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CHAPTER 10

SPECIAL CONDITIONS

Chapter 10: Special Conditions

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10-1 Special Conditions

The best shoring system design in the world is of little value if the soil being supported does not act as contemplated by the designer. Potential adverse soil properties and changing conditions need to be considered and monitored. This chapter reviews several of the more common challenges.

Proper placement of anchor blocks (similar to ground anchors) is one challenge, and easily overlooked. Anchors placed within a soil failure wedge will not provide the resistance value needed when soil movement in the active zone occurs. Additional information regarding anchors may be found in the *USS Steel Sheet Piling Design Manual* (note that additional information on this resource can be found in Appendix D, *Sheet Piles*).

Another challenge is soil movement within the shoring system. Cohesive soils tend to expand and may push upward into an excavation. Expanding soils may also produce additional forces on the shoring system, and may induce lateral movement of the shoring system. Soil rising in an excavation indicates that soil is settling somewhere else. Water rising in an excavation can lead to quick conditions, while water moving horizontally can transport soil particles, possibly leaving unwanted voids at critical locations. Soil heave (movement of the soil in the bottom of the excavation due the soil pressures outside the system) is another condition to be aware of.

Excavating in an area with a high water table or within a waterway is another challenging condition. A cofferdam shoring system is generally employed for these conditions. This chapter reviews the topic of ”piping” and stream flow pressures against the exposed sided of a cofferdam in which an unbalanced hydrostatic head occurs. The sizing of seal-course concrete often used at the bottom of a cofferdam is discussed in the Structure Construction (SC) *Foundation Manual.*

The last challenge presented is stability of the soils around the shoring system. A consideration for the stability of exposed slopes, and that of global stability of the system, is the potential for a failure due to slippage of the soil around the shoring system along a surface offering the least amount of resistance. This potential is present for most types of shoring systems, with the exception being two-sided systems in level ground (i.e., a traditional trench). Although global failures usually happen suddenly, occasionally there are indications of small slope movements hours and sometimes days before a global failure.

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

10-2 Use of Anchor Blocks

Lateral support for sheet pile and/or soldier pile walls can be provided by tie rods that extend to an anchor block (also called deadman anchor) sufficiently behind the face of the excavation. Each rod can be connected to a single anchor block or multiple blocks spaced along the rod. Occasionally, a shoring design may utilize not an individual block, but a continuous block (wall) running parallel to the excavated face. See Figure 10-1**.** Tie rod spacing is generally determined by the need to limit the deflection of the shoring face, and the maximum moment of the soldier pile.

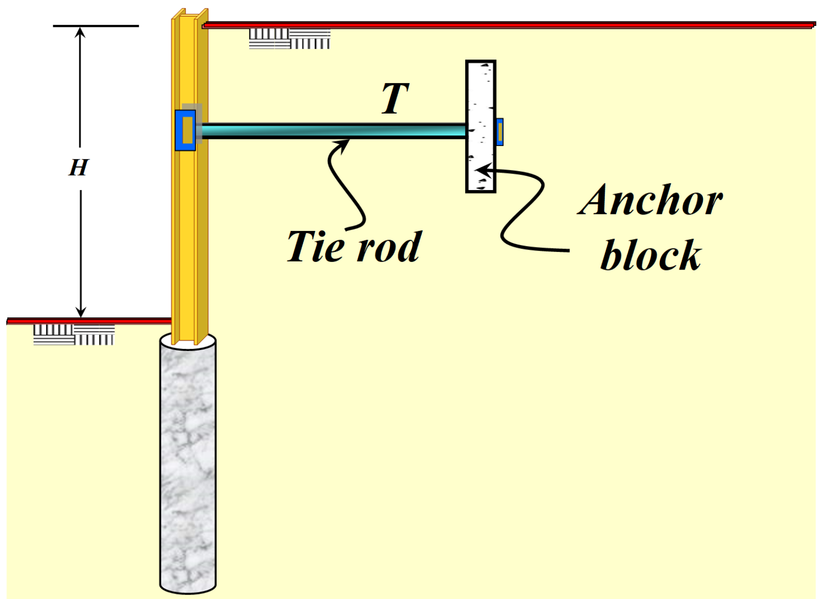


Figure 10-1. Anchor Block and Tie Rod

Using anchor blocks is simply another means for designing a restrained system, rather than using drilled ground anchors or struts. Use of an anchor block differs from ground anchors by how they develop resistance. An anchor block system's resistance is through the passive pressure developed in front of the anchor block, rather than soil-ground anchor bond strength along the bonded section of a drilled hole.

The size, shape, depth, and location of an anchor block affect the resistance capacity developed by that anchor. Figure 10-2 illustrates how the distance of the anchor block from the wall affects capacity.

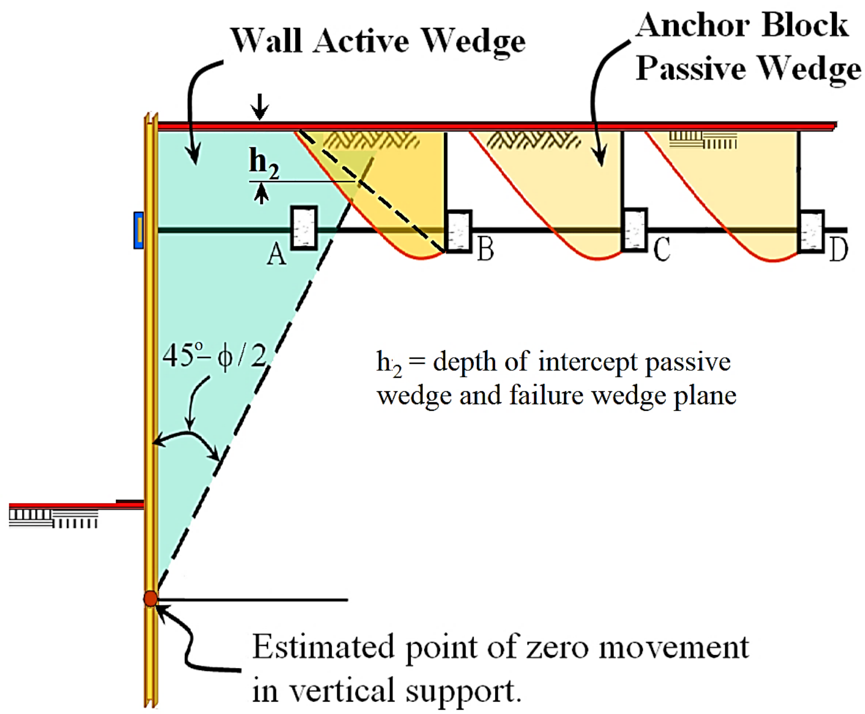


Figure 10-2. Anchor Block Position Relative to Wall Face and Failure Plain

1. Anchor block A is located inside active wedge and offers no resistance.
2. Anchor block B resistance is reduced due to overlap of the active wedge (wall) and the passive wedge (anchor).
    - Anchor reduction: (Granular soils)

$$\Delta P_p = \frac{\gamma h_2^2 (K_p - K_a)}{2} \tag{10-2-1}$$

    - This reduced portion becomes a load transferred to the wall as well.
3. Anchor block C develops full capacity but increases pressure on the wall.
4. Anchor block D is similar to Anchor block C. Anchor block D develops full capacity but increases pressure on the wall.

To follow are some considerations when dealing with anchor blocks. Anchor blocks should be placed against firm, undisturbed, or recompacted soil, and a safety factor of 2 is recommended for all anchor blocks. The capacities of anchor blocks are, of course, a function of the soil parameters. Other factors which affect the resistance of an anchor block include the depth of the anchor relative to the ground surface, and the proportions of the block and its spacing; i.e., whether it behaves as a continuous or a singular element. In the following section, we will examine properly located anchors at position **D** in Figure 10-2 (assuming cohesionless soils), beginning with a continuous block at or near the ground surface.

10-2.01 Anchor Block in Cohesionless Soil

10-2.01A Case A - Anchor blocks at or near the ground surface; d ≤ H/2

The forces acting on an anchor block at the ground surface are shown in Figure 10-3.

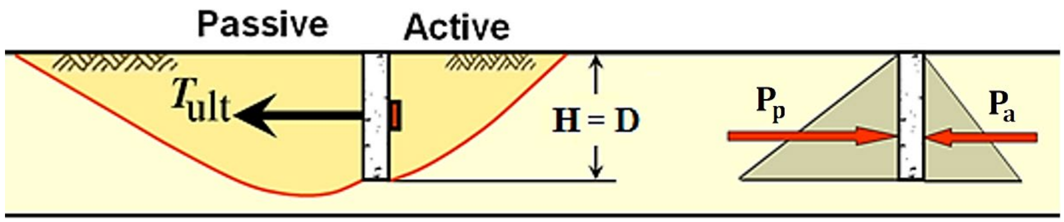


Figure 10-3. Anchor Block in Cohesionless Soil at the Ground Surface

When the block is not at the surface but the depth, **d**, is within **H/2,** it is assumed the influence of the block does extend to the ground surface as shown in Figure 10-4.

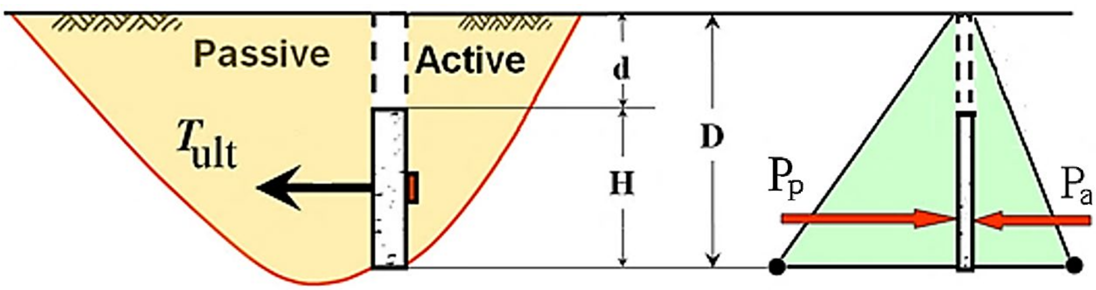


Figure 10-4. Continuous Anchor Block in Cohesionless Soil within H/2 of Ground Surface

Note: For Figures 10-3, 10-4, 10-8, and 10-11, the right side of each figure does not show the force pulling on the anchor block. That force is equal to **T**, and the maximum value for **T** is equal to **T_{ult}**.

The basic equation for a continuous anchor block to calculate the ultimate capacity is shown in Equation 10-2-1. If **L** is assumed to be 1 foot, the result is the capacity in pounds per linear foot of the block.

$$\mathbf{T_{ult} = L(P_p - P_a)} \qquad (10\text{-}2\text{-}2)$$

Where:

$$\mathbf{P_a = K_a \gamma \frac{D^2}{2}} \qquad (10\text{-}2\text{-}3)$$

$$\mathbf{P_p = K_p \gamma \frac{D^2}{2}} \qquad (10\text{-}2\text{-}4)$$

Substituting Equation 10-2-3 and Equation 10-2-4 into Equation 10-2-2 then:

$$\mathbf{T_{ult}} = \boldsymbol{\gamma}\frac{\mathbf{D^2}}{\mathbf{2}}\left(\mathbf{K_p} - \mathbf{K_a}\right)\mathbf{L} \qquad (10\text{-}2\text{-}5)$$

L = Length of the anchor block (depicted as "**w**" in Figure 10-5).

The conventional earth pressure theories using two-dimensional conditions corresponding to long (continuous) walls can be used to calculate the resistance force against the anchor block movement. An anchor block is considered continuous when its length exceeds its height by three or more times. The anchor block is otherwise considered to be isolated and has the advantage of an increased capacity by considering a three-dimensional analysis as described below.

In the case of isolated or short anchor blocks (see Figure 10-5 and 10-6 below), a larger passive pressure may develop because of three-dimensional effects due to a curved failure surface at both ends of the block, resulting in a wider passive zone in front of the anchor block, as shown below.

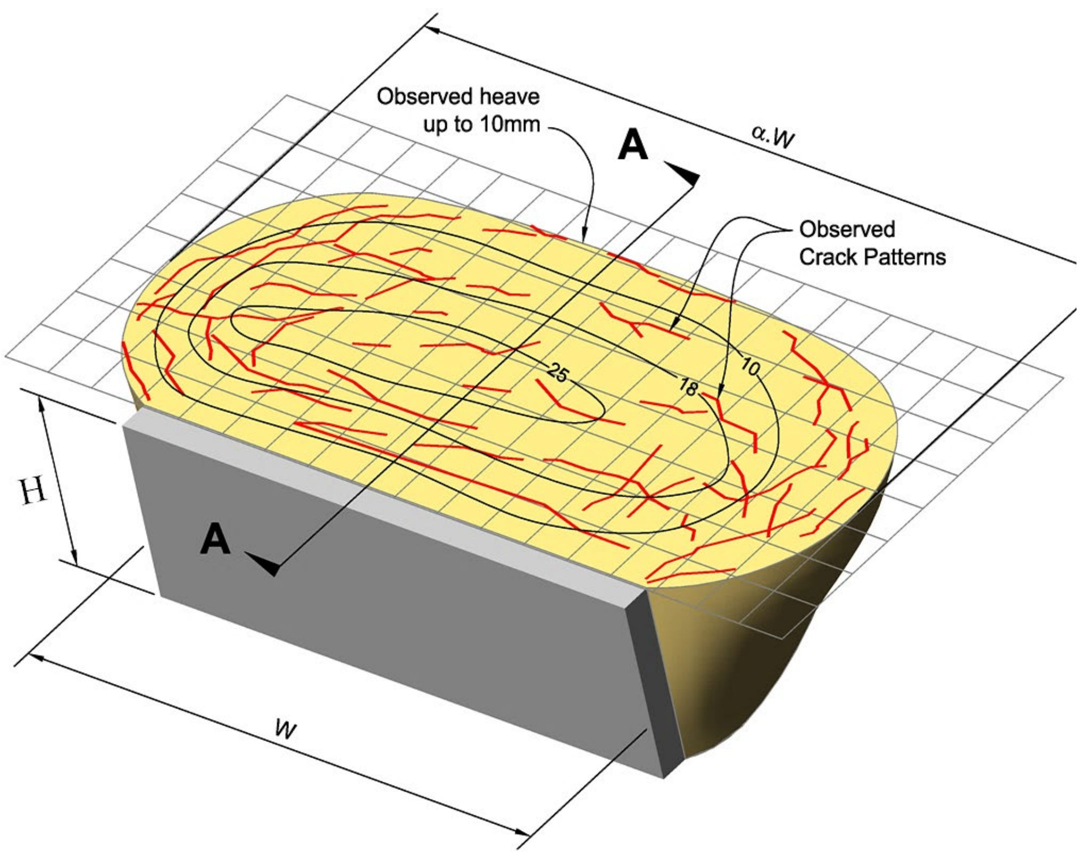


Figure 10-5. Anchor Block in 3D (Shamsabadi, A., Nordal, S., 2006)

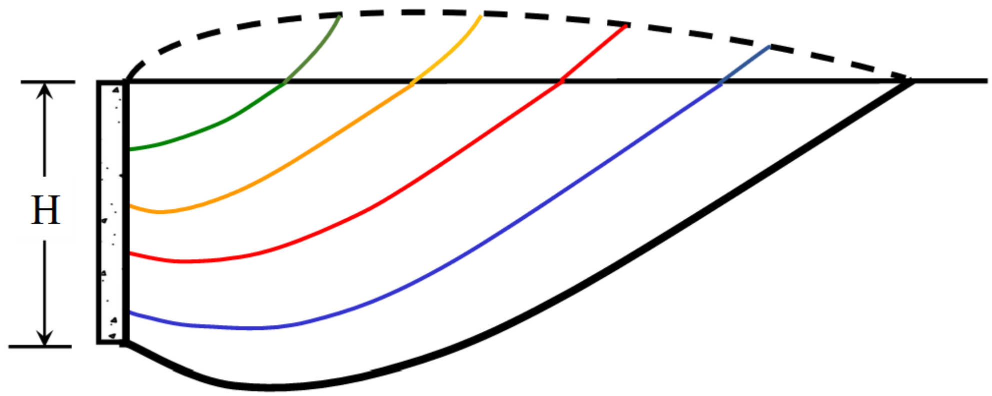


Figure 10-6. Section A-A (Shamsabadi, A., et al., 2007)

The ratio between three-dimensional and two-dimensional soil resistance varies with the soil friction angle and the depth below the ground surface. N. K. Ovesen studied and performed 32 different model tests for fully mobilized anchor blocks in granular soil. The resulting figures can be used to estimate the magnitude of the three-dimensional effects. Ovesen's method utilizes a three-dimensional factor, **R**, based on his test results, which differentiate the results of isolated versus continuous blocks. Figure 10-7 illustrates isolated anchor blocks near the ground surface.

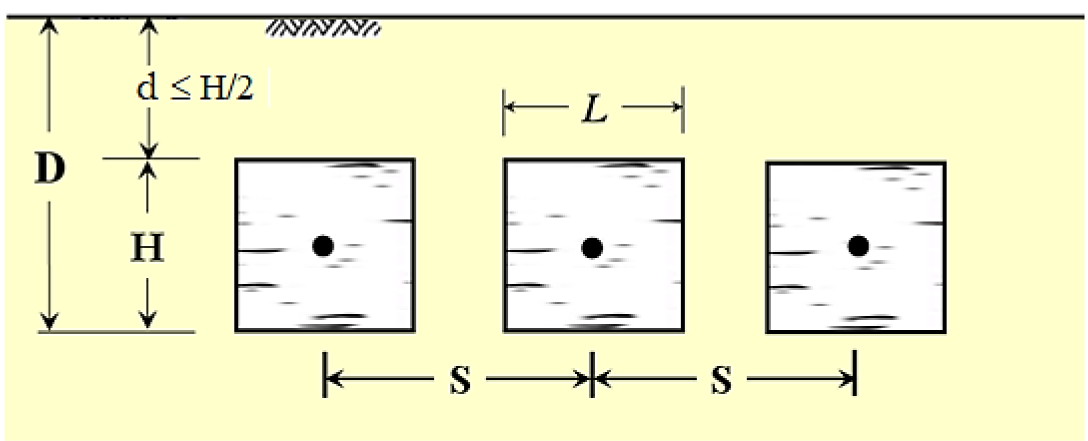


Figure 10-7. Isolated Anchor Blocks in Cohesionless Soil

$$\mathbf{T_{ult} = R\left[\gamma \frac{D^2}{2} \Delta KL\right]} \qquad (10\text{-}2\text{-}6)$$

$$\mathbf{\Delta K = (K_p - K_a)} \qquad (10\text{-}2\text{-}7)$$

L = Length of the anchor block.

$$\mathbf{R}/_{\Delta\mathbf{K}} = \mathbf{1} + \Delta\mathbf{K}^{2/3}\left[\mathbf{1.1E}^{4} + \frac{\mathbf{1.6B}}{\mathbf{1} + \mathbf{5}\frac{\mathbf{L}}{\mathbf{H}}} + \frac{\mathbf{0.4}\Delta\mathbf{KE}^{3}\mathbf{B}^{2}}{\mathbf{1} + \mathbf{0.05}\frac{\mathbf{L}}{\mathbf{H}}}\right] \quad (10\text{-}2\text{-}8)$$

Where:

$$\mathbf{B} = \mathbf{1} - \left(\frac{\mathbf{L}}{\mathbf{S}}\right)^{2} \quad (10\text{-}2\text{-}9)$$

and,

$$\mathbf{E} = \mathbf{1} - \frac{\mathbf{H}}{\mathbf{d} + \mathbf{H}} \quad (10\text{-}2\text{-}10)$$

10-2.01B Case B - Anchor block failure surfaces do not extend to the ground surface 1.5 ≤ D/H≤ 5.5

The forces acting on an anchor, which is not near the ground surface, are shown in Figure 10-8.

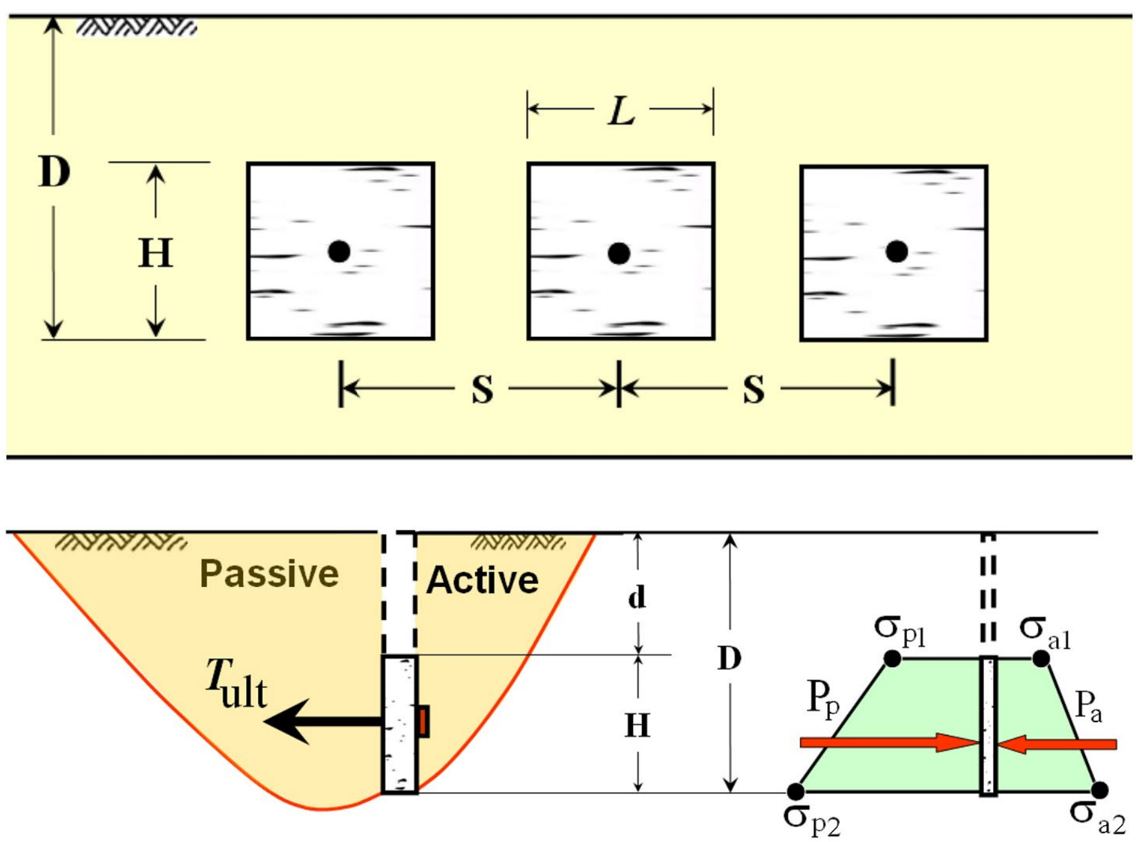


Figure 10-8. Anchor Block not near the Ground Surface: 1.5 ≤ D/H≤ 5.5

The basic equation to calculate the capacity of a continuous anchor block with length **L**, not extended near the ground surface, is shown in Equation 10-2-2.

$$\mathbf{T_{ult} = L(P_p - P_a)}$$

Where the $\mathbf{P_a}$ and $\mathbf{P_p}$ are the resultant forces from the areas of active and passive earth pressure developed in the front and back of the anchor block, as shown in Figure 10-8 and Equations 10-2-11 and 10-2-14.

$$\mathbf{P_a = \left[\frac{\sigma_{a1} + \sigma_{a2}}{2}\right] H} \qquad (10\text{-}2\text{-}11)$$

Where:

$$\mathbf{\sigma_{a1} = \gamma d K_a} \qquad (10\text{-}2\text{-}12)$$

and,

$$\mathbf{\sigma_{a2} = \gamma D K_a} \qquad (10\text{-}2\text{-}13)$$

$$\mathbf{P_p = \left[\frac{\sigma_{p1} + \sigma_{p2}}{2}\right] H} \qquad (10\text{-}2\text{-}14)$$

Where:

$$\mathbf{\sigma_{p1} = \gamma d K_p} \qquad (10\text{-}2\text{-}15)$$

and,

$$\mathbf{\sigma_{p2} = \gamma D K_p} \qquad (10\text{-}2\text{-}16)$$

Substituting Equation 10-2-11 and Equation 10-2-14 into Equation 10-2-2 then:

$$\mathbf{T_{ult} = L\left[\frac{1}{2}\left(\left(\gamma d K_p + \gamma D K_p\right) - \left(\gamma d K_a + \gamma D K_a\right)\right)\right] H} \qquad (10\text{-}2\text{-}17)$$

In case of isolated and short anchor blocks, the Ovesen's three-dimensional factor (**R**) must be estimated using Equation 10-2-8. Then use the following equation for $\mathbf{T_{ult}}$.

$$\mathbf{T_{ult} = R\,(L)\left[\frac{1}{2}\left(\left(\gamma d K_p + \gamma D K_p\right) - \left(\gamma d K_a + \gamma D K_a\right)\right)\right] H} \qquad (10\text{-}2\text{-}18)$$

10-2.02 Anchor Block in Cohesionless Soil Where 1.5 ≤ D/H ≤ 5.5 – Alternative Method

The chart shown in Figure 10-9 is based on sand of medium density (ϕ = 32.5 degrees). For other values of ϕ, a linear correlation may be made from (ϕ / 32.5 degrees). 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.

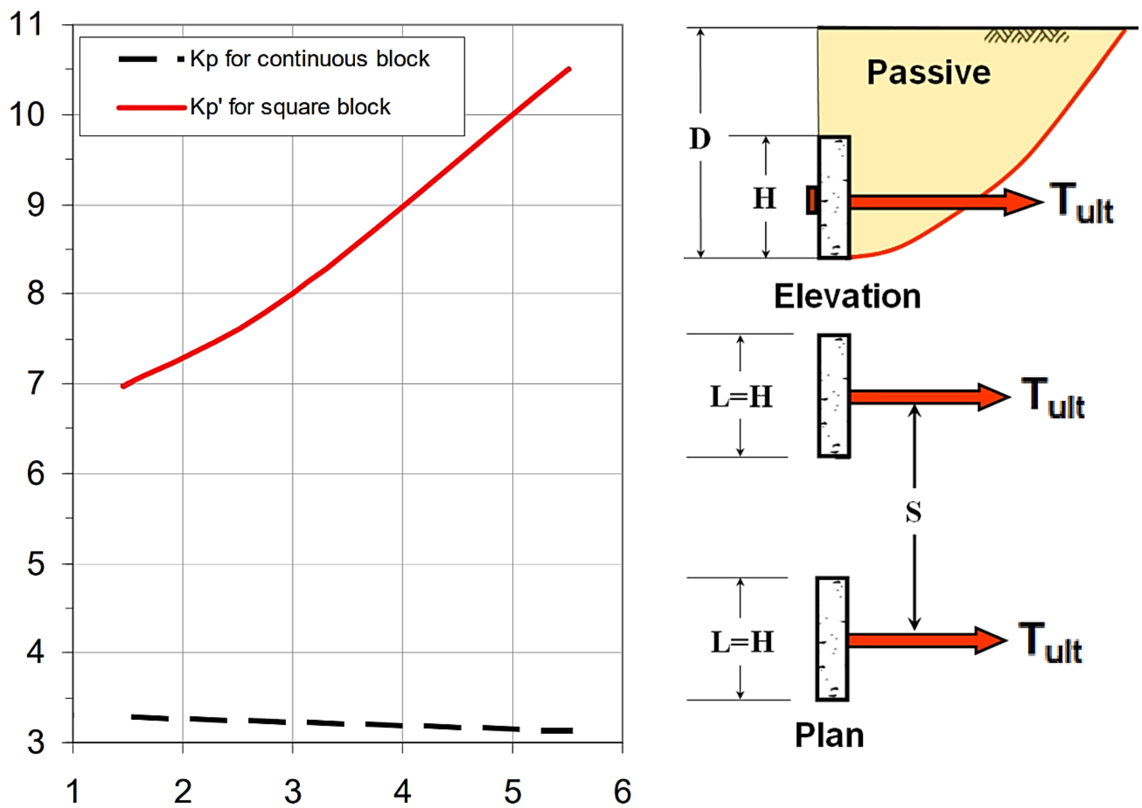


Figure 10-9. Anchor Block in Cohesionless Soil 1.5 ≤ D/H ≤ 5.5 – Alternate Method

$$\mathbf{T_{ult}} = [\boldsymbol{\gamma} \mathbf{D^2 K_P}' \,(\mathbf{L})]/\mathbf{2} \qquad (10\text{-}2\text{-}19)$$

10-2.03 Anchor Block in Cohesive Soil Near the Ground Surface d ≤ H/2

Recall from Chapter 4 that for cohesive soil, the pressure diagrams for the active and passive forces look as shown in Figure 10-10. Thus the forces acting on an anchor are shown in Figure 10-11. For the case of **d ≤ H/2**, where **H** is the height of the block, it is assumed that the anchor essentially extends to the ground surface. The capacity of the anchor depends upon whether it is considered continuous or short.

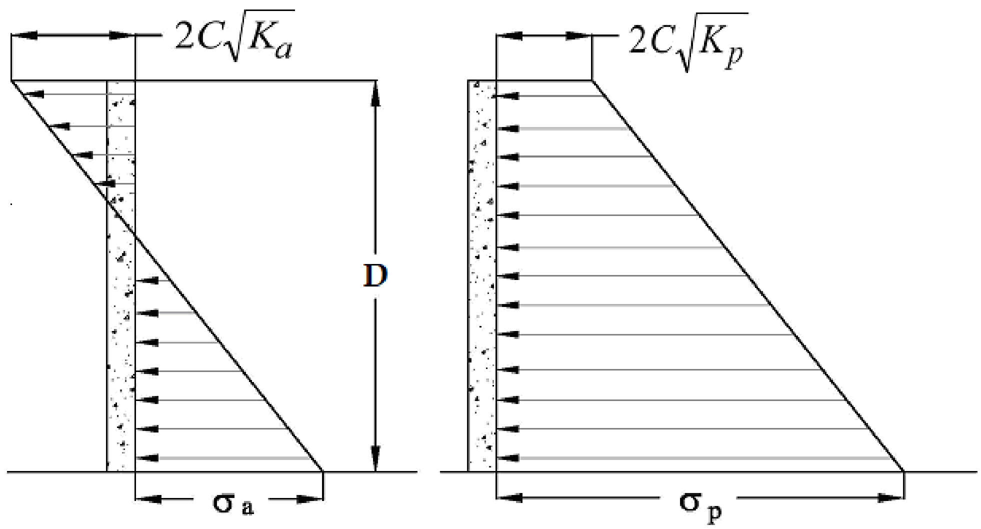


Figure 10-10. Cohesive Soil Pressure Diagrams

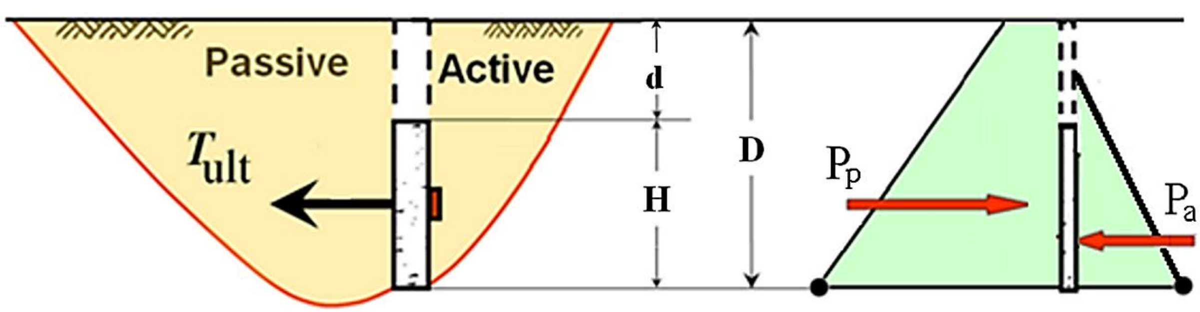


Figure 10-11. Anchor Block in cohesive soil near the ground surface d ≤ H/2

Where:

$$\sigma_p = \gamma D K_p + 2C\sqrt{K_p} \quad (10\text{-}2\text{-}20)$$

$$\sigma_a = \gamma D K_a - 2C\sqrt{K_a} \quad (10\text{-}2\text{-}21)$$

The pressure diagram shown in Figure 10-10 for cohesive soils assumes short load duration. Over a period of years, creep is likely to alter the pressure diagram. Therefore, conservative assumptions should be used in the analysis, such as **c** = 0 and ϕ = 27°.

The basic equation is:

$$\mathbf{T_{ult} = L(P_p - P_a)} \quad (10\text{-}2\text{-}22)$$

Where: **L** = Length of anchor block.

For continuous anchor blocks:

$$\mathbf{P_p = \frac{\gamma D^2 K_p}{2} + 2CD\sqrt{K_p}} \quad (10\text{-}2\text{-}23)$$

$$\mathbf{P_a = \frac{\left(\gamma D K_a - 2C\sqrt{K_a}\right)\left(D - \frac{2C}{\gamma}\right)}{2}} \quad (10\text{-}2\text{-}24)$$

It is recommended that the tension zone be neglected.

For short anchor blocks where $\mathbf{H \leq L}$:

$$\mathbf{T_{ult} = L(P_p - P_a) + 2CD^2} \quad (10\text{-}2\text{-}25)$$

10-2.04 Anchor Blocks in Cohesive Soil Where d ≥ H/2

The chart shown in Figure 10-12 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 10-12 is from *Strength of Deadmen Anchors in Clay*, Thomas R. Mackenzie, Master's Thesis Princeton University, Princeton, New Jersey, 1955.

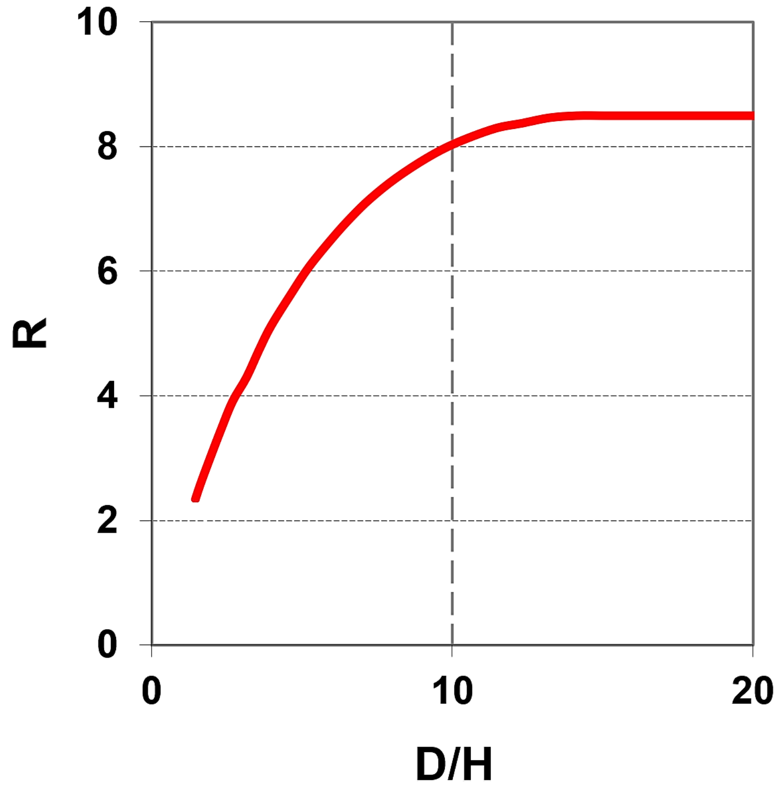


Figure 10-12. Anchor Block in Cohesive Soil d ≥ H/2

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

When using the graph in Figure 10-12, the reader needs to check that they are using a value of **D/H** that is greater than or equal to 1.5.

10-2.05 Example 10-1 Problem – Anchor Blocks

Given:

Check the adequacy of the Contractor's anchor blocks in the proposed shoring system shown in Figure 10-13. The 2 foot wide by 2 foot long anchor blocks are to be buried 3 feet below the ground surface. The required tie load on the wall is 11,000 lbs.

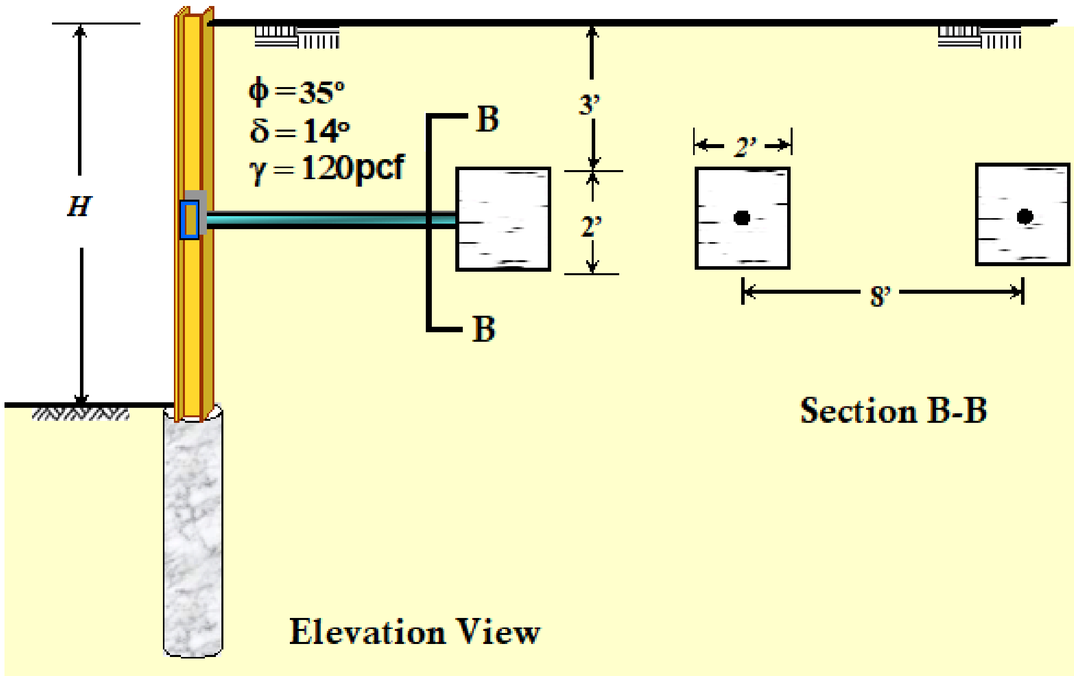


Figure 10-13. Anchor Block Example 10-1

Solution:

Step 1: Calculate active and passive earth pressure in the front and back of the anchor block. Begin by calculating the active and passive coefficients. Using the Coulomb equations, $\mathbf{K_a}$ = 0.27 and $\mathbf{K_p}$ = 6.27. The soil is cohesionless, and the cohesion value (**c** value) is equal to zero.

Since wall friction, δ, is included in the given information, these calculations take into account this friction (see Chapter 4, Section 4-3, *Developing Earth Pressures for Granular Soil,* for additional information on these topics).

$$\sigma_{a1} = \gamma d k_a \times \cos(\delta) = 120 \times 3 \times 0.27 \times \cos(14°) = 94.31 \quad (10\text{-}2\text{-}26)$$

$$\sigma_{a2} = \gamma H k_a \times \cos(\delta) = 120 \times 5 \times 0.27 \times \cos(14°) = 157.19 \quad (10\text{-}2\text{-}27)$$

$$P_a = \left[\frac{94.31 + 157.19}{2}\right] 2 = 251.50 \quad (10\text{-}2\text{-}28)$$

$$\sigma_{p1} = \gamma d K_p \times \cos(\delta) = 120 \times 3 \times 6.27 \times \cos(14°) = 2{,}190.15 \quad (10\text{-}2\text{-}29)$$

$$\sigma_{p2} = \gamma H K_p \times \cos(\delta) = 120 \times 5 \times 6.27 \times \cos(14°) = 3{,}650.25 \quad (10\text{-}2\text{-}30)$$

$$P_p = \left[\frac{2{,}190.15 + 3{,}650.25}{2}\right] 2 = 5{,}840.40 \quad (10\text{-}2\text{-}31)$$

Step 2: Use Ovesen's theory to estimate the magnitude of the three-dimensional effects **R**, using Equation 10-2-8.

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

$$\Delta K_{horz} = (K_p - K_a)\cos(\delta) = (6.27 - 0.27)\cos(14°) = 5.82 \quad (10\text{-}2\text{-}32)$$

$$B = 1 - \left(\frac{L}{S}\right)^2 = 1 - \left(\frac{2}{8}\right)^2 = 0.94 \quad (10\text{-}2\text{-}33)$$

$$E = 1 - \frac{H}{d + H} = 1 - \frac{2}{3 + 2} = 0.60 \quad (10\text{-}2\text{-}34)$$

$$R = 1 + 5.82^{2/3}\left[1.1 \times 0.6^4 + \frac{1.6 \times 0.94}{1 + 5\frac{2}{2}} + \frac{0.4 \times 5.82 \times 0.60^3 \times 0.94^2}{1 + 0.05\frac{2}{2}}\right] = 3.64$$

(10-2-35)

R = 3.64 > 2.0

Use: R=2 (see below)

The **R** value is the Ovesen factor that is equal to the 3-dimensional ultimate load divided by the 2-dimensional ultimate load. If the calculated **R** value is more than 2, then the **R** value that is used should be equal to 2.

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

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

(10-2-36)

Where **L** is the length of the anchor block.

$$FS = \frac{T_{ult}}{T} = \frac{22{,}355.6}{11{,}000.0} = 2.03 \quad (10\text{-}2\text{-}37)$$

10-3 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, $N_O = \gamma H/c$ (10-3-1)

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 10-14. The vertical load per unit length of the trench length at the bottom of the supports along line ***dc*** is the driving force to create heave in pounds per length of trench (plf). It can be calculated:

$$\mathbf{Q = W + (0.7B)q - S} \quad (10\text{-}3\text{-}2)$$

Where:

Q = Vertical load per unit length of trench.

W = Weight of soil column per unit length of trench $\mathbf{W = \gamma H\,(0.7B)}$.

B = Width of open excavation in feet.

q = Surcharge loading in psf.

S = Resistance of soil due to cohesion over depth of excavation,

S = (cH) in pound per unit length of the trench.

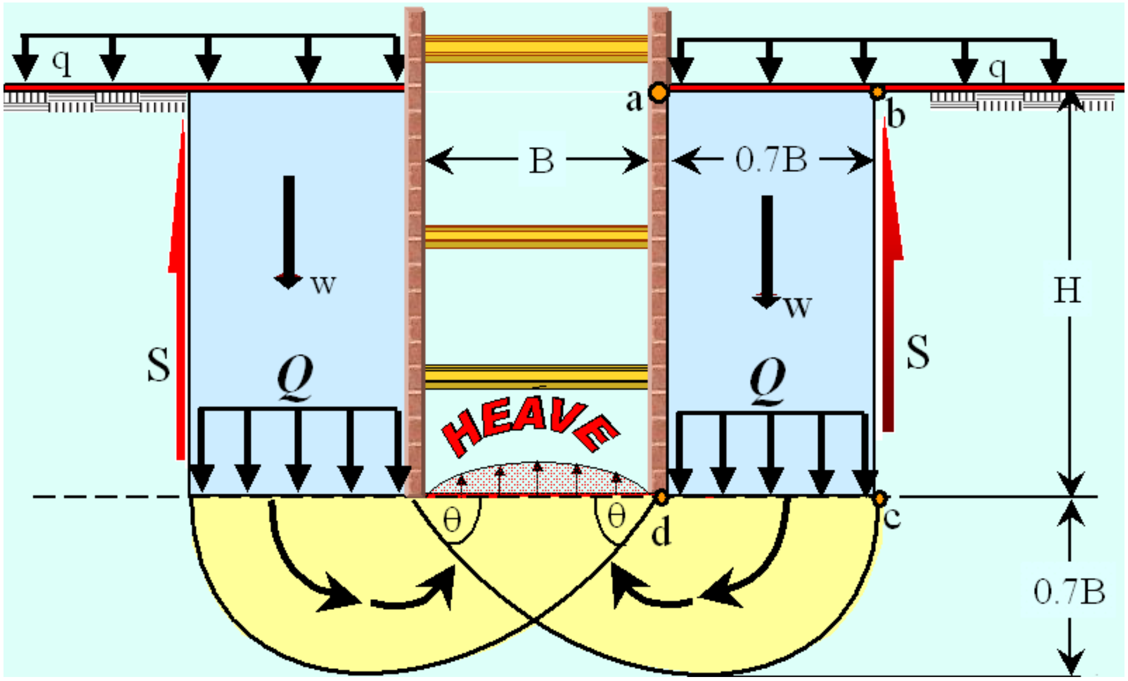


Figure 10-14. Bottom Heave

Through considering the mechanics of heave, the driving force may be treated as a load per unit length on a continuous foundation at the level of ***dc***, with the width of **0.7B**, and thus compare it to the bearing capacity analysis of a footing.

The resisting force is based on Terzaghi's bearing capacity theory by considering the driving force, **Q**, as a unit load from a foundation. The equation for the net ultimate load-carrying capacity per unit length per Terzaghi is:

$$\mathbf{Q_U = cN_C(0.7B)} \quad (10\text{-}3\text{-}3)$$

$\mathbf{Q_U}$ = Ultimate load carry capacity per unit length = Ultimate bearing capacity

c = Cohesion of soil in psf

$\mathbf{N_c}$ = Bearing capacity factor from Figure 10-15 and Equation 10-3-4

B = Width of open excavation in feet

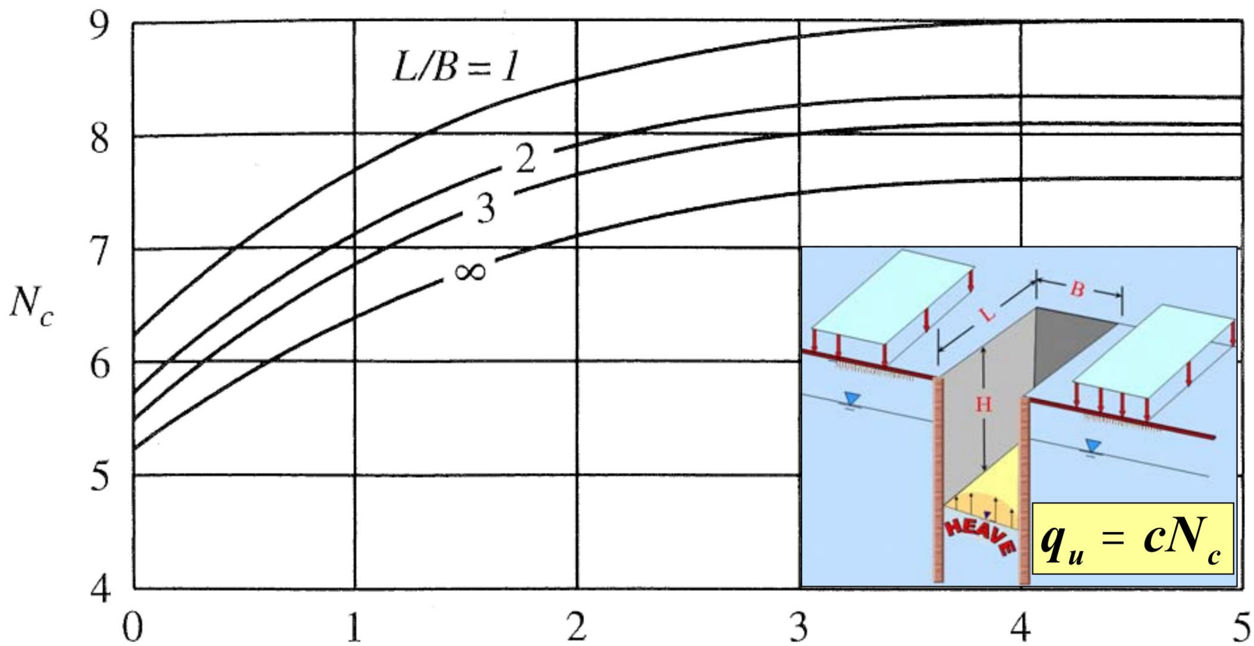


Figure 10-15. Bearing Capacity Factor from Bjerrum and Eide (1956)

The bearing capacity factor, **Nc**, shown in Figure 10-15, varies with the ratios of **H/B** and **L/B**. In general, for **H/B**:

$$\mathbf{N_{c(rectangle)}} = \mathbf{N_{c(square)}}\left(\mathbf{0.84 + 0.16\frac{B}{L}}\right) \text{ [from Bjerrum and Eide (1956)]}$$

(10-3-4)

Where:

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

B = Width of excavation in feet

L = Length of excavation in feet

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 only for excavations where **H>B**). When submerged or when installed in clay, another possible solution could be to over-excavate and construct a counterweight to the heaving force. Be aware that strutting a wall near its bottom will not prevent heave. Strutting only resists the lower shoring sides from rotating into the excavation.

10-3.01 Factor of Safety against Heave

The factor of safety (FS) against bottom heave as shown in Figure 10-16 is:

$$\mathbf{FS} = \frac{\mathbf{F_{RS}}}{\mathbf{F_{DR}}} = \frac{\mathbf{Q_U}}{\mathbf{Q}} \geq \mathbf{1.5} \qquad (10\text{-}3\text{-}5)$$

Where:

$\mathbf{F_{RS}}$ = Resisting Force = $\mathbf{Q_U}$ from Equation 10-3-3.

$\mathbf{F_{DR}}$ = Driving Force = **Q** from Equation 10-3-2.

Note that Equation 10-3-2 has a term of negative **S.**
S (resistance of soil due to cohesion over depth of excavation) is equal to **cH**, and is modelled as a negative driving force.

To reduce the risk of heave, it is recommended that a minimum safety factor of 1.5 should be used.

This factor of safety is based on the assumption that the clay layer is homogeneous, and at least to a depth of **0.7B** below the bottom of the excavation. However, if a hard layer of rock or rocklike material is within **0.7B** of the bottom of the excavation, then the failure surface will need to be modified to some extent. The depth to the hard layer of rock or rocklike material is **D**. **D** is measured from the excavation line to the top of the rock or rocklike material, and that depth is less than or equal to **0.7B**. The following steps would be made for this condition:

1. In Figure 10-14, the vertical distance of **0.7B** would be changed to **D**.
2. In Figure 10-14, the horizontal distance of **0.7B** would be changed to **D**.
3. In Equation 10-3-2, W would be changed from **γH** (**0.7B**) to **γHD**.
4. In Equation 10-3-2, the width for the surcharge would be changed from **0.7B** to **D**. The term of (**0.7B**)**q** would be changed to (**D**)**q**.
5. In Equation 10-3-2, the value of **S** (resistance of soil due to cohesion) would not change.
6. In Equation 10-3-3, the width for the bearing capacity would be changed from **0.7B** to **D**. The value of $\mathbf{Q_u}$ would be changed from $\mathbf{cN_c}$(**0.7B**) to $\mathbf{cN_c(D)}$.

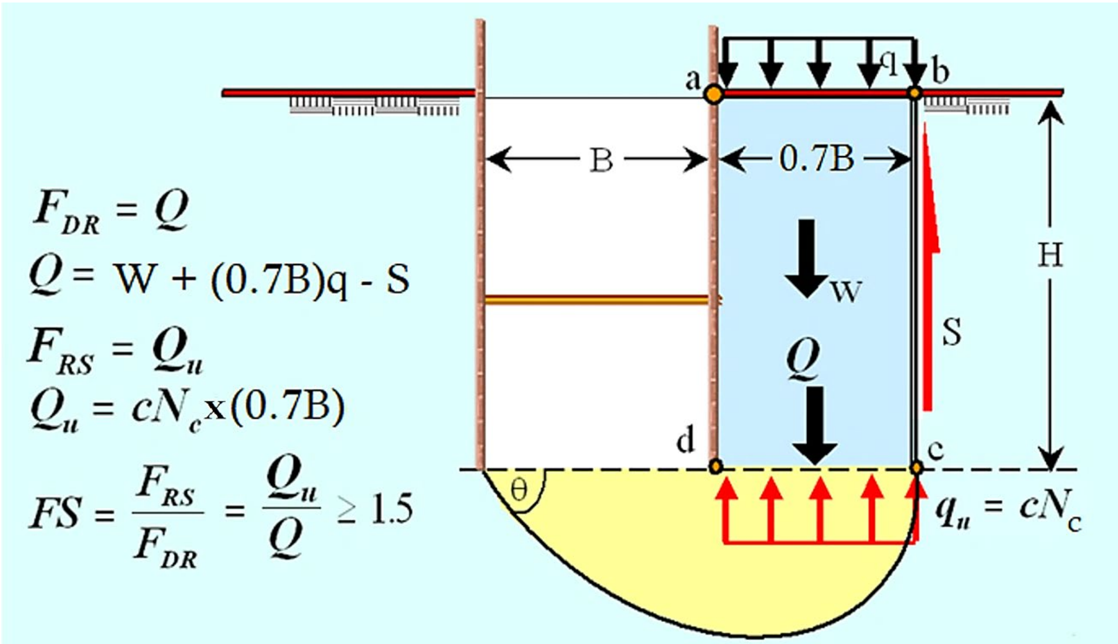


Figure 10-16. Driving and Resisting Forces and the Factor of Safety

10-3.02 Example 10-2 Problem – Heave Factor of Safety

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

q = 300 psf, **c** = 500 psf, γ = 120 pcf

Solution:

$\mathbf{N_{c(square)}}$ as determined from Figure 10-15 for H/B= 2 is 8.5. This is the value for L/B=1. However, the L/B value for this example is = 3.

$\mathbf{N_c}$ from Figure 10-15 (for H/B = 2 and L/B=3) is = 7.6.

Bearing capacity = **c x $\mathbf{N_c}$ x (0.7B)** = (500 psf) x (7.6) x (0.7 x 15 feet) ≈ 40 kip /ft

$$F_{DR} = W + (0.7B)q - S$$

$$W = (10.5 \times 30)0.120 = 37.8$$

$$\textbf{Surcharge} = (0.7)15(0.3) = 3.15$$

$$S = 0.5 \times 30 = 15$$

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

$$F_{RE} = q_u(0.7B) = 3.8 \times 10.5 \approx 40.0$$

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

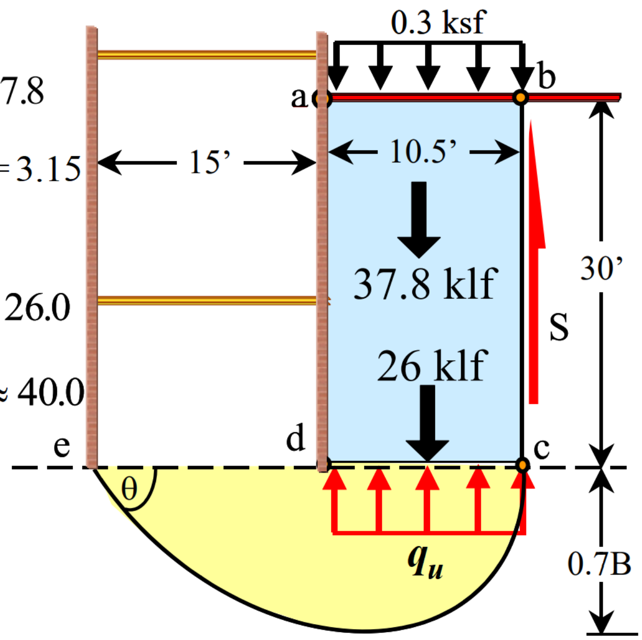


Figure 10-17. Heave Example Problem Calculations

(Note: figure only shows one side for simplicity)

10-4 Piping

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

One solution to this problem is to equalize the unbalanced hydraulic head by either allowing the excavation to fill with water or lower the water table outside the excavation by dewatering. If dewatering is used, the flow rate into the excavation will decrease, the shear strength of the soil will increase, and the soil will stop acting as a liquid. On Caltrans projects, one of the common methods used to protect or mitigate against piping is the use of a seal course. Refer to the [Foundation Manual](), Chapter 12, *Cofferdams and Seal Courses*, 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. The *USS Steel Sheet Piling Design Manual* contains charts on page 65 giving lengths of sheet pile embedment, which will result in an adequate factor of safety against piping. These charts are of particular interest and a good resource for cofferdams.

10-4.01 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 that can act along the sides of sheet piles as shown in Figure 10-18. The drag force, **D** (in pounds), is calculated with Equation 10-4-1 [from Ratay (1984)]:

$$\mathbf{D} = (\mathbf{A})(\mathbf{C_d})(\boldsymbol{\gamma}_\mathbf{w})\frac{\mathbf{V}^2}{\mathbf{2g}} \quad (10\text{-}4\text{-}1)$$

Where:

A	=	Projected area of the obstruction normal to the current in ft²
$\mathbf{C_d}$	=	Coefficient of drag
γ_w	=	Water unit weight in lbs/ft³
V	=	Velocity of the current in ft/sec
g	=	Acceleration due to gravity in ft/sec²

In English units, the numerical value of γ_ω is approximately equal to **2g**, without regard to units. (Recall that the weight of water is 62.4 lbs/ft³, and the acceleration of gravity is 32.2 ft/s²). Thus, Equation 10-4-1 can be simplified as follows:

$$\mathbf{D} = (\mathbf{A})(\mathbf{C_d})(\mathbf{V}^2) \quad (10\text{-}4\text{-}2)$$

Where:

A	=	as defined above
V	=	as defined above
$\mathbf{C_d}$	=	Coefficient of drag, lbs sec²/ft⁴ (Note: $\mathbf{C_d}$ is not dimensionless in the above equation for **D** to be in lbs.)
D	=	Drag force in lbs

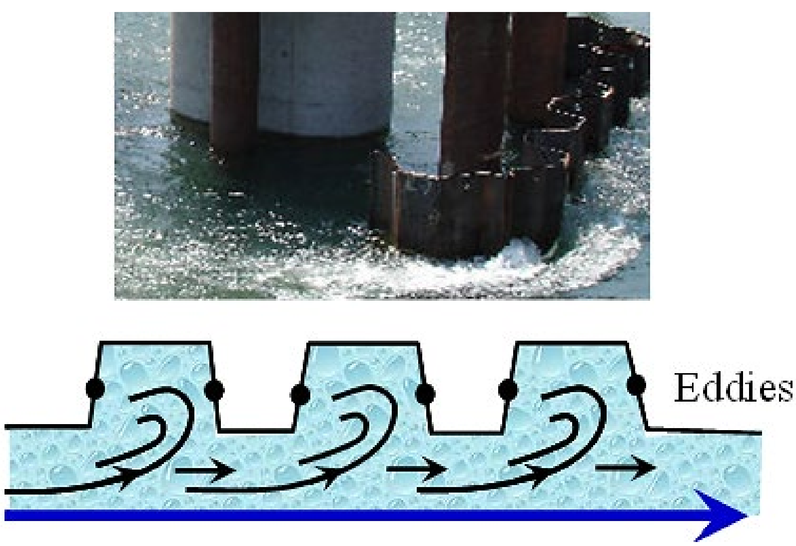


Figure 10-18. Hydraulic Forces on Cofferdams

Considering the roughness along the sides of the obstructions (as for a sheet pile cofferdam) the practical value for $\mathbf{C_d}$ = 2.0.

$$\mathbf{D} = \mathbf{2AV^2} \qquad (10\text{-}4\text{-}3)$$

The drag load, **D**, is applied in the same manner as a wind rectangular load on the loaded height of the obstruction (falsework or guyed elements).

Example: Determine the drag force on a 6-foot wide sheet pile cofferdam placed vertically in water with average depth of 6 feet flowing at 4 feet per second. For this example: $\mathbf{C_d}$ = 2.0.

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

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

The drag load, **D,** may then be added as an additional live load force distributed over the projected area of the cofferdam. If applied as a point load it would be placed at the centroid of the projected area, i.e., at the center of the 6-ft by 6-ft area for the example above. It is more appropriate to apply as a per square foot-loading, as illustrated in the calculation below. The applied load per square-foot generally would not govern for the stress in the sheet piles.

Drag Load = 1152 lbs/36 ft^2 = 32 psf

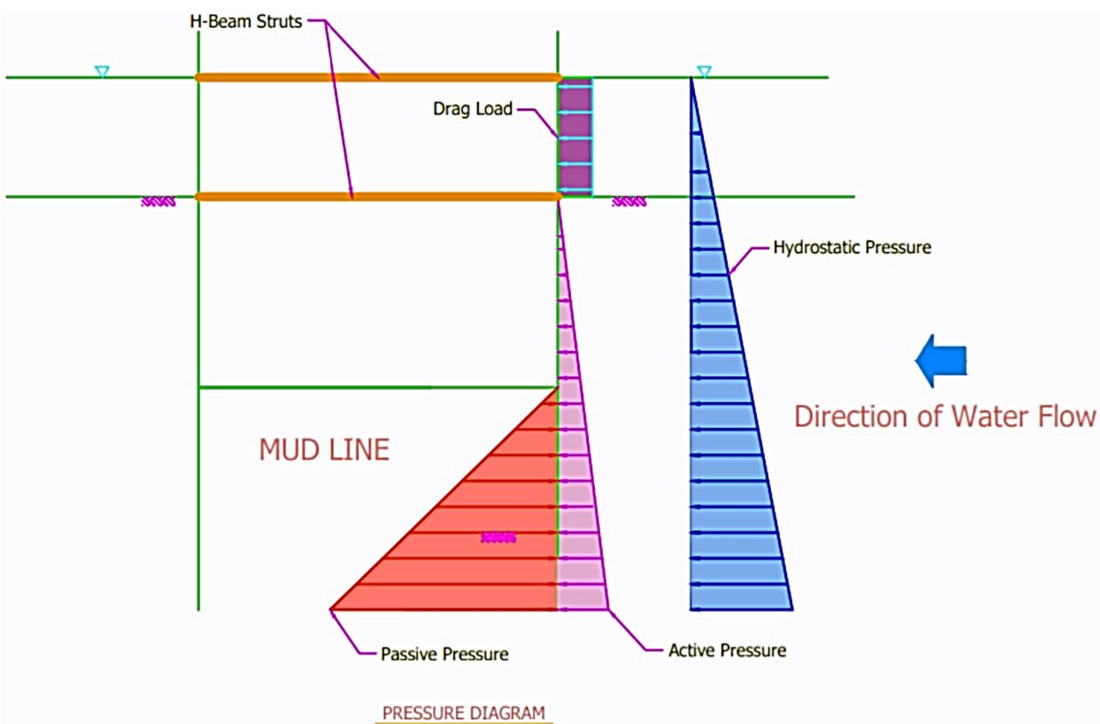


Figure 10-19. Drag and Hydraulic Forces on a Cofferdam

For the example above, illustrated in Figure 10-19, the Engineer may determine that the additional live load is negligible compared to the overall loads to the cofferdam system. It will likely be the primary contributor to the loading for the upper strut both in the final condition and in the sequence of construction. However, for a system nearing allowable capacities, the additional drag force produced by the water flow could require consideration.

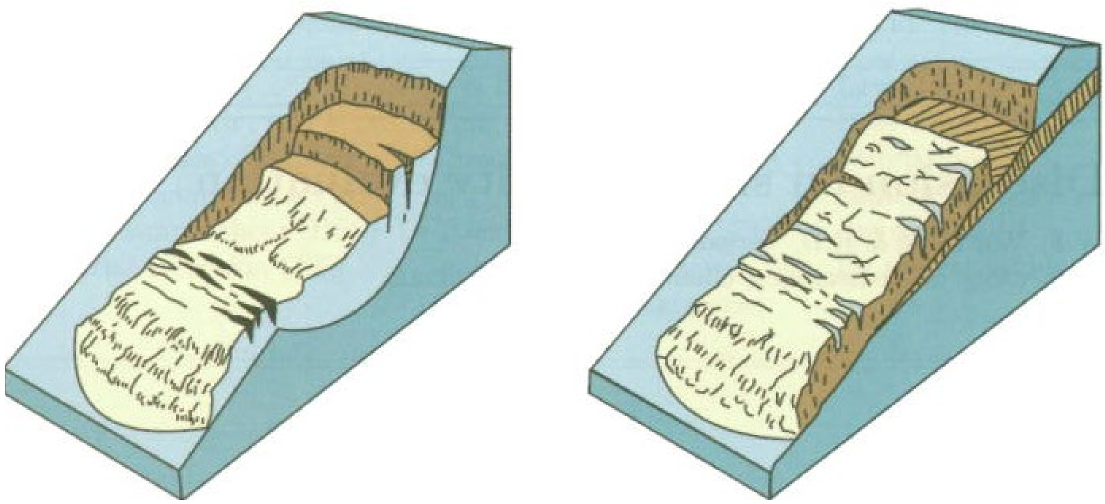
The characteristic of a cofferdam differs from a typical sheet pile system only in that it is intended to address both hydrostatic and flowing water. This added variable requires additional attention to the installation sequence proposed by the Contractor, as each step will require analysis of the system. Always obtain and review the Contractor's installation sequence. While the procedures vary by contractor, a typical installation may include the following:

1. Construct and install outside waler-ring template and anchor per the Contractor's submittal.
2. Install sheet piling to specified elevation.
3. Dewater cofferdam to elevation described in the Contractor's submittal to facilitate the installation of the first level of interior bracing. Verify calculations for cofferdam system in this configuration.
4. Install interior waler and struts and any required diagonal cross bracing struts. Verify cofferdam load capacities with bracing system installed.
5. Place concrete filler required at diagonal cross bracing struts.
6. Confirm Contractor's seal course thickness is adequate to resist hydrostatic pressure and verify use of proper cofferdam vent during seal course placement as required by the *Contract Specifications*, Section 19-3.03 B(4), *Earthwork – Structure Excavation and Backfill – Construction – Cofferdams.*
7. Excavate to specified elevation (usually bottom of seal course).
8. Place seal course concrete underwater and cure.
9. Dewater cofferdam. Verify cofferdam calculations at this configuration, fully dewatered, and braced with seal course placed.
10. Clean and prepare top of seal course for footing subgrade.

It should be noted that the Contractor's cofferdam system may require multiple levels of interior waler and struts, and the procedure of excavation and dewatering noted in the submittal should be followed closely.

10-5 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 10-20 shows some of the most common patterns of slope failure in soil. The slope failure of rocks is beyond the scope of this manual.



a-Rotational Slide b-Translational Slide

Figure 10-20. Common Pattern of Soil Slope Failure (USGS)

A stability analysis prepared by a Geotechnical engineer or Geologist should be requested from the Contractor when it appears that shoring or a cut slope presents a possibility of some form of slip failure. The discussion and examples that follow are intended to give the Engineer the ability to anticipate conditions needing further analysis. Geotechnical Services in Sacramento has the capability of performing computer-aided stability analysis to verify a contractor’s submitted analysis, and it is recommended the Engineer utilize this resource when they are tasked with performing a slope stability review.

The discussion and examples that follow for Fellenius and Bishop methods are provided to simply show the concept of the failure mechanism and analysis thereof. It is very important to be in agreement on the soil parameters with the Contractor, as these will have a significant effect on the factor of safety calculated, especially cohesion.

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 10-20. The popularity of limit-equilibrium methods is primarily due to their relative simplicity, and the many years of experience analyzing slope failures.

Construction surcharges such as equipment and stockpiled materials, may cause excavation instabilities and should be considered when performing a slope stability analysis. The slope stability analysis involves the following:

1. Obtain surface geometry, stratigraphy, and subsurface information.
2. Determine soil properties.
3. Determine soil-structure interaction, such as the presence of sheet piles, soldier piles, ground anchors, soil nails, and so forth.
4. Determine surcharge loads.
5. 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 found*,* utilizing multiple potential failure surfaces. Circular solutions to slope stability have been developed primarily due to the ease of this geometry during 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 10-20 (Rotational Slide) and Figure 10-21. Non-circular failure surfaces can be caused by adversely dipping bedding planes, zones of weak soil, or unfavorable ground water conditions.

10-5.01 Rotational Slides

Stability analysis of slopes with circular failure surfaces can be explained using a method of slices. Figure 10-21a shows an arc or a circle, AB, representing a trial failure surface. The soil above the trial surface is divided into a number of slices and given an incremental designation. The forces acting on a typical slice "**i**" are shown as b, c and d of Figure 10-21. The ordinary method of slices (Figure 10-21b), which is the simplest method, does not consider interslice forces acting on the side of the slices. The Simplified Bishop's Method of Slices (Figure 10-21c) accounts only for the horizontal interslice forces while more refined methods, such as Spencer's solution (Figure 10-21d), account for both vertical and horizontal interslice forces acting on each side of the slice.

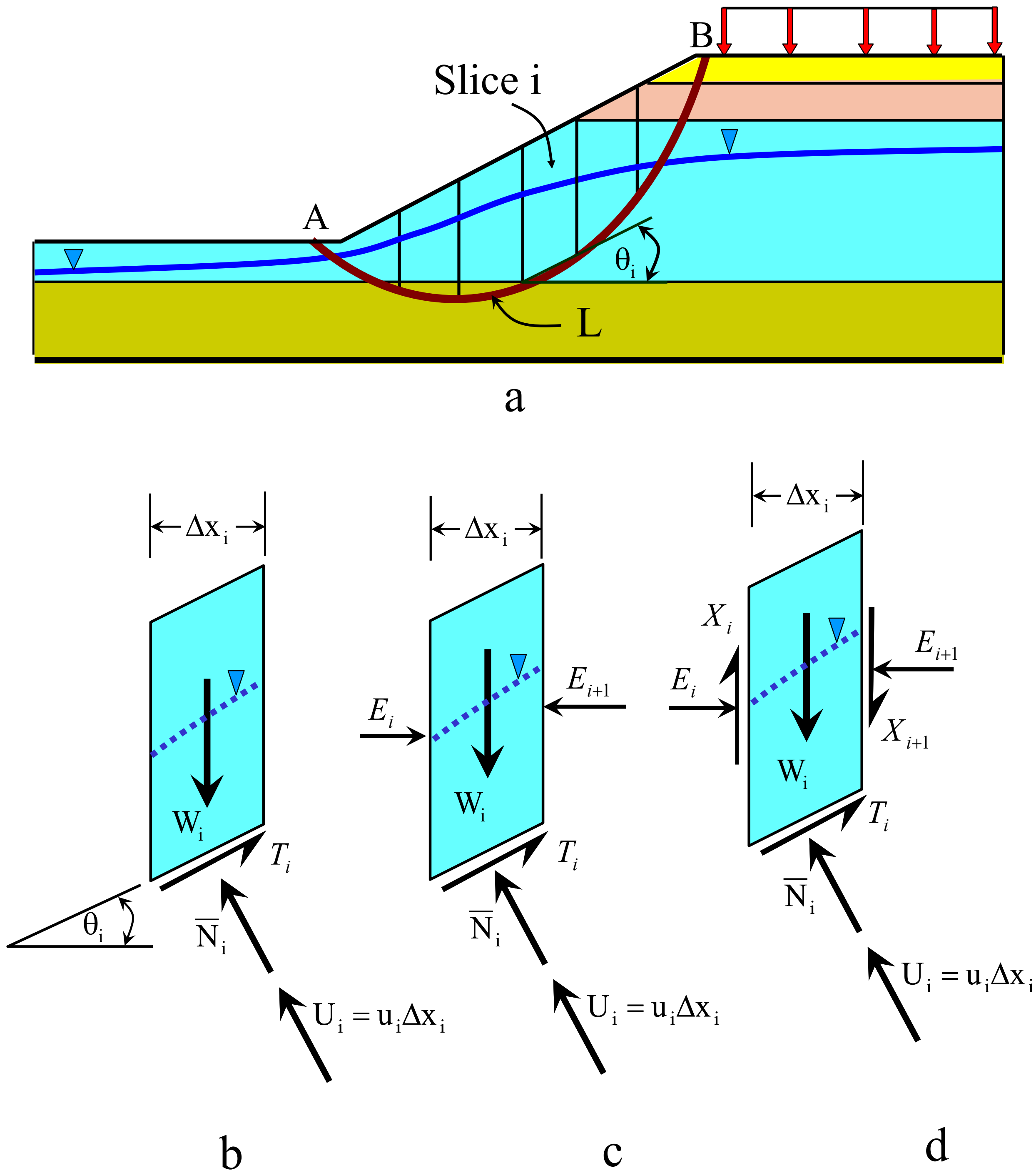


Figure 10-21. Method of Slices and Forces Acting on a Slice

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

- Fellenius Method of Slices (Figure 10-21b).
- Simplified Bishop Method of Slices (Figure 10-21c).
- Spencer and Janbu Method of Slices (Figure 10-21d).

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 10-22.

The basic equation for each of these methods is:

$$\mathbf{FS} = \frac{\bar{\mathbf{C}}\mathbf{L} + \tan\bar{\boldsymbol{\phi}} \sum_{i=1}^{i=n} \bar{\mathbf{N}}_i}{\sum_{i=1}^{i=n} \mathbf{W}_i \sin\boldsymbol{\theta}_i} \qquad (10\text{-}5\text{-}1)$$

Nomenclature

FS = Factor of safety

$\mathbf{FS_a}$ = Assumed factor of safety

i = Represents the current slice

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

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

$\mathbf{W_i}$ = Weight of the slice

$\bar{\mathbf{N}}_i$ = Effective normal force

$\boldsymbol{\theta}_i$ = Angle from the horizontal of a tangent at the center of the slice along the slip surface.

$\mathbf{T_i}$ = Shear force at base of slice.

$\mathbf{u_i}$ = Pore-water pressure force on a slice

$\mathbf{U_i}$ = Resultant neutral (pore-water pressure) force

$\Delta \mathbf{l_i}$ = Length of the failure arc cut by the slice. Note that as the slices get smaller, the values of $\Delta \mathbf{x_i}$ and $\Delta \mathbf{l_i}$ converge.

$\Delta \mathbf{x_i}$ = the width of a slice

L = Length of the entire failure arc

Note: The angle, **θ**, (measured from the horizontal) is shown in Figure 10-21 and is equal to the angle that is measured from the vertical in Figure 10-22. In Figure 10-21, the shear force at the base of the slice (**T**i) acts at this angle, and this angle is measured from the horizontal.

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

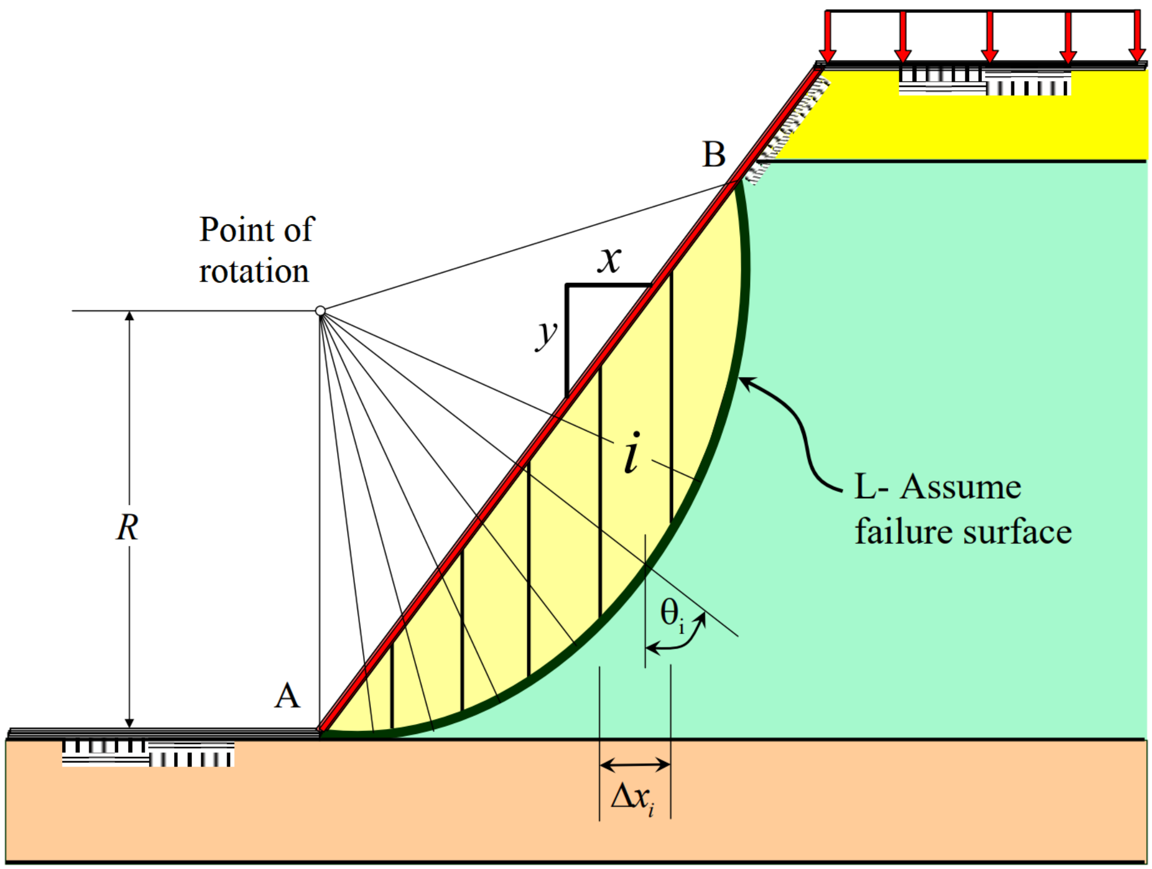


Figure 10-22. A Trial Surface and Potential Slices for Fellenius, Simplified Bishop, and Spencer and Janbu, Methods of Slices

10-5.02 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 is conservative but is widely used in practice because of its early origins and simplicity.

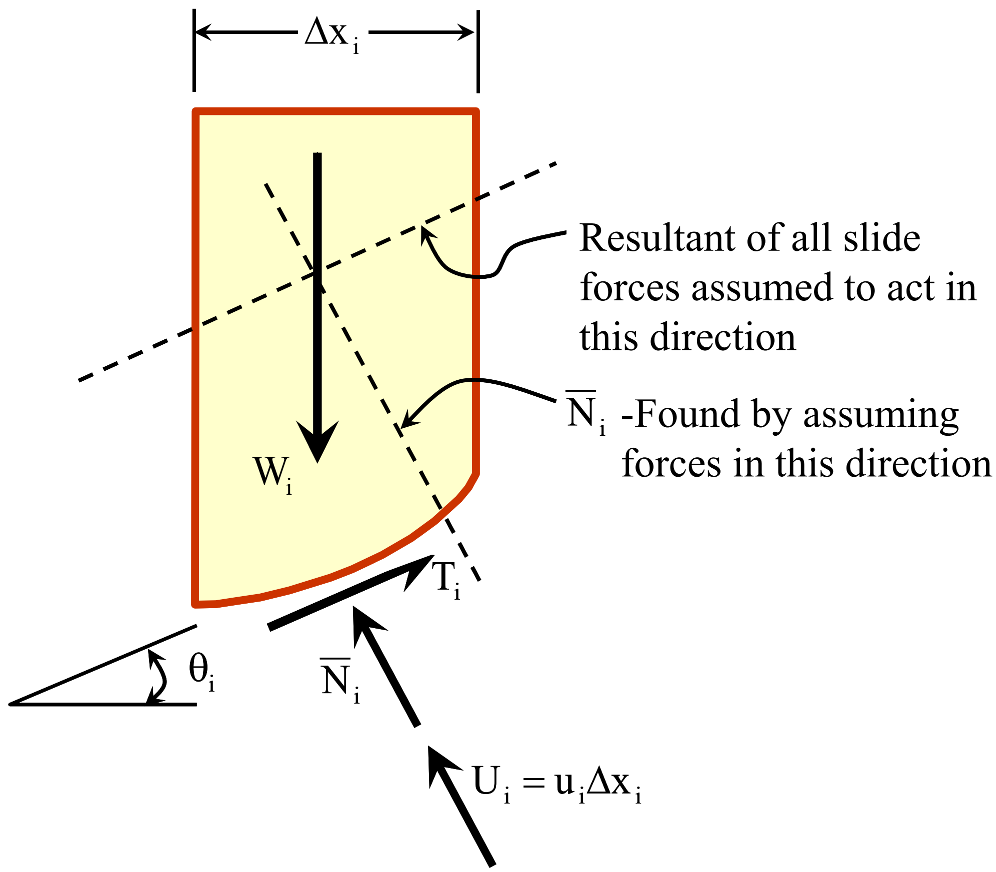


Figure 10-23. Slice i, Fellenius Method

$$\bar{N}_i = W_i \cos\theta_i - u_i \Delta l_i \qquad (10\text{-}5\text{-}2)$$

Recall that for small slices,

$$\Delta x_i \approx \Delta l_i \qquad (10\text{-}5\text{-}3)$$

Thus, the basic equation becomes:

$$FS = \frac{\bar{C}L + \tan\bar{\phi}\sum_{i=1}^{i=n}(W_i \cos\theta_i - u_i \Delta x_i)}{\sum_{i=1}^{i=n} W_i \sin\theta_i} \qquad (10\text{-}5\text{-}4)$$

The procedure is to investigate many possible failure planes, with different centers and radii, to identify the possible failure arc with the lowest factor of safety. Since this may take hundreds of iterations and is thus ideally solved with specialized software, Structure Construction staff should contact the DES Geotechnical unit for assistance.

10-5.02A Example 10-3 Problem – Fellenius Method

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

Solution:

The trial failure mass is divided into 6 slices with equal width as shown in Figure 10-24. Each slice makes an angle θ with respect to horizontal as shown; note relationship between angles from vertical and horizonal.

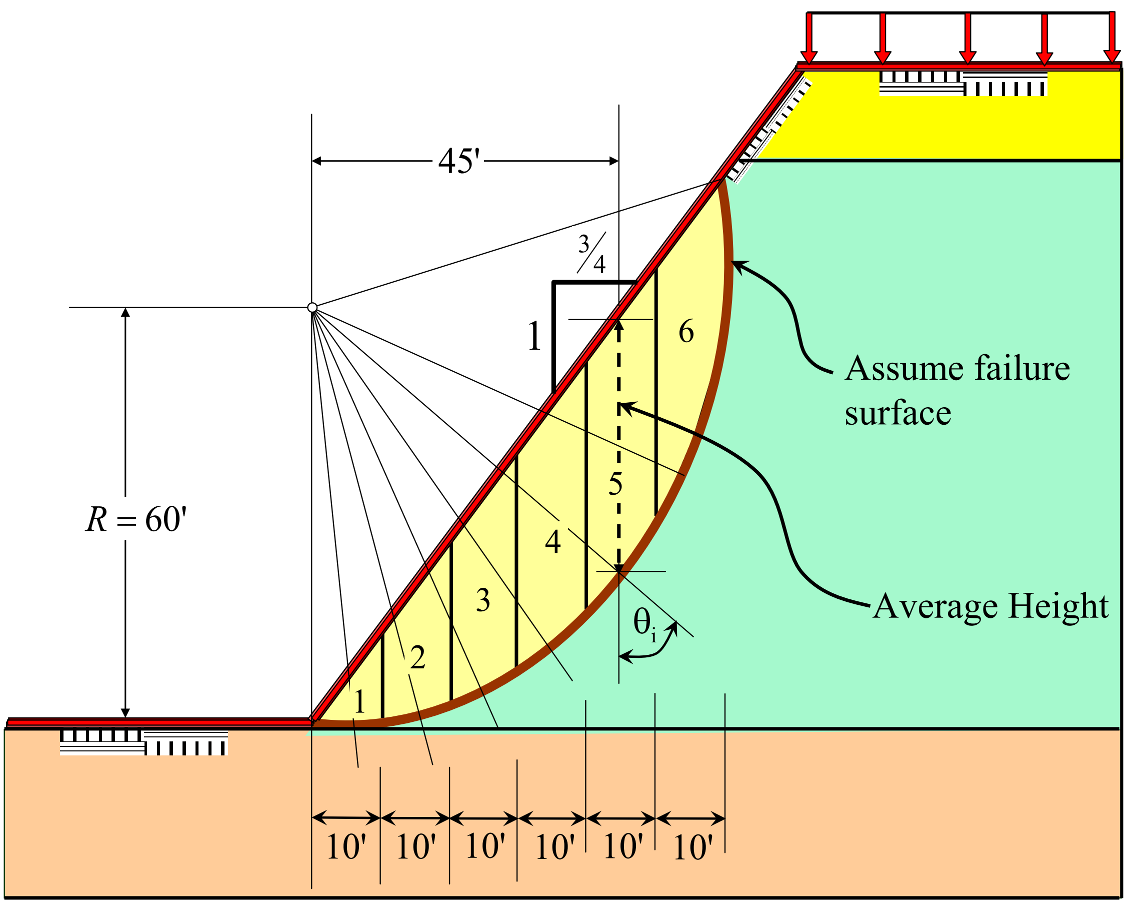


Figure 10-24. Example of Fellenius and Bishop Method of Slices

Table 10-1. Fellenius Table of Slices, Part 1

Angles θ_i (°)	Average Height (ft)	Slice Weights (kips/ft)
$\theta_1 = \sin^{-1}(5/60) = 4.78°$	6.46	$W_1 = (6.46)(10)(0.115) = 7.43$
$\theta_2 = \sin^{-1}(15/60) = 14.48°$	18.09	$W_2 = (18.09)(10)(0.115) = 20.81$
$\theta_3 = \sin^{-1}(25/60) = 24.62°$	27.88	$W_3 = (27.88)(10)(0.115) = 32.06$
$\theta_4 = \sin^{-1}(35/60) = 35.69°$	35.40	$W_4 = (35.40)(10)(0.115) = 40.71$
$\theta_5 = \sin^{-1}(45/60) = 48.59°$	39.69	$W_5 = (39.69)(10)(0.115) = 45.64$
$\theta_6 = \sin^{-1}(55/60) = 66.44°$	37.31	$W_6 = (37.31)(10)(0.115) = 42.91$

Table 10-2. Fellenius Table of Slices, Part 2

Slice	θ_i (°)	W_i (kips/ft)	$W_i \sin\theta_i$ (kips/ft)	$W_i \cos\theta_i$ (kips/ft)	$\bar{N}_i$ (kips/ft)
1	4.78	7.43	0.62	7.40	7.40
2	14.48	20.81	5.20	20.15	20.15
3	24.62	32.06	13.36	29.14	29.14
4	35.69	40.71	23.75	33.07	33.07
5	48.59	45.64	34.23	30.19	30.19
6	66.44	42.91	39.33	17.15	17.15
			Σ = 116.49		Σ = 137.09

L = 113.55 ft (by geometry)

$$FS = \frac{(0.2)\,(113.55) + (0.577)\,(137.09)}{116.49} = 0.87 < 1 \qquad (10\text{-}5\text{-}5)$$

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.

10-5.03 Bishop Method

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

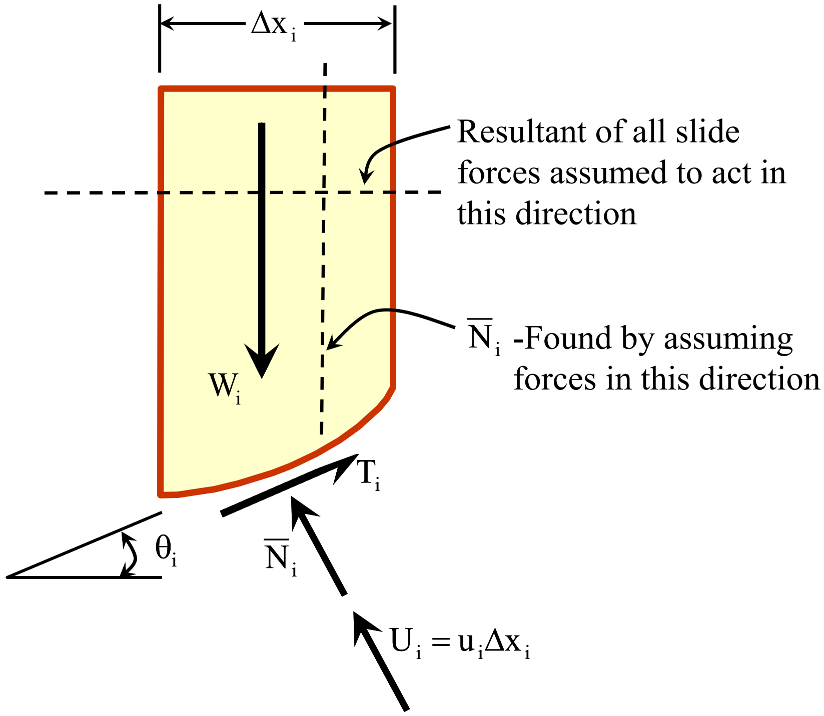


Figure 10-25. Slice i, Bishop Method

$$\bar{N}_i = \frac{W_i - u_i\Delta x_i - \frac{\bar{C}\Delta x_i \tan\,\theta_i}{FS_a}}{\cos\,\theta_i\left\{1 + \frac{\tan\,\theta_i \tan\,\bar{\phi}}{FS_a}\right\}} \quad (10\text{-}5\text{-}6)$$

The basic equation becomes:

$$FS = \frac{\sum_{i=n}^{i=n}\left(\frac{\bar{C}\Delta x_i + (W_i - u_i\Delta x_i)\tan\bar{\phi}}{M_i}\right)}{\sum_{i=n}^{i=n} W_i \sin\theta_i} \quad (10\text{-}5\text{-}7)$$

Where:

$$M_i = \cos\theta_i\left\{1 + \frac{\tan\theta_i \tan\bar{\phi}}{FS_a}\right\} \quad (10\text{-}5\text{-}8)$$

For the Bishop Method, the factor of safety must be assumed (FS_a) and trial-and-error iterations are required to determine the solution. The assumed FS_a converge on the factor of safety for that trial failure plane. Close agreement between the assumed FS_a and the calculated FS indicate that the selection of the center and radius is near the target value.

10-5.03A Example 10-4 Problem – Bishop Method

Given: γ = 115 pcf ϕ = 30° θ = 200 psf No Groundwater

Solution:

Table 10-3. Bishop Table of Slices, Part 1

Column	A	B	C	D	E	F	G
Slice	θ_I (deg)	W_i (k/ft)	$\bar{C}\,\Delta x_i$ (k/ft)	$W_i \tan\bar{\phi}$ (k/ft)	$\cos\theta_i$	$\tan\theta_i \tan\bar{\phi}$	C + D (kips/ft)
1	4.78	7.43	2	4.29	1.00	0.05	6.29
2	14.48	20.81	2	12.01	0.97	0.15	14.01
3	24.62	32.06	2	18.51	0.91	0.26	20.51
4	35.69	40.71	2	23.50	0.81	0.41	25.50
5	48.59	45.64	2	26.35	0.66	0.65	28.35
6	66.44	42.91	2	24.77	0.40	1.32	26.77

Table 10-4. Bishop Table of Slices, Part 2

Column	Ha	Hb	Ia	Ib	J
Slice	M_i FS_a = 1.5	M_i FS_a = 0.8	G/Ha (kips/ft) FS_a = 1.5	G/Hb (kips/ft) FS_a = 0.8	$W_i \sin\theta_I$ (kips/ft)
1	1.03	1.06	6.11	5.93	0.62
2	1.06	1.15	13.21	12.18	5.20
3	1.07	1.21	19.17	16.95	13.36
4	1.04	1.23	24.52	20.72	23.75
5	0.95	1.20	29.84	23.63	34.23
6	0.75	1.06	35.69	25.25	39.33
			Σ = 128.54	Σ = 104.66	Σ = 116.49

For FS_a = 1.5:

$$FS = \frac{128.54}{116.49} = 1.10 \qquad (10\text{-}5\text{-}9)$$

For FS_a = 0.8:

$$FS = \frac{104.66}{116.49} = 0.90 \qquad (10\text{-}5\text{-}10)$$

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 calculated based on field measured water levels.

10-5.04 Translational Slide

For excavations with soil layers dipping toward the excavation, or when 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 10-26. This surface would be assumed along the interface of the upper sliding soil and the weaker soil below it.

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 must be used to analyze slopes with a specific plane of weakness. Note that for Figure 10-26, the soil layers depicted are at an incline that would also need to be accounted for. A down sloping angle in the direction of the larger active mass would contribute to the forces that the passive block would need to resist.

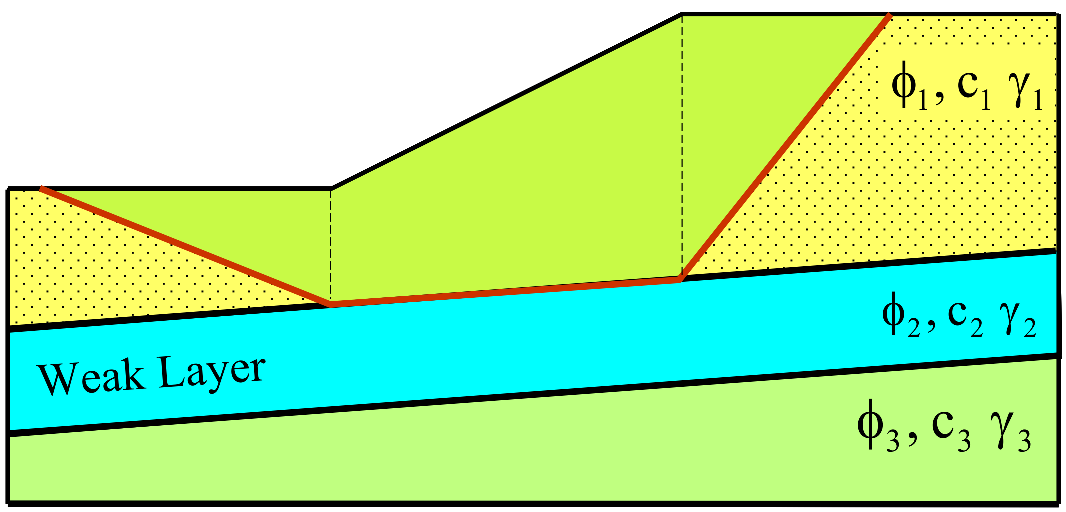


Figure 10-26. Mechanism of Translational Slide

Figure 10-27 also depicts a stratified soil consisting of three layers and a potential sliding mass. The force equilibrium of the blocks or wedges is more sensitive to shear forces than moment equilibrium as shown in Figure 10-27. 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.

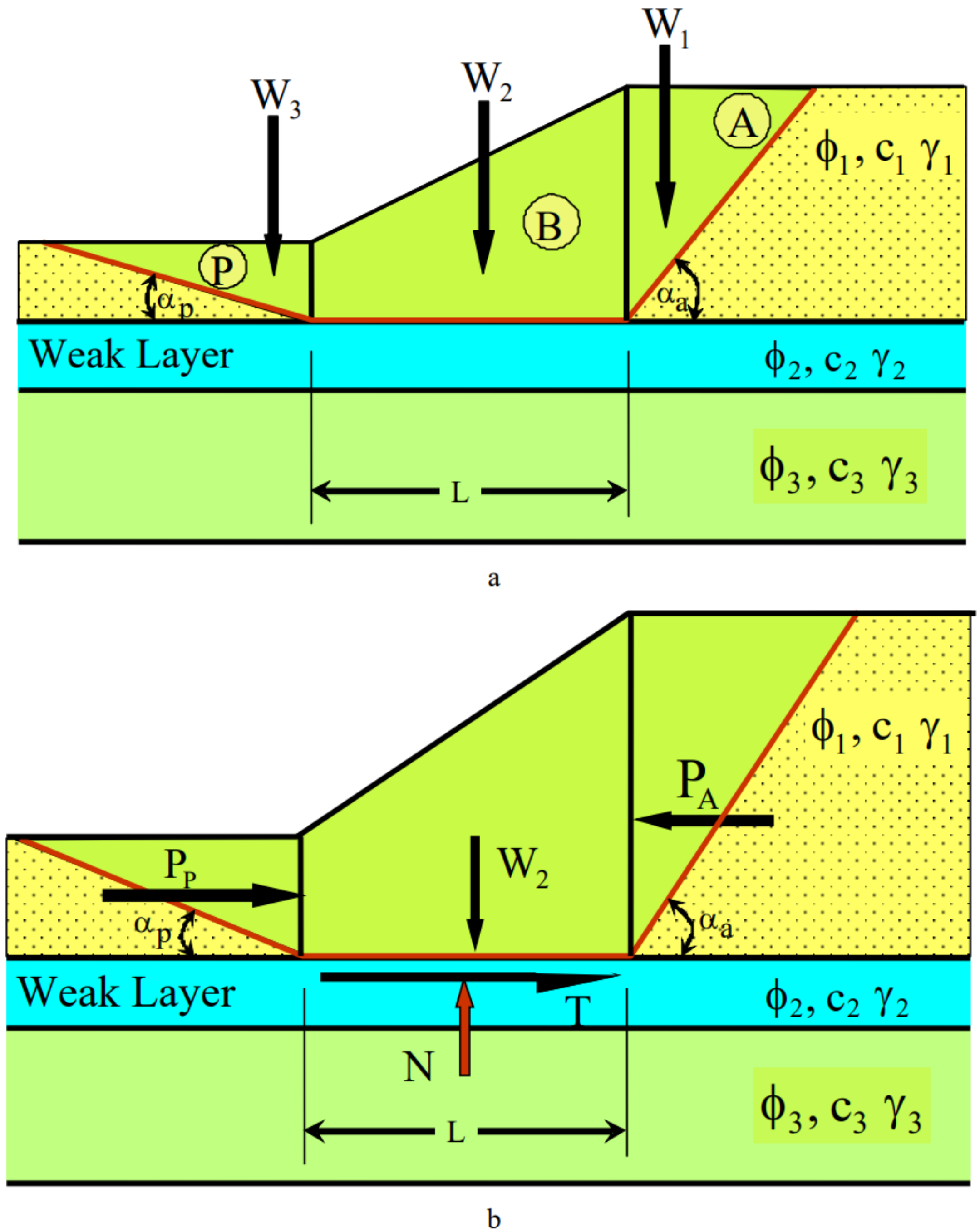


Figure 10-27. Mechanism of Translational Slide

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:

$$\mathbf{FS} = \frac{\mathbf{T} + \mathbf{P_p}}{\mathbf{P_a}} \quad (10\text{-}5\text{-}11)$$

In which:

$$\mathbf{T} = \mathbf{c_2} \times \mathbf{L} + \mathbf{W_2} \times \tan \boldsymbol{\phi_2} \quad (10\text{-}5\text{-}12)$$

Where:

T	=	tangential resistance force at the base of Block B
$\mathbf{c_2}$	=	unit cohesion along base of the Block B
L	=	length of base of Block B
$\mathbf{P_a}$	=	resultant active pressure on Block B $= W_1 \tan(\alpha_a - \phi_1)$
$\mathbf{P_p}$	=	passive pressure on Block B $= W_3 \tan(\alpha_P + \phi_1)$
$\mathbf{W_1}$	=	weight of section of Block A
$\mathbf{W_2}$	=	weight of section of Block B
$\mathbf{W_3}$	=	weight of section of Block P
$\boldsymbol{\alpha_a}$	=	failure plane angle with horizontal for active pressure
$\boldsymbol{\alpha_p}$	=	failure plane angle with horizontal for passive pressure
$\boldsymbol{\phi_1}$	=	internal friction angle of soil for Block A
$\boldsymbol{\phi_2}$	=	internal friction angle of weaker underlying soil
FS	=	factor of safety

Additional notes and observations for the translational slide:

1. Block B is the middle block, and it is the key element. A free body diagram can be drawn for Block B.
2. The bottom of Block B is horizontal in Figure 10-27. If the bottom of block B has a slope downward to the left, then the factor of safety would be reduced.
3. There is only one driving force that is acting on Block B, and that driving force is the resultant Rankine active earth pressure force on the right side of Block B.
4. At the bottom of Block B, there is a resisting force from the cohesion of the weak layer (due to the cohesion value for the weak layer), and there is a resisting force from the friction of the weak layer (due to the phi (ϕ) angle for the weak layer).
5. On the left side of Block B, there is a resisting force from the resultant Rankine passive earth pressure.
6. The factor of safety is equal to the sum of the resisting forces divided by the sum of the driving forces.

10-5.04A Example 10-5 Problem – Translational Slide

Calculate the factor of safety for a translational slide for a given failure surface, as shown below in Figure 10-28.

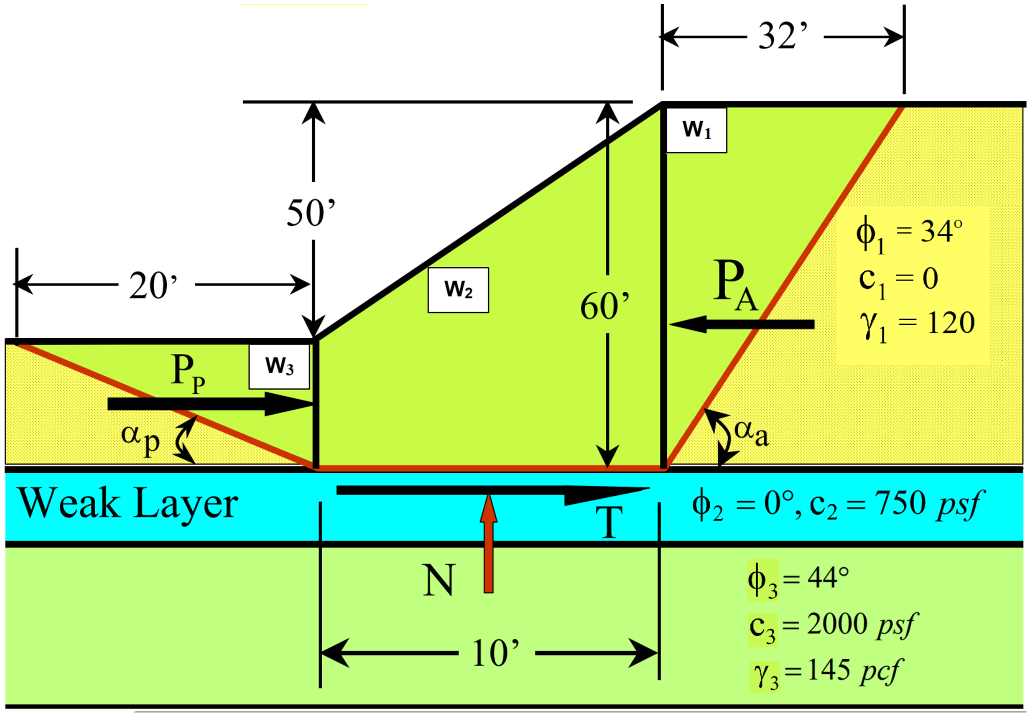


Figure 10-28. Example of a Translational Slide

Solution:

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

Note that these values for α_a and α_p are close to the values one would obtain when following the more rigorous method outlined in Section 4-3.02, *Active and/or Passive Earth Pressure*, of this manual.

Calculating the weight of the soil blocks:

$$W_1 = \frac{(32')(60')}{2}\left(\frac{120}{1000}\right) = 115.2 \text{ kip/ft} \qquad (10\text{-}5\text{-}13)$$

$$W_2 = \frac{(10' + 60')(10')}{2}\left(\frac{120}{1000}\right) = 42.0 \text{ kip/ft} \qquad (10\text{-}5\text{-}14)$$

$$W_3 = \frac{(20')(10')}{2}\left(\frac{120}{1000}\right) = 12.0 \text{ kip/ft} \qquad (10\text{-}5\text{-}15)$$

The tangential resistance force at the bottom of the block with weight **W2**, due to the cohesion of the weak layer is calculated as:

$$T = W_2 Tan(\phi_2) + c_2 L = (42)\left(Tan(0°)\right) + \frac{(750 \text{ psf})(10')}{1000 \text{ lb/k}} = 7.5 \text{ kip/ft}$$

(10-5-16)

Calculate the resultant active and passive forces acting on the soil block with weight **W2**:

$$P_a = (115.2)(Tan(62° - 34°)) \approx 61.3 \text{ kip/ft} \quad (10\text{-}5\text{-}17)$$

$$P_p = (12.0)(Tan(26.6° + 34°)) \approx 21.3 \text{ kip/ft} \quad (10\text{-}5\text{-}18)$$

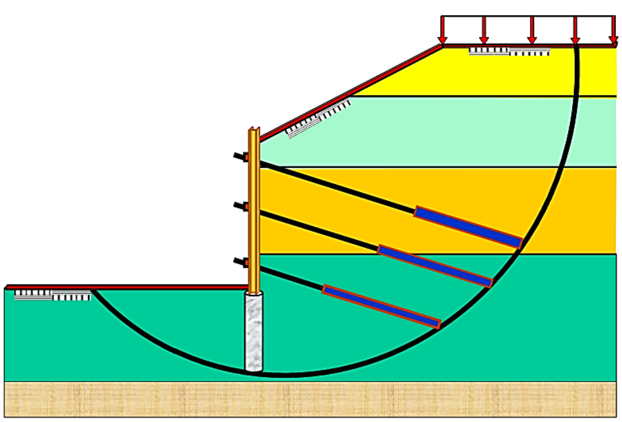
$$FS = \frac{7.5 + 21.3}{61.3} = 0.47 \quad (10\text{-}5\text{-}19)$$

Note that if the length of **W2** was increased to 100 feet, the FS ≈ 1.57.

Depending on the reliability of the soil properties, the duration, the level of risk desired, as well as other considerations, a factor of safety of 1.5 may be appropriate. A discussion with DES Geotechnical is always appropriate.

10-5.05 Stability Analysis of Shoring Systems

Deep-seated stability failure should be investigated for major shoring systems such as ground anchor walls. The slip surface passes behind the anchors and underneath the base tip of the vertical structural members as shown in Figure 10-29. 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 ground anchor systems as shown in Figure 10-29**.** The trial surface must extend to the depth of the excavation to calculate the minimum factor of safety of 1.25. The un-bonded length must extend beyond the failure surface.


a. Deep Seated

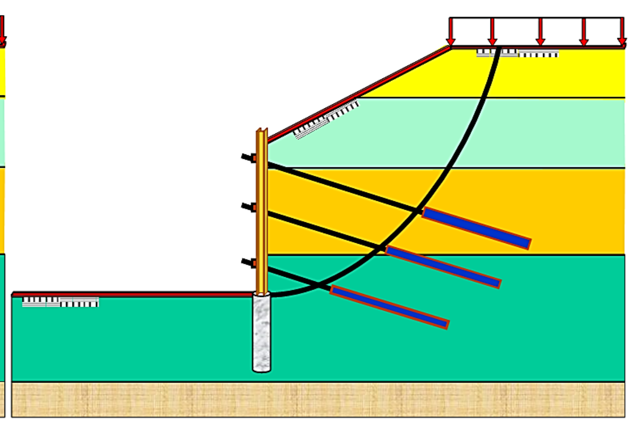

b. Local System

Figure 10-29. Stability Failure Modes