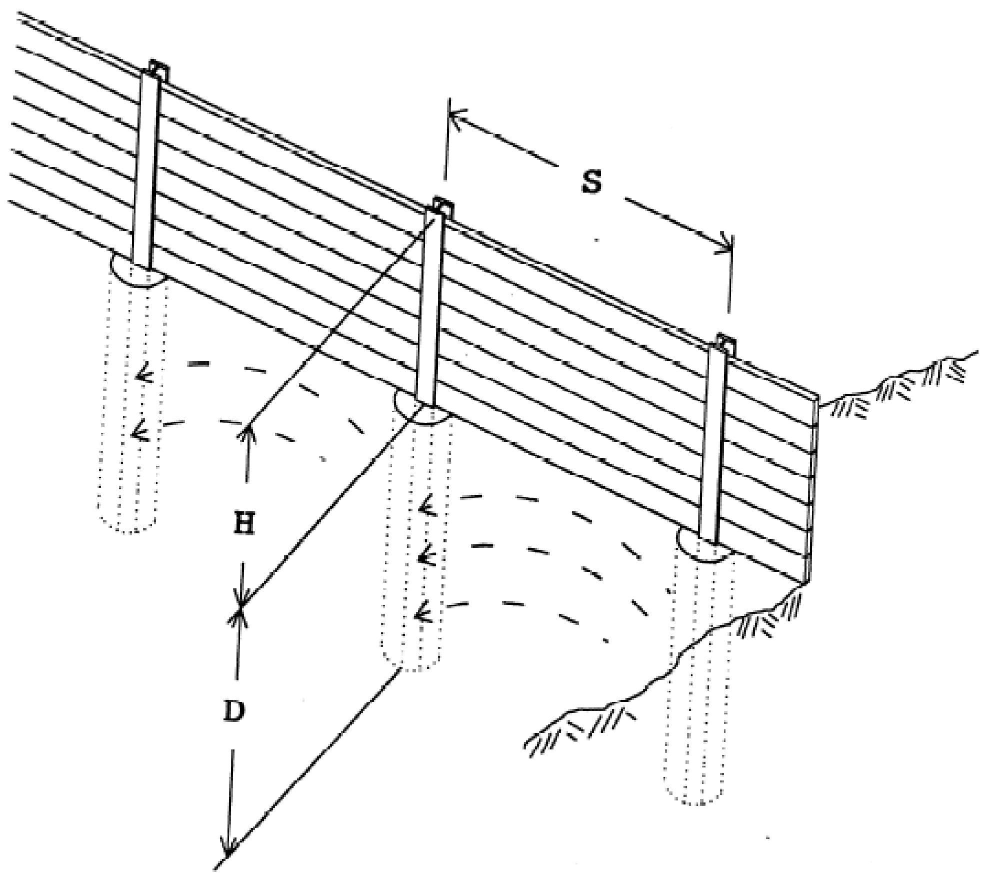
# CHAPTER 7

# UNRESTRAINED SHORING SYSTEMS



# Chapter 7: Unrestrained Shoring Systems

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# 7-1 Types of Unrestrained Shoring Systems

Unrestrained shoring systems or non-gravity cantilevered walls (hereafter simply cantilevered) are constructed of vertical structural members consisting of partially embedded continuous sheet piles or soldier piles. Continuous sheet pile retaining walls may be constructed with steel sheet piles with interlocking edges, driven precast pre-stressed concrete sheet piles, or other materials. The sheet piles are driven side by side into the ground and form a continuous vertical wall. Because of the large deflections that may develop, cantilever sheet pile retaining walls are mainly used for temporary excavations not greater than about 18 feet. However, the use of struts and/or walers can increase the wall height, making it a restrained system; see Chapter 8, *Restrained Shoring Systems,* of this manual. Figure 7-1 shows a typical cantilever sheet pile retaining wall for permanent work with a concrete cap.

**Figure 7-1. Sheet Pile Wall with Cap Beam**

Soldier pile retaining walls may be constructed with driven piles (steel, treated timber, or precast concrete), or the piles may be placed in drilled holes and backfilled with concrete, slurry, sand, pea-gravel, or similar approved material. A soldier pile may also be a cast-in-place reinforced concrete pile. Lagging is placed between soldier pile vertical elements and could be treated timber, reinforced shotcrete, reinforced cast-in-place concrete, precast concrete panels, or steel plates. This type of wall depends on passive resistance of the foundation material and the moment resisting capacity of the vertical structural members for stability. The maximum height is limited to competence of the foundation material and the moment resisting capacity of the vertical structural members. Figure 7-2 shows a typical soldier pile retaining wall with timber lagging.


**Figure 7-2. Soldier Pile Wall with Cap Beam**

The general design procedure for soldier pile walls is to assume one half the pile spacing on either side of the pile acts as a panel loaded with active soil pressures and surcharge loading above the depth of the excavation. The portion of the soldier pile below the depth of the excavation is likewise loaded with both active and passive soil pressures and surcharge loading. Resistance to the lateral movement or overturning (about any point) of the soldier pile is furnished by the passive resistance of the soil below the excavation on both sides of the soldier pile.

# 7-2 Effective Pile Width

The effective width (**d**) of a soldier pile is generally considered to be the dimension of the soldier pile taken parallel to the line of the wall for driven piles or drilled piles backfilled with material other than concrete such as gravel. The effective width of the soldier piles may be taken as the diameter of the drilled hole when 4-sack or better concrete is used below the excavation line. Soil arching, however, can greatly increase the effective width described above as shown in Figure 7-3 below. Arching of the soil between soldier piles can increase the effective width of a soldier pile up to 3 times for granular soil and 2 times for cohesive soils.

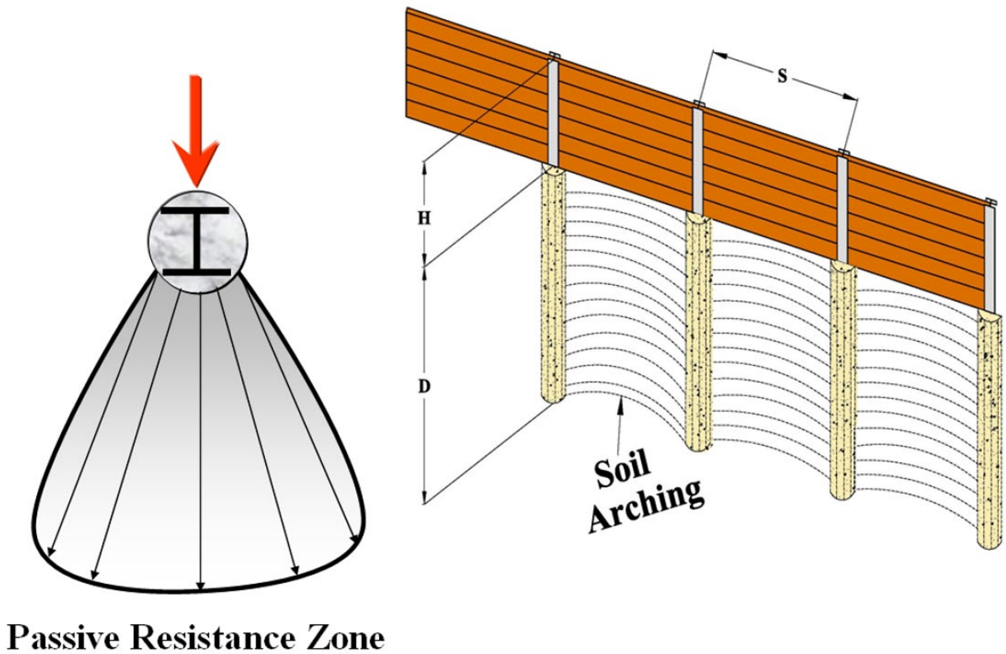



**Figure 7-3. Soldier Pile with Soil Arching Below Excavation**

Numerous full-scale pile experiments have shown the passive resistance in front of an isolated pile is a three-dimensional model for the sake of analysis. Therefore, the passive resistance in front of a pile must be adjusted due to the effect of soil arching beyond the effective pile width.

The soil arching factor for granular soils is a function of the soil friction angle (ϕ) as shown below.

$$\text{Arching Capability Factor, } \mathbf{f} = 0.08 \times \emptyset \leq 3 \qquad \text{(7-2-1)}$$

The Adjusted Pile Width is limited to actual pile spacing.

$$\text{Adjusted Pile Width} = (\text{Effective Width} \times \mathbf{f}) \leq \text{Pile Spacing} \qquad \text{(7-2-2)}$$

The arching capability for cohesive soil ranges between 1 and 2 as shown in Figure 7-4.

| CONSISTENCY | VERY SOFT | SOFT | MEDIUM | STIFF | VERY STIFF | HARD |
|---|---|---|---|---|---|---|
| $q_u$ = unconfined comp. strength (PSF) | 500 | 1000 | 2000 | 4000 | 8000 | |
| Unit Weight (PCF) Saturated | 100-120 | | 110-130 | 120-140 | | 130+ |
| Arching Capability | 1 to 2 | 1 to 2 | 2 | 2 | 2 | |

VERY SOFT: Exudes from fingers when squeezed in hand.
SOFT: Molded by light finger pressure.
MEDIUM: Molded by strong finger pressure.
STIFF: Indent by thumb.
VERY STIFF: Indent by thumb nail.
HARD: Difficult to indent by thumb nail.

**Figure 7-4. Arching Capability for Cohesive Soil**

Below the excavation depth, the adjusted pile width is used for the passive resistance in front of the pile. Any active loadings (including surcharge loadings) on the back of the pile below the excavation depth use only the effective width of the pile, effectively fixing the arching factor at 1. Again, the adjusted pile width cannot exceed the pile spacing.

# 7-3 System Deflection

The point of fixity, or the point of zero (0) deflection, of the "shoring support" of a cantilevered system is a significant assumption made for the analysis of a shoring system. The point of fixity, below the excavation line as shown in Figure 7-5 below, will affect the pile embedment, shears, moments, and deflection. The point of fixity is defined as a percentage of the embedment depth, **D,** which varies from 0 to 0.75D. For unrestrained shoring systems in most stiff to medium dense soils, a value of 0.25D may be assumed and this value is used as the default value in the Structure Construction (SC) Trenching & Shoring Check Program. A greater value may be used for loose sand or soft clay.

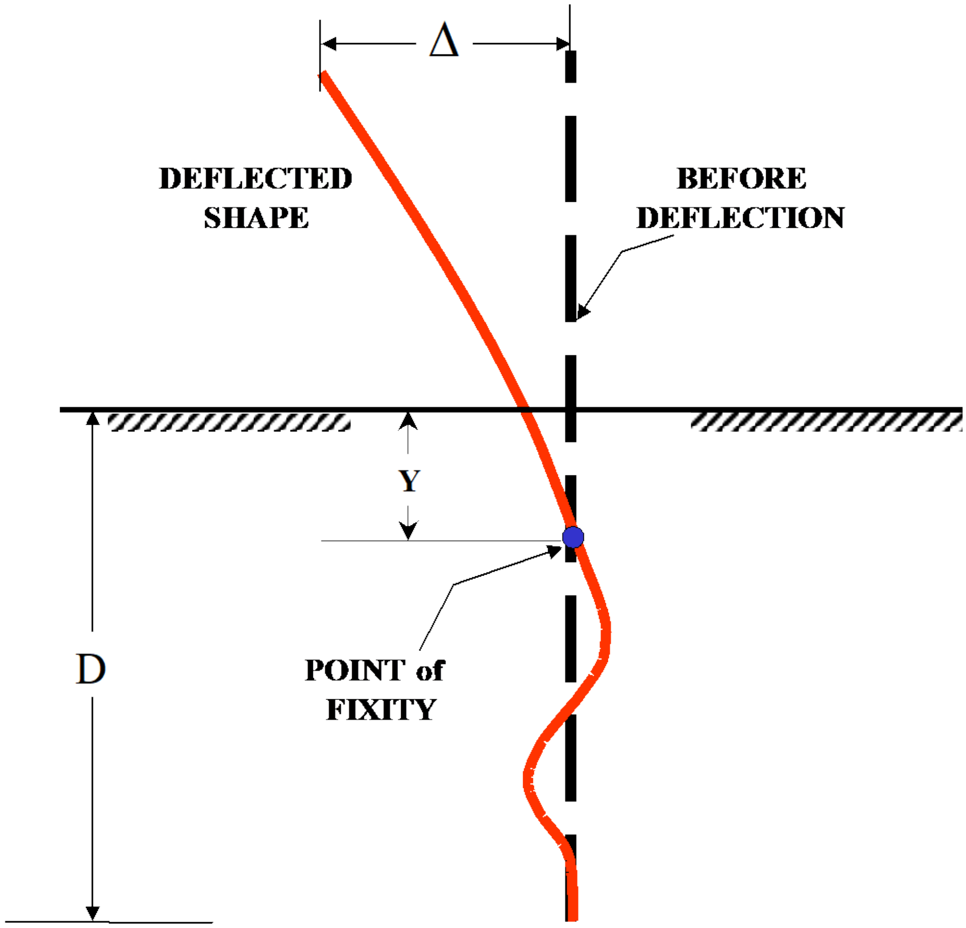



**Figure 7-5. Deflected Shape for Unrestrained System**

Calculating deflections of temporary shoring systems can be complicated. The total deflection of a shoring system is a combination of the deflection of the structural supporting member due to bending, and the movement of the entire system (usually thought of as rotation) within the embedded portion of the system. Deflection calculations are required for any shoring system adjacent to the railroad or high-risk structures. Generally, the taller a shoring system becomes, the more likely it is to be subject to large lateral deflections. The amount of allowable deflection or movement is inversely proportional to the sensitivity to movement of what is being shored. Thus, it will be up to the Engineer's judgment as to what degree of analysis will be performed. Bear in mind that except for the railroad as discussed in Chapter 9, *Railroads*, of this manual, there are no guidelines on the maximum allowable lateral deflection of the shoring system. For other high-risk structures, allowable deflections are on a case-by-case basis.

Calculations to approximate shoring deflection are normally performed per standard beam analysis methods. This approximation does not account for other factors that may be contributory, such as rotation of a cantilevered shoring or long-term movement of the ground anchors for a restrained system. The deflection can either be determined from double integration of the moment diagram or by multiplying the area under the moment diagram times its moment arm beginning from the top of the pile to a depth '**D**' below the dredge line. Although these methods described above are for standard beam analysis, it should be pointed out that shoring systems do not necessarily act as standard beams with point supports. Instead, for calculating a realistic deflection for a shoring system a Soil-Structure Interaction (SSI) analysis using a **p-y** approach or a finite element method, must be performed. The SSI method of analysis is beyond the scope of this manual and the Engineer is encouraged to contact the SC Falsework Engineer in SC HQ in Sacramento. An example problem showing deflection calculations can be found in Appendix B, *Example Problems*.

# 7-4 Soil Pressure Distribution for Layered Soil

As discussed in Chapter 4, *Earth Pressure Theory and Application*, the horizontal pressure exhibited is a function of the soil unit weight, the depth, and the earth pressure coefficient. Thus, in a uniform soil the pressure generally grows with depth. When there are layered soils in the system, the pressure diagram can develop discontinuities at the divisions between soil layers due to the property changes. As depicted in Figure 7-6 below, these interface pressure changes are represented by using sigma, **σ**, with a "**+**" or a "**-**" to represent the pressure based on the upper soil properties and lower soil properties, respectively. Thus, for a shoring system in layered soils, it is very important to be mindful of these nuances while developing the soil pressure distribution for each individual soil layer as shown in Figure 7-6.

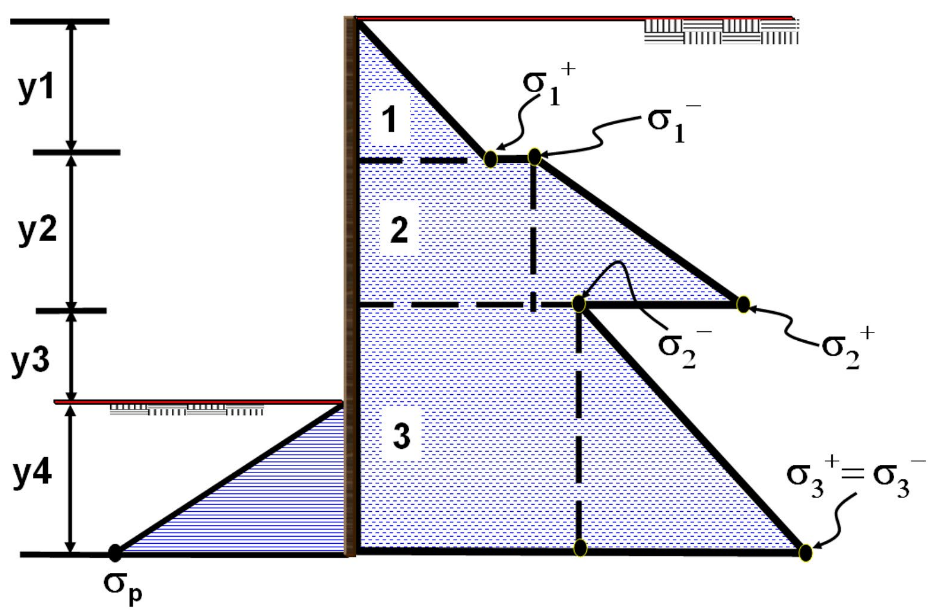



**Figure 7-6. Pressure Diagram of Multilayer Soil Pressure**

The vertical pressure of the soil in layer 1 is the unit weight of the soil multiplied by the depth. The horizontal pressure against the shoring at this depth is dependent on the $\mathbf{K_a}$

of the soil. Thus, sigma, σ, will have different values at the depth of **y1** due to the unique soil properties of layer 1 and layer 2. Figure 7-6 above, depicts layer 1 as having a smaller value of $K_a$ and layer 2 having a larger $K_a$. This manual uses the nomenclature of $\sigma^+$ to represent the pressure on the shoring "just above" the layer change and $\sigma^-$ to represent the pressure on the shoring "just below" the layer change.

# 7-5 Lateral Earth Pressures for Unrestrained Shoring Systems

Cantilever retaining walls are analyzed by assuming that the vertical structural member rotates at point O, at the distance $D_O$ below the excavation line, as illustrated in Figure 7-7. As a result, the mobilized active pressure develops above point O in the back of the wall and below point O in the front of the wall. The mobilized passive pressure develops in front of the wall above point O and at the back of the wall below point O. The realistic load distribution is shown in Figure 7-8b, and the idealized pressure distribution is shown in Figure 7-8c.

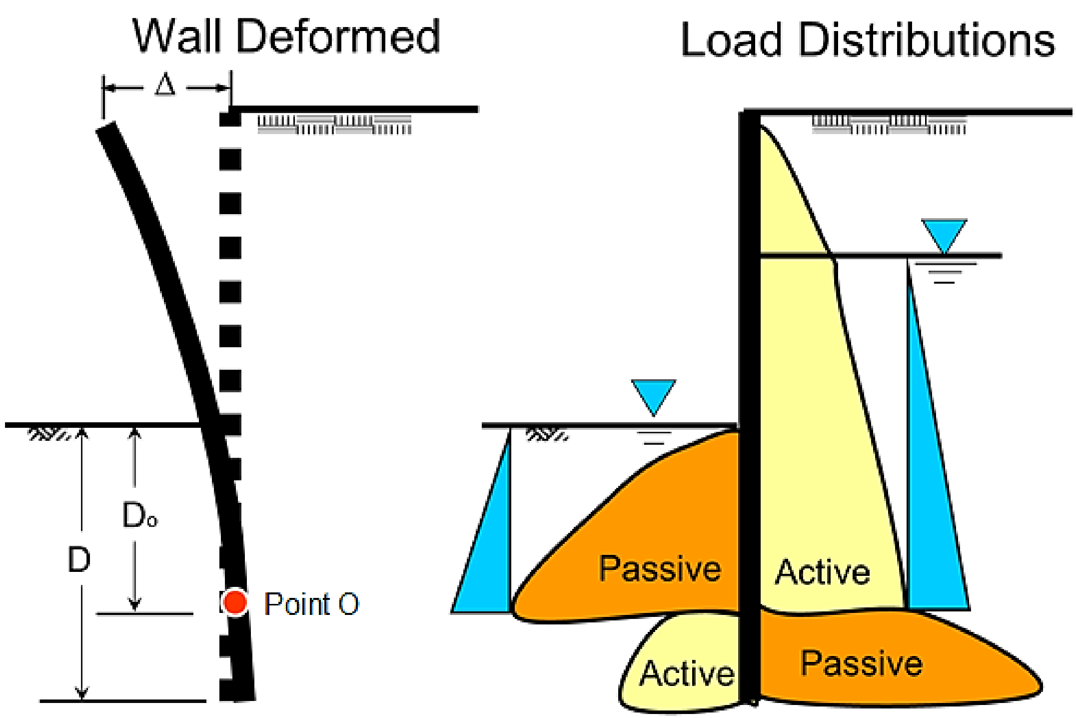



**Figure 7-7. Non-Gravity Cantilever Retaining Walls Loading Diagram**

There are at least two acceptable methods for analyzing the soil pressures for the unrestrained systems; the Rigorous Method and the Simplified Method. Examples of these methods are detailed below. Both of these methods solve for the embedment depth of the system to be stable by summarizing the moments these soil pressures exert to "Drive" the system over into the excavation, and offsetting the soil pressures "Resisting" these moments from the opposite direction. In the examples below, these will be referred to as "Driving Moments" and "Resisting Moments" within the shoring system.

## 7-5.01 The Rigorous Method

For the Rigorous Method, the idealized load distribution is shown in Figure 7-8. The load distribution is a combination of active and passive pressure, and it extends below point O down to the tip of pile.

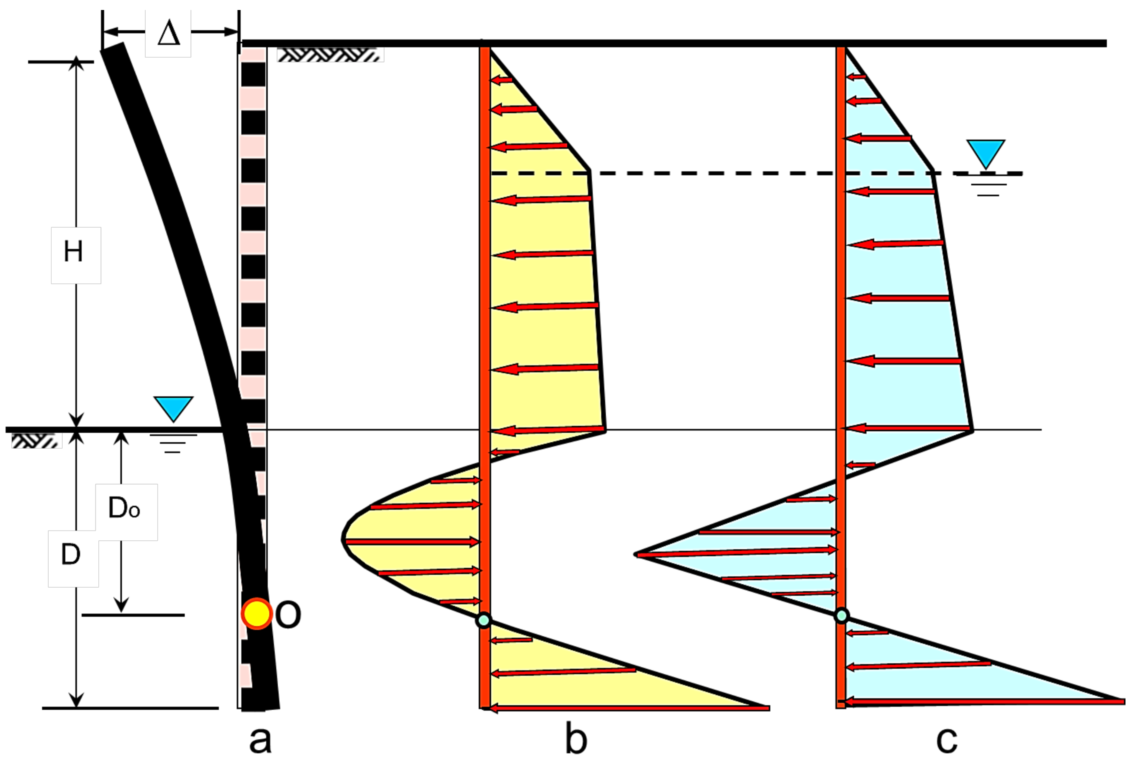



**Figure 7-8. Cantilever Sheet Pile Walls Idealized Loading Diagram**

Figure 7-9 shows the shear and moment diagrams for a continuous sheet pile wall with an idealized pressure distribution for the Rigorous Method. Shoring utilizing sheet piles will have a different pressure distribution below the excavation (see Figure 7-9) than a soldier pile (see Figure 7-10) due to pile spacing and effective pile width. Use this method to solve for the embedment depth (**D**), maximum shear, maximum moment, and deflection.

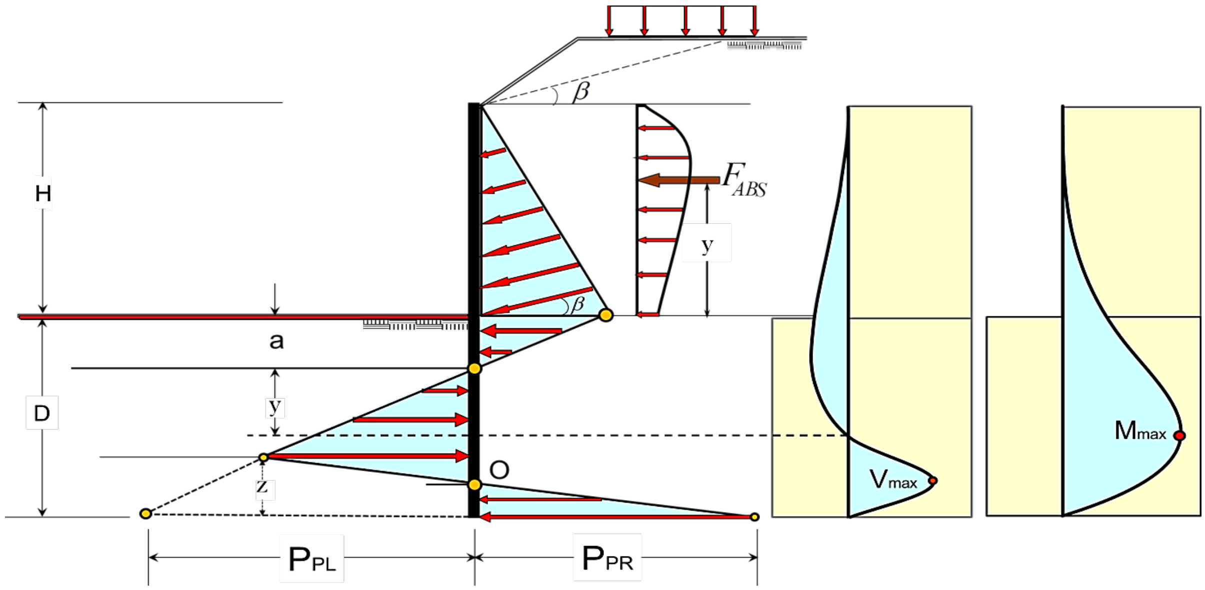



**Figure 7-9. Sheet Pile Walls Idealized Pressure Distribution and Shear and Moment Diagram**

For the sheet pile system shown above, the loading analysis is simpler as the wall loadings can be calculated on a per-foot basis. The figure below for a soldier pile system must account for the pile spacing as a tributary area loaded on the retained side, and the pile spacing plus soil arching, on the resistive side. This is what produces the discontinuity (step down) in the pressure diagram visible in the figure. This difference also applies to using the Simplified Method, which is discussed next.

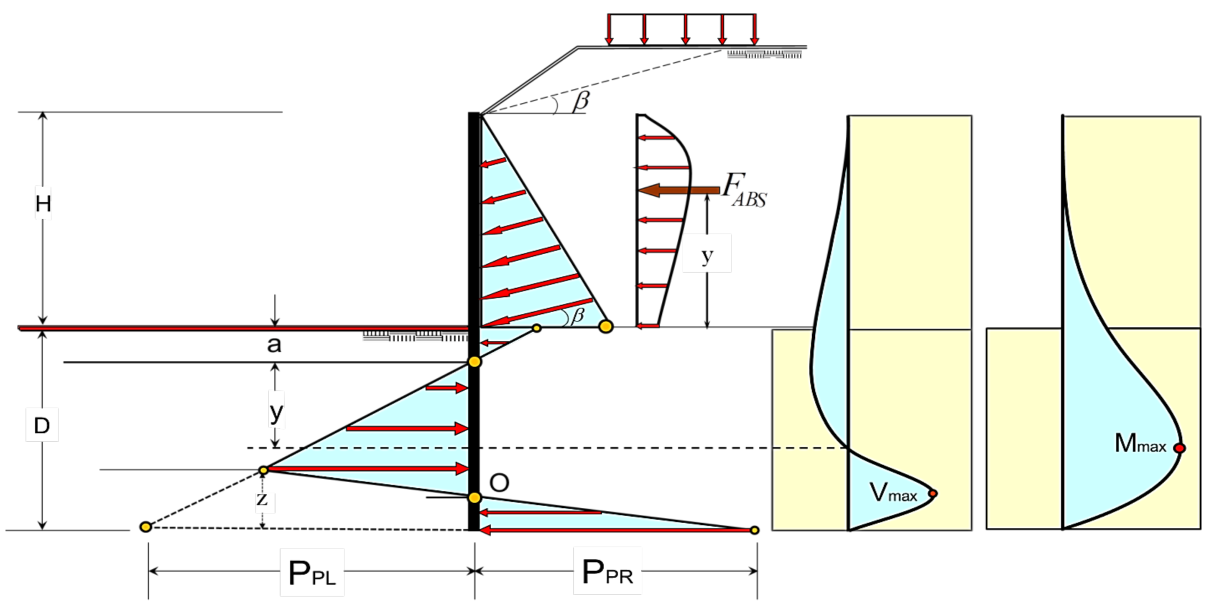
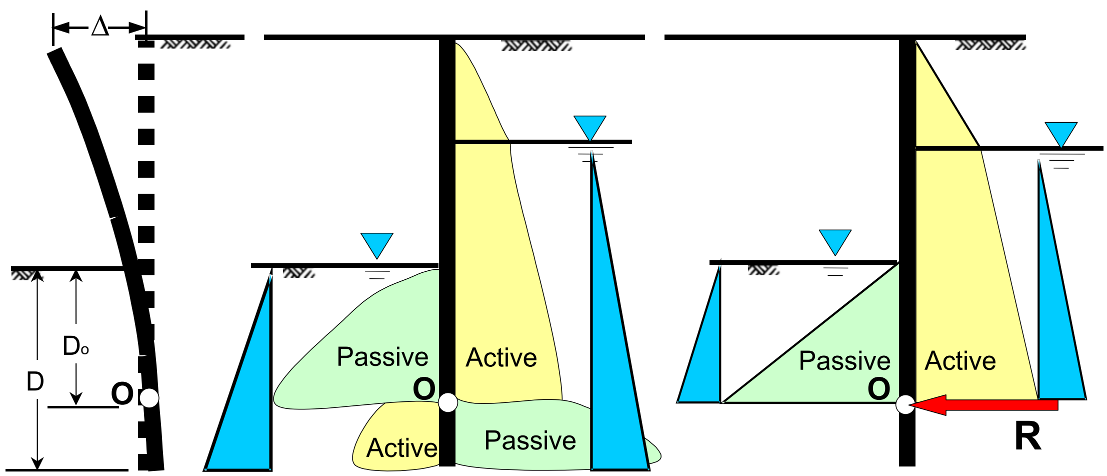



**Figure 7-10. Soldier Pile Idealized Pressure Distribution and Shear and Moment Diagram**

## 7-5.02 The Simplified Method

Load distributions for typical cantilever earth retaining systems using the simplified procedure are shown in Figure 7-11. Cantilever retaining walls are analyzed by assuming that the vertical structural member rotates at point O, at the distance **$D_O$**, below the excavation line as shown in Figure 7-11(a). The realistic load distribution is shown in (b). As a result, the mobilized active pressure develops above point O in the back of the wall and below point O in the front of the wall. The mobilized passive pressure develops in front of the wall above point O, and at the back of the wall below point O.

The simplified load distribution is shown in Figure 7-11(c). Force **R** is assumed at point O to compensate the resultant net active and passive pressure below point of rotation at point O. The calculated depth, **D**, is determined by increasing **$D_O$** by 20 percent to approximate the total embedment depth of the vertical wall element. The 20 percent increase is not a factor of safety. It accounts for the rotation of the length of vertical wall element below point O as shown in Figure 7-11(a). (**D** = **$1.2D_O$**, AASHTO 3.11.5.6).




(a)- Wall Deformed (b) - Load (c) - Simplified Load Simplification

**Figure 7-11. Cantilever Sheet Pile Walls Idealized Loading Diagram**

The following procedure is used for the check of a cantilever wall using the Simplified Method, as illustrated in (c) of Figure 7-11:

1. Calculate active/passive earth pressure to an arbitrary point, O, at the depth of **$D_o$** below the excavation line.
2. Take moments about point O to eliminate force **R** and determine embedment depth **$D_O$**. There will be driving moments tipping the system forward, and resisting moments working to keep the system upright. These become balanced at the depth **$D_O$** .

3. Increase $\mathbf{D_O}$ by 20 percent ($\mathbf{D}$ = 1.2 $D_O$).
4. Calculate **R** by summation of forces in horizontal direction (**R** ≤ 0; if **R** is larger than zero, increase **D**).
5. Calculate maximum bending moment ($\mathbf{M_{max}}$) and maximum shear force ($\mathbf{V_{max}}$) to check the vertical structural member.
6. Then check the lagging and any other members of the system (see Chapter 6, *Structural Design of Shoring Systems*).

## 7-5.03 Variations of Pressure Diagrams for Other Soil Configurations

Below are examples of various pressure distributions for analysis of unrestrained shoring systems with the Simplified Method, based on differing soil profiles.

For a shoring system with a single layer of granular soil, Figure 7-12 may be used to determine the lateral earth pressure distribution.

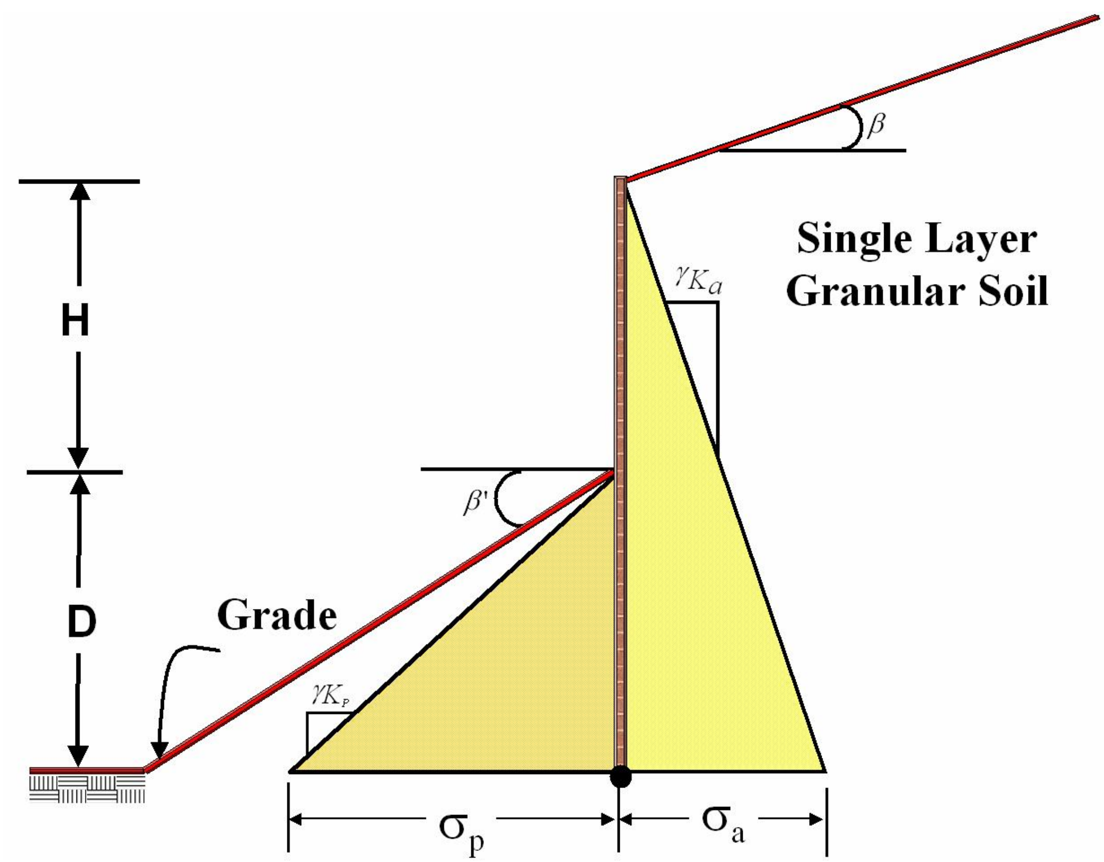



**Figure 7-12. Loading Diagram for Single Layer**

For a shoring system with a multi-layer granular soil, Figure 7-13 may be used to determine the lateral earth pressure distribution.

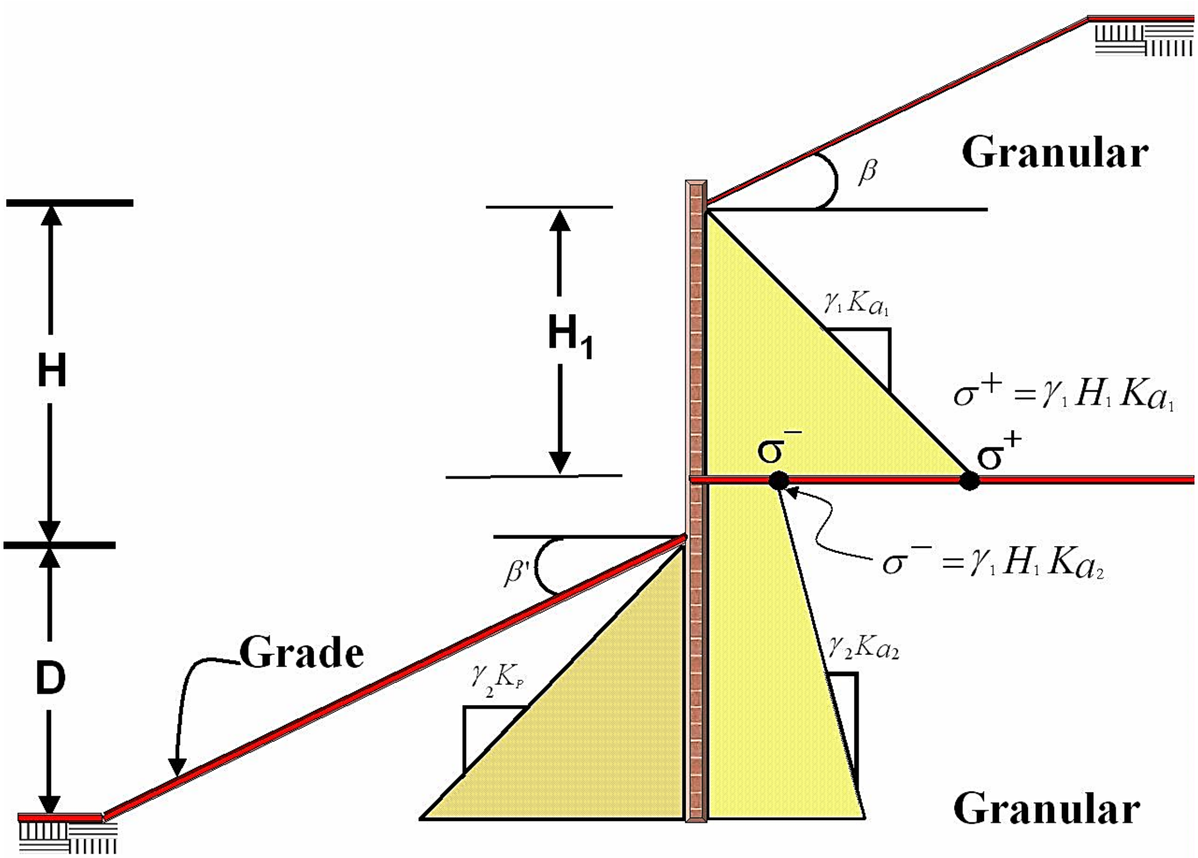



**Figure 7-13. Loading Diagram for Multi-Layer Granular Soil**

For a shoring system that is embedded in granular soils and retaining cohesive soils, Figure 7-14 may be used to determine the lateral earth pressure distribution.

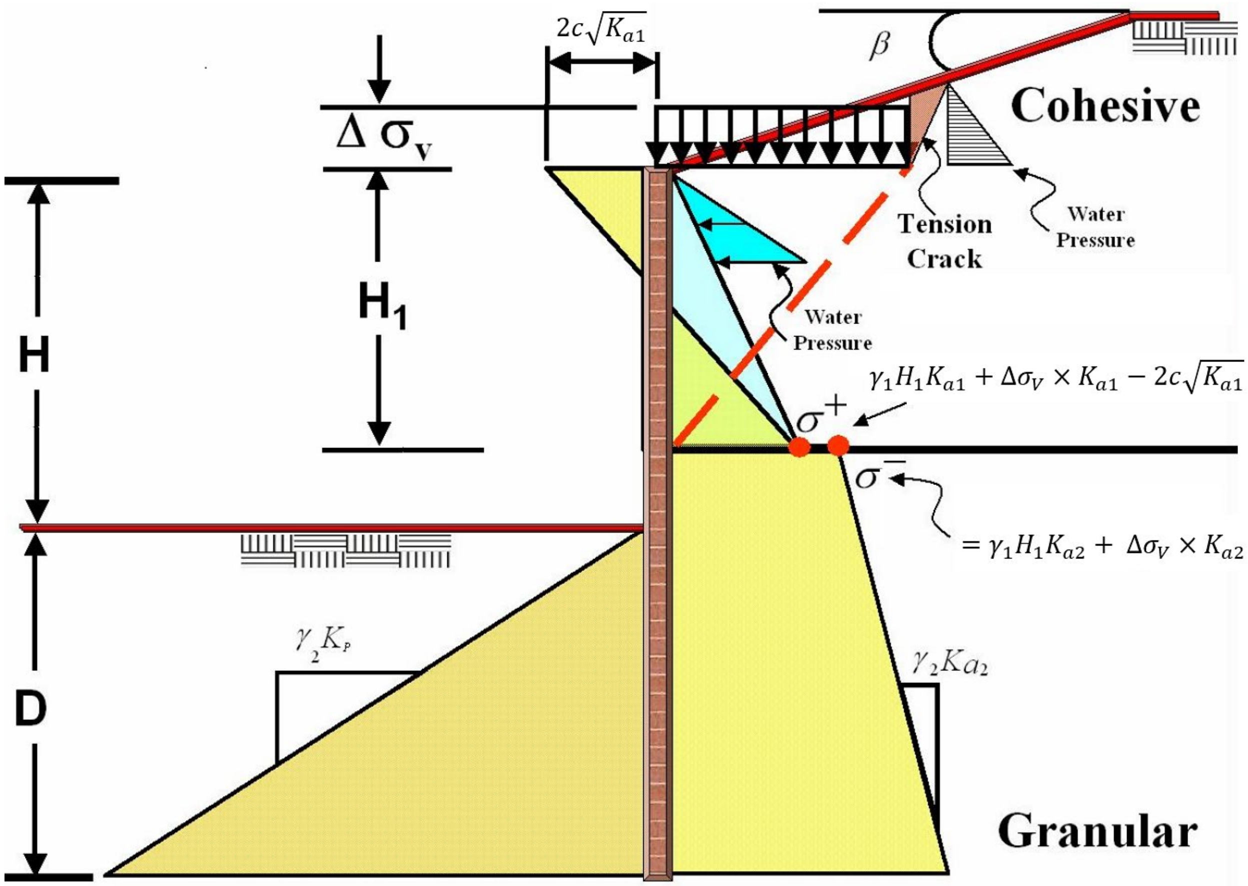



**Figure 7-14. Loading Diagram for Multi-Layer – Cohesive over Granular Soil**

For a shoring system that is embedded in (supported by) cohesive soils and retaining granular soil, Figure 7-15 may be used to determine the lateral earth pressure distribution.

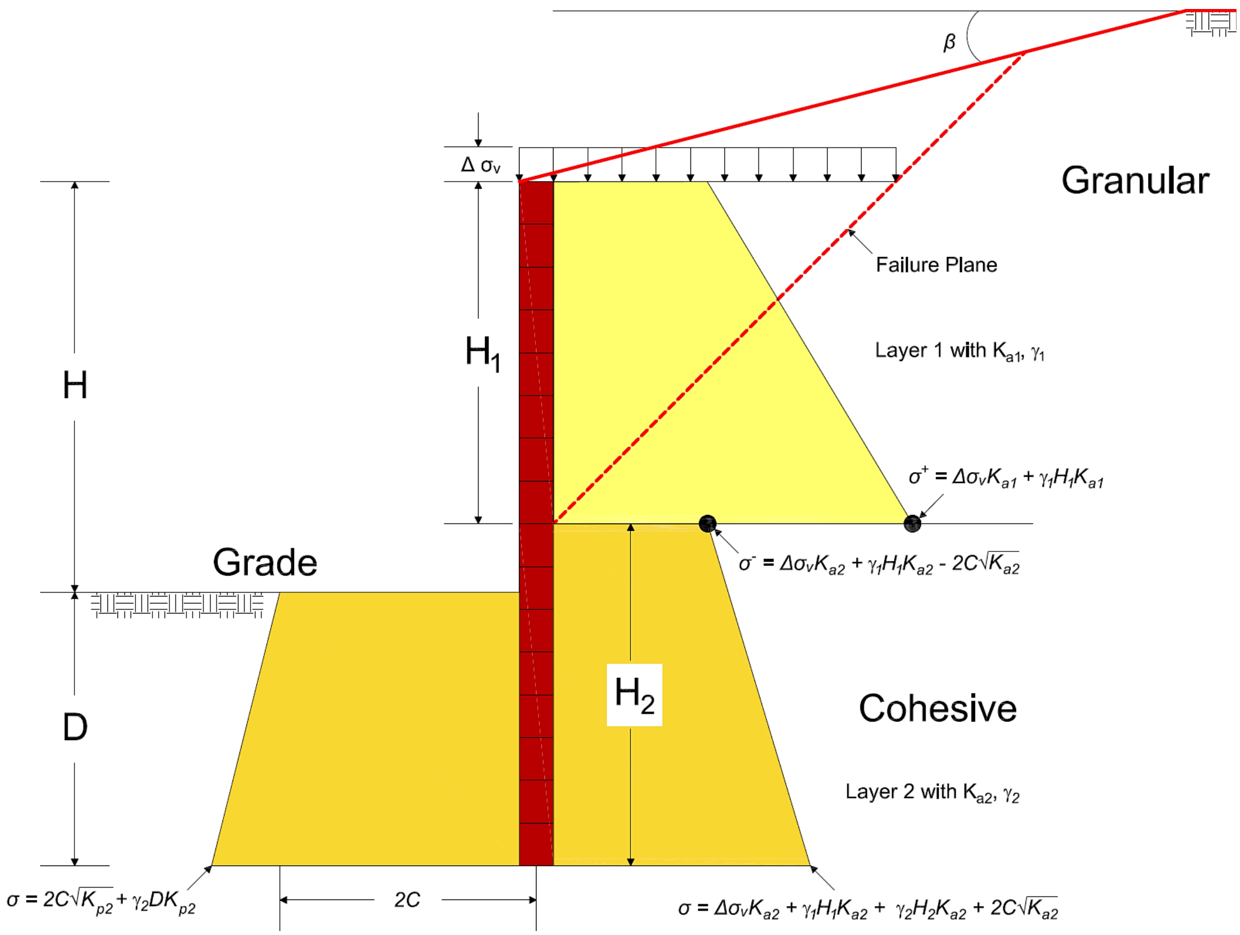



**Figure 7-15. Loading Diagram for Granular over Cohesive Soil**

For a shoring system with a multi-layer cohesive soil, Figure 7-16 may be used to determine the lateral earth pressure distribution.

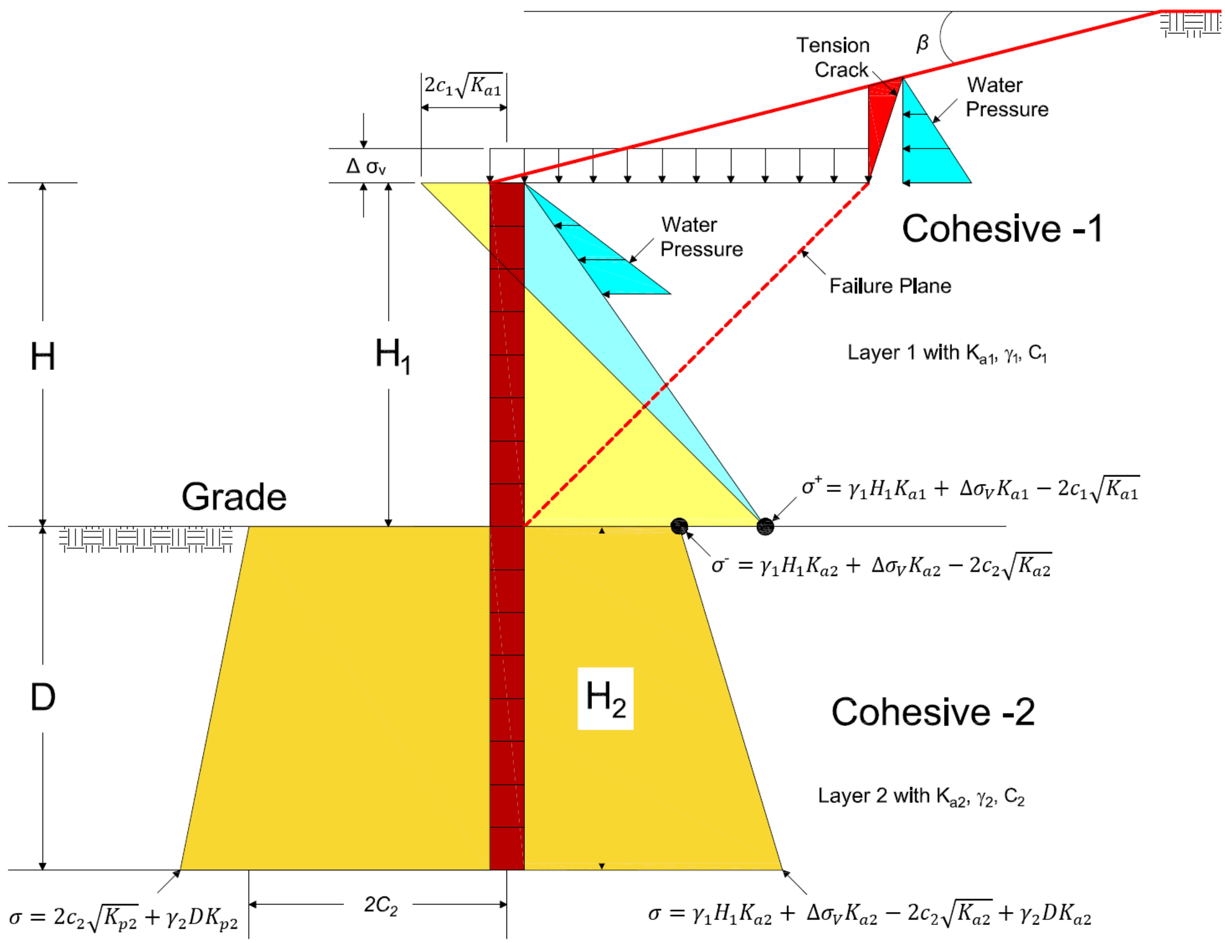



**Figure 7-16. Loading Diagram for Multi-Layer Cohesive Soil**

The active lateral earth pressure on the embedded wall element can be determined for Figure 7-12 through Figure 7-16 with these general steps:

1. Treat the sloping backfill above the top of the wall within the active failure wedge as an additional surcharge (**Δσv**).
2. The portion of the negative loading at the top of the wall due to cohesion is ignored.
3. Any hydrostatic pressure in the tension crack needs to be considered.

## 7-5.04 Example 7-1A: Cantilevered Soldier Pile Wall by Rigorous Method

Using the Rigorous Method perform a shoring check for a W14 x 120 cantilevered soldier-pile-lagging wall with piles placed at 8 feet on center, encased in 2-foot diameter holes, filled with 4-sack concrete. The pressure of 72 psf is the minimum lateral construction surcharge acting on the timber lagging that is caused by typical construction loading. The soil properties are shown below, in Figure 7-17.

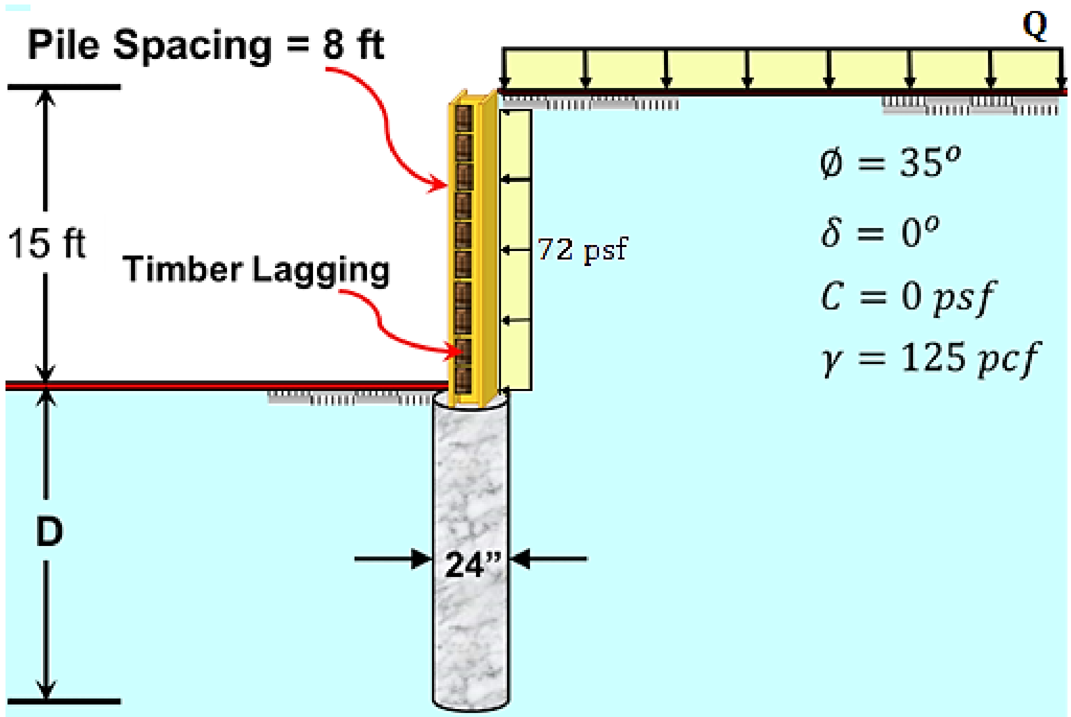



**Figure 7-17. Cantilevered Soldier-Pile-Lagging Wall Example Properties**

Solve for the following using the Rigorous Method:

1. Calculate active & passive earth pressures
2. Determine pile embedment, **D**
3. Calculate maximum shear & moment
4. Calculate service deformation
5. Calculate timber lagging deflection.

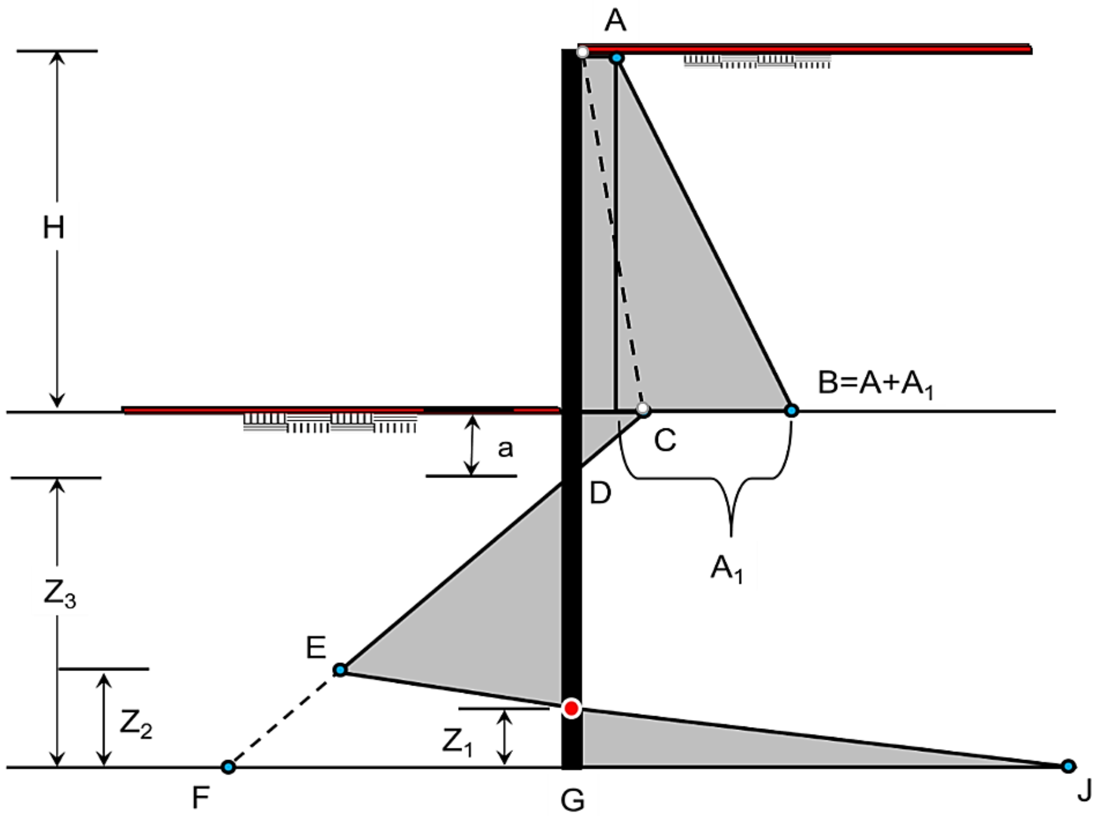



**Figure 7-18. Rigorous Assumed Loading Diagram**

**Determine Active and Passive Earth Pressures:**

Calculate active and passive earth pressure coefficients: since the wall friction (***δ***) is zero, use Rankine earth pressure theory to calculate the active and passive earth pressure coefficients, as outlined in Chapter 4.

$$K_a = \tan^2\left(45 - \frac{\emptyset}{2}\right) = \tan^2\left(45 - \frac{35}{2}\right) = 0.271 \quad (7\text{-}5\text{-}1)$$

$$K_p = \tan^2\left(45 + \frac{\emptyset}{2}\right) = \tan^2\left(45 + \frac{35}{2}\right) = 3.690 \quad (7\text{-}5\text{-}2)$$

Note: Rankine Theory tends to underestimate the passive earth pressure. It is recommended to use the Log-Spiral-Rankine Model to calculate the passive earth force.

Since this is a soldier pile system, soil arching for the passive resistance needs to be included. Calculate the arching factor, **f** :

$$f = 0.08\,\emptyset = (0.08 \times 35) = 2.8 \quad (7\text{-}5\text{-}3)$$

From the given information, lowercase "**a**" can be easily calculated and will be needed to find the pressures at each point. The calculations below use the slope of the pressure change with depth (slope of the line). The slope is based on the combination of the active and passive earth pressure coefficients and begins with the pressure at point C. The depth of "**a**" is at the point the earth pressure is equal to zero.

$$0 = \sigma(\text{at C}) - a[\text{slope of the line}] \quad (7\text{-}5\text{-}4)$$

$$(\text{slope of the line}) = \left[\gamma\left(K_p f - K_a\right)\right] \quad (7\text{-}5\text{-}5)$$

$$0 = \sigma(\text{at C}) - a\left[\gamma\left(K_p f - K_a\right)\right] \quad (7\text{-}5\text{-}6)$$

$$a = \frac{\sigma(\text{at C})}{\gamma(\text{pcf}) \times \left(K_p f - K_a\right)} \quad (7\text{-}5\text{-}7)$$

$$a = \frac{125 \text{ pcf} \times 15 \text{ ft} \times 0.271}{(125 \text{ pcf})((3.69 \times 2.8) - 0.271)} = \frac{508 \text{ psf}}{1258 \text{ pcf}} = 0.404 \text{ ft} \quad (7\text{-}5\text{-}8)$$

Note: In the above equation, **f** is the arching capability factor. This factor is applied to passive pressures below the excavation line for soldier pile systems. Calculate the earth pressure distribution in kips/ft at each node of the diagram. This implies multiplying each pressure to account for the soldier pile spacing at the various points in Figure 7-18.

- Point A - Lateral load due to minimum construction surcharge above the excavation line only (Note in this example the minimum construction surcharge is taken farther than 10 feet below the top of the excavation):

$$A = 0.072 \times 8 = 0.576 \text{ kips/ft} \quad (7\text{-}5\text{-}9)$$

- Point $A_1$ - Active lateral load at excavation level on the wall:

$$A_1 = 0.125 \times 15 \times 0.271 \times 8 = 4.065 \text{ kips/ft} \quad (7\text{-}5\text{-}10)$$

- Point C - Active lateral load at excavation level from top on the soldier pile:

$$C = 0.125 \times 15 \times 0.271 \times 2 = 1.01625 \text{ kips/ft} \quad (7\text{-}5\text{-}11)$$

- Point F - Passive lateral load in front of the dredge line at embedment depth:

$$F = (0.125 \times Z_3 \times ((3.69 \times 2.8) - 0.271) \times 2) = 2.51525 Z_3 \text{ kips/ft} \quad (7\text{-}5\text{-}12)$$

- Point J - Active lateral load distribution at embedment depth:

$$\mathrm{J} = \big(0.125 \times (\mathrm{Z_3} + 0.404) \times \big((3.69 \times 2.8) - 0.271\big) \times 2\big) + (0.125 \times 15 \times 3.69 \times 2.8 \times 2) = 2.51525\ \mathrm{Z_3} + 39.7612\ \text{ kips/ft} \quad (7\text{-}5\text{-}13)$$

Calculate resultant earth forces (**P**) and apply $\sum \mathbf{F} = \mathbf{0}$. The applied forces on the wall are the areas of the distributed loads, as illustrated in Figure 7-19.

1. Calculate active earth force due to surcharge full height of the wall **H**, **P<sub>sur</sub>**:

$$\mathrm{P_{sur}} = 0.576 \frac{\text{kips}}{\text{ft}} \times 15 \text{ ft} = 8.64 \text{ kips} \quad (7\text{-}5\text{-}14)$$

2. Calculate active earth force above dredge line, $\mathbf{P_1}$:

$$\mathrm{P_1} = \frac{1}{2} \times 4.065 \frac{\text{kips}}{\text{ft}} \times 15 \text{ ft} = 30.4875 \text{ kips} \quad (7\text{-}5\text{-}15)$$

3. Calculate active earth forces below dredge line, $\mathbf{P_2}$:

$$\mathrm{P_2} = \frac{1}{2} \times 1.01625 \frac{\text{kips}}{\text{ft}} \times 0.404 = 0.2053 \text{ kips} \quad (7\text{-}5\text{-}16)$$

4. Calculate passive earth forces below dredge line. For simplification, take (Area FEJ) – (Area FDG):

$$\text{Area FEJ} = \mathrm{P_3} = \frac{1}{2} \times \left(2.51525\ \mathrm{Z_3} \frac{\text{kips}}{\text{ft}} + \left(2.51525\ \mathrm{Z_3} + 39.7612\ \frac{\text{kips}}{\text{ft}}\right)\right) \times \mathrm{Z_2} = 2.51525\ \mathrm{Z_3 Z_2} + 19.881\ \mathrm{Z_2} \text{ kips} \quad (7\text{-}5\text{-}17)$$

$$\text{Area FDG} = \mathrm{P_4} = \frac{1}{2} \times 2.51525\ \mathrm{Z_3} \frac{\text{kips}}{\text{ft}} \times \mathrm{Z_3} \text{ ft} = 1.257625\ \mathrm{Z_3^2} \text{ kips} \quad (7\text{-}5\text{-}18)$$

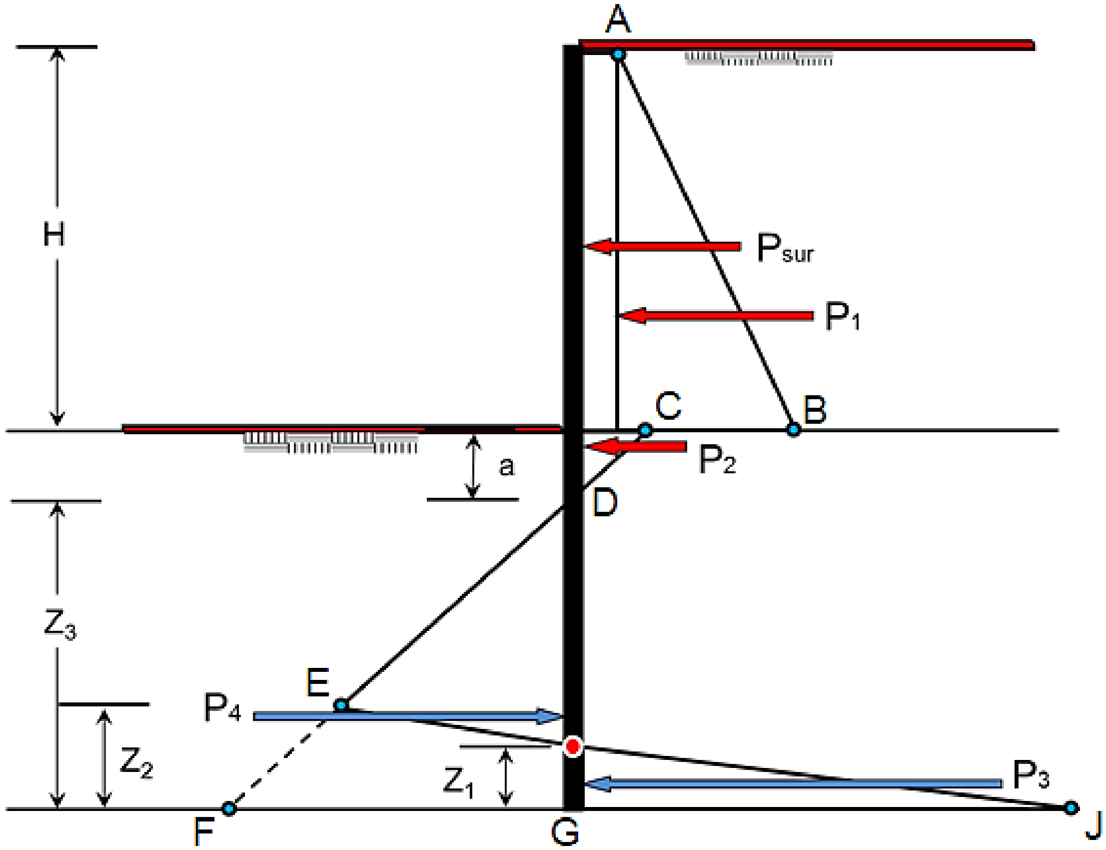



**Figure 7-19. Force Diagram for Rigorous Method**

Set up equations sum of forces and sum of moments to solve for variables **$Z_2$** and **$Z_3$**:

$$\Sigma F = 0$$

$$8.64 + 30.4875 + 0.2053 + (2.51525\, Z_3 Z_2 + 19.881\, Z_2) - 1.257625\, Z_3^2 = 0$$

(7-5-19)

Simplify and solve for **$Z_2$**:

$$Z_2 = \frac{1.257625 Z_3^2 - 39.3328}{2.51525 Z_3 + 19.881} \tag{7-5-20}$$

$$\Sigma M = 0$$

$$(8.64 \times (Z_3 + 0.404 + 7.5)) + (30.4875 \times (Z_3 + 0.404 + 5)) + \left(0.2053 \times \left(Z_3 + \frac{2(0.404)}{3}\right)\right) + \left((2.51525 Z_3 Z_2 + 19.881 Z_2) \times \frac{Z_2}{3}\right) - \left(1.257625\, Z_3^2 \times \left(\frac{Z_3}{3}\right)\right) = 0 \quad (7\text{-}5\text{-}21)$$

Simplify and collect like terms:

$$39.3328\, Z_3 + 233.1003 + 0.83842\, Z_3 Z_2^2 + 6.627\, Z_2^2 - 0.41921\, Z_3^3 = 0 \quad (7\text{-}5\text{-}22)$$

Solve for $\mathbf{Z_2}$ and $\mathbf{Z_3}$ by using iteration to achieve both simplified equations to equal 0:

$$Z_2 = 3.351 \text{ ft} \quad \& \quad Z_3 = 13.122 \text{ ft} \quad (7\text{-}5\text{-}23)$$

**Determine Embedment Depth (without a Safety Factor):**

$$\text{Total Embedment Depth} = Z_3 + a = 13.122 + 0.404 = 13.526 \text{ ft} \quad (7\text{-}5\text{-}24)$$

**Calculate Maximum Shear:**

Maximum shear occurs when the load diagram crosses zero (see Figure 7-20). In this case, the loading crosses zero at two locations so the area of the load diagram has to be calculated before the first zero point and after the second zero point. The largest value of the two areas will be $\mathbf{V_{max}}$. Usually, it will be the area of loading below the pivot point (second zero load location) because this is where the largest passive pressure is acting at the base of the wall.

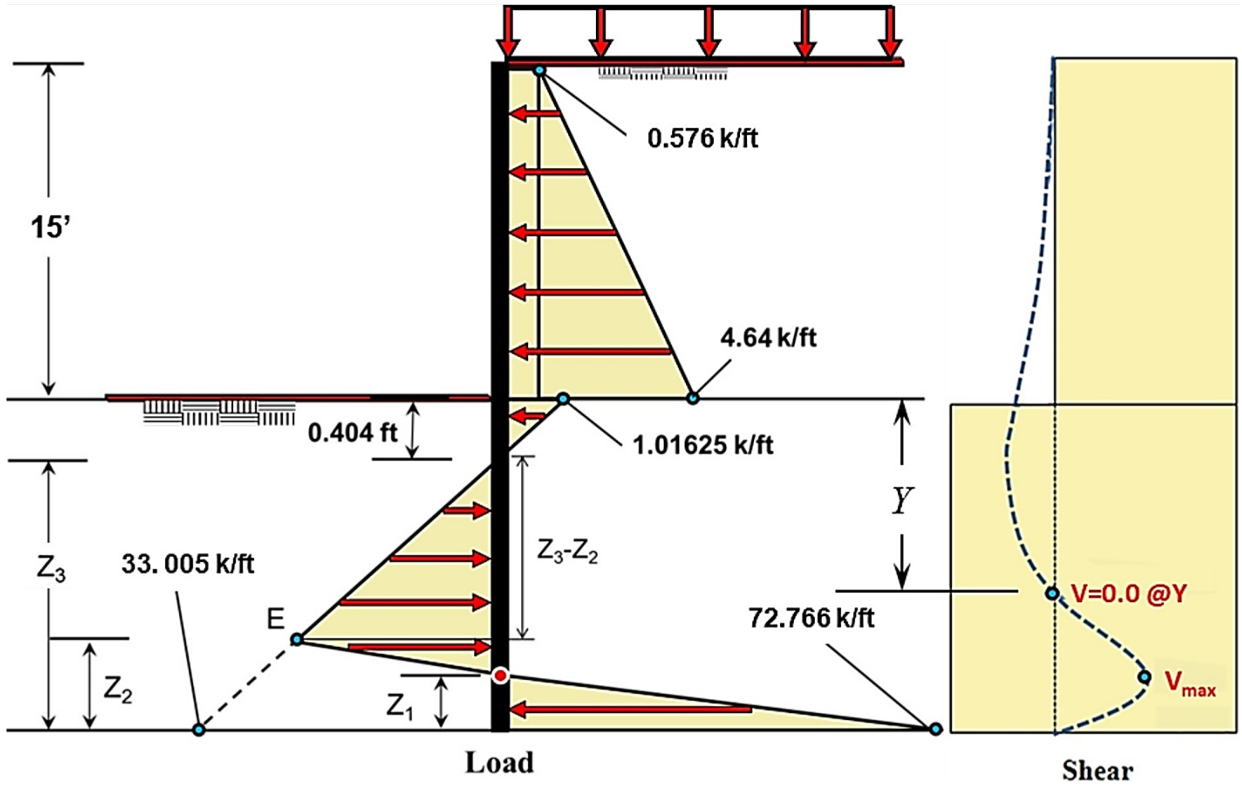



**Figure 7-20. Net Load and Shear Diagram**

Find pressure (kips/ft) at point E, using similar triangles:

$$\frac{33.005 \text{ kips/ft}}{13.122 \text{ ft}} = \frac{\text{E}}{(13.122 - 3.351)\text{ft}} \quad => \quad \text{E} = 24.576 \text{ Kips/ft} \tag{7-5-25}$$

Use similar triangles again to calculate $\mathbf{Z_1}$:

$$\frac{3.351 \text{ ft}}{(24.576 + 72.766) \text{ kips/ft}} = \frac{\text{Z}_1}{72.766 \text{ kips/ft}} \quad => \quad \text{Z}_1 = 2.505 \text{ ft} \tag{7-5-26}$$

Calculate Shear, $\mathbf{V_{max}}$:

$$\text{V}_{\text{max}} = \frac{1}{2} \times \left(72.766 \frac{\text{kips}}{\text{ft}}\right) \times (2.505 \text{ ft}) = 91.14 \text{ kips} \tag{7-5-27}$$

**Calculate Maximum Moment:**

The maximum moment is located at distance **Y** below the excavation line where the shear is equal to zero, as illustrated in Figure 7-21. Therefore, the summation of horizontal forces at the distance **Y** must be set to equal zero.

Passive earth pressure at **Y** below the dredge line (Y = y + 0.404 ft):

$$P_P = \frac{1}{2}[0.125 \times 1 \times y((3.69 \times 2.8) - 0.271) \times 2]\, y$$
$$= 1.257625y^2 \text{ kips/ft} \qquad (7\text{-}5\text{-}28)$$

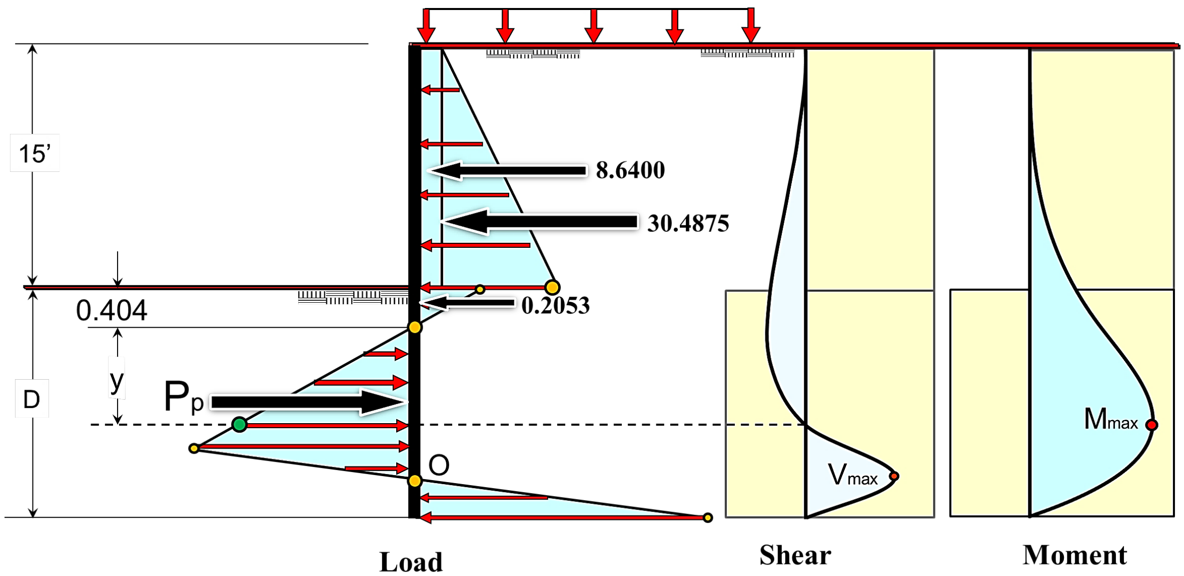



**Figure 7-21. Location of Zero Shear and Maximum Moment for Soldier Pile**

Set up equation, sum of forces: $\sum F_x = 0$ (7-5-29)

$$1.257625y^2 = 8.64 + 30.4875 + 0.2053 \qquad (7\text{-}5\text{-}30)$$

$$1.257625y^2 = 39.3328 \Rightarrow \quad y = 5.592 \text{ ft} \qquad (7\text{-}5\text{-}31)$$

$$Y = 5.592 + 0.404 = 6.00 \text{ ft (below the dredge line)} \qquad (7\text{-}5\text{-}32)$$

$$M_{max} = \begin{cases} 8.64\times(7.5+6.00)+30.4875\times(5+6.00)+0.2053\times\left(6.00-\frac{0.404}{3}\right) \\ -\ 1.257625\times5.592^2\times\left(\frac{5.592}{3}\right) \end{cases} \qquad (7\text{-}5\text{-}33)$$

$$M_{max} = 379.90 \text{ kip-ft} \qquad (7\text{-}5\text{-}34)$$

The Shear and Moment Diagram along the pile length is shown in the Figure 7-22 and the Deflection Diagram is shown in Figure 7-23. These are from the SC Trenching and Shoring (T&S) Program.

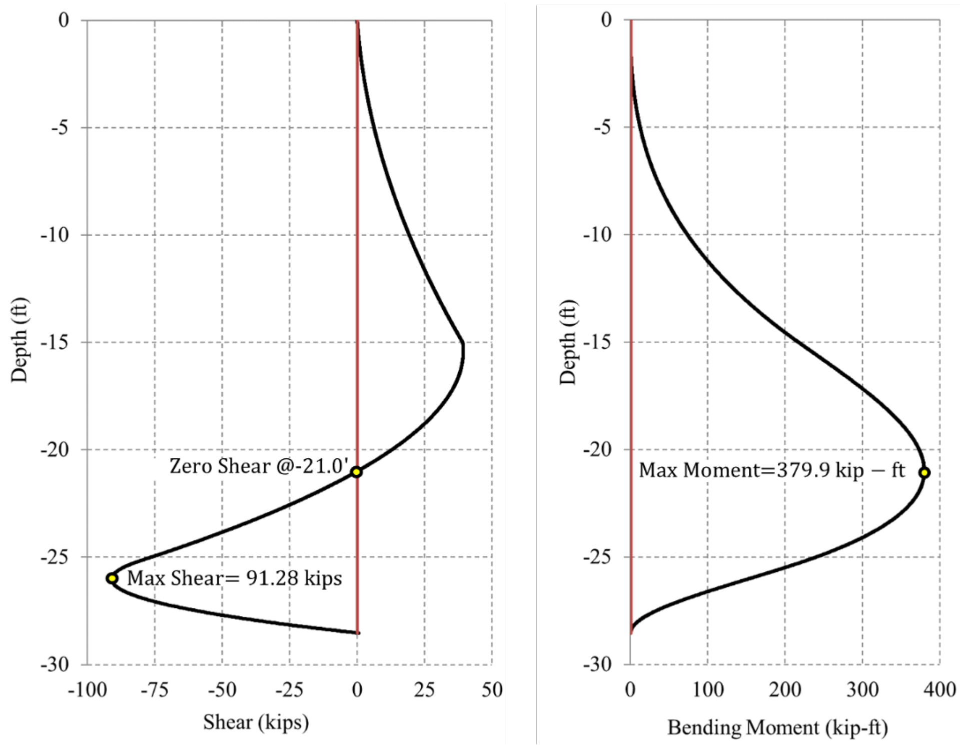



**Figure 7-22. Shear and Moment Diagram (CT_T&S Program)**

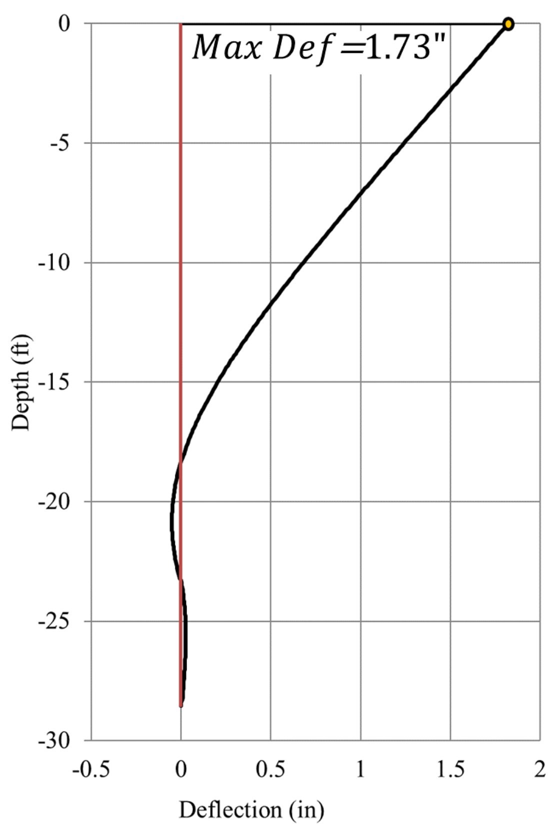



**Figure 7-23. Deflection (CT_T&S Program)**

Example lagging calculations are based on Chapter 6, *Structural Design of Shoring Systems*, Section 6-5, *Lagging*. A reduction factor of 0.6 will be used to reduce the soil distribution behind the lagging. See Section 6-5.01, *Example Lagging Calculations,* of this manual.

## 7-5.05 Example 7-1B: Cantilevered Soldier Pile Wall by Simplified Method

Perform a shoring check for a W14x120 cantilevered soldier-pile-lagging wall with piles at 8 feet on center encased in 2-foot diameter holes filled with 4-sack concrete, using the AASHTO simplified procedure. The pressure of 72 psf is the minimum lateral construction surcharge acting on the timber lagging that is caused by typical construction loading. The soil properties are shown below.

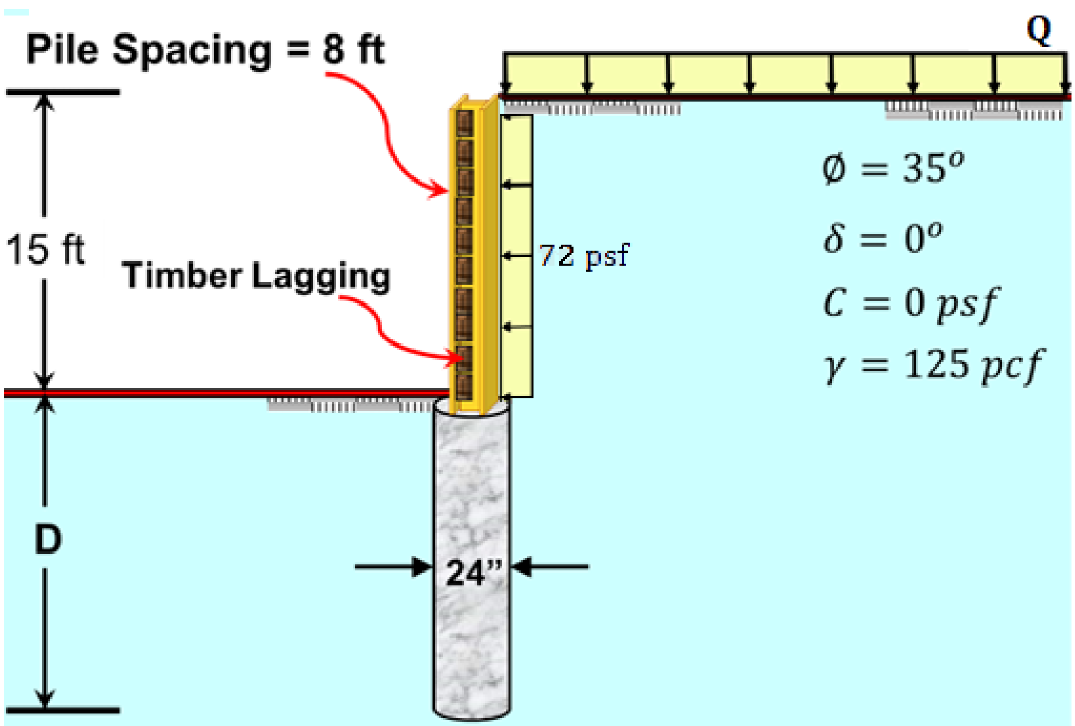



**Figure 7-24. Cantilevered Soldier-Pile-Lagging Wall Cross-Section**

Use these steps:

1. Calculate active & passive earth pressures
2. Determine pile embedment, **D**
3. Calculate maximum shear & moment
4. Calculate service deformation
5. Calculate timber lagging deflection.

**Determine active and passive earth pressures:**

- Calculate active and passive earth pressure coefficients. Since the wall friction (***δ***) is zero, use Rankine earth pressure theory to calculate the active and passive earth pressure coefficients:

$$K_a = \tan^2\left(45 - \frac{\emptyset}{2}\right) = \tan^2\left(45 - \frac{35}{2}\right) = 0.271 \quad (7\text{-}5\text{-}35)$$

$$K_p = \tan^2\left(45 + \frac{\emptyset}{2}\right) = \tan^2\left(45 + \frac{35}{2}\right) = 3.690 \quad (7\text{-}5\text{-}36)$$

Note: Rankine theory tends to underestimate the passive earth pressure. It is recommended to use the Log-Spiral model to compute the passive earth force.

- Calculate earth pressure distribution:

Lateral load due to minimum construction surcharge above the excavation line only:

$$\sigma_{sur} = 0.072 \text{ ksf} \quad (7\text{-}5\text{-}37)$$

Lateral load distribution at excavation level:

$$\sigma = 0.125 \text{ x } 15 \text{ x } 0.271 = 0.508 \text{ ksf} \quad (7\text{-}5\text{-}38)$$

Active lateral load distribution of the soil below the dredge line at depth **$D_o$**:

$$\sigma_{AD_O} = 0.508 + (0.271 \text{ x } 0.125 \text{ x } D_O) = (0.508 + 0.0339\ D_O) \text{ksf} \quad (7\text{-}5\text{-}39)$$

Passive Lateral load distribution in front of the wall, at depth **$D_o$**:

$$\sigma_{pD_O} = 0.125 \text{ x } 3.69 \text{ x } D_O = 0.461\ D_O \text{ ksf} \quad (7\text{-}5\text{-}40)$$

Calculate resultant earth forces:

1. Calculate active earth force due to surcharge **$P_{AS}$**:

$$P_{As} = 15 \text{ ft} \times 8 \text{ ft} \times 0.072 \text{ ksf} = 8.64 \text{ kips} \quad (7\text{-}5\text{-}41)$$

2. Calculate active earth force above dredge line using 8-foot spacing, **$P_{A1}$**:

$$P_{A1}=\frac{15}{2}\ \text{ft}\times 8\ \text{ft}\times 0.508\ \text{ksf}=30.48\ \text{kips} \quad (7\text{-}5\text{-}42)$$

3. Calculate active earth force below dredge line using 2-foot hole diameter, **$P_{A2}$**:

$$P_{A21}=D_0\times 2\times 0.508=1.016\ D_0\ \text{kips} \quad (7\text{-}5\text{-}43)$$

$$P_{A22}=0.0339D_0\times\left(\frac{D_0}{2}\right)\times 2=0.0339\ D_0^2\ \text{kips} \quad (7\text{-}5\text{-}44)$$

- Calculate passive earth force below dredge line using 2-foot hole diameter and soil arching capability factor, **$P_p$**:

$$f=0.08\times\emptyset=(0.08\times 35)=2.8 \quad (7\text{-}5\text{-}45)$$

$$P_p=0.461D_0\times\left(\frac{D_0}{2}\right)\times 2\times f=0.461D_0\times\left(\frac{D_0}{2}\right)\times 5.60=1.291D_0^2\ \text{kips} \quad (7\text{-}5\text{-}46)$$

These calculated values are summarized in Figure 7-25.

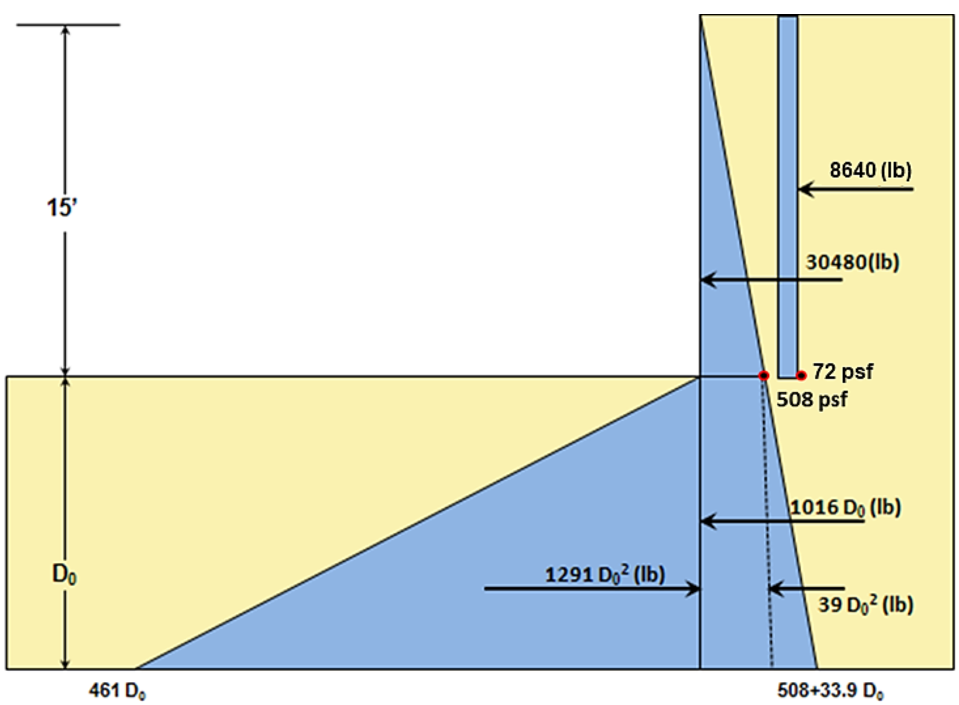



**Figure 7-25. Force Diagram**

Calculate driving and resisting moments as shown in Table 7-1 and Table 7-2:

**Table 7-1. Driving Moments**

| Driving Force (kips) | Arm (ft) | Driving Moment $M_{DR}$ |
|---|---|---|
| 8.64 | $7.5 + D_0$ | $64.8 + 8.64D_0$ |
| 30.48 | $5 + D_0$ | $152.4 + 30.48D_0$ |
| $1.016D_0$ | $\frac{D_0}{2}$ | $0.508D_0^2$ |
| $0.0339D_0^2$ | $\frac{D_0}{3}$ | $0.0113D_0^3$ |

**Table 7-2. Resisting Moments**

| Resisting Force (kips) | Arm (ft) | Resisting Moment $M_{RS}$ |
|---|---|---|
| $1.291D_0^2$ | $\frac{D_0}{3}$ | $0.430D_0^3$ |

$$M_{DR} = 0.0113\ D_0^3 + 0.508\ D_0^2 + 30.48\ D_0 + 152.4 + 8.64\ D_0 + 64.8 \quad (7\text{-}5\text{-}47)$$

$$M_{RS} = 0.43\ D_0^3 \quad (7\text{-}5\text{-}48)$$

**Calculate embedment depth:**

$$M_{DR} = 0.0113\ D_0^3 + 0.508\ D_0^2 + 30.48\ D_0 + 152.4 + 8.64\ D_0 + 64.8 \quad (7\text{-}5\text{-}49)$$

$$D_0^3 - 1.2133\ D_0^2 - 93.432\ D_0 - 518.75 = 0 \quad (7\text{-}5\text{-}50)$$

Solving for $\mathbf{D_o}$: $D_0 = 12.272\ \text{ft}$ (7-5-51)

Determine **D** to account for pile embedment required below point O.

$$D = 1.2D_0 = 14.73\ \text{ft} \quad (7\text{-}5\text{-}52)$$

Note that this embedment depth, **D**, does not have a safety factor applied.

**Calculate maximum moment:**

The maximum moment is located at distance **Y** below the excavation line where the shear is equal to zero. Therefore, the summation of horizontal forces at the distance **Y** must be set to equal zero.

$$\sum F_x = 0 \tag{7-5-53}$$

$$1.2571Y^2 - 1.016Y - 39.12 = 0 \tag{7-5-54}$$

$$Y^2 - 0.808Y - 31.119 = 0 \quad => \quad Y = 5.997 \text{ ft (below the dredge line)} \tag{7-5-55}$$

$$V_{max} = 1.291 \times 12.272^2 - 8.64 - 30.48 - 1.016 \text{ x } 12.272 - 5.11 = 137.729 \text{ kips} \tag{7-5-56}$$

$$M_{max} = \left\{ \begin{array}{c} 8.64\times(7.5+5.997)+30.48\times(5+5.997) + 1.016\times5.997\times\left(\frac{5.997}{2}\right) \\ + 0.0339\times5.997^2\times\left(\frac{5.997}{3}\right) \\ -1.291\times5.997^2\times\left(\frac{5.997}{3}\right) \end{array} \right\} \tag{7-5-57}$$

$$M_{max} = 379.697 \text{ kip-ft} \tag{7-5-58}$$

Figure 7-26 displays the shear and moment diagram using the Simplified Method, and Figure 7-27 displays the deflection diagram.

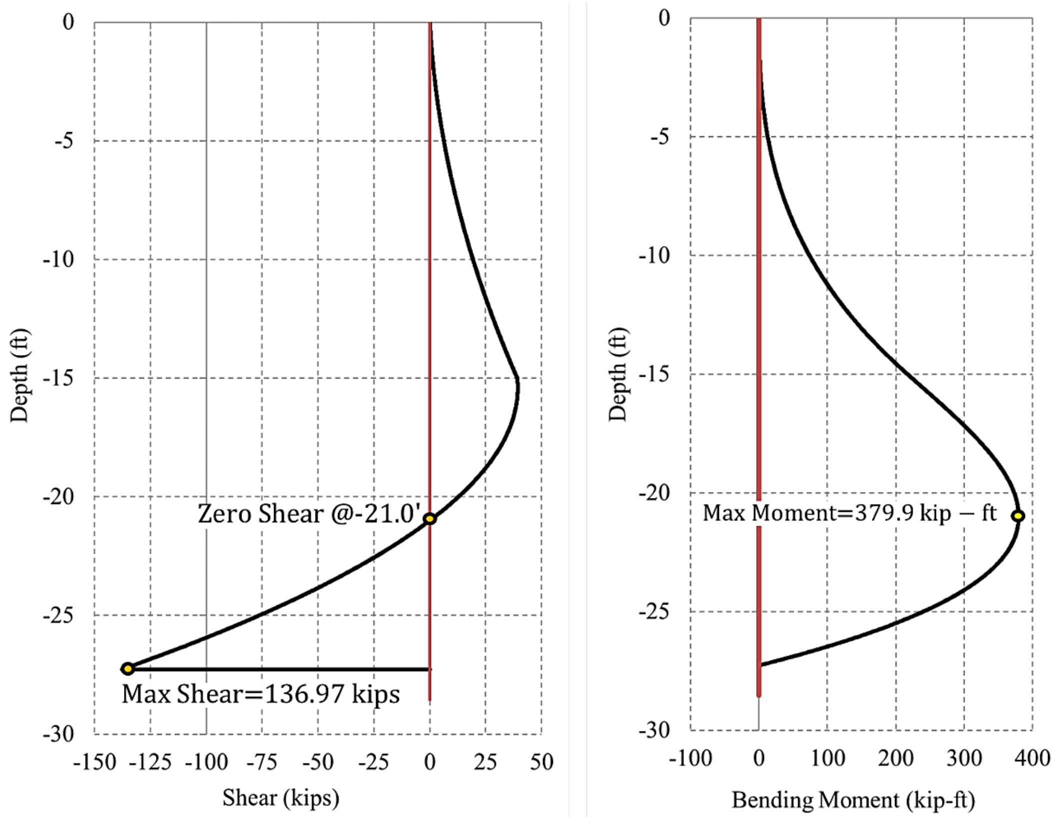



**Figure 7-26. Shear and Moment Diagram (CT_T&S Program)**

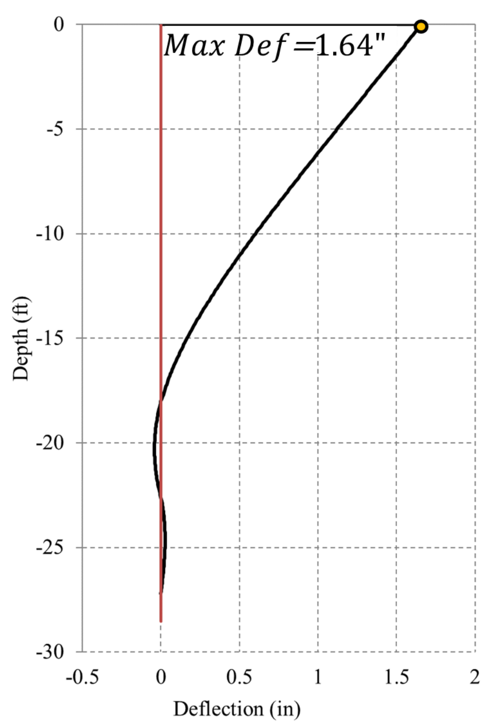



**Figure 7-27. Deflection by Simplified Method (CT_T&S Program)**

Table 7-3 below is a summary of these two methods of shoring system analysis. The Simplified Method is inherently slightly more conservative for embedment depth and shear. As a reminder to the reader, both the Rigorous and the Simplified methods only apply to unrestrained (cantilevered) shoring systems.

**Table 7-3. Comparison of Results Between Simplified Method and Conventional Method**

| Characteristic | Conventional | Simplified |
|---|---|---|
| Depth, **D** (ft) | 13.53 | *14.73 |
| Shear, **V** (kips) | 91.28 | 136.97 |
| Moment, **M** (kip-ft) | 379.90 | 379.70 |
| Deflection ∆ (in) | 1.73 | 1.64 |

Note: Simplified depth **D**, is increased by 20% (**D** = 1.2$D_o$, where $D_o$ = 12.3). The depth, **D**, shown above only used a safety factor of 1.0.

## 7-5.06 Example 7-2: Cantilevered Soldier Pile Wall, Simplified Method with Two Soil Layers above the Excavation

For a shoring system subjected to the lateral load of two different soil layers above the excavation, calculate the total required horizontal force using Rankine earth pressure theory. See Example 5 from Appendix B, *Example Problems*, for a related example.

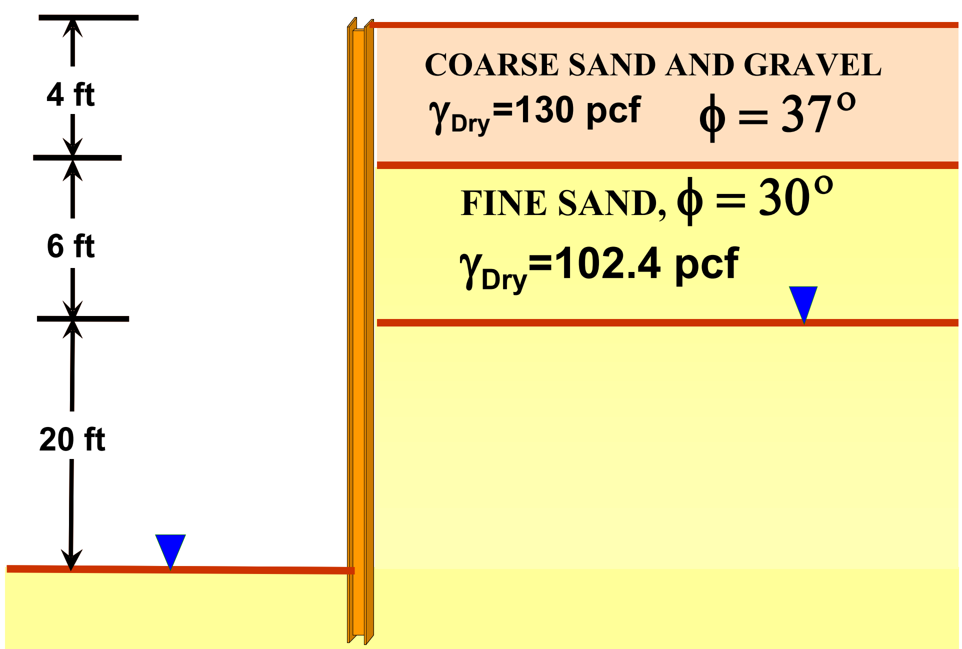



**Figure 7-28. Diagram of Shoring Cross Section and Soil Properties**

Determine:

- Calculate and plot earth pressure distribution.
- Calculate the total force on the shoring system.

Solution:

$$K_{a_1} = \tan^2\left(45 - \frac{\emptyset}{2}\right) = \tan^2\left(45 - \frac{37}{2}\right) = 0.249 \quad (7\text{-}5\text{-}59)$$

$$K_{a_2} = \tan^2\left(45 - \frac{\emptyset}{2}\right) = \tan^2\left(45 - \frac{30}{2}\right) = 0.333 \quad (7\text{-}5\text{-}60)$$

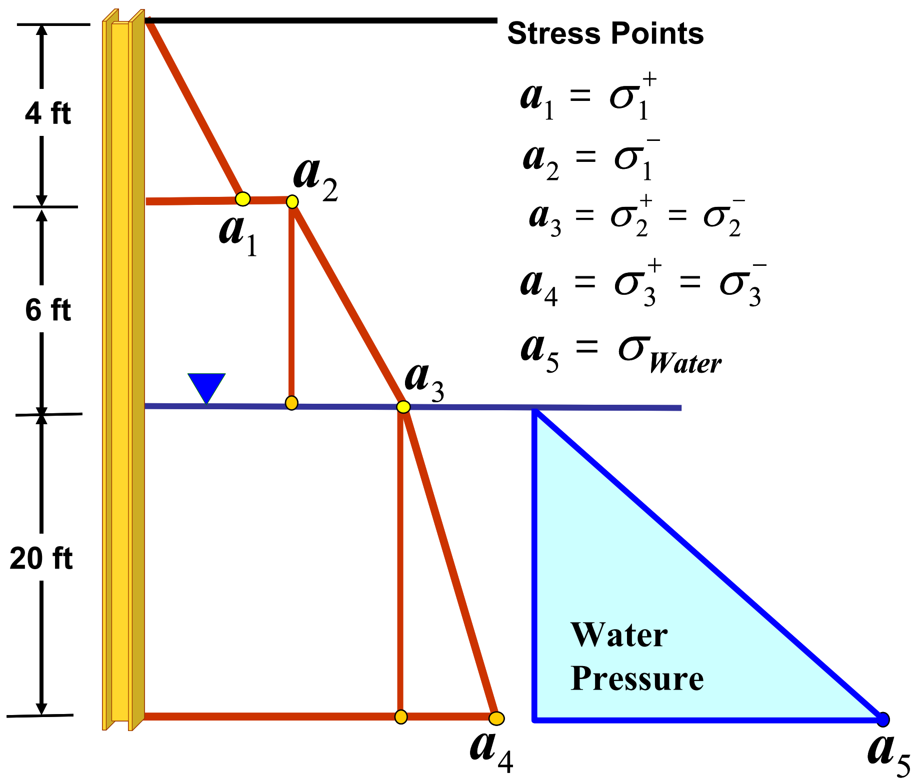



**Figure 7-29. Pressure Loading Diagram**

In Figure 7-29 above and the analysis below, the subscripted numbers refer to the soil layer. The superscripted **+** refers to the stress at the indicated soil layer due the material above the layer line based on **$K_a$** of that soil. The superscripted **–** refers to the stress at the indicated soil layer for the material above the layer line based on the **$K_a$** of the soil below the layer line.

$$\sigma_1^+ = (130\text{pcf})(4\text{ft})(0.249) = 129.48 \text{ psf} \quad (7\text{-}5\text{-}61)$$

$$\sigma_1^- = (130\text{pcf})(4\text{ft})(0.333) = 173.16 \text{ psf} \quad (7\text{-}5\text{-}62)$$

$$\sigma_2^+ = 173.16 + (102.40\text{pcf})(6\text{ft})(0.333) = 377.76 \text{ psf} \quad (7\text{-}5\text{-}63)$$

$$\sigma_2^+ = \sigma_2^- = 377.76 \text{ psf} \quad (7\text{-}5\text{-}64)$$

$$\sigma_3^+ = \sigma_3^- = 377.76 + (102.40 - 62.40)(20)(0.333) = 644.16 \text{ psf} \quad (7\text{-}5\text{-}65)$$

Water Pressure (at excavation line):

$$\sigma_{a5} = 20(62.4\text{pcf}) = 1{,}248.0 \text{ psf} \quad (7\text{-}5\text{-}66)$$

Driving Forces:

$$F_1 = \frac{1}{2}(4\text{ft})(129.48 \text{ psf}) = 258.96 \text{ lb/ft} \quad (7\text{-}5\text{-}67)$$

$$F_2 = (6\text{ft})(173.16 \text{ psf}) = 1{,}038.96 \text{ lb/ft} \quad (7\text{-}5\text{-}68)$$

$$F_3 = \frac{1}{2}(6\text{ft})(377.76 - 173.16 \text{ psf}) = 613.80 \text{ lb/ft} \quad (7\text{-}5\text{-}69)$$

$$F_4 = (20\text{ft})(377.76 \text{ psf}) = 7{,}555.20 \text{ lb/ft} \quad (7\text{-}5\text{-}70)$$

$$F_5 = \frac{1}{2}(20\text{ft})(644.16 - 377.76 \text{ psf}) = 2{,}664.00 \text{ lb/ft} \quad (7\text{-}5\text{-}71)$$

$$F_6 = \frac{1}{2}(20\text{ft})(1248 \text{ psf}) = 12{,}480 \text{ lb/ft} \quad (7\text{-}5\text{-}72)$$

Net Forces:

$$F_{TOTAL} = 24{,}610.92 \text{ lb/ft} \quad (7\text{-}5\text{-}73)$$

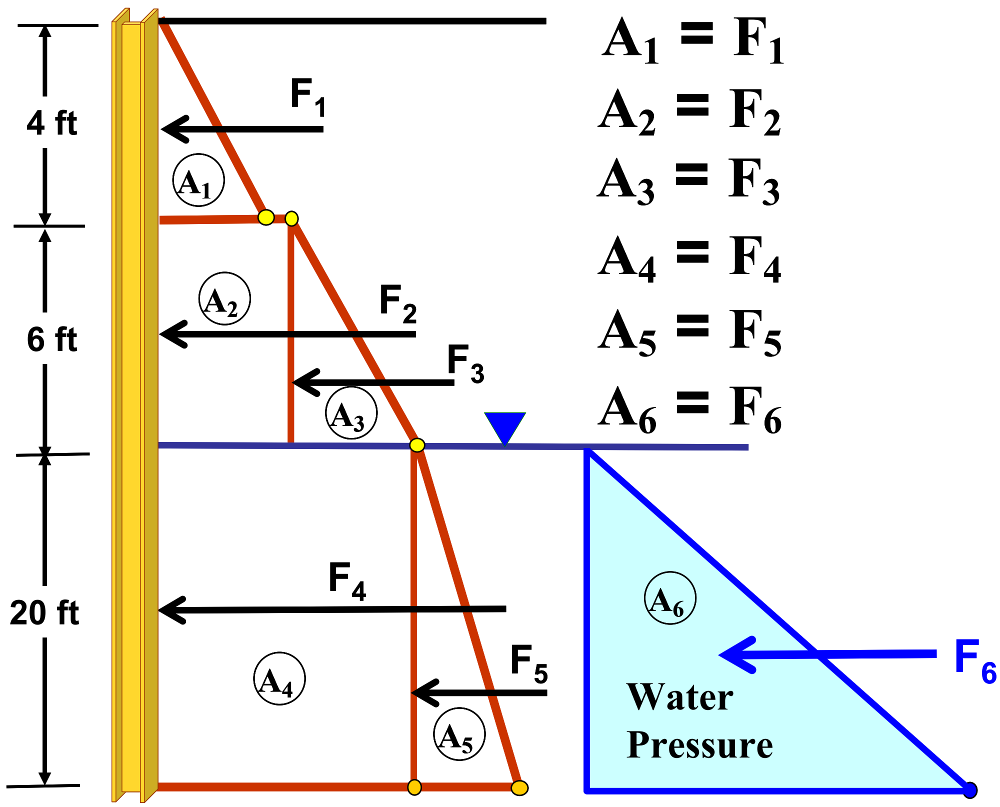



**Figure 7-30. Force Loading Diagram**

## 7-5.07 Example 7-3: Cantilevered Soldier Pile Wall, Simplified Method

Using the Simplified Method, check the adequacy of the cantilevered soldier pile wall in granular 2-layered soil with a negative slope in the front of the wall. The soldier pile is an HP12X84, 50 ksi steel beam placed in a 2-foot diameter hole filled with 4-sack concrete. Refer to Figure 7-31.

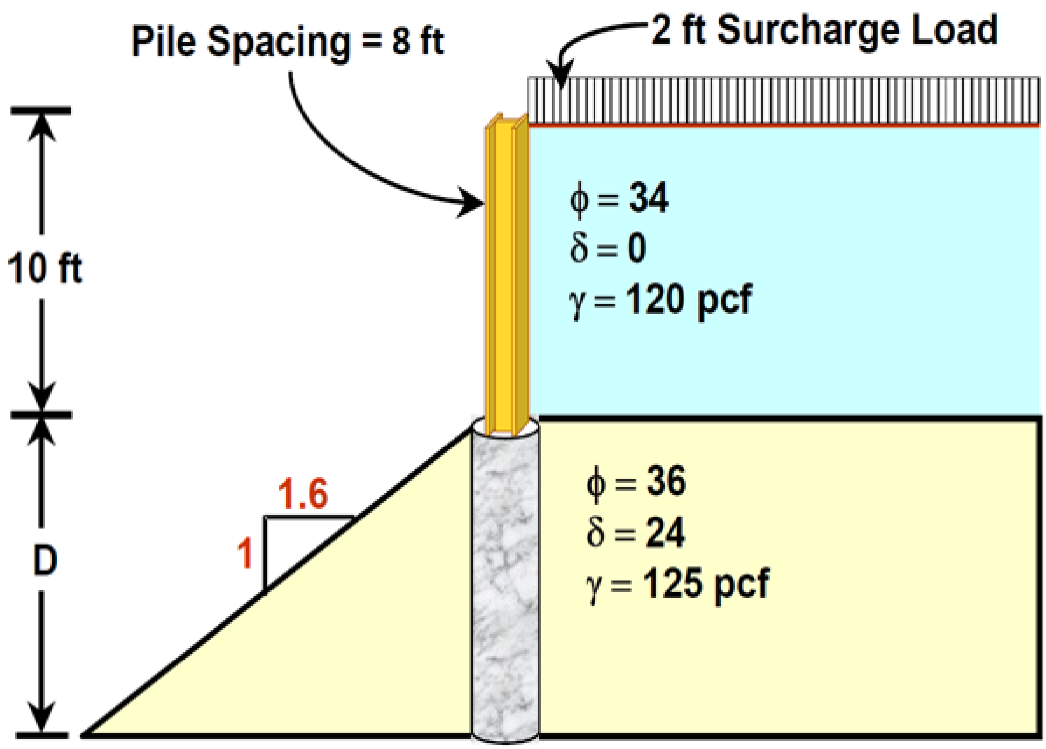



**Figure 7-31. Soldier Pile Wall with Sloping Ground, Example 7-3**

Determine:

1. Active & passive earth pressures
2. Pile embedment, **D**
3. Maximum moment.

Solution:

**Calculate the active & passive earth pressures for layer 1:**

$$K_{a_1} = \tan^2\left(45 - \frac{\emptyset}{2}\right) = \tan^2\left(45 - \frac{34}{2}\right) = 0.283 \quad (7\text{-}5\text{-}74)$$

Use Coulomb theory to calculate active earth pressure in layer 2. Note that due to layer 2 having a friction angle, **δ**, between the soil and the shoring, the active and passive pressures act at an angle (to the horizontal), and thus will need to be converted to the horizontal value.

$$K_{a2} = \frac{\cos^2\phi}{\cos\delta\left[1 + \sqrt{\frac{\sin(\delta + \phi)\sin\phi}{\cos\delta}}\right]^2} \qquad (7\text{-}5\text{-}75)$$

$$K_{a2} = \frac{\cos^2(36)}{\cos(24)\left[1 + \sqrt{\frac{\sin(24 + 36)\sin(36)}{\cos(24)}}\right]^2} = 0.235 \qquad (7\text{-}5\text{-}76)$$

The passive horizontal earth pressure coefficient $\mathbf{K_{ph}}$ is calculated using the method outlined in Section 4-6, *Log-Spiral Passive Earth Pressure,* and Figure 4-20, as shown below:

Calculate **δ/ϕ**:

$$\frac{\boldsymbol{\delta}}{\boldsymbol{\phi}} = \frac{24}{36} = 0.67 \qquad (7\text{-}5\text{-}77)$$

Calculate **β/ϕ**:

$$\frac{\boldsymbol{\beta}}{\boldsymbol{\phi}} = -\frac{32}{36} = -0.89 \qquad (7\text{-}5\text{-}78)$$

Where beta (**β)** is the slope of the backfill.

Use Log-Spiral to determine the passive soil coefficient, $\mathbf{K_p}$**,** using Figure 4-20. Determine $\mathbf{K_p}$ from Figure 4-20: $\mathbf{K_p}$ **= 1.65**

Determine the reduction factor **R**, using the ratio of **δ/ϕ** (thru interpolation of Figure 4-20): **R = 0.8**

$$\mathbf{K'_p} = \mathbf{K_p} \times \mathbf{R} = 1.65 \times 0.8 = 1.32 \qquad (7\text{-}5\text{-}79)$$

$\mathbf{K_p}$ is acting at an angle due to the wall friction angle, δ, of 24 degrees. Thus, calculate $\mathbf{K_p}$ acting in the horizontal, $\mathbf{K_{ph}}$:

$$\mathbf{K_{ph}} = \mathbf{K'_p} \times \mathbf{cos}(\boldsymbol{\delta}) = 1.32 \times \cos(24) = 1.20 \qquad (7\text{-}5\text{-}80)$$

Where:

**δ** = Friction angle between soil and shoring members (in this case, it is with the embedded piles).

**Φ** = Effective friction angle of soil.

**$K_a$** = Coefficient of active lateral earth pressure.

**$K_p$** = Coefficient of passive lateral earth pressure.

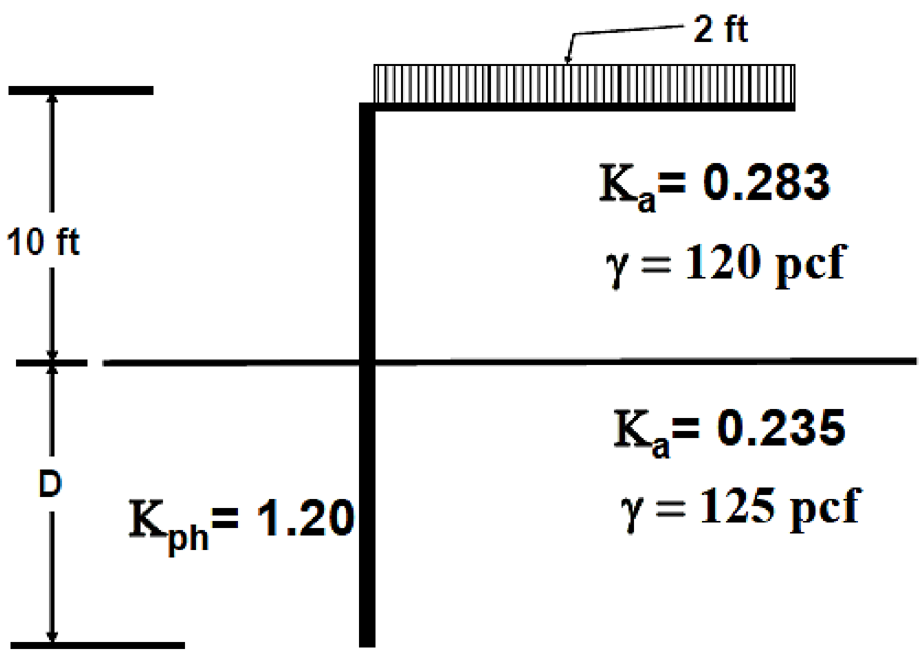



**Figure 7-32. Active and Passive Earth Pressure Coefficients**

Calculate earth pressure distribution:

Lateral load due to surcharge above the excavation line only:

$$\sigma_{Sur} = (120) \times (2) \times (0.283) = 68 \text{ psf; use } 72 \text{ psf minimum} \quad (7\text{-}5\text{-}81)$$

Note: Surcharge is assumed to act uniformly for the top 10 feet only.

Lateral load distribution for the first layer:

$$\sigma^{+} = 72 + [(120) \times (10) \times (0.283)] = 411.6 \text{ psf; use } 412 \text{ psf} \quad (7\text{-}5\text{-}82)$$

Lateral load distribution for the second layer at the soil boundary:

$$K_{ah} = K_a \times \cos(\delta) = 0.235 \times \cos(24) = 0.215 \quad (7\text{-}5\text{-}83)$$

$$\sigma^{-} = (120) \times (10) \times (0.215) = 258 \text{ psf} \quad (7\text{-}5\text{-}84)$$

Lateral load distribution for the second layer at depth **D**:

$$\sigma_D = 258 + (125)(0.215)D = (258 + 26.88D) \text{ psf} \quad (7\text{-}5\text{-}85)$$

Passive lateral load distribution for the second layer in the front of depth **D**:

$$\sigma_{PD} = (125)(1.2)D = 150D \text{ psf} \quad (7\text{-}5\text{-}86)$$

Calculate active earth pressure due to surcharge $\mathbf{P_{AS}}$:

$$P_{AS} = 72 \times 10 = 720 \text{ plf} \quad (7\text{-}5\text{-}87)$$

Calculate active earth pressure for the first soil layer $\mathbf{P_{A1}}$:

$$P_{A1} = \left[(412 - 72)\left(\frac{10}{2}\right)\right] = 1700 \text{ plf} \quad (7\text{-}5\text{-}88)$$

Calculate active earth pressure for the second soil layer $\mathbf{P_{A2}}$:

$$P_{A21} = 258D \text{ plf} \quad (7\text{-}5\text{-}89)$$

$$P_{A22} = \left[26.88 \times D \times \left(\frac{D}{2}\right)\right] = 13.44D^2 \text{ plf} \quad (7\text{-}5\text{-}90)$$

Calculate passive earth pressure for the second soil layer $\mathbf{P_p}$:

$$P_P = \left[150 \times D \times \left(\frac{D}{2}\right)\right] = 75D^2 \text{ plf} \quad (7\text{-}5\text{-}91)$$

Because the pile spacing is equal to 4 times the effective width of the pile and the arching factor is limited to a maximum of 3, the arching factor will be used to increase the effective pile width for the passive forces below the dredge line. Only the effective width of the pile **should be used** for the active forces below the dredge line because the arching factor is not applicable there. Figure 7-33 provides the total pressure for each area per foot of wall. Use these in reference to Table 7-4 and Table 7-5 for calculating the Driving and Resisting moments.

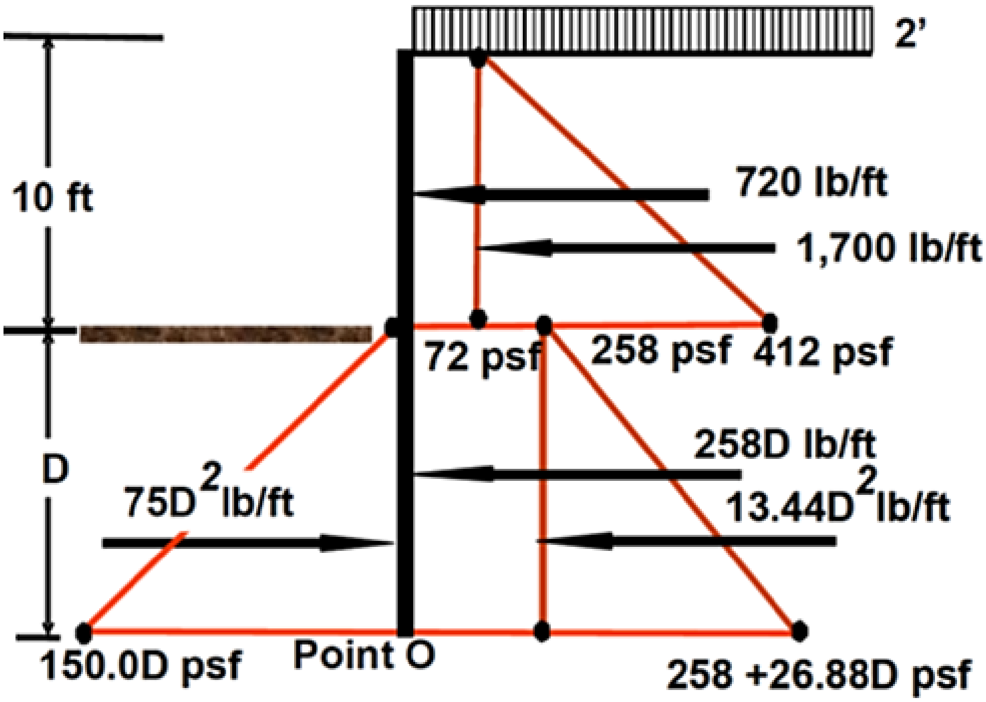



**Figure 7-33. Pressure Diagram**

**Table 7-4. Calculate Driving Moment ($M_{DR}$) about Point "O" using Figure 7-33**

| Driving Force = **$P_a$** × Spacing | Moment Arm(ft.) | Driving Moment **$M_{DR}$** |
|---|---|---|
| 720 × 8 = 5760 | 5+D | 5760D+28800 |
| 1700 × 8 = 13600 | 10/3+D | 13600D+45333 |
| 258D × 2 = 516D | D/2 | 258 $D^2$ |
| 13.44$D^2$ × 2 = 26.88$D^2$ | D/3 | 8.96 $D^3$ |

**Table 7-5. Calculate the Resisting Moment ($M_{RS}$) about Point "O" using Figure 7-33**

| Resisting Force = **$P_p$** × Spacing | Moment Arm(ft.) | Resisting Moment **$M_{RS}$** |
|---|---|---|
| 75$D^2$ × (5.76) = 432$D^2$ | D/3 | 144 $D^3$ |

$$M_{DR} = 8.96D^3 - 258D^2 - 19360D - 74134 = 0 \quad (7\text{-}5\text{-}92)$$

$$M_{RS} = 144D^3 \quad (7\text{-}5\text{-}93)$$

For the external stability use the safety factor of 1.3 to calculate the embedment depth and then increase it by 20 percent to account for the Simplified Method.

$$\frac{M_{RS}}{M_{DR}} = 1.3 \quad (7\text{-}5\text{-}94)$$

$$\left(\frac{144}{1.3}\right)D^3 - 8.96D^3 - 258D^2 - 19360D - 74134 = 0 \quad (7\text{-}5\text{-}95)$$

$$D^3 - 2.53D^2 - 190.2D - 728.2 = 0 \ \longrightarrow D = 16.6 \text{ ft} \quad (7\text{-}5\text{-}96)$$

Increase **D** by 20 percent:

$$16.6 \times 1.20 \longrightarrow D = 19.9 \text{ ft} \quad (7\text{-}5\text{-}97)$$

Calculate embedment depth using a factor of safety (FS) equal to 1.0 for the purposes of calculating the shear and moments.

$$\frac{M_{RS}}{M_{DR}} = 1.0 \quad (7\text{-}5\text{-}98)$$

$$144D^3 - 8.96D^3 - 258D^2 - 19360D - 74134 = 0 \quad (7\text{-}5\text{-}99)$$

$$D^3 - 1.91D^2 - 143.3D - 549 = 0 \ \longrightarrow D = 14.4 \text{ ft} \quad (7\text{-}5\text{-}100)$$

Increase **D** by 20 percent:

$$14.4 \times 1.20 \rightarrow D = 17.3 \text{ ft} \quad (7\text{-}5\text{-}101)$$

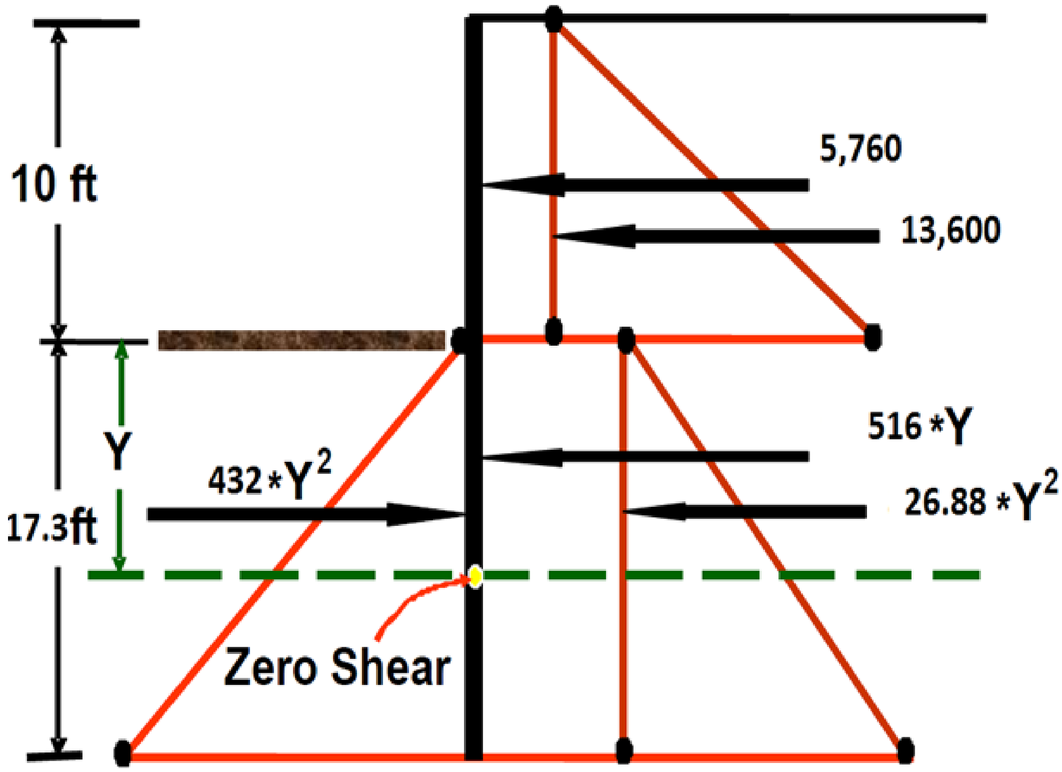



**Figure 7-34. Location of Zero Shear and Maximum Moment**

Calculate zero shear location using Figure 7-34:

$$432Y^2 - 26.88Y^2 - 516Y - 19360 = 0 \qquad (7\text{-}5\text{-}102)$$

$$Y^2 - 1.3Y - 47.8 = 0 \qquad (7\text{-}5\text{-}103)$$

$$Y = 7.59 \text{ ft Below the dredge line} \qquad (7\text{-}5\text{-}104)$$

Based on zero shear location, maximum moment can be calculated as below:

$$M_{max} = 5760(5 + 7.59) + 13600\left(\frac{10}{3} + 7.59\right) + (516 \times 7.59)\left(\frac{7.59}{2}\right) + (26.88 \times 7.59^2)\left(\frac{7.59}{3}\right) - (432 \times 7.59^2)\left(\frac{7.59}{3}\right) \qquad (7\text{-}5\text{-}105)$$

$$M_{max} = 176893 \text{ lb-ft} \qquad (7\text{-}5\text{-}106)$$

$$F_b = 0.60F_y = 0.60(50\text{ ksi}) = 30\text{ ksi} \tag{7-5-107}$$

Recall: $S_x$ = 106 in$^3$ for an HP 12x84

$$S_{Required} = \frac{M_{max}}{F_b} = \frac{176.893 \times 12}{30} = 70.8\text{ in}^3 < 106\text{ in}^3 : O.K \tag{7-5-108}$$

In summary, using the Simplified Method, the adequacy of the 10-foot cantilevered soldier pile wall for a granular 2-layered soil was completed to calculate the active and passive earth pressures, the pile embedment (**D**), and the maximum moment. Deflection and structural integrity of the lagging material were not included in this example.

The pile embedment required was determined to be 19.9 feet with a safety factor of 1.3. Use this value to compare with the Contractor's value in the shop drawings. The calculated maximum moment was used to verify the adequacy of the steel beam provided and was acceptable. Keep in mind that the calculations would have to be done if any of the variables change, such as excavation height, pile spacing, beam type, or soil conditions. See Example 4 from Appendix B, *Example Problems*, for a related example.