Design of Sheet Pile Walls Using TRULINE Composite Wall Sections –

Design Methods and Examples

A Report Presented to

Formtech Enterprises, Inc./ TRULINE
126 Ben Burton Circle
Bogart, Georgia 30622

by

Ensoft, Inc.
3003 Howard Lane
Austin, Texas 78728

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Introduction

This report presents the recommended structural capacities for Truline composite wall sections and a summary of design methods commonly used to design cantilevered and anchored sheet pile structures. Several design examples are provided to illustrate the application of these design methods for sheet pile walls using the Truline 800 Series composite wall sections.

Reinforcement Options

Three options for the placement of reinforcement in the composite wall sections are typically used. These options are shown in Figures 1 to 3. Also shown in these figures are the dimensions b_w, d, and h that describe the geometry of the reinforcement placement.

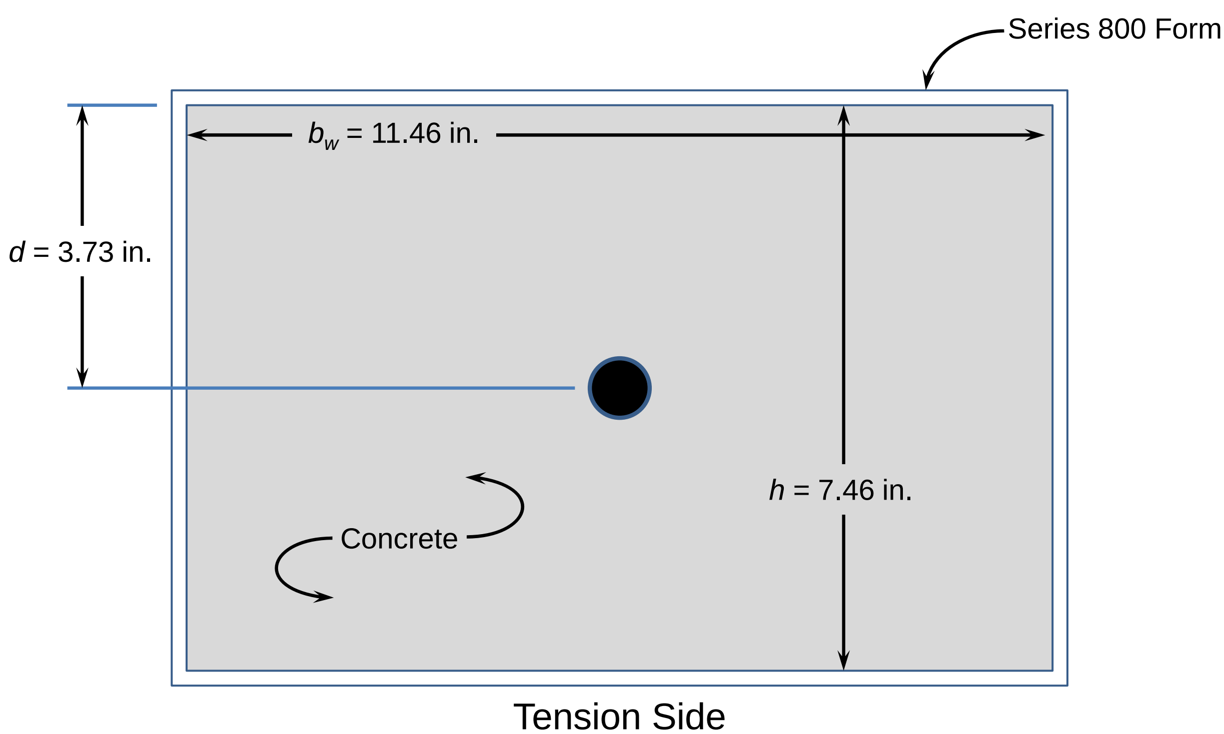

Figure 1 – Reinforced Sections – 1 Rebar

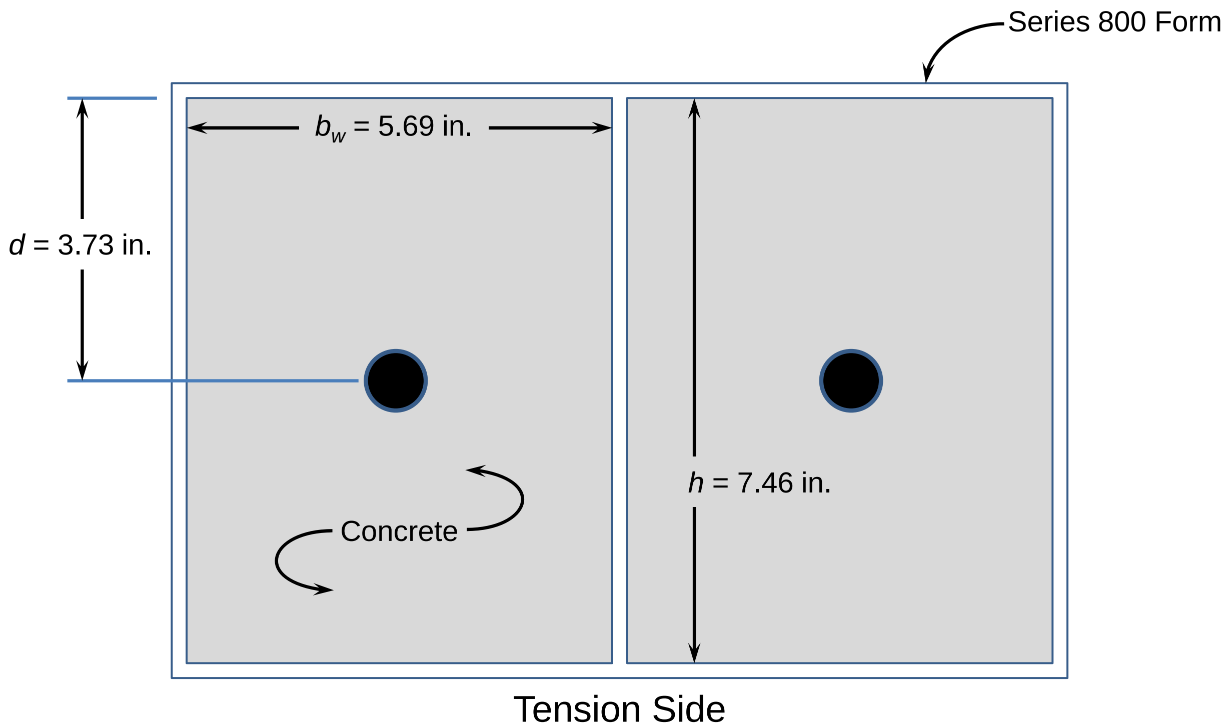

Figure 2 – Reinforced Sections – 2 Rebar

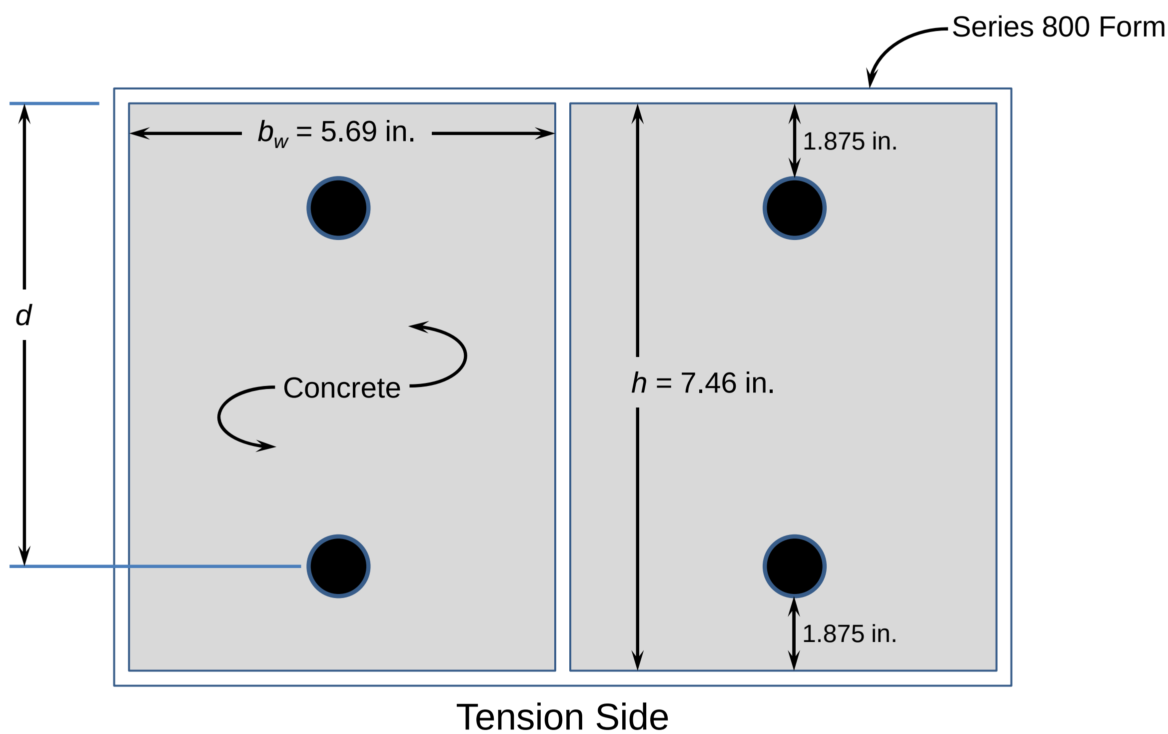

Figure 3 – Reinforced Sections – 4 Rebar

Recommended Structural Capacities of Truline 800 Series Composite Wall Sections

Structural Capacity and Bending Stiffness

The tables presented in this section are the recommended values for factored structural moment capacities and effective bending stiffness of Truline 800 Series PVC sheet pile sections filled with reinforced concrete. The factored moment capacities were determined from nonlinear moment vs. curvature behavior computed using LPile 2012. The nominal moment capacities were determined when the maximum compressive strain in concrete reached 0.003 in/in. The reported ultimate (factored) moment capacities were computed by multiplying the nominal moment capacity by a strength reduction factor, ϕ, of 0.65. The reported bending stiffnesses are for moment levels equal to the ultimate moment capacity and are for cracked sections.

The choice of a strength reduction factor of 0.65 is conservative. This is a value typically used for structural sections with combined flexure and axial load (see Chapter 10 in ACI 318-08). The rational for selection of this value is the similarity in construction methods for placement of concrete into the Truline sections to that used for drilled shaft foundations. In construction, concrete is placed from the top of section by free-fall if placed in the dry or by tremie or pumping if placed in the wet. Typically, vibration cannot be used due to the depth of concrete placement. In addition, no opportunity exists for direct visual inspection because the construction is not above ground and formwork is not used.

A reasonable argument can be made to raise the strength reduction factor to 0.90 if the wall sections are constructed above ground and moved into place. Presumably, in this type of construction, the use of vibration to consolidate concrete will be possible and no placement of concrete will be made in the wet. An evaluation of the analyses used to evaluate moment capacity and bending stiffness found that the moment capacity of all sections were controlled by tension reinforcement, thus permitting use of a strength reduction factor of 0.90 (see ACI 318-08 section R9.3.2.2).

Values of moment capacity and bending stiffness were computed for a range of concrete compressive strengths and reinforcement option. All values are reported in values per foot width of section.

Table 1 presents moment capacities of the Truline sections without concrete.

Table 1 – Allowable Moment Capacity and Bending Stiffness of Sections without Concrete

All pile sections must be filled with gravel or other material such as soil, sand, pebble, etc. to ensure the web is fully supported and the shear load is transferred from flange to flange by the fill material. Shear load must be applied by continuous beam or waler on the face of the wall.

Allowable Moment Capacity,* in-lbs/ft	Bending Stiffness, *EI*, lbs-in^2/ft
53,100	25,080,000

*Based on full scale performance test by Architectural Testing, Inc. Report #70174.01-122-44, not theoretical calculations.

Factored Moment Capacities

Tables 2 through 4 present moment capacities for concrete-filled Truline sections with various reinforcement and concrete compressive strength options

Bending Stiffness

Tables 5 through 7 present the computed bending stiffnesses of concrete-filled Truline sections with various reinforcement and concrete compressive strength options.

Table 2 – 800 Series - Factored Moment Capacities in-lbs/ft width – Reinforced Sections – 1 Rebar

Bar Size	Concrete Compressive Strength, f'_c psi				
	3,000	3,500	4,000	4,500	5,000
No. 4	41,300	42,500	43,400	44,200	44,700
No. 5	43,100	44,600	45,600	46,400	47,100
No. 6	52,300	56,100	58,100	59,600	60,800
No. 7	56,900	63,500	69,100	73,400	75,800
No. 8	59,800	67,600	74,700	81,000	86,600
No. 9	62,300	70,500	78,300	85,600	92,400
No. 10	65,000	73,800	82,000	89,900	97,500
No. 11	67,500	76,800	85,300	93,600	101,600
No. 14	73,200	82,900	92,300	101,400	110,000

Table 3 – 800 Series - Factored Moment Capacities in-lbs/ft width – Reinforced Sections - 2 Rebar

Bar Size	Concrete Compressive Strength, f'_c psi				
	3,000 psi	3,500 psi	4,000 psi	4,500 psi	5,000 psi
No. 4	64,500	65,900	66,800	67,500	68,000
No. 5	86,100	88,900	91,000	92,600	93,900
No. 6	104,500	112,000	116,000	118,900	121,300
No. 7	113,800	127,000	138,200	146,600	151,500
No. 8	119,700	135,100	149,300	162,000	173,100
No. 9	124,600	141,100	156,700	171,300	184,900
No. 10	129,900	147,400	164,000	179,900	195,000
No. 11	135,000	153,100	170,600	187,200	203,200
No. 14	146,500	165,900	184,600	202,700	220,100

Table 4 – 800 Series - Factored Moment Capacities in-lbs/ft width – Reinforced Sections - 4 Rebar

Bar Size	Concrete Compressive Strength, f'_c psi				
	3,000	3,500	4,000	4,500	5,000
No. 4	95,900	100,900	104,900	108,100	109,900
No. 5	127,100	132,800	138,100	143,100	147,700
No. 6	161,200	167,900	174,100	179,800	185,300
No. 7	201,600	209,100	216,200	222,800	229,000
No. 8	248,200	256,600	264,500	271,900	278,900

Table 5 – 800 Series - Bending Stiffness, lbs-in^2/ft width for – Reinforced Sections – 1 Rebar

Bar Size	Concrete Compressive Strength, f'_c psi				
	3,000	3,500	4,000	4,500	5,000
No. 4	64,920,000	67,370,000	69,380,000	71,080,000	72,560,000
No. 5	68,430,000	71,100,000	73,310,000	75,170,000	76,790,000
No. 6	81,460,000	84,950,000	88,160,000	90,900,000	93,280,000
No. 7	96,350,000	100,210,000	103,770,000	107,100,000	110,350,000
No. 8	111,380,000	115,960,000	120,110,000	123,910,000	127,450,000
No. 9	125,280,000	130,780,000	135,620,000	140,090,000	144,140,000
No. 10	140,620,000	147,160,000	153,050,000	158,290,000	163,130,000
No. 11	154,500,000	162,020,000	168,900,000	175,050,000	180,630,000
No. 14	181,660,000	191,420,000	200,140,000	208,040,000	215,310,000

Table 6 – 800 Series - Bending Stiffness, lb-in^2/ft width for – Reinforced Sections – 2 Rebar

Bar Size	Concrete Compressive Strength, f'_c psi				
	3,000 psi	3,500 psi	4,000 psi	4,500 psi	5,000 psi
No. 4	109,000,000	112,300,000	115,000,000	117,400,000	119,400,000
No. 5	136,700,000	142,000,000	146,400,000	150,100,000	153,300,000
No. 6	162,800,000	169,800,000	176,200,000	181,600,000	186,400,000
No. 7	192,600,000	200,300,000	207,400,000	214,100,000	220,600,000
No. 8	222,700,000	231,800,000	240,100,000	247,700,000	254,800,000
No. 9	250,500,000	261,500,000	271,200,000	280,100,000	288,200,000
No. 10	281,200,000	294,400,000	306,100,000	316,500,000	326,200,000
No. 11	309,000,000	324,200,000	337,800,000	350,100,000	361,200,000
No. 14	363,300,000	382,800,000	400,300,000	416,100,000	430,600,000

Table 7 – 800 Series - Bending Stiffness, lbs-in^2/ft width for – Reinforced Sections – 4 Rebar

Bar Size	Concrete Compressive Strength, f'_c psi				
	3,000	3,500	4,000	4,500	5,000
No. 4	234,820,000	239,000,000	242,900,000	246,235,000	249,500,000
No. 5	309,785,000	317,402,000	323,775,000	330,000,000	334,400,000
No. 6	383,022,000	394,125,000	404,646,000	413,000,000	420,000,000
No. 7	459,000,000	475,658,000	489,700,000	501,500,000	511,750,000
No. 8	537,000,000	558,815,000	576,993,000	592,869,000	606,500,000

Shear Capacity

The structural capacity in shear is computed as the sum of the shear capacity of the Truline sheet pile section and the reinforced concrete. Thus, the nominal shear capacity, V_n, can be expressed as

$$V_n = V_c + V_s + V_F$$

Where: V_c is the shear strength provided by the concrete, V_s is the shear strength provided by the steel shear reinforcement, and V_F is the shear strength provided by the Truline sheet piling. The allowable shear force for Truline sections without concrete is shown in Table 8.

Table 8 – Allowable Shear Capacity of Truline Sections without Concrete

Truline Section	Allowable Shear Capacity,* lbs/ft
Series 800	6,300

* All pile sections must be filled with gravel or other material such as soil, sand, pebble, etc. to ensure the web is fully supported and the shear load is transferred from flange to flange by the fill material. Shear load must be applied by continuous beam or waler on the face of the wall.

For concrete members subjected to shear and flexure only, the shear strength provided by the concrete is (Eq. 11-3 from ACI 318-08):

$$V_c = 2\lambda\sqrt{f_c'}b_w d$$

Where: λ = 1.0 for normal weight concrete, b_w is the width of the section, and d is the distance from the extreme compression edge to the centroid of the tension reinforcement.

The shear capacities of sections without steel shear reinforcement are shown in Table 9.

Table 9 – 800 Series Shear Capacity/ft width

$V_c + V_F$	Concrete Compressive Strength, f'_c psi				
	3,000	3,500	4,000	4,500	5,000
lbs/cell	5,440	5,620	5,790	5,950	6,100
lbs/ft	10,880	11,240	11,580	11,900	12,200

Additional shear strength can be provided by the addition of shear reinforcement steel to the section. The shear strength of the steel shear reinforcement is computed using

$$V_s = \frac{A_v f_{yt} d}{s}$$

Where: A_v is the area of shear reinforcement within spacing s, f_{yt} is the yield stress of the shear reinforcement, up to a maximum of 60,000 psi, and d is distance from the compression edge of the concrete to the centroid of the tension reinforcement.

The spacing s is limited to a maximum of d/2 by ACI 318-08 Section 11.4.5.1. This means s will be no greater than 4.4 inches for No. 5 bars down to 4.2 inches for No. 11 bars.

ACI 318-08 Section 11.4.6.1 requires a minimum shear reinforcement, $A_{v,min}$, when V_u exceeds 0.5 ϕ V_c. for sections greater than 10 inches in thickness. The requirement for minimum shear reinforcement does not apply to the 800 Series section since the depth of the concrete section is 7.46 inches is less than 10 inches.

The ultimate (factored) shear capacity is computed using a strength reduction factor, ϕ, of 0.75 (see ACI 318-08 Section 9.3.2.3).

$$V_u \leq \phi V_n = \phi\left(V_c + V_s + V_F\right)$$

As an example, the factored shear capacity of an 800 Series wall, with f'_c = 4,000 psi, Grade 60, No. 8 vertical reinforcement, and Grade 60, No. 3 shear reinforcement is

$$\phi V_n = 0.75\left(11{,}580 \text{ lb/ft} + 4{,}000 \text{ lb/ft}\right) = 15{,}800 \text{ lb/ft}$$

Comparison of Concrete-Filled Sections and Sections without Concrete

The increase in moment capacity and bending stiffness achieved by filling the sections with reinforced concrete varies in proportion to the concrete compressive strength and amount of steel reinforcement. The ratios of increase for moment capacity and bending stiffness are presented in Tables 10 and 11.

The increase in bending stiffness is much larger than the increase in moment capacity for all sections. The increase in moment capacity for concrete-filled walls allows higher wall sections to be constructed. The increase in bending stiffness for concrete-filled walls results in lower lateral deflections of the walls.

Table 10 – Ratio of Concrete-filled Sections to Sections without Concrete -- Moment Capacities -- 4 Rebar Reinforced Sections

Bar Size	Concrete Compressive Strength, f'_c psi				
	3,000	3,500	4,000	4,500	5,000
No. 4	1.81	1.90	1.97	2.03	2.07
No. 5	2.39	2.50	2.60	2.69	2.78
No. 6	3.03	3.16	3.28	3.38	3.49
No. 7	3.79	3.94	4.07	4.19	4.31
No. 8	4.67	4.83	4.98	5.12	5.25

Table 11 – Ratio of Concrete-filled Sections to Sections without Concrete -- Bending Stiffnesses -- 4 Rebar Reinforced Sections

Bar Size	Concrete Compressive Strength, f'_c psi				
	3,000	3,500	4,000	4,500	5,000
No. 4	9.36	9.53	9.69	9.82	9.95
No. 5	12.35	12.66	12.91	13.16	13.33
No. 6	15.27	15.71	16.13	16.47	16.75
No. 7	18.30	18.97	19.53	20.00	20.40
No. 8	21.41	22.28	23.01	23.64	24.18

Analysis and Design of Cantilever Sheet Pile Walls

A cantilever sheet pile wall consists of sheet piling driven deep enough into the ground to become a fixed, vertical cantilever. In a limit equilibrium analysis, this type of wall is supported by a lateral earth pressure equal to the difference between the active and passive earth pressures above and below a point of rotation. This type of wall is only suitable for walls of moderate height.

In a limit equilibrium analysis, the earth pressure assumed to be acting on a cantilever sheet pile wall in cohesive soil is that shown in Figure 4. The earth pressure acting on the wall is assumed as the difference between the passive and active earth pressures. The wall is assumed not to move laterally at the pivot point. The distributions of earth pressures are different for cohesive and cohesionless soils. In addition, it is possible for the earth pressures in cohesive soils to vary with time, particularly when excavations are made in front of the wall.

The analysis procedure for cantilever sheet pile walls in granular, cohesionless soils is as follows:

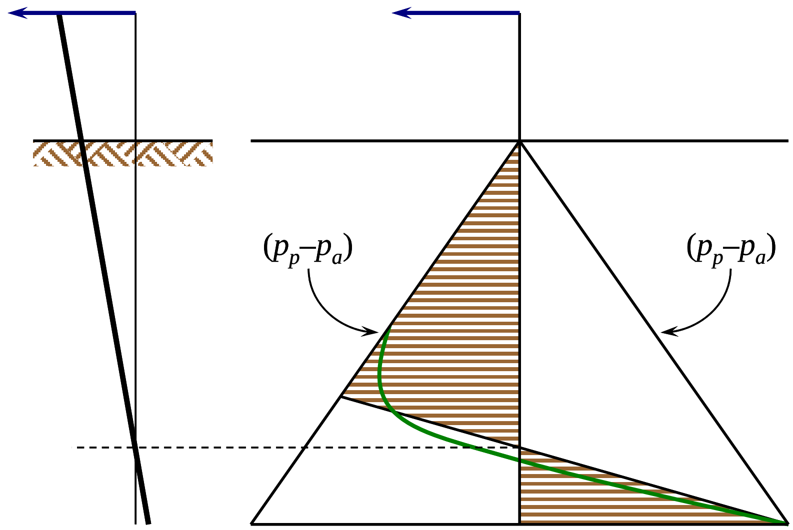

Figure 4 – Earth Pressures Acting on a Cantilever Sheet Pile Wall

(1) Assume a trial depth of penetration, *D*. A starting value can be obtained from Table 12.

Table 12 – Approximate Values for Required Depths for Cantilever Sheet Pile Walls in Cohesionless Soils

SPT Blowcount, N_{SPT}, blows/ft	Relative Density, D_r	Depth of Penetration
0-4	Very loose	2.0 *H*
5-10	Loose	1.5 *H*
11-30	Medium	1.25 *H*
31-50	Dense	1.0 *H*
Over 50	Very dense	0.75 *H*

H = height above the dredge line.

(2) Determine the active and passive lateral earth pressure distributions on both sides of the wall.

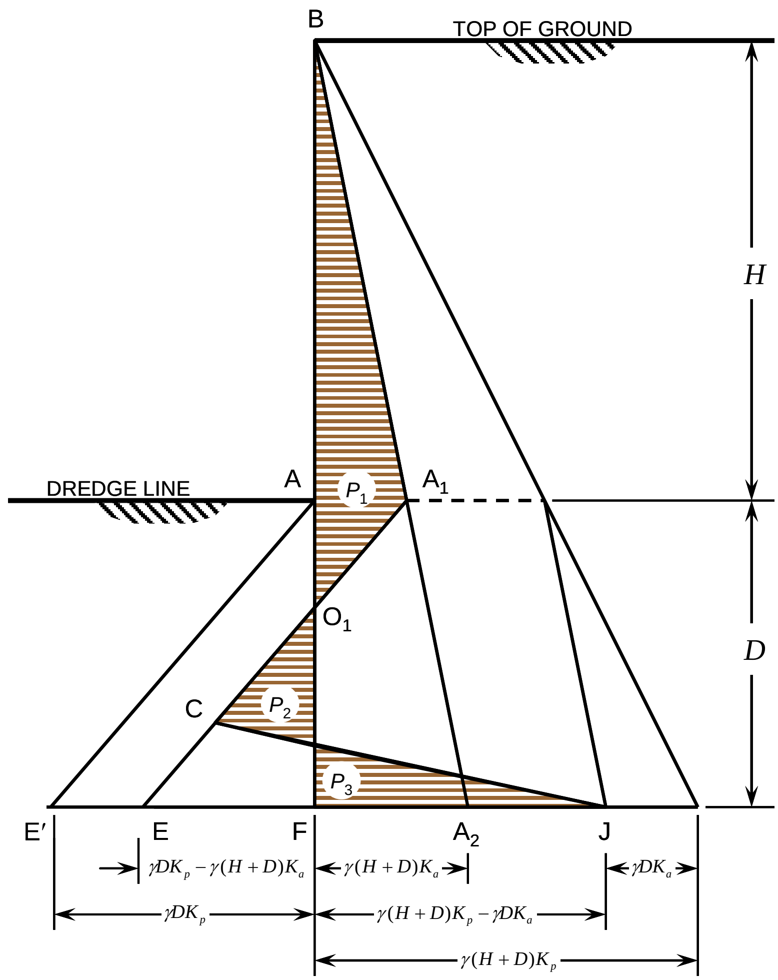

Figure 5 – Resultant Earth Pressure Diagram for Cantilever Sheet Pile in Cohesionless Soil

(3) Determine the depth of wall needed to achieve static equilibrium for forces acting in the horizontal direction.

$$\vec{F}_{EA_1A_2} - \overleftarrow{F}_{FBA_2} - \overleftarrow{F}_{ECJ\,0=0} \quad \text{...} \quad (1)$$

Solve Equation 1 for the distance *Z*. For uniform cohesionless soil:

$$Z = \frac{K_p D^2 - K_a (H+D)^2}{(K_p - K_a)(H+2D)} \quad \text{..} \quad (2)$$

Take moments about the tip of the wall at point *F* and check if the sum of moments is equal to zero. Revise the depth of penetration *D* until convergence (sum of moments equal to zero) is achieved.

Add 20 to 40 percent to the calculated depth of penetration. This results in a factor of safety of approximately 1.5 to 2.0. Alternatively, one may use a reduced value of the passive earth pressure coefficient for design. A typical value is 50 to 75 percent of the maximum passive resistance.

Compute the maximum bending moment developed in the wall *prior to increasing the depth by 20 to 40 percent*.

The lateral displacement can be estimated by assuming that the wall is fixed at a depth of ½*D* and loaded by a triangular load equal to the actual applied active loading. The lateral movement at any distance y below the top of the wall is computed by

$$\delta = \frac{P_t}{60EI\ell^2}\left(y^5 - 5\ell^4 y + 4\ell^5\right) \quad (3)$$

Design Example of Cantilever Sheet Pile Wall in Cohesionless Soil

The dimensions and soil properties of the example problem for a sheet pile wall in cohesionless soil is shown in Figure 6. The procedure for analyzing the wall is as follows.

Compute Wall Pressures Acting on Wall

The pressures in units of psf acting at points A1, A2, B, C, E, and J shown in Figure 5 are:

$$\begin{aligned}
p_{A1} &= \gamma H K_a = (115)(8)(0.271) = 249.3 \\
p_{A2} &= p_{A1} + \gamma' D K_a = 249.3 + 17.62D \\
p_E &= \gamma' D\left(K_p - K_a\right) - p_{A1} = 222.2D - 249.3 \\
p_J &= \gamma' D\left(K_p - K_a\right) + \gamma H K_p = 222.2D + 3{,}395.
\end{aligned}$$

From statics $\sum F_h = 0$ or $\tfrac{1}{2} H p_{A1} + \left(p_{A1} + p_{A2}\right)\frac{D}{2} + \left(p_E + p_J\right)\frac{z}{2} - \left(p_E + p_{A2}\right)\frac{D}{2} = 0$

Solving for *z*:

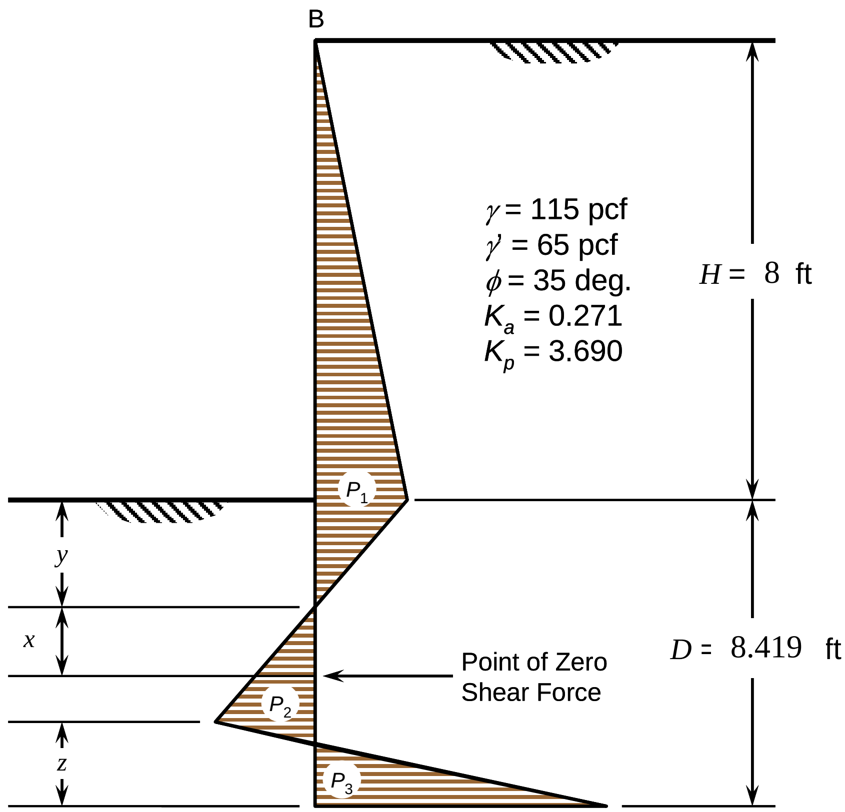

Figure 6 – Example Problem for Sheet Pile Wall in Cohesionless Soil

$$z = \frac{(p_E - p_{A1})D - Hp_{A1}}{p_E - p_J}$$

Summing moments about the bottom of the wall:

$$\sum M = 0 = ½\, Hp_{A1}\left(D + \frac{H}{3}\right) + p_{A1}\frac{D^2}{2} + (p_E + p_J)\frac{z^2}{6} - (p_E + p_{A2})\frac{D^2}{6} + (p_{A2} - p_{A1})\frac{D^2}{6}$$

The above equations can be solved by trial and error by assuming a value for *D*, computing *z*, and computing the sum of moments, varying *D* until the computed sum of moments is zero. Alternatively, the equations can be programmed in an electronic spreadsheet program and using the Goal Seek tool to obtain a solution. The solution for the above problem finds

$$D = 8.419\text{ ft}$$
$$z = 1.388\text{ ft}$$

Depth of Penetration for Design

The depth of penetration is determined by increasing the value of D by 20 to 40 percent. In this case, D = 10.1 to 11.8 ft. Use D = 11 ft.

Compute Location of Zero Shear Force

The point of zero shear force requires the computation of the depths x (depth of zero shear force) and y (depth of zero net pressure on wall). The depth of zero net pressure is computed using

$$y = \frac{p_{A1}}{\gamma'\left(K_p - K_a\right)} = \frac{249.3}{65(3.690 - 0.271)} = 1.122 \text{ ft}$$

Compute Maximum Moment Developed in Wall

The force resultants P_1, P_2, and P_3 are computed using:

$$\begin{aligned}
P_1 &= \tfrac{1}{2} p_{A1} H = \tfrac{1}{2}(249.3)(8) = 997 \text{lb} \\
P_2 &= \tfrac{1}{2} p_{A1} y = \tfrac{1}{2}(249.3)(1.122) = 139.8 \text{ lb} \\
P_1 + P_2 &= P_3 = \tfrac{1}{2}\gamma'\left(K_p - K_a\right)x^2 \\
x &= \sqrt{\frac{2\left(P_1 + P_2\right)}{\gamma'\left(K_p - K_a\right)}} = \sqrt{\frac{2(997.2 + 139.8}{65(3.690 - 0.271)}} = 3.199 \text{ ft} \\
P_3 &= \tfrac{1}{2}\gamma'\left(K_p - K_a\right)x^2 = 1{,}137 \text{ lb}
\end{aligned}$$

The maximum moment is computed using

$$M_{\max} = P_1\ell_1 + P_2\ell_2 - P_3\ell_3$$

$$\ell_1 = \left(\frac{H}{3} + y + x\right) = 6.987 \text{ ft}$$

$$\ell_2 = \left(\frac{2y}{3} + x\right) = 3.947 \text{ ft}$$

$$\ell_3 = \frac{x}{3} = 1.066 \text{ ft}$$

$$\begin{aligned}
M_{\max} &= (997.2)(6.987) + (139.8)(3.947) - (1{,}137)(1.066) \\
&= 6{,}307 \text{ ft - lbs} \\
&= 75{,}700 \text{in - lbs}
\end{aligned}$$

Consulting Tables 2 to 4, any of the 1-rebar sections with No. 9 and larger with 4,000 psi concrete or any of the 2-rebar No. 5 or larger or any of the 4-rebar options are suitable for this wall.

Note Regarding Passive Earth Pressure Coefficient

In the above design example, the passive earth pressure coefficient was computed assuming that the wall is vertical and frictionless and that the ground surface behind the wall is horizontal. For these conditions, the computation of the passive earth pressure coefficient using Rankine theory is both appropriate and conservative.

It is common practice when designing steel sheet pile walls to consider the effect of wall friction on the passive earth pressure coefficient. When doing so, many designers use the coefficients developed by Caquot and Kerisel (1948). These coefficients are illustrated in Figure A1 of the Appendix of this report and are reported in NAVFAC DM 7.02 (1986) in Figure 6 on page

7.2-67. This manual is available in PDF format for download from http://portal.tugraz.at/portal/page/portal/Files/i2210/files/eng_geol/NAVFAC_DM7_02.pdf.

Design Example of Cantilever Sheet Pile Wall in Cohesive Soil

Cantilever sheet pile structures are typically used for small walls. The analysis that follows was originally developed by Blum (1931) and is for under short-term loading conditions. For long-term loading conditions, the analysis is made using the analysis for cantilever sheet pile walls in cohesionless soils using the fully drained shearing properties of the soil presented above.

The earth pressures acting on a cantilever sheet pile wall in cohesive soils are shown in Figure 7.

The dimensions and soil properties for this design example are:

$$H = 14 \text{ ft},\ \gamma = 120 \text{ pcf},\ \gamma' = 60 \text{ pcf},\ c = 500 \text{ psf}$$

Compute the following quantities:

$$H_0 = 2c/\gamma = 8.333 \text{ ft}$$

$$H - H_0 = 5.67 \text{ ft}$$

$$\gamma H - 2\,c = 680 \text{ psf}$$

$$4c - \gamma H = 320 \text{ psf}$$

$$4c + \gamma H = 3{,}680 \text{ psf}$$

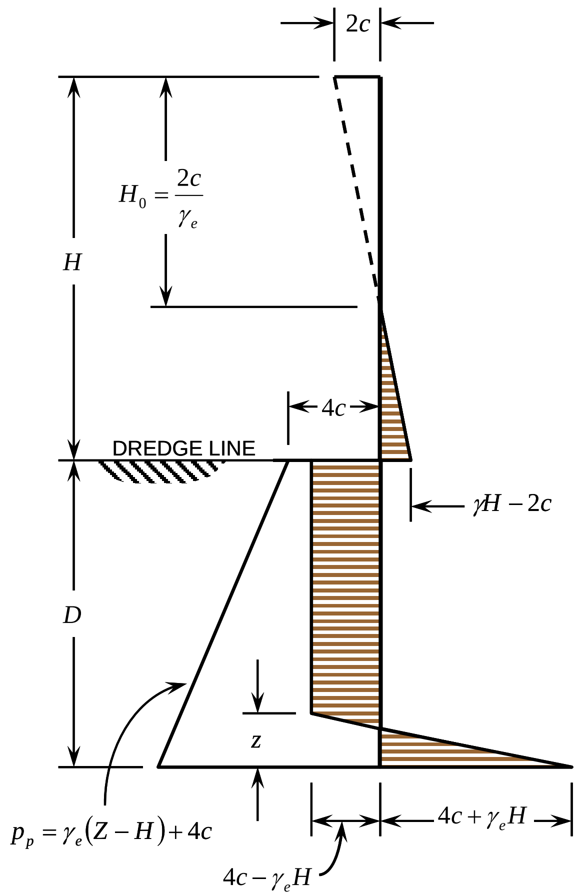

Figure 7 – Earth Pressures for Cantilever Sheet Pile Wall in Cohesive Soil

Sum horizontal forces:

$$\tfrac{1}{2}(\gamma H - 2c)H - H_0 + \frac{8cz}{2} - (4c - \gamma H)D = 0$$

Solve for depth *z*

$$z = \frac{2D(4c - \gamma H) - (\gamma H - 2c)(H - H_0)}{8c}$$

Sum moments about the bottom of wall

$$\sum M = \tfrac{1}{2}(\gamma H - 2c)(H - H_0) - \left(D + \frac{H - H_0}{3}\right) + \frac{8cz^2}{6} - (4c - \gamma H)\frac{D^2}{2} = 0$$

<u>Strategy for Computations:</u>

1. Assume a value for *D*.
2. Calculate depth *z*.
3. Calculate sum of moments about the bottom of wall.
4. Repeat until convergence (sum of moments) is achieved.

Alternatively, one may solve the quadratic equation (sum of moments equation) for *D*. The solution for the above given data:

$$z = 1.300 \text{ ft}$$

$$D = 14.147 \text{ ft}$$

The solution values are quite sensitive to the input values. If the value of cohesion is varied plus or minus 5 percent the values of H = 22.532 ft for c = 475 psf and H = 9.75 ft for c = 525 psf. Similar sensitivity is found for slight variations in unit weight and geometry.

<u>Selection of Wall Section</u>

The selection of the wall section is made based on the moment developed in the wall. The design moment is computed at the dredge line. For the above conditions, the moment is

$$\begin{aligned} M &= \tfrac{1}{6}(H - H_0)^2(\gamma H - 2c)(1\,\text{ft}) \\ &= \tfrac{1}{6}(5.67\text{ ft})^2(680\text{ psf})(1\text{ ft}) \\ &= 3{,}640\text{ ft - lb} \\ &= 43{,}700\text{ in - lb} \end{aligned}$$

Checking the level of moment developed in the wall against the allowable moment capacity values shown in Tables 2 through 4, an 800 Series section with the following combinations of concrete compressive strength and reinforcement size will be acceptable for this application for short-term, undrained conditions:

- a concrete compressive strength of 3,500 psi and 1-bar centered reinforced with No. 5 bar
- a concrete compressive strength of 3,000 psi and 2-bar centered reinforced with No. 4 bars
- a concrete compressive strength of 3,000 psi and 4-bar reinforced with No. 4 bars.

Analysis and Design of Anchored Sheet Pile Walls

A number of design methodologies are used to design anchored sheet pile walls. The USS Steel Sheet Pile Design Manual provides the details on the free earth support method, Rowe's moment reduction method, the fixed earth support method (equivalent beam method), graphical methods, and design using the Danish rules. Each of these methods is a "hand computation" method that assumes relatively simple soil profiles and do not require use of a computer program. For complicated problems, including staged construction, the computer program PYWall from Ensoft, Inc. may be considered for use. PYWall considers the nonlinear lateral load-transfer properties of the soil and may consider multiple levels of tiebacks, struts, and braces. Additional information about PYWall may be obtained from *www.ensoftinc.com*.

Free Earth Support Method

Anchored walls are supported by passive resistance at the toe of the wall and the anchor tie rods at the top of the wall. Wall heights may extend up to 25 feet, depending on local soil conditions. A procedure for the free earth support method is the following:

1. Compute the active and passive lateral pressured using appropriate coefficients of lateral earth pressures. (See Figure 8)
2. Calculate the weight of overburden and surcharge loads at the dredge level, $\gamma' H$.
3. Calculate the point of zero pressure using

$$y = \frac{\gamma' H\, K_a}{p_p - p_a} \qquad (4)$$

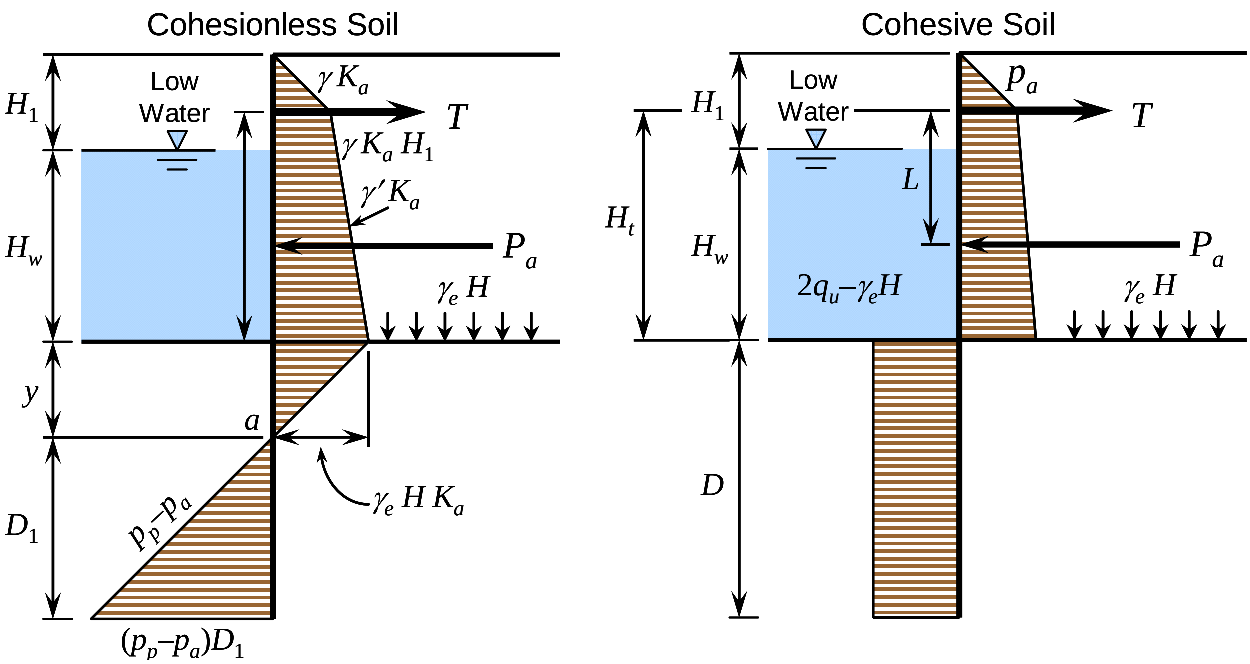

Figure 8 – Earth Pressure Distributions Used in Design of Anchored Sheet Piling by Free Earth Support Method for Cohesionless and Cohesive Soils

4. Calculate P_a, the resultant force of the earth pressure above point a, and its distance, L, below the tie rod elevation.

Static equilibrium is attained by making the wall deep enough that the moment due to the net passive pressure will balance the moment due to the resultant active force, P_a. Sum moments about the tie rod level.

$$\sum M = (L)(P_a) - \tfrac{1}{2}(p_p - p_a)D_1^2(H_t + y + \tfrac{2}{3}D_1) = 0 \quad \text{............................} \quad (5)$$

Solving for D_1; usually a trial and error solution is used. Alternatively, the equation can be solved using the Goal Seek option in an electronic spreadsheet program.

Compute the tension in the tie rod by

$$T = P_a - \tfrac{1}{2}(p_p - p_a)D_1^2$$

The maximum bending moment occurs at the point of zero shear force in the wall below the tie rod elevation.

Select the appropriate sheet pile section for the maximum moment developed.

Add 20 to 40 percent to D_1 to provide a margin of safety or divide the passive resistance force P_p by a factor of safety of 1.5 to 2.0.

Design Example for Anchored Sheet Pile Wall in Granular Soil Using Free-Earth Support Method

Compute Distribution of Earth Pressures Acting on Wall

The earth pressures values at points B, C1, C2, and E (see Figure 9) are:

$$p_B = \gamma H_1 K_a = (110)(5)(0.271) = 149.0 \text{ psf}$$

$$p_{C1} = P_B + \gamma H_w K_a = 149.0 + 60(13)(0.271) = 360.4 \text{ psf}$$

$$p_{C2} = [\gamma_e H_1 + \gamma' H_w]K_a = [110(5) + 60(13)]0.271 = 368.2 \text{ psf}$$

$$p_E = \gamma'(K_p - K_a)D_1 = 65(3.613 - 0.271)D_1 = 216.8D_1$$

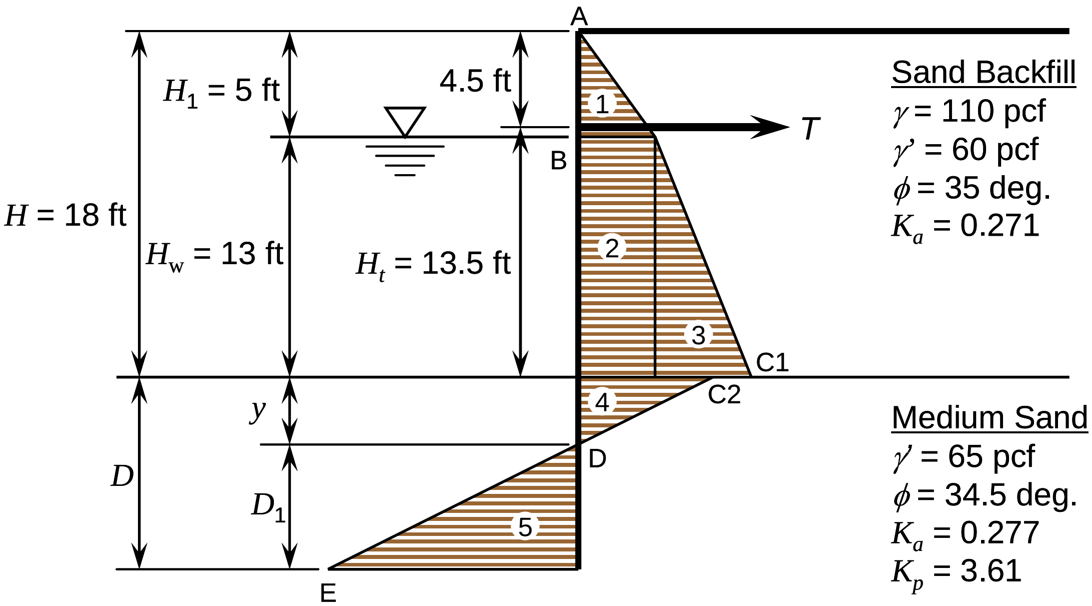

Figure 9 – Earth Pressure Distributions Used in Design Example for Anchored Sheet Pile Wall in Cohesionless Soil

Compute Depth of Point of Zero Net Pressure

The depth of zero net pressure below the dredge line is computed by

$$y = \frac{p_{C2}}{\gamma'(K_p - K_a)} = \frac{360.4}{65(3.614 - 0.271)} = 1.662 \text{ ft}$$

Compute Force Resultants of Pressures Acting on Wall

The resultant forces acting on the wall are:

$$P_1 = \tfrac{1}{2} H_1 p_B = (\tfrac{1}{2})(5)(149.0) = 372.6 \text{ lb}$$

$$P_2 = H_w p_B = (13)(149.0) = 1{,}938. \text{ lb}$$

$$P_3 = \tfrac{1}{2} H_w (p_{C1} - p_B) = (\tfrac{1}{2})(13)(360.4 - 149.0) = 1{,}374. \text{ lb}$$

$$P_4 = \tfrac{1}{2} p_{C2} y = (\tfrac{1}{2})(368.2)(1.662) = 306.0 \text{ lb}$$

$$P_5 = \tfrac{1}{2} p_E D_1 = (\tfrac{1}{2})(216.8 D_1) D_1 = 108.4 D_1^2$$

Compute Sum of Moments

Equate the sum of moments acting about the tieback to zero and solve for D_1.

Table 13 – Sum of Moment Computations

Force	Force, lbs	Arm, ft	Moment, ft-lbs
P_1	372.6	-1.167	-435
P_2	1,938.	7.00	13,563
P_3	1,374.	9.167	12,594
P_4	306.0	14.608	4,470
P_5	$-108.4D_1^2$	$13.5 + 1.662 + 2D_1/3$	$-1{,}644D_1 - 72.28D_1^2$
		Total	$-72.28D_1^3 - 1{,}644.D_1^2 + 30{,}192.$

Solve for D_1 and Select Value of D

Solve for D_1 by trial and error or by spreadsheet solution using the Goal Seek function.

$$D_1 = 3.956 \text{ ft}$$

$$D = D_1 + y = 3.956 + 1.662 = 5.62 \text{ ft}$$

Evaluate P_5 using value of D_1.

$$P_5 = -108.4D_1^2 = -108.4(3.956)^2 = -1{,}696 \text{ lb}$$

To provide a margin of safety, increase D by 20 to 40 percent (6.74 to 7.86 ft).

Use D = 7.5 ft for the design.

Compute Tension Force in Tieback

$$\begin{aligned} T &= P_1 + P_2 + P_3 + P_4 + P_5 \\ &= 372.6 + 1{,}937 + 1{,}374 + 306.0 - 1{,}696 \\ &= -2{,}294 \text{ lb/ft width} \end{aligned}$$

For sizing of tieback, increase by 33%: Use 3,340 lbs/ft width for design. The total force in an individual tieback will depend on the lateral spacing between tiebacks.

Prior to designing the tieback, it is necessary for the geotechnical engineer to perform a slope stability analysis to determine the position of the potential slip surface. The anchor block for the tieback must be located beyond the slip surface in order for the tieback to perform as designed.

Compute Location of Point of Maximum Moment

The location of the maximum moment, x, is below the water level at location of zero shear force. In this case, it is possible to write a quadratic equation using the horizontal tieback forces.

$$T + P_1 + p_b x + \gamma_1' K_{a1} x^2$$

$$8.13x^2 + 149.0x - 1{,}921 = 0$$

$$x = 8.732 \text{ ft below water}$$

Compute Maximum Moment in Wall Section

The maximum moment is computed at the point of zero shear force. The maximum moment developed in the wall is

$$\begin{aligned} M_{\max} &= -P_1\left(\frac{H_1}{3} + x\right) - p_B\frac{x^2}{2} - \tfrac{1}{2}(\gamma_1' K_{a1})(1\text{ ft})\frac{x^3}{3} - T\left(x + (H_t - H_w)\right) \\ &= -9{,}815 \text{ ft - lb} \\ &= -117{,}800 \text{ in - lbs} \end{aligned}$$

The magnitude of bending moment is higher than any of the moment capacities for 800 Series sections with 1-bar centered reinforced options presented in Table 2.

Checking the options for 2-bar centered reinforcement presented in Table 3, the reinforcement bar size combined with concrete compressive strengths shown below are acceptable for this wall section:

- No. 8 or larger for f'_c = 3000 psi
- No. 7 or larger for f'_c = 3,500 psi and higher
- No. 6 or larger for f'_c = 4,500 psi and higher

Checking the options for 4-bar reinforcement presented in Table 4, No. 5 or larger bars combined with concrete compressive strength of 3,000 psi or high are acceptable for this wall section:

Design Example Using PYWall 2013

The computer application PYWall 2013 from Ensoft (*www.ensoftinc.com*) considers soil-structure interaction by using a generalized beam-column model and analyzes the behavior of a flexible retaining wall or soldier-pile wall with or without tiebacks or bracing systems. Unlike the limit equilibrium analysis methods discussed previously, PYWall solves the nonlinear differential equation for a beam column that ensures compatibility of displacements of the wall and resistance forces exerted by the soil. In addition, PYWall can include the force versus deformation behavior of the tiebacks used for anchored walls.

The output from PYWall includes computation and graphs of lateral deflection, bending moment, and shear force versus wall elevation.

The earth pressures acting on an anchored wall are computed from the Rankine earth pressure resultant and the geometry of the wall above the line of excavation. The Rankine earth pressure resultant is computed for simple, uniform conditions without a water table present using

$$P_{AE} = \frac{1}{2} K_A \gamma H^2 - 2c\sqrt{K_A} H \quad \text{..} \quad (6)$$

If the ground slope is inclined at an angle of β, the active earth pressure coefficient is computed using

$$K_A = \cos\beta \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}} \quad \text{..} \quad (7)$$

If the ground slope is flat, the active earth pressure coefficient is computed using

$$K_A = \tan^2\left(45° - \frac{\phi}{2}\right) = \frac{1 - \sin\phi}{1 + \sin\phi} \quad \text{..} \quad (8)$$

$$\sigma_a = \frac{1.3 P_{AE}}{2/3\,H} \quad \text{...} \quad (9)$$

When a water table is present, the computations of Rankine earth pressures must account for the different effective unit weight of soil above and below the water table. In this example, the depth of the water table is 5 feet below the ground surface.

The soil properties for this example are shown in Table 14.

Table 14 – Example Soil Properties for Example Using PYWall

Soil Type	Depth Range, ft	Total Unit Weight	Cohesion	ϕ	ε_{50}	k, pci
Drained Clay	0-12 ft	124 pcf	-NA-	20 deg.	-NA-	10
Stiff Clay w/o Free Water	12–23 ft 0–276 in.	124 pcf 0.0718 pci	1,750 psf 12.15 psi	-NA-	0.007	600

The Rankine active earth pressure coefficient for the drained clay layer is computed using

$$K_A = \tan^2\left(45° - \frac{\phi}{2}\right) = \tan^2\left(45° - \frac{20°}{2}\right) = 0.490 \quad \text{...............................} \quad (10)$$

The active earth pressure resultant is computed by summing the resultants for the three sections of the active earth pressure diagram shown in Figure 10.

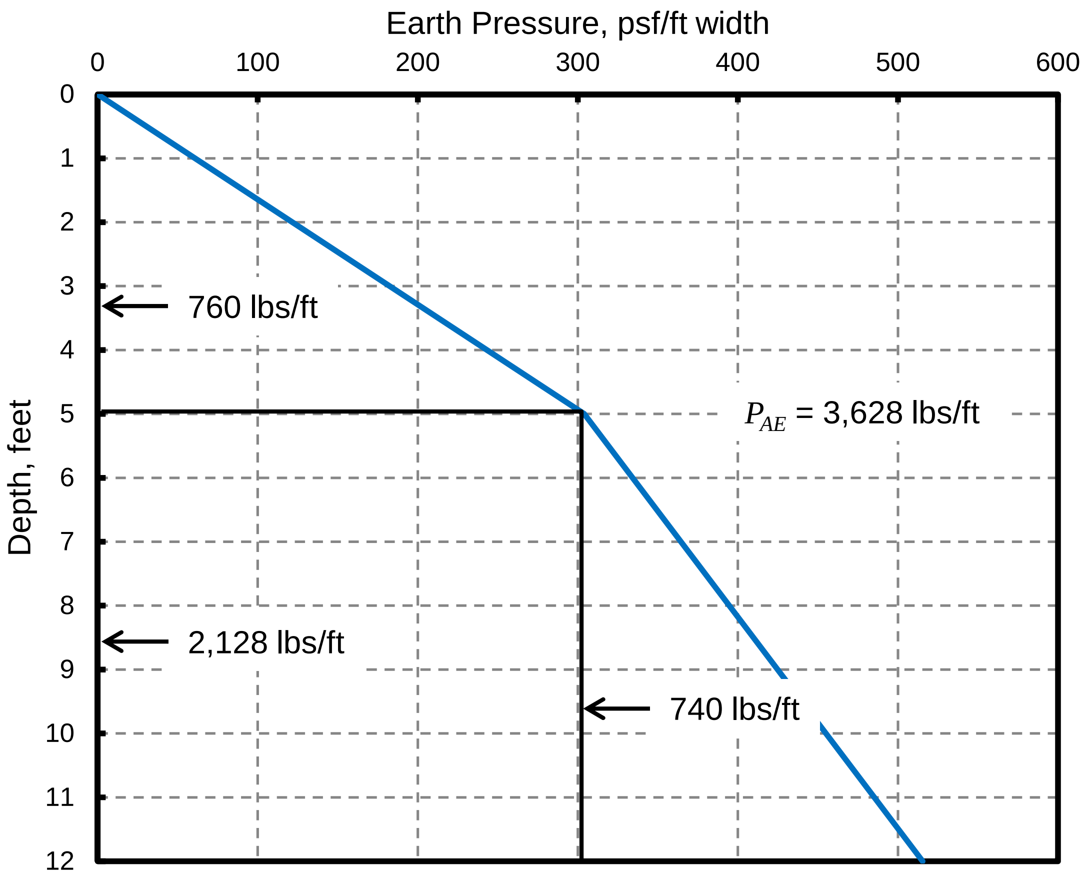

Figure 10 Rankine Active Earth Pressures

The earth pressure diagram that can either be input or computed by PYWall is shown in Figure 11 as a function of wall geometry.

The design pressure is

$$\sigma_a = \frac{1.3 P_{AE}}{2/3\,H} = \frac{1.3(3{,}628\ \text{lbs/ft})}{8\ \text{ft}} \frac{1\ \text{ft}}{12\ \text{in}} = 49.2\ \text{lbs/in/ft} \quad \text{..........................} \quad (11)$$

The design earth pressure diagram is based on the recommendations of the Technical Manual for PYWall 2013. The geometry of the earth pressure diagram for the geometry of the example of a 20-ft high wall with a tie-back anchor one foot below the top of wall is shown in Figure 12.

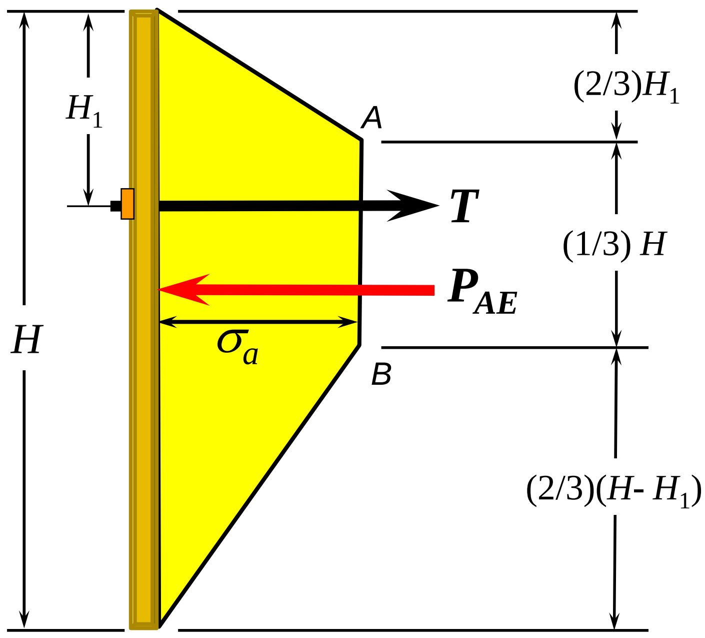

Figure 11 Recommended Earth Pressure Diagram for Anchored Flexible Retaining Structure

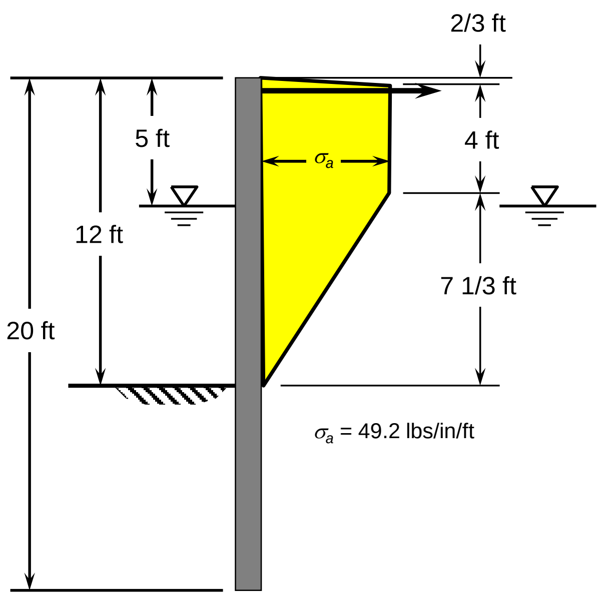

Figure 12 – Input Wall Pressure for Example

Equivalent Spring Constant for Tieback Anchor Rod

The tieback anchor will be modeled as a linear spring based on the extension of the tieback anchor bar attached to a unyielding anchor block at a depth one foot below the top of wall. The spring constant is equal to

$$k_{tieback} = \frac{AE}{L\ s_{tieback}} \quad (12)$$

Where A = cross sectional area of tieback anchor bar, E = Young's modulus, L = length of anchor bar, and $s_{tieback}$ = horizontal spacing between tieback anchor bars. For a No. 9 bar, 25 ft long, and a spacing of 6 ft, the anchor spring constant for a 12-inch wide section of wall is

$$k_{tieback} = \frac{AE}{L\ s_{tieback}} = \frac{(1\,\text{in}^2)(29{,}000{,}000\text{ psi})(12\text{ in})}{(300\text{ in})(72\text{ in})} = 16{,}110\text{ lbs/inch} \quad (13)$$

Computed Results

The input value of bending stiffness is for a 12-inch wide vertical section, as is listed in Tables 5 and 7. In a PYWall analysis, the moment developed is related to the bending stiffness of the wall. In general, the wall deflection will decrease as the bending stiffness increases. So, it is necessary to try different reinforcement options to determine the most economical wall section to use. A set of computed results is shown in Table 15. In every case, f'_c = 4,000 psi and the section is 4-bar reinforced.

Table 15 –Results Computed by PYWall 2013 for Example Problem

Wall Reinforcement	EI, lbs-in^2	M_{ult}, in-lbs	M_{max}, in-lbs	M_{max}/M_{ult}	Maximum Deflection inches	Anchor Force lbs/ft
No. 4	242,900,000	104,900	65,600	0.625	0.601	2,410
No. 5	323,775,000	138,100	65,700	0.476	0.481	2,410
No. 6	404,646,000	174,100	65,730	0.378	0.409	2,410
No. 7	489,700,000	216,200	65,690	0.304	0.358	2,410
No. 8	576,993,000	264,500	65,590	0.248	0.321	2,410

An examination of these results finds that a 800 Series wall section with a concrete compressive strength of 4,000 psi and reinforced with No. 4 bars is the minimum section that will work for this example problem. However, the computed wall deflection decreases if larger reinforcement is used.

The Figures 13 through 15 are graphs of lateral deflection, bending moment and shear force versus depth below the top of wall for the 800 Series wall section with a concrete compressive strength of 4,000 psi and 4-bar reinforced with No. 5 bars.

The lateral deflection profile is shown in Figure 13. The peak deflection is 0.481 inches and is developed at an elevation approximately 5.5 feet below the top of the wall. The deflection of the top of the wall is restrained to be approximately 0.1 inches by the tie-back anchor.

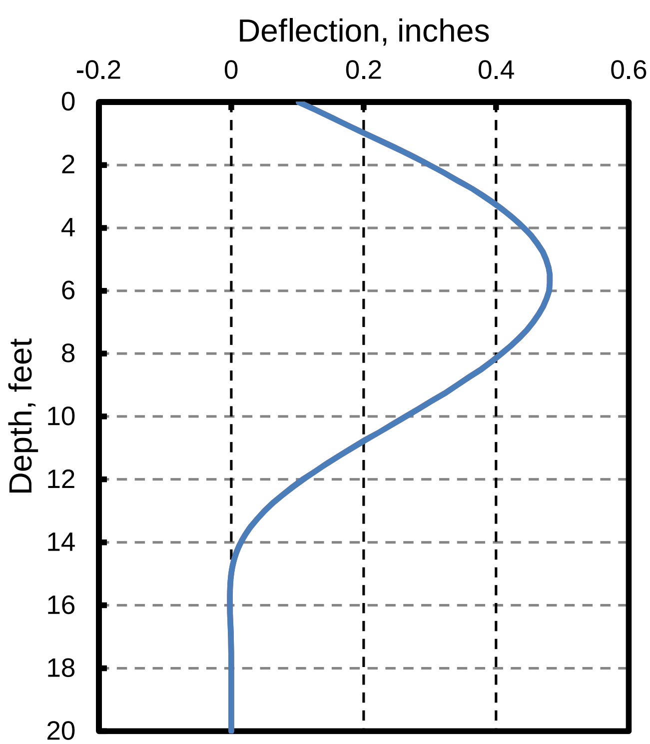

Figure 13 – Lateral Deflection vs. Depth Below Top of Wall

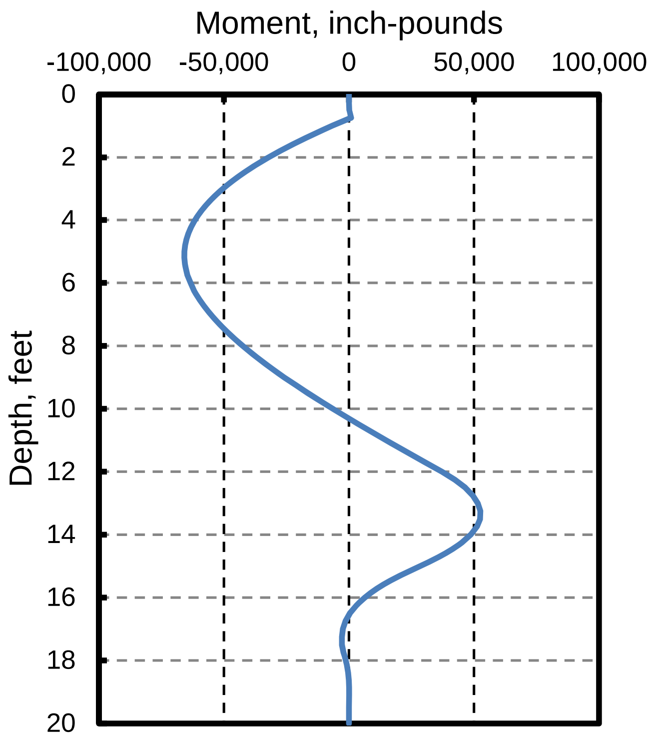

Figure 14 – Bending Moment vs. Depth Below Top of Wall

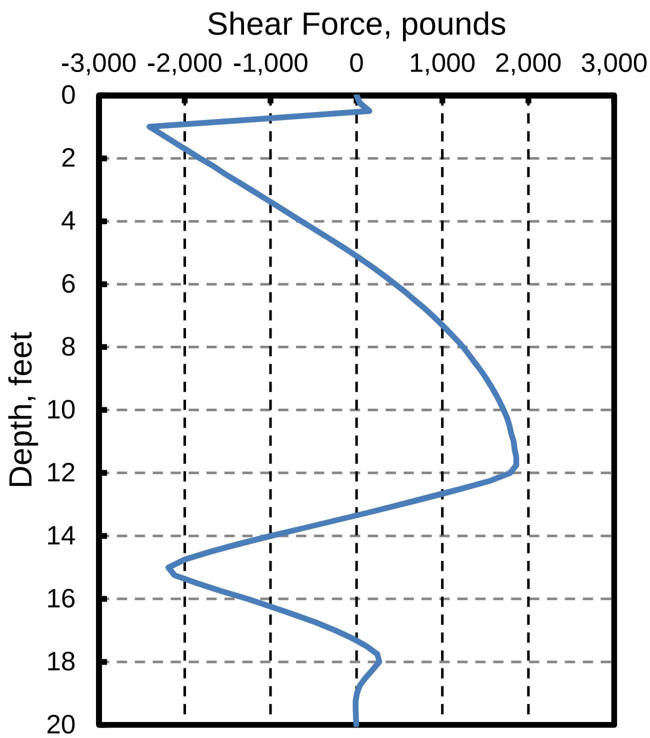

Figure 15 – Shear Force vs. Depth Below Top of Wall

The bending moment profile in the wall is shown in Figure 14. The peak moment is 65,700 in-lbs/ft of wall and occurs at an elevation approximately 5.25 feet below the top of the wall. This is just above the location for the peak lateral deflection. The bending moment in the wall is close to zero from the top of wall to the depth of the tie-back anchor at 1 foot.

The shear force profile is shown in Figure 15. There are two locations of high shear force in the wall and one location of high shear force at the connection point for the tieback anchor. The two high shear force values are 1,860 lbs/ft width just above the dredge line (12 ft) and a value of 2,190 lbs/ft width at 15 feet below the top of the wall.

The maximum shear force in the wall is due to the tieback and is 2,410 lbs/ft width. The actual force developed in the tieback will depend on the horizontal spacing between tiebacks. For example, if the tiebacks are spaced 6 ft apart on centers, the force in an individual tieback is computed to be 14,460 lbs.

A summary of the structural performance of the wall in the design example is presented in Table 16. The moment capacity of the section without concrete is 53,100 in-lbs/ft of wall (see Table 1) and the maximum moment developed is 65,700 in-lbs/ft of wall, so the use of a reinforced concrete-filled section is required for this example. The shear capacity of the section without concrete is 6,300 lbs/ft and for the reinforced concrete filled section is 11,580 lbs/ft. Both of these values exceed the maximum computed shear force in the wall of 2,410 lbs/ft. Once the requirements for structural limit states are satisfied, the designer should use maximum computed wall deflection as the criteria for selection of the amount of reinforcement in the wall.

Table 16 Summary of Structural Performance for Design Example

Structural Limit State	Computed for Design Example	Structural Capacity of Section without Concrete	Structural Capacity of 4-rebar reinforced with No. 5 bars
Bending Moment	65,700 in-lbs/ft	53,100 in-lbs/ft Not OK	138,100 in-lbs/ft OK
Shear Force	2,410 lbs/ft	6,300 lbs/ft OK	11,580 lbs/ft OK

Optimization of the Design

It is possible to optimize the design by reducing or eliminating the steel reinforcement below the depth in the wall where it is needed. In the design example, the lateral deflection is very small below 14 ft below the top of wall and the bending moment developed in this section is below the structural moment capacity of section without concrete. Thus, placement of a gravel fill can be substituted for the reinforced concrete fill below 14 ft below the top of the wall.

If the maximum tolerable deflection developed in the wall is larger than 0.5 inches, one can explore the possibility of substituting either a 1-bar centered or 2-bar centered reinforcement option for the 4-bar reinforcement used in the design example. One should recognize that when the bending stiffness of the wall is reduced, the lateral deflection developed in the wall will increase. Thus, it is necessary to repeat the analysis of wall deflections for each reinforcement option being considered.

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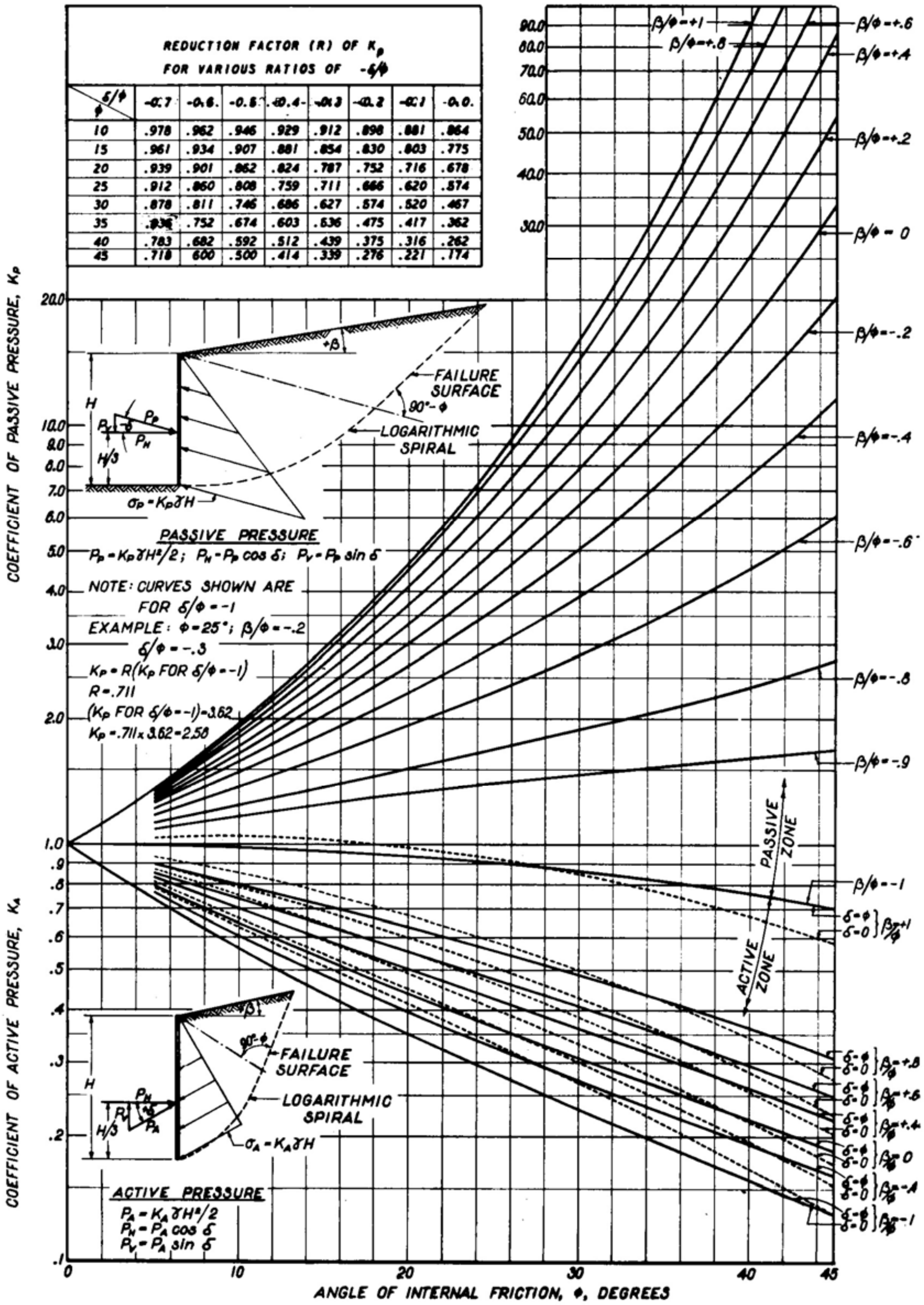
Appendix A – Earth Pressure Coefficients for Wall Friction and Sloping Backfill



ϕ \ δ/ϕ	-0.7	-0.6	-0.5	-0.4	-0.3	-0.2	-0.1	-0.0
10	.978	.962	.946	.929	.912	.898	.881	.864
15	.961	.934	.907	.881	.854	.830	.803	.775
20	.939	.901	.862	.824	.787	.752	.716	.678
25	.912	.860	.808	.759	.711	.666	.620	.574
30	.878	.811	.746	.686	.627	.574	.520	.467
35	.836	.752	.674	.603	.536	.475	.417	.362
40	.783	.682	.592	.512	.439	.375	.316	.262
45	.718	.600	.500	.414	.339	.276	.221	.174

Figure A1 – Active and Passive Earth Pressure Coefficients with Wall Friction and Sloping Backfill (from NAVFAC ,1986 and Caquot and Kerisel, 1948)