

To: Dave Peterson, P.E., Structures Project Manager
From: August Arles, Geotechnical Engineer via Callie Ewald, P.E., Geotechnical Engineering Manager
Date: September 11th, 2026
Subject: Mount Holly BF 1443(56) – Integral Abutment Recommendations

1.0 INTRODUCTION

As requested, we have completed our geotechnical and geological analyses for the replacement of Bridge No. 64 as part of the Mount Holly BF 1443(56) project. Bridge No. 64 carries North Bowlsville Rd. (TH-17) over the Mill River Branch, and is located approximately 50 ft southwest from the intersection of VT Route 103 and TH-17 in the town of Mount Holly, VT. The subject project consists of replacing the existing structure on a new alignment with an integral abutment bridge founded on driven piles. Contained herein are the results of our geotechnical recommendations for pile supported integral abutments as determined using 2024 AASHTO *LRFD Bridge Design Specifications*.

2.0 FIELD INVESTIGATION

An initial field investigation was conducted from July 20th through August 17th, 2021, which consisted of two standard penetration borings and is summarized in the Preliminary Geotechnical Information Report submitted by Eric Denardo, P.E., to Laura Stone, P.E., on September 24th, 2021. Due to a change in the alignment for the replacement structure, a subsequent field investigation was conducted by Seaboard Drilling, overseen by WSP Inc, between January 23rd, and February 8th, 2024. The second investigation consisted of three standard penetration borings within the new alignment of the bridge, and the revised report was submitted on April 3rd, 2025. The two reports summarize the subsurface investigations and findings. Information from these reports was used to estimate the soil and rock material parameters used in these analyses. The boring logs from both investigations can be found at the end of this report.

3.0 ANALYSIS

Developed by Ensoft Inc., LPile, version 2026.13.02, is a multi-aspect software that allows the user to analyze a single pile in two dimensions. The program estimates pile deflection, shear, bending moment, and soil response with respect to depth in nonlinear soils through the use of generated p-y curves based on soil and rock parameters input by the user. For this integral abutment analysis, utilizing the simplified design method for integral abutment foundation design, only a single pile was modeled in the LPile software. Analysis and recommendations have been developed in accordance with the 2024 AASHTO *LRFD Bridge Design Specifications* and the 2008 VTrans *Integral Abutment Bridge Design Guidelines*.

3.1 Loads: Final unfactored loads were submitted by David Peterson, P.E., as part of the Geotechnical Request Form dated April 10th, 2026. The unfactored loads used in the analysis can be found in Table 3.1.1 and 3.1.2 for Abutments 1 and 2, respectively. Our common practice, as outlined in the 2008 VTrans Integral Abutment Manual, is to apply vertical live and dead loading, as well as longitudinal effects from thermal deformations, braking forces, and rotation due to live loading. LPILE does not consider the longitudinal and transverse stiffness provided by the entire bridge structure; it models a single pile standing alone. Due to this, it is assumed the longitudinal braking forces and wind forces are to be resisted by the stiffness of the integral structure, and therefore not accounted for in design.

Table 3.1.1 Unfactored Loads Abutment 1

Load Type	Load Type	Value	Elevation	Direction
Superstructure Dead Load*	DC	545.3 kips	1285.47 ft	Vertical
Vehicular Live Load	LL	100.5 kips	1285.47 ft	Vertical
Wind on Live Load	WL	+/- 6 kips	1298.47 ft	Transverse
		+/- 2.4 kips	1298.47 ft	Longitudinal
Thermal Movement	TU, Δ_t	1.0 in	1285.47 ft	Longitudinal

*Includes pile cap self-weight

Table 3.1.2 Unfactored Loads Abutment 2

Load Type	Load	Value	Elevation	Direction
Superstructure Dead Load*	DC	545.3 kips	1278.98 ft	Vertical
Vehicular Live Load	LL	100.5 kips	1278.98 ft	Vertical
Wind on Live Load	WL	+/- 6 kips	1291.98 ft	Transverse
		+/- 2.4 kips	1291.98 ft	Longitudinal
Thermal Movement	TU, Δ_t	1.0 in	1278.98 ft	Longitudinal

*Includes pile cap self-weight

The abutments were analyzed for both scour and non-scour conditions. A scour elevation of 1273.0 feet (ft) was provided in the Final Loads and Deformation Sheet submitted April 10th, 2026. This elevation results in freestanding pile lengths of 12.47 ft and 5.98 ft below the pile cap for Abutment 1 and Abutment 2, respectively. All Strength and Service limit states were evaluated under scour conditions.

Using the loads in Tables 3.1.1 and 3.1.2, and AASHTO LRFD Bridge Design Specifications Table 3.4.1-1, Strength Limit States I through V, Service Limit States I through IV, and Extreme Limit State I were evaluated.

3.2 Soil Profile: Results from the subsurface investigation completed in January and February 2024 were used to develop the soil profile for each abutment and corresponding models in FB-MultiPier. B-201 was used for Abutment 1, and B-202 was used for Abutment 2.

The ground elevation at B-201 was approximately 1306.5 ft. Bedrock was encountered at a depth of 61.0 ft below the ground surface (bgs) corresponding to an approximate elevation of 1245.5 ft. Groundwater was measured before drilling operations on

January 29th, 2024, was measured at a depth of 6.8 ft bgs, corresponding to an approximate elevation of 1299.7 ft.

The ground elevation at B-202 was approximately 1289.3 ft. Bedrock was encountered at a depth of 68.0 ft bgs corresponding to an approximate elevation of 1221.3 ft. Groundwater was measured after drilling on February 8th, 2024, at a depth of 10.7 ft bgs corresponding to a depth of 1278.6 ft.

The soil parameters used in the analysis for Abutment 1 and Abutment 2 are displayed below in Tables 3.2.1 and 3.2.2, respectively. Rock parameters used for both abutments are summarized in Table 3.2.3.

Table 3.2.1 FB-MultiPier Analysis Soil Parameters – Abutment 1 (B-201)

Elevation (ft)	Description	Friction Angle (deg.)	Unit Weight (pcf)	Subgrade Modulus (pci)
1285.39 – 1245.5	Very Dense SAND, some Silt	38	125	225
<1245.5	Bedrock (GNEISS)	-	160	-

Table 3.2.2 FB-MultiPier Analysis Soil Parameters – Abutment 2 (B-202)

Elevation (ft)	Description	Friction Angle (deg.)	Unit Weight (pcf)	Subgrade Modulus (pci)
1278.68 – 1274	Medium Dense SAND, some Gravel	36	120	90
1274 – 1221.3	Very Dense SAND, some Silt	38	125	125
<1221.3	Bedrock (GNEISS)	-	160	-

Table 3.2.3 FB-MultiPier Analysis Rock Parameters

Parameter	Value
Uniaxial Compressive Strength (psi)	9728
Poisson's Ratio	0.22
Geologic Strength Index, GSI	55
Intact Rock Modulus (psi)	28

3.3 Modeling: The models were analyzed for Strength, Service, and Extreme Limit combinations as described in Section 3.1. As detailed in the Abutment 1 and Abutment 2 Details Sheets from the Final Plans dated June 22nd, 2026, both pile caps will be 28 ft long and 3 ft wide. Bottom of pile cap elevations of 1285.39 ft and 1278.68 ft were used in the analysis for Abutments 1 and 2, respectively. LPile only models singular

piles and not the full foundational structure, therefore, axial loads were assumed to be distributed evenly across the piles. Axial forces and thermal deflection from contraction and expansion were applied to the top of the pile.

All piles were assumed to be driven plumb and oriented for weak axis bending in a single row for each abutment. The piles were modeled as 40 ft long and 57.7 ft long from the bottom of pile cap, extending to and bearing on competent bedrock for Abutments 1 and 2, respectively.

4.0 RESULTS

4.1 Pile Stresses: Four HP 12x84 piles were modeled for both the non-scour and scour conditions at each abutment. The models were designed for strength limit state under non-scour and scour conditions. The piles were checked for combined axial compression and flexure under the non-scour and scour conditions using the requirements of AASHTO LRFD 6.9.2.2, 6.9.4.1, and 6.10.8.2. Following the simplified design method for a pile design using the 2008 VTrans Integral Abutment Guidelines, LPILE analyses were performed by first applying the factored axial load and thermal deflection at the top of each pile. The resultant forces and unbraced lengths were then evaluated through structural checks following AASHTO requirements to determine if the pile would develop a plastic hinge. If a plastic hinge developed, the plastic moment was then applied at the top of the pile along with the original axial load and thermal deflection. A second structural check was conducted to determine if the 2nd segment would fail under the applied loading conditions. This approach was determined to be the most appropriate method for evaluating a single row of piles for bridges that meet the criteria of the simplified design method. For both abutments, Strength I governed the strength limit states under non-scour and scour conditions.

The output from LPILE was used to calculate the factored structural and flexure pile resistance as well as the moment that would cause a plastic hinge in the pile, in accordance with the VTrans 2008 Integral Abutment Bridge Design Guidelines. LPILE outputs as well as calculated values are displayed below in Table 4.1.

Table 4.1 L Pile Output for AASHTO Governing Limit States

Abutment	Limit State	Max. Axial (kips)	Max. Applied Moment (kip-ft)	Plastic Moment Developed (kip-ft)	2 nd Pile Segment Interaction	Top Segment Unbraced Length (inches)	Fixity* (feet)
1	Strength I (Non- Scour)	214.4	206.7	185.7	0.64	45.4	19.7
	Strength I/Extreme I (Scour)	214.4	60.6	177.5	0.48	109.5	27.5
2	Strength I (Non-Scour)	214.4	195.3	185.3	0.60	51.4	15.2
	Strength I/Extreme I (Scour)	214.4	101.0	182	0.50	82.2	25

* Measured from bottom of pile cap

As shown in Table 4.1, in the scour condition, the maximum applied moment is less than the plastic moment calculated; therefore a plastic hinge does not develop in the top segment of the pile in these conditions given these loadings. Based on modeling, a foundation consisting of 4 HP 12x84 piles spaced at 8.0 ft (96 inch) on-center was found to satisfy design requirements. Anticipated pile layout is detailed below in Figure 4.1. For the strength limit state, structural capacity calculations based on the resultant forces resulted in interactions less than 1.0, which satisfied design standards called out in AASHTO LRFD 6.9.2.2, 6.9.4.1, and 6.10.8.2. For the service limit state, deflections were found to not exceed 1.0 inches for the non-scour condition.

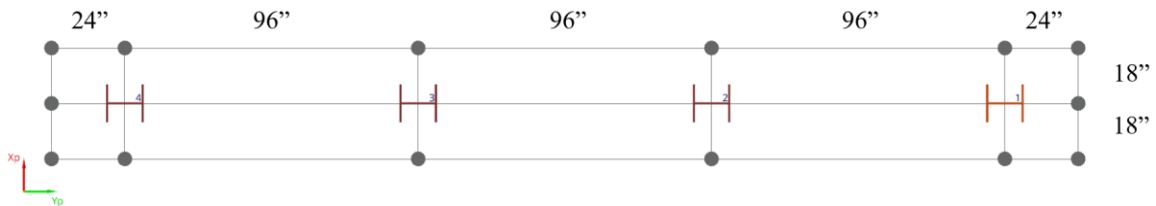


Figure 4.1 Abutment 1 and 2 Pile Layout

4.2 Driving Resistances: Soil conditions were evaluated in all of the borings to determine the drivability of H-piles through the materials to reach minimum tip elevation. Past experience and information collected suggest that the HP 12x84 piles can be driven through the soils encountered at both abutments by pile-driving equipment commonly used by contractors in the region. Difficult drilling conditions or indications of weathered rock were not encountered during drilling. The estimated length to gain geotechnical resistance for Abutment 1 and 2 was determined to be 37.4 ft and 35.3 ft respectively, corresponding to an approximate elevation of 1247.99 ft and 1243.38 ft, respectively. Section 10.7.8 of the AASHTO LRFD BDS stipulates that the maximum tension and compression stresses allowed in the piles shall not exceed $\sigma = 0.9 \cdot \phi_{da} \cdot f_y$. ϕ_{da} as defined in AASHTO LRFD 6.5.4.2 as 1.0, resulting in a maximum induced stress in the pile of $0.9 \cdot f_y$ or 45 ksi for grade 50 (50 ksi) piles.

4.3 Nominal Axial Pile Resistance: The piles are assumed to be driven to and seated on bedrock. All required axial capacity will be generated from the end bearing of the piles on rock. The nominal bearing resistance, R_N , shall be factored using the resistance factors, Φ_{dyn} , in Table 10.5.5.2.3-1 of the AASHTO LRFD Bridge Design Specifications. The factored resistance, R_R , may be taken as $R_R = \Phi_{dyn} \cdot R_N$. The resistance factor, Φ_{dyn} , which should be applied to those piles bearing in either soil or on rock to attain the factored resistance, is 0.65. The use of 0.65 requires a minimum of 2 dynamic tests performed during installation, with a minimum of one per abutment, in accordance with Table 10.5.5.2.3-1 of the AASHTO LRFD BDS. The remaining piles should be calibrated by wave equation analysis. Given the loads provided in Tables 3.1.1 and 3.1.2, the nominal axial pile resistance, or resistance the piles should be driven to, is 330 kips for both abutments.

4.4 Pile Cap Design: Based on discussions with Structures and the recommendations below in Section 5.2, we recommend placing a minimum of 25 inches of elasticized

expanded polystyrene (EPS) foam against the back of both abutment walls. The addition of the EPS at this specific structure is due to stability requirements outlined below including the use of geogrid. The placement of EPS foam will aid in the stability of the abutment and act to absorb any movement induced by soil structure into the bridge, as well as provide an elastic material in the gap between the geogrid system and the backwall of the abutment. The minimum width of foam was calculated using equation 5-2 assuming all movement occurs at a single abutment based on the grade, found in the VDOT Structural Design Manual Chapter 17. For Abutment 1, the EPS foam should be placed between the abutment wall and the GRS. Screenshots of the stability modeling are included in Appendix E.

$$EPS_t = 11[0.01h + 0.67(\Delta h)] \quad \text{Eq. 5-2}$$

There is no specification for EPS foam in the spec book, the special provision entitled “Geogrid-compressible inclusion” used in the Waterbury IM 089-2(43) plan set or the more recent Hartford (60) project can be referenced for use when developing notes on this project.

However, it is our understanding that the backwalls will still be designed as horizontal beams resisting lateral earth pressures as is standard practice. The lateral earth pressure is generated by movement of the abutment either into the soil mass (passive earth pressure) or away from the soil mass (active earth pressure). Passive earth pressure conditions may govern during warmer periods as the structure expands, while active earth pressure conditions may control during colder periods as the superstructure contracts.

Per the Final Plans, a height of 13 feet from the bottom of the pile cap to finished grade was assumed for Abutment 1 and Abutment 2. A thermal movement of 1.0 inches was used to evaluate lateral earth pressures on the abutments. The backfill unit weight is assumed to be 140 pcf with an internal friction angle of 34 degrees. The backfill is assumed to be free-draining granular material therefore, no hydrostatic pressure is assumed to develop behind the abutment walls.

Per AASHTO LRFD Bridge Design Specifications Table C3.11.1-1, full passive earth pressure requires a wall rotation, ratio of lateral movement to wall height, greater than 0.02 for medium dense cohesionless backfill. Based on the geometry and thermal movement, the calculated wall rotation ratio is approximately 0.006, therefore, full passive earth pressure is not expected to develop. Figure 3.10.8-1 from the MassDOT Bridge Manual was used to calculate the movement dependent partial passive earth pressure coefficient, K_P , for each abutment.

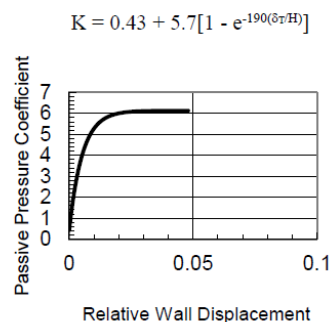


Figure 4.4.1. MassDOT Figure 3.10.8-1 Plot of Passive Pressure Coefficient, K , vs. Relative Wall Displacement, δ_T/H

This partial passive earth pressure coefficient was then used to calculate the unfactored passive lateral earth pressure, P_p , in accordance with AASHTO LRFD Bridge Design Specifications Figure 3.11.5.4-1.

$$P_p = \frac{1}{2} \cdot \gamma_{soil} \cdot H_{abut}^2 \cdot K_p \quad (\text{AASHTO Figure 3.11.5.4-1})$$

The resultant passive lateral earth pressure force acts at a height of $H/3$ above the base of the wall, corresponding to elevations of 1289.8 ft and 1283.3 ft for Abutments 1 and 2, respectively. The Rankine method was used to determine the full active earth pressure, and AASHTO LRFD Bridge Design Specifications Section 3.11.5.2 was used to determine the at-rest lateral earth pressure coefficient for the granular backfill. The results and recommendations are summarized in Table 4.4.1.

Table 4.4.1: Recommended Lateral Earth Pressures at the Proposed Abutments

Abutment	Abutment Height	Partial Passive Earth Pressure Coefficient, K_p	Passive Lateral Earth Pressure, P_p (Kip/ft)	Elevation (ft)
1	13	4.44	52.47	1289.8
2	13	4.44	52.47	1283.3

4.5 Downdrag Analysis: Negative skin friction, or downdrag, is considered when there is relative settlement between the pile and soil according to AASHTO 3.11.8. The proposed roadway varies significantly in grade with the existing site and as a result will require large amounts of fill. The proposed roadway will require up to 13.9 and 10.6 ft of fill to be placed to bring the roadway to the required grade at Abutment 1 and Abutment 2, respectively. A settlement analysis was completed according to the Hough method; 0.27 inches of settlement are anticipated for both Abutment 1 and Abutment 2. Due to the dense and granular nature of the material, the settlement is minimal and should occur during and immediately after construction, and additional axial loading to the HP 12 x 84 piles is not anticipated.

4.6 Settlement Analysis: Settlement of the abutments is anticipated to be negligible due to the piles being driven to very dense material or bedrock. Any settlement that does occur should be caused by the elasticity in the piles, which should occur as the piles are loaded

4.7 Global Stability: Due to the large amounts of added fill, global stability analyses were conducted to evaluate the external stability of the embankment fill and abutments. Using Slide2, developed by RocScience, slope stability analyses were performed evaluating failures through the fill and native layers of soil using the Spencer Method and a cuckoo search with surface altering optimizations for noncircular surfaces. These

analyses were performed for post-construction static load conditions and included material properties previously described in Section 3. STA 10+80 and STA 102+00 were used to model the soil slopes at the existing bridge and new bridge alignments post construction, respectively, and STA 11+50 was used to model the stability at the abutments of the new bridge. Some sections require adjustments and those are outlined below in Section 5.2.

5.0 RECOMMENDATIONS

5.1 The following table provides a summary of the requirements for the piles at Abutment 1 and 2.

Table 2. Summary of requirements of H-piles at each abutment

Requirement	Abutment 1 and 2
Pile Size	HP 12x84
Number of Piles per Abutment	4
Pile Orientation	Weak Axis
Pile Spacing	96 in on-center
Minimum Pile Embedment*	27.5 ft
	25 ft
Method of Installation	Driven pile
Nominal Axial Pile Resistance	330 kips

*Length of pile below bottom of pile cap

Pile lengths below the bottom of pile cap for estimating purposes should be assumed to be 40 ft for Abutment 1 and 58 ft for Abutment 2.

5.2 Multiple cross sections were analyzed to assess global stability of the approach embankments and abutments, resulting in the following recommendations:

STA 10+80: For STA 10+80, it was determined that no modification of the new slope geometry detailed in the Final Plans, dated June 2026, would be required to ensure a minimum factor of safety (FS) against failure of 1.3. A factor of safety of 1.3 is equivalent to a resistance factor of 0.75, which according to AASHTO LRFD Section 11.6.3.7, is the resistance factor for overall stability for a slope where the geotechnical parameters and subsurface stratigraphy are well defined.

STA 102+00: To produce a FS of 1.3 for the approach embankment slopes evaluated utilizing a cross section at 102+00, we recommend extending the toe of the left side slope of the planned roadway to the bottom of the existing drainage basin, continuing the 1H:1.5V slope as detailed in the plans. Type IV stone should be keyed in at the bottom and extend halfway up the slope, where Type II can then be placed on the face of the slope to the crest. A detail of the new slope requirements can be found in Appendix E.

STA 11+50: Modeling of the bridge abutments, as detailed in the Final Plans, resulted in a FS above 1.3 for Abutment 2 but well below 1.3 for Abutment 1. The lower factor of safety appears to be driven by the large amount of new fill and the steep slope of the new bridge approach. While the structure itself is going to provide some resistance to the slope's stability, it is best practice to neglect any such resistance for this evaluation. To increase the factor of safety, we recommend utilizing geogrid in layers behind Abutment 1 to force a deeper slip surface and corresponding adequate factor of safety. To raise the FS above 1.3, 7 layers of geogrid are required with a minimum allowable tensile force of 1750 lbs/ft, spaced 2 ft vertically, and extended a minimum of 7.5 ft from the back of the foam inclusion behind the abutment. The installation of the geogrid should follow manufacturer's recommendations and Section 217 of the VTrans 2024 *Standard Specifications for Construction*. A typical section detailing requirements is attached to the end of this report in Appendix D. A summary of the minimum design recommendations for the geogrid at Abutment 1 can be seen below in Table 5.2.

Table 5.1: Recommended Geosynthetic Design Parameters (Abut 1)

Design Parameters	Value
Min. Allowed Tensile Strength T_{al} (lbs/ft)	1,750
Number of Reinforcement Layers	7
Length of Primary Reinforcement (ft)	7.5
Reinforcement Vertical Spacing (ft)	2

The geogrid reinforcement is required to be constructed with a wrapped face to attain full strength and act as modeled. The geogrid re-embedment length should be at least 4 feet. As this material is behind the abutment, Granular Backfill for Structures should be used to ensure free-draining properties while also including the appropriate strength characteristics for the geogrid application. A detail is shown in the attached typical section.

5.3 Construction Considerations:

5.3.1 Cofferdams/Temporary Earthwork Support: Cofferdams or temporary shoring may be necessary during construction of the abutments. If required, the Contractor should be reminded that Section 208.06 of VTrans' 2024 *Standard Specifications for Construction* indicates that "The Contractor shall prepare detailed plans and a schedule of operations for each cofferdam specified in the Contract. Construction drawings shall be submitted in accordance with Subsection 105.06."

5.3.2 Construction Dewatering: Temporary construction dewatering may be required to construct the foundations. Temporary dewatering can likely be accomplished by open pumping from shallow sumps, temporary ditches, and trenches within and around the excavation limits. Sumps should be provided with filters suitable to prevent pumping of fine-grained soil particles. The water trapped by the temporary dewatering controls should be discharged to settling basins or an approved filter "sock" so that the fine particles suspended in the discharge have

adequate time to “settle out” prior to discharge. All effluent water, or discharge, should comply with all applicable permits and regulations.

5.3.3 Placement and Compaction of Soils: Fills should be placed systematically in horizontal layers not more than 12 inches in thickness, prior to compaction. Cobbles larger than 8 inches should be removed from the fill prior to placement. Compaction equipment should preferably consist of large, self-propelled vibratory rollers. Where hand-guided equipment is used, such as a small vibratory plate compactor, the loose lift thickness shall not exceed 6 inches. Cobbles larger than 4 inches should be removed from the fill prior to placement.

Embankment fills should be compacted to a dry density of no less than 95% of the maximum dry density determined in accordance with AASHTO T-99, Method C. Granular Backfill for Structures, or other select materials placed within the roadway base section, shall be compacted to a dry density of 95% of the maximum dry density determined in accordance with AASHTO T-99.

5.4 Design Parameters: Engineering properties of common construction materials are shown in Table 5.3.1. These values should be used when designing the substructure units. It is recommended that values of K_o be used for calculating earth pressures where the structure is not allowed to deflect longitudinally, away from or into the retained soil mass. Values for K_a should be utilized for an active earth pressure condition where the structure is moving away from the soil mass and K_p where the structure is moving toward the soil mass. The design earth pressure coefficients are based on horizontal surfaces (non-sloping backfill) and a vertical wall face.

Table 5.4.1: Engineering Properties of Construction Materials

	703.04 – Granular Borrow	704.08 – Granular Backfill for Structures
Unit Weight, γ (lbs/ft ³):	130	140
Internal Friction Angle, ϕ (degrees):	32	34
Coefficient of Friction, f		
- mass concrete cast against soil:	0.45	0.55
- soil against precast/formed concrete:	0.40	0.48
Active Earth Pressure Coef., K_a :		
Passive Earth Pressure Coef., K_p :	3.26	3.54
At-Rest Earth Pressure Coefficient, K_o :	0.47	0.44

6.0 CONCLUSION

If any further analysis is needed or if you would like to discuss this report, please contact us by email. Final LPILE input files used in the analyses are located in the <M:\Projects\12j624\MaterialsResearch\LPILE Models> on the M/drive.

Review by: Megan Messina, EIT, Geotechnical Engineer MNH

Appendices: Appendix A: Boring Logs (6 Pages)
Appendix B: LPile Input Tables, Output Graphs and Tables
Appendix C: MathCad Calculation Sheets – Structural Checks
Appendix D: MathCad Calculation Sheets – Lateral Earth
Pressure Calculations
Appendix E: Slide Modeling Outputs

cc: Electronic Read File/MG
Project File/CEE
AJA

["Z:\Highways\CMB\GeotechEngineering\Projects\Mount Holly BF 1443\(56\)\REPORTS\Mount Holly BF 1443\(56\)
Integral Abutment Recommendations.docx"](#)

Appendix A

Boring Logs



STATE OF VERMONT
AGENCY OF TRANSPORTATION
CONSTRUCTION AND
MATERIALS BUREAU
CENTRAL LABORATORY

BORING LOG

Route 103 Mount Holly
BF 1443 (56)
Bridge 64 Route 103 Mount Holly

Boring No.: **B-201**
Page No.: 1 of 3
Pin No.: 12j642
Checked By: KAR

Boring Crew: Mike Kern (Seaboard), T. Jiang (WSP)
Date Started: 1/24/24 Date Finished: 1/30/24
VTSPG NAD83: N 345573.10 ft E 1546936.00 ft
Station: 101+33.00 Offset: 8.9' LT
Ground Elevation: 1306.5 ft

Casing Sampler
Type: H.S.A. & WB SS, NX
I.D.: 4 in 1.5 in
Hammer Wt: 300 lb. 140 lb.
Hammer Fall: 30 in. 30 in.
Hammer/Rod Type: Auto/NWJ
Rig: Mobile B-53 $C_F = 1.44$

Groundwater Observations

Date	Depth (ft)	Notes
01/29/24	6.8	9:50 AM - drilling
01/30/24	11.0	11:40 AM - drilling

Depth (ft)	Strata (1)	CLASSIFICATION OF MATERIALS (Description)	Run (Dip deg.)	Core Rec. % (RQD %)	Drill Rate minutes/ft	Blows/6" (N Value)	Moisture Content %	Gravel %	Sand %	Fines %
5		1.5 ft - 1.8 ft, Driller noted that a cobble was encountered at 1.5 feet below ground surface. 2.0 ft - 4.0 ft, A-2-4, SM, dark brown, wet, medium dense, silty fine to medium SAND, some fine gravel, trace organics and roots, poorly-graded., Rec. = 0.8 ft, [Fill]				14-16-6-12 (22)				
		5.0 ft - 7.0 ft, A-4, SM, brown, wet, medium dense, silty fine to medium SAND, some gravel, poorly-graded., Rec. = 1.4 ft, [Fill]				8-9-8-7 (17)	15.0	21.0	43.0	36.0
10		10.0 ft - 12.0 ft, A-4, SM, brown, wet, medium dense, silty fine to medium SAND, some medium to coarse gravel, poorly-graded., Rec. = 1.0 ft, [Fill]				13-13-15-13 (28)				
15		15.0 ft - 17.0 ft, A-4, ML, dark brown, wet, hard, SILT, some sand little gravel, non-plastic., Rec. = 1.1 ft, [Glacial Till]				22-32-32-29 (64)	8.6	12.0	35.0	53.0
20		20.0 ft - 22.0 ft, A-2-4, SM, gray, wet, medium dense, silty fine to medium SAND, little fine to medium gravel, poorly-graded., Rec. = 1.4 ft, [Glacial Till]				9-13-16-20 (29)				
25		25.0 ft - 27.0 ft, A-2-4, GM, brown, wet, very dense, medium to coarse GRAVEL, some fine to medium sand, some silt, poorly-graded., Rec. = 1.7 ft, [Glacial Till]				25-36-42-28 (78)	10.4	45.0	23.0	32.0
30		30.0 ft - 30.8 ft, A-2-4, SM, brown, wet, very dense, fine to medium SAND, some gravel, some silt, poorly-graded., Rec. = 0.5 ft, [Glacial Till]				34-50/3" (R)				

Notes: 1. Stratification lines represent approximate boundary between material types. Transition may be gradual.
2. N Values have not been corrected for hammer energy. C_F is the hammer energy correction factor.
3. Water level readings have been made at times and under conditions stated. Fluctuations may occur due to other factors than those present at the time measurements were made.



STATE OF VERMONT
AGENCY OF TRANSPORTATION
CONSTRUCTION AND
MATERIALS BUREAU
CENTRAL LABORATORY

BORING LOG

Route 103 Mount Holly
BF 1443 (56)
Bridge 64 Route 103 Mount Holly

Boring No.: **B-201**

Page No.: 2 of 3

Pin No.: 12j642

Checked By: KAR

Boring Crew: Mike Kern (Seaboard), T. Jiang (WSP)

Date Started: 1/24/24 Date Finished: 1/30/24

VTSPG NAD83: N 345573.10 ft E 1546936.00 ft

Station: 101+33.00 Offset: 8.9' LT

Ground Elevation: 1306.5 ft

Casing Sampler
Type: H.S.A. & WB SS, NX
I.D.: 4 in 1.5 in
Hammer Wt: 300 lb. 140 lb.
Hammer Fall: 30 in. 30 in.
Hammer/Rod Type: Auto/NWJ
Rig: Mobile B-53 $C_E = 1.44$

Groundwater Observations

Date	Depth (ft)	Notes
01/29/24	6.8	9:50 AM - drilling
01/30/24	11.0	11:40 AM - drilling

Depth (ft)	Strata (1)	CLASSIFICATION OF MATERIALS (Description)	Run (Dip deg.)	Core Rec. % (RQD %)	Drill Rate minutes/ft	Blows/6" (N Value)	Moisture Content %	Gravel %	Sand %	Fines %
35		35.0 ft - 35.8 ft, Driller noted that a boulder was encountered from 35-37ft bgs.								
40		42.0 ft, Driller noted a change in drilling conditions at approximately 42 feet bgs.								
45		45.0 ft - 45.4 ft, A-2-4, SM, brown, wet, very dense, fine to medium SAND, some gravel, some silt, poorly-graded., Rec. = 0.3 ft, [Glacial Till]				55/5" (R)				
50		50.0 ft - 50.3 ft, A-2-4, SM, brown, wet, very dense, fine to medium SAND, some gravel, poorly-graded., Rec. = 0.3 ft, [Glacial Till]				50/3" (R)				
55		55.0 ft - 55.3 ft, A-2-4, SM, gray, wet, very dense, silty fine to coarse SAND, some medium to coarse gravel, poorly-graded., Rec. = 0.2 ft, [Glacial Till]				50/3" (R)				
60		61.0 ft - 66.0 ft, Greenish-gray, coarse-grained, strong (R4), GNEISS, moderately weathered (W3); discontinuities horizontal to high angle dipping, very closely to closely spaced, open. Average 5.4 fractures per foot.. NX, [MOUNT HOLLY COMPLEX]	C-1	48 (0)	6 11 6	Top of Bedrock @ 61.0 ft				

Notes:

1. Stratification lines represent approximate boundary between material types. Transition may be gradual.
2. N Values have not been corrected for hammer energy. C_E is the hammer energy correction factor.
3. Water level readings have been made at times and under conditions stated. Fluctuations may occur due to other factors than those present at the time measurements were made.



STATE OF VERMONT
AGENCY OF TRANSPORTATION
CONSTRUCTION AND
MATERIALS BUREAU
CENTRAL LABORATORY

BORING LOG

Route 103 Mount Holly
BF 1443 (56)
Bridge 64 Route 103 Mount Holly

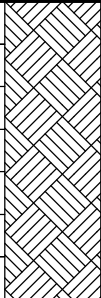
Boring No.: **B-201**
Page No.: 3 of 3
Pin No.: 12j642
Checked By: KAR

Boring Crew: Mike Kern (Seaboard), T. Jiang (WSP)
Date Started: 1/24/24 Date Finished: 1/30/24
VTSPG NAD83: N 345573.10 ft E 1546936.00 ft
Station: 101+33.00 Offset: 8.9' LT
Ground Elevation: 1306.5 ft

Casing Sampler
Type: H.S.A. & WB SS, NX
I.D.: 4 in 1.5 in
Hammer Wt: 300 lb. 140 lb.
Hammer Fall: 30 in. 30 in.
Hammer/Rod Type: Auto/NWJ
Rig: Mobile B-53 $C_E = 1.44$

Groundwater Observations

Date	Depth (ft)	Notes
01/29/24	6.8	9:50 AM - drilling
01/30/24	11.0	11:40 AM - drilling

Depth (ft)	Strata (1)	CLASSIFICATION OF MATERIALS (Description)	Run (Dip deg.)	Core Rec. % (RQD %)	Drill Rate minutes/ft	Blows/6" (N Value)	Moisture Content %	Gravel %	Sand %	Fines %
65					27					
					20					
		66.0 ft - 71.0 ft, Greenish-gray, coarse-grained, strong (R4), GNEISS, slightly weathered (W2); discontinuities horizontal to high angle dipping, very closely to closely spaced, open. Average 2 fractures per foot.. NX, [MOUNT HOLLY COMPLEX]	C-2	80 (10.8)	9					
					24					
					9					
					11					
70					11					
Hole stopped @ 71.0 ft										
75		Remarks: - Hollow stem augers used from the ground surface to an approximate depth of 10 feet below the ground surface (bgs), the driller switched to casing and drive wash methods at 10 ft bgs. - Bedrock was encountered at 61 feet bgs, first bedrock core was started at 61 feet bgs. - Boring terminated at 71 feet bgs. - Borehole was backfilled with bentonite chips within the bedrock core section from 61-71 feet bgs and sand and drill cuttings from 61 feet bgs to ground surface. - Four (4) boring attempts were completed closer to the south abutment location; three of these attempts encountered shallow refusals on probable boulders, at an approximate depth range of 2 to 6 ft bgs. One of the attempts encountered a drainage culvert at a depth of 3.2 ft bgs. - Northing and Easting coordinates are estimated based on swing ties to known features and the borehole locations were not surveyed.								
80										
85										
90										
95										

Notes:

1. Stratification lines represent approximate boundary between material types. Transition may be gradual.
2. N Values have not been corrected for hammer energy. C_E is the hammer energy correction factor.
3. Water level readings have been made at times and under conditions stated. Fluctuations may occur due to other factors than those present at the time measurements were made.



STATE OF VERMONT
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MATERIALS BUREAU
CENTRAL LABORATORY

BORING LOG

Route 103 Mount Holly
BF 1443 (56)
Bridge 64 Route 103 Mount Holly

Boring No.: **B-202A**
Page No.: 1 of 1
Pin No.: 12j642
Checked By: KAR

Boring Crew: Mike Kern (Seaboard), T. Jiang (WSP)
Date Started: 1/23/24 Date Finished: 1/23/24
VTSPG NAD83: N 345732.90 ft E 1547022.00 ft
Station: 103+16.00 Offset: 3.0' LT
Ground Elevation: 1289.3 ft

Casing Sampler
Type: H.S.A. & WB SS
I.D.: 4 in 1.5 in
Hammer Wt: N.A. 140 lb.
Hammer Fall: N.A. 30 in.
Hammer/Rod Type: Auto/NWJ
Rig: Mobile B-53 $C_E = 1.44$

Groundwater Observations

Date	Depth (ft)	Notes

Depth (ft)	Strata (1)	CLASSIFICATION OF MATERIALS (Description)	Blows/6" (N Value)	Moisture Content %	Gravel %	Sand %	Fines %
		0.0 ft - 1.1 ft, ASPHALT					
		1.1 ft - 2.5 ft, Driller notes multiple cobbles encountered at 1.1-2.5 feet bgs.					
		2.5 ft - 3.3 ft, A-2-4, SM, brown, dry, very dense, silty fine to medium SAND, poorly-graded., Rec. = 0.8 ft, [Fill]	34-50/4" (R)				
5		5.0 ft - 7.0 ft, A-2-4, SM, brown, dry, medium dense, fine to medium SAND, some fine to coarse gravel, some silt, poorly-graded., Rec. = 1.0 ft, [Fill]	20-15-6-5 (21)	10.6	25.0	53.0	22.0
10		10.0 ft - 11.3 ft, A-1-b, GP, brown, dry, FRACTURED ROCK, some gravel, some fine sand, poorly-graded., Rec. = 0.3 ft, [Fill]	22-34-50/3" (R)				
		Hole stopped @ 12.0 ft					
15		Remarks: - Hollow stem augers used from the ground surface to an approximate depth of 8 feet below the ground surface (bgs), the driller switched to casing and drive wash methods at 8 ft bgs. - Boring terminated at 12 feet bgs due to 5 feet of broken casing from 7 to 12 feet bgs. - Boring backfilled with drill cuttings and cold-patch asphalt to ground surface. - Northing and Easting coordinates are estimated based on swing ties to known features and the borehole locations were not surveyed.					
20							
25							
30							

Notes:

1. Stratification lines represent approximate boundary between material types. Transition may be gradual.
2. N Values have not been corrected for hammer energy. C_E is the hammer energy correction factor.
3. Water level readings have been made at times and under conditions stated. Fluctuations may occur due to other factors than those present at the time measurements were made.



STATE OF VERMONT
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CONSTRUCTION AND
MATERIALS BUREAU
CENTRAL LABORATORY

BORING LOG

Route 103 Mount Holly
BF 1443 (56)
Bridge 64 Route 103 Mount Holly

Boring No.: **B-202B**
Page No.: 1 of 3
Pin No.: 12j642
Checked By: KAR

Boring Crew: Dale Griffin (Seaboard), A. Martin (WSP)
Date Started: 2/07/24 Date Finished: 2/08/24
VTSPG NAD83: N 345733.00 ft E 1547025.00 ft
Station: 103+17.00 Offset: 1.0' LT
Ground Elevation: 1289.3 ft

Casing: WB
Sampler: SS, NX
Type: WB
I.D.: 4 in
1.5 in
Hammer Wt: 140 lb.
140 lb.
Hammer Fall: 30 in.
30 in.
Hammer/Rod Type: Auto/NWJ
Rig: Mobile B-53
 $C_F = 1.44$

Groundwater Observations

Date	Depth (ft)	Notes
02/08/22	10.7	8:10 AM - drilling

Depth (ft)	Strata (1)	CLASSIFICATION OF MATERIALS (Description)	Run (Dip deg.)	Core Rec. % (RQD %)	Drill Rate minutes/ft	Blows/6" (N Value)	Moisture Content %	Gravel %	Sand %	Fines %
		0.0 ft - 1.0 ft, ASPHALT								
5		5.0 ft - 5.6 ft, Advanced past boulder with the spin shoe casing from approximately 5.0 to 5.6 feet bgs., [Fill] 6.0 ft - 8.0 ft, Advanced past boulder with the spin shoe casing from approximately 6.0 to 8.0 feet bgs., [Fill]								
10		9.5 ft - 12.0 ft, Advanced past boulder with the spin shoe casing from approximately 9.5 to 12.0 feet bgs., [Fill] 12.5 ft - 13.5 ft, Advanced past boulder with the spin shoe casing from approximately 12.5 to 13.5 feet bgs., [Fill]								
15		15.0 ft - 17.0 ft, A-4, ML, gray, wet, hard, SILT, some fine to medium sand, little gravel, non-plastic., Rec. = 1.9 ft, [Glacial Till]				28-26-23-22 (49)				
20		20.0 ft - 22.0 ft, A-2-4, SM, gray, wet, very dense, fine to medium SAND, some silt, trace gravel, poorly-graded., Rec. = 2.0 ft, [Glacial Till]				21-31-32-29 (63)	11.5	3.0	63.0	34.0
25		25.0 ft - 27.0 ft, A-4, ML, gray, wet, hard, fine to medium sandy SILT, trace gravel, non-plastic., Rec. = 1.4 ft, [Glacial Till] 26.0 ft, Advanced past multiple cobbles with the roller bit from approximately 26 to 30 feet bgs.				20-24-28-30 (52)				
30		30.0 ft - 32.0 ft, A-4, ML, gray, wet, hard, SILT, some fine to coarse sand, little gravel, non-plastic., Rec. = 2.0 ft, [Glacial Till]				30-40-47-41 (87)	10.3	20.0	26.0	54.0

Notes:

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STATE OF VERMONT
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MATERIALS BUREAU
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BORING LOG

Route 103 Mount Holly
BF 1443 (56)
Bridge 64 Route 103 Mount Holly

Boring No.: **B-202B**

Page No.: 2 of 3

Pin No.: 12j642

Checked By: KAR

Boring Crew: Dale Griffin (Seaboard), A. Martin (WSP)

Date Started: 2/07/24 Date Finished: 2/08/24

VTSPG NAD83: N 345733.00 ft E 1547025.00 ft

Station: 103+17.00 Offset: 1.0' LT

Ground Elevation: 1289.3 ft

Casing: WB
Sampler: SS, NX
Type: WB
I.D.: 4 in
1.5 in
Hammer Wt: 140 lb.
140 lb.
Hammer Fall: 30 in.
30 in.
Hammer/Rod Type: Auto/NWJ
Rig: Mobile B-53
 $C_E = 1.44$

Groundwater Observations

Date	Depth (ft)	Notes
02/08/22	10.7	8:10 AM - drilling

Depth (ft)	Strata (1)	CLASSIFICATION OF MATERIALS (Description)	Run (Dip deg.)	Core Rec. % (RQD %)	Drill Rate minutes/ft	Blows/6" (N Value)	Moisture Content %	Gravel %	Sand %	Fines %
35		35.0 ft - 35.8 ft, A-4, ML, gray, wet, hard, SILT, some fine to coarse sand, little gravel, non-plastic., Rec. = 0.8 ft, [Glacial Till] 35.8 ft, Advanced past multiple cobbles with the roller bit from approximately 35.8 to 37 feet bgs.				27-50/3" (R)				
40		40.0 ft - 40.8 ft, A-2-4, SM, grayish brown, wet, very dense, fine to medium SAND, some gravel, some silt, poorly-graded., Rec. = 0.8 ft, [Glacial Till]				39-50/3" (R)	14.5	34.0	37.0	29.0
45		45.0 ft - 46.3 ft, A-4, ML, grayish brown, wet, hard, fine to medium sandy SILT, little gravel, non-plastic., Rec. = 1.3 ft, [Glacial Till] 46.3 ft, Advanced past multiple cobbles with the roller bit from approximately 46.3 to 49.5 feet bgs.				49-62-50/3" (R)				
50		50.0 ft - 51.0 ft, A-4, ML, grayish brown, wet, hard, SILT, some fine sand, little gravel, non-plastic., Rec. = 0.9 ft, [Glacial Till]				37-64-50/0" (R)	15.2	17.0	30.0	53.0
55										
60		60.0 ft - 60.1 ft, A-2-4, SM, light brown, wet, very dense, fine to medium SAND, some silt, poorly-graded., Rec. = 0.1 ft, [Glacial Till]				50/1" (R)				

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STATE OF VERMONT
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MATERIALS BUREAU
CENTRAL LABORATORY

BORING LOG

Route 103 Mount Holly
BF 1443 (56)
Bridge 64 Route 103 Mount Holly

Boring No.: **B-202B**
Page No.: 3 of 3
Pin No.: 12j642
Checked By: KAR

Boring Crew: Dale Griffin (Seaboard), A. Martin (WSP)
Date Started: 2/07/24 Date Finished: 2/08/24
VTSPG NAD83: N 345733.00 ft E 1547025.00 ft
Station: 103+17.00 Offset: 1.0' LT
Ground Elevation: 1289.3 ft

Casing: WB Sampler: SS, NX
Type: I.D.: 4 in 1.5 in
Hammer Wt: 140 lb. 140 lb.
Hammer Fall: 30 in. 30 in.
Hammer/Rod Type: Auto/NWJ
Rig: Mobile B-53 $C_E = 1.44$

Groundwater Observations

Date	Depth (ft)	Notes
02/08/22	10.7	8:10 AM - drilling

Depth (ft)	Strata (1)	CLASSIFICATION OF MATERIALS (Description)	Run (Dip deg.)	Core Rec. % (RQD %)	Drill Rate minutes/ft	Blows/6" (N Value)	Moisture Content %	Gravel %	Sand %	Fines %
65										
		66.0 ft - 68.0 ft, Advanced with roller bit into bedrock.								
		68.0 ft - 73.0 ft, Gray, fine to coarse-grained, strong (R4) to very strong (R5), GNEISS, slightly weathered (W2); discontinuities low to high angle dipping, very closely to closely spaced, open. Average 2.4 fractures per foot.. NX, [MOUNT HOLLY COMPLEX]	C-1	92 (35)	4					
70					4					
					4					
					4					
					4					
		73.0 ft - 78.0 ft, Gray, fine to coarse-grained, medium strong (R3), GNEISS, slightly weathered (W2); discontinuities low angle to moderately dipping, very closely to closely spaced, open. Average 3 fractures per foot.. NX, [MOUNT HOLLY COMPLEX]	C-2	62 (7)	2					
75					2.5					
					2.5					
					2.5					
					3					
		Hole stopped @ 78.0 ft								
80		Remarks: - Bedrock was encountered at 66 feet bgs, first bedrock core was started at 68 feet bgs. - Boring terminated at 78 feet bgs. - Borehole was backfilled with bentonite chips within the bedrock core section from 68-78 feet bgs, gravel from 68 feet bgs to bottom of pavement, and cold-patch asphalt to ground surface. - Northing and Easting coordinates are estimated based on swing ties to known features and the borehole locations were not surveyed.								
85										
90										
95										

Notes: 1. Stratification lines represent approximate boundary between material types. Transition may be gradual.
2. N Values have not been corrected for hammer energy. C_E is the hammer energy correction factor.
3. Water level readings have been made at times and under conditions stated. Fluctuations may occur due to other factors than those present at the time measurements were made.

Appendix B

LPile Analysis

Abutment Loads and Deformations

Pin: 12j642
Project: Mt Holly BO 1443(56)

DATE: JMP 8-12-2025
Checked by: DRP 3-25-26

Abutment 1 Loads, in Kips					
Load Type	Longitudinal		Transverse		Vertical
	Magnitude	Elevation	Magnitude (k)	Elevation	Magnitude (k)
DC	---	---	---	---	545.3
DW	---	---	---	---	---
LL	---	---	---	---	100.529
CE	---	---	---	---	---
BR	+/- 21.6	1298.47	---	---	---
WS	---	---	---	---	---
WL	+/- 2.4	1298.47	+/- 6	1298.47	---

Component Dead Loads, DC
Wearing Surface and Utility Loads, DW
Live Load (LL)
Centrifugal Force (CE)
Breaking Force (BR)
Wind Pressure on Structures, WS
Wind Pressure on Vehicles, WL

ICE LOAD (IC)

Abutment 2 Loads, in Kips					
Load Type	Longitudinal		Transverse		Vertical
	Magnitude	Elevation	Magnitude	Elevation	Magnitude
DC	---	---	---	---	545.3
DW	---	---	---	---	---
LL	---	---	---	---	100.529
CE	---	---	---	---	---
BR	+/- 21.6	1291.98	---	---	---
WS	---	---	---	---	---
WL	+/- 2.4	1291.98	+/- 6	1291.98	---

	Total (kip-ft)	Weak Axis (kip-ft)	Strong Axis (kip-ft)
Braking Moments			
Abut 1	280.8	280.800	0.000
Abut 2	280.8	280.800	0.000

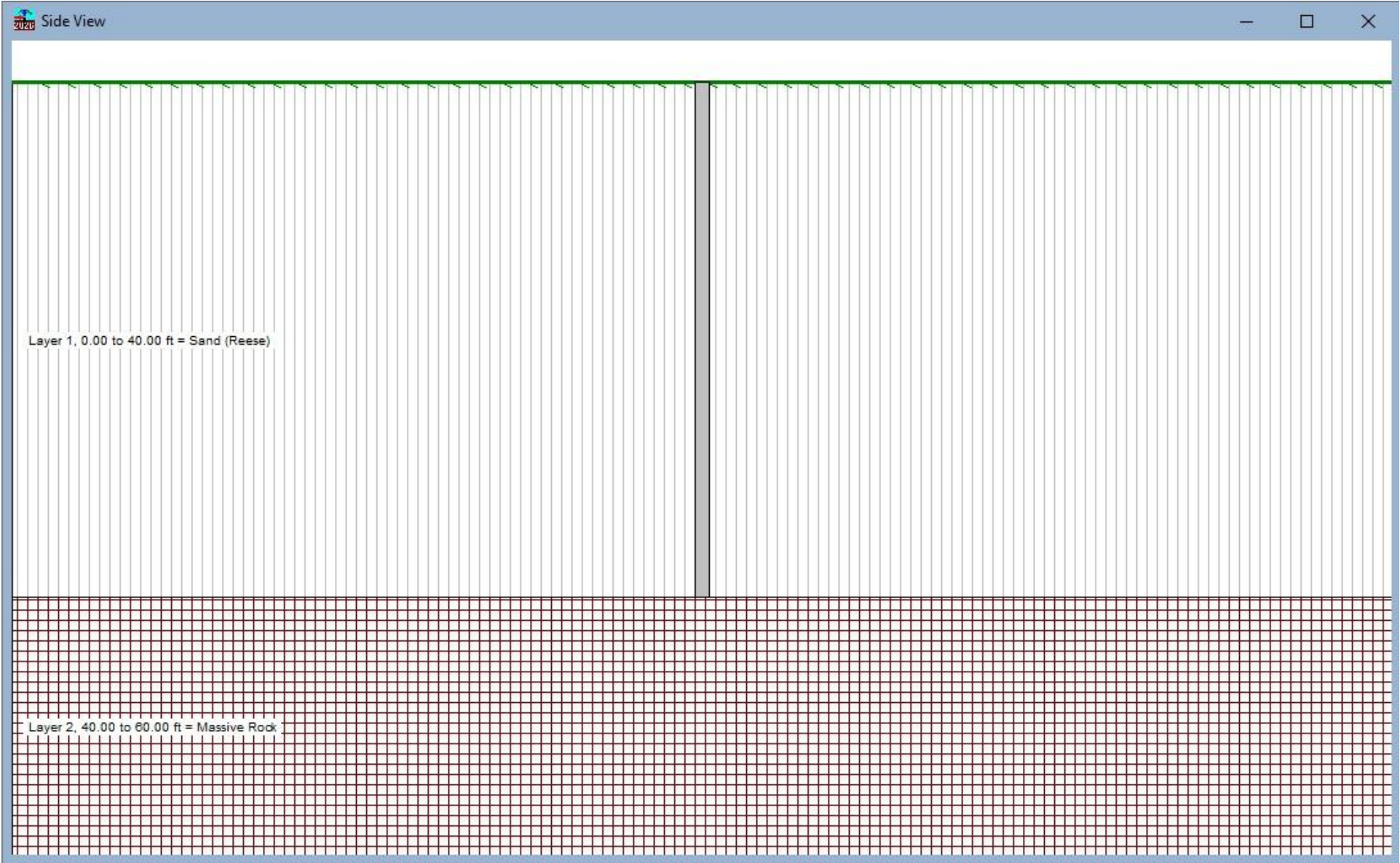
Sign Convention for Loads			
Longitudinal	Positive is upstation		
Transverse	Positive is right when looking upstation		
Vertical	Positive is downward		
	Total (in)	Weak Axis (in)	Strong Axis (in)
Expansion	1	1.000	0.000
Contraction	1	1.000	0.000

Note, due to grade, assume all movement is in one direction

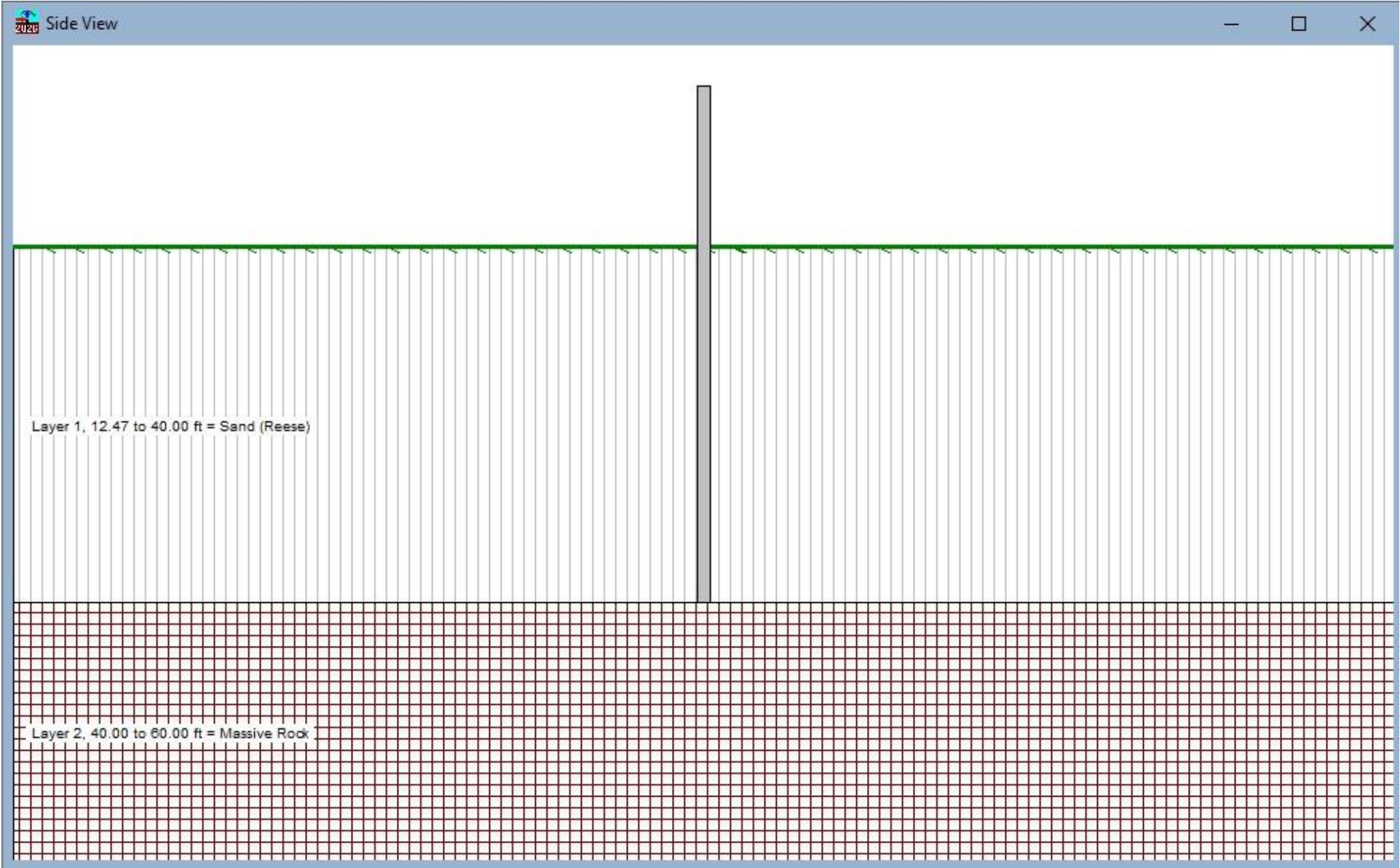
Strong axis orientation may be warranted at low end.

*Reported temperature deformations are maximums that could occur in each direction, but do not occur under the same case. The maximum expansion would occur if the abutment closure pours occur at 32°F. The maximum contraction would occur if the abutment closure pour occurred at 90°F.

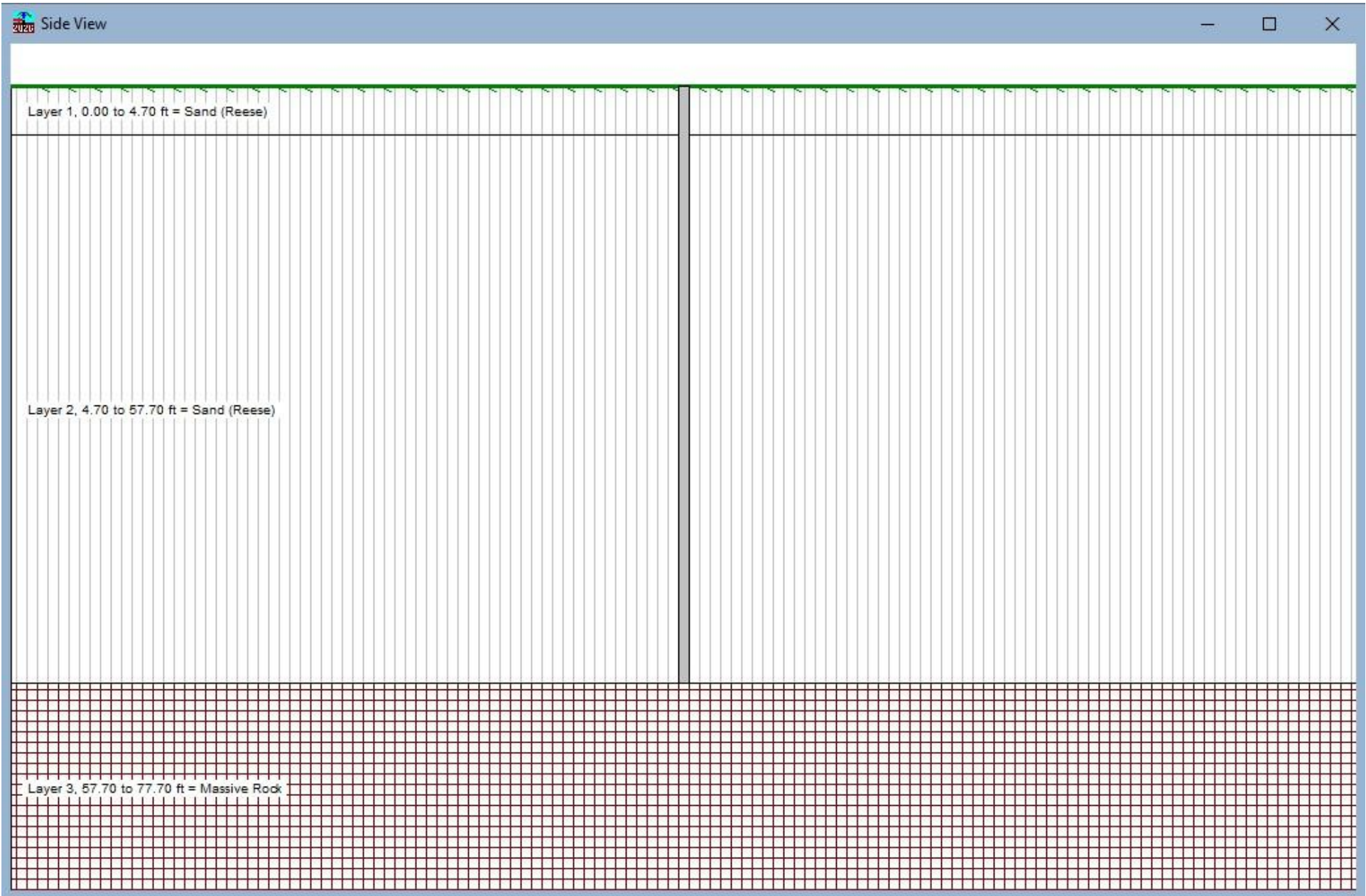
Freestanding Heights in Scour Condition			
Abutment #	Bottom of Abutment	Scour Elevation	Freestanding Height (ft)
1	1285.47	1273.00	12.47
2	1278.98	1273.00	5.98



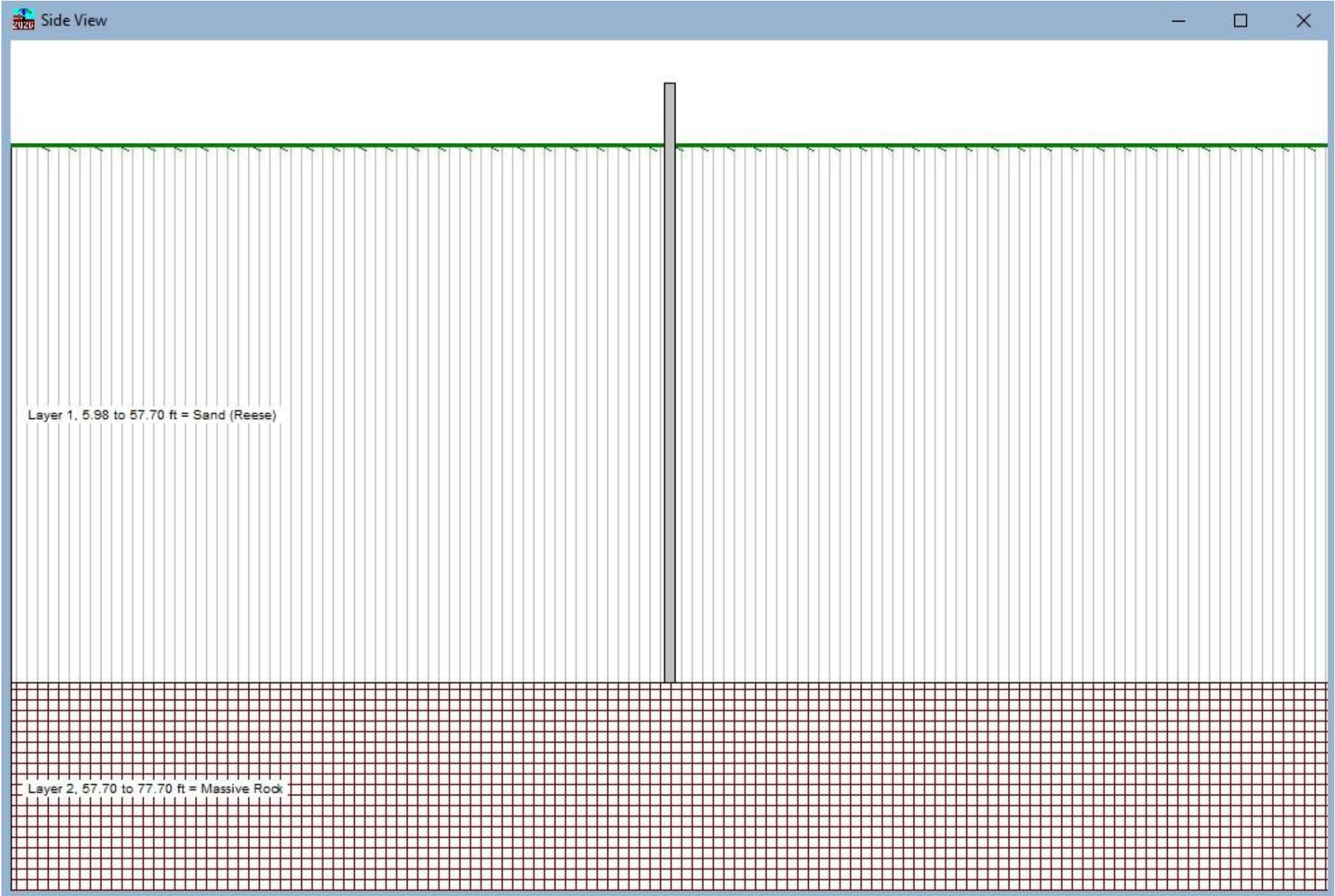
Abutment 1 Non-Scour Soil Profile



Abutment 1 Scour Soil Profile



Abutment 2 Non-Scour Soil Profile



Abutment 2 Scour Soil Profile

Soil Layers

	Select p-y Curve Type	Vertical Depth Below Pile Head	Vertical Depth Below Pile Head	Press Button to Enter
	from Drop-down List	of Top of Soil Layer (ft)	of Bottom of Soil Layer (ft)	Soil Properties
1	Sand (Reese)	0	40	1: Sand (Reese, et al.)
2	Massive Rock	40	60	2: Massive Rock Properties

Add Row

Insert Row

Delete Row

All positive depth coordinates are defined as vertical distances below the pile-head.
If the pile-head is embedded below the ground surface, the top layer must extend from the ground surface (defined by a negative vertical depth) to some point below the pile head.
Select the p-y soil type using the drop-down list in the left table column.

Abutment 1 Soil Layers

Sand (Reese, et al.) 1

1=Top, 2=Bottom	Effective Unit	Friction Angle,	Non-default
	Weight, (lbs/ft^3)	(DEG)	k, (lbs/in^3)
1	125	38	225
2	62.6	38	125

Copy Values from Top to Bottom

☐ Use Enhanced Model

LPile linearly interpolates over vertical depth to compute values between the top and bottom of the layer.

LPile will substitute a default value for k if the input value equals zero.

Abutment 1 Soil Parameters

Massive Rock Properties 2

1=Top, 2=Bottom	Effective Unit	Uniaxial Comp.	Select rock type,	Poisson's	Geologic Strength	(OPTION 1) Intact	(OPTION 2) Rock
	Weight, (lbs/ft^3)	Strength, (lbs/in^2)	Hoek-Brown Material Index, mi	Ratio	Index, GSI	Rock Modulus, (lbs/in^2)	Mass Modulus, (lbs/in^2)
1	97.6	9728	(metamorphic) gneiss, mi = 28	0.22	55	28	0
2	97.6	9728	(metamorphic) gneiss, mi = 28	0.22	55	28	0

Copy Values from Top to Bottom

Enter values for Material Index and Geologic Strength Index (GSI) for the top of layer (Row 1) only. Any values entered for bottom of layer (Row 2) will be ignored.

Input is always required for both input options of values of Effective Unit Weight, Uniaxial Compressive Strength, Material Index, Poisson's ratio, and Geologic Strength Index.

OPTION 1 is to input a non-zero value of INTACT ROCK MODULUS and to enter ZERO for ROCK MASS MODULUS. In Option 1, values of Rock Mass Modulus will be computed by LPile.

OPTION 2 is to input a non-zero value for ROCK MASS MODULUS. In Option 2, if the input value Rock Mass Modulus is non-zero, any value entered for Intact Rock Modulus will be ignored.

LPile uses interpolation to compute values of parameters at all intermediate depths between the top and bottom of the rock layer, except for Material Index and GSI.

Abutment 1 Rock Parameters

Soil Layers

	Select p-y Curve Type	Vertical Depth Below Pile Head	Vertical Depth Below Pile Head	Press Button to Enter
	from Drop-down List	of Top of Soil Layer (ft)	of Bottom of Soil Layer (ft)	Soil Properties
1	Sand (Reese) ▾	0	4.7	1: Sand (Reese, et al.)
2	Sand (Reese) ▾	4.7	57.7	2: Sand (Reese, et al.)
3	Massive Rock ▾	57.7	77.7	3: Massive Rock Properties

Add Row

Insert Row

Delete Row

All positive depth coordinates are defined as vertical distances below the pile-head.
If the pile-head is embedded below the ground surface, the top layer must extend from the ground surface (defined by a negative vertical depth) to some point below the pile head.
Select the p-y soil type using the drop-down list in the left table column.

Abutment 2 Soil Layers

Massive Rock Properties 3

1=Top, 2=Bottom	Effective Unit	Uniaxial Comp.	Select rock type,	Poisson's	Geologic Strength	(OPTION 1) Intact	(OPTION 2) Rock
	Weight, (lbs/ft^3)	Strength, (lbs/in^2)	Hoek-Brown Material Index, mi	Ratio	Index, GSI	Rock Modulus, (lbs/in^2)	Mass Modulus, (lbs/in^2)
1	97.6	9728	(metamorphic) gneiss, mi = 28 ▾	0.22	65	28	0
2	97.6	9728	(metamorphic) gneiss, mi = 28 ▾	0.22	65	28	0

Copy Values from Top to Bottom

Enter values for Material Index and Geologic Strength Index (GSI) for the top of layer (Row 1) only. Any values entered for bottom of layer (Row 2) will be ignored.

Input is always required for both input options of values of Effective Unit Weight, Uniaxial Compressive Strength, Material Index, Poisson's ratio, and Geologic Strength Index.

OPTION 1 is to input a non-zero value of INTACT ROCK MODULUS and to enter ZERO for ROCK MASS MODULUS. In Option 1, values of Rock Mass Modulus will be computed by LPile.

OPTION 2 is to input a non-zero value for ROCK MASS MODULUS. In Option 2, if the input value Rock Mass Modulus is non-zero, any value entered for Intact Rock Modulus will be ignored.

LPile uses interpolation to compute values of parameters at all intermediate depths between the top and bottom of the rock layer, except for Material Index and GSI.

Abutment 2 Rock Parameters

Sand (Reese, et al.) 1

1=Top, 2=Bottom	Effective Unit	Friction Angle,	Non-default
	Weight, (lbs/ft^3)	(DEG)	k, (lbs/in^3)
1	120	36	90
2	57.6	36	60

Copy Values from Top to Bottom

☐ Use Enhanced Model

LPile linearly interpolates over vertical depth to compute values between the top and bottom of the layer.

LPile will substitute a default value for k if the input value equals zero.

Abutment 2 Layer 1 Soil Parameters

Sand (Reese, et al.) 2

1=Top, 2=Bottom	Effective Unit	Friction Angle,	Non-default
	Weight, (lbs/ft^3)	(DEG)	k, (lbs/in^3)
1	62.6	38	125
2	62.6	38	125

Copy Values from Top to Bottom

☐ Use Enhanced Model

LPile linearly interpolates over vertical depth to compute values between the top and bottom of the layer.

LPile will substitute a default value for k if the input value equals zero.

Abutment 2 Layer 2 Soil Parameters

Abutment 1 NonScour Ouput Forces

Load Combo	Pu (kips)	Mutop (k-ft)	Mu2nd (k-ft)	lb top (in)	lb 2nd (in)	Vmax (kips)	Mp (k-in)	Plastic hinge?	Mutop (k-ft)	Mu2nd (k-ft)	lb top (in)	lb 2nd (in)	Vmax (kips)	2nd Interaction
STR I	214.4	206.7	91.2	45.4	126.0	58.0	2228.4	Yes	185.7	89.3	42.9	128.5	54.4	0.64
STR II	204.3	206.9	91.1	45.5	126.0	58.1	2264.3	Yes	188.7	89.5	43.3	127.1	55.0	0.63
STR III	170.4	207.6	91.0	47.7	124.0	58.5	2383.6	Yes	198.6	91.0	44.5	126.6	56.8	0.51
STR IV	204.5	206.9	91.1	45.4	126.1	58.1	2263.7	Yes	188.6	89.5	43.2	127.1	55.0	0.63
STR V	204.3	206.9	91.1	45.5	126.0	58.1	2264.3	Yes	188.7	89.5	43.3	127.1	55.0	0.63

Abutment 2 NonScour Output Forces

Load Combo	Pu (kips)	Mutop (k-ft)	Mu2nd (k-ft)	lb top (in)	lb 2nd (in)	Vmax (kips)	Mp (k-in)	Plastic hinge?	Mutop (k-ft)	Mu2nd (k-ft)	lb top (in)	lb 2nd (in)	Vmax (kips)	2nd Interaction
STR I	214.4	195.3	78.5	51.4	135.3	49.0	2223.0	Yes	185.3	77.8	50.0	136.0	47.2	0.60
STR II	204.3	195.9	79.1	51.4	134.9	49.3	2259.2	Yes	188.3	78.5	50.3	135.5	47.8	0.55
STR III	170.4	196.7	78.9	51.4	135.1	49.7	2380.9	No						0.44
STR IV	204.5	195.9	79.1	51.2	135.0	49.3	2258.7	Yes	188.2	78.5	50.3	135.5	47.8	0.59
STR V	204.3	195.9	79.1	51.4	134.9	49.3	2259.2	Yes	188.3	78.5	50.3	135.5	47.8	0.55

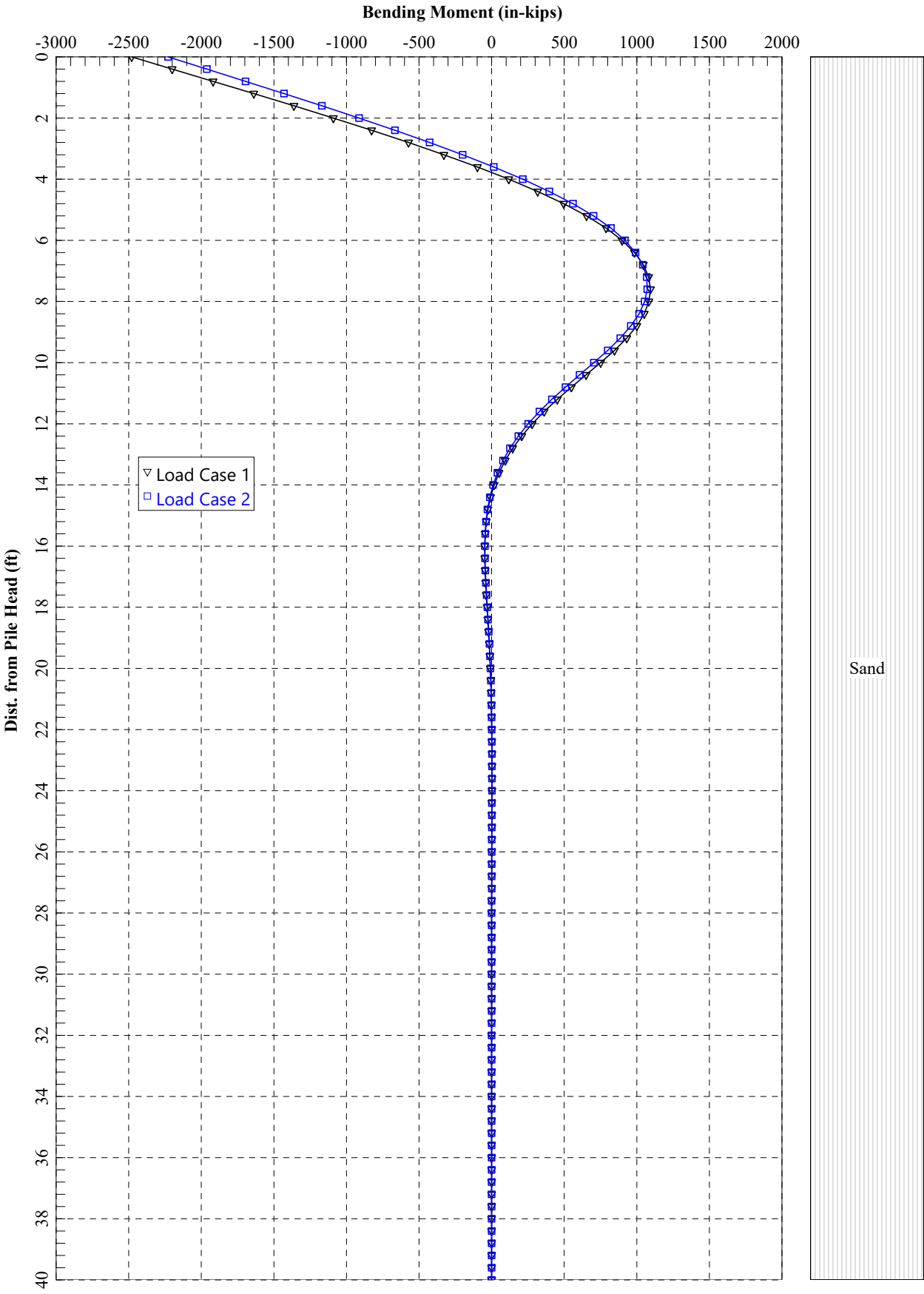
Abutment 1 Scour Output Forces

Load Combo	Pu (kips)	Mutop (k-ft)	Mu2nd (k-ft)	lb top (in)	lb 2nd (in)	Vmax (kips)	Mp (k-in)	Plastic Hinge?	2nd Interaction
STR I	214.4	60.6	37.5	109.5	184.2	7.5	2130.1	No	0.5
STR II	204.3	60.6	37.5	109.5	172.3	7.5	2170.8	No	0.5
STR III	170.4	60.9	37.4	109.5	172.6	7.5	2307.1	No	0.4
STR IV	204.5	60.6	37.5	109.5	172.3	7.5	2170.0	No	0.5
STR V	204.3	60.6	37.5	109.5	172.3	7.5	2170.8	No	0.5

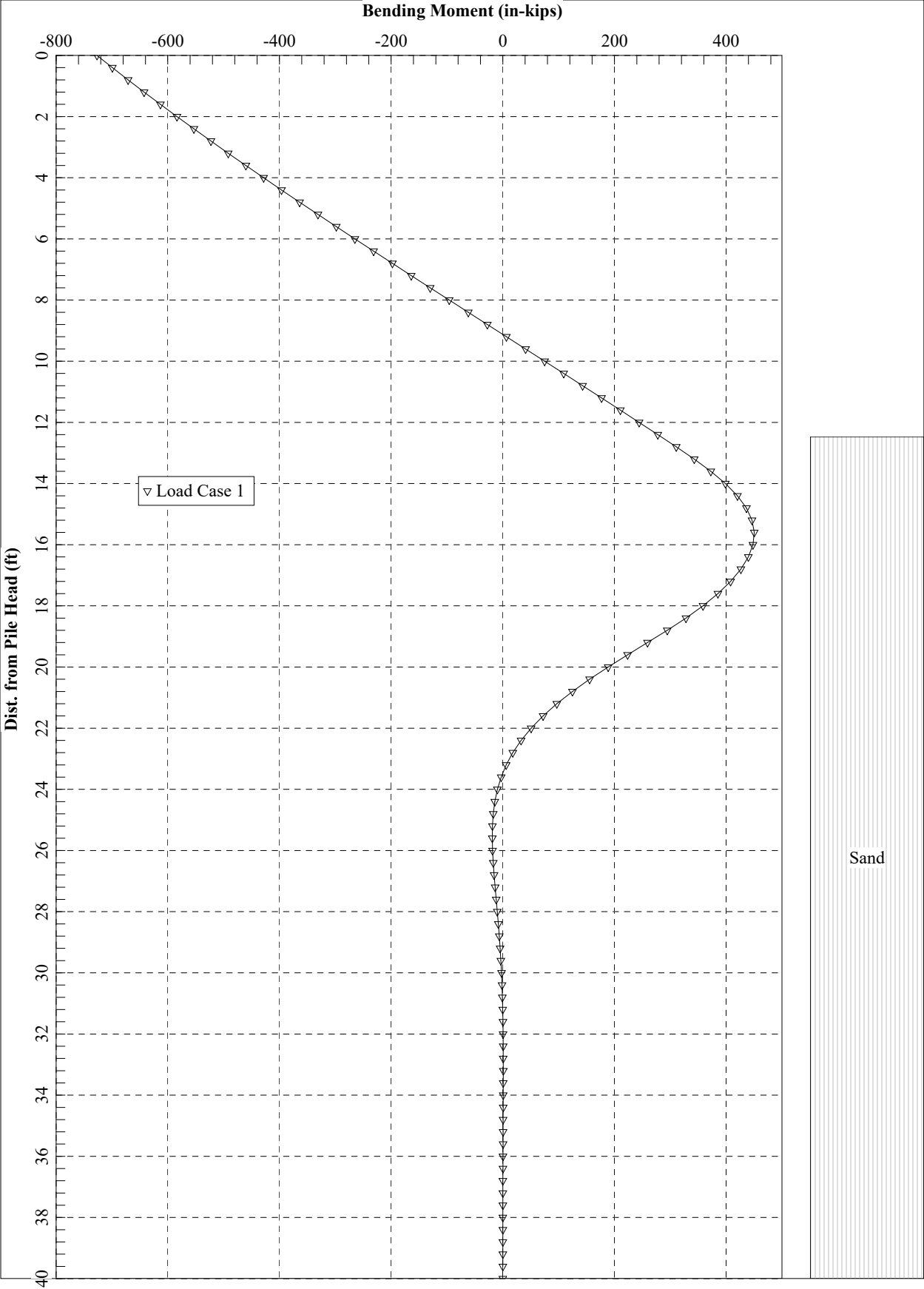
Abutment 2 Scour Output Forces

Load Combo	Pu (kips)	Mutop (k-ft)	Mu2nd (k-ft)	lb top (in)	lb 2nd (in)	Vmax (kips)	Mp (k-in)	Plastic Hinge?	2nd Interaction
STR I	214.4	101.0	47.7	82.2	160.3	13.6	2183.4	No	0.5
STR II	204.3	101.0	47.7	82.2	160.3	13.7	2221.5	No	0.5
STR III	170.4	101.2	47.5	82.2	160.9	13.9	2349.4	No	0.4
STR IV	204.5	101.0	47.7	82.2	160.5	13.7	2220.8	No	0.5
STR V	204.3	101.0	47.7	82.2	160.3	13.7	2221.5	No	0.5

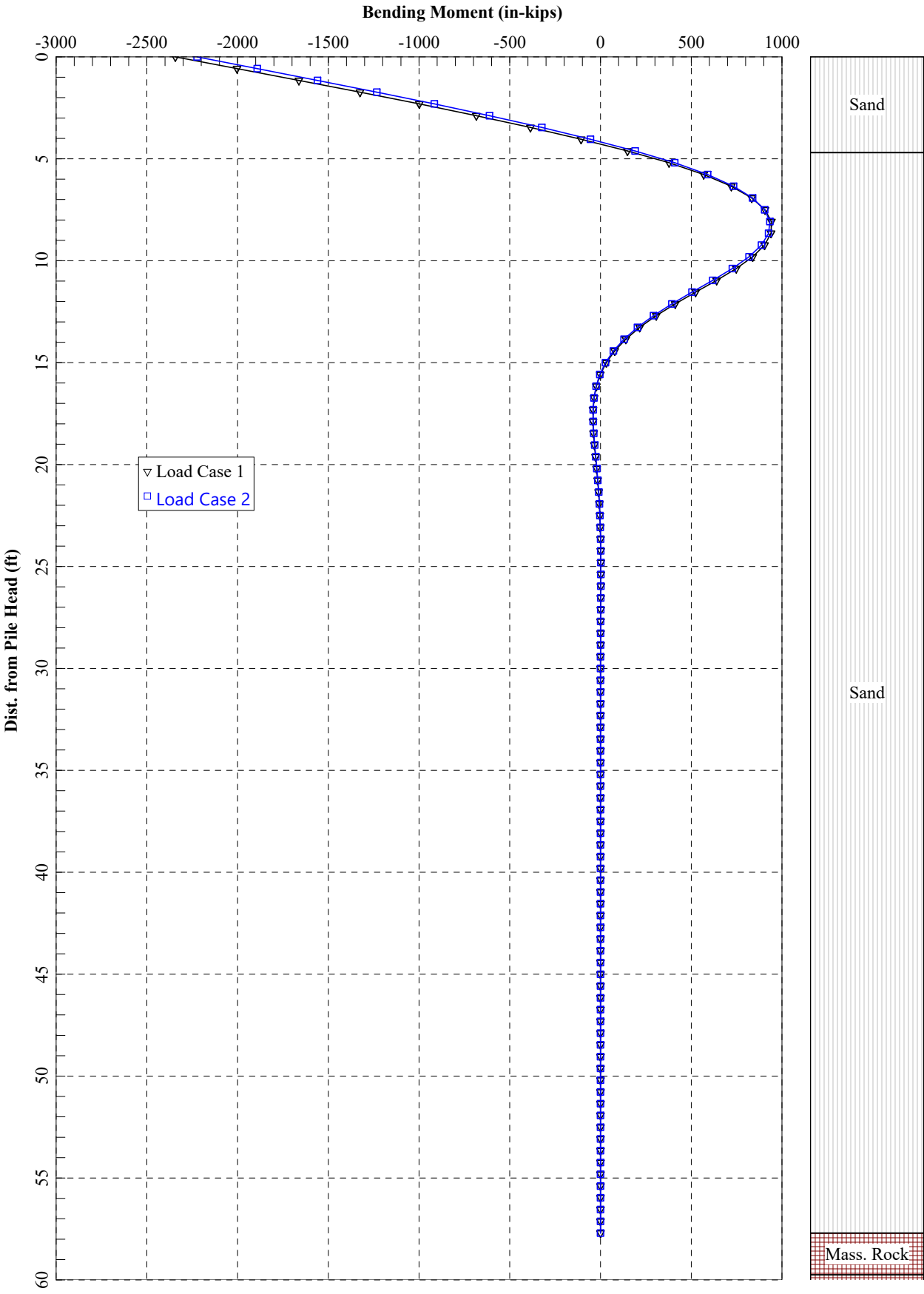
Abutment 1 Strength 1 Non Scour



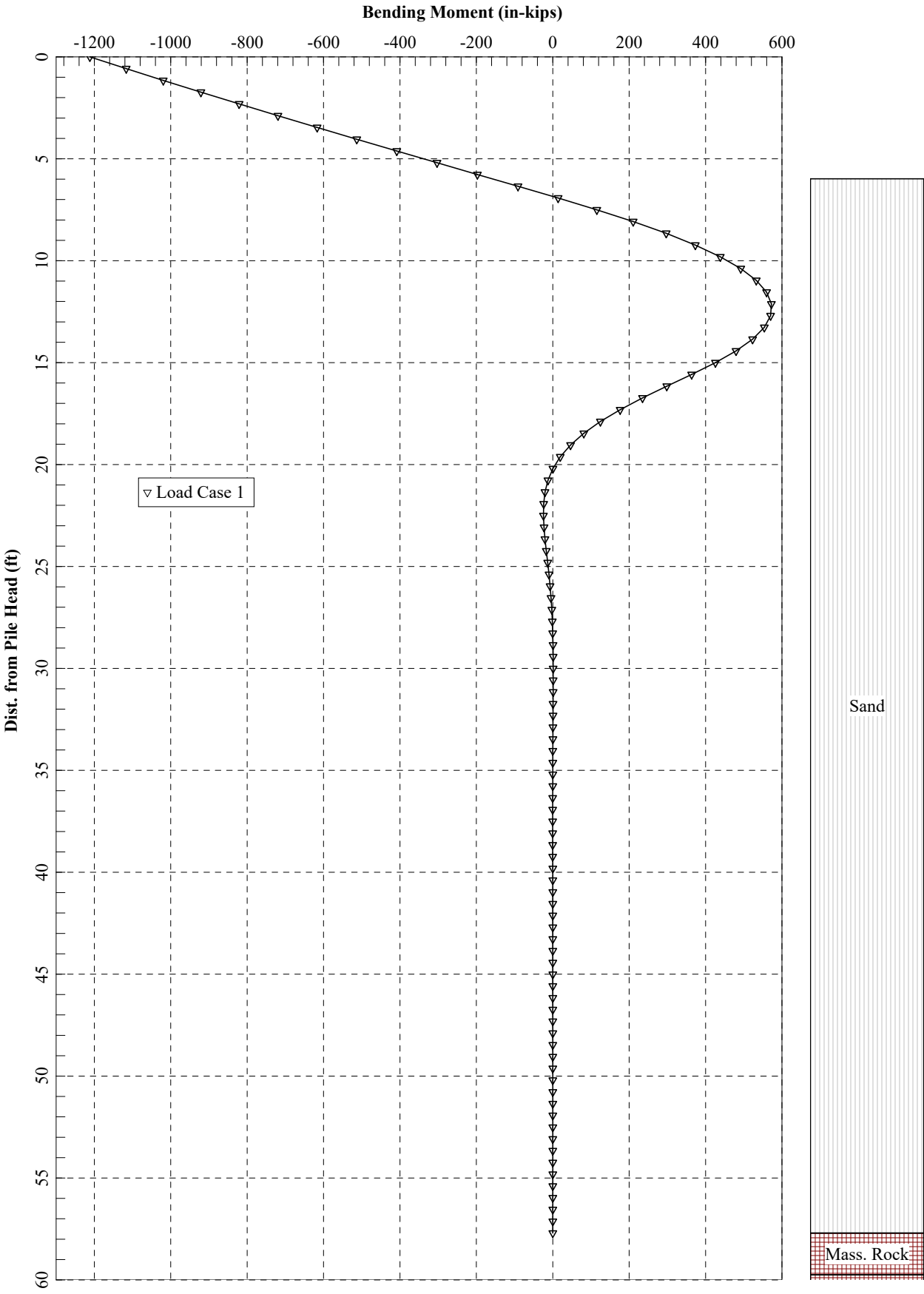
Abutment 1 Strength 1 Scour



Abutment 2 Strength 1 Non Scour



Abutment 2 Strength 1 Non Scour



Appendix C

MathCAD Structural Sheets

Integral Abutment Design Worksheet (See IAB Manual 2010 Appendix B Design Example)

Project Name: Mount Holly BF 1442(56)

FB-Pier Corresponding File Name: Abutment 1 STR I NS.in

Analysis By: A. Arles 6/01/2026 Piles Oriented for Weak Axis Bending

Pile Size Inputs:

Pile := HP 12x84 ▾

$$b_f = 12.3 \text{ in} \quad t_f = 0.685 \text{ in} \quad t_w = 0.685 \text{ in} \quad d = 12.3 \text{ in} \quad k_o = 1.375 \text{ in} \quad A_s = 24.6 \text{ in}^2$$

$$S_y = 34.6 \text{ in}^3 \quad Z_y = 53.2 \text{ in}^3 \quad r_y = 2.94 \text{ in}$$

$$S_x = 106 \text{ in}^3 \quad Z_x = 120 \text{ in}^3 \quad r_x = 5.14 \text{ in}$$

$$t := d - 2 \cdot k_o = 9.55 \text{ in} \quad \lambda_f := \frac{b_f}{2 \cdot t_f} = 8.978 \quad \lambda_w := \frac{t}{t_w} = 13.942$$

$$F_y := 50 \cdot \text{ksi}$$

$$E := 29000 \cdot \text{ksi}$$

FB-Pier Inputs:

$$P_u := 214.4 \cdot \text{kip}$$

$$Mu_{top} := 206.7 \cdot \text{kip} \cdot \text{ft}$$

$$Mu_{2nd} := 91.2 \cdot \text{kip} \cdot \text{ft}$$

$$lb_{top} := 45.4 \text{ in}$$

$$lb_{2nd} := 126 \cdot \text{in}$$

$$V_u := 58 \cdot \text{kip}$$

Resistance Factors

$$\phi_{cl} := 0.50$$

For lower zone of the pile

$\phi_{cu} := 0.70$ For the upper zone of the pile

$\phi_f := 1.0$ For the upper zone of the pile

Nominal Axial Pile Resistance
(Check Pile Size chosen)

$$R_n := \frac{P_u}{\phi_{cl}} \quad R_n = 428.8 \text{ kip}$$

$$A_{sreq} := \frac{R_n}{(0.8 \cdot F_y)} \quad A_{sreq} = 10.72 \text{ in}^2$$

Check if Area Required is less than chosen pile area:

$$A_{sreq} < A_s = 1 \quad (\text{Should see 1})$$

Elastic Flexural Buckling Resistance

(k_{top} = 1.2 first, 2.1 if hinge forms)

$$k_{top} := 1.2$$

$$k_{2nd} := 1.0$$

Calculate Column Slenderness Factors

$$\lambda_{top} := \left(\frac{k_{top} \cdot lb_{top}}{r_y \cdot \pi} \right)^2 \cdot \left(\frac{F_y}{E} \right) \quad \lambda_{top} = 0.06$$

Check:

$$\lambda_{top} < 2.25 = 1 \quad (\text{Should see 1})$$

$$\lambda_{2nd} := \left(\frac{k_{2nd} \cdot lb_{2nd}}{r_y \cdot \pi} \right)^2 \cdot \left(\frac{F_y}{E} \right) \quad \lambda_{2nd} = 0.321$$

$$\lambda_{2nd} < 2.25 = 1 \quad (\text{Should see 1})$$

$$P_{etop} := \pi^2 \cdot \frac{E \cdot A_s}{\left(\frac{k_{top} \cdot lb_{top}}{r_y} \right)^2} = (2.05 \cdot 10^4) \text{ kip}$$

AASHTO 6.9.4.1.2-1

$$P_{e2nd} := \pi^2 \cdot \frac{E \cdot A_s}{\left(\frac{k_{2nd} \cdot lb_{2nd}}{r_y} \right)^2} = (3.833 \cdot 10^3) \text{ kip}$$

$$Q := 1.0$$

use 1 for nonslender elements

AASHTO C6.9.4.1.1

$$P_n := Q \cdot F_u \cdot A_s = (1.23 \cdot 10^3) \text{ kip}$$

Nominal Structural Pile Resistance

AASHTO 6.9.4.2.1

AASHTO 6.9.4.2.1-1

$$Pn_{top} := \left\| \begin{array}{l} \text{if } \frac{P_{etop}}{P_o} \geq 0.44 \\ \left\| \begin{array}{l} 0.658 \left(\frac{P_o}{P_{etop}} \right) \cdot P_o \\ \text{if } \frac{P_{etop}}{P_o} < 0.44 \\ \left\| \begin{array}{l} 0.877 \cdot P_{etop} \end{array} \right. \end{array} \right. \end{array} \right\|$$

AASHTO 6.9.4.2.1-2

$$Pn_{top} = 1199.5 \text{ kip}$$

$$Pn_{2nd} := \begin{cases} \text{if } \frac{P_{e2nd}}{P_o} \geq 0.44 \\ \quad \left\| 0.658 \left(\frac{P_o}{P_{e2nd}} \right) \cdot P_o \right\| \\ \text{if } \frac{P_{e2nd}}{P_o} < 0.44 \\ \quad \left\| 0.877 \cdot P_{e2nd} \right\| \end{cases}$$

$$Pn_{2nd} = 1075.43 \text{ kip}$$

$$Pn_{bot} := 0.66^{0.0} \cdot F_y \cdot A_s$$

$$Pn_{bot} = 1230 \text{ kip}$$

Factored Structural Pile Resistance

$$Pr_{top} := \phi cu \cdot Pn_{top}$$

$$Pr_{top} = 839.65 \text{ kip}$$

$$Pr_{2nd} := \phi cu \cdot Pn_{2nd}$$

$$Pr_{2nd} = 752.8 \text{ kip}$$

$$Pr_{bot} := \phi cl \cdot Pn_{bot}$$

$$Pr_{bot}=615 \text{ kip}$$

Check to make sure pile size is not too big:

$$\frac{P_u}{Pr_{top}} > 0.2 = 1 \quad \text{(Should see 1)}$$

$$\frac{P_u}{Pr_{2nd}} > 0.2 = 1 \quad \text{(Should see 1)}$$

$$\frac{P_u}{Pr_{bot}} < 1 = 1 \quad \text{(Should see 1)}$$

Calculate Nominal Flexural Resistance

$$\lambda_f = 8.978$$

Flange Slenderness

$$\lambda_{pf} := 0.38 \cdot \sqrt{\frac{E}{F_y}} \quad \lambda_{pf} = 9.152$$

Compact Flange

AASHTO 6.12.2.2.1-4

$$\lambda_{rf} := 0.56 \cdot \sqrt{\frac{E}{0.7 \cdot F_y}} \quad \lambda_{rf} = 16.12$$

Noncompact Flange

AASHTO 6.12.2.2.1-5

$$M_n := \left\| \begin{array}{l} \text{if } \lambda_{pf} < \lambda_f < \lambda_{rf} \\ \left\| \left(1 - \left(1 - \frac{S_y}{Z_y} \right) \cdot \left(\frac{\lambda_f - \lambda_{pf}}{0.45 \cdot \sqrt{\frac{E}{F_y}}} \right) \right) \cdot F_y \cdot Z_y \right\| \\ \text{if } \lambda_f \leq \lambda_{pf} \\ \left\| Z_y \cdot F_y \right\| \end{array} \right\|$$

AASHTO 6.12.2.2.1-2

$$M_n = 2660 \text{ in}\cdot\text{kip}$$

Calculate Flexural Resistance

$$\phi_f := 1$$

$$M_r := \phi_f \cdot M_n$$

$$M_r = 2660 \text{ in}\cdot\text{kip}$$

Check Moment that will cause plastic hinge

$$M_p := \frac{9}{8} \cdot \left(1 - \frac{P_u}{Pr_{top}} \right) \cdot M_r$$

$$M_p = 2228.4 \text{ in}\cdot\text{kip}$$

$$M_p = 185.7 \text{ ft}\cdot\text{kip}$$

$$Mu_{top} > M_p = 1$$

(Should see 1)

Since the Applied moment exceeds the moment that would create a plastic hinge, it can be assumed that the pile head has plastically deformed.

Now, make sure the 2nd segment DOES NOT develop a plastic hinge. A plastic hinge in this segment will cause the pile to fail!

$$I := \left\| \begin{array}{l} \text{if } \frac{P_u}{Pr_{2nd}} \geq 0.2 \\ \left\| \frac{P_u}{Pr_{2nd}} + \frac{8}{9} \cdot \left(\frac{Mu_{2nd}}{M_r} \right) \right\| \\ \text{if } \frac{P_u}{Pr_{2nd}} < 0.2 \\ \left\| \frac{P_u}{2.0 \cdot Pr_{2nd}} + \left(\frac{Mu_{2nd}}{M_r} \right) \right\| \end{array} \right\|$$

AASHTO 6.9.2.2-2

AASHTO 6.9.2.2-1

$$P_u = 214.4 \text{ kip}$$

$$I = 0.651 \quad I < 1 = 1$$

(Should see 1)

Now Check for Shear

$$\phi_v := 1.0$$

$$k_v := 1.2 \quad \text{AISC Steel Manual, G7}$$

$$C_v := 1.0$$

$$A_w := 2 \cdot b_f \cdot t_f \quad A_w = 16.851 \text{ in}^2$$

$$V_n := 0.6 \cdot F_y \cdot A_w \cdot C_v \quad V_n = 505.53 \text{ kip}$$

$$V_r := \phi_v \cdot V_n \quad V_r = 505.53 \text{ kip}$$

$$\frac{V_u}{V_r} < 1 = 1 \quad \text{(Should see 1)}$$

Integral Abutment Design Worksheet (See IAB Manual 2010 Appendix B Design Example)

Project Name: Mount Holly BF 1442(56)

FB-Pier Corresponding File Name: Abutment 1 STR I NS 2nd.in

Analysis By: A. Arles

6/01/2026

Piles Oriented for Weak Axis Bending

Pile Size Inputs:

Pile := HP 12x84 ▾

$$b_f = 12.3 \text{ in} \quad t_f = 0.685 \text{ in} \quad t_w = 0.685 \text{ in} \quad d = 12.3 \text{ in} \quad k_o = 1.375 \text{ in} \quad A_s = 24.6 \text{ in}^2$$

$$S_y = 34.6 \text{ in}^3 \quad Z_y = 53.2 \text{ in}^3 \quad r_y = 2.94 \text{ in}$$

$$S_x = 106 \text{ in}^3 \quad Z_x = 120 \text{ in}^3 \quad r_x = 5.14 \text{ in}$$

$$t := d - 2 \cdot k_o = 9.55 \text{ in} \quad \lambda_f := \frac{b_f}{2 \cdot t_f} = 8.978 \quad \lambda_w := \frac{t}{t_w} = 13.942$$

$$F_y := 50 \cdot \text{ksi}$$

$$E := 29000 \cdot \text{ksi}$$

FB-Pier Inputs:

$$P_u := 214.4 \cdot \text{kip}$$

$$Mu_{top} := 185.7 \cdot \text{kip} \cdot \text{ft}$$

$$Mu_{2nd} := 89.3 \cdot \text{kip} \cdot \text{ft}$$

$$lb_{top} := 42.9 \text{ in}$$

$$lb_{2nd} := 128.5 \cdot \text{in}$$

$$V_u := 54.4 \cdot \text{kip}$$

Resistance Factors

$$\phi_{cl} := 0.50$$

For lower zone of the pile

$\phi_{cu} := 0.70$ For the upper zone of the pile

$\phi_f := 1.0$ For the upper zone of the pile

Nominal Axial Pile Resistance
(Check Pile Size chosen)

$$R_n := \frac{P_u}{\phi_{cl}} \quad R_n = 428.8 \text{ kip}$$

$$A_{sreq} := \frac{R_n}{(0.8 \cdot F_y)} \quad A_{sreq} = 10.72 \text{ in}^2$$

Check if Area Required is less than chosen pile area:

$$A_{sreq} < A_s = 1 \quad (\text{Should see 1})$$

Elastic Flexural Buckling Resistance

($k_{top} = 1.2$ first, 2.1 if hinge forms)

$$k_{top} := 2.1$$

$$k_{2nd} := 1.0$$

Calculate Column Slenderness Factors

$$\lambda_{top} := \left(\frac{k_{top} \cdot lb_{top}}{r_y \cdot \pi} \right)^2 \cdot \left(\frac{F_y}{E} \right) \quad \lambda_{top} = 0.164$$

Check:

$$\lambda_{top} < 2.25 = 1 \quad (\text{Should see 1})$$

$$\lambda_{2nd} := \left(\frac{k_{2nd} \cdot lb_{2nd}}{r_y \cdot \pi} \right)^2 \cdot \left(\frac{F_y}{E} \right) \quad \lambda_{2nd} = 0.334$$

$$\lambda_{2nd} < 2.25 = 1 \quad (\text{Should see 1})$$

$$P_{etop} := \pi^2 \cdot \frac{E \cdot A_s}{\left(\frac{k_{top} \cdot lb_{top}}{r_y} \right)^2} = (7.498 \cdot 10^3) \text{ kip}$$

AASHTO 6.9.4.1.2-1

$$P_{e2nd} := \pi^2 \cdot \frac{E \cdot A_s}{\left(\frac{k_{2nd} \cdot lb_{2nd}}{r_y} \right)^2} = (3.686 \cdot 10^3) \text{ kip}$$

$$Q := 1.0$$

use 1 for nonslender elements

AASHTO C6.9.4.1.1

$$P_n := Q \cdot F_u \cdot A_n = (1.23 \cdot 10^3) \text{ kip}$$

Nominal Structural Pile Resistance

AASHTO 6.9.4.2.1

AASHTO 6.9.4.2.1-1

$$Pn_{top} := \left\| \begin{array}{l} \text{if } \frac{P_{etop}}{P_o} \geq 0.44 \\ \left\| 0.658 \left(\frac{P_o}{P_{etop}} \right) \cdot P_o \right\| \\ \text{if } \frac{P_{etop}}{P_o} < 0.44 \\ \left\| 0.877 \cdot P_{etop} \right\| \end{array} \right\|$$

AASHTO 6.9.4.2.1-2

$$Pn_{top} = 1148.39 \text{ kip}$$

$$Pn_{2nd} := \left\| \begin{array}{l} \text{if } \frac{P_{e2nd}}{P_o} \geq 0.44 \\ \left\| 0.658 \left(\frac{P_o}{P_{e2nd}} \right) \cdot P_o \right\| \\ \text{if } \frac{P_{e2nd}}{P_o} < 0.44 \\ \left\| 0.877 \cdot P_{e2nd} \right\| \end{array} \right\|$$

$$Pn_{2nd} = 1069.65 \text{ kip}$$

$$Pn_{bot} := 0.66^{0.0} \cdot F_y \cdot A_s$$

$$Pn_{bot} = 1230 \text{ kip}$$

Factored Structural Pile Resistance

$$Pr_{top} := \phi_{cu} \cdot Pn_{top}$$

$$Pr_{top} = 803.87 \text{ kip}$$

$$Pr_{2nd} := \phi_{cu} \cdot Pn_{2nd}$$

$$Pr_{2nd} = 748.76 \text{ kip}$$

$$Pr_{bot} := \phi_{cl} \cdot Pn_{bot}$$

$$Pr_{bot} = 615 \text{ kip}$$

Check to make sure pile size is not too big:

$$\frac{P_u}{Pr_{top}} > 0.2 = 1 \quad (\text{Should see 1})$$

$$\frac{P_u}{Pr_{2nd}} > 0.2 = 1 \quad (\text{Should see 1})$$

$$\frac{P_u}{Pr_{bot}} < 1 = 1 \quad (\text{Should see 1})$$

Calculate Nominal Flexural Resistance

$$\lambda_f = 8.978$$

Flange Slenderness

$$\lambda_{pf} := 0.38 \cdot \sqrt{\frac{E}{F_y}} \quad \lambda_{pf} = 9.152$$

Compact Flange

AASHTO 6.12.2.2.1-4

$$\lambda_{rf} := 0.56 \cdot \sqrt{\frac{E}{0.7 \cdot F_y}} \quad \lambda_{rf} = 16.12$$

Noncompact Flange

AASHTO 6.12.2.2.1-5

$$M_n := \left\| \begin{array}{l} \text{if } \lambda_{pf} < \lambda_f < \lambda_{rf} \\ \left\| \left(1 - \left(1 - \frac{S_y}{Z_y} \right) \cdot \left(\frac{\lambda_f - \lambda_{pf}}{0.45 \cdot \sqrt{\frac{E}{F_y}}} \right) \right) \cdot F_y \cdot Z_y \right\| \\ \text{if } \lambda_f \leq \lambda_{pf} \\ \left\| Z_y \cdot F_y \right\| \end{array} \right\|$$

AASHTO 6.12.2.2.1-2

$$M_n = 2660 \text{ in}\cdot\text{kip}$$

Calculate Flexural Resistance

$$\phi_f := 1$$

$$M_r := \phi_f \cdot M_n$$

$$M_r = 2660 \text{ in}\cdot\text{kip}$$

Check Moment that will cause plastic hinge

$$M_p := \frac{9}{8} \cdot \left(1 - \frac{P_u}{Pr_{top}} \right) \cdot M_r$$

$$M_p = 2194.4 \text{ in}\cdot\text{kip}$$

$$M_p = 182.9 \text{ ft}\cdot\text{kip}$$

$$Mu_{top} > M_p = 1$$

(Should see 1)

Since the Applied moment exceeds the moment that would create a plastic hinge, it can be assumed that the pile head has plastically deformed.

Now, make sure the 2nd segment DOES NOT develop a plastic hinge. A plastic hinge in this segment will cause the pile to fail!

$$I := \left\| \begin{array}{l} \text{if } \frac{P_u}{Pr_{2nd}} \geq 0.2 \\ \left\| \frac{P_u}{Pr_{2nd}} + \frac{8}{9} \cdot \left(\frac{Mu_{2nd}}{M_r} \right) \right\| \\ \text{if } \frac{P_u}{Pr_{2nd}} < 0.2 \\ \left\| \frac{P_u}{2.0 \cdot Pr_{2nd}} + \left(\frac{Mu_{2nd}}{M_r} \right) \right\| \end{array} \right\|$$

AASHTO 6.9.2.2-2

AASHTO 6.9.2.2-1

$$P_u = 214.4 \text{ kip}$$

$$I = 0.644 \quad I < 1 = 1$$

(Should see 1)

Now Check for Shear

$$\phi_v := 1.0$$

$$k_v := 1.2 \quad \text{AISC Steel Manual, G7}$$

$$C_v := 1.0$$

$$A_w := 2 \cdot b_f \cdot t_f \quad A_w = 16.851 \text{ in}^2$$

$$V_n := 0.6 \cdot F_y \cdot A_w \cdot C_v \quad V_n = 505.53 \text{ kip}$$

$$V_r := \phi_v \cdot V_n \quad V_r = 505.53 \text{ kip}$$

$$\frac{V_u}{V_r} < 1 = 1 \quad \text{(Should see 1)}$$

Integral Abutment Design Worksheet (See IAB Manual 2010 Appendix B Design Example)

Project Name: Mount Holly BF 1442(56)

FB-Pier Corresponding File Name: Abutment 1 STR I NS.in

Analysis By: A. Arles 6/01/2026 Piles Oriented for Weak Axis Bending

Pile Size Inputs:

Pile := HP 12x84 ▾

$$b_f = 12.3 \text{ in} \quad t_f = 0.685 \text{ in} \quad t_w = 0.685 \text{ in} \quad d = 12.3 \text{ in} \quad k_o = 1.375 \text{ in} \quad A_s = 24.6 \text{ in}^2$$

$$S_y = 34.6 \text{ in}^3 \quad Z_y = 53.2 \text{ in}^3 \quad r_y = 2.94 \text{ in}$$

$$S_x = 106 \text{ in}^3 \quad Z_x = 120 \text{ in}^3 \quad r_x = 5.14 \text{ in}$$

$$t := d - 2 \cdot k_o = 9.55 \text{ in} \quad \lambda_f := \frac{b_f}{2 \cdot t_f} = 8.978 \quad \lambda_w := \frac{t}{t_w} = 13.942$$

$$F_y := 50 \cdot \text{ksi}$$

$$E := 29000 \cdot \text{ksi}$$

FB-Pier Inputs:

$$P_u := 214.4 \cdot \text{kip}$$

$$Mu_{top} := 206.7 \cdot \text{kip} \cdot \text{ft}$$

$$Mu_{2nd} := 89.3 \cdot \text{kip} \cdot \text{ft}$$

$$lb_{top} := 45.4 \text{ in}$$

$$lb_{2nd} := 126.1 \cdot \text{in}$$

$$V_u := 58 \cdot \text{kip}$$

Resistance Factors

$$\phi_{cl} := 0.50$$

For lower zone of the pile

$\phi_{cu} := 0.70$ For the upper zone of the pile

$\phi_f := 1.0$ For the upper zone of the pile

Nominal Axial Pile Resistance
(Check Pile Size chosen)

$$R_n := \frac{P_u}{\phi_{cl}} \quad R_n = 428.8 \text{ kip}$$

$$A_{sreq} := \frac{R_n}{(0.8 \cdot F_y)} \quad A_{sreq} = 10.72 \text{ in}^2$$

Check if Area Required is less than chosen pile area:

$$A_{sreq} < A_s = 1 \quad (\text{Should see 1})$$

Elastic Flexural Buckling Resistance

(k_{top} = 1.2 first, 2.1 if hinge forms)

$$k_{top} := 1.2$$

$$k_{2nd} := 1.0$$

Calculate Column Slenderness Factors

$$\lambda_{top} := \left(\frac{k_{top} \cdot lb_{top}}{r_y \cdot \pi} \right)^2 \cdot \left(\frac{F_y}{E} \right) \quad \lambda_{top} = 0.06$$

Check:

$$\lambda_{top} < 2.25 = 1 \quad (\text{Should see 1})$$

$$\lambda_{2nd} := \left(\frac{k_{2nd} \cdot lb_{2nd}}{r_y \cdot \pi} \right)^2 \cdot \left(\frac{F_y}{E} \right) \quad \lambda_{2nd} = 0.321$$

$$\lambda_{2nd} < 2.25 = 1 \quad (\text{Should see 1})$$

$$P_{etop} := \pi^2 \cdot \frac{E \cdot A_s}{\left(\frac{k_{top} \cdot lb_{top}}{r_y} \right)^2} = (2.05 \cdot 10^4) \text{ kip}$$

AASHTO 6.9.4.1.2-1

$$P_{e2nd} := \pi^2 \cdot \frac{E \cdot A_s}{\left(\frac{k_{2nd} \cdot lb_{2nd}}{r_y} \right)^2} = (3.827 \cdot 10^3) \text{ kip}$$

$$Q := 1.0$$

use 1 for nonslender elements

AASHTO C6.9.4.1.1

$$P_n := Q \cdot F_u \cdot A_s = (1.23 \cdot 10^3) \text{ kip}$$

Nominal Structural Pile Resistance

AASHTO 6.9.4.2.1

AASHTO 6.9.4.2.1-1

$$Pn_{top} := \left\| \begin{array}{l} \text{if } \frac{P_{etop}}{P_o} \geq 0.44 \\ \left\| 0.658 \left(\frac{P_o}{P_{etop}} \right) \cdot P_o \right\| \\ \text{if } \frac{P_{etop}}{P_o} < 0.44 \\ \left\| 0.877 \cdot P_{etop} \right\| \end{array} \right\|$$

AASHTO 6.9.4.2.1-2

$$Pn_{top} = 1199.5 \text{ kip}$$

$$Pn_{2nd} := \left\| \begin{array}{l} \text{if } \frac{P_{e2nd}}{P_o} \geq 0.44 \\ \left\| 0.658 \left(\frac{P_o}{P_{e2nd}} \right) \cdot P_o \right\| \\ \text{if } \frac{P_{e2nd}}{P_o} < 0.44 \\ \left\| 0.877 \cdot P_{e2nd} \right\| \end{array} \right\|$$

$$Pn_{2nd} = 1075.2 \text{ kip}$$

$$Pn_{bot} := 0.66^{0.0} \cdot F_y \cdot A_s$$

$$Pn_{bot} = 1230 \text{ kip}$$

Factored Structural Pile Resistance

$$Pr_{top} := \phi_{cu} \cdot Pn_{top}$$

$$Pr_{top} = 839.65 \text{ kip}$$

$$Pr_{2nd} := \phi_{cu} \cdot Pn_{2nd}$$

$$Pr_{2nd} = 752.64 \text{ kip}$$

$$Pr_{bot} := \phi_{cl} \cdot Pn_{bot}$$

$$Pr_{bot} = 615 \text{ kip}$$

Check to make sure pile size is not too big:

$$\frac{P_u}{Pr_{top}} > 0.2 = 1 \quad \text{(Should see 1)}$$

$$\frac{P_u}{Pr_{2nd}} > 0.2 = 1 \quad \text{(Should see 1)}$$

$$\frac{P_u}{Pr_{bot}} < 1 = 1 \quad \text{(Should see 1)}$$

Calculate Nominal Flexural Resistance

$$\lambda_f = 8.978$$

Flange Slenderness

$$\lambda_{pf} := 0.38 \cdot \sqrt{\frac{E}{F_y}} \quad \lambda_{pf} = 9.152$$

Compact Flange

AASHTO 6.12.2.2.1-4

$$\lambda_{rf} := 0.56 \cdot \sqrt{\frac{E}{0.7 \cdot F_y}} \quad \lambda_{rf} = 16.12$$

Noncompact Flange

AASHTO 6.12.2.2.1-5

$$M_n := \left\| \begin{array}{l} \text{if } \lambda_{pf} < \lambda_f < \lambda_{rf} \\ \left\| \left(1 - \left(1 - \frac{S_y}{Z_y} \right) \cdot \left(\frac{\lambda_f - \lambda_{pf}}{0.45 \cdot \sqrt{\frac{E}{F_y}}} \right) \right) \cdot F_y \cdot Z_y \right\| \\ \text{if } \lambda_f \leq \lambda_{pf} \\ \left\| Z_y \cdot F_y \right\| \end{array} \right\|$$

AASHTO 6.12.2.2.1-2

$$M_n = 2660 \text{ in}\cdot\text{kip}$$

Calculate Flexural Resistance

$$\phi_f := 1$$

$$M_r := \phi_f \cdot M_n$$

$$M_r = 2660 \text{ in}\cdot\text{kip}$$

Check Moment that will cause plastic hinge

$$M_p := \frac{9}{8} \cdot \left(1 - \frac{P_u}{Pr_{top}} \right) \cdot M_r$$

$$M_p = 2228.4 \text{ in}\cdot\text{kip}$$

$$M_p = 185.7 \text{ ft}\cdot\text{kip}$$

$$Mu_{top} > M_p = 1$$

(Should see 1)

Since the Applied moment exceeds the moment that would create a plastic hinge, it can be assumed that the pile head has plastically deformed.

Now, make sure the 2nd segment DOES NOT develop a plastic hinge. A plastic hinge in this segment will cause the pile to fail!

$$I := \left\| \begin{array}{l} \text{if } \frac{P_u}{Pr_{2nd}} \geq 0.2 \\ \left\| \frac{P_u}{Pr_{2nd}} + \frac{8}{9} \cdot \left(\frac{Mu_{2nd}}{M_r} \right) \right\| \\ \text{if } \frac{P_u}{Pr_{2nd}} < 0.2 \\ \left\| \frac{P_u}{2.0 \cdot Pr_{2nd}} + \left(\frac{Mu_{2nd}}{M_r} \right) \right\| \end{array} \right\|$$

AASHTO 6.9.2.2-2

AASHTO 6.9.2.2-1

$$P_u = 214.4 \text{ kip}$$

$$I = 0.643 \quad I < 1 = 1$$

(Should see 1)

Now Check for Shear

$$\phi_v := 1.0$$

$$k_v := 1.2 \quad \text{AISC Steel Manual, G7}$$

$$C_v := 1.0$$

$$A_w := 2 \cdot b_f \cdot t_f \quad A_w = 16.851 \text{ in}^2$$

$$V_n := 0.6 \cdot F_y \cdot A_w \cdot C_v \quad V_n = 505.53 \text{ kip}$$

$$V_r := \phi_v \cdot V_n \quad V_r = 505.53 \text{ kip}$$

$$\frac{V_u}{V_r} < 1 = 1 \quad \text{(Should see 1)}$$

Integral Abutment Design Worksheet (See IAB Manual 2010 Appendix B Design Example)

Project Name: Mount Holly BF 1442(56)

FB-Pier Corresponding File Name: Abutment 2 STR I NS.in

Analysis By: A. Arles 6/01/2026 Piles Oriented for Weak Axis Bending

Pile Size Inputs:

Pile := HP 12x84 ▾

$$b_f = 12.3 \text{ in} \quad t_f = 0.685 \text{ in} \quad t_w = 0.685 \text{ in} \quad d = 12.3 \text{ in} \quad k_o = 1.375 \text{ in} \quad A_s = 24.6 \text{ in}^2$$

$$S_y = 34.6 \text{ in}^3 \quad Z_y = 53.2 \text{ in}^3 \quad r_y = 2.94 \text{ in}$$

$$S_x = 106 \text{ in}^3 \quad Z_x = 120 \text{ in}^3 \quad r_x = 5.14 \text{ in}$$

$$t := d - 2 \cdot k_o = 9.55 \text{ in} \quad \lambda_f := \frac{b_f}{2 \cdot t_f} = 8.978 \quad \lambda_w := \frac{t}{t_w} = 13.942$$

$$F_y := 50 \cdot \text{ksi}$$

$$E := 29000 \cdot \text{ksi}$$

FB-Pier Inputs:

$$P_u := 214.4 \cdot \text{kip}$$

$$Mu_{top} := 195.3 \cdot \text{kip} \cdot \text{ft}$$

$$Mu_{2nd} := 78.5 \cdot \text{kip} \cdot \text{ft}$$

$$lb_{top} := 51.4 \text{ in}$$

$$lb_{2nd} := 135.3 \cdot \text{in}$$

$$V_u := 49 \cdot \text{kip}$$

Resistance Factors

$$\phi_{cl} := 0.50$$

For lower zone of the pile

$\phi_{cu} := 0.70$ For the upper zone of the pile

$\phi_f := 1.0$ For the upper zone of the pile

Nominal Axial Pile Resistance
(Check Pile Size chosen)

$$R_n := \frac{P_u}{\phi_{cl}} \quad R_n = 428.8 \text{ kip}$$

$$A_{sreq} := \frac{R_n}{(0.8 \cdot F_y)} \quad A_{sreq} = 10.72 \text{ in}^2$$

Check if Area Required is less than chosen pile area:

$$A_{sreq} < A_s = 1 \quad (\text{Should see 1})$$

Elastic Flexural Buckling Resistance

(k_{top} = 1.2 first, 2.1 if hinge forms)

$$k_{top} := 1.2$$

$$k_{2nd} := 1.0$$

Calculate Column Slenderness Factors

$$\lambda_{top} := \left(\frac{k_{top} \cdot l_{b_{top}}}{r_y \cdot \pi} \right)^2 \cdot \left(\frac{F_y}{E} \right) \quad \lambda_{top} = 0.077$$

Check:

$$\lambda_{top} < 2.25 = 1 \quad (\text{Should see 1})$$

$$\lambda_{2nd} := \left(\frac{k_{2nd} \cdot l_{b_{2nd}}}{r_y \cdot \pi} \right)^2 \cdot \left(\frac{F_y}{E} \right) \quad \lambda_{2nd} = 0.37$$

$$\lambda_{2nd} < 2.25 = 1 \quad (\text{Should see 1})$$

$$P_{etop} := \pi^2 \cdot \frac{E \cdot A_s}{\left(\frac{k_{top} \cdot l_{b_{top}}}{r_y} \right)^2} = (1.6 \cdot 10^4) \text{ kip}$$

AASHTO 6.9.4.1.2-1

$$P_{e2nd} := \pi^2 \cdot \frac{E \cdot A_s}{\left(\frac{k_{2nd} \cdot l_{b_{2nd}}}{r_y} \right)^2} = (3.325 \cdot 10^3) \text{ kip}$$

$$Q := 1.0$$

use 1 for nonslender elements

AASHTO C6.9.4.1.1

$$P_n := Q \cdot F_u \cdot A_n = (1.23 \cdot 10^3) \text{ kip}$$

Nominal Structural Pile Resistance

AASHTO 6.9.4.2.1

AASHTO 6.9.4.2.1-1

$$Pn_{top} := \left\| \begin{array}{l} \text{if } \frac{P_{etop}}{P_o} \geq 0.44 \\ \left\| 0.658 \left(\frac{P_o}{P_{etop}} \right) \cdot P_o \right\| \\ \text{if } \frac{P_{etop}}{P_o} < 0.44 \\ \left\| 0.877 \cdot P_{etop} \right\| \end{array} \right\|$$

AASHTO 6.9.4.2.1-2

$$Pn_{top} = 1191.05 \text{ kip}$$

$$Pn_{2nd} := \left\| \begin{array}{l} \text{if } \frac{P_{e2nd}}{P_o} \geq 0.44 \\ \left\| 0.658 \left(\frac{P_o}{P_{e2nd}} \right) \cdot P_o \right\| \\ \text{if } \frac{P_{e2nd}}{P_o} < 0.44 \\ \left\| 0.877 \cdot P_{e2nd} \right\| \end{array} \right\|$$

$$Pn_{2nd} = 1053.55 \text{ kip}$$

$$Pn_{bot} := 0.66^{0.0} \cdot F_y \cdot A_s$$

$$Pn_{bot} = 1230 \text{ kip}$$

Factored Structural Pile Resistance

$$Pr_{top} := \phi_{cu} \cdot Pn_{top}$$

$$Pr_{top} = 833.73 \text{ kip}$$

$$Pr_{2nd} := \phi_{cu} \cdot Pn_{2nd}$$

$$Pr_{2nd} = 737.48 \text{ kip}$$

$$Pr_{bot} := \phi_{cl} \cdot Pn_{bot}$$

$$Pr_{bot} = 615 \text{ kip}$$

Check to make sure pile size is not too big:

$$\frac{P_u}{Pr_{top}} > 0.2 = 1 \quad \text{(Should see 1)}$$

$$\frac{P_u}{Pr_{2nd}} > 0.2 = 1 \quad \text{(Should see 1)}$$

$$\frac{P_u}{Pr_{bot}} < 1 = 1 \quad \text{(Should see 1)}$$

Calculate Nominal Flexural Resistance

$$\lambda_f = 8.978$$

Flange Slenderness

$$\lambda_{pf} := 0.38 \cdot \sqrt{\frac{E}{F_y}} \quad \lambda_{pf} = 9.152$$

Compact Flange

AASHTO 6.12.2.2.1-4

$$\lambda_{rf} := 0.56 \cdot \sqrt{\frac{E}{0.7 \cdot F_y}} \quad \lambda_{rf} = 16.12$$

Noncompact Flange

AASHTO 6.12.2.2.1-5

$$M_n := \left\| \begin{array}{l} \text{if } \lambda_{pf} < \lambda_f < \lambda_{rf} \\ \left\| \left(1 - \left(1 - \frac{S_y}{Z_y} \right) \cdot \left(\frac{\lambda_f - \lambda_{pf}}{0.45 \cdot \sqrt{\frac{E}{F_y}}} \right) \right) \cdot F_y \cdot Z_y \right\| \\ \text{if } \lambda_f \leq \lambda_{pf} \\ \left\| Z_y \cdot F_y \right\| \end{array} \right\|$$

AASHTO 6.12.2.2.1-2

$$M_n = 2660 \text{ in}\cdot\text{kip}$$

Calculate Flexural Resistance

$$\phi_f := 1$$

$$M_r := \phi_f \cdot M_n$$

$$M_r = 2660 \text{ in}\cdot\text{kip}$$

Check Moment that will cause plastic hinge

$$M_p := \frac{9}{8} \cdot \left(1 - \frac{P_u}{Pr_{top}} \right) \cdot M_r$$

$$M_p = 2223 \text{ in}\cdot\text{kip}$$

$$M_p = 185.2 \text{ ft}\cdot\text{kip}$$

$$Mu_{top} > M_p = 1$$

(Should see 1)

Since the Applied moment exceeds the moment that would create a plastic hinge, it can be assumed that the pile head has plastically deformed.

Now, make sure the 2nd segment DOES NOT develop a plastic hinge. A plastic hinge in this segment will cause the pile to fail!

$$I := \left\| \begin{array}{l} \text{if } \frac{P_u}{Pr_{2nd}} \geq 0.2 \\ \left\| \frac{P_u}{Pr_{2nd}} + \frac{8}{9} \cdot \left(\frac{Mu_{2nd}}{M_r} \right) \right\| \\ \text{if } \frac{P_u}{Pr_{2nd}} < 0.2 \\ \left\| \frac{P_u}{2.0 \cdot Pr_{2nd}} + \left(\frac{Mu_{2nd}}{M_r} \right) \right\| \end{array} \right\|$$

AASHTO 6.9.2.2-2

AASHTO 6.9.2.2-1

$$P_u = 214.4 \text{ kip}$$

$$I = 0.606 \quad I < 1 = 1$$

(Should see 1)

Now Check for Shear

$$\phi_v := 1.0$$

$$k_v := 1.2 \quad \text{AISC Steel Manual, G7}$$

$$C_v := 1.0$$

$$A_w := 2 \cdot b_f \cdot t_f \quad A_w = 16.851 \text{ in}^2$$

$$V_n := 0.6 \cdot F_y \cdot A_w \cdot C_v \quad V_n = 505.53 \text{ kip}$$

$$V_r := \phi_v \cdot V_n \quad V_r = 505.53 \text{ kip}$$

$$\frac{V_u}{V_r} < 1 = 1 \quad \text{(Should see 1)}$$

Integral Abutment Design Worksheet (See IAB Manual 2010 Appendix B Design Example)

Project Name: Mount Holly BF 1442(56)

FB-Pier Corresponding File Name: Abutment 2 STR I NS 2nd.in

Analysis By: A. Arles

6/01/2026

Piles Oriented for Weak Axis Bending

Pile Size Inputs:

Pile := HP 12x84 ▾

$$b_f = 12.3 \text{ in} \quad t_f = 0.685 \text{ in} \quad t_w = 0.685 \text{ in} \quad d = 12.3 \text{ in} \quad k_o = 1.375 \text{ in} \quad A_s = 24.6 \text{ in}^2$$

$$S_y = 34.6 \text{ in}^3 \quad Z_y = 53.2 \text{ in}^3 \quad r_y = 2.94 \text{ in}$$

$$S_x = 106 \text{ in}^3 \quad Z_x = 120 \text{ in}^3 \quad r_x = 5.14 \text{ in}$$

$$t := d - 2 \cdot k_o = 9.55 \text{ in} \quad \lambda_f := \frac{b_f}{2 \cdot t_f} = 8.978 \quad \lambda_w := \frac{t}{t_w} = 13.942$$

$$F_y := 50 \cdot \text{ksi}$$

$$E := 29000 \cdot \text{ksi}$$

FB-Pier Inputs:

$$P_u := 214.4 \cdot \text{kip}$$

$$Mu_{top} := 185.3 \cdot \text{kip} \cdot \text{ft}$$

$$Mu_{2nd} := 77.8 \cdot \text{kip} \cdot \text{ft}$$

$$lb_{top} := 50 \text{ in}$$

$$lb_{2nd} := 136 \cdot \text{in}$$

$$V_u := 47.2 \cdot \text{kip}$$

Resistance Factors

$$\phi_{cl} := 0.50$$

For lower zone of the pile

$\phi_{cu} := 0.70$ For the upper zone of the pile

$\phi_f := 1.0$ For the upper zone of the pile

Nominal Axial Pile Resistance
(Check Pile Size chosen)

$$R_n := \frac{P_u}{\phi_{cl}} \quad R_n = 428.8 \text{ kip}$$

$$A_{sreq} := \frac{R_n}{(0.8 \cdot F_y)} \quad A_{sreq} = 10.72 \text{ in}^2$$

Check if Area Required is less than chosen pile area:

$$A_{sreq} < A_s = 1 \quad (\text{Should see 1})$$

Elastic Flexural Buckling Resistance

(k_{top} = 1.2 first, 2.1 if hinge forms)

$$k_{top} := 2.1$$

$$k_{2nd} := 1.0$$

Calculate Column Slenderness Factors

$$\lambda_{top} := \left(\frac{k_{top} \cdot lb_{top}}{r_y \cdot \pi} \right)^2 \cdot \left(\frac{F_y}{E} \right) \quad \lambda_{top} = 0.223$$

Check:

$$\lambda_{top} < 2.25 = 1 \quad (\text{Should see 1})$$

$$\lambda_{2nd} := \left(\frac{k_{2nd} \cdot lb_{2nd}}{r_y \cdot \pi} \right)^2 \cdot \left(\frac{F_y}{E} \right) \quad \lambda_{2nd} = 0.374$$

$$\lambda_{2nd} < 2.25 = 1 \quad (\text{Should see 1})$$

$$P_{etop} := \pi^2 \cdot \frac{E \cdot A_s}{\left(\frac{k_{top} \cdot lb_{top}}{r_y} \right)^2} = (5.52 \cdot 10^3) \text{ kip}$$

AASHTO 6.9.4.1.2-1

$$P_{e2nd} := \pi^2 \cdot \frac{E \cdot A_s}{\left(\frac{k_{2nd} \cdot lb_{2nd}}{r_y} \right)^2} = (3.29 \cdot 10^3) \text{ kip}$$

$$Q := 1.0$$

use 1 for nonslender elements

AASHTO C6.9.4.1.1

$$P_n := Q \cdot F_u \cdot A_n = (1.23 \cdot 10^3) \text{ kip}$$

Nominal Structural Pile Resistance

AASHTO 6.9.4.2.1

AASHTO 6.9.4.2.1-1

$$Pn_{top} := \left\| \begin{array}{l} \text{if } \frac{P_{etop}}{P_o} \geq 0.44 \\ \left\| 0.658 \left(\frac{P_o}{P_{etop}} \right) \cdot P_o \right\| \\ \text{if } \frac{P_{etop}}{P_o} < 0.44 \\ \left\| 0.877 \cdot P_{etop} \right\| \end{array} \right\|$$

AASHTO 6.9.4.2.1-2

$$Pn_{top} = 1120.47 \text{ kip}$$

$$Pn_{2nd} := \left\| \begin{array}{l} \text{if } \frac{P_{e2nd}}{P_o} \geq 0.44 \\ \left\| 0.658 \left(\frac{P_o}{P_{e2nd}} \right) \cdot P_o \right\| \\ \text{if } \frac{P_{e2nd}}{P_o} < 0.44 \\ \left\| 0.877 \cdot P_{e2nd} \right\| \end{array} \right\|$$

$$Pn_{2nd} = 1051.85 \text{ kip}$$

$$Pn_{bot} := 0.66^{0.0} \cdot F_y \cdot A_s$$

$$Pn_{bot} = 1230 \text{ kip}$$

Factored Structural Pile Resistance

$$Pr_{top} := \phi_{cu} \cdot Pn_{top}$$

$$Pr_{top} = 784.33 \text{ kip}$$

$$Pr_{2nd} := \phi_{cu} \cdot Pn_{2nd}$$

$$Pr_{2nd} = 736.3 \text{ kip}$$

$$Pr_{bot} := \phi_{cl} \cdot Pn_{bot}$$

$$Pr_{bot} = 615 \text{ kip}$$

Check to make sure pile size is not too big:

$$\frac{P_u}{Pr_{top}} > 0.2 = 1 \quad \text{(Should see 1)}$$

$$\frac{P_u}{Pr_{2nd}} > 0.2 = 1 \quad \text{(Should see 1)}$$

$$\frac{P_u}{Pr_{bot}} < 1 = 1 \quad \text{(Should see 1)}$$

Calculate Nominal Flexural Resistance

$$\lambda_f = 8.978$$

Flange Slenderness

$$\lambda_{pf} := 0.38 \cdot \sqrt{\frac{E}{F_y}} \quad \lambda_{pf} = 9.152$$

Compact Flange

AASHTO 6.12.2.2.1-4

$$\lambda_{rf} := 0.56 \cdot \sqrt{\frac{E}{0.7 \cdot F_y}} \quad \lambda_{rf} = 16.12$$

Noncompact Flange

AASHTO 6.12.2.2.1-5

$$M_n := \left\| \begin{array}{l} \text{if } \lambda_{pf} < \lambda_f < \lambda_{rf} \\ \left\| \left(1 - \left(1 - \frac{S_y}{Z_y} \right) \cdot \left(\frac{\lambda_f - \lambda_{pf}}{0.45 \cdot \sqrt{\frac{E}{F_y}}} \right) \right) \cdot F_y \cdot Z_y \right\| \\ \text{if } \lambda_f \leq \lambda_{pf} \\ \left\| Z_y \cdot F_y \right\| \end{array} \right\|$$

AASHTO 6.12.2.2.1-2

$$M_n = 2660 \text{ in}\cdot\text{kip}$$

Calculate Flexural Resistance

$$\phi_f := 1$$

$$M_r := \phi_f \cdot M_n$$

$$M_r = 2660 \text{ in}\cdot\text{kip}$$

Check Moment that will cause plastic hinge

$$M_p := \frac{9}{8} \cdot \left(1 - \frac{P_u}{Pr_{top}} \right) \cdot M_r$$

$$M_p = 2174.5 \text{ in}\cdot\text{kip}$$

$$M_p = 181.2 \text{ ft}\cdot\text{kip}$$

$$Mu_{top} > M_p = 1$$

(Should see 1)

Since the Applied moment exceeds the moment that would create a plastic hinge, it can be assumed that the pile head has plastically deformed.

Now, make sure the 2nd segment DOES NOT develop a plastic hinge. A plastic hinge in this segment will cause the pile to fail!

$$I := \left\| \begin{array}{l} \text{if } \frac{P_u}{Pr_{2nd}} \geq 0.2 \\ \left\| \frac{P_u}{Pr_{2nd}} + \frac{8}{9} \cdot \left(\frac{Mu_{2nd}}{M_r} \right) \right\| \\ \text{if } \frac{P_u}{Pr_{2nd}} < 0.2 \\ \left\| \frac{P_u}{2.0 \cdot Pr_{2nd}} + \left(\frac{Mu_{2nd}}{M_r} \right) \right\| \end{array} \right\|$$

AASHTO 6.9.2.2-2

AASHTO 6.9.2.2-1

$$P_u = 214.4 \text{ kip}$$

$$I = 0.603 \quad I < 1 = 1$$

(Should see 1)

Now Check for Shear

$$\phi_v := 1.0$$

$$k_v := 1.2 \quad \text{AISC Steel Manual, G7}$$

$$C_v := 1.0$$

$$A_w := 2 \cdot b_f \cdot t_f \quad A_w = 16.851 \text{ in}^2$$

$$V_n := 0.6 \cdot F_y \cdot A_w \cdot C_v \quad V_n = 505.53 \text{ kip}$$

$$V_r := \phi_v \cdot V_n \quad V_r = 505.53 \text{ kip}$$

$$\frac{V_u}{V_r} < 1 = 1 \quad \text{(Should see 1)}$$

Integral Abutment Design Worksheet (See IAB Manual 2010 Appendix B Design Example)

Project Name: Mount Holly BF 1442(56)

FB-Pier Corresponding File Name: Abutment 2 STR I S.in

Analysis By: A. Arles 6/01/2026 Piles Oriented for Weak Axis Bending

Pile Size Inputs:

Pile := HP 12x84 ▾

$$b_f = 12.3 \text{ in} \quad t_f = 0.685 \text{ in} \quad t_w = 0.685 \text{ in} \quad d = 12.3 \text{ in} \quad k_o = 1.375 \text{ in} \quad A_s = 24.6 \text{ in}^2$$

$$S_y = 34.6 \text{ in}^3 \quad Z_y = 53.2 \text{ in}^3 \quad r_y = 2.94 \text{ in}$$

$$S_x = 106 \text{ in}^3 \quad Z_x = 120 \text{ in}^3 \quad r_x = 5.14 \text{ in}$$

$$t := d - 2 \cdot k_o = 9.55 \text{ in} \quad \lambda_f := \frac{b_f}{2 \cdot t_f} = 8.978 \quad \lambda_w := \frac{t}{t_w} = 13.942$$

$$F_y := 50 \cdot \text{ksi}$$

$$E := 29000 \cdot \text{ksi}$$

FB-Pier Inputs:

$$P_u := 214.4 \cdot \text{kip}$$

$$Mu_{top} := 101 \cdot \text{kip} \cdot \text{ft}$$

$$Mu_{2nd} := 47.7 \cdot \text{kip} \cdot \text{ft}$$

$$lb_{top} := 82.2 \text{ in}$$

$$lb_{2nd} := 160.3 \cdot \text{in}$$

$$V_u := 13.6 \cdot \text{kip}$$

Resistance Factors

$$\phi_{cl} := 0.50$$

For lower zone of the pile

$\phi_{cu} := 0.70$ For the upper zone of the pile

$\phi_f := 1.0$ For the upper zone of the pile

Nominal Axial Pile Resistance
(Check Pile Size chosen)

$$R_n := \frac{P_u}{\phi_{cl}} \quad R_n = 428.8 \text{ kip}$$

$$A_{sreq} := \frac{R_n}{(0.8 \cdot F_y)} \quad A_{sreq} = 10.72 \text{ in}^2$$

Check if Area Required is less than chosen pile area:

$$A_{sreq} < A_s = 1 \quad (\text{Should see 1})$$

Elastic Flexural Buckling Resistance

($k_{top} = 1.2$ first, 2.1 if hinge forms)

$$k_{top} := 1.2$$

$$k_{2nd} := 1.0$$

Calculate Column Slenderness Factors

$$\lambda_{top} := \left(\frac{k_{top} \cdot lb_{top}}{ry \cdot \pi} \right)^2 \cdot \left(\frac{F_y}{E} \right) \quad \lambda_{top} = 0.197$$

Check:

$$\lambda_{top} < 2.25 = 1 \quad (\text{Should see 1})$$

$$\lambda_{2nd} := \left(\frac{k_{2nd} \cdot lb_{2nd}}{ry \cdot \pi} \right)^2 \cdot \left(\frac{F_y}{E} \right) \quad \lambda_{2nd} = 0.519$$

$$\lambda_{2nd} < 2.25 = 1 \quad (\text{Should see 1})$$

$$P_{etop} := \pi^2 \cdot \frac{E \cdot A_s}{\left(\frac{k_{top} \cdot lb_{top}}{ry} \right)^2} = (6.255 \cdot 10^3) \text{ kip}$$

AASHTO 6.9.4.1.2-1

$$P_{e2nd} := \pi^2 \cdot \frac{E \cdot A_s}{\left(\frac{k_{2nd} \cdot lb_{2nd}}{ry} \right)^2} = (2.368 \cdot 10^3) \text{ kip}$$

$$Q := 1.0$$

use 1 for nonslender elements

AASHTO C6.9.4.1.1

$$P_n := Q \cdot F_u \cdot A_s = (1.23 \cdot 10^3) \text{ kip}$$

Nominal Structural Pile Resistance

AASHTO 6.9.4.2.1

AASHTO 6.9.4.2.1-1

$$Pn_{top} := \left\| \begin{array}{l} \text{if } \frac{P_{etop}}{P_o} \geq 0.44 \\ \left\| 0.658 \left(\frac{P_o}{P_{etop}} \right) \cdot P_o \right\| \\ \text{if } \frac{P_{etop}}{P_o} < 0.44 \\ \left\| 0.877 \cdot P_{etop} \right\| \end{array} \right\|$$

AASHTO 6.9.4.2.1-2

$$Pn_{top} = 1132.82 \text{ kip}$$

$$Pn_{2nd} := \left\| \begin{array}{l} \text{if } \frac{P_{e2nd}}{P_o} \geq 0.44 \\ \left\| 0.658 \left(\frac{P_o}{P_{e2nd}} \right) \cdot P_o \right\| \\ \text{if } \frac{P_{e2nd}}{P_o} < 0.44 \\ \left\| 0.877 \cdot P_{e2nd} \right\| \end{array} \right\|$$

$$Pn_{2nd} = 989.7 \text{ kip}$$

$$Pn_{bot} := 0.66^{0.0} \cdot F_y \cdot A_s$$

$$Pn_{bot} = 1230 \text{ kip}$$

Factored Structural Pile Resistance

$$Pr_{top} := \phi_{cu} \cdot Pn_{top}$$

$$Pr_{top} = 792.97 \text{ kip}$$

$$Pr_{2nd} := \phi_{cu} \cdot Pn_{2nd}$$

$$Pr_{2nd} = 692.79 \text{ kip}$$

$$Pr_{bot} := \phi_{cl} \cdot Pn_{bot}$$

$$Pr_{bot} = 615 \text{ kip}$$

Check to make sure pile size is not too big:

$$\frac{P_u}{Pr_{top}} > 0.2 = 1 \quad \text{(Should see 1)}$$

$$\frac{P_u}{Pr_{2nd}} > 0.2 = 1 \quad \text{(Should see 1)}$$

$$\frac{P_u}{Pr_{bot}} < 1 = 1 \quad \text{(Should see 1)}$$

Calculate Nominal Flexural Resistance

$$\lambda_f = 8.978$$

Flange Slenderness

$$\lambda_{pf} := 0.38 \cdot \sqrt{\frac{E}{F_y}} \quad \lambda_{pf} = 9.152$$

Compact Flange

AASHTO 6.12.2.2.1-4

$$\lambda_{rf} := 0.56 \cdot \sqrt{\frac{E}{0.7 \cdot F_y}} \quad \lambda_{rf} = 16.12$$

Noncompact Flange

AASHTO 6.12.2.2.1-5

$$M_n := \left\| \begin{array}{l} \text{if } \lambda_{pf} < \lambda_f < \lambda_{rf} \\ \left\| \left(1 - \left(1 - \frac{S_y}{Z_y} \right) \cdot \left(\frac{\lambda_f - \lambda_{pf}}{0.45 \cdot \sqrt{\frac{E}{F_y}}} \right) \right) \cdot F_y \cdot Z_y \right\| \\ \text{if } \lambda_f \leq \lambda_{pf} \\ \left\| Z_y \cdot F_y \right\| \end{array} \right\|$$

AASHTO 6.12.2.2.1-2

$$M_n = 2660 \text{ in}\cdot\text{kip}$$

Calculate Flexural Resistance

$$\phi_f := 1$$

$$M_r := \phi_f \cdot M_n$$

$$M_r = 2660 \text{ in}\cdot\text{kip}$$

Check Moment that will cause plastic hinge

$$M_p := \frac{9}{8} \cdot \left(1 - \frac{P_u}{Pr_{top}} \right) \cdot M_r$$

$$M_p = 2183.4 \text{ in}\cdot\text{kip}$$

$$M_p = 182 \text{ ft}\cdot\text{kip}$$

$$Mu_{top} > M_p = 0$$

(Should see 1)

Since the Applied moment exceeds the moment that would create a plastic hinge, it can be assumed that the pile head has plastically deformed.

Now, make sure the 2nd segment DOES NOT develop a plastic hinge. A plastic hinge in this segment will cause the pile to fail!

$$I := \left\| \begin{array}{l} \text{if } \frac{P_u}{Pr_{2nd}} \geq 0.2 \\ \left\| \frac{P_u}{Pr_{2nd}} + \frac{8}{9} \cdot \left(\frac{Mu_{2nd}}{M_r} \right) \right\| \\ \text{if } \frac{P_u}{Pr_{2nd}} < 0.2 \\ \left\| \frac{P_u}{2.0 \cdot Pr_{2nd}} + \left(\frac{Mu_{2nd}}{M_r} \right) \right\| \end{array} \right\|$$

AASHTO 6.9.2.2-2

AASHTO 6.9.2.2-1

$$P_u = 214.4 \text{ kip}$$

$$I = 0.501 \quad I < 1 = 1$$

(Should see 1)

Now Check for Shear

$$\phi_v := 1.0$$

$$k_v := 1.2 \quad \text{AISC Steel Manual, G7}$$

$$C_v := 1.0$$

$$A_w := 2 \cdot b_f \cdot t_f \quad A_w = 16.851 \text{ in}^2$$

$$V_n := 0.6 \cdot F_y \cdot A_w \cdot C_v \quad V_n = 505.53 \text{ kip}$$

$$V_r := \phi_v \cdot V_n \quad V_r = 505.53 \text{ kip}$$

$$\frac{V_u}{V_r} < 1 = 1 \quad \text{(Should see 1)}$$

Appendix D

MathCAD Lateral Earth Pressure

Lateral Earth Pressure Calculations 2026

Project: Eden BF 029-2(15)

Assumptions:

1. 704.08 – Granular Backfill for Structures Engineering properties
2. The backfill surface behind each abutment is assumed to be approximately horizontal.
3. The backfill is assumed to be free draining (i.e., no water pressure is allowed to build up behind the abutment wall).

Inputs: Enter values in green boxes

	Abutment 1	Abutment 2
Abutment Height (ft): (bottom of the pile cap to finished grade)	$H_{A1} := 13 \text{ ft}$	$H_{A2} := 13 \text{ ft}$
Abutment Bottom of Footing Elevation (ft):	$BoF_{A1} := 1285.47 \text{ ft}$	$BoF_{A2} := 1278.98 \text{ ft}$
Expansion (ft):	$\delta_T := 0.083 \text{ ft}$	

Granular Backfill Engineering Properties

Unit weight (pcf): $\gamma_{soil} := 140 \text{ pcf}$

Friction Angle (degrees): $\phi := 34^\circ$

1. Calculate expected wall rotation for the integral abutment to select earth pressure case

Per AASHTO LRFD Table C3.11.1-1 (Ref. 2), in order to develop full passive earth pressure, wall rotation (the ratio of lateral abutment movement to abutment height) must exceed 0.02 for a medium dense cohesionless backfill such as the proposed Granular Backfill for Structures abutment backfill.

$$\text{Maximum wall rotation, Abutment 1} = R_{A1} := \frac{\delta_T}{H_{A1}} \quad R_{A1} = 0.006$$

$$\text{Maximum wall rotation, Abutment 2} = R_{A2} := \frac{\delta_T}{H_{A2}} \quad R_{A2} = 0.006$$

Since the anticipated wall rotation does not exceed 0.02, full passive earth pressure is not expected to develop. The magnitude of partial earth pressure will be calculated using the procedure in the MassDOT LRFD Bridge Manual Section 3.10.8

2. Calculate the movement-dependent passive earth pressure coefficient for conditions between full passive and at-rest earth pressure.

$$K_{P1} := 0.43 + 5.7 \left(1 - (e)^{(-190) \cdot (R_{A1})} \right) \quad K_{P1} = 4.44$$

$$K_{P2} := 0.43 + 5.7 \left(1 - (e)^{(-190) \cdot (R_{A2})} \right) \quad K_{P2} = 4.44$$

3. Determine the unfactored passive pressure P_p acting on the abutments.

$$P_{P1} := \frac{1}{2} \cdot \gamma_{soil} \cdot H_{A1}^2 \cdot K_{P1} \quad P_{P1} = 52.47 \frac{\text{kip}}{\text{ft}}$$

$$P_{P2} := \frac{1}{2} \cdot \gamma_{soil} \cdot H_{A2}^2 \cdot K_{P2} \quad P_{P2} = 52.47 \frac{kip}{ft}$$

The resultant lateral earth load acts at a height of H/3 above the base of the wall

$$E_{A1} := \frac{H_{A1}}{3} + BoF_{A1} \quad E_{A1} = (1.3 \cdot 10^3) \text{ ft}$$

$$E_{A2} := \frac{H_{A2}}{3} + BoF_{A2} \quad E_{A2} = (1.3 \cdot 10^3) \text{ ft}$$

4. Using the Rankine theory calculate full passive earth pressure coefficient, active earth pressure coefficient and at-rest lateral earth pressure coefficient for granular backfill for structures

Full Passive Earth Pressure Coefficient:

$$K_p := \tan^2 \left(45^\circ + \frac{\phi}{2} \right) \quad K_p = 3.54$$

Active Earth Pressure Coefficient:

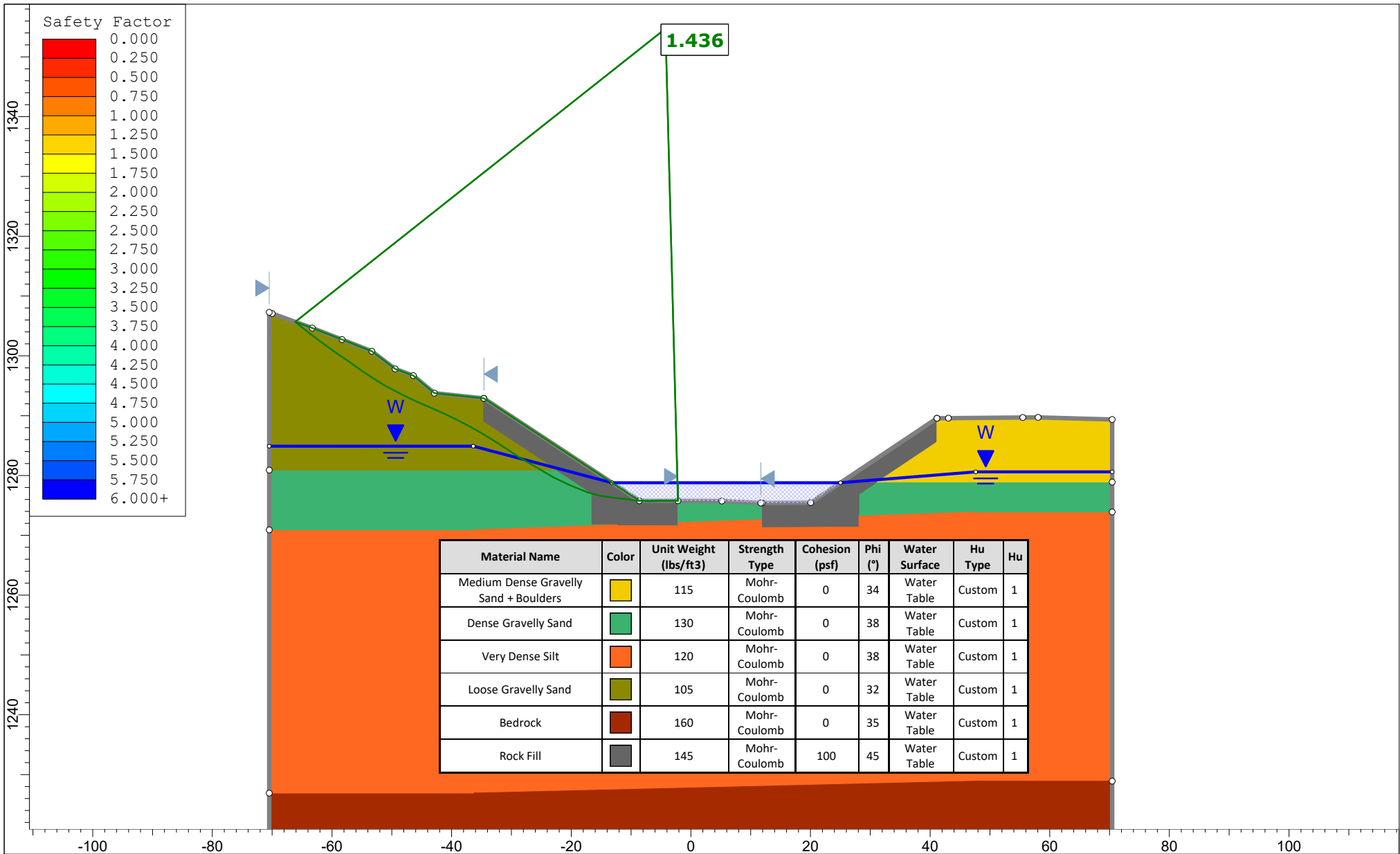
$$K_a := \tan^2 \left(45^\circ - \frac{\phi}{2} \right) \quad K_a = 0.28$$

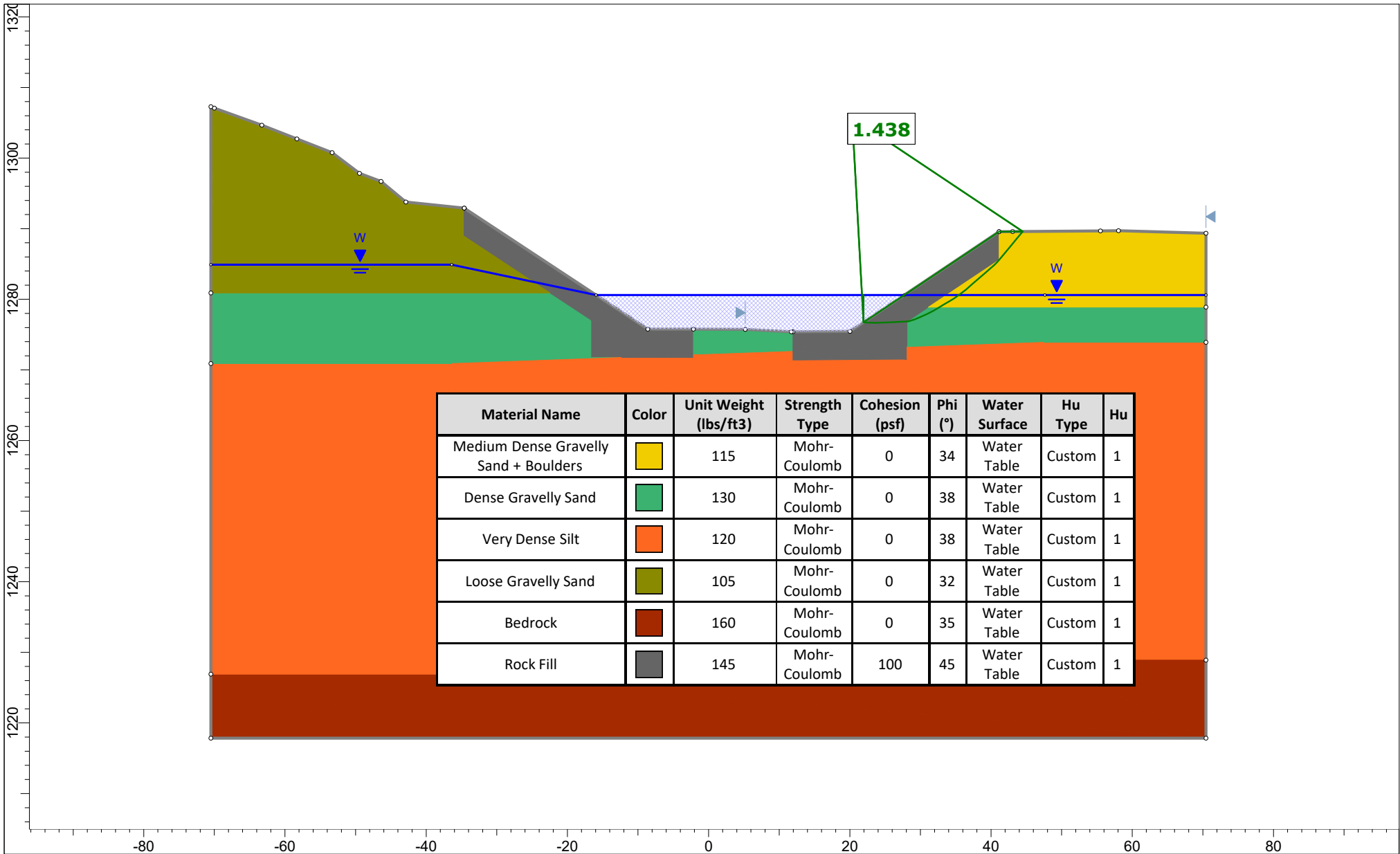
At Rest Lateral Earth Pressure Coefficient:

$$K_0 := 1 - \sin(\phi) \quad K_0 = 0.44$$

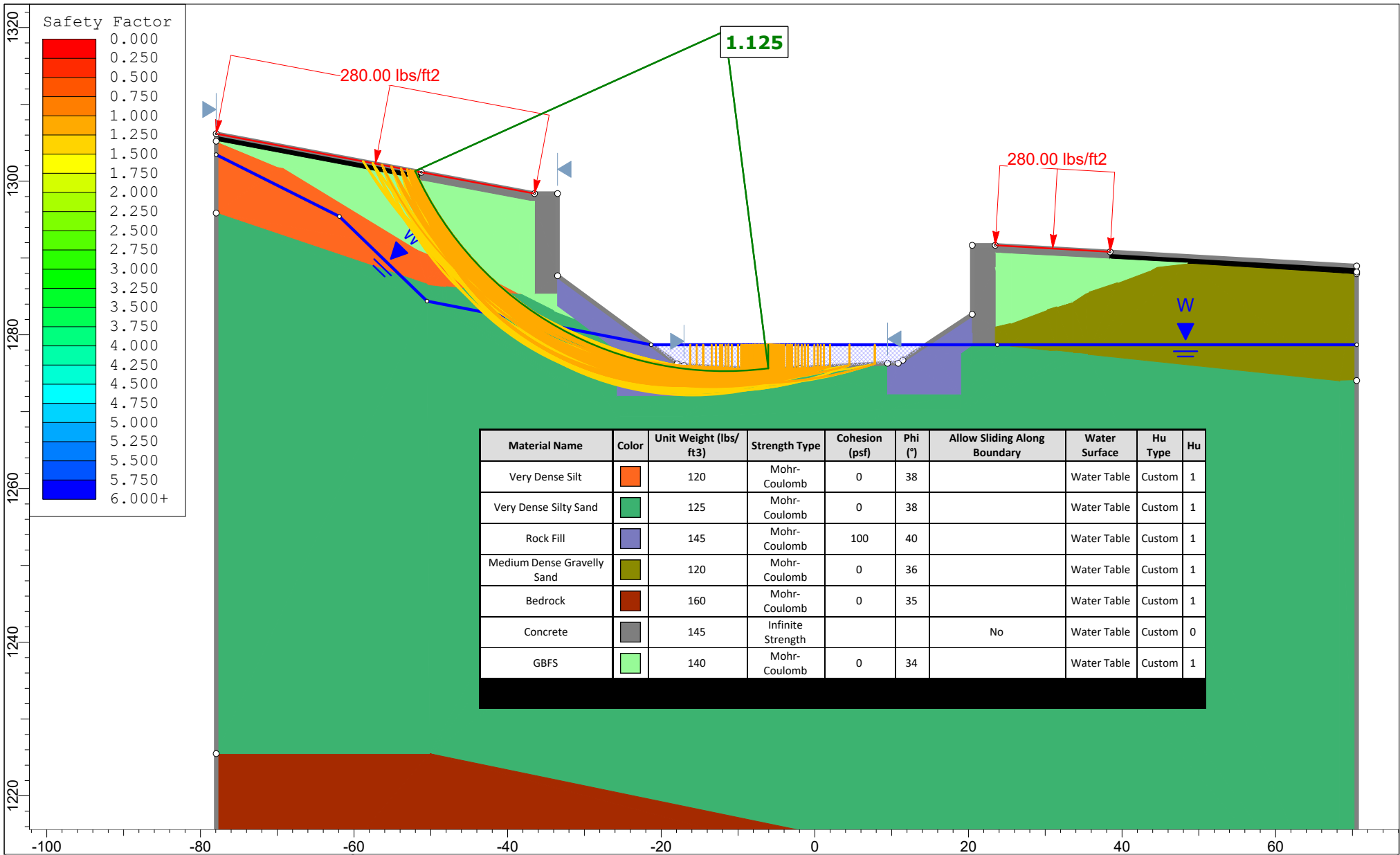
Appendix E

Slide2 Modeling

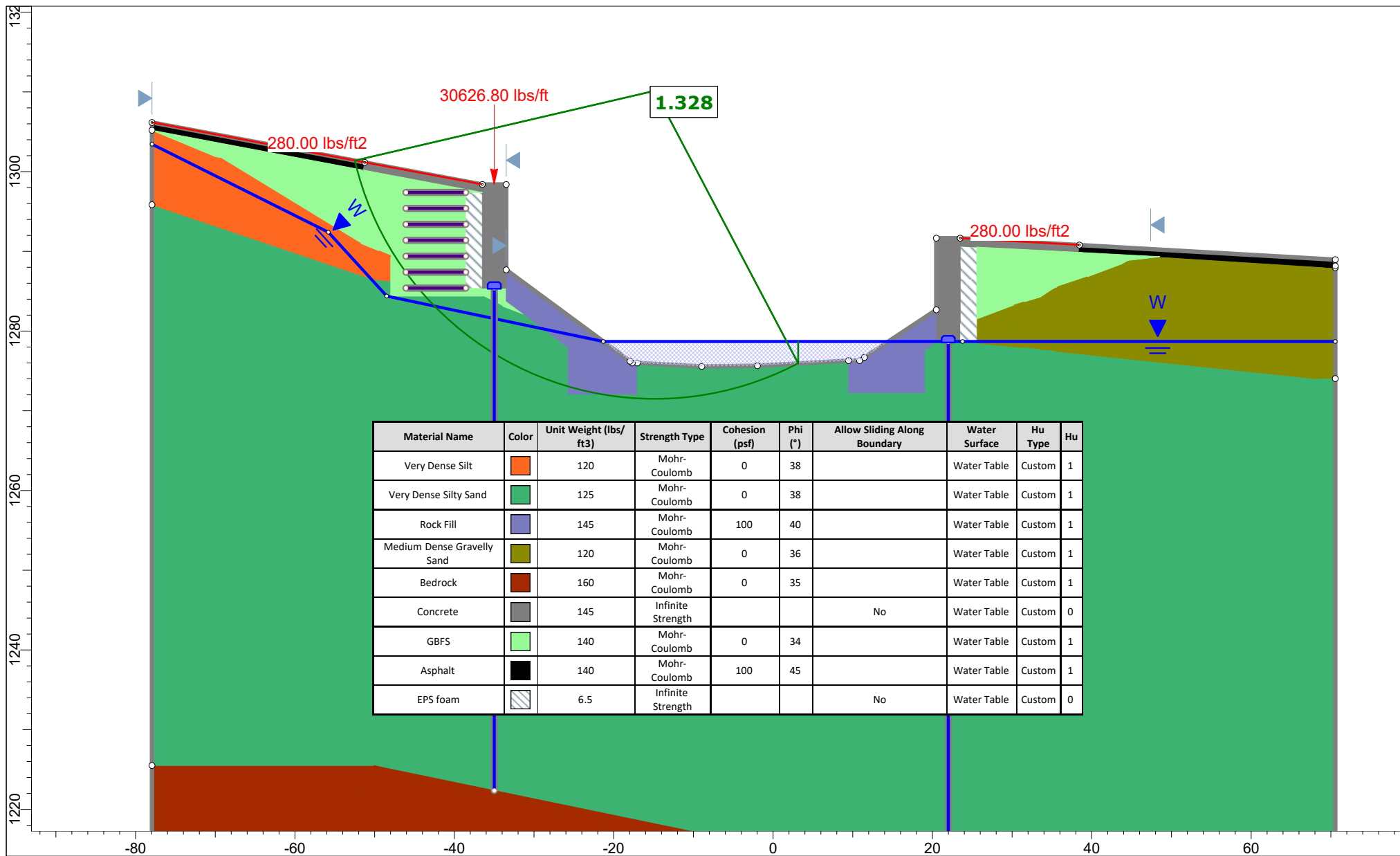


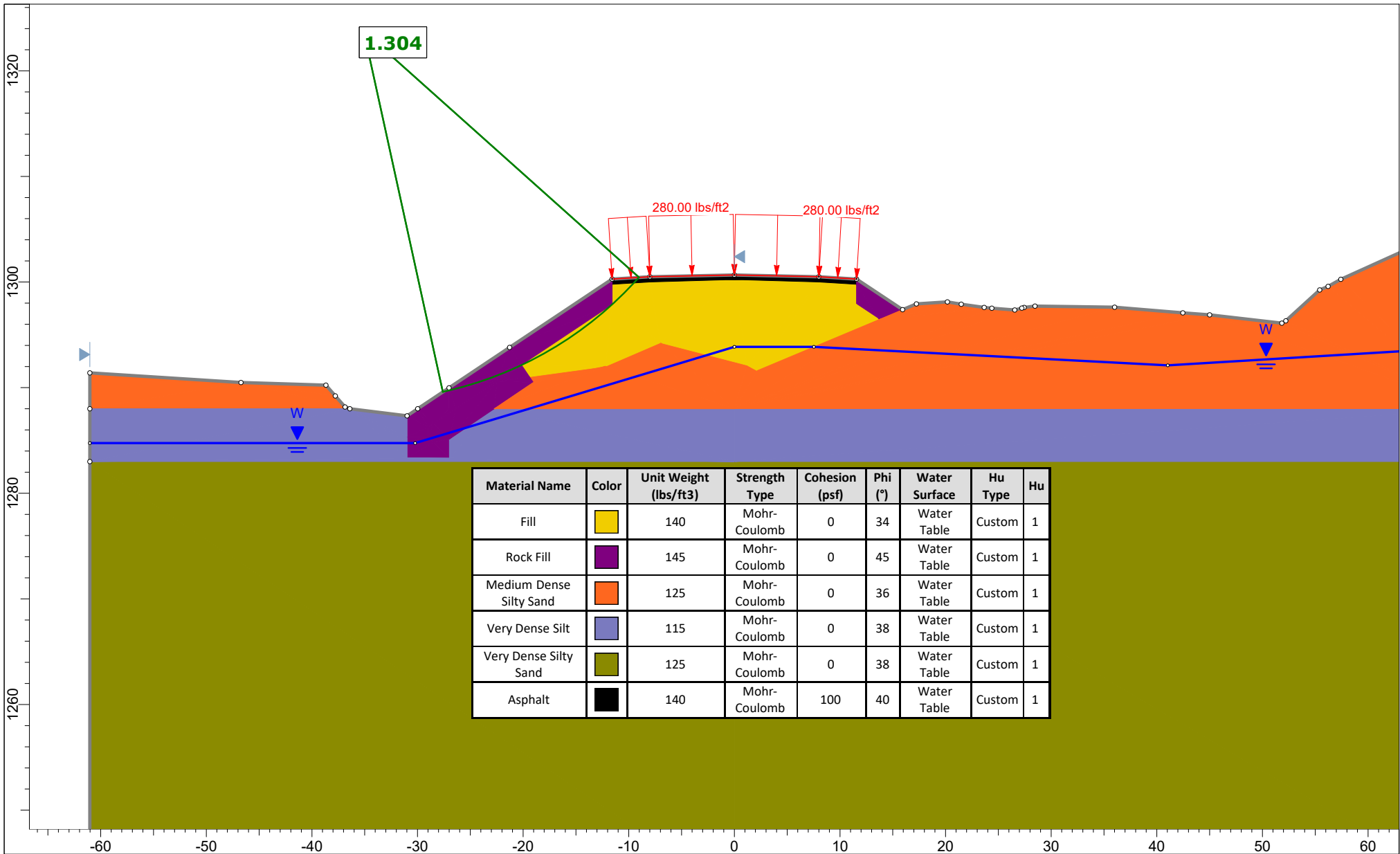


Project				Slide2 - An Interactive Slope Stability Program			
Group		Right Side		Scenario		Master Scenario	
Drawn By				Company			
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Project			Slide2 - An Interactive Slope Stability Program		
Group		Geogrid	Scenario		Abut 1 No Geogrid
Drawn By			Company		
Date		8/6/2026, 3:54:23 PM	File Name		11+50.slmd





Project

Slide2 - An Interactive Slope Stability Program

Group

Extended Slope

Scenario

Half Type IV Slope

Drawn By

Company

Date

7/22/2026, 2:53:09 PM

File Name

102+00.slmd

