AGENCY OF TRANSPORTATION **OFFICE MEMORANDUM**

To: Cory Burrall, P.E., Structures Project Manager

AJA CEE

From: August Arles, Geotechnical Engineer via Callie Ewald, P.E., Geotechnical Engineering Manager

Date: May 6th, 2026

Subject: Reading ER P23-1(404) – Geotechnical Recommendations

1.0 INTRODUCTION

As requested, we have completed our geotechnical assessment of the proposed foundations for the new bridge construction as part of the Reading ER P23-1(404) project. This project consists of the construction of a new bridge founded on pile supported abutments carrying VT Route 106 over the North Branch Black River in the town of Reading, VT. The new bridge is located approximately 1.2 miles south of the intersection of VT Route 106 and Mill Street (TH 43). Contained herein are the results of our geotechnical analysis and design parameter recommendations as determined using the 2024 AASHTO LRFD Bridge Design Specifications.

2.0 FIELD INVESTIGATION

A field investigation was conducted by VTrans between March 21st, 2025, and April 14th, 2025. The findings of this investigation are detailed in the Geotechnical Data Report dated May 12th, 2025. Please refer to this report for a detailed description of the field sampling and testing, laboratory analysis of soils samples, boring logs, and boring layout plan. Additionally, boring logs are attached to the end of this report in Appendix 1.

3.0 SOIL PROFILE

The following soil strata have been identified based on our review of the boring logs and laboratory testing. It should be noted that groundwater elevations are subject to change given the fact that boreholes were generally left open for a short period of time. Because groundwater elevations can fluctuate seasonally and are affected by temperature and precipitation, groundwater may be encountered during construction when not previously noted on the boring logs.

3.1 Abutment 1 (B-101, B-103)

The ground surface elevations at B-101 and B-103 were approximately 728.2 ft and 728.1 ft, respectively. Bedrock was encountered at a depth of 25.1 feet (ft) below ground surface (bgs) in B-101 and 30 ft bgs in B-103, corresponding to elevations of 703.1 ft and 698.1 ft, respectively. The soil parameters used in the analysis for Abutment 1 are displayed below in Table 3.1.

Table 3.1 FB-MultiPier Analysis Soil Parameters – Abutment 1 (B-101, B-103)

Elevation (ft)	Description	Friction Angle (deg.)	Unit Weight (pcf)	Subgrade Modulus (pci)	Shear Modulus (ksi)	Nominal Skin Friction (psf)	Poisson's Ratio
718 – 716.1	Med. Dense SAND and GRAVEL	36	125	225	2.33	942.31	0.40
716.1 – 703.1/698.1	Dense, SAND, some Gravel, little Silt	36	125	125	2.11	1618.19	0.35
<703.1/698.1	Bedrock (GNEISS)	-	165	-	3631.15	-	0.22

3.2 Abutment 2 (B-102a, B-104):
The ground surface elevation at both B-102a and B-104 was approximately 728.5 ft. Bedrock was encountered at a depth of 22.0 ft bgs in B-102a and 25 ft in B-104, corresponding to an elevation of 706.5 ft and 703.5 ft, in B-102a and B-104, respectively. The soil parameters used in the analysis for Abutment 2 are displayed below in Table 3.2.

Table 3.2 FB-MultiPier Analysis Soil Parameters – Abutment 2 (B-102a, B-104)

Elevation (ft)	Description	Friction Angle (deg.)	Unit Weight (pcf)	Subgrade Modulus (pci)	Shear Modulus (ksi)	Ultimate Skin Friction (psf)	Poisson's Ratio
718 – 706.5/703.5	Med. Dense SAND and GRAVEL	36	125	125	3.36	1323.22	0.35
<706.5/703.5	Bedrock (GNEISS)	-	165	-	3631.15	-	0.22

3.3 Rock Parameters
A summary of the rock core findings are listed in Appendix 2 as well as available in the attached boring logs. Information from the cores indicated hard Gneiss present at the boring locations. Cores were recovered and brought back to the VTrans Construction and Materials Bureau Laboratory for further evaluation and the results of this evaluation can be found in the Geotechnical Report dated May 13th, 2025. The bedrock had an average rock mass rating (RMR) of 50, indicating fair rock. Rock parameters used for both abutments are summarized in Table 3.3.1 and were correlated from the rock testing data and AASHTO 2024 Standard Specifications.

Table 3.3.1 FB-MultiPier Analysis Rock Parameters

Parameter	Value
Unconfined Compressive Strength (ksf)	1399.26
Poisson's Ratio	0.22
Shear Modulus (ksi)	3631.15

4.0 ANALYSIS

Developed by the Florida Bridge Software Institute, FB-MultiPier, version 6.0.0, is a multi-aspect software that allows the user to analyze a bridge pier system in three dimensions. Its analysis factors in the subsurface strata, pile group including cap, and the structural capabilities of the pier system. For this pile supported abutment analysis, only the piles and cap were modeled in FB-MultiPier.

4.1 Geometry

Final geometry for the substructures were provided by Cory Burrall, VTrans Structures Project Manager, in an email dated January 20th, 2026. Both abutments will consist of a pile cap that is 57 ft long and 9.5 ft wide, Abutment 1 is a maximum of 10.41 ft tall, and Abutment 2 is a maximum of 10.77 ft tall. The bottom of both abutment caps will be set at an elevation of 716.0 ft. Abutment 1 will be supported by two rows of eight HP 14x89 steel piles spaced 90 inches (in) center to center. Abutment 2 will be supported by two rows of eleven HP 14x89 steel piles spaced 63 in center to center. In order to reduce the group effect on the front row of piles, the pile rows were spaced at 70 in center to center, based on guidance from AASHTO Section 10.7.2.4. For both abutments, the front row of piles are battered at a 1H:6V angle and the back row of piles are plumb. All piles are oriented for strong axis bending. Figure 4.1 and Figure 4.2 below detail the pile layout at both Abutment 1 and Abutment 2.

Figure 4.1 Pile Layout For Abutment 1

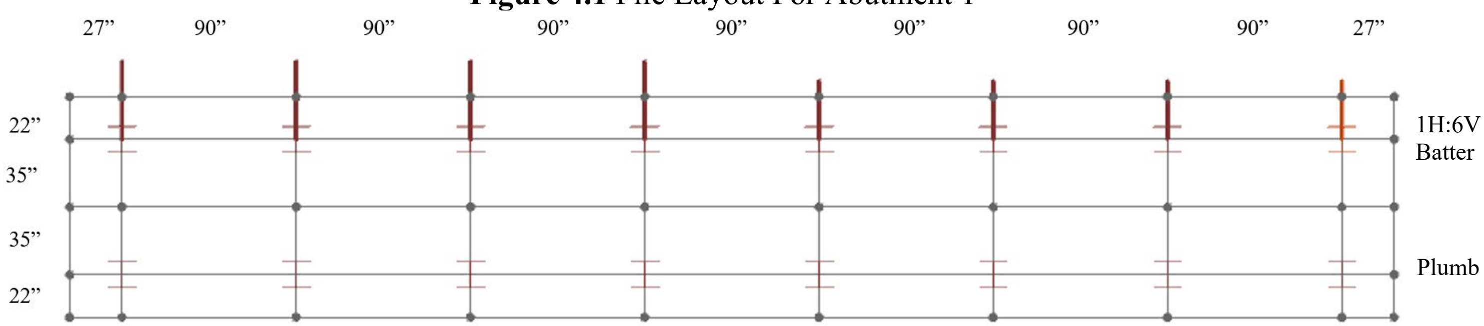
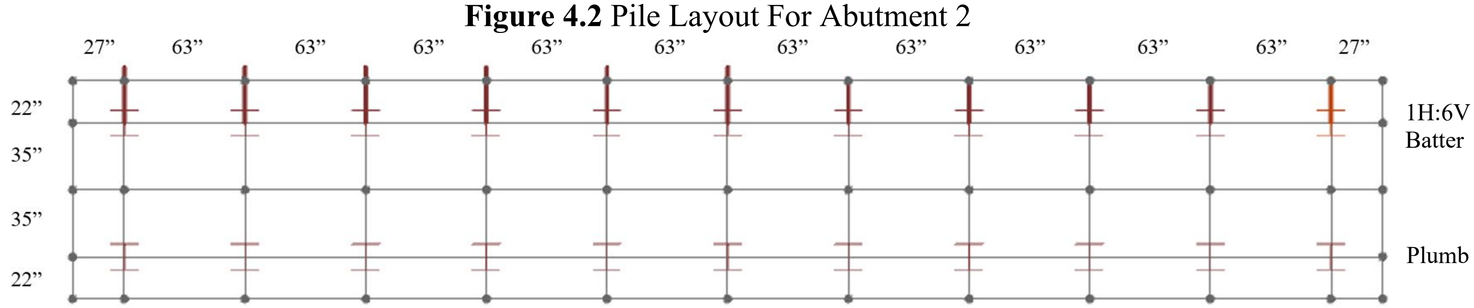


Figure 4.2 Pile Layout For Abutment 2



4.2 Loads

Final unfactored loads were provided by Cory Burrall, via an email dated January 20th, 2026. Per AASHTO Section 3.4.1, the unfactored loads were assessed at all Strength, Service, and Extreme Event limit states with the appropriate load factors as specified in AASHTO Table 3.4.1-1.

4.3 Modeling

The models were analyzed for all Strength, Service, and Extreme Event load combinations as described in Section 4.2. A detailed layout of each pile cap and pile group geometry can be found attached to this report in Appendix 3.4. The unfactored input loads can be found in Table 4.3.1 and Table 4.3.2, as well as in Appendix 3. It should be noted that scour conditions govern for Strength Limit and the Service Limit. Extreme Limit State was evaluated for check scour condition only.

Two rows of piles were modeled, the back row of piles were driven plumb, the front row of piles were modeled at a 1H:6V batter. For Abutment 2, due to shallow bedrock encountered during drilling operations, at an approximate depth of 9.5 ft below the proposed bottom of footing elevation, the plumb row of piles were modeled as predrilled 4.5 ft into rock. The piles were modeled to best replicate conditions of pre-drilled holes in rock for the pile installment. The piles were modeled as placed in a 20.2 in diameter hole predrilled through the soil and rock and then backfilled with sand. All piles in Abutment 1 were modeled as being driven to and seated on rock.

All piles were assumed to be driven for strong axis bending. The plumb piles were modeled as 14.9 ft and 16.0 ft long piles for Abutment 1 and Abutment 2, respectively, due to the angle of the batter, the front row of the piles was modeled to be 0.5 ft longer for both abutments to ensure contact with bedrock.

Table 4.3.1 Unfactored Input Loads at Abutment 1

Load Type	Load	Loads (kips)*			Moment on Pile Group (kip-ft)	
		Y	X	Z	Mx	My
Superstructure Dead	DC	0	0	870.49	-212.019	0
Dead Load Wearing Surfaces	DW	0	0	53.53	-6.61	0
Vehicular Live	LL	0	0	189.38	94.69	0
Braking Force	BR	16	14.5	0	203.36	184.295
Live Surcharge	LS	93.2	0	0	605.8	0
Horizontal Earth Pressure	EH	317.69	0	0	1374.96	0
Vertical Earth Pressure	EV	0	0	328	-274.208	0
Wind on Structure	WS	5.5	5.9	0	69.905	74.989
Wind on Live	WL	3.58	3.75	0	45.5018	47.6625

**X indicates transverse, Y indicates longitudinal, and Z indicates vertical loads*

Table 4.3.2 Unfactored Input Loads at Abutment 2

Load Type	Load	Loads and Shear Forces (kips)*			Moment on Pile Group (kip-ft)	
		Y	X	Z	Mx	My
Superstructure Dead	DC	0	0	908.65	-89.6808	0
Dead Load Wearing Surfaces	DW	0	0	53.53	-6.61	0
Vehicular Live	LL	0	0	189.38	94.69	0
Braking Force	BR	16	14.5	0	203.36	184.295
Live Surcharge	LS	96	0	0	662.4	0
Horizontal Earth Pressure	EH	404.51	0	0	1868.84	0
Vertical Earth Pressure	EV	0	0	423	-330.786	0
Wind on Structure	WS	5.5	5.9	0	69.905	74.989
Wind on Live	WL	3.58	3.75	0	45.5018	47.6625

**X indicates transverse, Y indicates longitudinal, and Z indicates vertical loads*

4.4 Scour Analysis

Scour elevations at the bridge location were provided Fianna Barrows in an email dated January 23rd, 2026. Design Scour elevation was determined to be 716.0 ft and Check Scour elevation was determined to be 715.8 ft. All loads were applied to each abutment under scour conditions.

5.0 RESULTS

5.1 Pile Stresses

Two rows of eight and two rows of eleven HP14x89 steel piles, for Abutment 1 and Abutment 2, respectively, were found to satisfy design requirements at each abutment for the Strength Limit State and to limit deflection to less than 1.0 inch in the Service Limit State under scour conditions. FB-MultiPier analyses were performed by applying each loading combination, which consisted of longitudinal, transverse, and vertical loads along with their respective moments, at the top of each pile. For both abutments, Strength I was determined to be the governing cases for maximum loads and moments in Strength Limit State when evaluating for structural capacity of the piles for non-scour and scour conditions. Additionally, Service II was determined to be the governing case resulting in maximum deflections under scour conditions for both abutments, respectively. The piles were designed for 0.0 ft and 0.2 feet of freestanding length in the design scour and check scour conditions, respectively, for both abutments.

Due to the shallow depth of bedrock, some pre-drilling into the bedrock is required to meet design requirements. Efforts were made to minimize the amount of pre-drilling, and final design resulted in only the back row piles in Abutment 2 requiring pre-drilling and embedment into a rock socket.

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. Maximum resultant forces and deflections from the FB MultiPier analyses can be found below in Table 5.1.1 and Table 5.1.2. Resultant forces and deflection for each load combination can be found attached to the end of this report in Appendix 3.7.

Table 5.1.1 Resultant Forces at Abutment 1

Row	Load Combination	Max. Shear X-dir. (kips)	Max. Shear Y-dir. (kips)	Max. Moment About X-axis (k-ft)	Max. Moment About Y-axis (k-ft)	Max. Axial Compression (kips)	Max. Lateral Deflection Y-dir. (in)
Front Row	Strength I	2.16	26.34	144.23	10.49	249.77	-
	Service II	1.50	18.52	85.46	6.75	180.42	0.17
Back Row	Strength I	1.81	22.04	124.41	8.73	28.29	-
	Service II	1.37	14.38	74.13	6.19	32.24	0.02

Table 5.1.2 Resultant Forces at Abutment 2

Row	Load Combination	Max. Shear X-dir. (kips)	Max. Shear Y-dir. (kips)	Max. Moment About X-axis (k-ft)	Max. Moment About Y-axis (k-ft)	Max. Axial Compression (kips)	Max. Lateral Deflection Y-dir. (in)
Front Row	Strength I	2.06	23.72	110.56	10.57	213.32	-
	Service II	1.54	17.23	71.81	7.02	151.78	0.20
Back Row	Strength I	1.23	27.56	147.81	6.14	21.38	-
	Service II	0.95	16.24	83.72	4.29	17.95	0.20

5.2 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 14x89 piles can be driven through the soils encountered at Abutment 1 by pile-driving equipment commonly used by contractors in the region. The borings taken in the vicinity of Abutment 2 noted cobbles and boulders directly above bedrock, requiring pre-drilling for both the piles seated on bedrock and the plumb piles requiring rock sockets.

Section 10.7.8 of the AASHTO LRFD Bridge Design Specifications stipulates that the maximum tension and compression stresses allowed in the piles shall not exceed $\sigma=0.9^* \varphi_{da}{}^*f_y$. 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^*f_y or 45 ksi for grade 50 (50 ksi) piles.

5.3 Nominal Axial Pile Resistance

Due to the depth of bedrock encountered in the borings, the piles not predrilled into rock are assumed to be seated on bedrock and reach nominal axial resistance through end bearing 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 $R_R = \varphi_{dyn} * R_n$. The resistance factor, φ_{dyn}, which should be applied to those piles bearing 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 in accordance with Table 10.5.5.2.3-1 of the AASHTO LRFD Bridge Design Specification. The remaining piles should be calibrated by wave equation analysis. Given the loads provided in Tables 5.1.1 and Table 5.1.2, the nominal axial pile resistance, or resistance the piles should be driven to, is 383 kips for all piles for Abutment 1. Both dynamic tests will occur on Abutment 1 as Abutment 2 will require all piles to be pre-drilled.

For Abutment 2, due to the need for pre-drilling for all of the piles, no dynamic test will be performed, and the resistance factor, φ_{dyn}, is 0.5. The plumb piles will be driven to refusal and seated into the rock socket, and the battered piles will be driven to refusal and seated on rock. The nominal axial resistance for all piles in Abutment 2 is 426 kips.

5.5 Downdrag Analysis

Negative skin friction, or downdrag, is considered when the relative settlement between the pile and soil equals or exceeds 0.4 inches according to AASHTO 3.11.8. The proposed roadway does not vary significantly in grade with the existing roadway and as a result will not require large amounts of fill. Therefore, neither settlement nor downdrag due to an additional roadway surcharge is expected.

5.6 Settlement Analysis

Settlement of the abutment is anticipated to be negligible due to the piles being driven to bedrock. Any settlement that does occur should be caused by the elasticity in the piles, which should occur as the piles are loaded.

6.0 RECOMMENDATIONS

6.1 H-Pile Foundations

For Abutment 1, we anticipate driving plumb and battered piles to bedrock. Based on the depth to bedrock, required pile embedment, and in-situ soil conditions, we anticipate the piles to be driven to depth without the need for pre-drilled rock sockets at Abutment 1. For Abutment 2, both in-situ soil conditions and design requirements necessitate pre-drilling for piles extending to rock and into pre-drilled rock sockets to achieve the minimum pile embedment requirements.

The following table provides a summary of the requirements for the piles at Abutment 1 and 2 and should be included in the plans.

Table 6.1 Summary of requirements of H-piles at each abutment

Requirement	Abutment 1	Abutment 2
Pile Size	HP 14x89	
Number of Piles per Abutment	16	22
Pile Spacing (CTC)	90 inches laterally 70 inches longitudinally	63 inches laterally 70 inches longitudinally
Pile Orientation	Strong Axis	
Pile Batter	Plumb, Back Row	Plumb, Back Row
	1H:6V, Front Row	1H:6V, Front Row
Minimum Pile Embedment*	13 ft	14.5 ft
Estimated Pile Length*	13 ft, Back Row	15 ft, Back Row
	13.5 ft, Front Row	15.5 ft, Front Row
Minimum Rock Socket	N/A	1.5 ft, Back Row N/A, Front Row
Method of Installation	Driven Pile	Pre-drilled and Driven
Nominal Axial Pile Resistance	383 kips	426 kips, Predrilled

*Length of pile below bottom of pile cap.

For the back row of piles in Abutment 2, the piles were modeled as being pre-drilled into the rock 4.5 ft based on the highest bedrock encountered in the field investigation. If bedrock is encountered higher than anticipated, additional pre-drilling would be required to reach minimum pile embedment depth. For the piles to have enough lateral resistance to the applied loads, the back row of piles should be drilled into rock a minimum of 1.5 ft. Therefore, a note should be included on the plans to require the back row of piles at Abutment 2 to meet a minimum pile embedment of 14.5 feet below the bottom of pile and a minimum depth of rock socket of 1.5 feet.

Item 546.1 and 546.2, Pre-Excavation of Abutment Piles in Earth and Rock, should be included in the contract to facilitate the pre-drilling. For Abutment 2, we recommend including a total pre-excavation quantity of 260 ft in earth and 40 ft in rock for estimating purposes.

6.2 Construction Considerations:

6.2.1 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."

6.2.2 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.

6.2.3 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.

6.3 Design Parameters

Engineering properties of common construction materials are shown in Table 6.3.1. 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 6.3.1: 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, °(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:	0.31	0.28
Passive Earth Pressure Coef., K_p:	3.26	3.54
At-Rest Earth Pressure Coefficient, K_o:	0.47	0.44

7.0 CONCLUSION

If any further analysis is needed or if you would like to discuss this report, please reach out to our section via email. Final FB-MultiPier input files used in the analyses are located in the folder M:\Projects\23b770\MaterialsResearch on the M/drive.

Reviewed by: Eric Denardo, P.E., Geotechnical Engineer END

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

Attachments: Appendix 1 – Boring Logs (11 Pages)
Appendix 2 – Rock Core Tables (2 Pages)
Appendix 3 – FB MultiPier Analysis (56 Pages)
3.1 – Supplied Load Tables and Cap Geometry
3.2 – Calculated Load Tables
3.3 – FB MultiPier Soil and Rock Parameters
3.4 – FB MultiPier Pile and Cap Geometry
3.5 – FB MultiPier Nodal Loading Schematic
3.6 – FB MultiPier Output Graphs
3.7 – Maximum Forces and Displacement Tables
3.8 – Structural Resistance Calculations

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Appendix 1

Boring Logs

STATE OF VERMONT
AGENCY OF TRANSPORTATION
MATERIALS AND CONSTRUCTION BUREAU
GEOTECHNICAL ENGINEERING SECTION

BORING LOG

Reading
ER P23-1(404)
VT Route 106 Bridge No. 12

Boring No.: B-101

Pin No.: 23b770
Checked By: Arles

Boring Crew: Thurston, Lubas, Degener
Date Started: 03/21/2025 Date Finished: 04/01/2025
Coordinates: N:347125.5, E:1630434.5
Station: 320+12 Offset: 6.1 ft RT
Ground Elevation: 728.2 ft

	Casing	Sampler
Type:	WB	SPT
I.D.:	4 in	1.5 in
Hammer Wt:	N.A.	140 lb
Hammer Fall:	N.A.	30 in.

Hammer/Rod type: Auto / AWJ
Rig: Diedrich D-25 C_E = 1.45

Groundwater Observations

Date	Depth (ft)	Status
04/01/25	11.7'	After Drilling (AD)

Depth (ft)	Graphic Log	AASHTO Classification	Material Description and Notes	Field Data: Recovery (ft) (%)	Field Data: Blow Counts (N-Value)	Field Data: Run Number	Lab Data: Moisture Content (%)	Lab Data: Gravel (%)	Lab Data: Sand (%)	Lab Data: Fines (%)
			0.0 ft - 0.5 ft, Field Note: ASPHALT							
		A-1-a	0.5 ft - 2.5 ft, Lab Classification: Dry to Moist, Brown, GRAVEL, some Sand, trace Silt, Rollercone Cleanout 1.9 ft - 2.5 ft	1.1	9-26-13-28 (39)		7.4	84.3	12.4	3.3
			2.5 ft - 4.5 ft, Field Description: Moist, Brownish Gray, SAND and GRAVEL	0.6	8-9-3-2 (12)					
5		A-1-a	4.5 ft - 5.8 ft, Lab Classification: Moist, Brownish Gray, SAND and GRAVEL, trace Silt, Refusal at 6.3 ft	0.9	4-5-7-50/4" (12)		8.7	56.7	35.3	8
	NR		6.5 ft - 6.8 ft, Field Note: NO RECOVERY, Refusal at 6.8 ft, 50 blows per 6 in	0	50/4" (R)					
10			8.5 ft - 10.5 ft, Field Description: Dry to Moist, Brownish Gray, SAND and GRAVEL, Rollercone Cleanout 9.5 ft - 10.5 ft	0.4	21-13-5-7 (18)					
		A-1-a	10.5 ft - 11.7 ft, Lab Classification: Dry to Moist, Brown, GRAVEL, some Sand, trace Silt, Refusal at 11.7 ft, 10 blows no movement	0.7	10-10-15-10/1" (25)		14.4	60.2	32.9	6.9
		A-2-4	12.5 ft - 14.5 ft, Lab Classification: Dry to Moist, Gray, SAND, some Gravel, little Silt	0.8	4-19-16-14 (35)					
15			14.5 ft - 16.5 ft, Lab Classification: Moist, Brownish Gray, SAND, some Gravel, little Silt	0.9	10-13-17-15 (30)		15.1	27.3	57.4	15.3

Notes:
1. Stratification lines represent approximate boundary between material types. Transition may be gradual.
2. N-values have not been corrected for hammer energy. CE is the hammer energy correction factor.
3. Water level readings have been made at times and under conditions stated. Water levels are likely to vary.:

VTrans Working to Get You There — Vermont Agency of Transportation STATE OF VERMONT AGENCY OF TRANSPORTATION MATERIALS AND CONSTRUCTION BUREAU GEOTECHNICAL ENGINEERING SECTION	**BORING LOG** **Reading** **ER P23-1(404)** **VT Route 106 Bridge No. 12**	Boring No.: B-101 Page No.: 2 of 2 Pin No.: 23b770 Checked By: Arles

Boring Crew: Thurston, Lubas, Degener
Date Started: 03/21/2025 Date Finished: 04/01/2025
Coordinates: N:347125.5, E:1630434.5
Station: 320+12 Offset: 6.1 ft RT
Ground Elevation: 728.2 ft

	Casing	Sampler
Type:	WB	SPT
I.D.:	4 in	1.5 in
Hammer Wt:	N.A.	140 lb
Hammer Fall:	N.A.	30 in.
Hammer/Rod type:	Auto / AWJ	
Rig:	Diedrich D-25	C_E = 1.45

Groundwater Observations

Date	Depth (ft)	Status
04/01/25	11.7'	After Drilling (AD)

Depth (ft)	Graphic Log	AASHTO Classification	Material Description and Notes	Field Data: Recovery (ft) (%)	Field Data: Blow Counts (N-Value)	Field Data: Run Number	Lab Data: Moisture Content (%)	Lab Data: Gravel (%)	Lab Data: Sand (%)	Lab Data: Fines (%)
			24.0 ft - 24.1 ft, Field Note: NO RECOVERY, Refusal @ 24.1 ft, 10 blows no movement							
25			24.1 ft - 25.1 ft, Field Note: fine grained BOULDER							
			25.1 ft - 29.0 ft, light gray Magnetite-biotite-quartz-plagioclase GNEISS, foliated, hard, fresh/unweathered, Joint surfaces are rough, some rust staining along joints. NX	4.8 (96%)		R-1				
30			29.0 ft - 34.0 ft, light gray Magnetite-biotite-quartz-plagioclase GNEISS, foliated, hard, fresh to very slightly weathered, Joint surfaces are rough. NX	2.9 (58%)		R-2				

End of Boring at 34 ft
Hole Collapse at 2.8 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. CE is the hammer energy correction factor.
3. Water level readings have been made at times and under conditions stated. Water levels are likely to vary.:

SOIL BORING

STATE OF VERMONT
AGENCY OF TRANSPORTATION
MATERIALS AND CONSTRUCTION BUREAU
GEOTECHNICAL ENGINEERING SECTION

Reading
ER P23-1(404)
VT Route 106 Bridge No. 12

Boring No.: B-102

Pin No.: 23b770
Checked By: Arles

Boring Crew: Thurston, Lubas, Degener
Date Started: 04/02/2025 Date Finished: 04/02/2025
Coordinates: N:347232.2, E:1630426.3
Station: 321+18 Offset: 13.2 ft LT
Ground Elevation: 728.5 ft

	Casing	Sampler
Type:	WB	SPT
I.D.:	4 in	1.5 in
Hammer Wt:	N.A.	140 lb
Hammer Fall:	N.A.	30 in.

Hammer/Rod type: Auto / AWJ
Rig: Diedrich D-25 $C_E = 1.45$

Groundwater Observations

Date	Depth (ft)	Status

Depth (ft)	Graphic Log	AASHTO Classification	Material Description and Notes	Field Data: Recovery (ft) (%)	Field Data: Blow Counts (N-Value)	Lab Data: Moisture Content (%)	Lab Data: Gravel (%)	Lab Data: Sand (%)	Lab Data: Fines (%)
			0.0 ft - 0.5 ft, Field Note: ASPHALT						
			0.5 ft - 2.5 ft, Field Description: Dry to Moist, Gray, GRAVEL, some Sand		12-10-8-8 (18)				
			2.5 ft - 4.5 ft, Field Description: Dry to Moist, Gray, SAND and GRAVEL, wood in sample		6-11-21-18 (32)				
5			4.5 ft - 5.7 ft, Field Description: Dry to Moist, Gray, SAND and GRAVEL, Refusal at 5.7 ft		13-24-50/2" (R)				
			5.0 ft - 6.5 ft, Field Note: Casing would not advance past 5.0 ft, NX Cleanout 5 ft - 6.5 ft						
10	NR NR NR NR NR NR NR		6.5 ft - 11.5 ft, Field Note: NO RECOVERY, 5 ft core run. Assumed cobbles and boulders						

Hole abandoned at 11.5 ft
Water Table not measured during drilling

Notes:
1. Stratification lines represent approximate boundary between material types. Transition may be gradual.
2. N-values have not been corrected for hammer energy. CE is the hammer energy correction factor.
3. Water level readings have been made at times and under conditions stated. Water levels are likely to vary.:

STATE OF VERMONT
AGENCY OF TRANSPORTATION
MATERIALS AND CONSTRUCTION BUREAU
GEOTECHNICAL ENGINEERING SECTION

BORING LOG

Reading
ER P23-1(404)
VT Route 106 Bridge No. 12

Boring No.: B-102a
Page No.: 1 of 1
Pin No.: 23b770
Checked By: Arles

Boring Crew: McGinley, Lubas, Degener
Date Started: 04/07/2025 Date Finished: 04/08/2025
Coordinates: N:347231.3, E:1630432.2
Station: 321+18 Offset: 7.3 ft LT
Ground Elevation: 728.4 ft

	Casing	Sampler
Type:	WB	SPT
I.D.:	4 in	1.5 in
Hammer Wt:	N.A.	140 lb
Hammer Fall:	N.A.	30 in.
Hammer/Rod type:	Auto / AWJ	
Rig:	Diedrich D-25	C_E = 1.45

Groundwater Observations

Date	Depth (ft)	Status
04/08/25	6.3'	After Drilling (AD)

Depth (ft)	Graphic Log	AASHTO Classification	Material Description and Notes	Field Data: Recovery (ft) (%)	Field Data: Blow Counts (N-Value)	Field Data: Run Number	Lab Data: Moisture Content (%)	Lab Data: Gravel (%)	Lab Data: Sand (%)	Lab Data: Fines (%)
			0.0 ft - 4.0 ft, Field Note: Advanced casing to 4 ft without sampling							
5			4.0 ft - 6.0 ft, Field Description: Dry to Moist, Brownish Gray, GRAVEL, some Sand	1.1	11-10-14-19 (24)					
			6.0 ft - 7.5 ft, Field Description: Dry to Moist, Brownish Gray, SAND, some Gravel, Refusal at 7.5 ft, NX Cleanout 6 ft - 8 ft	0.8	17-19-10/1" (R)					
	NR		8.0 ft - 9.4 ft, Field Description: NO RECOVERY, Refusal at 9.4 ft, Fractured Rock in end of Sampler		23-20-50/5" (R)					
10		A-1-b	10.0 ft - 12.0 ft, Lab Classification: Moist, Brownish Gray, SAND and GRAVEL, trace Silt	1.2	7-6-22-26 (28)		12.5	45.7	47.7	6.6
			12.0 ft - 14.0 ft, Lab Classification: Moist, Brownish Gray, SAND, some Gravel, little Silt, Metal shavings in sample	1.8	40-22-20-21 (42)		13.4	35.7	51.5	12.8
15			15.0 ft - 17.0 ft, Field Description: Moist, Brownish Gray, SAND, some Gravel	0.9	22-24-20-18 (44)					
		A-1-b	17.0 ft - 18.0 ft, Lab Classification: Dry to Moist, Brown, SAND, some Gravel, trace Silt, Refusal at 18 ft, 10 blows no movement	1	19-18-10/1" (R)		14.3	31.9	30.2	7.9
20			18.0 ft - 22.0 ft, Field Note: fine grained BOULDER	0.4 (13%)		R-1				
				0.9 (90%)		R-2				
25			22.0 ft - 27.0 ft, Magnetite-biotite-quartz-plagioclase GNEISS, hard, fresh to very slightly weathered, Joint surfaces are rough, Rust stains/oxidation along joints at 24.4 ft	4.9 (98%)		R-3				

End of Boring at 27 ft
Hole Collapse at 2.8 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. CE is the hammer energy correction factor.
3. Water level readings have been made at times and under conditions stated. Water levels are likely to vary.:

STATE OF VERMONT
AGENCY OF TRANSPORTATION
MATERIALS AND CONSTRUCTION BUREAU
GEOTECHNICAL ENGINEERING SECTION

BORING LOG

Reading
ER P23-1(404)
VT Route 106 Bridge No. 12

Boring No.: B-103

Pin No.: 23b770
Checked By: Arles

Boring Crew: Thurston, McGinley, Lubas
Date Started: 04/01/2025 Date Finished: 04/14/2025
Coordinates: N:347140.5, E:1630419.2
Station: 320+26 Offset: 10.7 ft LT
Ground Elevation: 728.1 ft

	Casing	Sampler
Type:	WB	SPT
I.D.:	4 in	1.5 in
Hammer Wt:	N.A.	140 lb
Hammer Fall:	N.A.	30 in.
Hammer/Rod type:	Auto / AWJ	
Rig:	Diedrich D-25	C_E = 1.45

Groundwater Observations

Date	Depth (ft)	Status

Depth (ft)	Graphic Log	AASHTO Classification	Material Description and Notes	Field Data: Recovery (ft) (%)	Field Data: Blow Counts (N-Value)	Field Data: Run Number	Lab Data: Moisture Content (%)	Lab Data: Gravel (%)	Lab Data: Sand (%)	Lab Data: Fines (%)
0.0 - 0.5			0.0 ft - 0.5 ft, Field Note: ASPHALT, Spun Casing to bedrock without sampling							
6.0 - 8.0			6.0 ft - 8.0 ft, Field Note: NX Cleanout 6 ft - 8 ft							
29.0 - 30.0			29.0 ft - 30.0 ft, Field Note: Rollercone Cleanout 29 ft - 30 ft							
30.0 - 35.0			30.0 ft - 35.0 ft, light gray Magnetite-biotite-quartz-plagioclase GNEISS, foliated, hard, fresh to very slightly weathered, Joint surfaces are rough, Rust stains on joints 30 ft - 30.3 ft. 32.3 ft - 35 ft very micaeous/biotite, NX	4.4 (88%)		R-1				
35.0 - 40.0			35.0 ft - 40.0 ft, light gray Magnetite-biotite-quartz-plagioclase GNEISS, hard, fresh to very slightly weathered, Occasional oxidation stains, NX	4.8 (96%)		R-2				
40.0 - 43.8			40.0 ft - 43.8 ft, light gray Magnetite-biotite-quartz-plagioclase GNEISS, foliated, hard, fresh to very slightly weathered, Occasional oxidation stains, NX	3.6 (72%)		R-3				

End of Boring at 43.8 ft
Hole Collapse at 3.8 ft
Water table not measured during drilling

Notes:
1. Stratification lines represent approximate boundary between material types. Transition may be gradual.
2. N-values have not been corrected for hammer energy. CE is the hammer energy correction factor.
3. Water level readings have been made at times and under conditions stated. Water levels are likely to vary.:

STATE OF VERMONT AGENCY OF TRANSPORTATION MATERIALS AND CONSTRUCTION BUREAU GEOTECHNICAL ENGINEERING SECTION	# BORING LOG **Reading** **ER P23-1(404)** **VT Route 106 Bridge No. 12**	Boring No.: B-104 Page No.: 1 of 1 Pin No.: 23b770 Checked By: Arles

Boring Crew: Thurston, Lubas, McGinley, Degener

Date Started: 04/09/2025 Date Finished: 04/09/2025

Coordinates: N:347204.9, E:1630448.2

Station: 320+93 Offset: 11.4 ft RT

Ground Elevation: 728.5 ft

	Casing	Sampler
Type:	WB	SPT
I.D.:	4 in	1.5 in
Hammer Wt:	N.A.	140 lb
Hammer Fall:	N.A.	30 in.
Hammer/Rod type:	Auto / AWJ	
Rig:	Diedrich D-25	C_E = 1.45

Groundwater Observations

Date	Depth (ft)	Status

Depth (ft)	Graphic Log	AASHTO Classification	Material Description and Notes	Field Data: Recovery (ft) (%)	Field Data: Blow Counts (N-Value)	Field Data: Run Number	Lab Data: Moisture Content (%)	Lab Data: Gravel (%)	Lab Data: Sand (%)	Lab Data: Fines (%)
0.0–25.0			0.0 ft - 25.0 ft, Field Note: Casing Advanced to rock without sampling							
25.0–30.0			25.0 ft - 30.0 ft, light gray Magnetite-biotite-quartz-plagioclase GNEISS, hard, fresh to very slightly weathered, Rust stains on joints, some oxidation stains through out	4.4 (88%)		R-1				
30.0–35.0			30.0 ft - 35.0 ft, light gray Magnetite-biotite-quartz-plagioclase GNEISS, hard, fresh to very slightly weathered	5.1 (102%)		R-2				
35.0–40.0			35.0 ft - 40.0 ft, light gray Magnetite-biotite-quartz-plagioclase GNEISS, foliated, hard, fresh to very slightly weathered, Joint surfaces are rough, rust stains on joints, some oxidation stains through out	5.1 (102%)		R-3				

End of Boring at 40 ft
Hole Collapse at 9.1 ft
Water table not measured during drilling

Notes:
1. Stratification lines represent approximate boundary between material types. Transition may be gradual.
2. N-values have not been corrected for hammer energy. CE is the hammer energy correction factor.
3. Water level readings have been made at times and under conditions stated. Water levels are likely to vary.:

Appendix 2

Rock Core Tables

Appendix 3

FB MultiPier Analysis

Table A2.1 Rock Core Table

Run Number	Core Size	Start Depth (ft)	End Depth (ft)	Recovery (%)	RQD (%)	Dip (deg)	Lithologic Description	RMR
B-101, R-1	NX	24	29	96	51	65/ 0-10	From 24 to 25.1 ft - probable BOULDERS. 25.2 ft to 29 ft, Light-gray to whitish-gray-weathering, magnetite-biotite-quartz-plagioclase GNEISS. Foliated, joint surfaces are rough. Some rust staining along joints. Hard, fresh to very slightly weathered. Fair Rock	59
B-101, R-2	NX	29	34	58	32	0-10	Light-gray to whitish-gray-weathering, magnetite-biotite-quartz-plagioclase GNEISS. Foliated, joint surfaces are rough. Hard, fresh to very slightly weathered. Fair Rock.	56
B-102a, R-3*	NX	22	27	98	62	15-30	Light-gray to whitish-gray-weathering, magnetite-biotite-quartz-plagioclase GNEISS. Foliated, joint surfaces are rough. Rust stains / oxidation along joints at 24.4ft. Hard, fresh to very slightly weathered. Good Rock.	64
B-103, R-1	NX	30	35	88	0	5-15	Light-gray to whitish-gray-weathering, magnetite-biotite-quartz-plagioclase GNEISS. Foliated, joint surfaces are rough. Rust stains on joints 30ft to 30.3ft. Occasional oxidation stains from 30.3ft to 35ft. 32.3ft to 35ft very micaceous/ biotite. Hard, fresh to very slightly weathered. Fair Rock.	42
B-103, R-2	NX	35	40	96	33	10-35	Light-gray to whitish-gray-weathering, magnetite-biotite-quartz-plagioclase GNEISS. Occasional oxidation stains. Hard, fresh to very slightly weathered. Fair Rock	47
B-103, R-3	NX	40	43.8	95	25	20	Light-gray to whitish-gray-weathering, magnetite-biotite-quartz-plagioclase GNEISS. Foliated, joint surfaces are rough. Occasional oxidation stains. Hard, fresh to very slightly weathered. Fair Rock.	49

**B-102a R-1, R-2 encountered suspected boulders and did not recover any bedrock*

Table A2.2 Rock Core Table

Run Number	Core Size	Start Depth (ft)	End Depth (ft)	Recovery (%)	RQD (%)	Dip (deg)	Lithologic Description	RMR
B-104, R-1	NX	25	30	88	70	35-45	Light-gray to whitish-gray-weathering, magnetite-biotite-quartz-plagioclase GNEISS. Foliated, joint surfaces are rough. Rust stains on joints, some oxidation stains through out. Hard, fresh to slightly weathered. Fair Rock.	57
B-104, R-2	NX	30	35	102	71	15-30	Light-gray to whitish-gray-weathering, magnetite-biotite-quartz-plagioclase GNEISS. Foliated, joint surfaces are rough. Rust stains on joints, some oxidation stains through out. Hard, fresh to slightly weathered. Fair Rock.	55
B-104, R-3	NX	35	40	102	96	10-40	Light-gray to whitish-gray-weathering, magnetite-biotite-quartz-plagioclase GNEISS. Foliated, joint surfaces are rough. Rust stains on joints, some oxidation stains through out. Hard, fresh to slightly weathered. Good Rock.	62

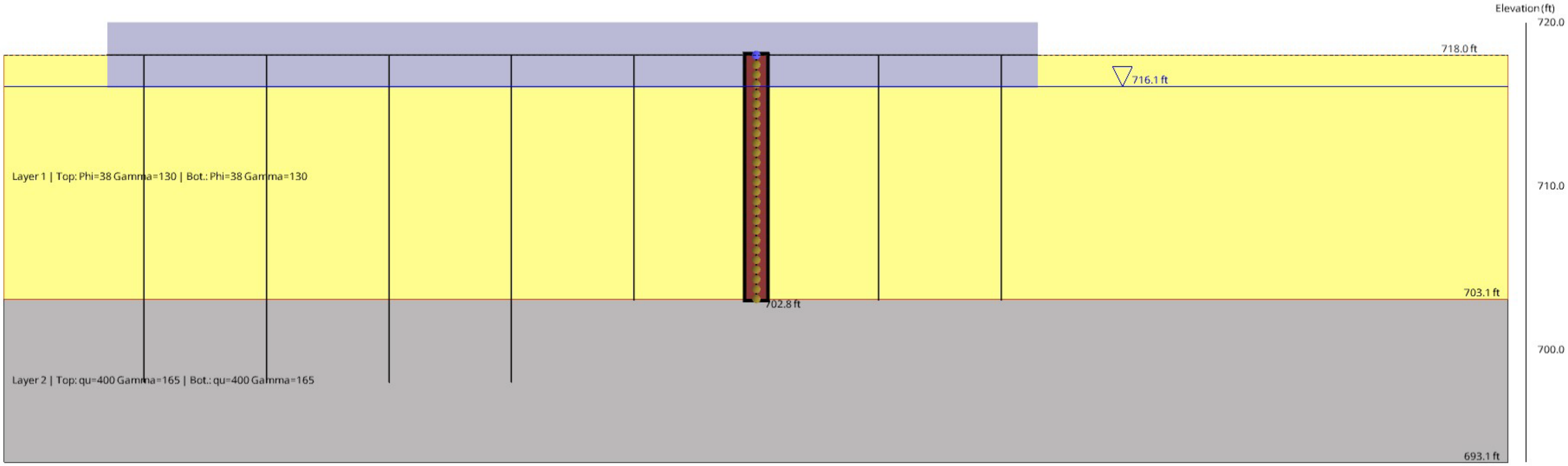

Figure 3.3.1 Abutment 1 Soil Profile

Table 3.3.1 Abutment 1 Soil Parameters

Soil Set	Soil Layer	Soil Type	Top Layer Elevation (ft)	Bottom Layer Elevation (ft)	Unit Weight (pcf)	Internal Friction Angle (deg)	Subgrade Modulus (lb/in^3)	Unconfined Compressive Strength (psf)	Small Strain Shear Modulus (ksi)	Unit Side Friction (psf)	Shear Modulus (ksi)	Torsional Shear Stress (psf)	Small Strain Shear Modulus (ksi)	Nominal Tip Resistance (kips)
1	1	Cohesionless	718.0	703.1	130	38	125		2.11	1646.24	2.11	1646.24		
1	2	Rock	703.1	693.1	165			400	3631.15	1646.24	3631.15	1646.24	3631.15	1044.00
2	1	Cohesionless	718.0	703.1	130	38	125		2.11	1646.24	2.11	1646.24		
2	2	Rock	703.1	693.1	165			400	3631.15	1646.24	3631.15	1646.24	3631.15	1044.00
3	1	Cohesionless	718.0	698.1	130	38	125		2.11	1646.24	2.11	1646.24		
3	2	Rock	698.1	688.1	165			400	3631.15	1646.24	3631.15	1646.24	3631.15	1044.00
4	1	Cohesionless	718.0	698.1	130	38	125		2.11	1646.24	2.11	1646.24		
4	2	Rock	698.1	688.1	165			400	3631.15	1646.24	3631.15	1646.24	3631.15	1044.00

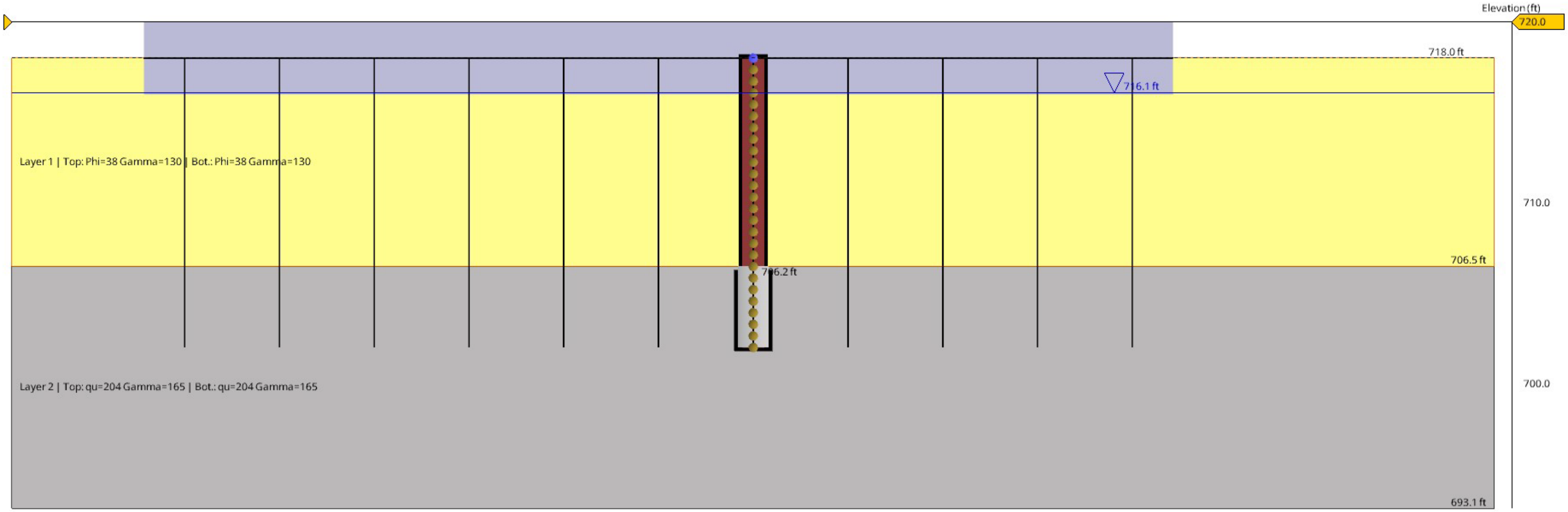

Figure 3.3.2 Abutment 2 Soil Profile

Table 3.3.2 Abutment 2 Soil Parameters

Soil Set	Soil Layer	Soil Type	Top Layer Elevation (ft)	Bottom Layer Elevation (ft)	Unit Weight (pcf)	Internal Friction Angle (deg)	Subgrade Modulus (lb/in^3)	Unconfined Compressive Strength (psf)	Small Strain Shear Modulus (ksi)	Unit Side Friction (psf)	Shear Modulus (ksi)	Torsional Shear Stress (psf)	Small Strain Shear Modulus (ksi)	Nominal Tip Resistance (kips)
1	1	Cohesionless	716.0	703.1	130	38	125		2.11	1646.24	2.11	1646.24		
1	2	Rock	703.1	693.1	165			400	3631.15	1646.24	3631.15	1646.24	3631.15	1044
2	1	Cohesionless	716.0	703.1	130	38	125		2.11	1646.24	2.11	1646.24		
2	2	Rock	703.1	693.1	165			400	3631.15	1646.24	3631.15	1646.24	3631.15	1044
3	1	Cohesionless	716.0	698.1	130	38	125		2.11	1646.24	2.11	1646.24		
3	2	Rock	698.1	688.1	165			400	3631.15	1646.24	3631.15	1646.24	3631.15	1044
4	1	Cohesionless	716.0	698.1	130	38	125		2.11	1646.24	2.11	1646.24		
4	2	Rock	698.1	688.1	165			400	3631.15	1646.24	3631.15	1646.24	3631.15	1044

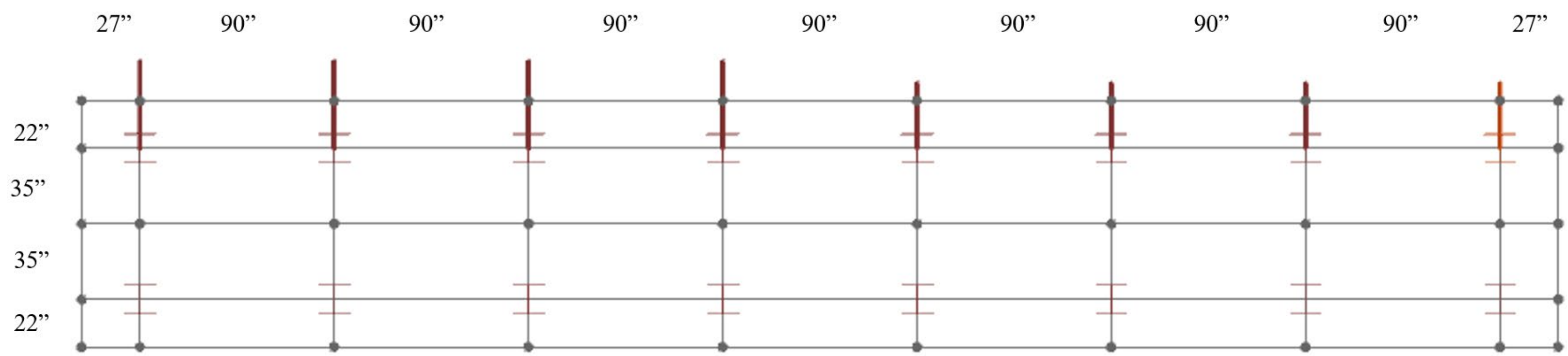

Figure 3.4.1 Abutment 1 Pile Cap Geometry

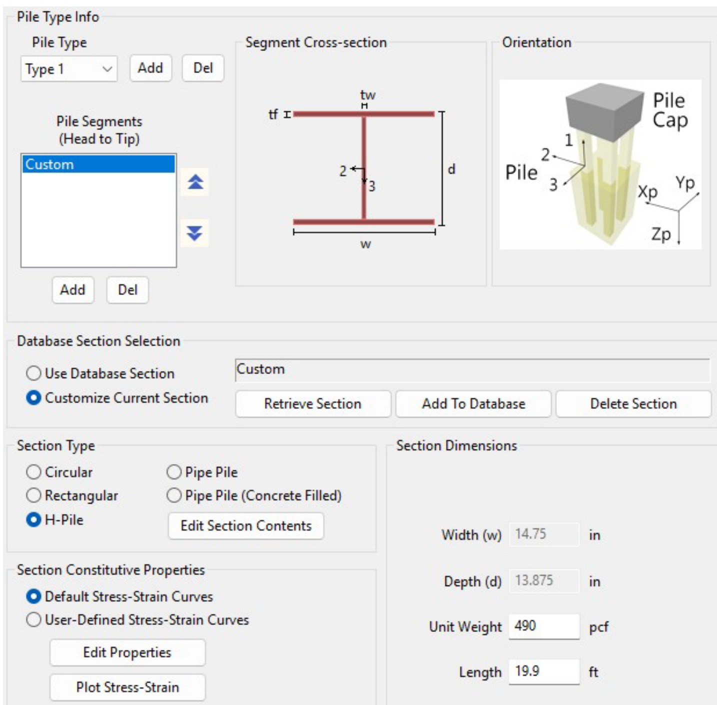

Figure 3.4.2 A1 Pile Type 1 Details

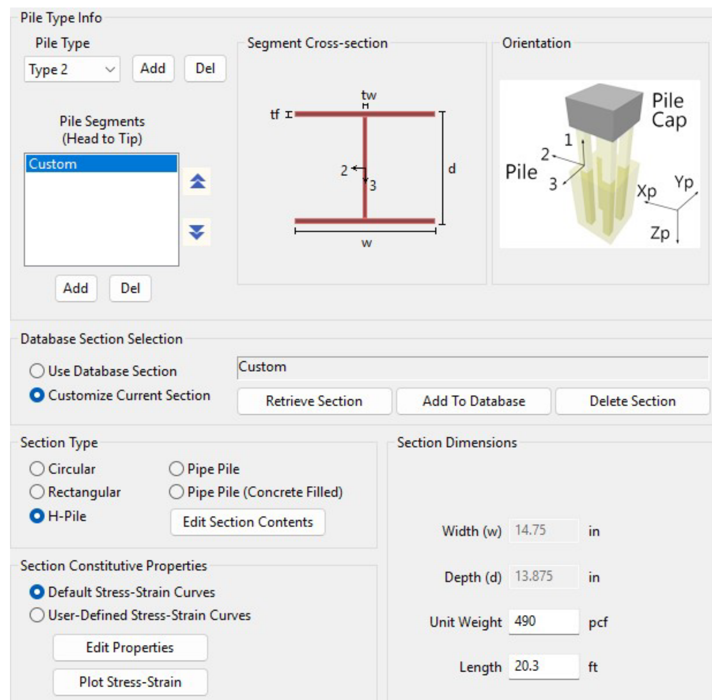

Figure 3.4.3 A1 Pile Type 2 Details

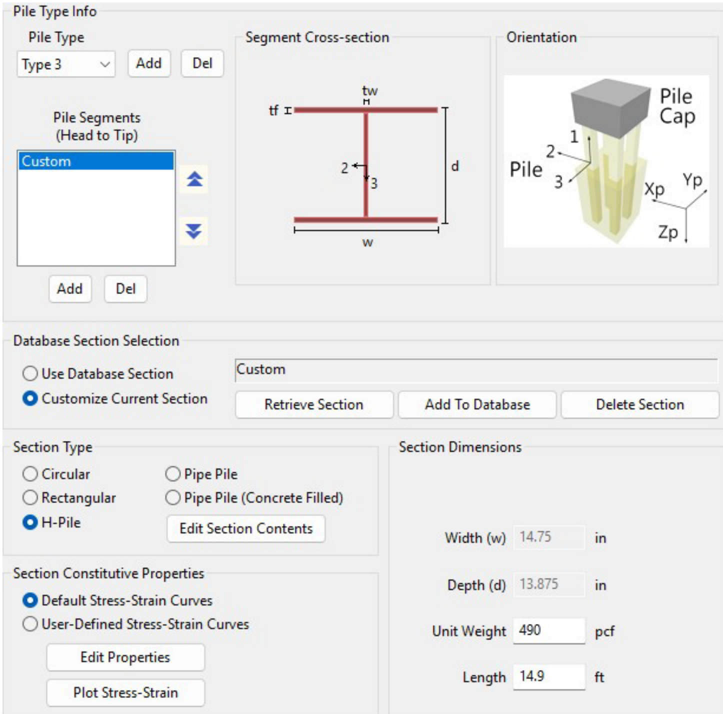

Figure 3.4.4 A1 Pile Type 3 Details

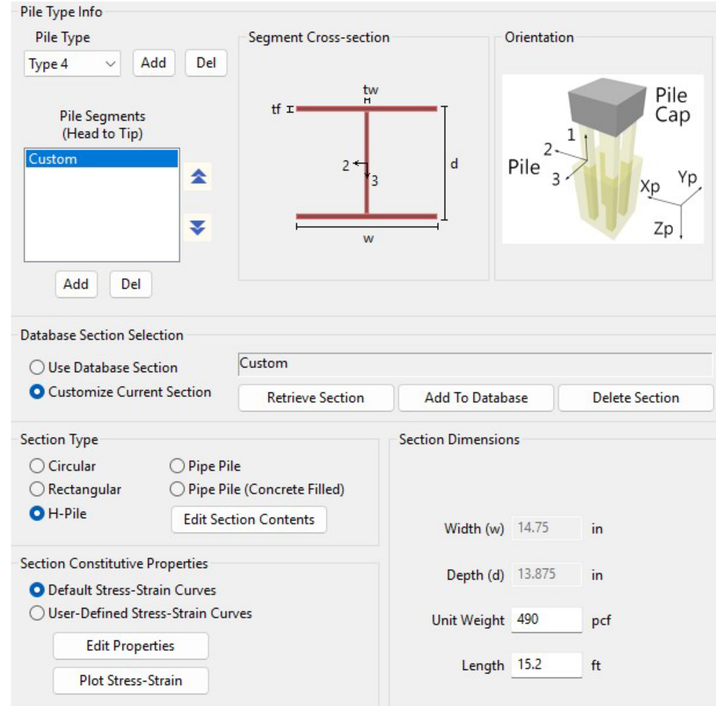

Figure 3.4.5 A1 Pile Type 4 Details

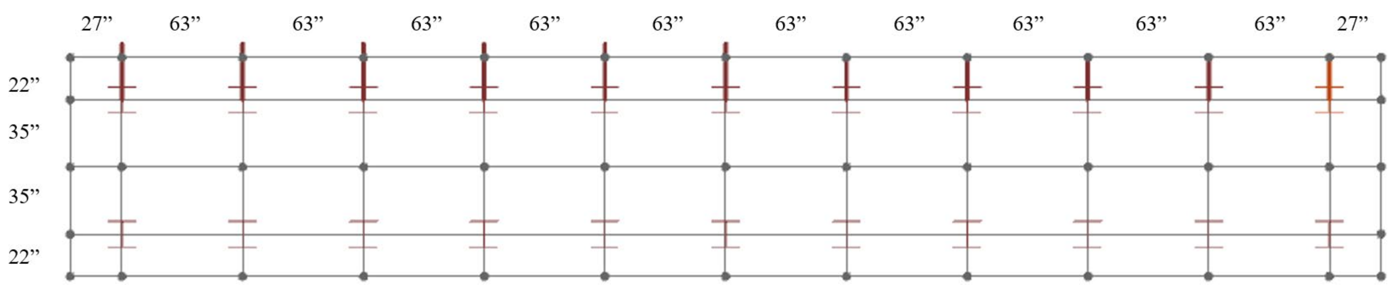

Figure 3.4.6 Abutment 2 Pile Cap Geometry

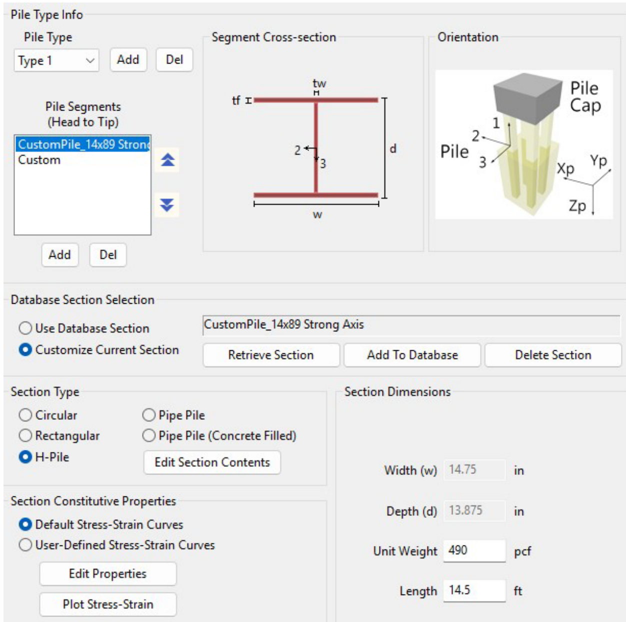

Figure 3.4.7 A2 Pile Type 1 Details

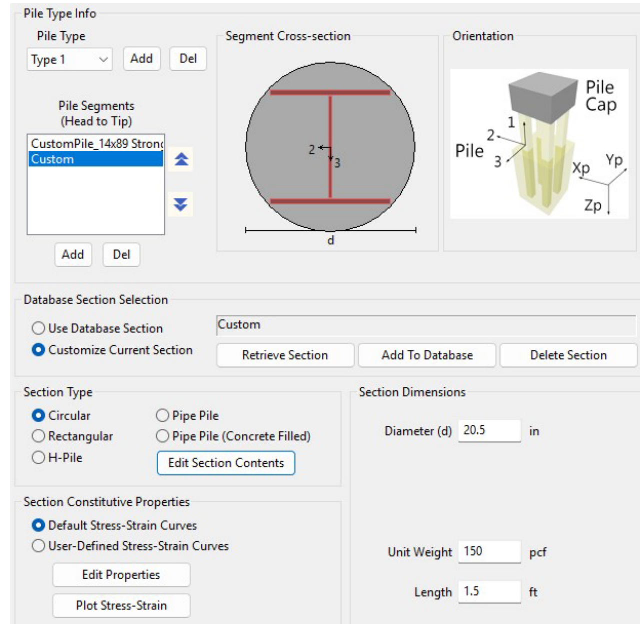

Figure 3.4.8 A2 Pile Type 1 Details

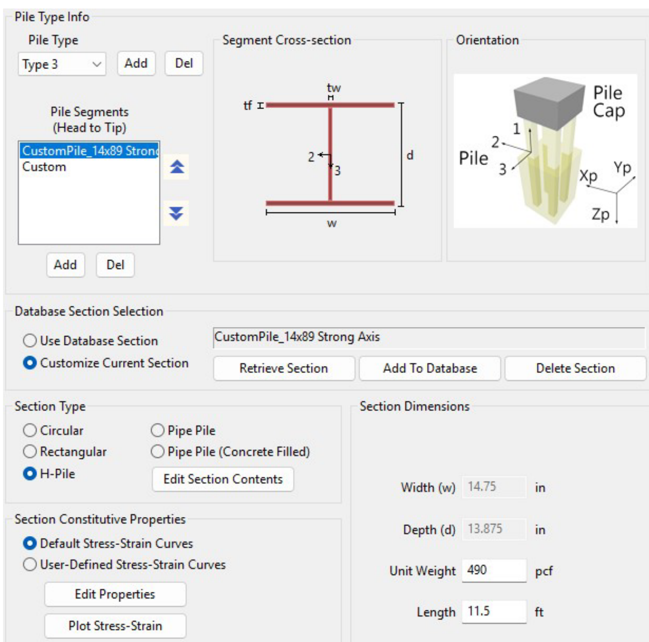

Figure 3.4.9 A2 Pile Type 3 Details

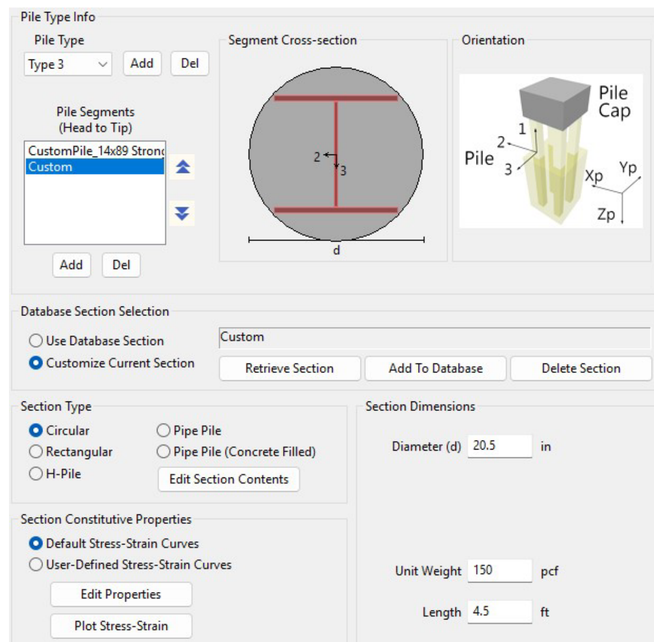

Figure 3.4.10 A2 Pile Type 4 Details

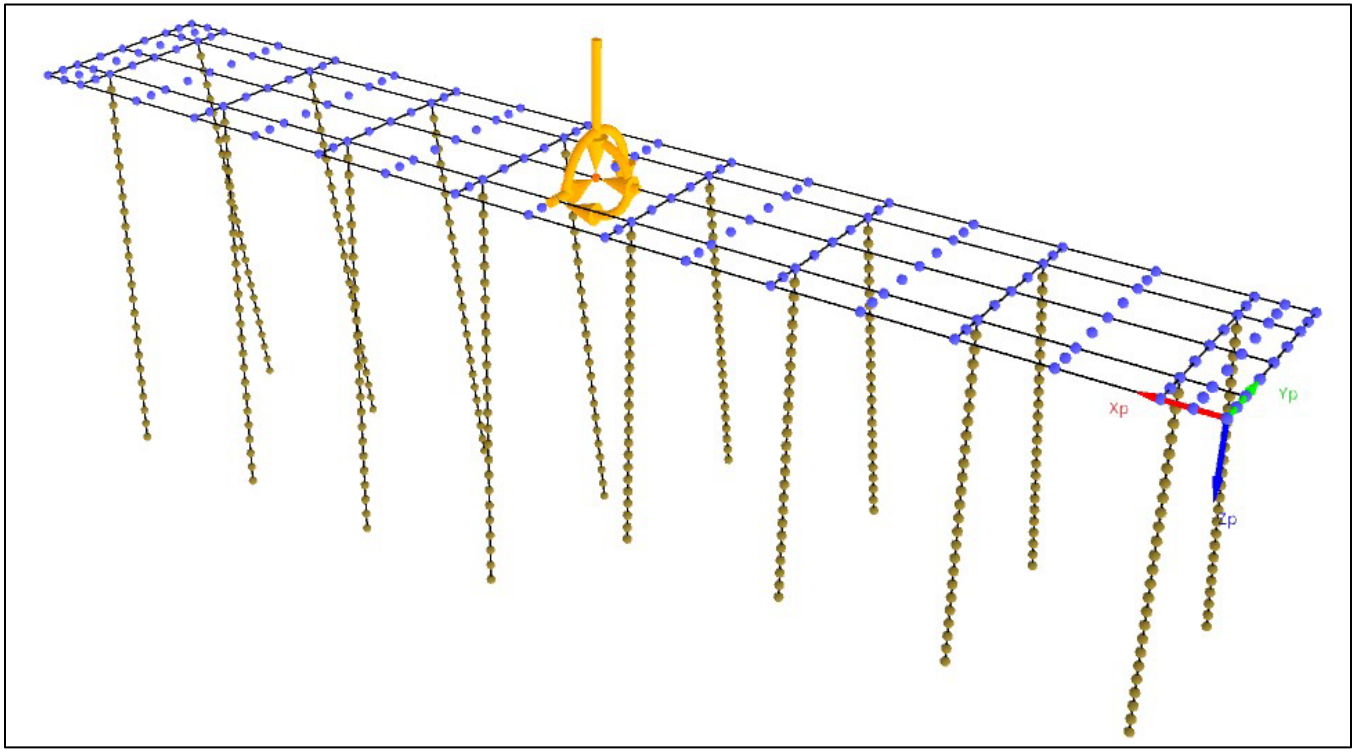

Figure 3.5.1 Abutment 1 Loading Schematic

Load Case	Node	Force Xp	Force Yp	Force Zp	Moment Xp	Moment Yp	Moment Zp	Displacement?
PreLoad	1	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	N/A
1	111	25.4000	667.6000	1942.6000	2999.1000	322.5000	0.0000	☐
2	111	19.6000	624.0000	1866.9000	2999.1000	322.5000	0.0000	☐
3	111	5.9000	482.0000	1611.2000	1487.2000	75.0000	0.0000	☐
4	111	0.0000	476.5000	1828.8000	1364.3000	0.0000	0.0000	☐
5	111	29.2000	633.0000	1866.9000	2752.9000	371.4000	0.0000	☐
6	111	24.2000	436.0000	1441.4000	1901.4000	306.9000	0.0000	☐
7	111	18.9000	459.7000	1498.2000	2057.1000	239.6000	0.0000	☐
8	111	14.5000	426.9000	1441.4000	1786.0000	184.3000	0.0000	☐
9	111	5.9000	323.2000	1441.4000	979.8000	12.5000	0.0000	☐

Figure 3.5.2 Abutment 1 Load Table

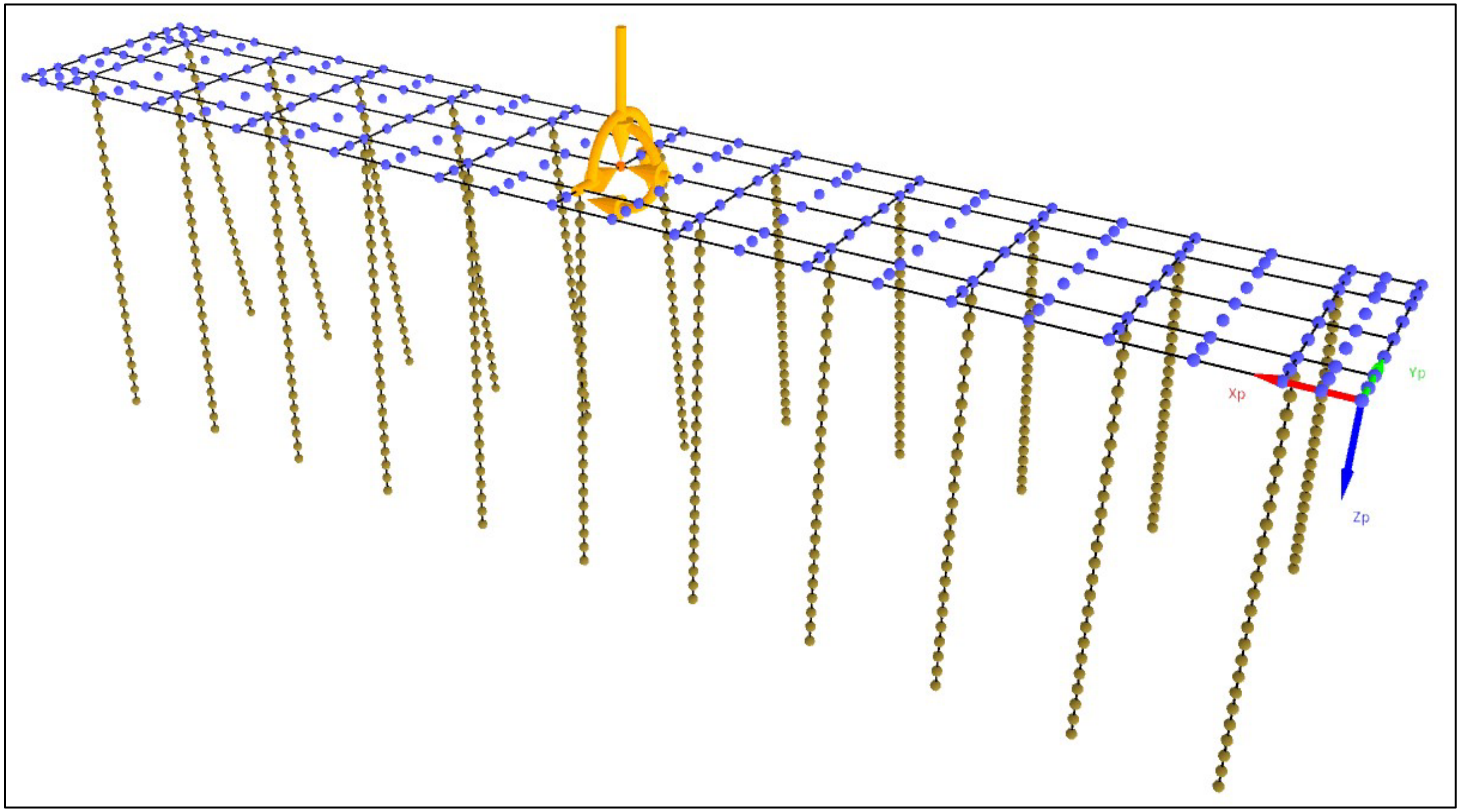

Figure 3.5.3 Abutment 2 Loading Schematic

Load Case	Node	Force Xp	Force Yp	Force Zp	Moment Xp	Moment Yp	Moment Zp	Displacement?
PreLoad	1	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	N/A
1	44	25.4000	802.8000	2118.6000	3915.5000	322.5000	0.0000	☐
2	44	19.6000	758.0000	2042.8000	3915.5000	322.5000	0.0000	☐
3	44	5.9000	612.3000	1787.2000	2304.6000	75.0000	0.0000	☐
4	44	0.0000	606.8000	2014.3000	2212.3000	0.0000	0.0000	☐
5	44	29.2000	767.0000	2042.8000	3646.7000	371.4000	0.0000	☐
6	44	24.2000	525.6000	1574.6000	2517.6000	306.9000	0.0000	☐
7	44	18.9000	550.1000	1631.4000	2690.3000	239.6000	0.0000	☐
8	44	14.5000	516.5000	1574.6000	2402.2000	184.3000	0.0000	☐
9	44	5.9000	410.0000	1574.6000	1539.5000	12.5000	0.0000	☐

Figure 3.5.4 Abutment 2 Load Table

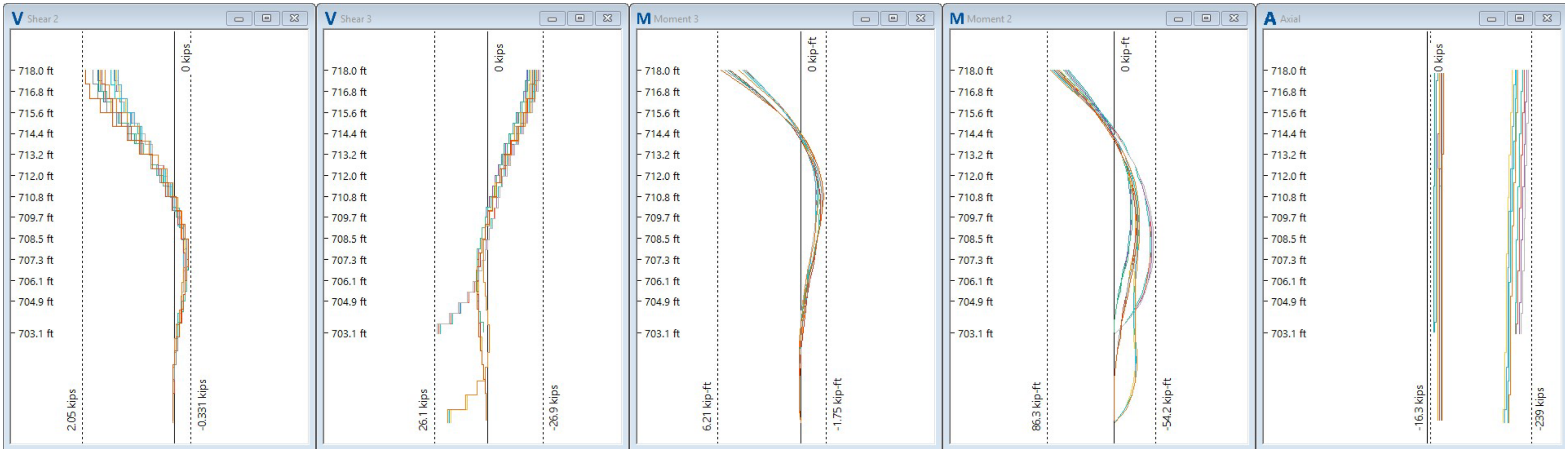

Figure 3.6.1 Abutment 1 Strength I Non-Scour Output Forces

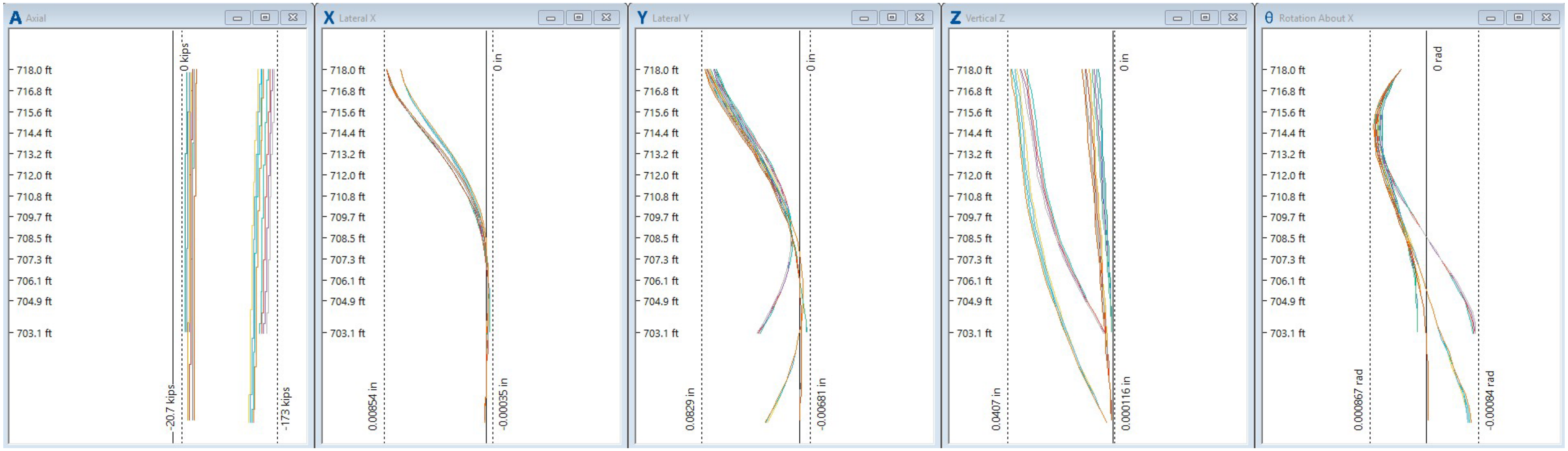

Figure 3.6.2 Abutment 1 Service II Non-Scour Output Forces

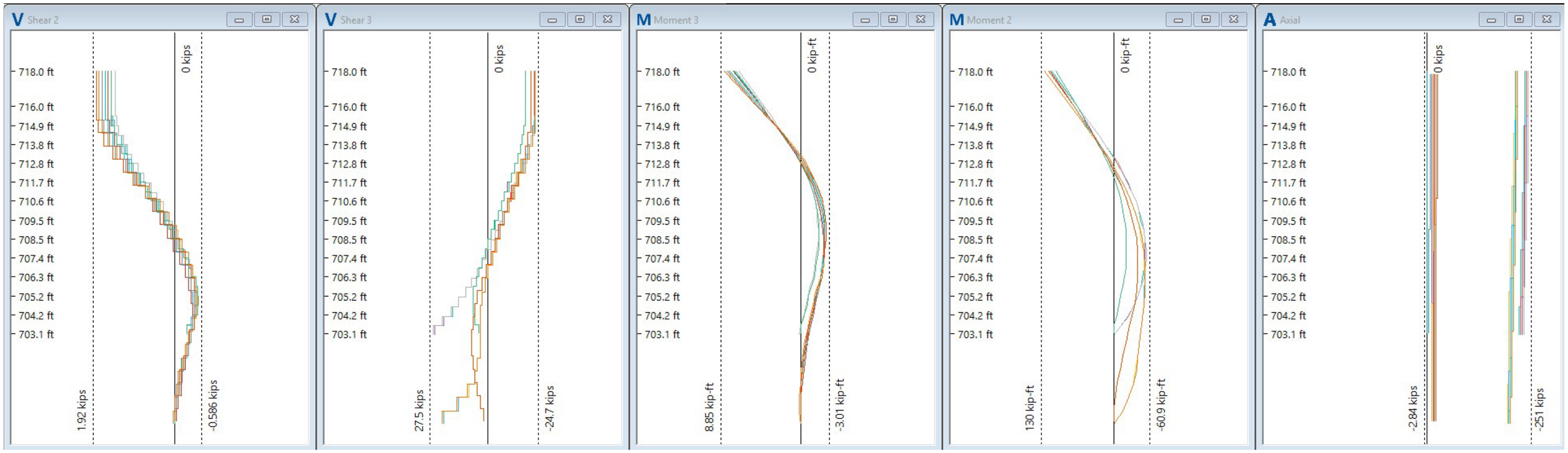

Figure 3.6.3 Abutment 1 Strength I Scour Output Forces

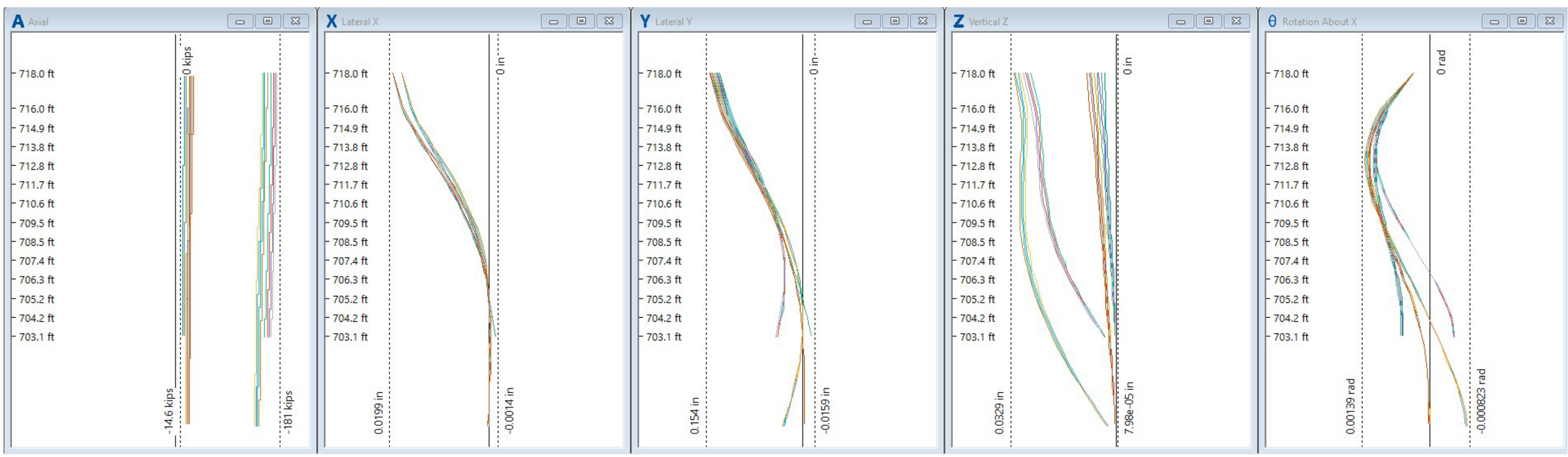

Figure 3.6.4 Abutment 1 Service II Design Scour Output Forces

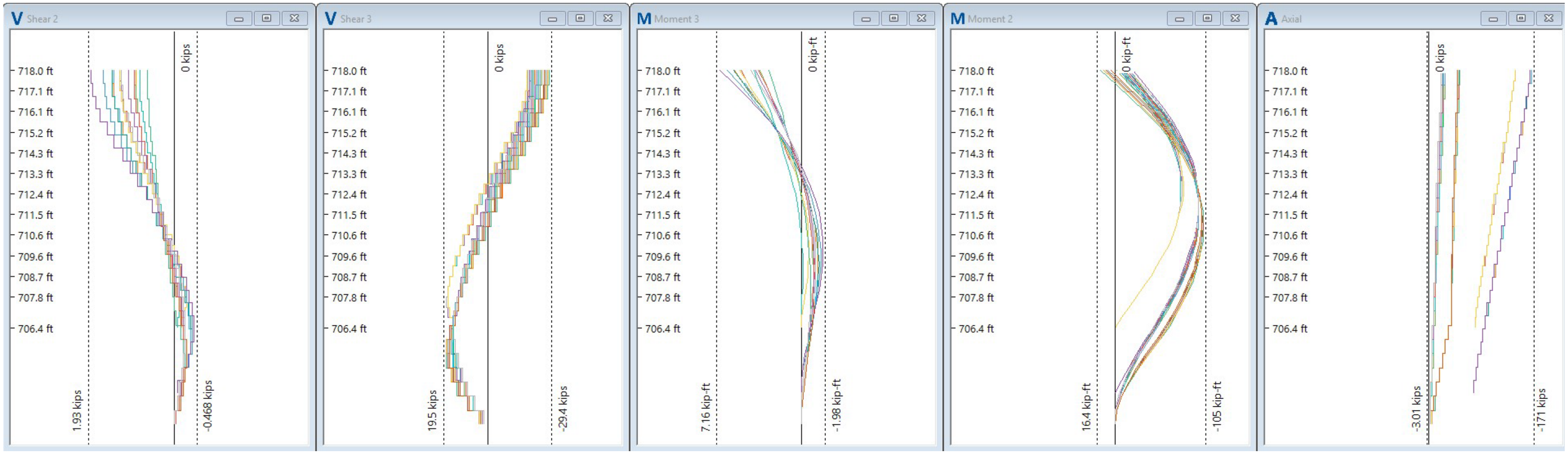

Figure 3.6.5 Abutment 2 Strength I Non-Scour Output Forces

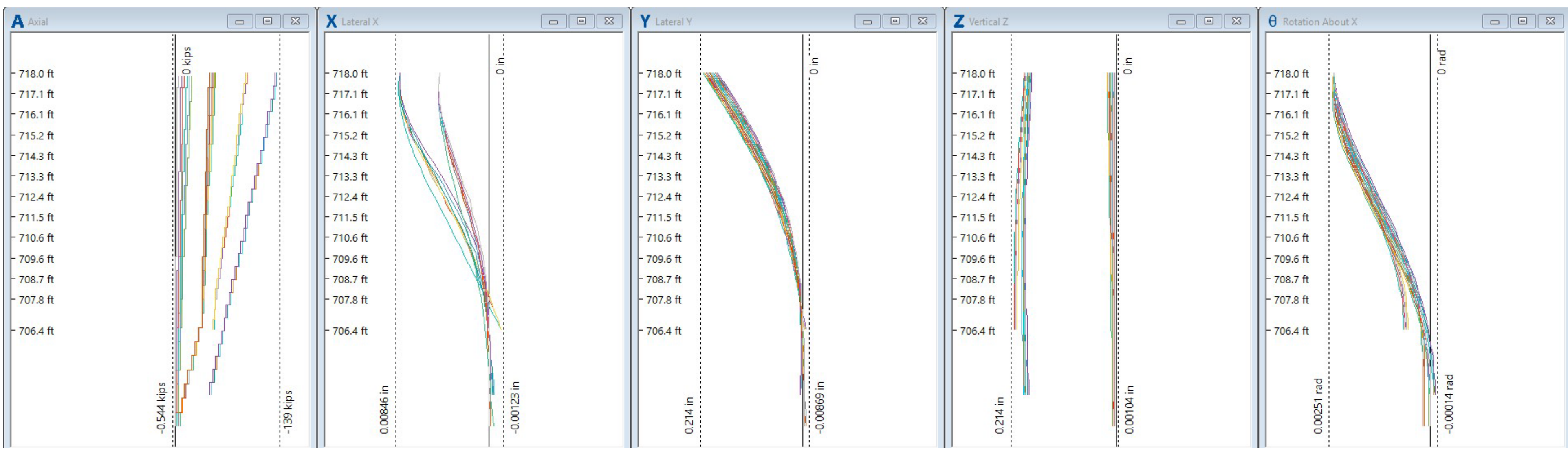

Figure 3.6.6 Abutment 2 Service II Non-Scour Output Forces

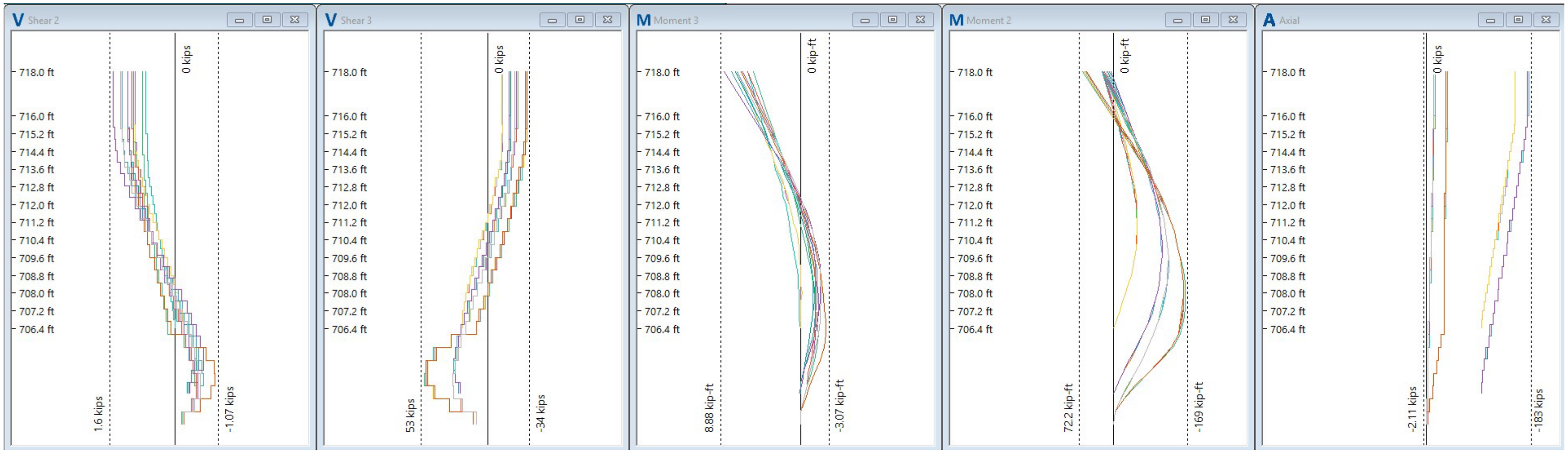

Figure 3.6.7 Abutment 2 Strength I Scour Output Forces

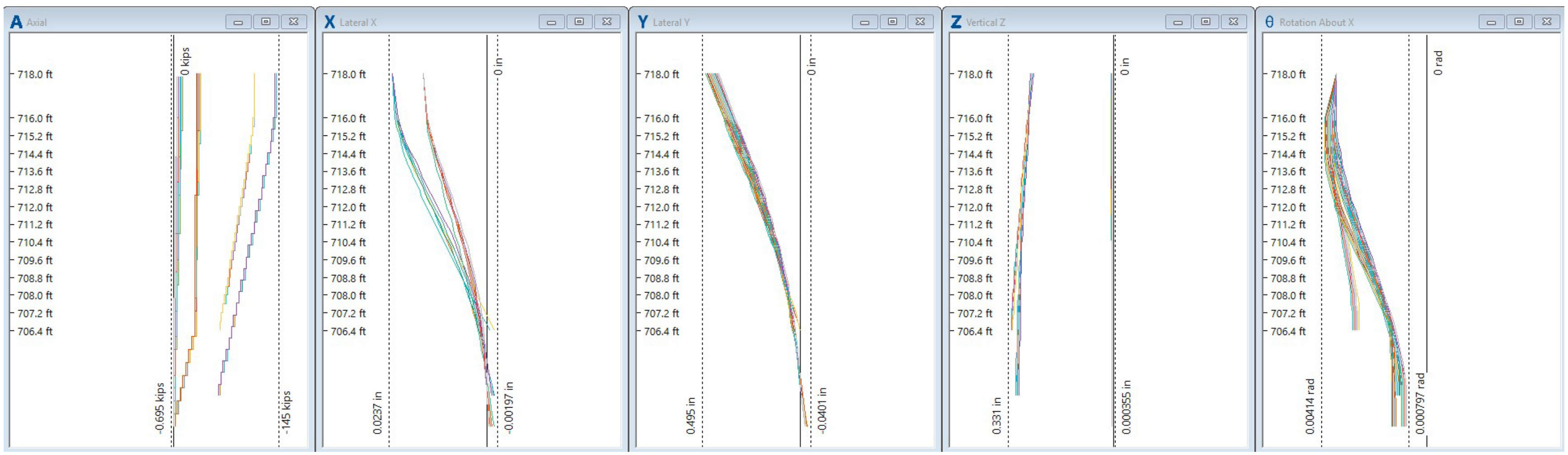

Figure 3.6.8 Abutment 2 Service II Design Scour Output Forces

Table 3.7.1 Abutment 1 Non Scour FB Pier Output Forces

FB MultiPier Final Output Loads Abutment 1 Non-Scour								
Load Case	Shear x (k)	Shear Y (k)	Momx (kft)	Momy (kft)	Axial (k)	Disp x (in)	Disp y (in)	Rotatex (rad)
Strength I	**2.05**	**26.92**	**86.29**	**6.21**	**238.79**	**0.01**	**0.14**	**0.001**
Strength II	1.63	24.88	75.78	4.81	229.42	0.01	0.13	0.001
Strength III	0.55	19.34	59.90	1.46	170.74	0.00	0.09	0.001
Strength IV	0.11	19.45	55.90	0.46	180.70	0.00	0.08	0.001
Strength V	2.31	25.59	80.91	6.93	225.86	0.01	0.13	0.001
Service I	1.92	17.70	49.31	5.49	164.50	0.01	0.08	0.001
Service II	**1.53**	**18.65**	**51.90**	**4.35**	**173.13**	**0.01**	**0.08**	**0.001**
Service III	1.19	17.35	48.16	3.36	161.35	0.01	0.08	0.001
Service IV	0.51	14.42	33.55	1.36	135.81	0.00	0.05	0.001

Table 3.7.2 Abutment 1 Design Scour FB Pier Output Forces

FB MultiPier Final Output Loads Abutment 1 Design Scour								
Load Case	Shear x (k)	Shear Y (k)	Momx (kft)	Momy (kft)	Axial (k)	Disp x (in)	Disp y (in)	Rotatex (rad)
Strength I	**1.92**	**27.49**	**129.99**	**8.85**	**251.12**	**0.03**	**0.27**	**0.002**
Strength II	1.50	26.19	116.40	6.80	240.80	0.02	0.24	0.002
Strength III	0.48	20.11	89.74	2.04	180.01	0.01	0.16	0.001
Strength IV	0.06	20.50	82.40	0.34	188.42	0.00	0.15	0.001
Strength V	2.20	26.15	122.69	10.08	237.87	0.04	0.25	0.002
Service I	1.82	18.94	74.89	7.81	171.98	0.02	0.14	0.001
Service II	**1.44**	**19.90**	**79.53**	**6.22**	**181.10**	**0.02**	**0.15**	**0.001**
Service III	1.12	18.56	72.65	4.75	168.53	0.01	0.14	0.001
Service IV	0.49	15.02	45.96	1.92	139.48	0.01	0.08	0.001

Table 3.7.3 Abutment 1 Check Scour FB Pier Output Forces

FB MultiPier Final Output Loads Abutment 1 Check Scour								
Load Case	Shear x (k)	Shear Y (k)	Momx (kft)	Momy (kft)	Axial (k)	Disp x (in)	Disp y (in)	Rotatex (rad)
Extreme I	1.11	18.59	74.19	4.84	168.85	0.02	0.14	0.001
Extreme II	0.59	16.34	62.78	2.47	147.86	0.01	0.11	0.001

Table 3.7.4 Abutment 2 Non Scour FB Pier Output Forces

FB MultiPier Final Output Loads Abutment 2 Non-Scour								
Load Case	Shear x (k)	Shear Y (k)	Momx (kft)	Momy (kft)	Axial (k)	Disp x (in)	Disp y (in)	Rotatex (rad)
Strength I	**1.93**	**29.39**	**104.72**	**7.16**	**171.34**	**0.02**	**0.48**	**0.005**
Strength II	1.56	27.34	96.86	5.76	167.56	0.01	0.44	0.005
Strength III	0.68	21.82	57.87	2.59	143.19	0.00	0.24	0.003
Strength IV	0.30	21.06	60.93	1.30	147.95	0.00	0.25	0.003
Strength V	2.15	27.86	94.14	7.83	166.07	0.02	0.43	0.005
Service I	1.81	18.22	49.07	6.02	135.49	0.01	0.19	0.002
Service II	**1.48**	**19.10**	**53.51**	**5.05**	**139.47**	**0.01**	**0.21**	**0.003**
Service III	1.20	17.89	47.42	4.12	134.12	0.01	0.19	0.002
Service IV	0.68	13.89	33.63	2.50	121.88	0.00	0.12	0.001

Table 3.7.5 Abutment 2 Design Scour FB Pier Output Forces

FB MultiPier Final Output Loads Abutment 2 Design Scour								
Load Case	Shear x (k)	Shear Y (k)	Momx (kft)	Momy (kft)	Axial (k)	Disp x (in)	Disp y (in)	Rotatex (rad)
Strength I	**1.60**	**53.05**	**168.87**	**8.88**	**182.67**	**0.04**	**1.16**	**0.010**
Strength II	1.30	47.32	152.99	7.12	178.08	0.03	1.04	0.009
Strength III	0.58	27.14	89.58	3.08	149.46	0.01	0.57	0.005
Strength IV	0.23	27.83	93.15	1.32	154.23	0.01	0.60	0.005
Strength V	1.83	46.85	150.36	9.98	176.69	0.04	1.02	0.008
Service I	1.68	21.11	72.66	8.32	140.10	0.03	0.45	0.004
Service II	**1.36**	**23.19**	**79.20**	**6.85**	**144.56**	**0.02**	**0.49**	**0.004**
Service III	1.12	20.36	70.33	5.57	138.55	0.02	0.43	0.004
Service IV	0.62	14.92	48.98	3.10	124.42	0.01	0.28	0.002

Table 3.7.6 Abutment 2 Check Scour FB Pier Output Forces

FB MultiPier Final Output Loads Abutment 2 Check Scour								
Load Case	Shear x (k)	Shear Y (k)	Momx (kft)	Momy (kft)	Axial (k)	Disp x (in)	Disp y (in)	Rotatex (rad)
Extreme I	1.10	21.26	71.72	5.64	138.65	0.02	0.45	0.004
Extreme II	0.69	17.39	58.31	3.55	128.36	0.01	0.36	0.003

Structural Pile Check - **2010 AASHTO LRFD Bridge Designs Specifications**
VTrans Soils & Foundations Unit - January 2011

6.15 - PILES: Piles shall be designed as structural members capable of safely supporting all imposed loads.

6.15.3 Compressive Resistance

6.15.3.1 - Axial Compression: For piles under axial load, the factored resistance of pile in compression, Pr, shall be taken as specified in 6.9.2.1 using the resistance factors in 6.5.4.2.

6.15.3.2 - Combined Axial Compression and Flexure: Pile subjected to axial load and flexure shall be designed in accordance with 6.9.2.2 using the resistance factors in 6.5.4.2.

6.15.3.3 - Buckling: Instability of piles which extend through water or air shall be accounted for and specified in 6.9. Pile which extend through water or air shall be assumed fixed at some depth below the ground. Stability shall be determined in accordance with 6.9 for compression members using an equivalent length of pile equal to the laterally unsupported length, plus an embedded depth to fixity.

How the MathCad Sheet Works:

1. Perform an analysis in FB-Pier, extract both moments, axial load, unbraced length and depth to fixity from largest moment diagram versus depth.
2. Enter the largest moments and axial loads, along with unbraced length and depth to fixity below to attain the 'worst case scenario' pile interaction. To hone in on the interaction coefficient of a specific pile; enter moments, axial, and lengths specific to that pile.

(Fill in GREEN Boxes and Use List Box to Choose Pile Size!)

INPUT VARIABLES: *ENGLISH UNITS*

FB-MultiPier Input ***(IN GREEN)***:

Factored flexural moment about the x-axis, $M_{ux} := 1035.5 \cdot kip \cdot in$

Factored flexural moment about the y-axis, $M_{uy} := 57.4 \cdot kip \cdot in$

Factored compressive load, $P_u := 238.79 \cdot kip$

Unbraced Length, $L_b := 194.7 \cdot in$

Be Aware of Units!

(Distance between two points of
inflection on moment diagram)

Depth to Fixity, $l := 240.2 \cdot in$

(Distance between top of pile and point of negligible
moment and deflection in the pile)

Choose Pile Size:

$Pile :=$ HP 14x89

Design Considerations:

Pile Head Condition:

$k := 1.2$

(See Section 4.6.2.5 for
effective length factors
for various conditions)

Grade of Steel:

$E := 29000 \cdot ksi$

$F_y := 50 \cdot ksi$

Note: Must change these values if not Grade 50 Steel

Pile Size Inputs:

$table_{pile} :=$

"HP 12x84"	12.3	0.685	0.685	12.3	1.375	24.6	34.6	53.2	2.94	106	120	5.14	4.24	3.4
"HP 12x74"	12.2	0.61	0.605	12.2	1.3125	21.8	30.4	46.6	2.92	93.8	105	5.11	2.98	3.3
"HP 12x63"	12.1	0.515	0.515	12.1	1.25	18.4	25.3	38.7	2.88	79.1	88.3	5.06	1.83	3.3
"HP 14x117"	14.9	0.805	0.805	14.9	1.5	34.4	59.5	91.4	3.59	172	194	5.96	8.02	4.1
"HP 14x102"	14.8	0.705	0.705	14.8	1.375	30.1	51.4	78.8	3.56	150	169	5.92	5.39	4.
"HP 14x89"	14.7	0.615	0.615	14.7	1.3125	26.1	44.3	67.7	3.53	131	146	5.88	3.59	4.0

$b_f := \text{vlookup}\left(Pile, table_{pile}, 1\right)_0 \cdot in$

$t_f := \text{vlookup}\left(Pile, table_{pile}, 2\right)_0 \cdot in$

$t_w := \text{vlookup}\left(Pile, table\right.$

$d := \text{vlookup}\left(Pile, table_{pile}, 4\right)_0 \cdot in$

$ko := \text{vlookup}\left(Pile, table_{pile}, 5\right)_0 \cdot in$

$A_s := \text{vlookup}\left(Pile, table\right.$

$S_y := \text{vlookup}\left(Pile, table_{pile}, 7\right)_0 \cdot in^3$

$Z_y := \text{vlookup}\left(Pile, table_{pile}, 8\right)_0 \cdot in^3$

$ry := \text{vlookup}\left(Pile, table\right.$

$S_x := \text{vlookup}\left(Pile, table_{pile}, 10\right)_0 \cdot in^3$

$Z_x := \text{vlookup}\left(Pile, table_{pile}, 11\right)_0 \cdot in^3$

$rx := \text{vlookup}\left(Pile, table\right.$

Selected Pile Properties:

$b_f = 14.7\ in$ $t_f = 0.615\ in$ $t_w = 0.615\ in$ $d = 14.7\ in$ $ko = 1.313\ in$ $A_s = 26.1\ in^2$

$S_y = 44.3\ in^3$ $Z_y = 67.7\ in^3$ $ry = 3.53\ in$ $J_{const} = 3.59$

$D_c := \frac{d}{2}$ Dc = dep

$rts = 4.05\ in$

$S_x = 131\ in^3$ $Z_x = 146\ in^3$ $rx = 5.88\ in$

$t := d - 2 \cdot ko = 12.075\ in$ $\lambda_f := \frac{b_f}{2 \cdot t_f} = 11.951$ $\lambda w := \frac{t}{t_w} = 19.634$

6.15.2 - Structural Resistance Factors:

The specified resistance ϕc a factors for axial resistance of 0.50 to 0.70 for piles in compression without bending shall be applied only to the axial capacity of the pile. The ϕc factors of 0.70 and 0.80 and the ϕf factor 1.0 shall be applied to the combined axial and flexural resistance of the pile in the interaction equation for the compression and flexure terms, respectively. See Section C6.15.2 for further discussion.

Notes:

(Combined axial & flexural)

$\phi_c := 0.70$

$\phi_f := 1.0$

(Axial Only)

$\phi_{ca} := 0.60$

- Flexure occurs primarily toward the head of the pile. The upper zone of the pile is less likely to experience damage during driving.
- 0.60 between severe and good driving conditions; VTrans uses 0.65 if dynamic pile testing planned

6.9.4.1 - Nominal Compressive Resistance

Calculate the elastic critical buckling strength (Pe) and the equivalent nominal yield resistance (Pe).. The variables in these equations differ between the 2010 version of AASHTO and all editions prior. The 2010 edition uses different variables to allow for more convenient calculation of the nominal resistance for members subject to buckling modes in addition to, or other than, flexural buckling, and to allow for the consideration of compression members with slender elements (C6.9.4.1.1).
* The depth to fixity is used rather than the unbraced length because when calculating axial resistance, the length between points of fixity on the column should be considered.

$P_e := \frac{\pi^2 \cdot E}{\left(\frac{k \cdot l}{\ }\right)^2} \cdot A_s$

$\left(ry \right)$ $P_e = \left(1.12 \cdot 10^3\right)$ ***kip***

Slender Element Reduction (6.9.4.2): $Q := 1.0$

$P_o := Q \cdot F_y \cdot A_s$

$P_o = \left(1.305 \cdot 10^3\right)$ ***kip***

$$P_n := \begin{cases} \left(0.658^{\frac{P_o}{P_e}}\right) \cdot P_o & \text{if } \dfrac{P_e}{P_o} \geq 0.44 \\ 0.877 \cdot P_e & \text{if } \dfrac{P_e}{P_o} < 0.44 \end{cases}$$

$\left(\dfrac{P_o}{P_e}\right) = 1.165$

$P_n = 801.475$ ***kip***

Factored Compressive Resistance:

$P_r := \phi_c \cdot P_n$ $P_r = 561.032$ ***kip***

$\left(\phi c\right)$ is used here because upper zone of pile controls)

A6.3 - Flexural Resistance Based on the Compression Flange

For unbraced lengths in which the member is prismatic, the nominal flexural resistance based on the compression flange, Mnc, shall be taken as the smaller of the local buckling resistance determined in A6.3.2 (Mn1), and the lateral torsional buckling resistance detemined in A6.3.3 (Mn2).

A6.3.2 - Local Buckling Resistance

Slenderness ratio for the compression flange, $\lambda_f = 11.951$

l

Limiting slendernesss ratio for a compact flange, $\lambda_{pf} := 0.38 \cdot \sqrt{\dfrac{E}{F_y}}$

Limiting slenderness ratio for a non-compact flange, $\lambda_{rf} := 0.95 \cdot \sqrt{\dfrac{(0.76 \cdot E)}{F_y}}$

*See CA6.3.2 for language regarding kc (flange local buckling

coeffiecient) value of 0.76 for rolled shapes

$$M_{n1} := \begin{cases} \text{if } \lambda_f > \lambda_{pf} \\ \quad \left(\left(\left(1 - \left(1 - \frac{(0.7 \cdot F_y \cdot S_x)}{Z_x \cdot F_y}\right) \cdot \frac{(\lambda_f - \lambda_{pf})}{\lambda_{rf} - \lambda_{pf}}\right) \cdot (Z_x \cdot F_y)\right)\right) \\ \text{if } \lambda_f \le \lambda_{pf} \\ \quad Z_x \cdot F_y \end{cases}$$

Local Buckling Resistance:

$$M_{n1} = \left(6.596 \cdot 10^3\right) \ in \cdot kip$$

A6.3.3 - Lateral Torsional Buckling Resistance

Limiting unbraced length to achieve the nominal flexural resistance under uniform bending,

(Compact unbraced length limit for a member subject to uniform major-axis bending)

$$L_p := 1.0 \cdot rts \cdot \sqrt{\frac{E}{F_y}}$$

$$L_p = 97.537 \ in$$

Limiting unbraced length to achieve the nominal onset of yielding in either flange under uniform bending with consideration of compression flange residual stress effects

(Noncompact unbraced length limit)

$$L_r := 1.95 \cdot rts \cdot \left(\frac{E}{.7 \cdot F_y}\right) \cdot \sqrt{J_{const}} \cdot \sqrt{1 + \sqrt{1 + 6.76 \cdot \left(\frac{(0.7 \cdot F_y)}{E} \cdot \frac{1}{J_{const}}\right)^2}}$$

$$L_r = \left(1.753 \cdot 10^4\right) \ in$$

$$1 + 6.76 \cdot \left(\frac{(0.7 \cdot F_y)}{E} \cdot \frac{1}{J_{const}}\right.$$

$$M_{n2} := \begin{cases} \text{if } L_p < L_b < L_r \\ \quad \left(1.0 \cdot \left(1 - \left(1 - \frac{(0.7 \cdot F_y \cdot S_x)}{Z_x \cdot F_y}\right) \cdot \left(\frac{(L_b - L_p)}{L_r - L_p}\right)\right) \cdot (Z_x \cdot F_y)\right) \\ \text{if } L_b < L_p \\ \quad Z_x \cdot F_y \end{cases}$$

$$M_{n2} = \left(7.285 \cdot 10^3\right) \ in \cdot kip$$

*If Segment is RED, Lb>Lr, See Section A6.3.3, Mn = Fcr*Sxc < Rpc*Myc*

M.n2 should be less than plastic moment, or else plastic moment should control --->

should control

$$M_{n.2} := \begin{array}{|l} \text{if } M_{n2} \le Z_x \cdot F_y \\ \quad \| M_{n2} \\ \text{if } Z_x \cdot F_y < M_{n2} \\ \quad \| Z_x \cdot F_y \end{array}$$

:

$$M_{px} := Z_x \cdot F_y$$

$$M_{px} = \left(7.3 \cdot 10^3\right) \; \textit{in·kip}$$

Lateral Torsional Buckling Resistance:

$$M_{n.2} = \left(7.285 \cdot 10^3\right) \; \textit{in·kip}$$

$$M_{nc} := \begin{array}{|l} \text{if } M_{n1} < M_{n.2} \\ \quad \| M_{n1} \\ \text{if } M_{n.2} < M_{n1} \\ \quad \| M_{n.2} \end{array}$$

(To determine if local or buckling resistance controls)

$$M_{nc} = \left(6.596 \cdot 10^3\right) \; \textit{in·kip}$$

Factored flexural resistance based on the compression flange:

$$M_{rx} := 1.0 \cdot M_{nc}$$

$$M_{rx} = \left(6.596 \cdot 10^3\right) \; \textit{in·kip}$$

6.12.2 Nominal Flexural Resistance

6.12.2.2 Noncomposite Members, 6.12.2.2.1 applies to I and H shaped members, the nominal flexural resistance for flexure about the weak axis is:

$$M_{n3} := \begin{array}{|l} \text{if } \lambda_{pf} < \lambda_f \le \lambda_{rf} \\ \quad \left\| \left(\left(1 - \left(1 - \dfrac{S_y}{Z_y} \right) \cdot \left(\dfrac{\left(\lambda_f - \lambda_{pf}\right)}{0.45 \cdot \sqrt{\dfrac{E}{F_y}}} \right) \right) \cdot \left(F_y \cdot Z_y\right) \right) \right. \\ \text{if } \lambda_f \le \lambda_{pf} \\ \quad \| \left(Z_y \cdot F_y\right) \end{array}$$

$$M_{n3} = \left(3.083 \cdot 10^3\right) \; \textit{in·kip}$$

$$Z_y \cdot F_y = \left(3.385 \cdot 10^3\right) \; \textit{in·kip}$$

Factored Flexural Resistance:

$$M_{ry} := 1.0 \cdot M_{n3}$$

$$M_{ry} = \left(3.083 \cdot 10^3\right) \; \textit{in·kip}$$

6.9.2.2 - Combined Axial Compression and Flexure

The axial compressive load (Pu), and the concurrent moments (Mux & Muy), calculated for the factored loadings by elastic analytical procedures shall satisfy the relationship specified below:

$$Interaction := \begin{array}{l} \text{if } \dfrac{P_u}{P_r} \geq 0.2 \\ \quad \left(\left(\dfrac{P_u}{P_r} \right) + \left(\dfrac{8.0}{9.0} \cdot \left(\dfrac{M_{ux}}{M_{rx}} + \dfrac{M_{uy}}{M_{ry}} \right) \right) \right) \\ \text{if } \dfrac{P_u}{P_r} < 0.2 \\ \quad \left(\left(\dfrac{P_u}{2 \cdot P_r} \right) + \left(\dfrac{M_{ux}}{M_{rx}} + \dfrac{M_{uy}}{M_{ry}} \right) \right) \end{array}$$

$$Interaction = 0.582$$

Interaction must be less than 1.0 to satisfy design requirements!

<u>Structural Pile Check</u> - 2010 AASHTO LRFD Bridge Designs Specifications

VTrans Soils & Foundations Unit - January 2011

6.15 - PILES: Piles shall be designed as structural members capable of safely supporting all imposed loads.

6.15.3 Compressive Resistance

6.15.3.1 - <u>Axial Compression</u>: For piles under axial load, the factored resistance of pile in compression, Pr, shall be taken as specified in 6.9.2.1 using the resistance factors in 6.5.4.2.

6.15.3.2 - <u>Combined Axial Compression and Flexure</u>: Pile subjected to axial load and flexure shall be designed in accordance with 6.9.2.2 using the resistance factors in 6.5.4.2.

6.15.3.3 - <u>Buckling</u>: Instability of piles which extend through water or air shall be accounted for and specified in 6.9. Pile which extend through water or air shall be assumed fixed at some depth below the ground. Stability shall be determined in accordance with 6.9 for compression members using an equivalent length of pile equal to the laterally unsupported length, plus an embedded depth to fixity.

<u>How the MathCad Sheet Works:</u>

1. Perform an analysis in FB-Pier, extract both moments, axial load, unbraced length and depth to fixity from largest moment diagram versus depth.
2. Enter the largest moments and axial loads, along with unbraced length and depth to fixity below to attain the 'worst case scenario' pile interaction. To hone in on the interaction coefficient of a specific pile; enter moments, axial, and lengths specific to that pile.

(Fill in GREEN Boxes and Use List Box to Choose Pile Size!)

<u>INPUT VARIABLES:</u> *ENGLISH UNITS*

FB-MultiPier Input *(IN GREEN)*:

Factored flexural moment about the x-axis, $M_{ux} := 1559.9 \cdot kip \cdot in$

Factored flexural moment about the y-axis, $M_{uy} := 103.1 \cdot kip \cdot in$

Factored compressive load, $P_u := 251.12 \cdot kip$

Unbraced Length, $L_b := 178.1 \cdot in$

Be Aware of Units!

(Distance between two points of inflection on moment diagram)

Depth to Fixity, $l := 240.2 \cdot in$

(Distance between top of pile and point of negligible moment and deflection in the pile)

Choose Pile Size:

$Pile :=$ HP 14x89

Design Considerations:

Pile Head Condition:

$k := 1.2$

(See Section 4.6.2.5 for effective length factors for various conditions)

Grade of Steel:

$E := 29000 \cdot ksi$

$F_y := 50 \cdot ksi$

Note: Must change these values if not Grade 50 Steel

Pile Size Inputs:

$table_{pile} :=$

"HP 12x84"	12.3	0.685	0.685	12.3	1.375	24.6	34.6	53.2	2.94	106	120	5.14	4.24	3.4
"HP 12x74"	12.2	0.61	0.605	12.2	1.3125	21.8	30.4	46.6	2.92	93.8	105	5.11	2.98	3.3
"HP 12x63"	12.1	0.515	0.515	12.1	1.25	18.4	25.3	38.7	2.88	79.1	88.3	5.06	1.83	3.3
"HP 14x117"	14.9	0.805	0.805	14.9	1.5	34.4	59.5	91.4	3.59	172	194	5.96	8.02	4.1
"HP 14x102"	14.8	0.705	0.705	14.8	1.375	30.1	51.4	78.8	3.56	150	169	5.92	5.39	4.
"HP 14x89"	14.7	0.615	0.615	14.7	1.3125	26.1	44.3	67.7	3.53	131	146	5.88	3.59	4.0

$b_f := \text{vlookup}(Pile, table_{pile}, 1)_0 \cdot in$

$t_f := \text{vlookup}(Pile, table_{pile}, 2)_0 \cdot in$

$t_w := \text{vlookup}(Pile, table$

$d := \text{vlookup}(Pile, table_{pile}, 4)_0 \cdot in$

$ko := \text{vlookup}(Pile, table_{pile}, 5)_0 \cdot in$

$A_s := \text{vlookup}(Pile, table$

$S_y := \text{vlookup}(Pile, table_{pile}, 7)_0 \cdot in^3$

$Z_y := \text{vlookup}(Pile, table_{pile}, 8)_0 \cdot in^3$

$ry := \text{vlookup}(Pile, table$

$S_x := \text{vlookup}(Pile, table_{pile}, 10)_0 \cdot in^3$

$Z_x := \text{vlookup}(Pile, table_{pile}, 11)_0 \cdot in^3$

$rx := \text{vlookup}(Pile, table$

Selected Pile Properties:

$b_f = 14.7\ in$ $\quad t_f = 0.615\ in$ $\quad t_w = 0.615\ in$ $\quad d = 14.7\ in$ $\quad ko = 1.313\ in$ $\quad A_s = 26.1\ in^2$

$S_y = 44.3\ in^3$ $\quad Z_y = 67.7\ in^3$ $\quad ry = 3.53\ in$ $\quad J_{const} = 3.59$

$S_x = 131\ in^3$ $\quad Z_x = 146\ in^3$ $\quad rx = 5.88\ in$ $\quad rts = 4.05\ in$

$D_c := \frac{d}{2}$ Dc = dep

$t := d - 2 \cdot ko = 12.075\ in$ $\quad \lambda_f := \frac{b_f}{2 \cdot t_f} = 11.951$ $\quad \lambda w := \frac{t}{t_w} = 19.634$

6.15.2 - Structural Resistance Factors:

The specified resistance ϕc a factors for axial resistance of 0.50 to 0.70 for piles in compression without bending shall be applied only to the axial capacity of the pile. The ϕc factors of 0.70 and 0.80 and the ϕf factor 1.0 shall be applied to the combined axial and flexural resistance of the pile in the interaction equation for the compression and flexure terms, respectively. See Section C6.15.2 for further discussion.

Notes:

(Combined axial & flexural)

$\phi_c := 0.70$

$\phi_f := 1.0$

(Axial Only)

$\phi_{ca} := 0.60$

- Flexure occurs primarily toward the head of the pile. The upper zone of the pile is less likely to experience damage during driving.
- 0.60 between severe and good driving conditions; VTrans uses 0.65 if dynamic pile testing planned

6.9.4.1 - Nominal Compressive Resistance

Calculate the elastic critical buckling strength (Pe) and the equivalent nominal yield resistance (Pe).. The variables in these equations differ between the 2010 version of AASHTO and all editions prior. The 2010 edition uses different variables to allow for more convenient calculation of the nominal resistance for members subject to buckling modes in addition to, or other than, flexural buckling, and to allow for the consideration of compression members with slender elements (C6.9.4.1.1).
* The depth to fixity is used rather than the unbraced length because when calculating axial resistance, the length between points of fixity on the column should be considered.

$$P_e := \frac{\pi^2 \cdot E}{\left(\frac{k \cdot l}{\ }\right)^2} \cdot A_s$$

$\left(r_y \right)$ $P_e = \left(1.12 \cdot 10^3\right)$ ***kip***

Slender Element Reduction (6.9.4.2): $Q := 1.0$

$P_o := Q \cdot F_y \cdot A_s$

$P_o = \left(1.305 \cdot 10^3\right)$ ***kip***

$$P_n := \begin{Vmatrix} \text{if } \dfrac{P_e}{P_o} \geq 0.44 \\ \quad \left\| \left(0.658^{\frac{P_o}{P_e}}\right) \cdot P_o \right. \\ \text{if } \dfrac{P_e}{P_o} < 0.44 \\ \quad \left\| 0.877 \cdot P_e \right. \end{Vmatrix}$$

$\left(\dfrac{P_o}{P_e}\right) = 1.165$

$P_n = 801.475$ ***kip***

Factored Compressive Resistance:

$P_r := \phi_c \cdot P_n$ $P_r = 561.032$ ***kip***

(ϕc) is used here because upper zone of pile controls)

A6.3 - Flexural Resistance Based on the Compression Flange

For unbraced lengths in which the member is prismatic, the nominal flexural resistance based on the compression flange, Mnc, shall be taken as the smaller of the local buckling resistance determined in A6.3.2 (Mn1), and the lateral torsional buckling resistance detemined in A6.3.3 (Mn2).

A6.3.2 - Local Buckling Resistance

Slenderness ratio for the compression flange, $\lambda_f = 11.951$

l

Limiting slendernesss ratio for a compact flange, $\lambda_{pf} := 0.38 \cdot \sqrt{\dfrac{E}{F_y}}$

Limiting slenderness ratio for a non-compact flange, $\lambda_{rf} := 0.95 \cdot \sqrt{\dfrac{(0.76 \cdot E)}{F_y}}$

*See CA6.3.2 for language regarding kc (flange local buckling

coeffiecient) value of 0.76 for rolled shapes

$$M_{n1} := \begin{cases} \text{if } \lambda_f > \lambda_{pf} \\ \quad \left(\left(\left(1 - \left(1 - \frac{(0.7 \cdot F_y \cdot S_x)}{Z_x \cdot F_y}\right) \cdot \frac{(\lambda_f - \lambda_{pf})}{\lambda_{rf} - \lambda_{pf}}\right) \cdot (Z_x \cdot F_y)\right)\right) \\ \text{if } \lambda_f \le \lambda_{pf} \\ \quad Z_x \cdot F_y \end{cases}$$

Local Buckling Resistance:

$$M_{n1} = (6.596 \cdot 10^3)\ in \cdot kip$$

A6.3.3 - Lateral Torsional Buckling Resistance

Limiting unbraced length to achieve the nominal flexural resistance under uniform bending,

(Compact unbraced length limit for a member subject to uniform major-axis bending)

$$L_p := 1.0 \cdot rts \cdot \sqrt{\frac{E}{F_y}} \qquad L_p = 97.537\ in$$

Limiting unbraced length to achieve the nominal onset of yielding in either flange under uniform bending with consideration of compression flange residual stress effects

(Noncompact unbraced length limit)

$$L_r := 1.95 \cdot rts \cdot \left(\frac{E}{.7 \cdot F_y}\right) \cdot \sqrt{J_{const}} \cdot \sqrt{1 + \sqrt{1 + 6.76 \cdot \left(\frac{(0.7 \cdot F_y)}{E} \cdot \frac{1}{J_{const}}\right)^2}} \qquad L_r = (1.753 \cdot 10^4)\ in$$

$$1 + 6.76 \cdot \left(\frac{(0.7 \cdot F_y)}{E} \cdot \frac{1}{J_{const}}\right.$$

$$M_{n2} := \begin{cases} \text{if } L_p < L_b < L_r \\ \quad \left(1.0 \cdot \left(1 - \left(1 - \frac{(0.7 \cdot F_y \cdot S_x)}{Z_x \cdot F_y}\right) \cdot \left(\frac{(L_b - L_p)}{L_r - L_p}\right)\right) \cdot (Z_x \cdot F_y)\right) \\ \text{if } L_b < L_p \\ \quad Z_x \cdot F_y \end{cases}$$

$$M_{n2} = (7.287 \cdot 10^3)\ in \cdot kip$$

*If Segment is RED, Lb>Lr, See Section A6.3.3, Mn = Fcr*Sxc < Rpc*Myc*

M.n2 should be less than plastic moment, or else plastic moment should control --->

should control :

$$M_{n.2} := \begin{cases} M_{n2} & \text{if } M_{n2} \le Z_x \cdot F_y \\ Z_x \cdot F_y & \text{if } Z_x \cdot F_y < M_{n2} \end{cases}$$

$$M_{px} := Z_x \cdot F_y$$

$$M_{px} = \left(7.3 \cdot 10^3\right) \; \textit{in·kip}$$

Lateral Torsional Buckling Resistance:

$$M_{n.2} = \left(7.287 \cdot 10^3\right) \; \textit{in·kip}$$

$$M_{nc} := \begin{cases} M_{n1} & \text{if } M_{n1} < M_{n.2} \\ M_{n.2} & \text{if } M_{n.2} < M_{n1} \end{cases}$$

(To determine if local or buckling resistance controls)

$$M_{nc} = \left(6.596 \cdot 10^3\right) \; \textit{in·kip}$$

Factored flexural resistance based on the compression flange:

$$M_{rx} := 1.0 \cdot M_{nc}$$

$$M_{rx} = \left(6.596 \cdot 10^3\right) \; \textit{in·kip}$$

6.12.2 Nominal Flexural Resistance

6.12.2.2 Noncomposite Members, 6.12.2.2.1 applies to I and H shaped members, the nominal flexural resistance for flexure about the weak axis is:

$$M_{n3} := \begin{cases} \left(\left(1 - \left(1 - \dfrac{S_y}{Z_y}\right) \cdot \left(\dfrac{\left(\lambda_f - \lambda_{pf}\right)}{0.45 \cdot \sqrt{\dfrac{E}{F_y}}}\right)\right) \cdot \left(F_y \cdot Z_y\right)\right) & \text{if } \lambda_{pf} < \lambda_f \le \lambda_{rf} \\ \left(Z_y \cdot F_y\right) & \text{if } \lambda_f \le \lambda_{pf} \end{cases}$$

$$M_{n3} = \left(3.083 \cdot 10^3\right) \; \textit{in·kip}$$

$$Z_y \cdot F_y = \left(3.385 \cdot 10^3\right) \; \textit{in·kip}$$

Factored Flexural Resistance:

$$M_{ry} := 1.0 \cdot M_{n3}$$

$$M_{ry} = \left(3.083 \cdot 10^3\right) \; \textit{in·kip}$$

6.9.2.2 - Combined Axial Compression and Flexure

The axial compressive load (Pu), and the concurrent moments (Mux & Muy), calculated for the factored loadings by elastic analytical procedures shall satisfy the relationship specified below:

$$Interaction := \begin{cases} \text{if } \dfrac{P_u}{P_r} \geq 0.2 \\ \quad \left(\left(\dfrac{P_u}{P_r} \right) + \left(\dfrac{8.0}{9.0} \cdot \left(\dfrac{M_{ux}}{M_{rx}} + \dfrac{M_{uy}}{M_{ry}} \right) \right) \right) \\ \text{if } \dfrac{P_u}{P_r} < 0.2 \\ \quad \left(\left(\dfrac{P_u}{2 \cdot P_r} \right) + \left(\dfrac{M_{ux}}{M_{rx}} + \dfrac{M_{uy}}{M_{ry}} \right) \right) \end{cases}$$

$Interaction = 0.688$

Interaction must be less than 1.0 to satisfy design requirements!

Structural Pile Check - 2010 AASHTO LRFD Bridge Designs Specifications
VTrans Soils & Foundations Unit - January 2011

6.15 - PILES: Piles shall be designed as structural members capable of safely supporting all imposed loads.

6.15.3 Compressive Resistance

6.15.3.1 - Axial Compression: For piles under axial load, the factored resistance of pile in compression, Pr, shall be taken as specified in 6.9.2.1 using the resistance factors in 6.5.4.2.

6.15.3.2 - Combined Axial Compression and Flexure: Pile subjected to axial load and flexure shall be designed in accordance with 6.9.2.2 using the resistance factors in 6.5.4.2.

6.15.3.3 - Buckling: Instability of piles which extend through water or air shall be accounted for and specified in 6.9. Pile which extend through water or air shall be assumed fixed at some depth below the ground. Stability shall be determined in accordance with 6.9 for compression members using an equivalent length of pile equal to the laterally unsupported length, plus an embedded depth to fixity.

How the MathCad Sheet Works:

1. Perform an analysis in FB-Pier, extract both moments, axial load, unbraced length and depth to fixity from largest moment diagram versus depth.
2. Enter the largest moments and axial loads, along with unbraced length and depth to fixity below to attain the 'worst case scenario' pile interaction. To hone in on the interaction coefficient of a specific pile; enter moments, axial, and lengths specific to that pile.

(Fill in GREEN Boxes and Use List Box to Choose Pile Size!)

INPUT VARIABLES: *ENGLISH UNITS*

FB-MultiPier Input *(IN GREEN)*:

Factored flexural moment about the x-axis, $M_{ux} := 1256.6 \cdot kip \cdot in$

Factored flexural moment about the y-axis, $M_{uy} := 34.2 \cdot kip \cdot in$

Factored compressive load, $P_u := 171.34 \cdot kip$

Unbraced Length, $L_b := 185.4 \cdot in$

Be Aware of Units!

(Distance between two points of inflection on moment diagram)

Depth to Fixity, $l := 192.0 \cdot in$

(Distance between top of pile and point of negligible moment and deflection in the pile)

Choose Pile Size:

$Pile :=$ HP 14x89

Design Considerations:

Pile Head Condition:

$k := 1.2$

(See Section 4.6.2.5 for effective length factors for various conditions)

Grade of Steel:

$E := 29000 \cdot ksi$

$F_y := 50 \cdot ksi$

Note: Must change these values if not Grade 50 Steel

Pile Size Inputs:

$table_{pile} :=$

"HP 12x84"	12.3	0.685	0.685	12.3	1.375	24.6	34.6	53.2	2.94	106	120	5.14	4.24	3.4
"HP 12x74"	12.2	0.61	0.605	12.2	1.3125	21.8	30.4	46.6	2.92	93.8	105	5.11	2.98	3.3
"HP 12x63"	12.1	0.515	0.515	12.1	1.25	18.4	25.3	38.7	2.88	79.1	88.3	5.06	1.83	3.3
"HP 14x117"	14.9	0.805	0.805	14.9	1.5	34.4	59.5	91.4	3.59	172	194	5.96	8.02	4.1
"HP 14x102"	14.8	0.705	0.705	14.8	1.375	30.1	51.4	78.8	3.56	150	169	5.92	5.39	4.
"HP 14x89"	14.7	0.615	0.615	14.7	1.3125	26.1	44.3	67.7	3.53	131	146	5.88	3.59	4.0

$b_f := \text{vlookup}\left(Pile, table_{pile}, 1\right)_0 \cdot in$ $\quad t_f := \text{vlookup}\left(Pile, table_{pile}, 2\right)_0 \cdot in$ $\quad t_w := \text{vlookup}\left(Pile, table_{pile}\right.$

$d := \text{vlookup}\left(Pile, table_{pile}, 4\right)_0 \cdot in$ $\quad ko := \text{vlookup}\left(Pile, table_{pile}, 5\right)_0 \cdot in$ $\quad A_s := \text{vlookup}\left(Pile, table\right.$

$S_y := \text{vlookup}\left(Pile, table_{pile}, 7\right)_0 \cdot in^3$ $\quad Z_y := \text{vlookup}\left(Pile, table_{pile}, 8\right)_0 \cdot in^3$ $\quad ry := \text{vlookup}\left(Pile, table\right.$

$S_x := \text{vlookup}\left(Pile, table_{pile}, 10\right)_0 \cdot in^3$ $\quad Z_x := \text{vlookup}\left(Pile, table_{pile}, 11\right)_0 \cdot in^3$ $\quad rx := \text{vlookup}\left(Pile, table\right.$

Selected Pile Properties:

$b_f = 14.7\ in$ $t_f = 0.615\ in$ $t_w = 0.615\ in$ $d = 14.7\ in$ $ko = 1.313\ in$ $A_s = 26.1\ in^2$

$S_y = 44.3\ in^3$ $Z_y = 67.7\ in^3$ $ry = 3.53\ in$ $J_{const} = 3.59$ $D_c := \frac{d}{2}$ Dc = dep

$S_x = 131\ in^3$ $Z_x = 146\ in^3$ $rx = 5.88\ in$ $rts = 4.05\ in$

$t := d - 2 \cdot ko = 12.075\ in$ $\lambda_f := \frac{b_f}{2 \cdot t_f} = 11.951$ $\lambda w := \frac{t}{t_w} = 19.634$

6.15.2 - Structural Resistance Factors:

The specified resistance ϕc a factors for axial resistance of 0.50 to 0.70 for piles in compression without bending shall be applied only to the axial capacity of the pile. The ϕc factors of 0.70 and 0.80 and the ϕf factor 1.0 shall be applied to the combined axial and flexural resistance of the pile in the interaction equation for the compression and flexure terms, respectively. See Section C6.15.2 for further discussion.

Notes:

(Combined axial & flexural) $\phi_c := 0.70$

$\phi_f := 1.0$

(Axial Only) $\phi_{ca} := 0.60$

- Flexure occurs primarily toward the head of the pile. The upper zone of the pile is less likely to experience damage during driving.
- 0.60 between severe and good driving conditions; VTrans uses 0.65 if dynamic pile testing planned

6.9.4.1 - Nominal Compressive Resistance

Calculate the elastic critical buckling strength (Pe) and the equivalent nominal yield resistance (Pe).. The variables in these equations differ between the 2010 version of AASHTO and all editions prior. The 2010 edition uses different variables to allow for more convenient calculation of the nominal resistance for members subject to buckling modes in addition to, or other than, flexural buckling, and to allow for the consideration of compression members with slender elements (C6.9.4.1.1).

* The depth to fixity is used rather than the unbraced length because when calculating axial resistance, the length between points of fixity on the column should be considered.

$$P_e := \frac{\pi^2 \cdot E}{\left(\frac{k \cdot l}{\ }\right)^2} \cdot A_s$$

$\left(ry \right)$ $P_e = \left(1.754 \cdot 10^3\right)$ ***kip***

Slender Element Reduction (6.9.4.2): $Q := 1.0$

$P_o := Q \cdot F_y \cdot A_s$

$P_o = \left(1.305 \cdot 10^3\right)$ ***kip***

$$P_n := \begin{Vmatrix} \text{if } \dfrac{P_e}{P_o} \geq 0.44 \\ \quad \begin{Vmatrix} \left(0.658^{\frac{P_o}{P_e}}\right) \cdot P_o \end{Vmatrix} \\ \text{if } \dfrac{P_e}{P_o} < 0.44 \\ \quad \begin{Vmatrix} 0.877 \cdot P_e \end{Vmatrix} \end{Vmatrix}$$

$\left(\dfrac{P_o}{P_e}\right) = 0.744$

$P_n = 955.729$ ***kip***

Factored Compressive Resistance:

$P_r := \phi_c \cdot P_n$ $P_r = 669.01$ ***kip***

(ϕc) is used here because upper zone of pile controls)

A6.3 - Flexural Resistance Based on the Compression Flange

For unbraced lengths in which the member is prismatic, the nominal flexural resistance based on the compression flange, Mnc, shall be taken as the smaller of the local buckling resistance determined in A6.3.2 (Mn1), and the lateral torsional buckling resistance detemined in A6.3.3 (Mn2).

A6.3.2 - Local Buckling Resistance

Slenderness ratio for the compression flange, $\lambda_f = 11.951$ l

Limiting slendernesss ratio for a compact flange, $\lambda_{pf} := 0.38 \cdot \sqrt{\dfrac{E}{F_y}}$

Limiting slenderness ratio for a non-compact flange, $\lambda_{rf} := 0.95 \cdot \sqrt{\dfrac{(0.76 \cdot E)}{F_y}}$

*See CA6.3.2 for language regarding kc (flange local buckling

coeffiecient) value of 0.76 for rolled shapes

$$M_{n1} := \begin{array}{l} \text{if } \lambda_f > \lambda_{pf} \\ \quad \left(\left(\left(1 - \left(1 - \frac{(0.7 \cdot F_y \cdot S_x)}{Z_x \cdot F_y}\right) \cdot \frac{(\lambda_f - \lambda_{pf})}{\lambda_{rf} - \lambda_{pf}}\right) \cdot (Z_x \cdot F_y)\right)\right) \\ \text{if } \lambda_f \le \lambda_{pf} \\ \quad Z_x \cdot F_y \end{array}$$

Local Buckling Resistance:

$$M_{n1} = (6.596 \cdot 10^3) \ \boldsymbol{in \cdot kip}$$

A6.3.3 - Lateral Torsional Buckling Resistance

Limiting unbraced length to achieve the nominal flexural resistance under uniform bending,

(Compact unbraced length limit for a member subject to uniform major-axis bending)

$$L_p := 1.0 \cdot rts \cdot \sqrt{\frac{E}{F_y}} \qquad L_p = 97.537 \ \boldsymbol{in}$$

Limiting unbraced length to achieve the nominal onset of yielding in either flange under uniform bending with consideration of compression flange residual stress effects

(Noncompact unbraced length limit)

$$L_r := 1.95 \cdot rts \cdot \left(\frac{E}{.7 \cdot F_y}\right) \cdot \sqrt{J_{const}} \cdot \sqrt{1 + \sqrt{1 + 6.76 \cdot \left(\frac{(0.7 \cdot F_y)}{E} \cdot \frac{1}{J_{const}}\right)^2}} \qquad L_r = (1.753 \cdot 10^4) \ \boldsymbol{in}$$

$$1 + 6.76 \cdot \left(\frac{(0.7 \cdot F_y)}{E} \cdot \frac{1}{J_{const}}\right)$$

$$M_{n2} := \begin{array}{l} \text{if } L_p < L_b < L_r \\ \quad \left(1.0 \cdot \left(1 - \left(1 - \frac{(0.7 \cdot F_y \cdot S_x)}{Z_x \cdot F_y}\right) \cdot \left(\frac{(L_b - L_p)}{L_r - L_p}\right)\right) \cdot (Z_x \cdot F_y)\right) \\ \text{if } L_b < L_p \\ \quad Z_x \cdot F_y \end{array}$$

$$M_{n2} = (7.286 \cdot 10^3) \ \boldsymbol{in \cdot kip}$$

*If Segment is RED, Lb>Lr, See Section A6.3.3, Mn = Fcr*Sxc < Rpc*Myc*

M.n2 should be less than plastic moment, or else plastic moment should control --->

should control

:

$$M_{n.2} := \begin{cases} M_{n2} & \text{if } M_{n2} \le Z_x \cdot F_y \\ Z_x \cdot F_y & \text{if } Z_x \cdot F_y < M_{n2} \end{cases}$$

$$M_{px} := Z_x \cdot F_y$$

$$M_{px} = \left(7.3 \cdot 10^3\right) \ in \cdot kip$$

Lateral Torsional Buckling Resistance:

$$M_{n.2} = \left(7.286 \cdot 10^3\right) \ in \cdot kip$$

$$M_{nc} := \begin{cases} M_{n1} & \text{if } M_{n1} < M_{n.2} \\ M_{n.2} & \text{if } M_{n.2} < M_{n1} \end{cases}$$

(To determine if local or buckling resistance controls)

$$M_{nc} = \left(6.596 \cdot 10^3\right) \ in \cdot kip$$

Factored flexural resistance based on the compression flange:

$$M_{rx} := 1.0 \cdot M_{nc}$$

$$M_{rx} = \left(6.596 \cdot 10^3\right) \ in \cdot kip$$

6.12.2 Nominal Flexural Resistance

6.12.2.2 Noncomposite Members, 6.12.2.2.1 applies to I and H shaped members, the nominal flexural resistance for flexure about the weak axis is:

$$M_{n3} := \begin{cases} \left(\left(1 - \left(1 - \dfrac{S_y}{Z_y}\right) \cdot \left(\dfrac{\left(\lambda_f - \lambda_{pf}\right)}{0.45 \cdot \sqrt{\dfrac{E}{F_y}}}\right)\right) \cdot \left(F_y \cdot Z_y\right)\right) & \text{if } \lambda_{pf} < \lambda_f \le \lambda_{rf} \\ \left(Z_y \cdot F_y\right) & \text{if } \lambda_f \le \lambda_{pf} \end{cases}$$

$$M_{n3} = \left(3.083 \cdot 10^3\right) \ in \cdot kip$$

$$Z_y \cdot F_y = \left(3.385 \cdot 10^3\right) \ in \cdot kip$$

Factored Flexural Resistance:

$$M_{ry} := 1.0 \cdot M_{n3}$$

$$M_{ry} = \left(3.083 \cdot 10^3\right) \ in \cdot kip$$

6.9.2.2 - Combined Axial Compression and Flexure

The axial compressive load (Pu), and the concurrent moments (Mux & Muy), calculated for the factored loadings by elastic analytical procedures shall satisfy the relationship specified below:

$$Interaction := \begin{array}{l} \text{if } \dfrac{P_u}{P_r} \geq 0.2 \\ \quad \left(\left(\dfrac{P_u}{P_r} \right) + \left(\dfrac{8.0}{9.0} \cdot \left(\dfrac{M_{ux}}{M_{rx}} + \dfrac{M_{uy}}{M_{ry}} \right) \right) \right) \\ \text{if } \dfrac{P_u}{P_r} < 0.2 \\ \quad \left(\left(\dfrac{P_u}{2 \cdot P_r} \right) + \left(\dfrac{M_{ux}}{M_{rx}} + \dfrac{M_{uy}}{M_{ry}} \right) \right) \end{array}$$

$$Interaction = 0.435$$

Interaction must be less than 1.0 to satisfy design requirements!

<u>Structural Pile Check</u> - 2010 AASHTO LRFD Bridge Designs Specifications
VTrans Soils & Foundations Unit - January 2011

6.15 - PILES: Piles shall be designed as structural members capable of safely supporting all imposed loads.

6.15.3 Compressive Resistance

6.15.3.1 - <u>Axial Compression</u>: For piles under axial load, the factored resistance of pile in compression, Pr, shall be taken as specified in 6.9.2.1 using the resistance factors in 6.5.4.2.

6.15.3.2 - <u>Combined Axial Compression and Flexure</u>: Pile subjected to axial load and flexure shall be designed in accordance with 6.9.2.2 using the resistance factors in 6.5.4.2.

6.15.3.3 - <u>Buckling</u>: Instability of piles which extend through water or air shall be accounted for and specified in 6.9. Pile which extend through water or air shall be assumed fixed at some depth below the ground. Stability shall be determined in accordance with 6.9 for compression members using an equivalent length of pile equal to the laterally unsupported length, plus an embedded depth to fixity.

<u>How the MathCad Sheet Works:</u>

1. Perform an analysis in FB-Pier, extract both moments, axial load, unbraced length and depth to fixity from largest moment diagram versus depth.
2. Enter the largest moments and axial loads, along with unbraced length and depth to fixity below to attain the 'worst case scenario' pile interaction. To hone in on the interaction coefficient of a specific pile; enter moments, axial, and lengths specific to that pile.

(Fill in GREEN Boxes and Use List Box to Choose Pile Size!)

<u>INPUT VARIABLES:</u> *ENGLISH UNITS*

FB-MultiPier Input *(IN GREEN)*:

Factored flexural moment about the x-axis, $M_{ux} := 2039.9 \cdot kip \cdot in$

Factored flexural moment about the y-axis, $M_{uy} := 50.9 \cdot kip \cdot in$

Factored compressive load, $P_u := 184.81 \cdot kip$

Unbraced Length, $L_b := 171.5 \cdot in$

Be Aware of Units!

(Distance between two points of
inflection on moment diagram)

Depth to Fixity, $l := 192 \cdot in$

(Distance between top of pile and point of negligible
moment and deflection in the pile)

Choose Pile Size:

$Pile :=$ HP 14x89

Design Considerations:

Pile Head Condition:

$k := 1.2$

(See Section 4.6.2.5 for
effective length factors
for various conditions)

Grade of Steel:

$E := 29000 \cdot ksi$

$F_y := 50 \cdot ksi$

*Note: Must change these
values if not Grade 50 Steel*

Pile Size Inputs:

$table_{pile} :=$

"HP 12x84"	12.3	0.685	0.685	12.3	1.375	24.6	34.6	53.2	2.94	106	120	5.14	4.24	3.4
"HP 12x74"	12.2	0.61	0.605	12.2	1.3125	21.8	30.4	46.6	2.92	93.8	105	5.11	2.98	3.3
"HP 12x63"	12.1	0.515	0.515	12.1	1.25	18.4	25.3	38.7	2.88	79.1	88.3	5.06	1.83	3.3
"HP 14x117"	14.9	0.805	0.805	14.9	1.5	34.4	59.5	91.4	3.59	172	194	5.96	8.02	4.1
"HP 14x102"	14.8	0.705	0.705	14.8	1.375	30.1	51.4	78.8	3.56	150	169	5.92	5.39	4.
"HP 14x89"	14.7	0.615	0.615	14.7	1.3125	26.1	44.3	67.7	3.53	131	146	5.88	3.59	4.0

$b_f := \text{vlookup}\left(Pile, table_{pile}, 1\right)_0 \cdot in$ $\quad t_f := \text{vlookup}\left(Pile, table_{pile}, 2\right)_0 \cdot in$ $\quad t_w := \text{vlookup}\left(Pile, table_{p}\right.$

$d := \text{vlookup}\left(Pile, table_{pile}, 4\right)_0 \cdot in$ $\quad ko := \text{vlookup}\left(Pile, table_{pile}, 5\right)_0 \cdot in$ $\quad A_s := \text{vlookup}\left(Pile, table\right.$

$S_y := \text{vlookup}\left(Pile, table_{pile}, 7\right)_0 \cdot in^3$ $\quad Z_y := \text{vlookup}\left(Pile, table_{pile}, 8\right)_0 \cdot in^3$ $\quad ry := \text{vlookup}\left(Pile, table\right.$

$S_x := \text{vlookup}\left(Pile, table_{pile}, 10\right)_0 \cdot in^3$ $\quad Z_x := \text{vlookup}\left(Pile, table_{pile}, 11\right)_0 \cdot in^3$ $\quad rx := \text{vlookup}\left(Pile, table\right.$

Selected Pile Properties:

$b_f = 14.7\ in$ $t_f = 0.615\ in$ $t_w = 0.615\ in$ $d = 14.7\ in$ $ko = 1.313\ in$ $A_s = 26.1\ in^2$

$S_y = 44.3\ in^3$ $Z_y = 67.7\ in^3$ $ry = 3.53\ in$ $J_{const} = 3.59$

$D_c := \frac{d}{2}$ Dc = dep

$S_x = 131\ in^3$ $Z_x = 146\ in^3$ $rx = 5.88\ in$ $rts = 4.05\ in$

$t := d - 2 \cdot ko = 12.075\ in$ $\lambda_f := \frac{b_f}{2 \cdot t_f} = 11.951$ $\lambda w := \frac{t}{t_w} = 19.634$

6.15.2 - Structural Resistance Factors:

The specified resistance ϕc a factors for axial resistance of 0.50 to 0.70 for piles in compression without bending shall be applied only to the axial capacity of the pile. The ϕc factors of 0.70 and 0.80 and the ϕf factor 1.0 shall be applied to the combined axial and flexural resistance of the pile in the interaction equation for the compression and flexure terms, respectively. See Section C6.15.2 for further discussion.

Notes:

(Combined axial & flexural) $\phi_c := 0.70$

$\phi_f := 1.0$

(Axial Only) $\phi_{ca} := 0.60$

- Flexure occurs primarily toward the head of the pile. The upper zone of the pile is less likely to experience damage during driving.
- 0.60 between severe and good driving conditions; VTrans uses 0.65 if dynamic pile testing planned

6.9.4.1 - Nominal Compressive Resistance

Calculate the elastic critical buckling strength (Pe) and the equivalent nominal yield resistance (Pe).. The variables in these equations differ between the 2010 version of AASHTO and all editions prior. The 2010 edition uses different variables to allow for more convenient calculation of the nominal resistance for members subject to buckling modes in addition to, or other than, flexural buckling, and to allow for the consideration of compression members with slender elements (C6.9.4.1.1).

* The depth to fixity is used rather than the unbraced length because when calculating axial resistance, the length between points of fixity on the column should be considered.

$$P_e := \frac{\pi^2 \cdot E}{\left(\frac{k \cdot l}{\ }\right)^2} \cdot A_s$$

$\left(\frac{}{ry} \right)$ $P_e = (1.754 \cdot 10^3)$ ***kip***

Slender Element Reduction (6.9.4.2): $Q := 1.0$

$P_o := Q \cdot F_y \cdot A_s$

$P_o = (1.305 \cdot 10^3)$ ***kip***

$$P_n := \begin{cases} \left(0.658^{\frac{P_o}{P_e}}\right) \cdot P_o & \text{if } \frac{P_e}{P_o} \geq 0.44 \\ 0.877 \cdot P_e & \text{if } \frac{P_e}{P_o} < 0.44 \end{cases}$$

$\left(\frac{P_o}{P_e}\right) = 0.744$

$P_n = 955.729$ ***kip***

Factored Compressive Resistance:

$P_r := \phi_c \cdot P_n$ $P_r = 669.01$ ***kip***

(ϕc) is used here because upper zone of pile controls)

A6.3 - Flexural Resistance Based on the Compression Flange

For unbraced lengths in which the member is prismatic, the nominal flexural resistance based on the compression flange, Mnc, shall be taken as the smaller of the local buckling resistance determined in A6.3.2 (Mn1), and the lateral torsional buckling resistance detemined in A6.3.3 (Mn2).

A6.3.2 - Local Buckling Resistance

Slenderness ratio for the compression flange, $\lambda_f = 11.951$

l

Limiting slendernesss ratio for a compact flange, $\lambda_{pf} := 0.38 \cdot \sqrt{\frac{E}{F_y}}$

Limiting slenderness ratio for a non-compact flange, $\lambda_{rf} := 0.95 \cdot \sqrt{\frac{(0.76 \cdot E)}{F_y}}$

*See CA6.3.2 for language regarding kc (flange local buckling

coeffiecient) value of 0.76 for rolled shapes

$$M_{n1} := \begin{Vmatrix} \text{if } \lambda_f > \lambda_{pf} \\ \quad \left\| \left(\left(\left(1 - \left(1 - \frac{(0.7 \cdot F_y \cdot S_x)}{Z_x \cdot F_y} \right) \cdot \frac{(\lambda_f - \lambda_{pf})}{\lambda_{rf} - \lambda_{pf}} \right) \cdot (Z_x \cdot F_y) \right) \right) \right. \\ \text{if } \lambda_f \le \lambda_{pf} \\ \quad \left\| Z_x \cdot F_y \right. \end{Vmatrix}$$

Local Buckling Resistance:

$$M_{n1} = \left(6.596 \cdot 10^3\right) \; \boldsymbol{in \cdot kip}$$

A6.3.3 - Lateral Torsional Buckling Resistance

Limiting unbraced length to achieve the nominal flexural resistance under uniform bending,

(Compact unbraced length limit for a member subject to uniform major-axis bending)

$$L_p := 1.0 \cdot rts \cdot \sqrt{\frac{E}{F_y}}$$

$$L_p = 97.537 \; \boldsymbol{in}$$

Limiting unbraced length to achieve the nominal onset of yielding in either flange under uniform bending with consideration of compression flange residual stress effects

(Noncompact unbraced length limit)

$$L_r := 1.95 \cdot rts \cdot \left(\frac{E}{.7 \cdot F_y} \right) \cdot \sqrt{J_{const}} \cdot \sqrt{1 + \sqrt{1 + 6.76 \cdot \left(\frac{(0.7 \cdot F_y)}{E} \cdot \frac{1}{J_{const}} \right)^2}}$$

$$L_r = \left(1.753 \cdot 10^4\right) \; \boldsymbol{in}$$

$$1 + 6.76 \cdot \left(\frac{(0.7 \cdot F_y)}{E} \cdot \frac{1}{J_{const}} \right)$$

$$M_{n2} := \begin{Vmatrix} \text{if } L_p < L_b < L_r \\ \quad \left\| \left(1.0 \cdot \left(1 - \left(1 - \frac{(0.7 \cdot F_y \cdot S_x)}{Z_x \cdot F_y} \right) \cdot \left(\frac{(L_b - L_p)}{L_r - L_p} \right) \right) \cdot (Z_x \cdot F_y) \right) \right. \\ \text{if } L_b < L_p \\ \quad \left\| Z_x \cdot F_y \right. \end{Vmatrix}$$

$$M_{n2} = \left(7.288 \cdot 10^3\right) \; \boldsymbol{in \cdot kip}$$

*If Segment is RED, Lb>Lr, See Section A6.3.3, Mn = Fcr*Sxc < Rpc*Myc*

M.n2 should be less than plastic moment, or else plastic moment should control --->

$$M_{n.2} := \begin{cases} M_{n2} & \text{if } M_{n2} \le Z_x \cdot F_y \\ Z_x \cdot F_y & \text{if } Z_x \cdot F_y < M_{n2} \end{cases}$$

$M_{px} := Z_x \cdot F_y$

$M_{px} = \left(7.3 \cdot 10^3\right)$ ***in·kip***

Lateral Torsional Buckling Resistance:

$M_{n.2} = \left(7.288 \cdot 10^3\right)$ ***in·kip***

$$M_{nc} := \begin{cases} M_{n1} & \text{if } M_{n1} < M_{n.2} \\ M_{n.2} & \text{if } M_{n.2} < M_{n1} \end{cases}$$

(To determine if local or buckling resistance controls)

$M_{nc} = \left(6.596 \cdot 10^3\right)$ ***in·kip***

Factored flexural resistance based on the compression flange:

$M_{rx} := 1.0 \cdot M_{nc}$ $\quad$ $M_{rx} = \left(6.596 \cdot 10^3\right)$ ***in·kip***

6.12.2 Nominal Flexural Resistance

6.12.2.2 Noncomposite Members, 6.12.2.2.1 applies to I and H shaped members, the nominal flexural resistance for flexure about the weak axis is:

$$M_{n3} := \begin{cases} \left(\left(1 - \left(1 - \dfrac{S_y}{Z_y}\right) \cdot \left(\dfrac{\left(\lambda_f - \lambda_{pf}\right)}{0.45 \cdot \sqrt{\dfrac{E}{F_y}}}\right)\right) \cdot \left(F_y \cdot Z_y\right)\right) & \text{if } \lambda_{pf} < \lambda_f \le \lambda_{rf} \\ \left(Z_y \cdot F_y\right) & \text{if } \lambda_f \le \lambda_{pf} \end{cases}$$

$M_{n3} = \left(3.083 \cdot 10^3\right)$ ***in·kip*** $\quad$ $Z_y \cdot F_y = \left(3.385 \cdot 10^3\right)$ ***in·kip***

Factored Flexural Resistance:

$M_{ry} := 1.0 \cdot M_{n3}$ $\quad$ $M_{ry} = \left(3.083 \cdot 10^3\right)$ ***in·kip***

6.9.2.2 - Combined Axial Compression and Flexure

The axial compressive load (Pu), and the concurrent moments (Mux & Muy), calculated for the factored loadings by elastic analytical procedures shall satisfy the relationship specified below:

$$Interaction := \begin{Vmatrix} \text{if } \dfrac{P_u}{P_r} \geq 0.2 \\ \quad \left\| \left(\left(\dfrac{P_u}{P_r} \right) + \left(\dfrac{8.0}{9.0} \cdot \left(\dfrac{M_{ux}}{M_{rx}} + \dfrac{M_{uy}}{M_{ry}} \right) \right) \right) \right. \\ \text{if } \dfrac{P_u}{P_r} < 0.2 \\ \quad \left\| \left(\left(\dfrac{P_u}{2 \cdot P_r} \right) + \left(\dfrac{M_{ux}}{M_{rx}} + \dfrac{M_{uy}}{M_{ry}} \right) \right) \right. \end{Vmatrix}$$

$$Interaction = 0.566$$

Interaction must be less than 1.0 to satisfy design requirements!