil through grout (or concrete) in the unbonded region since this length is considered to be within the zone of the failure wedge.

As an alternative, for small diameter drilled holes (6 inches or less) a plastic sheathing may be used over the unbonded length of the tendon to separate the tendon from the grout (see Figure 9-27). The sheathing permits the tendon to be grouted full length before proof testing. A void must be left between the top of the grout and the soldier pile to allow for movement of the grout column during testing.

Research has shown that small diameter tiebacks develop most of their capacity in the bonded length despite the additional grout in the unbonded length zone. This phenomenon is not true for larger diameter tieback anchors.

Generally the Contractor will specify an alignment load of 5% to 10% of the design load, which is initially applied to the tendon to secure the jack against the anchor head and stabilize the setup. The load is then increased until the proof load is achieved. Generally a maximum amount of time is specified to reach proof load. Once the proof load is attained, the load hold period begins. Movement of the tieback anchor is normally measured by using a dial indicator gage mounted on a tripod independent of the tieback and shoring and positioned in a manner similar to that shown in Figure 9-30.

The tip of the dial indicator gage is positioned against a flat surface perpendicular to the centerline of the tendon. (This can be a plate secured to the tendon). The piston of the jack may be used in lieu of a plate if the jack is not going to have to be cycled during the test. As long as the dial indicator gage is mounted independently of the shoring system, only movement of the anchor due to the proof load will be measured. Continuous jacking to maintain the specified proof load during the load hold period is essential to offset losses resulting from anchor creep or movement of the shoring into the supporting soil.

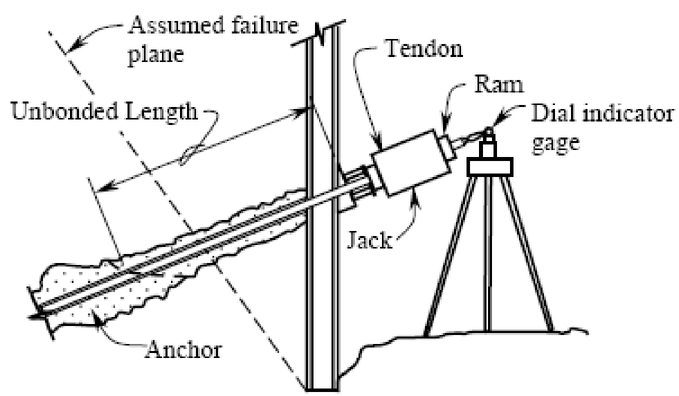


Figure 9-30. Proof Testing

Measurements from the dial indicator gage are taken periodically during the load hold period in accordance with the contractor's approved plan. The total movement measured during the load hold period of time is compared to the allowable value indicated on the approved shoring plans to determine the acceptability of the anchor.

It is important that the proof load be reached quickly. When excessive time is taken to reach the proof load, or the proof load is held for an excessive amount of time before beginning the measurement of creep movement, the creep rate indicated will not be representative. For the proof test to be accurate, the starting time must begin when the proof load is first reached.

As an alternative to measuring movement with a dial indicator gage, the contractor may propose a "lift-off test". A "lift-off test" compares the force on the tieback at seating to the force required to lift the anchor head off of the bearing plate. The comparison should be made over a specified period of time. The lost force can be converted into creep movement to provide an estimate of the amount of creep over the life of the shoring system.

Use of the "lift-off test" may not accurately predict overall anchor movement. During the time period between lock-off and lift-off, the tieback may creep and the wall may move into the soil. These two components cannot be separated. If the test is done accurately, results are likely to be a conservative measure of anchor movement. The Offices of Structure Construction recommends the use of a dial indicator gage to monitor creep rather than lift-off tests.

9.5.3.7 Evaluation of Creep Movement

Long-term tieback creep can be estimated from measurements taken during initial short term proof testing: In effect, measurements made at the time of proof testing can be extrapolated to determine anticipated total creep over the period the shoring system is in use if it is assumed that the anchor creep is roughly modeled by a curve described by the "log" of time.

The general formula listed below for the determination of the anticipated long-term creep is only an estimate of the potential anchor creep and should be used in conjunction with periodic monitoring of the wall movement. This formula will not accurately predict anchor creep for soft cohesive soils.

Based on the assumed creep behavior, the following formula can be utilized to evaluate the long-term effects of creep:

General formula:

$$\Delta_{2-3} = C\left[\log_{10}\left(\frac{T_3}{T_2}\right)\right]$$

Where:

C = $\Delta_{1-2} / \left[\log_{10}\left(T_2 / T_1\right)\right]$

Δ = Creep movement (inches) specified on the plans for times T_1, T_2, or T_3 (or measured in the field)

T_1 = Time of first movement measurement during load hold period (usually 1 minute after proof load is applied)

T_2 = Time of last movement measurement during load hold period.

T_3 = Time the shoring system will be in use.

If using a "lift-off test" to estimate the creep movement, the following approximation needs to be made for substitution into the above equation:

$$\Delta_{1-1} = \left(P_1 - P_2\right)\frac{L_U}{AE}$$

Where:

P_1 = Force at seating

P_2 = Force at lift off

L_u = L_u + 0 to 5 feet of the bonded length necessary to develop the tendon

A = Area of strand or bar in anchor

E = Modulus of elasticity of the strand or bar in anchor

Example 9-6 demonstrates the calculation of long-term creep.

9.5.3.8 Wall Movement and Settlement

As a rule of thumb, the settlement of the soil behind a tieback wall, where the tiebacks are locked-off at a high percentage of the design force, can be approximated as equal to the movement at the top of the wall caused by anchor creep and deflection of the piling. Reference is made to Section 6.3 titled "DEFLECTION" of CHAPTER 6.

If a shoring system is to be in close proximity to an existing structure where settlement might be detrimental, significant deflection and creep of the shoring system would not be acceptable. If a shoring system will not affect permanent structures or when the shoring might support something like a haul road, reasonable lateral movement and settlement can be tolerated.

9.5.3.9 Performance Testing

Performance testing is similar to, but more extensive, than proof testing. Performance testing is used to establish the movement behavior for a tieback anchor at a particular site. Performance testing is not normally specified for temporary shoring, but it can be utilized to identify the causes of anchor movement. Performance testing consists of incremental loading and unloading of a tieback anchor in conjunction with measuring movement.

9.5.3.10 Lock-Off Force

The lock-off force is the percentage of the required design force that the anchor wedges or anchor nut is seated at after seating losses. A value of $0.8T_{DESIGN}$ is typically recommended as the lock-off force but lower or higher values are used to achieve specific design needs.

One method for obtaining the proper lock-off force for strand systems is to insert a shim plate under the anchor head equal to the elastic elongation of the tendon produced by a force equal to the proof load minus the lock-off load. A correction for seating of the wedges in the anchor head is often subtracted from the shim plate thickness. To determine the thickness of the shim plate you may use the following equation:

$$t_{shim} = \frac{\left(P_{proof} - P_{lockoff}\right)L}{AE} - \Delta L$$

Where,

t_{shim} = thickness of shim

P_{proof} = Proof load

$P_{lockoff}$ = Lock-off load

A = Area of tendon steel (bar or strands)

E = Modulus of Elasticity of strand or bar

ΔL = seating loss

L = Elastic length of tendon (usually the unbonded length + 3 to 5 feet of the bonded length necessary to develop the tendon

Seating loss can vary between ⅜” to ⅝" for strand systems. The seating loss should be determined by the designer of the system and verified during installation. Often times, wedges are mechanically seated minimizing seating loss resulting in the use of a lesser value for the seating loss. For thread bar systems, seating loss is much less than that for strand systems and can vary between 0" to l/16".

After seating the wedges in the anchor head at the proof load, the tendon is loaded, the shim is removed and the whole anchor head assembly is seated against the bearing plate.

9.5.3.11 Corrosion Protection

The contractor's submittal must address potential corrosion of the tendon after it has been stressed.

For very short-term installations in non-corrosive sites corrosion protection may not be necessary. The exposed steel may not be affected by a small amount of corrosion that occurs during its life.

For longer term installations grouting of the bonded and unbonded length is generally adequate to minimize corrosion in most non-corrosive sites. Encapsulating or coating any un-grouted portions (anchor head, bearing plate, wedges, strand, etc.) of the tieback system may be necessary to guard against corrosion.

For long-term installations or installations in corrosive sites, more elaborate corrosion protection schemes may be necessary. (Grease is often used as a corrosion inhibitor). Figure 9-31 depicts tendons encapsulated in pre-greased and pre-grouted plastic sheaths generally used for permanent installations.

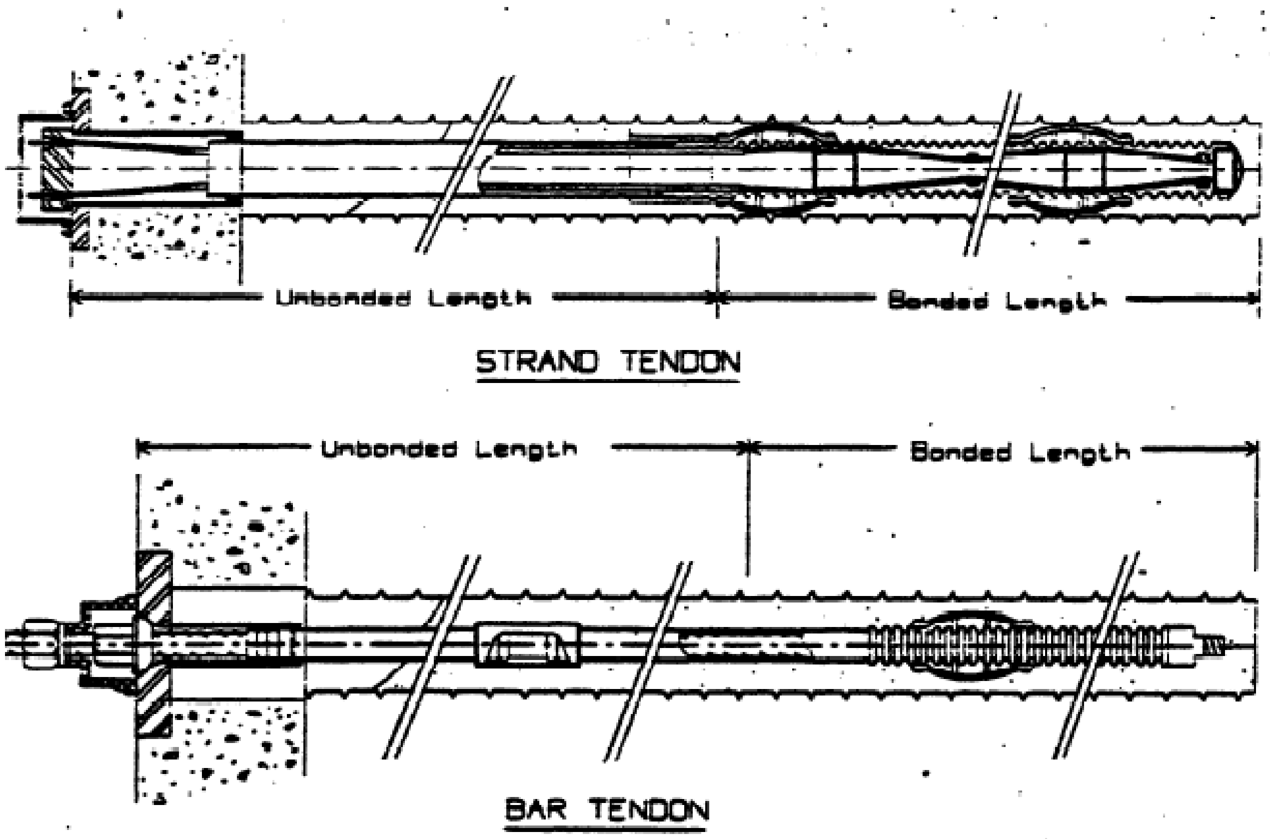


Figure 9-31. Bar or Strand Tendons

9.5.3.12 Steps for Checking Tieback Shoring Submittal

1 Review plans submittal for completeness.

2. Determine K_a and K_p

3 Develop pressure diagrams.

4 Determine forces.

5 Determine the moments around the top of the pile (or some other convenient location).

6 Solve for depth (D), for both lateral and vertical loads, and tieback force (T_H).

7 Check pile section.

8 Check anchor capacity.

9. Check miscellaneous details.
10. Check adequacy of tieback test procedure.
11. Review corrosion proposal.
12. General: Consider effects of wall deflection and subsequent soil settlement on any surface feature behind the shoring wall.

9.5.3.13 Example 9-6 Tieback Testing

Determine the long-term effects of creep.

9.5.3.13.1 Measurement and Time Method

Given:

The shoring plans indicate that a proof load shall be applied in 2 minutes or less then the load shall be held for ten minutes. The test begins immediately upon reaching the proof load value. Measurements of movement are to be taken at 1, 4, 6, 8 and 10 minutes. The proof load is to be 133% of the design load. The maximum permissible movement between 1 and 10 minutes of time will not exceed 0.1 inches. All tiebacks are to be tested. The system is anticipated to be in place for 1 year.

Solution:

Δ = 0.1 inches

T_1 = 1 minute

T_2 = 10 minutes

$$T_3 = (1Y)\left(365\frac{D}{Y}\right)\left(24\frac{H}{D}\right)\left(60\frac{M}{H}\right) = 525{,}600 \text{ minutes}$$

$$C = \frac{\Delta_{1-2}}{\left[\log_{10}\left(\frac{T_2}{T_1}\right)\right]} = \frac{0.1}{\left[\log_{10}\left(\frac{10}{1}\right)\right]} = 0.1$$

$$\text{Long-term } \Delta_{2-3} = (C)\log_{10}\left(\frac{T_3}{T_2}\right) = (0.1)\log_{10}\left(\frac{525{,}600}{10}\right) = 0.47 \text{ inches} \approx \frac{1}{2} \text{ inch}$$

The proof load and duration of test are reasonable and exceed the minimums shown in Table 9-2. Applying the proof load in a short period of time and beginning the test immediately upon reaching that load ensure the test results will be meaningful and can be compared to the calculated long-term creep movement for the anchor.

If the shoring system were in close proximity to an existing structure that could not tolerate an l/2 inch of settlement the design would not be acceptable. If the shoring would not affect permanent structures or when the shoring might support something like a haul road, the anticipated movement would be tolerable.

9.5.3.13.2 Lift Off Load Method

Given:

Lift off test will be performed 24 hours after wedges are seated (1 minute). The force at seating the wedges will be 83,000 pounds and the lift off force will be no less than 67,900 pounds.

L $\approx$ 20 ft which is the unbonded length of 15' + 5'

A = 0.647 in²

E = 28x10^6 psi

T_2 = 1 minute, this is the time the wedges are seated

$$\Delta_{1-2} \approx \frac{(P_1 - P_2)L}{AE}$$

$$\approx \frac{(83{,}000 - 67{,}900)(20)(12)}{(0.647)(28x10^6)} \approx 0.2 \text{ in}$$

$$C \approx \frac{0.2}{\left[\log_{10}\left(\frac{1440}{1}\right)\right]} = 0.06$$

$$\text{Long term } \Delta_{2-3} \approx (C)\log_{10}\left(\frac{T_3}{T_2}\right) = (0.06)\log_{10}\left(\frac{525{,}600}{1}\right)$$

$$\approx 0.34 \text{ inches } \approx \frac{5}{16} \text{ inch}$$

9.6 *SUMMARY*

The Department has an obligation with respect to trenching and shoring work. Be informed of legal responsibilities and requirements (Refer to CHAPTER 1).

Soil Mechanics (Geotechnical Engineering) is not a precise science. Be aware of the effects assumptions can make. Simplified engineering analysis procedures can be used for much of the trenching and shoring work that will be encountered.

The actual construction work is of equal importance to the engineering design or planning. The Contractor and the Engineer must always be alert to changed conditions and must take appropriate action. Technical assistance is available. The Engineer at the jobsite must be able to recognize when he needs help. The need for good engineering judgment is essential.

Work involving railroads requires additional controls and specific administrative procedures.

The following is a summary of D.O.T. policy in regard to trench and excavation shoring work:

1 The law (State Statute, Section 137.6) requires that a California registered engineer review the Contractor's plans for temporary structures in connection with State Highway work. Shoring plans are included in this category.

2 The Resident Engineer will ascertain that the Contractor has obtained a proper excavation or trenching permit from Cal/OSHA before any work starts, and that the permit (or copy) is properly posted at the work site.

3 If the trench is less than 20 feet deep and the Contractor submits a plan in accordance with the Construction Safety Order Standard Details, it is not necessary to have the plans prepared by a Professional Engineer. The Resident Engineer will confirm that the Contractor's plan does indeed conform to the Cal/OSHA Standard Details and need not make an independent engineering analysis.

4 If a trench is over 20 feet in depth, or if the Cal/OSHA Details cannot be used; the plans must be prepared by a Professional Engineer.

5. When shoring plans are designed by firms specializing in temporary support systems and soil restraint (including sloping), good engineering judgment is to prevail for review. Shoring designs by such firms may appear less conservative when analyzed using the methods proposed in this Manual. Consequently, the shoring plan may need to be reviewed in the manner in which it was designed.
6. If the Contractor's shoring plan deviates from the Construction Safety Order Details, the plan must be prepared by a California Registered Professional Engineer and the reviewing Engineer will perform a structural analysis.
7. For any shoring work that requires review and approval by a Railroad, the Sacramento OSC Office will be the liaison between the project and the Railroad. The Structure Representative will submit the Contractor's shoring plans to OSC Sacramento after review. The review should be so complete that the plans are ready for approval. The Structure Representative should inform the Contractor of the proper procedure, and the time, required for Railroad review and approval.
8. Any revisions to plans should be done by the plan originator or by his authorized representative. Minor revisions may be made on plans but the revisions should be initialed and dated by the person making the changes.