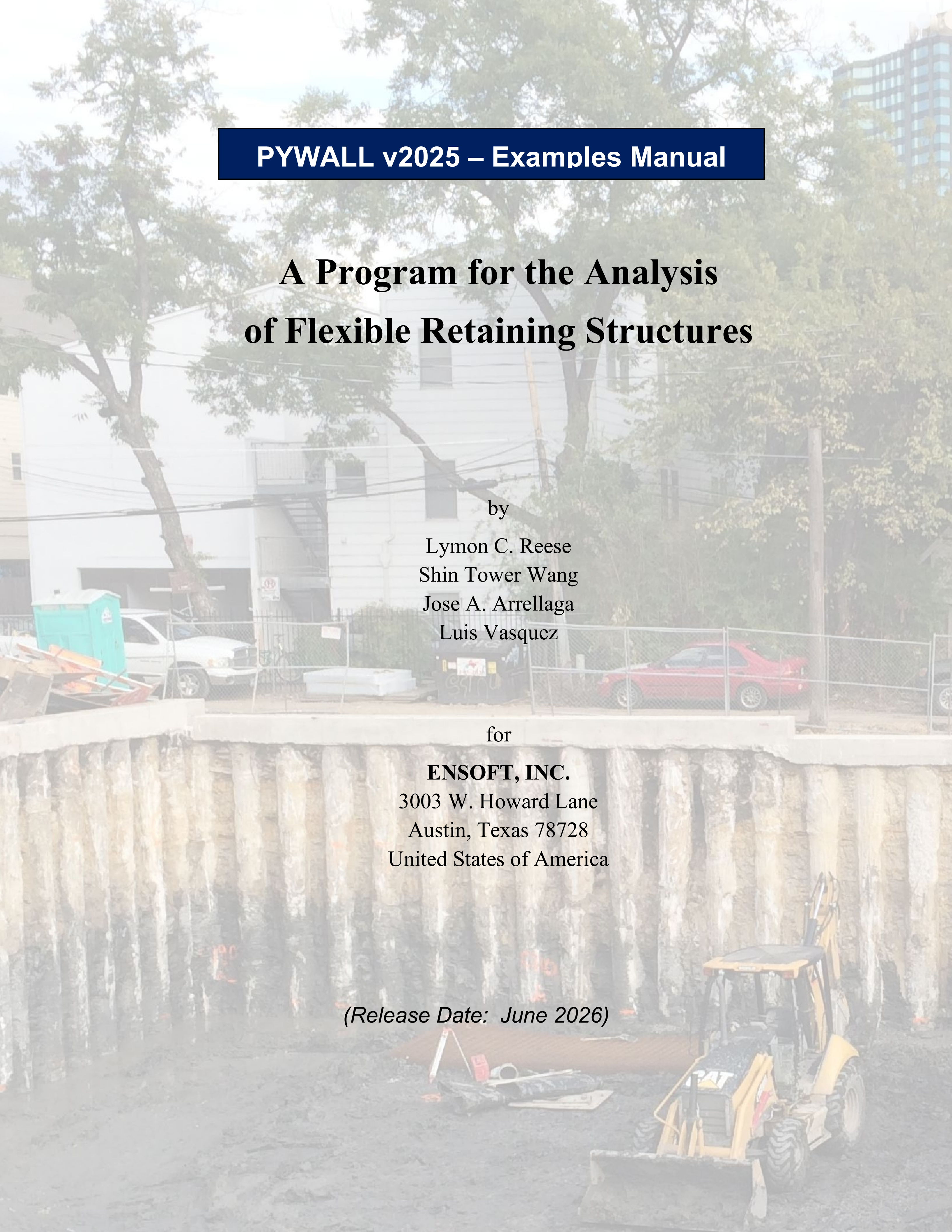

PYWALL v2025 – Examples Manual

A Program for the Analysis of Flexible Retaining Structures

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Example 1. Elastic Sheet Pile Wall in Granular Soil

1.1.1 Example 1 – Elastic Sheet Pile Wall in Granular Soil

1.1.2 Problem Description

The model for this example was presented in a July 1975 publication from United States Steel (USS) titled Steel Sheet Piling Design Manual. The published example uses the traditional limit-equilibrium method to analyze and design the wall penetration and the size of the steel section.

The free height of the wall (cantilever portion) is 14 ft. The sheet pile is embedded in granular soils, and the groundwater is pumped out of the excavation side (Figure 1.1). The granular soil above the excavation line has a total unit weight of 115 lb/ft^3 and an internal friction angle of 35^o. The granular soil below the excavation line has a submerged unit weight of 65 lb/ft^3 and an internal friction angle of 35^o. The total length of the sheet pile was proposed to be 27.5 ft based on the limit-equilibrium method.

USS recommends that active earth pressures should be computed using Coulomb's theory if the wall friction angle is taken into consideration. If the wall friction angle (δ) is zero with the flat backfill, the active earth pressure coefficient from Coulomb's theory is equal to that of the Rankine theory. Therefore, K_A (0.277) is equal to $\tan^2(45 - \phi/2)$. USS also recommends that passive earth pressure should be computed using the Coulomb theory with an appropriate safety factor. In this example, the wall friction angle is assumed to be 0.5ϕ. Therefore, the coefficient of the passive earth pressure of 6.56 was used in the limit-equilibrium analysis. PYWALL also follows the Coulomb equation with the wall friction angle equal to 0.5ϕ in generating the passive earth resistance for a continuous sheet-pile wall.

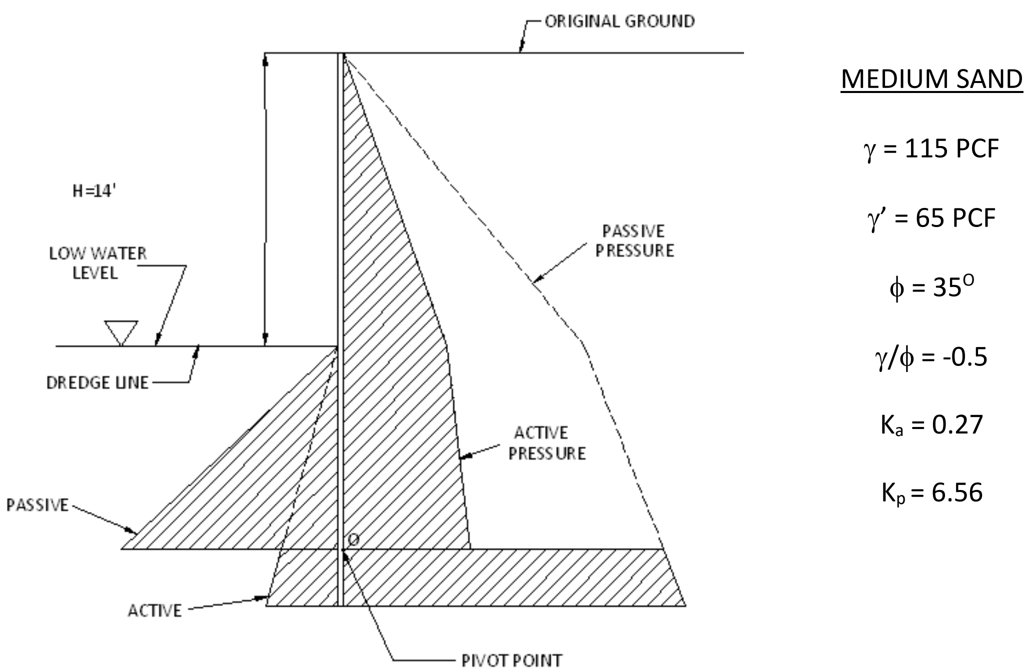


Figure 1.1 Pressure Distribution of Cantilevered Sheet-Pile Wall in Granular Soil

1.1.3 Computations Based on the Limit-Equilibrium Method

The assumed pressure diagram based on the conventional Rankine theory is presented in Figure 1.2.

From Statics, the following two conditions must be satisfied

(1) $\Sigma FH = 0$

In Terms of Areas:

Area (BAA_1) + Area (AA_1A_2F) + Area (ECJ) – Area (EA_1A_2) = 0

Or:

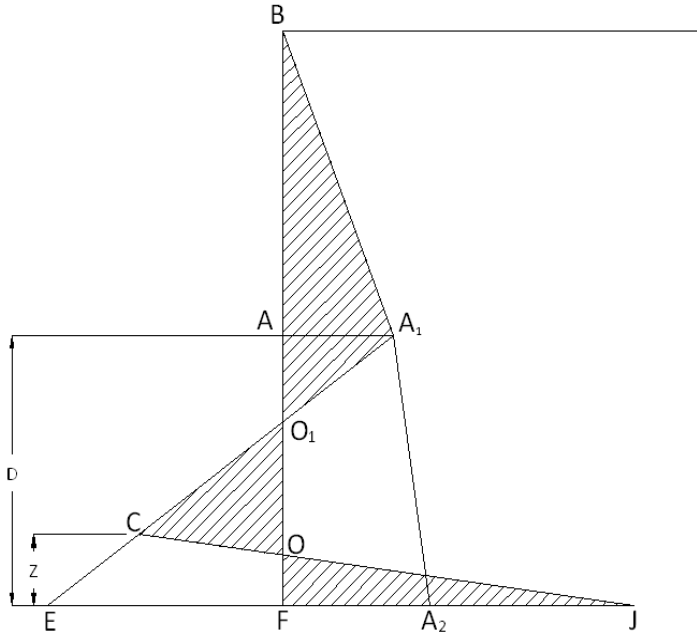
$$\frac{1}{2}(H)PA_1 + (PA_1 + PA_2)\frac{D}{2} + (P_E + P_J)\frac{Z}{2} - (P_E + P_{A2})\frac{D}{2} = 0$$

Solving For Z:

$$Z = \frac{(P_E - P_{A1})D - HP_{A1}}{P_E + P_J}$$

(2) ΣM about point F (at bottom of wall) must be Zero:

$$\Sigma M_F = \frac{1}{2}(H)\,P_{A1}\,(D + \frac{H}{3}) + (P_{A1})\frac{D^2}{2} + (P_E + P_J)\frac{Z^2}{6} - (P_E + P_{A2})\frac{D^2}{6} + (P_{A2} - P_{A1})\frac{D^2}{6} = 0$$



CALCULATION OF WALL PRESSURES:

$P_{A1} = \gamma_e H K_a = (115)(14.0)(0.27) = 435$ PSF

$P_{A2} = P_{A1} + \gamma_e D K_a = 435 + (65)(0.27)D$

$= 435 + 17.6D$

$P_E = \gamma_e D (K_p - K_a) - P_{A1} = 65D (6.29) - 435$

$= 408.9D - 435$

$P_J = \gamma_e D (K_p - K_a) + \gamma_e H K_P = 65D (6.29)$

$+ 115 (14)(6.56)$

$P_J = 408.9D + 10562$

Figure 1.2 Pressure diagram for conventional analysis of Example 1

Method of Solution:

1. Assume a depth of penetration, D
2. Calculate X (Point of Zero Shear)
3. Substitute X into ΣM_F and check if zero. Adjust D and recalculate if necessary.

1.1.3.1 Assume Depth of Penetration

(1) Try an initial D = 10.5 FT

P_{A1} = 435 PSF P_{A2} = 620 PSF P_J = 14855 PSF P_E = 3858 PSF

$$Z = \frac{(3858-435)(10.5)-(14)(435)}{14855+3858} = \frac{29852}{18713} = 1.60 \text{ FT}$$

$$\Sigma M_F = \frac{1}{2}(14)\,435\,(10.5+4.67) + (435)\frac{(10.5)^2}{2} + (620-435)\frac{(10.5)^2}{6} + (3858+14855)\frac{(1.60)^2}{6}$$

$$-\,(3858+620)\frac{(10.5)^2}{6}$$

ΣM_F = 43193 + 23979 + 3399 +7984 – 82283

ΣM_F = -728 FT.-LB. SAY O.K. D = 10.5 FT works OK.

However, to assure a margin of safety, D may be increased by 20 to 40% or, alternatively, a reduced passive earth pressure coefficient could be used.

Therefore:

USE D = 13.5 FT (INCREASE = 28.5%)

1.1.3.2 Locate Point of Zero Shear

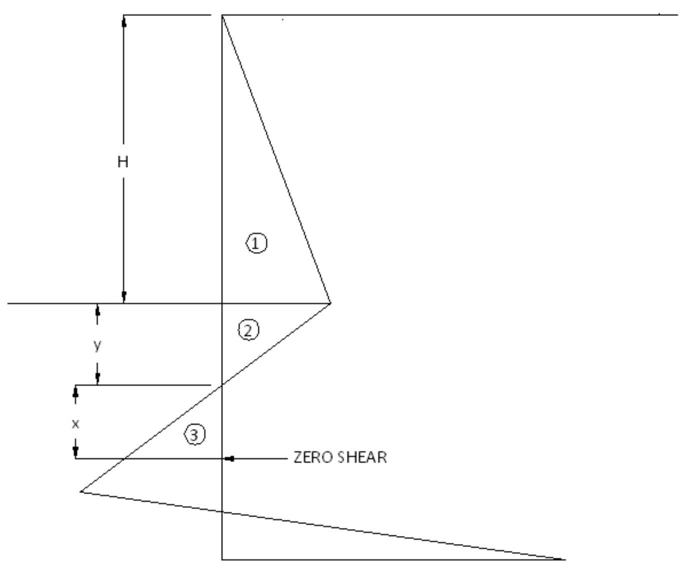


Figure 1.3 Shear diagram for conventional analysis

$$y = \frac{P_{A1}}{(K_P-K_a)\gamma'} = \frac{435}{65\,(6.29)} = 1.06 \text{ FT}$$

SAY 1.0 FT

$$P1 = \frac{1}{2}\,PA1\;H = \frac{1}{2}(435)(14) = 3040 \text{ LB}$$

P2 = $\frac{1}{2}$ PA1 y = $\frac{1}{2}$ (435)(218) = 218 LB

$\frac{1}{2}\gamma'$ ($K_P - K_a$) x^2 = $P_1 + P_2$

$$x^2 = \frac{2(P_1 + P_2)}{\gamma'\,(K_P - K_a)}$$

$$x^2 = \frac{2(3040+218)}{65(6.29)} = \frac{2(3258)}{407} = 16$$

x = 4.0 FT

1.1.3.3 Calculate Maximum Moment and Design the Section

$P_3 = \frac{1}{2}\gamma'$ ($K_P - K_a$) 4^2 = $P_1 + P_2$ = 3280 LB

$M_{MAX} = P_1 L_1 + P_2 L_2 - P_3 L_3$

$$L_1 = (\frac{H}{3} + y + x)$$

$$L_2 = (\frac{2y}{3} + x)$$

$$L_3 = \frac{x}{3}$$

$$M_{MAX} = 3040(\frac{14}{3} + 1.0 + 4.0) + 218(\frac{2(1)}{3} + 4.0) - 3280(\frac{4.0}{3})$$

M_{MAX} = 29300 + 1030 - 4360 = 26000 FT-LBS

Try regular carbon grade (ASTM A328); f_s= 25 KSI (allowable stress)

Required section modulus = $\frac{M_S}{f_S} = \frac{26000 \, X \, 12}{25000}$ = 12.5 IN^3

Must use MZ–27

Try Gr. 45 steel (ASTM A572 GR45), with f_s= 29 KSI (allowable stress)

Required section modulus = $\frac{26000 \, X \, 12}{29000}$ = 10.76 IN^3

Therefore can USE PDA–27 with S = 10.7 IN^3 (alternate section)

1.1.3.4 Proposed Design Based on Limit-Equilibrium Method

Based on the traditional limit-equilibrium analysis and design, the proposed total wall length is 27.5 ft and the calculated maximum bending moment was 26,000 ft-lbs. The steel section used to construct the wall was selected to be PDA-27 Gr 45 steel, with moment of inertia equal to 53.0 in^4 per pile, elastic section modulus equal to 30.2 in^3 per linear ft, and area equal to 11.91 in^2 per pile.

1.1.4 Computations Based on the *p-y* Curve Method (PYWALL)

The *p-y* Curve Method, also known as the SSI Method (Soil-Structure Interaction Method), is the numerical solution used within the PYWALL computer software. The retaining-wall model that is being analyzed is the same as described earlier in Section 1.1.2.

1.1.4.1 Input-Data for Initial PYWALL Model

An initial model in PYWALL is made with the input-data file *Example 1a - Sheet Pile in Sand - Initial.py7d* that is installed with the program. Some of the model details are as follows:

- For convenience in the comparisons, the unit width of retaining wall used in the PYWALL analysis was selected as 12 inches. See Data > Cross Section > Edit Section 1 where it was selected an initial General section type to represent the PDA-27 sheet pile that has a Moment of Inertia $I_{yy} = 39.8\ in^4$ (per ft of wall length) and using the default Young's Modulus of Elasticity $E_s = 29{,}000{,}000\ psi$. The Equivalent Diameter of 12 inches was also selected to correspond to the entered flexural properties (per ft of wall length).
- As in the limit-equilibrium method, the initial PYWALL model estimates a wall length (total wall height or total length of each sheet pile) as 27.5 ft. This height of the wall was divided in 110 increments (thus resulting in 3-in increment length). See Data > Wall Properties where other inputs are left as default. In the button Enter Section Data the user associates the previously-defined wall section (Cross Section Index = 1) to the entire length of the sheet pile (total wall height), from 0 to 27.5 ft.
- The soil input parameters defined under Data > Soil Layer Data are summarized in Table 1.1. Notice that a second layer of sand is defined with a higher total unit weight of 127.4 pcf so that it agrees with the problem statement indicating 65 pcf of submerged unit weight (62.4 pcf is used internally as water weight in PYWALL). The bottom of the second soil layer is selected at a depth that is slightly deeper than the estimated wall tip.

Soil Type	Layer Thickness (ft)	Total Unit Weight (lbs/ft^3)	Friction Angle (deg)	K_{py} (lbs/in^{3}s)
Sand	14	115	35	30
Sand	16	127.4	35	30

Table 1.1 Soil Input Parameters for Example 1

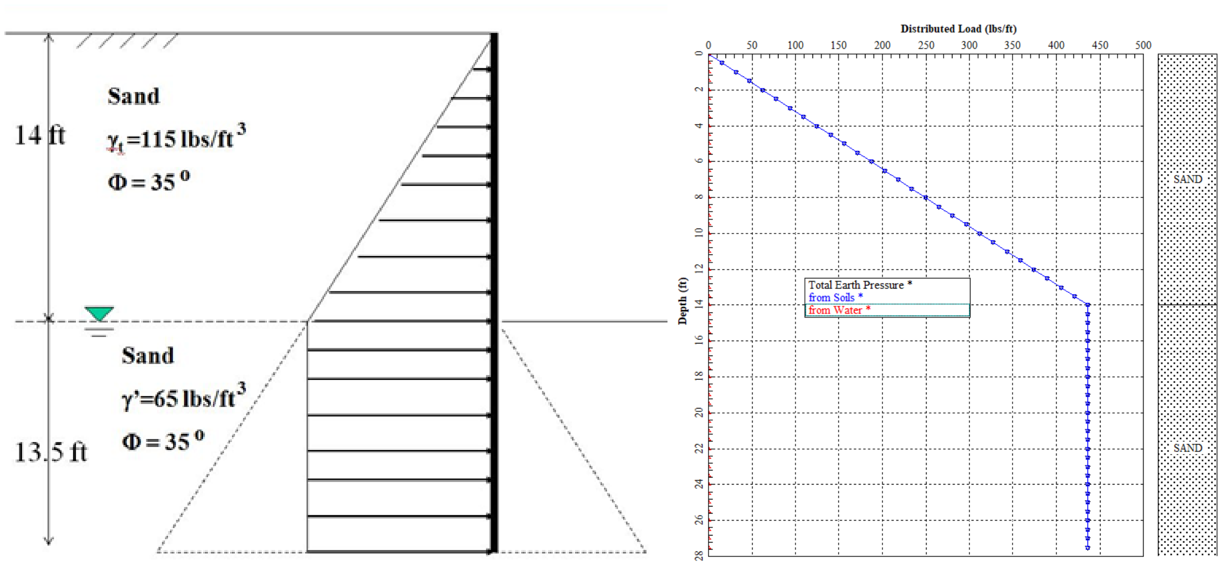
- Under Data > Wall Configuration the model defines the 14-ft free height of wall (cantilever portion) and depths to water table on both sides of the wall. For a continuous sheet-pile wall (CASE 2 in referenced graphics at middle right for this dialog box) the width parameters for soil resistance and active pressures are equal ($W_p = W_a$). For convenience, the user previously selected (under Data > Cross Section) a 12-in width. A check mark next to Get From Cross Sections will reproduce the value previously entered under Data > Cross Section > Equivalent Diameter.

1.1.4.2 Check of Internally-Generated Forces

The PYWALL program generated the triangular distribution of active earth pressure above the excavation line (Options > Earth-Pressure Options > Generate Triangular-Distribution Earth-Pressure) and the soil resistance (*p-y* curves) below the excavation line (Options > Generate p-y Curves).

A simple hand calculation of the earth pressure behind the wall at the bottom of the excavation can be made to check the earth pressure computed by the program. The depth of the cantilever portion is 14 ft and the active earth pressure computed by the program at the bottom of the excavation (Graphics > Earth Pressure Combined) is 436.3 lbs/ft (per ft of width, so = 436.3 psf), which is very close to the $P_{A1} = 435$ *psf* previously calculated in Figure 1.2 by the limit-equilibrium method.

The *p-y* curve method treats the earth pressure above the excavation line as external distributed loads. However, the active earth pressure below the excavation line on the backfill side is also present due to the surcharge of soil mass above the excavation line. Therefore, the earth pressure of 436.3 lb/ft computed at the final excavation depth is extended by PYWALL constantly below on the backfill side to represent the load from the surcharge, as shown in Figure 1.4 and in Graphics > Earth Pressure Combined.



a. Schematics **b. Computed Active Loading in PYWALL Model**

Figure 1.4 Earth-Pressure Distribution Used in the *p-y* Curve Method for Example 1a

A soil resistance (the nonlinear *p-y* curve) is generated by the program at each depth increment below the excavation line to represent the resistance/support from the soil layers. The program solves the force equilibrium and displacement compatibility by using iterative procedures as discussed in the accompanying Technical Manual. The soil resistance curves (*p-y* curves) below the excavation line for this example were generated as shown in Figure 1.5 (Graphics > p-y Curves) based on the corresponding *p-y* criteria described in the accompanying Technical Manual of PYWALL. In Figure 1.5 it is noticeable that the soil resistance (*p*) at the backfill side (upper left quadrant) is higher than those at the excavation side (lower right quadrant) due to the difference in the overburden pressure.

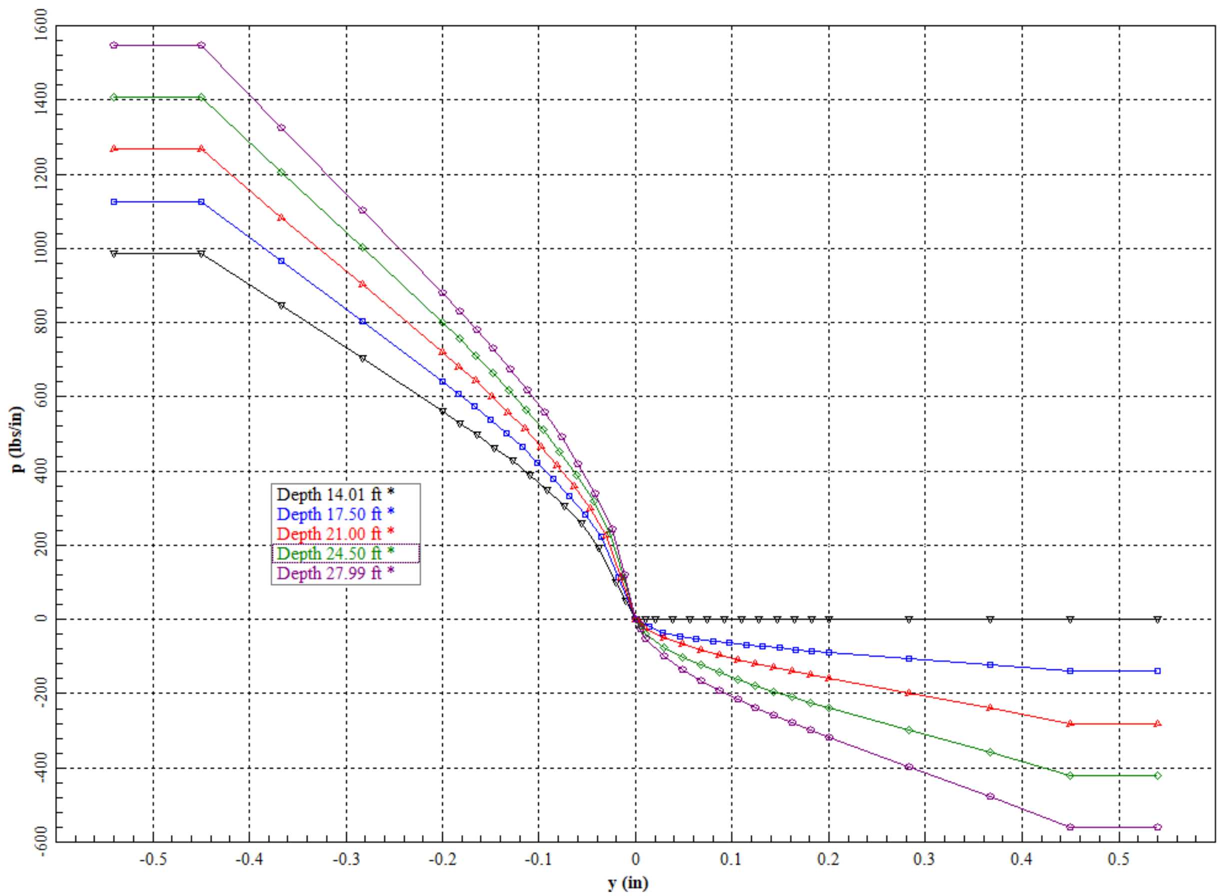


Figure 1.5 Soil resistance (*p*-*y*) curves generated for Example 1a

1.1.4.3 Check of Bending Moment, Shear and Deflection

The deflection curve, shear curve, and bending moment curve (Figure 1.6) versus wall depth were also computed by the *p*-*y* curve method because of the available options from the SSI model (select Graphics > Deflection, Moment and Shear to see all three result curves together in one graph, or alternatively each curve can also be selected individually). It is interesting to point out that the maximum bending moment was computed as 301.5 in-kips at a depth of 18.5 ft below the top of wall, which was very close to the maximum bending moment of 26,000 ft-lbs (312 in-kips) at depth of 19.06 ft (H+y+x) calculated previously using on the limit-equilibrium method (Section 1.1.3.3).

The maximum bending moment described above can be sustained with a PDA-27 sheet pile using Gr. 45 steel (as previously checked with service stress at end of Section 1.1.3.3). However, when observing the deflection profile of this wall (Graphics > Deflection vs Depth) the top deflection of 6.7 inches seems excessive for the 14-ft excavation height. Notice that the traditional limit-equilibrium method has difficulties in computing the deflection curve correctly since it is based on the limit-state soil resistance instead of the actual mobilized soil resistance that is used in PYWALL.

This large deflection at the top of the retaining wall predicted by the *p*-*y* curve method also provides a reason for the designer to check the deflection tolerance for the retaining wall and adjust the design accordingly. Increasing the wall section's rigidity will help to reduce the deflections. Program PYWALL provides a convenient and rational way to investigate different options for optimizing the final design.

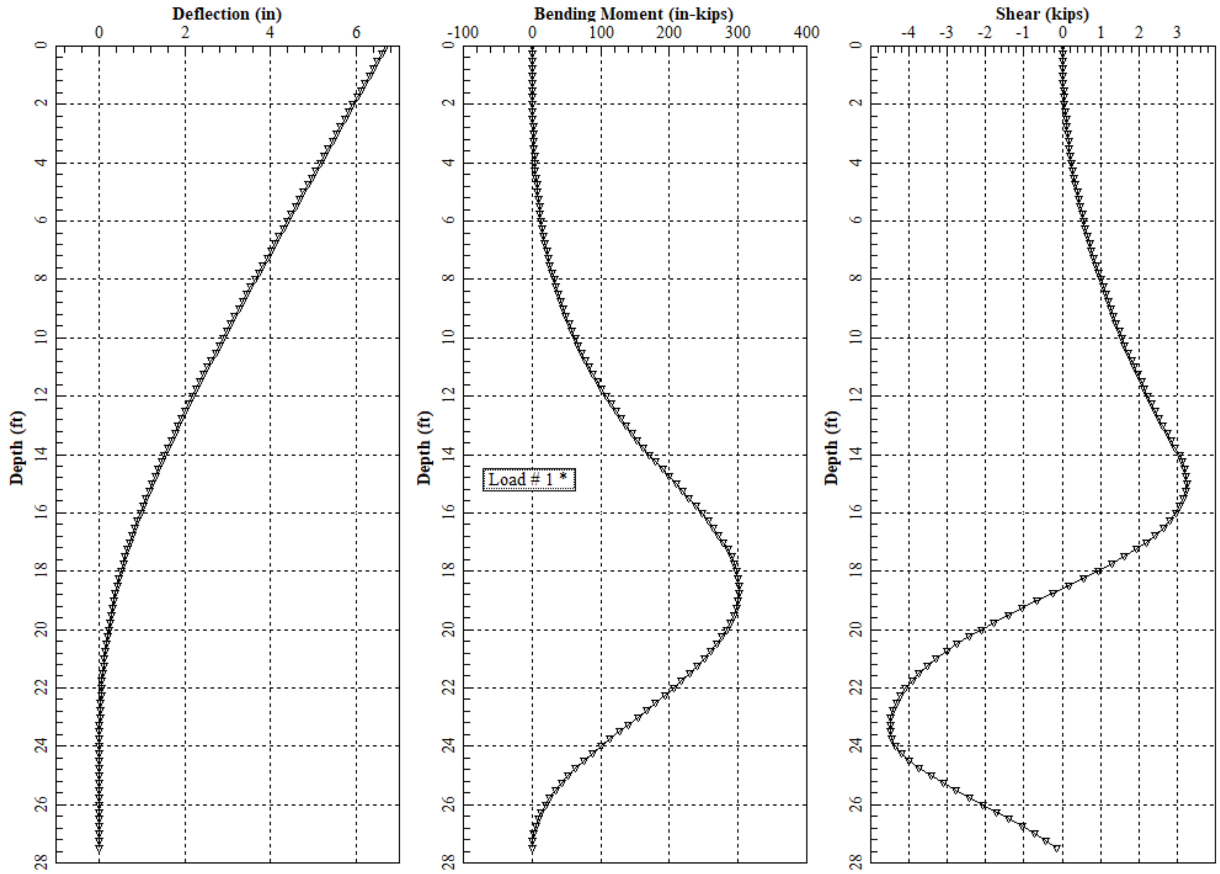


Figure 1.6 Deflection, Shear and Bending Moment Curves from the PYWALL Model for Example 1a

1.1.4.4 Second PYWALL Model

In order to control deflections, a sheet pile of deeper profile (with higher flexural stiffness) than the initially-selected PDA-27 is needed. A second model in PYWALL is analyzed (*Example 1b - Sheet Pile in Sand - Final PZ22.py7d*) with a PZ-22 sheet pile having a moment of inertia $I_{yy}=91.1\ in^4$ (per ft of wall length) or $167\ in^4$ (per sheet pile unit width of 22 inches) which is much higher than the $39.8\ in^4$ (per ft of wall length) from the previous trial with PDA-27 sheet piles.

In the new model (*Example 1b - Sheet Pile in Sand - Final PZ22.py7d*) the sheet pile cross section properties are selected using the sheet piling database that is included in PYWALL (Data > Cross Sections > Edit Section 1). The analysis is now made using the width (or length, depending on point of view) of a single sheet pile instead of the foot length of wall. See Figure 1.7 for reference.

As expected, the maximum lateral movement at the top of the wall is now reduced to 3.4 inches (from previous 6.7 inches with PDA-27). This is shown in Figure 1.8 (which also shows the soil stratigraphy and wall height by selecting Plot > Show Soil Layer when observing the plot). This level of deflection may be now more acceptable for most situations. If further reductions in deflections are necessary then the user may need to select an even stiffer sheet pile, like a PZ-27 or even PZ-32. Those evaluations can be made easily within PYWALL with results obtained immediately.

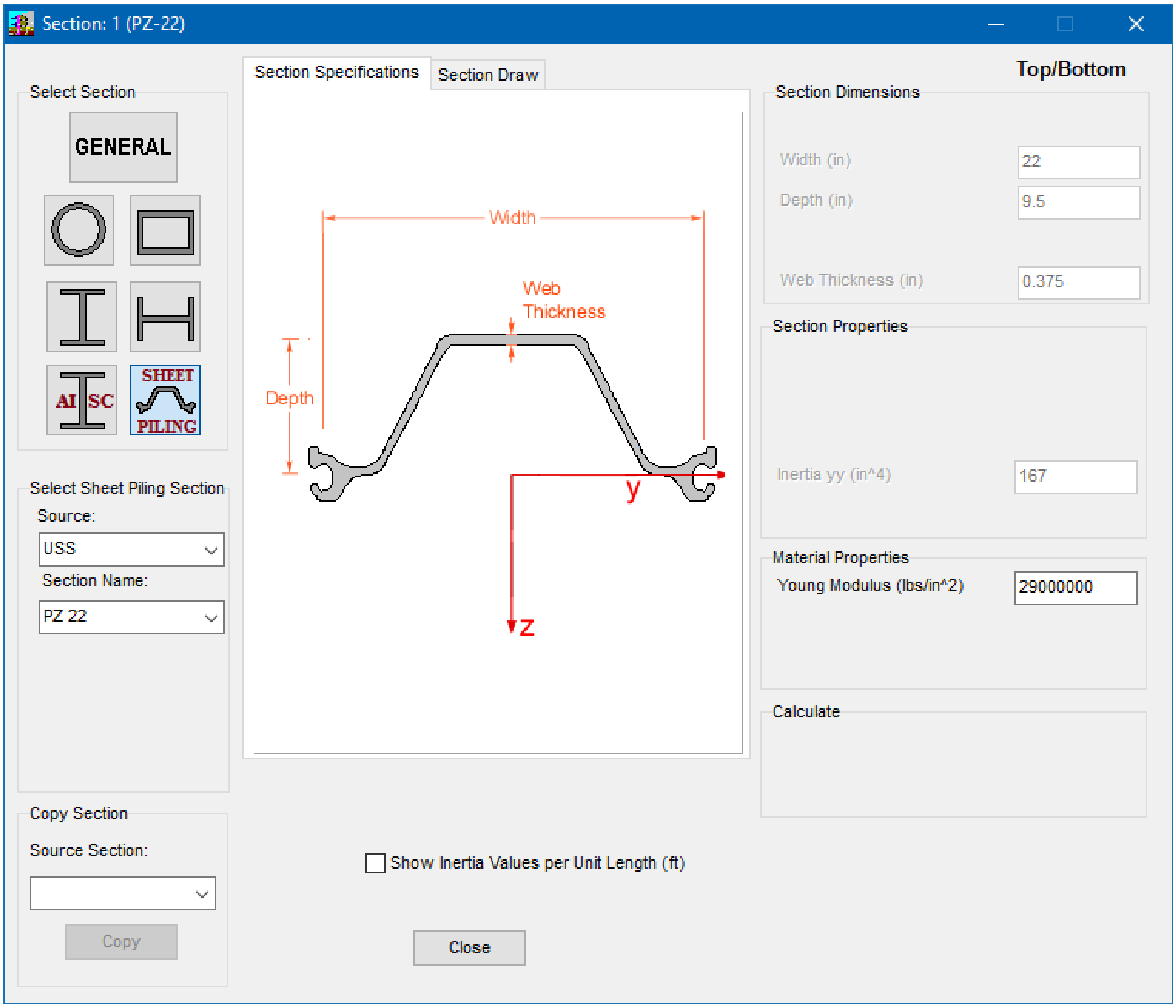


Figure 1.7 Automatic PZ-22 Sheet Pile Properties for Example 1b

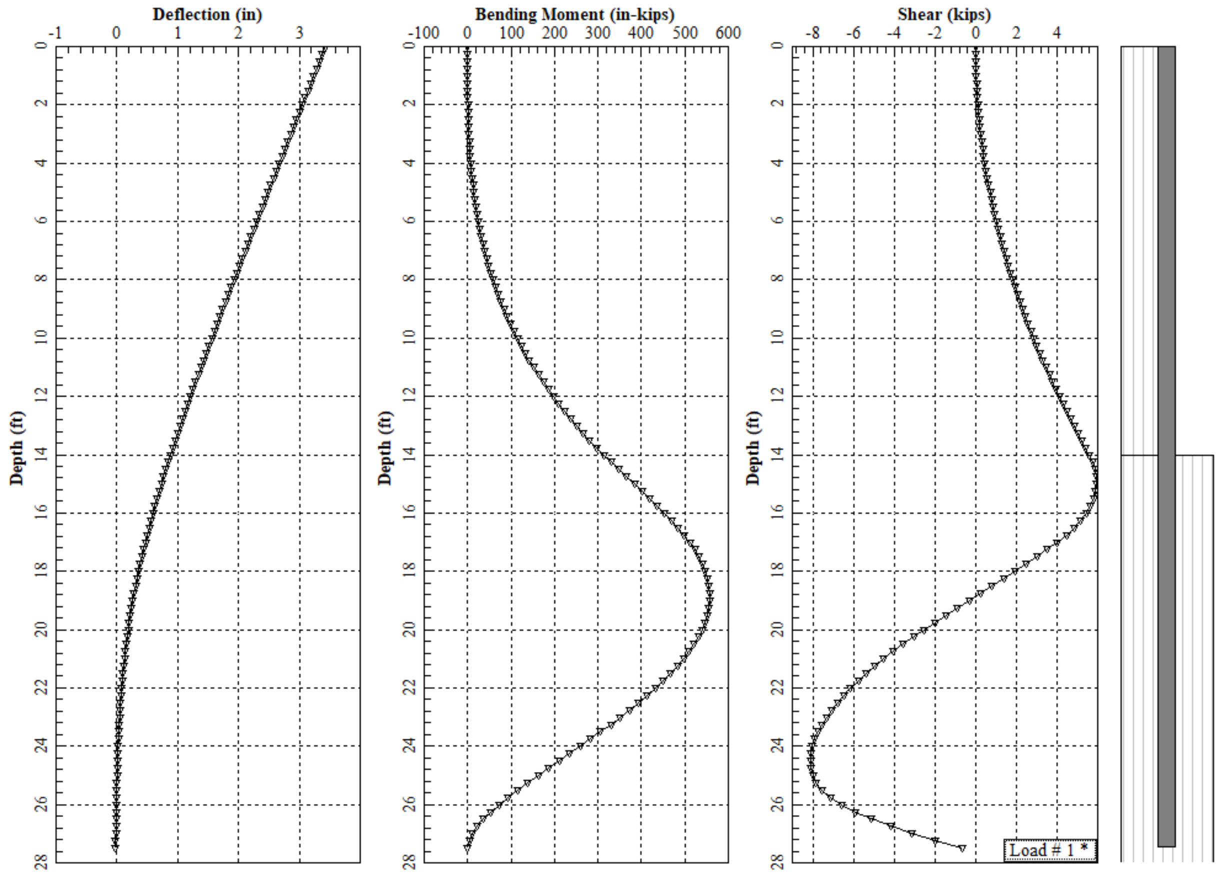


Figure 1.8 Results using PZ-22 Sheet Pile in Example 1b

1.1.5 Study of Total Wall Height for Example 1

The soil-structure interaction (SSI) method used in PYWALL allows the designer to make a theoretical evaluation of critical wall penetration. A computer run can be made for a trial total wall height (H_{wi}) to solve for the corresponding lateral deflection at top of wall (y_{ti}) so a point may be plotted as indicated in the sample graphics on Figure 1.9. The user can now make a series of runs (models in PYWALL) with varying wall depths to create the various points in the graph. In theory, the connection of points for the deflection at the top of the pile yields the curve in Figure 1.9.

A critical depth can be defined as the total wall height where the top deflection starts to increase, which is at approximately 40-ft in Figure 1.9. For the sample project in Figure 1.9, if the top deflection of about 4.3 inches is acceptable to the specific project, for a long-term design the engineer would choose a wall height of about 42 feet (a couple of feet beyond critical).

The critical penetration graph is not always precisely as the one in Figure 1.9. For the case of Example 1 the user can perform several models changing the total wall length to a wide range (but keeping the same pile increment length at 3 inches, to avoid mathematical approximations). The top wall deflection versus total wall length results for Example 1 are plotted in Figure 1.10. The slight increase of top wall deflection with wall length after the critical penetration of 27.5 feet is due to increased pressure in the cohesionless soil below the excavation. If the top movement in the range of 3.3 to 3.6 inches is acceptable for this retaining wall project, then a penetration of about 30 feet would be sufficient based on the plot of critical wall height.

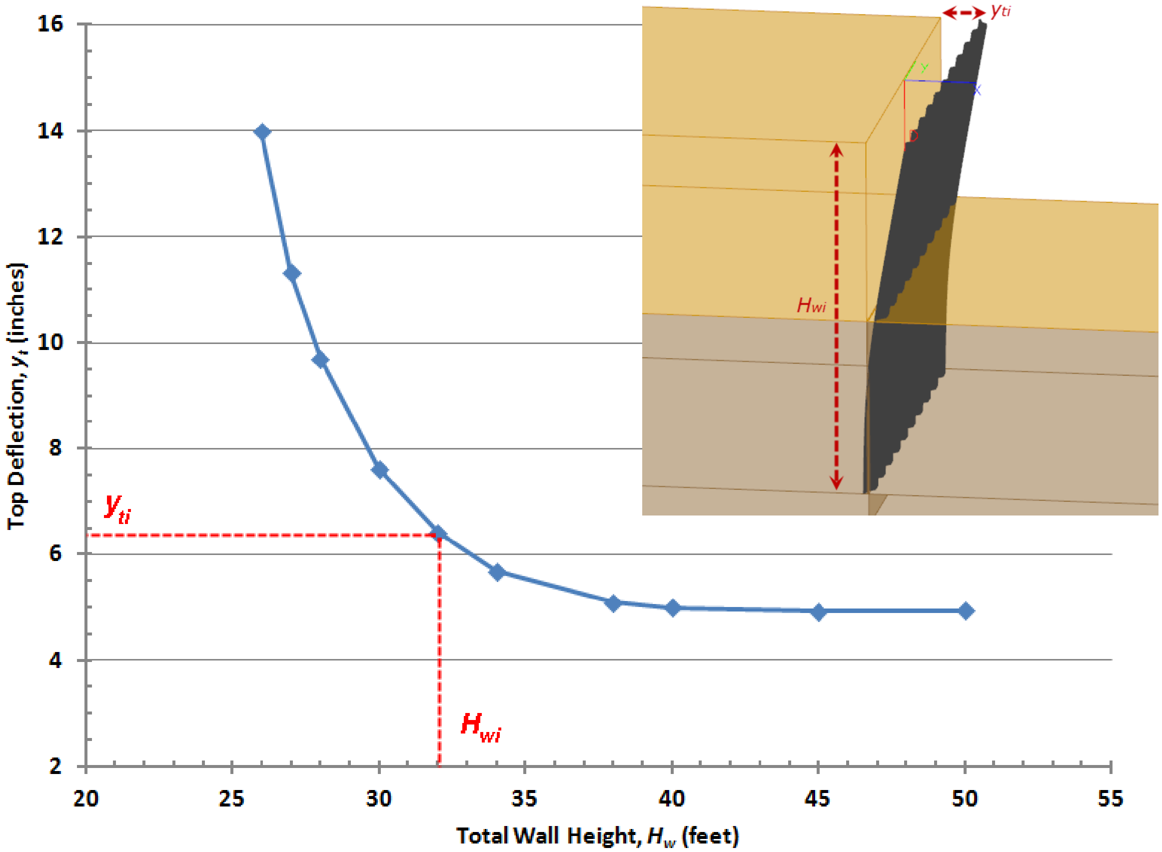


Figure 1.9 Theoretical Critical Wall Height

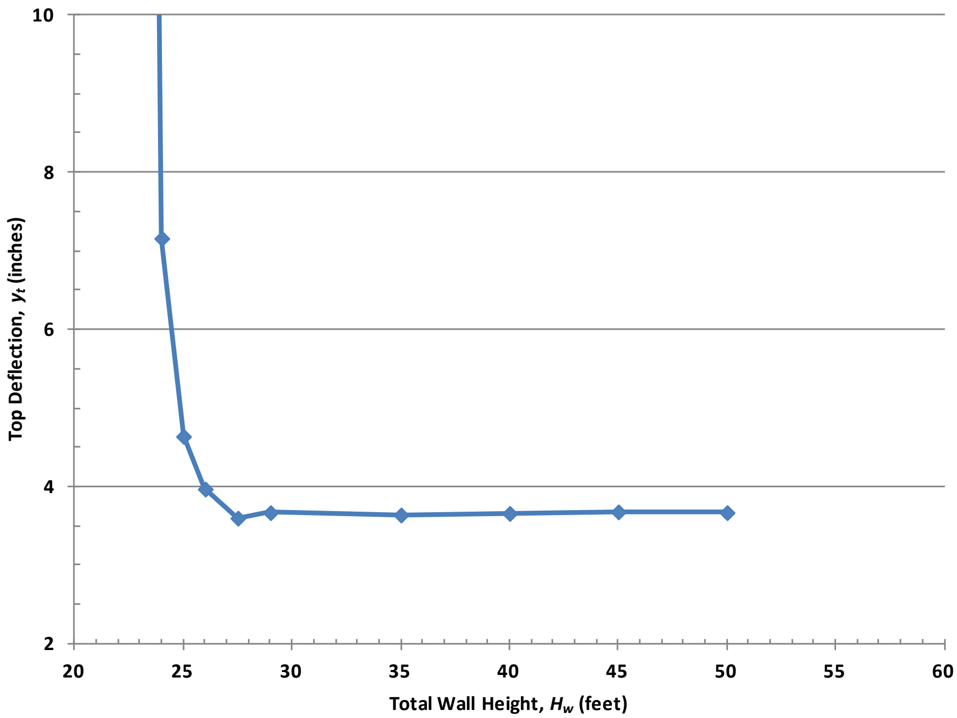


Figure 1.10 Solving for Critical Wall Height for Example 1b

1.1.6 Comments on Analytical Methods for Example 1

The distribution of the soil pressure and soil resistance is rational based on the *p-y* curve method. Structural engineers can get the information, such as the maximum bending moment, maximum shear, and the deflection profile, easily for the final design. The limit-equilibrium method has difficulties in computing the deflection curve because it is based on the limit-state soil resistance instead of the mobilized soil resistance.

In addition, computations are tedious using the traditional limit-equilibrium method. To simplify the computations, the soil layer below the excavation line has to be treated as a single uniform soil layer in order to calculate the required pile penetration for equilibrium with a high-order equation. Thus, for layered soils below the excavation it will be very difficult to solve by hand calculations based on the traditional method. Moreover, layered soils are commonly encountered in real applications. The *p-y* Curve Method was developed based on recognized structural theories. With the *p-y* Curve Method from PYWALL, both equilibrium and compatibility are automatically satisfied when the solution converges.

1.1.7 Input and Output Data Files for Example 1

Users can read Section 2.1.1 (7) of the User's Manual for reference on the location of placement of the input and output data files for the example files installed with this program. The default installation directory is the following: *(Root Drive) c:\Ensoft\PYwall2025-Examples*. The input data files for all examples presented in this manual are installed automatically with the program.

The input-data filenames for Example 1 are the following:

Example 1a - Sheet Pile in Sand - Initial.py8d

Example 1b - Sheet Pile in Sand - Final PZ22.py8d

The output-data filenames for Example 1 are the following:

Example 1a - Sheet Pile in Sand - Initial.py8o

Example 1b - Sheet Pile in Sand - Final PZ22.py8o

Example 2. Cantilevered Sheet Pile Wall in Cohesive Soil

2.1 Example 2 – Cantilevered Sheet Pile Wall in Cohesive Soil

2.1.1 Problem Description

Similar to Example 1, this is another case presented in the July 1975 publication from United States Steel (USS) titled Steel Sheet Piling Design Manual. The limit-equilibrium method is used to calculate the wall penetration and the maximum bending moment for the case of cohesive soils.

Cohesive soils may impose small active forces (in some cases even negative forces, which are ignored in PYWALL). In many cases the design of the retaining wall section for those conditions is done with the assumption of drained conditions for long-term behavior. This example is such a case, where the geotechnical designer suggested the replacement of the cohesive soils with a loose sand with zero cohesive strength and a friction angle of 27° to simulate long-term conditions (as indicated in Figure 2.1).

The total length of the sheet pile was proposed to be 28.5 ft based on the limit-equilibrium method. The free height of the wall (cantilever portion) is 14 ft and the embedment length is 14.5 ft. The soft clay above the excavation line has a total unit weight of 120 lb/ft^3 and selected cohesive strength of 500 psf. A pressure distribution for this model is shown in Figure 2.1.

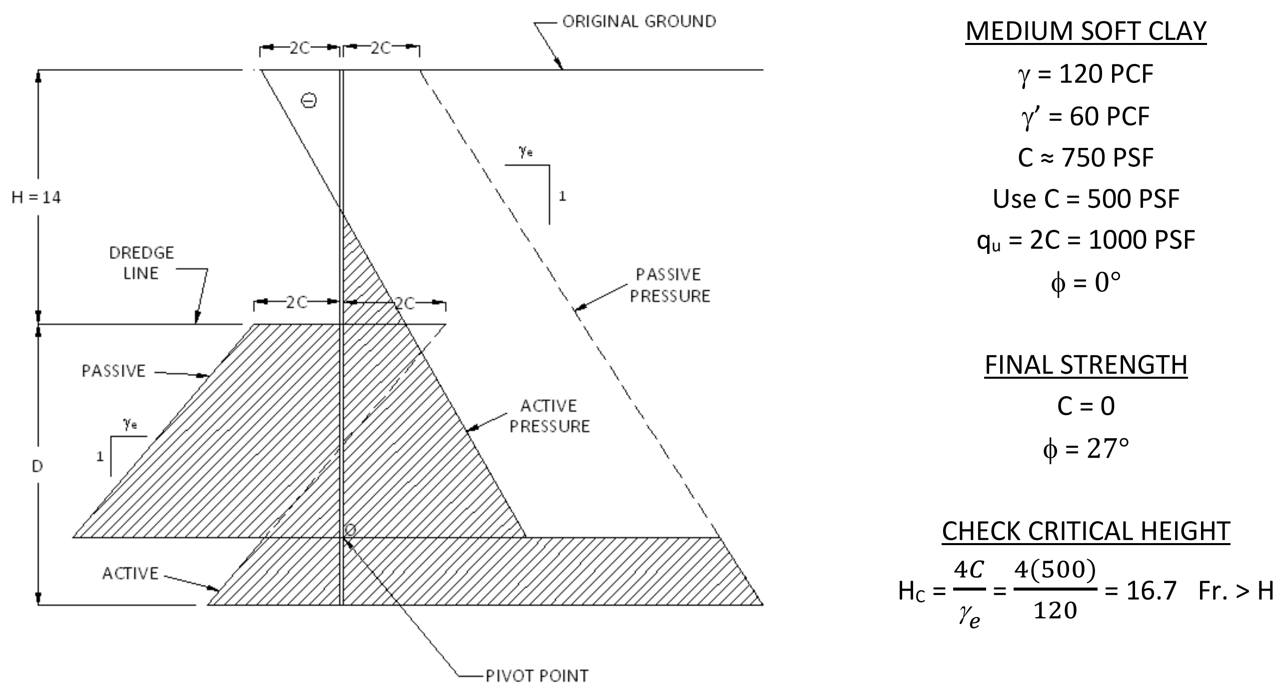


Figure 2.1 Pressure Distribution of Cantilevered Sheet-Pile Wall in Cohesive Soil

2.1.2 Computation Based on the Limit-Equilibrium Method

In the active state, the saturated cohesive soil initially develops a pressure diagram as shown in Figure 2.2.

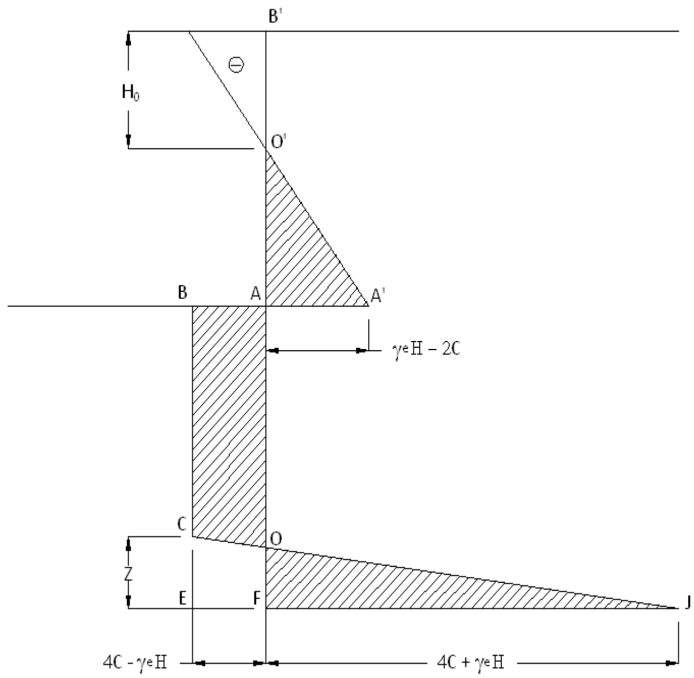


Figure 2.2 Net Pressure Diagram for Conventional Analysis of Example 2

First, determine the Wall Pressures:

Ho = Point of Zero Pressure (at top in Figure 2.2)

$$H_O = \frac{2c}{\gamma} = \frac{1000 \text{ psf}}{120 \text{ pcf}} = 8.3 \text{ ft}$$

$$H - H_O = 14.0 - 8.3 = 5.7 \text{ ft}$$

$$\gamma H - 2c = 120(14.0) - 1000 = 680 \text{ psf}$$

$$4c - \gamma H = 4(500) - 120(14.0) = 320 \text{ psf}$$

$$4c + \gamma H = 4(500) + 120(14.0) = 3680 \text{ psf}$$

From Statics, the following conditions must be satisfied:

(1) ΣFH = 0

In Terms of Areas:

Area (O'A'A) + Area (CJE) – Area (BAFE) = 0

Or:

$$\frac{1}{2}(\gamma H - 2c)H - H_O + \frac{8C(Z)}{2} - (4c - \gamma H)D = 0$$

Solving for Z:

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

(2) ΣM about point F (at bottom of wall) must be Zero:

$$\Sigma M_F = \frac{1}{2}(\gamma H - 2c)(H - H_O)(D + \frac{H-H_O}{3}) + \frac{8cZ^2}{6} - (4c - \gamma H)\frac{D^2}{2} = 0$$

Method of Solution:

1. Assume a depth of penetration, D

2. Calculate Z (Point of Zero Shear, $\Sigma F_H = 0$)

3. Substitute Z into ΣM_F and check if zero. Adjust D and recalculate if necessary.

2.1.2.1 Assume Depth of Penetration

$$Z = \frac{2(320)D - 5.7(680)}{8(500)} = 0.16D - 0.97$$

$$\Sigma M_F = \frac{1}{2}(680)(5.7)(D + 1.90) + \frac{8(500)Z^2}{6} = \frac{320D^2}{2} = 0$$

$$1938(D + 1.9) + 666.7Z^2 - 160D^2 = 0$$

OR

$$Z^2 = \frac{160D^2 - 1938(D + 1.90)}{666.7}$$

TRY D = 14.0

Z = 0.16(14.0) – 0.97 = 1.27 FT

CHECK $\Sigma M_F = 0$

$$Z^2 = \frac{160(14.0)^2 - 1938(14 + 1.90)}{666.7} = 0.82$$

Z = 0.9 FT < 1.27 FT NOT GOOD (Increase D)

TRY D = 14.5

Z = 0.16(14.5) – 0.97 = 1.35 FT

CHECK $\Sigma M_F = 0$

$$Z^2 = \frac{160(14.5)^2 - 1938(14.5 + 1.90)}{666.7} = 2.79$$

Z = 1.65 FT >1.35 FT SAY O.K. and use <u>D = 14.5 FT</u>

NOTE: D can be found directly by substituting expression [Z = 0.16D – 0.97] into the equation [$1938(D + 1.9) + 666.7Z^2 - 160D^2 = 0$]. This reduces to the quadratic equation [$142.9D^2 - 1731.1D - 4309.5 = 0$] and by this method D = 14.23 FT and Z = 1.31 FT.

2.1.2.2 Locate Point of Zero Shear

Based on Figure 2.2, the depth below the excavation line, where the shear is zero was calculated as:

X = (680) (14-8.3)(0.5) /(320) = 6.06 ft.

2.1.2.3 Calculate Maximum Moment

The maximum bending moment should develop at the depth of zero shear, which is 20.06 ft (= 14 + 6.06) below the top of the pile. The maximum bending moment developed at 20.06 ft was:

M_{max} = (680) (5.7)(0.5)(6.06+5.7/3) – (320) (6.06) (6.06) (0.5) = 9550 ft-lbs

2.1.2.4 Comment on Tension Zone in Cohesive Soil

The theoretical effect of cohesion creates a tension zone near the top. Consequently, it reduces the active earth pressure on the wall. In common practice, the tension is not considered in the soil and the negative earth pressure shown in the pressure diagram is ignored in the limit-equilibrium computation (as well as in PYWALL). However, the soil tension may create a crack/separation and may also fill with water. In such condition, the hydrostatic water pressure in the crack zone should be considered as a positive earth pressure in the computational models.

2.1.3 Computations Results Based on the *p-y* Curve Method (PYWALL)

The *p-y* Curve Method, also known as the SSI Method (Soil-Structure Interaction Method), is the numerical solution used within the PYWALL computer software. The retaining-wall model that is being analyzed is the same as described earlier in Section 1.1.2.

2.1.3.1 Input-Data for Initial PYWALL Model

The PYWALL model is made with the input-data file *Example 2a - Sheet Pile in Clay.py7d* that is installed with the program. Some of the model details are as follows:

- For convenience in the comparisons, the unit width of retaining wall used in the PYWALL analysis was selected as 12 inches. See Data > Cross Section > Edit Section 1 where the user selected an initial General section type to represent a PDA-27 sheet pile that has a Moment of Inertia $I_{yy} = 39.8\ in^4$ (per ft of wall length) and using the default Young's Modulus of Elasticity $E_s = 29{,}000{,}000\ psi$. The Equivalent Diameter of 12 inches was also selected to correspond to the entered flexural properties (per ft of wall length).
- As in the limit-equilibrium method, the PYWALL model estimates a wall height (total length of each sheet pile) as 28.5 ft. This height of the wall was divided with the 114 increments (so as to obtain even 3-in increment length). See Data > Wall Properties where other inputs are left as default. In the button Enter Section Data the user associates the previously-defined wall section (Cross Section Index = 1) to the entire length of the sheet pile (total wall height), from 0 to 28.5 ft.
- The soil input parameters defined under Data > Soil Layer Data are summarized in Table 2.1. Notice that a second layer of clay is defined with a total unit weight of 122.4 pcf so that it agrees with the problem statement indicating 60 pcf of submerged unit weight (62.4 pcf is used internally in PYWALL as water weight). The bottom of the second soil layer is made at a depth that is slightly deeper from the estimated wall tip.

Soil Type	Layer Thickness (ft)	Total Unit Weight (lbs/ft^3)	Cohesive Strength (psf)	E50
Soft Clay	14	120	500	0.01
Soft Clay	15	122.4	500	0.01

Table 2.1 Soil Input Parameters for Example 2

- Under Data > Wall Configuration the model defines the 14-ft free height of wall (cantilever portion) and depths to water table on both sides of the wall. For a continuous sheet-pile wall

(Case 2 in reference graphics at middle right) the width parameters for soil resistance and active pressures are equal ($W_p = W_a$). For convenience, the user previously selected (under Data > Cross Section) a 12-in width. A check mark next to Get From Cross Sections will reproduce the value previously entered under Data > Cross Section.

2.1.3.2 Check of Internally-Generated Forces

The PYWALL program generates the triangular distribution of active earth pressure above the excavation line (Options > Earth-Pressure Options > Generate Triangular-Distribution Earth-Pressure) and the soil resistance (*p-y* curves) below the excavation line (Options > Generate p-y Curves).

Results from the PYWALL model in Graphics > Earth Pressure Combined indicate that the tension zone of pressure in the wall extends to the depth of 8 ft. This is in agreement with the calculations from the traditional limit-equilibrium method in Section 2.1.2 where $H_o = 8.3\ ft$ (difference is related to nodal points that are every half foot increment for earth pressures in PYWALL).

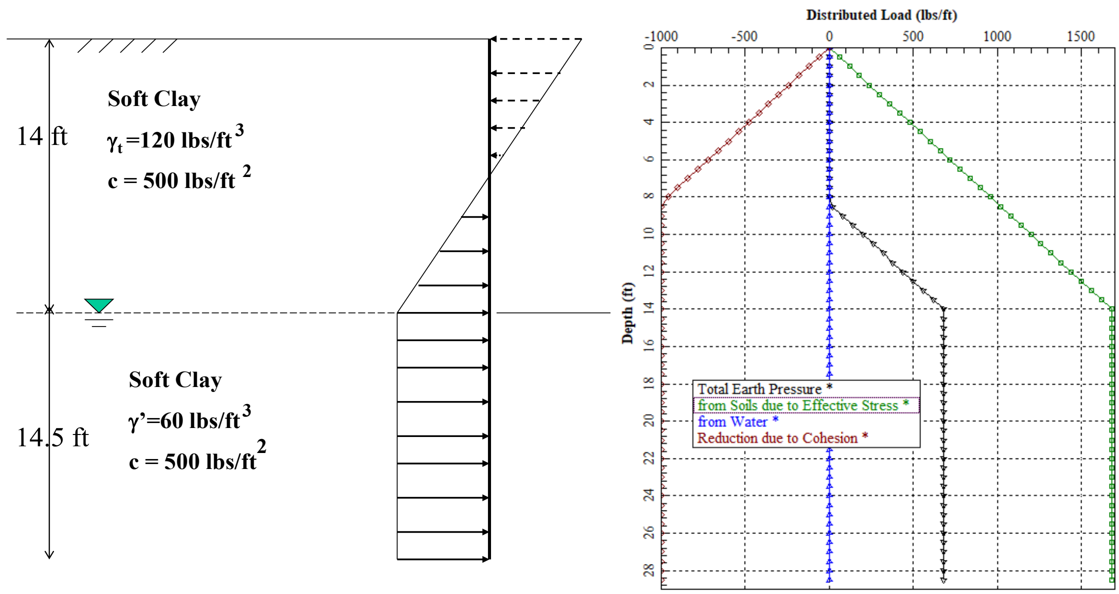
As it can be observed at 14-ft depth in Graphics > Earth Pressure Combined, the net active earth pressure computed by PYWALL at the bottom of the excavation is 680 lb/ft (per ft of width, so = 680 psf). This value is equal to the $\gamma H\text{-}2c = 680\ psf$ previously calculated in Section 2.1.2 by the limit-equilibrium method.

The *p-y* curve method treats the earth pressure above the excavation line as external distributed loads. The active earth pressure below the excavation line on the backfill side is significant due to the surcharge of soil mass above the excavation line. Therefore, the earth pressure of 680 lb/ft computed at the final excavation depth is extended constantly below on the backfill side to represent the load from the surcharge, as shown in Figure 2.3 and in Graphics > Earth Pressure Combined.

A soil resistance (the nonlinear *p-y* curve) is generated by the PYWALL program at each depth increment below the excavation line to represent the resistance/support from the soil layers. The program solves the force equilibrium and displacement compatibility by using iterative procedures as discussed in the accompanying Technical Manual. The soil resistance curves (*p-y* curves) below the excavation line for this example were generated as shown in Figure 2.4 (Graphics > p-y Curves) based on the corresponding *p-y* criteria described in the accompanying Technical Manual of PYWALL. In Figure 2.4 it is noticeable that the soil resistance (*p*) at the backfill side (upper left quadrant) is higher than those at the excavation side (lower right quadrant) due to the difference in the overburden pressure.

2.1.3.3 Check of Bending Moment, Shear and Deflection

The deflection curve, shear curve, and bending moment curve (Figure 2.5) versus wall depth were also computed by the *p-y* curve method because of the available options from the SSI model (select Graphics > Deflection, Moment and Shear to see all three result curves together in one graph, or alternatively each curve can also be selected individually, also select Plot > Show Soil Layer to view the soil stratigraphy next to the plots). It is interesting to point out that the maximum bending moment was computed as 110 in-kips at a depth of 19.8 ft below the top, which was close to the maximum bending moment of 9,550 ft-lbs (114 in-kips) at depth of 20.06 ft (H+y+x) calculated previously using on the limit-equilibrium method (Section 2.1.2.3). The small difference between the two methods is likely from the soil resistance that is accounted more accurately in PYWALL based on deflection (or actual mobilization of the structure).



a. Schematics **b. Computed Active Loading in PYWALL Model**

Figure 2.3 Earth-Pressure Distribution Used in the *p-y* Curve Method for Example 2a

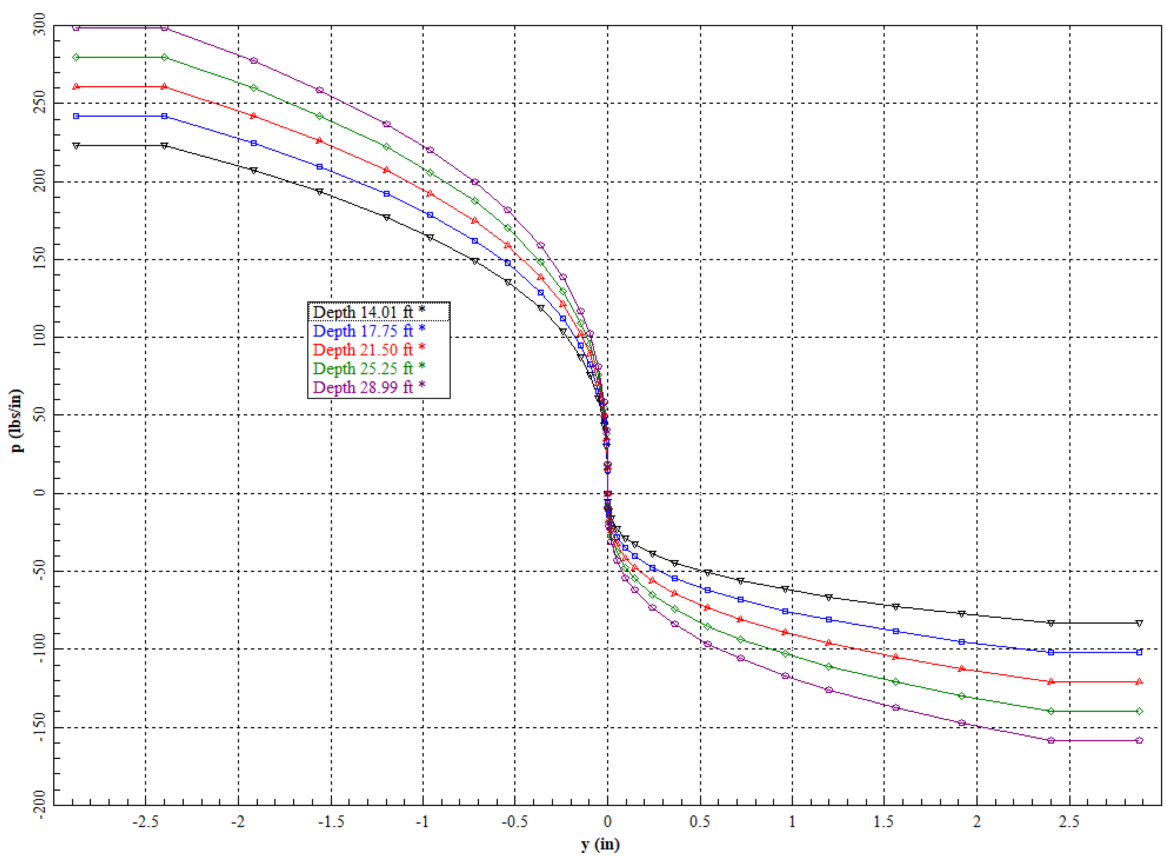


Figure 2.4 Soil Resistance (*p-y*) Curves Generated for Example 2a

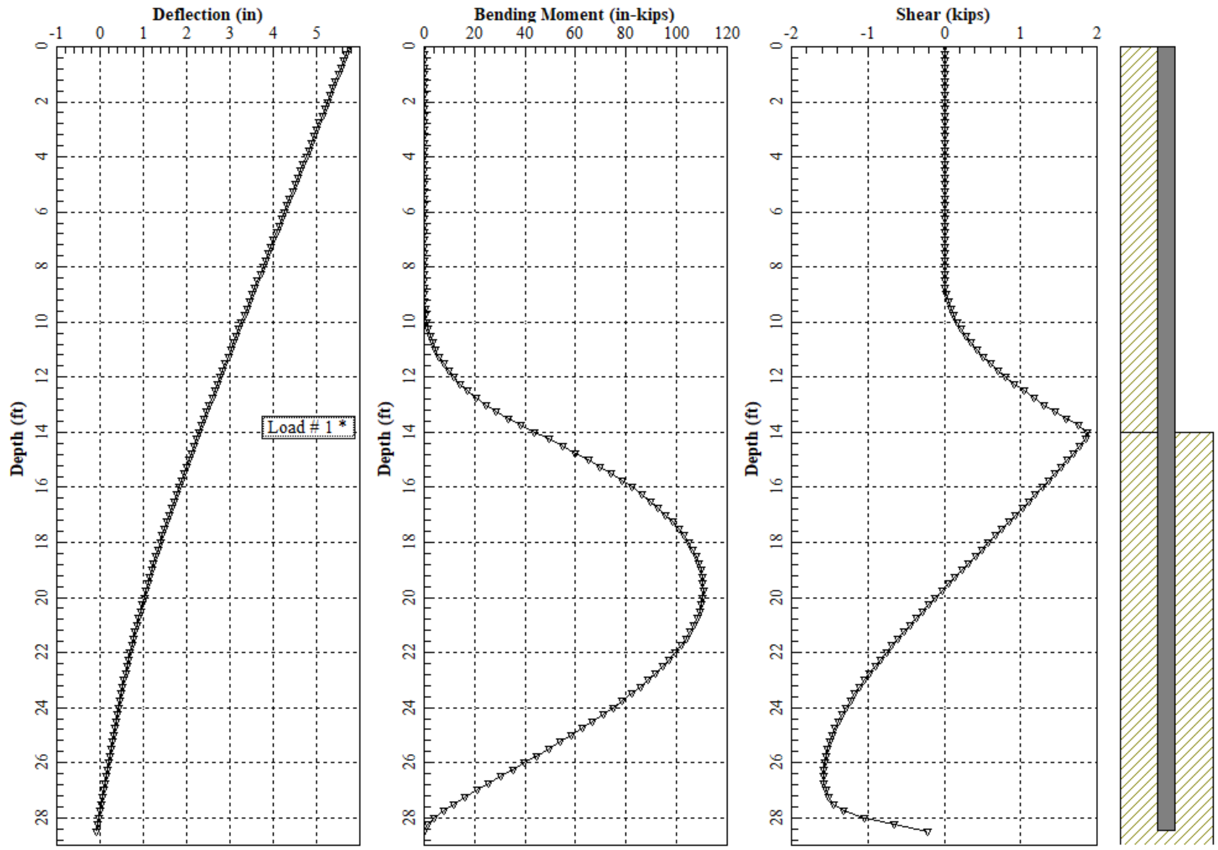


Figure 2.5 Computed deflection, shear and bending moment profiles based on the *p-y* Curve Method for Example 2a

When using the PDA-27 sheet pile section, the deflection profile of this wall (Graphics > Deflection vs Depth) indicates a top deflection of 5.8 inches which is considerable over a 14-ft excavation height. The limit-equilibrium method has difficulties in computing the deflection curve correctly because it is based on the limit-state soil resistance instead of the actual mobilized soil resistance that is used in PYWALL.

This large deflection at the top of the retaining wall predicted by the *p-y* curve method also provides a reason for the designer to check the deflection tolerance and adjust the design accordingly. Increasing pile embedment length or the section rigidity will reduce the wall deflections. Program PYWALL provides a convenient and rational way to investigate different options for optimizing the final design.

2.1.3.4 Improved Design for Example 2

To reduce top wall deflections and ensure a penetration beyond critical the design for Example 2 was changed to a PZ-27 sheet pile and with a total wall height of 42 feet. The final model was made with file *Example 2b - Sheet Pile in Clay PZ27.py7d* which reduced the top wall deflection to about 2.5 inches, as shown in Figure 2.6.

2.1.4 Short-Term and Long-Term Analyses for Example 2

It has been well-recognized that the design of retaining structures should be based on the service life of the structure. If the retaining structure is a permanent structure and is envisioned for long-term service, the design in cohesive soils should be based on the soil parameters with drained conditions. The soft clay in this example under the drained condition may be modeled with an internal friction angle of 27º (30º below excavation) and zero cohesive strength. The model was made with file *Example 2c - Short and Long Term Analyses.py7d* with some results shown in Figure 2.7.

The top wall deflection and maximum bending moment increase significantly as shown in Figure 2.7 based on such long-term soil properties. This example is thus helpful to show that the program PYWALL exhibits the flexibility and ease-of-use for the study of both short-term and long-term conditions with a constructed model.

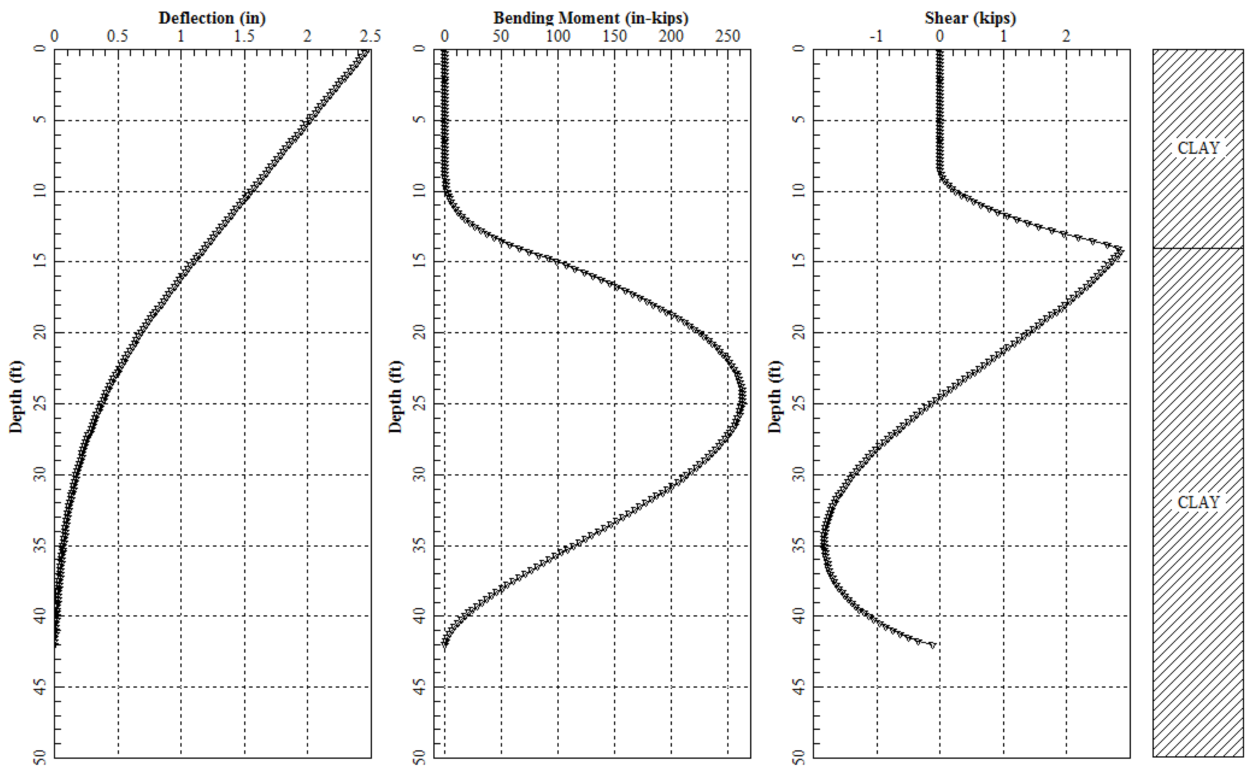


Figure 2.6 PZ27 Sheet Pile for Final Design on Example 2b

The top wall deflection and maximum bending moment increase significantly as shown in Figure 2.7 based on such long-term soil properties. The user should check that the maximum bending moment of 896 in-Kips for the long-term conditions are acceptably below yielding (for Gr.36 Steel yielding would start at around 1,630 in-Kips in PZ-27 sheet piles).

This example is helpful to show that the program PYWALL exhibits the flexibility and ease-of-use for the study of both short-term and long-term conditions with a constructed model.

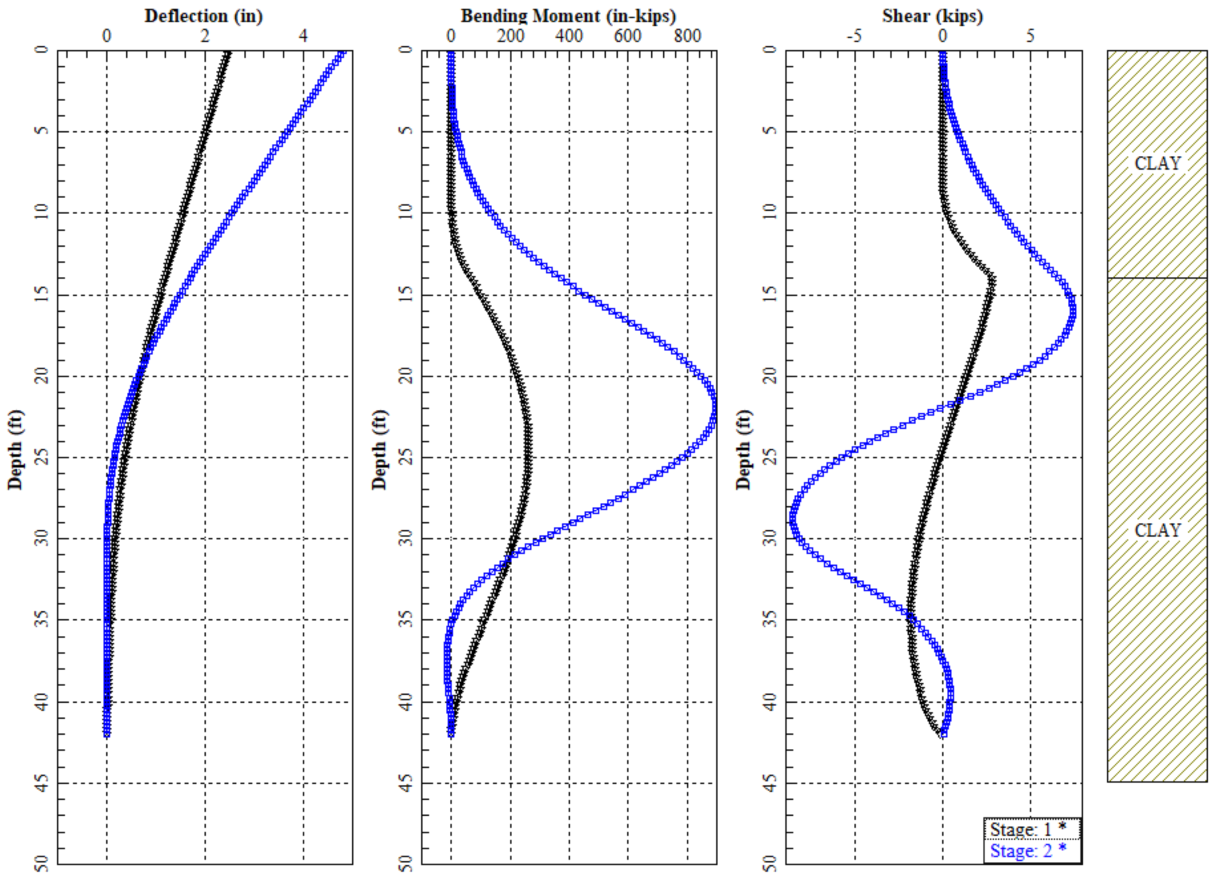


Figure 2.7 Comparisons of Short-Term and Long-Term Conditions for Example 2c

2.1.5 Input and Output Data Files for Example 2

Users can read Section 2.1.1 (7) of the User's Manual for reference on the location of placement of the input and output data files for the example files installed with this program. The default installation directory is the following: *(Root Drive) c:\Ensoft\PYwall2025-Examples*. The input data files for all examples presented in this manual are installed automatically with the program.

The input-data filenames for Example 2 are the following:

Example 2a - Sheet Pile in Clay.py8d

Example 2b - Sheet Pile in Clay PZ27.py8d

Example 2c - Short and Long Term Analyses.py8d

The output-data filenames for Example 2 are the following:

Example 2a - Sheet Pile in Clay.py8o

Example 2b - Sheet Pile in Clay PZ27.py8o

Example 2c - Short and Long Term Analyses.py8o

Example 3. Soldier-Pile Wall with Multiple Tiebacks

3.1 Example 3 – Soldier-Pile Wall with Multiple Tiebacks

3.1.1 Problem Description

This example is introduced for validating the adequacy of PYWALL for a retaining wall on a tall excavation formed by soldier piles spaced at 8-ft on center with multiple levels of tiebacks in one layer of soil. This example is provided to compare active pressures and tieback forces obtained from traditional limit-equilibrium analyses against those from a PYWALL model. Furthermore, this example is also useful to illustrate design limitations from the traditional analytical methods and to show the expediency of PYWALL for the evaluation of various modeling changes to reach an ultimate design that is both safe and efficient. Finally, the example allows PYWALL to introduce methods for optimization of the engineering solution for this particular excavation profile.

The main features of the modeled system are included in Figure 3.1. The soldier pile is an HP8x36 steel beam placed in a 2-feet diameter hole filled with 4 sack concrete. The top of the HP8x36 soldier piles is 5 ft below the original ground surface with a transition abutment slope (V:5-ft to H:11-ft) near the wall. In addition to the loads due to earth pressure, an additional surcharge load (300 psf) is also considered. The spacing between HP8x36 soldier piles is 8 ft, which is filled with timber lagging.

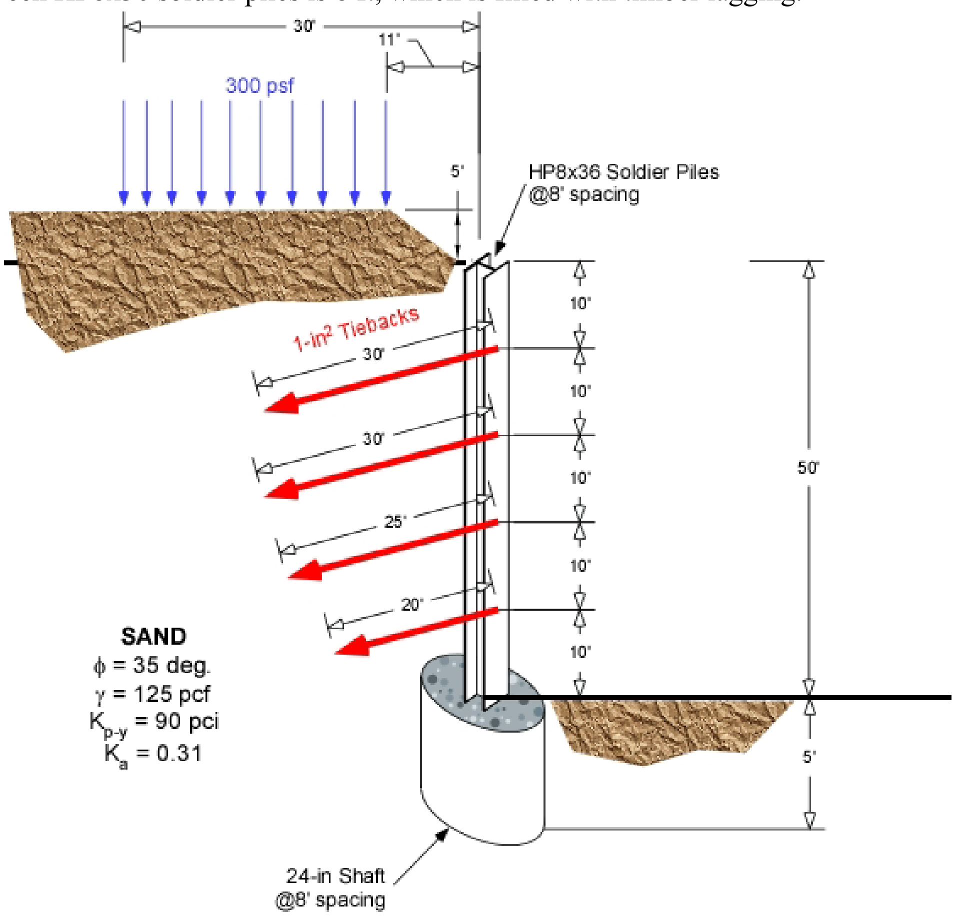


Figure 3.1 Representative Sketch for Example 3

3.1.2 Hand Computations of Active Pressure Based on Limit Equilibrium

The lateral earth pressure distribution for the analysis of braced or anchored walls constructed in cohesionless soils may be determined using Figure 3.2.

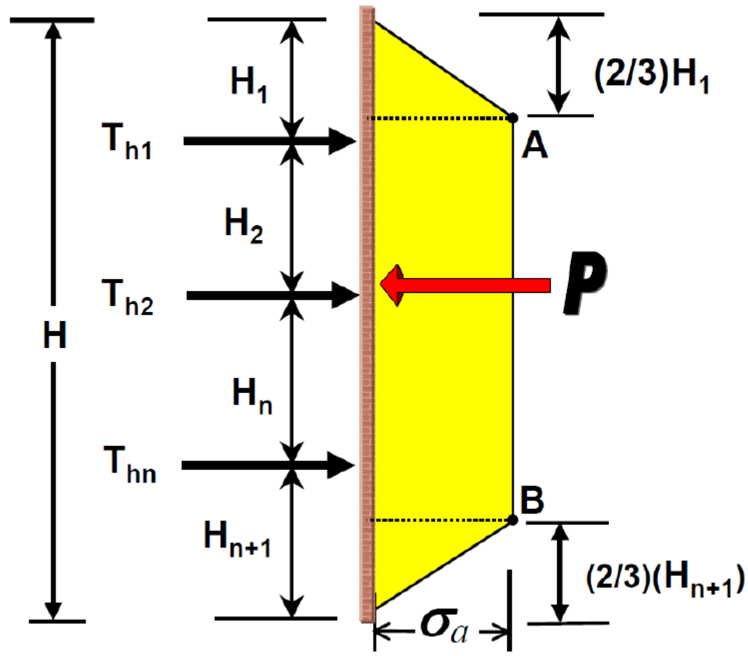


Figure 3.2 Diagram of Trapezoidal Earth Pressure Distribution in Sand

3.1.2.1 Active Earth Pressure from Soils

The coefficient of active earth pressure is $K_a = 0.310$. The maximum ordinate (σ_a) of the pressure diagram is determined as follows. A full height (5 ft) of surcharge was assumed starting from the face of the wall. The transition soil slope (V:5-ft to H:11-ft) near the wall is insignificant and was ignored in this example.

The lateral load distribution at the excavation line (σ_{a1}) is calculated as follows:

$$\sigma_{a1} = \gamma * (H{=}50\ ft + 5\ ft) * Ka = (125)(55)(0.310) = 2{,}131\ psf$$

For a spacing of 8 ft:

$$\sigma_{a1} = (2131)\ (8) = 17{,}048\ \text{lb/ft} = 1{,}421\ \text{lb/in} \quad (3.1)$$

The total lateral load on the wall (50 ft only) above the excavation line:

$$P = (0.5)\ (\gamma) * (55\ \text{ft} + 5\ \text{ft})\ (50\ \text{ft}) * Ka = (0.5)(125)(60)(50)(0.310) = 58{,}125\ \text{lb per ft width}$$

The active stress at Points A and B of the trapezoidal earth pressure (apparent earth pressure) diagram is:

$$\sigma_{A,B} = \frac{(1.3)(P)}{H - \left(\frac{H_1}{3}\right) - \left(\frac{H_{n+1}}{3}\right)} = \frac{(1.3)(58125)}{(50 - \frac{10}{3} - \frac{10}{3})} = 1{,}743.8\ lb/ft^2$$

For the wall with 8-ft spacing, pressure at Points A and B is:

$$\sigma_{A,B} = 1{,}743.8\ (8) = 13{,}950\frac{lb}{ft} = 1{,}162.5\ lb/inch \tag{3.2}$$

3.1.2.2 Active Earth Pressure from Additional Surcharge (Boussinesq)

The load at top of the wall is calculated based on the diagram in Figure 3.3.

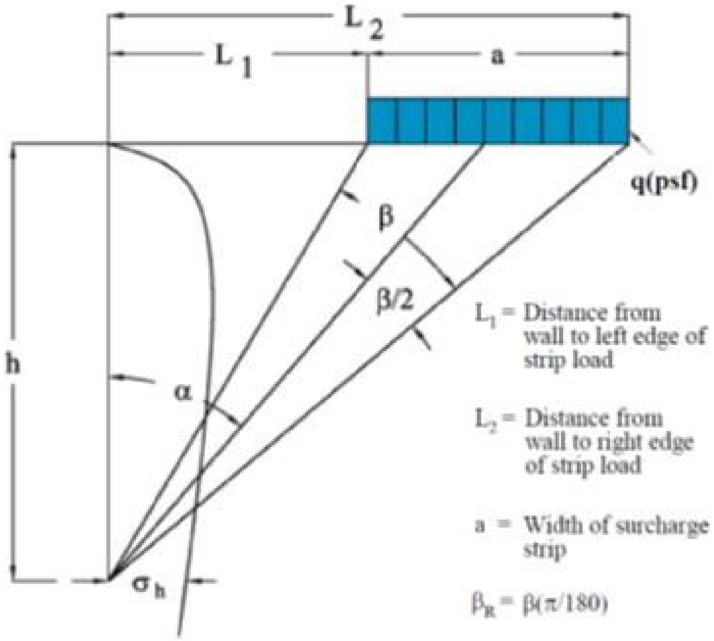


Figure 3.3 Boussinesq's Diagram of Additional Surcharges at Top of Wall

$\beta = tan^{-1}(L_2/h) - tan^{-1}(L_1/h) = tan^{-1}(30/5) - tan^{-1}(11/5) = 0.261$

$\alpha = tan^{-1}(L_1/h) + \beta/2 = tan^{-1}(11/5) + 0.261/2 = 1.275$

$Q_S = 0.3\ ksf$

$\sigma_h = 2Q_S * (\beta_R - sin\beta cos2\alpha)/(\pi)$

$\sigma_h = 2*300*[0.261 - \sin(0.261)\cos(2*1.275)]/(\pi) = 91\ psf$

Hand computations of the lateral pressures at depth of 0 ft (top of the wall), 5 ft, 10 ft, and 50 ft (dredge level) are presented in Table 3.1.

Depth (ft)	Location	Load (psf)	Pressure (lb/ft) for 8-ft spacing
0	Top of wall	91	728
5	5 ft below the top	117	936
10	10 ft below the top	105	840
20	20 ft below the top	65	520
50	dredge level	14	112

Table 3.1 Hand Computations of Boussinesq's Loads & Pressures for Example 3

3.1.2.3 Approximate Hand Computation of Tieback Forces

Tieback forces can be computed by hand using the simplified approach from the "AASHTO Hinge Model" which has the assumption that there is zero moment at all tieback locations except the top one. This was incorporated as a new feature in PYWALL v2019 where the user can place a checkmark selecting the AASHTO Hinge Model under Data > Tieback (Ground Anchor) Data (see Section 3.2.13 of this User's Manual). However, the user should also keep in mind that PYWALL can compute tieback forces without this simplified assumption (which is necessary for hand computations).

The main load components acting on the wall due to active forces (in purple arrows) from the soil and those from the additional surcharges (Boussinesq, in green arrows) can be numbered as P1 through P14, as represented in Figure 3.4. The resulting tieback forces (in horizontal direction) are represented in red arrows as T1 through T4 in Figure 3.4.

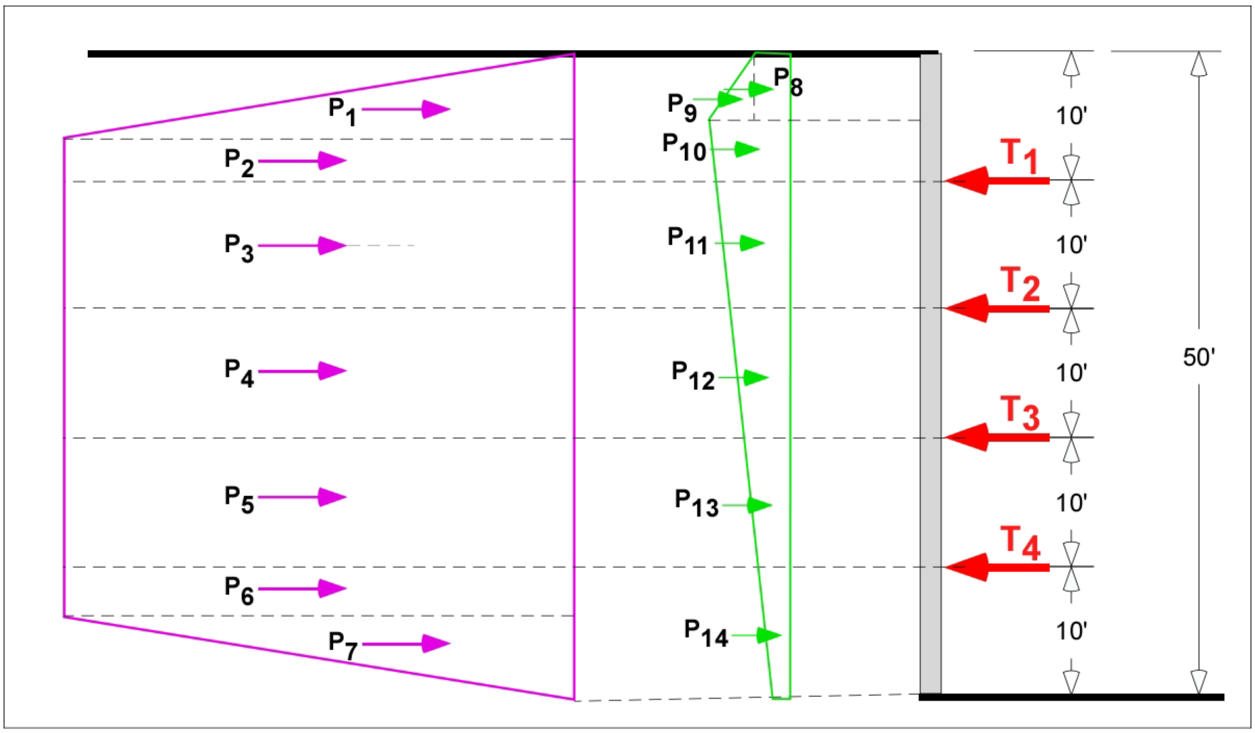


Figure 3.4 Simplified Diagram for All Tieback Forces

A free-body diagram for the upper tieback force T1 (and part of T2) is included in Figure 3.5. The various lengths represented in gray in Figure 3.5 are computed as follows:

L1 = 1/3 (6.67') + 3.33' = 5.55'

L2 = 1/2 (3.33') = 1.67'

L3 = 1/2 (5') = 2.5'

L8 = 1/2 (5') + 5' = 7.5'

L9 = 1/3 (5') + 5' = 6.67'

L10 = 1/2 (5') = 2.5' (approx.)

L11 = 1/2 (5') = 2.5' (approx.)

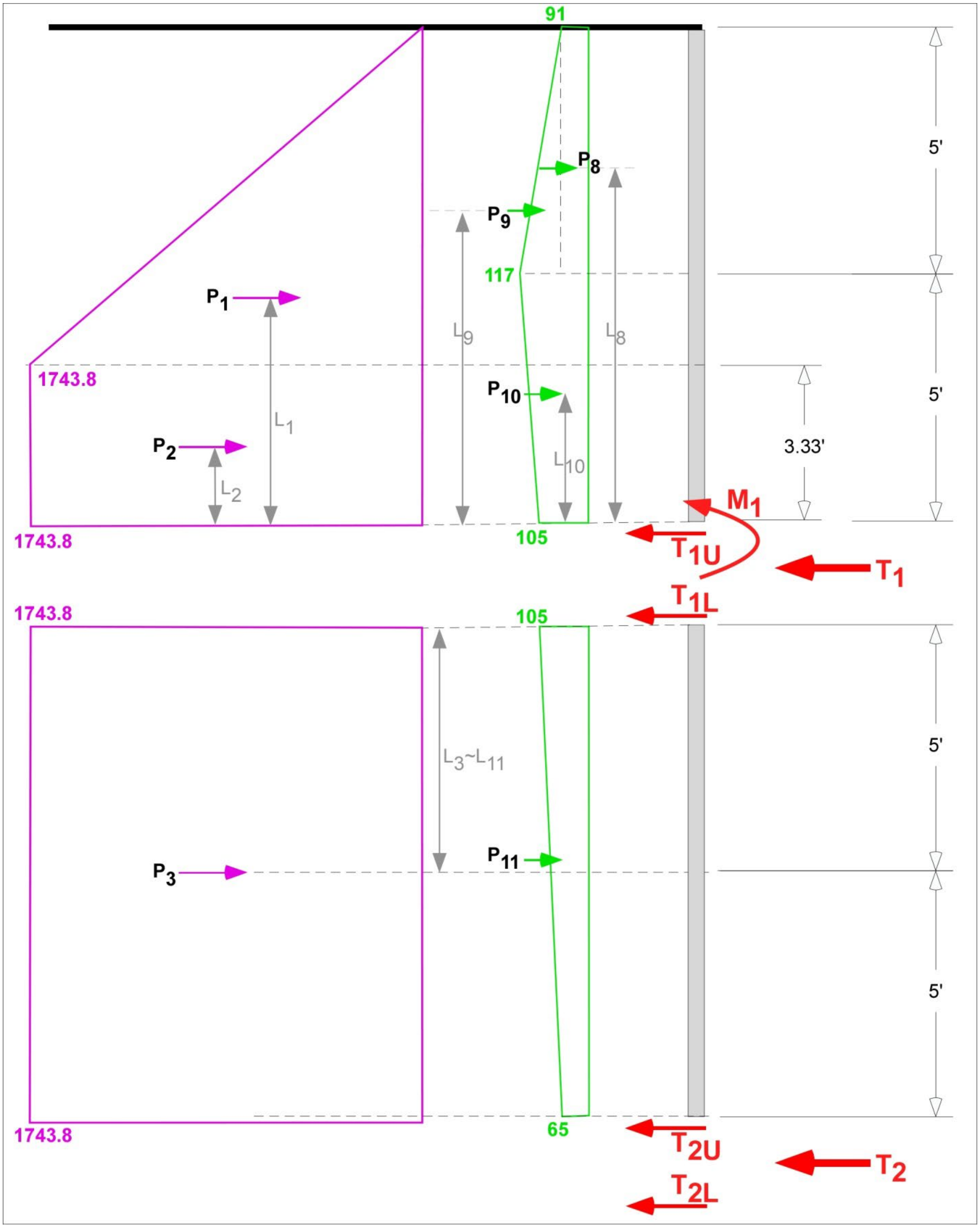


Figure 3.5 Free-Body Diagram for T1 and T2

The various forces with purple arrows acting on the 8-ft length of wall in Figure 3.5 are computed as follows:

P1 = 1/2 (6.67') (1743.8 psf) (8') = 46,525 lbs

P2 = (3.33') (1743.8 psf) (8') = 46,455 lbs

P3 = (10') (1743.8 psf) (8') = 139,504 lbs

P4 = (10') (1743.8 psf) (8') = 139,504 lbs

P8 = (5') (91 psf) (8') = 3,640 lbs

P9 = 1/2 (5') (117 psf - 91 psf) (8') = 520 lbs

P10 = (5') 1/2 (117 psf + 105 psf) (8') = 4,440 lbs

P11 = (10') 1/2 (105 psf + 65 psf) (8') = 6,800 lbs

P12 = (10') 1/2 (65 psf + 37 psf) (8') = 4,080 lbs

The moment M1 acting on the 8-ft length of wall in Figure 3.5 can be estimated as follows:

M1 = P1 x L1 + P2 x L2 + P8 x L8 + P9 x L9 + P10 x L10

M1 = 46,525 x 5.55 + 46,455 x 1.67 + 3,640 x 7.5 + 520 x 6.67 + 4,440 x 2.5

M1 = 377,662 lbs-ft **(3.3)**

The tieback forces T1 and T2 acting on the 8-ft length of wall in Figure 3.5 can be estimated as follows:

T1 = T1U + T1L

T2 = T2U + T2L

Where:

T1U = P1 + P2 + P8 + P9 + P10

T1L = P3/2 + M1/10' + P11/2

T2U = P3/2 - M1/10' + P11/2

T2L = P4/2 + P12/2

After calculations:

T1U = 101,580 lbs

T1L = 110,918 lbs

Results in:

T1 =212,498 lbs = 212.5 kips **(3.4)**

T2U = 35,386 lbs

T2L = 71,792 lbs

Results in:

T2 = 107,178 lbs = 107.2 kips **(3.5)**

3.1.3 Active Pressures from PYWALL and Comparisons

The initial PYWALL model for this Example is available with the input-data file labeled *Example 3a - Soldier Pile in Sand with Tiebacks.py8d* that is installed with the program. The user can open this file and will be able to observe all input/output data for the model.

The active pressure diagrams generated by PYWALL are presented in Figure 3.6. The total soil active pressure at the dredge level = 15,500 lb/ft (green line) + 1,550 lb/ft (red line) = 17,050 lb/ft, closely follows the hand calculation of 17,048 lb/ft presented in Eq. (3.1).

The active stress at Points A and B of the trapezoidal earth pressure (apparent earth pressure) diagram in Figure 3.2 is 13,948 lb/ft in PYWALL versus 13,950 lb/ft from Eq. (3.2) based on hand computations.

Comparisons of Boussinesq pressures on the retaining wall as obtained from PYWALL (purple line in Figure 3.6) and those from hand computations (Table 3.1) are presented in Table 3.2.

Depth (ft)	Location	Pressure (lb/ft) for 8-ft spacing	
		From PYWALL (purple line)	From Hand Computations
0	Top of wall	700	728
5	5 ft below the top	936	936
10	10 ft below the top	846	840
20	20 ft below the top	519	520
50	dredge level	113	112

Table 3.2 Comparisons of Boussinesq Pressures for Example 3a

Notice that the checks above are for the service limit state. Different load factors, such as 1.35 for soil earth pressure and 1.75 for surcharge load may be applied for LRFD design checks (if desired, those factors can be easily introduced by the user editing the PYWALL model for Example 3a).

3.1.4 Moment and Tieback Forces from PYWALL and Comparisons

To compare with the simplified hand computations the PYWALL model had a checkmark in AASHTO Hinge Model (zero bending moment at all tieback locations except the top one) under Data > Tieback Data.

The bending moment at the location of the first tie back (at 10 ft) in the PYWALL model of Example 3a was computed as 4,534 in-Kips (377,833 ft-lbs). This value of bending moment closely follows the hand calculation of 377,662 lbs-ft presented in Eq. (3.3).

A diagram of horizontal tieback forces generated by the PYWALL model of Example 3a is presented in Figure 3.7. The top tieback has a force T1 = 220.4 kips which closely follows the hand calculation of 212.5 kips presented in Eq. (3.4). The second tieback has a force T2 = 110.8 kips which also closely follows the hand calculation of 107.2 kips presented in Eq. (3.5). Additional comparisons with equal success can be made with the third and fourth tiebacks T3 and T4 following similar simplified hand computations.

3.1.5 Lateral Movements, Bending Moment and Shear from PYWALL

Selecting Graphics > Deflection, Moment and Shear plus Plot > Show Soil Layer allows the user to display a graphics of all three variables with respect to depth, as shown in Figure 3.8. The traditional methods of analyses for retaining walls have difficulties in estimating the lateral movements of the model, whereas PYWALL includes the estimates while using the traditional loads (or "active forces," as shown with good comparisons in previous sections), actual flexural stiffness of the modeled wall element and the well-known concepts of nonlinear *p-y* response curves for the soil/rock strata.

In this particular example, as indicated in Figure 3.8, it is noticeable the large movements expected for this wall, particularly in the top section prior to the first tieback (with a maximum of 11.3 inches). The bending moment is also noticeable but likely still within the capacity of the selected HP8x36 steel shape.

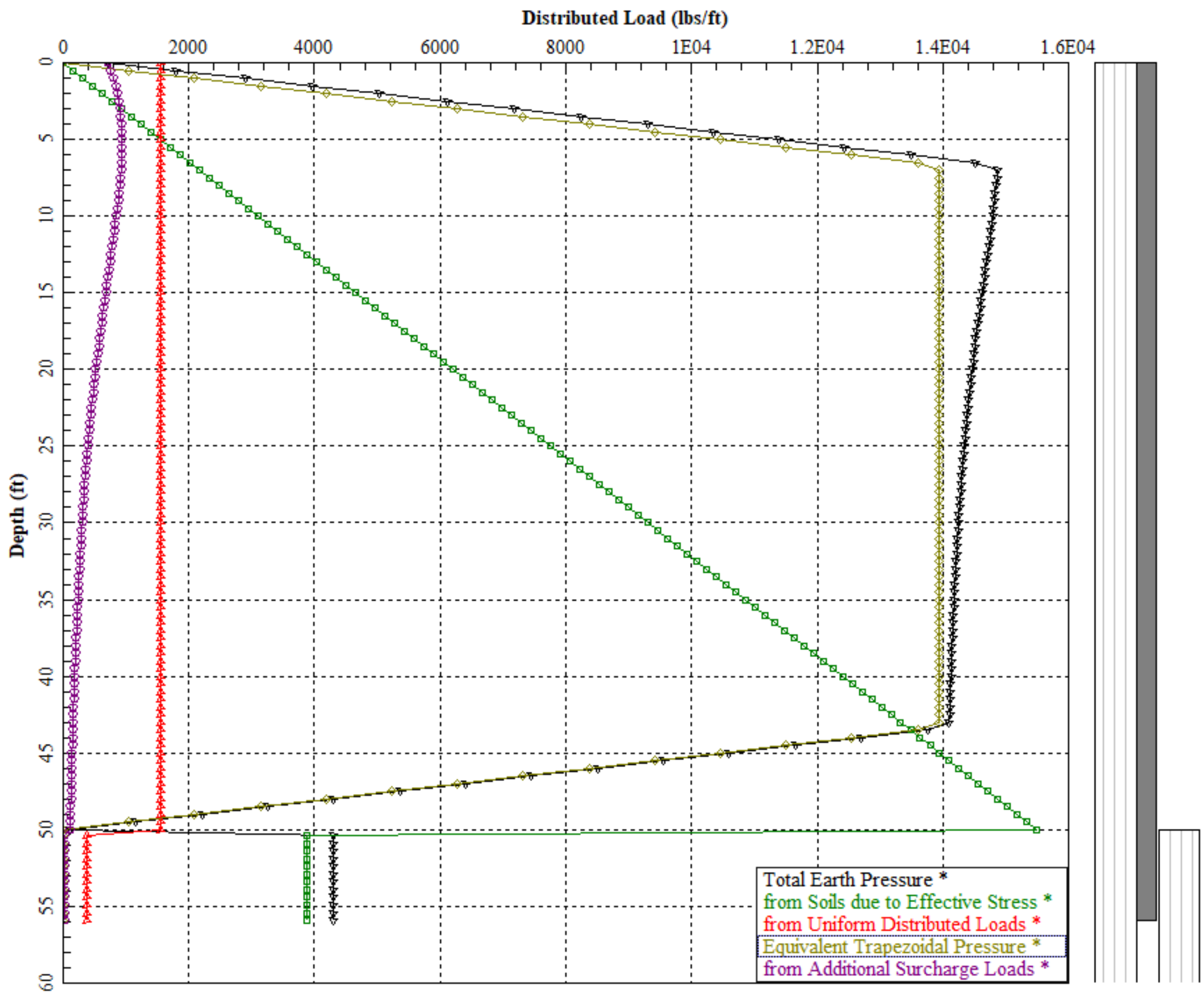


Figure 3.6 Active Pressures from PYWALL for Example 3a

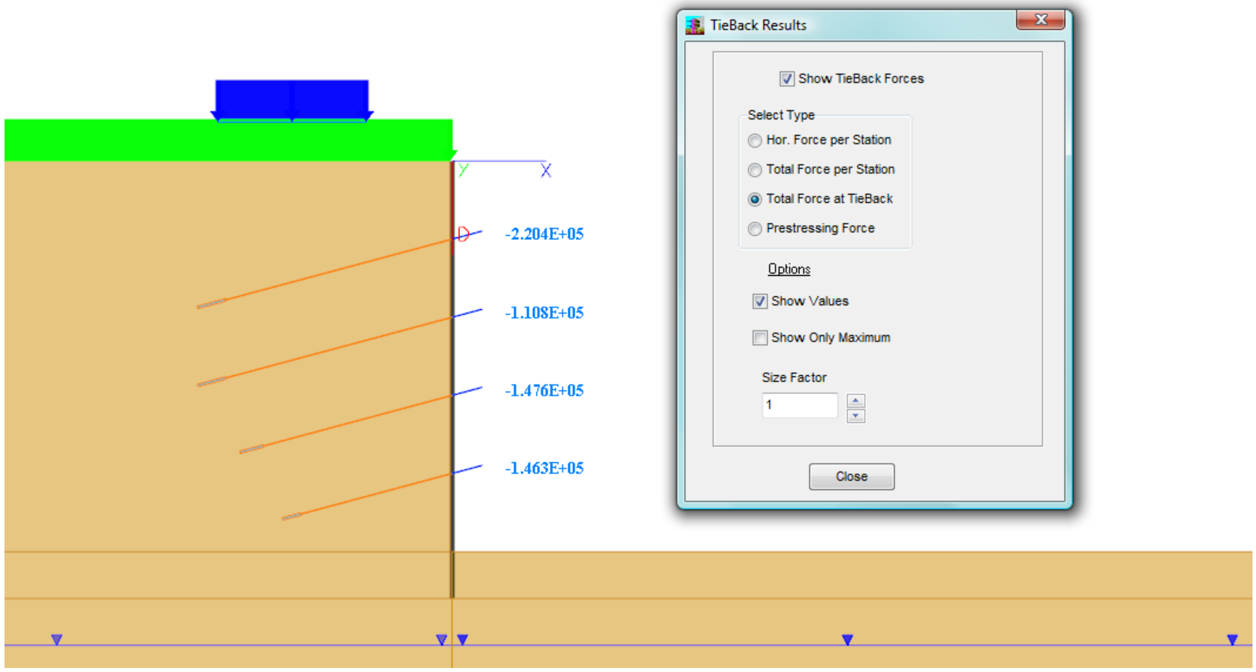


Figure 3.7 Tieback Forces from PYWALL for Example 3a

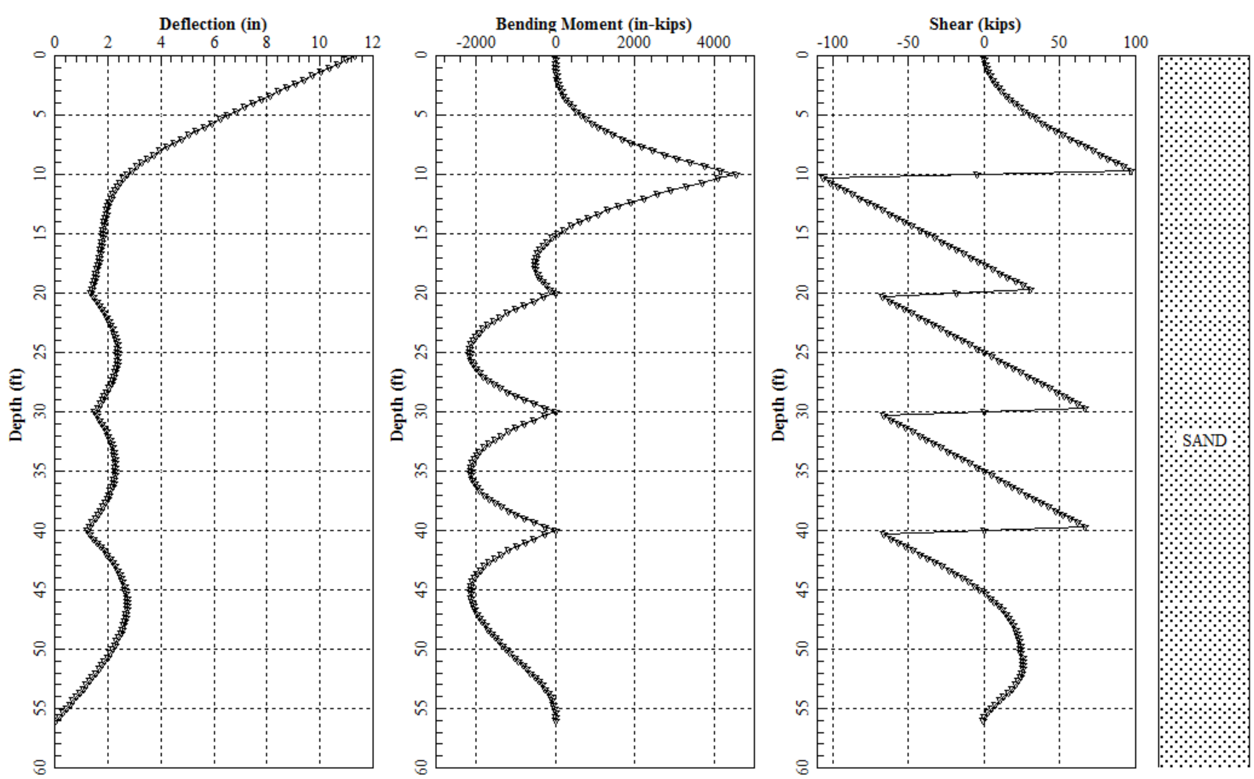


Figure 3.8 Deflection-Moment-Shear from PYWALL for Example 3a

3.1.6 Model Optimization in PYWALL

Detailed observations of the graphical results in Figure 3.8 for the initial model can bring up ideas for several aspects that can be improved. In this example we will evaluate fast and practical optimization methods that are easily incorporated in PYWALL starting from the initial model.

3.1.6.1 Prestressing in Tiebacks

The displacement at each tieback location can be reduced with the introduction of prestressing forces on each tieback (in Prestressing Force column under Data > Tieback (Ground Anchor) Data) which can be initially estimated as the lower values from the resulting tieback forces (Figure 3.7). This is done in the input data file labeled *Example 3b - Prestressed Tiebacks.py8d* that is installed with the program. For simplification, the same prestressing force of 110 kips was added in each tieback (T1 to T4).

As indicated in the new output graphics in Figure 3.9, the small displacements in each tieback location (at 10, 20, 30 and 40 ft) were reduced to near zero. The initial objective is thus satisfactory, even though the maximum deflection was reduced only slightly at the top of the wall.

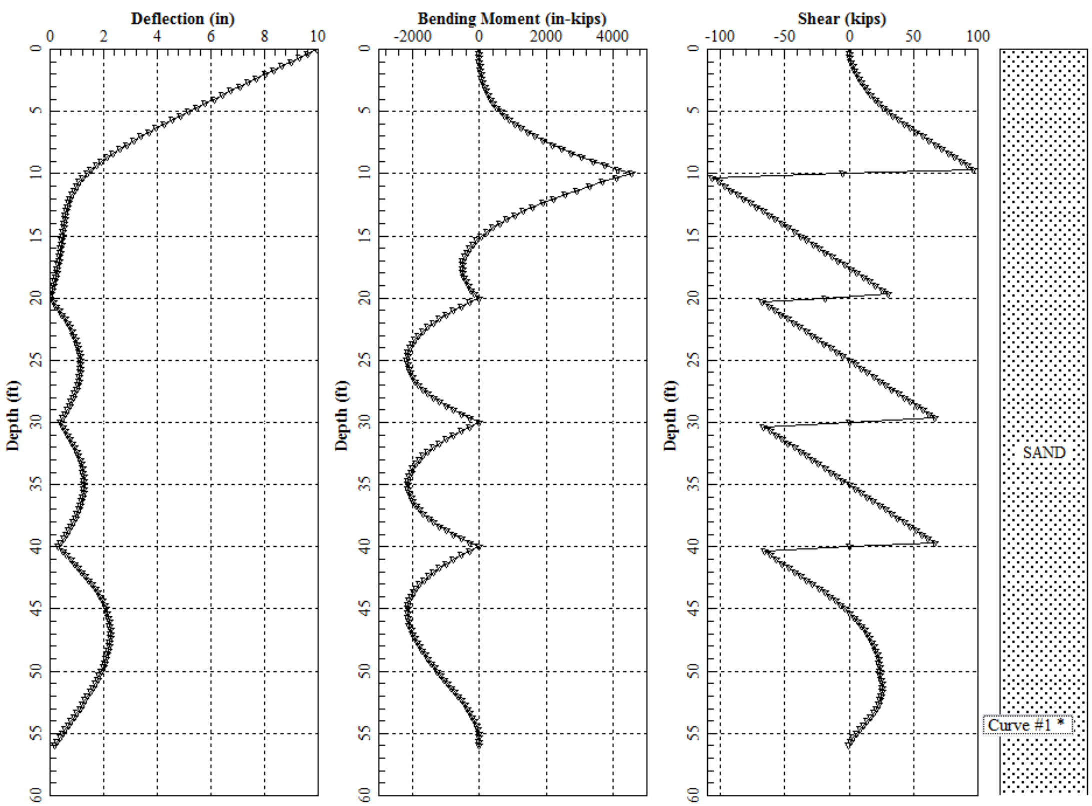


Figure 3.9 Deflection-Moment-Shear from PYWALL for Example 3b

3.1.6.2 Improvement in Tieback Locations

Observing the deflections, bending moments and shear in Figure 3.9 it is easy to see that the vertical placements of the various tiebacks can be optimized so they are more effective in controlling deflections along the entire wall. This is done in the input data file labeled *Example 3c - Optimized Tieback Locations.py8d* that is installed with the program. New locations of tiebacks were introduced in the Depth from top of wall column under Data > Tieback (Ground Anchor) Data with new tieback placements at 7, 19, 30 and 41 ft.

As indicated in the new output graphics in Figure 3.10 the overall displacements were reduced considerably and the maximum displacements between tiebacks were also made more uniform. The designer, however, must notice that such an irregular placement of tiebacks may be difficult to control in the field so the contractor must be warned carefully regarding the importance of these tieback depths!

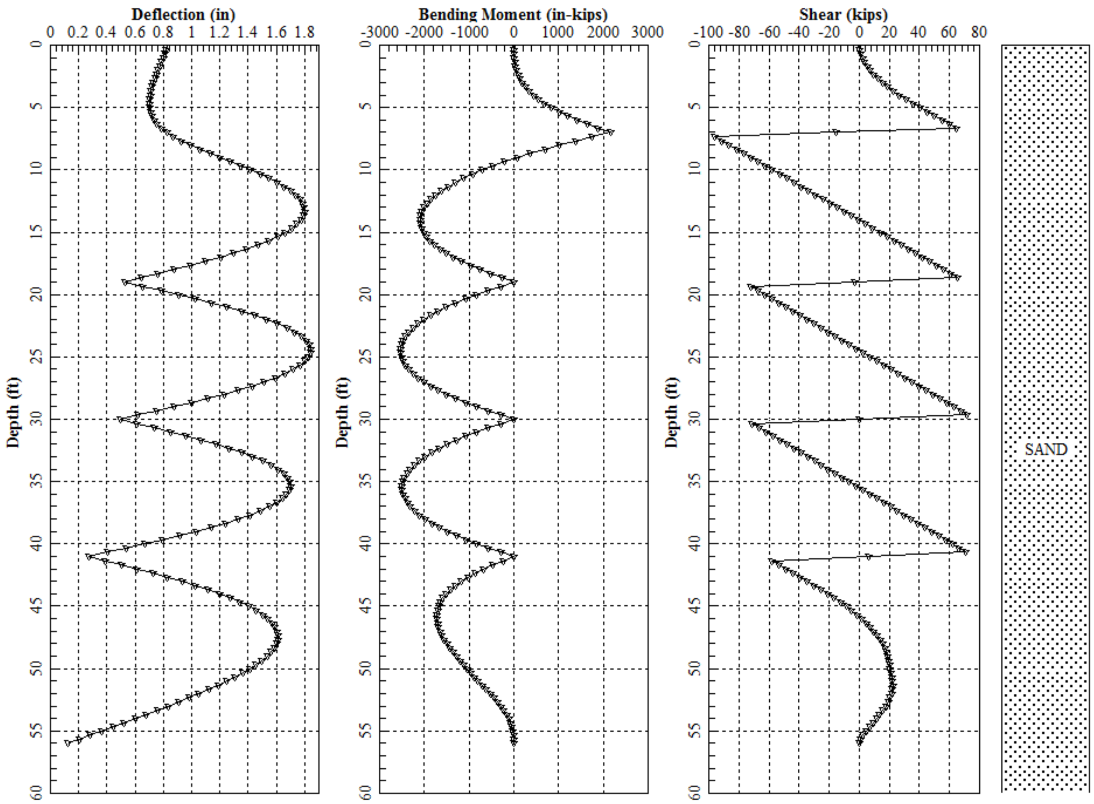


Figure 3.10 Deflection-Moment-Shear from PYWALL for Example 3c

3.1.6.3 Final Optimization

A final optimization can be made with a small increase in the prestressing forces on the tiebacks. In fact, if the contractor is agreeable, it would be best to use different prestressing forces in the various tieback locations.

For demonstration purposes, new prestressing forces of 140, 130, 120 and 110 kips (from top to bottom) are implemented in the input data file labeled *Example 3d - Final Elastic Model.py8d* that is installed with the program. This variation of forces from highest at top and reduced towards the lower levels is in accordance to the tieback lengths (Unbonded anchor length) in Figure 3.1.

The new output graphics in Figure 3.11 indicates another reduction of displacements from the initial and even previous models.

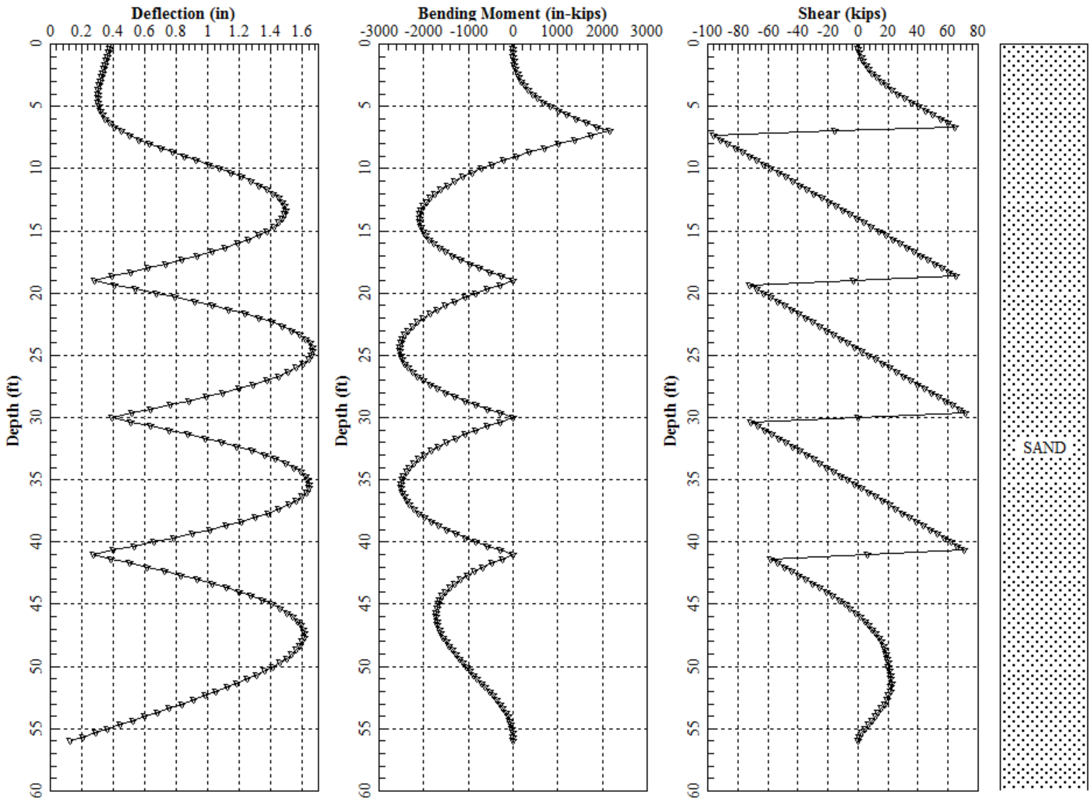


Figure 3.11 Deflection-Moment-Shear from PYWALL for Example 3d

3.1.6.4 Final Nonlinear Model

PYWALL v2019 introduced the new feature that models the flexural stiffness of the selected wall section(s) using nonlinear elements. All previous models of Example 3 were made with elastic elements for the wall sections. Such is useful for initial approximations but not representative for a final design, particularly when loads are near the capacity of the wall sections.

A quick check of the AISC Steel Manual indicates that the plastic moment capacity of the HP8x36 steel shape (with Gr. 50 steel) is 126 kip-ft = 1,512 kip-in. A comparison of this value with the maximum bending moments in Figure 3.11 indicates that the HP8x36 will not work, since there is a maximum moment demand slightly over 2,000 in-kips at the level of the first tieback. The same conclusion can be made when checking the shear capacity of this section (96.4 kips) against the maximum shear in Figure 3.11.

New nonlinear sections were entered under Data > Cross Sections for the top length of the wall (plain steel shape) and the bottom wall embedment into soil (steel shape as structural insert in concrete). The final section that seemed to work for this model was an HP10x57 Gr50 Steel, as shown in Figure 3.12. This was implemented in the input data file labeled *Example 3e - Nonlinear Model.py8d* that is installed with the program.

The final output graphics in Figure 3.13 indicates that the section may work well for this particular model. However, the final selection depends on the particular code that the designer must follow for this wall design (incorporating phi factors for the section capacity and load factors for the various active load components).

Useful output graphics for report presentations can be created using the Computation > 3D View and Show 3d View > Wall Diagrams by selecting appropriate output graphs and values. Some of those outputs are included in Figure 3.14 through Figure 3.16.

In this model, for the final design the user may choose the simplified option of the AASHTO Hinge Model (zero bending moment at all tieback locations except the top one) under Data > Tieback Data.

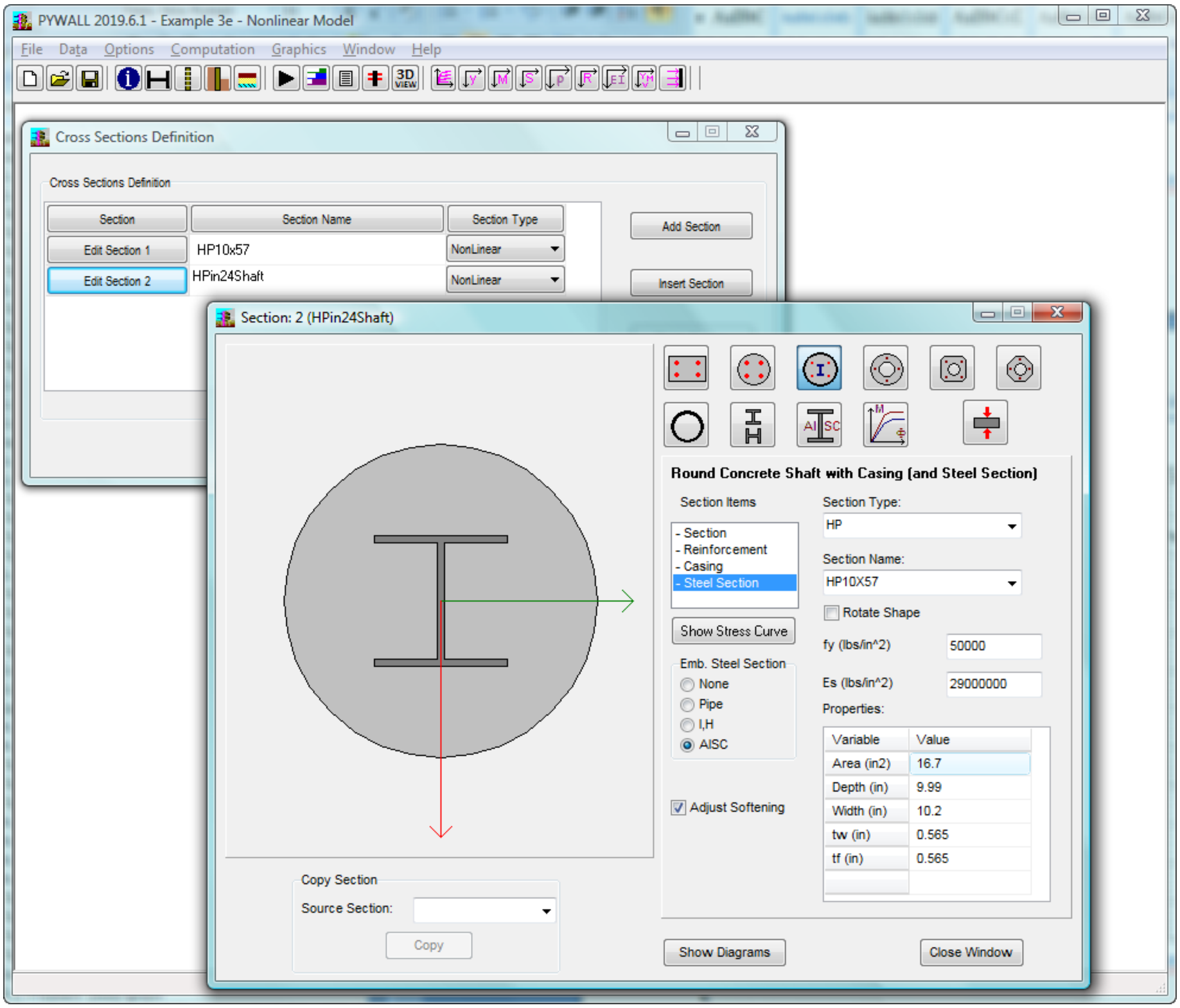


Figure 3.12 Nonlinear Sections for Example 3e

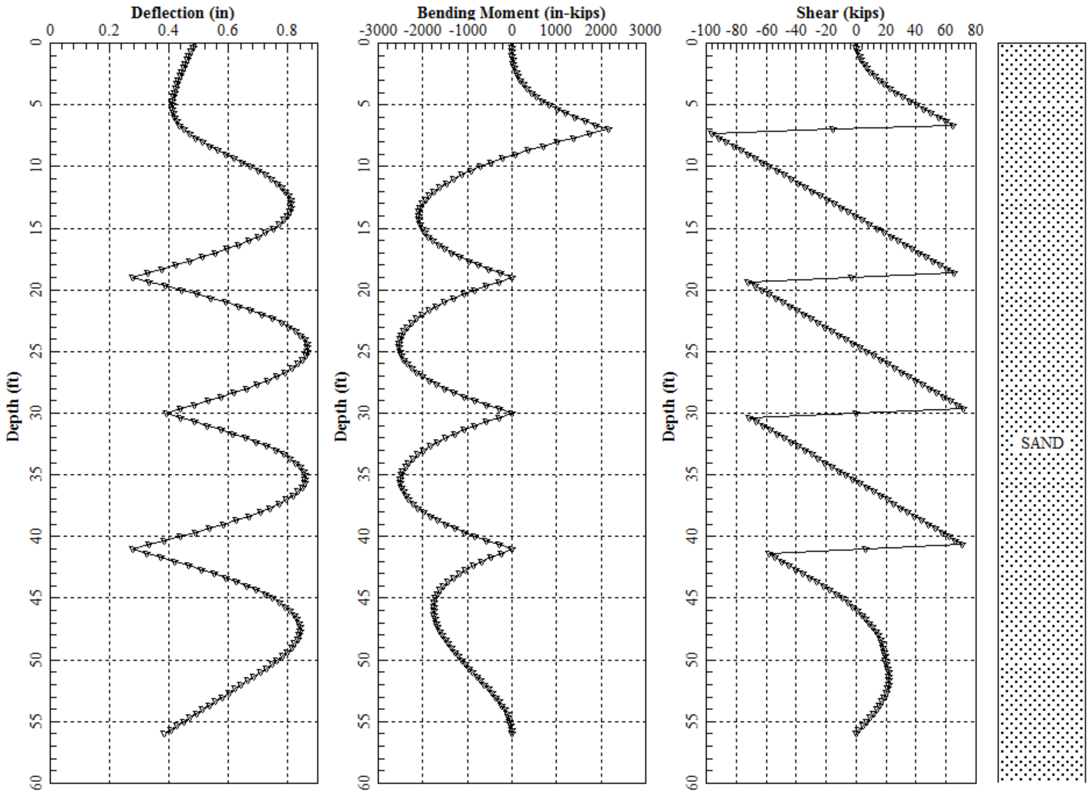


Figure 3.13 Deflection-Moment-Shear from PYWALL for Example 3e

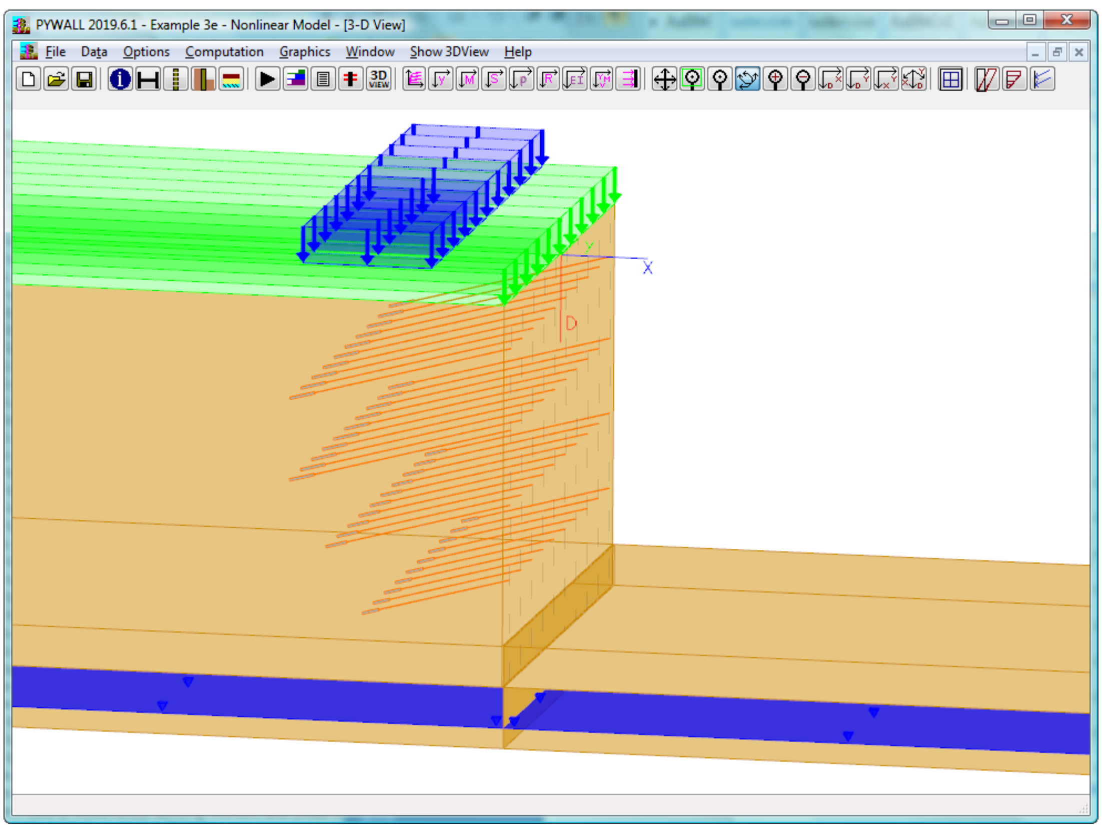


Figure 3.14 3D View of Wall Model

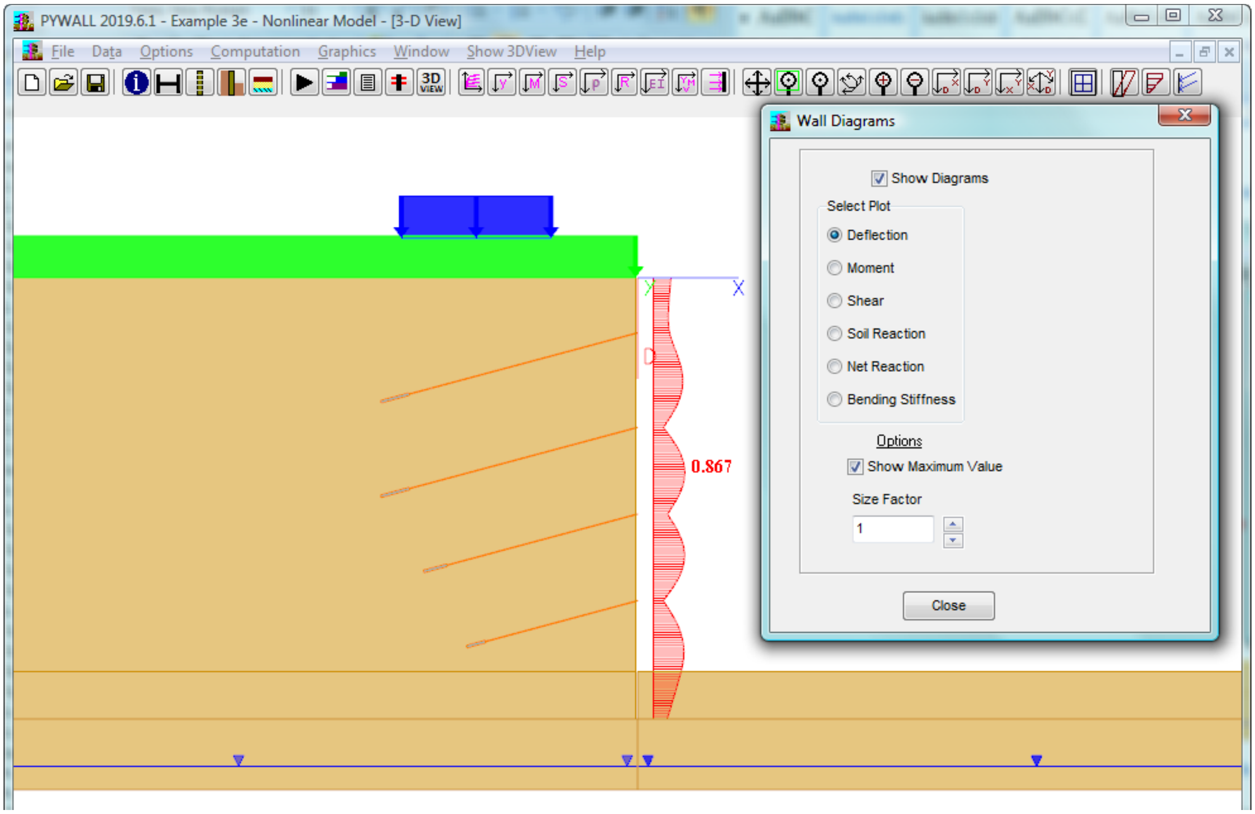


Figure 3.15 Presentation Graphics of Wall Deflections

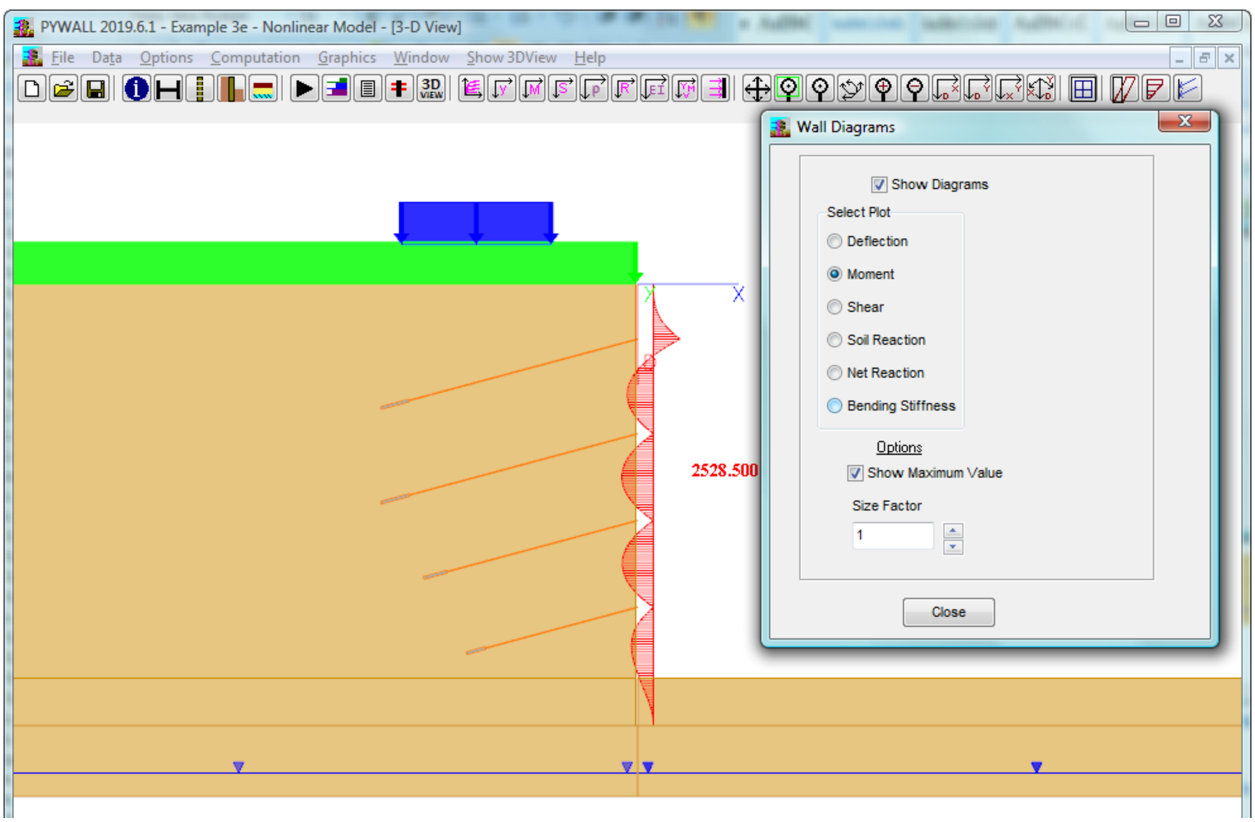


Figure 3.16 Presentation Graphics of Bending Moments on Wall

3.1.7 Input and Output Data Files for Example 3

Users can read Section 2.1.1 (7) of the User's Manual for reference on the location of placement of the input and output data files for the example files installed with this program. The default installation directory is the following: *(Root Drive) c:\Ensoft\PYwall2025-Examples*. The input data files for all examples presented in this manual are installed automatically with the program.

The input-data filenames for Example 3 are the following:

Example 3a - Soldier Pile in Sand with Tiebacks.py8d

Example 3b - Prestressed Tiebacks.py8d

Example 3c - Optimized Tieback Locations.py8d

Example 3d - Final Elastic Model.py8d

Example 3e - Nonlinear Model.py8d

The output-data filenames for Example 3 are the following:

Example 3a - Soldier Pile in Sand with Tiebacks.py8o

Example 3b - Prestressed Tiebacks.py8o

Example 3c - Optimized Tieback Locations.py8o

Example 3d - Final Elastic Model.py8o

Example 3e - Nonlinear Model.py8o

Example 4. LRFD Analysis of Sheet-Pile Wall for Static and Seismic Conditions

4.1 Example 4 – LRFD Analysis of Sheet-Pile Wall for Static and Seismic Conditions

4.1.1 Problem Description

This example is helpful to show how the PYWALL program can be used rationally for the static and seismic analysis of a cantilever sheet-pile wall. The wall to be analyzed is presented in Figure 4.1, along with the soil layer information. The results based on PYWALL can compare directly with those following the conventional limit-equilibrium method. This example has been prepared based on problems evaluated on draft reports used by the NCHRP Project 12-70, with results partly contained on NCHRP Report 611 from 2008. The sheet pile selected for the analysis has bending stiffness (EI) of 1.453 x 10^7 ft^2-lb/ft and yield stress of 50 ksi. The design consideration for the factor of safety will follow the AASHTO LRFD Method.

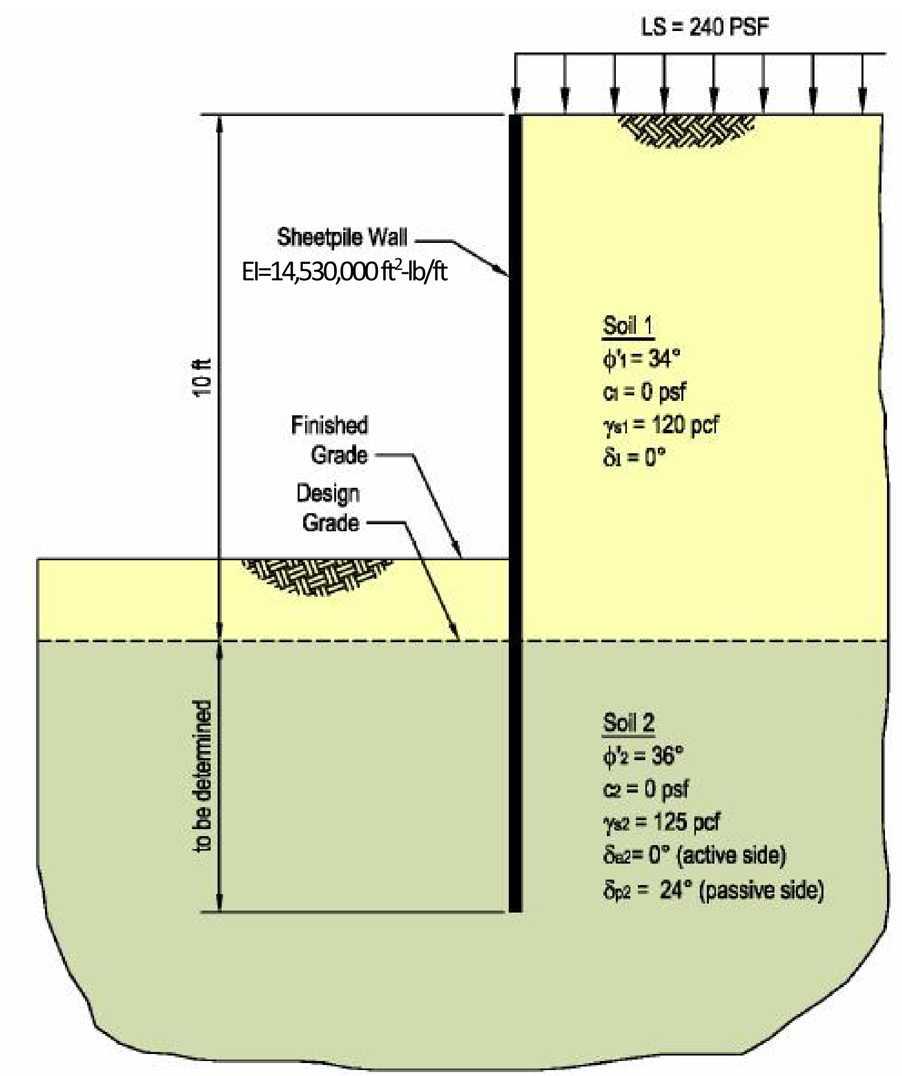


Figure 4.1 Geometry and Soil Information for a Cantilever Sheet-Pile Wall

4.1.2 Static Active Earth Pressure Using AASHTO LRFD Method

The parameters shown in Table 4.1 were used to estimate the earth pressure and the wall dimensions for the static load case labeled Strength I Load Case following the AASHTO LRFD method.

Category	Soil 1	Soil 2
γ (load factor for horizontal earth pressure)	1.5	1.5
γ_{LS} (load factor for surcharge load)	1.75	1.75
ϕ (resistance factor for passive pressure)	-	0.75
ϕ' (effective internal friction angle)	34°	36°
K_a (Active earth pressure coefficient)	0.283	0.260
K_p (Passive earth pressure coefficient)	-	8.2
γ_s (soil unit weight)	120 lbs/ft^3	125 lbs/ft^3
δ (wall friction angle)	0	24°

Table 4.1 Summary of Design Parameters for the Static Load Case (Strength I)

Wall friction angle (δ) was assumed to be zero on the active side. For the passive side, however, a wall friction angle of 24° (2/3 ϕ) was used in the limit-equilibrium analysis. The designer should evaluate these assumptions on project-specific basis. Figure 4.2 shows the factored pressure distribution for the Strength I Load Case. The active earth pressure from the backfill at the depth of excavation grade (10 ft below the top) was calculated as:

$$P_{a1} = (\gamma_s)\,(H)\,(K_a)\,(\gamma) = (120)\,(10)\,(0.283)\,(1.5) = 509 \text{ psf}$$

The active earth pressure from the surcharge at the depth of design grade (10 ft below the top) was calculated as:

$$P_{aLS} = (LS)\,(K_a)\,(\gamma_{LS}) = (240)\,(0.283)\,(1.75) = 119 \text{ psf}$$

Using the limit-equilibrium procedure and the above parameters, the minimum embedment depth (Do) was calculated to be 8.57 ft. An increase of the minimum embedment depth (Do) by 20% is the recommendation that is sometimes used in practice. Therefore, the total embedment length (D) for this example determined based on the limit-equilibrium method is 10.3 ft.

Regarding this specific topic, the user should keep in mind that the Soil-Structure Interaction (SSI) method used in PYWALL may be more rational in selecting the total embedment length because it allows the design engineers to study the stability of the system by changing the embedment length incrementally (see Section 1.1.5 for an example).

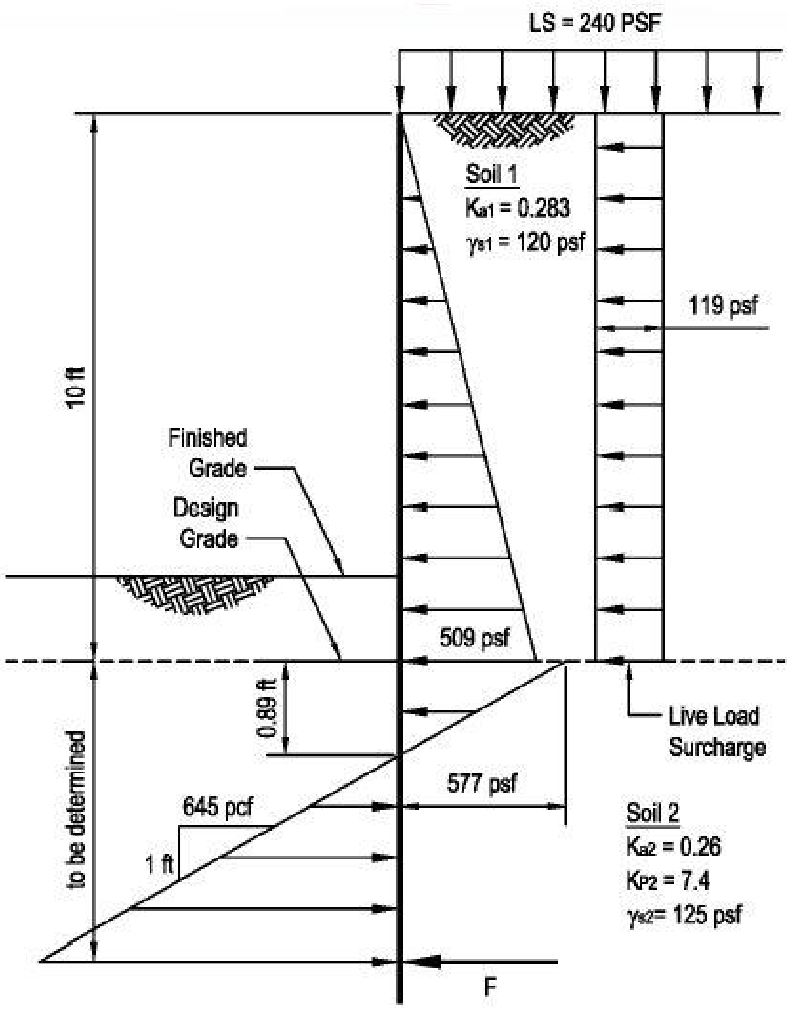


Figure 4.2 Distribution of Factored Earth Pressure for the Static Load Case

The shear and bending moment diagrams computed based on the limit-equilibrium method are presented in Figure 4.3. The maximum shear is approximately 16 kips at 18.6 ft below the top of the sheet pile and the maximum bending moment of approximately 27,272 lbs-ft is at 14.4 ft below the top of the sheet pile.

Figure 4.3 Calculated Earth Pressure, Shear, and Bending Moment Diagrams Based on the Limit-Equilibrium Method for the Static Load Case

The active pressures for the Static Load Case on the sheet-pile wall are summarized in Table 4.2.

Depth, ft	Active Earth pressure, psf	Surcharge Load, psf	Total applied pressure, psf
0	0	119	119
10	509	119	628
10	468	109	577
20.3	468	109	577

Table 4.2 Summary of Active-Earth Pressure on the Sheet-Pile Wall for the Strength I - Static Load Case

4.1.3 Static Analysis Based on the *p-y* Curve Method (PYWALL)

In the PYWALL model (*Example 4a - LRFD Analysis of Sheet-Pile Wall for Static Case.py8d*) the earth pressure above the excavation line (from 0 to 25 ft) is treated as the external load. The external load in PYWALL is assumed to remain as a constant below the excavation line at the backfill side.

The soil resistance below the excavation following the *p-y* curve method will be represented by nonlinear springs (*p-y* curves) for the force-deformation relationship. The modulus of these *p-y* curves (k_{py})

was assumed to be 60 and 90 lbs/in^3 respectively for the soils with internal friction angles of 32^o and 36^o. The soil properties for each layer shown in Table 4.3 were used in PYWALL for the static analysis.

Soil Type	Layer Thickness (ft)	Total Unit Wt (lbs/ft^3)	Internal Friction Angle (deg)	K_{p-y} (lbs/in^3)
Sand	10	120	34	60
Sand	15	125	36	90

Table 4.3 Soil Input Parameters for Example 4

In this example, a load factor of 1.5 was recommended for soil earth pressure and a load factor of 1.75 was recommended for surcharge load (Table 4.1). Since the program PYWALL applies the same load factor to both, the soil earth pressure and surcharge load, the input data for the surcharge (240 psf originally) should be adjusted by a factor 1.167 (1.75/1.5) in order to reach the same factored load on surcharge in PYWALL as those used in the limit-equilibrium method. The PYWALL example thus uses the following entries under the Data > Wall Configuration menu:

Uniform Distr. Load from soils at top of backfill = 280

Modification Factor for Active Earth Pressure = 1.5

The resulting active earth pressures automatically calculated within PYWALL for the Static Load Case are shown in Figure 4.4. The values shown in Figure 4.4 are approximately the same as those obtained from hand computations in Table 4.2.

The resulting deflection, shear and bending moment diagrams in the PYWALL model are presented in Figure 4.5. The maximum shear from PYWALL is approximately 5.7 kips at about 18 ft below the top of the sheet pile, compared with 16 kips at 18.6 ft below the top of the sheet pile from the limit-equilibrium analysis. The difference is significant and mainly caused by the assumption of full soil mobilization in the limit-equilibrium method.

The maximum bending moment based on the PYWALL model is approximately 24,300 ft-lbs (292 in-kips) at a depth of 13.8 ft below the top of the wall, compared with 27,272 lbs-ft at 14.4 ft below the top of the sheet pile from the limit-equilibrium method. The maximum bending moment from the PYWALL model is thus similar to the one from the conventional limit-equilibrium analysis.

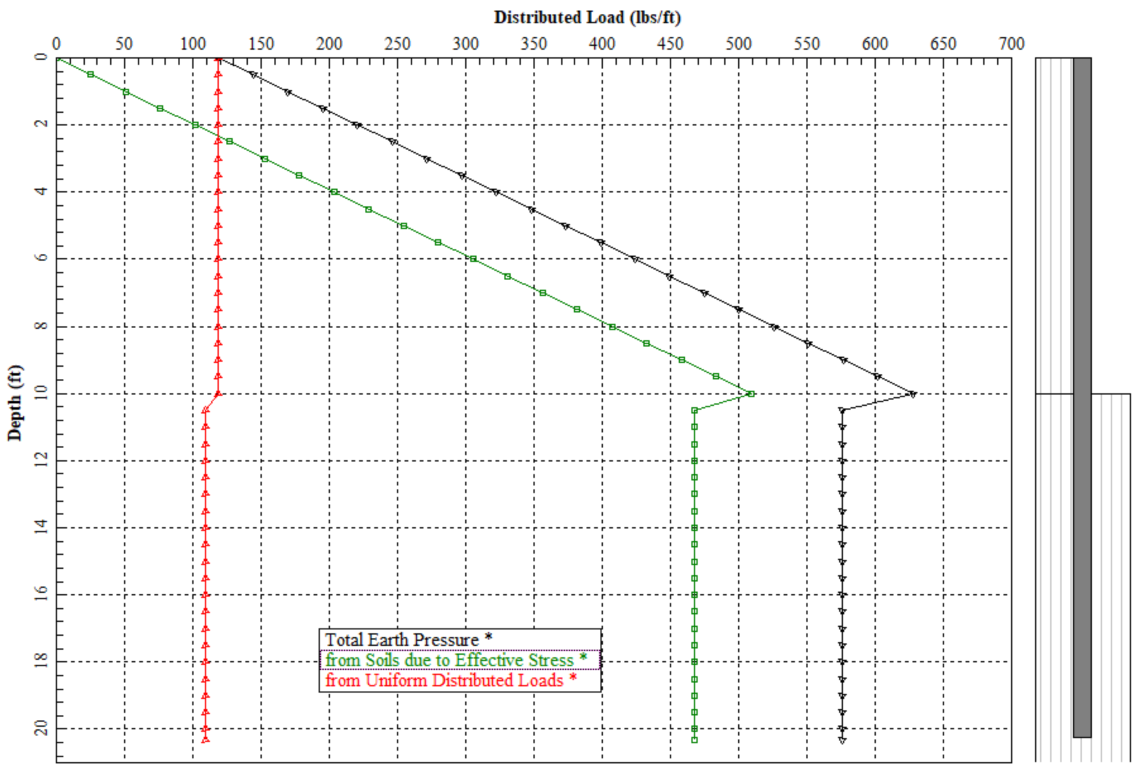


Figure 4.4 Computed Active Forces from PYWALL for the Static Load Case in Example 4a

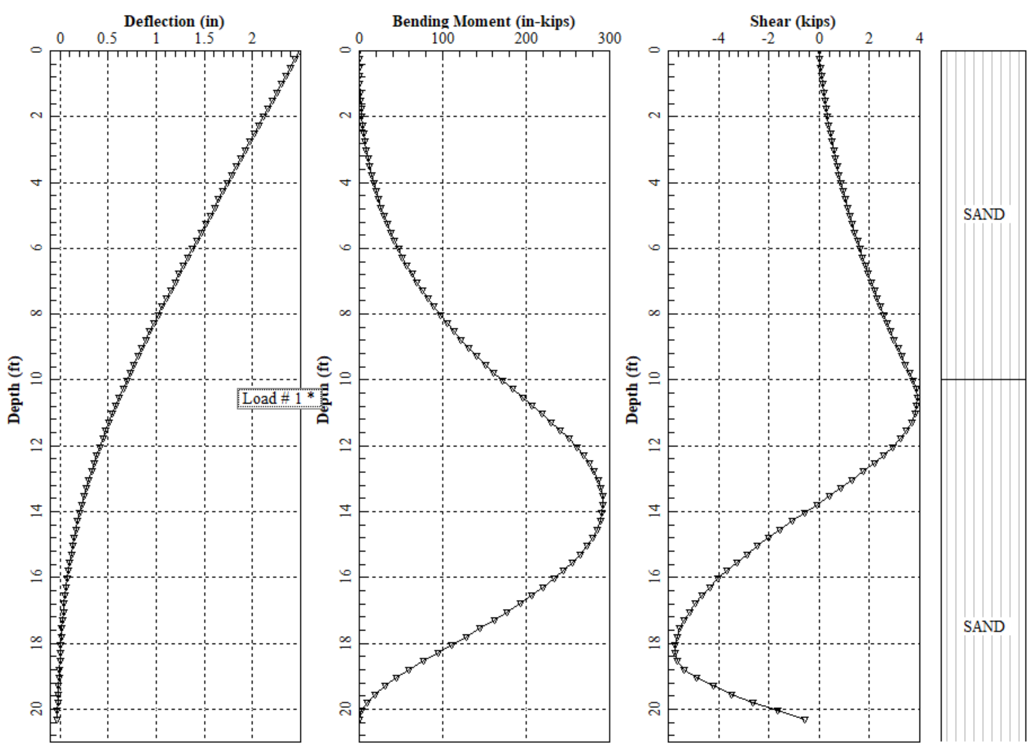


Figure 4.5 Deflection, Shear and Bending Moment Curves from the PYWALL Model for the Static Load Case in Example 4a

4.1.4 Seismic Procedures Using AASHTO LRFD Method

The peak ground acceleration coefficient, k_{max}=0.4 was selected for the seismic-load case. It was further assumed that a small amount of movement of the wall at the excavation level was permissible for the seismic design event, as long as the wall did not collapse. This assumption thus allows the use of a seismic coefficient equal to (0.5)(k_{max}). The Mononobe-Okabe (M-O) equation was used to estimate the seismic earth pressure for a non-cohesive backfill and the earth pressure distribution on the free height of the wall was assumed to be uniform. The parameters in Table 4.4 were used to estimate the earth pressure and the wall dimension for seismic loads under the seismic load case labeled Extreme Event I, following the AASHTO LRFD method.

The seismic lateral earth pressure above the excavation depth was applied as a uniform load, acting on the free height of the wall (top 10 ft), as shown in Figure 4.6. As the form of seismic active and passive pressure distribution below the excavation level is uncertain, it was simply assumed they are similar to static pressure distributions.

The same procedure used for the Strength I Load Case was used for the Extreme I Load Case (seismic condition) except that the load and resistance factors were changed to 1.0 for the seismic condition. According to AASHTO, the percentage of the live load surcharge to be used in seismic design (γ_{EQ}) should be determined on a project-specific basis. For this example, 100% of the live load surcharge was assumed to be present under the seismic condition (γ_{EQ} = 1.0).

Category	Soil 1	Soil 2
γ (load factor for horizontal earth pressure)	1.0	1.0
γ_{EQ} (load factor for surcharge load)	1.0*	1.0*
ϕ (resistance factor for passive pressure)	-	1.0
ϕ' (effective internal friction angle)	34°	36°
K_{ae} (Active earth pressure coefficient)	0.602	0.561
K_{pe} (Passive earth pressure coefficient)	-	6.15
γ_s (soil unit weight)	120 lbs/ft^3	125 lbs/ft^3
δ (wall friction angle)	0	24°
* γ_{EQ} can be determined on a project-specific basis		

Table 4.4 Summary of Design Parameters for Extreme I Load Case (Seismic)

Figure 4.6 shows the factored pressure distribution for the Extreme I Load Case. The active earth pressure from the backfill at the depth of excavation grade (10 ft below the top) based on Mononobe-Okabe (M-O) equation was calculated as:

$$P_{a1} = (0.5)\,(\gamma_s)\,(H)\,(K_{ae})\,(\gamma) = (0.5)\,(120)\,(10)\,(0.602)\,(1.0) = 361 \text{ psf}$$

The active earth pressure from the surcharge at the depth of design grade (10 ft below the top) was calculated as:

$P_{aLS} = (LS)\ (K_a)\ (\gamma_{EQ}) = (240)\ (0.283)\ (1.0) = 68$ psf

The active pressures for the Seismic Load Case on the sheet-pile wall are summarized in Table 4.5.

Using the limit-equilibrium procedure and the above parameters, the minimum embedment depth (D_o) under the seismic condition was calculated to be 9.79 ft. An increase of the minimum embedment depth (Do) by 20% is the recommendation that is sometimes used in practice. Therefore, the total embedment length (D) for the seismic design determined based on the limit-equilibrium method is 11.75 ft.

The shear and bending moment diagrams computed based on the limit equilibrium method are presented in Figure 4.7. The maximum shear is approximately 19.8 kips at 20 ft below the top of the sheet pile and the maximum bending moment of approximately 39,906 lbs-ft is at 15 ft below the top of the sheet pile.

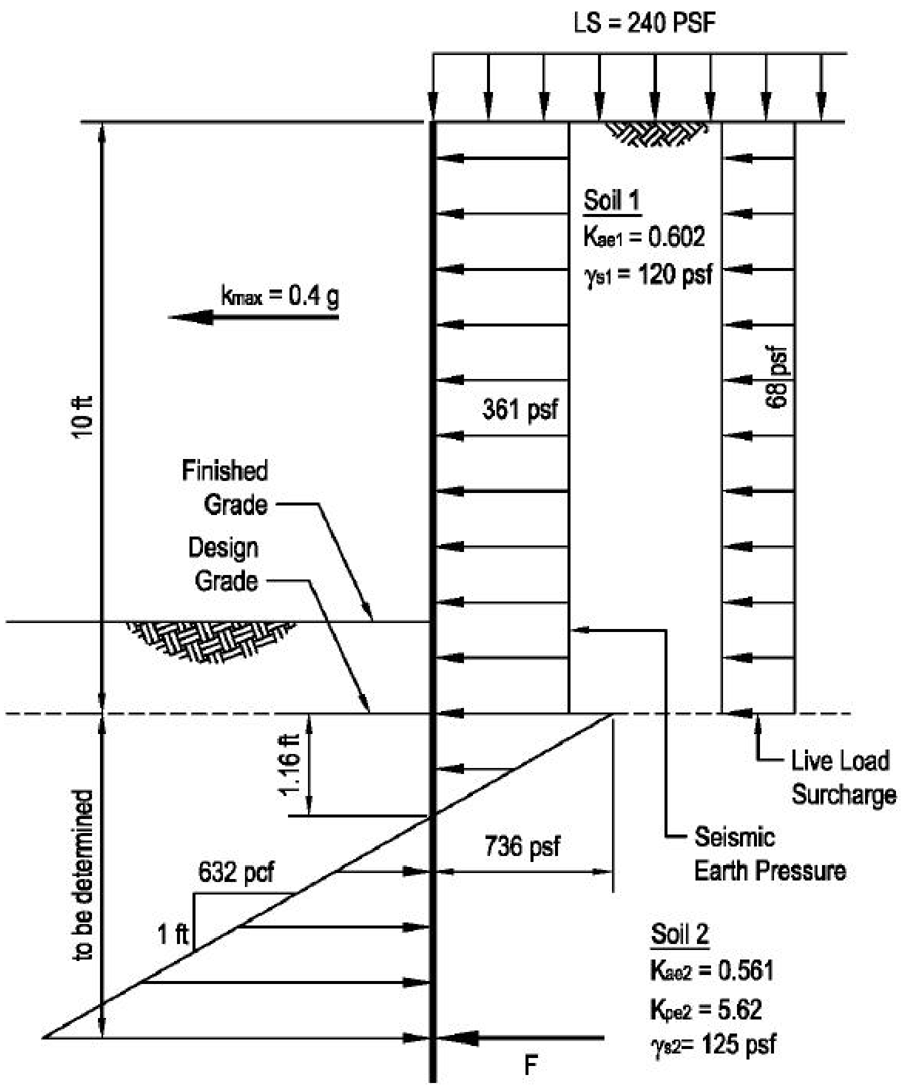


Figure 4.6 Distribution of Factored Earth Pressure for Extreme I Load Case (kmax =0.4)

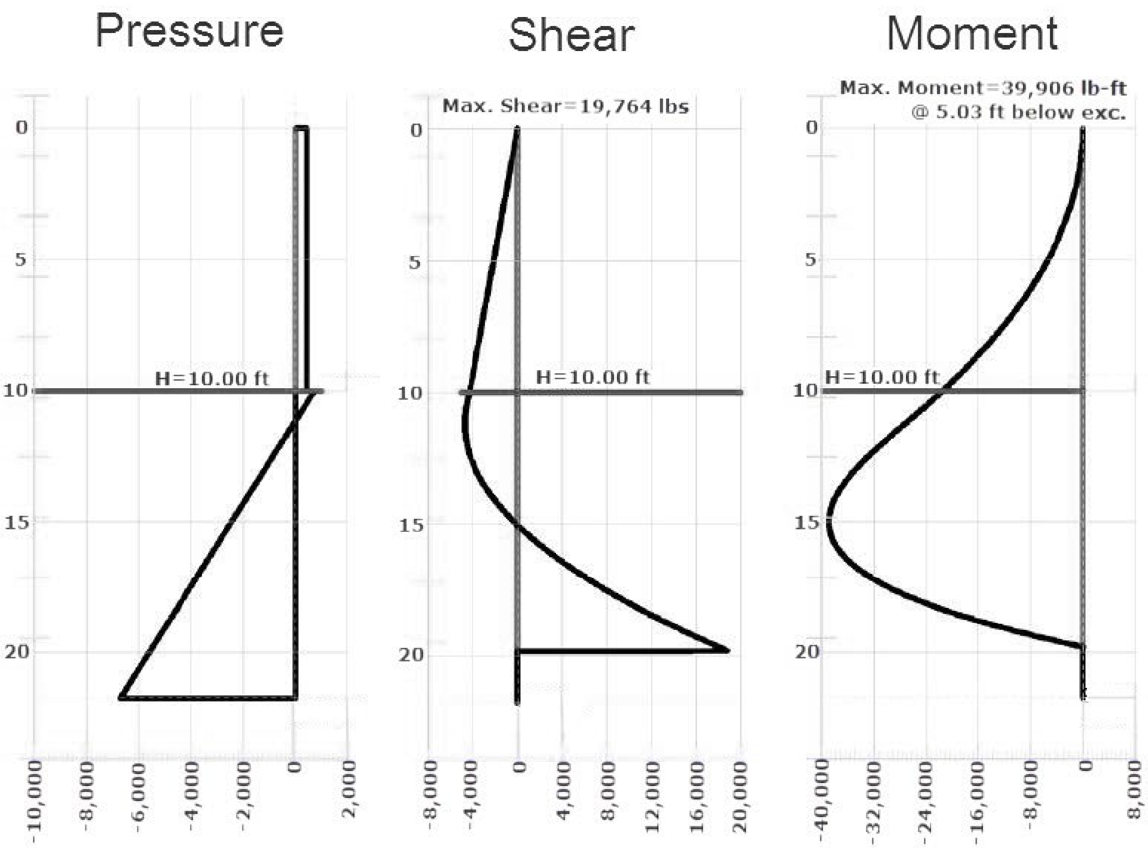


Figure 4.7 Calculated Earth Pressure, Shear, and Bending Moment Diagrams Based on the Limit-Equilibrium Method for the Seismic Case

4.1.5 Seismic Analysis Based on the *p-y* Curve Method (PYWALL)

As discussed for static loads, the earth pressure above the excavation line (from 0 to 10 ft) is treated as the external load. The external load is assumed to remain as a constant below the excavation line at the backfill side. For the case of Extreme I Seismic Load Case, the model in PYWALL (*Example 4b - LRFD Analysis of Sheet-Pile Wall for Seismic Case.py8d*) uses the Options > Earth-Pressure Options > Earth-Pressure Specified by User since the user needs to enter the active loads computed earlier (from Table 4.5) into Data > Distributed Active Earth Pressure.

The soil resistance below the excavation following the *p-y* curve method will be represented by nonlinear springs (*p-y* curves) for force-deformation relationship. The modulus of the *p-y* curves (k_{py}) was assumed to be 90 lbs/in^3 for the soil with an internal frictional angle of 36°. The soil properties for each layer, previously shown in Table 4.3, were used in PYWALL for the seismic case.

No strength reduction factor is applied on the *p-y* curves because they represent the force-displacement relationship. However, the cyclic *p-y* criteria (Options > Use Cyclic Model for p-y Curves) are recommended for seismic consideration because the degradation effect on soil resistance is partially included in those response curves. Keep in mind that additional field tests under cyclic loading are needed in order to improve the understanding on soil degradation.

Depth, ft	Active Earth pressure, psf	Surcharge Load, psf	Total applied pressure, psf
0	361	68	429
10	361	68	429
10	673.2	62.4	736
21.75	673.2	62.4	736

Table 4.5 Summary of Active Earth Pressure on the Sheet-Pile Wall for the Seismic Load Case

The resulting deflection, shear and bending moment diagrams computed by the program PYWALL are presented in Figure 4.8. The maximum shear from the SSI/PYWALL method is approximately 6.8 kips at about 18.3 ft below the top of the sheet pile, compared with 19.8 kips at 20 ft below the top of the sheet pile from the limit-equilibrium analysis. The difference is significant and mainly caused by the assumption of full soil mobilization in the limit-equilibrium method.

The maximum bending moment is approximately 33,080 ft-lbs (397 in-kips) at a depth of 14 ft below the top of the wall based on the PYWALL model, compared with 39,906 lbs-ft at 15 ft below the top of the sheet pile from the limit-equilibrium method.

4.1.6 Concluding Comments

1. Since the focus of this example was to illustrate the benefits of using PYWALL compared to the limit-equilibrium analysis, the seismic earth pressure was estimated using the Mononobe-Okabe method and inputted into PYWALL. A uniform distribution of the seismic earth pressure was assumed on the free-height wall. The load factor and the earth pressure coefficient for surcharge loads under seismic conditions are still poorly understood within the profession and should be used cautiously for different applications.
2. The results show that the sheet-pile wall analyzed using PYWALL, based on the AASHTO LRFD guidelines, performed very well for seismic conditions. It provides a rational way to aid the structural designer in obtaining a more refined evaluation of the actual performance of a flexible retaining wall, especially for the cases with seismic loading.
3. The limit equilibrium solution assumes that the ultimate passive pressure is fully mobilized along the entire embedded wall. Such an assumption may be reasonable up to about 5-ft depth below the excavation grade where sufficiently large wall deflection (greater than 1 inch) was calculated. However, the potential error of this assumption increases rapidly with depth when the wall deflection becomes very small due to the deformation of the wall.
4. The assumption on the passive resistance in the limit-equilibrium solution shown in this example leads to significantly higher shear and slightly higher bending moment, which are not reasonable in design.
5. Notice that the global stability of the cantilevered wall for seismic conditions should be evaluated separately with some other program (such as UTEXAS or the simpler EnSlope from ENSOFT) based on methods of slope-stability analyses.

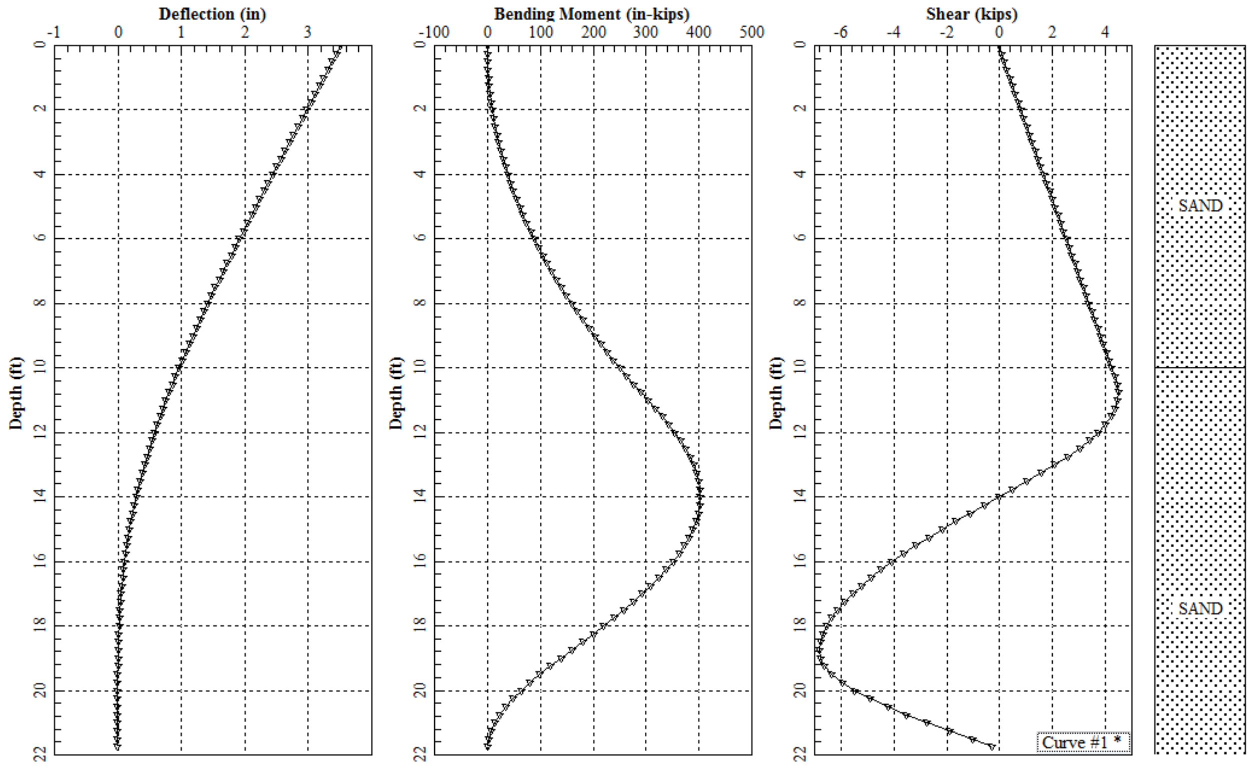


Figure 4.8 Deflection, Bending Moment and Shear Curves from the PYWALL for the Seismic Load Case in Example 4b

4.1.7 Input and Output Data Files for Example 4

Users can read Section 2.1.1 (7) of the User's Manual for reference on the location of placement of the input and output data files for the example files installed with this program. The default installation directory is the following: *(Root Drive) c:\Ensoft\PYwall2025-Examples*. The input data files for all examples presented in this manual are installed automatically with the program.

The input-data filenames for Example 4 are the following:

Example 4a - LRFD Analysis of Sheet-Pile Wall for Static Case.py8d

Example 4b - LRFD Analysis of Sheet-Pile Wall for Seismic Case.py8d

The output-data filenames for Example 4 are the following:

Example 4a - LRFD Analysis of Sheet-Pile Wall for Static Case.py8o

Example 4b - LRFD Analysis of Sheet-Pile Wall for Seismic Case.py8o

Example 5. Anchored Sheet-Pile Wall for Static and Seismic Conditions

5.1 Example 5 –Anchored Sheet-Pile Wall for Static and Seismic Conditions

5.1.1 Problem Description

This example illustrates the analysis of an anchored continuous wall using PYWALL under static and seismic loading conditions. The wall to be analyzed is presented in Figure 5.1, along with the soil layer information. The functions available in PYWALL can help the designer to compare the results and wall behavior under different levels of seismic loading conveniently and efficiently.

The wall for this example is 25 ft in height and involves a single unit of homogeneous sandy soil behind the wall as well as the foundation soils below the excavation grade. The unit weight of the sand is assumed to be 120 lb/ft^3 and the internal friction angle is 32 degrees. The sheetpile selected for the analysis has bending stiffness (EI) of 1.453 x 10^7 ft^2-lb/ft and yield stress of 50 ksi. The design consideration for the factor of safety will follow the AASHTO LRFD Method.

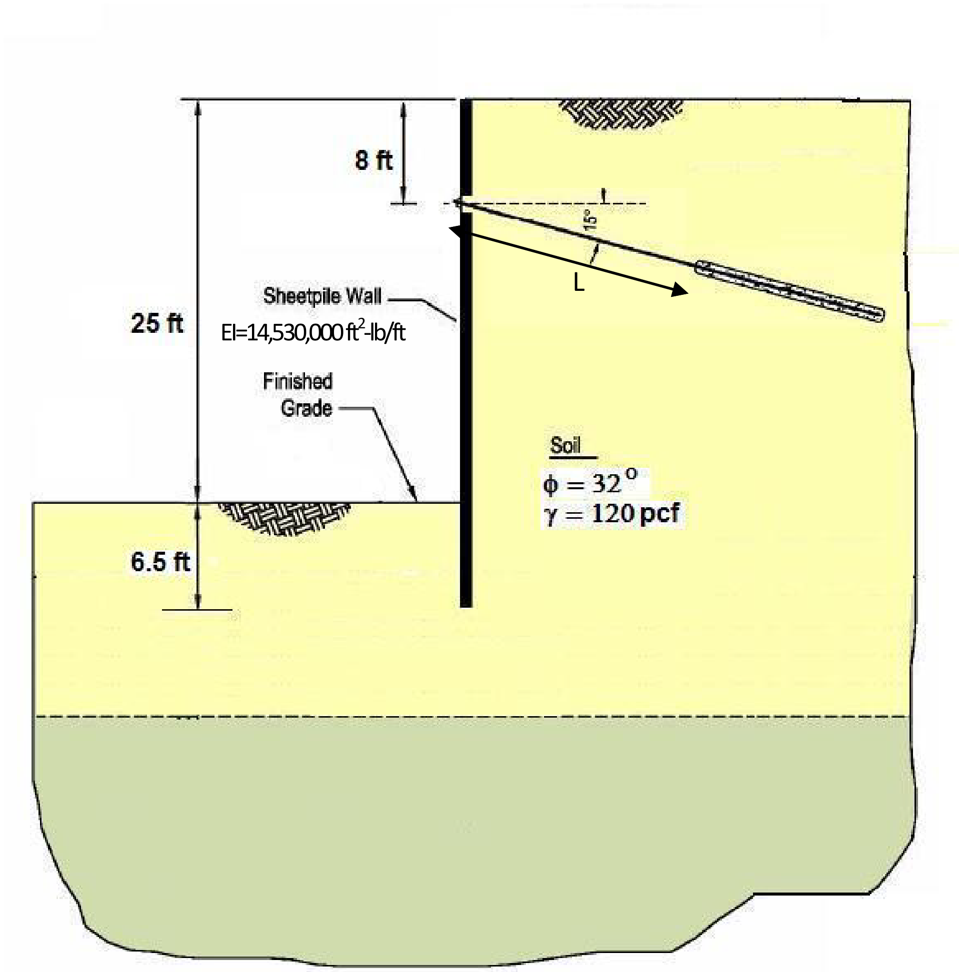


Figure 5.1 Geometry and Soil Information for Anchored Sheet-Pile Wall in Example 5

5.1.2 Static Active Earth Pressure Using AASHTO LRFD Method

Similar to the method for cantilever sheet-pile walls in Example 4 (Table 4.1), the loading condition is based on the active earth pressure theory. In this particular example, using the Rankine earth pressure theory with $\phi = 32^o$, the static active pressure coefficient (K_a) is calculated to be 0.307. The tieback anchor increases the wall/soil stiffness, which leads to an increase in the earth pressure.

A load factor of 1.3 is generally applied to the active earth pressure above the finished grade as the external load on the anchored walls. The total active earth pressure from the backfill above the depth of excavation grade (25 ft below the top) can be calculated as:

$$P_A = 1/2\ (\gamma_s)\ (H)^2\ (K_a)\ (\gamma) = (0.5)\ (120)\ (25)^2\ (0.307)\ (1.3) = 14{,}966 \text{ lbs/per ft}$$

The trapezoidal earth pressure distribution shown in Figure 5.2 is recommended by the Federal Highway Administration (FHWA) when a single tieback is used. The earth pressure P based on the trapezoidal geometry was calculated as:

$$P = P_A/(2H/3) = (14{,}966)/(0.667{*}25) = 898 \text{ psf} = 74.8 \text{ lb/in/ft}$$

The earth pressure increases linearly from 0 at the top to 898 psf at 5.33 ft (two-third of the anchor depth). The constant earth pressure of 898 psf at depth of 5.33 ft is extended to the depth of 13.67 ft (=5.33 + 25/3) and then decreases linearly to zero at depth of 25 ft (the final grade).

In conjunction with the trapezoidal earth pressure above the excavation level, an additional loading from the active earth pressure should be considered for the portion of the anchored pile wall beneath the excavation level. This set of earth-pressure loads increases from 921 psf (=0.307*120*25) at the excavation level to 1160 psf (=0.307*120*31.5) at the tip of the wall. The load was calculated using the static Rankine active earth pressure theory without a 1.3 amplification factor. It is an implicit assumption that the soil zone beneath the excavation level would be sufficiently far away from the tieback anchor and hence, would not be affected by the anchor stiffness. Figure 5.3 shows the loads from the active-pressure distribution for the static load case. In addition to the earth-pressure loads, Figure 5.3 also shows the passive soil resistance below the excavation level on the left side of the wall.

Wall friction angle (δ) was assumed to be zero on the active side. However, the designer should evaluate these assumptions on project-specific basis.

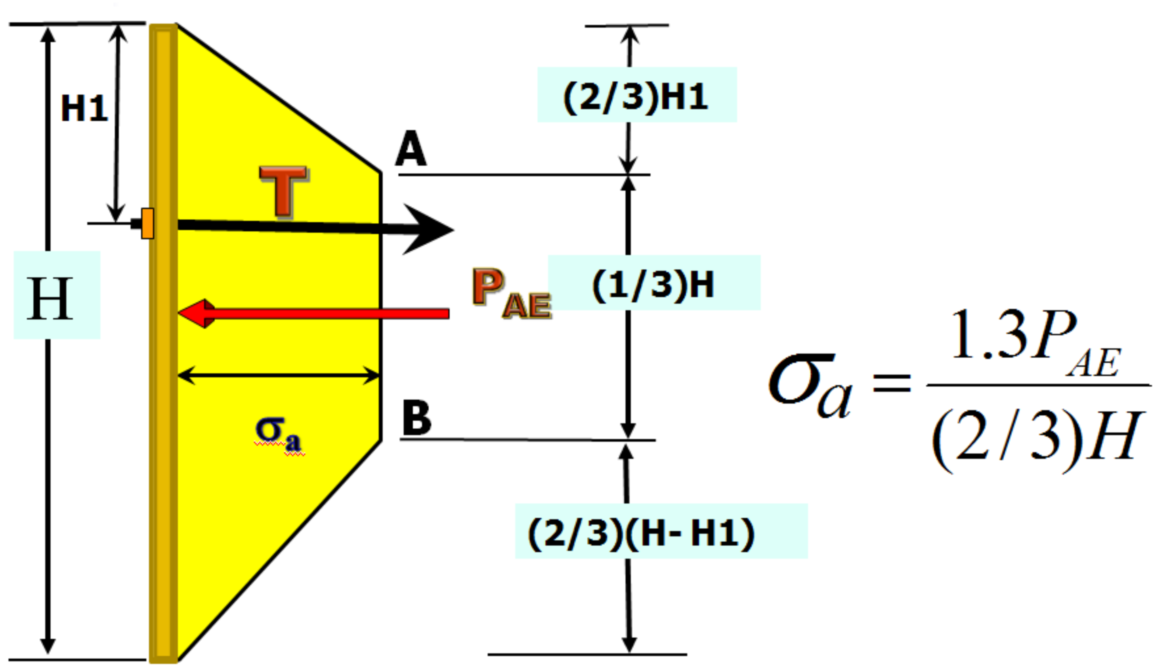


Figure 5.2 Distribution of Trapezoidal Earth Pressure for Single-Anchor Walls

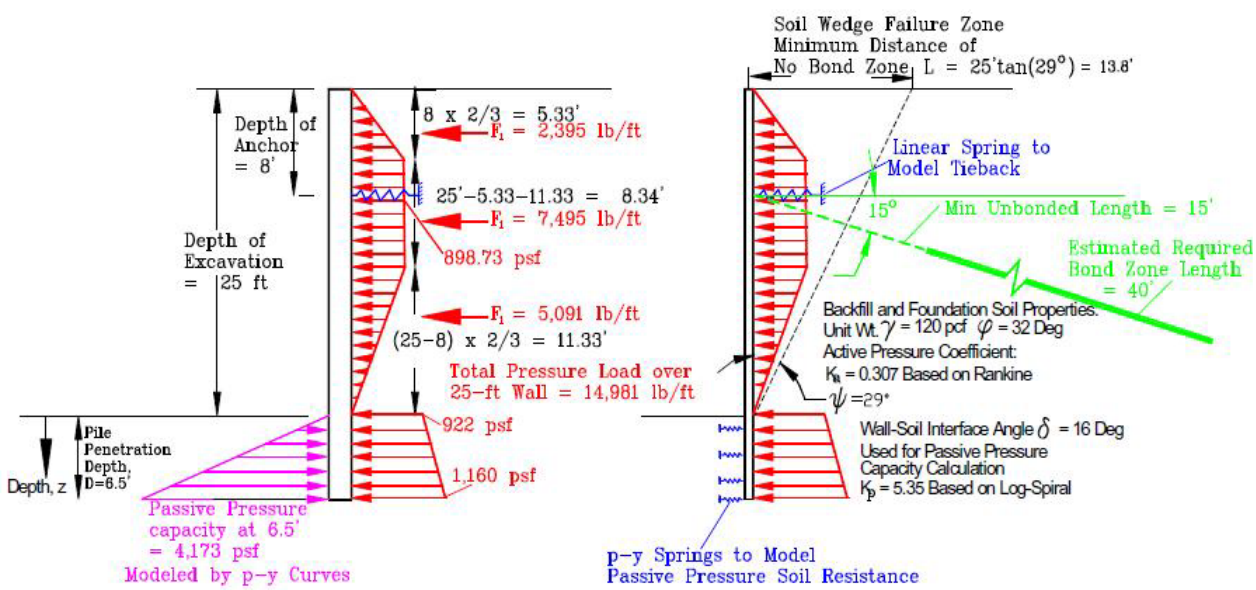


Figure 5.3 Distribution of Earth Pressure for Static Load Case in Example 5a

5.1.3 Traditional Analysis of Tieback Anchor

The total load from the active earth pressure above the excavation level is 14,966 lbs per foot of wall width. Assuming a lateral spacing between anchors of 8 ft, the design load per anchor would be 14,966 x

8 = 119,728 lbs (120 kips) per anchor. The anchors are assumed to be constructed using steel strands with an ultimate tensile strength of 270 ksi. The cross-sectional area of the strand is estimated below.

A_{cross} = 120 kips/(0.6*270 ksi) = 0.74 in^2

In addition to the basic design load (120 kips) for the anchor, the design load also needs to evaluate loads associated with proof testing. A factor at about 1.5 is typically specified for the proof-test condition. After proof testing to this higher load (180 kips), the anchors are relaxed to a theoretical lock-off design load (prestressing force). A high-strength reduction factor (0.8) is used for the required cross-section area under the temporary load. Therefore, the required cross section area for temporary proof test will be:

A_{proof} = 1.5*120 kips/(0.8*270 ksi) = 0.833 in^2

The length parameter L on the tieback needs to be estimated based on the unbonded length of the anchor as shown in the right side of Figure 5.1. Based on the Rankine active failure wedge, the lateral distance on the ground surface needs to be at least 13.8 feet for this example. An unbonded length of 15 ft was thus assumed for this example. Notice that it is necessary for the designer to check the boundary line of the Rankine failure wedge under static/seismic influences and make sure to have sufficient unbonded length passing through the active failure zone.

5.1.4 Static Analysis Based on the *p-y* Curve Method (PYWALL)

In the PYWALL model the earth pressure above the excavation line (from 0 to 25 ft) is treated as the external load. The external load in PYWALL is assumed to remain as a constant below the excavation line at the backfill side.

The soil resistance below the excavation following the *p-y* curve method is represented by nonlinear springs (*p-y* curves) for the force-deformation relationship. The modulus of these *p-y* curves (k_{py}) is assumed to be 50 lbs/in^3 for the cohesionless soil with an internal friction angle of 32^o. The soil properties for the PYWALL model are shown in Table 5.1.

Soil Type	Layer Thickness (ft)	Total Unit Wt (lbs/ft^3)	Internal Friction Angle (deg)	K_{p-y} (lbs/in^3)
Sand	25	120	32	50
Sand	10	120	32	50

Table 5.1 Soil Input Parameters for Example 5

The automatic computation of trapezoidal load distribution was selected for this example in PYWALL: Options > Earth-Pressure Options > Generate Trapezoidal-Distribution Pressure.

The tieback information computed in the previous section was entered in the PYWALL model under Data > Tieback Data > Enter Tieback Data.

The resulting active earth pressures automatically calculated within PYWALL for the Static Load Case are shown in Figure 5.4. The values shown in Figure 5.4 are approximately the same as those obtained from hand computations in Figure 5.3 (values in red color).

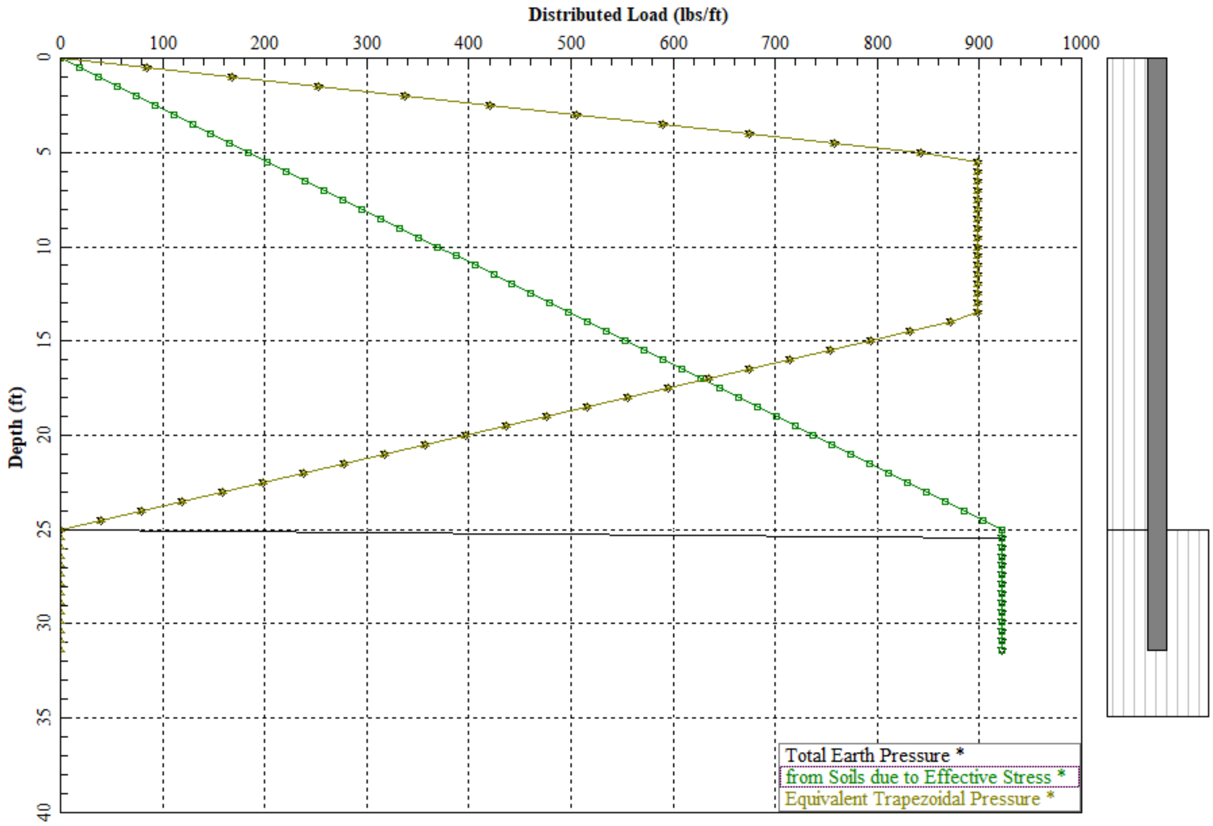


Figure 5.4 Computed Active Forces from PYWALL for the Static Load Case in Example 5

The resulting deflection, shear and bending moment diagrams in the PYWALL model are presented in Figure 5.5. The maximum bending moment based on the PYWALL model is approximately 20,670 ft-lbs (248 in-kips) at a depth of 17.3 ft below the top of the wall.

The total tieback force (in the axis of the tieback) was computed in PYWALL at 104 kips. This is in general agreement for the design concepts assumed in the previous section for this tieback.

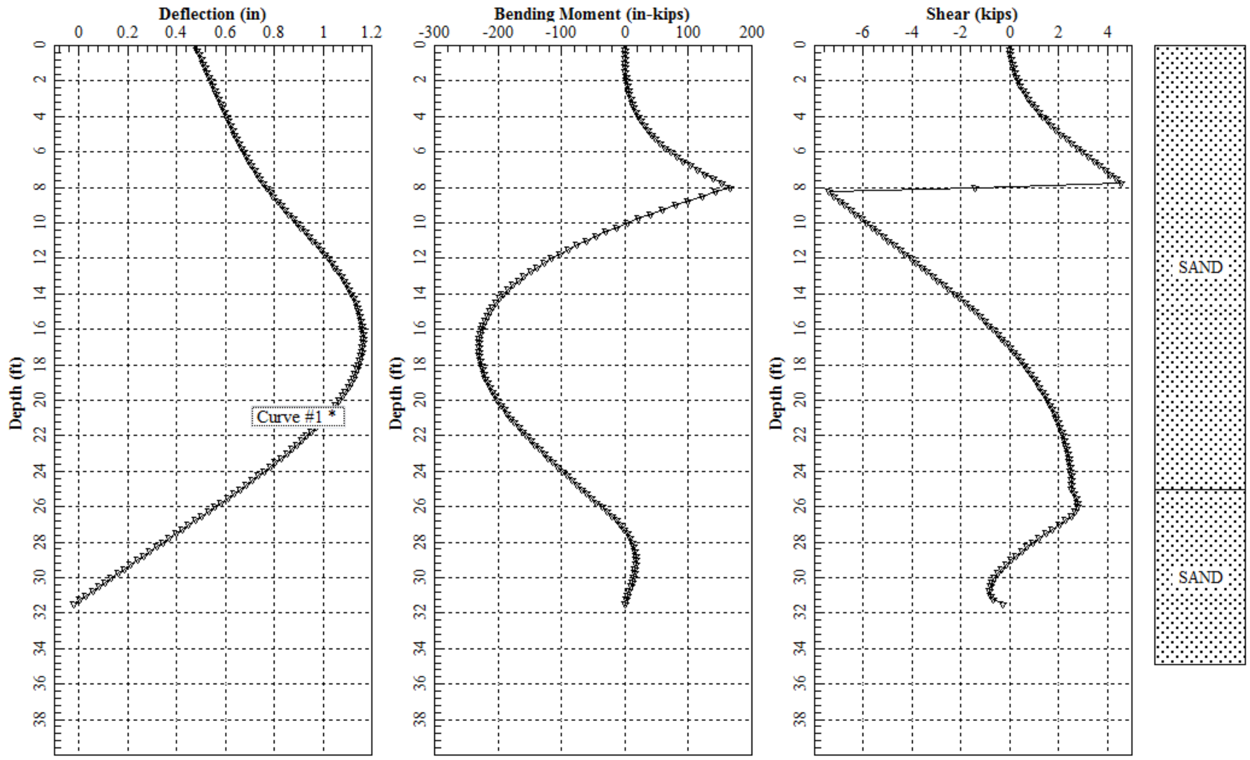


Figure 5.5 Deflection, Shear and Bending Moment Curves from the PYWALL Model for the Static Load Case in Example 5a

5.1.5 Active Earth Pressure for Seismic Loading

In addition to the static earth pressure previously shown in Figure 5.3, an incremental earth pressure for various horizontal seismic coefficients (k_{max}) can be calculated by following the Mononobe-Okabe equations (applicable to homogeneous and cohesionless soils), with the computed values shown in Table 5.2. Similar to the static loading condition, the conventional dynamic active pressure is increased by 30 percent to account for the stiffer anchor system on the earth pressure distribution above the excavation level.

Seismic Coefficient, k_{max}	0.1	0.2	0.4
$K_{a,\ static}$	0.307	0.307	0.307
$K_{a,seismic\ (from\ M\text{-}O\ equations)}$	0.37	0.44	0.65
$K_{a,\ net}$	0.063	0.133	0.343
Total active load from $K_{a,net}$ (seismic – static) = (1/2) ($K_{a,net}$) (γ) $(H)^2$ (1.3)	3,071 lb/ft	6,484 lb/ft	16,721 lb/ft
Uniformly Distributed Pressure over 25 ft	122.8 psf	259.4 psf	668.9 psf

Table 5.2 Summary of Active Earth Pressure for Seismic Loads on Example 5b

5.1.6 Seismic and Static Analyses with PYWALL

The feature Options > Conduct Staged-Construction Analysis was used in this example so the results from all the loading conditions (static plus 3 seismic conditions) are displayed in the same graph. The construction stages 1 through 4 are thus used to represent the different loading. Soil and wall properties are kept unchanged between the four dummy "construction" stages.

Figure 5.6 presents the results of the PYWALL analyses for the anchored wall under static and seismic loading conditions. Notice that the results from the Static load case are similar to those from the previous model using static conditions. The maximum bending moment is approximately 554 in-kips at a depth of 18.5 ft below the top for seismic case 4 (with k_{max} = 0.4g), compared with 248 in-kips at 17.3 ft below the top of the sheet pile from the static case (approximately 220% increase).

The SSI analyses also indicate (Figure 5.7) that the tieback forces would be 104, 124, 147 and 214 kips per anchor (8-ft spacing) for (1) static load case, (2) seismic case with k_{max} = 0.1g, (3) seismic case with k_{max} = 0.2g, and (4) seismic case with k_{max} = 0.4g, respectively. Notice that the anchor force under the seismic loading of 0.4g is over the anchor capacity of 180 kips (23 kips x 8 ft spacing) and the anchor would be overloaded in this case.

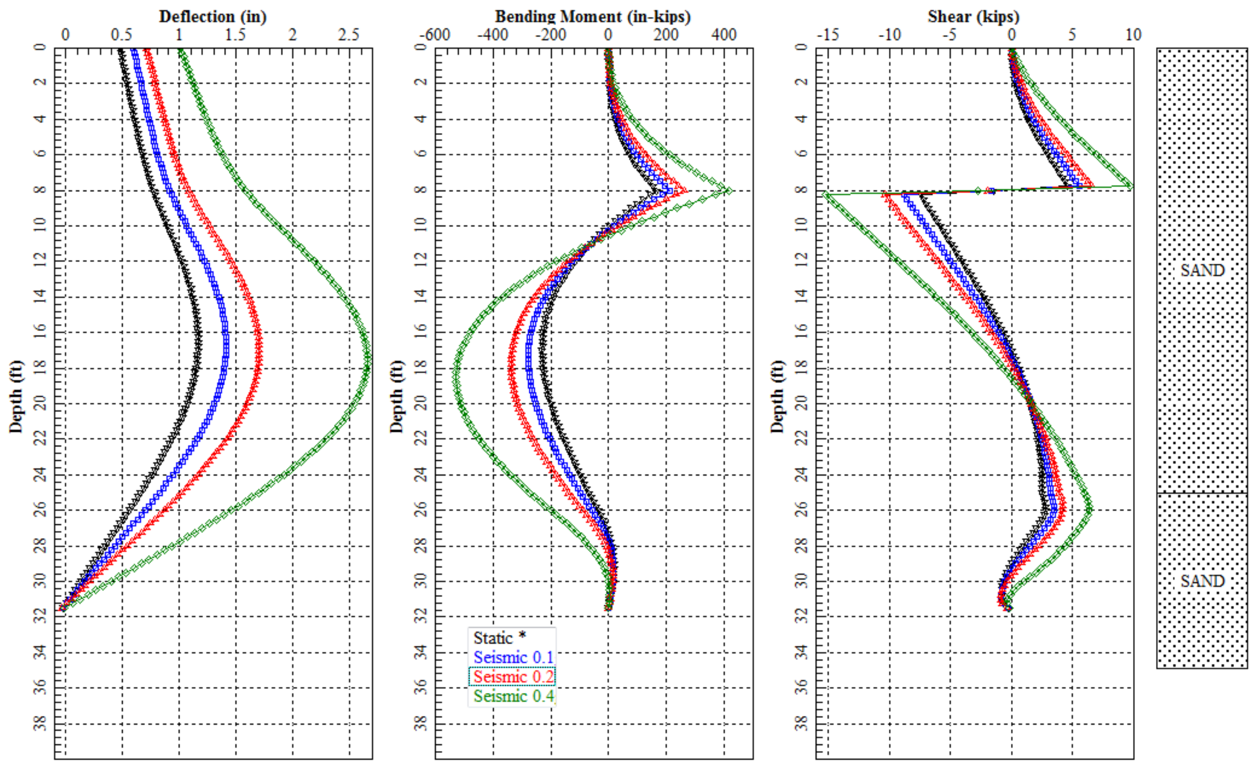


Figure 5.6 Deflection, Shear and Bending Moment Curves from the PYWALL Model for the Static and Seismic Load Cases in Example 5b

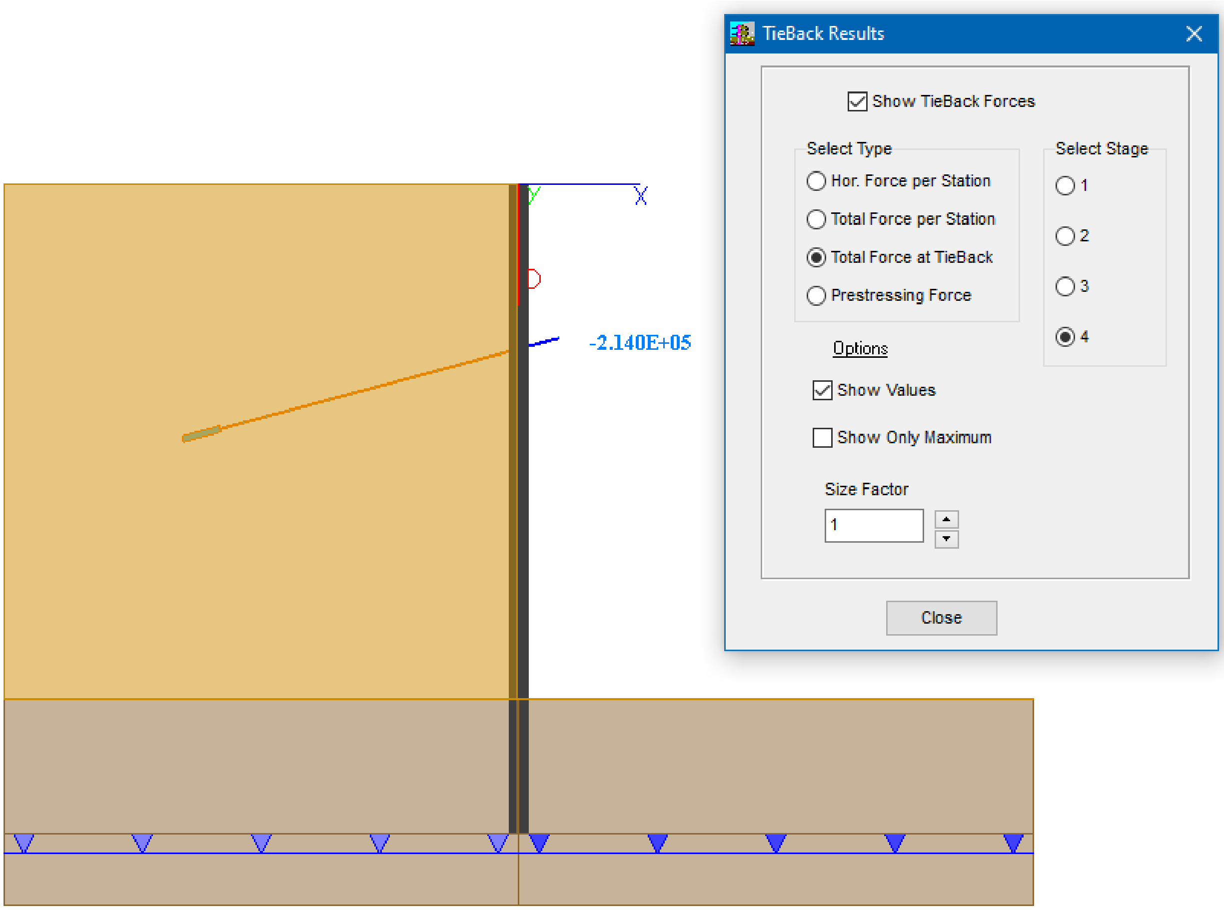


Figure 5.7 Total Tieback Force for Seismic Load Case 4 in Example 5b

5.1.7 Concluding Comments

1. The global stability of the anchored wall for seismic conditions should be evaluated separately based on the method of slope-stability analyses.
2. The seismic earth pressure was estimated using the Mononobe-Okabe method. A uniform distribution of the net seismic earth pressure (after subtracting the static earth pressure with trapezoidal distribution) was assumed on the free-height wall for easy comparison with various earthquake coefficient k_{max}. It is relatively convenient and simple for parametric studies.
3. The results show that the program PYWALL for the analysis of this anchored sheet-pile wall is rational and adaptable for seismic conditions. It provides a reliable way to aid the structural designer in obtaining a more refined evaluation of the actual behavior of flexible anchored wall.

5.1.8 Input and Output Data Files for Example 5

Users can read Section 2.1.1 (7) of the User's Manual for reference on the location of placement of the input and output data files for the example files installed with this program. The default installation directory is the following: *(Root Drive)c:\Ensoft\PYwall2025-Examples*. The input data files for all examples presented in this manual are installed automatically with the program.

The input-data filenames for Example 5 are the following:

Example 5a - Analysis of Anchored Sheet-Pile Wall for Static Case.py8d

Example 5b - Analysis of Anchored Sheet-Pile Wall for Seismic Conditions.py8d

The output-data filenames for Example 5 are the following:

Example 5a - Analysis of Anchored Sheet-Pile Wall for Static Case.py8o

Example 5b - Analysis of Anchored Sheet-Pile Wall for Seismic Conditions.py8o

Example 6. Anchored Timber-Lagging Wall in Stiff Clay

6.1 Example 6 – Anchored Timber-Lagging Wall in Stiff Clay

6.1.1 Problem Description

Timber-lagging walls are widely used as temporary retaining systems in basement excavation or cofferdam construction. Timber lagging is the system of wall-panels made-up of thick timber planks spanning between the steel H piles. The thick timber planks transfer the earth pressure to the steel H piles. The spacing between two support piles is typically from 6 ft to 8 ft. For deep excavations, timber-lagging walls need additional support from tiebacks or earth anchors at different levels. The conventional method of limit equilibrium has limitations in analyzing the behavior of the timber-lagging wall because simplified assumptions are used in solving the reaction at each level of tiebacks. The *p-y* curve method in PYWALL allows the tiebacks to be modeled as elastic springs, which take into account the elastic elongation of the tieback at each location.

The final depth of the excavation for this example is proposed to be 24 ft below the top grade. The subsurface within the top 40 ft at the excavation site is fissured medium stiff clay near the top and becomes stiff clay below the excavation depth. The clay has a total unit weight of 125 lb/ft^3 and cohesive strength of 1800 lb/ft^2. The water table is at 15 ft below the surface of the excavation. The cohesive strength of stiff clay below the excavation depth is of 5,000 lb/ft^2. The soil input parameters are summarized in Table 6.1.

The embedment length of the H piles (HP13x60) is 10 ft and the tiebacks are installed at 6 ft and 14 ft below the ground surface.

Soil Type	Layer Thickness (ft)	Total Unit Weight (lbs/ft^3)	Cohesive Strength (psf)	E_{50}
Fissured Stiff Clay	24	125	1800	0.005
Stiff Clay	20	125	5000	0.005

Table 6.1 Soil Input Parameters for Timber Lagging Wall on Example 6

6.1.2 Earth Pressure Distribution

The distribution of earth pressure for a braced wall is different from the triangular distribution of earth pressure on unbraced walls (Terzaghi and Peck, 1967). The earth pressure is thus calculated based on the recommended pressure-distribution diagram for braced walls by assuming no significant wall movement at the excavation level. The earth pressure on each soldier pile should include the soil in the tributary area, i.e., the soil mass with a width equal to the spacing between two soldier piles. The unit for distributed loads on soldier piles is “line load/depth” on each pile, such as lbs/inch, lbs/ft, or kN/m per soldier pile.

Active earth pressures for this model are not computed internally within PYWALL since those are based on the Rankine equation, see Section 6.2 of the Technical Manual. When cohesive soils or even cohesive silts (c > 0, and phi very small) are involved, PYWALL automatically considers undrained analyses which use the total stress (total unit weight) and no hydrostatic water pressure (so basically, no free water is considered in clay layers).

Instead, calculation of the trapezoidal distribution of earth pressure behind the wall based on the apparent earth pressure recommended by Terzaghi and Peck (1967) is presented in Figure 6.1. The distribution of active earth pressure was terminated at the excavation depth, and no active earth pressure was considered below the excavation depth for this example because the supporting layer is a very stiff clay (c_u = 5000 psf). Notice that if the active earth pressure below the excavation level should be considered for non-cohesive layers, the tributary width for the active earth pressure below the excavation level should be only the width of the H-pile (12 inches), not the spacing (8 ft) between soldier piles.

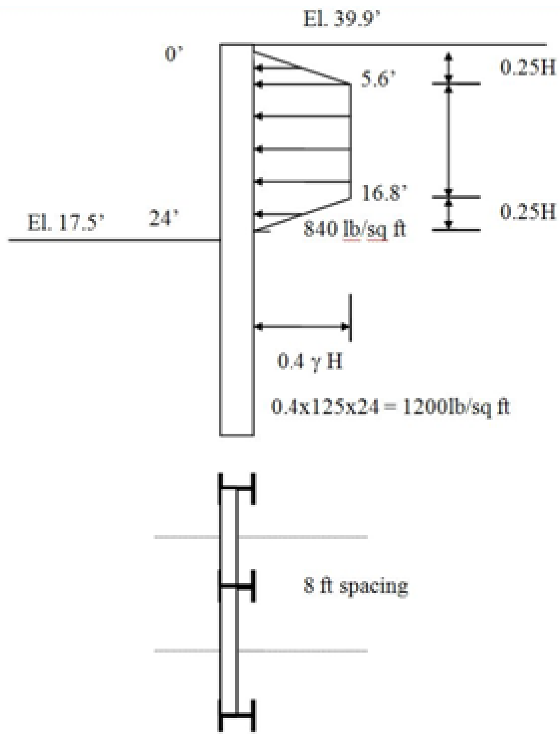


Figure 6.1 Trapezoidal Pressure Distribution on Braced Walls

6.1.3 Soil Resistance (p-y curves) and Tiebacks

The total length of the soldier pile is 34 ft and it was divided into 134 increments with 3.0 in. of increment length. The flange width of the soldier pile (HP12x53) is 12 inches and it was used for computations of the soil resistance (*p-y*) curves below the excavation depth. Notice that the previously computed active earth pressure took into account the earth pressure over 8-ft span width, which is the tributary area of earth pressure on each soldier pile.

The anchor rod of each tieback (located at 6 and 14-ft from top of wall) has a diameter of 1.0 inch (for a cross-sectional area of 0.7854 in^2), a free length of 30 ft, and is spaced at 8 ft apart along the length of the wall. The tiebacks were installed on every soldier pile and the calculated stiffness represented the stiffness on each soldier pile. The tiebacks are installed with a prestressing force of 75 kips per location and are angled at 25 degrees to the horizontal. Tieback information was entered in the PYWALL model under Data > Tieback Data > Enter Tieback Data.

Notice that in some cases the designer may install tiebacks at every other soldier pile. PYWALL allows the user to model arbitrary spacing on each level of tieback.

6.1.4 Numerical Solution from PYWALL

The solution from PYWALL (filename: *Example 6a - Anchored Timber-Lagging Wall in Stiff Clay.py8d*) based on the earth pressure, anchor stiffness, and the nonlinear *p-y* curves is presented in Figure 6.2. The maximum deflection is well under 0.5 inches. The maximum bending-moment on the soldier pile, which has a moment of inertia (I) of 393 in^4, is approximately 650,000 in-lbs. The maximum bending stress for a HP12x53 soldier pile is thus:

$$S = (M_{max})\ (y) / I = (650{,}000)(11.78/2)/393 = 9.7 \text{ ksi}$$

which is much lower than the allowable bending stress with Grade 36 steel (23.7 ksi = 0.66x36 ksi). The computed reaction of the tiebacks are 79 and 86 kips for the top and bottom anchors, respectively. These loads are about 73% of the capacity of a 150 ksi anchor rod (if such are used in this design).

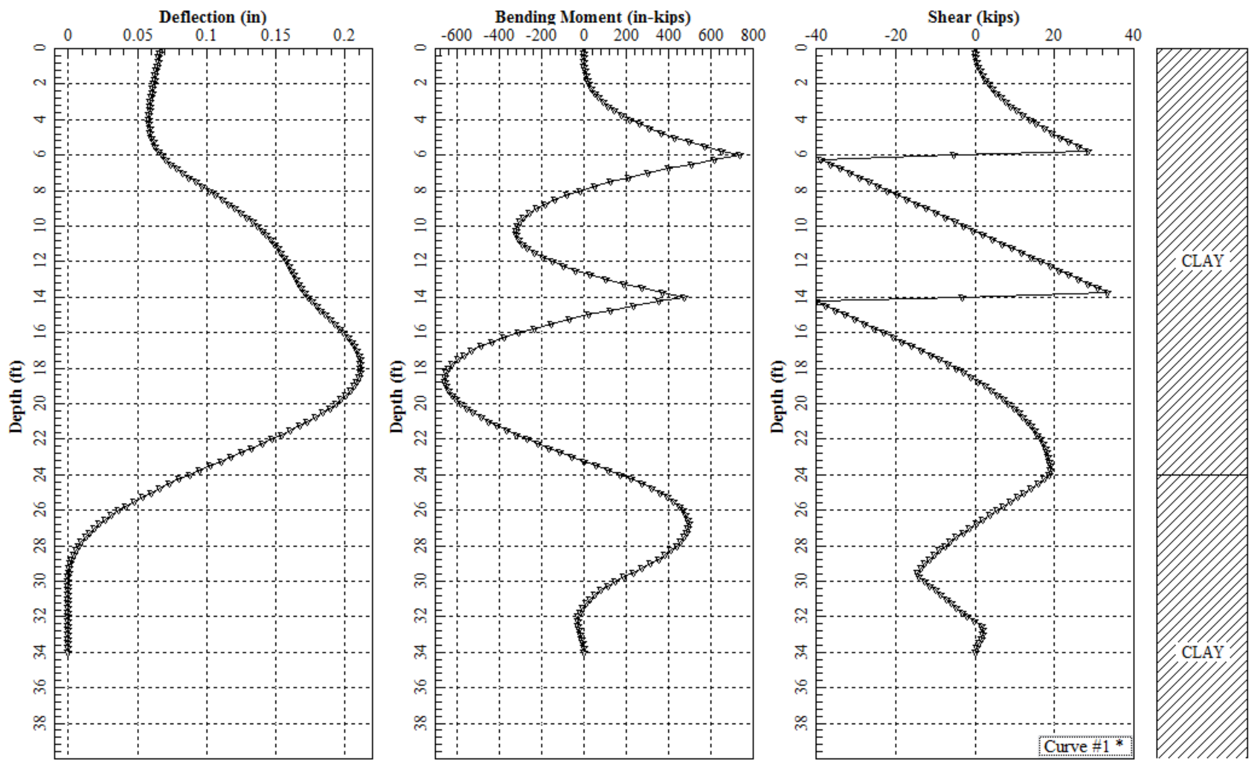


Figure 6.2 Deflection, Bending Moment and Shear Curves from the PYWALL Model for Example 6a

6.1.5 Check the Excavation Safety Before Tieback Installation

The file *Example 6b - Anchored Timber-Lagging Wall in Stiff Clay-Check.py8d* includes a check of the response of the wall before the installation of the first level of tieback (at 6 ft from top of wall). Calculation of the earth pressure on the 8-ft cantilever wall (assume 2 ft of over-excavation below the 6-ft level of the first level of tieback) before the first row of tieback installation is needed and is entered in the PYWALL model using Options > Earth-Pressure Options > Earth –Pressure Specified by User under Data > Distributed Active Earth Pressure

The fissured clay may be filled with water so an equivalent earth pressure of 45 psf was assumed for this check. The active earth pressure at 8 ft below the ground is 2,880 lbs/ft (= 45 psf*8 ft*8 ft). In the output results from this PYWALL model check, the maximum bending-moment on the soldier pile before installation of the first level of tieback is approximately 630,000 in-lbs. The maximum bending stress for the HP12x53 soldier pile is thus:

$$S = (M_{max})\ (y) / I = (630{,}000)(11.78/2)/393 = 9.5 \text{ ksi}$$

which is less than the allowable bending stress with Grade 36 steel 23.7 ksi (=0.66 x36 ksi).

6.1.6 Check the Stability against Bottom Heave

The designer should also check the stability due to the heave of the bottom excavation. The total overburden pressure on the back of the retaining wall at excavation depth is about 3000 psf (= 125 pcf * 24 ft). The allowable bearing capacity at the excavation line is 5.7 (Cu)/(F.S.) = (5.7) (5000)/3.0 = 9500 psf (using a factor of safety of 3), which is greater than the overburden pressure. There is thus no heave problem at this site.

6.1.7 Input and Output Data Files for Example 6

Users can read Section 2.1.1 (7) of the User's Manual for reference on the location of placement of the input and output data files for the example files installed with this program. The default installation directory is the following: *(Root Drive) c:\Ensoft\PYwall2025-Examples*. The input data files for all examples presented in this manual are installed automatically with the program.

The input-data filenames for Example 6 are the following:

Example 6a - Anchored Timber-Lagging Wall in Stiff Clay.py8d

Example 6b - Anchored Timber-Lagging Wall in Stiff Clay-Check.py8d

The output-data filenames for Example 6 are the following:

Example 6a - Anchored Timber-Lagging Wall in Stiff Clay.py8o

Example 6b - Anchored Timber-Lagging Wall in Stiff Clay-Check.py8o

Example 7. Analyzing a Drilled-Shaft Wall in Sand

7.1 Example 7 – Analyzing a Drilled-Shaft Wall in Sand

7.1.1 Problem Description

Drilled shafts with or without tiebacks have been used widely for retaining structures in highway underpasses. Drilled-shaft walls can provide large flexural rigidity and, thus, can support a cantilever wall with a height over 25 ft. If the deflection at the top of the wall is over the allowable limit, which may cause some damage on the neighboring structures, tiebacks or earth anchors can be applied at specific heights.

Drilled shafts with 36-in OD and 60 ft in total length were used for a permanent retaining system in this example. The height of the free-standing section (wall height) is 30 ft. The drilled shafts were placed with 4.5 ft of center-to-center spacing (Figure 7.1). The top 45 ft of soil is medium dense to dense sand, underlain by very stiff clay embedded with clay shale. The undrained shear strength of the clay shale is about 8 ksf. The soil parameters, which were recommended for design based on site investigations, are presented in Table 7.1.

Soil Type	Layer Thickness (ft)	Total Unit Wt (lbs/ft^3)	Friction Angle (deg)	K_{p-y} (lbs/in.^{3} s)	Cohesive Strength (psf)	E_{50}
Dense sand	45	125	38	120	-	-
Clay shale	30	125	-	-	8000	0.005

Table 7.1 Soil Input Parameters for Drilled Shaft Wall on Example 7

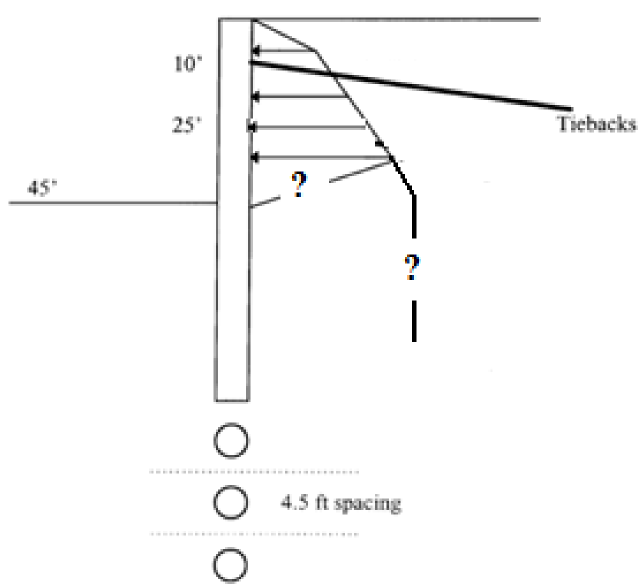


Figure 7.1 Geometry and Arrangement of Drilled-Shaft Retaining Wall for Example 7

7.1.2 Earth Pressure Distribution

The distribution of earth pressure for drilled shafts without tiebacks can be calculated based on a triangular distribution of earth pressure on cantilevered walls. If multi rows of tiebacks are used to restrain the deflection, the distribution of active earth pressure may change to a trapezoidal shape, depending on field conditions.

The triangular distribution of soil earth pressure is employed for this example for drilled-shaft walls without tiebacks. The earth pressure on each drilled shaft should include the soil in the tributary area, i.e., the soil mass with a width equal to the spacing between two shafts, which was 4.5 ft in this application. The unit for distributed loads on drilled shafts is "line load/depth" on each shaft, such as lbs/inch, lbs/ft, or kN/m per drilled shaft. The distribution of active earth pressure shown in Table 7.2 was calculated by hand using Rankine's theory.

Depth (ft)	Lateral Load	
	(lbs/in)	(lbs/ft)
0	0	0
10	111	1,332
20	222	2,664
30	333	3,996

Table 7.2 Earth-Pressure Distribution for Example 7a

PYWALL extends the active earth pressures below the excavation. However, notice that the active earth pressure computed within PYWALL below the excavation will be reduced since the pressure only applies to the pile diameter of 36 inches (instead of the pile spacing of 4.5-ft above the excavation). However, the designer may consider that the clay/shale below the excavation may arch the small 6-in spacing between shafts and thus produce a higher loading. Such an effect can be modeled by PYWALL if the user computes the pressure outside the program and enters the loads using Options > Earth Pressure Specified by User and Data > Distributed Active Earth Pressure.

7.1.3 Soil Resistance (*p-y* curves) and Reduction Factors

The length of the drilled shaft is 60 ft and it was divided into 120 increments that are 6.0 inches in length. The diameter of the drilled shaft is 36 inches and it was used in computation of the soil resistance (*p-y*) curves below the excavation depth. A group reduction factor should be considered in computations of the *p-y* curves because the drilled shafts were installed with a close spacing (center-to-center spacing = 1.5 diameter). The reduction factor can be as low as 0.5 if drilled shafts are in contact with each other without any spacing. The reduction factor is computed internally by PYWALL based on the widths introduced in the Data > Wall Configuration menu. For this example, the PYWALL program computed a *p*-reduction factor of 0.67 (see Computation > Edit Output Text in the section labeled P-Y CURVES DATA).

The soil resistance curves (*p-y* curves) below the excavation line for this example were generated as shown in Figure 7.2 based on the criteria described in the Technical Manual. It is noticeable that the soil resistance (*p*) at the backfill side is significantly higher than those at the excavation side in the sand layer due to the difference in overburden pressure. Notice that some of the *p-y* curves in Figure 7.2 may not be continued beyond certain values of deflection (*y*). These *p-y* curves are considered constant by internal computations in the PYWALL program for deflections beyond the display limits in Figure 7.2.

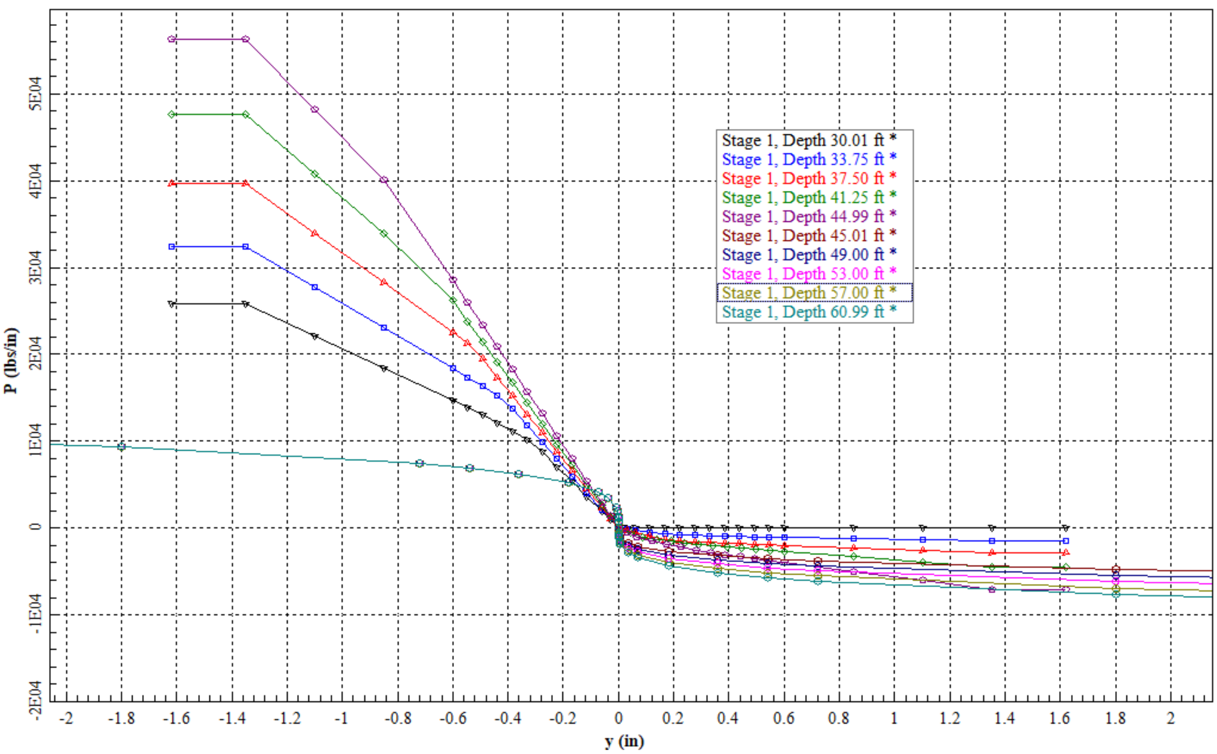


Figure 7.2 Soil Resistance Curves Generated by PYWALL for Example 7a

7.1.4 Numerical Solution from PYWALL

PYWALL can analyze the bending capacity and nonlinear flexural rigidity (EI) for the structural members. The ultimate bending capacity of the cross section (36 inches) was predicted to be approximately 15,000 in-kips (Figure 7.3) if 4000 psi of the compressive strength of concrete and 2% steel ratio (20 #-9 bars) were specified. As shown in Figure 7.3, the user can enter the rebar sizes and material yield strength for the specified cross section. The PYWALL program calculates the bending capacity of the user-specified section and also applies the nonlinear EI (nonlinear bending moment versus curvature) in the analysis.

The drilled-shaft model solved by PYWALL for this example is included in filename: *Example 7a - Drilled Shaft Wall in Sand without Tieback.py8d*. The computed wall behavior based on the earth pressure, linear or nonlinear flexural rigidity of drilled shaft, and the nonlinear *p-y* curves is presented in Figure 7.4. The feature Options > Conduct Staged-Construction Analysis was used in this example so the results from both pile materials (elastic and nonlinear) are displayed in the same graph. The construction stages 1 and 2 are thus used to represent the different responses due to the pile material. Soil, wall and loading properties are kept unchanged between these two dummy "construction" stages.

For the cantilever condition, a nonlinear EI is generated by the program and a constant EI, which was inputted by the user based on the gross section, were both considered in the analysis for a comparison.

The deflection computed based on a nonlinear EI is significantly higher than that from the case with a constant EI. The difference in bending moments and shears between those two cases is not significant as compared to the deflection. The maximum moment at a depth of 38 ft below the top of the drilled shaft was computed to be approximately 11,200 in-kips for the linear-elastic case, which is lower than the nominal unfactored ultimate bending capacity of the reinforced concrete section (user must calculate the capacity of the wall section outside PYWALL based on applicable code and using applicable resistance factors). The maximum deflection at the top of the drilled shaft is unacceptable for a retaining system so tiebacks are necessary on this application.

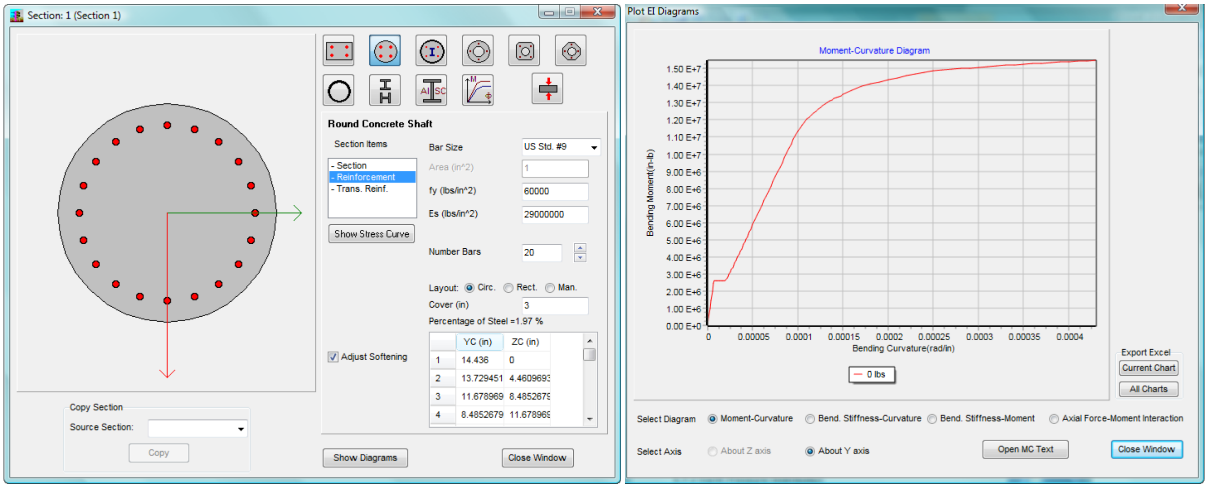


Figure 7.3 Computed Ultimate Bending Capacity for Drilled Shaft with 36-in OD, 4000 psi Compressive Strength and 2% of Steel Ratio.

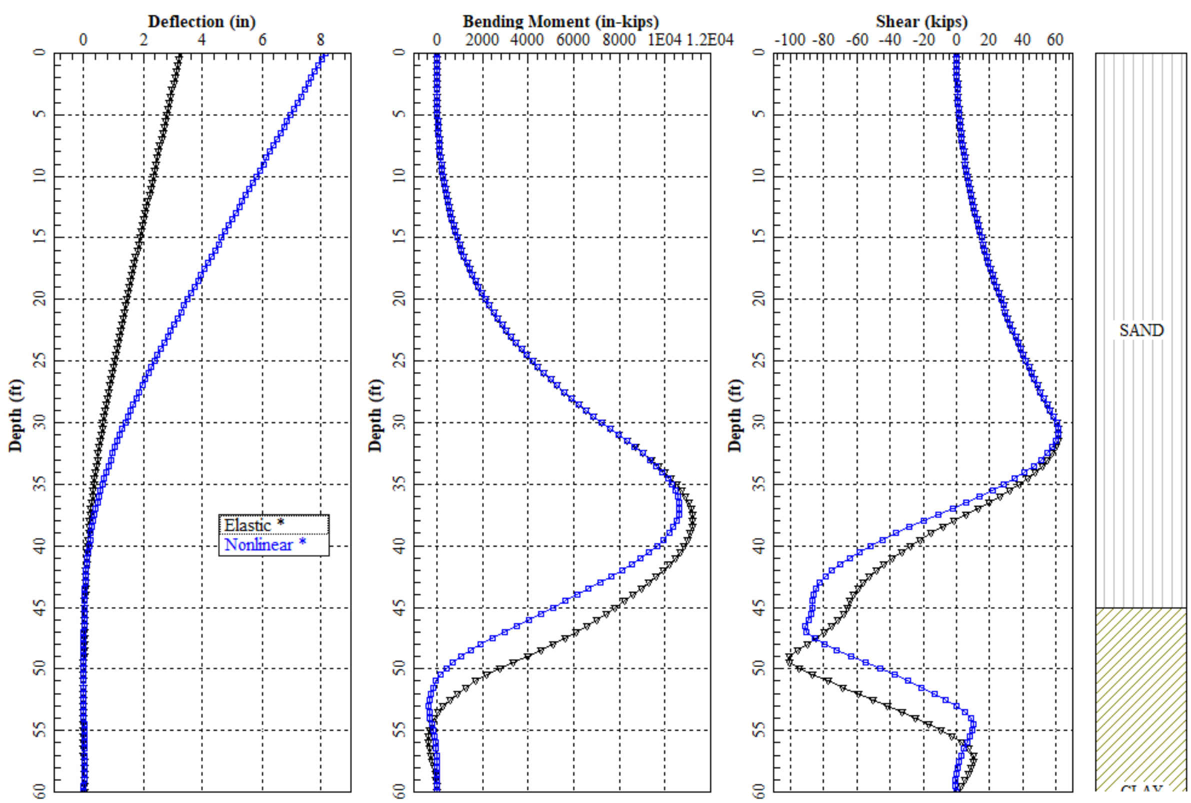


Figure 7.4 Deflection, Bending Moment and Shear Curves from the PYWALL Model for Example 7a Without Tiebacks

7.1.5 PYWALL Model with Tiebacks

When a cantilever wall is over 25-ft in height, the generated bending moment or the deflections on the drilled shaft can be very large and difficult to handle. In some of those cases, it may be necessary to use tiebacks to reduce the deflection and bending moment on the drilled shaft.

To control the large top deflection, in this example they modeler may choose a 1-inch diameter deformed bar (with a cross-sectional area of 0.85 in^2), installed in every other shaft (every 9 ft) at an angle of 25 degrees to the horizontal, at a depth of 10 ft below the top of the wall and initially without any prestressing force. It is also assumed that the free length of the tieback is 30 ft and the rod material is high-strength steel. The assumed free length only influences the elastic deformation so it can be adjusted on a final design with only minor influences to the PYWALL model.

The tiebacks are assumed to be installed on every other drilled shaft (every 9 ft) and some of the PYWALL model (from filename: *Example 7b - Drilled Shaft Wall in Sand with Tieback.py8d*) results are presented in Figure 7.5. The maximum deflection for the nonlinear model was reduced from 8.4 inches to 1.2 inches, which may be a more acceptable range for this retaining system. The maximum bending-moment at 38 ft below the top of the wall was also reduced from 11,400 to 4,000 in-kips, which is also a significant reduction.

7.1.6 PYWALL Model with Tiebacks Using Trapezoidal Loading

When using anchored walls, the modeler would normally choose a trapezoidal distribution of the earth pressures. In PYWALL this is done very simply and quickly by selecting Options > Earth-Pressure Options > Generate Trapezoidal-Distribution Pressure. It must be done for both construction stages (which in this model are used for elastic and nonlinear wall elements). The new model is distributed with filename: *Example 7c - Drilled Shaft Wall in Sand with Tieback and Trapezoidal Loads.py7d*.

The user may select Graphics > Earth Pressure Combined to see all the various components of earth pressure in this project (for each Stage), including the original triangular distribution that are used as basis for the trapezoidal distribution.

As shown in Figure 7.6, the maximum deflection for the nonlinear model is increased from 1.2 inches to almost 2.4 inches, and the maximum bending-moment was also increased from 4,000 in-kips to almost 6,800 in-kips.

In checking the total force in the tieback (either in Computation > 3D View and Show 3D View > Tieback Results or at the end of the output text file in Computation > Edit Output Text), as shown in Figure 7.7, the user will realize that the 115.8 kips may be too close or even exceed the ultimate capacity of many 1-in high-strength bars. For instance, a 1-inch bar with 150 ksi is rated at 127.5 kips loading at ultimate. If the resulting load is not admissible (according to applicable codes), the modeler may choose a larger tieback or install a smaller tieback on each wall element.

In performing further optimizations, the modeler may also want to reduce the reinforcement in the wall elements since the demand of bending moments was reduced considerably when adding the tieback.

The benefits for using the tieback at a depth of 10 ft on a tall drilled-shaft wall is remarkable, as demonstrated in this example.

7.1.7 Input and Output Data Files for Example 7

Users can read Section 2.1.1 (7) of the User's Manual for reference on the location of placement of the input and output data files for the example files installed with this program. The default installation directory is the following: *(Root Drive) c:\Ensoft\PYwall2025-Examples*. The input data files for all examples presented in this manual are installed automatically with the program.

The input-data filenames for Example 7 are the following:

Example 7a - Drilled Shaft Wall in Sand without Tieback.py8d

Example 7b - Drilled Shaft Wall in Sand with Tieback.py8d

Example 7c - Drilled Shaft Wall in Sand with Tieback and Trapezoidal Loads.py8d

The output-data filenames for Example 7 are the following:

Example 7a - Drilled Shaft Wall in Sand without Tieback.py8o

Example 7b - Drilled Shaft Wall in Sand with Tieback.py8o

Example 7c - Drilled Shaft Wall in Sand with Tieback and Trapezoidal Loads.py8o

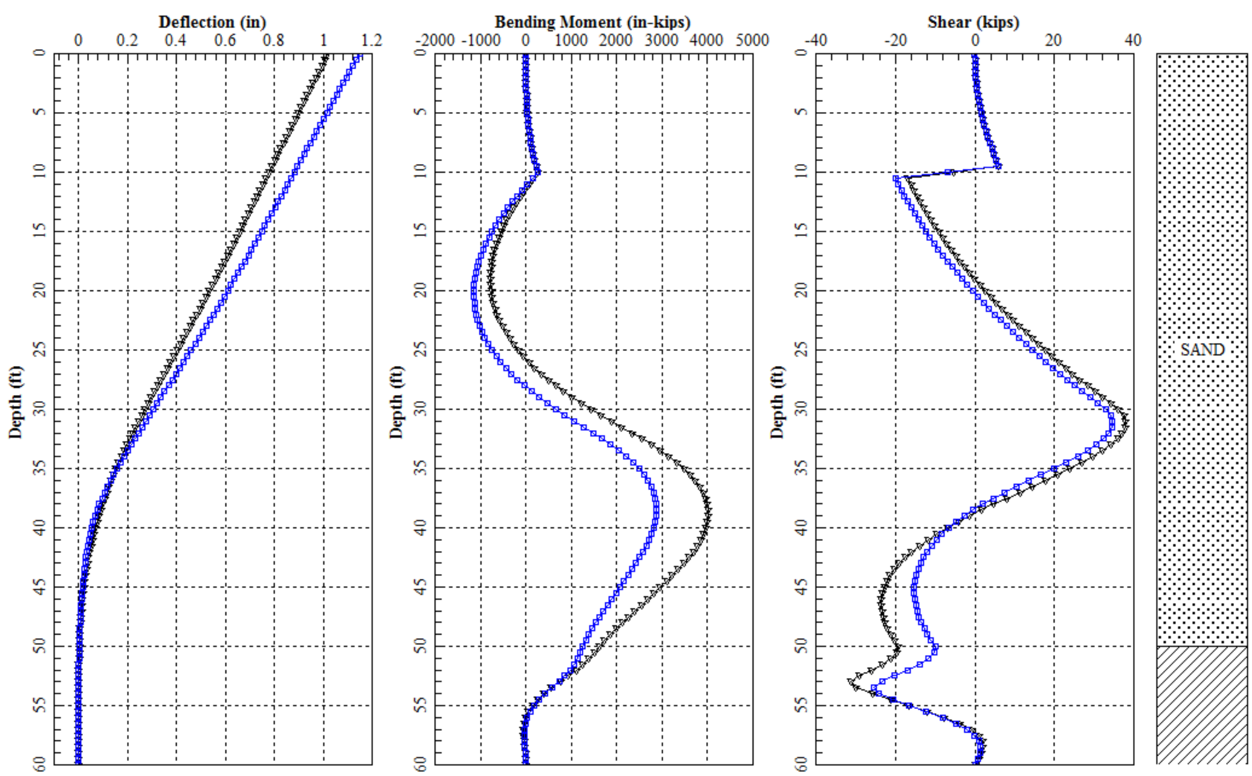


Figure 7.5 Deflection, Shear and Bending Moment Curves from the PYWALL Model for Example 7b With Tiebacks

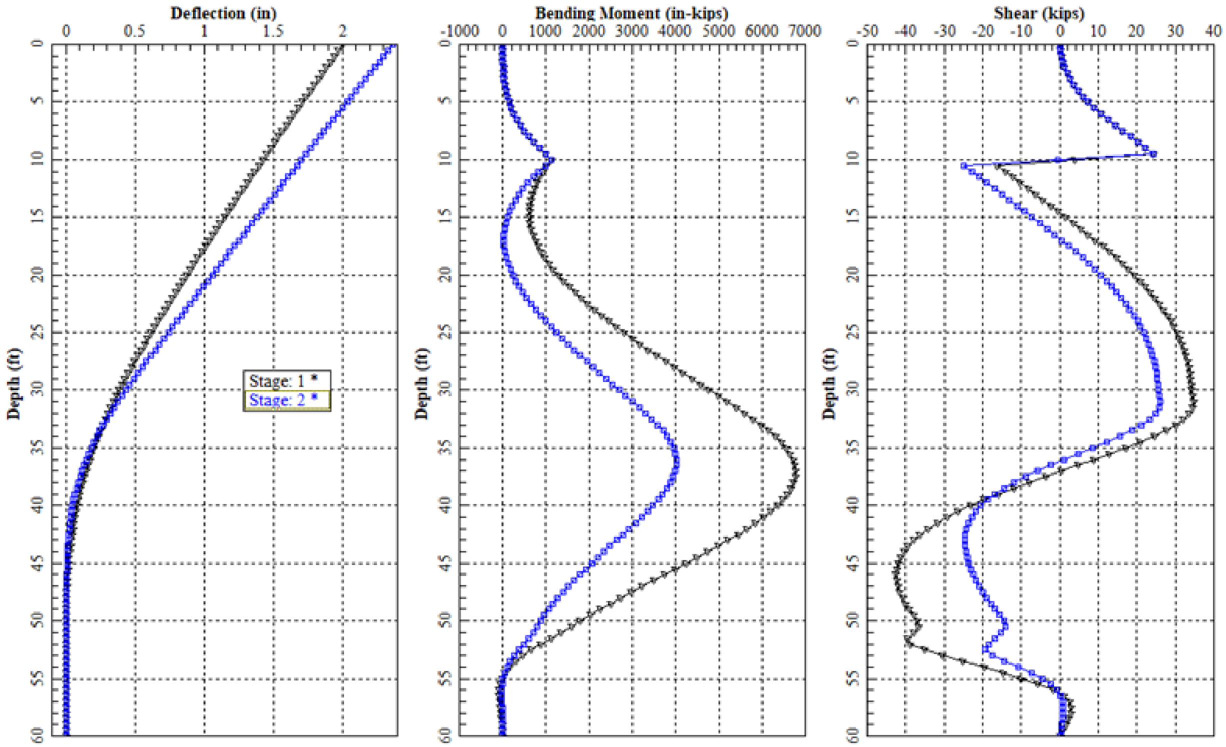


Figure 7.6 Deflection, Shear and Bending Moment Curves from the PYWALL Model for Example 7c With Tiebacks and Trapezoidal Distribution

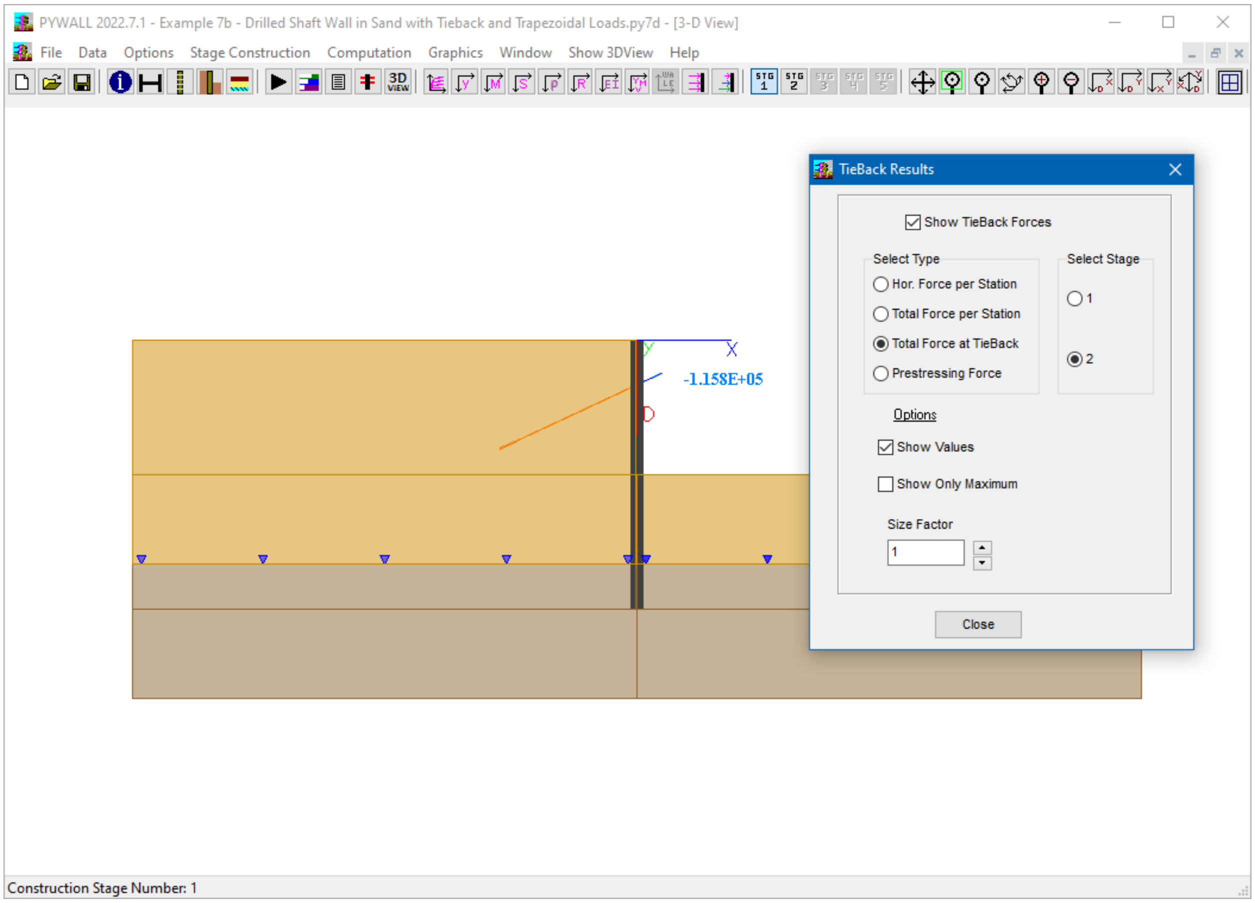


Figure 7.7 3D View with Tieback Reaction Force for Example 7c

Example 8. Lateral Springs for Struts and Soils on Braced Trench Excavation

8.1 Example 8 – Lateral Springs for Struts and Soils on Braced Trench Excavation

8.1.1 Problem Description

The model is illustrated in Figure 8.1, this example is intended to demonstrate the use of PYWALL to analyze a retaining system for a braced trench with user-specified springs representing the struts. The example also shows the usage of user-specified loading (calculated outside the PYWALL program) and user-specified soil resistances (also computed outside PYWALL).

The trench is 120 in. wide and is lined with continuous sheet piling. Wales are used so that soil pressure on each five-foot length of trench is transferred to timber struts located at the third points of the trench depth. The excavation depth is set to be 30 ft below the ground level. The sheet piles in this example have a total length of 40 ft with only 10 ft penetration below the excavation level. As indicated in Figure 8.1b, the flexural stiffness (F = EI) of the analyzed sheet-pile section in this model is provided as 5×10^9 lb-in^2, which is approximately equal to five times the flexural stiffness per foot of a PDA27 sheet pile.

8.1.2 Earth Pressures

Since they will be specified by the user, the PYWALL model for this example (filename: *Example 8 - Lateral Springs for Struts and Soil on Braced Trench Excavation.py8d*) requires entries for applied loading and resistances. For simplicity in the numerical entries on this model, the depth of sheet piling was divided into 80 six-inch increments, as indicted in the entries under the Data > Wall Properties dialog box.

The distribution of earth pressure can be calculated based on a triangular distribution of earth pressure on cantilevered walls. Applied lateral soil pressure is assumed to be as shown in Figure 8.1b, which was computed per 5-ft length of wall (similar to the flexural stiffness of the analyzed wall section that was also provided for a 5-ft length of wall). For this example, the active earth pressure from the top to the toe of the sheet pile was entered selecting Options > Earth Pressure Options > Earth-Pressure Specified by User and Data > Distributed Active Earth Pressure, as indicated in Figure 8.2.

8.1.3 Struts and Soil Resistances

Two rows of fir braces are assumed to be used to restrain the deflection into the trench excavation. The lateral stiffness (spring) of 2.67×10^5 lbs/in. is used for each lateral bracing support at depths of 10 ft and 20 ft. This value was computed based on the axial deformation of the single strut (EA/L) and on the proportion of that stiffness that corresponds to the longitudinal width of wall used for computation of applied soil loads and wall stiffness (per foot of wall) and the longitudinal spacing between struts (5 ft). A general formulation for computing the stiffness of the bracing strut is the following:

$$K_b' = (E_b A_b / L_b)\left(\frac{W_a}{S_b}\right)$$

Where:

K_b' = the equivalent stiffness of the bracing strut

E_b = Modulus of Elasticity of the bracing strut (approx. equal to 1,775,000 lb/in^2)

A_b = cross sectional area of the bracing strut (approx 9.5”x9.5” or 90 in^2)

L_b = length of each bracing strut (defined as 10 ft or 120 in)

W_a = longitudinal width of wall used for computation of applied soil loads (defined as 12 in)

S_b = longitudinal spacing between struts (defined as 5 ft or 60 in)

Therefore:

$$K_b' = \left(\frac{1{,}775{,}000 \text{ x } 9.5\text{x}9.5}{120}\right)\left(\frac{12}{60}\right) \sim 267{,}000 \text{ lb/in}$$

The lateral restraint provided by the soil against the embedded sheet piling is also provided in Figure 8.1b and simulated by a series of springs with stiffness varying in simple proportion to depth from zero at the bottom of the excavation to 12,500 lb/in at the tip of the sheet piles (this value represents the resistance for a 5-ft length of wall).

The resistances from the bracing struts and the soil are entered into PYWALL by selecting Options > Apply Lateral Springs and Rotation Restraints and Data > Lateral Springs and Rotational Restraints > Edit Spring Stiffness with the values shown in Figure 8.3. The top two entries, at 10 ft and 20 ft from top of wall, are for the bracing struts. The entries that follow, from 30 ft to 40 ft from top of wall, are for the soil springs.

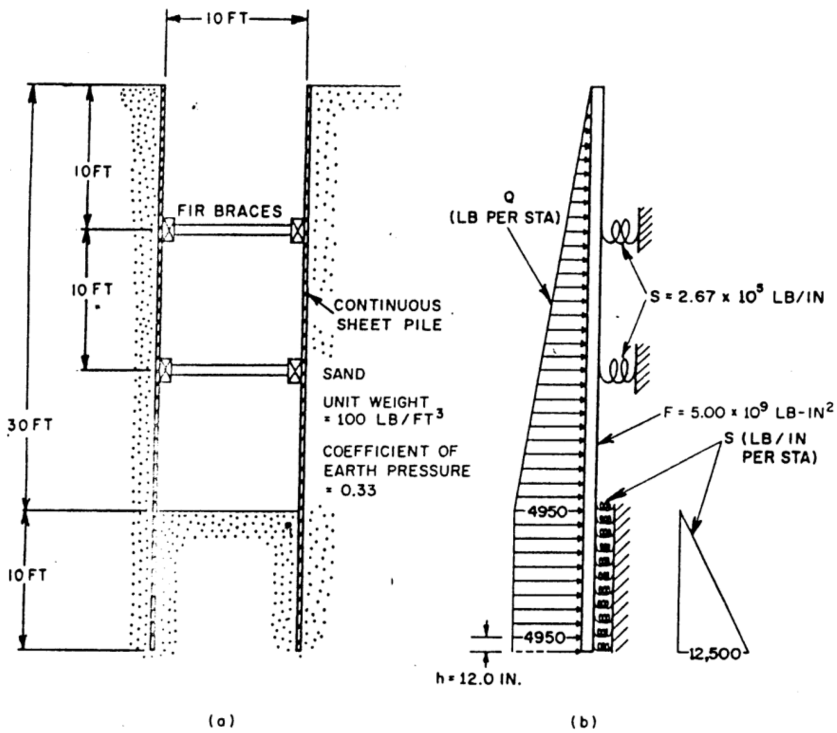


Figure 8.1 Model of Braced Trench Excavation

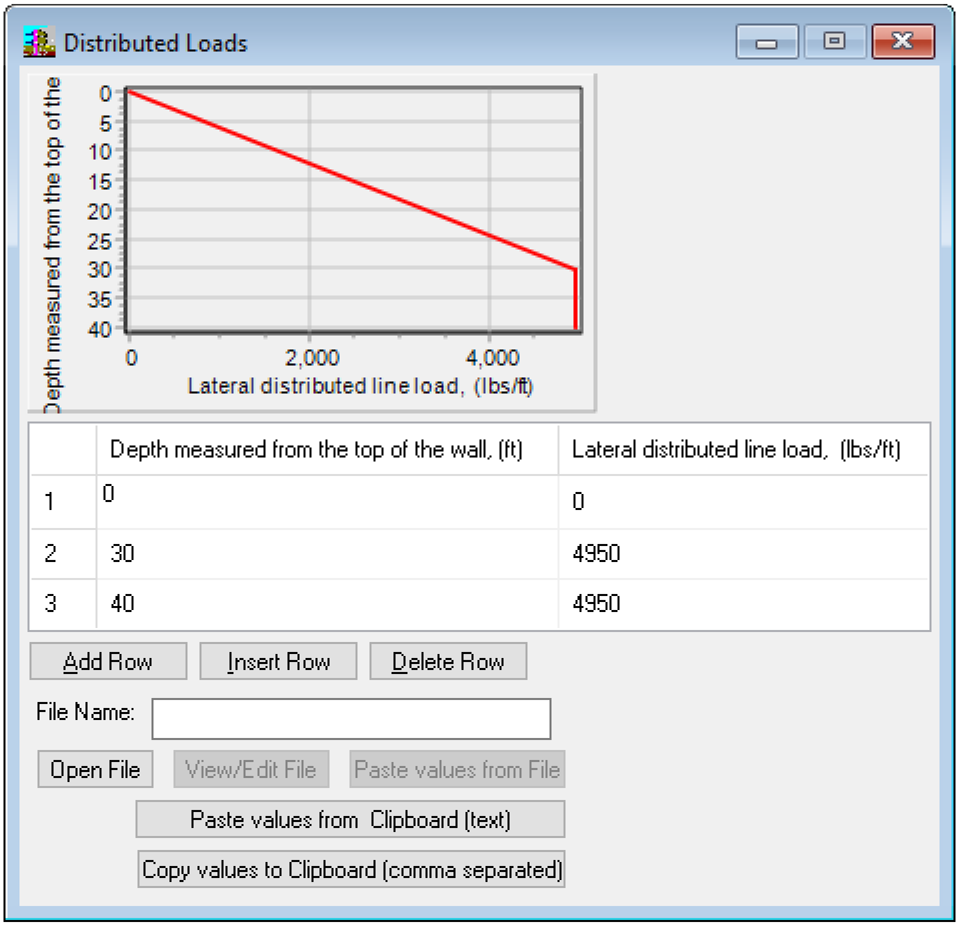


Figure 8.2 Applied Active Pressure on PYWALL Model of Braced Trench Excavation

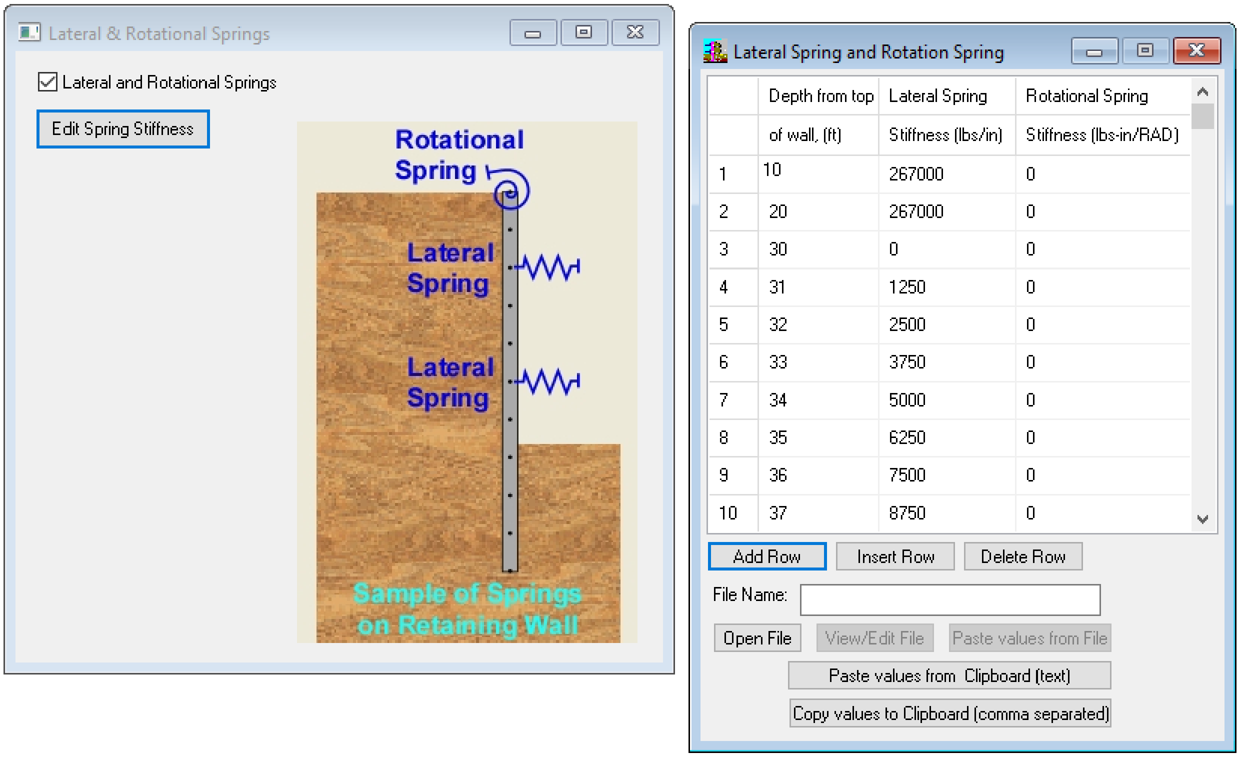


Figure 8.3 Applied Resistance Springs on PYWALL Model of Braced Trench Excavation

8.1.4 Numerical Solution from PYWALL

The solution from PYWALL in terms of curves of deflected shape, bending moment and shear distribution with depth is presented in Figure 8.4. Maximum deflection of about 1.3 inches is predicted for the region around the bottom of the trench. The lower brace at 20-ft from the top is compressed by about 0.21 inches.

The compression forces on the struts or braces can be observed in the Graphics > Net Force vs Depth shown in Figure 8.5. At 20-ft depth the lower brace has a negative reaction of -55,820 lb which added to the positive 1,640 lb/sta provides a net reaction of 57,460 lbs. A similar computation yields about 12,540 lbs on the upper brace at 10-ft depth.

The two largest values of bending moment are predicted near the point of greatest sheet piling deflection near the bottom of the trench and at the lower trench brace (20 ft). The selection of a stiffer sheet piling or an increase of the braces may help to reduce the predicted deflections.

8.1.5 Input and Output Data Files for Example 8

Users can read Section 2.1.1 (7) of the User's Manual for reference on the location of placement of the input and output data files for the example files installed with this program. The default installation directory is the following: *(Root Drive) c:\Ensoft\PYwall2025-Examples*. The input data files for all examples presented in this manual are installed automatically with the program.

The input-data filenames for Example 8 are the following:

Example 8 - Lateral Springs for Struts and Soil on Braced Trench Excavation.py8d

The output-data filenames for Example 8 are the following:

Example 8 - Lateral Springs for Struts and Soil on Braced Trench Excavation.py8o

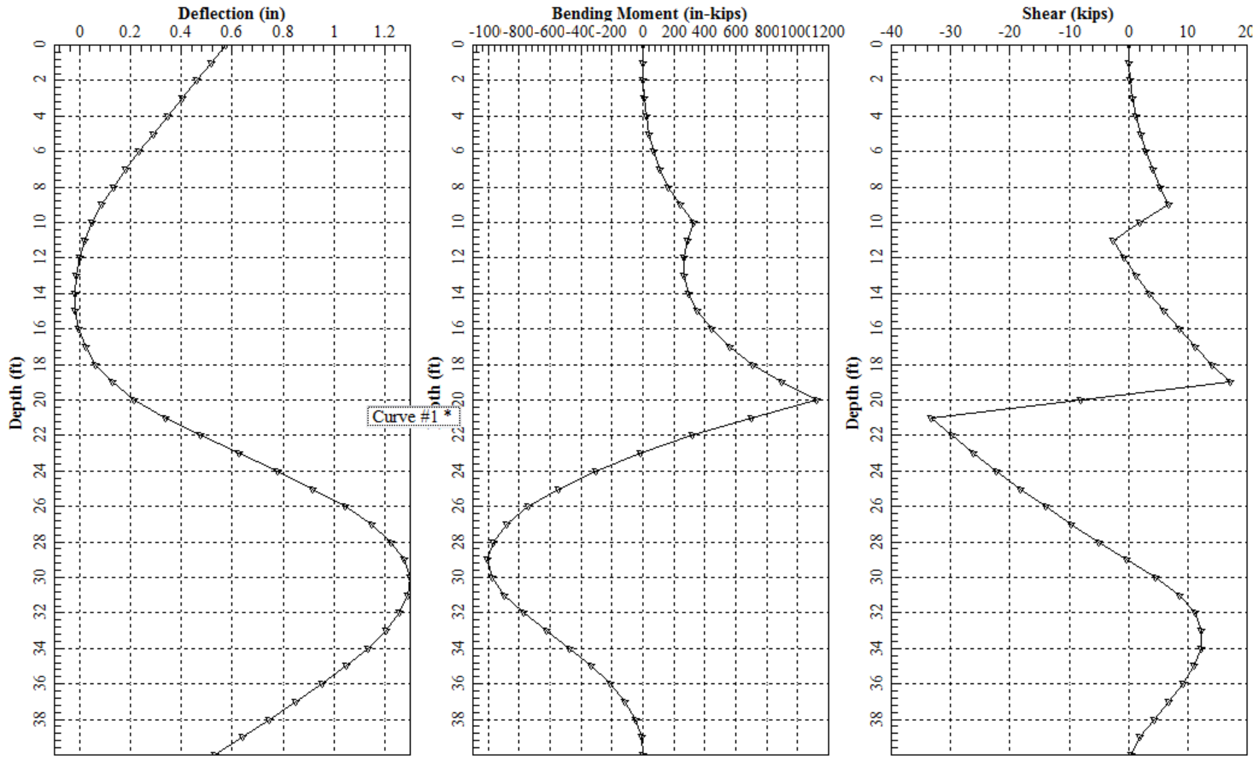


Figure 8.4 Deflection, Shear and Bending Moment Curves from the PYWALL Model for the Braced Trench in Example 8

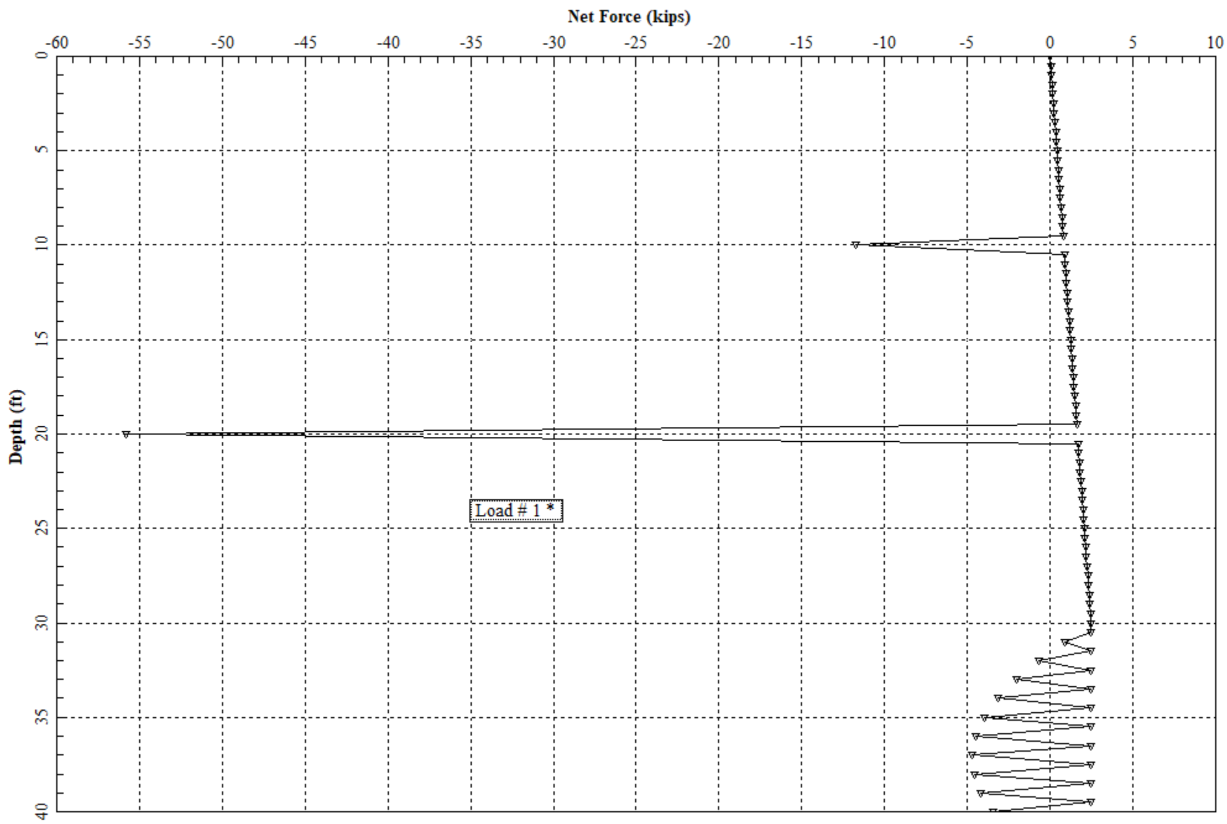


Figure 8.5 Net Force vs. Depth for Example 8

Example 9. User-Specified Lateral Spring, External *p-y* Curves & External Loads on Sheet-Pile Wall

9.1 Example 9 – User-Specified Lateral Spring, External *p-y* Curves & External Loads on Sheet-Pile Wall

9.1.1 Problem Description

This example is included to show a number of features from PYWALL: i) the input of user-specified (external) *p-y* curves, ii) the use of a lateral spring (elastic) to model the tieback anchor and iii) the input of user-specified distributed loads (active pressure). The example is also used to compare the results of the user-specified inputs with those that are automatically calculated by PYWALL for the same model.

The data for this example was detailed in Reese, et al (2011) as originated from Peck, et al. (1974), mentioning that units from the original publication were converted from English to SI units. Figure 9.1 shows the geometry of the problem and the earth pressure distribution as calculated using the Rankine equations (assuming that the wall has deflected to allow the mobilization of the minimum active pressures). The earth-pressure diagram includes the effects of the surcharge load and the sudden draw down of the water table. The sheet pile for the continuous wall to be analyzed is also presented in Figure 9.1, along with the distribution of earth pressures on the wall.

The required data are the maximum bending moment in the sheet pile wall, the load on the anchor rod, and the factor of safety against a deep-seated failure. The anchor rods are spaced 2.44 m apart and wales are assumed to be used to transfer the load from the wall to the anchors. The actual wales will not be analyzed in this example. The sheet pile that was selected for analysis is a PDA27, a classification available in the United States. The dimensions of the cross section of the sheet pile in Figure 9.1 are given in English units, with other properties provided in SI Units per meter length of sheet pile, including the Moment of Inertia = 54,347,000 mm^4 ($5.437x10^{-5}$ m^4) which is inputted into PYWALL. The steel is ASTM A-572 GR 50 with a minimum yield stress of 345,000 kPa (50,000 psi) and modulus of elasticity of $200x10^6$ kN/m^2.

9.1.2 Soil Resistance (*p-y*) Curves and Earth Pressure

For the user-specified *p-y* curves, if wall friction is neglected, a possible solution is to formulate two sets of *p-y* curves, one set for behind the wall where the overburden exists and the other set in front of the wall below the dredge line. Haliburton (1968) presented such an approach; however, experimental evidence is lacking on the nonlinear variation of these separate sets of curves as a function of the displacement of the wall. Therefore, simplified assumptions are provided in this example so as to allow the commercially available *p-y* curves to be modified in a rational manner.

For the simplified user-specified *p-y* curves, the following assumptions are made: (1) the active-pressure condition will exist in the soil behind the wall (retained soil) above the dredge line; (2) the *p-y* curves for a continuous wall are the same as those of a widely-spaced pile except that the *p*-values are reduced by one-half (to account for group effects); and (3) the *p-y* curves for the wall below the dredge line may be constructed initially by using the recommended values on the passive-pressure side (inside the wall) and subtracting from the ultimate *p*-values of the active pressure (outside the wall) due to the overburden pressure.

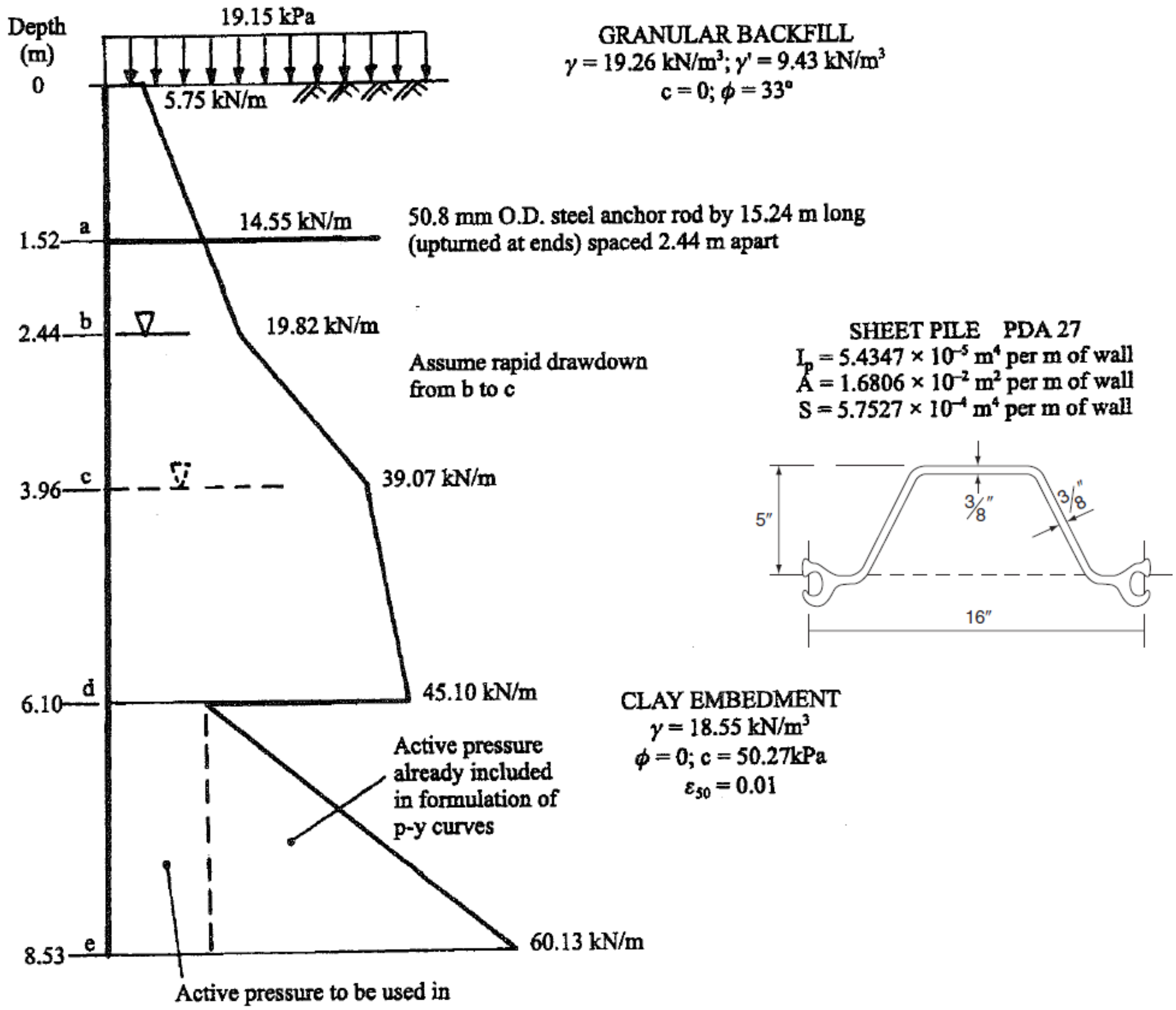


Figure 9.1 Earth Pressure and Sheet-Pile Section of Anchored Bulkhead in Example 9a

The *p-y* curves already include the effect of active pressure for ground-level conditions. As discussed below, the previously mentioned assumption (3) may be modified as noted.

The *p-y* curves for the clay below the dredge line were computed in Reese, et al (2011) with the following modifications: the ultimate *p*-values for the clay, with the inward movement of the wall, are reduced by the active pressure behind the wall, due to the overburden above the dredge line. The ultimate values of *p* were not reduced initially for the wall as related to the pile with the view that checks will be made after reviewing the results. The resulting *p-y* curves from Reese, et al (2011) are shown in Table 9.1.

The PYWALL model uses the Options > Apply User-Specified p-y Curves (External) and the user can enter the curves under Data > Sets of External p-y Curves at depths of 6.10 m and 8.53 m from top of wall. The PYWALL program interpolates linearly the *p-y* curves between those two input depths.

0 m Below the Dredge Line	2.43 m Below the Dredge Line

y, m	p, kN/m	y, m	p, kN/m
0	0	0	0
0.0125	24.0	0.0125	56.9
0.025	30.2	0.025	71.7
0.05	38.1	0.05	90.4
0.1	48.0	0.1	113.9
0.2	60.5	0.2	143.4
10	60.5	10	143.4

Table 9.1 *p-y* Curves for the Soil below the Dredge Line (from Reese, et al. 2011)

9.1.3 Spring Stiffness of Anchor Rod

The anchor rods have a diameter of 50.8 mm, a length of 15.24 m and spacing at 2.44 m apart along the length of the wall. The anchor resistance can be modeled in PYWALL by specifying the stiffness of an equivalent lateral spring at the anchor location. The engineer must compute the stiffness of the equivalent spring by relating the deflection of the anchor rod to the load in the anchor rod using the $d = PL/AE$ equation, where d is the deflection of the anchor rod in m, P is the load in the rod in kN, L is the length of the rod in m (15.24 m), A is the cross sectional area of the steel rod in m^2 (0.002027 m^2), and E is the modulus of elasticity of steel (200,000,000 kN/m^2).

$$Stiffness = P/d = AE/L = (0.002027 \times 200{,}000{,}000)/15.24 = 26600\ kN/m$$

However, since the anchors are located 2.44 meters apart, the user must divide the stiffness by a factor equal to the anchor spacing to obtain the distributed stiffness, since wales will be used in the real wall to distribute the load along its length. The stiffness of the lateral spring needs to be applied at the closest nodal point, which represents the resistance force from the anchor to that increment. The designer must multiply the anchor stiffness by 1 meter, which is the width of the wall section that is being analyzed, and then divide by the spacing of the anchors. The equivalent anchor stiffness that the designer needs to input into the program is:

$$Equivalent\ stiffness = (26600\ (kN/m) * 1\ (m))/(2.44\ (m)) = 10900\ kN/m$$

The PYWALL model uses the Options > Apply Lateral Springs and Rotational Restraints and the user can enter the equivalent anchor stiffness under Data > Lateral Springs and Rotational Restraints > Edit Spring Stiffness at 1.505 m from top of wall.

9.1.4 Numerical Solution from PYWALL for Model with User-Inputted Data

The drilled-shaft model solved by PYWALL for the user-inputted data of soil springs, anchor spring and active pressure is included in filename: *Example 9a - User-Specified Anchor Spring, p-y Curves and Loads on Sheet Pile Wall.py8d*. The computed wall behavior based on the earth pressure, linear or nonlinear flexural rigidity of drilled shaft, and the nonlinear *p-y* curves is presented in Figure 9.2.

The maximum moment in the sheet-pile wall is 127 kN-m, yielding a bending stress (M_{max}/Section Modulus per meter) of 127/0.00057527 = 220,766 kPa, which is substantially less than the 345,000 kPa at minimum yield for ASTM A572 GR 50. For comparison, the corresponding solution employing static equilibrium gives a bending stress of 171,700 kPa.

The computed force in the anchor rod can be obtained in various ways: i) by checking the lateral movement at the anchor rod location (0.00863 m at depth of 1.505 m) and multiplying against the lateral spring stiffness (0.00863 x 10900 = 94.1 kN); ii) by checking the change in net reaction at the depth of the anchor rod (0.73 + 93.3 = 94.0 kN); or iii) by checking the result at the end of the output text file for Station 30 (Node 30 at 1.505 m) which indicates 94.1 kN.

Notice that the anchor rod force is provided by PYWALL for the modeled one-meter length/width of the wall. The force in the anchor rod for a spacing of 2.44 m is 229.6 kN (94.1 x 2.44), yielding a tensile stress of 113,300 kPa ($229.6/3.14 \times 0.0254^2$), which is a nominal value. The corresponding force computed by statics was 277 kN.

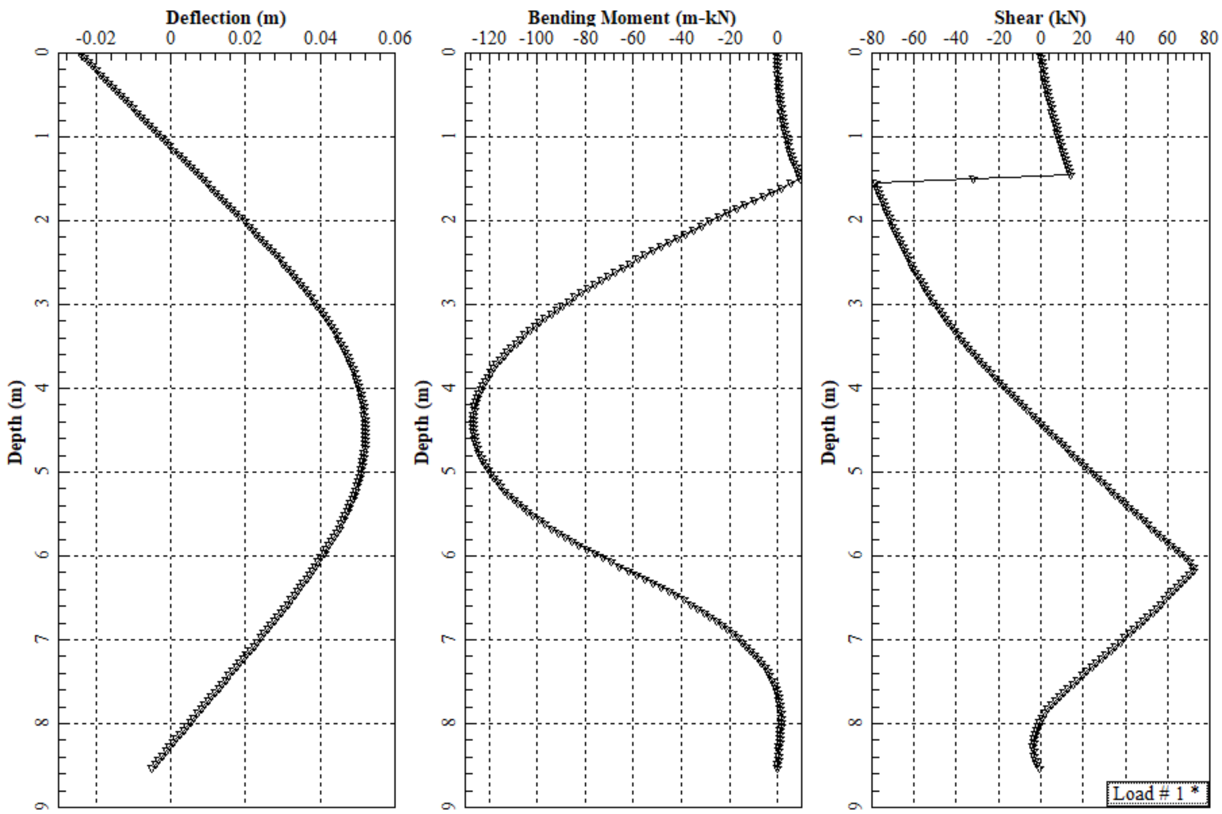


Figure 9.2 Deflection, Bending Moment and Shear Curves from the PYWALL Model for Example 9a with User-Inputted Data

9.1.5 PYWALL Model with Internal Data Computations

A model (*Example 9b - SheetPile Wall with Loads-Resistance-Anchor from PYWALL.py8d*) was prepared for the same sheet-pile wall in Example 9a but using the PYWALL features for internal computations of nonlinear soil springs, active earth pressure in triangular distribution and for calculating anchor stiffness. The new model incorporated the following options from PYWALL

Options > Earth-Pressure Options > Generate Triangular-Distribution Earth Pressure

Options > Generate p-y Curves

Options > Apply Tieback (Ground Anchor)

The various components that contribute to the Total Earth Pressure computed internally by PYWALL for Example 9b are shown in Figure 9.3. The program computed: 5.65 kN/m at ground surface, 19.2 kN/m at 2.4 m from top of wall, 38.7 kN/m at 3.96 m from top of wall, 44.3 kN/m at 6.10 m from top of wall and a constant 15.1 kN/m for depths below the excavation. These values are similar to those in Figure 9.1 for Example 9a.

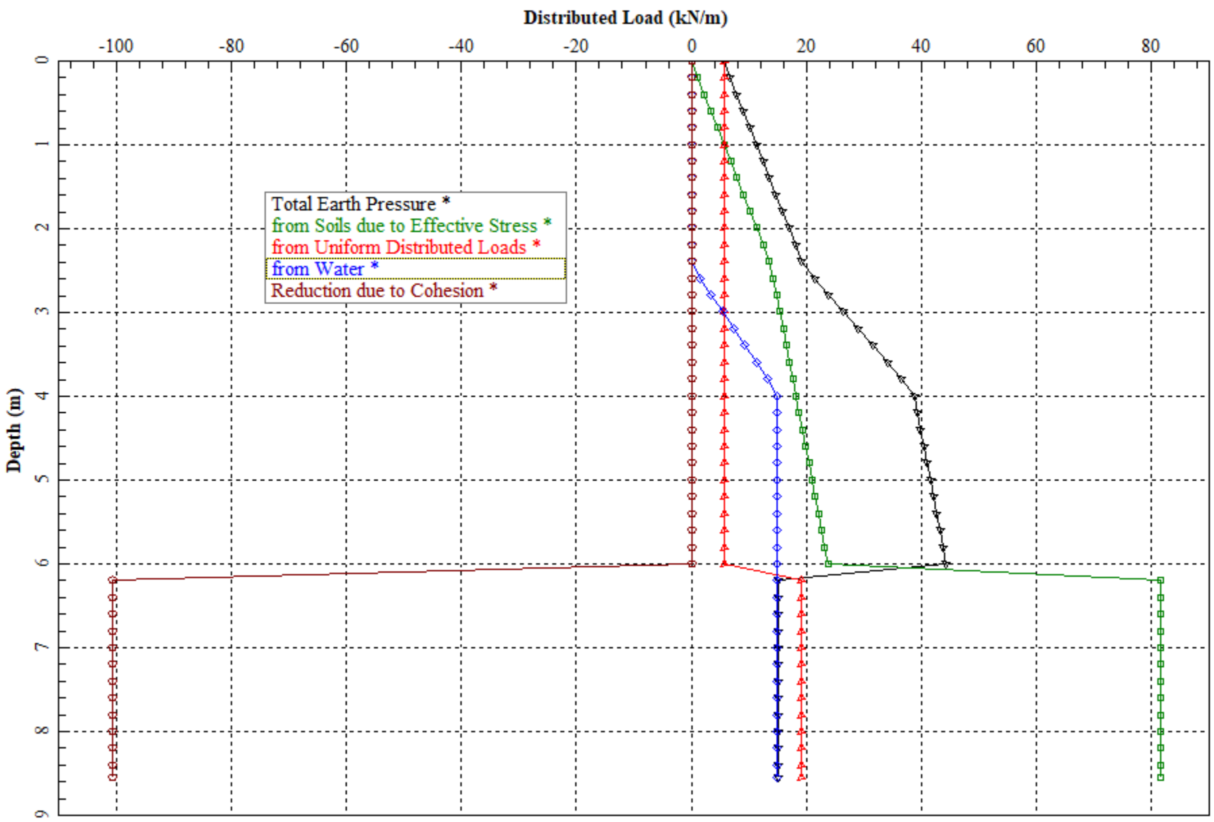


Figure 9.3 Combined Earth Pressures from the PYWALL Model for Example 9b with Automatic Internal Data Computations

The computed wall behavior based on the earth pressure, linear or nonlinear flexural rigidity of drilled shaft, and the nonlinear *p-y* curves is presented in Figure 9.4. The computed values are similar but slightly smaller than those previously computed (Figure 9.2) using the approximated user-inputted data for active loads, soil response curves and anchor response.

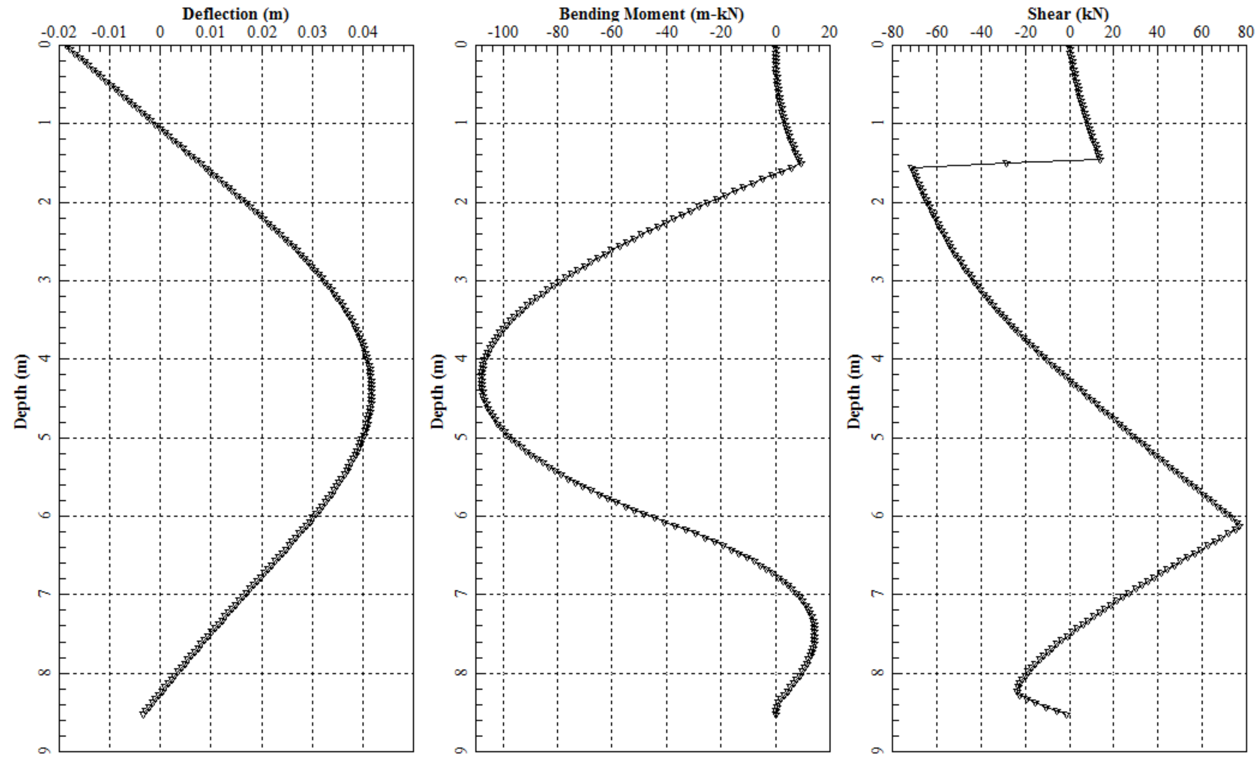


Figure 9.4 Deflection, Bending Moment and Shear Curves from the PYWALL Model for Example 9b with Automatic Internal Data Computations

9.1.6 PYWALL Model with Factored Loads

The soil pressures against the wall were increased by factors to find the loading that would cause yielding of the steel in the sheet-pile wall. Load factors for the soil pressures can be easily incorporated into the previous model for Example 9b under Data > Wall Configuration > Modification Factor for Active Earth Pressure.

The new model *Example 9c - SheetPile Wall with Factored Loads.py8d* with a multiplier of 1.6 was found to generate a maximum bending moment of 193.4 kN/m that is very close to yielding on the sheet pile. The examination of the results of the computation with the modest load factor gives further insight into the functioning of the wall.

9.1.7 Hand Checks for Deep-Seated Failure

Following the procedure suggested by Peck, et al. (1974) a deep-seated failure will occur if the vertical pressure behind the wall at the level of the dredge line is greater than the bearing capacity of the clay inside the wall. The vertical pressure behind the wall is equal to 19.15 + (6.10)(19.26) - (6.10-3.96)(9.80) = 115.66 kN/m^2. The bearing capacity of the clay may be taken at 5c or (5)(50.27) = 251.35 kN/m^2. The factor of safety against a deep-seated failure, then, is (251.35)/(115.66) = 2.17. Thus, the computations show the wall to be safe against a deep-seated failure.

9.1.8 Concluding Comments for Example 9

Necessary input for the PYWALL computer program can be prepared easily to solve the problem of the anchored bulkhead. While the equations of static equilibrium can be employed to obtain reasonable results for the case of a single anchor rod, a solution using statics for multiple anchors would be unsatisfactory. The solution provided by PYWALL gives the designer considerable insight into the actual response of the wall to earth pressures.

The designer normally applies a factor of safety to the distribution of the earth pressures. The factor of safety can be used to account for uncertainty in the earth pressures. With PYWALL the user can find rather quickly the maximum load factor that will initiate a structural failure in the wall elements.

9.1.9 Input and Output Data Files for Example 9

Users can read Section 2.1.1 (7) of the User's Manual for reference on the location of placement of the input and output data files for the example files installed with this program. The default installation directory is the following: *(Root Drive) c:\Ensoft\PYwall2025-Examples*. The input data files for all examples presented in this manual are installed automatically with the program.

The input-data filenames for Example 9 are the following:

Example 9a - User-Specified Anchor Spring, p-y Curves and Loads on Sheet Pile Wall.py8d

Example 9b - SheetPile Wall with Loads-Resistance-Anchor from PYWALL.py8d

Example 9c - SheetPile Wall with Factored Loads.py8d

The output-data filenames for Example 9 are the following:

Example 9a - User-Specified Anchor Spring, p-y Curves and Loads on Sheet Pile Wall.py8o

Example 9b - SheetPile Wall with Loads-Resistance-Anchor from PYWALL.py8o

Example 9c - SheetPile Wall with Factored Loads.py8o

Example 10. Construction Stages on Drilled-Shaft Wall

10.1 Example 10 – Construction Stages on Drilled-Shaft Wall

10.1.1 Problem Description

This example is made after published technical information regarding the design of retaining walls for the 16-km long trench section of the Alameda Corridor project in Los Angeles. That design was controlled by the wall stiffness necessary to meet a specified wall deflection criteria and by the minimum depth of shaft embedment required to avoid kick-out failure during excavation (Roth et al. 2001).

The middle section of the Alameda Corridor consists of a 10-m deep, 15.5-m wide trench with two rail tracks and a service road running at the bottom. The trench walls consist of 0.9-m diameter, cast-in-place concrete drilled shafts spaced 1.2 m on center, with shotcrete applied between the piles. The total length of the cast-in-place drilled shaft is 15.6 m from the ground surface. The trench walls were supported by permanent precast concrete struts at the top and a concrete slab at the bottom. In order to speed up construction, the trench was to be excavated without temporary struts. Figure 10.1 shows sketches of the construction process represented in four simplified stages.

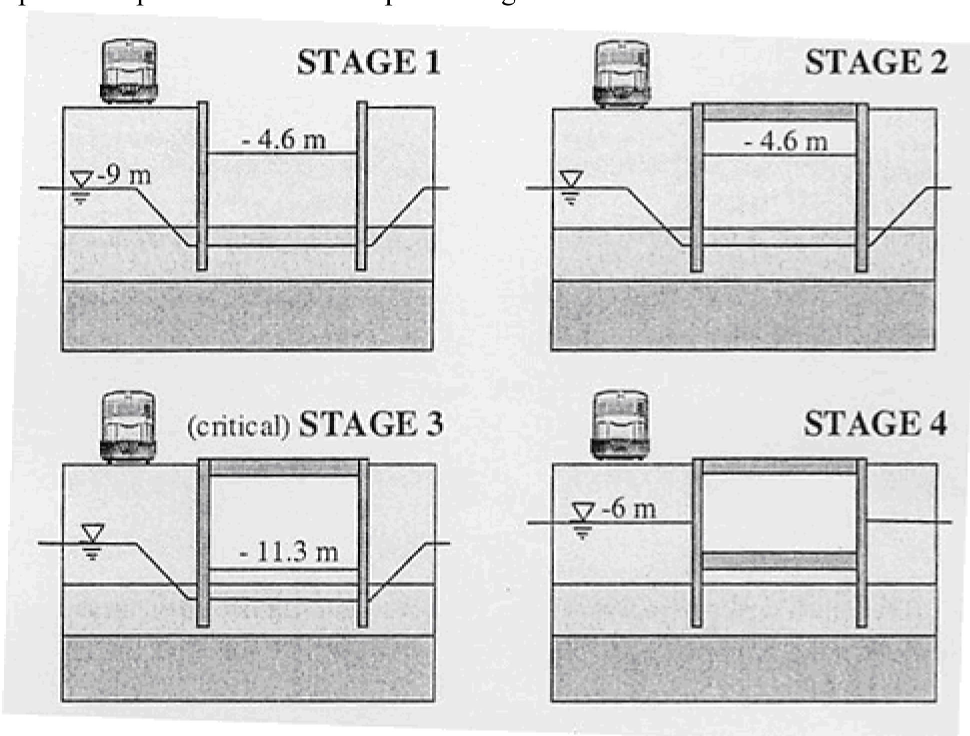


Figure 10.1 Construction Stages of Example 10

10.1.2 Soil and Structure Properties

The soil profiles at the North and South sides are presented in Figure 10.2. Typically, the top 9 to 12 m of the subsurface consists of a loose to medium dense silt underlain by approximately 5 m of medium dense sand and silty sand. Very dense sand was encountered at depth of 14 m (South side) to 17 m (North side). The distribution of earth pressure can be calculated based on a triangular distribution of earth pressure. The moment of inertia of the drilled shaft with 0.9-m in outside diameter was provided to be 0.029 m^4 and the modulus of the concrete is approximately 21 MPa.

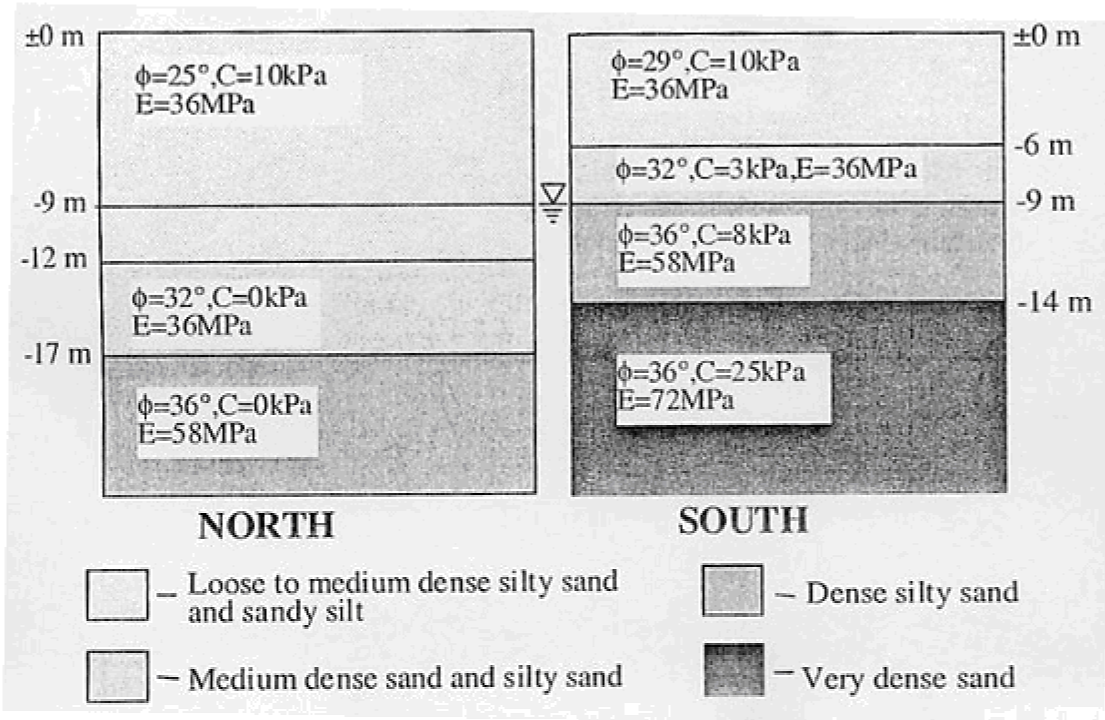


Figure 10.2 Soil Profiles for Example 10

10.1.3 Initial Excavation for Stage 1

As shown in Figure 10.1, during the construction at Stage 1 the trench was excavated to 4.6 m without any temporary struts for support. This system is similar to a cantilever wall and the deflection at the top was recorded as approximately 2 cm.

A PYWALL model (input data labeled: *Example 10a - Construction Stages on Drilled-Shaft Wall - Stages 1N and 1S.py8d*) was prepared for Stage 1 alone but using two dummy "construction" stages to represent the different soil profiles at the North and South locations. In the model/data file, the North Profile from Figure 10.2 is represented as Stage 1 and the South Profile as Stage 2.

The triangular earth pressure was computed by PYWALL internally based on the distribution of Rankine's active earth pressure. The pressures computed internally by PYWALL are displayed in Figure 10.3. The active earth pressure below the excavation (below 4.6 m) is reduced since the pressure only applies to the pile diameter of 0.9 m (instead of the pile spacing of 1.2 m above the excavation). There are also reductions at other depths representing the changes in soil properties. Notice that the user may desire to increase the pressure below the excavation with the argument that soil arching may still produce a higher load on the embedded piles.

The predicted deflections from the PYWALL for Stage 1 for the North and South soil profiles are displayed in Figure 10.4. Notice that the predicted maximum deflection of 0.025 m at top of wall for the South Soil Profile is in close proximity to the deflection of 0.02 m that was reportedly measured in Stage 1. The designer should keep in mind that the field measurements of wall deflection may be difficult to achieve and perhaps lack of sufficient precision. In any event, the close prediction from the PYWALL model is indeed very remarkable.

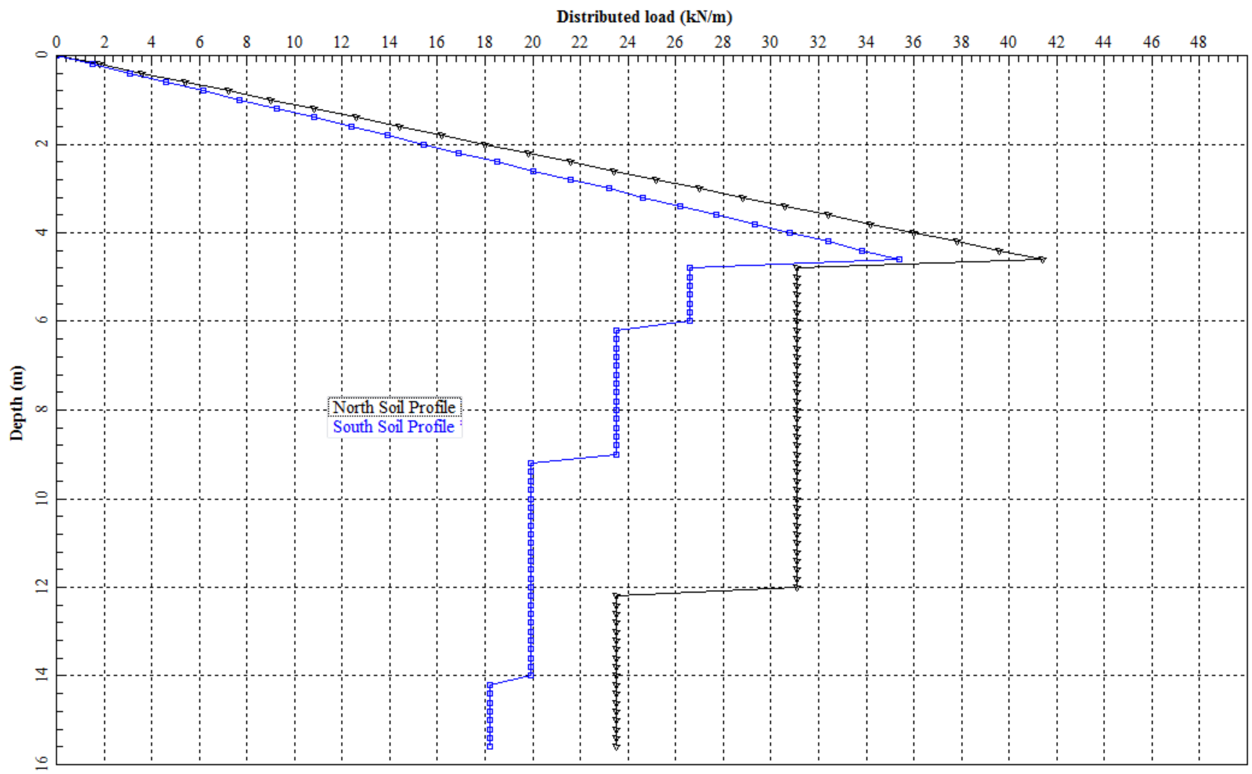


Figure 10.3 Active Earth Pressures from PYWALL for Stage 1 in Example 10a

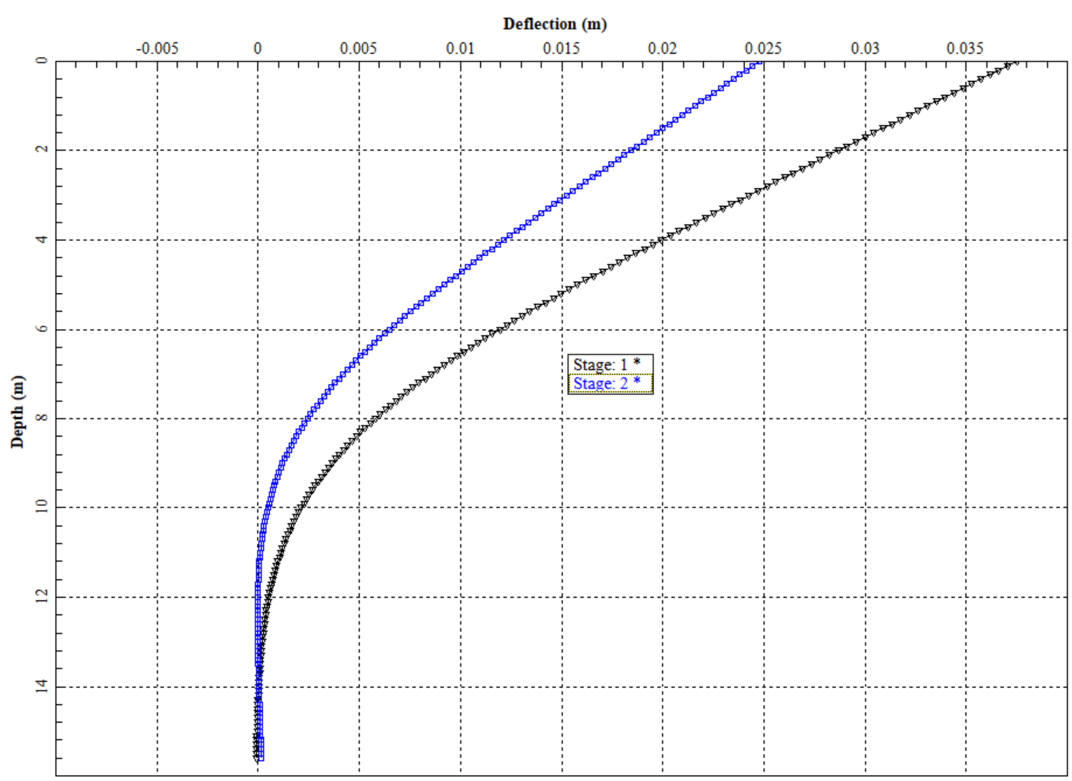


Figure 10.4 Deflection vs Depth for North and South Soil Profiles - Stage 1 (Example 10a)

10.1.4 Construction Stages and Modeling

A permanent strut was installed in Stage 2 at the top of the wall after the excavation reached 4.6 meters. After Stage 2 the trench was excavated to the depth of 11.3 m before the bottom slab was poured. The measured deflections for the excavations at 7.6 m, 9.1 m, 10.1 m, and 11.3 m were reported by the design team as shown in Figure 10.5.

PYWALL (input data labeled: *Example 10b - Construction Stages on Drilled-Shaft Wall.py8d*) was used to model the wall behavior during excavations at 4.6 m (Stage 1), 7.6 m (Stage 2), 9.1 m (Stage 3) and 11.3 m (Stage 4). The lateral strut was installed at the top of the wall model using Options > Apply Restraint on Deflection or Slope with a set deflection of 0.02 m at node zero (top of wall) under Data > Boundary Condition on Deflection or Slope for Stages 2 through 4.

The predicted deflection profiles from the PYWALL model are shown in Figure 10.6. These deflections from PYWALL are in general agreement with the reported measured data from the designers. The critical stage 4 was when the excavation reached 11.3 m and before pouring of the concrete slab. The measured maximum deflection was reported as approximately 8 cm at a depth of 7 meters, compared to the prediction of 6 cm at 7-m depth from the PYWALL model.

Several factors may contribute for the small differences between predicted and measured deflections:

- The bending stiffness (EI) of the concrete shaft may need to be reduced due to the areas of tensile concrete crack under large bending moments. Such a response from the piles would result in an increase of the predicted deflections. PYWALL can be used to model the nonlinear behavior of the pile but the reinforcement layout (vertical and lateral for confinement) and actual measured concrete strength on the piles are not available.
- The active earth pressure extended below the excavation line has a higher impact for deep excavations and particularly when the embedded portion of the shaft moves laterally below the excavation, as it was predicted (and somehow also measured) for this model.
- Field measurements of lateral wall movements may not be very precise and it is not known the methods used for this specific project.

Despite the factors mentioned above, the predictions from the PYWALL model are very close to those of the measurements. It is obvious that PYWALL provides a rational tool for engineers to analyze retaining walls in a practical manner with the ability to quickly evaluate a multitude of possible scenarios.

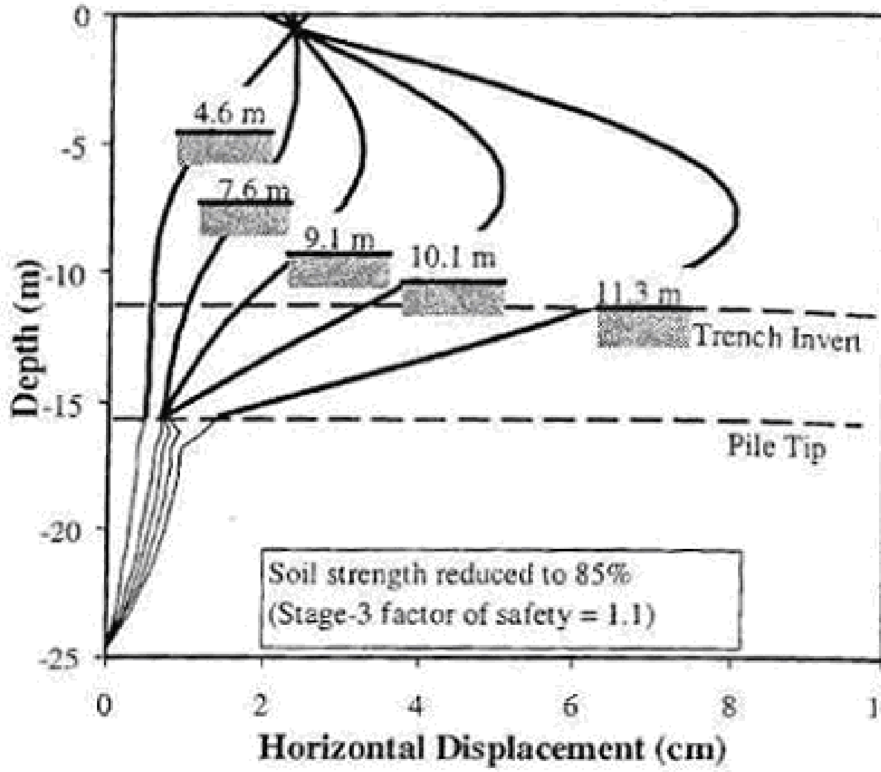


Figure 10.5 Wall Deflection from Field Measurements of Example 10

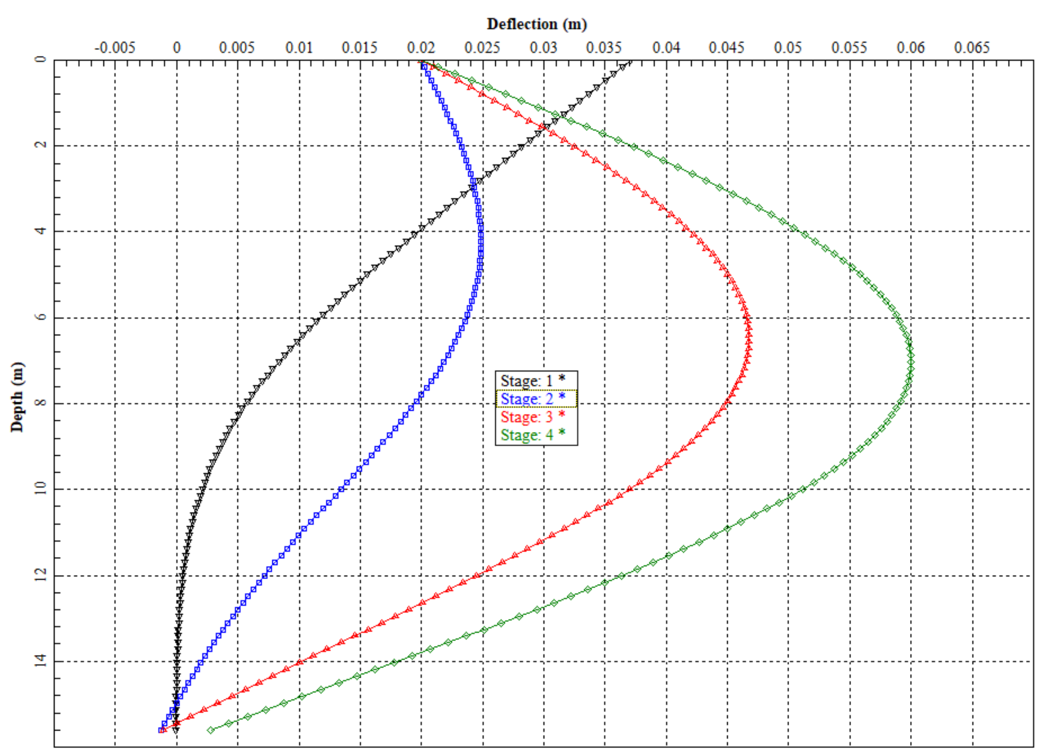


Figure 10.6 Predicted Deflection vs Depth at Different Stages for Example 10b

10.1.5 Input and Output Data Files for Example 10

Users can read Section 2.1.1 (7) of the User's Manual for reference on the location of placement of the input and output data files for the example files installed with this program. The default installation directory is the following: *(Root Drive) c:\Ensoft\PYwall2025-Examples*. The input data files for all examples presented in this manual are installed automatically with the program.

The input-data filenames for Example 10 are the following:

Example 10a - Construction Stages on Drilled-Shaft Wall - Stages 1N and 1S.py8d

Example 10b - Construction Stages on Drilled-Shaft Wall.py8d

The output-data filenames for Example 10 are the following:

Example 10a - Construction Stages on Drilled-Shaft Wall - Stages 1N and 1S.py8o

Example 10b - Construction Stages on Drilled-Shaft Wall.py8o

Example 11. Anchored Soldier Pile Wall with Trapezoidal Soil Pressure

11.1 Example 11 – Anchored Soldier Pile Wall with Trapezoidal Soil Pressure

11.1.1 Problem Description

This example is included as an exercise for the analysis of an anchored retaining wall based on trapezoidal soil-pressure distribution. Figure 11.1 shows the problem geometry, in which a cut-and-cover excavation is required for construction of a roadway tunnel. The final excavation depth will be 8 m and the excavation will be supported by a soldier pile and timber lagging wall incorporating two levels of anchors. The high water table is below the bedrock surface, and the vehicular live load surcharge (LS = q_s= 12 kN/m^2) on the backfill is applied as shown in the figure.

During the subsurface exploration, it was determined that the foundation soils consist of medium dense sand to a depth of 13 m below the ground surface, underlain by hard, sandstone bedrock. The anchors will derive their capacity wholly within the medium dense sand. Only the final excavation stage is analyzed in this example.

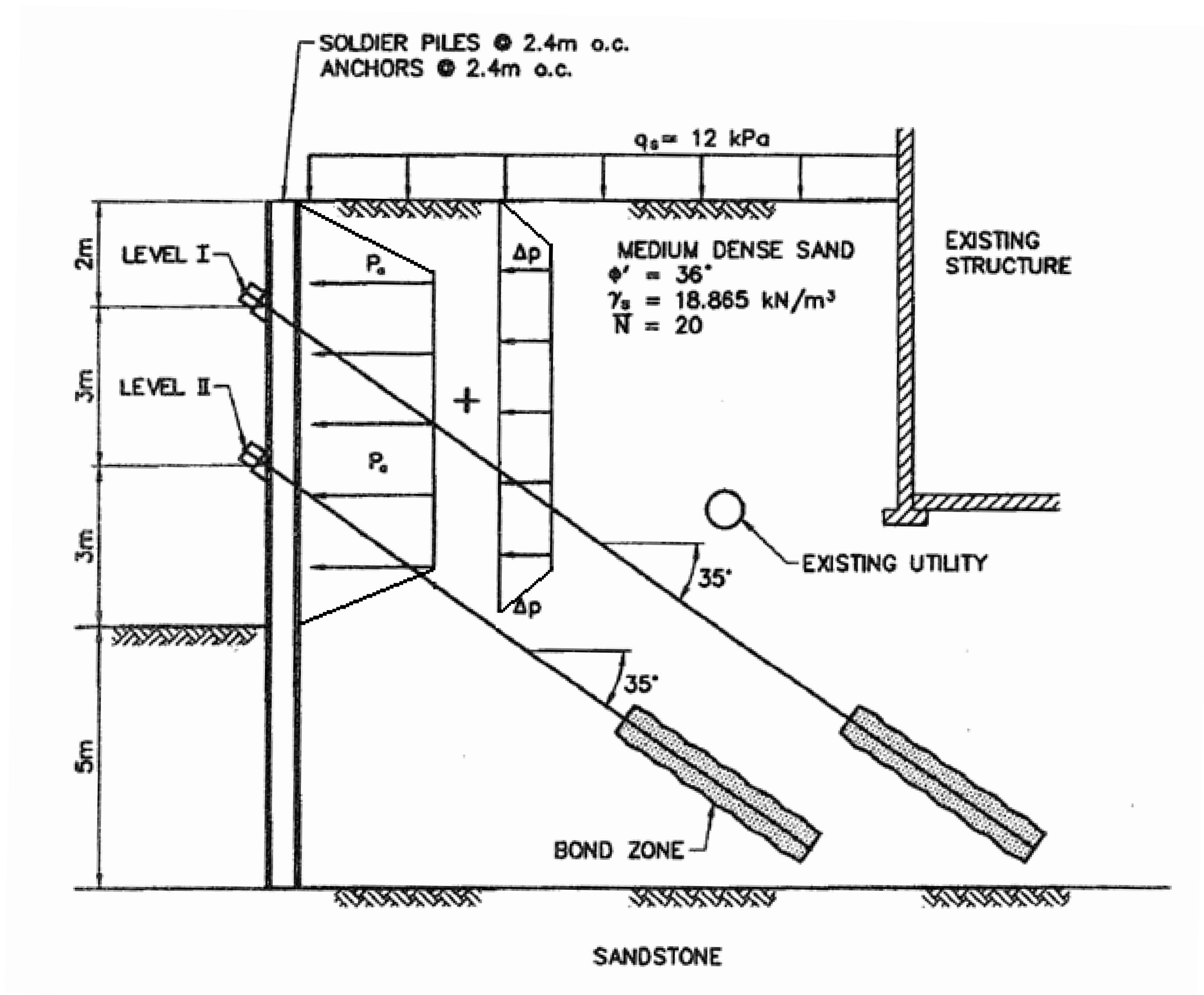


Figure 11.1 Geometry and Arrangement of Soldier Pile Wall for Example 11

11.1.2 Properties of Soldier Piles and Anchors

The soldier piles considered in this design are subjected to flexure due to the imposed earth pressure and to axial load from the two levels of anchors. Therefore, the structural analysis of these soldier piles must consider the interaction effects of combined axial load and flexure. Structural steel of HP12x63 (HP310x94 in SI Units) is selected for the proposed soldier piles. The properties of the HP section are listed below.

Flange width: 307 mm

Flange depth: 302 mm

Flange and web thickness: 13.1 mm

Section area: $11.9\text{x}10^{-3}$ m^2

Moment of Inertia (strong axis), I_{xx}: $190\text{x}10^{-6}$ m^2

Steel yield stress, F_y: $250\text{x}10^3$ kPa

Modulus of elasticity, E: $200\text{x}10^6$ kPa

Pile length: 13 m

The anchors for this example are of 25.4 mm in outside diameter with cross-sectional area of 506 mm^2 and unbonded lengths of 9.5 m and 6 m respectively for the top and bottom anchors.

11.1.3 Hand Computations of Earth Pressure Distribution

The distribution of earth pressure for drilled shafts without tiebacks can be calculated based on a triangular distribution of earth pressure on cantilevered walls. If multi rows of tiebacks are used to restrain the deflection, the distribution of active earth pressure may change to a trapezoidal shape, dependent on field conditions. In general, the triangular distribution can be calculated based on Rankine's equation as follows:

$Ka = \tan^2(45^o-\phi/2) = \tan^2(45^o-18^o) = 0.26$

σ_v (at surface) = 12 kN/m^2 (uniform surcharge)

σ_v (at 8m) = (18.865*8)+12 = 162.9 kN/m^2 (soil weight + uniform surcharge)

Pa (at surface) = Ka σ_v = (0.26) (12) = 3.12 kN/m^2 (uniform surcharge)

Pa (at 8m) = Ka σ_v = (0.26) (162.9) = 42.35 kN/m^2 (soil weight + uniform surcharge)

P_{sum} = (Wa) ($\sum P_a$) = (2.4m) (3.12+42.35) (8)/2 = 436.5 kN

The apparent earth pressure diagrams shown in Figure 11.2 are commonly used for anchored walls. Calculations for this example yield the following:

H (wall height) = 8 m

H_1 (first tieback) = 2 m

H_{n+1} (last tieback) = 8-5 = 3 m

SEG_1 (2/3 H_1) = 1.33 m

SEG_{N+1} (2/3 H_n+$_1$) = 2 m

Total Load = 1.3 (P_{sum}) = (1.3) (436.5) = 567.5

P = (Total Load)/(H-0.333*(SEG_1+SEG_{n+1})) = (567.5) /(6.89) = 82.4 kN/m^2

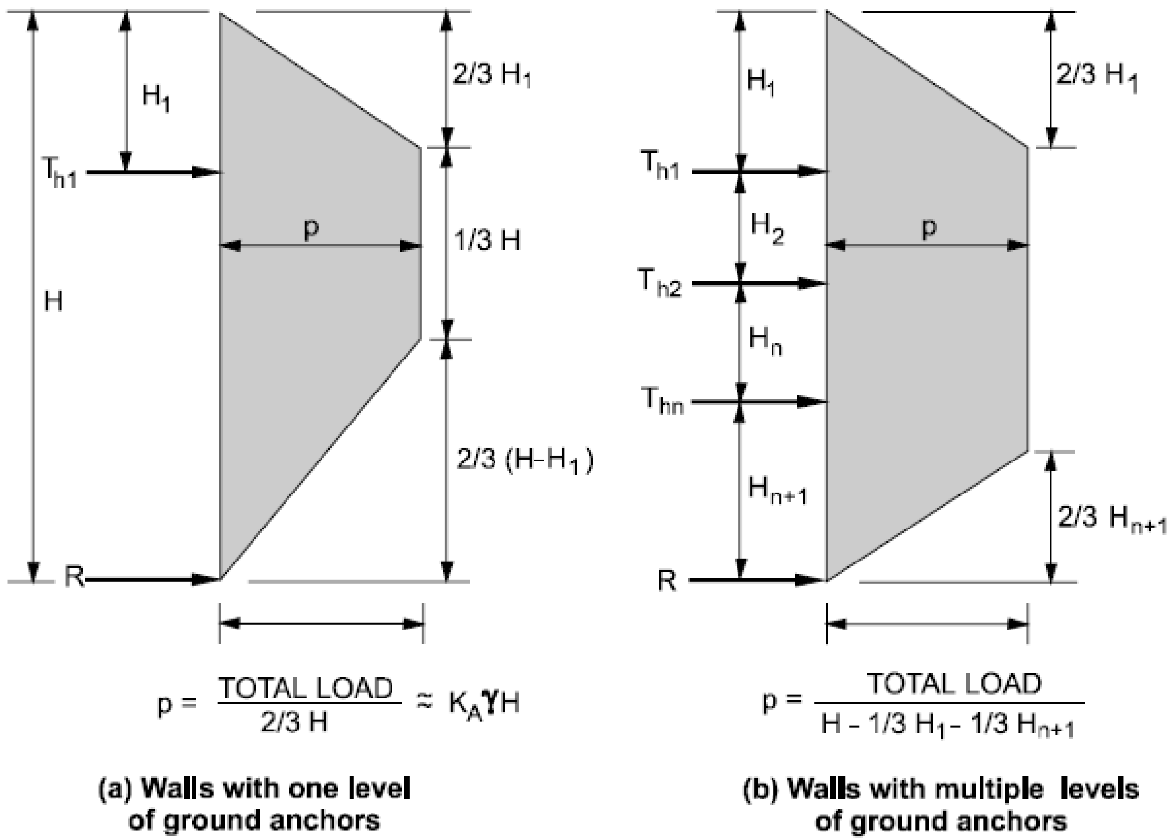


Figure 11.2 Recommended Apparent Earth Pressure Diagram for Anchored Walls (FHWA-IF-99-015)

The earth pressure on each H-pile should include the soil in the tributary area, i.e., the soil mass with a width equal to the spacing between two piles, which was 2.4 m this application. The unit for distributed loads on drilled shafts is "line load/depth" on each pile, such as lbs/inch, or kN/m per pile. The distribution of active earth pressure shown in Table 11.1 was calculated using the Rankine's theory modified with the trapezoidal shape.

Depth (m)	Lateral Load (kN/m)
0	7.5
1.33	82.4
6	82.4
8	0
8	13
13	13

Table 11.1 Hand Computations of Earth Pressure Distribution for Example 11

As shown in Table 11.1, the soil pressure below the excavation level due to the earth pressure from the backfill should be included in the computation by using the *p-y* resistance curves below the excavation level. However, the tributary width is only the width of the H-pile (Wp) as indicated in the calculation below.

Pa (below excavation level) = Ka σ_v W_p= (0.26) (162.9) (0.308) = 13 kN/m^2

11.1.4 Numerical Solution from PYWALL

The input data file *Example 11a - Anchored soldier pile with trapezoidal loads - linear elastic.py8d* represents the model described in previous sections for Example 11. The Example 11a defines a linear-elastic material for the modeled soldier pile. The model incorporates the following options from PYWALL:

Options > Earth-Pressure Options > Generate Triangular-Distribution Earth Pressure

Options > Generate p-y Curves

Options > Apply Tieback (Ground Anchor)

The various components that contribute to the Total Earth Pressure computed internally by PYWALL for Example 11a are shown in Figure 11.3. The trapezoidal pressure is represented by the olive lines and derived from the Rankine pressure in green lines. The Total Earth Pressure in black lines from the PYWALL model are very close to those in Table 11.1 defined earlier based on approximate hand computations.

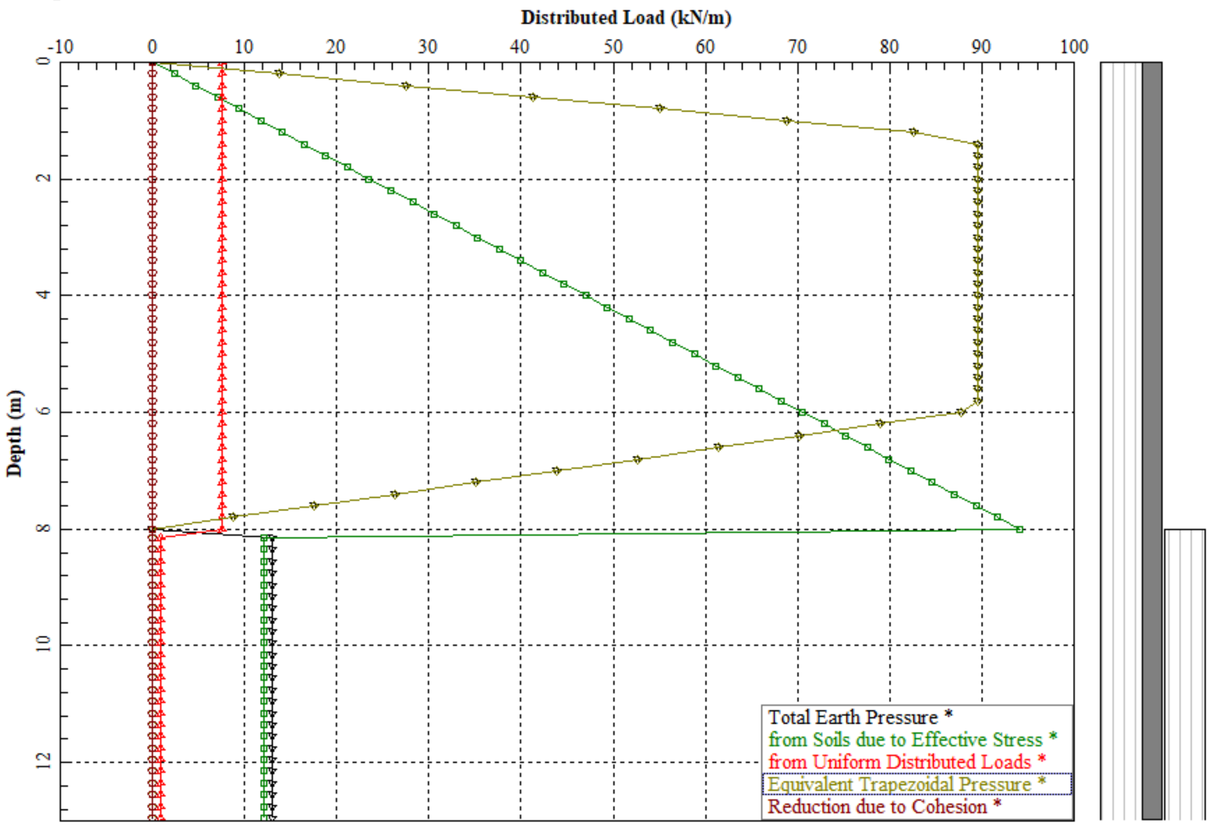


Figure 11.3 Combined Earth Pressures from the PYWALL Model for Example 11a

The computed deflection, bending moment, and shear versus depth from top of wall are shown in Figure 11.4. The maximum deflection at the top of the wall is 0.038 m and the maximum bending moment at the location of the first level of tiebacks is 81.6 kN-m.

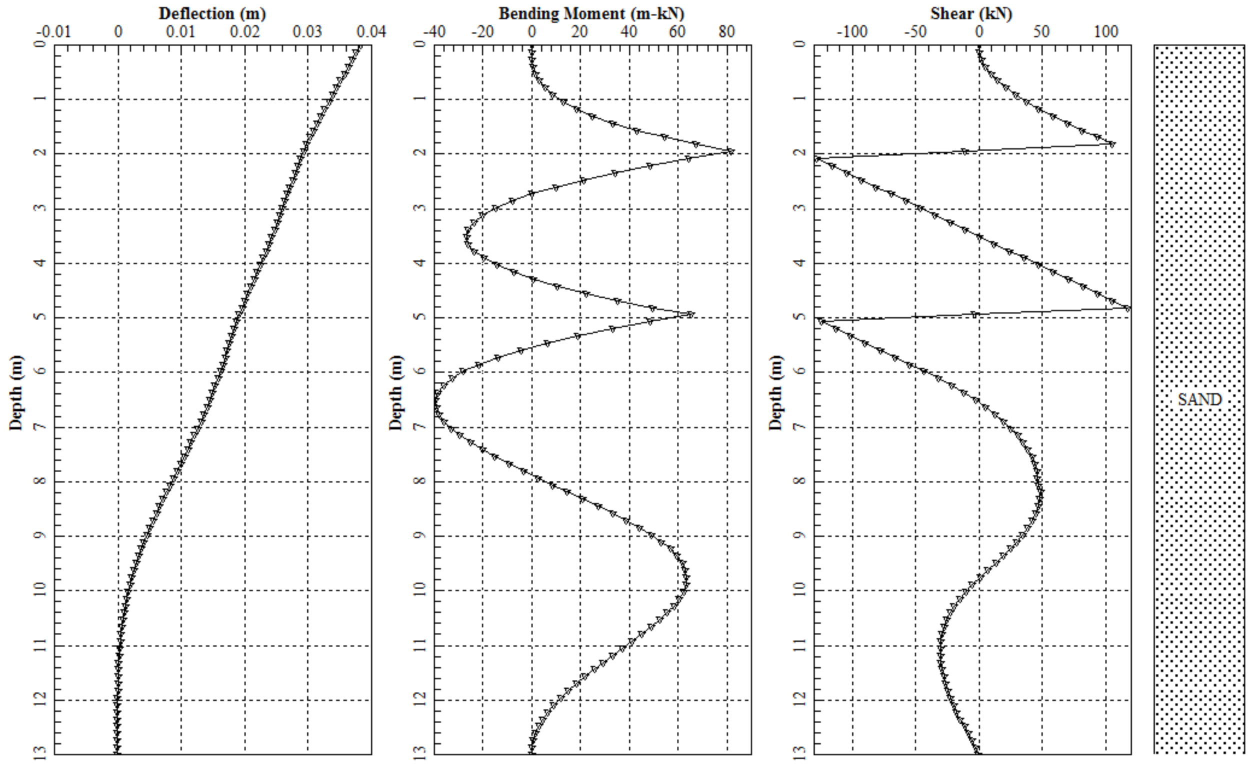


Figure 11.4 Deflection, Bending Moment and Shear Curves from the PYWALL Model for Example 11a with Linear-Elastic Piles and Zero Load Factor

The maximum forces developed in the tiebacks are shown in Figure 11.5 and approximately equal to 312 and 323 kN respectively for top and bottom tiebacks. Notice that these forces are for each tieback placed on each soldier pile (every 2.4 m).

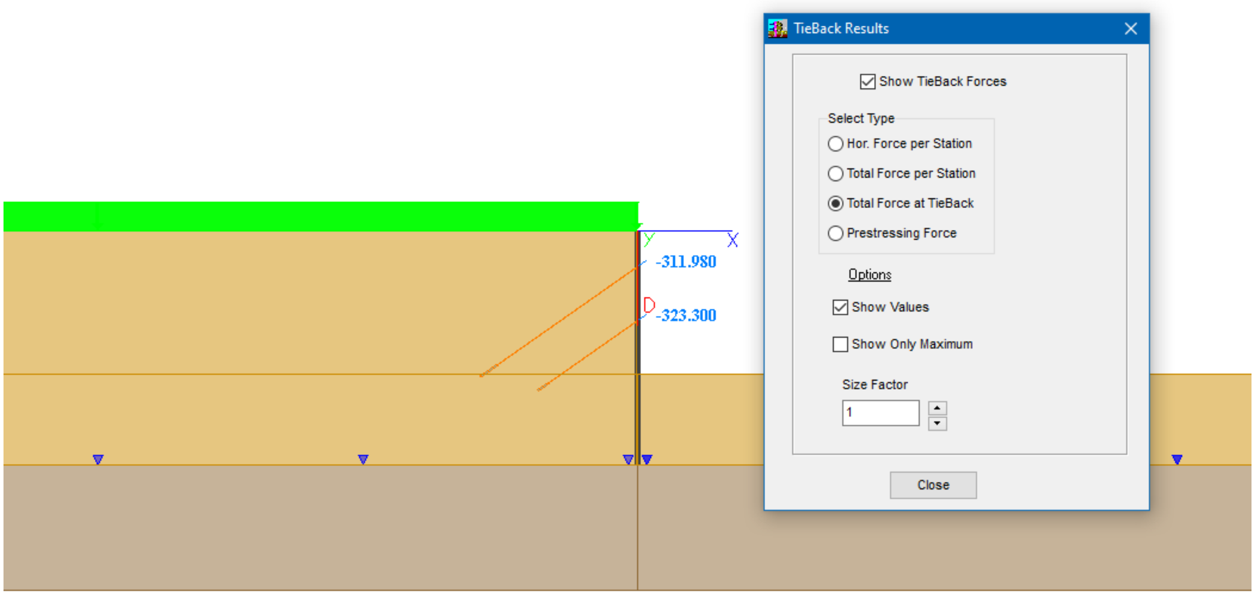


Figure 11.5 Maximum Tieback Forces from the PYWALL Model for Example 11a

11.1.5 Loading Factors and Nonlinear Response

The active earth pressure (including the uniform surcharge load), which was calculated earlier based on Rankine's theory, does not consider any load factors. If a load factor of 1.5 is required in the design specification, the user can specify the modification factor on the calculated active earth pressure as shown in Figure 11.6. In addition, in order to obtain results that are more appropriate for higher loadings (resulting in higher bending moments), the structural section for the soldier pile was changed to an HP12x63 with nonlinear flexural response (Figure 11.7).

The new PYWALL model with load factors and nonlinear flexural stiffness is reflected in file *Example 11b - Anchored soldier pile with trapezoidal loads - factored nonlinear.py8d*.

Notice that the PYWALL program also allows the user to enter concentrated surcharge loads (such as point loads, line loads, or strip loads) as described in Section 3.2.11 of the User's Manual. It should be noted that the load factor in Figure 11.6 will not be applied to the earth pressure based on the Boussinesq equations for those concentrated surcharge loads from Section 3.2.11 of the User's Manual. If a load factor is also needed for those loads, the user should enter them in the column labeled Modification Factor for Surcharge Loads in the Data > Additional Surcharge Loads.

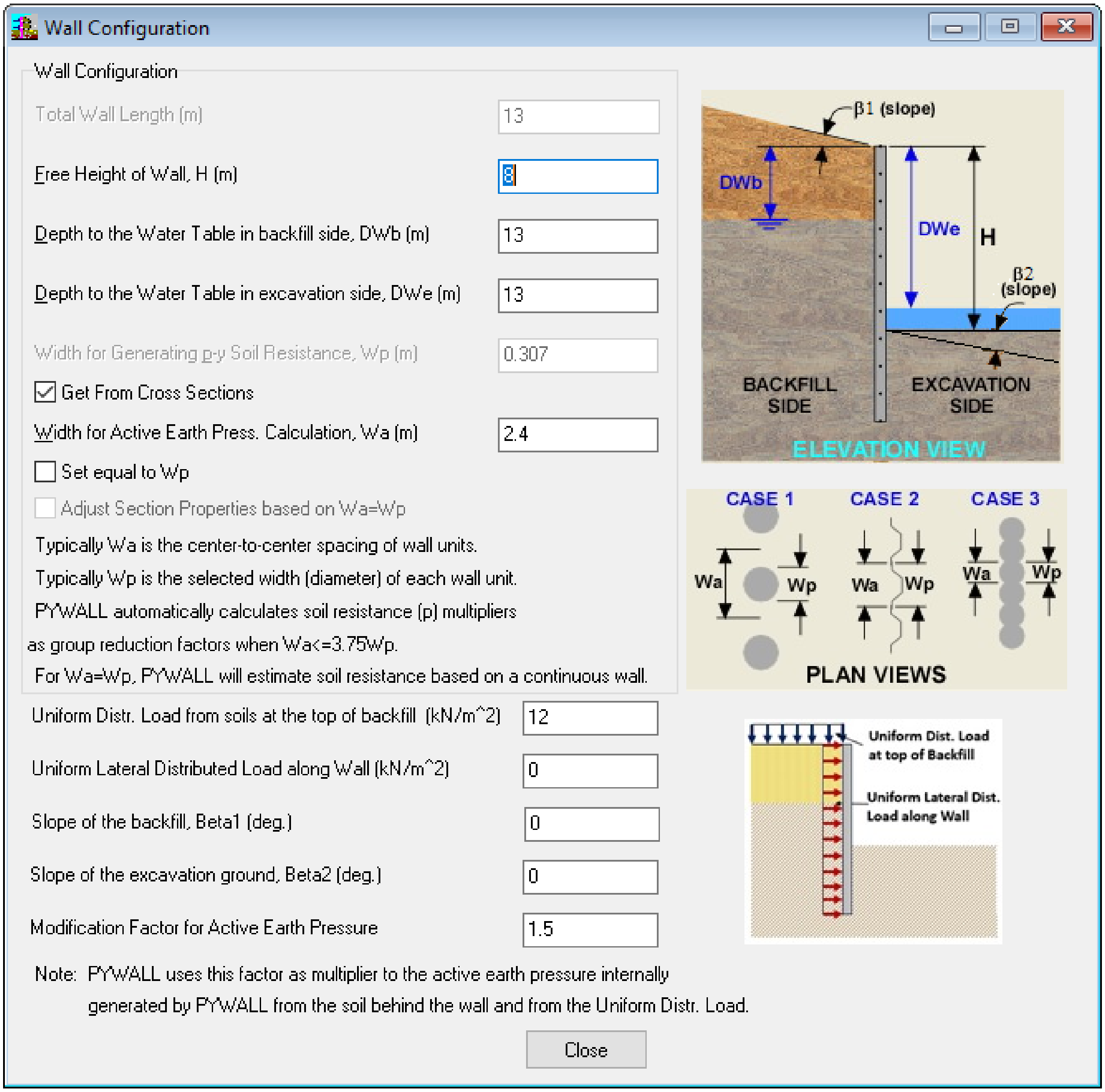


Figure 11.6 Data > Wall Configuration Dialog Box for Example 11b

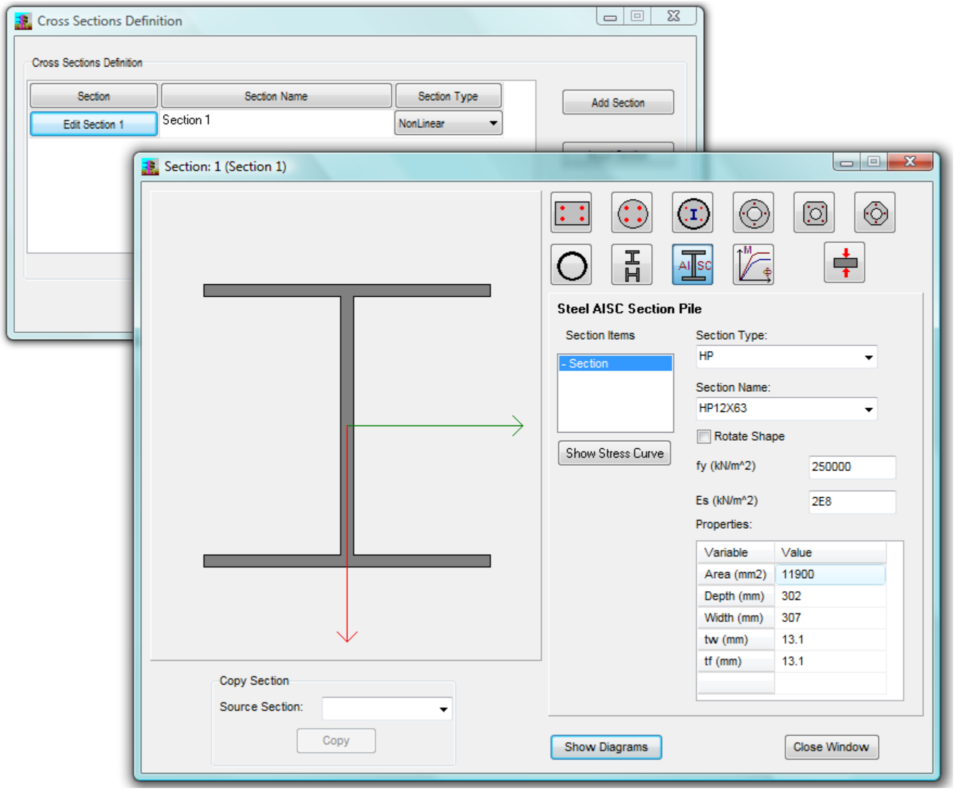


Figure 11.7 Nonlinear Pile Section for Example 11b

11.1.6 PYWALL Solutions for Factored Loading and Nonlinear Response

The computed deflection, bending moment, and shear with a specified load factor of 1.5 are shown in Figure 11.8. The maximum deflection at the top of the wall is 0.057 m and the maximum bending moment at the first level of tiebacks is 122.5 kN-m under this new loading condition. The maximum bending moment on the wall both seem to be acceptable when compared to the ultimate bending capacity of the H-pile section:

$$My = (\sigma_y)\,(I)\,/y = (250{,}000)\,(0.000196)\,/0.152 = 322 \text{ kN-m} >> 122.5 \text{ kN-m}$$

The maximum force developed in the tie back is approximately 490 kN. This maximum tieback force for factored loads is also below the ultimate tension capacity of a tieback with high-strength steel (yielding of 1,034 MPa or 150 ksi):

$$P_{ult} = (A)\,(f_y) = (506\text{x}10^{-6})\,(1{,}034{,}000) = 523 \text{ kN} > 490 \text{ kN}$$

Notice that the load demand on the tieback may not be acceptable when using resistance factors required by applicable codes.

The design of the wall on this example is thus controlled by either the tieback forces or the acceptable level of deflection. The designer must be careful when analyzing for higher loads since the PYWALL program models the tiebacks with linear-elastic properties.

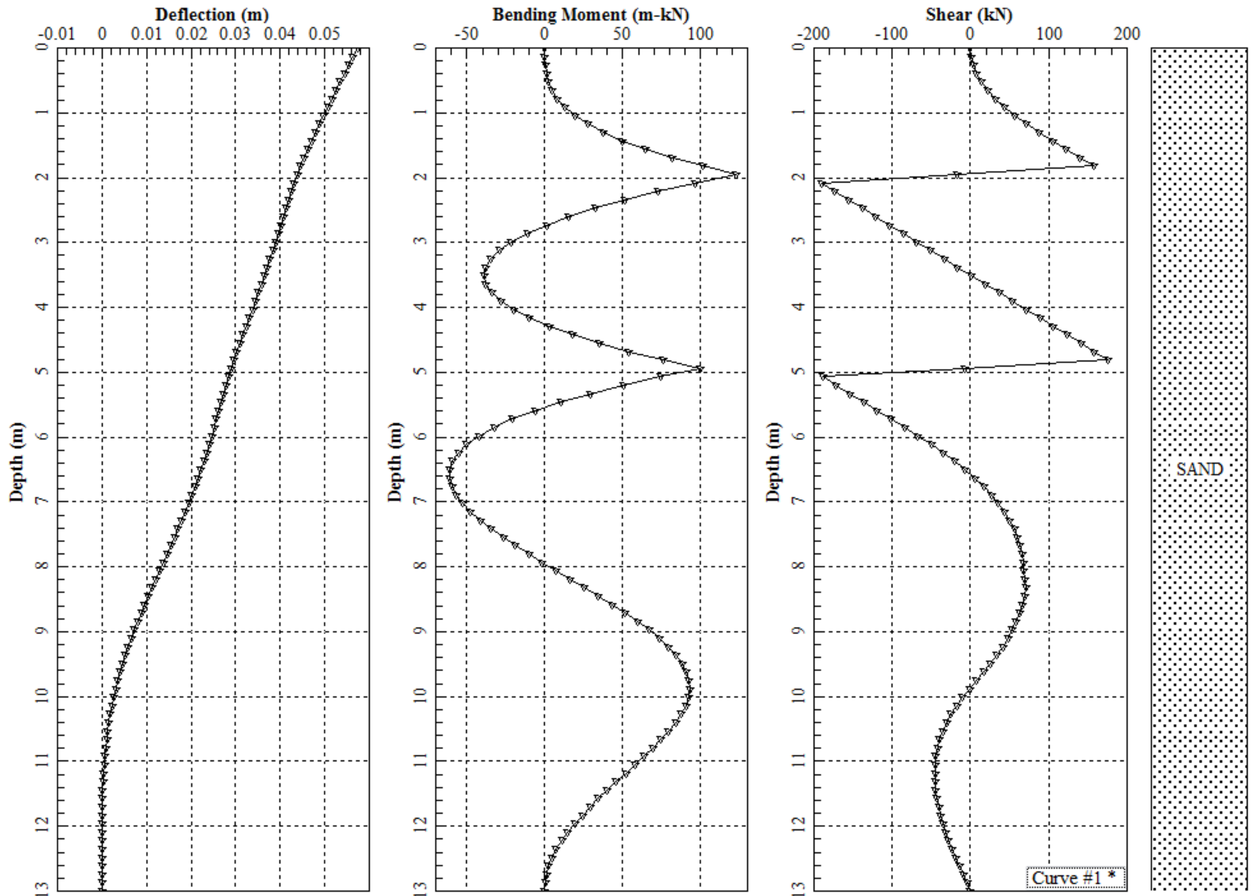


Figure 11.8 Deflection, Bending Moment and Shear Curves from the PYWALL Model for Example 11b with Nonlinear Piles and 1.5 Load Factor

11.1.7 Input and Output Data Files for Example 11

Users can read Section 2.1.1 (7) of the User's Manual for reference on the location of placement of the input and output data files for the example files installed with this program. The default installation directory is the following: *(Root Drive) c:\Ensoft\PYwall2025-Examples*. The input data files for all examples presented in this manual are installed automatically with the program.

The input-data filenames for Example 11 are the following:

Example 11a - Anchored soldier pile with trapezoidal loads - linear elastic.py8d

Example 11b - Anchored soldier pile with trapezoidal loads - factored nonlinear.py8d

The output-data filenames for Example 11 are the following:

Example 11a - Anchored soldier pile with trapezoidal loads - linear elastic.py8o

Example 11b - Anchored soldier pile with trapezoidal loads - factored nonlinear.py8o

Example 12. Earth Pressure Generated Based on Wall Movement of Post-Tensioned Tieback Walls

12.1 Example 12 – Earth Pressure Generated Based on Wall Movement of Post-Tensioned Tieback Walls

12.1.1 Problem Description

The earth pressure above the excavation level may not be in the active state if the wall is pushed into the backfill by high prestressing forces of multi-level tiebacks. When the wall starts to deflect into the backfill soil due to prestressing forces on tieback anchors, the soil-resistant pressure may increase towards the maximum passive earth pressure. The earth pressure in the active state, at rest, and passive state is determined by the movement of the wall. PYWALL provides an option for computing the earth pressures based on the actual wall movements at each wall increment. This example is prepared to demonstrate this new feature on a sample multi-level tieback wall.

Figure 12.1 shows the wall geometry, in which an 11-m excavation is required for construction of a commercial building. The natural site conditions consist of loose sands (γ = 15.7 kN/m^3, ϕ = 32^o) within the top 11 m and dense sand (γ = 18.6 kN/m^3, ϕ = 38^o) below the excavation level. Post-tensioned tiebacks were installed at four different levels during construction. The final depth of excavation is 11 m and the excavation will be supported by soldier piles and timber lagging. The water table is 17 m below the ground surface, and uniform surcharge of 9.8 kN/m^2 is considered on top of the backfill.

The anchors were post-tensioned at each construction stage. The location of the anchor and the post-tension force are listed in Table 12.1. The elastic modulus of steel rods is 200x10^6 kPa and they are considered to be deformed high-strength bars of 1,100 MPa yield strength.

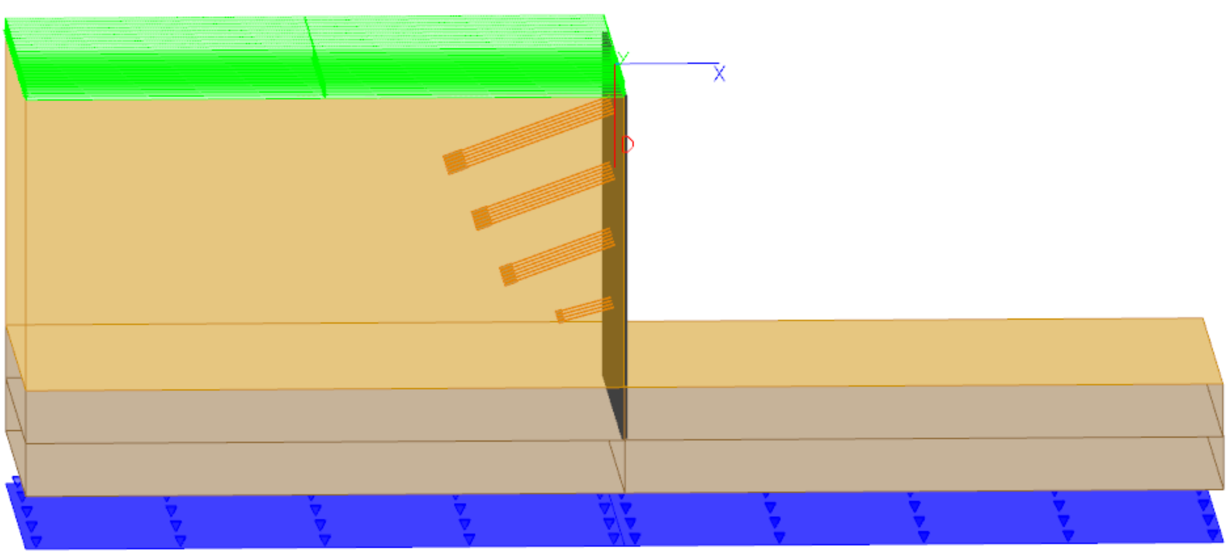


Figure 12.1 Geometry and Arrangement of a Multi-level Tieback Wall in Example 12

Depth, m	Cross sectional area of tie-rod, mm²	Unbonded length, m	Angle from Horizontal, deg.	Anchor spacing in each level, m	Prestressing force, kN
1.5	380.1	6	20	2.7	300
4.0	380.1	5	20	2.7	180
6.5	380.1	4	20	2.7	380
9.0	380.1	2	15	3.6	245

Table 12.1 Anchor Details for Example 12

12.1.2 Properties of Soldier Pile Structure

Soldier piles with lagging are probably the most common shoring solution for urban construction. Tiebacks are used along with the soldier piles to provide additional stability to the retaining system. Soldier piles can be driven if H-section steel is used or drilled and concreted for the cast-in-place concrete piles.

The soldier piles considered in this example are small cast-in-place concrete piles (f'c = 20,000 kPa) with a diameter of 0.25 m and spaced at 0.9 m between the piles, as shown in Figure 12.2. Six longitudinal steel bars with sizes of 16 mm and 20 mm are placed for reinforcement as indicated in the figure. The total length of cast-in-place piles is 13 m. Notice that the drilled size and reinforcing layout on this example is very unique and may only be used in temporary cases with expert control and construction (this is because a small deviation of the rebar layout may have considerable in the nonlinear response of the shaft).

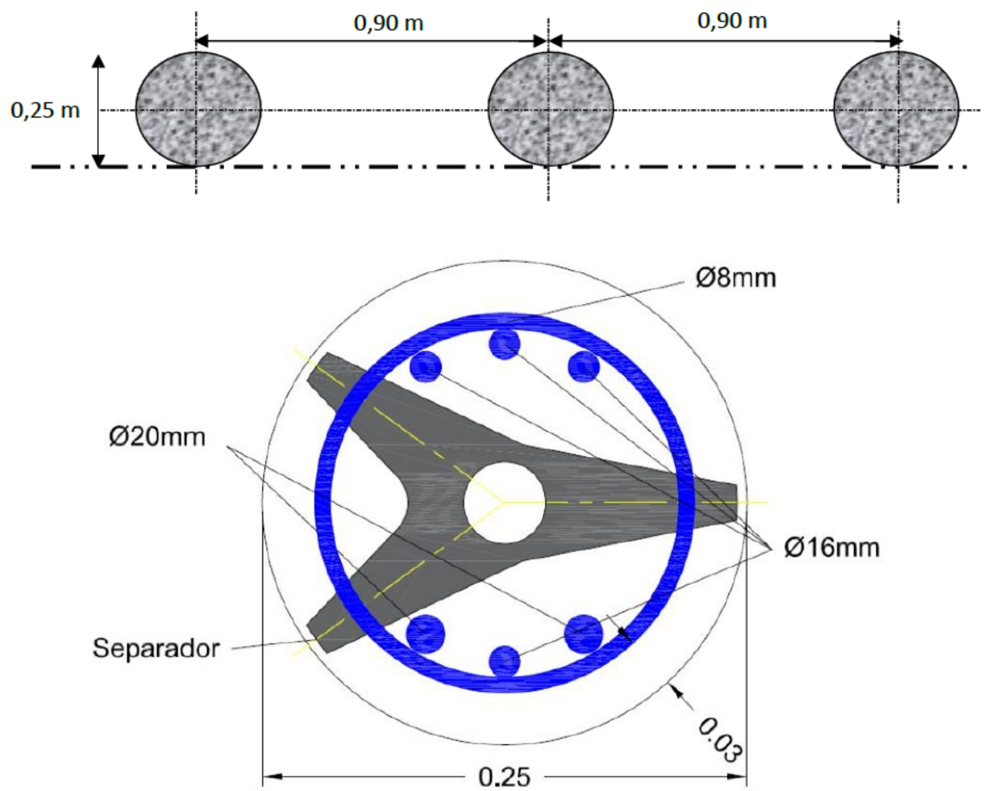


Figure 12.2 Soldier-Pile Details for Example 12

A nonlinear section as detailed in Figure 12.3 is used for this example. The soldier pile has one uniform section and the resulting bending moment versus curvature (in radians) is shown at the bottom right in Figure 12.3. Notice that a manual input of rebar layout (detailed at bottom left in Figure 12.3) was made for this example. The location of the 2 bars of 20-mm at the top causes the highest flexural stiffness for a wall moving into the excavation and was selected for this example (implies that the two larger bars are in the backfill side).

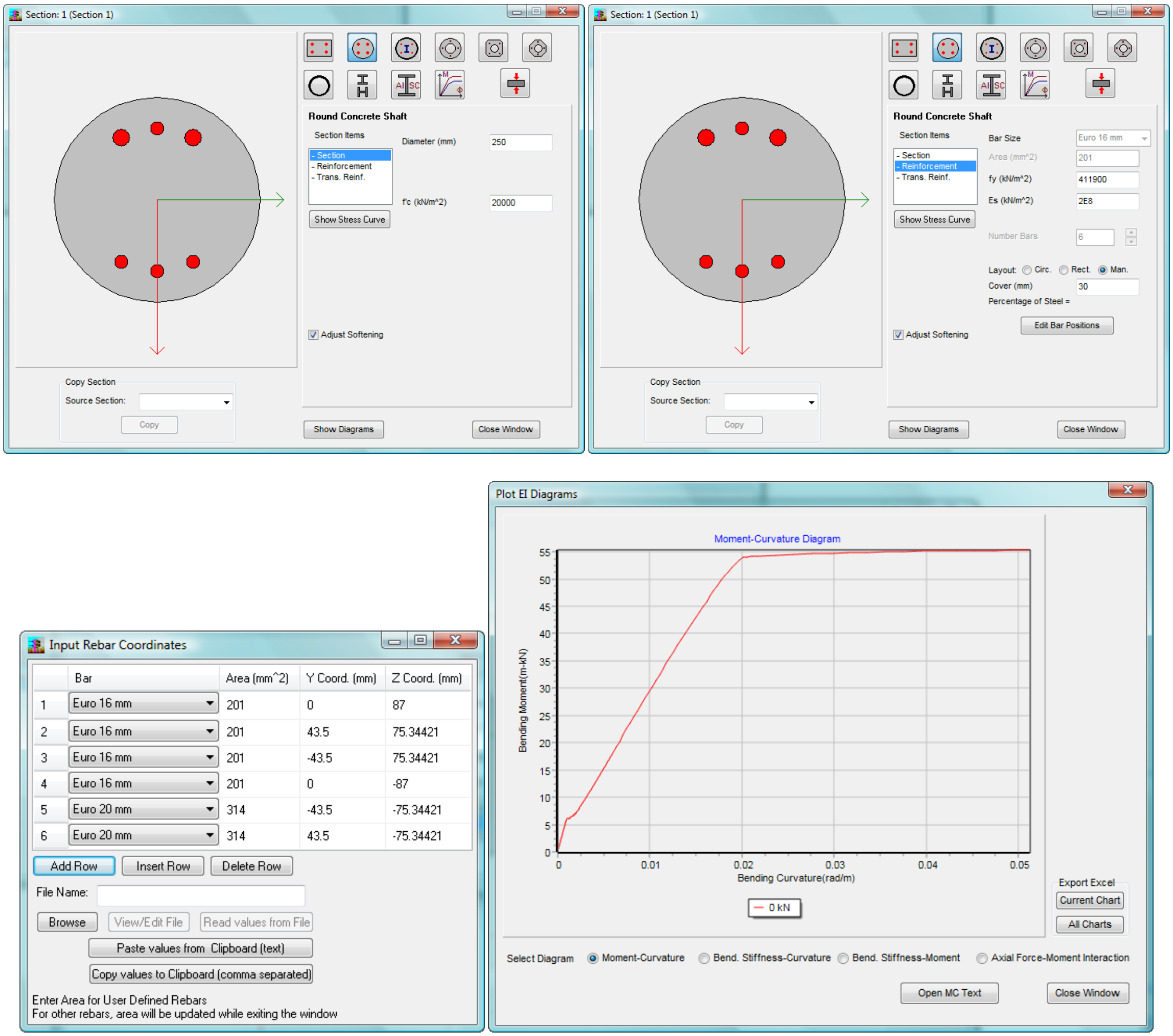


Figure 12.3 Nonlinear Shaft Model for Example 12

12.1.3 Earth Pressure Affected by Wall Movement Caused by Prestressing Forces

The distribution of earth pressure on the wall with multiple-level tiebacks can be calculated based on a trapezoidal-shape distribution of earth pressure as defined in other examples. However, during the staged construction for basement excavations, field monitoring data commonly show that the initial wall movement is toward the backfill due to the strong prestressing forces at the early stages. In such case, the earth pressure of interest may lie between the 'at rest' pressure and the 'passive' pressure depending on the wall movement. For such conditions the PYWALL program provides a function for generating the earth pressure based on the nonlinear curve in Figure 12.4. The earth pressure (including the uniform surcharge load) will be calculated based on the wall movement at each location of the wall using the iterative solution procedures. This is explained in Section 3.9 of the accompanying Technical Manual.

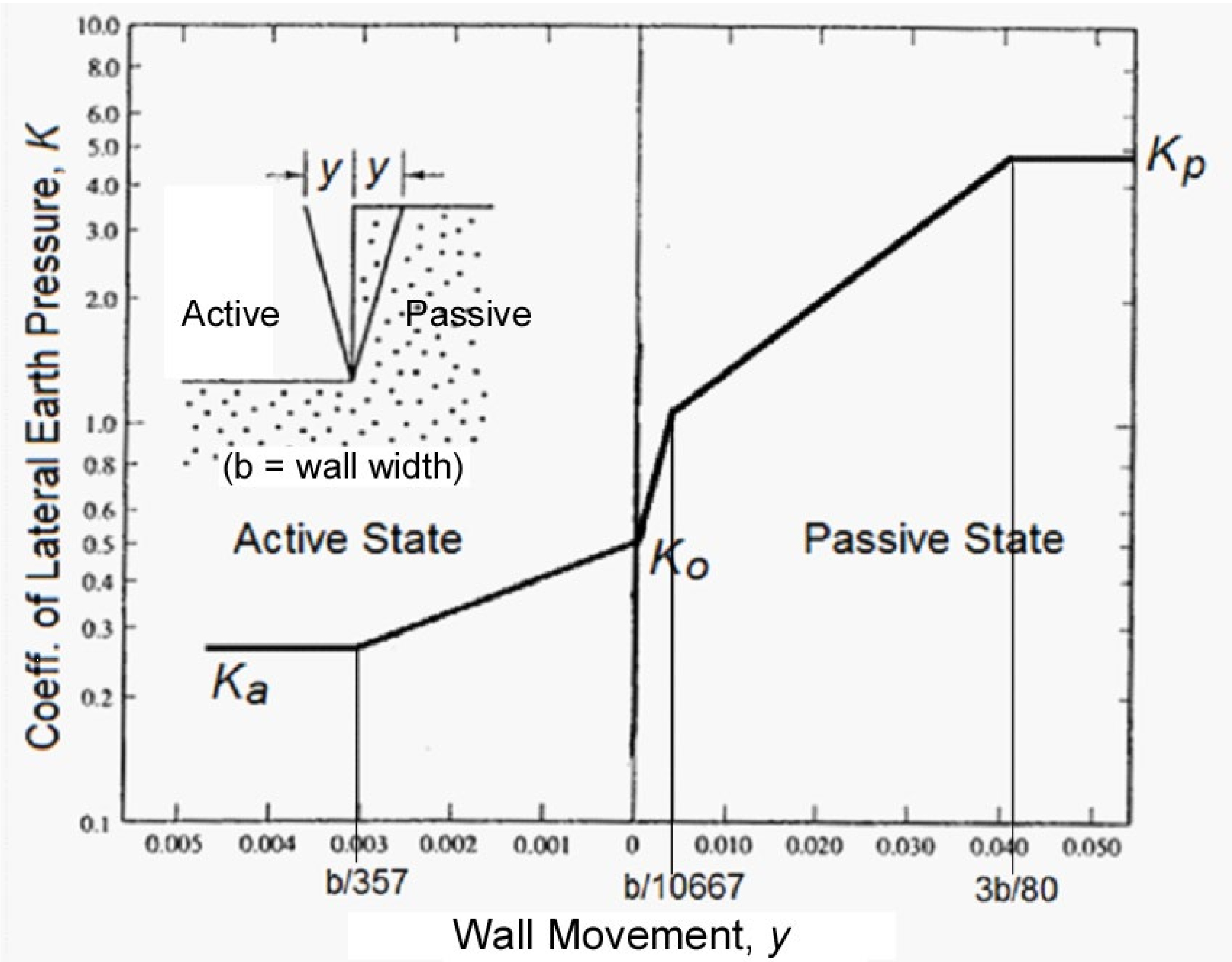


Figure 12.4 Nonlinear Relationship Between Earth Pressure and Wall Movement

12.1.4 Numerical Solution for Serviceability State

The input data file *Example 12a - MultiLevel Anchored Wall with Pressures Based on Movements.py8d* represents the model described in previous sections for Example 12. The computed deflection, bending moment, and shear under the serviceability state while taking into account the variation of earth pressure with the wall movement are shown in Figure 12.5. The maximum deflection at the top of the wall is -0.005 m (into the backfill) and the maximum bending moment at the location of the third level of tiebacks is 28 kN-m. Based on the bottom right graph in Figure 12.3, the ultimate bending capacity of the cast-in-place concrete section is approximately 54 kN-m.

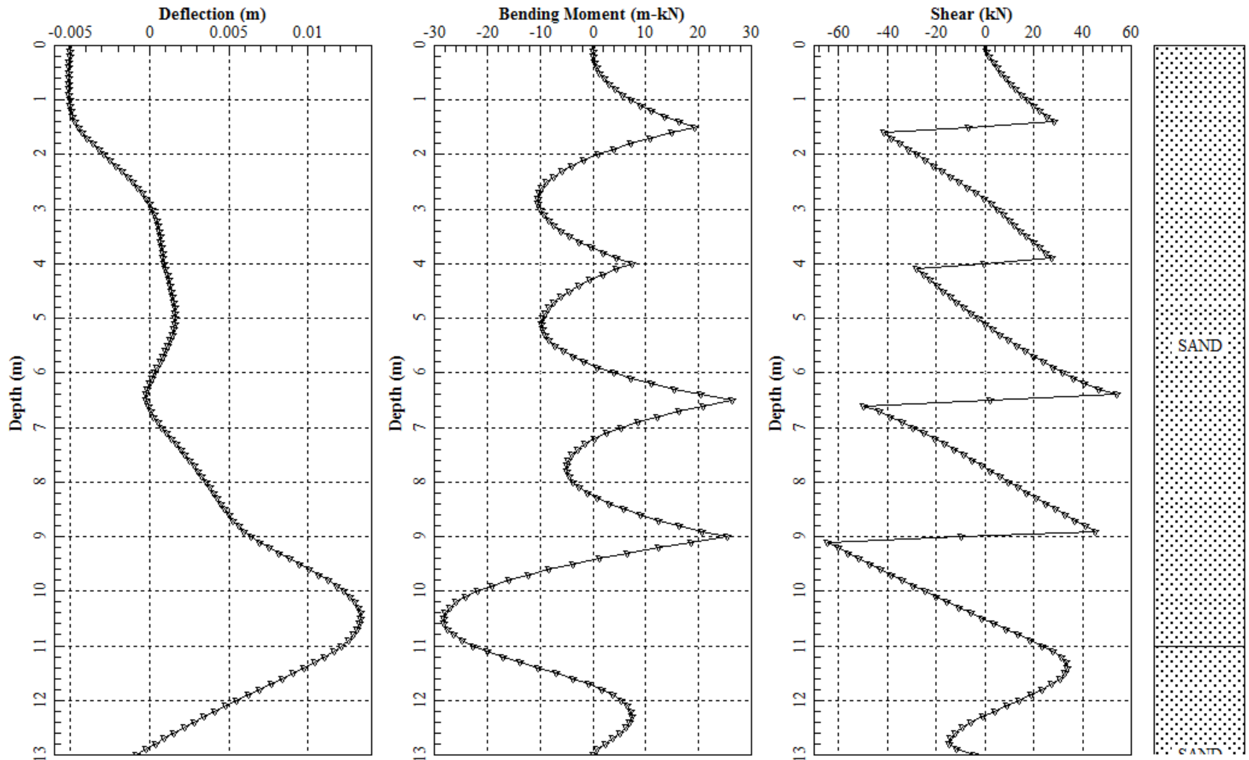


Figure 12.5 Deflection, Bending Moment and Shear Curves from the PYWALL Model for Example 12a with Varying Earth Pressures According to Wall Movements

The maximum forces developed in the tiebacks are shown in Figure 12.6 and approximately equal to 243, 194, 376 and 489 kN respectively for the four tiebacks from top to bottom. Notice that these forces are for each tieback placed at spacing of 2.7 m for the top three and at 3.6 m for the bottom.

The maximum tieback force of 489 kN is even beyond the ultimate tension capacity (designer must reduce the capacity using the appropriate resistance, ϕ, phi factor) of a tieback with high-strength steel (yielding of 1,100 MPa or 160 ksi):

$$P_{ult} = (A)\ (f_y) = (380.1\text{x}10^{-6})\ (1{,}100{,}000) = 419 \text{ kN} < 489 \text{ kN NG!!}$$

For this particular model the designer will need to reduce the spacing between tiebacks in the bottom level or use larger tiebacks.

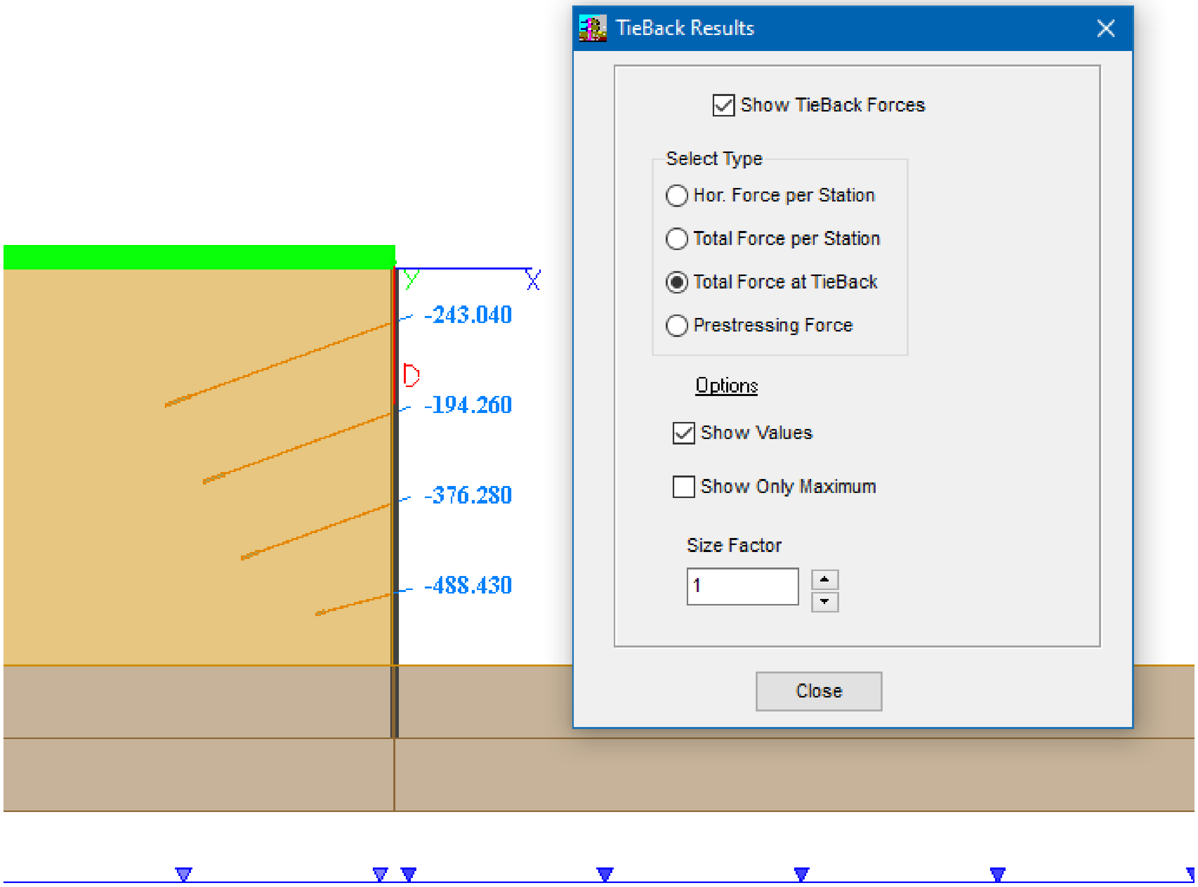


Figure 12.6 Maximum Tieback Forces from the PYWALL Model for Example 12a

12.1.5 Numerical Solution Using Trapezoidal Pressure

The input data file *Example 12b - MultiLevel Anchored Wall with Trapezoidal Pressures.py8d* has a model similar to the previous but with the earth pressures generated automatically within PYWALL based on trapezoidal distribution under the same serviceability state.

The computed deflection, bending moment, and shear for the new model are shown in Figure 12.7. The maximum deflection at the top of the wall is reduced to 0.003 m (into the backfill). The resulting curves of bending moment and shear are also smaller than those in the previous model (Figure 12.5). In addition, the tieback forces are also smaller, with a maximum of 347 kN (reduced from 489 kN) at the bottom tieback.

The reason for these reductions is because the assumption of wall movements towards the excavation side generates a much lower pressure when using the trapezoidal distribution. The designer in this situation may prudently select the more conservative approach used in the previous Example 12a or simply use the at rest (*Ko*) pressure for an even more conservative approach (which is perhaps the more common conservative simplification that is used in practice).

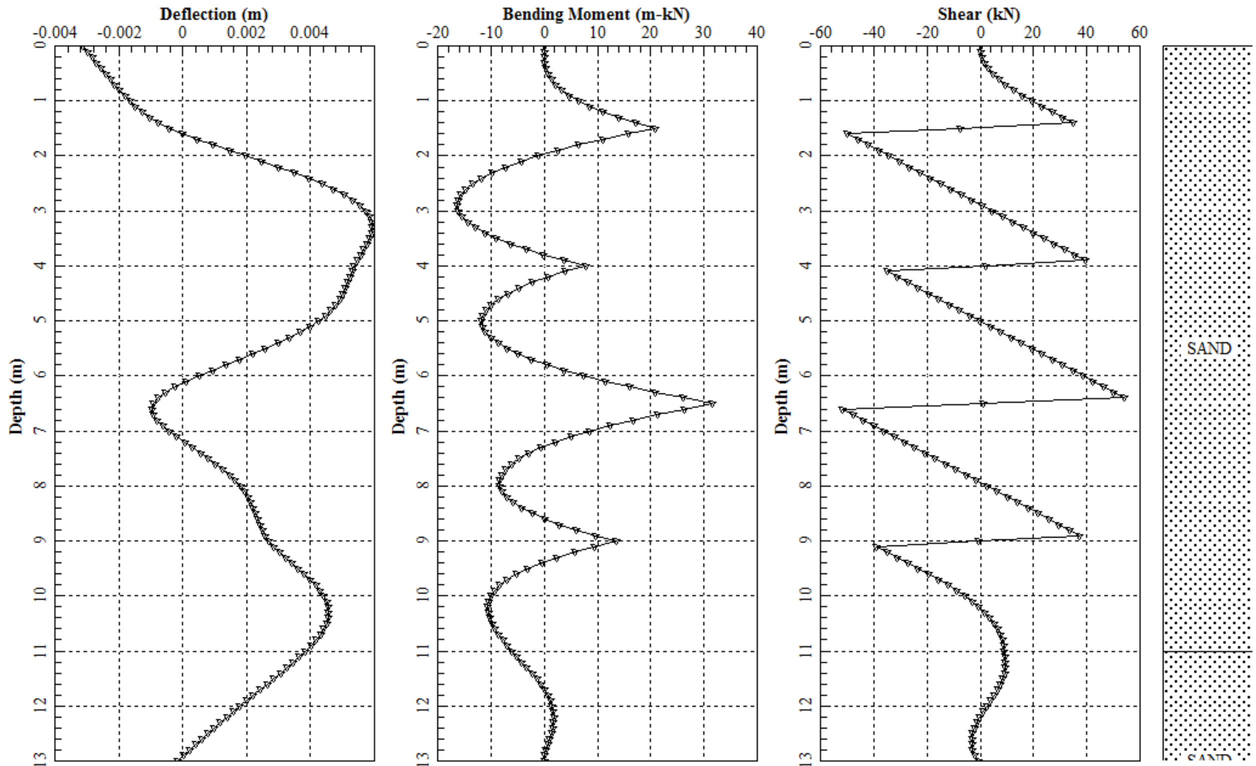


Figure 12.7 Deflection, Bending Moment and Shear Curves from the PYWALL Model for Example 12b with Trapezoidal Earth Pressures

12.1.6 Input and Output Data Files for Example 12

Users can read Section 2.1.1 (7) of the User's Manual for reference on the location of placement of the input and output data files for the example files installed with this program. The default installation directory is the following: *(Root Drive) c:\Ensoft\PYwall2025-Examples*. The input data files for all examples presented in this manual are installed automatically with the program.

The input-data filenames for Example 12 are the following:

Example 12a - MultiLevel Anchored Wall with Pressures Based on Movements.py8d

Example 12b - MultiLevel Anchored Wall with Trapezoidal Pressures.py8d

The output-data filenames for Example 12 are the following:

Example 12a - MultiLevel Anchored Wall with Pressures Based on Movements.py8o

Example 12b - MultiLevel Anchored Wall with Trapezoidal Pressures.py8o

Example 13. Analysis of Sheet-Pile Wall with Struts and Wales

13.1 Analysis of Sheet-Pile Wall with Struts and Wales

13.1.1 Problem Description

This example illustrates the analysis of an anchored continuous wall with struts and wales using PYWALL under static conditions. The wall to be analyzed is presented in Figure 13.1, along with the soil layer information. The functions available in PYWALL can help the designer to review the results on the different elements of the continuous wall conveniently and efficiently.

The wall for this example is 60 ft in total length under a stratified soil. The sheetpile selected for the analysis is a PZ 27. The model includes three level of struts which are separated 18 ft. The struts have an area of 20 in^2, a modulus of elasticity of 29,000,000 psi and a length of 10 ft. Wales are considered at each level of struts.

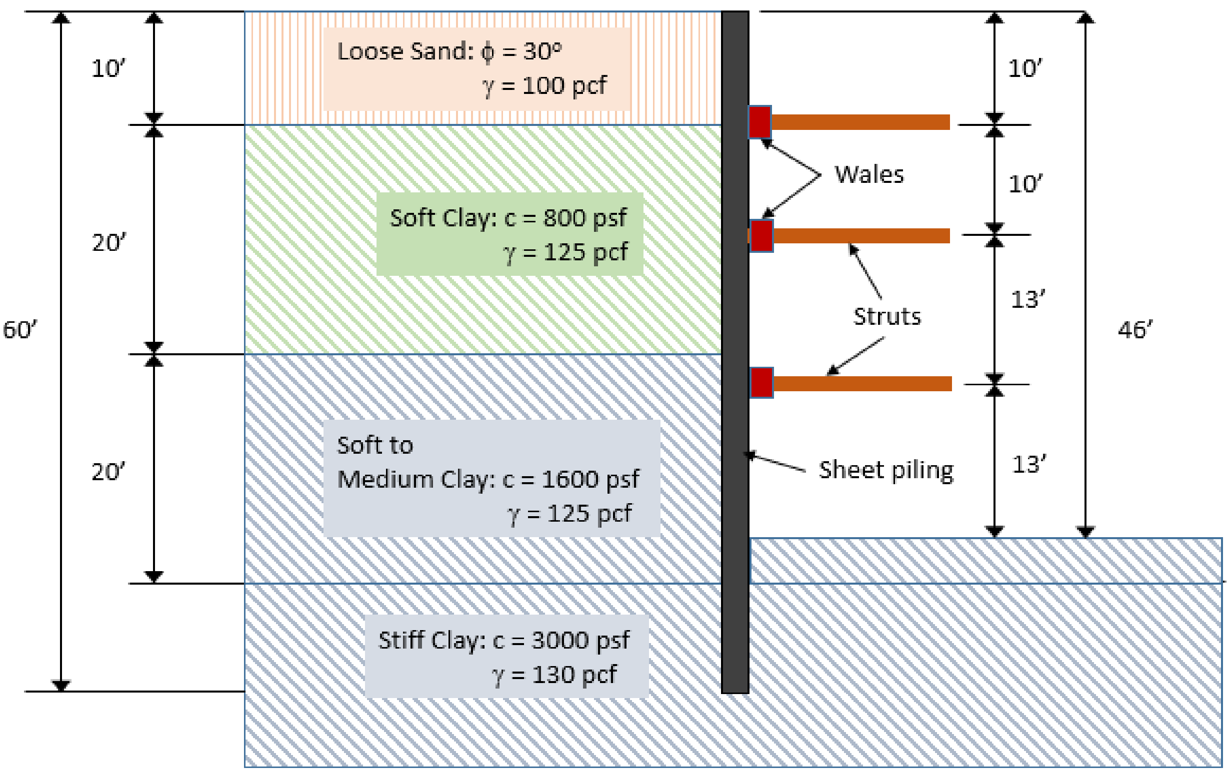


Figure 13.1 Geometry and Soil Information for Anchored Sheet-Pile Wall

13.1.2 Static Active Earth Pressure Using Peck and Terzaghi Method

Since the thickness of soil layers is more than 50% clay, the maximum design pressure can be estimated by Peck and Terzaghi's using one of the two given equations:

$$P_A = (\gamma_a)(H) - 4\,c_a \quad \text{if } (\gamma_a)(H)/c_a >= 6$$

$$P_A = 0.3\,(\gamma_a)(H) \quad \text{if } (\gamma_a)(H)/c_a < 6$$

Where γ_a is the average specific weight and c_a is the equivalent average cohesion over the depth H.

The average specific weight is estimated as:

$$\gamma_a = (\Sigma\ \gamma_i\ H_i)/H$$

The equivalent average cohesion is estimated as:

$$c_a = (\Sigma\ \gamma_i\ k_i\ H_i^2\ \tan(\phi_i) + \Sigma\ c_i\ H_i)/H$$

Therefore, for this example the average specific weight is calculated as:

$$\gamma_a = (100\ (10) + 125\ (20) + 125\ (16))/46 = 120 \text{ pcf}$$

And the equivalent average cohesion is calculated as:

$$c_a = (100\ (1)\ (10)^2 \tan(30) + 800\ (20) + 1600\ (16))/46 = 1030 \text{ psf}$$

Therefore,

$$(\gamma_a)\ (H)/c_a = (120 * 46 / 1030) = 5.35 < 6$$

Then, the maximum design pressure is calculated as:

$$P_A = 0.3 * 120\ (46) = 1656 \text{ psf} = 1656 \text{ lb/ft/per ft} = 138 \text{ lb/in/per ft}$$

The trapezoidal earth pressure distribution is presented in Figure 13.2

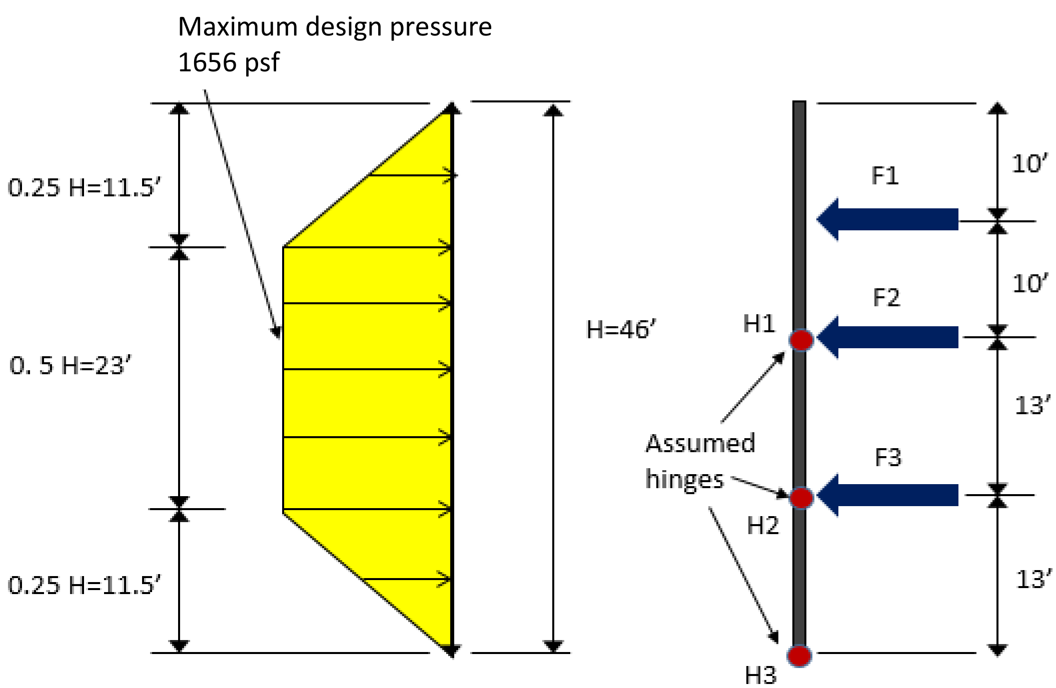


Figure 13.2 Distribution of Trapezoidal Earth Pressure and Location of Axial Forces at Struts

13.1.3 Estimation of Forces in Struts

Assuming hinges at location of struts (with the exception of the top one) and the excavation level, the axial forces in the struts can be estimated.

Taking moments about H1, in order to get F1 (at depth 10'):

F1(10) – 8.5*(1.66)*8.5/2 - 11.5*(1.66)/2*(8.5+11.5/3) = 0

F1 = 17.77 kip/ft

Taking moments about H2, in order to get F2 (at depth 20'):

17.77*23 + F2(13) – 21.5*(1.66)*21.5/2 - 11.5*(1.66)/2*(21.5+11.5/3) = 0

F2 = 16.67 kip/ft

Taking moments about H3, in order to get F3 (at depth 33'):

17.77*36 + 16.67*26 + F3(13) –11.5*(1.66)/2*(11.5*2/3) - 23*(1.66)*(11.5+23/2)

- 11.5*(1.66)/2*(34.5+11.5/3) = 0

F3 = 18.77 kip/ft

The total force in each strut is:

F1 = 17.77*18 = 319.86 kip

F2 = 16.67*18 = 300.06 kip

F3 = 18.77*18 = 337.86 kip

13.1.4 Estimation of Forces in Wales

The response of a wale can be assumed somewhere between that of a continuous beam on several supports (struts) and a beam of a single span. For this example, considering a continuous beam (Figure 13.3), the maximum moment can be estimated as:

$$M = w\,L^2/10$$

Where: w is the equivalent load in the wale that is equal to the force in the struts for unit length and L is the distance between struts.

For the wale at F1 (at depth 10'), w=17.77 kip/ft, then M=17.77*18^2/10=576 kip-ft.

For the wale at F2 (at depth 20'), w=16.67 kip/ft, then M=16.67*18^2/10=540 kip-ft.

For the wale at F3 (at depth 33'), w=18.77 kip/ft, then M=18.77*18^2/10=608 kip-ft.

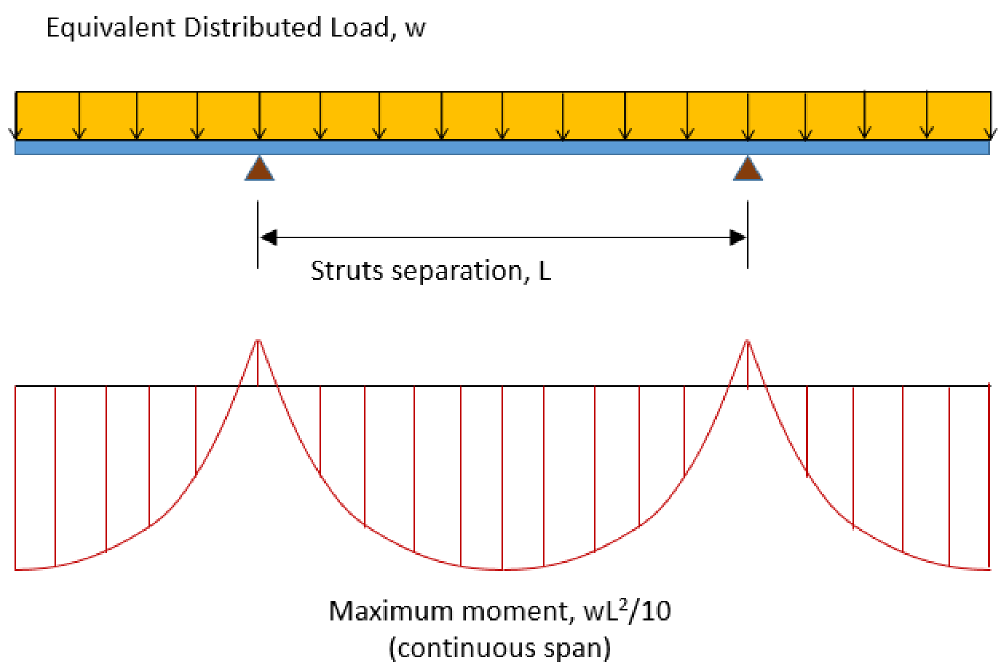


Figure 13.3 Equivalent distributed load and moment in wales

13.1.5 Analysis Based on the *p-y* Curve Method (PYWALL)

In the PYWALL model the earth pressure above the excavation line (from 0 to 46 ft) is treated as the external load. The external load in PYWALL is assumed to remain as a constant below the excavation line at the backfill side.

The soil resistance below the excavation following the *p-y* curve method is represented by nonlinear springs (*p-y* curves) for the force-deformation relationship. The soil properties for the PYWALL model are shown in Table 13.1. The k_{py} value for the sand layer is estimated by the program based in the friction angle and the ε_o value for the clay layers are estimated by the program based in the cohesive strength

Soil Type	Layer Thickness (ft)	Total Unit Wt (lbs/ft^3)	Internal Friction Angle (deg)	Cohesive Strength (lbs/ft^2)
Sand	10	100	30	
Soft Clay	20	125		800
Stiff Clay	20	125		1600
Stiff Clay	16	130		3000

Table 13.1 Soil Input Parameters for Example 13

The automatic computation of trapezoidal load distribution using the Peck and Terzaghi method was selected for this example in PYWALL: Options > Earth-Pressure Options > Generate Trap.-Distr. Pressure (Anchored Walls, Peck and Terzaghi).

The struts information presented before was entered in the PYWALL model under Data > Strut/Raker Data > Enter Strut/Raker Data.

The resulting active earth pressures automatically calculated within PYWALL are shown in Figure 13.4 (Graphics > Earth Pressure Distribution). The values shown in Figure 13.4 are approximately the same as those previously obtained from hand computations in Figure 13.2.

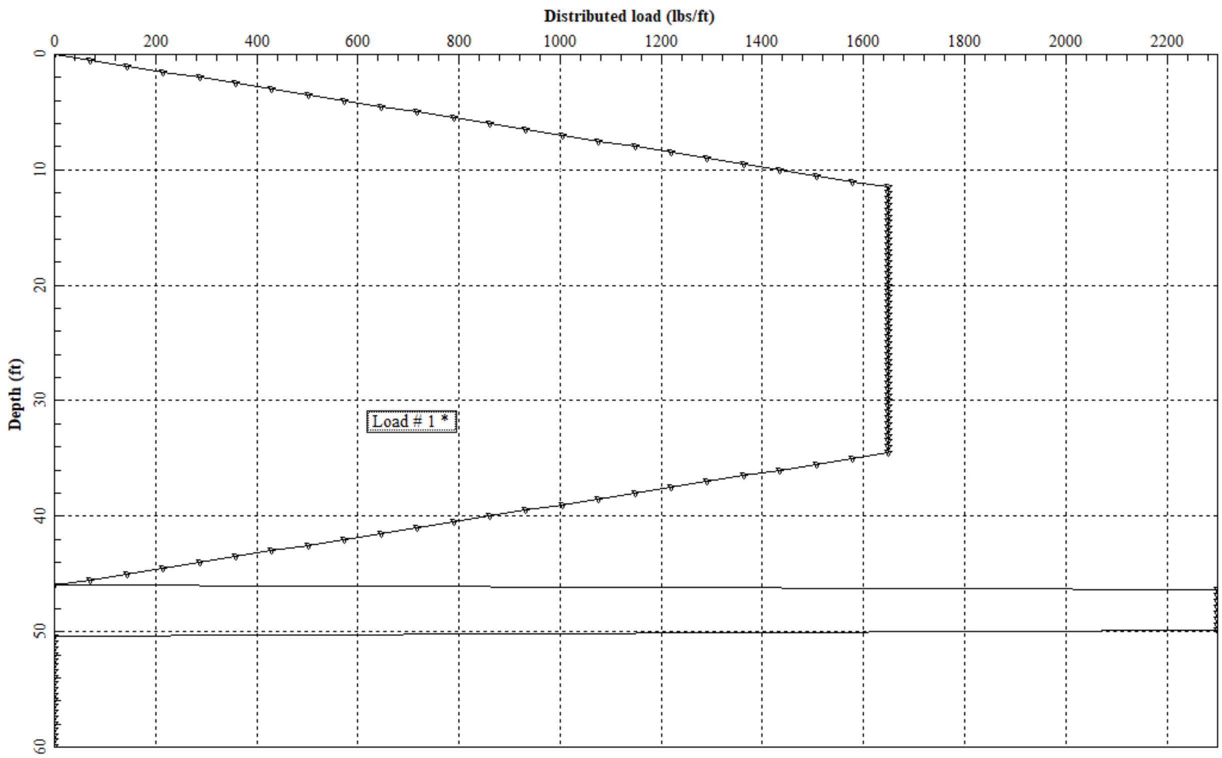


Figure 13.4 Computed Active Forces from PYWALL in Example 13

The resulting deflection, shear and bending moment diagrams of the retaining wall in the PYWALL model are presented in Figure 13.5 (Graphics > Deflection, Moment and Shear). The maximum bending moment based on the PYWALL model is approximately 34,583 ft-lbs (415 in-kips) at a depth of 26 ft below the top of the wall.

The forces in the struts, obtained in tabular form at the end of the output text file (Computation > View Output Text), are presented in Table 13.2. The results from PYWALL are in agreement with the hand computations presented in Section 13.1.3.

Effects on wales are presented graphically in Figure 13.6 (Graphics > Wales Effects) and in Table 13.3 (obtained from Computation > View Output Text at the end of the output text file). The results from PYWALL are in agreement with the hand computations presented in Section 13.1.4.

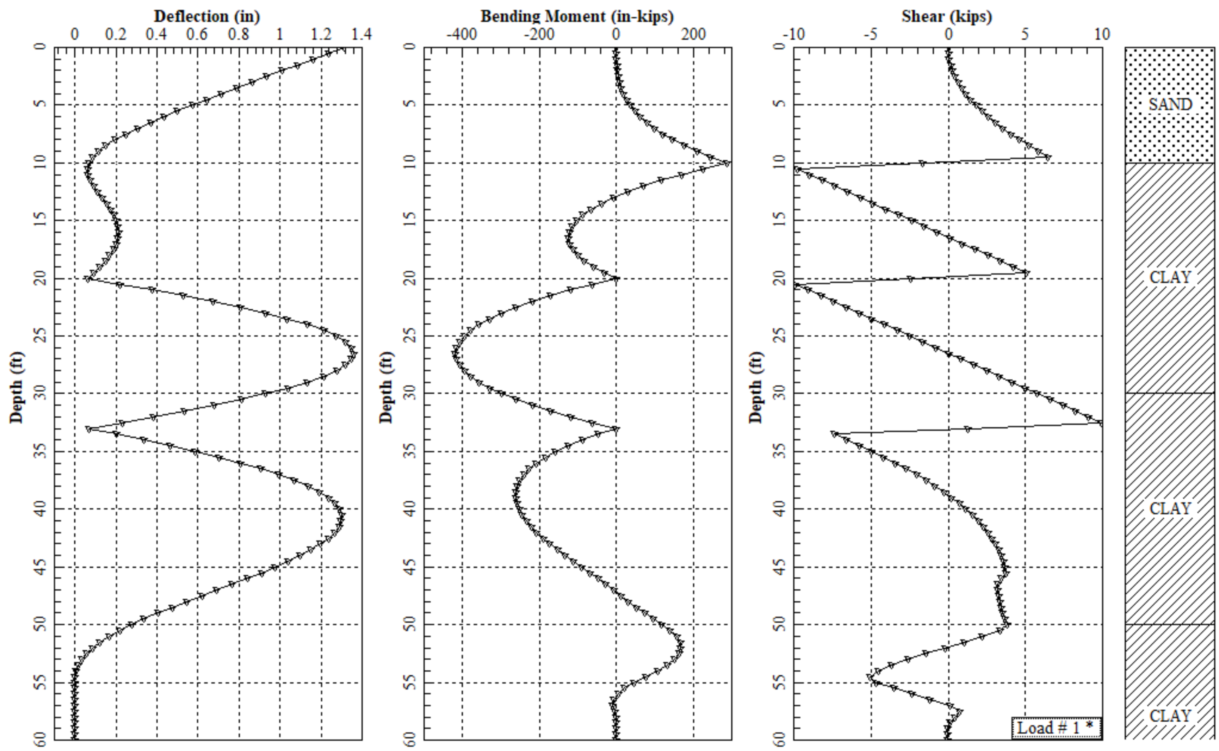


Figure 13.5 Deflection, Shear and Bending Moment Curves from the PYWALL Model for Example 13

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STRUT/RAKER RESULTS

N	STA	X FT	HOR.FORCE/STA. LBS	FORCE/STA. LBS	FORCE/STRUT LBS
1	20	1.000E+01	-1.7655E+04	-1.7655E+04	-3.1779E+05
2	40	2.000E+01	-1.6582E+04	-1.6582E+04	-2.9848E+05
3	66	3.300E+01	-1.8953E+04	-1.8953E+04	-3.4116E+05

Table 13.2 Estimated Forces in Struts/Rakers from PYWALL Model for Example 13

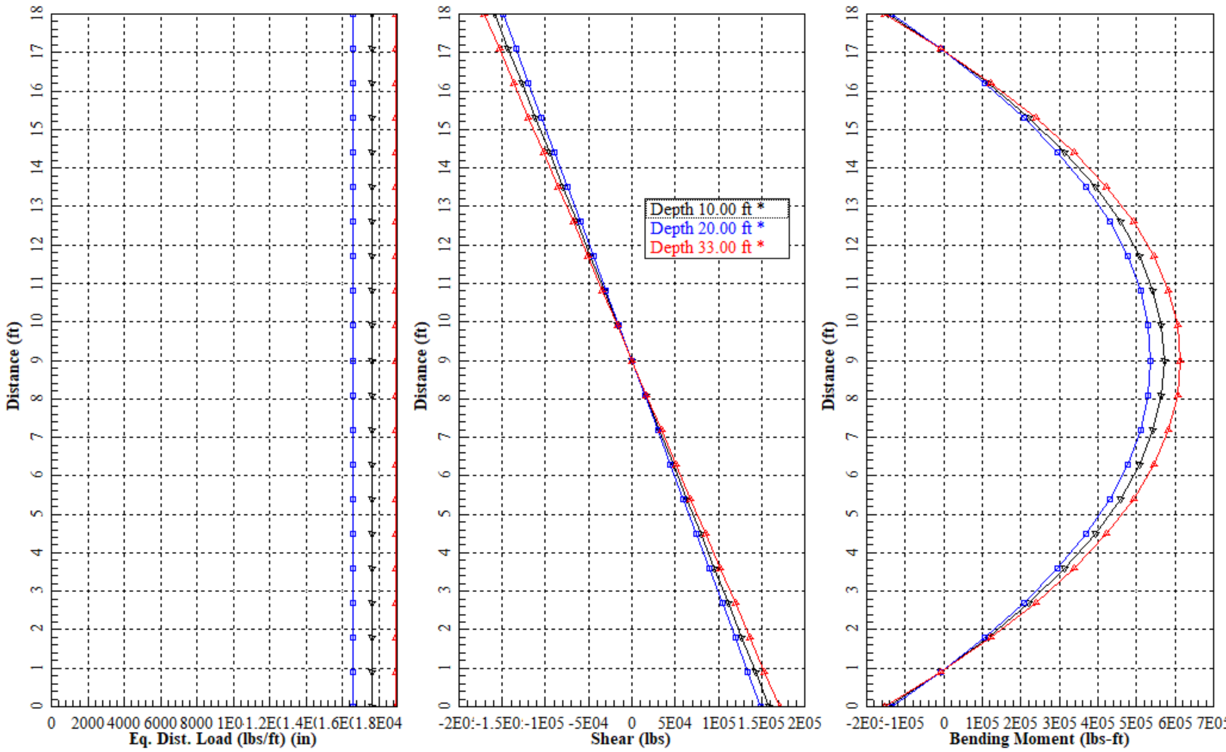

Figure 13.6 Graphics of Estimated Load and Demands of Wales in Example 13

WALES RESULTS

N	TYPE	X FT	ANCHOR SPA. FT	HORIZ. REACT. LBS	EQ. DL LBS/ FT	MAX. MOMENT LBS- FT
1	CONTINUOUS	1.000E+01	1.8000E+01	3.1779E+05	1.7655E+04	5.7202E+05
2	CONTINUOUS	2.000E+01	1.8000E+01	2.9848E+05	1.6582E+04	5.3726E+05
3	CONTINUOUS	3.300E+01	1.8000E+01	3.4116E+05	1.8953E+04	6.1408E+05

Table 13.3 Table of Estimated Forces of Wales in Example 13

13.1.6 Concluding Comments

The results show that the program PYWALL for the analysis of this anchored sheet-pile wall is rational and adaptable. It provides a reliable way to aid the designer in obtaining a more refined evaluation of the actual behavior of flexible anchored wall.

13.1.7 Input and Output Data Files for Example 13

Users can read Section 2.1.1 (7) of the User's Manual for reference on the location of placement of the input and output data files for the example files installed with this program. The default installation directory is the following: *(Root Drive) c:\Ensoft\PYwall2025-Examples*. The input data files for all examples presented in this manual are installed automatically with the program.

The input-data filename for this Example is the following:

Example 13 - Sheet Pile with Struts and Wales.py8d

The output-data filename for this Example is the following:

Example 13 - Sheet Pile with Struts and Wales.py8o

Installation Notes

All input and output files for the examples of PYWALL v2025 are installed with the program. During program installation, the user is provided with an option to select a drive and directory for the installation of example files (see Section 2.1.1 (7) of the separate User's Manual). If it was not changed by the user, the example files can be found in the following default installation directory:

(Root Drive) c:\Ensoft\PYwall2025-Examples

The input-data files for models created in PYWALL v2025 have extensions of the type *filename.py8d* (such as *Example1.py8d*), as indicated in Table 2.1 of the separate User's Manual.

List of Technical References

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