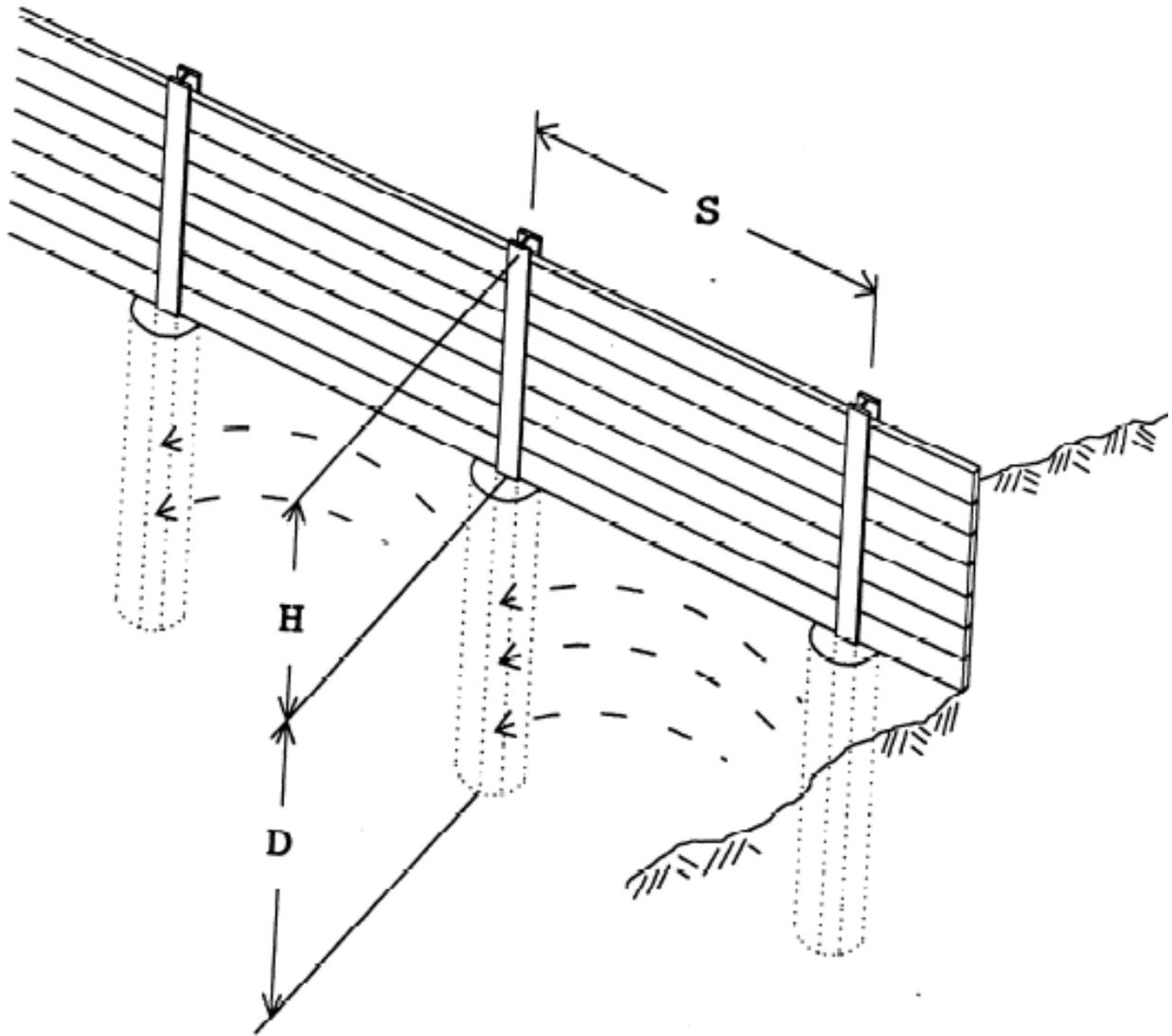# CHAPTER 6

UNRESTRAINED

SHORING SYSTEMS

6.0 *TYPES OF UNRESTRAINED SHORING SYSTEMS*

There are two types of unrestrained shoring systems, sheet pile walls and soldier pile walls. Continuous sheet pile retaining walls may be constructed with driven precast prestressed concrete sheet piles or steel sheet piles with interlocking edges. 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. Figure 6-1 shows a typical cantilever sheet pile retaining wall.

Figure 6-1. Sheet Pile Wall with Cap Beam

Soldier pile retaining walls may be constructed with driven piles (steel, timber or concrete) or they may be placed in drilled holes and backfilled with concrete, slurry, sand, pea-gravel or similar material. A soldier pile could 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, therefore its maximum height is limited to competence of the foundation material and the moment resisting capacity of the vertical structural members. The economical height of this type of wall is generally limited to a maximum height of 18 feet. Figure 6-2 shows a typical soldier pile retaining wall.

Figure 6-2. Soldier Pile Wall with Cap Beam

6.1 *LATERAL EARTH PRESSURES FOR UNRESTRAINED SHORING SYSTEMS*

Non-gravity 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 6-3 (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 6-3 (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% to approximate the total embedment depth of the vertical wall element. The 20% 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 6-3.
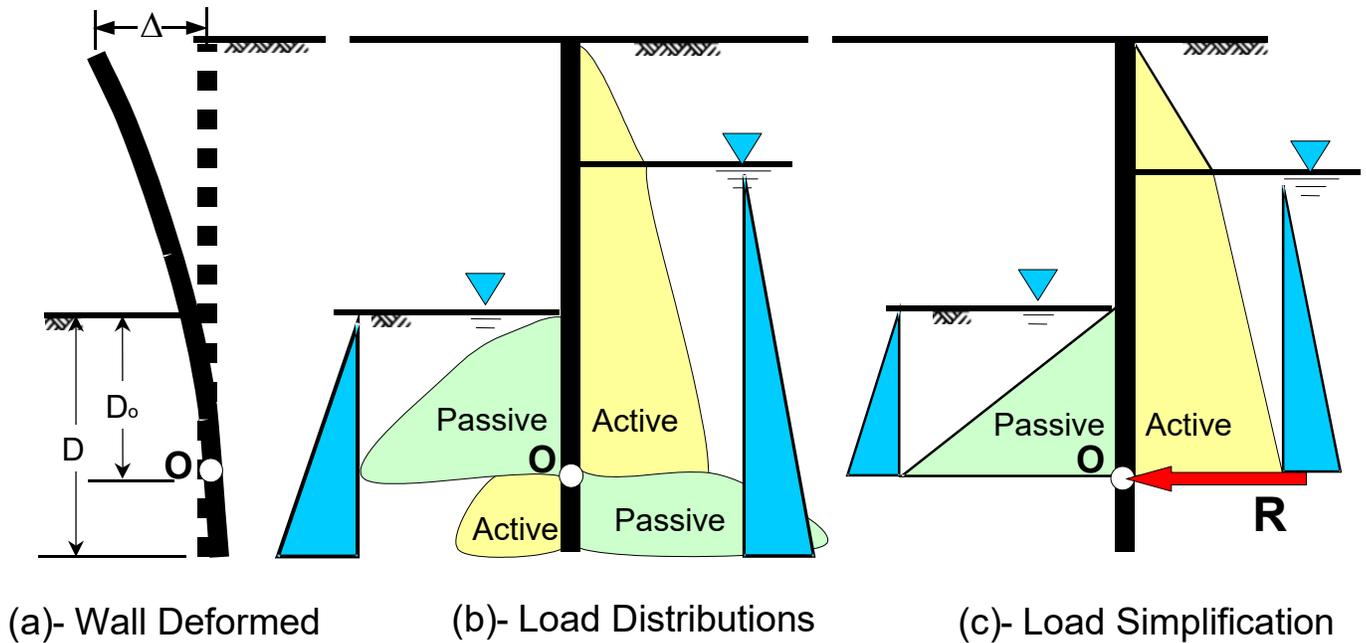

Figure 6-3. Cantilever Retaining Walls

For unrestrained shoring systems, depending on the site soil profile, the simplified lateral earth pressure distribution shown in Figure 6-4 through Figure 6-8 may be used.

For walls with vertical elements embedded in a single layer of granular soil and retaining granular soil, Figure 6-4 may be used to determine the lateral earth pressure distribution for a cantilever shoring system.
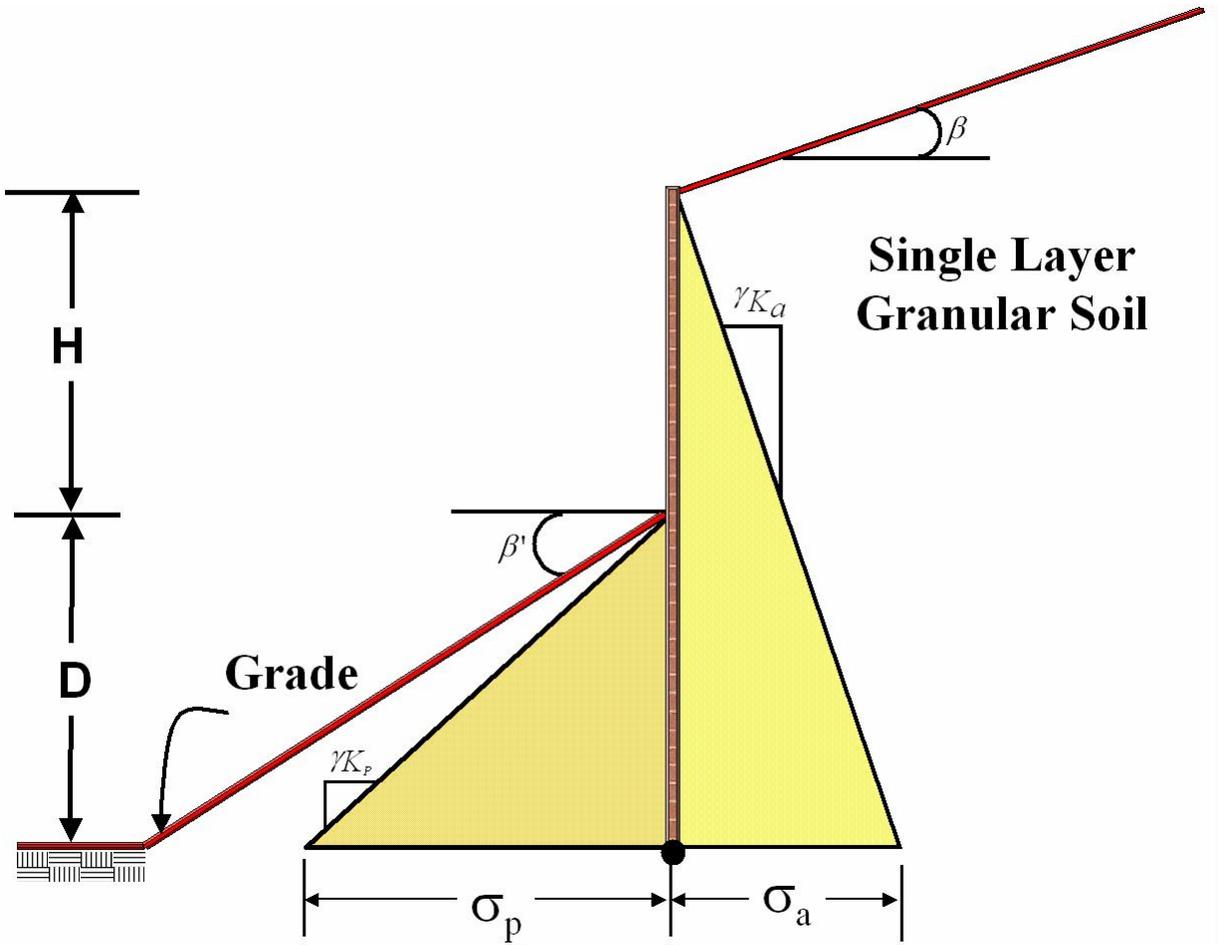

Figure 6-4. Loading Diagram for Single Layer

For walls with vertical elements embedded in multi-layer granular soil and retaining granular soil, Figure 6-5 may be used to determine the lateral earth pressure distribution for a cantilever shoring system.
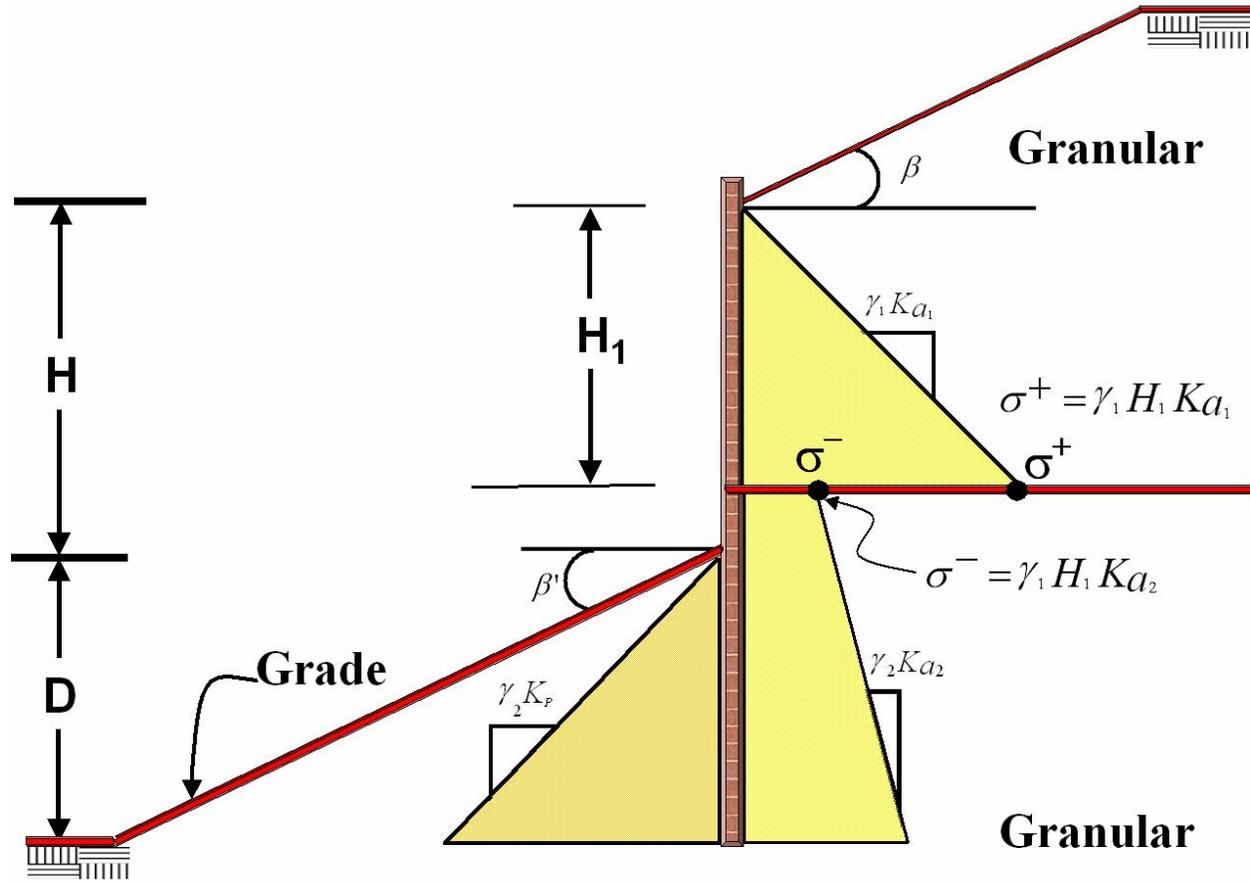

Figure 6-5. Loading Diagram for Multi-Layer Soil

If walls support or are supported by cohesive soils, the walls may be designed by the total stress method of analysis and undrained shear strength parameters. For the latter, the simplified lateral earth pressure distribution shown in Figure 6-6, Figure 6-7, and Figure 6-8 may be used.
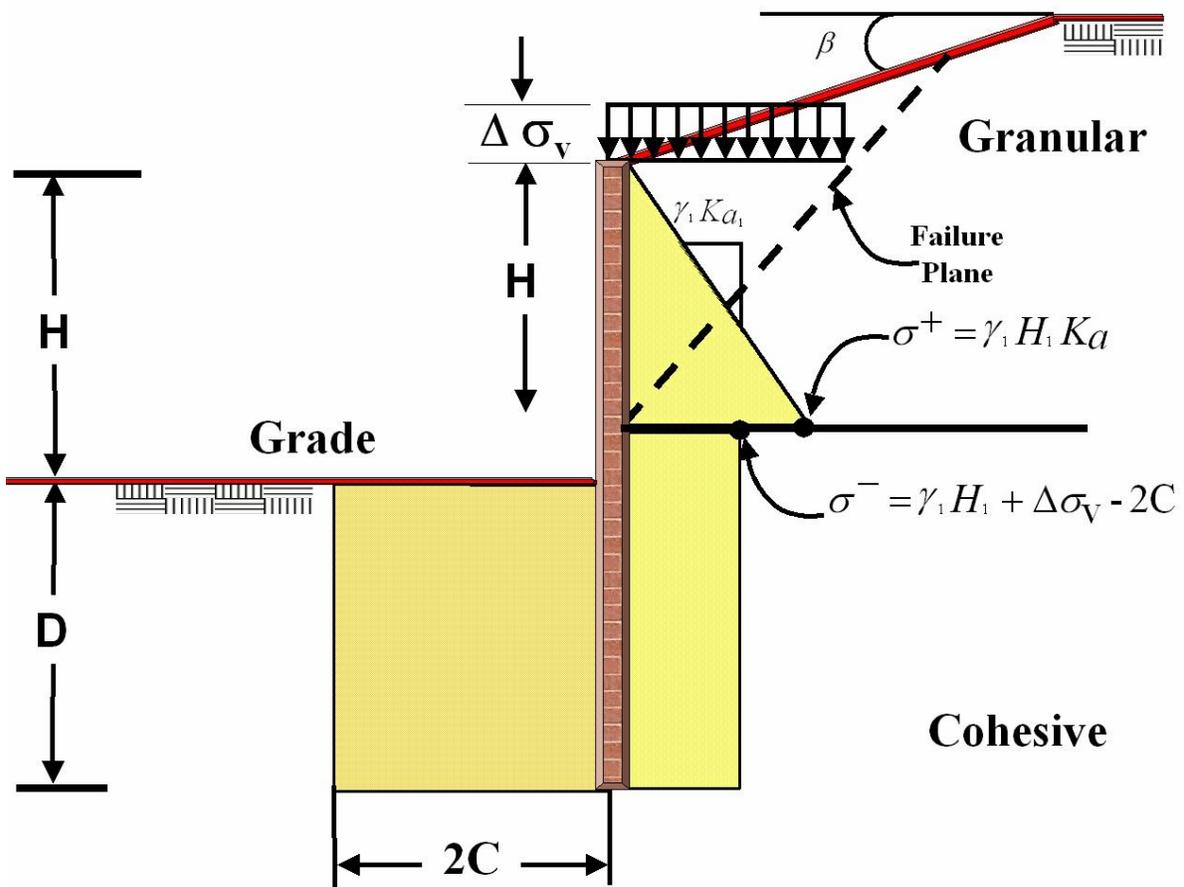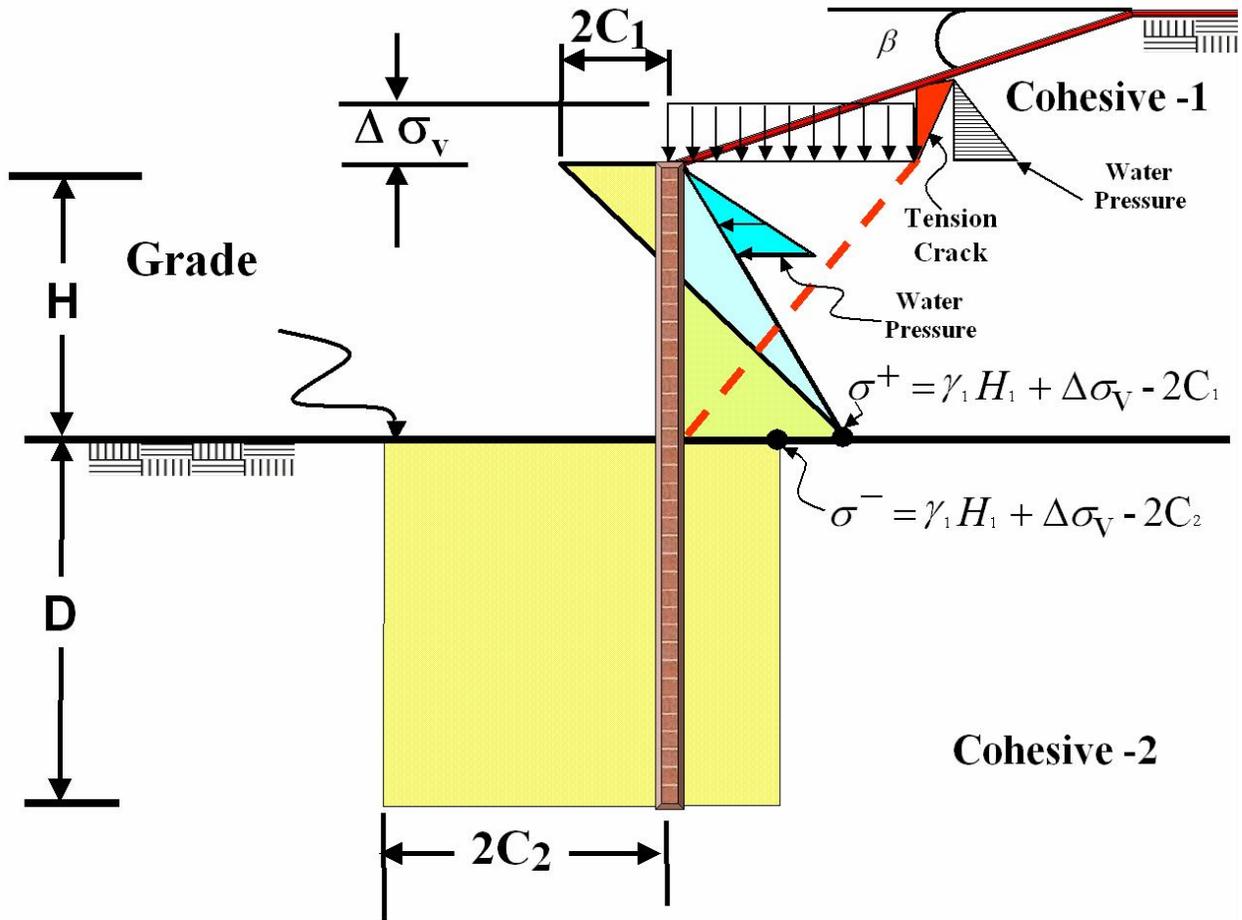

Figure 6-6. Loading Diagram for Multi-Layer

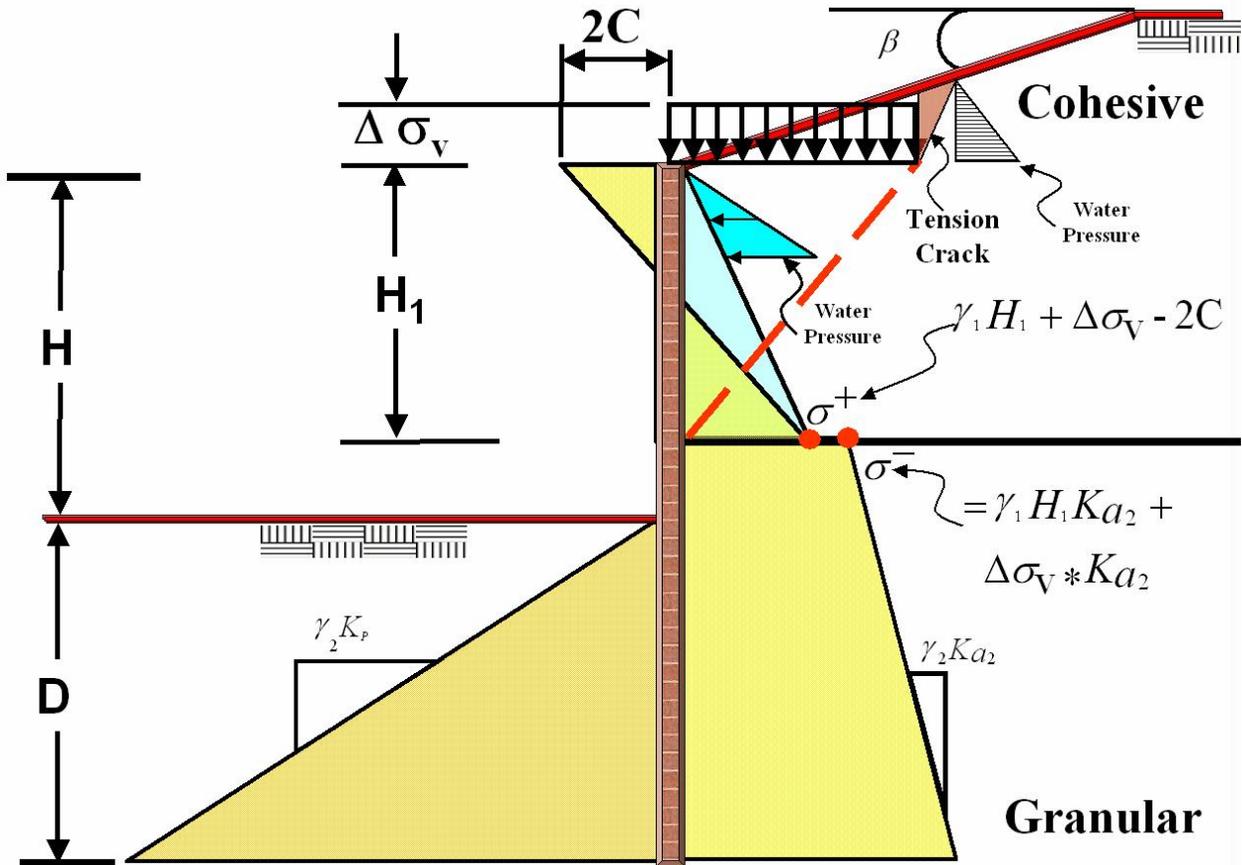Figure 6-7. Loading Diagram for Multi-Layer

Figure 6-8. Loading Diagram for Multi-Layer

To determine the active lateral earth pressure on the embedded wall element shown above:

- Treat the sloping backfill above the top of the wall within the active failure wedge as an additional surcharge ($\Delta\sigma_v$).

- The portion of the negative loading at the top of the wall due to cohesion is ignored

- Any hydrostatic pressure in the tension crack needs to be considered.

- The ratio of total overburden pressure to undrained shear strength (*NS*) must be < 3 at the design grade in front of wall.

- The active lateral earth pressure acting over the wall height (*H*) shall not be less than *0.25* times the effective overburden pressure at any depth, or *0.036* KSF/FT of wall height, which ever is greater.

6.2 *EFFECTIVE 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. 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. Soil arching however, can greatly increase the effective width described above. See Figure 6-9. 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.
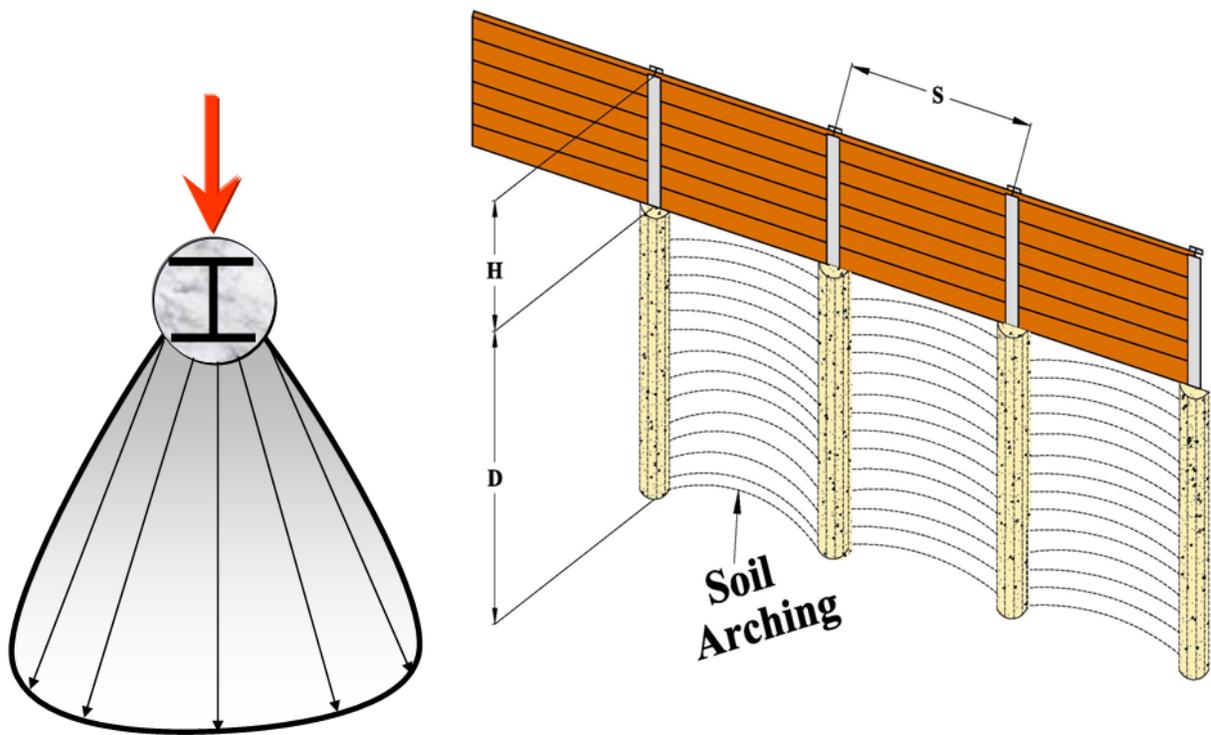
Figure 6-9. Soldier Pile with Arching

Numerous full-scale pile experiments have shown the passive resistance in front of an isolated pile is a three dimensional problem as shown in Figure 6-9. Two dimensional classical earth pressure theories under estimates the passive resistance in front of a soldier pile. Therefore, the passive resistance in front of a pile calculated by classical earth pressure theories shall be multiplied by the

adjusted pile width. The adjusted pile width is a function of the effective width of the pile and the soil friction angle (ϕ) as shown below.

$$\text{Adjusted Pile Width} = \text{Effective Width} * \text{Arching Capability Factor} \qquad \text{Eq. 6-1}$$

Table 6-1. Arching Capability Factor

Pile Spacing (s)	Arching Capability Factor
$\leq 3 * d$	3
$> 3 * d$	$0.08 * \phi \ (\leq 3)$

Where:

Effective Width = Width of the pile as described above.

d = Effective Width

ϕ = Internal friction angle of the soil in degrees

For granular soils, if the pile spacing is 3 times the effective width (d) or less the arching capability factor may be taken as 3. The arching capability for cohesive soil ranges between 1 and 2 as shown in Table 6-2.

Table 6-2. Arching Capability for Cohesive Soil

```
   CONSISTENCY            VERY                              VERY
                          SOFT       SOFT      MEDIUM    STIFF    STIFF      HARD

   q  = unconfined comp.
    u
   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.
```

Below the excavation depth the adjusted pile width is used for any active loadings (including surcharge loadings) on the back of the pile as well as for the passive resistance in front of the pile. The adjusted pile width cannot exceed the pile spacing and when the adjusted pile width equals the pile spacing, soldier pile systems can be analyzed in the same manner as sheet pile systems.

6.3 *DEFLECTION*

Calculating deflections of temporary shoring systems can be complicated. 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 yield large lateral deflections. The amount of deflection or movement that is allowable inversely proportional to the sensitivity to movement of what is being shored. Thus it will be up to the Engineer's good judgment as to what degree of analysis will be performed. Bear in mind that except for the Railroad as discussed in CHAPTER 8 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 based on case by case basis.

Typical deflection calculations are normally performed per standard beam analysis methods. 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 typical shoring systems do not necessarily act as standard beams supported by 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 shall be performed. The SSI method of analysis is beyond the scope of this Manual and the Engineer is encouraged to contact the Trenching and Shoring Specialist in Sacramento.
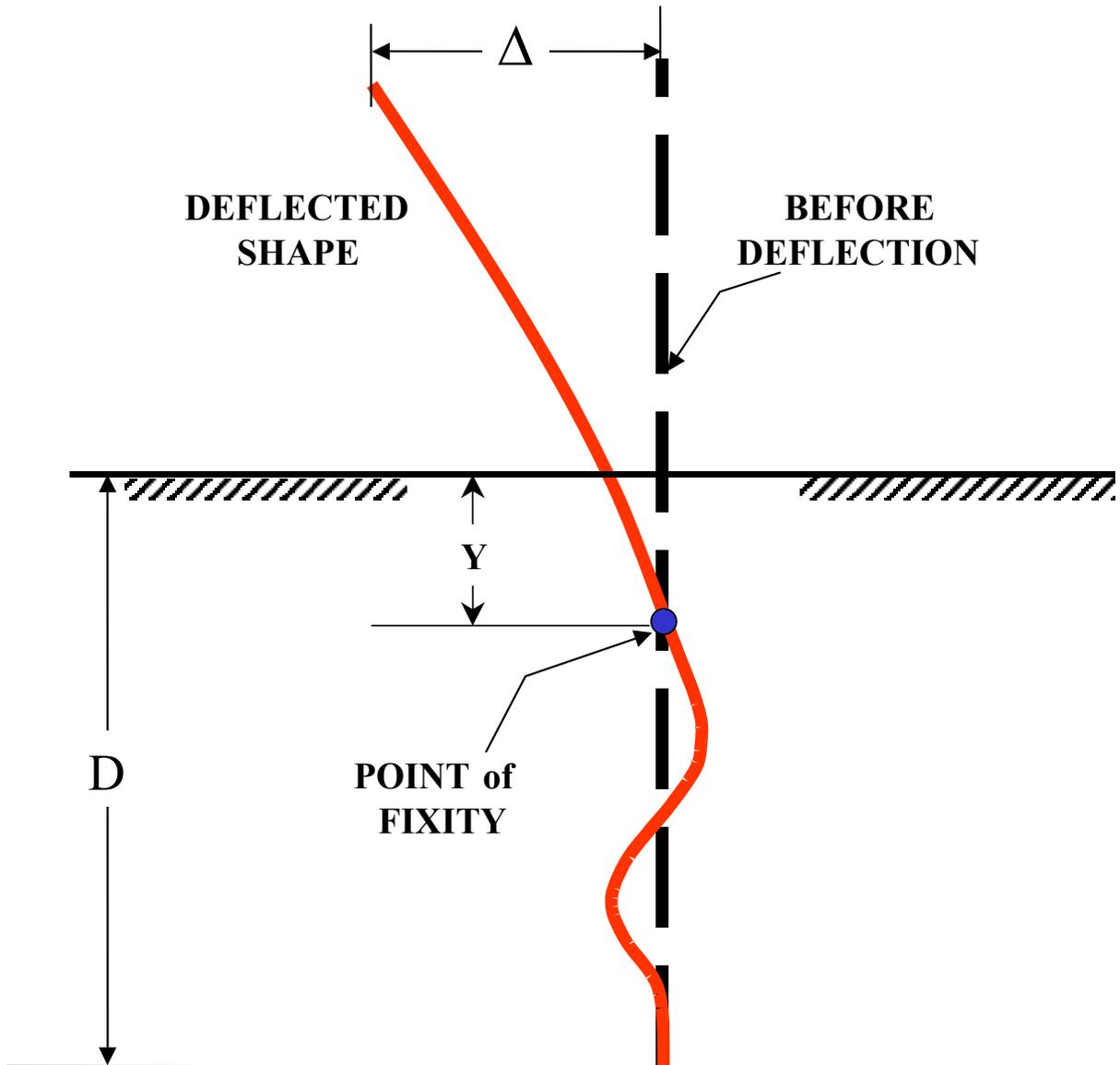
Figure 6-10. Deflected Shape for Unrestrained System

For the simple beam analysis method, one important issue that needs to be considered when calculating deflections is the Point of Fixity, or the point of zero (0) deflection, below the excavation line as shown in Figure 6-10. 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. A greater value may be used for loose sand or soft clay. It should be noted that the simple beam method of analysis alluded to above is only approximate.

6.4 *SOIL PRESSURE DISTRIBUTION FOR LAYERED SOIL*

For a shoring system in layered soils it is very important to develop appropriate soil pressure distribution for each individual soil layer as shown in Figure 6-11.
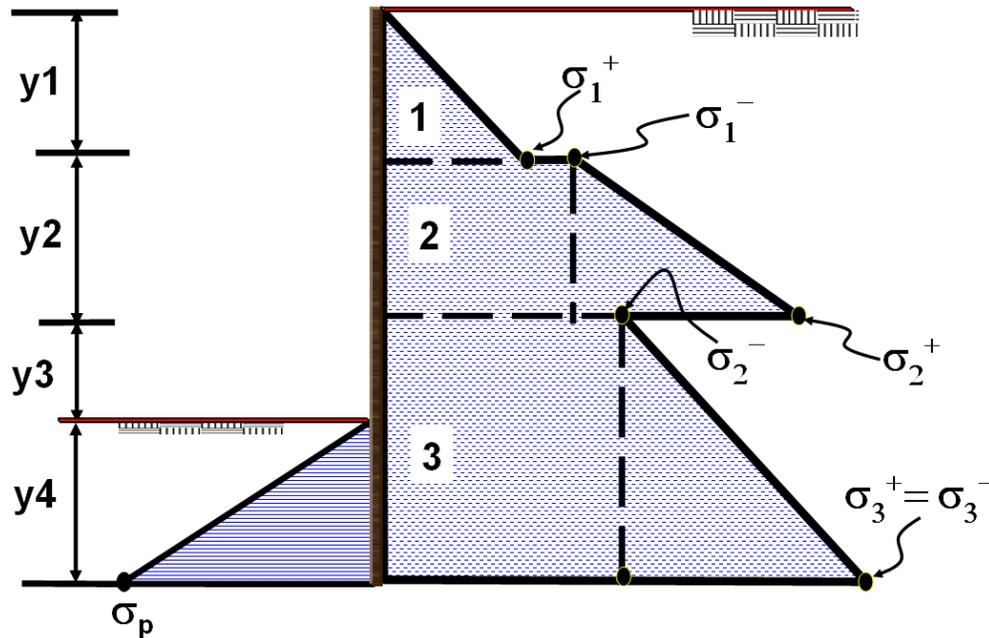

Figure 6-11. Multilayer soil pressure

The following procedure is used for the check of a Cantilever wall (see Figure 6-3):

1. Calculate Active/Passive Earth Pressure to an arbitrary point, O, at the distance, D_O, below the excavation line.

2. Take a moment about Point O to eliminate Force R and determine embedment depth D_O.

3. Increase D_O by 20 percent ($D = 1.2D_O$)

4. Calculate R by summation of forces in horizontal direction ($R \leq 0$, if R is larger than zero, increase D)

5. Calculate Maximum Bending Moment (M_{MAX}) and Maximum Shear Force (V_{MAX}) to check the vertical structural member and lagging.

6.4.1 Example 6-1 Cantilevered Soldier Pile Wall

For a shoring system subjected to the lateral load given below calculate the total required horizontal force using the Rankine earth pressure theory.
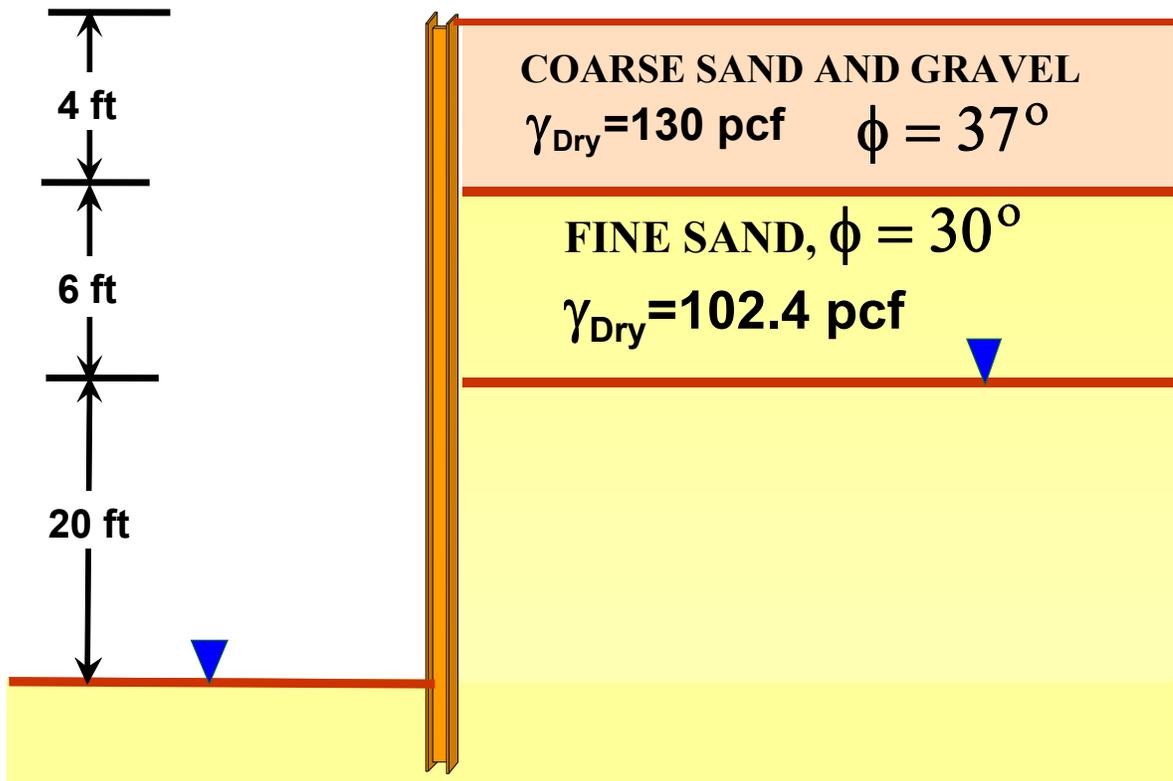

4 ft

6 ft

20 ft

COARSE SAND AND GRAVEL
γ_{Dry}=130 pcf $\phi = 37^o$

FINE SAND, $\phi = 30^o$
γ_{Dry}=102.4 pcf

Figure 6-12. Example 6-1

Solution:

- Calculate and plot earth pressure distribution.

- Calculate the total force on the shoring system.

$$K_{a1} = \tan^2\left(45 - \frac{\phi}{2}\right) = \tan^2\left(45 - \frac{37}{2}\right) = 0.249$$

$$K_{a2} = \tan^2\left(45 - \frac{\phi}{2}\right) = \tan^2\left(45 - \frac{30}{2}\right) = 0.333$$
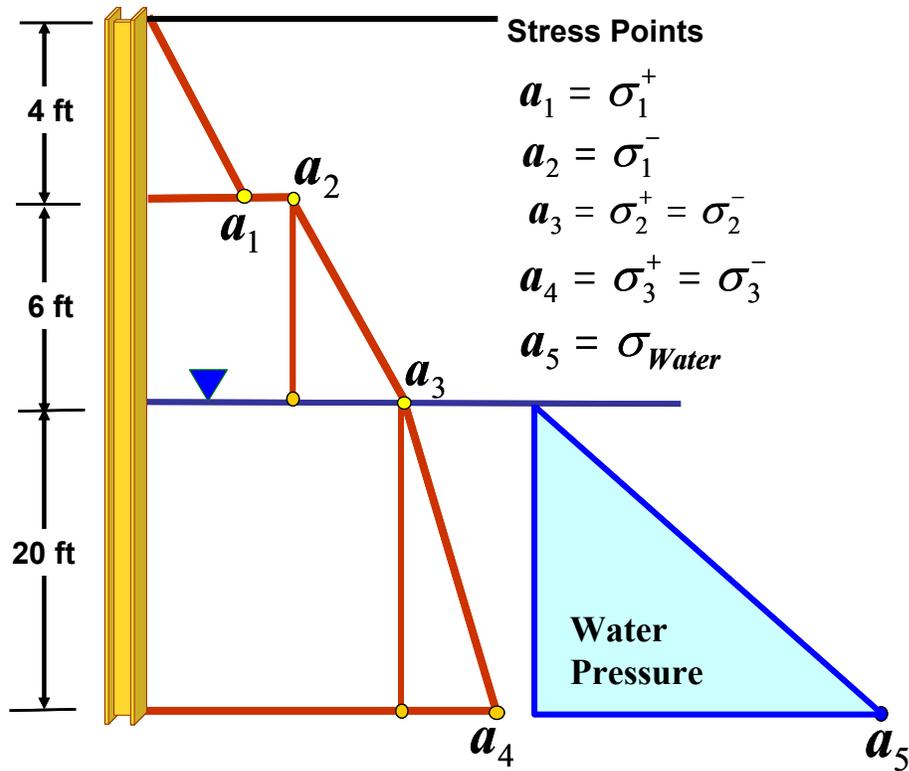
Figure 6-13. Pressure Loading Diagram

In the figure 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 Ka of that soil. The superscripted – refers to the stress at the indicated soil layer for the material above the layer line based on the Ka of the soil below the layer line.

$$\sigma_1^+ = (130\,\text{pcf})(4\,\text{ft})(0.249) = 129.48\,\text{psf}$$
$$\sigma_1^- = (130\,\text{pcf})(4\,\text{ft})(0.333) = 173.16\,\text{psf}$$
$$\sigma_2^+ = 173.16 + (102.40\,\text{pcf})(6\,\text{ft})(0.333) = 377.76\,\text{psf}$$
$$\sigma_2^+ = \sigma_2^- = 377.76\,\text{psf}$$
$$\sigma_3^+ = \sigma_3^- = 377.76 + (102.40 - 62.40)(20)(0.333) = 644.16\,\text{psf}$$

Water Pressure
$$\sigma_{a5} = 20(62.4\,\text{pcf}) = 1{,}248.0\,\text{psf}$$

DRIVING FORCES:

$$F_1 = \frac{1}{2}(4\,\text{ft})(129.48\,\text{psf}) = 258.96 \text{ lb/ft}$$

$$F_2 = (6\,\text{ft})(173.16\,\text{psf}) = 1,038.96 \text{ lb/ft}$$

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

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

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

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

THE NET FORCES:

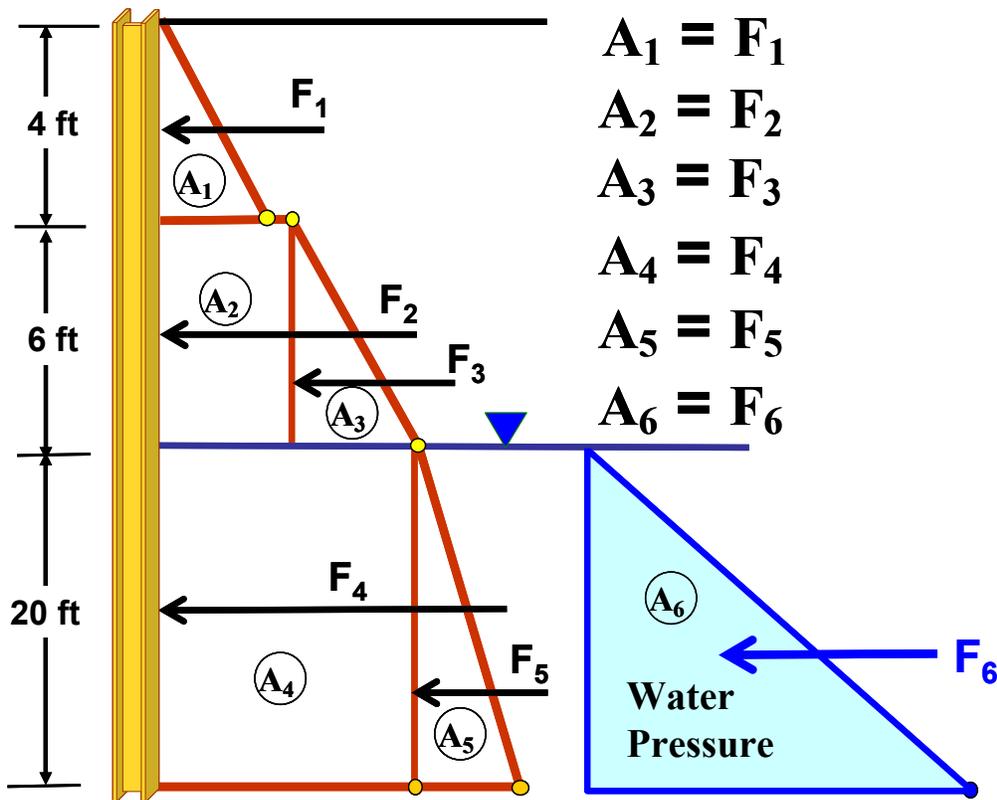$$F_{TOTAL} = 24,610.92 \text{ lb/ft}$$

Figure 6-14. Force Loading Diagram

6.4.2 Example 6-2 Cantilevered Soldier Pile Wall

Check the adequacy of the cantilevered soldier pile wall in granular layered-soil with negative slope in the front of the wall. The soldier pile is an HP12x84 steel beam placed in a 2 feet diameter hole filled with 4 sack concrete.

Pile Spacing = 6 ft

2 ft Surcharge Load

10 ft

1.6

1

D

$\phi = 34$
$\delta = 0$
$\gamma = 120 \text{ pcf}$

$\phi = 36$
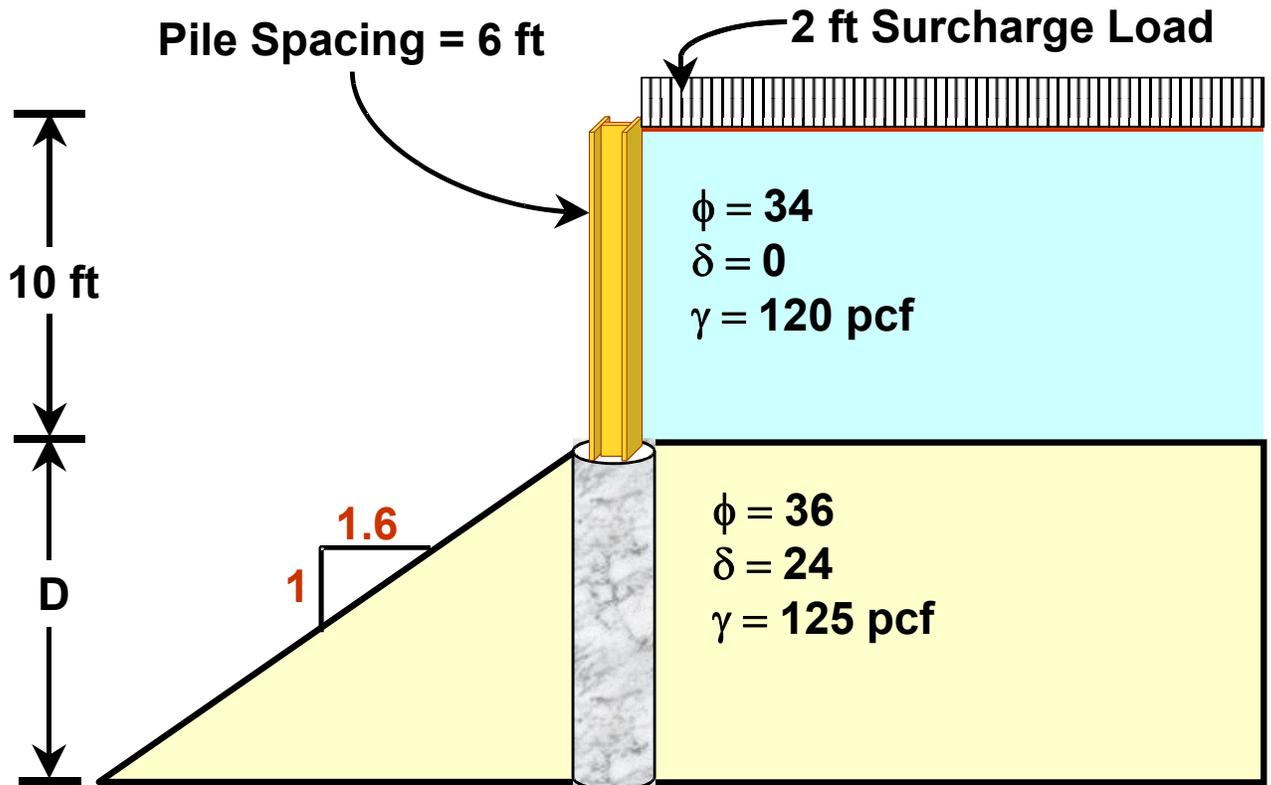$\delta = 24$
$\gamma = 125 \text{ pcf}$

Figure 6-15. Soldier pile with sloping ground Example 6-2

For a factor of safety (FS) = 1.3

Solution:

1. Active & Passive Earth Pressures.
2. Pile Embedment D.
3. Maximum Moment.

Calculate the Active & Passive Earth Pressures:

$$K_{a1} = \tan^2\left(45 - \frac{\phi}{2}\right) = \tan^2\left(45 - \frac{34}{2}\right) = 0.283$$

Use Coulomb theory to calculate active earth pressure below the dredge line.

$$K_{a2} = \frac{\cos^2\phi}{\cos\delta\left[1+\sqrt{\dfrac{\sin(\delta+\phi)\sin\phi}{\cos\delta}}\right]^2}$$

$$K_{a2} = \frac{\cos^2(36)}{\cos24\left[1+\sqrt{\dfrac{\sin(24+36)\sin36}{\cos36}}\right]^2} = 0.235$$

The passive horizontal earth pressure coefficient K_{ph} is calculated using Figure 4-37 as shown below:

- Calculate $^\delta/_\phi$: $\quad 24/36 = 0.67$.

- Calculate $^\beta/_\phi$: $\quad -32/36 = -0.89$.

- Determine K_p from Figure 4-37: $K_P = 1.65$

- Calculate reduction factor R using the ratio of $^\delta/_\phi$. $R = 0.8$

- Calculate K_{ph}:

  $K_{ph} = K_p * R * \cos(\delta) = 1.65 * 0.8 * \cos(24°) = 1.20$

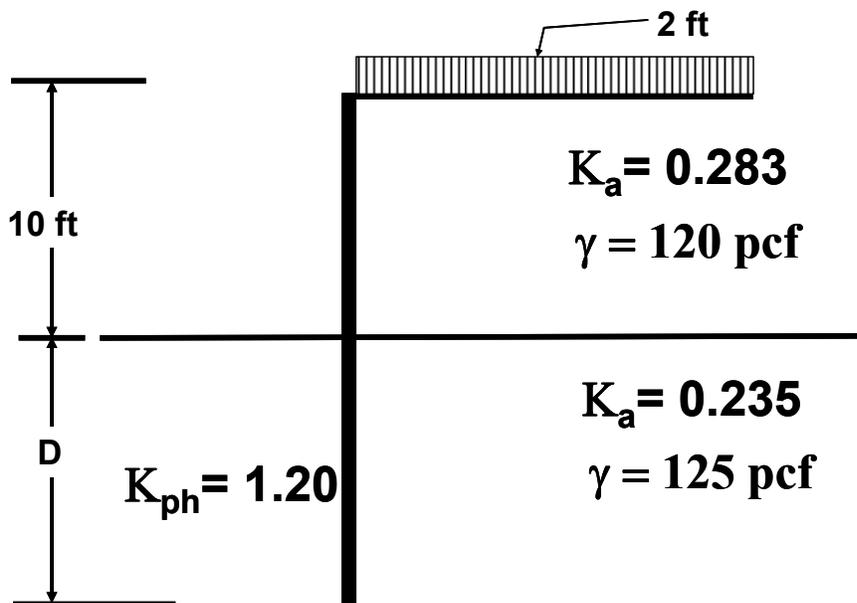

Figure 6-16. Active and Passive Earth Pressure Coefficients

Calculate earth pressure distribution:

Lateral load due to surcharge above the excavation line only:

$$\sigma_{sur} = (120)(2)(0.283) = 68 \text{ psf use 72 psf minimum (See Section 4.8.1)}$$

Lateral load distribution for the first layer:

$$\sigma^+ = 72 + (120)(10)(0.283) = 411.6 \text{ psf use } 412 \text{ psf}$$

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

$$k_{ah} = k_a \cos(\delta) = 0.235 * \cos(24) = 0.215$$

$$\sigma^- = (120)(10)(0.215) = 258.0 \text{ psf}$$

Lateral load distribution for the second layer at depth D:

$$\sigma_D = 258.0 + (125)(0.215)D = 258.0 + 26.88D \text{ psf}$$

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

$$\sigma_{pD} = (125)(1.2)D = 150.0D \text{ psf}$$

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

$$P_{As} = (72)(10) = 720 \text{ plf}$$

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

$$P_{A1} = \left[(412 - 72)\left(\frac{10}{2}\right) \right] = 1,700 \text{plf}$$

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

$$P_{A21} = 258.0 * D = 258.0D \text{ plf}$$

$$P_{A22} = \left[(26.88)(D)\left(\frac{D}{2}\right) \right] = 13.44D^2 \text{ plf}$$

Calculate passive earth pressure for the second soil layer P_P:

$$P_P = \left[(150)(D)\left(\frac{D}{2}\right) \right] = 75.0D^2 \text{ plf}$$

Because the pile spacing is equal to 3 times the effective width of the pile, the soldier pile wall can be analyzed in the same manner as a sheet pile wall.
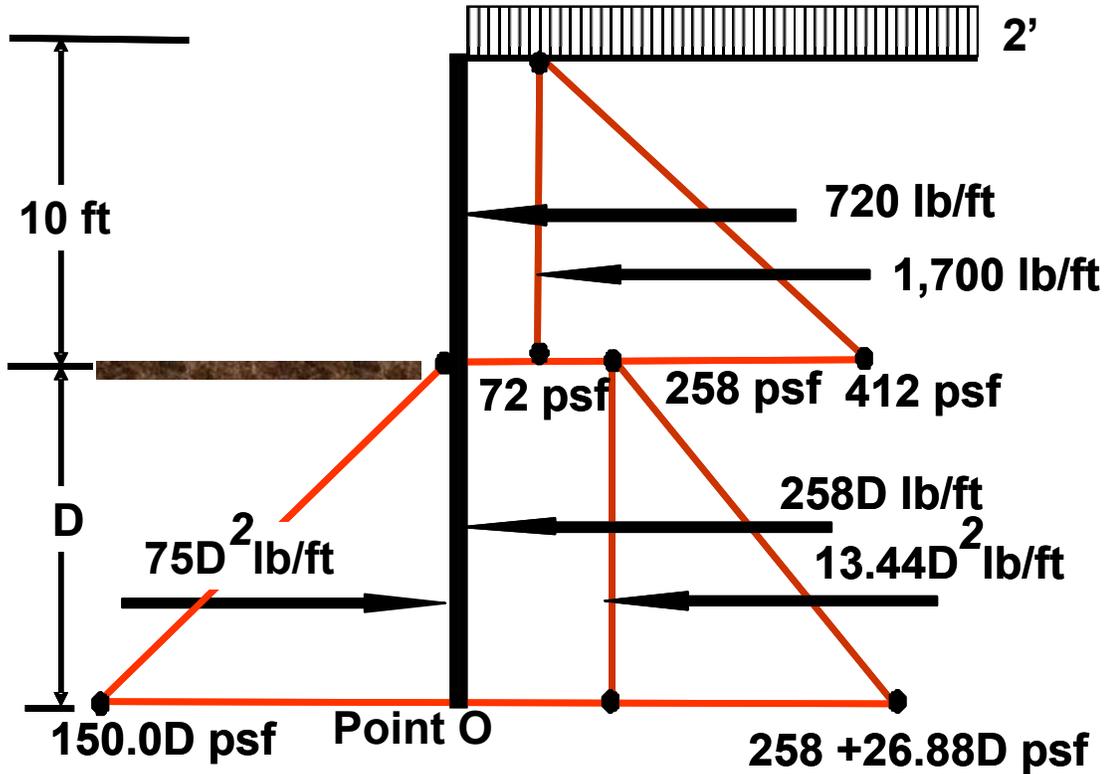
Figure 6-17. Pressure Diagram

Calculate Driving Moment (M_{DR}) and Resisting Moment (M_{RS}) about Point O.

Driving Force = Pa x Spacing	Arm (ft)	Driving Moment M_{DR}
720*6= 4,320	5+D	4,320D+21,600
1,700*6 = 10,200	10/3+D	10,200D+34,000
258*6 =1,548D	D/2	$774D^2$
$13.44D^2$*6 = 80.64 D^2	D/3	$26.88D^3$
Resisting Force = Pp x Spacing	Arm (ft)	Resisting Moment M_{RS}
$75D^2$*6 = 450 D^2	D/3	$150D^3$

$$M_{DR} = 26.88\ D^3 + 774\ D^2 + 14{,}520\ D + 55{,}600$$

$$M_{RS} = 150\ D^3$$

Calculate embedment depth using a factor of safety (FS) equal to 1.3.

$$FS = \left[\frac{M_{RS}}{M_{DR}}\right] = 1.3$$

$$\left(\frac{150}{1.3}\right)D^3 - 26.88D^3 - 774.0D^2 - 14,520.0D - 55,600.0 = 0.0$$

$$D^3 - 8.75D^2 - 164.06D - 628.22 = 0.0 \Rightarrow D = 19.07 \text{ ft}$$

Increase D by 20%.

19.07*1.2= 22.88 ft   Use D=23.0 ft

Calculate Maximum Moment.

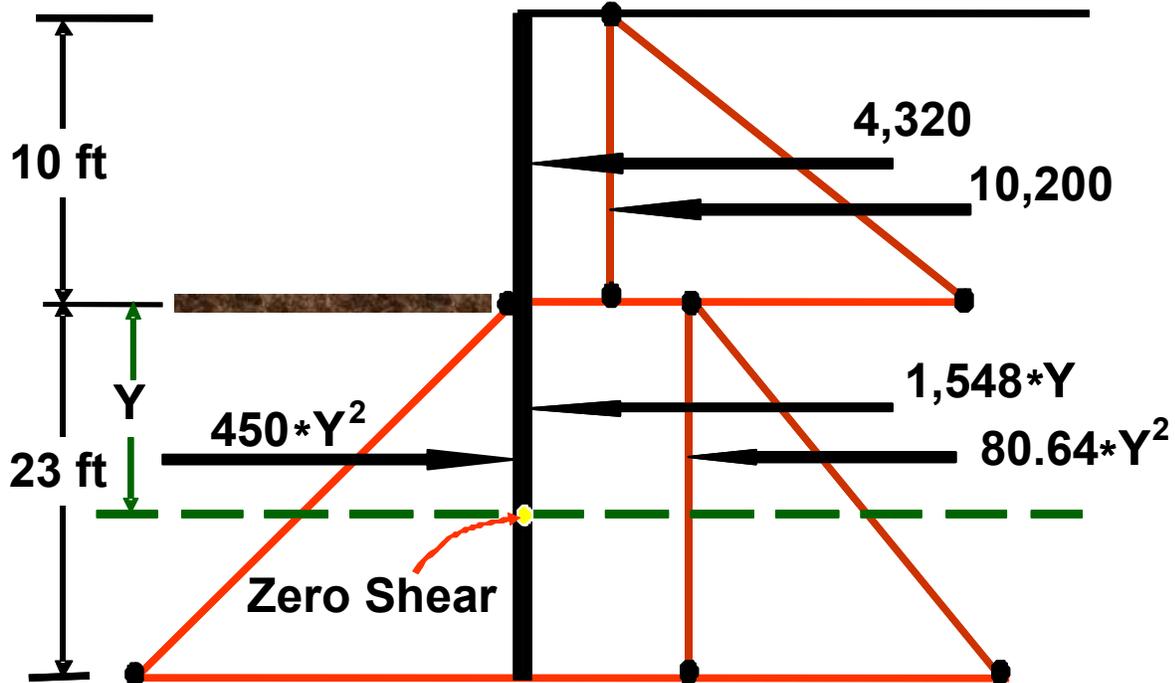
Figure 6-18. Location of Zero Shear and Maximum Moment

$$450Y^2 - 80.64Y^2 - 1548Y - 14,520 = 0.0$$

$$Y^2 - 4.19Y - 39.31 = 0.0$$

Y = 8.71 ft. Below the dredge line.

$$M_{max} = 4,320.00(5 + 8.71) + 10,200.00\left(\frac{10}{3} + 8.71\right) + 1,548.00(8.71)\left(\frac{8.71}{2}\right) +$$

$$80.64\left(8.71^2\right)\left(\frac{8.71}{3}\right) - 450.00\left(8.71^2\right)\left(\frac{8.71}{3}\right)$$

$$M_{max} = 159,430 \ lb - ft = 159.43 \ k - ft.$$

$$F_b = 0.66F_y = 0.66(36) = 23.67 \ ksi$$

$$S_{required} = \frac{M_{max}}{F_b} = \frac{159.43 * 12}{23.67ksi} = 80.83 \ in^3 < 106 \ in^3 \ \therefore \ ok$$

6.4.3 Example 6-3 Deflection of a Cantilevered Soldier Pile Wall

The calculation to determine the deflected shape follows. It is noted that there is no specification that limits the deflection of a shoring system. See Table 8-1 for specific Railroad limitations on the deflection of shoring systems. It is essential that the Engineer exercise good engineering judgment when checking a shoring submittal for deflection.

The Engineer is also reminded that the method described below yields only approximate deflections. If the shoring system is adjacent to a Railroad or other high risk structure then a more rigorous approach may be necessary. See Section 6.3 DEFLECTION, Section 7.3.3 Deflection, and Section 8.3 DEFLECTION CALCULATION for more information.

To determine the deflected shape, it will be necessary to plot the shear and moment diagrams. Also, the unfactored Depth D_o needs to be based on the driving moment equaling the resisting moment: $M_{DR} = M_{RS}$. From above:

$$150D_O{}^3 - 26.88D_O{}^3 - 774D_O{}^2 - 14{,}520D_O - 55{,}600 = 0$$
$$123.12D_O{}^3 - 774D_O{}^2 - 14{,}520D_O - 55{,}600 = 0$$
$$D_O{}^3 - 6.29D_O{}^2 - 117.93D_O - 451.59 = 0$$
$$D_O \approx 15.66 \text{ ft}$$

Develop the loading diagram based on combined active and passive pressures below the excavation line:

Determine the slope of Line FCG:

$$S_{FCG} = (150D_O - 26.88D_O)(6') = 738.72D_O$$

Determine distance y to max shear below the excavation line:
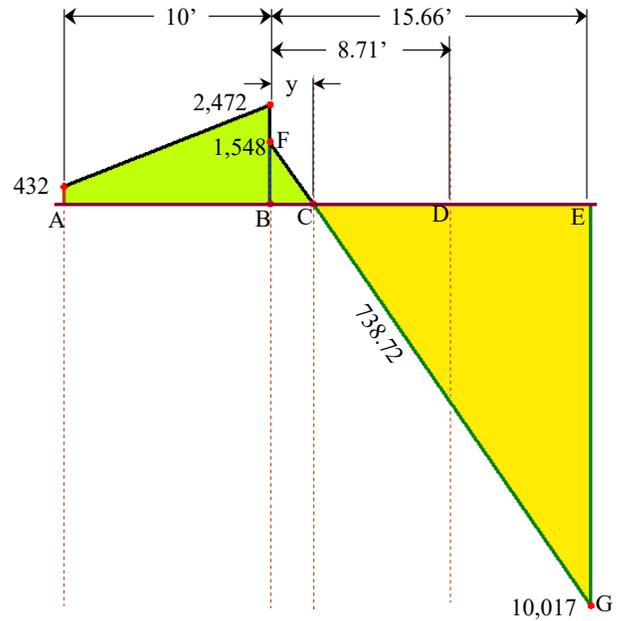
$$y = \frac{1548}{738.72} = 2.1\,ft$$

Figure 6-19. Loading Diagram

Determine the negative shears at:

Point B:

$$V_B = \frac{1}{2}(432 + 2472)(10') = 14,520 \text{ lbs}$$

Point C: (Max negative shear)

$$V_C = 14,520 + \frac{1}{2}(1548)(2.1') = 16,145 \text{ lbs}$$

Determine positive shear at Point E:

$$V_E = \frac{1}{2}(738.72)(8.71'-2.1'+15.66'-2.1')$$
$$(15.66'-8.71') = 51,777 \text{ lbs}$$
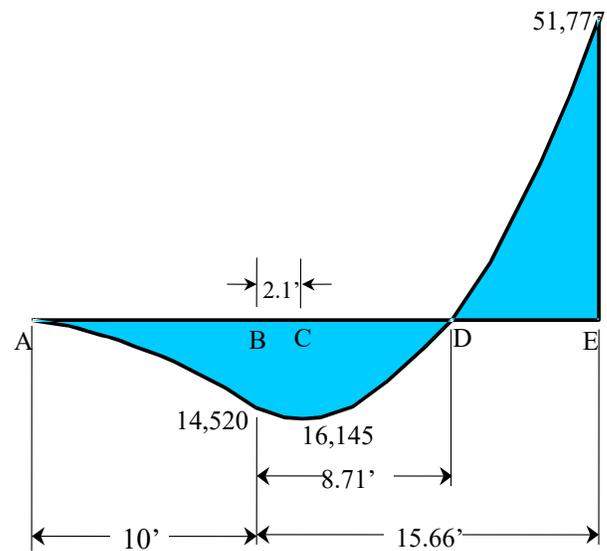
Maximum shear in beam is at depth $D_O = 15.66$ ft.

Figure 6-20. Shear Diagram

Draw the moment Diagram

From the Loading Diagram:

Determine Moment at Point B:

$$M_B = \begin{bmatrix} (432)(10') \left(\dfrac{10'}{2} \right) + \\ \dfrac{1}{2}(2{,}472 - 432)(10') \left(\dfrac{10'}{3} \right) \end{bmatrix}$$

$$M_B = 55{,}600 \text{ lb} - ft$$

From the Shear diagram:

Determine Moment at Point C:

$$M_C = 55{,}600 + (14{,}520)(2.1') +$$
$$\tfrac{2}{3}(16{,}145 - 14{,}520)(2.1') = 88{,}367 \text{ lb} - ft$$

Determine Moment at Point D:

$$M_D = 88{,}367 + \tfrac{2}{3}(16{,}145)(8.71' - 2.1') = 159{,}513 \text{ lb} - ft$$

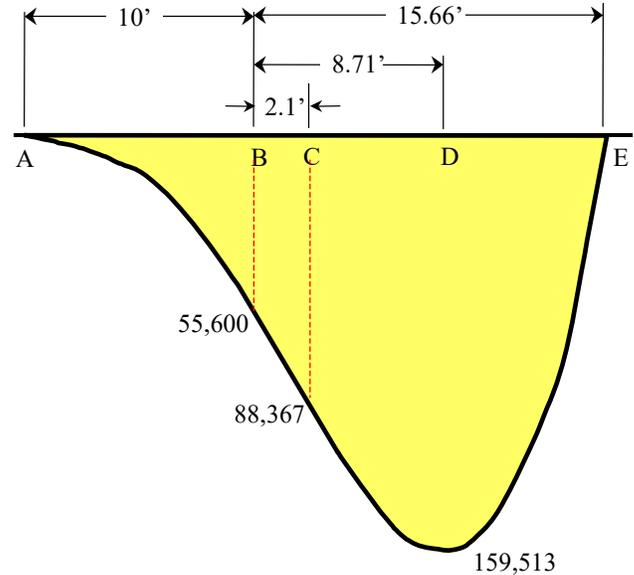NOTE: M_D is the maximum moment and it does differ slightly from that calculated above.

Figure 6-21. Moment Diagram

Determine the deflected shape of the beam:

Determine the depth to Point of Fixity (PoF) below excavation line. (See Figure 8-11.)

$$\text{PoF} = (0.25)(D_o) = (0.25)(15.66') = 3.91'$$

Determine δ_C.

First, calculate Moment at B, 1.81' beyond Max. Neg. Shear (i.e. 13.91'–12.1'):

Determine Shear at B:

$$V_B = V_{12.1} - \tfrac{1}{2}(738.72)(1.81')(1.81')$$

$$V_B = 16{,}145 - 1{,}210 = 14{,}935 \text{ lbs}$$

Determine Moment at B:

$$M_B = 88{,}367 + (14{,}935)(1.81') + \tfrac{3}{4}(16{,}145 - 14{,}935)(1.81') = 116{,}892 \text{ ft - lbs}$$
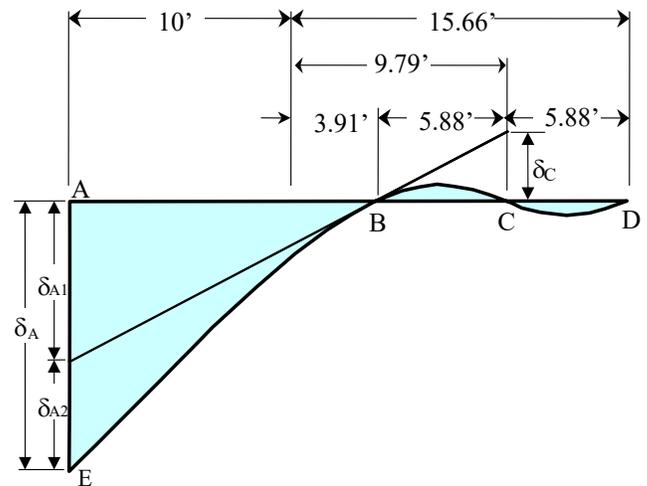

Figure 6-22. Deflected Shape

Next, in Figure 6-22, point C is assumed to act at half the distance between the PoF and the tip of the pile. This assumption appears to bring the ultimate results to a more realistic value.

Next, calculate Moment at C, 1.08 ft beyond maximum moment point (i.e.19.79'-18.71'):

Figure 6-23. Moment Area diagram

Determine Shear at C: (Ref. Figure 6-19 and Figure 6-20)

$$V_C = \left\{ \begin{array}{l} \dfrac{1}{2}(738.72) \\ [(8.71'-2.1')+(9.79'-2.1')](1.08') \end{array} \right\}$$

$V_C = 5{,}704 \text{ lbs}$

Determine Moment at C: (Ref. Figure 6-21)

$$M_C = (159{,}513) - \frac{1}{3}(5{,}704)(1.08') = 157{,}459 \text{ ft - lbs}$$

Using the developed moment area diagram in Figure 6-23 Calculate δ_C due to moment area C to B: (i.e. Take moments about C.) (Ref. Figure 6-22)

$$\delta_C = \left[\begin{array}{l} (116{,}892)(4.8')\left(\dfrac{4.8'}{2}+1.08'\right)+\dfrac{3}{4}(159{,}513-116{,}892)(4.8')\left(\dfrac{3}{7}4.8'+1.08'\right)+ \\ (157{,}459)\left(\dfrac{1.08^2}{2}\right)+\dfrac{3}{4}(159{,}513-157{,}459)(1.08')\left(\dfrac{3}{5}\right)(1.08') \end{array} \right]\left(\dfrac{1728}{(650)(3E7)}\right)$$

$$\delta_C = (1{,}952{,}534+481{,}350+91{,}830+1{,}078)\left(\frac{1728}{(650)(3E7)}\right) = 0.224 \text{ in}$$

Note the Moment of Inertia of soldier beam HP12x84 is 650 in[4].

Calculate δ_{A1} due to slope of tangent line at Point B. (See Figure 8-13.) (Ref. Figure 6-22)

$$\delta_{A1} = \delta_C\left(\frac{13.91'}{5.88'}\right) = (0.224)\left(\frac{13.91'}{5.88'}\right) = 0.530 \text{ in}$$

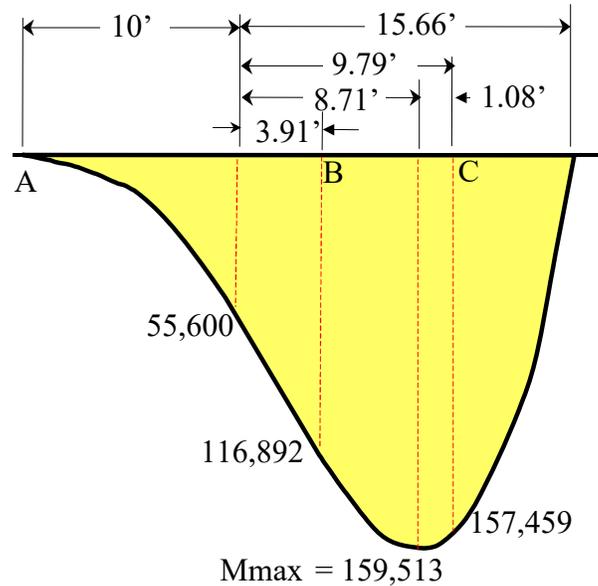Calculate δ_{A2} due to moment area A to B: (e.g. Take moment about A.)

Note that for this calculation, the combined developed moment area diagram in Figure 6-23 will not be used. Instead separate moment area diagrams for the surcharge load and for the active and passive pressures will be created as shown in Figure 6-24. The latter method is used for additional accuracy because there is approximately an 11% error when using the combined developed moment area diagram as compared to separate moment area diagrams. Table 6-3 shows the calculations that use Figure 6-24.

Figure 6-24. Redeveloped shear and moments diagrams
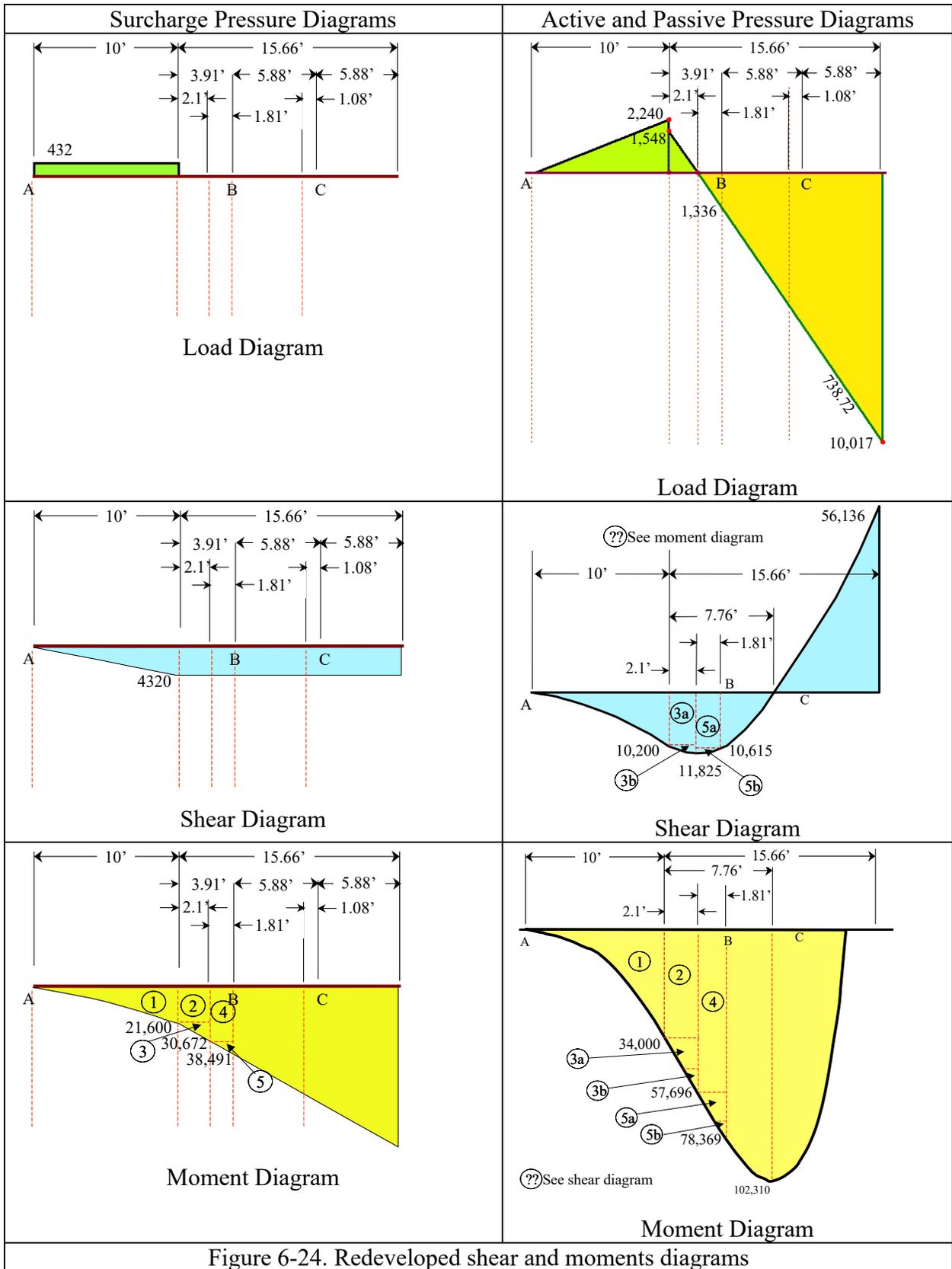
For location 3a use triangular shape based on 10,200 * 2.1 = 21,420.

For location 3b use 4th degree curve shape based on $\frac{2}{3}(1,625)(2.1) = 2,275$.

For location 5a use triangular shape based on 10,615 * 1.81 = 19,213.

For location 5b use 4th degree curve shape based on $\frac{2}{3}(1,210)(1.81) = 1,460$.

Table 6-3. Calculations for deflection

Loc	Area	Moment Arm	Area Moment	Loc	Area	Moment Arm	Area Moment
①	$\frac{1}{3}(21,600)(10)$	$\frac{3}{4}(10)$	540,000	①	$\frac{1}{4}(34,000)(10)$	$\frac{4}{5}(10)$	680,000
②	$(21,600)(2.1)$	$10+\left(\frac{2.1}{2}\right)$	501,228	②	$(34,000)(2.1)$	$10+\left(\frac{2.1}{2}\right)$	788,970
③	$\frac{1}{2}(9,072)(2.1)$	$10+\left(\frac{2}{3}\right)2.1$	108,592	③a	$\frac{1}{2}(21,420)(2.1)$	$10+\left(\frac{2}{3}\right)2.1$	256,397
				③b	$\frac{1}{4}(2,275)(2.1)$	$10+\left(\frac{5}{7}\right)2.1$	13,735
④	$(30,672)(1.81)$	$12.1+\left(\frac{1.81}{2}\right)$	721,990	④	$(56,558)(1.81)$	$12.1+\left(\frac{1.81}{2}\right)$	1,331,322
⑤	$\frac{1}{2}(7,819)(1.81)$	$12.1+\left(\frac{2}{3}\right)1.81$	94,161	⑤a	$\frac{1}{2}(19,213)(1.81)$	$12.1+\left(\frac{2}{3}\right)1.81$	231,373
				⑤b	$\frac{1}{4}(1,460)(1.81)$	$12.1+\left(\frac{5}{7}\right)1.81$	8,848
		Total	1,965,970			Total	3,310,645

The deflection δ_{A2} due to moment area from A to B is the summation of the totals above.

$$\delta_{A2} = (1,965,970 + 3,310,645)\left(\frac{1728}{(650)(3E7)}\right) = 0.467 \text{ in}$$

Total deflection at A:

$$\delta_A = \delta_{A1} + \delta_{A2} = 0.530 + 0.467 = 0.997 \text{ in}$$

The results above compare closely with the Caltrans Trenching and Shoring Check Program. See Figure 6-28.

Results from Caltrans Trenching and Shoring Program (Custom Module)
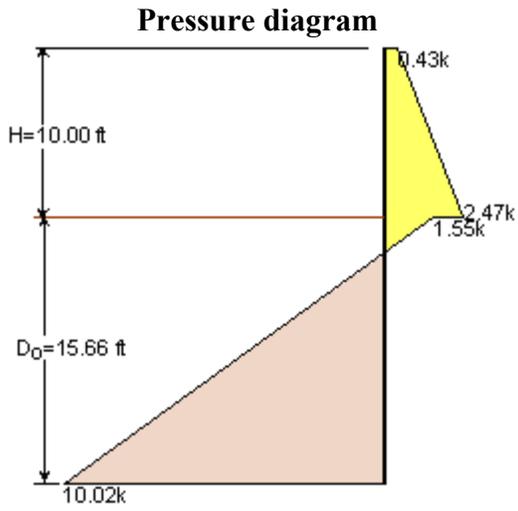
Pressure diagram

Figure 6-25. Loading Diagram

Max Shear 51.62 k

Figure 6-26. Shear Diagram

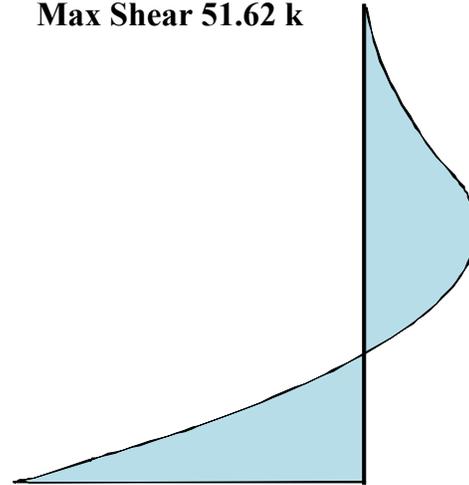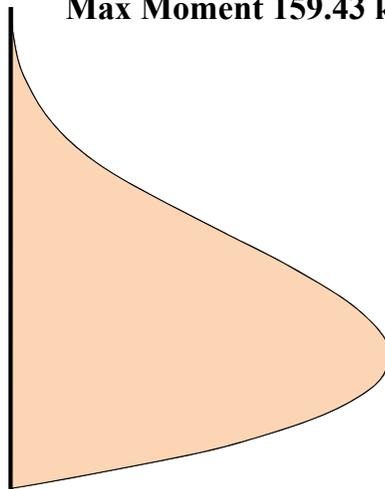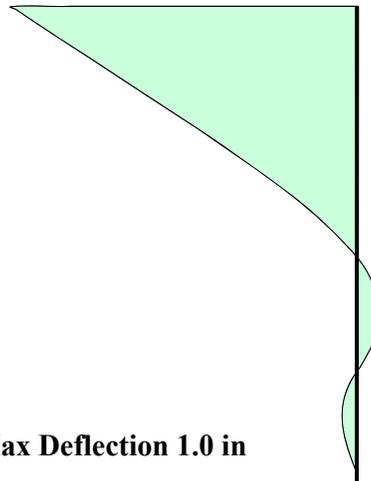**Max Moment 159.43 k-ft**

Figure 6-27. Moment Diagram

Max Deflection 1.0 in

Figure 6-28. Deflection Diagram