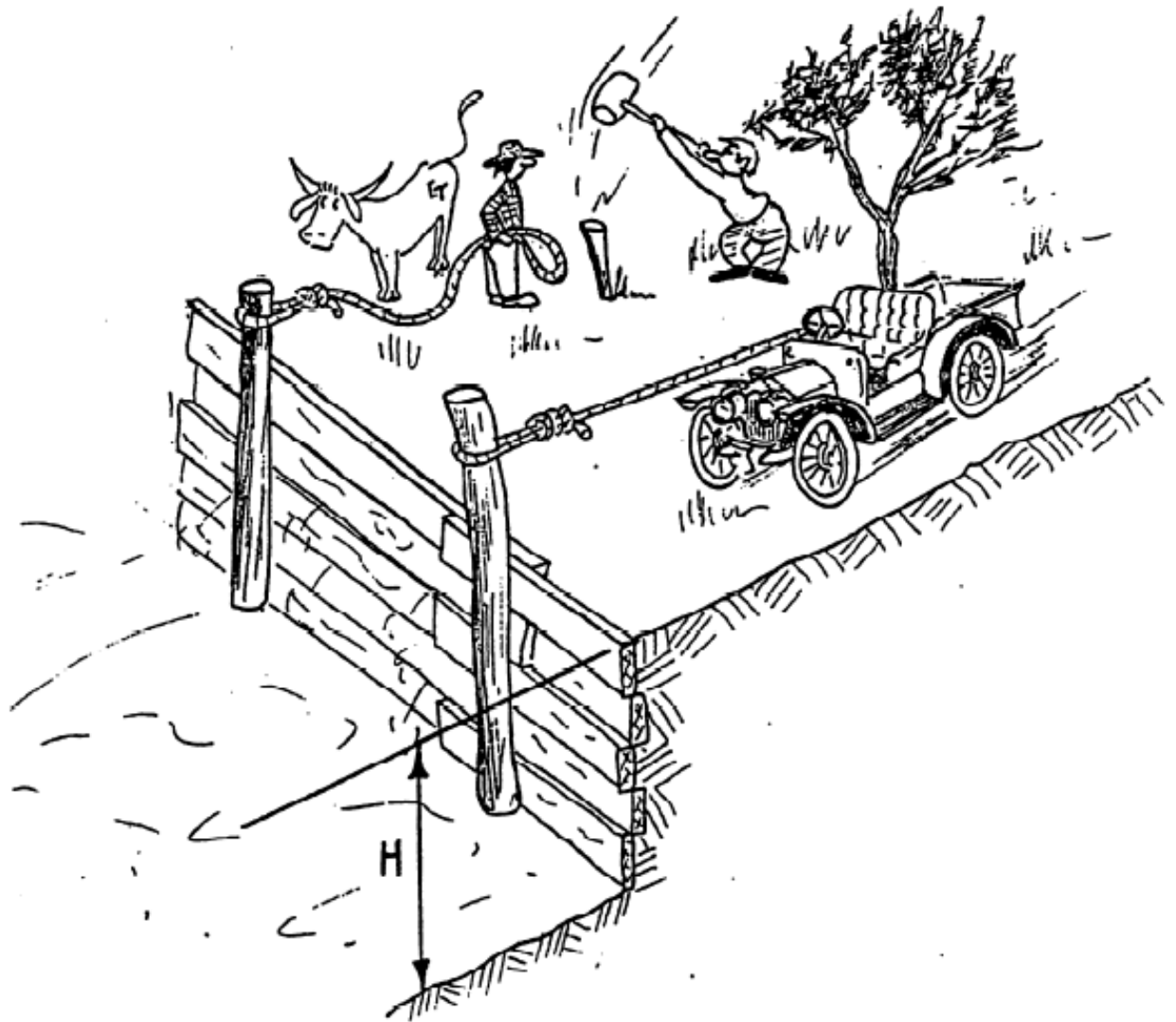


CHAPTER 7

RESTRAINED SHORING SYSTEMS



7.0 LATERAL EARTH PRESSURES FOR RESTRAINED SHORING SYSTEMS

An anchored wall includes an exposed height (H) over which soil is retained. Also, an embedded depth (D) may provide vertical and lateral support in addition to either structural anchors or ground anchors, see Figure 7-1. In developing the lateral earth pressure for braced or anchored walls, consideration must be given to the wall displacement that may affect adjacent structures and/or underground utilities.

The lateral earth pressure acting on the wall is determined by the soil type and the appropriate earth pressure theory. Generally, the earth pressure increases with depth against a wall. But for braced or tieback walls this is not the case. A trapezoidal shaped apparent earth pressure distribution needs to be developed for this type of wall system.

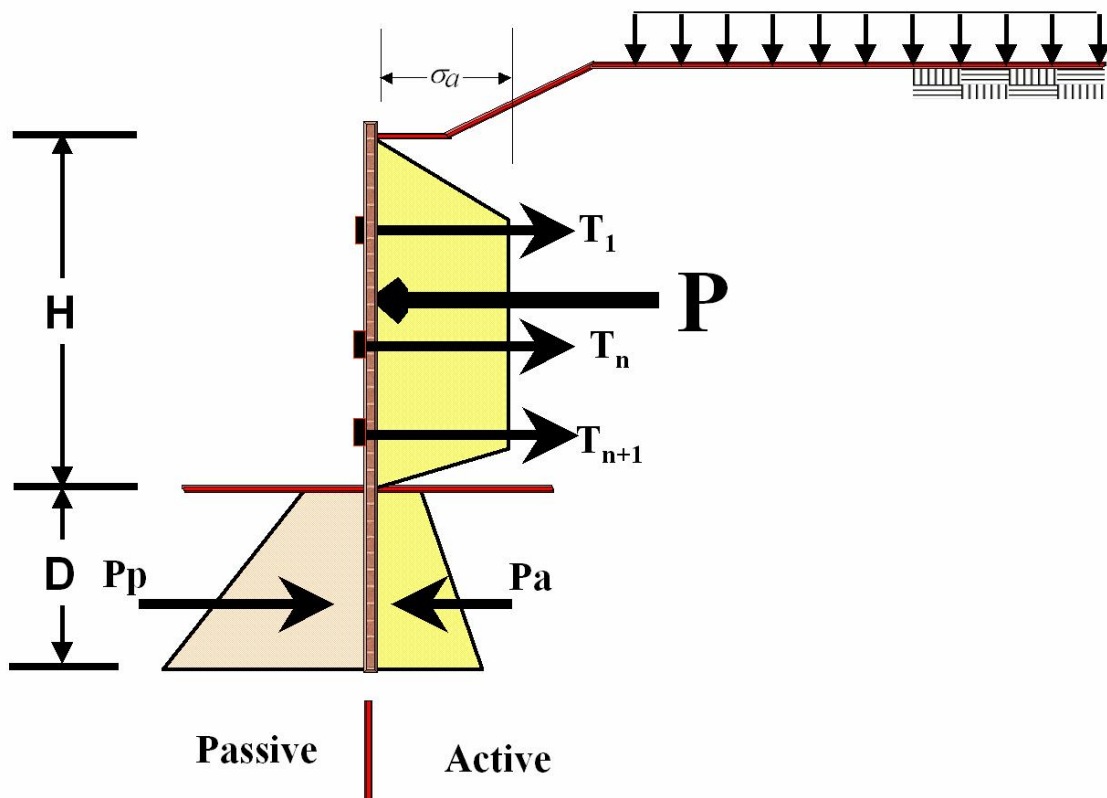


Figure 7-1. Lateral Earth Pressure for Anchored/Braced Walls

7.1 COHESIONLESS SOILS

The lateral earth pressure distribution for braced or anchored walls constructed in cohesionless soils may be determined using Figure 7-2 for single braced/tieback walls and Figure 7-3 for multiple braced/tieback walls.

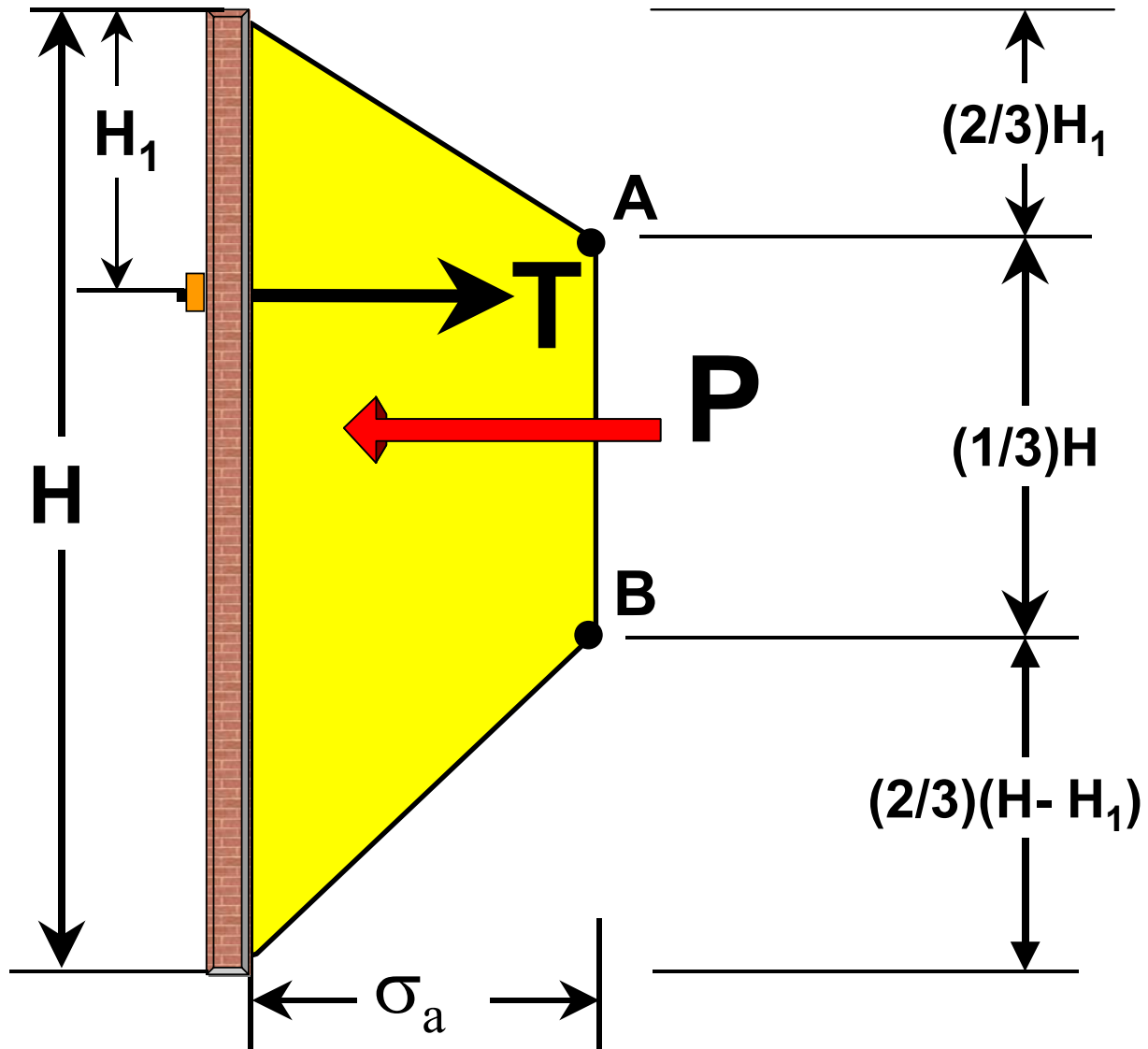


Figure 7-2. Pressure Diagram for Single Anchored/Braced Wall

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The maximum ordinate (σ_a) of the pressure diagram is determined as follows:

For walls with a single level of anchors or braces:

$$\sigma_a = \frac{f * P}{\left(\frac{2}{3}\right)H} = \frac{1.3 * P}{\left(\frac{2}{3}\right)H} \quad \text{Eq. 7-1}$$

Where the factor f is a constant to convert triangular pressure distribution to trapezoidal pressure distribution as is shown below.

Triangular pressure distribution:

$$P = \left(\frac{1}{2}\gamma\right)(h^2)(K_a)$$

Trapezoidal pressure distribution:

$$P_T = 0.65 \left[\gamma(h^2)(K_a) \right]$$

Let triangular pressure distribution equal to trapezoidal pressure distribution:

$$P_T = \left[\frac{1}{2}\gamma(h^2)(K_a) \right] f = 0.65 \left[\gamma(h^2)(K_a) \right] \Rightarrow f = \frac{0.65\gamma \left[(h^2)(K_a) \right]}{\left(\frac{1}{2}\gamma \right) \left[(h^2)(K_a) \right]} = 1.3$$

Therefore:

$$P_T = 1.3P \quad \text{Eq. 7-2}$$

The lateral active horizontal earth pressure for the multilevel anchors wall is shown in Figure 7-3:

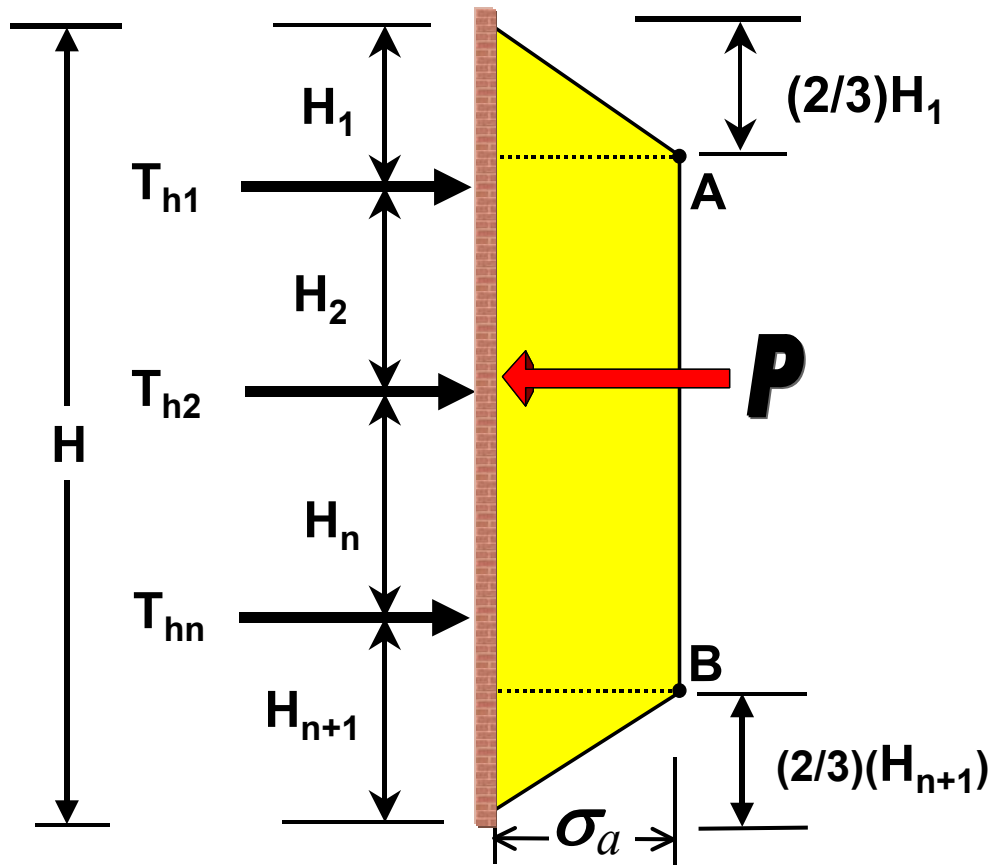


Figure 7-3. Pressure Diagram for Multi Anchored/Braced Wall

Eq. 7-3 may be used to calculate the active earth pressure for the multiple tieback wall.

$$\sigma_a = \frac{1.3P}{\left[H - \frac{1}{3}(H_1 + H_{n+1}) \right]}$$

Eq. 7-3

RESTRAINED SHORING SYSTEMS

Where:

σ_a = maximum ordinate of pressure diagram.

P = total lateral load required to be applied to the wall.

H = wall height.

H_1 = distance from ground surface at top of wall to uppermost level of anchors.

H_{n+1} = distance from the grade at bottom of a wall to lowermost level of anchors.

n = number of anchors.

T_{hn} = horizontal component of the anchor force at level n .

P_a = active lateral earth pressure below dredge line.

P_p = passive lateral earth pressure below dredge line.

7.2 COHESIVE SOILS

The lateral earth pressure distribution for cohesive soils is related to the stability number (N_s), which is defined as:

$$N_s = \frac{\gamma_s(H)}{C} \quad \text{Eq. 7-4}$$

Where:

γ_s = total unit weight of soil.

H = wall height.

C = average undrained shear strength of soil.

7.2.1 Stiff to Hard

For braced or anchored walls in stiff to hard cohesive soils with the stability number (N_s) less than or equal to 4 the lateral earth pressure may be determined using Figure 7-4, with the maximum ordinate (σ_a) of the pressure diagram determined as:

$$\sigma_a = 0.2(\gamma_s)(H) \quad \text{to} \quad 0.4(\gamma_s)(H) \quad \text{Eq. 7-5}$$

Where:

σ_a = maximum ordinate of trapezoidal pressure diagram.

γ_s = total unit weight of soil.

H = wall height.

7.2.2 Soft to Medium Stiff

The lateral earth pressure on a restrained shoring system in soft to medium stiff cohesive soils with the stability number equal to or larger than 6 may be determined, using Figure 7-4 for which the maximum ordinate (σ_a) of the pressure diagram is determined as:

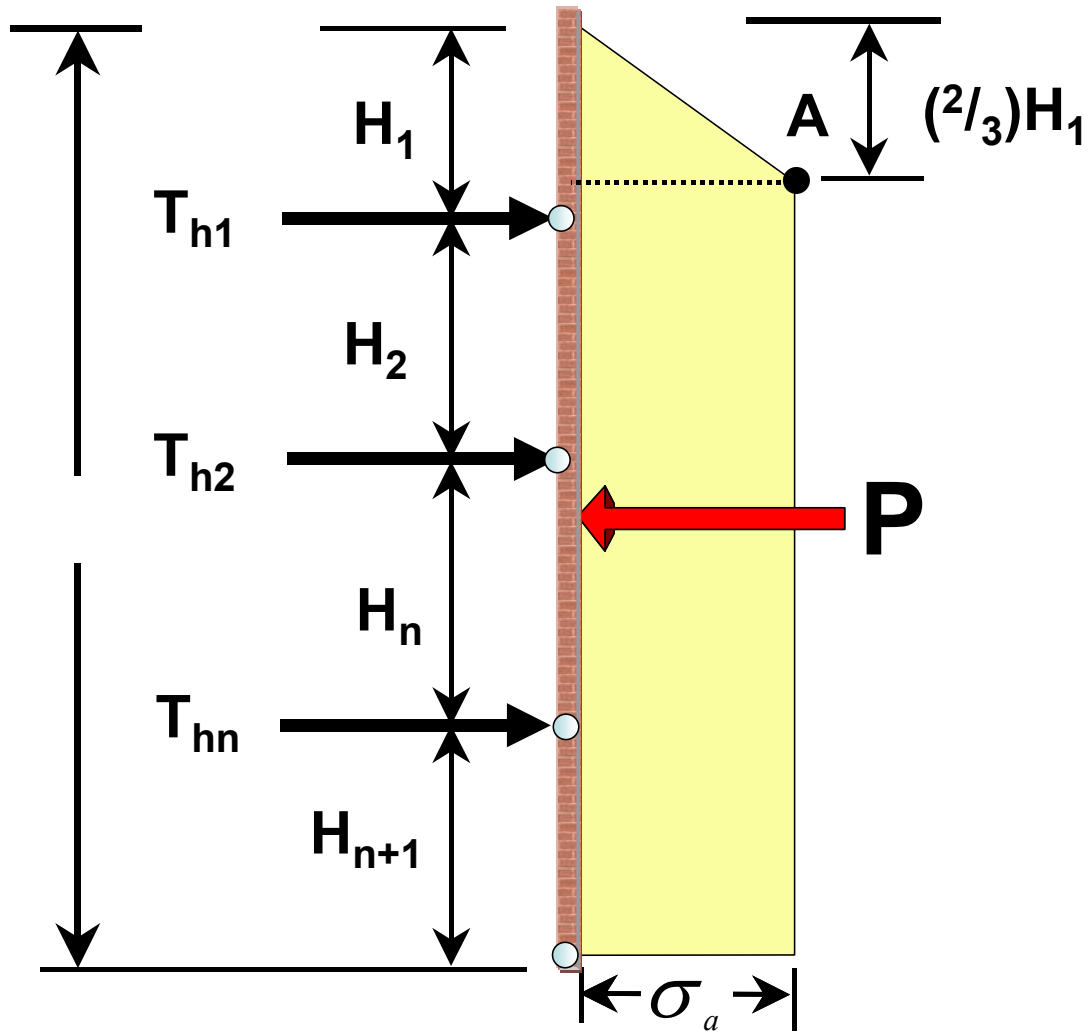


Figure 7-4. Pressure Diagram for Multi Anchored/Braced Wall for Cohesive Backfill

$$\sigma_a = (K_a)(\gamma_s)(H)$$

Eq. 7-6

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Where:

σ_a = maximum ordinate of pressure diagram.

K_a = coefficient of active lateral earth pressure.

γ_s = total unit weight of soil.

H = wall height.

The coefficient of active lateral earth pressure (K_a) may be determined using Eq. 7-7.

$$K_a = 1 - \frac{4(C1)}{(\gamma_s)(H)} + 2\sqrt{2} + \frac{D}{H} \left[\frac{1 - 5.14(C2)}{(\gamma_s)(H)} \right] \geq 0.22 \quad \text{Eq. 7-7}$$

Where:

C1 = undrained shear strength of retained soil.

C2 = undrained shear strength of soil below grade in front of wall.

γ_s = total unit weight of retained soil.

H = wall height.

D = depth from the grade in front of the wall to the potential failure surface below.

For soils with $4 < N_s < 6$, use the larger σ_a from Eq. 7-5 and Eq. 7-6.

7.3 CALCULATION PROCEDURES

7.3.1 Single Tieback/Brace System

The following procedure is used for the analysis of a Single Tieback/Brace System wall including any surcharge as shown in Figure 7-5:

1. Determine the Earth Pressure Coefficients using the classical Earth Pressure Theories described in the previous section.
2. Convert the active earth pressure above the excavation line to a trapezoidal earth pressure.
3. Take moments about the tieback to calculate embedment depth D , using a factor of safety of 1.3.
4. Take moments about the tieback to calculate embedment depth D , using a factor of safety of 1.0 to calculate tieback load T in following step 5.
5. Set summation of forces equal to zero in horizontal direction to calculate tieback/brace force T .
6. Calculate Maximum Bending Moment (M_{MAX}) and Maximum Shear Force (V_{MAX}) to analyze the vertical structural member and lagging.

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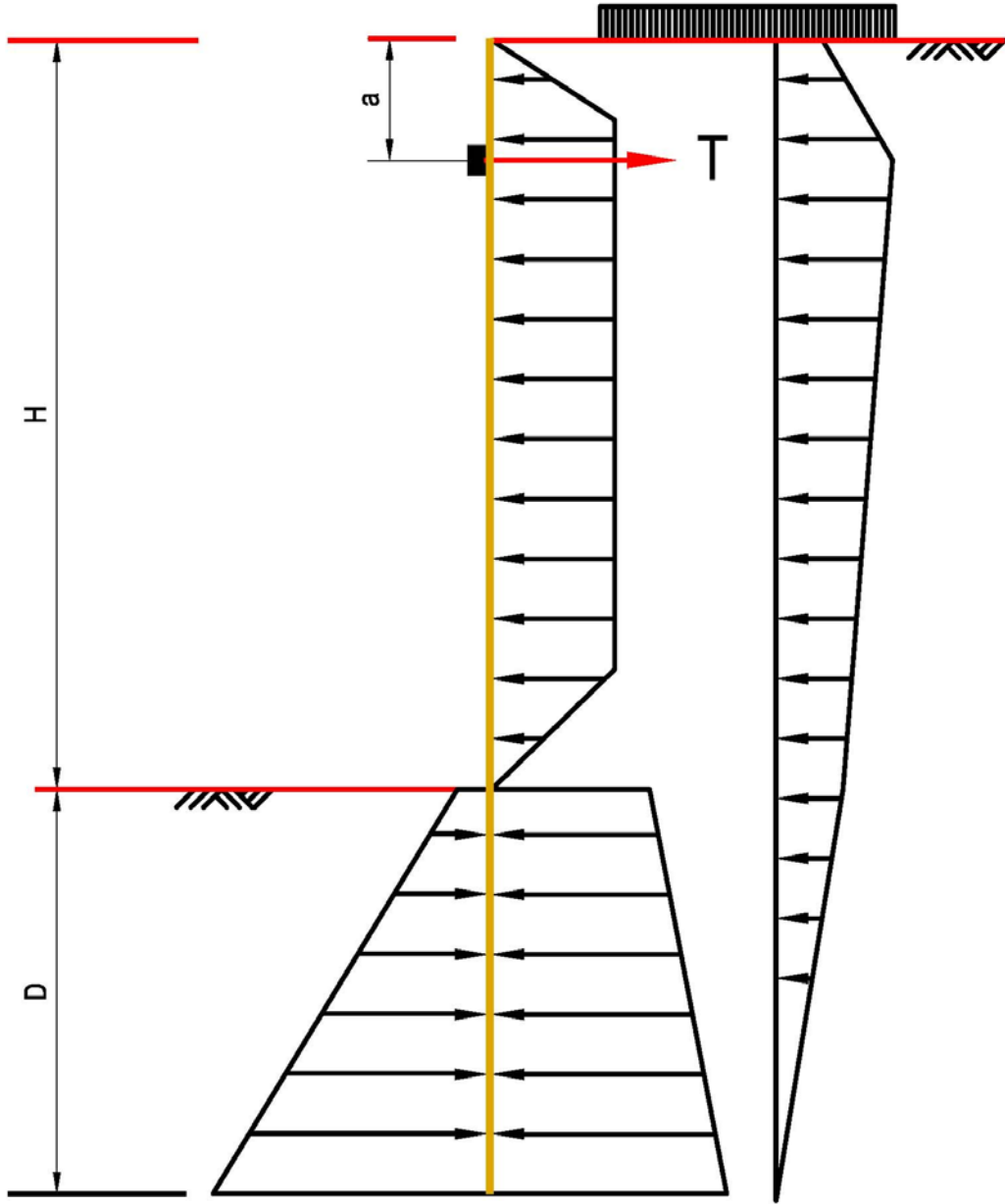


Figure 7-5. Single Tieback System

7.3.2 Multiple Tieback/Brace System

Depending on the backfill properties trapezoidal soil's pressure diagram shown in Figure 7-3 and Figure 7-4 are used for the analysis of multiple tieback systems. Figure 7-6 shows a simple trapezoidal pressure diagram for a multiple tieback system. The beam is divided into three types of spans.

- Starting Cantilever Span S_1 .
- Interior Spans S_n .
- Embedment Span S_D .

Per the FHWA the two methods used to calculate the embedment depth, D , and tieback load, T , are the Hinge Method and the Tributary Area Method. The Tributary Area Method balances only summation of forces, which results in a large moment at the tip of the pile. The Hinge Method satisfies the force and moment equilibrium in that the shear and moment equal zero at the tip of the pile. Both finite element and beam spring models show the same trend.

The detailed procedure is shown below:

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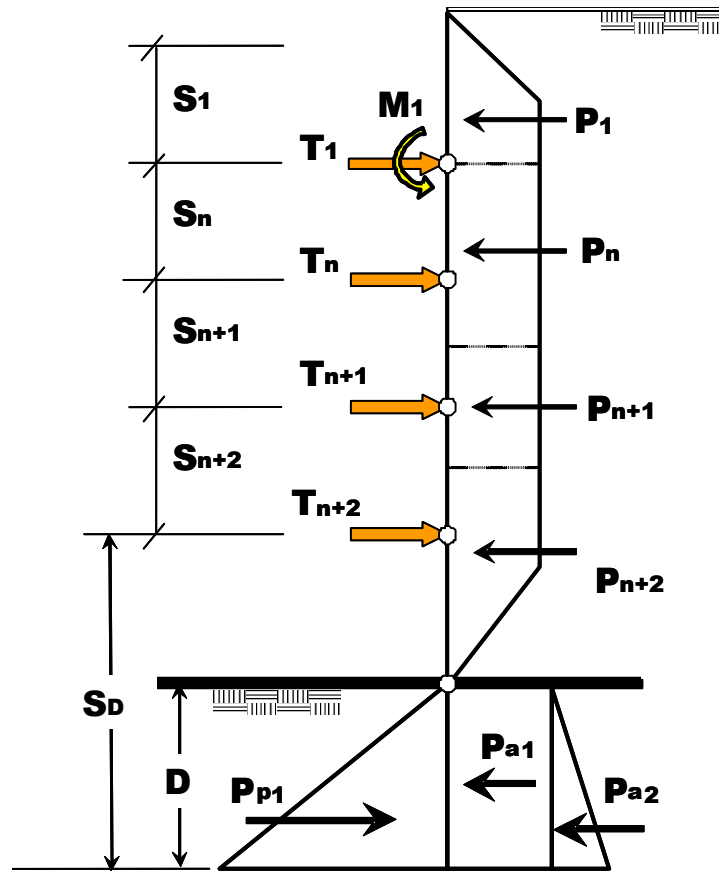


Figure 7-6. Multiple Tieback System

The Hinge Method as shown in Figure 7-6 and Figure 7-7 is used to solve multiple Tieback/Brace systems.

- Take moments M_1 about the upper level tieback due to cantilever action of the soil pressure above the upper tieback. The moments at the remaining tiebacks are assumed to be zero (0).
- Use combination of the moment M_1 and tributary area to calculate the remaining tieback loads except the last tieback load.
- Calculate last tieback load T_{n+2} .
 - Calculate embedment depth D by taking moments about the last tieback. (Set Driving Moment = Resisting Moment.)
 - Set summation of forces equal to zero in horizontal direction to calculate the last tieback load T_{n+2} .
- Take moments about the last tieback to calculate embedment depth D using a factor of safety of 1.3 for external stability.

$M_I =$ Moment Due to load P_1

$$T_{1U} = (P_1 + P_2)$$

$$T_{1L} = \left(\frac{P_3}{2} + \frac{M_1}{S_1} \right)$$

$$T_{2U} = \left(\frac{P_3}{2} - \frac{M_1}{S_1} \right)$$

$$T_{2L} = \left(\frac{P_4}{2} \right)$$

$$T_{3U} = \left(\frac{P_4}{2} \right)$$

$$T_{3L} = \left(\frac{P_5}{2} \right)$$

$$T_{4U} = \left(\frac{P_5}{2} \right)$$

$$T_{4L} = (P_6 + P_7 + P_{a1} + P_{a2} - P_{p1})$$

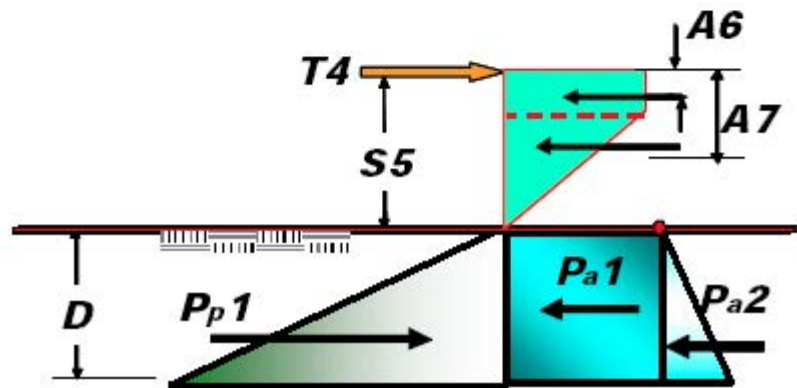
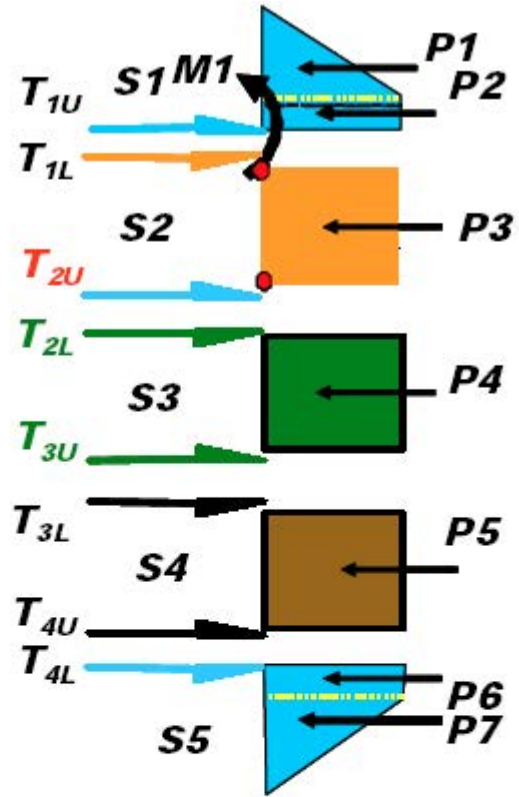


Figure 7-7. Detail Hinge Method for Tieback Analysis

7.3.3 Deflection

A general discussion of deflection for unrestrained temporary shoring systems was described in the previous chapter. The same approach applies when calculating the deflection of restrained shoring systems. For simple beam analysis, the deflection at the supports along the vertical element of the shoring system is assumed to be zero (0) as shown in Figure 7-8. The Point of Fixity varies from 0.25D to 0.8D below the excavation level and is a function of the effective pile diameter and soil type.

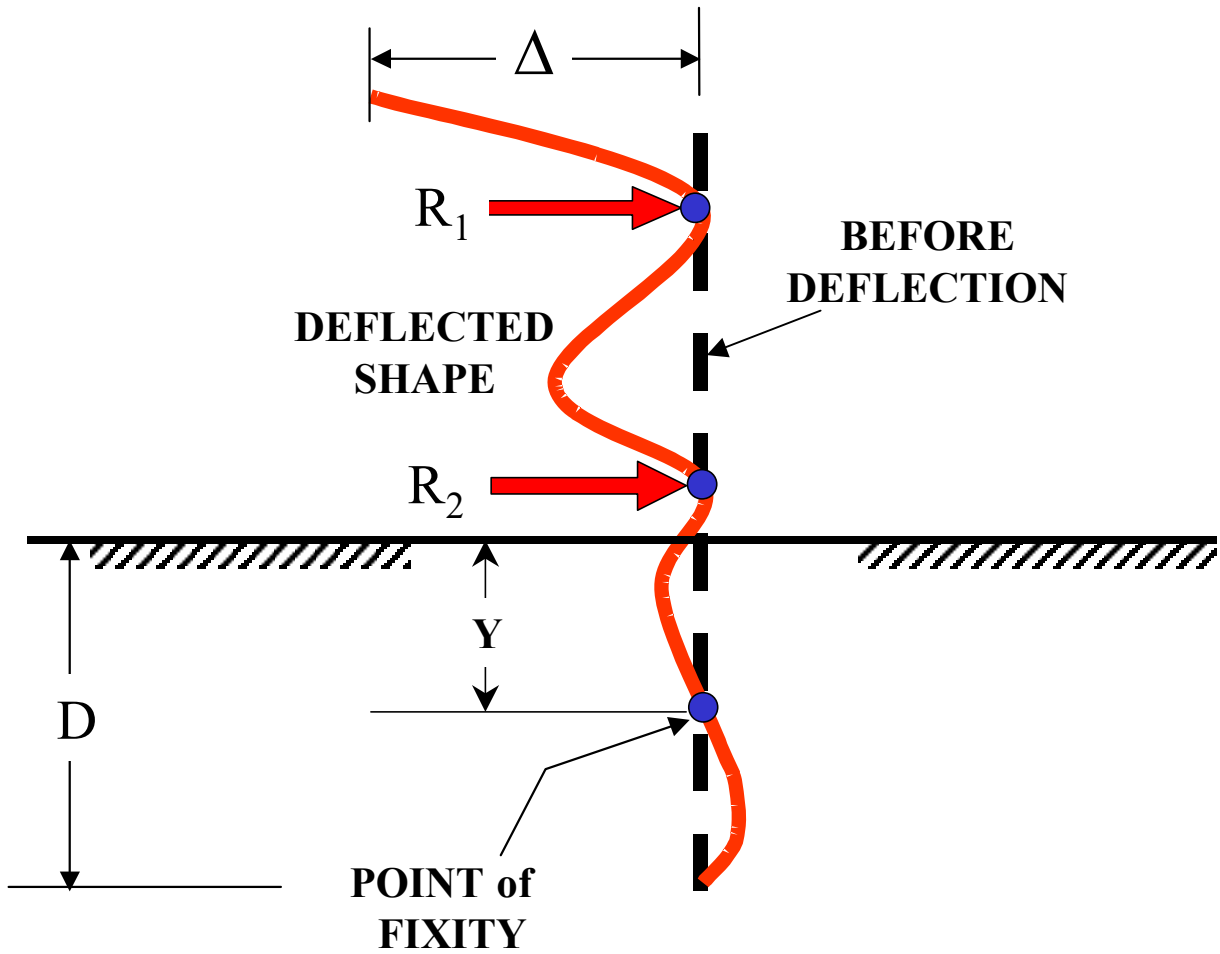


Figure 7-8. Deflected Shape for Restrained System

7.3.4 Example 7-1 Single Tieback Sheet Pile Wall

Check the adequacy of a single tieback sheet pile wall with a single soil layer shown below with a tieback spacing = 10 feet. The sheet pile section is a PZ22, steel grade 42 ksi.

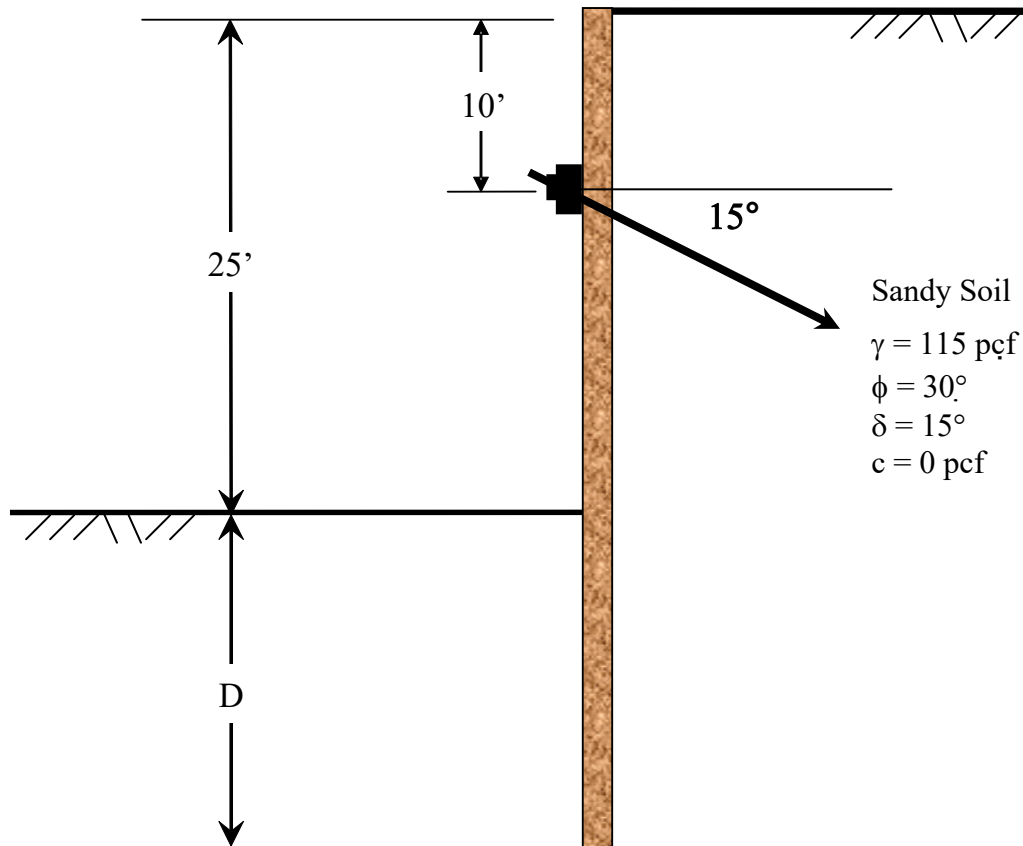


Figure 7-9. Example 7-1

Determine:

1. Active & Passive Earth Pressures.
2. Pile Embedment D with $FS = 1.3$.
3. Tieback Load with $FS = 1.0$.
4. Maximum Shear, Maximum Moment.

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Structural properties of sheet pile section PZ22 are:

- Section Modulus per foot of wall width: $S = 18.10 \text{ in}^3$.
- Moment of Inertia per foot of wall width: $I = 84.70 \text{ in}^4$.
- Radius of Gyration per foot of wall width: $r = 3.62 \text{ in}$.
- Area per foot of wall width: $A = \frac{I}{r^2} = \frac{84.7 \text{ in}^4}{(3.62 \text{ in})^2} = 6.46 \text{ in}^2$

Develop the pressure diagram:

From Rankine's Theory: $K_a = \frac{1}{3}$. Using the Log Spiral Theory, from Figure 4-37: $K_p = 4.7$.

Also, since the wall friction angle (δ) is 0:

$$K_{ph} = K_p = 4.7.$$

The lateral earth pressure distribution for the analysis of anchored walls constructed in cohesionless soils may be determined using Figure 7-10.

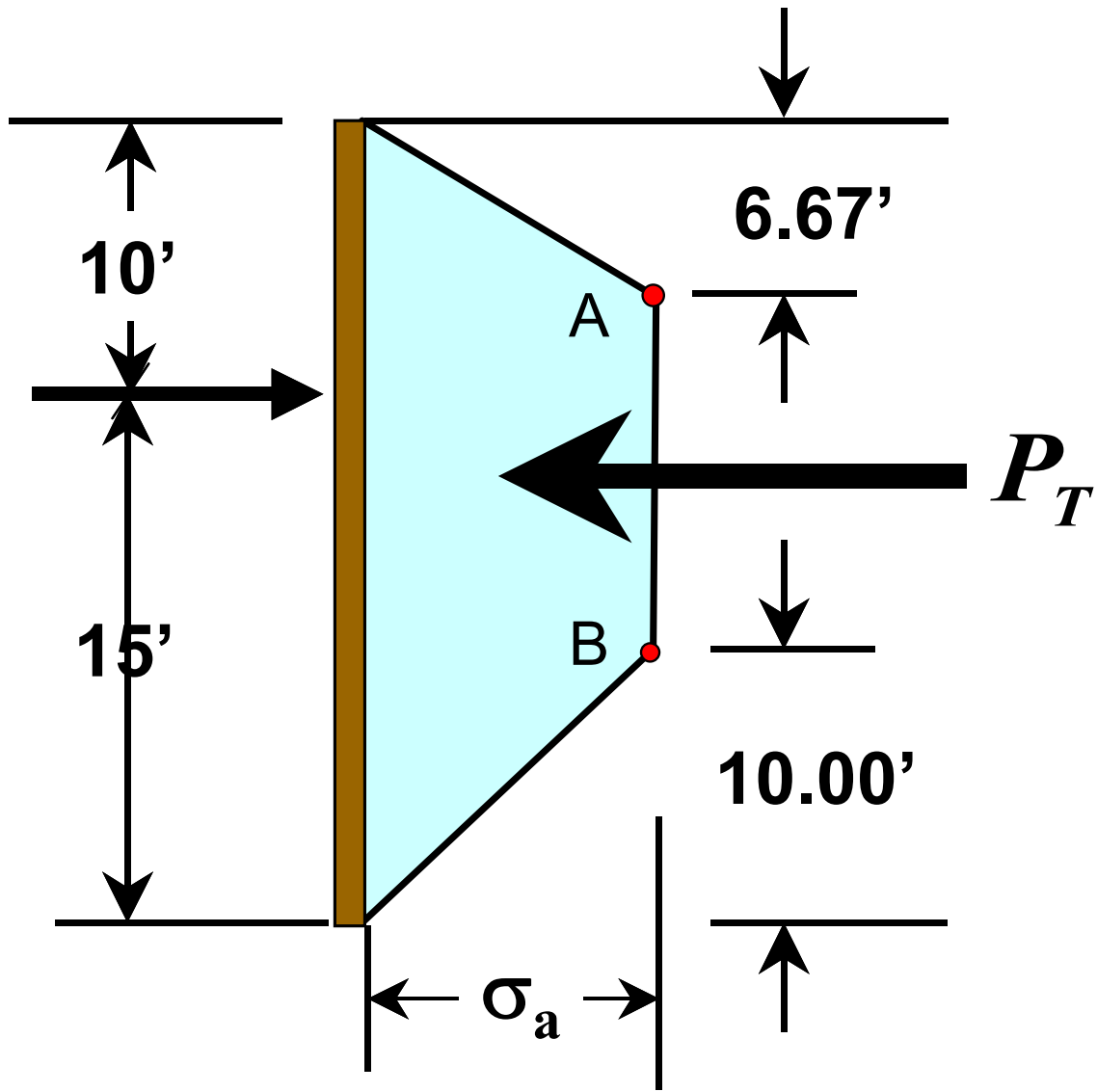


Figure 7-10. Pressure Diagram for Single Tieback Wall

The maximum ordinate (σ_a) of the pressure diagram is determined as follows:

$$\sigma_a = \frac{1.3P}{\left(\frac{2}{3}\right)h} = \frac{P_T}{\left(\frac{2}{3}\right)h}$$

Where the total active earth pressure for a triangular pressure distribution is calculated as follows:

$$P = \frac{1}{2} \gamma h^2 K_a$$

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Using Eq. 7-2: $P_T = 1.3 * P$

$$P = \left(\frac{1}{2}\right)(115)(25^2)\left(\frac{1}{3}\right) = 11,980 \text{ lb}$$

$$P_T = 1.3P = 1.3 * 11,980 = 15,574 \text{ lb}$$

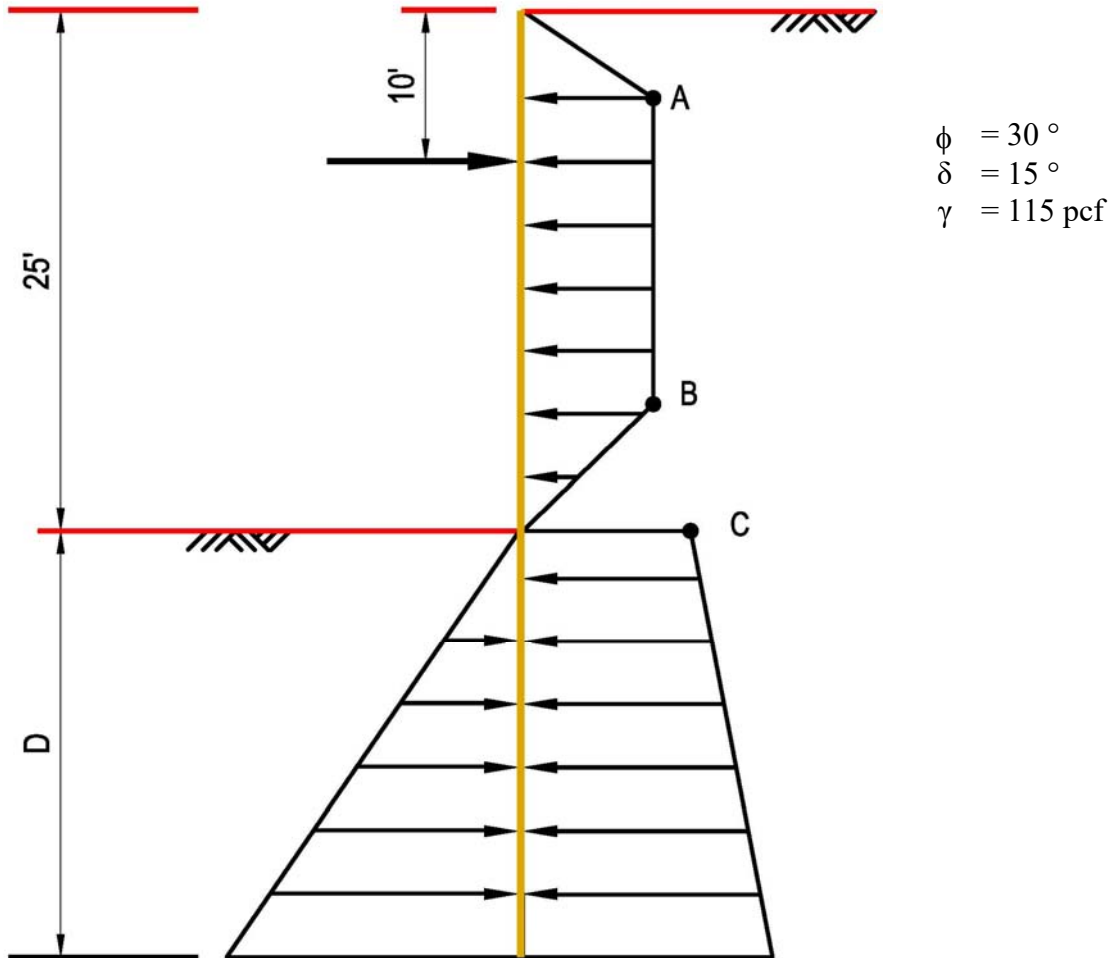


Figure 7-11. Pressure Diagram

Active stress at the point A and B as shown in Figure 7-11:

$$\sigma_a = \frac{15,574}{\left(\frac{2}{3}\right)25'} = 934.4 \text{ psf}$$

Active stress at the dredge line point C:

$$\sigma_c = (115)(25')\left(\frac{1}{3}\right) = 958.3 \text{ psf}$$

$$FS = \frac{M_R}{M_D}$$

Let FS=1.3.

$$M_R = 1.3M_D$$

Take moment about the tieback

$$M_D = \left[\begin{aligned} & (934.4) \left(\frac{6.67'}{2} \right) \left(3.33' + \frac{6.67'}{3} \right) - (934.4)(8.33')(0.835') \\ & - (934.4) \left(\frac{10'}{2} \right) (8.33') - (958.3) \left(15' + \frac{D}{2} \right) D \\ & - (0.5)(115) \left(\frac{1}{3} \right) \left(15' + \frac{2}{3} D \right) D^2 \end{aligned} \right]$$

As determined above: K_{ph} is 4.7.

$$M_R = \frac{1}{2} (115)(4.7) \left(15' + \frac{2}{3} D \right) D^2$$

Solve for D.

$$D^3 + 18.7D^2 - 114.53D - 223.98 = 0$$

$$D \approx 6.09 \text{ ft}$$

Solve for tieback force T by setting the resisting moment equal to driving moment as shown below:

$$M_R = M_D$$

Find D' :

$$D'^3 + 19.64D'^2 - 85.88D' - 167.95 = 0$$

$$D' \approx 4.89 \text{ ft}$$

$$\sum F_X = 0$$

$$\left[\begin{aligned} & - \left(\frac{25 + 8.33}{2} \right) (934.4) - \frac{1}{2} (115)(4.89^2) \frac{1}{3} - (958.3)(4.89) + \\ & \frac{1}{2} (115)(4.89^2)(4.7) \end{aligned} \right] (10) + T_H = 0$$

$$T_H = 142.54 \text{ kips} \quad \text{and} \quad T = \frac{142.54}{\cos(15^\circ)} = 147.57 \text{ kips}$$

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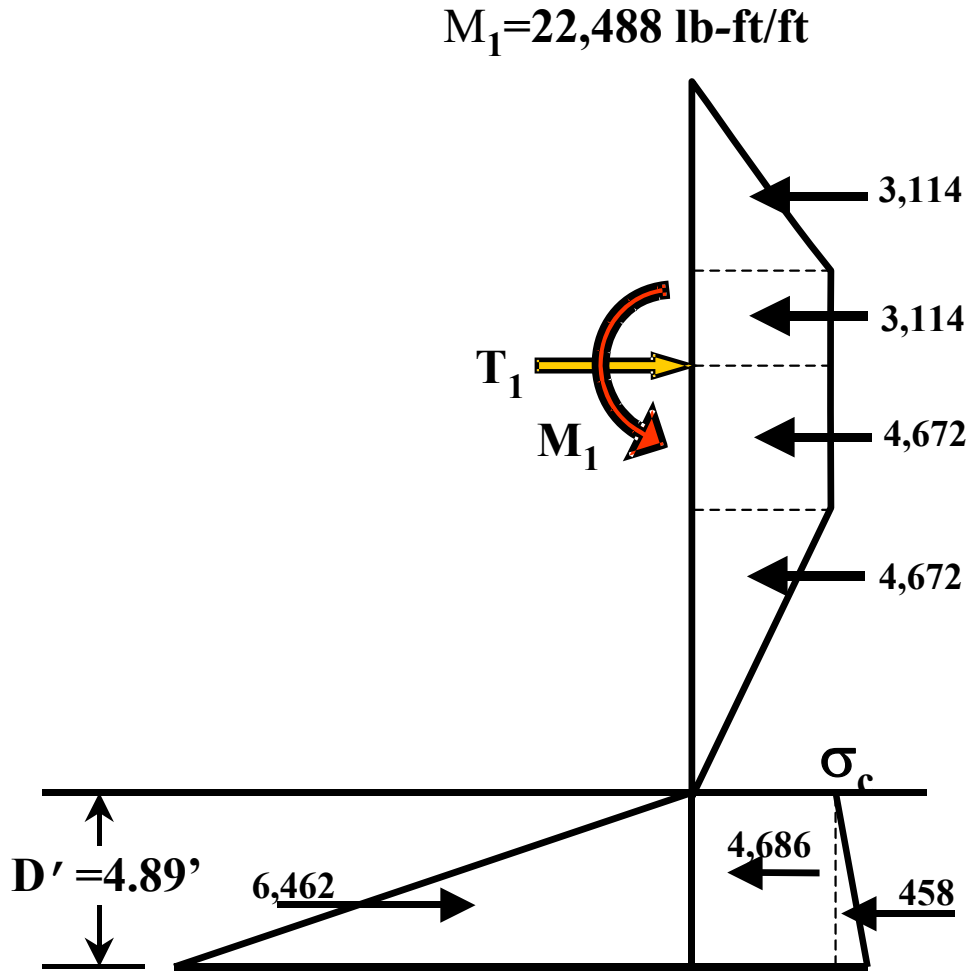


Figure 7-12. Pressure Diagram For Single-Tieback Above Dredge Line
Based on $M_R = M_D$.

The maximum shear in the beam is located at the Tieback.

$$T_{1U} = \frac{1}{2}(934.4)(6.667') + (934.4)(3.333') = 3,114 + 3,114 = 6,228 \text{ lbs}$$

$$T_{1L} = \frac{1}{2}(934.4)(10') + (934.4)(5') + (958.3)(4.89') + \frac{1}{2}(115)\left(\frac{1}{3} - 4.7\right)(4.89')^2 = 4,672 + 4,672 + 4,686 - 6,004 = 8,026 \text{ lbs}$$

Maximum shear is $T_{1L} = 8,026 \text{ lbs}$.

$$f_v = \frac{8,026 \text{ lbs}}{6.46 \text{ in}^2} = 1,242 \text{ psi} < 16,800 \text{ psi}$$

$F_v = 42,000 \text{ psi} * 0.4 \approx 17,000 \text{ psi} \therefore \text{PZ22 is satisfactory in shear.}$

Determine moment M_1 at top tieback due to cantilever loads:

$$M_1 = F_1 * \text{Mom arm of triangular load} + F_2 * \text{Mom arm of rectangular load}$$

$$\begin{aligned} M_1 &= \frac{1}{2}(934.4)(6.667)\left(3.333 + \frac{6.667}{3}\right) + (934.4)(3.333)\left(\frac{3.333}{2}\right) \\ &= (3,114)(5.555) + (3,114)(1.667) = 17,299 + 5,189 = 22,488 \text{ lb-ft} \end{aligned}$$

Determine moment at zero shear below tieback. Please refer to Figure 7-15 for shear diagram of single tieback. The point of zero shear is either located below the bottom of excavation or it is located between the tieback and the bottom of excavation. For this particular example problem, when the summation of forces in the horizontal direction includes the area below the bottom of excavation, a quadratic equation results with two possible roots. As shown below, one root lies at depth D but is not the root we are looking for. The other root is negative and therefore, cannot be used:

$$\begin{aligned} \sum F_H &= 8026 - 4672 - 4672 - 958.3y' - \frac{1}{2}(115)\left(\frac{1}{3} - 4.7\right)(y')^2 \\ &= -1318 - 958.3y' + 251.09y'^2 = 0 \end{aligned}$$

Solving:

$$y'^2 - 3.816y' - 5.25 = 0 \text{ yields: } y' = 4.89 \text{ ft and } y' = -1.07 \text{ ft}$$

Since the second root is invalid, the point of zero shear must be located above the bottom of excavation. Further, it can be surmised that the point of zero shear is located within the sloping portion of the load diagram below the tieback since:

$$T_{1L} - (934.4)(5') = 8,026 \text{ lbs} - 4,672 \text{ lbs} = 3,354 \text{ lbs} > 0$$

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The slope of the load line just above the dredge line is

$$934.4/10' = 93.44. \text{ Solving for } y':$$

$$(8,026 - 4,672) = \frac{1}{2}(934.4 + 934.4 - 93.44y')y' =$$

$$(2)(3,354) = 1,868.8y' - 93.44y'^2$$

$$\therefore 93.44y'^2 - 1,868.8y' + 6,708 = 0$$

$$y' = 4.69'$$

The point of zero shear is located $5' + 4.69' = 9.69'$ below

T_1 . Taking moments about the point of zero shear (O)

in Figure 7-13:

$$F_1 = \frac{1}{2}(93.44)(4.69')(4.69') = 1,027.6 \text{ lbs/ft}$$

$$F_2 = (93.44)(10' - 4.69')(4.69') = 2,326.7 \text{ lbs/ft}$$

$$M_{1-tip} = \left[\begin{array}{l} (8,026)(9.69') - (4,672)\left(\frac{5'}{2} + 4.69'\right) - \frac{2}{3}(1,027)(4.69') - \frac{1}{2}(2,326)(4.69') \\ - 22,488 \end{array} \right]$$

$$M_{1-tip} = 77,772 - 33,592 - 3,211 - 5,455 - 22,488 = 13,026 \text{ ft} \cdot \text{lbs/ft}$$

Therefore the maximum moment is at T_1 : $M_1 = 22,488 \text{ ft} \cdot \text{lbs/ft}$.

Check the bending stress in the sheet pile section:

$$f_b = \frac{22,488 \text{ ft} \cdot \text{lb} \cdot 12 \text{ in/ft}}{18.10 \text{ in}^3} = 14,909 \text{ psi}$$

$$F_b = 42,000 \text{ psi} \cdot 0.6 \approx 25,000 \text{ psi} \therefore \text{ Therefore, PZ22 is satisfactory in bending.}$$

The process to check deflection can be found in Example 6-2 and EXAMPLE 8-1 and will

not be calculated for this example. Figure 7-17 represents the deflected shape of the PZ22

sheet pile based on the Moment Area method; therefore, use these values with caution.

The deflection due to the cantilever is 0.20 inches. The maximum deflection is 0.23 inches

and is located about 9.6 ft below T_1 . The respective diagrams are shown in Figure 7-14

through Figure 7-17 and are for information only.

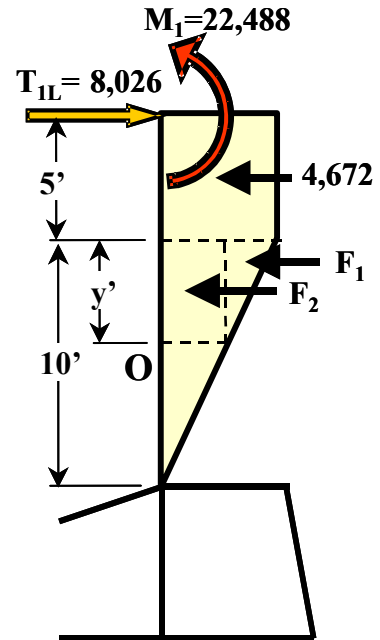


Figure 7-13. Zero shear

Caltrans Trenching and Shoring Check Program, Single Tiebacks

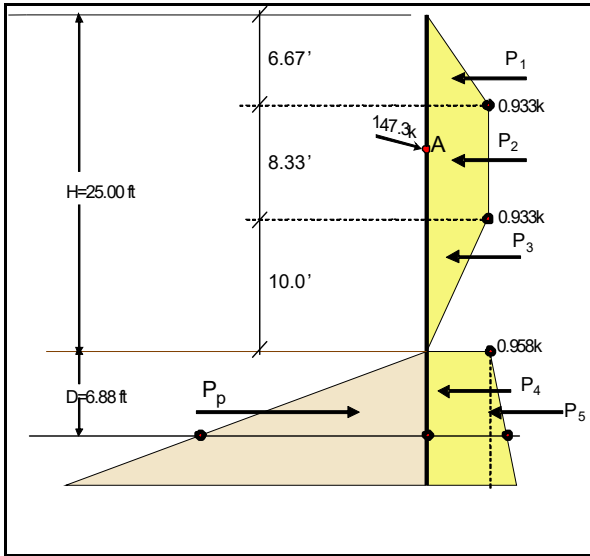


Figure 7-14. Pressure Diagram

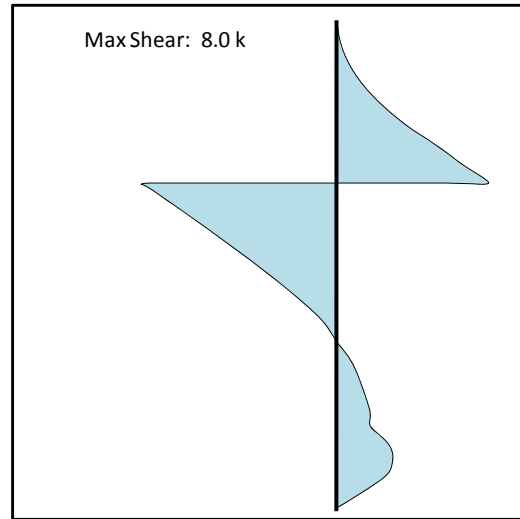


Figure 7-15. Shear Diagram

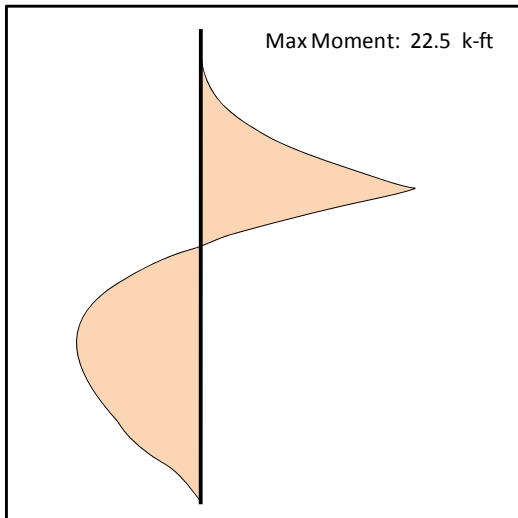


Figure 7-16. Moment Diagram

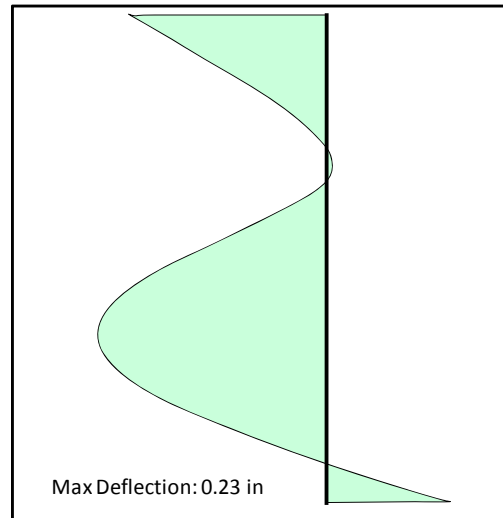


Figure 7-17. Deflection Diagram

7.3.5 Example 7-2 Multiple Tieback Sheet Pile Wall

Check the adequacy of a multiple tieback sheet pile wall with a single soil layer shown below with a tieback spacing = 10 feet. The sheet pile section is a PZ22, steel grade 42 ksi.

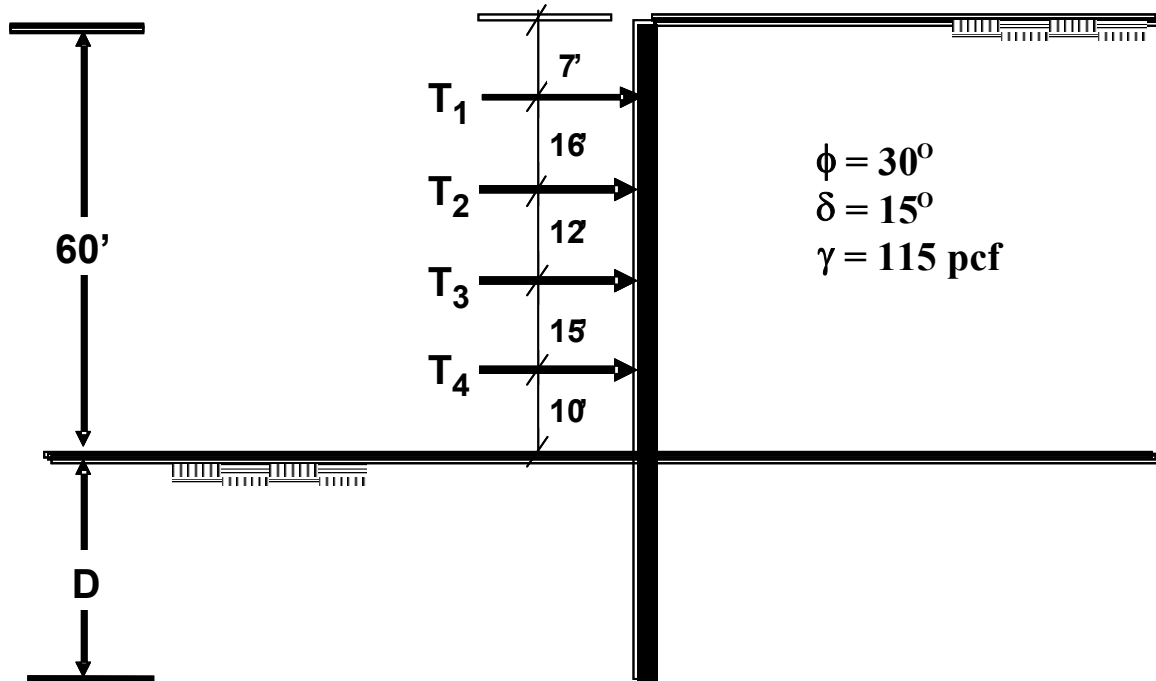


Figure 7-18. Example 7-2

Determine:

1. Active & Passive Earth Pressures.
2. Pile Embedment D with a FS of 1.3.
3. Tieback Loads with a FS of 1.0.
4. Maximum Shear, Moment and Deflection.

Structural properties of the PZ22 are:

- Section Modulus per foot of wall width: $S = 18.10 \text{ in}^3$.
- Moment of Inertia per foot of wall width: $I = 84.70 \text{ in}^4$.
- Radius of Gyration per foot of wall width: $r = 3.62 \text{ in}$.
- Area per foot of wall width: $A = \frac{I}{r^2} = \frac{84.7 \text{ in}^4}{(3.62 \text{ in})^2} = 6.46 \text{ in}^2$

Develop the pressure diagram:

From Rankine's Theory: $K_a = \frac{1}{3}$. Using the Log Spiral Theory, from Figure 4-37: $K_p = 4.7$.

The lateral earth pressure distribution for the analysis of braced or anchored walls constructed in cohesionless soils may be determined using Figure 7-19. The maximum ordinate (σ_a) of the pressure diagram is determined as follows:

$$\sigma_a = \left[\frac{P_T}{H - \frac{1}{3}(H_1 + H_2)} \right]$$

Where the total active earth pressure is calculated as follow:

$$P = \frac{1}{2}(115)(60^2) \left(\frac{1}{3} \right) = 69,000 \text{ lb/ft}$$

$$P_T = 1.3P = 1.3(69,000) = 89,700 \text{ lb/ft}$$

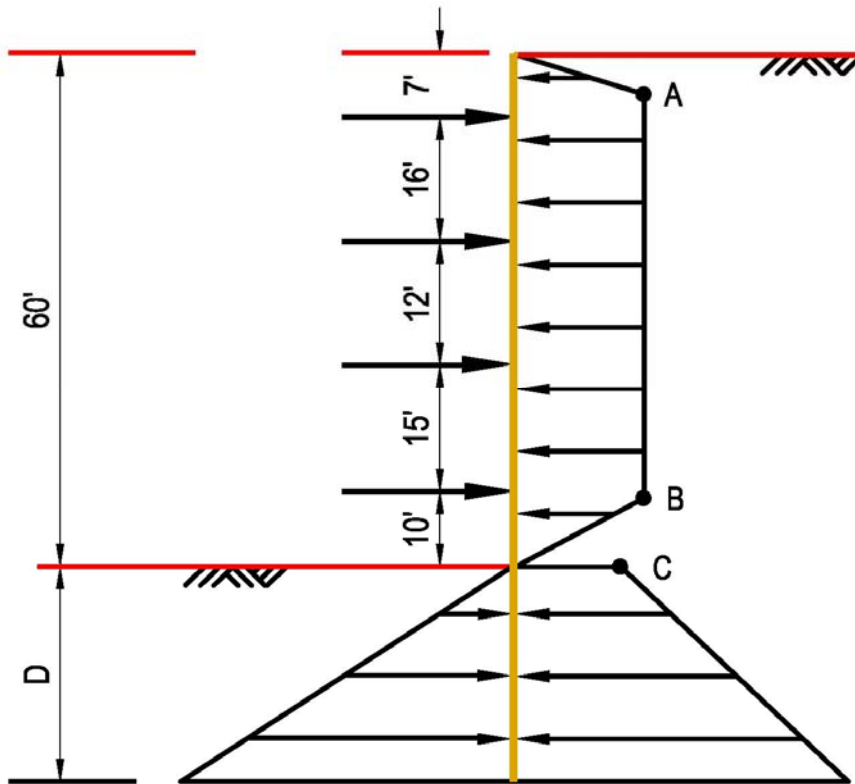


Figure 7-19. Pressure Diagram For Multi-Tieback

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Active stress at the point A and B as shown in Figure 7-19:

$$\sigma_a = \left[\frac{89,700}{60 - \frac{1}{3}(7+10)} \right] \approx 1,650 \text{ psf}$$

Active stress at the dredge line point C:

$$\sigma_c = (115)(60) \left(\frac{1}{3} \right) = 2,300 \text{ psf}$$

Determine forces due to trapezoidal pressure distribution:

$$F_1 = \frac{1}{2}(4.667)(1,650) \approx 3,850 \text{ lb}$$

$$F_2 = (2.333)(1,650) \approx 3,850 \text{ lb}$$

$$F_3 = (16)(1,650) = 26,400 \text{ lb}$$

$$F_4 = (12)(1,650) = 29,800 \text{ lb}$$

$$F_5 = (15)(1,650) = 24,750 \text{ lb}$$

$$F_6 = (3.33)(1,650) \approx 5,495 \text{ lb}$$

$$F_7 = \frac{1}{2}(6.67)(1,650) \approx 5,503 \text{ lb}$$

Determine moment M_1 at top tieback due to cantilever loads:

$M_1 = F_1 * \text{Mom arm of triangular load} + F_2 * \text{Mom arm of rectangular load}$

$$M_1 = (3850) \left(2.333 + \frac{4.667}{3} \right) + (3850) \left(\frac{2.333}{2} \right) \approx 19,462 \text{ lb - ft}$$

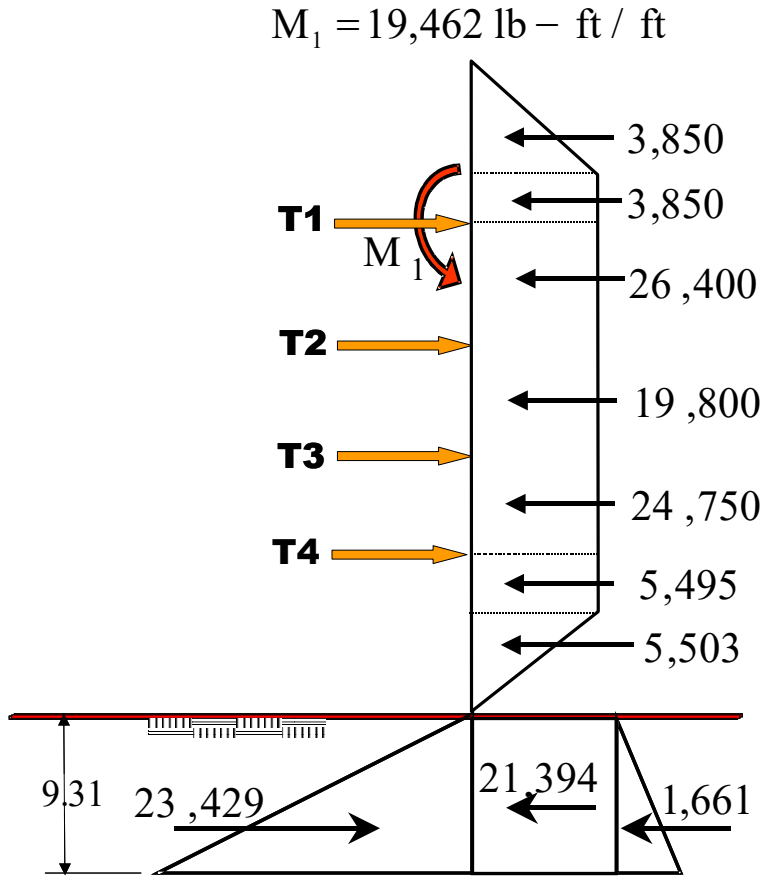


Figure 7-20. Pressure Diagram For Multi-Tieback Above Dredge Line

Note: D is calculated below.

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Determine tieback loads T_1 through T_3 and component T_{4U} . Component T_{4L} will be determined after D is calculated. Note, the subscript letters “U” refers to Upper and “L” refers to Lower components of each tieback. Also, the number 10 in the calculations below is the horizontal spacing of the tiebacks. Note that T_{1L} and T_{2U} include the effect of moment shear $19,462/16$ due to M_1 .

$$T_{1U} = 3,850 + 3,850 = 7,700 \text{ lb/ft}$$

$$T_{1L} = \left(\frac{26,400}{2} \right) + \left(\frac{19,462}{16} \right) \approx 14,416 \text{ lb/ft}$$

$$T_1 = \left[\frac{10(7,700 + 14,416)}{1000} \right] = 211.16 \text{ kips}$$

$$T_{2U} = \left(\frac{26,400}{2} \right) - \left(\frac{19,462}{16} \right) \approx 11,984 \text{ lb/ft}$$

$$T_{2L} = \left(\frac{19,800}{2} \right) = 9,900 \text{ lb/ft}$$

$$T_2 = \left[\frac{10(11,984 + 9,900)}{1000} \right] = 218.84 \text{ kips}$$

$$T_{3U} = T_{2L} = 9,900 \text{ lb/ft}$$

$$T_{3L} = \left(\frac{24,750}{2} \right) = 12,375 \text{ lb/ft}$$

$$T_3 = \left[\frac{10(9,900 + 12,375)}{1000} \right] = 222.75 \text{ kips}$$

$$T_{4U} = T_{3L} = 12,375 \text{ lb/ft}$$

Determine D' to calculate T_4 by taking moments about T_4 .

$$M_D = (5,495) \left(\frac{3.33}{2} \right) + (5,503) \left(3.33 + \frac{6.67}{3} \right) + (2300) \left(10 + \frac{D'}{2} \right) D' + \left(\frac{115}{2} \right) \left(\frac{1}{3} \right) \left(10 + \frac{2}{3} D' \right) D'^2$$

$$M_R = (4.7) \left(\frac{115}{2} \right) \left(10 + \frac{2}{3} D' \right) D'^2$$

Set $M_R = M_D$ and solve for D' .

$$D'^3 + 8.13D'^2 - 137.41D' - 237.24 = 0$$

$$D' \approx 9.31 \text{ ft}$$

Determine the lower component T_{4L} of tieback T_4 and calculate its load.

$$T_{4L} = 5,495 + 5,503 + (2,300)(9.31) + \left(\frac{115}{3}\right)\left(\frac{9.31^2}{2}\right) + (115)(4.7)\left(\frac{9.31^2}{2}\right)$$

$$T_{4L} = 5,495 + 5,503 + 21,413 + 1661 + 23,424 = 10,648 \text{ lb/ft}$$

$$T_4 = \left[\frac{10(12,375 + 10,648)}{1000} \right] = 230.23 \text{ kips}$$

Determine embedment D for external stability by taking moments about T_4 using factor of safety (FS)=1.3.

Set $M_R = 1.3M_D$ and solve for D .

$$D^3 + 5.86D^2 - 182.81D - 315.61 = 0$$

$$D \approx 11.83 \text{ ft}$$

The maximum shear in the beam is located at one of the tiebacks. By inspection of the above calculations the Maximum Shear is located at the lower component of Tieback 1:

$$V_{MAX} = T_{1L} = 14,416 \text{ lbs}$$

Check the shear stress in the sheet pile section:

$$f_v = \frac{14,416 \text{ lbs}}{6.46 \text{ in}^2} = 2,230 \text{ psi} < 16,800 \text{ psi}$$

$$F_v = 42,000 \text{ psi} * 0.4 \approx 17,000 \text{ psi} \therefore \text{PZ22 is satisfactory in shear.}$$

Determine the maximum moment:

The maximum negative moment is located at Tieback 1:

$$M_{MAX-NEG} = M_1 = 19,462 \text{ ft} \cdot \text{lbs/ft}$$

Determine the maximum positive moments between the tiebacks:

$$M_{1-2} = -19,462 + \frac{1}{2}(14,416)(8.74') = 43,536 \text{ ft} \cdot \text{lbs/ft}$$

$$M_{2-3} = \frac{1}{2}(9,900)\left(\frac{1}{2}12'\right) = 29,700 \text{ ft} \cdot \text{lbs/ft}$$

$$M_{3-4} = \frac{1}{2}(12,375)\left(\frac{1}{2}15'\right) = 46,406 \text{ ft} \cdot \text{lbs/ft}$$

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Determine the maximum positive moment between T_4 and tip of the pile:

Sum forces in the horizontal direction to find zero shear at distance y below the lowest tieback:

$$\begin{aligned}\sum F_x &= 5405 + 5503 + 2300y + \frac{1}{2}(115)\left(\frac{1}{3}y\right) - \frac{1}{2}(115)(4.7y) = 0 \\ &= 251.08y^2 - 2300y - 10,988 = 0\end{aligned}$$

The result yields $y = 12.63'$ or $y = -3.46'$. Neither of these values is located within distance D' of 9.31 ft and therefore is not valid. Therefore the point of zero shear is located above the dredge line. Further, it can be surmised that the point of zero shear is located within the sloping portion of the load diagram below the T_4 since:

$$T_{4L} - (1,650)(3.33') = 10,648 \text{ lbs} - 5,495 \text{ lbs} = 5,153 \text{ lbs} > 0$$

The following will determine the point of zero shear above the dredge line: See Figure 7-21. Solving for y' :
(The slope of the line in the area of the assumed zero shear is $1650/6.67' = 247.38$.)

$$\begin{aligned}(10,648 - 5,495) &= \frac{1}{2}(1,650 + 1650 - 247.38y')y' = \\ (2)(5,153) &= 3,300y' - 247.38y'^2 \\ \therefore 247.38y'^2 - 3,300y' + 10,306 &= 0 \\ y' &= 4.99 \text{ ft}\end{aligned}$$

The point of zero shear is located at $3.33' + 4.99' = 8.32'$ below T_4 . Taking moments about point O:

$$\begin{aligned}M_{4\text{-tip}} &= -\frac{1}{2}(1,699)(4.99') - \frac{2}{3}(3,018)(4.99') - (5,495)\left(4.99' + \frac{3.33'}{2}\right) + (10,648)(8.32') = \\ &= -4,239 - 10,040 - 36,569 + 88,591 = 37,743 \text{ ft} \cdot \text{lb/ft}\end{aligned}$$

Therefore the maximum moment is between T_3 and T_4 : $M_{3-4} = 46,406 \text{ ft} \cdot \text{lb/ft}$

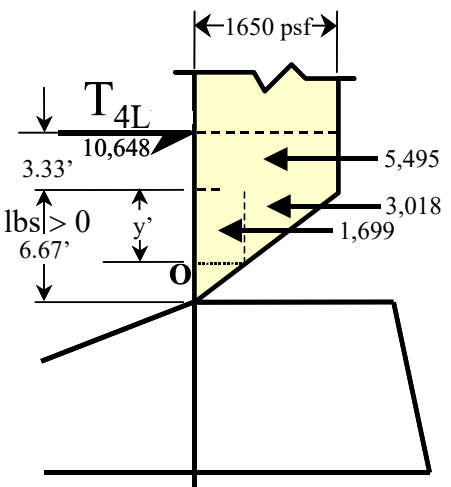


Figure 7-21. Point of zero shear

Check the bending stress in sheet pile section PZ22:

$$f_b = \frac{46,406 \text{ ft} \cdot \text{lb} * 12 \text{ in/ft}}{18.10 \text{ in}^3} = 30,766 \text{ psi}$$

$F_b = 42,000 \text{ psi} * 0.6 \approx 25,000 \text{ psi} \therefore$ Therefore, PZ22 is NOT satisfactory in bending.

The process used to check deflection can be found in Example 6-2 and EXAMPLE 8-1 and will not be calculated for this example. Figure 7-25 represents the deflected shape of the PZ22 sheet pile based on the Moment Area method; therefore, use these values with caution. The deflection due to the cantilever is 0.71 inches. The maximum deflection is 0.79 inches and is located about 8.6 ft below T_4 . The respective diagrams are shown in Figure 7-22 through Figure 7-25 and are for information only.

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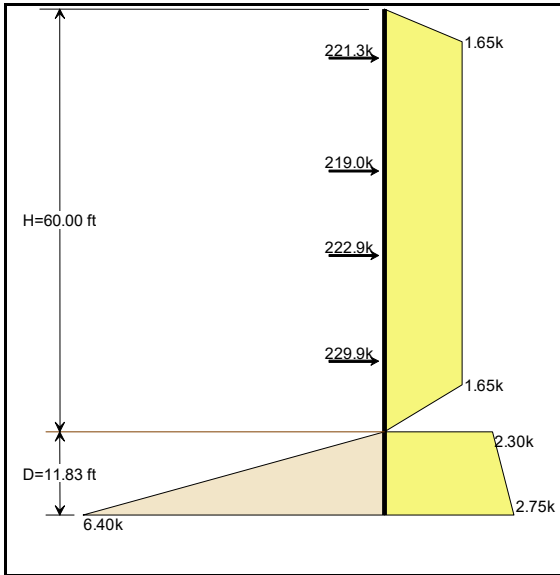


Figure 7-22. Pressure Diagram

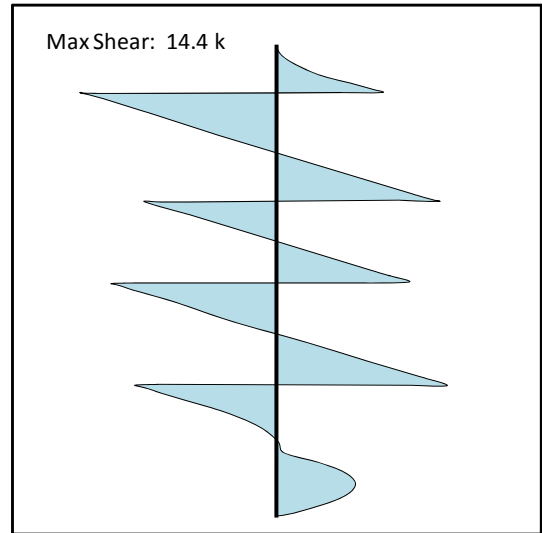


Figure 7-23. Shear Diagram

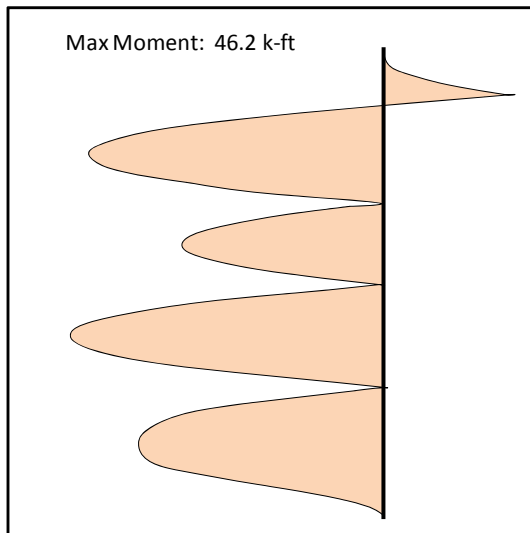


Figure 7-24. Moment Diagram

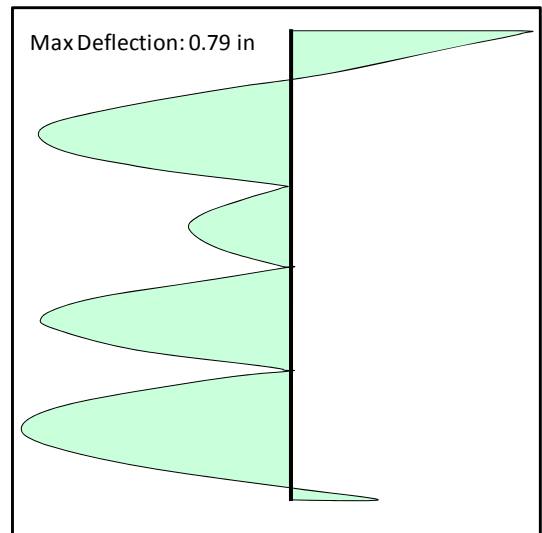


Figure 7-25. Deflection Diagram