APPENDIX K
DESIGN EXAMPLES

GEOTECHNICAL DESIGN MANUAL

January 2022

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APPENDIX K

DESIGN EXAMPLES

K.1 INTRODUCTION

The purpose of this Appendix is to provide design examples to assist the GEOR with conducting sophisticated analysis. Typically this analysis requires direct interaction with other team members, such as the SEOR. It is not the intent of the Appendix to provide examples of analysis that is concerned to be part of normal procedures (i.e., foundation analysis, SSL analysis, settlement analysis, etc.). Currently there is one design example, the use of foundation elements to resist lateral movements induced within an embankment. In Section 14.8, unstable ground refers to slope.

K.2 FOUNDATION ELEMENTS IN UNSTABLE GROUND

This design example is for the analysis required to use bridge foundation elements to restrain the movement of bridge embankment located above unstable ground. Prior to applying this procedure, the GEOR should have determined the SSL potential and estimated the residual strength of the liquefied soil. In addition, the GEOR should have determined that the ground will be unstable during the EE I limit state check. This method shall not be used to mitigate unstable ground during the Strength or Service limit state checks.

This example describes the load Case 1 (restrained ground displacement (see Chapter 14)), which is observed when the displacing soil crust is limited to the dimensions of the approach embankment (i.e., the width of the embankment). Therefore, it is assumed that the foundation is sufficiently stiff to partially restrain the movement of the failure mass.

Note that LPile 2019.11.06 for Networks was used to conduct the lateral pile analysis and that Slide2 9.010 was used to conduct the slope stability analysis. The use of these software by SCDOT does not imply endorsement of the software, nor establishes a requirement that these software be used by the GECs.

K.2.1 <u>Project description</u>

This example evaluates the pile supported end bent for a three-span continuous flat slab bridge, with an overall length of 100 ft located in Florence County, South Carolina. The site has a peak ground acceleration (PGA) of 0.35g and the moment magnitude (Mw) is 7.36 (Safety Evaluation Earthquake, SEE). Loadings consists of a combined design dead load and live load of 140 kips per pile (Table K-1). The foundation support for the end bents will be provided by HP14x73 steel piles. Figure K-1 sketches the soil strata and proprieties for this project. The abutment has limited width, allowing the foundation to provide a restrictive force against the soil displacement. Thus, the procedure for Case 1 described in Chapter 14 is used.

The main goal of the procedure is to find the displacement demand of the foundation due to the crustal movement. This example illustrates the methodology using a single bent analysis, which is sufficient for bridges with uniform foundations elements, and allows the demonstration of the "superpile" concept.

Table K-1, End Bent Design Pile Load Summary

Loading Case	Service Limit (kips)	Strength Limit (kips)	Extreme Event I (kips)
Axial – Compression (Fy)	98	140	60
Lateral – Transverse (Fx)	0.3	0.5	40
Lateral – Longitudinal (Fz)	1.3	2.2	39

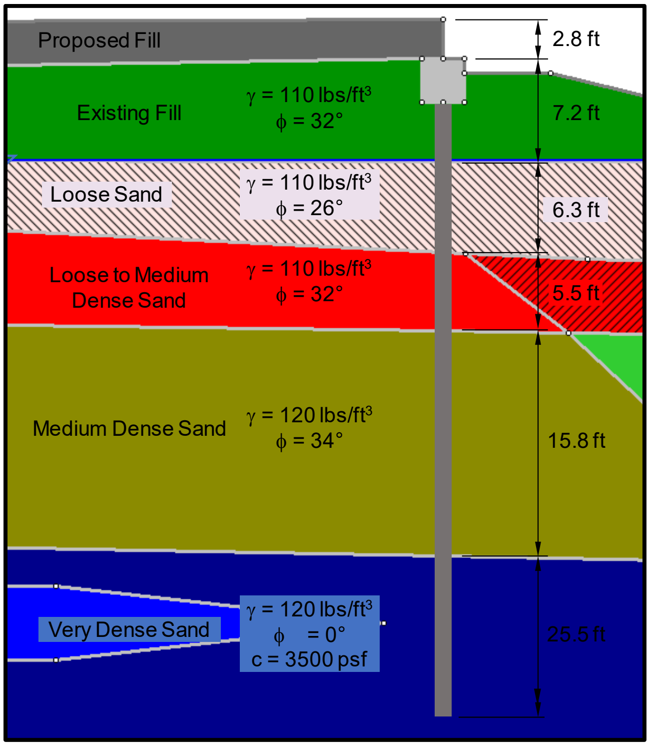


Figure K-1, Soil Properties

The objective of the first three steps is to find the foundation resistive force (shear demand) of the "superpile" due to the soil displacement.

K.2.2 Example Calculations

K.2.2.1 Foundation Model

For most bridges with uniform foundation elements, a single bent analysis is sufficient. The single bent analysis together with the concept of "superpile" allows the use of a lateral pile analysis software such as LPile. The first step is capturing the properties of the bent foundation by an equivalent (single) "superpile".

A non-linear elastic model is recommended. To create a non-linear "superpile" it is necessary to scale the moment-stiffness curve of a single pile by the number of piles in the group. The following will illustrate the use of LPile to generate the moment-stiffness curve for an HP 14x73 pile.

I. Generate the Moment-Stiffness curve for the typical pile.

a. Using the Nonlinear EI Only Mode computational option (Figure K-2), create the Moment-Stiffness curve for a single pile (Figure K-7). Define the single pile section type, dimensions and material properties (Figure K-3 to K-5). Please note that all figures are screen captures from LPile.

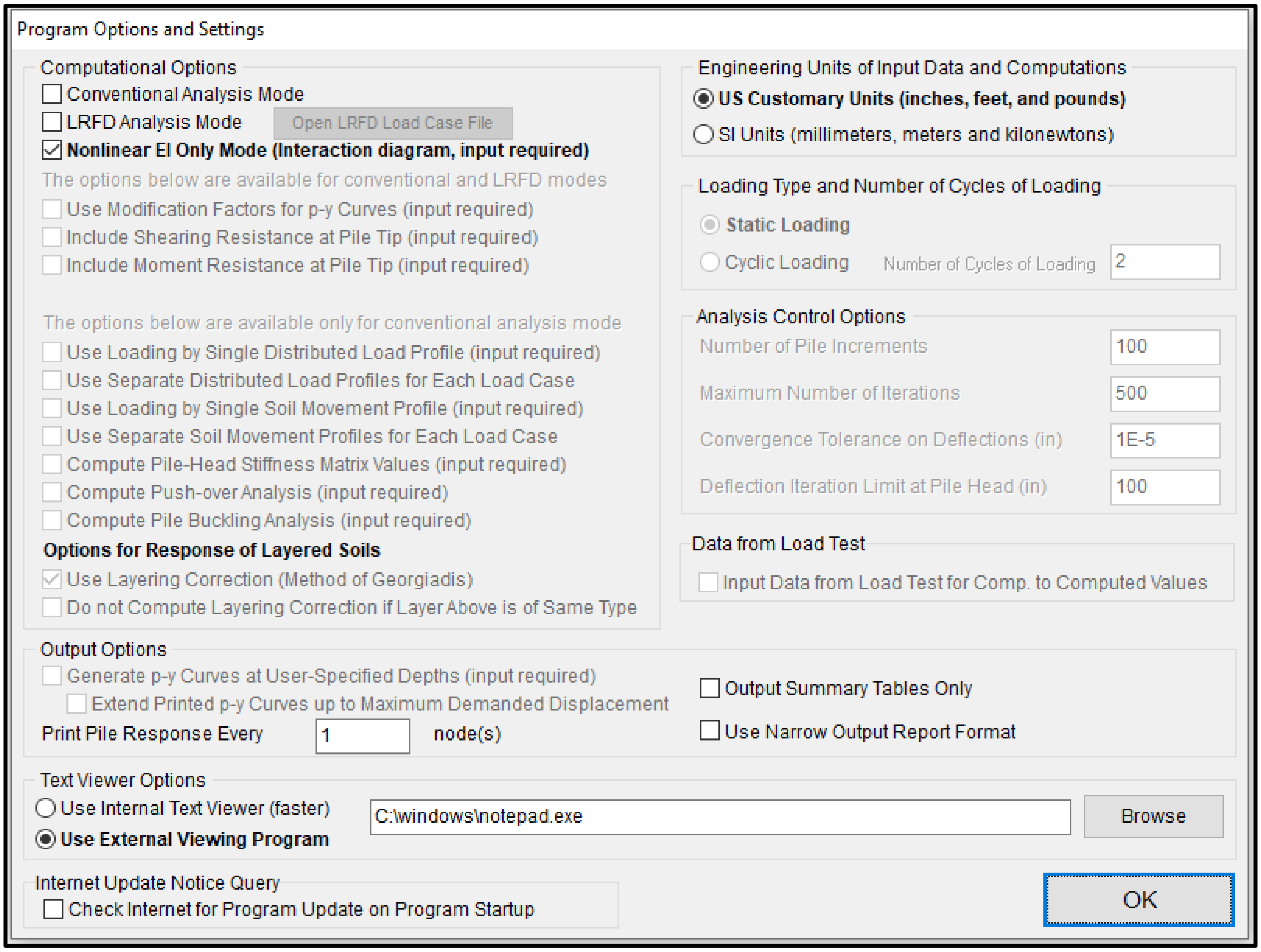


Figure K-2, Computational Options – Single Pile

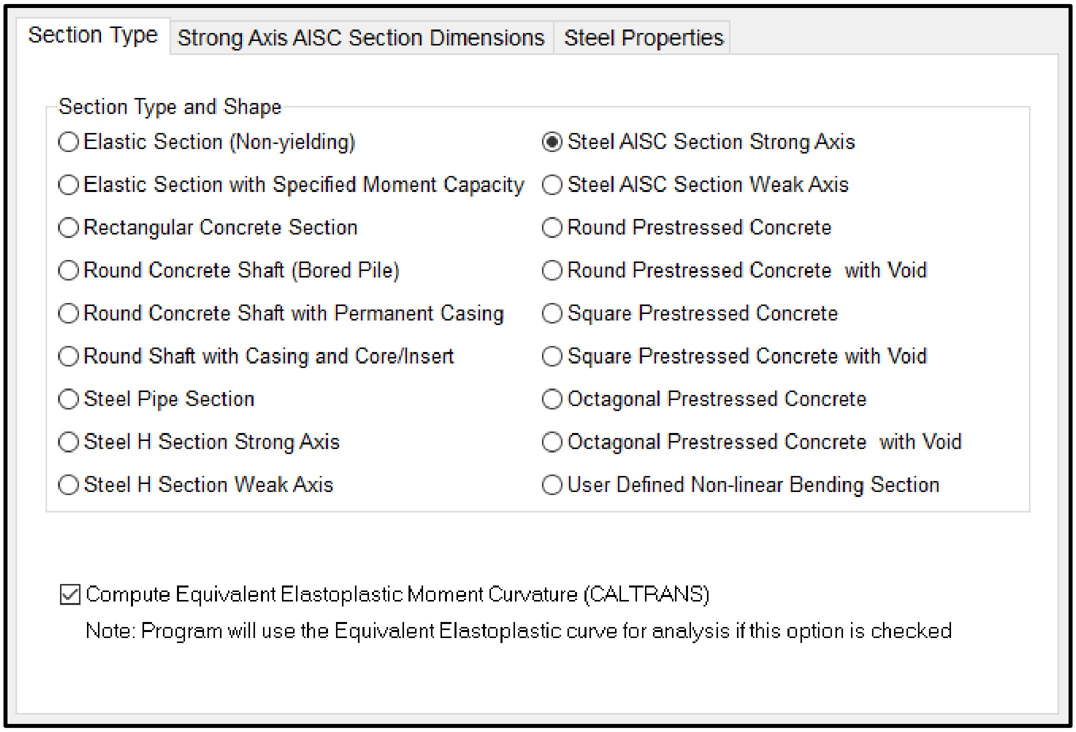


Figure K-3, Section Type – Single Pile

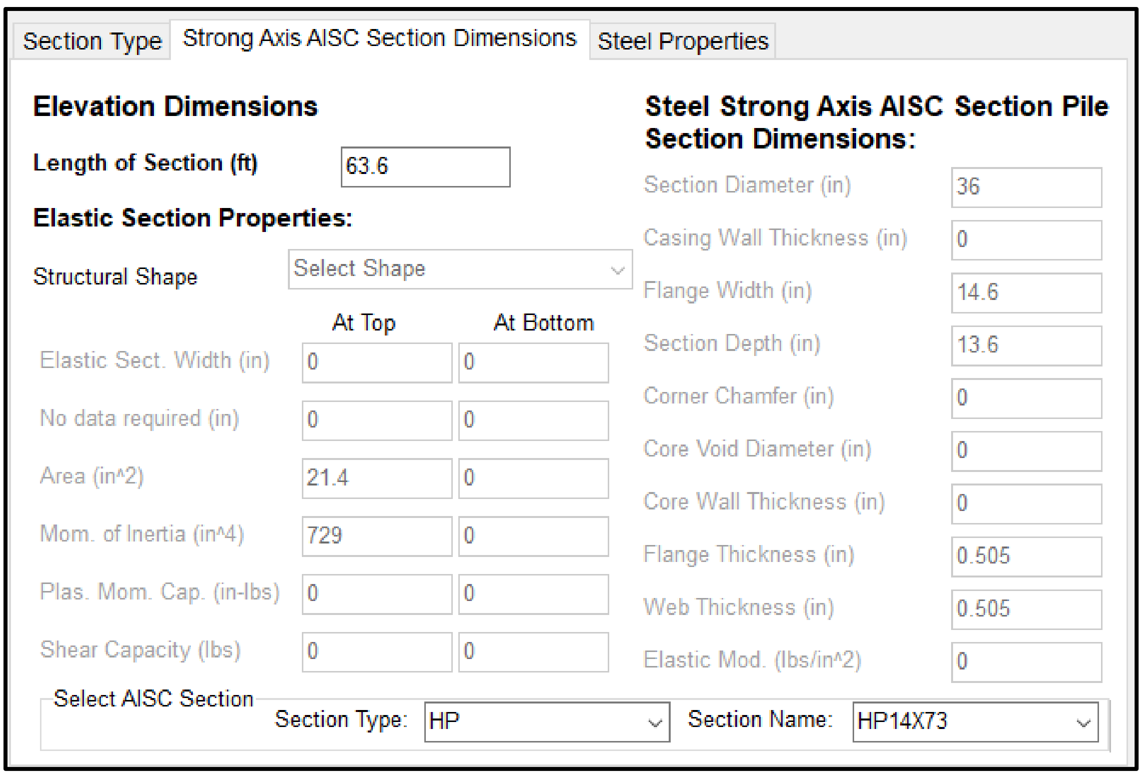


Figure K-4, HP 14 x 73 Section Dimensions

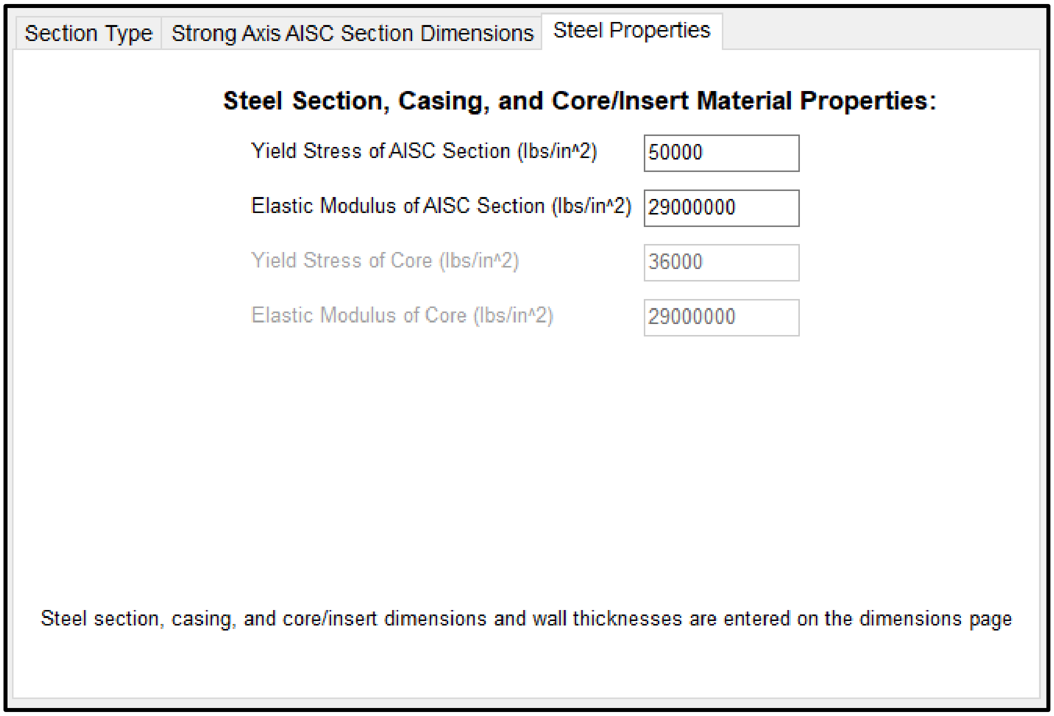


Figure K-5, HP 14 x 73 Steel Properties

b. Input the Axial Thrust loads (Figure K-6) for Interaction Diagram that includes the combination of dead and live loads.

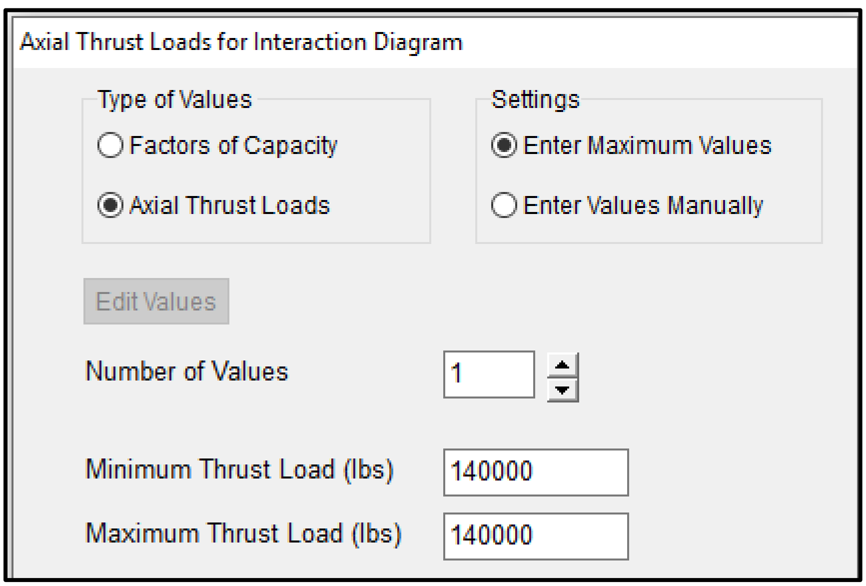
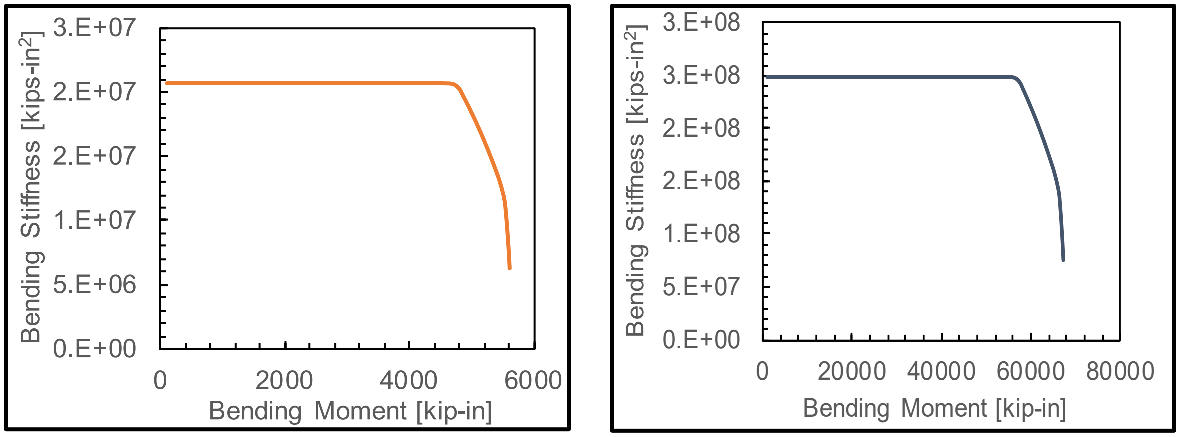


Figure K-6, Axial Thrust Loading

c. Run LPile to obtain the Moment-stiffness data from a single pile and scale up by the number of piles in the bent, n (Figure K-7a and b, respectively). For this example n = 12 piles.



a. Single pile Moment-Stiffness Curve (non-linear)

b. “Superpile” Moment-Stiffness Curve

Figure K-7, Moment-Stiffness Curves

The p-y curve for the cap/abutment-soil interaction is defined by a trilinear curve based on the ultimate crust load on the foundation elements, F_{ULT}, and the relative displacement required to achieve F_{ULT}, Δ_{MAX}. The trilinear curve is used as an input for the foundation model.

The efficiency factor is the average of the group reduction factors for each row in the pile group. For this methodology, the reduction factor for each row in the pile group is computed according with Chapter 16. For single row pile bents, which are the vast majority of the bridges in South Carolina, the group reduction factor is 1.0.

II. An example of the input required and the results of the development of the Force-Displacement Curve are provided. The numerical values assigned to the variables are assumed. Figure K-8 depicts the Idealized Pile-Cap Force-Displacement Curve.

Geometry:

n =	12		
B =	**14.0 in**	≈ 1.17 ft	B = Pile Diameter
c/c =	**86.0 in**		c/c = Pile Separation Center to Center
T =	**3.0 ft**		T = Pile cap thickness
W_T =	**83.7 ft**		W_T = Transverse Pile Cap Width
W_L =	**3.0 ft**		W_L = Longitudinal Pile Cap Width
D =	**2.8 ft**		D = Depth from Ground Surface to Top Cap

Crust data:

Select soil behavior **Sand-Like or c - ϕ soils**

γ =	**110.0 pcf**	γ = Effective Unit Weight
s_v =	**330.0 psf**	s_v = Mean Vertical Effective Stress
c =	**0.0 psf**	c = Undrained Shear Strength
ϕ =	**32°**	ϕ = Friction Angle $20 < \phi < 40$
δ =	**10.7°**	δ = Pile Cap-Soil Crust Interface Friction Angle

c' =	**0.0 psf**	c' = Cohesion Intercept
Z_c =	**10.0 ft**	Z_c = Crust Thickness
$\overline{H}$ =	**7.4 ft**	$\overline{H}$ = Average Pile Depth in the Crust
L_c =	**4.2 ft**	L_c = Length of Pile Extending Through Crust
α =	0.5	α = Adhesion Factor

Determination of F_{ULT}

K_a =	0.307	K_a = Crust materials active earth pressure coefficient

Case A:

Select location of piles	**Row 3 and higher**	
Center to center pile spacing	6 B	
Group reduction factor (GRF)	1	(GDM Chapter 16)

P_{ULT} =	17083 lb/ft	P_{ULT} = Ultimate lateral resisting force per unit length
$F_{PILES\text{-}A}$ =	867732 lbs	$F_{PILES\text{-}A}$ = Ultimate resistance of individual pile length above liquefied zone
$K_{p,log\text{-}spiral}$ =	4.16	K_p= Crust materials passive earth pressure coefficient
$K_{w,CASE\ A}$ =	1.41	K_W = Adjustment factor for wedge shaped failure surface
$F_{PASSIVE\text{-}A}$ =	487018 lbs	$F_{PASSIVE}$ = Passive force from compression of soil on up-slope face of foundation
$F_{SIDES\text{-}A}$ =	1119 lbs	F_{SIDES} = Friction or adhesion of soil moving along side of foundation
$F_{ULT\text{-}A}$ =	1355869 lbs	

Case B:

$K_{p,Rankine}$ =	3.25	K_p= Crust materials passive earth pressure coefficient
$K_{w,CASE\ B}$ =	1.32	K_W = Adjustment factor for wedge shaped failure surface
$F_{PASSIVE\text{-}B}$ =	354866 lbs	$F_{PASSIVE}$ = Passive force from compression of soil on up-slope face of foundation
$F_{SIDES\text{-}B}$ =	2697 lbs	F_{SIDES} = Friction or adhesion of soil moving along side of foundation
$F_{ULT\text{-}B}$ =	357564 lbs	

$F_{ULT\text{-}A}$ > **$F_{ULT\text{-}B}$** **Case B controls!**

Determination of Δ_{MAX}

f_{depth} =	0.01	f_{depth}= factor to account the effect of crust thickness with respect to the pile cap thickness

f_{width} = 0.99 f_{width}= factor to account the effect of crust thickness with respect to the pile cap width

Δ_{MAX} = 2.0 in

Table K-2, Data Point for Idealized Force-Displacement Behavior of Pile Cap

F_{ULT} [lbs]	p_{ULT} [lbs/in]	Δ_{MAX} [in]
0	0	0.00
178782	4966	0.51
357564	9932	2.03
357564	9932	4.07

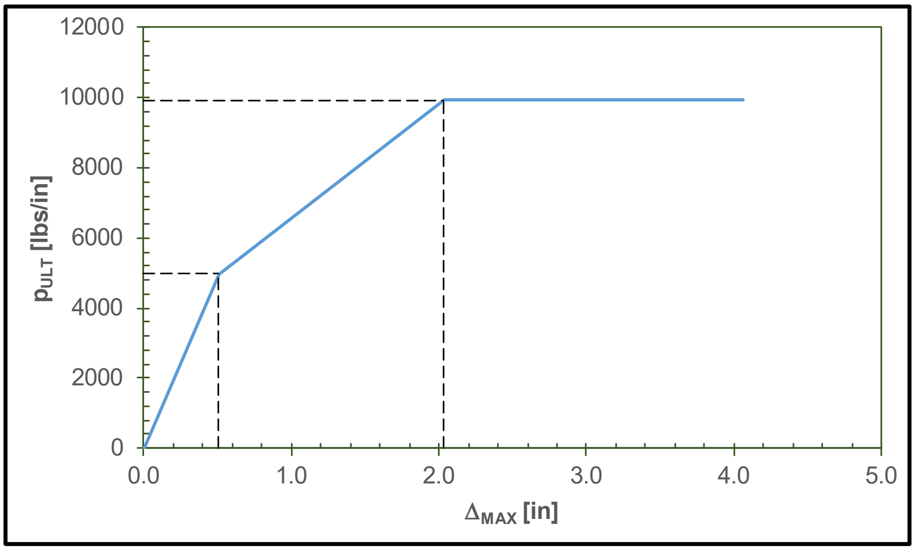


Figure K-8, Idealized Force-Displacement Behavior of Pile Cap.

In addition to the definition of the non-linear "superpile" and the p-y curve for the cap/abutment-soil interaction, define the rotational stiffness of the pile in the transverse direction, if applicable. For this case study and most bridges, a single row of piles does not provide an arm, therefore, x_i will be equal to zero and the rotational stiffness may be neglected.

III. Create the foundation model on LPile. Figure K-9 shows computational options on LPile for the foundation model. The pile head is modeled free to rotate and move laterally. Select in Lpile the pile-head loading conditions for shear and moment.

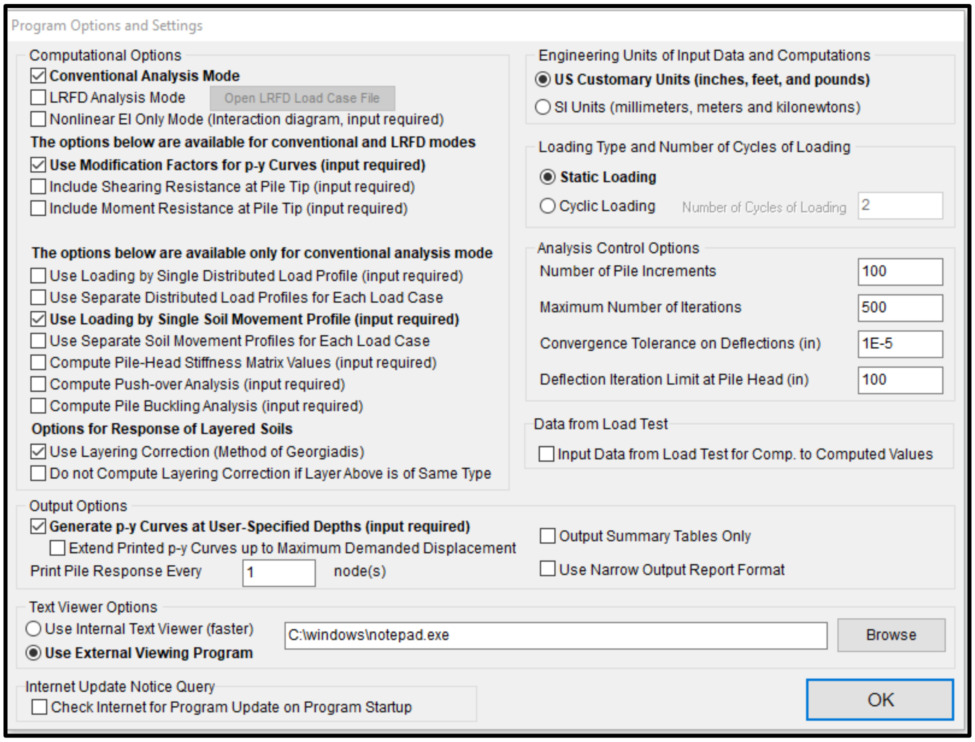


Figure K-9, Computational Options - "Superpile"

Define the non-linearity of the "superpile" selecting the "User Defined non-linear Bending Section" in the section type options (Figure K-10).

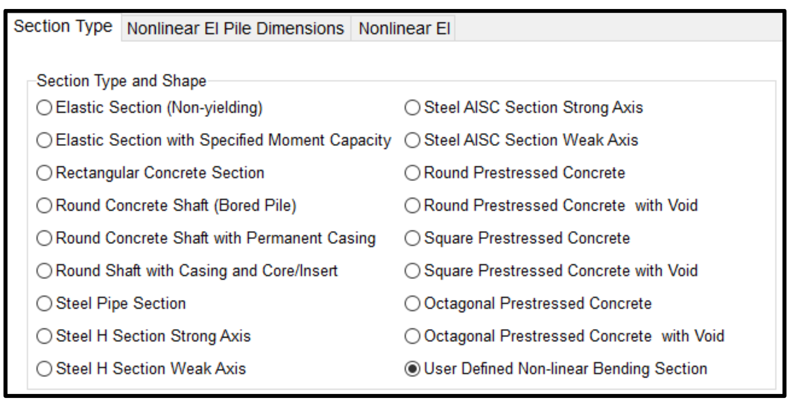


Figure K-10, Selection of Section Type - "Superpile"

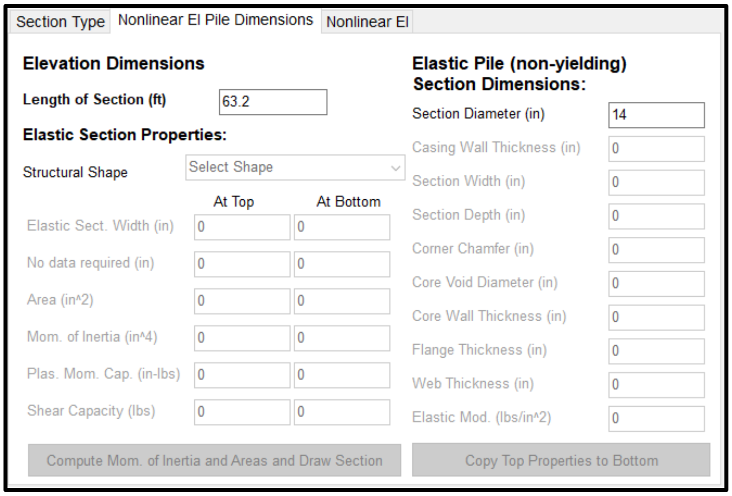
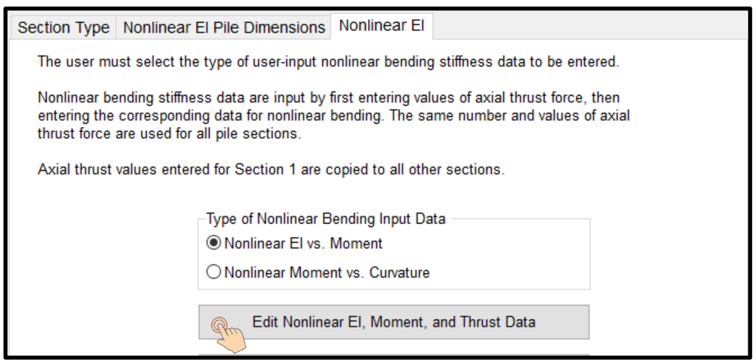
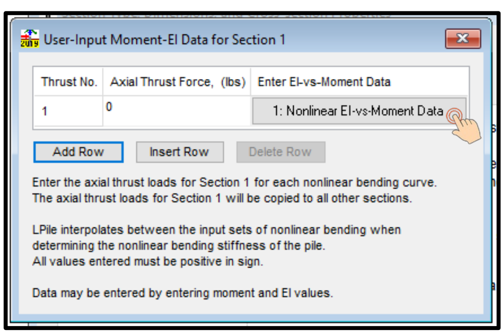


Figure K-11, “Superpile” dimensions and length

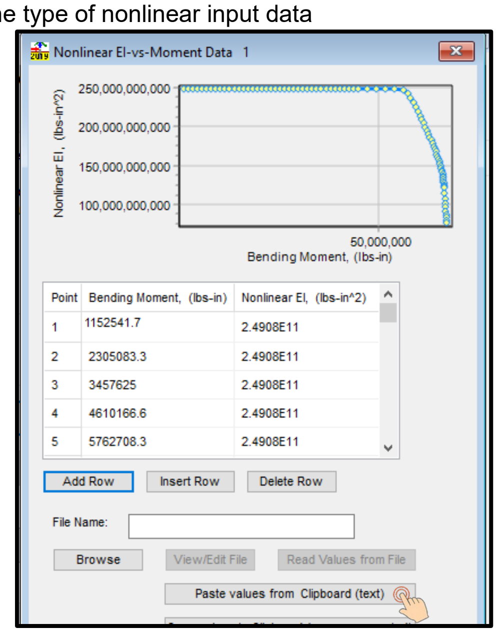
Copy data from moment-stiffness curve for the equivalent “superpile” and paste it in nonlinear EI-vs-Moment Data. Figure K-12 shows the step by step to input moment-stiffness curve for the foundation model.



(a) Select Nonlinear EI-vs-Moment as the type of nonlinear input data



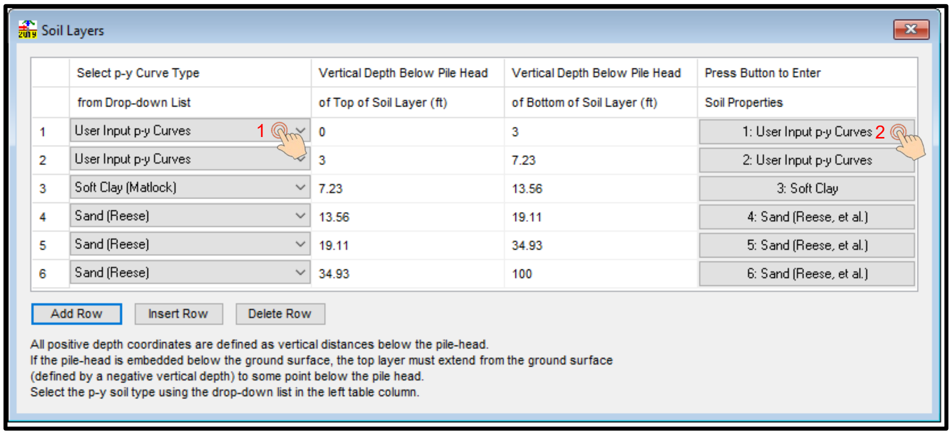
(b) Input the scaled curve by clicking on Nonlinear EI-vs-Moment Data



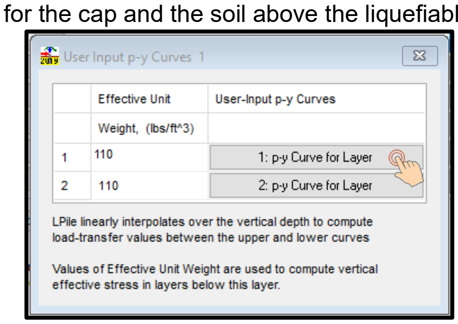
(c) Paste the data points

Figure K-12, “Superpile” Scaled Moment-Stiffness Curve Input

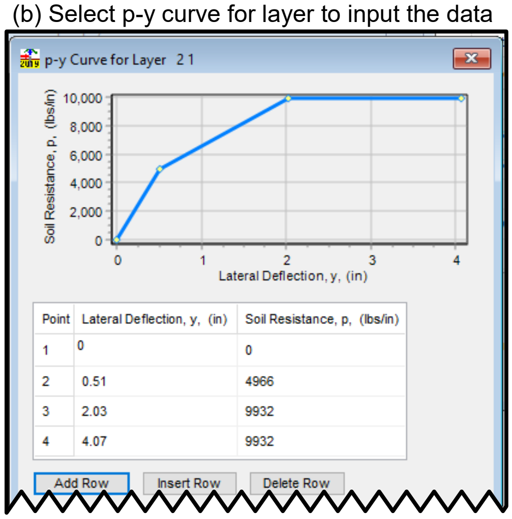
Input the soil layer properties. Use the option to input a user-defined p-y curve to model the crust (Figure K-13). Use the p-y curve for idealized force-displacement behavior generated previously.



(a) Select User Input p-y curves for the cap and the soil above the liquefiable layer and click on soil properties



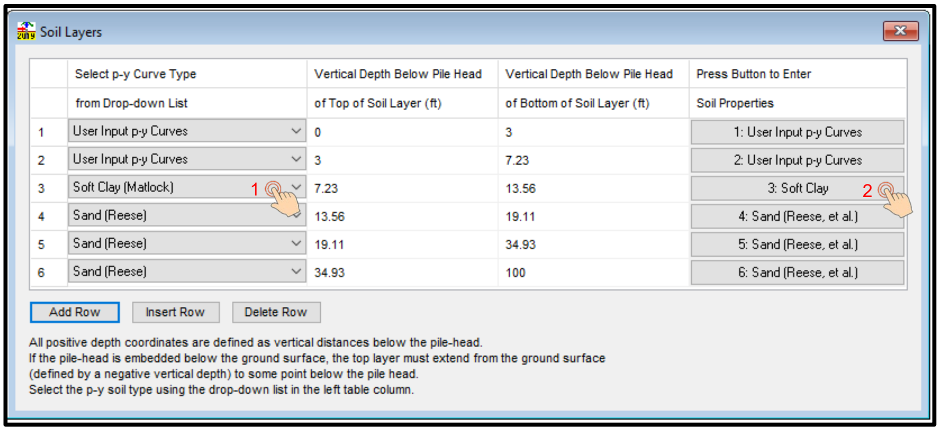
(b) Select p-y curve for layer to input the data



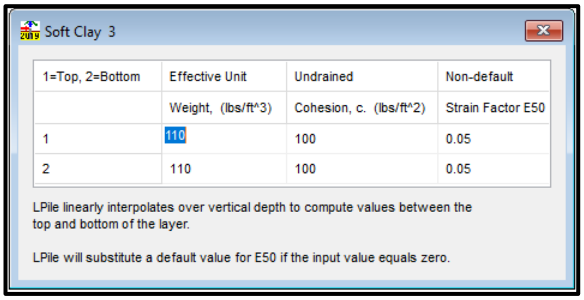
(c) Paste data to generate trilinear curve (Figure K-8)

Figure K-13, User-defined P-y curve for crust modeling

The liquefied sand could be modeled using the residual strength model or as a soft clay. In this example, the liquefied layer was modeled as a soft clay equating the cohesive strength equal to the residual strength of liquefied sand (Figure K-14). In addition, the strain factor is taken equal to 0.05 as recommended in the LPile Technical Manual.



(a) Select soft clay as p-y curve type, click on soil properties



(b) Input the corresponding data for the liquefiable layer

Figure K-14, Liquefied Soil Layer Modeled as Soft Clay

The corresponding "p" in the p-y curves for a single group pile must be scaled by a factor equal to the number of piles multiplied by the group efficiency factor. Modify the p-value of the curve using the option to modify the p-y factors under the data tab (Figure K-15).

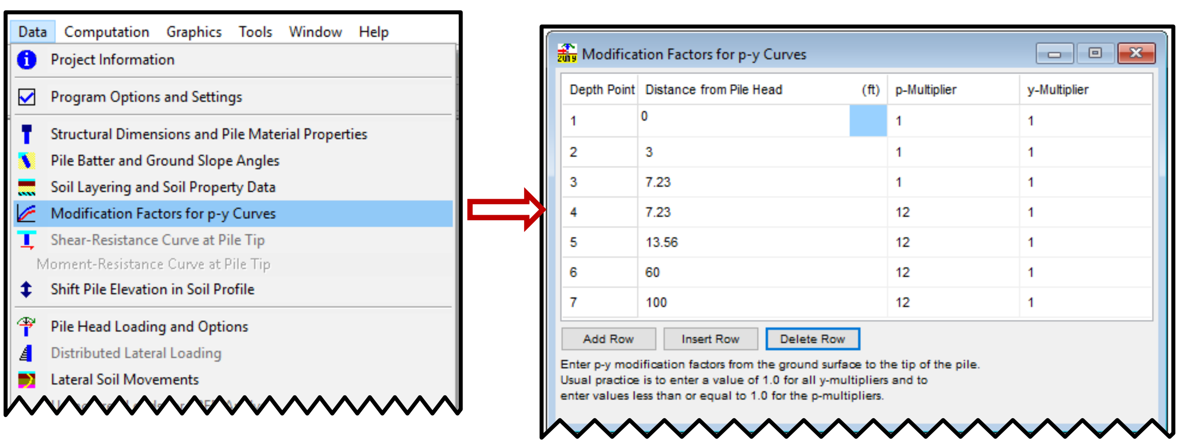


Figure K-15, p-value Modification to Create "Superpile"

The group reduction and efficiency factor are equal to 1.0. Multiply p-value by a factor equal to 12 corresponding to the number of piles in the cap. Do not account for group effects in the liquefied layer.

K.2.2.2 Bridge Longitudinal Resistance

The next step is to determine if the bridge can provide any lateral resistance to the movement of the abutments. This type of analysis is typically performed by the SEOR with input from the

GEOR. The restraining force from the bridge superstructure depends on the structural configuration, the characteristics of the embankment soils, as well as the capacity of the adjacent opposite abutments. The restraining force that develops at the abutment must be transferred to the intermediate bents and the opposite abutment. If superstructure restraint is considered in the lateral spreading analysis, it is recommended that a global structural model is used. This type of modeling is not anticipated being performed on typical SCDOT projects. For this example, it is assumed that there is not longitudinal resistance from bridge deck.

K.2.2.3 Foundation Model Displacement Analysis

A displacement analysis of the foundation model should be performed next. Impose a series of increasing soil displacement profiles on the foundation model (Figure K-16). For each displacement increment, determine the shear force on the foundation at the center of the liquefiable layer. Define this as the foundation resistive force corresponding to a given crustal displacement, R.

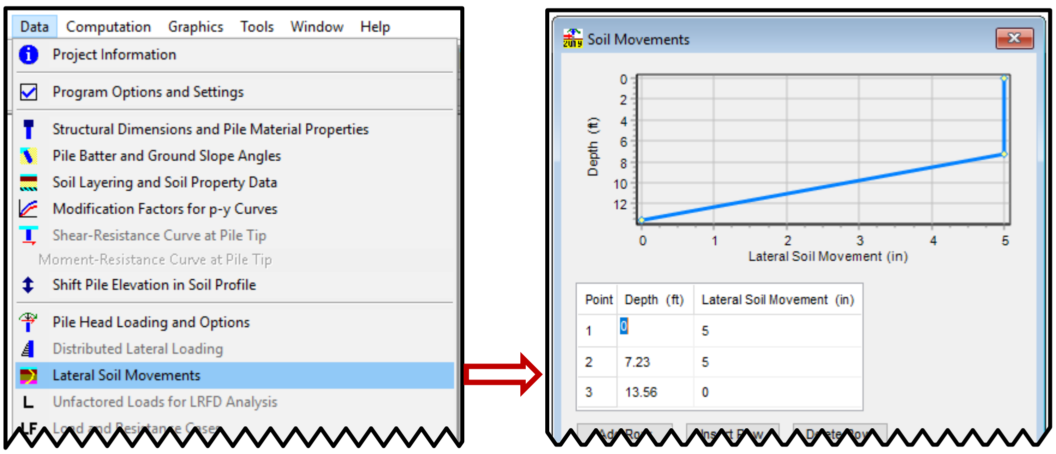


Figure K-16, "Superpile" Model Displacement Profile

The soil displacement is incremented every 2.5 in up to 20 in. For each point read the shear force in the "superpile" at the middle of the liquefiable layer and record it. To compensate the effect of the change of the sliding mass, use the running average of the shear force. A summary of the "superpile" analysis is presented in Table K-3 and plotted in Figure K-17.

Table K-3, Crustal Displacement Data Points and Foundation Resistive Force

Data point	d [in]	R [kips]	$R_{running\ avg}$ [kips]
1	2.5	357.99	358.0
2	5	578.42	468.2
3	7.5	608.69	515.0
4	10	613.9	539.8
5	12.5	617.37	555.3
6	15	619.51	566.0
7	17.5	621.28	573.9
8	20	622.82	580.0

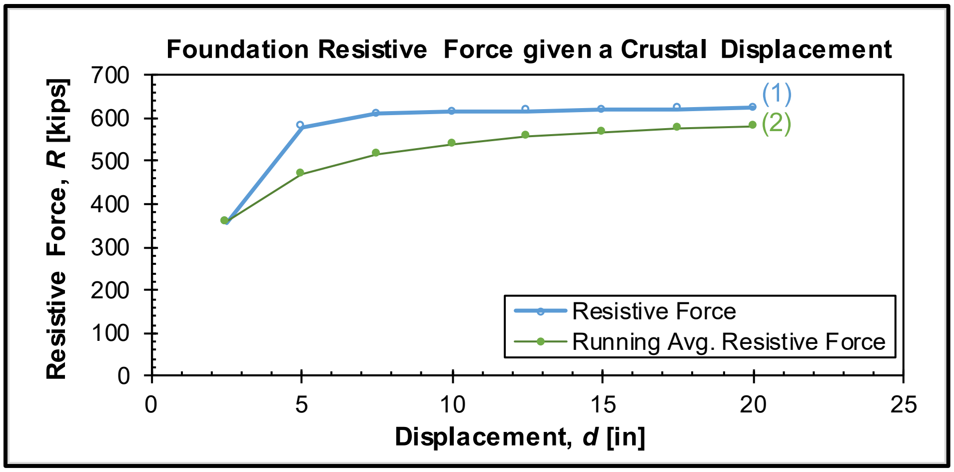
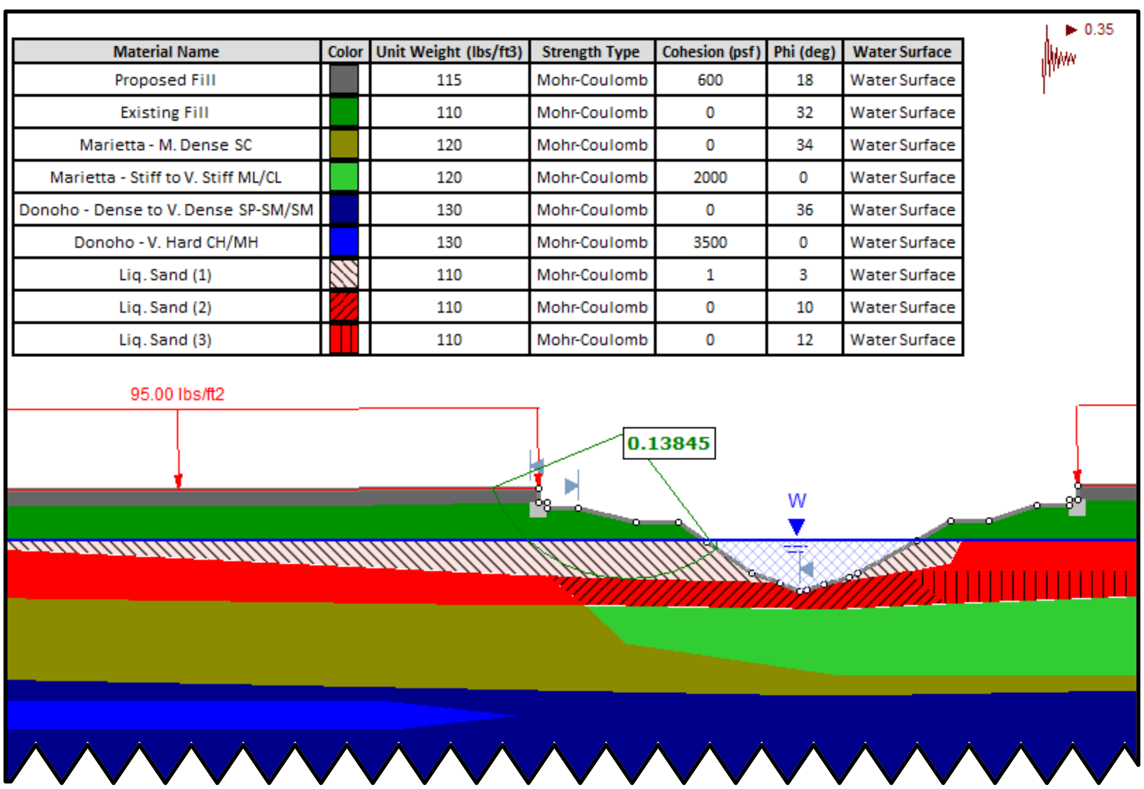


Figure K-17, Foundation resistive force given a crustal displacement

K.2.2.4 Bridge Embankment Slope Stability and Deformation Analyses

This example uses the stability model in Slide to determine the foundation resisting forces at the center of the liquefied layer for a series of horizontal accelerations.

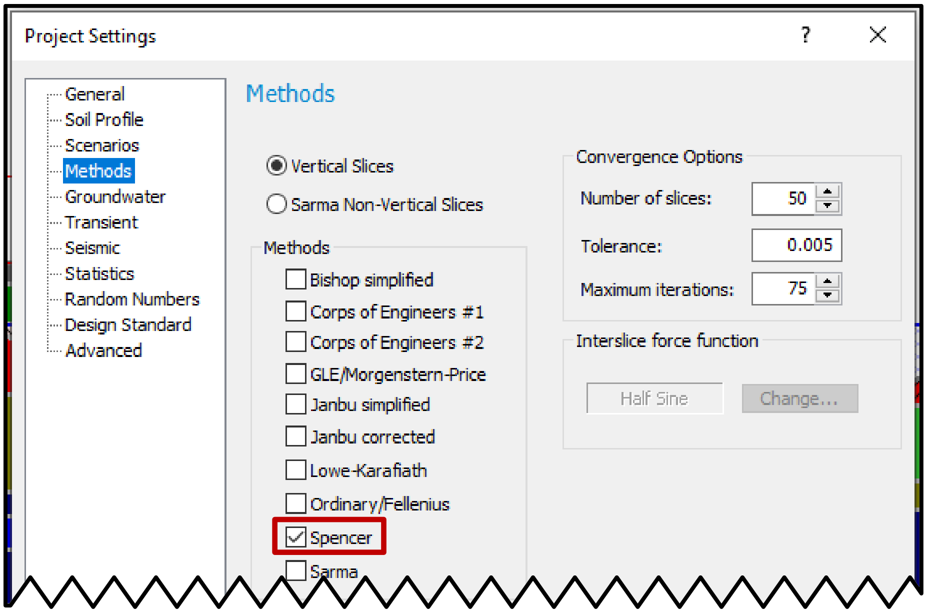
Run the slope stability analysis for the EE I with a peak ground acceleration equal to 0.35g to identify the location of the failure surface (Figure K-18). The failure surface needs to pass through the middle of the liquefied layer. A non-circular surface option and block search for this analysis is recommended (Figure K-20). Longitudinally, the failure surface is limited to four times the embankment thickness.



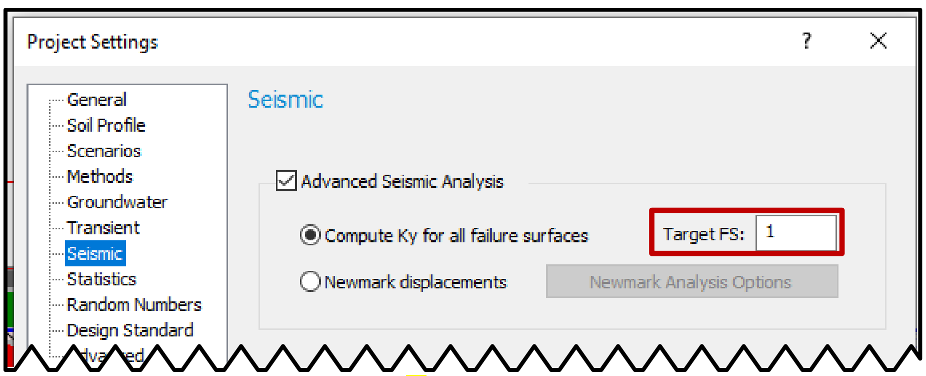
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
Proposed Fill		115	Mohr-Coulomb	600	18	Water Surface
Existing Fill		110	Mohr-Coulomb	0	32	Water Surface
Marietta - M. Dense SC		120	Mohr-Coulomb	0	34	Water Surface
Marietta - Stiff to V. Stiff ML/CL		120	Mohr-Coulomb	2000	0	Water Surface
Donoho - Dense to V. Dense SP-SM/SM		130	Mohr-Coulomb	0	36	Water Surface
Donoho - V. Hard CH/MH		130	Mohr-Coulomb	3500	0	Water Surface
Liq. Sand (1)		110	Mohr-Coulomb	1	3	Water Surface
Liq. Sand (2)		110	Mohr-Coulomb	0	10	Water Surface
Liq. Sand (3)		110	Mohr-Coulomb	0	12	Water Surface

Figure K-18, EE I Slope Stability Analysis

The following figures depict the options selected for the creation of the slope model. Select the Spencer option as the methodology of analysis, required per Chapter 17 and the seismic option to compute a yielding coeffcient with a target factor of safety equal to one (Figure K-19). Notice that the parameter "Ky" in Slide to refers to the yield coefficient (called k_c in Chapter 14) and not the yield acceleration (k_y) as defined in the Chapter 13.



(a) Select Spencer as analysis method



(b) Compute k_c for all failure surfaces

Figure K-19, Slope Stability Analysis Settings

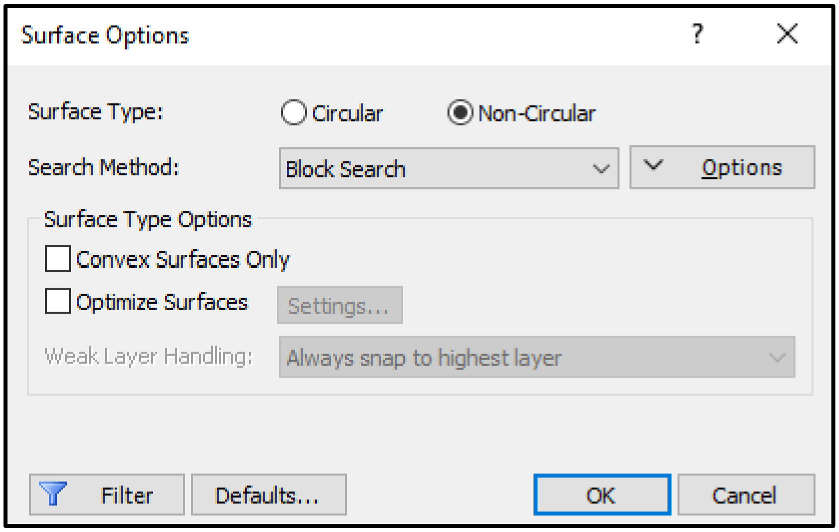


Figure K-20, Surface Option Selection

Place an additional vertex in the middle of liquefied layer directly below the cap to place the restraining force (point force) representative of the resistive force provided by the "superpile" (Figure K-21). Use the option to add a polyline under the block search option to place the failure surface in accordance with the vertical and horizontal limits considerations (Figure K-22). Create the block failure surface to pass slightly under this point of force application. If the point force is placed on the failure surface, the model does not take into account the force acting against the displacement of the crust. For this example, the failure surface is placed two inches below the application point of the resistive force.

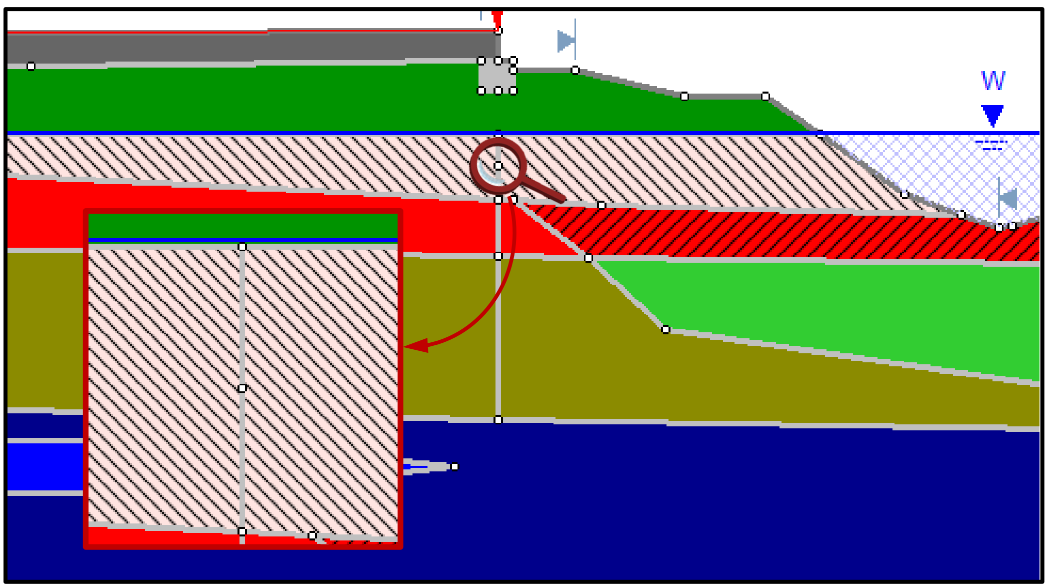


Figure K-21, Additional Vertex Location

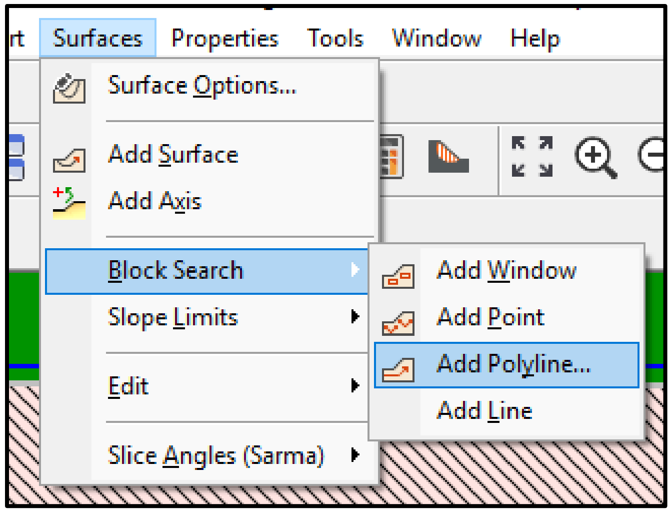


Figure K-22, Block Search Location Option

The height of the embankment in the longitudinal direction, H, is measured from the top of the roadway to the top of slope. For this case H = 19.2 ft, 4*H = 76.8 ft. The lateral constrain is placed at 63 ft behind the top of the roadway, which is less than 4H (Figure K-23).

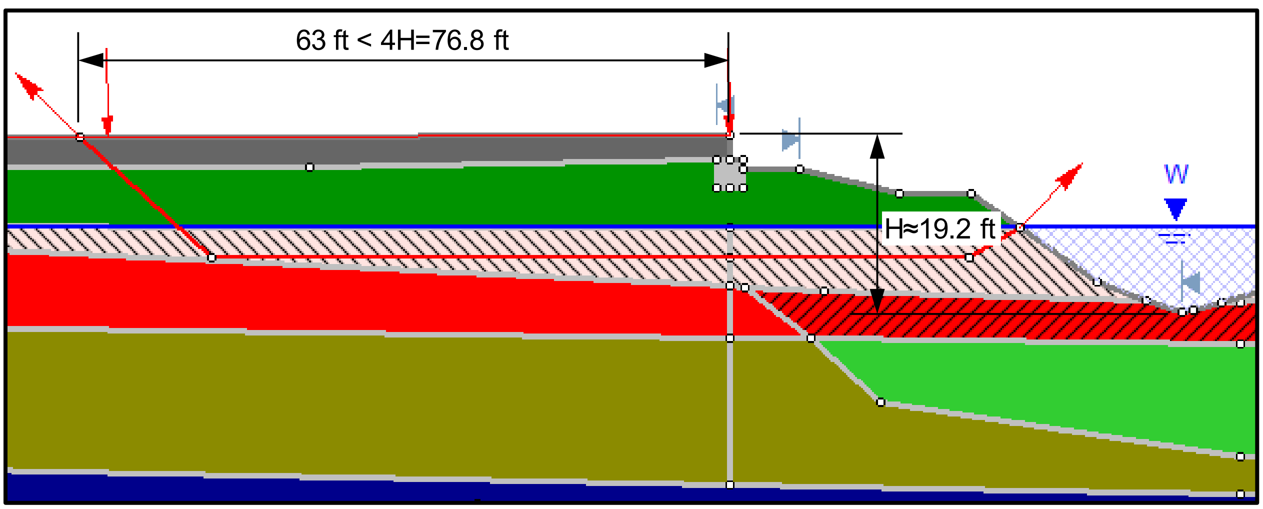


Figure K-23, Block Failure Surface Limits

The next step is to apply a constant force at the middle of the liquefied layer, representing the resistive force provided by the foundation, and find the corresponding yield coefficient, k_c, for a target factor of safety equal to 1. With the resultant yielding coefficient, calculate the corresponding displacement using Bray and Travasarou (2007) for the Newmark rigid sliding block case (Equation K-1).

Equation K-1

$$\boldsymbol{d = Exp\left[-0.22 - 2.83ln(k_c) - 0.333\left(ln(k_c)\right)^2 + 0.566ln(k_c)ln(PGA) + 3.04ln(PGA) - 0.244\left(ln(PGA)\right)^2 + 0.278(M_w - 7)\right]}$$

The first point of the curve is where the constant force equals to zero (Figure K-24):

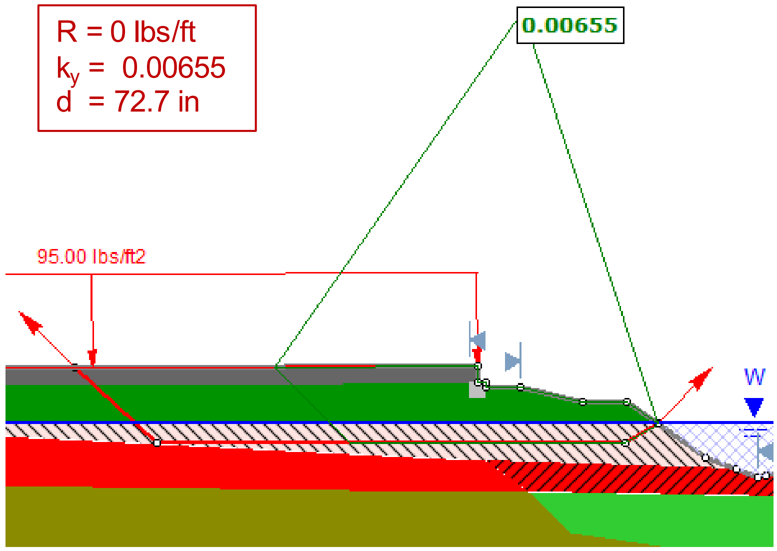


Figure K-24, Yield Coefficient at a Resistive Force Equal to zero

Next, apply a series of constant forces at the middle of the liquefied layer and determine the corresponding yield coefficient to generate the points for the curve (Figure K-25):

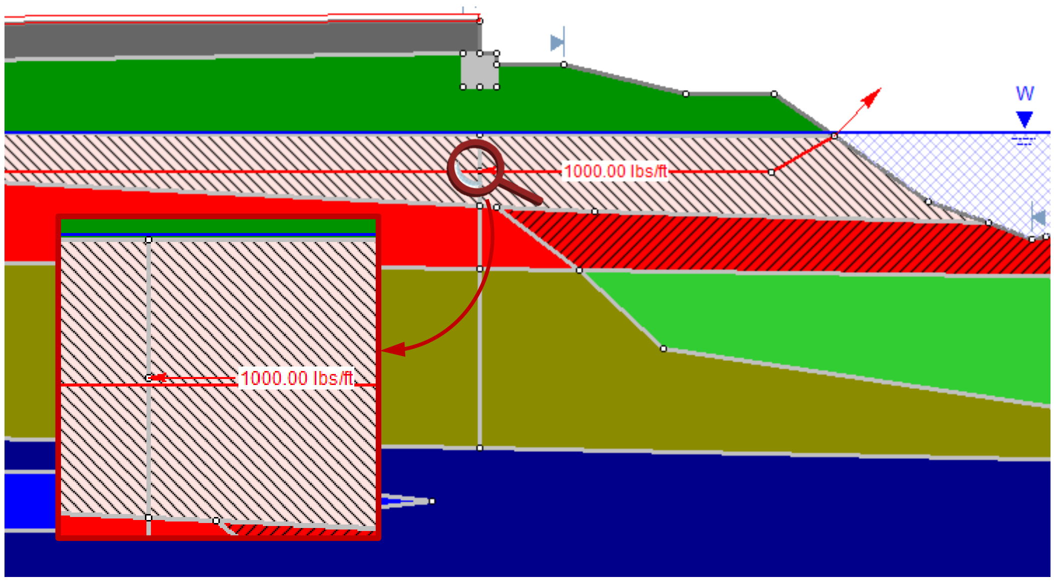
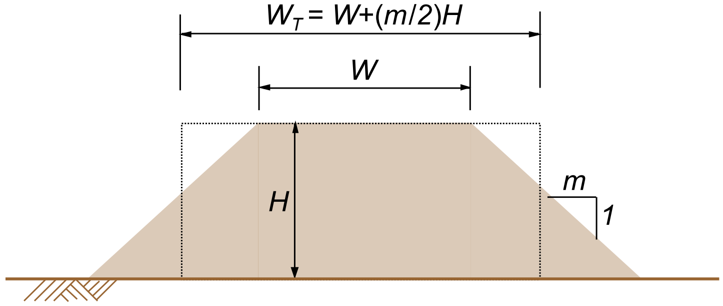


Figure K-25, Constant Force Application to Determine Yield Coefficient

Prior creating the curve, make an adjustment for non-rectangular embankment shape multiplying the force by the tributary width of the sliding mass, W_T (Figure K-31)



H =	**9.2 ft**
W =	**92.0 ft**
m =	**2**
W_T =	**101.2 ft**

Figure K-26, Non-rectangular embankment adjustment

Table K-4 shows the applied resistive force and the resultant yield coefficient, k_c. Plot the total resistive force provided for the embankment vs. the crust displacement calculated with Equation K-1 (Figure K-27).

Table K-4, Resistive Force Anticipated Crustal Displacement

R_{dist} [lbs/ft]	k_c	d [cm]	R [kips]	d [in]
0	0.00655	184.6	0	72.7
100	0.00783	180.2	10.1	70.9
1000	0.01376	145.3	101.2	57.2
2000	0.02584	88.9	202.4	35.0
3000	0.05438	35.4	303.6	13.9
4000	0.06646	25.9	404.8	10.2
5000	0.07535	21.1	506.0	8.3
6000	0.08418	17.4	607.2	6.8
7000	0.09299	14.5	708.4	5.7
8000	0.10194	12.2	809.6	4.8
9000	0.16818	4.3	910.8	1.7
10000	0.16818	4.3	1012.0	1.7

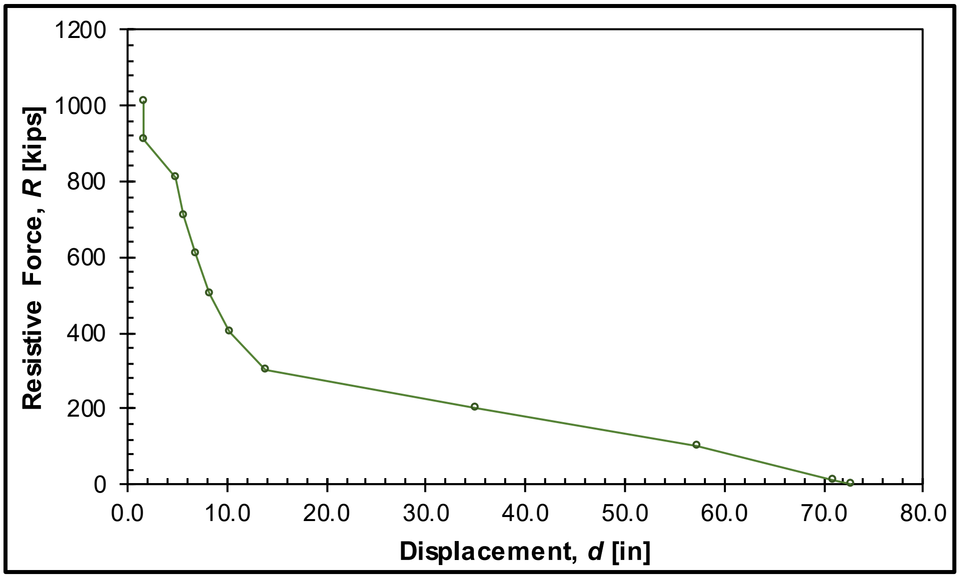


Figure K-27, Resistive Force vs Anticipated Crustal Displacement

K.2.2.5 Target Displacement Determination

Combine the results from Subsections K.2.1.3 and K.2.1.4 to determine the target displacement. Plot the displacement compatibility curve with these data (Figure K-28). Use the running average for the foundation resistive force given a crustal displacement and the expected crustal displacement for a given resistive force. The displacement corresponding to the intersection of these two curves represents the expected displacement demand on the foundation (Figure K-29).

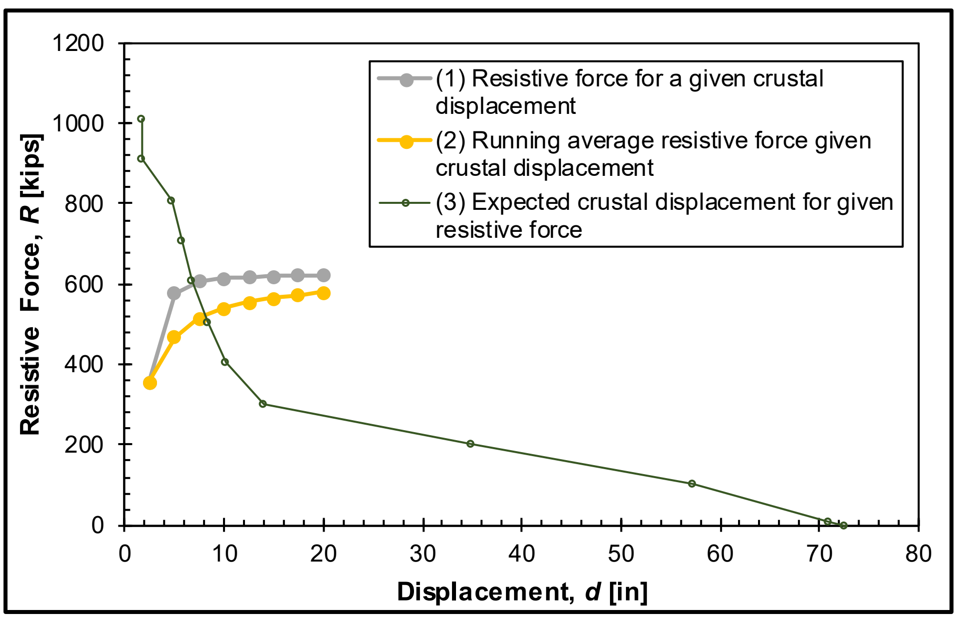


Figure K-28, Determination of compatible displacements

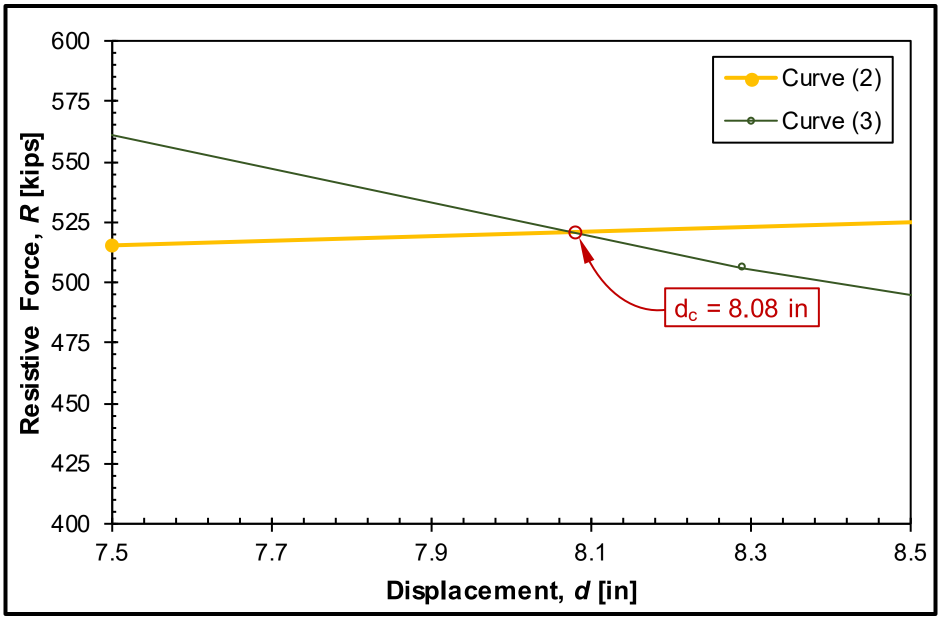


Figure K-29, Expected crustal displacement given a constant resistive force

K.2.2.6 Calculate Foundation Demands

The adequacy of the pile (structural) capacity is generally performed by the SEOR. The SEOR evaluates the performance of the foundation checking the displacement, shear and moment demand for an imposed soil displacement found in Subsection K.2.2.1. The methodology assumes that the unstable soil will occur during strong shaking and the inertial loading of the foundation must be considered in tandem with kinematic loading (Equation K-2). However, for the case of freestanding abutment it is recommended to ignore the inertial forces since the backwall is generally designed as a weak fuse. A soil profile with a maximum displacement equal to 8.1 inches is imposed on the foundation model in tandem with the inertial loads (Figure K-30).

Equation K-2

$$\mathbf{100\%\ kinematic \pm 50\%\ inertial} \rightarrow (\text{peak pile cap displacement, moment or shear})$$

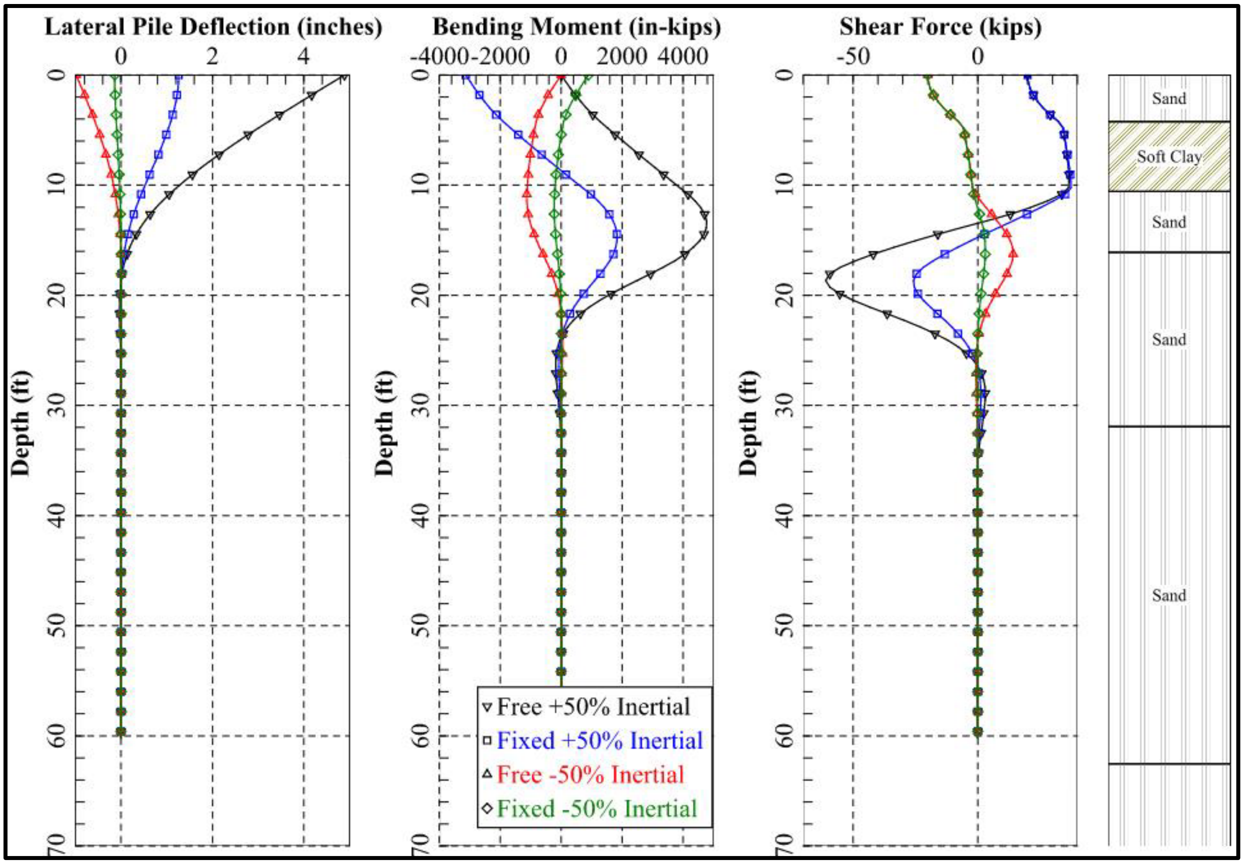


Figure K-30, Profile of Displacement, Moment and Shear (HP14x73 piles)

The moment demand is 4,775.3 kip-in at a depth of 13.25 ft from the surface. The capacity of the HP14x73 is 4,361 kip-in, which indicates the formation of a plastic hinge. Plastic hinging is allowed by the SDOT Seismic Design Specifications with the permission from the Regional Production Group (RPG), in consultation with the Geotechnical and Structural Design Support Engineer. Alternatively, the piles size could be increased to avoid the formation of plastic hinges. For this example, when the pile size was increased to HP14x89, the target displacement was 11% smaller (Figure K-31) and there were not plastic hinges forming due to the imposed displacement (Figure K-32). The HP14x89 has a moment demand of 5,939 kip-in and a capacity of 6,599 kip-in.

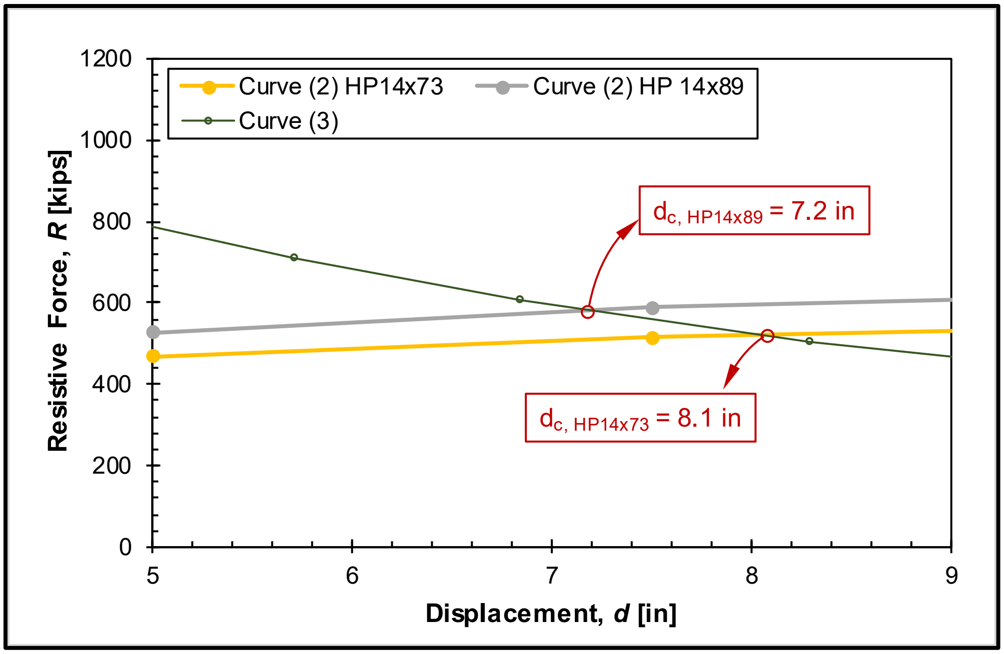


Figure K-31, Expected crustal displacement for HP14x89 piles

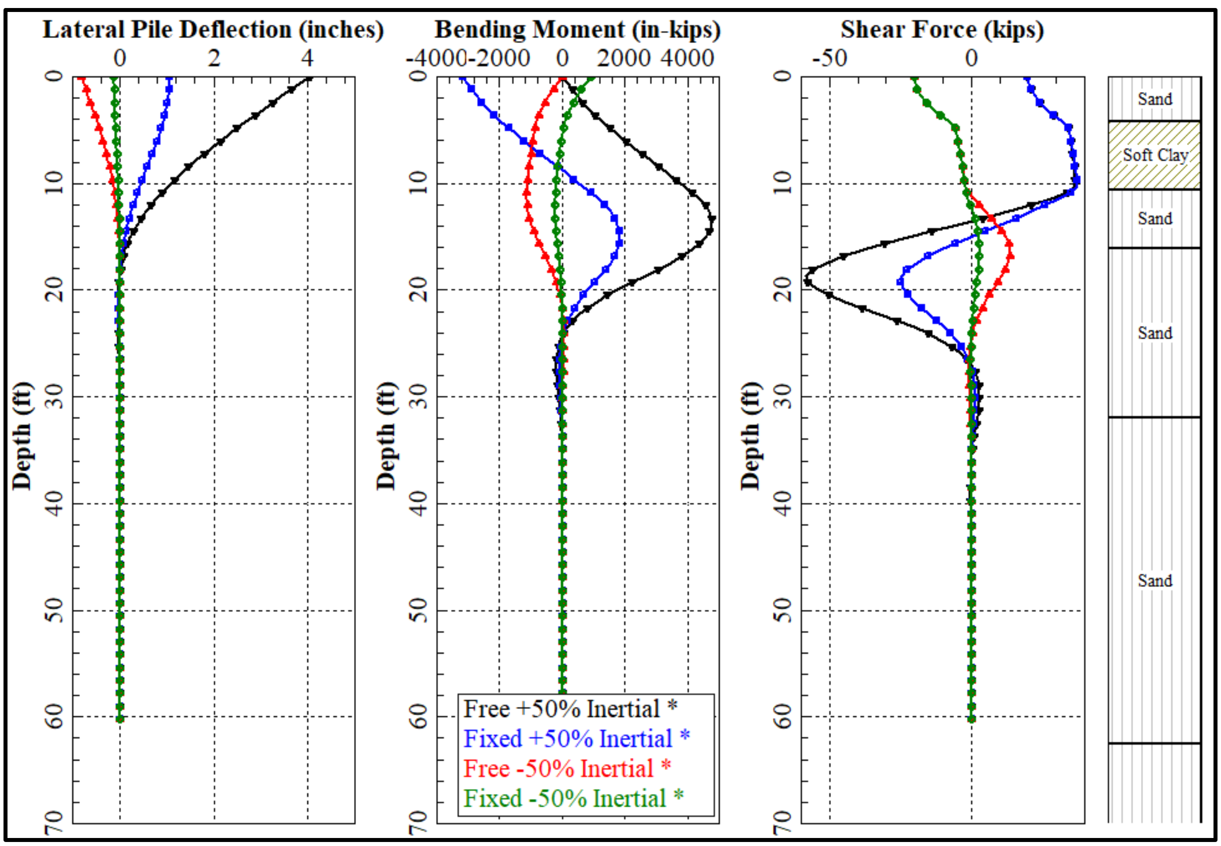


Figure K-32, Profile of Displacement, Moment and Shear (HP14x89 piles)

K.3 REFERENCES

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