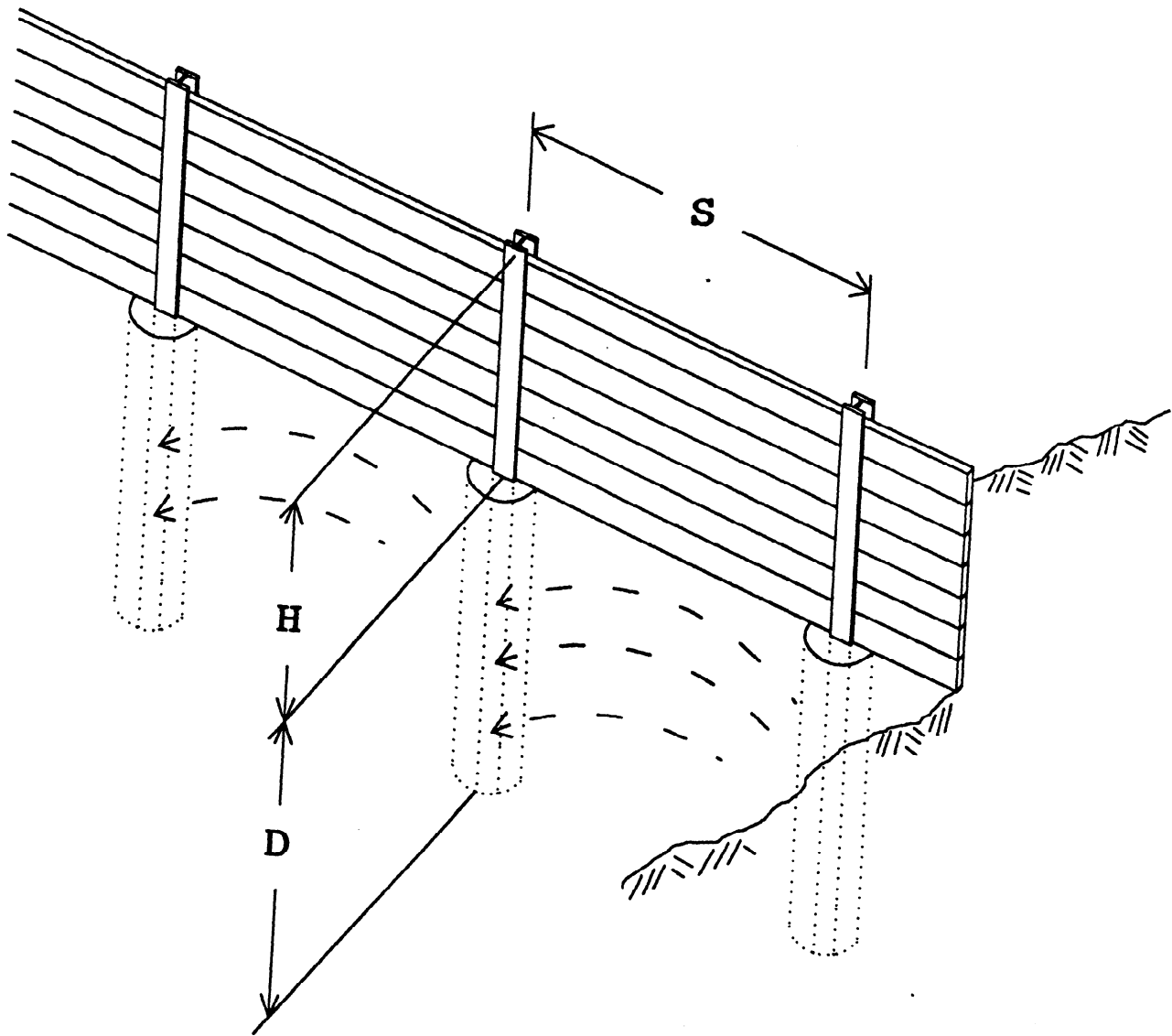


SOLDIER PILE SYSTEMS



SOLDIER PILES

SOLDIER PILE SYSTEMS

Soldier piles of varying materials and sections are used, often in conjunction with some form of lagging to support soils as a continuous wall above the depth of excavation. Soldier piling elements may consist of HP or wide flange sections, sheet pile sections, or CIDH piles. Lagging may consist of horizontally placed wood members, steel plates, or concrete sections.

Soil loads are transferred to soldier piles partially through the lagging and partially through soil arching. A semi-circular section of soil immediately behind the lagging may represent all the load that gets transmitted to the lagging. When the soil between soldier piles is capable of self support the soil loads will transfer to the adjacent soldier piles, and no lagging will be needed. This soil load transfer is referred to as soil arching. Compact or cohesive soils will demonstrate a greater ability for soil arching than will loose and cohesionless soils. However, the looser soils will tend to load the lagging more.

Stiff soils exhibit an ability to stand unsupported for some height for some period of time. This is evident by comparison to relatively small square or rectangular excavations where no shoring is used. The soil behind and along the cut faces transmit the lateral forces to the vertical corners through soil arching. Soldier piles act in the same manner as the vertical corners.

The general design procedure for soldier pile walls is to assume one half the pile spacing either side of the pile acts as a panel loaded with active soil pressures and surcharge loadings above the depth of excavation. The portion of soldier pile below the depth of excavation is likewise loaded with both active soil pressure and surcharge loadings.

Resistance to lateral movement or overturning (about any point) of the soldier pile is furnished by the passive resistance of the soil below the depth of excavation. The depth of pile penetration must be sufficient to prevent lateral movement or tip over (about the base) of the soldier pile system. To account for soil disturbance at the excavation elevation AASHTO recommends that any passive resistance be ignored or discounted for a distance equal to 1.5 times the effective pile diameter immediately below the depth of the excavation.

Soldier piles may be driven or they may be installed in drilled holes. Drilled holes may be backfilled with concrete, slurry, sand, pea-gravel or similar material after the soldier pile has been installed in the hole. Some soldier pile drilled holes are backfilled with concrete to the depth of excavation and then the remainder of the hole is filled with slurry to ground level. Slurry is generally considered to be a sand-cement mix placed wet enough to fill all voids. Backfill materials other than concrete are used when it is desirable to extract the soldier piles. When

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materials other than concrete are used to backfill drilled soldier pile holes some vibratory methods and/or jetting procedures need to be used. The backfill should be as compact as the native soil into which the pile is set.

The use of pea gravel backfill in lieu of concrete substantially decreases the passive resistance on all sides of the soldier pile. The pea gravel does not permit the soldier pile to act as a unit until sufficient soldier pile deflection compacts the pea gravel against the soil. Similar reasoning is true for compacted sand backfill.

Effective Width:

The effective width of a soldier pile is considered to be the dimension of the soldier pile taken parallel to the line of the wall for piles either driven, or placed in drilled holes backfilled with materials other than concrete. When hard rock concrete is used for the backfill of drilled holes the effective width of the soldier pile is the diameter of the drilled hole. Structural concrete is generally considered to be a 4 sack or better concrete mix. Properly placed lean concrete can also be effective. However, lean concrete must be sufficiently strong to prevent collapse of the hole, yet weak enough to be excavated easily. A lean concrete mix is normally about 1 to 2 sacks of cement per cubic yard with a minimum specified strength of 2,400 psi.

Experimentation (1970's and later) has determined that the passive resistance of cohesionless soils acts over a width greater than the effective width of the soldier pile. The pressure exerted by the laterally pulled soldier pile produces what amounts to a wedge shaped resisting soil configuration. This soil failure configuration offers a resistance similar to the resistance that would act over something wider than the effective pile width.

Because of the apparent increase in passive resistance previously mentioned, the effective widths of soldier piles installed in cohesionless soils may be increased by an adjustment factor (passive arching capability) of 0.08 times the internal friction angle of the soil (0.08ϕ), but not to exceed a value of 3.00.

This means for example, that the final adjusted design width for a soldier pile with a flange width of 14" installed in soil which has a ϕ angle of 38° , not installed in hard rock concrete, would be equivalent to $[(0.08)(38)](14/12) = [3.00 \text{ max}](14/12) = 3.5$ feet. The full value of 3.5 feet per soldier pile could be used provided the pile spacing is greater than 3' - 6". Care will have to be exercised to be sure that no more than 1 pile spacing is used for the final adjusted widths so that none of the widths overlap.

SOLDIER PILES

Adjustments:

Adjustment Factor = Arching Capability = 0.08ϕ (≤ 3.00)

Adjusted Pile Width = (Effective Width)(0.08ϕ) ≤ 1 Pile Spacing

For cohesive soils the adjustment factor for increasing the effective soldier pile width ranges between 1 to 2. Permissible adjustment factors are listed in Table 104.

For excavations adjacent to railroad tracks the AREA recommendations specify that ϕ and C values be reduced 15% for the effects of dynamic loading when these values are determined by a qualified soils analysis laboratory. See the railroad requirements in the appendix.

Below the excavation depth the final adjusted width may be used for the passively loaded side of the pile. The same final adjusted width may be used for the active and surcharge loading also. Some consultants have used only the effective width of the piling for the active and surcharge loaded width (not advisable).

Soldier Piles As Sheet Piling:

Soldier piling can be analyzed in the same manner as sheet piling when the active loaded width below the depth of excavation is assumed to be the same as the passive loaded width.

When soldier piling is analyzed in the same manner as sheet piling either the loaded panel width (the pile spacing) or the effective soldier pile width must be adjusted. Proportioning the soldier pile effective width to the pile spacing permits analysis on a per foot basis of wall as is done for sheet pile analysis. For example, a soldier pile spaced on 8'-0" centers having a final effective pile width of 2'-0" has an equivalent sheet pile width of 1'-0" above the excavation line and $2'/8' = 0.25$ foot width below that line. Completion of the soldier pile computations using sheet pile analysis is accomplished by increasing all answers for moments and shears by a factor equal to the soldier pile spacing, in this case 8 feet.

An easy method for converting from soldier pile to sheet pile analysis involves determining an Arching Factor (f). The value of f is determined by multiplying the adjustment factor (Passive Arching Capability listed in Table 10-1), by the effective pile width then dividing that result by the soldier pile spacing.

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f = Arching Factor

$$f = \frac{(\text{Passive Arching Capability}) (\text{Effective Pile Width})}{\text{Soldier Pile Spacing}}$$

Where: $f \leq 1.0$

The value f must be equal to or less than 1.0 to prevent overlap of the passive resisting lengths.

Assume the same values previously cited where the soldier pile spacing is 8'- 0", the pile is not encased in hard rock concrete, the pile flange width is 14", and the internal friction angle ϕ of the cohesionless soil is 38°.

$$f = \frac{[0.08(38)](14/12)}{8.0} = 0.438$$

A sheet pile analysis could then be made for the soldier piling as long as all equations used below the excavation line are factored by 0.438, and the final answers multiplied by 8 which equals the pile spacing of 8'- 0".

AASHTO Methodology:

The 1992 publication, Standard Specifications for Highway Bridges by AASHTO contains a simplified method for designing cantilever soldier piling in cohesionless soils. The methodology along with a sample problem is included near the end of this chapter. The AASHTO method permits the inclusion of surcharges. This design method requires that no passive resistance be counted within 1.5 times the effective pile width below the depth of excavation, The method also provides that the computed pile depth (D) be increased by 30% for temporary work.

The AASHTO method indicates that the adjusted pile width may be up to 3 times the effective pile width provided that the soldier pile spacing is equal to or greater than 5 times the effective pile width. Structures policy will be to use an adjusted pile width of 0.08ϕ (≤ 3.00) times the effective pile width provided this width does not exceed the soldier pile spacing.

SOLDIER PILES

GUIDELINES FOR REVIEW OF SOLDIER PILE

PASSIVE ARCHING CAPABILITIES

GRANULAR SOILS

<u>COMPACTNESS</u>	<u>VERY LOOSE</u>	<u>LOOSE</u>	<u>MEDIUM</u>	<u>DENSE</u>	<u>VERY DENSE</u>
Relative Density, D_r		15%	35%	65%	85%
Standard Penetration Resistance, $N = \text{Blows/ft}$		4	10	30	50
Angle of Internal Friction, ϕ		28	30	36	41
Unit Weight (PCF)					
Moist	100	95-125	110-130	110-140	130+
Submerged	60	55-65	60-70	65-85	75+
Arching Capability	0.08 ϕ	0.08 ϕ	0.08 ϕ	0.08 ϕ	0.08 ϕ

COHESIVE SOILS

<u>CONSISTENCY</u>	<u>VERY SOFT</u>	<u>SOFT</u>	<u>MEDIUM</u>	<u>STIFF</u>	<u>VERY STIFF</u>	<u>HARD</u>
$q_u = \text{unconfined comp. strength (PSF)}$	500	1000	2000	4000	8000	
Standard Penetration Resistance, $N = \text{Blows/Ft}$	2	4	8	16	32	
Unit Weight (PCF)						
Saturated	100-120		110-130	120-140		130+
Arching Capability	1 to 2	1 to 2	2	2	2	

TABLE 10 - 1 (TABLE 21)

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LAGGING

Wood lagging is commonly installed in front of, or behind the front flange of wide flange beam soldier piles. The procedure of installing lagging behind the back flange of the soldier piling is not recommended because the potential arching action of the supported soil will be destroyed. Lagging placed behind the front flange may be wedged back to provide tight soil, to lagging contact. Voids behind lagging should be filled with compacted material. Lagging may be installed with a maximum spacing up to 1 1/2" to permit seepage of moisture through the wall system. Movement of soil through the lagging spaces can be prevented by packing straw or hay in the spaces.

Construction grade lumber is the most common material used for lagging. Treated lumber is used when it is expected that the lagging will remain in place for a longer period of time or permanently.

Soil arching behind lagging is induced by lateral soil movement within the failure wedge. This soil movement causes the lagging to flex outward. The arching process induces a redistribution of soil pressure away from the center of the lagging toward the much stiffer soldier pile support. Because of this, the design load on the lagging may be taken as 0.6 times the theoretical or calculated pressure based on a simple span. Studies have shown that a maximum lagging pressure of 400 psf should be expected when surcharges are not affecting the system. Without soil arching, the pressure redistribution would not occur and reduced lagging loads should not be considered. For the arching effect to occur the back side of the soldier pile must bear against the soil.

- **Lagging design load = 0.6(shoring design load)**
- **Maximum lagging load may be 400 psf without surcharges**

Table 10-2 lists FHWA recommended minimum timber thickness for construction grade douglas fir lagging for a variety of soil classifications.

- **Competent Soils:** These soils include high internal friction angle sand or granular material or stiff to very stiff clays.
- **Difficult Soils:** These soils consist of loose or low friction angle cohesionless material, silty sands, and over consolidated clays which may expand laterally, especially in deep excavations.
- **Potentially Dangerous Soils:** The use of lagging with potentially dangerous soils is questionable.

The tabular values may be used for lagging where soil arching behind the lagging can develop. Tabular values should not be used for excavations adjacent to existing facilities including railroads. Lagging used in conjunction with surcharges should be analyzed separately.

SOLDIER PILES

RECOMMENDED THICKNESS OF WOOD LAGGING

WHEN SOIL ARCHING WILL BE DEVELOPED

(FOR LOCATIONS WITHOUT SURCHARGE LOADINGS)

Soil Description Classification	Unified	Depth	Recommended Thickness of Lagging (rough cut) for Clear Spans of:									
			5'	6'	7'	8'	9'	10'				
COMPETENT SOILS												
Silts or fine sand and silt above water table	ML, SM - ML											
Sands and gravels (medium dense to dense).	GW, GP, GM, GC, SW, SP, SM	0' to 25'	2"	3"	3"	3"	4"	4"				
Clays (stiff to very stiff); non-fissured	CL, CH	25' to 60'	3"	3"	3"	4"	4"	5"				
Clays, medium consistency and $\gamma_H/C < 5$.	CL, CH											
DIFFICULT SOILS												
Sands and silty sands, (loose).	SW, SP, SM											
Clayey sands (medium dense to dense) below water table.	SC	0' to 25'	3"	3"	3"	4"	4"	5"				
Clays, heavily over-consolidated fissured.	CL, CH	25' to 60'	3"	3"	4"	4"	5"	5"				
Cohesionless silt or fine sand and silt below water table	ML; SM - ML											
POTENTIALLY DANGEROUS SOILS (appropriateness of lagging is questionable)												
Soft clays $\gamma_H/C > 5$.	CL, CH	0' to 15'	3"	3"	4"	5"	-	-				
Slightly plastic silts below water table.	ML	15' to 25'	3"	4"	5"	6"	-	-				
Clayey sands (loose), below water table.	SC	25' to 35'	4"	5"	6"	-	-	-				

*Adapted and revised from the April 1976 Federal Highway Administration Report No. FHWA-RD-75-130.

TABLE 10- 2 (TABLE 22)

CANTILEVER SOLDIER PILES - GRANULAR SOIL

Basic soil - no surcharge

f = Arching factor
(See page 10-3)

$$P_{A1} = \gamma H K_2 = K_w H$$

$$P_{A'1} = f P_{A1}$$

$$P_{A2} = f \gamma D K_2 + P_{A'1}$$

$$P_E = f \gamma D (K_1 - K_2) - P_{A'1}$$

$$P_J = f \gamma D (K_1 - K_2) + f \gamma H K_2$$

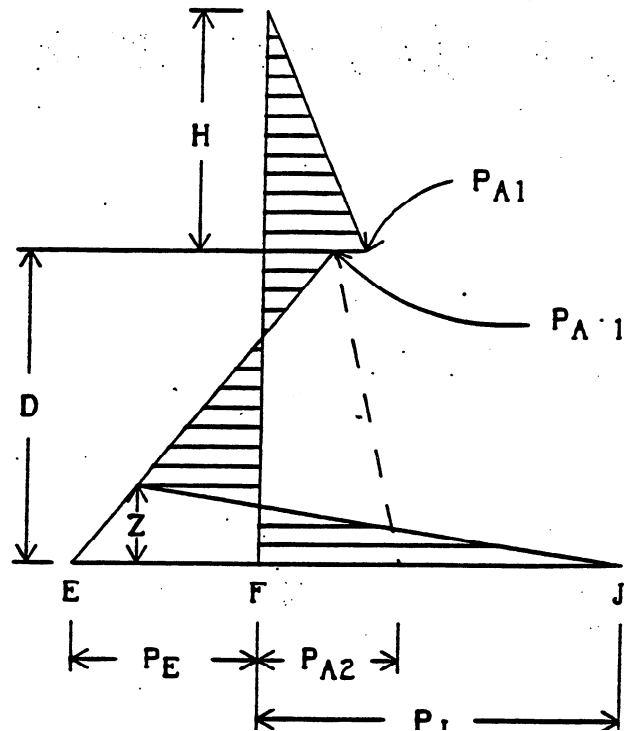


FIGURE 10-1

$$\Sigma F_H = 0 = (H) (P_{A1}) / 2 + (P_{A'1} + P_{A2}) (D) / 2 + (P_E + P_J) (Z) / 2 - (P_E + P_{A2}) (D) / 2$$

$$\therefore Z = \{ (P_E + P_{A2}) (D) - (H) (P_{A1}) - (P_{A'1} + P_{A2}) (D) \} / (P_E + P_J) = \{ P_E - P_{A'1} \} (D) - (H) (P_{A1}) \} / (P_E + P_J)$$

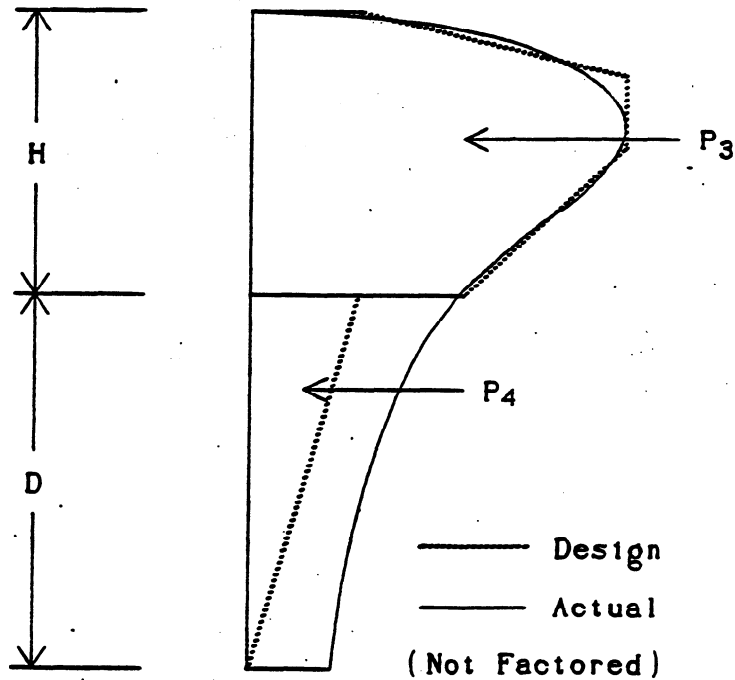
$$\Sigma M_F = 0 = \{ (H) (P_{A1}) / 2 \} [H/3 + D] + (P_{A'1} (D) [D/2] + \{ (P_{A2} - P_{A'1}) (D) / 2 \} [D/3] + \{ (P_E + P_J) (Z) / 2 \} [Z/3] - \{ (P_E + P_{A2}) (D) / 2 \} [D/3]$$

Surcharge Considerations:

In most real situations there will be some sort of surcharge present. The following pages demonstrate how two types of surcharges may be handled. Simplifying the pressure diagrams (using sound engineering judgement) should not alter the results significantly and may make the problem much easier to solve. The surcharge pressures can be added directly to the soil pressure diagram, or may be sketched separately. It will often be convenient to convert a long uniform surcharge to an equivalent height of soil, increase H by that amount, and then analyze the shoring with one pressure diagram.

SOLDIER PILES

For Boussinesq surcharges, the pressure diagram above the excavation depth is simplified so as to match the area as close as possible, while still allowing for ease of computation. The surcharge pressure immediately below the depth of excavation is adjusted by the Passive Arching Capability factor and then may be tapered to zero (for small surcharge) at the bottom end of the soldier pile.



BOUSSINESQ SURCHARGE

FIGURE 10 - 2

P_3 = Area under dashed line
above the excavation depth.

P_4 = Area under dashed line
below the excavation depth.

The forces and moments are then added to the equations on the previous page to solve for the total horizontal forces and moments and to arrive at the required depth (D).

To allow for a Safety Factor and sufficient embedment increase D by 20% - 40% or initially adjust K_p by using $K_p/1.5$ to $K_p/1.75$.

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SAMPLE PROBLEM 10-1: CANTILEVER SOLDIER PILE

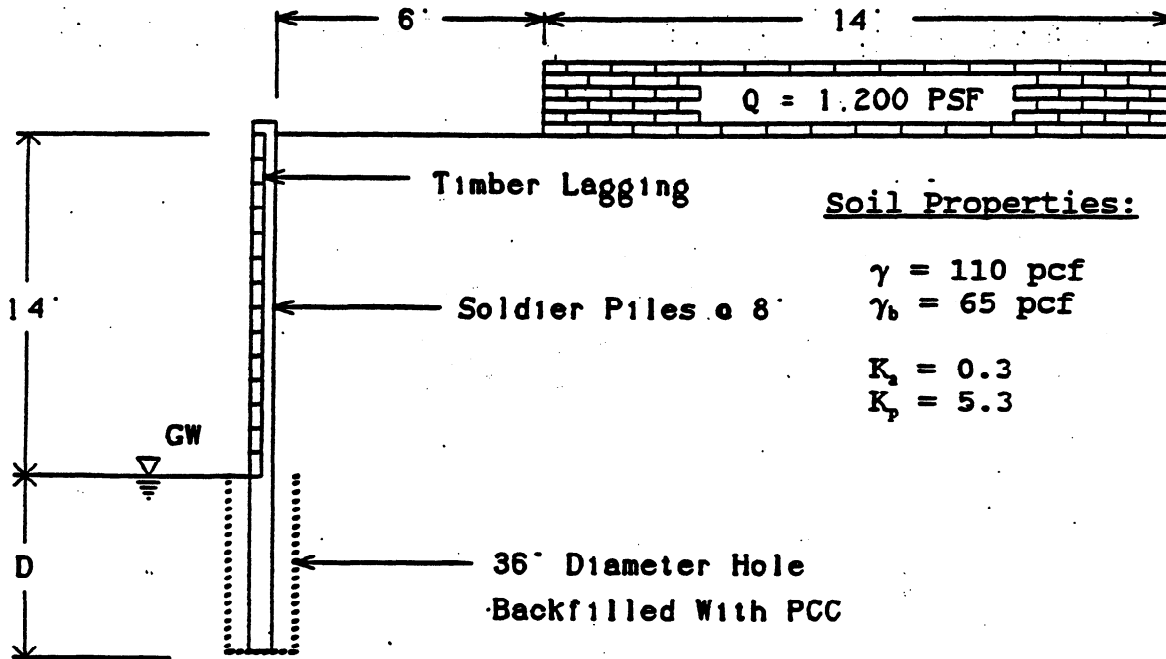


FIGURE 10 - 3

Determine Lateral Pressures: Soil parameters are arbitrary values chosen for simplicity

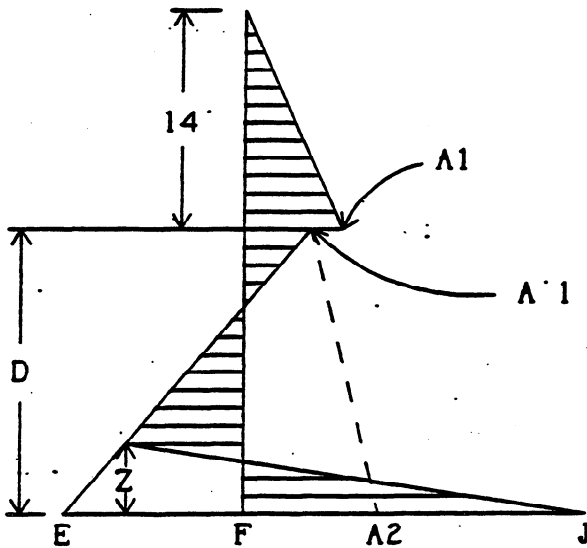


FIGURE 10 - 4

Soil pressure only.

Arching capability = 2
 $f = (2)(36/12)/8 = 0.75$

$$P_{A1} = \gamma H K_a = (110)(14)(0.3) = 462 \text{ psf}$$

$$P_{A1} = f P_{A1} = (0.75)(462) = 347 \text{ psf}$$

$$P_{A2} = f \gamma_b D K_a + P_{A1} = (0.75)(65)(0.3)D + 347 = 15D + 347$$

$$P_E = f \gamma_b D (K_p - K_a) - P_{A1} = (0.75)(65)(5.3 - 0.3)D - 347 = 244D - 347$$

$$P_J = f \gamma_b D (K_p - K_a) + f \gamma H K_p = (0.75)(65)(D)(5.0) + (0.75)(110)(14)(5.3) = 244D + 6,122$$

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Surcharge Pressure:

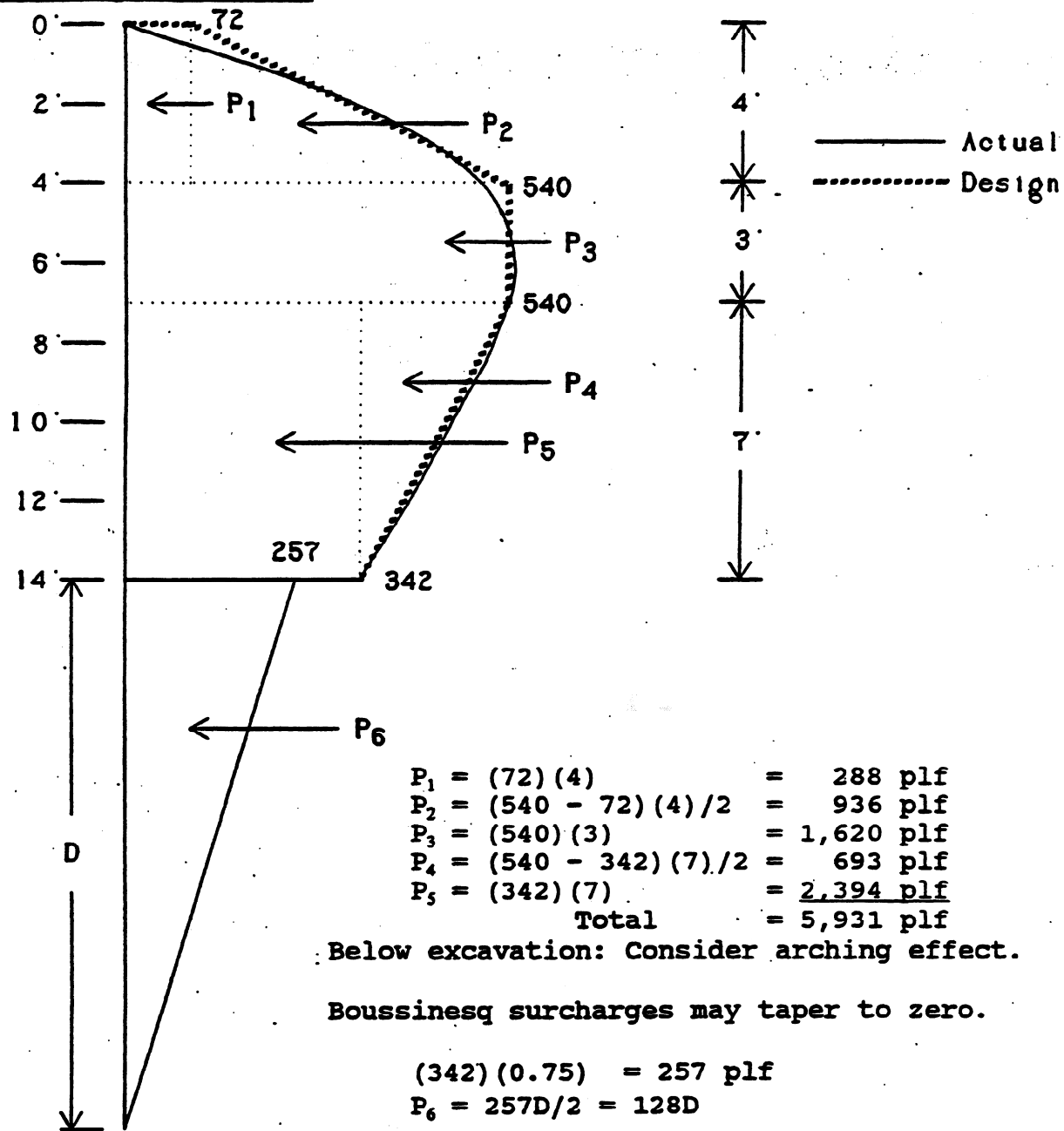


FIGURE 10 - 5

Determine D:

$\Sigma F_H = 0$

$= (14)(462)/2 + (347 + 15D + 347)(D)/2$
 $+ (244D - 347 + 244D + 6,122)(Z)/2 + 5,931 + 128D$
 $- (244D - 347 + 15D + 347)(D)/2$
 $= -122D^2 + 475D + 244DZ + 2,888Z + 9,165$
 $\therefore Z = (D^2 - 3.9D - 75.1)/(2D + 23.7)$

CALIFORNIA TRENCHING AND SHORING MANUAL

$$\begin{aligned} \Sigma M_F &= 0 \\ &= \{(14)(462)/2\}[D + 14/3] + 347D[D/2] + \{(15D)(D)/2\}[D/3] \\ &\quad + \{(244D - 347 + 244D + 6,122)(Z)/2\}[Z/3] + 288[D + 12] \\ &\quad + 936[D + 11.33] + 1,620[D + 8.5] + 693[D + 4.67] \\ &\quad + 2,394[D + 3.5] + 128D[2D/3] \\ &\quad - \{(244D - 347 + 15D + 347)(D)/2\}[D/3] \\ &= -41D^3 + 259D^2 + 9,165D + 81DZ^2 + 963Z^2 + 54,538 = 0 \end{aligned}$$

$$\therefore Z^2 = (D^3 - 6.3D^2 - 223.5D - 1,330.2)/(2.0D + 23.5)$$

By trial and error or other means $D = 22.30'$ & $Z = 4.91'$
 Use a safety factor of 30%: Use $D = 1.3(22.3) = 29.0'$

Find Maximum Moment:

(Composite section properties ignored)

Locate plane of zero shear (B).

$$\begin{aligned} Y &= P_{A1} / f\gamma_b(K_p - K_s) \\ &= 347 / (0.75)(65)(5.3 - 0.3) = 1.42' \end{aligned}$$

Surcharge pressure at A:

$$257(22.30 - 1.42)/22.30 = 241 \text{ psf}$$

Shear due to surcharge at A:

$$1.42(257 + 241)/2 + 5,931 = 6,285$$

Total shear at A:

$$\begin{aligned} &14(462)/2 + 347(1.42)/2 + 6,285 \\ &= 9,765 \text{ Lb/LF} \end{aligned}$$

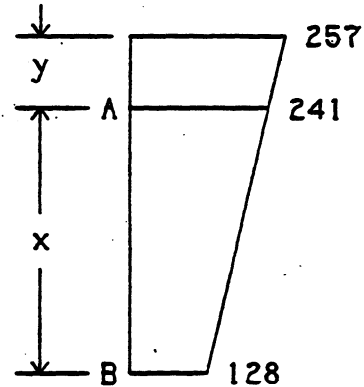
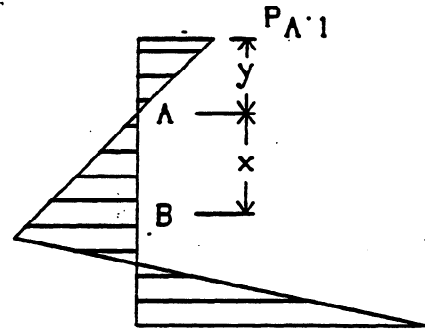


FIGURE 10 - 6

Shear for area between A & B = 9,765

$$\{f\gamma_b(K_p - K_s)x^2/2\} - (\{241 + (257)(22.30 - 1.42 - x)/22.30\}/2)x = 9,765$$

$$\text{Substituting: } 0.75(65)(5.3 - 0.3)x^2 - 241x + 5.76x^2 = 9,765$$

$$\text{Simplifying: } 127.6x^2 - 241x - 9,765 = 0$$

$$\text{Solving for } x: \quad x = 9.74'$$

Find Moment at B:

M due to soil pressure above A.

$$\{14(462)/2\}[15.83] + \{347(1.42)/2\}[10.69] = 53,828 \text{ Ft-Lb/LF}$$

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M due to soil pressure between A & B:

$$0.75(65)(5.3 - 0.3)(9.74)(9.74/2)[9.74/3] = 37,538 \text{ Ft-Lb/LF}$$

M due to surcharge above excavation:

$$288[23.16] + 936[22.49] + 1,620[19.66] + 693[15.83] \\ + 2,394[14.66] = 105,636 \text{ Ft-Lb/LF}$$

Surcharge pressure at B:

$$257(22.30 - 1.42 - 9.74)/22.30 = 128 \text{ psf}$$

M due to surcharge below excavation:

$$128(11.16)[11.16/2] + \{(257 - 128)(11.16)/2\}[11.16(2)/3] \\ = 13,326 \text{ Ft-Lb/LF}$$

$$M \text{ (Total)} = 8(53,828 - 37,538 + 105,636 + 13,326) \\ = 1,082,016 \text{ Ft-Lb}$$

$$S \text{ Required} = (1,082,016)(12)/22,000 = 590.2 \text{ in}^3$$

Use W30 x 191, S = 598 in³

Determine Lagging Needed:

By inspection, maximum load occurs at the depth of excavation.

$$M_{\text{max}} = wL^2/8 = (342 \text{ } 462)(8)^2/8 = 6,432 \text{ Ft-Lb} \\ \text{(With total soil arching } M_{\text{max}} = 400(8)^2/8 = 3,200 \text{ Ft-Lb)}$$

$$S \text{ Required} = 6,432(12)(0.6)/(1,500)(1.0)* = 30.9 \text{ in}^3$$

* 1.0 In lieu of 1.33 duration factor due to high risk building

Use 4 x 12's (rough lumber): S = 32 in³

This answer does not agree with values in Table 22 because the table does not provide for surcharge loadings.

Summary:

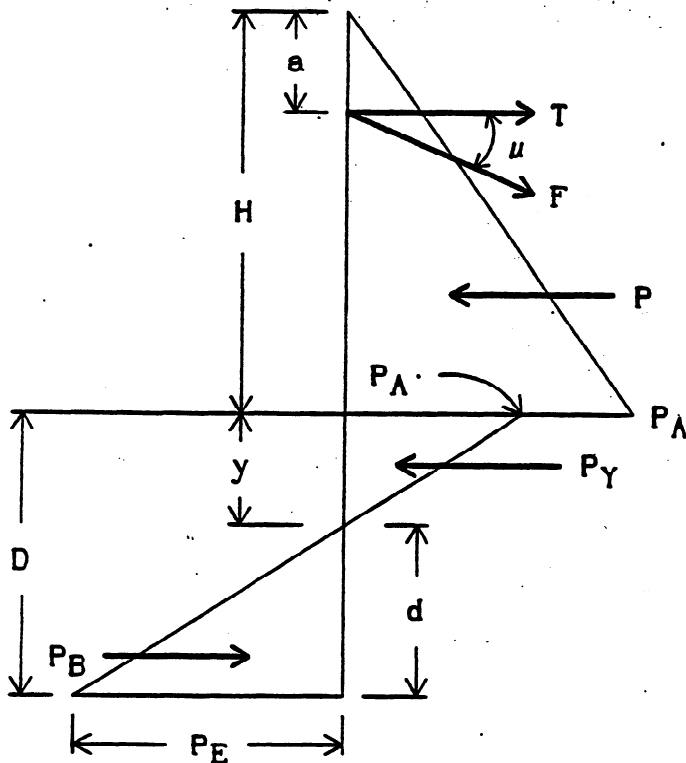
Use W30 X 191's - minimum length of 42' 8", placed in 36" diameter holes and backfilled to bottom of excavation with concrete.

Use 4 x 12's (Rough lumber) for lagging.

The size of the wide flange beam and the diameter of the drilled hole indicate that cantilevered soldier piles would not be the correct type of shoring for the conditions given.

CALIFORNIA TRENCHING AND SHORING MANUAL

SOLDIER PILES W/SINGLE TIEBACK - GRANULAR SOIL



Soil only (No surcharge)
 $a \approx 0.2H$ to $0.4H$

S = Soldier pile spacing

f = Arching factor

$$P_A = \gamma H K_a = K_w H$$

$$P = HP_A/2 = \gamma H^2 K_a / 2$$

$$= K_w H^2 / 2$$

$$P_{A'} = f P_A$$

$$y = P_{A'} / f \gamma (K_p - K_a)$$

$$P_Y = y P_{A'} / 2$$

$$P_E = f \gamma d (K_p - K_a)$$

$$P_B = d P_E / 2$$

FIGURE 10 - 7

Determine D:

$$\Sigma M_T = 0 = P[2H/3 - a] + P_Y[H-a + y/3] - P_B[H - a + y + 2d/3]$$

Solve for d by trial and error (or other means). For the first approximation try $d = H/4.5$.

$D = y + d$. Adjust the value of D or K_p for a safety factor.

Determine T: $T = S(P + P_Y - P_B)$ and $F = T/\cos\mu$

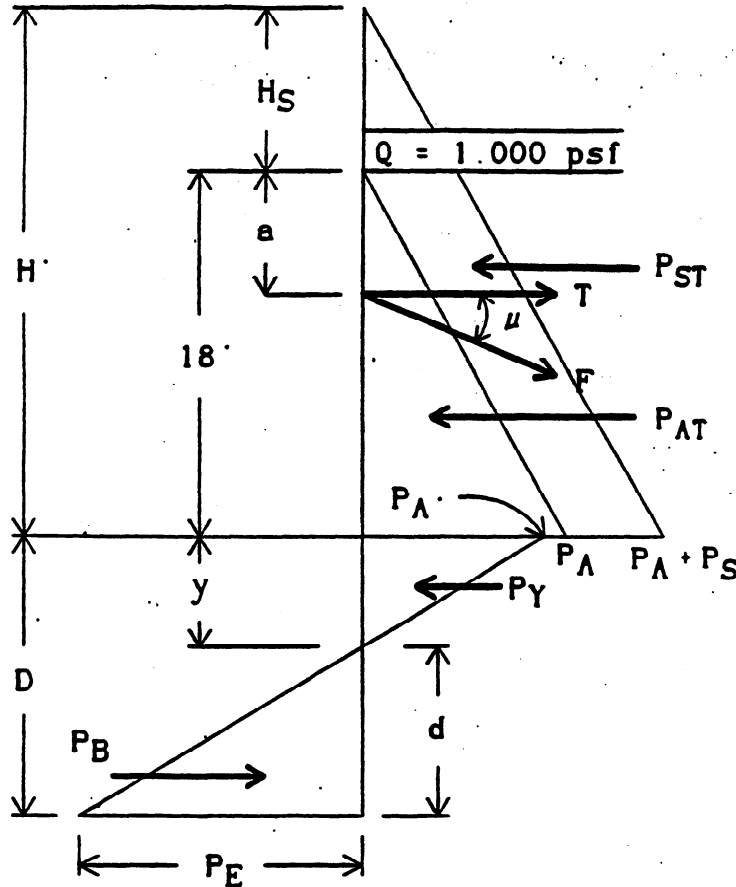
Find the maximum moment. This will generally be either the cantilever section above T or somewhere between T and the excavation level. Determine this second point by locating the point of zero shear.

Determine soldier pile section modulus required, size of lagging needed, and tieback requirements.

Note: A surcharge will normally be present (or the minimum surcharge or 72 psf will be used) and should be added to the equations shown above.

SOLDIER PILES

SAMPLE PROBLEM 10-2: SOLDIER PILE WITH SINGLE TIEBACK



Given:

$K_w = 35 \text{ pcf}$
 $\gamma = 115 \text{ pcf} \quad \mu = 20^\circ$
 $K_a = K_w/\gamma = 0.30$
 $K_p = \gamma/K_w = 3.29$
 $H = 18' \quad a = 6'$

Solution:

$Q = \text{Uniform surcharge}$
 $H_s = 1,000/115 = 8.7'$
 $H' = 18 + 8.7 = 26.7'$
 Pile spacing (S) = 6'
 Pile Hole Diameter = 24"
 Arching capability = 2.0
 $f = (2)(24/12)/6 = 0.67$
 $P_A = K_w H = 35(18) = 630$
 $P_A + P_s = K_w H' = 35(26.7) = 935 \text{ psf}$
 $P_s = 935 - 630 = 305 \text{ psf}$

FIGURE 10 - 8

$$P_{AT} = 630(18/2) = 5670 \text{ Lb/LF}$$

$$P_{ST} = 305(H) = 305(18) = 5490 \text{ Lb/LF}$$

$$P_A' = fP_A = 0.67(630) = 422 \text{ psf}$$

$$y = P_A'/f\gamma(K_p - K_a) = 422/[(0.67)(115)(3.29 - 0.30)] = 1.83$$

$$P_Y = yP_A'/2 = 1.83(422)/2 = 386 \text{ Lb/LF}$$

$$P_E = f\gamma(d)(K_p - K_a) = 0.67(115)(d)(3.29 - 0.30) = 230d$$

$$P_B = dP_E/2 = d(230d)/2 = 115d^2$$

Determine D:

$$\begin{aligned}
 \Sigma M_T = 0 &= P_{AT}[2H/3 - a] + P_Y[H - a + y/3] + P_{ST}[H/2] \\
 &\quad - P_B[H - a + y + 2d/3] \\
 &= 5,670[2(18.0)/3 - 6] + 386[18 - 6 + 1.83/3] \\
 &\quad + 5490[18/2 - 6] - (115d^2)[18 - 6 + 1.83 + 2d/3] \\
 &= 55,357.5 - 1590.5d^2 - 76.7d^3 = d^3 + 20.7d^2 - 721.7
 \end{aligned}$$

CALIFORNIA TRENCHING AND SHORING MANUAL

By trial and error, or by other means, $d = 5.27'$

$$D = d + y = 5.27 + 1.83 = 7.10'$$

$$\text{Increase } D \text{ by } 30\% \text{ for safety factor: } D = 1.3(7.1) = 9.2'$$

Determine T:

$$P_B = 115(5.27)^2 = 3,194 \text{ Lb/LF}$$

$$T = P_{AT} + P_{ST} + P_y - P_B = 5,670 + 5,490 + 386 - 3,194 = 8,352 \text{ Lb/LF}$$

$$F = T/\cos\mu = 8,352/\cos 20^\circ = 8,888 \text{ Lb/LF}$$

$$\text{Total } F = 6(8,888) = 53,328$$

Find Maximum Moment (See Note)

Locate point of zero shear (x).

$$8,352 = P_A(x/18)(x/2) + P_Sx$$

$$8,352 = 17.5x^2 + 305x$$

$$x = 14.8'$$

$$M = -(630)(14.8/18)(14.8/2)[14.8/3]$$

$$- (305)(14.8)[14.8/2]$$

$$+ (8,352)[14.8 - 6.0]$$

$$= 21,184 \text{ Ft Lb/LF}$$

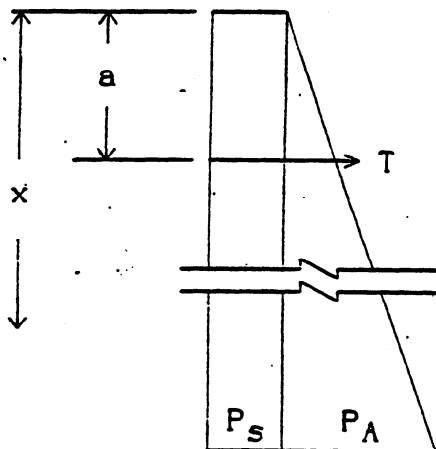


FIGURE 10 - 9

Check Cantilever:

$$M = \{(630)(6.0/18)(6.0/2)\}[6.0/3] = 6,750 \text{ Ft-Lb/LF}$$

This does not control

$$S \text{ Required} = 21,184(6)(12)/22,000 = 69.3 \text{ in}^3$$

$$\text{Use } W14 \times 53, \quad S = 77.8 \text{ in}^3$$

Determine lagging and tieback requirements.

See alternate analysis using AISC specifications at end of Chapter.

Note: When the soldier pile is encased in 4 sack or better concrete, the buried portion of the pile acts as a composite section which will have a large section modulus. When this is the case the moment at the excavation line may often be controlling.

SOLDIER PILES

SAMPLE PROBLEM 10-3: SOLDIER PILE WITH SINGLE TIEBACK

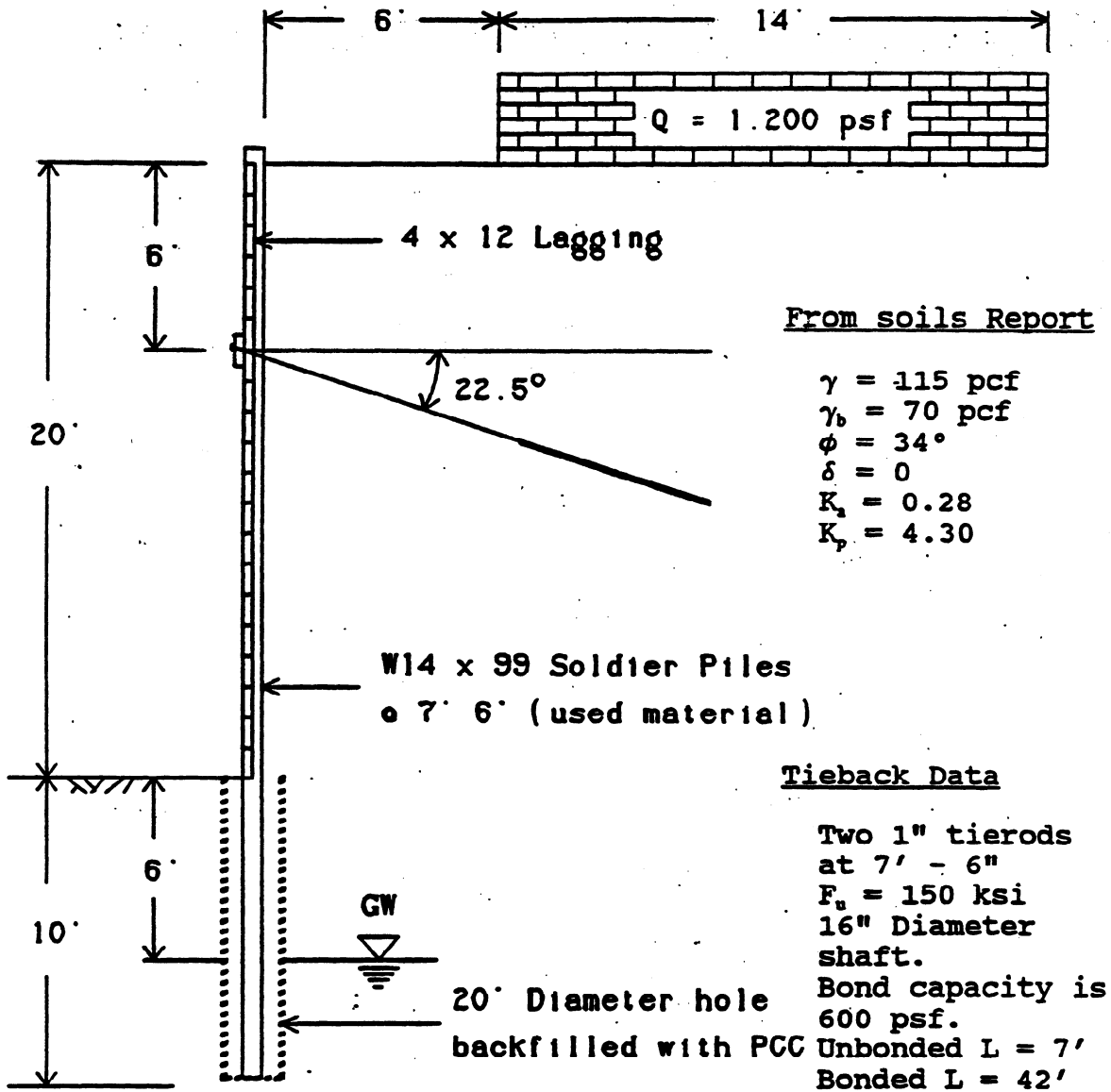


FIGURE 10 - 10

Determine Lateral Pressures:

FHWA recommends adjusting K_p by division with 1.5 in lieu of increasing D by 20% - 40%. (This is conservative)

Adjust K_p by 1.5. For this example $K_p = 4.3/1.5 = 2.87$

$$K_p - K_a = 2.87 - 0.28 = 2.59$$

Arching capability = $0.08(34^\circ) = 2.7$ Use 1.5 due to wet condition (Need to be conservative next to building).

$$f = 1.5(20/12)/7.5 = 0.33$$

CALIFORNIA TRENCHING AND SHORING MANUAL

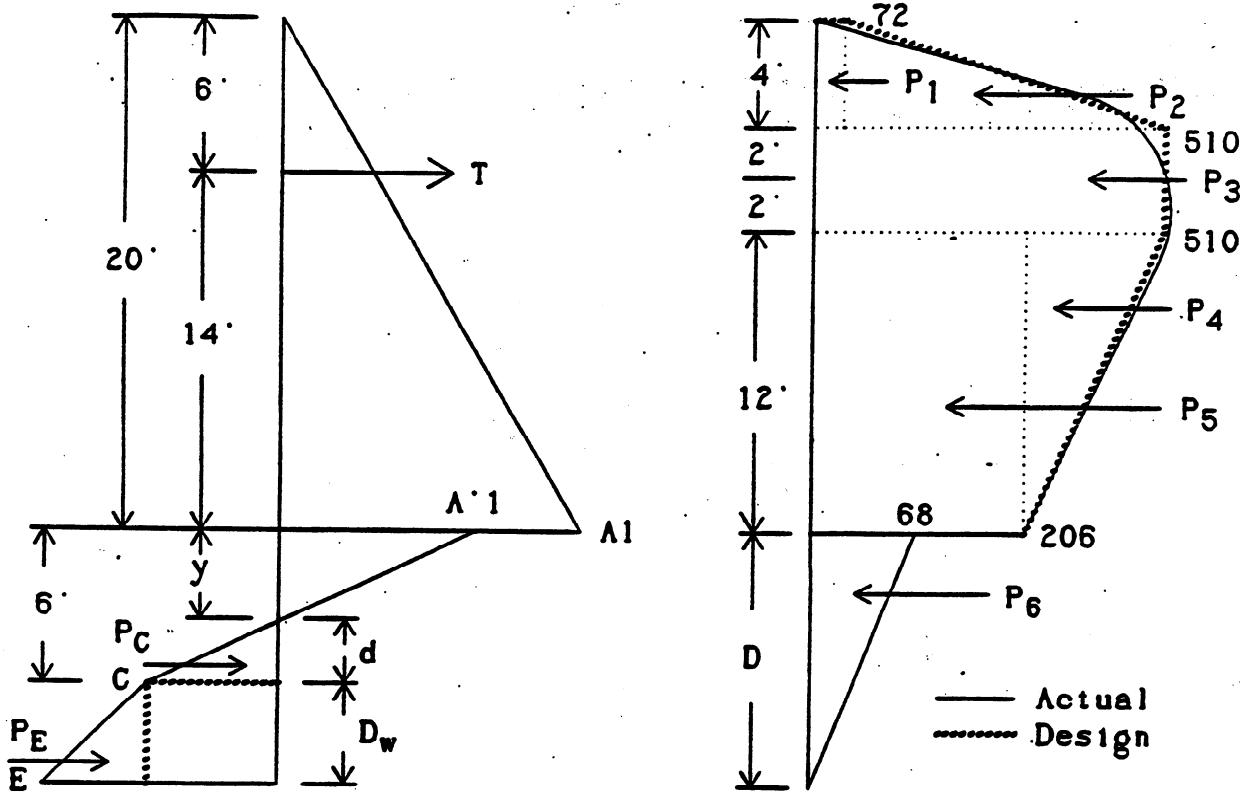


FIGURE 10 - 11

$$P_{A1} = \gamma HK_s = 115(20)(0.28) = 644 \text{ psf}$$

$$P_{A'1} = fP_{A1} = 0.33(644) = 213 \text{ psf}$$

$$Y = P_{A'1}/f\gamma(K_p - K_s) = 213/[(0.33)(115)(2.59)] = 2.17'$$

$$d = 6 - 2.17 = 3.83'$$

$$P_C = f\gamma d(K_p - K_s) = 0.33(115)(3.83)(2.59) = 376 \text{ psf}$$

$$P_E = P_C + f\gamma_b D_w(K_p - K_s) = 376 + 0.33(70)(2.59)D_w = 376 + 60D_w$$

$$P_1 = 72(4) = 288 \text{ psf}$$

$$P_2 = (510 - 72)(4)/2 = 876 \text{ psf}$$

$$P_3 = 510(4) = 2,040 \text{ psf}$$

$$P_4 = (510 - 206)(12)/2 = 1,824 \text{ psf}$$

$$P_5 = 206(12) = 2,472 \text{ psf}$$

Below excavation: Consider arching effect. Boussinesq surcharges may be tapered to zero at the calculated depth D.

$$206(0.33) = 68 \text{ psf}$$

$$P_6 = 68(6 + D_w)/2 = 204 + 34D_w$$

SOLDIER PILES

Determine D:

$$\Sigma m_T = 0$$

Moment due to surcharge:

$$\begin{aligned} & (204 + 34D_w)[14 + (6 + D_w)/3] + 2,472[8] + 1,824[6] \\ & + 2,040[0] - 876[3.33] - 288[4] \\ & = 11D_w^2 + 612D_w + 29,915 \end{aligned}$$

Moment due to soil:

$$\begin{aligned} & \{644(20)/2\}[7.33] + \{213(2.17)/2\}[14.72] \\ & - \{376(3.83)/2\}[18.72] - 376(D_w)[20 + D_w/2] \\ & - \{60D_w\}(D_w)/2[20 + (2/3)D_w] \\ & = -20D_w^3 - 788D_w^2 - 7,520D_w + 37,128 \end{aligned}$$

Combined moment:

$$\begin{aligned} 20D_w^3 + 777D_w^2 + 6,908D_w - 67,043 &= 0 \\ \text{or } D_w^3 + 39D_w^2 + 345D_w - 3,352 &= 0 \end{aligned}$$

From which $D_w = 5.62'$

$$D = D_w + 6 = 5.62 + 6 = 11.62' \quad (\text{Use } D = 11'-8")$$

Determine T:

$$\begin{aligned} \Sigma F_H &= 0 \\ &= 288 + 876 + 2,040 + 1,824 + 2,472 + \{204 + 34(5.62)\} \\ &+ 644(20)/2 + 213(2.17)/2 - 376(3.83)/2 - 376(5.62) \\ &- 60(5.62)(5.62/2) - T \end{aligned}$$

$$\therefore T = 10,785 \text{ Lb/LF}$$

$$\text{Total } T = 10,785(7.5) = 80,888 \text{ Lb}$$

Find Maximum Moment:

Check cantilever moment at T:

$$\begin{aligned} M &= \{644(6/20)(6/2)\}[6/3] + 288[2 + 4/2] + 876[2 + 4/3] \\ &+ 510(2)[2/2] \\ &= 6,251 \text{ Ft-Lb/LF} \end{aligned}$$

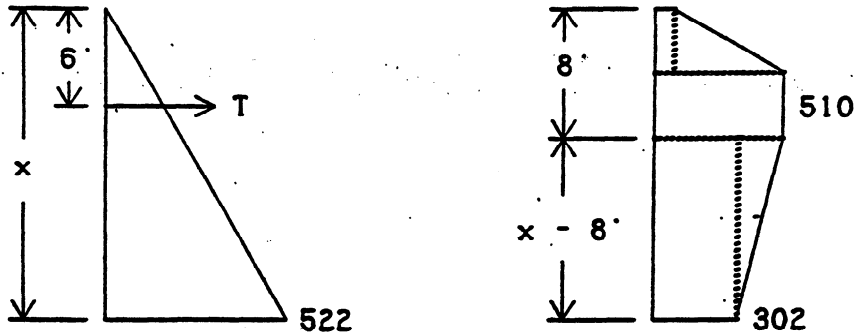


FIGURE 10 - 12

Locate point of zero shear:

$$10,785 - 288 - 876 - 2,040 - 644(x/20)(x/2) - \{x - 8\}\{510 + [206 + (510 - 206)(20 - x)/12]\}/2 = 0$$

$$x^2 + 207.6x - 3,632.5 = 0$$

$$\therefore x = 16.23'$$

Pressure at point of zero shear due to soil:

$$644(16.2/20) = 522 \text{ psf}$$

Pressure at point of zero shear due to surcharge:

$$206 + (510 - 206)(20 - 16.2)/12 = 302 \text{ psf}$$

Moment due to tieback and soil:

$$10,785[16.2 - 6] - \{(522)(16.2/2)\}[16.2/3] = 87,175 \text{ Ft-Lb/LF}$$

Moment due to surcharge:

$$\begin{aligned} M &= - 288[16.2 - 4/2] - 876[16.2 - 4(2/3)] \\ &\quad - 2,040[16.2 - 4 - 4/2] - 302(16.2 - 8)[(16.2 - 8)/2] \\ &\quad - \{(510 - 302)(16.2 - 8)/2\}[(16.2 - 8)(2/3)] \\ &= - 51,568 \text{ Ft-Lb/LF} \end{aligned}$$

Combined moment (assuming non-compact section):

$$87,175 - 51,568 = 35,607 \text{ Ft-Lb/LF} \quad \text{This controls.}$$

$$S \text{ Required} = 35,607(7.5)(12)/22,000 = 145.67 \text{ in}^3$$

$$S \text{ furnished} = 157 \text{ in}^3 > 145.67 \text{ in}^3 \quad \therefore \text{O.K.}$$

See alternate analysis using AISC specifications on page 10-21.

SOLDIER PILES

Check Lagging:

Consider arching effect on lagging. Multiply all pressure results by 0.6. By inspection, maximum moment occurs at the depth of excavation.

$$M_{\max} = wL^2/8 = (644 + 206)(7.5)^2/8 = 5,977 \text{ Ft-Lb}$$

$$S \text{ Required} = 5,977(12)(0.6)/1,500(1.0)* = 28.7 \text{ in}^3$$

* Load duration factor due to high risk building.

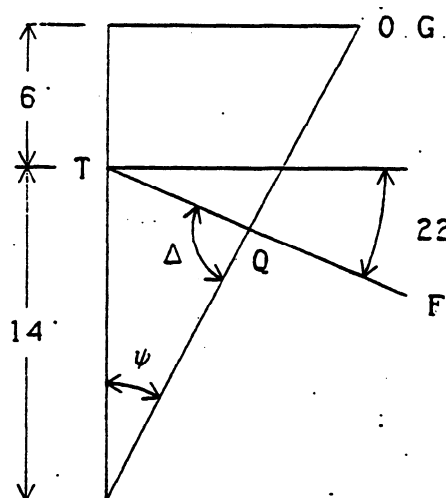
Use 4 x 12's (Rough lumber). $S = 32 \text{ in}^3$

Check Shear:

$$V = (7.5/2 - 0.33)(850)(0.6) = 1,744 \text{ Lb}$$

$$v = 3V/2A = 3(1,744)/2(4)(12) = 54.5 \text{ psi} < 140 \text{ psi} \therefore \text{O.K.}$$

Tiebacks:



$$F = 80,888/\text{COS } 22.5^\circ = 87,553 \text{ Lb}$$

$$\psi \approx 45^\circ - \phi/2 \approx 45^\circ - 34^\circ/2 \approx 28^\circ$$

$$\Delta = 180^\circ - (90^\circ - 22.5^\circ) - \psi = 84.5^\circ$$

$$TQ = \text{Unbonded length}$$

$$= (14 \sin 28^\circ)/\sin \Delta = 6.6'$$

$$6.6' < 7.0' \text{ provided } \therefore \text{O.K.}$$

Bond capacity given as 600 psf

FIGURE 10 - 13

$$\text{Bonded frictional resistance (per foot)} = \pi(16/12)(600)(L)$$

$$= 2,513(L) \text{ Lb/LF}$$

$$\text{Length needed} = 87,553/2,513 = 34.8' < 42' \therefore \text{O.K.}$$

$$\text{Safety Factor} = 42/34.8 = 1.21 = (21\%)$$

Note: Vertical downward component of tie may be used in conjunction with wedge weights in stability analysis to counteract slip circle failure.

$$\text{Vertical component} = 87,553 \sin 22.5^\circ = 33,505 \text{ Lb}$$

CALIFORNIA TRENCHING AND SHORING MANUAL

SOLDIER PILE WITH SINGLE TIEBACK

Soldier piles in the two previous problems were not checked for compressive stress or for the combined stresses due to the vertical component of the tieback force. AISC criteria may be used to check combined stresses. For Sample Problems 15 and 16 assume maximum unbraced length is the cantilever or the length between the tie and the point where the passive soil resistance becomes effective.

SAMPLE PROBLEM 10 - 2 - SOLDIER PILE: (ALTERNATE ANALYSIS)

Pile properties: $A = 15.6 \text{ in}^2$ $r_x = 5.89 \text{ in}$

Assume $k = 1$, then $KL/r = 15.8(12)/5.89 = 32.19$

From AISC: $M_r = 142.6 \text{ Ft-k}$ and $F_c = 19.79 \text{ ksi}$

Downward tie force = $F \sin \omega = 53.3 \sin 20^\circ = 18 \text{ k}$

$M_{\max} = 6(21.18) = 127.1 \text{ Ft-k}$

$$\begin{aligned} f_a/F_c + f_b/F_b &= f_a/F_c + M_{\max}/M_r \leq 1.0 \\ &= (18/15.6)/19.79 + 127.1/142.6 \\ &= 0.06 + 0.89 = 0.95 \leq 1.0 \quad \text{OK} \end{aligned}$$

SAMPLE PROBLEM 10 - 3 - SOLDIER PILE: (ALTERNATE ANALYSIS)

Pile properties: $A = 29.1 \text{ in}^2$ $r_x = 6.14 \text{ in}$

Assume $k = 1$; then $KL/r = 16.2(12)/6.14 = 31.7$

From AISC: $M_r = 288 \text{ Ft-k}$ and $F_c = 19.82 \text{ ksi}$

Downward tie force = $F \sin \omega = 87.6 \sin 22.5^\circ = 33.5 \text{ k}$

$M_{\max} = 7.5(35.6) = 267 \text{ Ft-k}$

$$\begin{aligned} f_a/F_c + f_b/F_b &= f_a/F_c + M_{\max}/M_r \leq 1.0 \\ &= (33.5/29.1)/19.82 + 267/288 \\ &= 0.06 + 0.93 = 0.99 \leq 1.0 \quad \text{OK} \end{aligned}$$

SOLDIER PILES

SAMPLE PROBLEM 10-4: SOLDIER PILE WITH RAKER

Given:

From Soils
Report:

$$\begin{aligned} \gamma &= 115 \text{ pcf} \\ \phi &= 35^\circ \\ \delta &= 0 \end{aligned}$$

Allowable Soil
Bearing = 2 ksf

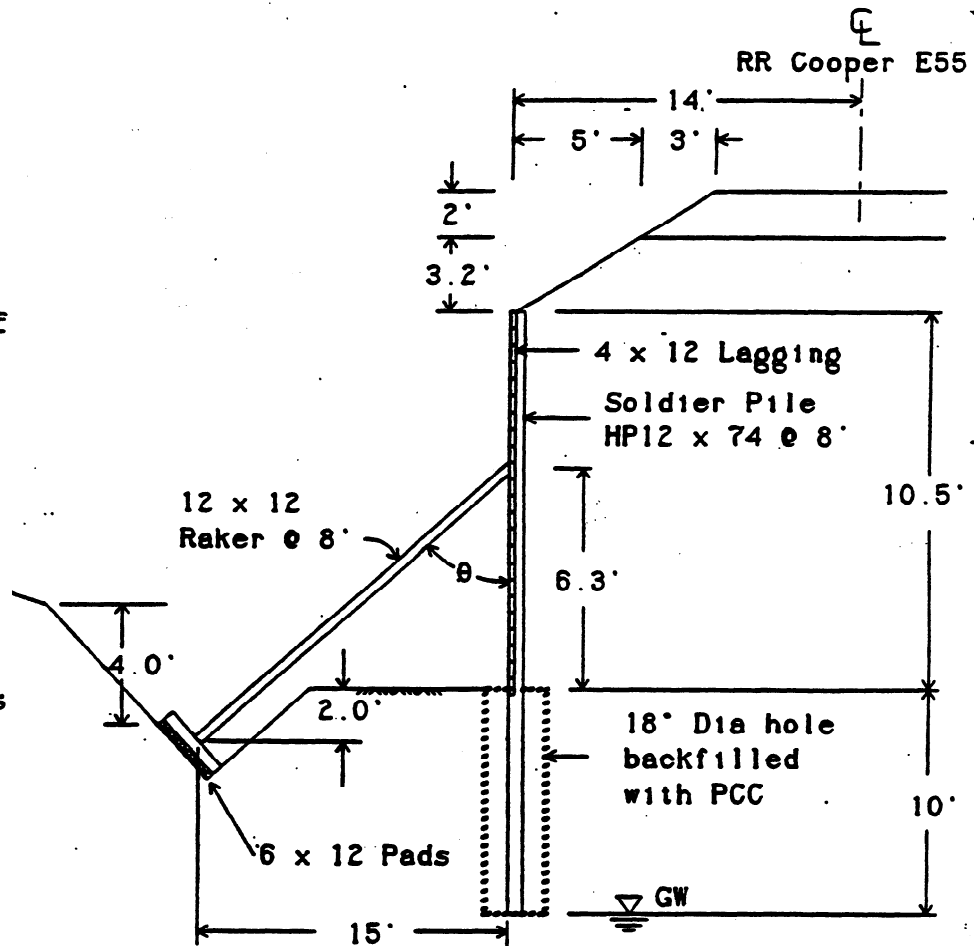
Loose Ballast:

$$\gamma = 115 \text{ pcf}$$

Rough lumber is
to be used

$$\begin{aligned} \beta &= \tan^{-1}(5.2/8) \\ &= 33^\circ \end{aligned}$$

$$\begin{aligned} \beta/\phi &= 33^\circ/35^\circ \\ &= 0.94 \end{aligned}$$



Solution:

FIGURE 10-14

NOTE: Assume that the back slope and ballast are one unit since their densities are equal and their angles are practically the same. If this was not the case, or if $\beta/\phi > 1$ then Trial Wedge or similar analysis would have to be used to solve the problem.

From Figure 8, $K_1 = 0.65$; $K_p = \tan^2(45^\circ + 35^\circ/2) = 3.69$

$$P_A = K_1 \gamma H = 0.65(115)(10.5) = 785 \text{ psf}$$

Adjust K_p per FHWA recommendation in lieu of increasing D 20%-40%.

Adjust K_p by 1.5: $K_p = 3.69/1.5 = 2.46$

$$K_p - K_1 = 2.46 - 0.65 = 1.8$$

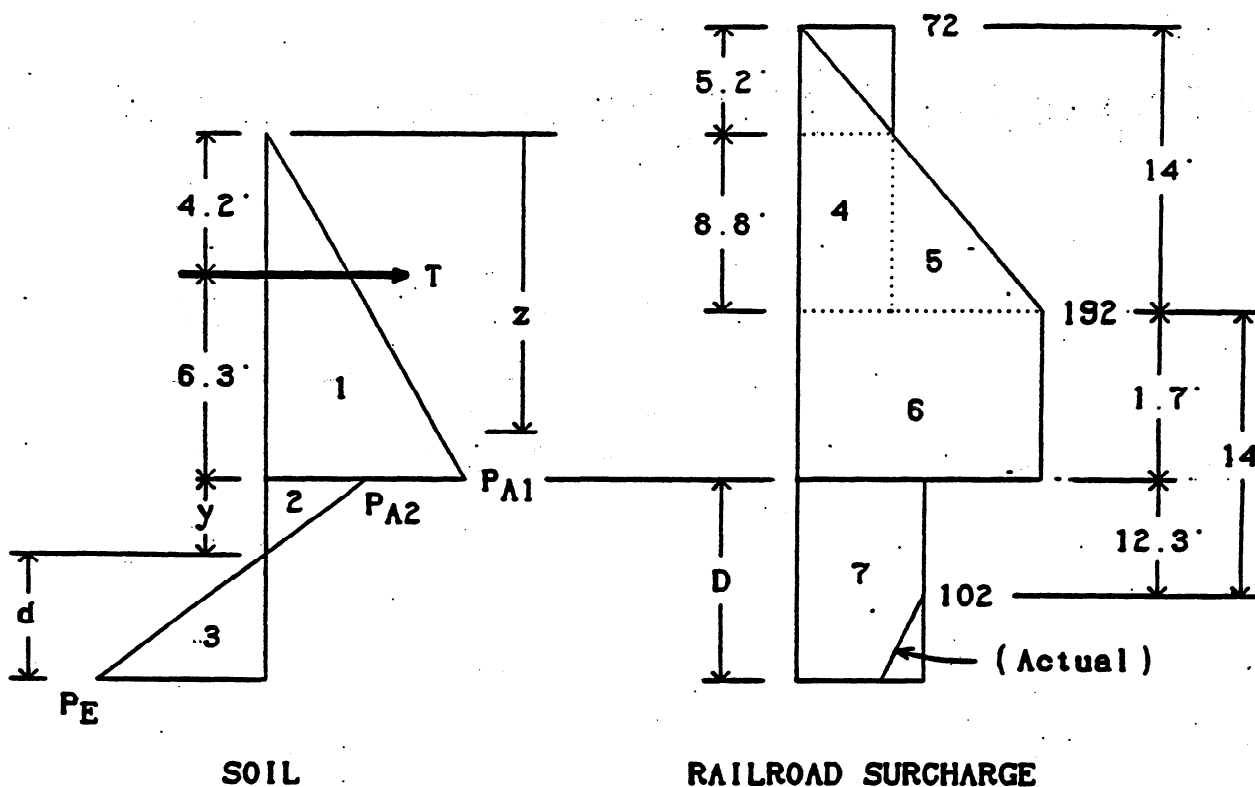


FIGURE 10 - 15

NOTE:- For ease of computation assume that the railroad surcharge has a rectangular shape below the excavation. The railroad surcharge is prorated from "CHART 3.6, LATERAL PRESSURE FOR COOPER RAILROAD LIVE LOAD": $279(E55/E80) = 192$ psf. Note that the top of the railroad surcharge diagram is always located at the elevation of the top of the rail.

Soil and surcharge act on adjusted width of soldier pile. Determine f and multiply all pressures below the excavation line by this factor.

Passive arching capability = $0.08(35^\circ) = 2.8$

Diameter of drilled hole = $18/12 = 1.5'$

Arching factor (f) = $2.8(1.5)/8 = 0.53$

$P_{A2} = 0.53P_{A1} = 0.53(785) = 416$ psf

Surcharge at excavation line = $0.53(192) = 102$ psf

$y = P_{A2}/f\gamma(K_p - K_s) = 416/[0.53(115)(1.8)] = 3.79'$

$P_E = f\gamma d(K_p - K_s) = 0.53(115)(1.8)d = 110d$

Determine D:

$D = d + y$

$\Sigma M_T = 0$

SOLDIER PILES

	<u>AREA</u>	<u>ARM</u>	<u>MOMENT</u>
1	$785(10.5)/2 = 4,121$	$2/3(10.5) - 4.2 = 2.80$	11,539
2	$416(3.79)/2 = 788$	$6.3 + 3.79/3 = 7.56$	5,957
4	$72(8.8) = 634$	$8.8/2 - 4.2 = 0.20$	127
5	$(192 - 72)(8.8)/2 = 528$	$2/3(8.8) - 4.2 = 1.66$	876
6	$192(1.7) = 326$	$6.3 - 1.7/2 = 5.45$	<u>1,777</u>
	<u>6,397</u>		<u>20,276</u>

3 AREA = $-110d(d/2) = -55d^2$
 ARM = $6.3 + 3.79 + 2/3(d) = 10.09 + 0.67d$
 MOMENT = $-554.95d^2 - 36.85d^3$

7 AREA = $102(3.79 + d) = 386.58 + 102d$
 ARM = $6.3 + (3.79 + d)/2 = 8.20 + 0.5d$
 MOMENT = $3,169.96 + 1,029.69d + 51d^2$

$\therefore 0 = 23,445.96 + 1,029.69d - 503.95d^2 - 36.85d^3$

Solve for d and D: $d = 6.37'$
 $D = d + y = 6.37 + 3.79 = 10.16'$

Determine T:

T = Σ Active areas - Σ passive areas
 $= 6,397 + 102(6.37 + 3.79) - (6.37)^2(110)/2 = 5,202 \text{ Lb/LF}$

Check Soldier Pile: Find M_{max}

By inspection, the point of zero shear will be above the excavation.

$0 = 785z/10.5(z/2) + 634 + 528 + 192(z - 8.8) - 5,202$
 $= 37.38z^2 - 4,040 + 192z - 1,689.60 = z^2 + 5.14z - 153.28$
 $\therefore z = 10.08' \approx 10.1'$

$M_{max} = 37.38(10.1)^2[10.1/3] + 634[10.1 - 8.8/2]$
 $+ 528[10.1 - 2/3(8.8)] + 192(10.1 - 8.8)[(10.1 - 8.8)/2]$
 $- 5,202[10.1 - 4.2]$
 $= 11,843 \text{ Ft-Lb/LF}$

Total Moment = $8(11,843) = 94,744 \text{ Ft-Lb}$

S Required = $M/F_b = 94,744(12)/22,0000 = 51.68 \text{ in}^3$
 S Furnished = $93.8 \text{ in}^3 > 51.68 \text{ in}^3$

CALIFORNIA TRENCHING AND SHORING MANUAL

Check Lagging:

Pressure is greatest at the bottom of the excavation:

$$785 + 192 = 977 \text{ psf}$$

$$M = wL^2/8 = 977(12)(8)^2/8 = 93,792 \text{ In-Lb}$$

$$S \text{ Required} = 93,792(0.6)/(1,500)(1.0)^* = 37.52 \text{ in}^3$$

* No load duration factor used when adjacent to railroads.

$$S \text{ for a } 4 \times 12 = 12(4)^2/6 = 32 \text{ in}^3 < 37.52 \text{ in}^3$$

$$S \text{ for a } 6 \times 12 = 12(6)^2/6 = 72 \text{ in}^3 \quad \text{Use } 6 \times 12 \text{ lagging}$$

$$V = (8/2 - 0.33)(977)(0.6) = 2,151 \text{ Lb}$$

$$f_v = 3V/2A = 3(2,151)/[2(6)(12)] = 44.8 \text{ psi} < 140 \text{ psi} \quad \therefore \text{OK}$$

Check Raker:

$$T = 8(5,202) = 41,616 \text{ Lb}$$

$$\text{Angle } \theta = \tan^{-1} 15/8.3 = 61^\circ$$

$$L = [(15)^2 + (8.3)^2]^{1/2} = 17.14 \text{ Ft}$$

$$\text{Axial load} = 41,616(17.14/15) = 47,553 \text{ Lb}$$

$$P/A = 47,553/(12)(12) = 330 \text{ psi}$$

$$\text{Allowable } F_c = 480,000/(L/d)^2 = 480,000/[(17.14)(12)/12]^2 = 1,634 \\ = 1,600 \text{ psi max}$$

$$330 < 1,600 \quad \therefore \text{OK}$$

Check Pad:

Determine allowable soil pressure under 6 x 12 pads:

(Use NAVFAC inclined load on inclined footing - See Appendix B)

$$\text{Angle of pad to horizontal} = 61^\circ$$

$$D/B = (4.0/5.0) = 0.8 \quad N_{\gamma q} \text{ From graph} = 20$$

$$q_{ult} = CN_{cq} + 1/2(\gamma B)N_{\gamma q} = 0 + 1/2(115)(5)(20) = 5,750 \text{ psf}$$

$$q_{Allowable}/FS = 5,750/2 = 2,875 > 2,000 \text{ psf} \quad \text{Use } 2,000 \text{ psf}$$

$$\text{Pad bearing area needed} = 47,553/2,000 = 23.78 \text{ Ft}^2$$

$$\text{Pad length needed} = 23.78/5.00 = 4.76 \text{ Ft}$$

$$\text{Pad cantiliver length} = (4.76 - 1.00)/2 = 1.88 \text{ Ft}$$

SOLDIER PILES

$$M \text{ (for 1 pad)} = wL^2/2 = 2,000(1.88)^2/2 = 3,534 \text{ Ft-Lb}$$

$$f_b = M/S = 3,534(12)/[12(6)^2/6] = 589 < 1,500 \text{ psi}$$

$$\text{Shear } V = 2,000(1.88 - 0.5) = 2,760 \text{ Lb}$$

$$f_v = 3V/2A = 3(2,760)/2(6 \times 12) = 58 < 140 \text{ psi}$$

Check Corbel:

$$\text{Raker to corbel crushing} = 47,553/12 \times 12 = 330 < 450 \text{ psi}$$

$$\text{Length for flexure} = (5.00 - 1.00)/2 = 2.00$$

$$\text{Load per foot of corbel} = 2,000(4.76) = 9,520 \text{ Lb/Ft}$$

$$M = wL^2/2 = 9,520(2)^2/2 = 19,040 \text{ Ft-Lb}$$

$$f_b = M/S = 19,040(12)/[12(12)^2/6] = 793 < 1,800 \text{ psi}$$

$$\text{Length for shear} = 5.00/2 - 1.00/2 - 1.00 = 1.00 \text{ Ft}$$

$$\text{Shear } V = 2,000(4.76)(1.00) = 9,520 \text{ Lb}$$

$$f_v = 3(9,520)/2(12 \times 12) = 99 < 144 \text{ psi}$$

Summary:

Use HP12 x 74 (equivalent or larger) soldier pile (D = 10'- 3").

Use 6 x 12's for lagging (may use 4 x 12 for upper half).

Use 12 X 12 raker.

Provide at least 5 x 5 pad.

CALIFORNIA TRENCHING AND SHORING MANUAL

SAMPLE PROBLEM 10-5: PREVIOUS PROBLEM WITH NO RAKER

Given:

Analyze previous problem using an H of 8 feet and no raker.

Solution:

Instead of adjusting K_p per FHWA recommendation, use a safety factor of 30% for D.

$$K_a = 0.65 \quad K_p = 3.69 \quad (\text{Arbitrary values})$$

$$K_p - K_a = 3.69 - 0.65 = 3.04$$

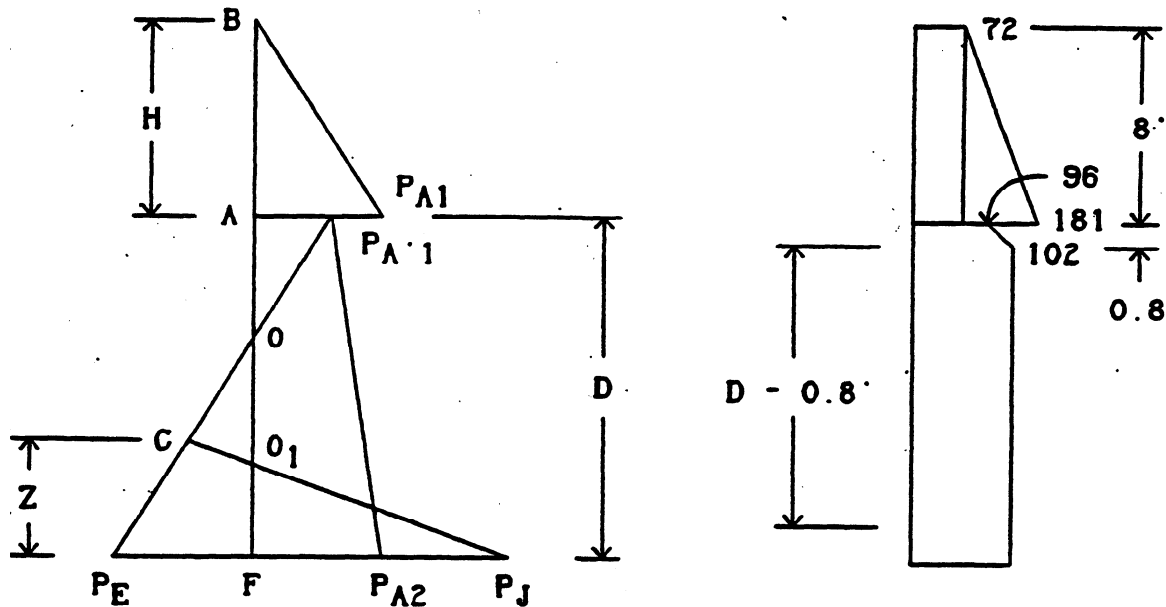


FIGURE 10 - 16

$$P_{A1} = K_a \gamma H = (0.65) (115) (8) = 598 \text{ psf}$$

$$P_{A'1} = (0.53) (598) = 317 \text{ psf}$$

$$P_{A2} = P_{A'1} + f \gamma D K_a = 317 + (0.53) (115) (0.65) D - 317 = 317 + 40D$$

$$P_E = f \gamma D (K_p - K_a) - P_{A'1} = (0.53) (115) (3.04) D - 317 = 185D - 317$$

$$P_J = f \gamma D (K_p - K_a) + f \gamma H K_p = (0.53) (115) (3.04) D + (0.53) (115) (8) (3.69) = 1,799 + 185D$$

SOLDIER PILES

Areas:

$$ABA_1 = 598(8)/2 = 2,392$$

$$AA'_1A_2F = (317 + 317 + 40D)D/2 = 317D + 20D^2$$

$$ECJ = (185D - 317 + 1,779 + 185D)Z/2 = 185DZ + 731Z$$

$$EA'_1A_2 = -(185D - 317 + 317 + 40D)D/2 = -113D^2 \quad (\text{negative area})$$

Surcharge area (for ease of computation assume surcharge uniform below excavation):

$$72(8) + (181 - 72)(8)/2 + (102)D = 1,012 + 102D$$

$$\Sigma F_H = 0 \quad \therefore ABA_1 + AA'_1A_2f + ECJ - EA'_1A_2 + \text{surcharge} = 0$$

$$2,392 + 317D + 20D^2 + 185DZ + 731Z - 113D^2 + 1,012 + 102D = 0$$

$$z = (D^2 - 4.51D - 36.60)/(1.99D + 7.86)$$

$$\Sigma M_P = 0$$

$$= ABA_1[D + 8/3] + (P_{A1})(D)[D/2] + (P_{A2} - P_{A1})(D/2)[D/3] \\ + ECJ[Z/3] - EA'_1A_2[D/3] + 576[8/2 + D] + 436[8/3 + D] \\ + 102D[D/2]$$

$$2,392[D + 2.7] + 158D^2 + 6.67D^3 + (185DZ + 731Z)[Z/3] \\ - 37.67D^3 + (2,304 + 576D) + (1,163 + 436D) + 51D^2 = 0$$

$$z^2 = (D^3 - 6.74D^2 - 109.81D - 320.17)/(1.99D + 7.86)$$

By trial and error, or by other means: $Z = 4.15'$ and $D = 16.88'$

Increase D by 30% $D = 1.30(16.88) = 21.9 \text{ Ft}$

Use $D = 22 \text{ Ft}$

Check Soldier Pile: Find m_{ax}

Locate point of zero shear.
Assume point between O & G).

$$AO/P_{A1} = D/(P_{A1} + P_E) \\ AO = 1.71'$$

$$OG = D - AO - z \\ = 16.88 - 1.71 - 4.15 = 11.02'$$

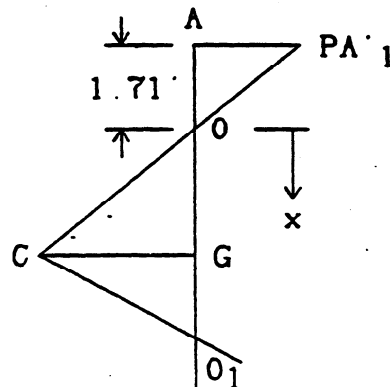


FIGURE 10 - 17

CALIFORNIA TRENCHING AND SHORING MANUAL

$$\begin{aligned} \text{Shear at point 0} &= 2,392 + 576 + 436 + 317(1.71)/2 + (102)(1.71) \\ &= 3,849 \text{ Lb/LF} \end{aligned}$$

$$3,849 + 102x = 185x(x/2)$$

$$92.5x^2 - 102x - 3,849 = 0$$

$$x^2 - 1.10x - 41.61 = 0 \quad \therefore x = 7.02' \text{ (and assumption is correct)}$$

$$AO + x = 1.71 + 7.02 = 8.73'$$

$$\begin{aligned} M_{\max} &= 2,392[8/3 + 8.73] + 576[8/2 + 8.73] + 436[8/3 + 8.73] \\ &\quad 271.0[(1.71)(2/3) + 7.02] + 102(8.73)[8.73/2] \\ &\quad - (185)(7.02)(7.02/2)[7.02/3] \\ &= 34,994 \text{ Ft-Lb/LF} \end{aligned}$$

Total soldier pile moment = moment per foot times pile spacing:

$$= 8(34,994) = 279,952 \text{ Ft-Lb}$$

$$S \text{ required} = M/F_b = 279,952(12)/22,000 = 152.7 \text{ in}^3$$

$$S \text{ furnished with HP12 x 84} = 106 \text{ in}^3 < 152.7 \text{ in}^3$$

$$\text{Use HP14 x 117, } S = 172 \text{ in}^3$$

A second point of zero shear occurs near the computed depth, but this point is not normally used for maximum moment.

Compare the moment computed above to the moment at the depth of excavation:

$$M = 2,392[8/3] + 576[8/2] + 436[8/3] = 9,846 < 34,994 \text{ Ft-Lb/Lf}$$

The portion of piling encased in sound concrete (generally, four sack or better) comprises a composite section usually having a large section modulus. If this is the case, the moment at or above the excavation elevation may be controlling to determine the critical section modulus.

SOLDIER PILES

CANTILEVER SOLDIER PILE - COHESIVE SOIL ($\phi = 0$ METHOD)

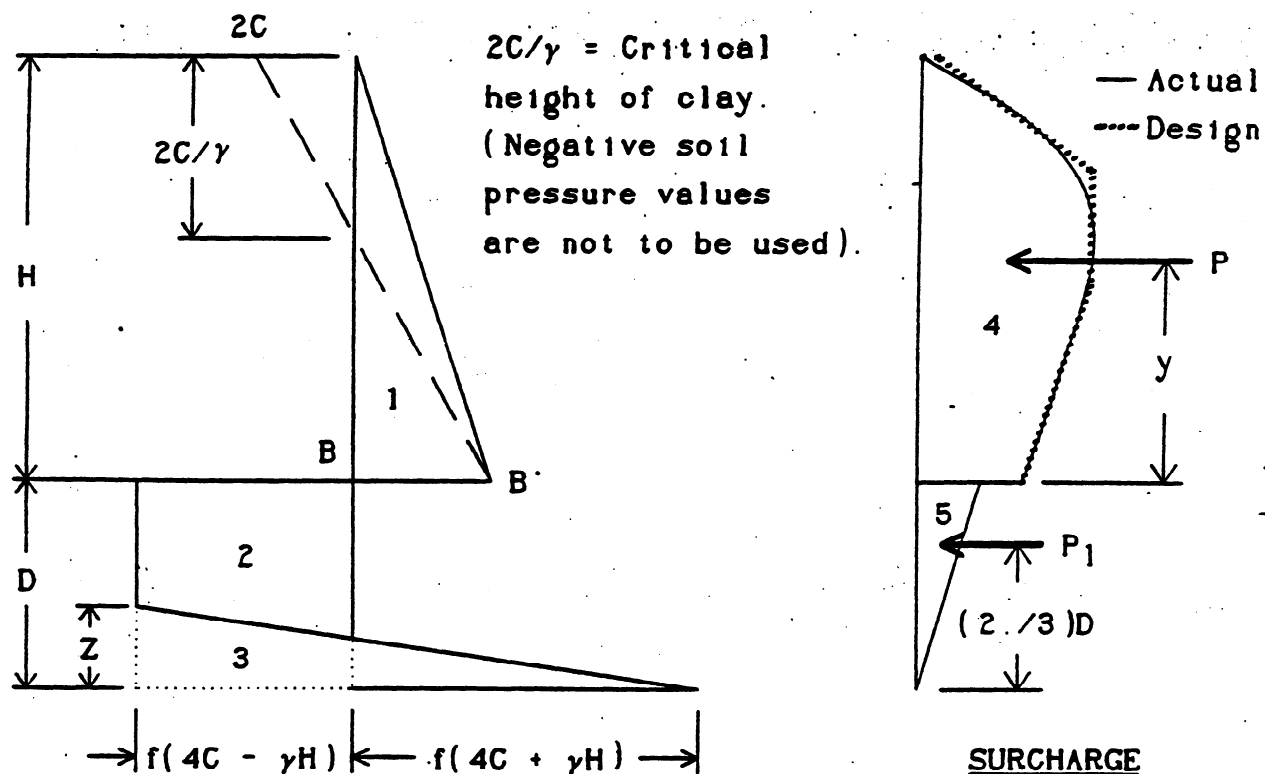


FIGURE 10-38

Use a safety factor of 50% - 70% with the clay or increase D by 20% - 40%. Critical height of wall = $H_c = 4C/\gamma$. Theoretically the wall will fail if $\gamma H_c > 4$.

$BB' = \gamma H - 2C \geq 0$. (If not, see note below)
f = Arching factor

1 = $BB' (H/2)$

2 = $f(4C - \gamma H)D$

3 = $\{f(4C - \gamma H) + f(4C + \gamma H)\} \{Z/2\} = 4fCZ$

4 = P = Area under dashed line above the excavation depth.

5 = P_1 = Area below the excavation depth (when used).

1) $\Sigma F_H = 0 = 1 - 2 + 3 + 4 + 5$ and $Z = (2 - 1 - 4 - 5) / 4fC$

2) $\Sigma M_{base} = 0 = 1[H + H/3] - 2[D/2] + 3[Z/3] + 4[D + y] + 5[2D/3]$

Solve equations 1) and 2) simultaneously for D and Z.
Determine maximum moment and section modulus required.
Determine lagging requirements.

*Note : If ϕ does not = 0, or if $BB' < 0$, see next page.

CALIFORNIA TRENCHING AND SHORING MANUAL

CANTILEVER SOLDIER PILE - COHESIVE SOIL (ALTERNATE METHOD)

This approach should be used only when $\phi \neq 0$, or when $BB' \leq 0$.

If $\phi \neq 0$, then $BB' = \gamma K_a H$ where $K_a = \tan^2(45^\circ - \phi/2)$

If $BB' < 0$, then assume $C = 0$ and $\phi = 20^\circ$ to 30° .
 $BB' = \gamma H K_a$ where $K_a = \tan^2(45^\circ - \phi/2)$

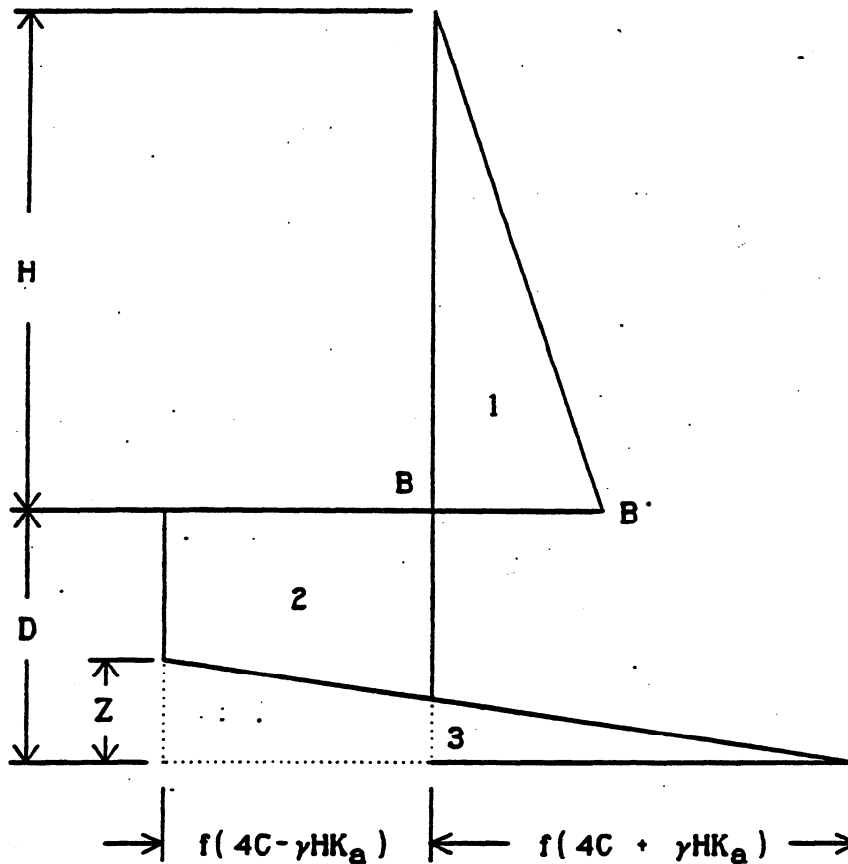


FIGURE 10 - 19

The procedure from this point on, including the addition of any surcharges, is identical to the " $\phi = 0$ method" outlined on the previous page.

SOLDIER PILES

SAMPLE PROBLEM 10-6: CANTILEVER SOLDIER PILE: COHESIVE SOIL

Given:

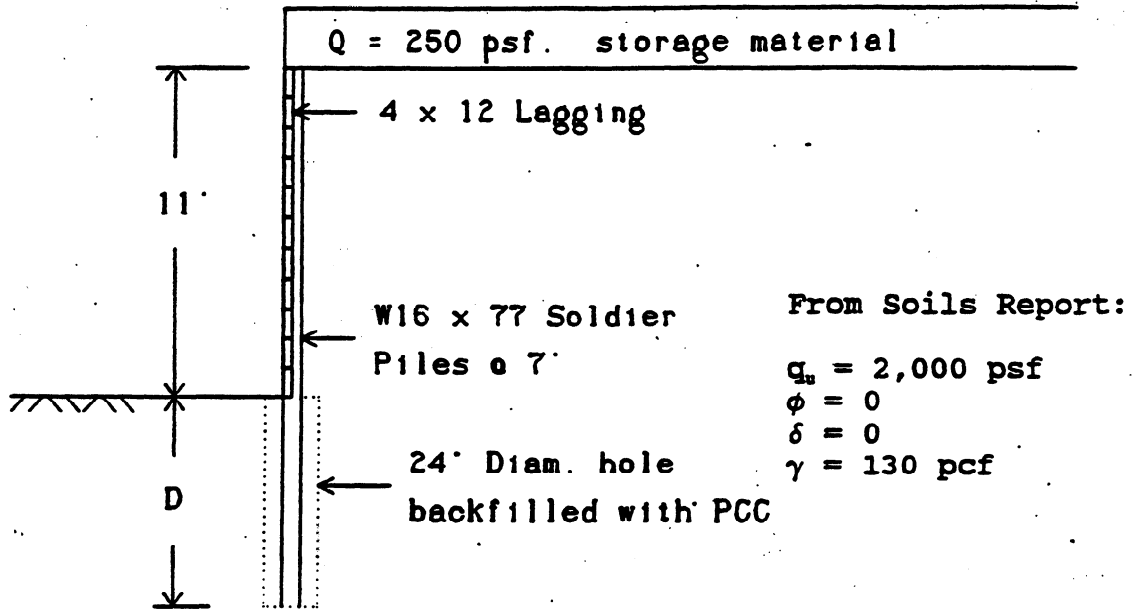


FIGURE 10 - 20

Solution:

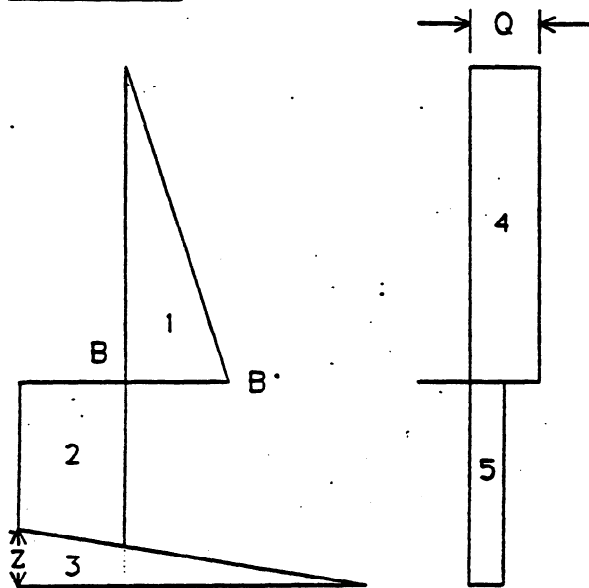


FIGURE 10 - 21

Use a safety factor of 1.6 for clay.

$$q_u/1.6 = 2,000/1.6 = 1,250 \text{ psf}$$

$$C = q_u/2 = 1,250/2 = 625 \text{ psf}$$

Arching capability = 2.0

$$f = 2(24/12)/7 = 0.57$$

Check critical height of wall:

$$H_c = 4C/\gamma$$

$$= 4(625)/130 = 19.2' > 11'$$

$$BB' = \gamma H - 2C$$

$$= 130(11) - 2(625)$$

$$= 180 \text{ psf}$$

AREAS:

$$1 = bb'h/2 = 180(11)/2 = 990$$

$$2 = F(4c - \gamma h)d = [(0.57)(4)(625) - 130(11)]D = 610D$$

$$3 = 4fCZ = 4(0.57)(625)Z = 1,425Z$$

$$4 = QH = 250(11) = 2,750$$

$$5 = fQD = 0.57(250)D = 143D$$

CALIFORNIA TRENCHING AND SHORING MANUAL

Determine D:

$$\begin{aligned}\Sigma F_H &= 0 \\ &= 990 - 610D + 1,425Z + 2,750 + 143D = 0\end{aligned}$$

$$3,740 + 1,425Z - 467D = 0$$

$$1) \quad Z = 0.33D - 2.62$$

$$\begin{aligned}\Sigma M_{base} &= 0 \\ &= 990[D + 11/3] - 610D[D/2] + 1,425Z[Z/3] + 2,750[D + 11/2] \\ &\quad + 143D[D/2] = 3,740D + 18,755 + 475Z^2 - 233D^2\end{aligned}$$

$$2) \quad Z^2 = 0.49D^2 - 7.87D - 39.48$$

By trial and error, or by other means, solve equations 1) and 2):

There is no need to increase D since a safety factor has already been applied to the clay.

Find Maximum Moment: (Assuming non-composite section):

Locate point of zero shear.

Shear at excavation elevation.

$$990 + 250(11) = 3,740 \text{ psf/LF}$$

Determine distance below excavation level to obtain shear equality:

$$\begin{aligned}610x - 143x &= 467x \text{ psf/LF} \\ 467x &= 3,740 \quad \therefore x = 3,740/467\end{aligned}$$

$$\begin{aligned}M_{max} &= 990[8.01 + 11/3] + 250(11)[8.01 + 11/2] \\ &\quad + 143(8.01)[8.01/2] - 610(8.01)[8.01/2] \\ &= 33,731 \text{ Ft-Lb/LF}\end{aligned}$$

$$\begin{aligned}S \text{ required} &= 33,731(7)(12)/22,000 = 128.8 \text{ in}^3 \\ S \text{ furnished} &= 134 \text{ in}^3 > 128.8 \text{ in}^3 \text{ OK}\end{aligned}$$

Often, with sound concrete below the depth of excavation, the moment occurring at that elevation may be used to determine the critical section modulus.

SOLDIER PILES

Check Lagging:

Consider arching effect on lagging. Multiply all loads by 0.6.
By inspection, maximum moment occurs at the depth of excavation.

$$M_{\max} = wL^2/8 = (180 + 250)(7)^2/8 = 2,634 \text{ Ft-Lb}$$

$$S \text{ required} = 2,634(12)(0.6)/(1,500) = 12.43 \text{ in}^3$$

$$S \text{ furnished (rough lumber)} = 32 \text{ in}^3 > 12.43 \text{ in}^3 \text{ OK}$$

$$V = (7/2 - 0.33)(180 + 250)(0.6) = 818 \text{ Lb}$$

$$v = 3V/2A = 3(818)/[2(4)(12)] = 25.6 \text{ psi} < 140 \text{ psi OK}$$

CALIFORNIA TRENCHING AND SHORING MANUAL

EFFECT OF SURCHARGE BELOW DEPTH OF EXCAVATION

Sample problems 10-1, 10-3, 10-6 and 22 (Appendix F) were recomputed using no surcharge below the depth of excavation to demonstrate the negligible difference in answers. A comparison of answers follows:

<u>SAMPLE PROBLEM 10-1</u>	<u>WITH SURCHARGE</u>	<u>WITHOUT SURCHARGE</u>
D	22.3'	21.1'
130% (D)	29.0'	27.4'
Z	4.91'	4.7'
M_{total}	1,082,016 Ft-Lb	1,009,632 Ft-Lb
S Required	590.2 in ³	550.7 in ³

SAMPLE PROBLEM 10-3

D_w	5.62'	5.2'
D	11.62' Use 11'- 8"	11.2' Use 11'-3"
T	10,785 Lb/LF	10,685 Lb/LF
Total T	80,888 Lb	80,138 Lb
Combined Moment	35,607 Ft-Lb/LF	34,587 Ft-Lb/LF
S Required	145.67 in ³	141.5 in ³

SAMPLE PROBLEM 10-6

Z	4.54'	5.1'
D	21.71'	18.0'
M_{max}	33,731 Ft-Lb/LF	30,220 Ft-Lb/LF
S Required	128.8 in ³	115.4 in ³

SAMPLE PROBLEM 22 (Appendix F)

Y	20.45'	19.77'
D	24.26'	23.63'

SOLDIER FILES

ACCEPTABLE ALTERNATE DESIGN METHODS WHICH HAVE BEEN USED

Cantilever System or Single Tie (Or Struttred) System:

Surcharge (S) may be limited to a depth of 10 feet or more, or to the elevation of the upper tie or strut (depicted as force T).

Dimension A is designers choice.

The location of point M is the designers choice. M may be located anywhere between points L to N. Point N is used for cantilever sheetpile or continuous walls.

Forces above L-N represent active loads on the soldier pile.

Passive forces are based on the effective pile diameter which includes the unitless number 3 (divided by an appropriate safety factor), times the pile diameter or drilled hole diameter, times K_p , times the unit weight of the soil.

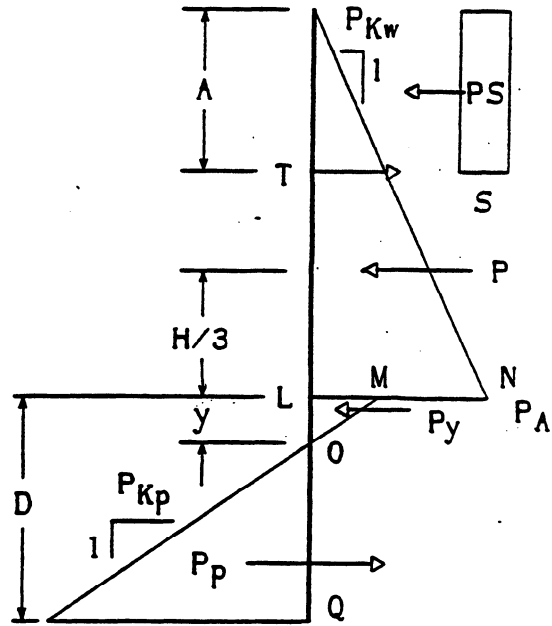


FIGURE 10 - 22

$$y = \frac{LM}{[3dK_p\gamma/S.F.]} \quad (\text{May not include contribution of surcharge}).$$

Cantilever Systems:

D is generally determined from moments taken about point Q. A safety factor against overturning should be included so that passive moments exceed active moments, all taken above point Q.

The section modulus of the soldier pile may be determined from moments taken about the point of zero shear.

Single Tie Or Strut System: (Only one method described below.)

T may be determined from moments of the forces above point O.

D may be determined from moments of the forces about T.

Check stability against overturning by taking moments about Q.

The section modulus required for the soldier pile is determined from the larger of the cantilever moment for the forces above point T, or from moments taken about the plane of zero shear.

CALIFORNIA TRENCHING AND SHORING MANUAL

AASHTO Soldier Pile Method

The figure below represents the soil pressures that may be used for cantilever soldier piles cohesionless soil. This figure (excluding surcharge) is an adaptation from the figure titled, Simplified Earth Pressure Distributions for Permanent Flexible Cantilevered Walls with Discrete Vertical Wall Elements, in AASHTO Standard Specifications For Highway Bridges.

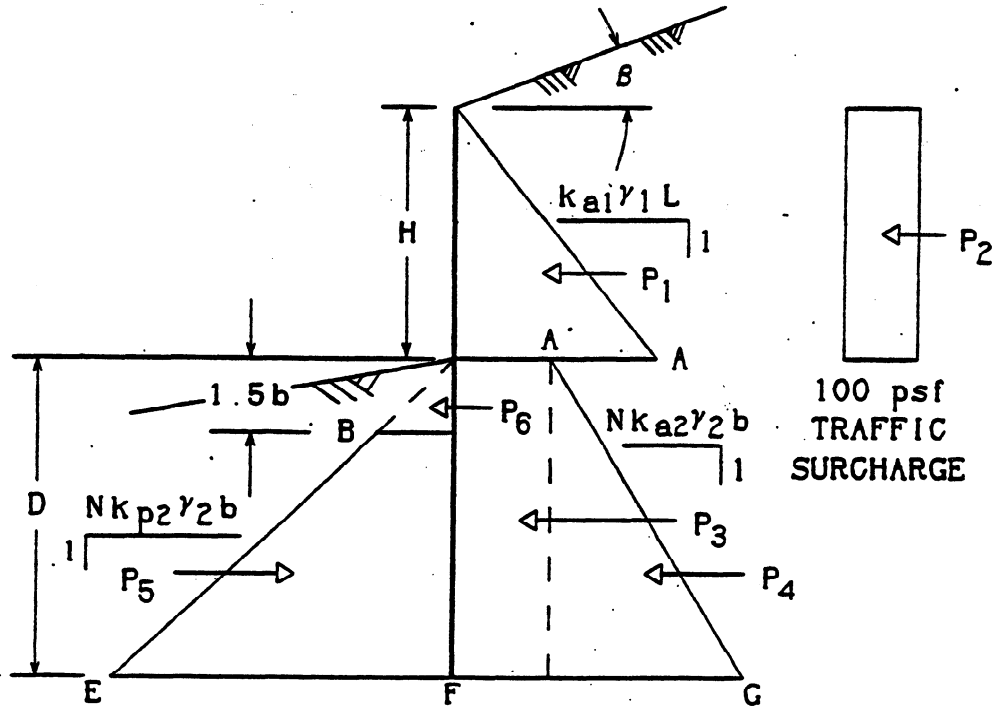


FIGURE 10 - 23

L = Soldier Pile Spacing
 N = Increase in effective pile width. AASHTO uses N = 3, (Caltrans uses N = 0.08φ). The value of Nb cannot exceed the soldier pile spacing "L" (AASHTO uses L ≤ 5b).
 b = Effective pile width: pile width or width of drilled hole backfilled with hard rock concrete.

PROJECT USE:

A-minimum safety factor of 50% is to be added to the computed embedment depth (D) for permanent flexible cantilever walls, otherwise 30% should be added for temporary construction.

For temporary construction the forces P₃ and P₄ may be ignored.

GENERAL PRESSURE EQUATIONS:

$$\begin{aligned}
 P_A &= K_{a1} \gamma_1 H(L) & P_B &= K_{p2} \gamma_2 (1.5b)(Nb) & \text{Schrg} &= P_s(L) \\
 P_{A'} &= K_{a2} \gamma_2 H(Nb) & P_G &= K_{a2} \gamma_2 D(Nb) & P_E &= K_{p2} \gamma_2 D(Nb)
 \end{aligned}$$

SOLDIER PILES

SAMPLE PROBLEM 10 - 7: USING THE AASHTO METHODOLOGY

$$\phi = 32^\circ \quad \beta = \phi$$

$$\gamma_1 = \gamma_2 = 110 \text{ pcf}$$

$$L = 7 \text{ Ft}$$

$$b = 1.33 \text{ Ft}$$

$$N = 0.08(32) = 2.56$$

$$Nb = 2.56(1.33) = 3.40$$

From Log-Spiral:

$$K_a = 0.80$$

$$K_p = 8.0(0.425) = 3.4$$

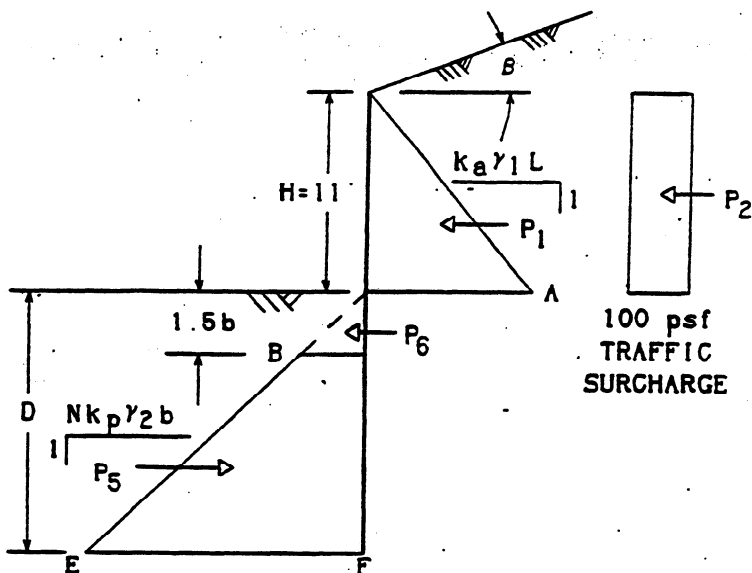


FIGURE 10-24

General Equations:

$$P_A = K_a \gamma_1 H(L) = 0.8(110)(11)(7) = \quad \quad \quad 6,776 \text{ Lb/Ft}$$

$$P_s = 100(L) = 100(7) = \quad \quad \quad 700$$

$$P_E = K_p \gamma_2 D(Nb) = 3.4(110)(3.4)D = \quad \quad \quad 1,271.6D$$

$$P_B = K_p \gamma_2 (1.5b)(Nb) = 3.4(110)(1.5)(1.33)(3.40) = 2,536.8$$

$$P_1 = P_A(H/2) = \quad \quad \quad 37,268 \text{ Lb}$$

$$P_2 = P_s H = \quad \quad \quad 7,700$$

$$P_6 = P_B(1.5b/2) = \quad \quad \quad 2,530.5$$

$$P_5 = P_E(D/2) = \quad \quad \quad - 635.8D^2$$

Determine D by Taking Moments About F:

$$0 = P_1[D + H/3] + P_2[D + H/2] + P_6[D - 2/3(1.5b)] - P_5[D/3]$$

$$= 37,268D + 136,649.3 + 7,700D + 42,350 + 2,530.5D - 3,365.6$$

$$\quad \quad \quad - 211.9D^3$$

$$= 211.9D^3 - 47,498.5D - 175,633.7 \quad \text{From which } D = 16.56'$$

$$\text{Use } D = 1.30(16.56) = 21.5 \text{ Ft}$$

CALIFORNIA TRENCHING AND SHORING MANUAL

Locate Depth To Plane Of Zero Shear:

(Use x in lieu of D in Figure 10 - 25)

$$P_1 + P_2 + P_6 = P_5$$

$$37,268 + 7,700 + 2,530.5 = 635.8(x^2)$$

$$x^2 = 74.71' \quad x = 8.64'$$

Determine Moment At Plane Of Zero Shear:

$$M = P_1[8.64 + H/3] + P_2[8.64 + H/2] + P_6[8.64 - 2/3(1.5b)] - P_5[8.64/3]$$

$$= 37,268[12.31] + 7,700[14.14] + 2,530.5[7.31] - 635.8(8.64)^2[2.88]$$

$$= 458,769 + 108,878 + 18,498 - 136,691$$

$$= 449,454 \text{ Ft-Lb}$$

Determine Section Modulus Required:

$$S = M(12)/22,000 = 449,454(12)/22,000 = 245 \text{ in}^3$$

W12 X 190 ($S = 263 \text{ in}^3$) Could be used (providing deflection is not a consideration).