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GMRC Bridge No. 132 Rehabilitation Project

Geotechnical Report

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State of Vermont Agency of Transportation (VTrans)

Bridge No. 132

GMRC Bridge No. 132 Rehabilitation Project

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1. Introduction

This geotechnical report presents our geotechnical recommendations for the proposed Green Mountain Railroad Corporation (GMRC) Bridge No. 132 Rehabilitation Project in Cavendish, Vermont. This report is subject to the limitations contained herein.

All elevations in this report are expressed in feet and are referenced to the North American Vertical Datum of 1988 (NAVD88). This report does not address environmental considerations.

2. Existing Conditions

Bridge No. 132 (herein referred to as Bridge 132) is located on the Green Mountain Railroad Corporation Line at Mile Post 24.31 in Cavendish, Vermont. Existing Bridge 132 is two-span skewed steel through plate girder structure constructed in 1893 to cross the Black River. 8″ x 15″ structural timber ties carry the track load directly to the two through girders. The abutments are constructed of stone masonry stems, bearing seats, backwalls and wingwalls. The pier is also stone masonry. The track alignment is tangent over the bridge, curved to the east on the south approach, and tangent on the north approach. Vermont State Route 103 crosses the track approximately 100′ from the bridge on the south approach. For purposes of this report, the track orientation is south to north, with increasing milepost to the north.

Bridge 132 has an overall girder length of 188′-7″, which includes a ½″ plate connecting each span. Each span has a total girder length of 94′-3″ and a design span of 91'-6″. The girders are spaced at 16′-1″. Each floor system span includes six interior cross frames. Bottom lateral bracing is present at the bottom portion of the girder and attaches to the girder just below the tie bearing angle. Bridge bearings are steel sole plates with steel shoes bolted to the underside of the girder bottom flange. The bridge is located within FEMA designated Zone AE – within the 100-year flood area.

The existing bridge does not support any utilities. A site locus plan is provided as Figure 1 in Appendix A.

3. Proposed Construction

The proposed construction includes the following items:

- The existing bridge superstructure will remain with some structure member replacements.
- Two temporary shoring towers will be constructed to support the existing bridge superstructure in place during center pier construction.
- The existing bridge pier will be demolished.
- A new bridge pier supported on two deep foundations will be constructed at the same location as the existing pier.
- Temporary shoring towers will be demolished after the new pier construction.
- The top course of stone in the existing abutment seats will be removed, and new precast concrete seats will be installed. New backwalls will be placed and configured to account for the necessary revision to the end diaphragms and bracing components due to modifications to the tie seat angle.

4. Local Geology

According to the USGS geology map of the State of Vermont, bedrock within the area of the Project site consists of Meta-ultramatic rocks or the Moretown Formation. Meta-ultramfic rocks mainly consist of Brown to white-weathering, green, massive, moderately to fully serpentinized dunite and peridotite and schistose serpentinite; rusty-weathering, medium-grained talc-carbonate rock and quartz-carbonate (magnesite) rock. The Moretown Formation mainly consists of Rusty-weathering, dark-gray biotite-muscovite-quartz (±garnet) schist, carbonaceous schist, and gray, splintery-fractured, biotitic sulfidic quartz schist. At the Bridge 132 Project site, the bedrock consisted of very hard, very slightly weathered to fresh, moderately to slightly fractured, medium grained, dark gray Schist.

Refer to Appendix C for rock core sample descriptions and rock core photos.

5. Subsurface Explorations

Under Jacobs supervision, a subsurface geotechnical exploration program was conducted at the Project site in September 2019. The subsurface exploration program included four test borings in the vicinity of the existing bridge pier and abutments, completed by New England Boring Contractors, Inc. (NEBC). The borings are designated B-1 through B-4.

The borings were advanced to depths ranging from 14 to 79.5 feet using rotary-wash drilling techniques with a 4-inch casing. Standard split-spoon soil samples were collected, and Standard Penetration Tests (SPT)[1] were performed using a 2 inch outside diameter, 2-foot long split-spoon sampler in accordance with ASTM D1586 at the depth intervals noted on the boring logs. Standard Penetration Tests (SPTs) and split spoon sampling of overburden soils were generally performed at 5-foot intervals. The SPTs were performed using a 140-lb automatic hammer. One 5-foot rock core sample was collected from boring B-2 using a diamond-bit NX-size double tube core barrel. The recovered rock core was placed in a wooden core box for transport and storage. In borings B-3 and B-4, roller bit drilling continued for approximately 2 feet to confirm rock after the drill bit hit an obstruction, indicating possible bedrock or boulder. The boreholes were backfilled with soil cuttings and sand to the existing grade upon completion.

Jacobs personnel observed the drilling and prepared logs for each boring. Soil samples were classified using the Modified Burmister Classification System. Refer to Figure 2 in Appendix A for approximate test boring locations. Borings logs prepared by Jacobs are included in Appendix C.

6. Laboratory Testing

No laboratory testing was performed on collected soil and rock samples.

7. Subsurface Conditions

The following generalized subsurface conditions at the site are interpretive and inferred from information obtained from the subsurface exploration program.

[1] The SPT consists of driving a 2-inch-outside-diameter split spoon sampler a total of 24 inches with a 140-pound hammer free-falling 30 inches. The number of hammer blows required to drive the sampler are recorded for each 6-inch interval. The number of blows required to advance the sampler from 6 to 18 inches of penetration is referred to as the "N Value".

The subsurface conditions at the Project site vary along the existing bridge alignment. Subgrade soils at the bridge abutments generally consisted of medium dense to very dense sand and gravel overlying possible bedrock or large boulders. Subgrade soils at the bridge pier generally consisted of dense to very dense silty sand overlying bedrock.

Refer to Figure 3 in Appendix A for the anticipated subsurface profile along Bridge 132.

The subsurface soil conditions, as encountered in the four 2019 borings, are summarized in Table 1 below. A description of the principal subsurface units is provided in the sub sections that follow.

Table 1: Summary of Subsurface Conditions

Boring No.	Approx. Ground Surface El. (ft.)	Approx. Overburden Thickness (ft.)	Approx. Top of Possible Bedrock El. (ft.)	Bottom of Boring El. (ft.)	Approx. Ground Water El. (ft.)
B-1	925.4[1]	63.5	861.9	861.9	NR[3]
B-2	927.9[1]	62.0	865.9	860.9	927.9
B-3	940.7	26.0	914.7[2]	912.7	NR[3]
B-4	939.9	12.2	927.7[2]	925.9	NR[3]

Notes:

1. Ground surface elevations in borings B-1 and B-2 are riverbed elevations. The drill rig was set up at the top of rail elevation (El. 941.4) for borings B-1 and B-2.
2. Top of rock elevations in borings B-3 and B-4 may actually be the top of large boulders.
3. "NR" indicates groundwater not recorded.
4. Ground surface elevation at boring locations were interpolated from topographic contours with NAVD88 vertical datum.

7.1 Soil

7.1.1 Sand and Gravel at Bridge Abutments

Sand and gravel overburden was encountered in the two abutment borings, B-3 and B-4 overlaying possible bedrock or large boulder. The sand and gravel overburden ranged in thickness from approximately 12 to 26 feet and consisted predominately of fine to coarse sand and gravel. SPT N-values ranged from 15 to greater than 100 blows per foot (bpf). Note SPT N-values in excess of 50 bpf in the sand and gravel layer may represent oversized materials such as coarse gravel, cobbles and/or boulders, and may not necessarily represent the relative density of the soil unit.

7.1.2 Silty Sand at Bridge Pier

Silty sand overburden was encountered in the two pier borings B-1 and B-2 overlaying bedrock. The silty sand overburden ranged in thickness from approximately 62.0 to 63.5 feet and consisted predominately of fine to medium sand and silt with varying amounts of gravel. SPT N-values ranged from 32 to greater than 100 blows per foot (bpf). Similar to the sand and gravel layer at the bridge abutments, SPT N-values in excess of 50 bpf in the silty sand layer may represent oversized materials such as coarse gravel, cobbles and/or boulders, and may not necessarily represent the relative density of the soil unit.

7.2 Bedrock

At the bridge pier location, bedrock was encountered at approximately 75.5 to 80.5 feet below the top of rail elevation on the bridge, corresponding to El. 865.9 to El. 860.9. In boring B-2, the bedrock consisted of very hard, very slightly weathered to fresh, moderately to slightly fractured, medium grained, dark gray schist. Rock recovery percentage was 100, and Rock Quality Designation (RQD) percentage was 83. Refer to Appendix C for the photograph of the rock cores.

At the abutments, possible bedrock was encountered at approximately 12.2 to 26 feet below the existing ground surface, corresponding to El. 927.7 to El. 914.7.

7.3 Groundwater

Groundwater levels were not measured during drilling at borings B-1, B-3, and B-4. Stream water elevation was measured in pier boring B-2 to be at El. 927.9 feet which is about 13.5 feet below the top of rail elevation. The stream water elevation is higher than the proposed top of footing (i.e., the entire footing is submerged).

Local or periodic fluctuations of stream water and groundwater levels should be expected, as levels may be influenced by season, precipitation, construction activity, and other factors. Therefore, groundwater elevations presented herein may not be representative of water levels encountered during construction.

8. Seismic Design Parameters

Our work included conducting a seismic site class evaluation to determine the appropriate site coefficients for structural design. For the current Project, seismic site classification followed the 2019 AREMA Manual for Railway Engineering, Chapter 9. Detailed seismic site class evaluation information is presented in Appendix E.

Based on our study of the average SPT-N values measured within the upper 100 feet of the recent test borings, the site may be classified as Site Class C.

A 475-year return period was used for this evaluation since it represents a rare earthquake frequency. Based on the above design earthquake and the site soils as indicated in the boring logs, the following seismic coefficients are recommended for structural design.

Table 2: Seismic Design Parameters

Ground Accelerations (% g)	
Peak Ground Acceleration (PGA)	3.7
0.2-Second Period Spectral Response Acceleration (S_s)	8.2
1.0-Second Period Spectral Response Acceleration (S_1)	2.7
Site Specific Parameters	
Site Classification	C
Site Factors	
USGS Site Factor - F_{pga}	1.2
USGS Site Factor – F_a	1.2
USGS Site Factor – F_v	1.7

Refer to Appendix E for full seismic design parameter calculations.

9. Liquefaction Potential

Based on the observed subsurface conditions, recorded water level, percentage of fines content and the recorded SPT N-values, the soil layers underlying the site are judged to be not susceptible to liquefaction.

10. Geotechnical Recommendations

10.1 Foundation Recommendations

Driven Pile Foundations

Based on the design loads, the existing subsurface conditions, and to minimize the railroad downtime, driven steel H-piles are considered the most appropriate foundation type for the new bridge pier.

Design and construction of driven steel H-piles should be performed in accordance with Part 4 of Chapter 8 and Parts 1.3 and 1.4 of Chapter 15 of the 2019 AREMA Manual. The recommended allowable stress for driven steel piles is 12,600 psi for axial load only. Accordingly, we recommend an allowable structural capacity of 362 kips (axial compression) for a 50 ksi HP 14x117 section, assuming a corrosion allowance of 1/16 inch.

All foundation piles shall be driven to the top of the rock. Generally, for pile foundations driven to bedrock, the available geotechnical resistance from the rock will likely be greater than the structural axial capacity (i.e., the available steel strength will control pile axial capacity).

For each center pier pile group, a total of nine driven HP vertical piles are planned in a 3 x 3 grid pattern with their strong axis aligned to resist longitudinal loads. The pile row spacing is 4 feet center-to-center in both directions.

Based on the foundation loads and the proposed pile cap configuration provided by the Jacobs Structural Engineer, the maximum and minimum estimated pile axial loads for the abutments are 283.4 kips and -60.2 kips, respectively, from the GROUP analyses. Using a factor of safety of 2.25, which assumes dynamic pile driving analyzer (PDA) testing is performed, the required ultimate pile capacity in compression is 637.7 kips. Using the same factor of safety, the required ultimate uplift capacity per pile is 135.5 kips.

To estimate the approximate driving depth to achieve the required pile capacity, static pile capacity analyses were performed using the computer programs APILE. Based on the results of the APILE analyses, we estimate that the center pier piles could achieve the required resistance at a depth of approximately 55 to 56 feet, which is close to the top of bedrock. We recommend all piles be driven to bedrock to achieve the required ultimate capacities. Based on an assumed pile tip elevation at the top of bedrock, the allowable uplift capacity of the pile will exceed the demand of 60.2 kips as determined through the GROUP analyses. To ensure bottom fixity, piles shall be driven to a minimum depth of 26 feet below the bottom of the pile cap (El. 892.9). Driven pile design calculations are presented in Appendix F.

Maximum Pile Group Reactions

A summary of the maximum pile/pile cap reactions at the center pier foundations from the GROUP analyses is presented in Table 3 below.

Table 3: Maximum Pile Group Reactions

Proposed Substructure	Axial Compression (kips)	Axial Tension (kips)	M_z (kip-ft)	M_y (kip-ft)	V_y (kips)	V_z (kips)	δ_y (inches)	δ_z (inches)
Center Pier	283.4	-60.2	47.6	62.9	13.1	17.9	0.185	0.193

Notes:
1) The governing load condition includes both Primary and Secondary loads.
2) A fixed pile-pile cap connection was assumed in the GROUP analyses.
3) Passive earth pressure on pile cap was not considered in the GROUP analyses.
4) Maximum pile reactions presented in the table may not occur on the same pile.
5) Maximum deflections were estimated at the center of the pile cap.

Settlement

For the proposed driven pile foundations, it is anticipated that pile settlements may generally be equivalent to the elastic compression of the pile, plus the inelastic compression within the bearing layer. The inelastic compression within the bedrock is anticipated to be small (less than ¼ inch). The elastic compression for the driven H-piles is expected to be less than ½ inch.

10.2 Precast Abutment Wall Lateral Earth Pressure

The following backfill parameters and assumptions are recommended for the design of the precast abutment walls, assuming that the walls are backfilled with free-draining Type 1 Backfill in accordance with Tables 8-5-1 and 8-5-2 of the AREMA Manual:

- assumed soil friction angle of 34 degrees (compacted Type 1 Backfill)
- a total soil unit weight of 125 pounds per cubic foot (pcf) (compacted Type 1 Backfill)
- horizontal backfill slope behind the wall
- an equivalent fluid pressure of 55 pcf (based on a recommended Ko of 0.44 for this site) for evaluation of at-rest lateral earth pressures

The recommended equivalent fluid pressure assumes that hydrostatic pressure is not present behind the walls and that the walls are fully drained. Free-draining material shall be used within a wedge behind the wall, bounded by a plane rising at 60 degrees to the horizontal from the bottom of the wall.

Additional lateral forces due to a seismic event and lateral pressures due to railroad surcharge loads should be applied as required by Part 5.3.1 of Chapter 8 of the AREMA Manual.

Where the calculated pressure behind the retaining wall is less than 250 pounds per square foot (psf), it should be increased to 250 psf to account for stresses created by compaction of fill behind the wall.

10.3 Temporary Support of Excavation and Shoring

The demolition of the existing pier and construction of the new pier will require temporary Support of Excavation (SOE) and a temporary shoring system to support the bridge superstructure. The construction of the new precast concrete backwall at the abutments will require temporary sloped excavation.

Possible temporary SOE systems include interlocked sheeting. However, the SOE systems should be selected and designed by the Contractor, based on the Contractor's own experience, means, and methods of construction.

We recommend that the temporary SOE and shoring systems be designed utilizing the following minimum soil properties and surcharge loads:

Temporary SOE:

- Active earth pressure coefficient of 0.33
- At-rest earth pressure coefficient of 0.5
- Passive earth pressure coefficient of 3.0
- Saturated unit weight of 120 pcf (below the water table)

Temporary Shoring:

- Temporary shoring for the pier shall support the superstructure vertical dead load, 10 MPH vertical and longitudinal live load (286K car), and wind load.
- Dead load girder reaction at the shoring location is 57 kips.
- Vertical live load girder reaction at the shoring location is 288 kips.

- Longitudinal live load girder reaction at the shoring location is 54 kips.

Temporary SOE and shoring systems should be selected by the Contractor and designed by an experienced Professional Engineer registered in the State of Vermont, and retained by the Contractor, and should be submitted to Jacobs for review and approval. Excavation sides that are cut back and sloped shall be no steeper than 1.5H:1V and should comply with the Occupational Safety and Health Administration (OSHA) Construction Industry Standards.

10.4 Scour Protection

The following measures are recommended for scour protection at the center pier:

- The center pier foundations are recommended to be installed with the bottom of the pile cap located at about 8 feet below the existing mudline (i.e., at approximate El. 919).
- Stone fill material with minimum size of 4 inches is recommended as backfill on top of the proposed pile cap.
- The cofferdam used for pier foundation construction will remain in place below the top of footing elevation to further mitigate the scour of soil under the footing.

11. Construction Considerations

11.1 Construction Dewatering and Temporary Excavation Support

We anticipate that sheet pile cofferdams will be installed within the river for lateral earth support and as part of the dewatering system for the construction of the center pier foundations. The cofferdam should be designed to include loads applied by the existing center pier footing, the maximum design hydrostatic pressure as well as ice and/or river current.

We recommend that the piles be installed after installation of the sheeting and bracing, and excavation to the required subgrade. In order to allow for construction of the pile cap to proceed in the dry, we recommend installing a tremie slab. Note that the uplift resistance of sheeting and piles may be used to limit the thickness of the tremie slab provided shear studs or other structural connections are used. Following installation of the tremie slab, the cofferdam can be pumped dry, and the pile cap constructed.

Alternatively, in lieu of a tremie slab, the Contractor should be prepared to manage and control the river water during foundation excavation, including seepage and hydraulic gradients that could result in instability of the subgrade to provide a relatively stable subgrade during pile installation and subsequent pile cap construction. Based on the granular nature of the existing natural soils, we anticipate that a significant number of wells will be required to manage large quantities of groundwater.

The Contractor should be responsible for selecting the dewatering methods based on his/her proposed methods and equipment used for excavation, as previously discussed. The method of dewatering will depend on time of year that the work is performed, depth of sheeting, and the size of the excavation.

11.2 Driven H-Piles

Prior to mobilizing to the site, the pile contractor should submit a wave equation analyses (WEAP) performed by his engineer considering the specific hammer that will be utilized during construction. The WEAP should establish estimated blow counts in order to develop the necessary capacities and to estimate dynamic stresses during driving. We recommend that the piles be driven to 2.25 times the required design axial compressive load, provided PDA is utilized as discussed below.

A static pile load test is deemed unnecessary at the site based on the expected subsurface conditions. Alternatively, we recommend performing dynamic pile testing with a Pile Driving Analyzer (PDA) on a minimum of one pile at each center pier foundation to establish the driving criteria and confirm the geotechnical pile capacity. The pile driving data should be signal-matched utilizing CAPWAP procedures. Dynamic testing shall consider the bearing strata to determine the need of restrike and timing.

Due to the dense sands and potential boulders present at the site, difficult driving conditions are anticipated. Therefore, consideration should be given to vibrating the piles for some depth prior to proceeding with impact driving. However, all piles shall be driven to their final elevation for a distance of at least 5 feet with an impact hammer. The intent of utilizing vibration techniques to partially install the piles is to allow the pile to become seated properly. Several attempts at installation (requiring a pull out and restart) may be required if cobbles/boulders are present near the surface of the riverbed.

We recommend that all H-pile tips should be equipped with pile points fabricated from cast steel conforming to ASTM A27 in order to protect the piles and help them penetrate small obstructions at the site. All H-piles should penetrate any existing fill, boulder obstructions, and be driven to bedrock. Driving stresses should be limited to less than 40 ksi, $0.8F_y$ of Grade 50 steel, per AREMA Chapter 8, Part C - 4.4.2.6.

11.3 Obstructions

No obstructions to rollerbit were observed in borings B-1 and B-2 during drilling. However, very dense sand was encountered in both borings based on spoon sampling below a depth of approximately 20 to 25 feet below the top of the rail. The presence of the very dense sand may impact the pile and sheet pile driving at the site.

Rollerbit obstructions were observed in borings B-3 and B-4 at depth of 26 and 12.2 feet, respectively, from the top of rail elevation. In both borings, roller bit was continued for approximately 2 feet with a consistent penetration rate after hitting obstruction, and the obstruction is confirmed to be either bedrock or large boulder.

We recommend the project specifications contain provisions for pre-augering and/or other measures to contend with the anticipated dense material in advance of pile installation. It is also recommended that a contingency plan be made for pile installation due to the probability of piles or cofferdam sheet piling stopping short.

11.4 Protection of Existing Facilities

Utilizing impact and/or vibration techniques with sheeting and H-piles will result in vibrations that could impact existing structures. We recommend that the existing bridge superstructure, abutment wall to remain, and railroad tracks be carefully monitored via survey techniques for movement during pile and sheeting installation.

11.5 Earthwork and Compaction

In accordance with Chapter 8, Part 5.5.1 of the AREMA Manual, material placed immediately adjacent to the precast abutment backwalls should consist of non-cohesive free draining material such that free flow of water is permitted behind the wall. Jacobs recommends that backfill placed behind the abutment retaining walls consist of Backfill Type 1 as specified in Table 8-5-1 of the AREMA Manual, similar to Granular Backfill for Structures in accordance with Article 704.08 of 2011 VTrans Standard Specification for Construction. Backfill Type 1 consists of coarse-grained soil without admixture of fine soil particles and is free-draining (i.e., sand, gravel, or broken stone). Weep holes are not required based on the limited distance the water would have to travel laterally around the abutment wall.

Fill should be compacted to at least 95 percent of the maximum dry density and within 2 percent of the optimum moisture content, as determined by the Standard Proctor Test (ASTM D-698), in accordance with Chapter 8 Part 5.5.2 of the AREMA Manual. Compaction should be performed in lifts not exceeding 12 inches in thickness. In confined areas, place only 6-inch layers and compact with manually operated, powered vibratory compaction equipment acceptable to the geotechnical engineer. Crushed stone should be placed in layers not more than 12 inches thick and compacted to an unyielding surface. Crushed stone should be wrapped in non-woven geotextile filter fabric similar to Mirafi 170N with a minimum overlap of at least two feet. Extra care should be used when compacting adjacent to walls. Compaction within 5 feet of abutment walls less than 15-feet-high, or within 10 feet of walls greater than 15-feet-high should be performed using a vibratory walk-behind roller or plate compactor. Fill must not be placed over frozen soil. Soil subgrades must be protected against frost both during and after construction.

11.6 Reuse of Excavated Materials

Based on the soil descriptions on the boring logs, it is anticipated that some of the existing on-site granular soils to be excavated for foundation construction could meet the gradation requirements for Type 1 Backfill. Soils not meeting the Type 1 Backfill specification may be reused in areas not requiring a free-draining material, provided that weather conditions are satisfactory, the moisture content can be controlled, and the materials can be compacted to the required density.

If any contaminated soils are exposed, these materials should be properly managed and disposed of in accordance with appropriate State and Federal regulations.

The Contractor should anticipate that areas immediately adjacent to the site will not be available to stockpile soils. Stockpiled soils may require the installation of run-off protection between drainage channels and the stockpile. Stockpiles of fill materials should be maintained to prevent material from fluctuating from the optimum moisture content, freezing, separating due to migration of fine-grained soils, and collection of snow or ice within the stockpiles. Reuse of on-site soils should be at the acceptance of the geotechnical engineer prior to placement.

12. Limitations

This report and the recommendations contained herein have been prepared for the exclusive use of the Vermont Agency of Transportation and their representatives for specific application to the geotechnical design and construction for the Green Mountain Railroad Corporation (GMRC) Bridge No. 132 Rehabilitation Project in Cavendish, Vermont.

This report was prepared in accordance with generally accepted soil and foundation engineering practices. No warranty, expressed or implied, is made. The analysis, design, and recommendations submitted in this report are based in part upon the data obtained from subsurface explorations available at the time of this report. Subsurface stratification variations between borings are anticipated. The reported groundwater levels only represent the water levels at the time noted on the logs. The nature and extent of variations between these explorations may not become evident until construction. If significant variations appear, or if there are changes in the nature, design, or location of the proposed structure, it may be necessary to reevaluate the recommendations of this report.

Unless noted on the boring logs, the lines designating the changes between fill, soil, and rock strata represent approximate boundaries. The transition between materials may be gradual or may occur between recovered samples. The stratification given on the boring logs, or described herein, is for use by Jacobs in its analyses and should not be used as the basis of design or construction cost estimates without realizing there can be variation from that shown or described.

The boring logs and related information depict subsurface conditions only at the specific locations and times when sampling was conducted. The passage of time may result in changes of conditions at or between the locations where sampling was conducted.

We appreciate the opportunity to be of service to you on this Project. Please contact us if you have any questions regarding this report.

Very truly yours,

Jacobs Engineering Group

Da Ha, PE, PhD
Geotechnical Engineer

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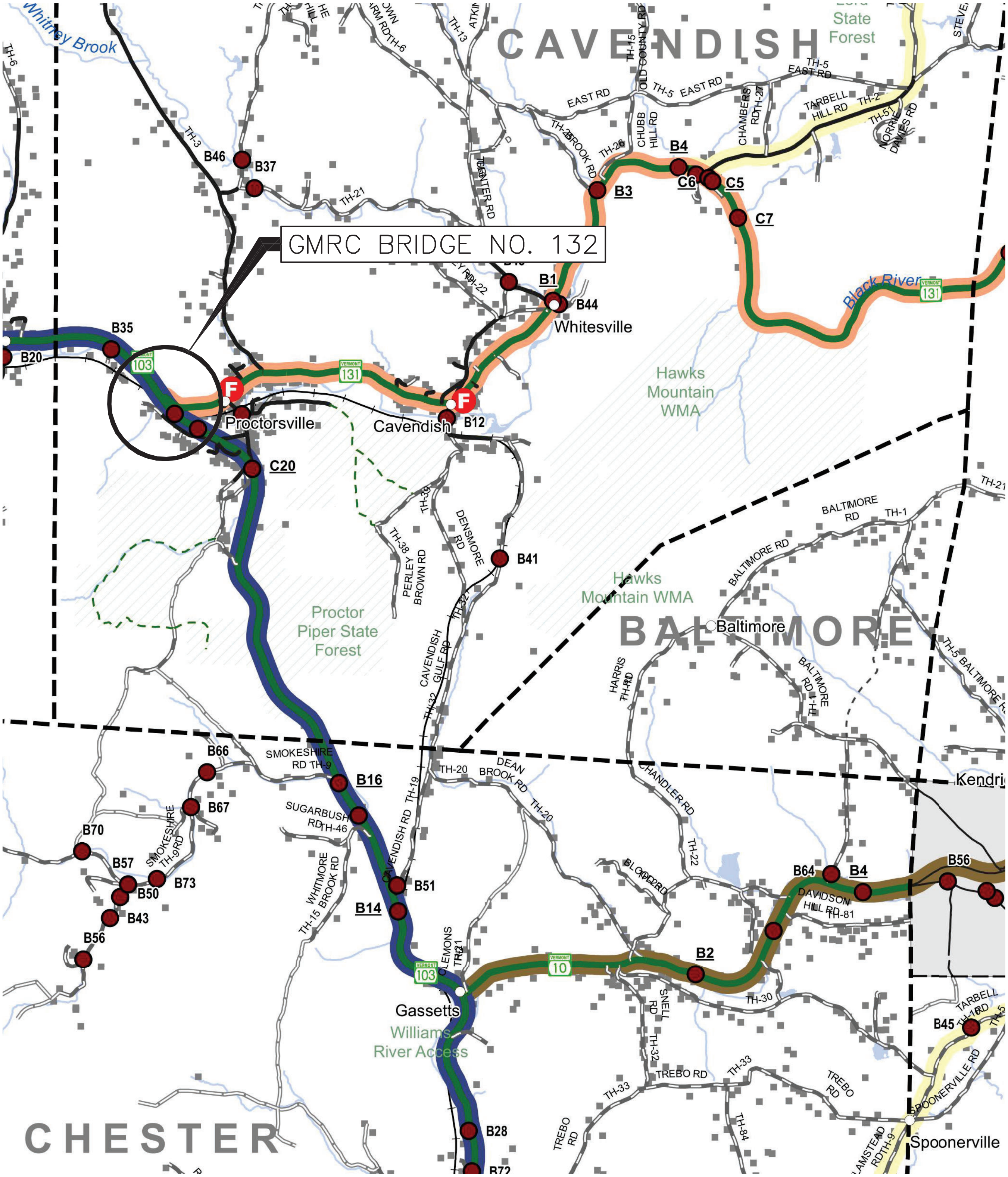
Appendix A. Figures and Plans



LOCUS PLAN

NOT TO SCALE

Jacobs

GMRC BRIDGE NO. 132
CAVENDISH, VERMONT
LOCUS PLAN

FIGURE 1

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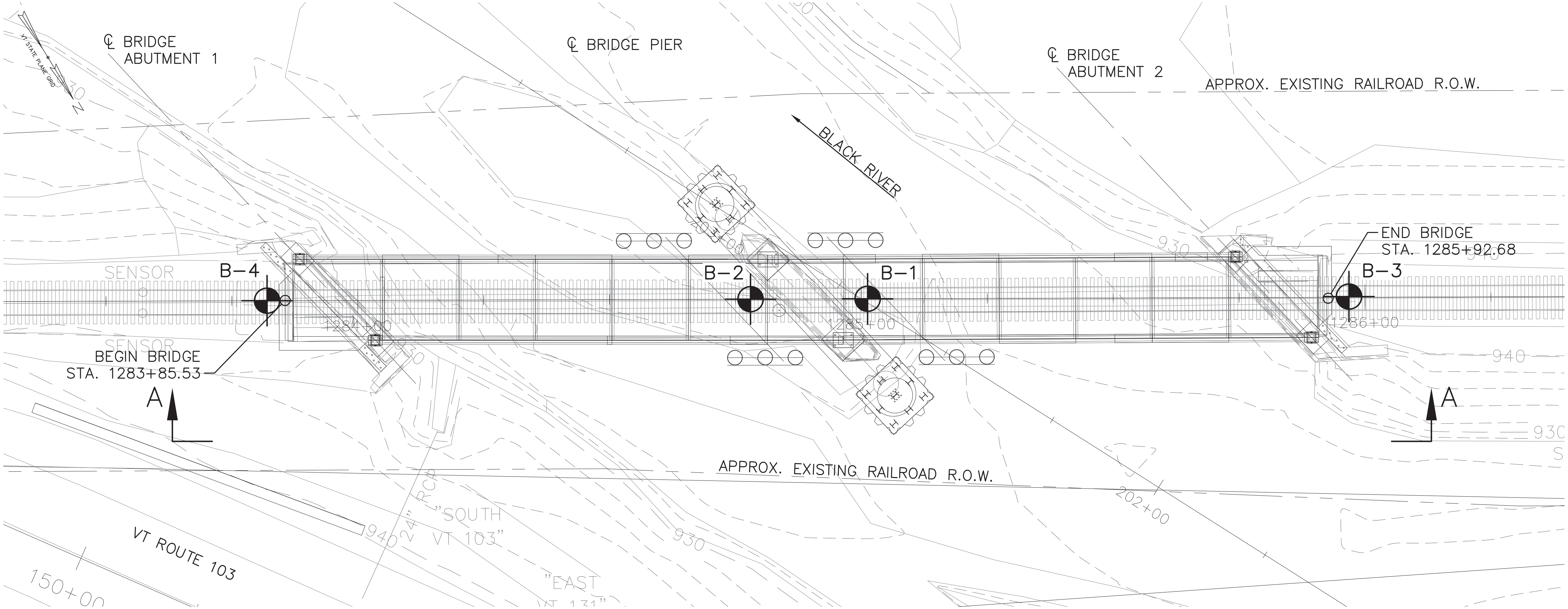
December 16, 2021



SUBSURFACE EXPLORATION PLAN

0 10 20
SCALE 1" = 20'–0"

NOTES:

1. BORING LOCATIONS WERE APPROXIMATELY DETERMINED BY TAPE MEASUREMENT AND LINE OF SIGHT FROM THE ORIGINAL STAKED LOCATION AND SITE FEATURES AND SHOULD BE CONSIDERED ACCURATE TO THE DEGREE IMPLIED BY THE METHOD USED.
2. FOR BORING LOGS SEE FINAL GEOTECHNICAL REPORT DATED DECEMBER 2021 APPENDIX C.
3. SURFACE DETAIL AND TOPOGRAPHY SHOWN HEREON WERE BASED ON SURVEY PERFORMED BY VERMONT SURVEY IN APRIL 2014.
4. CONTOURS AND ELEVATIONS SHOWN HEREIN ARE BASED ON THE NORTH AMERICAN VERTICAL DATUM (NAVD 88).

LEGEND:

B–1	BORINGS PERFORMED BY NEW ENGLAND BORING CORP. BORINGS WERE OBSERVED AND LOGGED BY JACOBS PERSONNEL BETWEEN SEPTEMBER 9 AND 13, 2019.
A A	SUBSURFACE PROFILE

DRAWN BY:	C. BAISLY
DESIGNED BY:	D. HA
CHECKED BY:	M. MAHDAVI
DATE:	DECEMBER 2021

Jacobs

GMRC BRIDGE NO. 132
CAVENDISH, VERMONT
SUBSURFACE EXPLORATION PLAN

FIGURE 2

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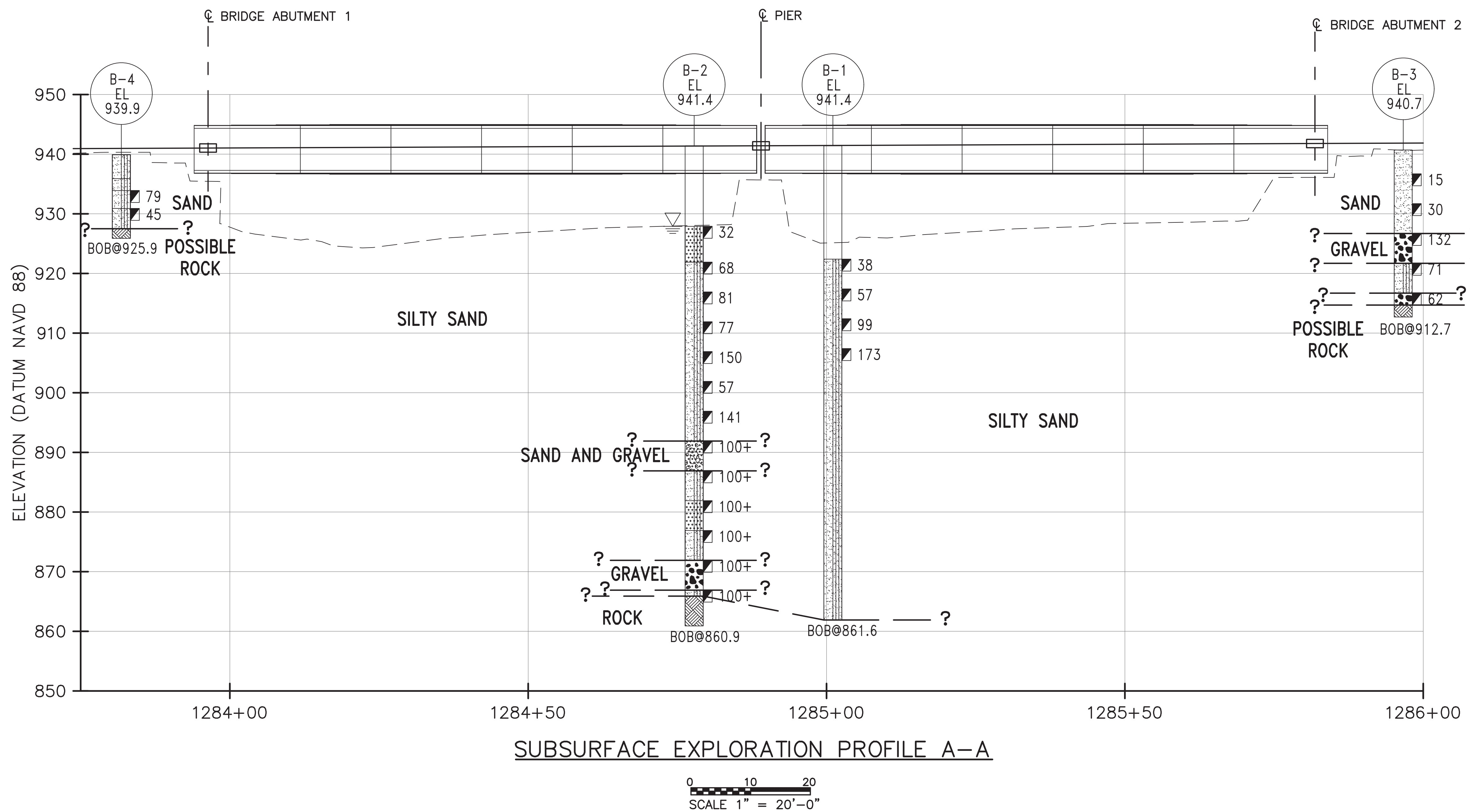
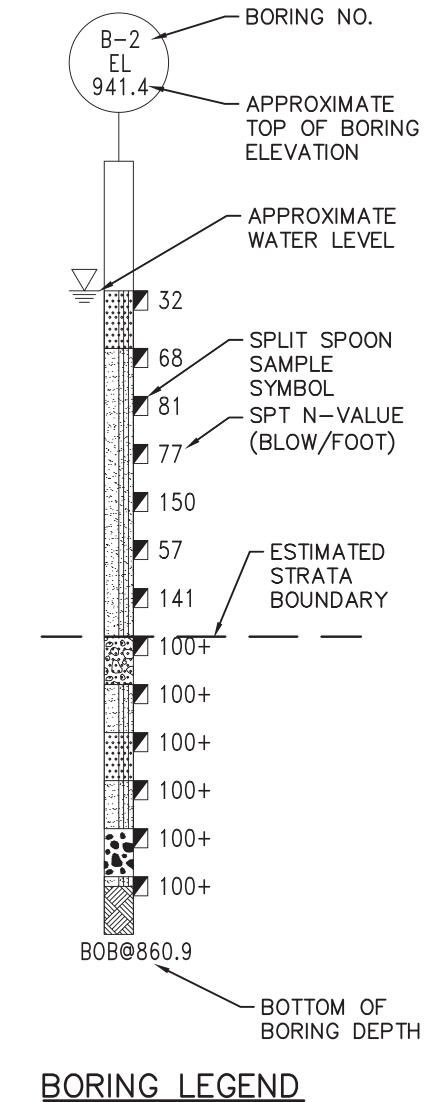
December 16, 2021





LEGEND

- USGS WELL–GRADED GRAVEL
- USGS POORLY–GRADED SAND WITH SILT
- USGS WELL–GRADED GRAVELY SAND
- USGS POORLY–GRADED SAND
- USGS WELL–GRADED SAND WITH SILT

NOTES:

1. THE STRATIFICATION LINES ARE BASED UPON WIDELY SPACED BORING LOCATIONS AND THUS REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES. ACTUAL TRANSITIONS MAY VARY FROM THOSE SHOWN. REFER TO BORING LOGS FOR SPECIFIC CONDITIONS ENCOUNTERED AT EACH BORING.
2. THE STRATA DESCRIPTIONS SHOWN ON THE PROFILE ARE GENERALIZED.
3. REFER TO FIGURE 2 FOR THE LOCATION OF THE SUBSURFACE PROFILE.
4. THE WATER LEVEL IN THE BORINGS REPRESENT LEVELS MEASURED DURING DRILLING AND UNDER CONDITIONS NOTED ON THE LOGS. FLUCTUATIONS IN THE WATER LEVEL MAY OCCUR DUE TO VARIATION IN RIVER LEVEL, RAINFALL, TEMPERATURE AND OTHER FACTORS DIFFERENT THAN AT THE TIME THESE MEASUREMENTS WERE MADE.

DRAWN BY:	C. BAISLY
DESIGNED BY:	D. HA
CHECKED BY:	M. MAHDAVI
DATE:	DECEMBER 2021

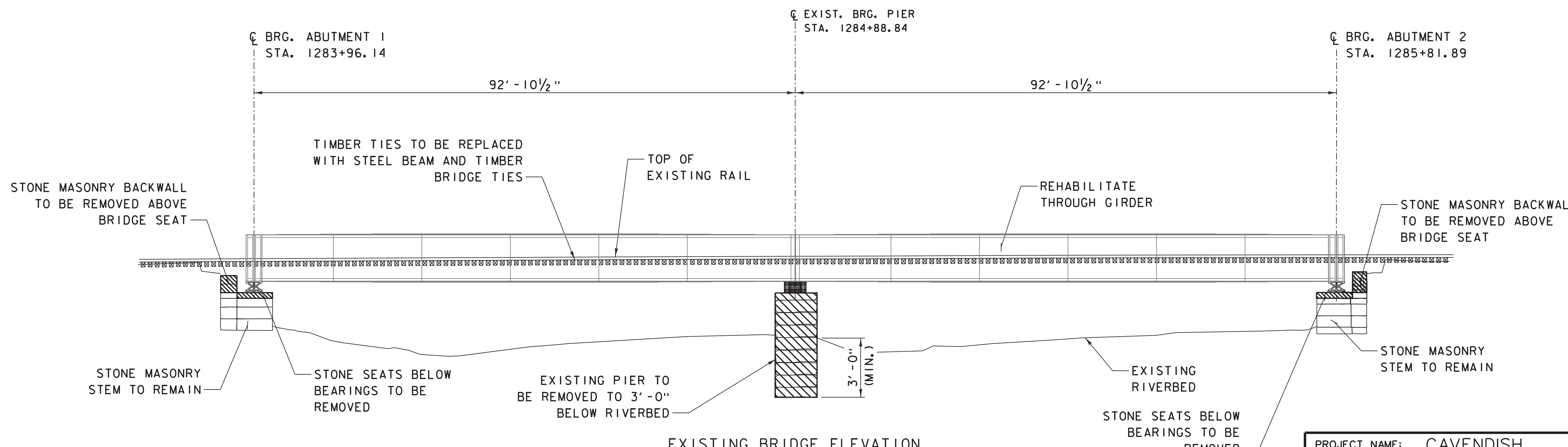
Jacobs

GMRC BRIDGE NO. 132
CAVENDISH, VERMONT
SUBSURFACE EXPLORATION PROFILE

FIGURE 3

O:\INFRASTRUCTURE\GEOTECHNICAL\VTRANS BRIDGE 132\GEOTECH MEMO\CAD\FIGURES.DWG
December 22, 2021

EXISTING BRIDGE PLAN
SCALE: 1" = 10′-0"



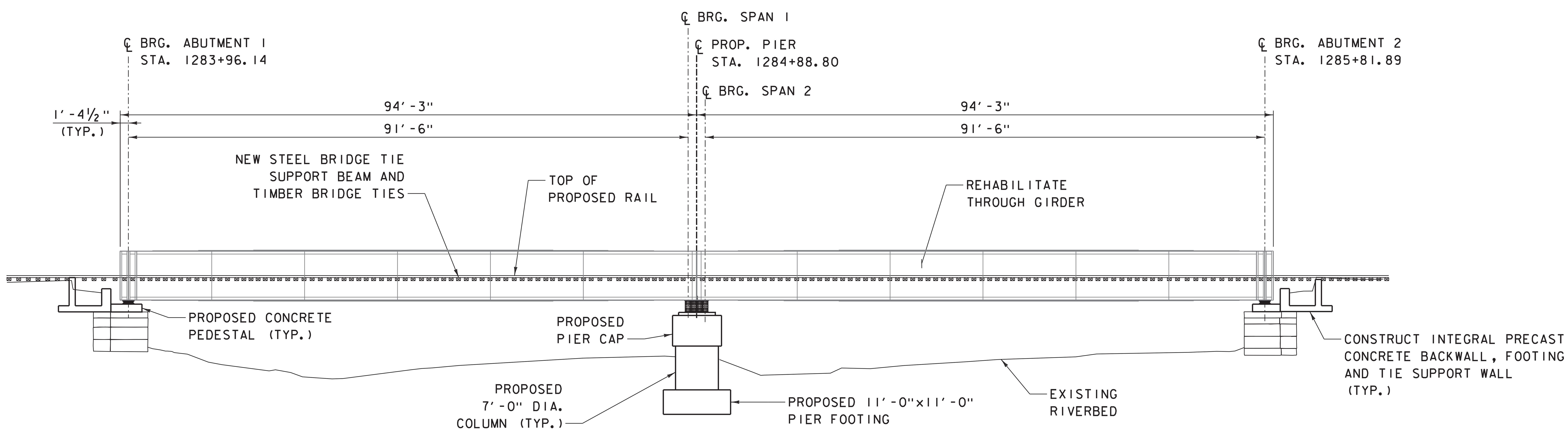
EXISTING BRIDGE ELEVATION
SCALE: 1" = 10′-0"

NOTE: RIVERBED TO BE FILLED IN AT PIER REMOVAL LOCATION USING ITEM 613.06, "STONE FILL, STREAM BED MATERIAL (PIER)" CONSISTING OF E-STONE FILL TYPE II.

Jacobs

PROJECT NAME: CAVENDISH	
PROJECT NUMBER: GMRC(24)	
FILE NAME: z15g155pe01.dgn	PLOT DATE: 11/17/2021
PROJECT LEADER: J. WILSON	DRAWN BY: S. GUNN
DESIGNED BY: A. WALL	CHECKED BY: S. HALLORAN
EXISTING BRIDGE PLAN & ELEVATION	SHEET 15 OF 98

PROPOSED BRIDGE PLAN
SCALE: 1" = 10'-0"



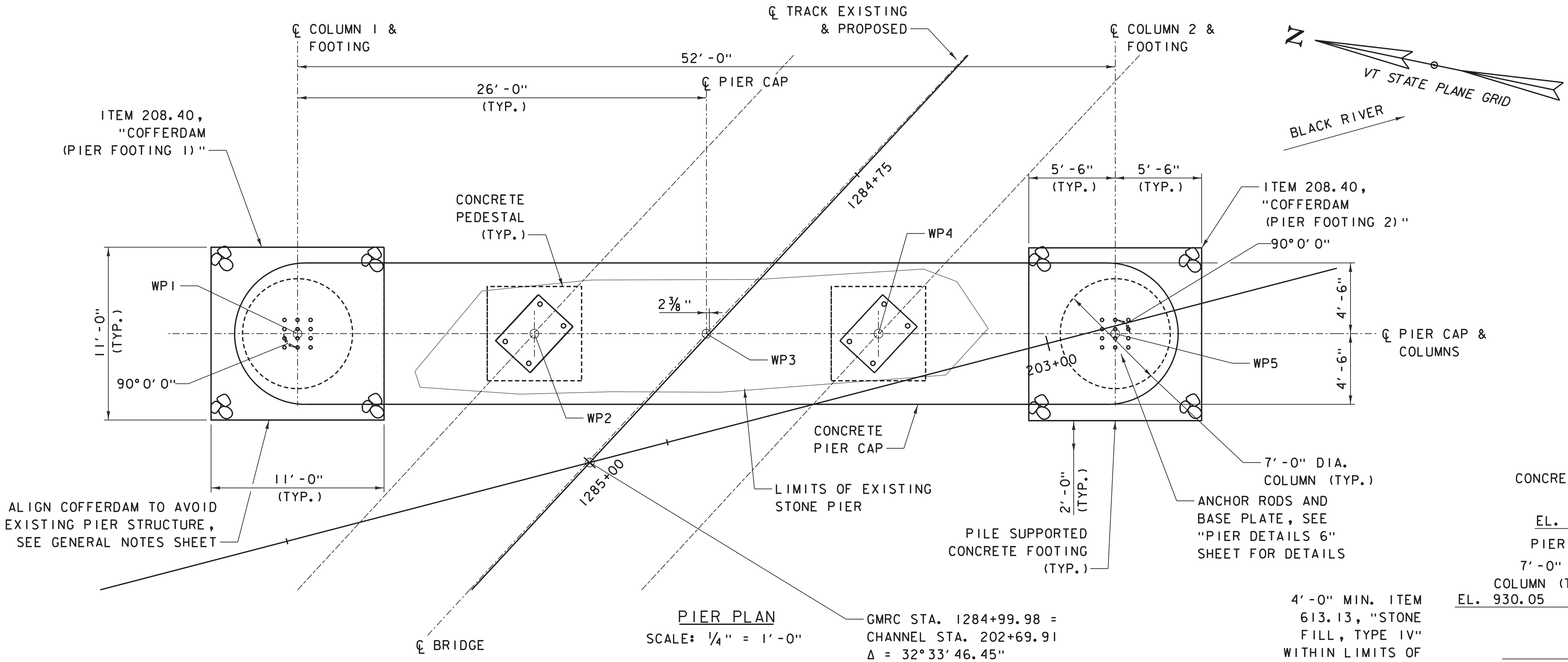
PROPOSED BRIDGE ELEVATION
SCALE: 1" = 10'-0"

PROJECT NAME: CAVENDISH
PROJECT NUMBER: GMRC(24)

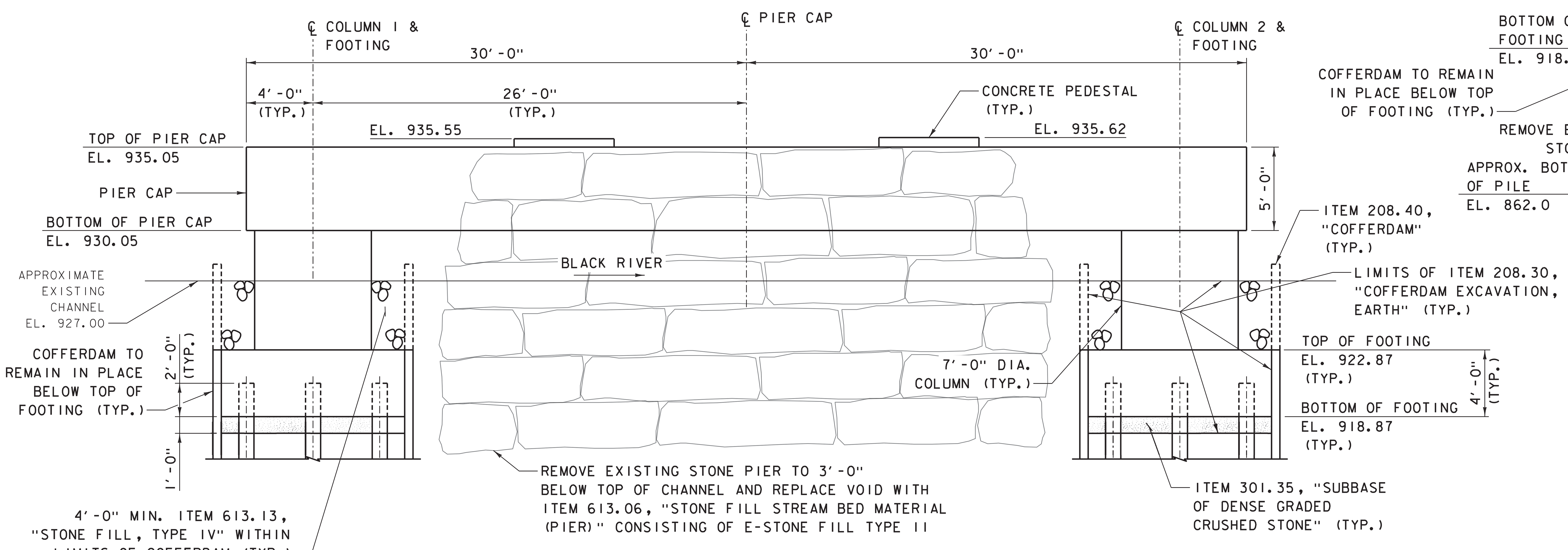
FILE NAME: z15g155pe02.dgn
PROJECT LEADER: J. WILSON
DESIGNED BY: A. WALL
PROPOSED BRIDGE PLAN & ELEVATION

PLOT DATE: 11/17/2021
DRAWN BY: S. GUNN
CHECKED BY: S. HALLORAN
SHEET 16 OF 98

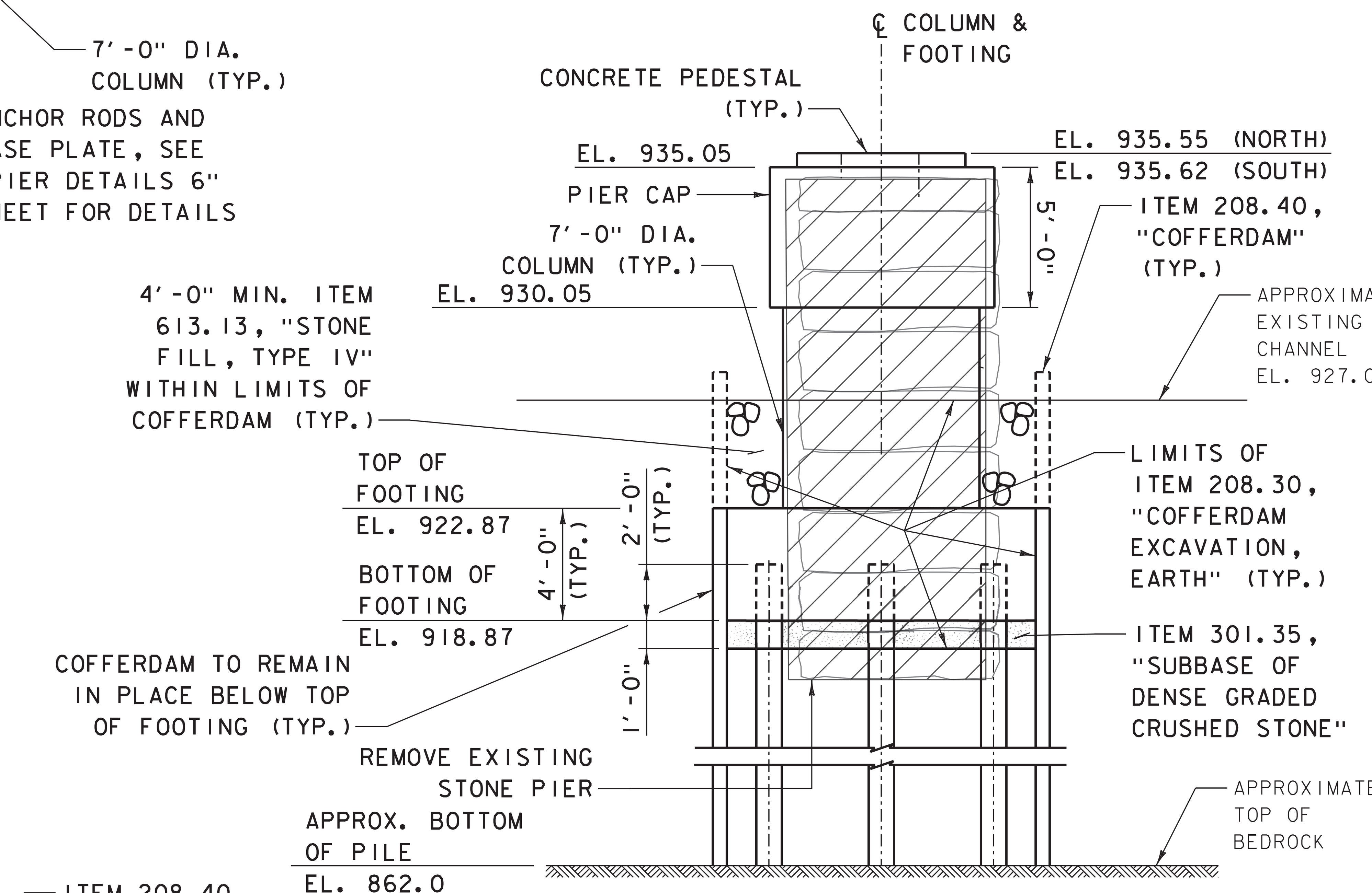
Jacobs



PIER PLAN
SCALE: ¼" = 1'-0"



PIER ELEVATION
SCALE: ¼" = 1'-0"



PIER SECTION
SCALE: ¼" = 1'-0"

WORKING POINT CONTROL

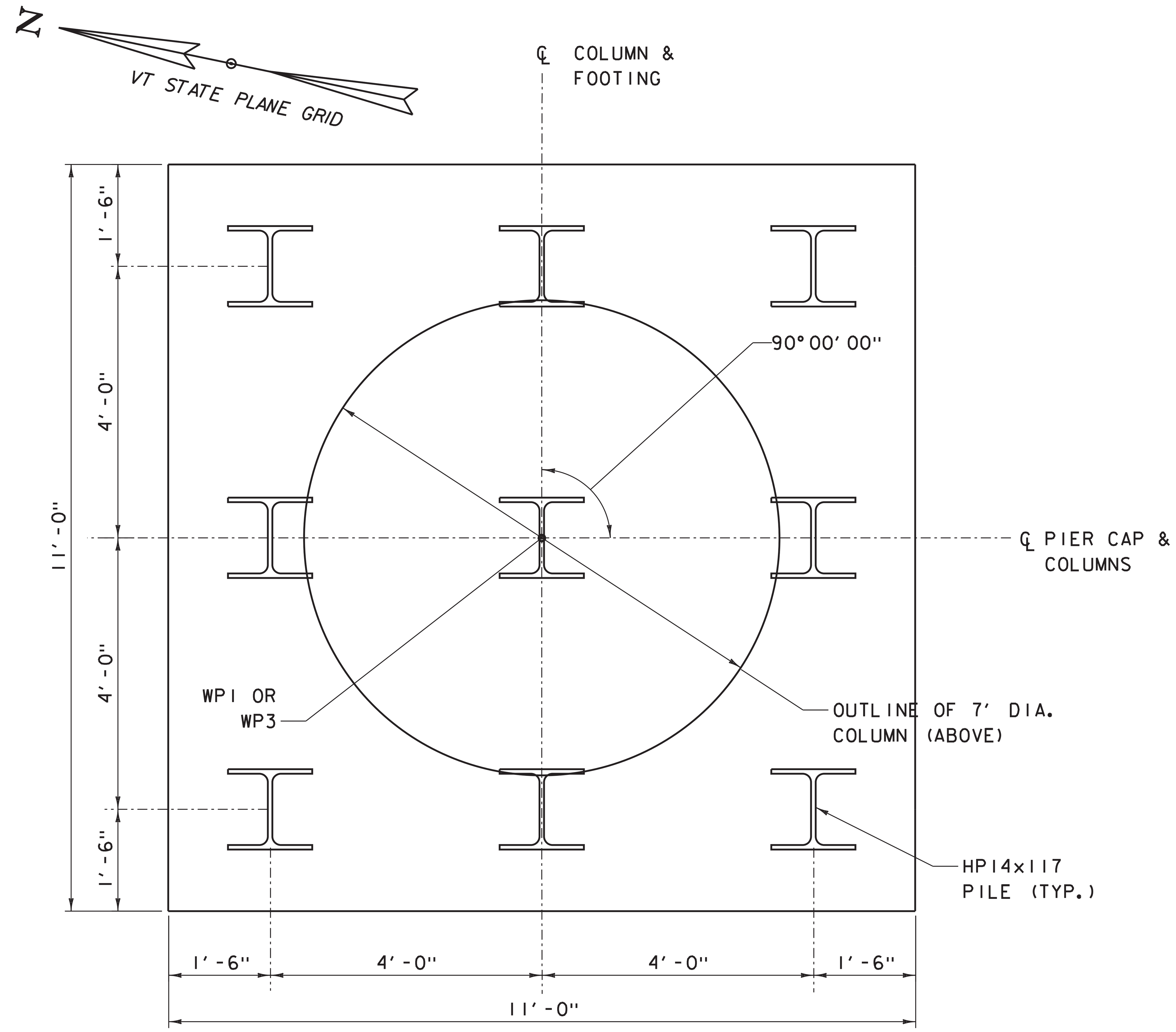
LOCATION	STATION	NORTHING	EASTING
WP1	1285+06.61, RT. 19.21'	321780.4263	1601189.3298
WP2	1284+96.37, RT. 8.17'	321765.6797	1601192.3956
WP3	1284+88.93, RT. 0.14'	321754.9658	1601194.6230
WP4	1284+81.49, LT. 7.88'	321744.2493	1601196.8509
WP5	1284+71.25, LT. 18.92'	321729.5101	1601199.9152

PROJECT NAME: CAVENDISH
PROJECT NUMBER: GMRC(24)

Jacobs

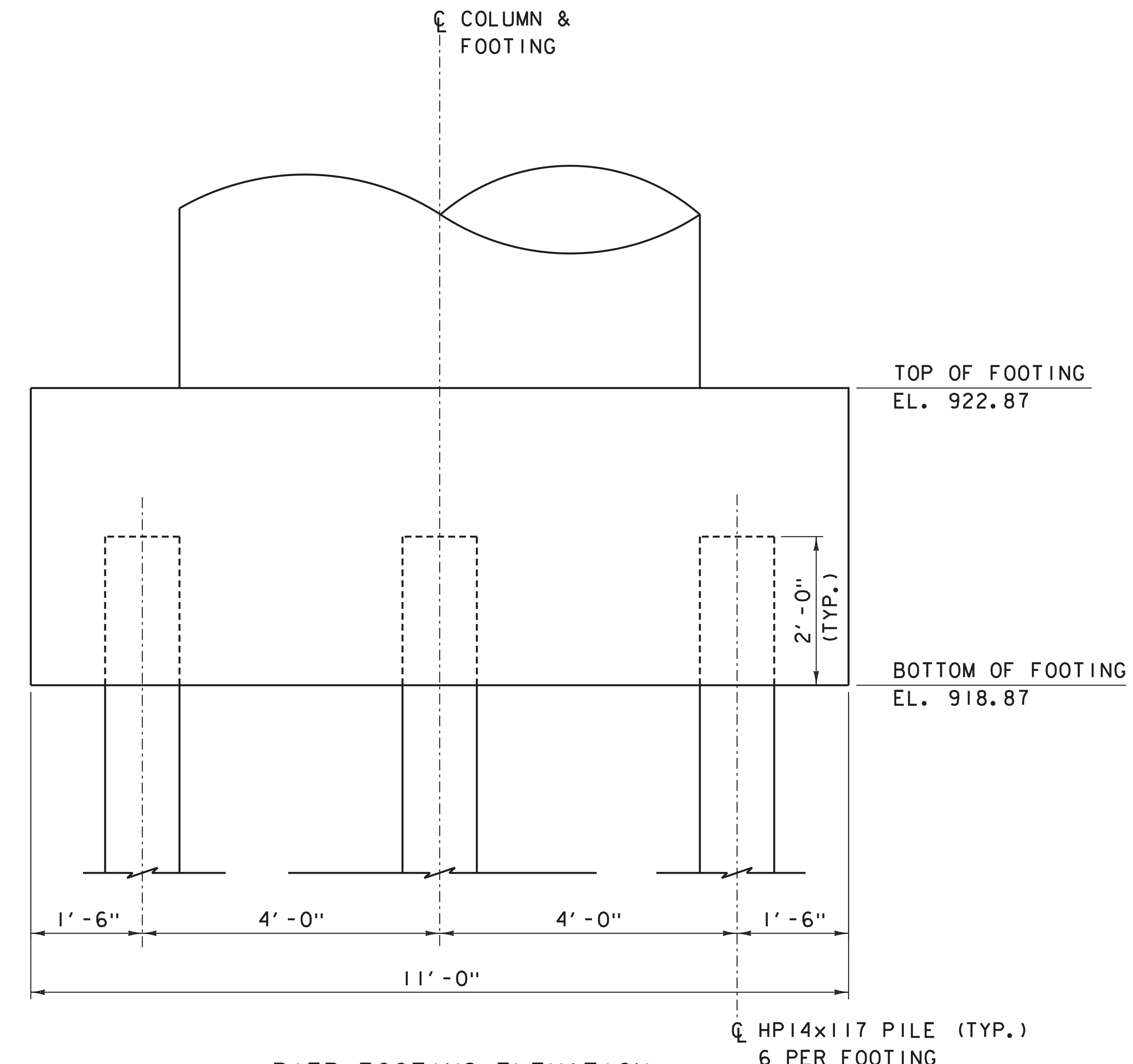
FILE NAME: z15g155pier01.dgn
PROJECT LEADER: J. WILSON
DESIGNED BY: A. WALL
PIER PLAN, ELEVATION & SECTION

PLOT DATE: 11/17/2021
DRAWN BY: S. GUNN
CHECKED BY: S. HALLORAN
SHEET 50 OF 98



PIER FOOTING PLAN

SCALE ¾" = 1'-0"



PIER FOOTING ELEVATION

SCALE ¾" = 1'-0"

PILE NOTES:

1. FURNISHING EQUIPMENT FOR DRIVING PILING WILL BE PAID UNDER ITEM 504.10, "FURNISHING EQUIPMENT FOR DRIVING PILING". STEEL PILING (HP 14x117) INSTALLATION, ENGINEERING ANALYSIS, SUBMISSIONS, SPLICES, REINFORCING, SURVEYING, PILE DRIVING SHOES, TEMPLATES AND ALL OTHER INCIDENTALS WILL BE PAID UNDER ITEM 505.20, "STEEL PILING, HP 14 X 117". DYNAMIC PILE LOAD TESTING WILL BE PAID UNDER ITEM 505.45,"DYNAMIC PILE LOAD TESTING".

2. PILE SHOES SHALL BE USED ON ALL PILES AND SHALL CONFORM TO SUBSECTION 505.04 (f).

3. PERFORM TWO DYNAMIC PILE LOAD TESTS, ONE PILE AT EACH PIER FOOTING.

4. STUDY SITE AND AVAILABLE DESCRIPTIONS OF SUBSURFACE SOIL, ROCK AND GROUNDWATER CONDITIONS PRIOR TO SUBMITTING A BID FOR THE WORK. CONTRACTOR SHALL DRAW ITS OWN CONCLUSIONS REGARDING SITE CONDITIONS, BASED UPON SITE VISIT(S) AND FROM OTHER AVAILABLE SOURCES. INFORMATION IN CONTRACT DOCUMENTS IS MADE AVAILABLE TO CONTRACTOR FOR INFORMATION ONLY AND SHALL NOT BE INTERPRETED AS A WARRANTY OF SUBSURFACE OR ENVIRONMENTAL CONDITIONS.

5. PROTECT THE ENVIRONMENT, STRUCTURES, UTILITIES, IMPROVEMENTS AND PROPERTIES IN THE VICINITY OF THE WORK FROM DAMAGE ASSOCIATED WITH PILE DRIVING AND OTHER RELATED OPERATIONS. IMMEDIATELY REPAIR ANY DAMAGE RESULTING FROM CONTRACTOR'S ACTIVITIES TO ENGINEER'S SATISFACTION.

2.25 650

6. DRIVE PILES TO END BEARING ON ROCK, TO APPROVED DRIVING CRITERIA AND TO THE REQUIRED ULTIMATE GEOTECHNICAL COMPRESSION CAPACITY OF 520 KIPS USING A FACTOR OF SAFETY OF 2.0. THE FINAL DRIVING CRITERIA SHALL BE APPROVED BY THE ENGINEER BASED ON THE RESULTS OF THE PDA TESTING AND CAPWAP ANALYSES.

7. ESTIMATED PILE LENGTH IS 59 FEET (INCLUDING 2 FEET EMBEDMENT IN PIER FOOTING). THE ACTUAL IN PLACE LENGTH MAY VARY. FINAL TOLERANCE ON CUTOFF ELEVATION SHALL NOT EXCEED +/- 2 INCHES.

8. THE MINIMUM PILE FIXITY DEPTH BELOW BOTTOM OF PILE CAP IS 26 FEET, CORRESPONDING TO ELEVATION 892.87 FEET.

9. DO NOT PREDRILL HOLES OR USE FOLLOWERS.

10. DRIVE ALL PILES PLUMB/VERTICAL AT ALL LOCATIONS. USE TEMPLATES OR OTHER METHODS TO POSITION THE PILES AT THEIR DESIGN LOCATION.

11. THE MAXIMUM ALLOWABLE PILE DRIVING STRESS IS 40 KSI (I.E., 0.8*Fy WHERE Fy = 50 KSI).

FOOTING AND COLUMN NOTES:

1. PIER COLUMN CONCRETE WILL BE PAID FOR UNDER ITEM 900.608, "SPECIAL PROVISION (PERFORMANCE-BASED CONCRETE, CLASS PCD)".

2. PIER FOOTING CONCRETE WILL BE PAID FOR UNDER ITEM 900.608, "SPECIAL PROVISION (PERFORMANCE-BASED CONCRETE, CLASS PCS)".

3. REINFORCEMENT IN PIER FOOTINGS AND COLUMNS WILL BE PAID FOR UNDER ITEM 507.11, "REINFORCING STEEL, LEVEL I" AND SHALL BE EPOXY COATED.

Jacobs

PROJECT NAME: CAVENDISH	
PROJECT NUMBER: GMRC(24)	
FILE NAME: z15g155pierdtl04.dgn	PLOT DATE: 11/17/2021
PROJECT LEADER: J. WILSON	DRAWN BY: S. GUNN
DESIGNED BY: A. WALL	CHECKED BY: S. HALLORAN
PIER DETAILS 4	SHEET 55 OF 98

Appendix B. Site Photos

Figure B1: Boring B-4 Drilling (adjacent to the East Abutment)

Figure B2: Boring B-2 Drilling (on the east side of the Center Pier)

Figure B3: Existing Center Pier - Looking West from the East Abutment

Figure B4: Existing Center Pier - Looking East from the West Abutment

Figure B5: Existing East Abutment

Figure B6: Existing West Abutment

Appendix C. Subsurface Exploration Logs and Rock Core Photos

Appendix C.1 Subsurface Exploration Logs

STATE OF VERMONT
AGENCY OF TRANSPORTATION
MATERIALS & RESEARCH SECTION
SUBSURFACE INFORMATION

BORING LOG

Cavendish, VT
E2X88317
VTrans Bridge 132

Boring No.: **B-1**
Page No.: 1 of 2
Pin No.:
Checked By: Da Ha

Boring Crew: P. LaBossiere, NEBC, N. Huaman, JEG
Date Started: 9/12/19 Date Finished: 9/13/19
VTSPG NAD83: N 321761.20 ft E 1601184.00 ft
Station: Offset:
Ground Elevation: 941.4 ft

	Casing	Sampler
Type:	WASH BORE	SS
I.D.:	4 in	1.5 in
Hammer Wt:	300 lb.	140 lb.
Hammer Fall:	30 in.	30 in.
Hammer/Rod Type:	Auto	
Rig:	MOBILE DRILL	C_E = 1.25

Groundwater Observations

Date	Depth (ft)	Notes

Depth (ft)	Strata (1)	CLASSIFICATION OF MATERIALS (Description)	Blows/6" (N Value)	Moisture Content %	Gravel %	Sand %	Fines %
5							
10							
15							
20		S1: dense, f-m SAND and SILT, little f-c Gravel, Dark brown, Wet, Rec. = 1.0 ft, 19.0 ft - 21.0 ft	30-19-19-9 (38)				
25		S2: very dense, f-m SAND, little Silt, trace f Gravel, Dark brown, Wet, Rec. = 0.92 ft, 24.0 ft - 26.0 ft	22-26-31-49 (57)				
30		S3: very dense, f-m SAND, little Silt, little f-c Gravel, Dark brown, Wet, Rec. = 1.25 ft, 29.0 ft - 31.0 ft	33-54-45-43 (99)				
35		S4: SAME AS ABOVE, Dark brown and gray, Wet, Rec. = 1.58 ft, 34.0 ft - 36.0 ft	36-47-126-118 (173)				
		36.0 ft - 79.5 ft, Drill Bit advanced to top of rock					
40							
45							

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

STATE OF VERMONT
AGENCY OF TRANSPORTATION
MATERIALS & RESEARCH SECTION
SUBSURFACE INFORMATION

BORING LOG

Cavendish, VT
E2X88317
VTrans Bridge 132

Boring No.: **B-1**

Pin No.:
Checked By: Da Ha

Boring Crew: P. LaBossiere, NEBC, N. Huaman, JEG
Date Started: 9/12/19 Date Finished: 9/13/19
VTSPG NAD83: N 321761.20 ft E 1601184.00 ft
Station: Offset:
Ground Elevation: 941.4 ft

	Casing	Sampler
Type:	WASH BORE	SS
I.D.:	4 in	1.5 in
Hammer Wt:	300 lb.	140 lb.
Hammer Fall:	30 in.	30 in.
Hammer/Rod Type:	Auto	
Rig:	MOBILE DRILL	C_E = 1.25

Groundwater Observations

Date	Depth (ft)	Notes

Depth (ft)	Strata (1)	CLASSIFICATION OF MATERIALS (Description)	Blows/6" (N Value)	Moisture Content %	Gravel %	Sand %	Fines %
55							
60							
65							
70							
75							
80		Hole stopped @ 79.5 ft	Top of Bedrock @ 79.5 ft				
85							
90							
95							

Remarks:
1. DRILL RIG WAS SET UP ON THE BRIDGE AT THE TOP OF RAIL ELEVATION. BORING GROUND ELEVATION REFERS TO THE TOP OF RAIL ELEVATION.

2. DISTANCE FROM THE TOP OF RAIL TO RIVERBED IS ABOUT 16 FT.

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

STATE OF VERMONT AGENCY OF TRANSPORTATION MATERIALS & RESEARCH SECTION SUBSURFACE INFORMATION	**BORING LOG** **Cavendish, VT** **E2X88317** **VTrans Bridge 132**

Boring No.: **B-2**
Page No.: 1 of 2
Pin No.:
Checked By: Da Ha

Boring Crew: P. LaBossiere, NEBC, N. Huaman, JEG
Date Started: 9/11/19 Date Finished: 9/12/19
VTSPG NAD83: N 321749.20 ft E 1601203.90 ft
Station: Offset:
Ground Elevation: 941.4 ft

	Casing	Sampler
Type:	WASH BORE	SS
I.D.:	4 in	1.5 in
Hammer Wt:	300 lb.	140 lb.
Hammer Fall:	30 in.	30 in.
Hammer/Rod Type:	Auto	
Rig:	MOBILE DRILL	C_E = 1.25

Groundwater Observations

Date	Depth (ft)	Notes
09/11/19	13.5	See Remark

Depth (ft)	Strata (1)	CLASSIFICATION OF MATERIALS (Description)	Run (Dip deg.)	Core Rec. % (RQD %)	Blows/6" (N Value)	Moisture Content %	Gravel %	Sand %	Fines %
5									
10									
15		S1: dense, f-c SAND and SILT, little f-c Gravel, gray, Moist, Rec. = 0.58 ft, 13.5 ft - 15.5 ft			7-13-19-20 (32)				
20		S2: very dense, f-m SAND and SILT, trace f-c Gravel, brownish gray, Moist, Rec. = 1.17 ft, 19.5 ft - 21.5 ft			20-25-33-49 (68)				
25		S3: very dense, Top 10": fine SAND and SILT, trace f-c Gravel, brownish gray, Wet, Rec. = 1.42 ft, 24.5 ft - 26.5 ft, Bottom 7": f-c GRAVEL (1 piece =1", the rest is f-m gravel)			27-50-31-33 (81)				
30		S4: very dense, f-m SAND and SILT, some f-c Gravel, Dark brown, Wet, Rec. = 1.67 ft, 29.5 ft - 31.5 ft			48-51-26-58 (77)				
35		S5: very dense, f-m SAND and SILT, little f-c Gravel, Dark brown/gray, Wet, Rec. = 0.5 ft, 34.5 ft - 36.5 ft			64-72-78-56 (150)				
40		S6: very dense, f-m SAND, little Silt, little f-c Gravel, Dark brown, Wet, Rec. = 1.25 ft, 39.5 ft - 41.5 ft, color of gravel ranged from black orange and white			31-27-30-37 (57)				
45		S7: very dense, f-m SAND, some Silt, some f-c Gravel, Dark brown, Wet, Rec. = 1.67 ft, 44.5 ft - 46.5 ft, color of gravel ranged from black orange and white			80-66-75-118 (141)				

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

STATE OF VERMONT AGENCY OF TRANSPORTATION MATERIALS & RESEARCH SECTION SUBSURFACE INFORMATION	BORING LOG **Cavendish, VT** **E2X88317** **VTrans Bridge 132**	Boring No.: **B-2** Page No.: 2 of 2 Pin No.: Checked By: Da Ha

Boring Crew: P. LaBossiere, NEBC, N. Huaman, JEG
Date Started: 9/11/19 Date Finished: 9/12/19
VTSPG NAD83: N 321749.20 ft E 1601203.90 ft
Station: Offset:
Ground Elevation: 941.4 ft

	Casing	Sampler
Type:	WASH BORE	SS
I.D.:	4 in	1.5 in
Hammer Wt:	300 lb.	140 lb.
Hammer Fall:	30 in.	30 in.
Hammer/Rod Type:	Auto	
Rig:	MOBILE DRILL	C_E = 1.25

Groundwater Observations

Date	Depth (ft)	Notes
09/11/19	13.5	See Remark

Depth (ft)	Strata (1)	CLASSIFICATION OF MATERIALS (Description)	Run (Dip deg.)	Core Rec. % (RQD %)	Blows/6" (N Value)	Moisture Content %	Gravel %	Sand %	Fines %
		S8: very dense, f-c SAND, some Gravel, little Silt, gray, Wet, Rec. = 0.5 ft, 49.5 ft - 50.33 ft, color of gravel ranged from black gray and white 175 blows with 300lb hammer, 50.5 ft - 54.5 ft			112-100				
55		S9: very dense, f-m SAND, some Silt, trace f-c Gravel, gray, Wet, Rec. = 0.33 ft, 54.5 ft - 55.25 ft			107-100				
60		S10: very dense, f-c SAND, some Silt, trace c Gravel, gray, Wet, Rec. = 0.92 ft, 59.5 ft - 60.42 ft			45-107				
65		S11: very dense, f-m SAND, some Silt, little f-c Gravel, gray, Wet, Rec. = 1.0 ft, 64.5 ft - 64.75 ft, Probably wash			100				
70		S12: very dense, f-c GRAVEL, gray-black-orange, Wet, Rec. = 0.13 ft, 69.5 ft - 69.75 ft			100				
75		S13: very dense, f-m SAND, some Silt, trace f-c Gravel, gray, Wet, Rec. = 0.33 ft, 74.5 ft - 74.83 ft			100				
		Top of Bedrock @ 75.5 ft							
		75.5 ft - 80.5 ft, Gray, SCHIST: Dark gray, very hard, medium grained, slightly to moderately fractured, very slightly weathered to fresh, horizontal to 45 degree dipping close joints, mineralization and discoloration present at the joints	C1	100 (83)					
80									

Hole stopped @ 80.5 ft

Remarks:
1. DRILL RIG WAS SET UP ON THE BRIDGE AT THE TOP OF RAIL ELEVATION. BORING GROUND ELEVATION REFERS TO THE TOP OF RAIL ELEVATION.

2. DISTANCE FROM THE TOP OF RAIL TO STREAM WATER IS ABOUT 13.5 FT.

85

90

95

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

VTrans Working to Get You There — Vermont Agency of Transportation	STATE OF VERMONT AGENCY OF TRANSPORTATION MATERIALS & RESEARCH SECTION SUBSURFACE INFORMATION	BORING LOG **Cavendish, VT** **E2X88317** **VTrans Bridge 132**	Boring No.: **B-3** Page No.: 1 of 1 Pin No.: Checked By: Da Ha

Boring Crew: P. LaBossiere, NEBC, N. Huaman, JEG
Date Started: 9/10/19 Date Finished: 9/10/19
VTSPG NAD83: N 321810.50 ft E 1601102.10 ft
Station: Offset:
Ground Elevation: 940.7 ft

	Casing	Sampler
Type:	WASH BORE	SS
I.D.:	4 in	1.5 in
Hammer Wt:	300 lb.	140 lb.
Hammer Fall:	30 in.	30 in.
Hammer/Rod Type:	Auto	
Rig:	MOBILE DRILL	C_E = 1.25

Groundwater Observations

Date	Depth (ft)	Notes

Depth (ft)	Strata (1)	CLASSIFICATION OF MATERIALS (Description)	Blows/6" (N Value)	Moisture Content %	Gravel %	Sand %	Fines %
		87 BLOWS with 300lb Hammer, 0.0 ft - 4.0 ft					
5		S1: medium dense, f-m SAND, little Silt, little f-c Gravel, Dark brown, Moist, Rec. = 0.71 ft, 4.0 ft - 6.0 ft	7-7-8-6 (15)				
10		S2: dense, f SAND, little m-c Gravel, gray, Moist, Rec. = 1.08 ft, 9.0 ft - 11.0 ft	19-16-14-7 (30)				
		148 BLOWS with 300 lb Hammer, 11.0 ft - 14.0 ft					
15		S3: very dense, f-c GRAVEL, trace f Sand, gray-brown, Wet, Rec. = 0.58 ft, 14.0 ft - 16.0 ft	50-79-53-62 (132)				
		141 BLOWS with 300 lb Hammer, 16.0 ft - 19.0 ft					
20		S4: very dense, f-m SAND, some Silt, little f-c Gravel, Gray brown, Wet, Rec. = 0.92 ft, 19.0 ft - 21.0 ft	47-36-25-14 (71)				
		279 BLOWS with 300 lb Hammer, 21.0 ft - 24.0 ft					
25		S5: very dense, f-c GRAVEL, trace Sand, gray, Wet, Rec. = 0.67 ft, 24.0 ft - 26.0 ft	29-29-33-24 (62)				
		Wash and roller bit for 12 minutes, 26.0 ft - 27.0 ft, After drilling to 27 ft had to stop drilling due to incoming train.					
		Advanced drilling by roller bit, 27.0 ft - 28.0 ft					
30		Hole stopped @ 28.0 ft					
		Remarks: Possible top of bedrock at 26 ft.					
35							
40							
45							

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

STATE OF VERMONT AGENCY OF TRANSPORTATION MATERIALS & RESEARCH SECTION SUBSURFACE INFORMATION	BORING LOG **Cavendish, VT** **E2X88317** **VTrans Bridge 132**	Boring No.: **B-4** Page No.: 1 of 1 Pin No.: Checked By: Da Ha

Boring Crew: P. LaBossiere, NEBC, N. Huaman, JEG
Date Started: 9/09/19 Date Finished: 9/09/19
VTSPG NAD83: N 321699.60 ft E 1601286.10 ft
Station: Offset:
Ground Elevation: 939.9 ft

	Casing	Sampler
Type:	WASH BORE	SS
I.D.:	4 in	1.5 in
Hammer Wt:	300 lb.	140 lb.
Hammer Fall:	30 in.	30 in.
Hammer/Rod Type:	Auto	
Rig:	MOBILE DRILL	C_E = 1.25

Groundwater Observations

Date	Depth (ft)	Notes

Depth (ft)	Strata (1)	CLASSIFICATION OF MATERIALS (Description)	Blows/6" (N Value)	Moisture Content %	Gravel %	Sand %	Fines %
5		S1: very dense, f-m SAND, some Silt, little f-c Gravel, Dark brown, Wet, Rec. = 0.33 ft, 4.0 ft - 6.0 ft	100-53-26-12 (79)				
10		S2: dense, f-m SAND, some Silt, little f-c Gravel, brown-gray, Wet, Rec. = 0.92 ft, 9.0 ft - 11.0 ft	45-19-26-36 (45)				
		127 BLOWS with 300 lb Hammer, 11.0 ft - 12.2 ft					
		Roller bit for approx. 2 minutes, penetrated 3", 12.2 ft - 12.4 ft, After roller bit to 12.4 ft, had to stop due to incoming train. Left casing in place, came back to borehole 30 minutes later.					
15		Roller bit for approx. 35 minutes, 12.4 ft - 14.0 ft					
		Hole stopped @ 14.0 ft					

Remarks:
Possible top of bedrock at 12.2 ft.

Depth scale: 5, 10, 15, 20, 25, 30, 35, 40, 45

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

Appendix C.2 Rock Core Photos

Figure C1: Boring B-2, C1 (75.5 – 80.5 ft) - Wet

Figure C2: Boring B-2, C1 (75.5 – 80.5 ft) – Partially Dry

Figure C3: Boring B-2, C1 (75.5 – 80.5 ft) – Close-up Photo #1

Figure C4: Boring B-2, C1 (75.5 – 80.5 ft) – Close-up Photo #2

Appendix D. Soil Parameters

Geotechnical Parameters for Pile Analyses (GROUP and APILE) – VTrans Bridge 132

Soil Layer	Layer Top Elevation	Layer Bottom Elevation	Top Strata Depth (ft)	Bottom Strata Depth (ft)	Moist Unit Weight (pcf)	Submerged Unit Weight (pcf)	Friction Angle (deg)	Subgrade modulus, k (pci)	Unconfined Compressive Strength (psi)
Sand	919	862	0	57	125	62.6	Ø = 36°	125 (below GWT)	-
Bedrock	862	849	57	70	150	87.6	-	-	4000

Notes:

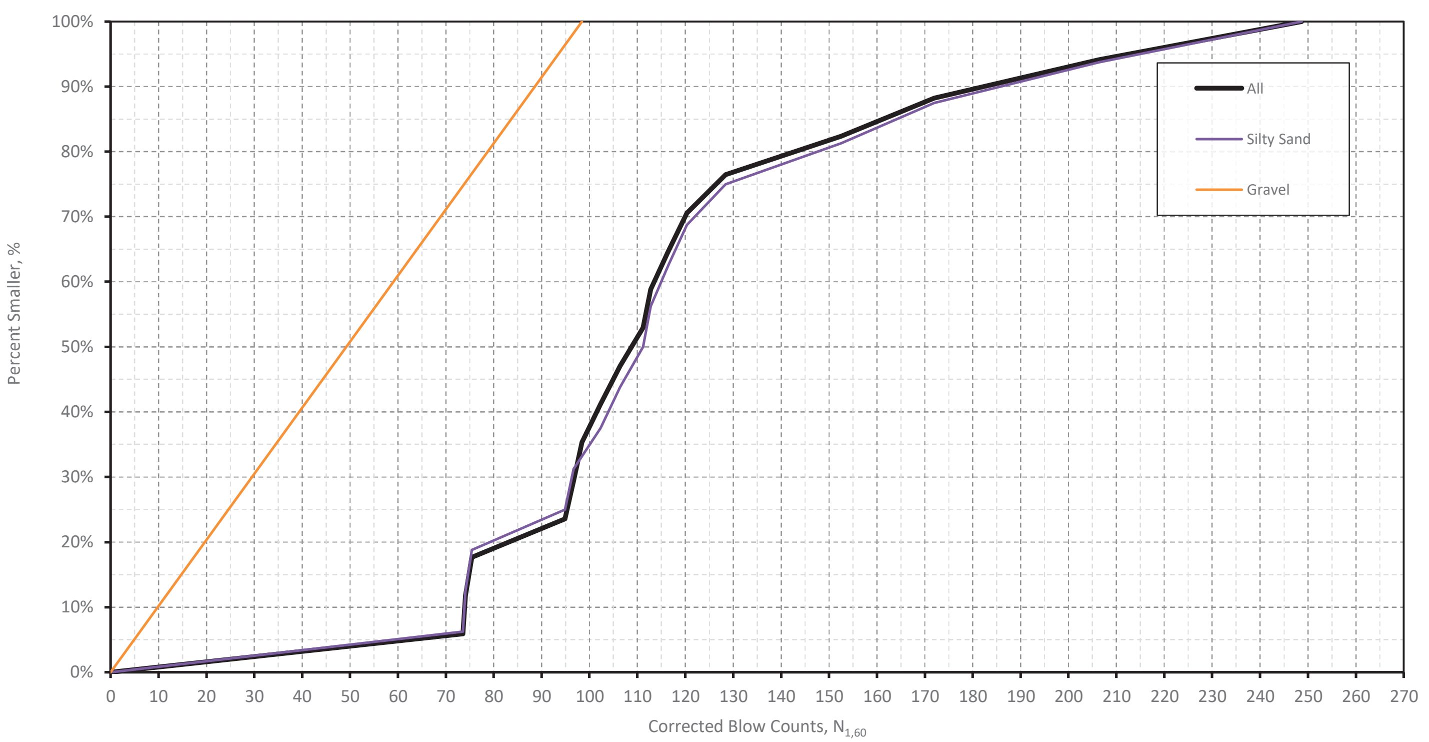
1. Soil profile in the table is based on borings B-1 and B-2. The top 8 feet of soil is neglected considering potential scour at the center pier.
2. Groundwater level is assumed at El. 927 feet (i.e., same as the riverbed elevation).
3. Bottom of abutment footing is assumed at El. 919 feet.
4. All depths are relative to the bottom of pile cap elevation.
5. A friction angle of 36 degrees was conservatively selected for the sand layer, although the SPT blow count correlations indicate higher values.
6. Unit weight and unconfined compressive strength of bedrock were estimated based on rock core descriptions in boring B-2 for Schist.

			Sheet No.	1
Project:	VTrans Bridge 132	Authored by: DH	Date:	12/10/2021
Job No.	E2X88317	Checked by: MM	Date:	12/20/2021

N Value Distribution and Inferred Friction Angle

Uses information from Borings: B-1, B-2/2A

Material	$N_{1,60}$ Values					Representative Value	Inferred ϕ (deg)
	35th %	Median	65th %	Geomean	Arithmean		
All	98.3	108.8	116.6	115.9	123.0		
Silty Sand	100.0	111.2	118.0	117.1	124.5	100	52
Gravel	34.4	49.2	64.0	98.4	98.4	98	51



All						
Boring	El.	Sample	Depth	Material	$N_{1,60}$ 0	% 0
B-2	900.9	S6	27.0	Silty Sand	73.6	6%
B-2	926.9	S1	1.0	Silty Sand	74.1	12%
B-1	921.4	S1	4	Silty Sand	75.4	18%
B-2	866.7	S13	61.2	Silty Sand	94.8	24%
B-1	916.4	S2	9.0	Silty Sand	96.7	29%
B-2	871.7	S12	56.2	Gravel	98.4	35%
B-2	876.7	S11	51.2	Silty Sand	102.3	41%
B-2	881.4	S10	46.5	Silty Sand	106.3	47%
B-2	886.5	S9	41.4	Silty Sand	111.2	53%
B-2	910.9	S4	17.0	Silty Sand	112.8	59%
B-2	891.4	S8	36.5	Silty Sand	116.4	65%
B-2	920.9	S2	7.0	Silty Sand	120.3	71%
B-2	915.9	S3	12.0	Silty Sand	128.4	76%
B-1	911.4	S3	14.0	Silty Sand	152.6	82%
B-2	895.9	S7	32	Silty Sand	171.9	88%
B-2	905.9	S5	22.0	Silty Sand	206.4	94%
B-1	906.4	S4	19.0	Silty Sand	248.7	100%

Silty Sand						
Boring	**El.**	**Sample**	**Depth**	**Material**	**$N_{1,60}$** 0	**%** 0
B-2	900.9	S6	27.0	Silty Sand	73.6	**6%**
B-2	926.9	S1	1.0	Silty Sand	74.1	**13%**
B-1	921.4	S1	4	Silty Sand	75.4	**19%**
B-2	866.7	S13	61.2	Silty Sand	94.8	**25%**
B-1	916.4	S2	9.0	Silty Sand	96.7	**31%**
B-2	876.7	S11	51.2	Silty Sand	102.3	**38%**
B-2	881.4	S10	46.5	Silty Sand	106.3	**44%**
B-2	886.5	S9	41.4	Silty Sand	111.2	**50%**
B-2	910.9	S4	17.0	Silty Sand	112.8	**56%**
B-2	891.4	S8	36.5	Silty Sand	116.4	**63%**
B-2	920.9	S2	7.0	Silty Sand	120.3	**69%**
B-2	915.9	S3	12.0	Silty Sand	128.4	**75%**
B-1	911.4	S3	14.0	Silty Sand	152.6	**81%**
B-2	895.9	S7	32	Silty Sand	171.9	**88%**
B-2	905.9	S5	22.0	Silty Sand	206.4	**94%**
B-1	906.4	S4	19.0	Silty Sand	248.7	**100%**

Gravel						
Boring	El.	Sample	Depth	Material	$N_{1,60}$ 0	% 0
B-2	871.7	S12	56.2	Gravel	98.4	**100%**

CALCULATION SHEET

Sheet No. 1

Project:	VTrans Bridge 132 Rehabilitation	Authored by:	DH	Date	12/10/2021
Job No.	E2X88317	Checked by:	MM	Date	12/20/2021

Corrected N Value for Estimation of Soil Strength Parameters

Boring No. B-1
Ground Surface Elevation 925.4 ft (NAVD 88)
Ground Water Depth during Drilling 0.0 ft
Hammer Efficiency 75 %

Hammer Type Automatic
Sampler Type Standard
SPT Rod Stickup 21.0 ft

References:
1. FHWA-IF-02-034 (2002)
2. FHWA-NHI-10-0.16 (May 2010)
3. NAVFAC DM-7 (March 1971)
4. IDOT AGMU Memo 10.2
5. Valiquette, Robinson, and Borden (2009)

Elevation of Sample	Boring Data: Sample No.	Sample Depth		Material	Borehole Diameter	SPT Correction Factor					Corrected SPT N Value, N_{60}	Vertical Effective Stress	Corrected SPT N Value, $N_{1,60}$
(ft)	-	(ft)	(blows/ft)	-	(in)	C_N	C_E	C_R	C_B	C_S	(blows/ft)	(psf)	(blows/ft)
921.4	S1	4	38	Silty Sand	4	1.67	1.25	0.95	1.00	1.00	**45.1**	270	**75.4**
916.4	S2	9	57	Silty Sand	4	1.40	1.25	0.97	1.00	1.00	**69.1**	608	**96.7**
911.4	S3	14	99	Silty Sand	4	1.25	1.25	0.98	1.00	1.00	**121.9**	946	**152.6**
906.4	S4	19	173	Silty Sand	4	1.15	1.25	1.00	1.00	1.00	**216.3**	1,284	**248.7**

Notation:
N_{60} = SPT blow count corrected for hammer efficiency (blows/ft)
$N_{1,60}$ = SPT blow count corrected for hammer efficiency and overburden pressure.
C_N = SPT correction factor for overburden pressure, C_N = min[0.77*log(40/σ'v),2]
C_E = SPT correction factor for hammer energy, C_E = 60/ER
C_R = SPT Correction factor for rod length, C_R = -8E-5*L^2+0.0083*L+0.7922
C_B = SPT Correction factor for borehole diameter, C_B = 1.0 if D < 4.5", 1.05 if 4.5" < D < 6", 1.15 if D > 6"
C_S = SPT Correction factor for sampler type, C_S = 1.0 if standard sampler, 1.2 if liner space without liners

Notes:
1) Overburden pressure calculated using a unit weight based on IDOT AGMU Memo 10.2
2) Rod length assumes 5' stickup during SPTs, L = depth + 5'

IDOT AGMU Memo 10.2 Unit Weights
Max Unit Weight: $\gamma_{granular, dry}$ (pcf)= 130
$\gamma_{granular, submerged}$ (pcf)= 67.6
Above Water Table: $\gamma_{granular} = 95*(N_{60})^{0.095}$ [pcf]
Below Water Table: $\gamma_{granular} = 105*(N_{60})^{0.07} - 62.4$ [pcf]
Above Water Table: $\gamma_{cohesive} = 121.5*(Qu)^{0.095}$ [pcf]
Below Water Table: $\gamma_{cohesive} = 121.5*(Qu)^{0.095} - 62.4$ [pcf]

CALCULATION SHEET

Sheet No. 1

Project:	VTrans Bridge 132 Rehabilitation	Authored by:	DH	Date	12/10/2021
Job No.	E2X88317	Checked by:	MM	Date	12/20/2021

Corrected N Value for Estimation of Soil Strength Parameters

Boring No.	B-2		**Hammer Type**	Automatic		References: 1. FHWA-IF-02-034 (2002)
Ground Surface Elevation	927.9	ft (NAVD 88)	**Sampler Type**	Standard		2. FHWA-NHI-10-0.16 (May 2010)
Ground Water Depth during Drilling	0.0	ft	**SPT Rod Stickup**	19.0	ft	3. NAVFAC DM-7 (March 1971)
Hammer Efficiency	75	%				4. IDOT AGMU Memo 10.2
						5. Valiquette, Robinson, and Borden (2009)

Elevation of Sample	Boring Data					SPT Correction Factor					Corrected SPT N Value, N_{60}	Vertical Effective Stress	Corrected SPT N Value, $N_{1,60}$
	Sample No.	Sample Depth	SPT N Value	Material	Borehole Diameter								
(ft)	-	(ft)	(blows/ft)	-	(in)	C_N	C_E	C_R	C_B	C_S	(blows/ft)	(psf)	(blows/ft)
926.9	S1	1	32	Silty Sand	4	2.00	1.25	0.93	1.00	1.00	**37.0**	68	**74.1**
920.9	S2	7	68	Silty Sand	4	1.48	1.25	0.95	1.00	1.00	**81.1**	473	**120.3**
915.9	S3	12	81	Silty Sand	4	1.30	1.25	0.97	1.00	1.00	**98.5**	811	**128.4**
910.9	S4	17	77	Silty Sand	4	1.19	1.25	0.99	1.00	1.00	**95.0**	1,149	**112.8**
905.9	S5	22	150	Silty Sand	4	1.10	1.25	1.00	1.00	1.00	**187.5**	1,487	**206.4**
900.9	S6	27	57	Silty Sand	4	1.03	1.25	1.00	1.00	1.00	**71.3**	1,825	**73.6**
895.9	S7	32	141	Silty Sand	4	0.98	1.25	1.00	1.00	1.00	**176.3**	2,163	**171.9**
891.4	S8	36.5	100	Silty Sand	4	0.93	1.25	1.00	1.00	1.00	**125.0**	2,467	**116.4**
886.5	S9	41.4	100	Silty Sand	4	0.89	1.25	1.00	1.00	1.00	**125.0**	2,799	**111.2**
881.4	S10	46.5	100	Silty Sand	4	0.85	1.25	1.00	1.00	1.00	**125.0**	3,143	**106.3**
876.7	S11	51.2	100	Silty Sand	4	0.82	1.25	1.00	1.00	1.00	**125.0**	3,461	**102.3**
871.7	S12	56.2	100	Gravel	4	0.79	1.25	1.00	1.00	1.00	**125.0**	3,799	**98.4**
866.7	S13	61.2	100	Silty Sand	4	0.76	1.25	1.00	1.00	1.00	**125.0**	4,137	**94.8**

Notation:
N_{60} = SPT blow count corrected for hammer efficiency (blows/ft)
$N_{1,60}$ = SPT blow count corrected for hammer efficiency and overburden pressure.
C_N = SPT correction factor for overburden pressure, C_N = min[0.77*log(40/σ'v),2]
C_E = SPT correction factor for hammer energy, C_E = 60/ER
C_R = SPT Correction factor for rod length, C_R = -8E-5*L^2+0.0083*L+0.7922
C_B = SPT Correction factor for borehole diameter, C_B = 1.0 if D < 4.5", 1.05 if 4.5" < D < 6", 1.15 if D > 6"
C_S = SPT Correction factor for sampler type, C_S = 1.0 if standard sampler, 1.2 if liner space without liners

Notes:
1) Overburden pressure calculated using a unit weight based on IDOT AGMU Memo 10.2
2) Rod length assumes 5' stickup during SPTs, L = depth + 5'

IDOT AGMU Memo 10.2 Unit Weights
Max Unit Weight: $\gamma_{granular, dry}$ (pcf)= 130
$\gamma_{granular, submerged}$ (pcf)= 67.6
Above Water Table: $\gamma_{granular} = 95*(N_{60})^{0.095}$ [pcf]
Below Water Table: $\gamma_{granular} = 105*(N_{60})^{0.07} - 62.4$ [pcf]
Above Water Table: $\gamma_{cohesive} = 121.5*(Qu)^{0.095}$ [pcf]
Below Water Table: $\gamma_{cohesive} = 121.5*(Qu)^{0.095} - 62.4$ [pcf]

Appendix E. Seismic Site Class Evaluation

Jacobs
120 St James Ave
Boston, MA 02116
617-242-9222

JOB	VTrans Bridge 132		
SUBJECT	Seismic Site Class		
CALCULATED BY	DH	DATE	12/9/2021
CHECKED BY	MM	DATE	12/21/2021

Seismic Site Class Summary

PURPOSE: Determine seismic site class for VTrans Bridge 132 in accordance with 2019 AREMA Manual for Railway Engineering, Chapter 9.

SUBSURFACE INFORMATION: SPT borings performed by New England Boring Contractors (NEBC) in 2019.

APPROACH:
(Per Table 9-1-6)

1) Check for the four categories of Site Class F requiring site-specific evaluation:
   - Soil vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, and collapsible weakly cemented soils.
   - Peats or highly organic clays (H > 10 feet of peat or highly organic clay, where H = thickness of soil)
   - Very high plasticity clays (H > 25 feet with PI > 75)
   - Very thick soft/medium stiff clays (H > 120 feet with s_u < 1.0 ksf)

2) Categorize the site using one of the V_s, N and s_u methods.
3) Determine the appropriate Site Class and Site Factors based on the boring-specific results.

SITE CLASS RESULTS PER BORING:

Boring	N_bar	Site Class
B-1	87	C
B-1	85	C
B-3	63	C
B-4	93	C

SITE CLASS: Per Table 9-1-6 and based on the site class results from the borings, we recommend using Site Class C.

Assume 475-year return period is used for the design and based on the design maps:

PGA = 0.037 (Figure 9-1-4)
S_S = 0.082 (Figure 9-1-5)
S_1 = 0.027 (Figure 9-1-6)

For Site Class C, we have the following site coefficients:

F_{pga} = 1.2 (Table 9-1-7)
F_a = 1.2 (Table 9-1-8)
F_v = 1.7 (Table 9-1-9)

Design Spectral Response Parameters

$PGA_M = F_{pga} * PGA$ = 0.044 (PGA_M = Maximum considered earthquake peak ground acceleration coefficient)
$S_{MS} = Ss \times F_a$ = 0.098 (S_{MS} = Maximum earthquake spectral response acceleration parameter at short period)
$S_{M1} = S_1 \times F_v$ = 0.046 (S_{M1} = Maximum earthquake spectral response acceleration parameter at 1 sec. period)

$S_{DS} = 2/3 \times S_{MS}$ = 0.066 (S_{DS} = Design earthquake spectral response acceleration parameter at short period)
$S_{D1} = 2/3 \times S_{M1}$ = 0.031 (S_{D1} = Design earthquake spectral response acceleration parameter at 1 sec. period)

Table 9-1-6. Site Class Definitions

Site Class	Soil Type and Profile
A	Hard rock with measured shear wave velocity, $\overline{v}_s > 5{,}000$ ft/s (1,500 m/s)
B	Rock with 2,500 ft/s (760 m/s) $< \overline{v}_s \leq$ 5,000 ft/s (1,500 m/s)
C	Very dense soil and soft rock with 1,200 ft/s (360 m/s) $< \overline{v}_s \leq$ 2,500 ft/s (760 m/s), or with either $\overline{N} > 50$ blows/ft (blows/0.3 m), or $\overline{s}_u > 2.0$ ksf (100 kPa)
D	Stiff soil with 600 ft/s (180 m/s) $\leq \overline{v}_s \leq$ 1,200 ft/s (360 m/s), or with either $15 \leq \overline{N} \leq$ 50 blows/ft (blows/0.3 m), or 1.0 ksf (50 kPa) $\leq \overline{s}_u \leq$ 2.0 ksf (100 kPa)
E	Soft soil with $\overline{v}_s < 600$ ft/s (180 m/s), or with either $\overline{N} < 15$ blows/ft (blows/0.3 m), or $\overline{s}_u < 1.0$ ksf (50 kPa), or any profile with more than 10 feet (3 m) of soft clay defined as soil with PI > 20, w $\geq$ 40 percent and $\overline{s}_u < 0.5$ ksf (25 kPa)
F	Soils requiring site-specific evaluations, such as: • Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, and collapsible weakly cemented soils. • Peats or highly organic clays (H > 10 feet (3 m) of peat or highly organic clay where H = thickness of soil) • Very high plasticity clays (H > 25 feet (7.6 m) with PI > 75) • Very thick soft/medium stiff clays (H > 120 feet (36 m) with $\overline{s}_u < 1.0$ ksf (50 k Pa)

Table 9-1-7. USGS Site Factor, F_{pga}

USGS Site Class	Peak Ground Acceleration Coefficient (PGA)[1]				
	PGA $\leq$ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA $\geq$ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F[2]	*	*	*	*	*

Notes:
[1]Use straight-line interpolation for intermediate values of PGA.
[2]Site-specific hazard analysis should be performed for all sites in Site Class F.

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JOB	VTrans Bridge 132		
SUBJECT	Seismic Site Class		
CALCULATED BY	DH	DATE	12/9/2021
CHECKED BY	MM	DATE	12/21/2021

Table 9-1-8. USGS Site Factor, F_a

USGS Site Class	Spectral Acceleration Coefficient at 0.2 second period (S_s)[1]				
	$S_s \leq$ 0.25	$S_s =$ 0.50	$S_s =$ 0.75	$S_s =$ 1.00	$S_s \geq$ 1.25
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F[2]	*	*	*	*	*

Notes:
[1]Use straight-line interpolation for intermediate values of S_s.
[2]Site-specific hazard analysis should be performed for all sites in Site Class F.

Table 9-1-9. USGS Site Factor, F_v

USGS Site Class	Spectral Acceleration Coefficient at 1.0 second period (S_1)[1]				
	$S_1 \leq$ 0.10	$S_1 =$ 0.20	$S_1 =$ 0.30	$S_1 =$ 0.40	$S_1 \geq$ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F[2]	*	*	*	*	*

Notes:
[1]Use straight-line interpolation for intermediate values of S_1.
[2]Site-specific hazard analysis should be performed for all sites in Site Class F.

ATTACHMENTS: Refer to the attached boring-specific calculation sheets for further information.

Authored by : DH 12/09/2021
Checked by : MM 12/21/2021

VTrans Bridge 132 - Site Class Evaluation

Boring No.	Sample No.	N Value	Di	Di/N_i	N_{bar}
B-1	S-1	38	6.5	0.17	
	S-2	57	5	0.09	
	S-3	99	5	0.05	
	S-4	100	47	0.47	
	Rock	100	36.5	0.37	
					87
	Total Depth =	100	Σ	1.14	
	Depth to Rock =	63.5			

$N = \Sigma Di / \Sigma(Di/Ni) =$ 100 / 1.14 = **87**

Per Table 9-1-6, N_{bar} > 50, Site Class C

Authored by : DH 12/09/2021
Checked by : MM 12/21/2021

VTrans Bridge 132 - Site Class Evaluation

Boring No.	Sample No.	N Value	Di	Di/N_i	N_{bar}
B-2	S-1	32	4	0.13	
	S-2	68	5	0.07	
	S-3	81	5	0.06	
	S-4	77	5	0.06	
	S-5	100	5	0.05	
	S-6	57	5	0.09	
	S-7	100	5	0.05	
	S-8	100	5	0.05	
	S-9	100	5	0.05	
	S-10	100	5	0.05	
	S-11	100	5	0.05	
	S-12	100	5	0.05	**85**
	S-13	100	3	0.03	
	Rock	100	38	0.38	
	Total Depth =	100	Σ	1.17	
	Depth to Rock =	62			

$N = \Sigma\, Di / \Sigma(Di/Ni) =$ 100 / 1.17 = **85**

Per Table 9-1-6, N_{bar} > 50, Site Class C

Authored by : DH 12/09/2021
Checked by : MM 12/21/2021

<u>VTrans Bridge 132 - Site Class Evaluation</u>

Boring No.	Sample No.	N Value	Di	Di/N_i	N_{bar}
B-3	S-1	15	7.5	0.50	**63**
	S-2	30	5	0.17	
	S-3	100	5	0.05	
	S-4	71	5	0.07	
	S-5	62	3.5	0.06	
	Rock	100	74	0.74	
	Total Depth =	100	Σ	1.58	
	Depth to Rock =	26			

$N = \Sigma Di / \Sigma(Di/Ni) =$ 100 / 1.58 = **63**

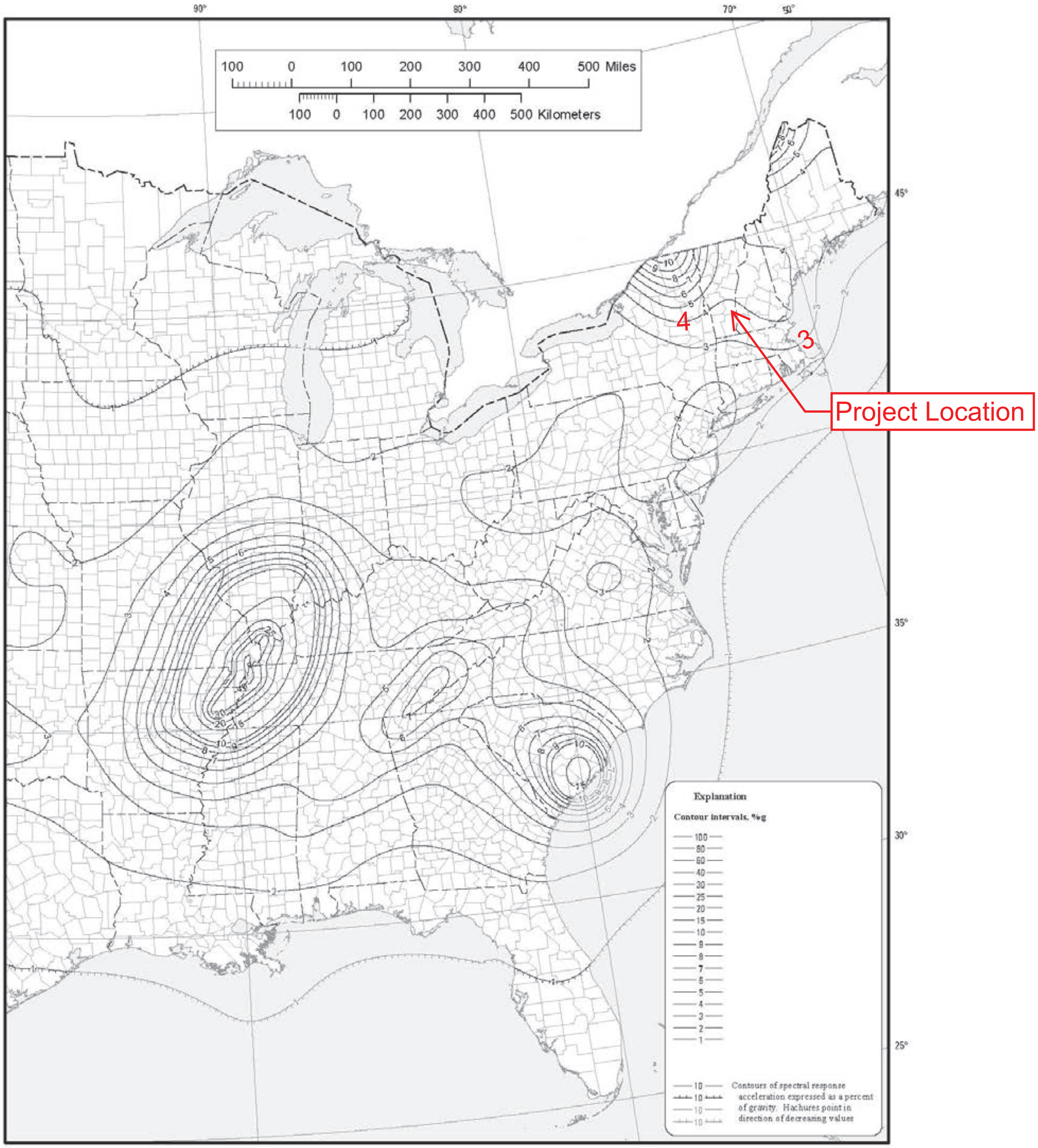
Per Table 9-1-6, N_{bar} > 50, Site Class C

VTrans Bridge 132 - Site Class Evaluation

Boring No.	**Sample No.**	**N Value**	**Di**	**D_i/N_i**	**N_{bar}**
B-4	S-1	79	7.5	0.09	**93**
	S-2	45	4.7	0.10	
	Rock	100	87.8	0.88	
	Total Depth =	100	Σ	1.08	
	Depth to Rock =	12.2			

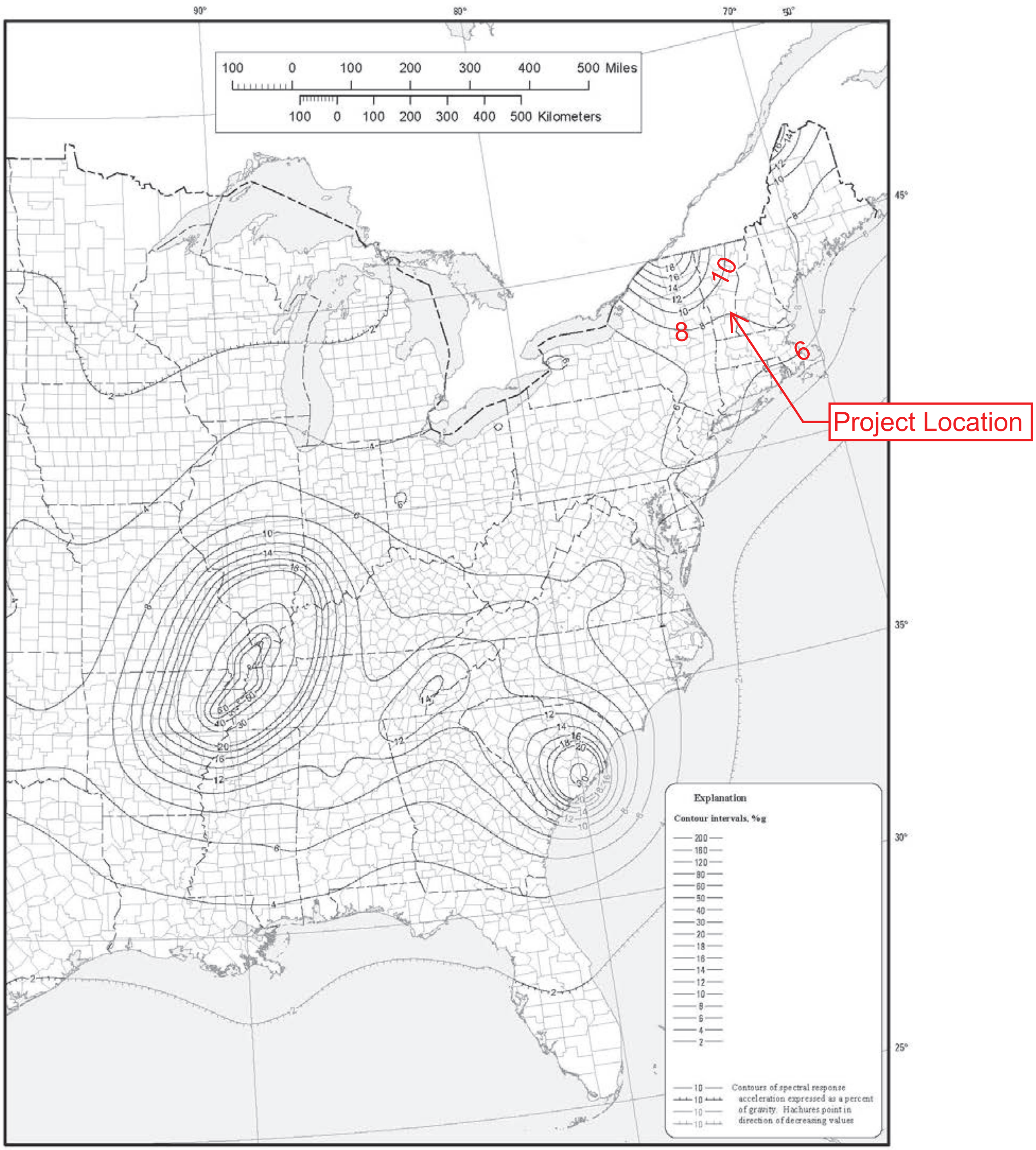
$N = \Sigma\, Di / \Sigma(Di/Ni) =$ 100 / 1.08 = **93**

Per Table 9-1-6, $N_{bar} > 50$, Site Class C



Peak Horizontal Acceleration
With a 475 Year Return Period, Site Class B

Figure 9-1-4. 475-year Return Period, Peak Ground Acceleration - United States (Continued)



Horizontal Spectral Response Acceleration for 0.2 Second Period (5 Percent of Critical Damping)
With a 475 Year Return Period, Site Class B

Figure 9-1-5. 475-year Return Period, 0.2 Second Period Spectral Response Acceleration - United States (Continued)

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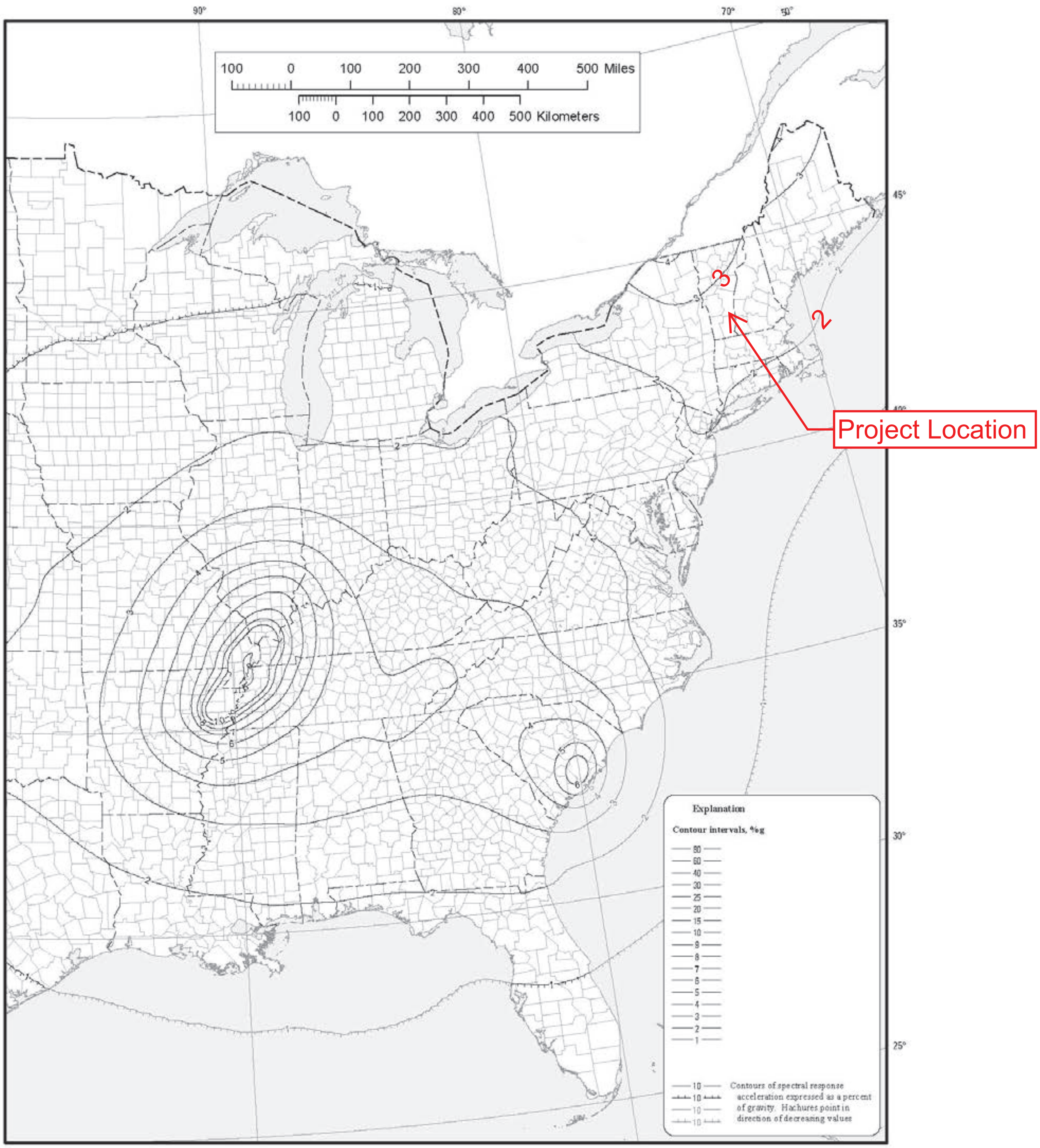
VOL 2
VOL 3
VOL 4



Horizontal Spectral Response Acceleration for 1.0 Second Period (5 Percent of Critical Damping)
With a 475 Year Return Period, Site Class B

Figure 9-1-6. 475-year Return Period, 1.0 Second Period Spectral Response Acceleration - United States (Continued)

Appendix F. Geotechnical Analysis

Jacobs	Authored by: DH	Date: 12/16/2021
	Checked by: MM	Date: 12/21/2021

Foundation Analysis of VTrans Bridge 132 Center Pier - Cavendish, VT

Purpose:

To analyze the lateral and axial capacities of the proposed driven H-pile foundation for the proposed new VTrans Bridge 132 center pier.

Procedure:

Ensoft GROUP software was used to analyze the reactions of new center pier pile group subjected to design loads, including lateral and axial loads. Ensoft APILE software was used to analyze the pile axial geotechnical capacity.

Model Input:

1.) Load cases applied to one single center pier footing are shown in Table F-1 and were provided by Jacobs structural engineer.
2.) The soil profile shown in Figure F-1 was modeled in GROUP. The soil parameters assigned to those soil profiles are shown in Table F-2 and are based soil information from borings B-1 and B-2.
3.) The fixed center pier will be supported by two footings. Each footing is supported by 9 driven HP 14x117 piles in a 3x3 grid pattern. The pile row spacing is 4-ft center-to-center in both directions. Figure F-2 shows the footing pile layout.
4.) 1/16-inch was deducted from the exterior dimensions of the H-pile to account for corrosion loss.
5.) Pile group effects (group reduction) were taken into account by software GROUP.

Analysis Assumptions:

1.) Groundwater was assumed at El. 927 feet. All soils in the model were assumed submerged.
2.) Pile-cap connections were assumed to be fixed.

Results

The maximum estimated pile reactions for Group I, Group II, and Group III load cases are presented in Table F-3. GROUP analysis result files and pile reaction diagrams for the design H-piles are attached.

Attachments

1.) Figure F-1: Center Pier Footing Design Soil Profile
2.) Figure F-2: Center Pier Footing Plan and Pile Layout
3.) Table F-1: Foundation Loads
4.) Table F-2: Soil Parameters for Lateral Analyses
5.) Table F-3: Maximum Pile and Cap Reactions
6.) GROUP Pile Reaction Diagrams
7.) GROUP Analyses Output
8.) H-pile Structural Capacity Check
9.) APILE Analysis Results

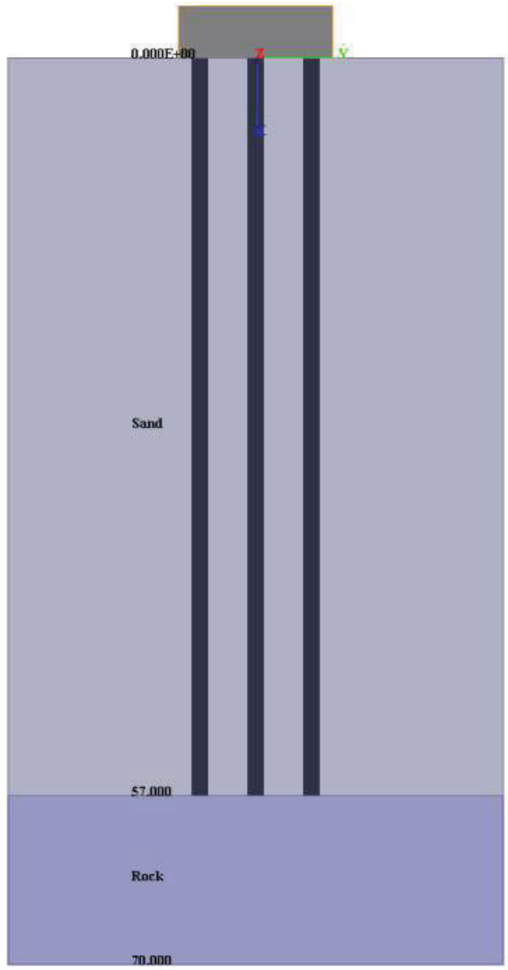


Figure F-1 Center Pier Footing Design Soil Profile

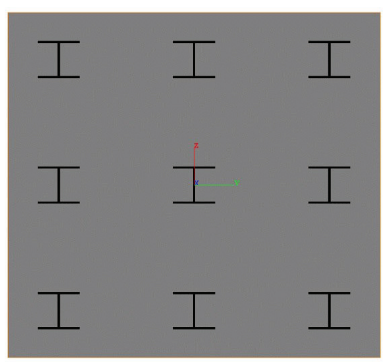


Figure F-2: Center Pier Footing Plan and Pile Layout
(Only one footing shown)

Jacobs

Authored by: DH　　Date: 12/16/2021
Checked by: MM　　Date: 12/21/2021

Foundation Analysis of VTrans Bridge 132 Center Pier - Cavendish, VT

Table F-1: Foundation Loads

Summary of Load Cases for Lateral Analysis - Center Pier Footing							**Group Input**				
Load Case No.	**Load Case Name**	**Vertical Fx**	**Long. Fz**	**Moment My**	**Trans. Fy**	**Moment Mz**	**Vertical Fx**	**Long. Fz**	**Moment My**	**Trans. Fy**	**Moment Mz**
		(k)	**(k)**	**(k-ft)**	**(k)**	**(k-ft)**	**(lb)**	**(lb)**	**(lb-in)**	**(lb)**	**(lb-in)**
1	Group I	1109.14	0	16.22	22.31	240.96	1109140	0	194640	22310	2891520
2	Group II	549.55	3.09	30.08	44.95	515.47	549550	3090	360960	44950	6185640
3	Group III	1102.85	116.07	-1227.93	158.83	1,896.26	1102850	116070	-14735160	158830	22755120

Table F-1 Notes:
1) Controlling loading case is highlighted in yellow.
2) Construction loading condition doesn't control the design per Jacobs structural engineer.

Table F-2: Soil Parameters for Lateral Analyses

Ground Surface Elev (ft): 927
Top of Pile Cap Elev (ft): 923
Bottom of Pile Cap Elev (ft): 919

Soil Layer(1)	**Layer Top Elevation**	**Layer Bot Elevation**	**Top Strata Depth (ft)**	**Bottom Strata Depth (ft)**	**Moist Unit Weight (pcf)**	**Sub. Unit Weight (pcf)**	**Friction Angle (deg)**	**Subgrade Modulus, k (pci)**	**Unconfined Compressive Strength (psi)**
Sand	919	862	0	57	125	62.6	$\emptyset = 36^{o}$	125	-
Bedrock	862	849	57	70	150	87.6	-	-	4000

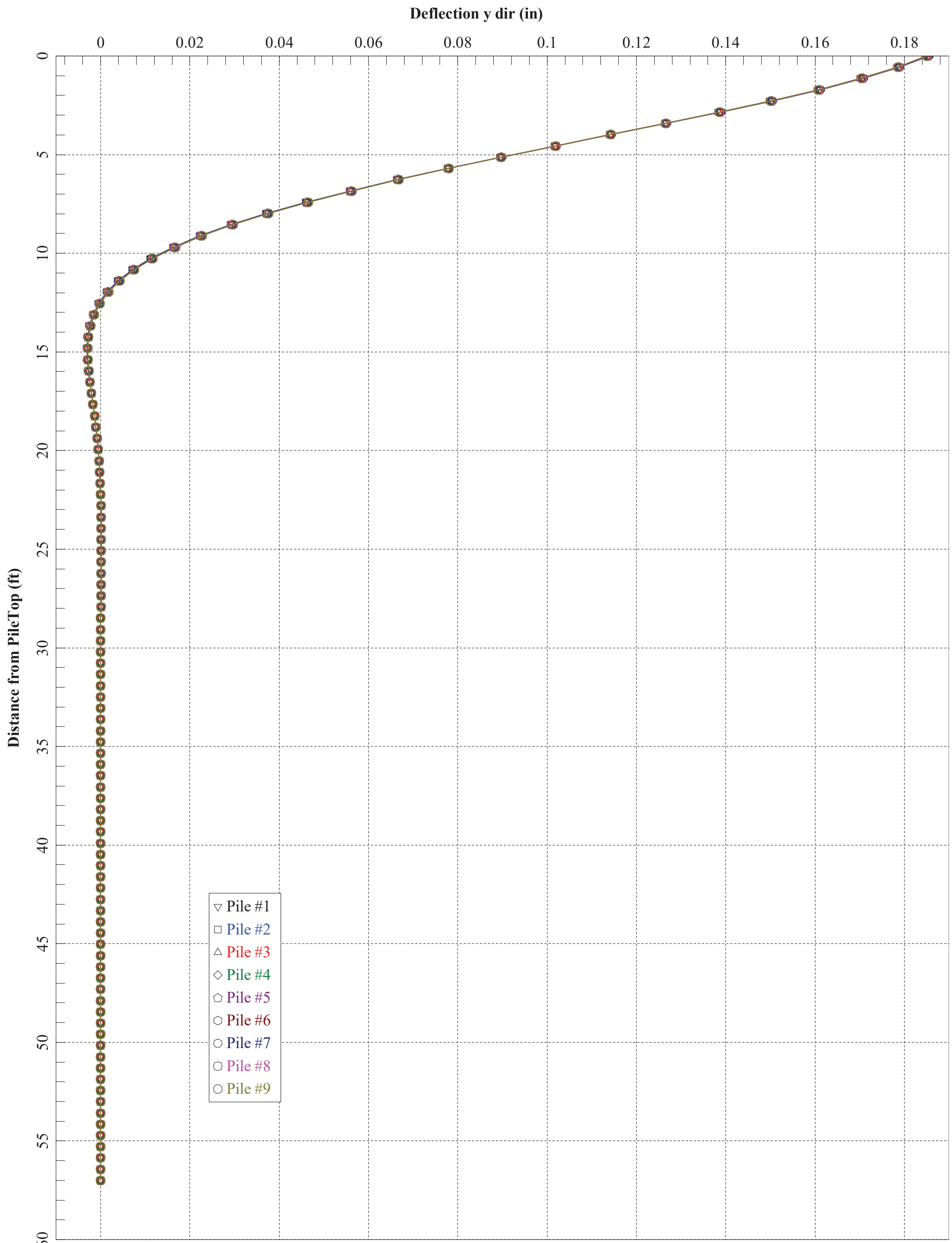
Table F-2 Notes:
1) Soil profile is based on borings B-1 and B-2. The top 8 feet of soil is neglacted considering potential scour at the center pier.
2) Groundwater table is assumed at Elevation 927 feet (i.e., same as the riverbed elevation).
3) All depths are relative to the bottom of pile cap elevation.
4) Unit weight and unconfined compressive strength of bedrock were conservatively estimated based on rock core descriptions in boring B-2 for Schist.

Table F-3 - Maximum Pile and Cap Reactions

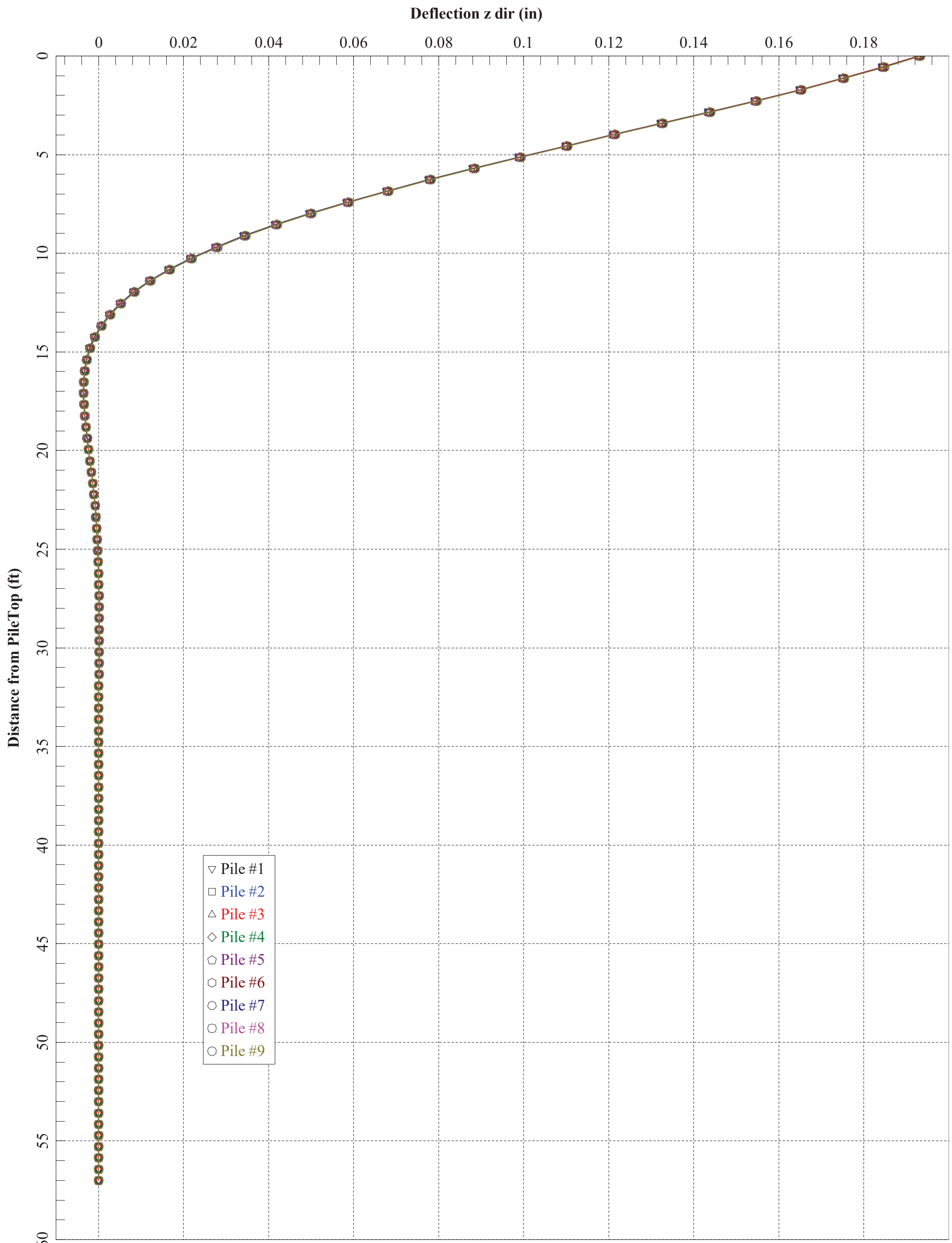
Structure	**Controlling Load Case**	**Pile Axial Load**		**Maximum Pile Moment and Shear**				**Deflection**	
		Max.	**Min.**	M_y	M_z	V_z	V_y	D_z	D_y
		(kips)	(kips)	(kip-ft)	(kip-ft)	(kips)	(kips)	(inches)	(inches)
Center Pier Footing (single)	Group I	135.8	110.6	5.2	0.1	2.5	1.5	0.014	0.000
	Group II	87.4	34.7	10.9	1.2	5.0	0.3	0.030	0.001
	Group III	283.4	-60.2	62.9	47.6	17.9	13.1	0.193	0.185

Table F-3 Notes:
1) The forces, reactions and deflections were estimated using computer software GROUP (Version 2019.11.10) produced by Ensoft Inc.
2) Structural loads were provided by Jacobs structural engineer.
3) Maximum axial compression and maximum bending may not occur on the same pile.

GROUP Pile Reaction Diagrams



Group III Loading



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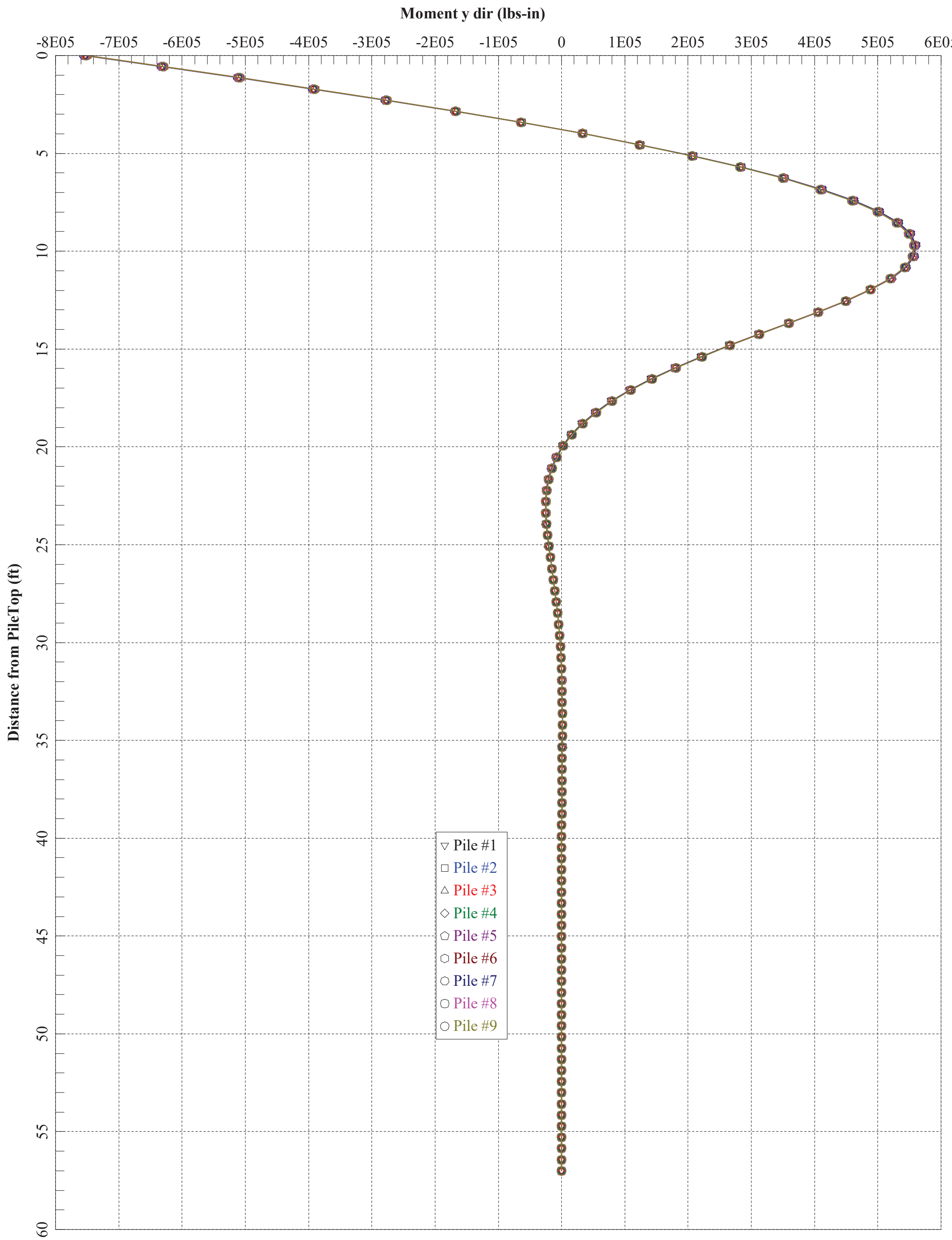
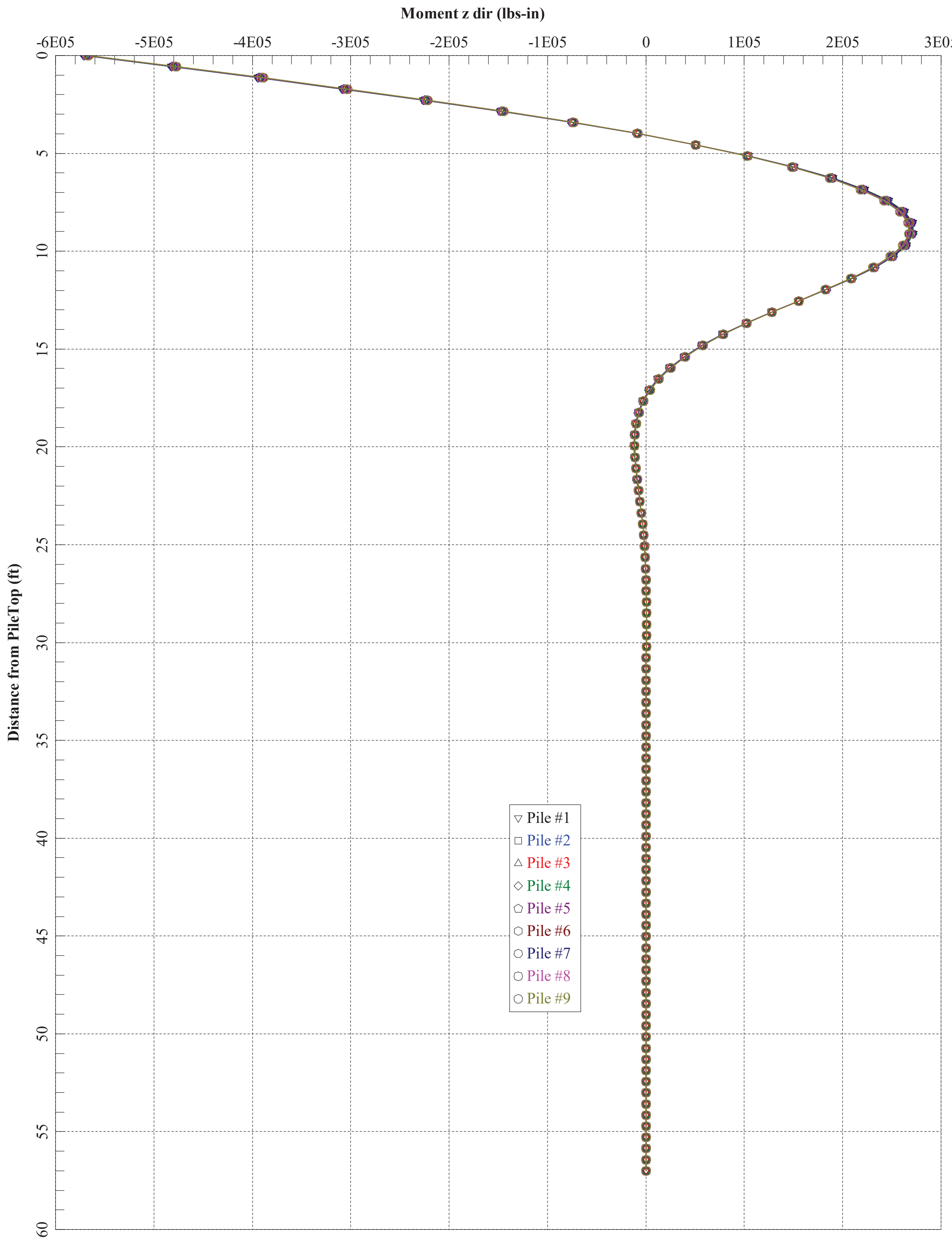
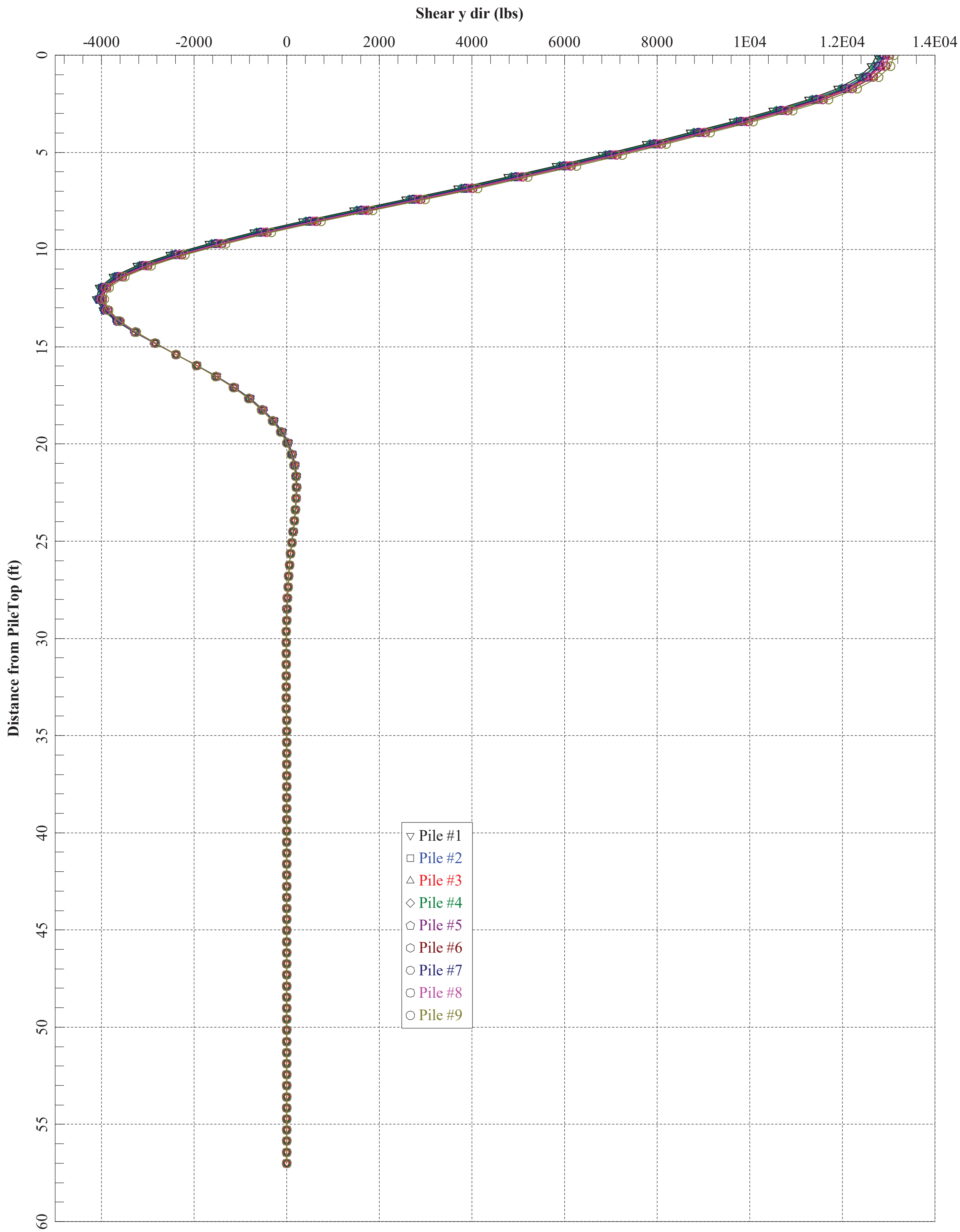


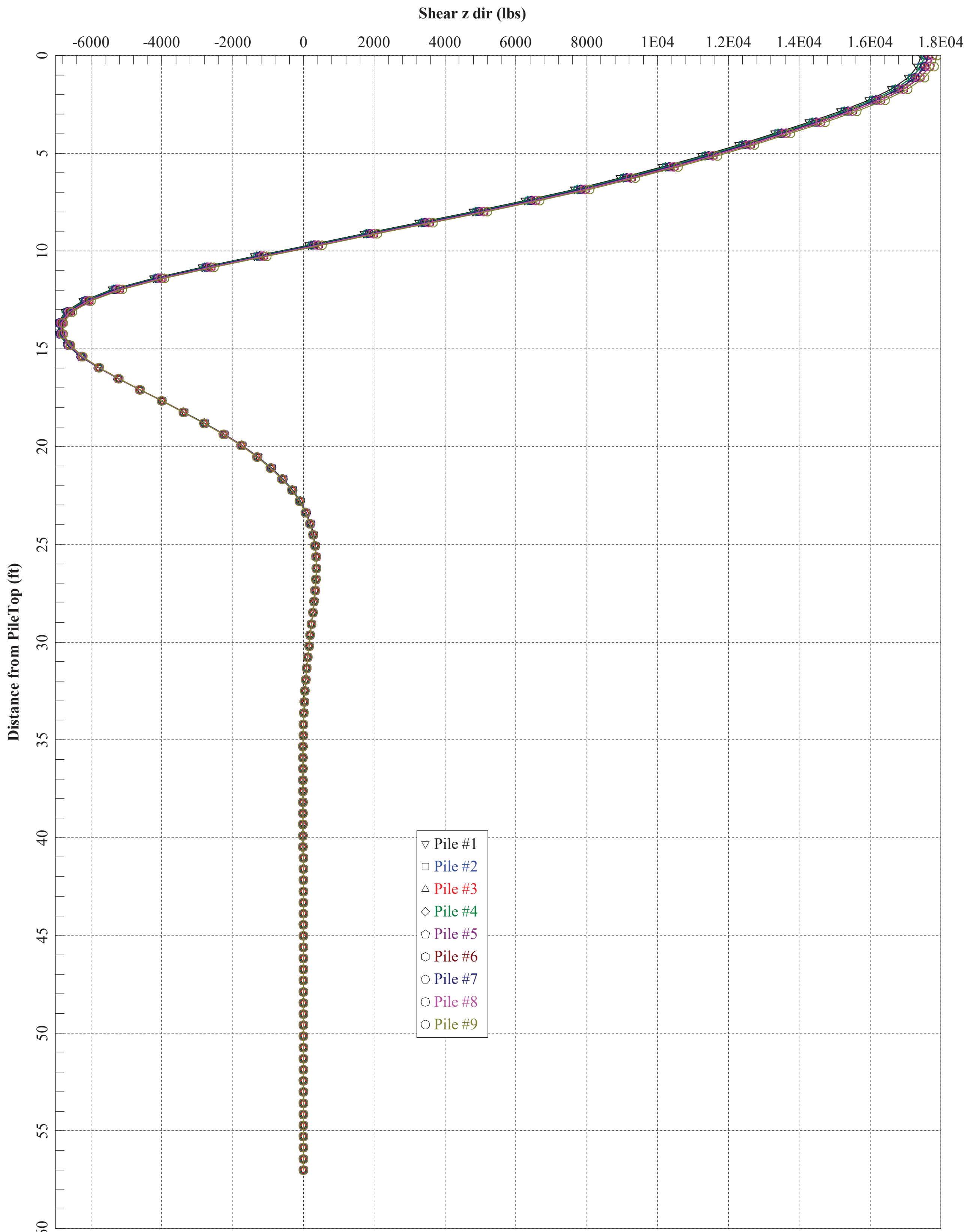
Group III Loading



Group III Loading



Group III Loading



Group III Loading

GROUP Analysis Output

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GROUP for Windows, Version 2019.11.10

Serial Number : 253581976

Analysis of A Group of Piles
Subjected to Axial and Lateral Loading

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Time and Date of Analysis

Date: December 16, 2021 Time: 10:57:42

***** COMPUTATION RESULTS *****

VTrans Bridge 132

***** LOAD CASES RESULTS *****

LOAD CASE : 1
CASE NAME : Group I
LOAD TYPE : Dead, DL

* TABLE L * COMPUTATION ON PILE CAP

* EQUIVALENT CONCENTRATED LOAD AT ORIGIN *

VERT. LOAD, LBS	HOR. LOAD Y, LBS	HOR. LOAD Z, LBS
1.10914E+06	0.00000	22310.0

MOMENT X ,LBS-IN	MOMENT Y,LBS-IN	MOMENT Z,LBS-IN

0.00000	2.89152E+06	1.94640E+05

* DISPLACEMENT OF GROUPED PILE FOUNDATION AT ORIGIN *

VERTICAL ,IN	HORIZONTAL Y,IN	HORIZONTAL Z,IN
0.0596188	-2.55774E-04	0.0135397

ANGLE ROT. X,RAD	ANGLE ROT. Y,RAD	ANGLE ROT. Z,RAD
-1.39307E-09	1.24690E-04	6.84206E-06

THE GLOBAL STRUCTURAL COORDINATE SYSTEM
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* PILE TOP DISPLACEMENTS *

PILE GROUP	DISP. X,IN	DISP. Y,IN	DISP. Z,IN	ROT. X,RAD	ROT. Y,RAD	ROT. Z,RAD
**********	***********	***********	***********	***********	***********	***********
1	0.065276	-2.5571E-04	0.013540	-1.3931E-09	1.2469E-04	6.8421E-06
2	0.059290	-2.5577E-04	0.013540	-1.3931E-09	1.2469E-04	6.8421E-06
3	0.053305	-2.5584E-04	0.013540	-1.3931E-09	1.2469E-04	6.8421E-06
4	0.065604	-2.5571E-04	0.013540	-1.3931E-09	1.2469E-04	6.8421E-06
5	0.059619	-2.5577E-04	0.013540	-1.3931E-09	1.2469E-04	6.8421E-06
6	0.053634	-2.5584E-04	0.013540	-1.3931E-09	1.2469E-04	6.8421E-06
7	0.065932	-2.5571E-04	0.013540	-1.3931E-09	1.2469E-04	6.8421E-06
8	0.059947	-2.5577E-04	0.013540	-1.3931E-09	1.2469E-04	6.8421E-06
9	0.053962	-2.5584E-04	0.013540	-1.3931E-09	1.2469E-04	6.8421E-06
MINIMUM	0.053305	-2.5584E-04	0.013540	-1.3931E-09	1.2469E-04	6.8421E-06
Pile N.	3	3	1	1	1	1
MAXIMUM	0.065932	-2.5571E-04	0.013540	-1.3931E-09	1.2469E-04	6.8421E-06
Pile N.	7	1	7	1	1	1

* PILE TOP REACTIONS *

PILE GROUP	FOR. X,LBS	FOR. Y,LBS	FOR. Z,LBS	MOM X,LBS-IN	MOM Y,LBS-IN	MOM Z,LBS-IN
**********	***********	***********	***********	***********	***********	***********
1	1.3461E+05	0.040189	2477.7	-0.097715	-6.2576E+04	1229.7
2	1.2259E+05	-1.4790E-03	2478.9	-0.097715	-6.2555E+04	1230.2
3	1.1058E+05	-0.043156	2480.1	-0.097715	-6.2533E+04	1230.7
4	1.3527E+05	0.041705	2477.7	-0.097715	-6.2578E+04	1229.6
5	1.2325E+05	3.6607E-05	2478.9	-0.097715	-6.2557E+04	1230.1
6	1.1124E+05	-0.041640	2480.1	-0.097715	-6.2535E+04	1230.6
7	1.3580E+05	0.042917	2477.6	-0.097715	-6.2580E+04	1229.6
8	1.2391E+05	1.5522E-03	2478.8	-0.097715	-6.2559E+04	1230.1
9	1.1190E+05	-0.040125	2480.1	-0.097715	-6.2537E+04	1230.6
MINIMUM	1.1058E+05	-0.043156	2477.6	-0.097715	-6.2580E+04	1229.6
Pile N.	3	3	7	1	7	4
MAXIMUM	1.3580E+05	0.042917	2480.1	-0.097715	-6.2533E+04	1230.7
Pile N.	7	7	3	1	3	3

THE PILE COORDINATE SYSTEM (LOCAL AXES)
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* PILE TOP DISPLACEMENTS *

PILE GROUP	DISP. x,IN	DISP. y,IN	DISP. z,IN	ROT. x,RAD	ROT. y,RAD	ROT. z,RAD
1	0.065276	-2.5571E-04	0.013540	-1.3931E-09	1.2469E-04	6.8421E-06
2	0.059290	-2.5577E-04	0.013540	-1.3931E-09	1.2469E-04	6.8421E-06
3	0.053305	-2.5584E-04	0.013540	-1.3931E-09	1.2469E-04	6.8421E-06
4	0.065604	-2.5571E-04	0.013540	-1.3931E-09	1.2469E-04	6.8421E-06
5	0.059619	-2.5577E-04	0.013540	-1.3931E-09	1.2469E-04	6.8421E-06
6	0.053634	-2.5584E-04	0.013540	-1.3931E-09	1.2469E-04	6.8421E-06
7	0.065932	-2.5571E-04	0.013540	-1.3931E-09	1.2469E-04	6.8421E-06
8	0.059947	-2.5577E-04	0.013540	-1.3931E-09	1.2469E-04	6.8421E-06
9	0.053962	-2.5584E-04	0.013540	-1.3931E-09	1.2469E-04	6.8421E-06
MINIMUM	0.053305	-2.5584E-04	0.013540	-1.3931E-09	1.2469E-04	6.8421E-06
Pile N.	3	3	1	1	1	1
MAXIMUM	0.065932	-2.5571E-04	0.013540	-1.3931E-09	1.2469E-04	6.8421E-06
Pile N.	7	1	7	1	1	1

* PILE TOP REACTIONS *

PILE GROUP	AXIAL,LBS	LAT. y,LBS	LAT. z,LBS	MOM x,LBS-IN	MOM y,LBS-IN	MOM z,LBS-IN
1	1.3461E+05	0.040189	2477.7	-0.097715	-6.2576E+04	1229.7
2	1.2259E+05	-1.4790E-03	2478.9	-0.097715	-6.2555E+04	1230.2
3	1.1058E+05	-0.043156	2480.1	-0.097715	-6.2533E+04	1230.7
4	1.3527E+05	0.041705	2477.7	-0.097715	-6.2578E+04	1229.6
5	1.2325E+05	3.6607E-05	2478.9	-0.097715	-6.2557E+04	1230.1
6	1.1124E+05	-0.041640	2480.1	-0.097715	-6.2535E+04	1230.6
7	1.3580E+05	0.042917	2477.6	-0.097715	-6.2580E+04	1229.6
8	1.2391E+05	1.5522E-03	2478.8	-0.097715	-6.2559E+04	1230.1
9	1.1190E+05	-0.040125	2480.1	-0.097715	-6.2537E+04	1230.6
MINIMUM	1.1058E+05	-0.043156	2477.6	-0.097715	-6.2580E+04	1229.6
Pile N.	3	3	7	1	7	4
MAXIMUM	1.3580E+05	0.042917	2480.1	-0.097715	-6.2533E+04	1230.7
Pile N.	7	7	3	1	3	3

PILE GROUP	STRESS,LBS/IN**2
1	4287.5
2	3938.1
3	3588.6
4	4306.7
5	3957.3
6	3607.8
7	4322.1
8	3976.5

9	3627.0
MINIMUM	3588.6
Pile N.	3
MAXIMUM	4322.1
Pile N.	7

* EFFECTS FOR LATERALLY LOADED PILE *

* MINIMUM VALUES AND LOCATIONS *

PILE	DISPL. y-DIR IN	DISPL. z-DIR IN	MOMENT z-DIR LBS-IN	MOMENT y-DIR LBS-IN	SHEAR y-DIR LBS	SHEAR z-DIR LBS	SOIL REACT y-DIR LBS/IN	SOIL REACT z-DIR LBS/IN	TOTAL STRESS LBS/IN**2	FLEX. RIG. z-DIR LBS-IN**2	FLEX. RIG. y-DIR LBS-IN**2
******	**********	**********	**********	**********	**********	**********	**********	**********	**********	**********	**********
1	-2.5571E-04	-3.7075E-04	-1235.4	-6.2576E+04	-0.9201	-636.37	-0.3569	-8.1178	3913.1	1.2847E+10	3.5380E+10
x(FT)	0.0000	14.250	0.5700	0.0000	16.530	11.400	2.2800	14.820	57.000	0.0000	0.0000
2	-2.5577E-04	-3.7039E-04	-1235.7	-6.2555E+04	-0.9175	-635.92	-0.3572	-8.1111	3563.8	1.2847E+10	3.5380E+10
x(FT)	0.0000	14.250	0.5700	0.0000	16.530	11.400	2.2800	14.820	57.000	0.0000	0.0000
3	-2.5584E-04	-3.7003E-04	-1235.9	-6.2533E+04	-0.9149	-635.46	-0.3574	-8.1044	3214.4	1.2847E+10	3.5380E+10
x(FT)	0.0000	14.250	0.5700	0.0000	16.530	11.400	2.2800	14.820	57.000	0.0000	0.0000
4	-2.5571E-04	-3.7077E-04	-1235.4	-6.2578E+04	-0.9203	-636.40	-0.3569	-8.1182	3932.2	1.2847E+10	3.5380E+10
x(FT)	0.0000	14.250	0.5700	0.0000	16.530	11.400	2.2800	14.820	57.000	0.0000	0.0000
5	-2.5577E-04	-3.7041E-04	-1235.6	-6.2557E+04	-0.9177	-635.95	-0.3572	-8.1115	3582.9	1.2847E+10	3.5380E+10
x(FT)	0.0000	14.250	0.5700	0.0000	16.530	11.400	2.2800	14.820	57.000	0.0000	0.0000
6	-2.5584E-04	-3.7005E-04	-1235.9	-6.2535E+04	-0.9151	-635.49	-0.3574	-8.1048	3233.6	1.2847E+10	3.5380E+10
x(FT)	0.0000	14.250	0.5700	0.0000	16.530	11.400	2.2800	14.820	57.000	0.0000	0.0000
7	-2.5571E-04	-3.7079E-04	-1235.4	-6.2580E+04	-0.9204	-636.43	-0.3569	-8.1186	3947.6	1.2847E+10	3.5380E+10
x(FT)	0.0000	14.250	0.5700	0.0000	16.530	11.400	2.2800	14.820	57.000	0.0000	0.0000
8	-2.5577E-04	-3.7043E-04	-1235.6	-6.2559E+04	-0.9178	-635.97	-0.3572	-8.1119	3602.1	1.2847E+10	3.5380E+10
x(FT)	0.0000	14.250	0.5700	0.0000	16.530	11.400	2.2800	14.820	57.000	0.0000	0.0000
9	-2.5584E-04	-3.7007E-04	-1235.8	-6.2537E+04	-0.9152	-635.52	-0.3574	-8.1052	3252.8	1.2847E+10	3.5380E+10
x(FT)	0.0000	14.250	0.5700	0.0000	16.530	11.400	2.2800	14.820	57.000	0.0000	0.0000
Min.	-2.5584E-04	-3.7079E-04	-1235.9	-6.2580E+04	-0.9204	-636.43	-0.3574	-8.1186	3214.4	1.2847E+10	3.5380E+10
Pile N.	3	7	3	7	7	7	3	7	3	1	1

* MAXIMUM VALUES AND LOCATIONS *

PILE	DISPL. y-DIR IN	DISPL. z-DIR IN	MOMENT z-DIR LBS-IN	MOMENT y-DIR LBS-IN	SHEAR y-DIR LBS	SHEAR z-DIR LBS	SOIL REACT y-DIR LBS/IN	SOIL REACT z-DIR LBS/IN	TOTAL STRESS LBS/IN**2	FLEX. RIG. z-DIR LBS-IN**2	FLEX. RIG. y-DIR LBS-IN**2
******	**********	**********	**********	**********	**********	**********	**********	**********	**********	**********	**********
1	1.6579E-05	0.013540	54.385	5.4476E+04	15.060	2478.5	0.2194	40.285	4287.5	1.2847E+10	3.5380E+10
x(FT)	8.5500	0.0000	13.680	6.8400	5.7000	0.0000	9.1200	3.9900	0.0000	0.0000	0.0000
2	1.6548E-05	0.013540	54.264	5.4464E+04	15.038	2479.7	0.2190	40.288	3938.1	1.2847E+10	3.5380E+10
x(FT)	8.5500	0.0000	13.680	6.8400	5.7000	0.0000	9.1200	3.9900	0.0000	0.0000	0.0000
3	1.6516E-05	0.013540	54.144	5.4452E+04	15.016	2480.8	0.2187	40.291	3588.6	1.2847E+10	3.5380E+10
x(FT)	8.5500	0.0000	13.680	6.8400	5.7000	0.0000	9.1200	3.9900	0.0000	0.0000	0.0000
4	1.6581E-05	0.013540	54.391	5.4476E+04	15.061	2478.5	0.2194	40.285	4306.7	1.2847E+10	3.5380E+10
x(FT)	8.5500	0.0000	13.680	6.8400	5.7000	0.0000	9.1200	3.9900	0.0000	0.0000	0.0000
5	1.6549E-05	0.013540	54.270	5.4465E+04	15.039	2479.6	0.2190	40.288	3957.3	1.2847E+10	3.5380E+10
x(FT)	8.5500	0.0000	13.680	6.8400	5.7000	0.0000	9.1200	3.9900	0.0000	0.0000	0.0000

6	1.6517E-05	0.013540	54.149	5.4453E+04	15.017	2480.8	0.2187	40.291	3607.8	1.2847E+10	3.5380E+10
x(FT)	8.5500	0.0000	13.680	6.8400	5.7000	0.0000	9.1200	3.9900	0.0000	0.0000	0.0000
7	1.6582E-05	0.013540	54.396	5.4477E+04	15.062	2478.5	0.2194	40.285	4322.1	1.2847E+10	3.5380E+10
x(FT)	8.5500	0.0000	13.680	6.8400	5.7000	0.0000	9.1200	3.9900	0.0000	0.0000	0.0000
8	1.6550E-05	0.013540	54.276	5.4466E+04	15.040	2479.6	0.2191	40.288	3976.5	1.2847E+10	3.5380E+10
x(FT)	8.5500	0.0000	13.680	6.8400	5.7000	0.0000	9.1200	3.9900	0.0000	0.0000	0.0000
9	1.6519E-05	0.013540	54.155	5.4454E+04	15.018	2480.7	0.2187	40.291	3627.0	1.2847E+10	3.5380E+10
x(FT)	8.5500	0.0000	13.680	6.8400	5.7000	0.0000	9.1200	3.9900	0.0000	0.0000	0.0000
Max.	1.6582E-05	0.013540	54.396	5.4477E+04	15.062	2480.8	0.2194	40.291	4322.1	1.2847E+10	3.5380E+10
Pile N.	7	1	7	7	7	3	7	3	7	1	1

LOAD CASE : 2
CASE NAME : Group II
LOAD TYPE : Dead, DL

* TABLE L * COMPUTATION ON PILE CAP

* EQUIVALENT CONCENTRATED LOAD AT ORIGIN *

VERT. LOAD, LBS	HOR. LOAD Y, LBS	HOR. LOAD Z, LBS
5.49550E+05	3090.00	44950.0

MOMENT X ,LBS-IN	MOMENT Y,LBS-IN	MOMENT Z,LBS-IN
0.00000	6.18564E+06	3.60960E+05

* DISPLACEMENT OF GROUPED PILE FOUNDATION AT ORIGIN *

VERTICAL ,IN	HORIZONTAL Y,IN	HORIZONTAL Z,IN
0.0286439	1.49625E-03	0.0301764

ANGLE ROT. X,RAD	ANGLE ROT. Y,RAD	ANGLE ROT. Z,RAD
-2.48894E-08	2.65168E-04	8.26272E-06

THE GLOBAL STRUCTURAL COORDINATE SYSTEM
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* PILE TOP DISPLACEMENTS *

PILE GROUP	DISP. X,IN	DISP. Y,IN	DISP. Z,IN	ROT. X,RAD	ROT. Y,RAD	ROT. Z,RAD
**********	***********	***********	***********	***********	***********	***********
1	0.040975	1.4974E-03	0.030175	-2.4889E-08	2.6517E-04	8.2627E-06
2	0.028247	1.4962E-03	0.030175	-2.4889E-08	2.6517E-04	8.2627E-06
3	0.015519	1.4950E-03	0.030175	-2.4889E-08	2.6517E-04	8.2627E-06

4	0.041372	1.4974E-03	0.030176	-2.4889E-08	2.6517E-04	8.2627E-06
5	0.028644	1.4962E-03	0.030176	-2.4889E-08	2.6517E-04	8.2627E-06
6	0.015916	1.4950E-03	0.030176	-2.4889E-08	2.6517E-04	8.2627E-06
7	0.041769	1.4974E-03	0.030177	-2.4889E-08	2.6517E-04	8.2627E-06
8	0.029041	1.4962E-03	0.030177	-2.4889E-08	2.6517E-04	8.2627E-06
9	0.016312	1.4950E-03	0.030177	-2.4889E-08	2.6517E-04	8.2627E-06
MINIMUM	0.015519	1.4950E-03	0.030175	-2.4889E-08	2.6517E-04	8.2627E-06
Pile N.	3	3	1	1	1	1
MAXIMUM	0.041769	1.4974E-03	0.030177	-2.4889E-08	2.6517E-04	8.2627E-06
Pile N.	7	1	7	1	1	1

* PILE TOP REACTIONS *

PILE GROUP	FOR. X,LBS	FOR. Y,LBS	FOR. Z,LBS	MOM X,LBS-IN	MOM Y,LBS-IN	MOM Z,LBS-IN
1	8.5820E+04	343.23	4989.0	-1.7458	-1.3056E+05	1.4630E+04
2	6.0265E+04	343.35	4994.4	-1.7458	-1.3046E+05	1.4625E+04
3	3.4709E+04	343.47	4999.8	-1.7458	-1.3036E+05	1.4620E+04
4	8.6616E+04	343.22	4989.0	-1.7458	-1.3058E+05	1.4630E+04
5	6.1061E+04	343.33	4994.4	-1.7458	-1.3048E+05	1.4625E+04
6	3.5506E+04	343.45	4999.9	-1.7458	-1.3038E+05	1.4620E+04
7	8.7413E+04	343.20	4989.1	-1.7458	-1.3060E+05	1.4630E+04
8	6.1857E+04	343.32	4994.5	-1.7458	-1.3049E+05	1.4624E+04
9	3.6302E+04	343.43	4999.9	-1.7458	-1.3039E+05	1.4619E+04
MINIMUM	3.4709E+04	343.20	4989.0	-1.7458	-1.3060E+05	1.4619E+04
Pile N.	3	7	1	1	7	9
MAXIMUM	8.7413E+04	343.47	4999.9	-1.7458	-1.3036E+05	1.4630E+04
Pile N.	7	3	6	1	3	1

THE PILE COORDINATE SYSTEM (LOCAL AXES)
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* PILE TOP DISPLACEMENTS *

PILE GROUP	DISP. x,IN	DISP. y,IN	DISP. z,IN	ROT. x,RAD	ROT. y,RAD	ROT. z,RAD
1	0.040975	1.4974E-03	0.030175	-2.4889E-08	2.6517E-04	8.2627E-06
2	0.028247	1.4962E-03	0.030175	-2.4889E-08	2.6517E-04	8.2627E-06
3	0.015519	1.4950E-03	0.030175	-2.4889E-08	2.6517E-04	8.2627E-06
4	0.041372	1.4974E-03	0.030176	-2.4889E-08	2.6517E-04	8.2627E-06
5	0.028644	1.4962E-03	0.030176	-2.4889E-08	2.6517E-04	8.2627E-06
6	0.015916	1.4950E-03	0.030176	-2.4889E-08	2.6517E-04	8.2627E-06
7	0.041769	1.4974E-03	0.030177	-2.4889E-08	2.6517E-04	8.2627E-06
8	0.029041	1.4962E-03	0.030177	-2.4889E-08	2.6517E-04	8.2627E-06
9	0.016312	1.4950E-03	0.030177	-2.4889E-08	2.6517E-04	8.2627E-06
MINIMUM	0.015519	1.4950E-03	0.030175	-2.4889E-08	2.6517E-04	8.2627E-06
Pile N.	3	3	1	1	1	1
MAXIMUM	0.041769	1.4974E-03	0.030177	-2.4889E-08	2.6517E-04	8.2627E-06
Pile N.	7	1	7	1	1	1

* PILE TOP REACTIONS *

PILE GROUP	AXIAL,LBS	LAT. y,LBS	LAT. z,LBS	MOM x,LBS-IN	MOM y,LBS-IN	MOM z,LBS-IN
**********	***********	***********	***********	***********	***********	***********
1	8.5820E+04	343.23	4989.0	-1.7458	-1.3056E+05	1.4630E+04
2	6.0265E+04	343.35	4994.4	-1.7458	-1.3046E+05	1.4625E+04
3	3.4709E+04	343.47	4999.8	-1.7458	-1.3036E+05	1.4620E+04
4	8.6616E+04	343.22	4989.0	-1.7458	-1.3058E+05	1.4630E+04
5	6.1061E+04	343.33	4994.4	-1.7458	-1.3048E+05	1.4625E+04
6	3.5506E+04	343.45	4999.9	-1.7458	-1.3038E+05	1.4620E+04
7	8.7413E+04	343.20	4989.1	-1.7458	-1.3060E+05	1.4630E+04
8	6.1857E+04	343.32	4994.5	-1.7458	-1.3049E+05	1.4624E+04
9	3.6302E+04	343.43	4999.9	-1.7458	-1.3039E+05	1.4619E+04
MINIMUM	3.4709E+04	343.20	4989.0	-1.7458	-1.3060E+05	1.4619E+04
Pile N.	3	7	1	1	7	9
MAXIMUM	8.7413E+04	343.47	4999.9	-1.7458	-1.3036E+05	1.4630E+04
Pile N.	7	3	6	1	3	1

PILE GROUP	STRESS,LBS/IN**2
**********	****************
1	3311.2
2	2567.7
3	1824.3
4	3334.5
5	2591.0
6	1847.5
7	3357.7
8	2614.2
9	1870.7
MINIMUM	1824.3
Pile N.	3
MAXIMUM	3357.7
Pile N.	7

* EFFECTS FOR LATERALLY LOADED PILE *

* MINIMUM VALUES AND LOCATIONS *

PILE	DISPL. y-DIR IN	DISPL. z-DIR IN	MOMENT z-DIR LBS-IN	MOMENT y-DIR LBS-IN	SHEAR y-DIR LBS	SHEAR z-DIR LBS	SOIL REACT y-DIR LBS/IN	SOIL REACT z-DIR LBS/IN	TOTAL STRESS LBS/IN**2	FLEX. RIG. z-DIR LBS-IN**2	FLEX. RIG. y-DIR LBS-IN**2
******	**********	**********	**********	**********	**********	**********	**********	**********	**********	**********	**********
1	-3.8690E-05	-7.5681E-04	-1.4630E+04	-1.3056E+05	-49.598	-1332.0	-0.8040	-17.094	2494.8	1.2847E+10	3.5380E+10
x(FT)	13.680	14.820	0.0000	0.0000	11.400	11.400	14.250	15.390	57.000	0.0000	0.0000
2	-3.8534E-05	-7.5528E-04	-1.4625E+04	-1.3046E+05	-49.439	-1329.6	-0.8012	-17.065	1751.9	1.2847E+10	3.5380E+10
x(FT)	13.680	14.820	0.0000	0.0000	11.400	11.400	14.250	15.390	57.000	0.0000	0.0000
3	-3.8379E-05	-7.5376E-04	-1.4620E+04	-1.3036E+05	-49.280	-1327.3	-0.7985	-17.036	1009.0	1.2847E+10	3.5380E+10
x(FT)	13.680	14.820	0.0000	0.0000	11.400	11.400	14.250	15.390	57.000	0.0000	0.0000
4	-3.8694E-05	-7.5688E-04	-1.4630E+04	-1.3058E+05	-49.602	-1332.1	-0.8040	-17.096	2517.9	1.2847E+10	3.5380E+10
x(FT)	13.680	14.820	0.0000	0.0000	11.400	11.400	14.250	15.390	57.000	0.0000	0.0000

5	-3.8538E-05	-7.5535E-04	-1.4625E+04	-1.3048E+05	-49.443	-1329.8	-0.8013	-17.067	1775.0	1.2847E+10	3.5380E+10
x(FT)	13.680	14.820	0.0000	0.0000	11.400	11.400	14.250	15.390	57.000	0.0000	0.0000
6	-3.8383E-05	-7.5383E-04	-1.4620E+04	-1.3038E+05	-49.284	-1327.4	-0.7986	-17.038	1032.1	1.2847E+10	3.5380E+10
x(FT)	13.680	14.820	0.0000	0.0000	11.400	11.400	14.250	15.390	57.000	0.0000	0.0000
7	-3.8698E-05	-7.5694E-04	-1.4630E+04	-1.3059E+05	-49.606	-1332.2	-0.8041	-17.097	2541.1	1.2847E+10	3.5380E+10
x(FT)	13.680	14.820	0.0000	0.0000	11.400	11.400	14.250	15.390	57.000	0.0000	0.0000
8	-3.8542E-05	-7.5542E-04	-1.4624E+04	-1.3049E+05	-49.447	-1329.9	-0.8014	-17.068	1798.2	1.2847E+10	3.5380E+10
x(FT)	13.680	14.820	0.0000	0.0000	11.400	11.400	14.250	15.390	57.000	0.0000	0.0000
9	-3.8386E-05	-7.5390E-04	-1.4619E+04	-1.3039E+05	-49.288	-1327.5	-0.7986	-17.039	1055.3	1.2847E+10	3.5380E+10
x(FT)	13.680	14.820	0.0000	0.0000	11.400	11.400	14.250	15.390	57.000	0.0000	0.0000
Min.	-3.8698E-05	-7.5694E-04	-1.4630E+04	-1.3059E+05	-49.606	-1332.2	-0.8041	-17.097	1009.0	1.2847E+10	3.5380E+10
Pile N.	7	7	1	7	7	7	7	7	3	1	1

* MAXIMUM VALUES AND LOCATIONS *

PILE	DISPL. y-DIR IN	DISPL. z-DIR IN	MOMENT z-DIR LBS-IN	MOMENT y-DIR LBS-IN	SHEAR y-DIR LBS	SHEAR z-DIR LBS	SOIL REACT y-DIR LBS/IN	SOIL REACT z-DIR LBS/IN	TOTAL STRESS LBS/IN**2	FLEX. RIG. z-DIR LBS-IN**2	FLEX. RIG. y-DIR LBS-IN**2
******	**********	**********	**********	**********	**********	**********	**********	**********	**********	**********	**********
1	1.5273E-03	0.030175	3279.6	1.1274E+05	343.57	4990.0	5.2267	79.555	3311.2	1.2847E+10	3.5380E+10
x(FT)	0.5700	0.0000	7.9800	7.4100	0.0000	0.0000	3.4200	2.8500	0.0000	0.0000	0.0000
2	1.5261E-03	0.030175	3274.8	1.1268E+05	343.58	4995.1	5.2229	79.557	2567.7	1.2847E+10	3.5380E+10
x(FT)	0.5700	0.0000	7.9800	7.4100	0.0000	0.0000	3.4200	2.8500	0.0000	0.0000	0.0000
3	1.5249E-03	0.030175	3269.9	1.1262E+05	343.60	5000.2	5.2191	79.559	1824.3	1.2847E+10	3.5380E+10
x(FT)	0.5700	0.0000	7.9800	7.4100	0.0000	0.0000	3.4200	2.8500	0.0000	0.0000	0.0000
4	1.5273E-03	0.030176	3279.7	1.1274E+05	343.55	4990.1	5.2265	79.557	3334.5	1.2847E+10	3.5380E+10
x(FT)	0.5700	0.0000	7.9800	7.4100	0.0000	0.0000	3.4200	2.8500	0.0000	0.0000	0.0000
5	1.5261E-03	0.030176	3274.8	1.1268E+05	343.57	4995.2	5.2227	79.559	2591.0	1.2847E+10	3.5380E+10
x(FT)	0.5700	0.0000	7.9800	7.4100	0.0000	0.0000	3.4200	2.8500	0.0000	0.0000	0.0000
6	1.5250E-03	0.030176	3270.0	1.1263E+05	343.59	5000.3	5.2189	79.561	1847.5	1.2847E+10	3.5380E+10
x(FT)	0.5700	0.0000	7.9800	7.4100	0.0000	0.0000	3.4200	2.8500	0.0000	0.0000	0.0000
7	1.5273E-03	0.030178	3279.7	1.1275E+05	343.54	4990.2	5.2264	79.558	3357.7	1.2847E+10	3.5380E+10
x(FT)	0.5700	0.0000	7.9800	7.4100	0.0000	0.0000	3.4200	2.8500	0.0000	0.0000	0.0000
8	1.5261E-03	0.030178	3274.9	1.1269E+05	343.56	4995.3	5.2226	79.560	2614.2	1.2847E+10	3.5380E+10
x(FT)	0.5700	0.0000	7.9800	7.4100	0.0000	0.0000	3.4200	2.8500	0.0000	0.0000	0.0000
9	1.5250E-03	0.030178	3270.0	1.1263E+05	343.57	5000.4	5.2188	79.562	1870.7	1.2847E+10	3.5380E+10
x(FT)	0.5700	0.0000	7.9800	7.4100	0.0000	0.0000	3.4200	2.8500	0.0000	0.0000	0.0000
Max.	1.5273E-03	0.030178	3279.7	1.1275E+05	343.60	5000.4	5.2267	79.562	3357.7	1.2847E+10	3.5380E+10
Pile N.	1	7	4	7	3	9	1	9	7	1	1

LOAD CASE : 3
CASE NAME : Group III
LOAD TYPE : Dead, DL

* TABLE L * COMPUTATION ON PILE CAP

* EQUIVALENT CONCENTRATED LOAD AT ORIGIN *

VERT. LOAD, LBS	HOR. LOAD Y, LBS	HOR. LOAD Z, LBS
1.10285E+06	1.16070E+05	1.58830E+05

MOMENT X ,LBS-IN	MOMENT Y,LBS-IN	MOMENT Z,LBS-IN
0.00000	2.27551E+07	-1.47352E+07

* DISPLACEMENT OF GROUPED PILE FOUNDATION AT ORIGIN *

VERTICAL ,IN	HORIZONTAL Y,IN	HORIZONTAL Z,IN
0.0639588	0.18515	0.19309

ANGLE ROT. X,RAD	ANGLE ROT. Y,RAD	ANGLE ROT. Z,RAD
-5.03744E-06	1.17502E-03	-7.89652E-04

THE GLOBAL STRUCTURAL COORDINATE SYSTEM
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* PILE TOP DISPLACEMENTS *

PILE GROUP	DISP. X,IN	DISP. Y,IN	DISP. Z,IN	ROT. X,RAD	ROT. Y,RAD	ROT. Z,RAD
**********	***********	***********	***********	***********	***********	***********
1	0.1583	0.1854	0.1928	-5.0374E-06	1.1750E-03	-7.8965E-04
2	0.1019	0.1852	0.1928	-5.0374E-06	1.1750E-03	-7.8965E-04
3	0.045461	0.1849	0.1928	-5.0374E-06	1.1750E-03	-7.8965E-04
4	0.1204	0.1854	0.1931	-5.0374E-06	1.1750E-03	-7.8965E-04
5	0.063959	0.1852	0.1931	-5.0374E-06	1.1750E-03	-7.8965E-04
6	7.5578E-03	0.1849	0.1931	-5.0374E-06	1.1750E-03	-7.8965E-04
7	0.082457	0.1854	0.1933	-5.0374E-06	1.1750E-03	-7.8965E-04
8	0.026055	0.1852	0.1933	-5.0374E-06	1.1750E-03	-7.8965E-04
9	-0.030346	0.1849	0.1933	-5.0374E-06	1.1750E-03	-7.8965E-04
MINIMUM	-0.030346	0.1849	0.1928	-5.0374E-06	1.1750E-03	-7.8965E-04
Pile N.	9	3	1	1	1	1
MAXIMUM	0.1583	0.1854	0.1933	-5.0374E-06	1.1750E-03	-7.8965E-04
Pile N.	1	1	7	1	1	1

* PILE TOP REACTIONS *

PILE GROUP	FOR. X,LBS	FOR. Y,LBS	FOR. Z,LBS	MOM X,LBS-IN	MOM Y,LBS-IN	MOM Z,LBS-IN
**********	***********	***********	***********	***********	***********	***********
1	2.8341E+05	1.2698E+04	1.7427E+04	-353.35	-7.5304E+05	5.7102E+05
2	1.9324E+05	1.2804E+04	1.7535E+04	-353.35	-7.5150E+05	5.6933E+05
3	9.4827E+04	1.2920E+04	1.7653E+04	-353.35	-7.4980E+05	5.6759E+05
4	2.2281E+05	1.2778E+04	1.7527E+04	-353.35	-7.5395E+05	5.7047E+05
5	1.3197E+05	1.2884E+04	1.7637E+04	-353.35	-7.5240E+05	5.6878E+05
6	1.8725E+04	1.3020E+04	1.7772E+04	-353.35	-7.5044E+05	5.6694E+05
7	1.6222E+05	1.2858E+04	1.7628E+04	-353.35	-7.5487E+05	5.6992E+05

8	5.5864E+04	1.2985E+04	1.7756E+04	-353.35	-7.5303E+05	5.6813E+05
9	-6.0216E+04	1.3124E+04	1.7894E+04	-353.35	-7.5103E+05	5.6626E+05
MINIMUM	-6.0216E+04	1.2698E+04	1.7427E+04	-353.35	-7.5487E+05	5.6626E+05
Pile N.	9	1	1	1	7	9
MAXIMUM	2.8341E+05	1.3124E+04	1.7894E+04	-353.35	-7.4980E+05	5.7102E+05
Pile N.	1	9	9	1	3	1

THE PILE COORDINATE SYSTEM (LOCAL AXES)
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* PILE TOP DISPLACEMENTS *

PILE GROUP	DISP. x,IN	DISP. y,IN	DISP. z,IN	ROT. x,RAD	ROT. y,RAD	ROT. z,RAD
**********	***********	***********	***********	***********	***********	***********
1	0.1583	0.1854	0.1928	-5.0374E-06	1.1750E-03	-7.8965E-04
2	0.1019	0.1852	0.1928	-5.0374E-06	1.1750E-03	-7.8965E-04
3	0.045461	0.1849	0.1928	-5.0374E-06	1.1750E-03	-7.8965E-04
4	0.1204	0.1854	0.1931	-5.0374E-06	1.1750E-03	-7.8965E-04
5	0.063959	0.1852	0.1931	-5.0374E-06	1.1750E-03	-7.8965E-04
6	7.5578E-03	0.1849	0.1931	-5.0374E-06	1.1750E-03	-7.8965E-04
7	0.082457	0.1854	0.1933	-5.0374E-06	1.1750E-03	-7.8965E-04
8	0.026055	0.1852	0.1933	-5.0374E-06	1.1750E-03	-7.8965E-04
9	-0.030346	0.1849	0.1933	-5.0374E-06	1.1750E-03	-7.8965E-04
MINIMUM	-0.030346	0.1849	0.1928	-5.0374E-06	1.1750E-03	-7.8965E-04
Pile N.	9	3	1	1	1	1
MAXIMUM	0.1583	0.1854	0.1933	-5.0374E-06	1.1750E-03	-7.8965E-04
Pile N.	1	1	7	1	1	1

* PILE TOP REACTIONS *

PILE GROUP	AXIAL,LBS	LAT. y,LBS	LAT. z,LBS	MOM x,LBS-IN	MOM y,LBS-IN	MOM z,LBS-IN
**********	***********	***********	***********	***********	***********	***********
1	2.8341E+05	1.2698E+04	1.7427E+04	-353.35	-7.5304E+05	5.7102E+05
2	1.9324E+05	1.2804E+04	1.7535E+04	-353.35	-7.5150E+05	5.6933E+05
3	9.4827E+04	1.2920E+04	1.7653E+04	-353.35	-7.4980E+05	5.6759E+05
4	2.2281E+05	1.2778E+04	1.7527E+04	-353.35	-7.5395E+05	5.7047E+05
5	1.3197E+05	1.2884E+04	1.7637E+04	-353.35	-7.5240E+05	5.6878E+05
6	1.8725E+04	1.3020E+04	1.7772E+04	-353.35	-7.5044E+05	5.6694E+05
7	1.6222E+05	1.2858E+04	1.7628E+04	-353.35	-7.5487E+05	5.6992E+05
8	5.5864E+04	1.2985E+04	1.7756E+04	-353.35	-7.5303E+05	5.6813E+05
9	-6.0216E+04	1.3124E+04	1.7894E+04	-353.35	-7.5103E+05	5.6626E+05
MINIMUM	-6.0216E+04	1.2698E+04	1.7427E+04	-353.35	-7.5487E+05	5.6626E+05
Pile N.	9	1	1	1	7	9
MAXIMUM	2.8341E+05	1.3124E+04	1.7894E+04	-353.35	-7.4980E+05	5.7102E+05
Pile N.	1	9	9	1	3	1

PILE GROUP	STRESS,LBS/IN**2
**********	****************
1	1.8657E+04

2	1.6007E+04
3	1.3116E+04
4	1.6890E+04
5	1.4220E+04
6	1.0896E+04
7	1.5123E+04
8	1.2000E+04
9	1.2093E+04

MINIMUM	1.0896E+04
Pile N.	6
MAXIMUM	1.8657E+04
Pile N.	1

* EFFECTS FOR LATERALLY LOADED PILE *

* MINIMUM VALUES AND LOCATIONS *

PILE	DISPL. y-DIR IN	DISPL. z-DIR IN	MOMENT z-DIR LBS-IN	MOMENT y-DIR LBS-IN	SHEAR y-DIR LBS	SHEAR z-DIR LBS	SOIL REACT y-DIR LBS/IN	SOIL REACT z-DIR LBS/IN	TOTAL STRESS LBS/IN**2	FLEX. RIG. z-DIR LBS-IN**2	FLEX. RIG. y-DIR LBS-IN**2
******	**********	**********	**********	**********	**********	**********	**********	**********	**********	**********	**********
1	-3.0194E-03	-3.5656E-03	-5.7102E+05	-7.5304E+05	-4111.0	-6934.1	-67.947	-91.770	8238.8	1.2847E+10	3.5380E+10
x(FT)	14.820	17.100	0.0000	0.0000	12.540	13.680	15.390	17.670	57.000	0.0000	0.0000
2	-2.9754E-03	-3.5410E-03	-5.6933E+05	-7.5150E+05	-4064.7	-6889.7	-67.169	-91.257	5617.5	1.2847E+10	3.5380E+10
x(FT)	14.820	17.100	0.0000	0.0000	12.540	13.680	15.390	17.670	57.000	0.0000	0.0000
3	-2.9278E-03	-3.5141E-03	-5.6759E+05	-7.4980E+05	-4014.4	-6841.1	-66.327	-90.696	2756.6	1.2847E+10	3.5380E+10
x(FT)	14.820	17.100	0.0000	0.0000	12.540	13.680	15.390	17.670	57.000	0.0000	0.0000
4	-2.9915E-03	-3.5531E-03	-5.7047E+05	-7.5395E+05	-4082.9	-6912.0	-67.476	-91.544	6477.2	1.2847E+10	3.5380E+10
x(FT)	14.820	17.100	0.0000	0.0000	12.540	13.680	15.390	17.670	57.000	0.0000	0.0000
5	-2.9472E-03	-3.5282E-03	-5.6878E+05	-7.5240E+05	-4036.0	-6867.1	-66.691	-91.025	3836.2	1.2847E+10	3.5380E+10
x(FT)	14.820	17.100	0.0000	0.0000	12.540	13.680	15.390	17.670	57.000	0.0000	0.0000
6	-2.8929E-03	-3.4972E-03	-5.6694E+05	-7.5044E+05	-3978.8	-6810.9	-65.733	-90.377	544.32	1.2847E+10	3.5380E+10
x(FT)	14.820	17.100	0.0000	0.0000	12.540	13.680	15.390	17.670	57.000	0.0000	0.0000
7	-2.9636E-03	-3.5404E-03	-5.6992E+05	-7.5487E+05	-4054.5	-6889.8	-67.003	-91.315	4715.6	1.2847E+10	3.5380E+10
x(FT)	14.820	17.100	0.0000	0.0000	12.540	13.680	15.390	17.670	57.000	0.0000	0.0000
8	-2.9123E-03	-3.5112E-03	-5.6813E+05	-7.5303E+05	-4000.4	-6836.9	-66.097	-90.705	1624.0	1.2847E+10	3.5380E+10
x(FT)	14.820	17.100	0.0000	0.0000	12.540	13.680	15.390	17.670	57.000	0.0000	0.0000
9	-2.8566E-03	-3.4793E-03	-5.6626E+05	-7.5103E+05	-3941.6	-6781.7	-65.115	-90.038	1750.5	1.2847E+10	3.5380E+10
x(FT)	14.820	17.100	0.0000	0.0000	12.540	14.250	15.390	17.670	57.000	0.0000	0.0000
Min.	-3.0194E-03	-3.5656E-03	-5.7102E+05	-7.5487E+05	-4111.0	-6934.1	-67.947	-91.770	544.32	1.2847E+10	3.5380E+10
Pile N.	1	1	1	7	1	1	1	1	6	1	1

* MAXIMUM VALUES AND LOCATIONS *

PILE	DISPL. y-DIR IN	DISPL. z-DIR IN	MOMENT z-DIR LBS-IN	MOMENT y-DIR LBS-IN	SHEAR y-DIR LBS	SHEAR z-DIR LBS	SOIL REACT y-DIR LBS/IN	SOIL REACT z-DIR LBS/IN	TOTAL STRESS LBS/IN**2	FLEX. RIG. z-DIR LBS-IN**2	FLEX. RIG. y-DIR LBS-IN**2
******	**********	**********	**********	**********	**********	**********	**********	**********	**********	**********	**********
1	0.1854	0.1928	2.7166E+05	5.6026E+05	1.2741E+04	1.7447E+04	165.45	228.76	1.8657E+04	1.2847E+10	3.5380E+10
x(FT)	0.0000	0.0000	9.1200	9.6900	0.0000	0.0000	7.4100	9.1200	0.0000	0.0000	0.0000
2	0.1852	0.1928	2.7031E+05	5.5903E+05	1.2833E+04	1.7549E+04	165.65	229.10	1.6007E+04	1.2847E+10	3.5380E+10

x(FT)	0.0000	0.0000	9.1200	9.6900	0.0000	0.0000	7.4100	9.1200	0.0000	0.0000	0.0000
3	0.1849	0.1928	2.6887E+05	5.5769E+05	1.2934E+04	1.7660E+04	165.90	229.48	1.3116E+04	1.2847E+10	3.5380E+10
x(FT)	0.0000	0.0000	9.1200	9.6900	0.0000	0.0000	7.4100	9.1200	0.0000	0.0000	0.0000
4	0.1854	0.1931	2.7096E+05	5.6006E+05	1.2812E+04	1.7544E+04	165.70	229.31	1.6890E+04	1.2847E+10	3.5380E+10
x(FT)	0.0000	0.0000	9.1200	9.6900	0.0000	0.0000	7.4100	9.1200	0.0000	0.0000	0.0000
5	0.1852	0.1931	2.6960E+05	5.5882E+05	1.2904E+04	1.7646E+04	165.91	229.66	1.4220E+04	1.2847E+10	3.5380E+10
x(FT)	0.0000	0.0000	9.1200	9.6900	0.0000	0.0000	7.4100	9.1200	0.0000	0.0000	0.0000
6	0.1849	0.1931	2.6801E+05	5.5728E+05	1.3023E+04	1.7774E+04	166.22	230.08	1.0896E+04	1.2847E+10	3.5380E+10
x(FT)	0.0000	0.0000	9.1200	9.6900	0.0000	0.0000	7.4100	9.1200	0.0000	0.0000	0.0000
7	0.1854	0.1933	2.7025E+05	5.5985E+05	1.2882E+04	1.7640E+04	165.95	229.86	1.5123E+04	1.2847E+10	3.5380E+10
x(FT)	0.0000	0.0000	9.1200	9.6900	0.0000	0.0000	7.4100	9.1200	0.0000	0.0000	0.0000
8	0.1852	0.1933	2.6874E+05	5.5840E+05	1.2993E+04	1.7760E+04	166.23	230.26	1.2000E+04	1.2847E+10	3.5380E+10
x(FT)	0.0000	0.0000	9.1200	9.6900	0.0000	0.0000	7.4100	9.1200	0.0000	0.0000	0.0000
9	0.1849	0.1933	2.6712E+05	5.5682E+05	1.3115E+04	1.7890E+04	166.56	230.69	1.2093E+04	1.2847E+10	3.5380E+10
x(FT)	0.0000	0.0000	9.1200	9.6900	0.0000	0.0000	7.4100	9.1200	0.0000	0.0000	0.0000
Max.	0.1854	0.1933	2.7166E+05	5.6026E+05	1.3115E+04	1.7890E+04	166.56	230.69	1.8657E+04	1.2847E+10	3.5380E+10
Pile N.	1	7	1	1	9	9	9	9	1	1	1

***** SUMMARY FOR LOAD CASES AND COMBINATIONS *****

***** LOAD CASES RESULTS *****

LOAD CASE : 1

* TABLE L * COMPUTATION ON PILE CAP

* EQUIVALENT CONCENTRATED LOAD AT ORIGIN *

LOAD X,LBS	LOAD Y,LBS	LOAD Z,LBS	MOM X,LBS-IN	MOM Y,LBS-IN	MOM Z,LBS-IN
1.10914E+06	0.00000	22310.0	0.00000	2.89152E+06	1.94640E+05

* DISPLACEMENT OF GROUPED PILE FOUNDATION AT ORIGIN *

DISP X,IN	DISP Y,IN	DISP Z,IN	ROT X,RAD	ROT Y,RAD	ROT Z,RAD
0.0596188	-2.55774E-04	0.0135397	-1.39307E-09	1.24690E-04	6.84206E-06

* PILE TOP DISPLACEMENTS, GLOBAL *

	DISP. X,IN	DISP. Y,IN	DISP. Z,IN	ROT. X,RAD	ROT. Y,RAD	ROT. Z,RAD
	***********	***********	***********	***********	***********	***********
MINIMUM	0.053305	-2.5584E-04	0.013540	-1.3931E-09	1.2469E-04	6.8421E-06
Pile N.	3	3	1	1	1	1
MAXIMUM	0.065932	-2.5571E-04	0.013540	-1.3931E-09	1.2469E-04	6.8421E-06
Pile N.	7	1	7	1	1	1

* PILE TOP REACTIONS, GLOBAL *

	FOR. X,LBS	FOR. Y,LBS	FOR. Z,LBS	MOM X,LBS-IN	MOM Y,LBS-IN	MOM Z,LBS-IN
	***********	***********	***********	***********	***********	***********
MINIMUM	1.1058E+05	-0.043156	2477.6	-0.097715	-6.2580E+04	1229.6
Pile N.	3	3	7	1	7	4
MAXIMUM	1.3580E+05	0.042917	2480.1	-0.097715	-6.2533E+04	1230.7

Pile N.	7	7	3	1	3	3

* PILE TOP DISPLACEMENTS, LOCAL *

	DISP. x,IN	DISP. y,IN	DISP. z,IN	ROT. x,RAD	ROT. y,RAD	ROT. z,RAD
	***********	***********	***********	***********	***********	***********
MINIMUM	0.053305	-2.5584E-04	0.013540	-1.3931E-09	1.2469E-04	6.8421E-06
Pile N.	3	3	1	1	1	1
MAXIMUM	0.065932	-2.5571E-04	0.013540	-1.3931E-09	1.2469E-04	6.8421E-06
Pile N.	7	1	7	1	1	1

* PILE TOP REACTIONS, LOCAL *

	AXIAL,LBS	LAT. y,LBS	LAT. z,LBS	MOM x,LBS-IN	MOM y,LBS-IN	MOM z,LBS-IN
	***********	***********	***********	***********	***********	***********
MINIMUM	1.1058E+05	-0.043156	2477.6	-0.097715	-6.2580E+04	1229.6
Pile N.	3	3	7	1	7	4
MAXIMUM	1.3580E+05	0.042917	2480.1	-0.097715	-6.2533E+04	1230.7
Pile N.	7	7	3	1	3	3

* EFFECTS FOR LATERALLY LOADED PILE *

PILE	DISPL. y-DIR IN	DISPL. z-DIR IN	MOMENT z-DIR LBS-IN	MOMENT y-DIR LBS-IN	SHEAR y-DIR LBS	SHEAR z-DIR LBS	SOIL REACT y-DIR LBS/IN	SOIL REACT z-DIR LBS/IN	TOTAL STRESS LBS/IN**2
******	**********	**********	**********	**********	**********	**********	**********	**********	**********
Min.	-2.5584E-04	-3.7079E-04	-1235.9	-6.2580E+04	-0.9204	-636.43	-0.3574	-8.1186	3214.4
Pile N.	3	7	3	7	7	7	3	7	3
Max.	1.6582E-05	0.013540	54.396	5.4477E+04	15.062	2480.8	0.2194	40.291	4322.1
Pile N.	7	1	7	7	7	3	7	3	7

LOAD CASE : 2

* TABLE L * COMPUTATION ON PILE CAP

* EQUIVALENT CONCENTRATED LOAD AT ORIGIN *

LOAD X,LBS	LOAD Y,LBS	LOAD Z,LBS	MOM X,LBS-IN	MOM Y,LBS-IN	MOM Z,LBS-IN
5.49550E+05	3090.00	44950.0	0.00000	6.18564E+06	3.60960E+05

* DISPLACEMENT OF GROUPED PILE FOUNDATION AT ORIGIN *

DISP X,IN	DISP Y,IN	DISP Z,IN	ROT X,RAD	ROT Y,RAD	ROT Z,RAD
0.0286439	1.49625E-03	0.0301764	-2.48894E-08	2.65168E-04	8.26272E-06

* PILE TOP DISPLACEMENTS, GLOBAL *

	DISP. X,IN	DISP. Y,IN	DISP. Z,IN	ROT. X,RAD	ROT. Y,RAD	ROT. Z,RAD
	***********	***********	***********	***********	***********	***********
MINIMUM	0.015519	1.4950E-03	0.030175	-2.4889E-08	2.6517E-04	8.2627E-06
Pile N.	3	3	1	1	1	1
MAXIMUM	0.041769	1.4974E-03	0.030177	-2.4889E-08	2.6517E-04	8.2627E-06
Pile N.	7	1	7	1	1	1

* PILE TOP REACTIONS, GLOBAL *

	FOR. X,LBS	FOR. Y,LBS	FOR. Z,LBS	MOM X,LBS-IN	MOM Y,LBS-IN	MOM Z,LBS-IN
	***********	***********	***********	***********	***********	***********
MINIMUM	3.4709E+04	343.20	4989.0	-1.7458	-1.3060E+05	1.4619E+04

```
    Pile N.          3            7            1            1            7            9
    MAXIMUM     8.7413E+04       343.47       4999.9      -1.7458  -1.3036E+05   1.4630E+04
    Pile N.          7            3            6            1            3            1

    * PILE TOP DISPLACEMENTS, LOCAL *
               DISP. x,IN   DISP. y,IN   DISP. z,IN   ROT. x,RAD   ROT. y,RAD   ROT. z,RAD
              ***********  ***********  ***********  ***********  ***********  ***********
    MINIMUM       0.015519   1.4950E-03     0.030175  -2.4889E-08   2.6517E-04   8.2627E-06
    Pile N.          3            3            1            1            1            1
    MAXIMUM       0.041769   1.4974E-03     0.030177  -2.4889E-08   2.6517E-04   8.2627E-06
    Pile N.          7            1            7            1            1            1

    * PILE TOP REACTIONS, LOCAL *
               AXIAL,LBS    LAT. y,LBS   LAT. z,LBS MOM x,LBS-IN MOM y,LBS-IN MOM z,LBS-IN
              ***********  ***********  ***********  ***********  ***********  ***********
    MINIMUM     3.4709E+04       343.20       4989.0      -1.7458  -1.3060E+05   1.4619E+04
    Pile N.          3            7            1            1            7            9
    MAXIMUM     8.7413E+04       343.47       4999.9      -1.7458  -1.3036E+05   1.4630E+04
    Pile N.          7            3            6            1            3            1

    * EFFECTS FOR LATERALLY LOADED PILE *

  PILE     DISPL.      DISPL.      MOMENT      MOMENT       SHEAR       SHEAR     SOIL REACT  SOIL REACT     TOTAL
           y-DIR       z-DIR       z-DIR       y-DIR       y-DIR       z-DIR       y-DIR       z-DIR       STRESS
            IN          IN        LBS-IN      LBS-IN        LBS         LBS        LBS/IN      LBS/IN     LBS/IN**2
 ******  **********  **********  **********  **********  **********  **********  **********  **********  **********
   Min. -3.8698E-05 -7.5694E-04 -1.4630E+04 -1.3059E+05     -49.606     -1332.2     -0.8041     -17.097      1009.0
Pile N.       7           7           1           7           7           7           7           7           3
   Max.  1.5273E-03    0.030178      3279.7  1.1275E+05      343.60      5000.4      5.2267      79.562      3357.7
Pile N.       1           7           4           7           3           9           1           9           7

  LOAD CASE :     3

  * TABLE L *   COMPUTATION ON PILE CAP

  * EQUIVALENT CONCENTRATED LOAD AT ORIGIN *
        LOAD X,LBS     LOAD Y,LBS     LOAD Z,LBS    MOM X,LBS-IN   MOM Y,LBS-IN   MOM Z,LBS-IN
        1.10285E+06    1.16070E+05    1.58830E+05        0.00000    2.27551E+07   -1.47352E+07

  * DISPLACEMENT OF GROUPED PILE FOUNDATION AT ORIGIN *
         DISP X,IN      DISP Y,IN      DISP Z,IN      ROT X,RAD      ROT Y,RAD      ROT Z,RAD
          0.0639588        0.18515        0.19309   -5.03744E-06    1.17502E-03   -7.89652E-04

    * PILE TOP DISPLACEMENTS, GLOBAL *
               DISP. X,IN   DISP. Y,IN   DISP. Z,IN   ROT. X,RAD   ROT. Y,RAD   ROT. Z,RAD
              ***********  ***********  ***********  ***********  ***********  ***********
    MINIMUM      -0.030346       0.1849       0.1928  -5.0374E-06   1.1750E-03  -7.8965E-04
    Pile N.          9            3            1            1            1            1
    MAXIMUM         0.1583       0.1854       0.1933  -5.0374E-06   1.1750E-03  -7.8965E-04
    Pile N.          1            1            7            1            1            1

    * PILE TOP REACTIONS, GLOBAL *
               FOR. X,LBS   FOR. Y,LBS   FOR. Z,LBS MOM X,LBS-IN MOM Y,LBS-IN MOM Z,LBS-IN
```

	***********	***********	***********	***********	***********	***********
MINIMUM	-6.0216E+04	1.2698E+04	1.7427E+04	-353.35	-7.5487E+05	5.6626E+05
Pile N.	9	1	1	1	7	9
MAXIMUM	2.8341E+05	1.3124E+04	1.7894E+04	-353.35	-7.4980E+05	5.7102E+05
Pile N.	1	9	9	1	3	1

* PILE TOP DISPLACEMENTS, LOCAL *

	DISP. x,IN	DISP. y,IN	DISP. z,IN	ROT. x,RAD	ROT. y,RAD	ROT. z,RAD
	***********	***********	***********	***********	***********	***********
MINIMUM	-0.030346	0.1849	0.1928	-5.0374E-06	1.1750E-03	-7.8965E-04
Pile N.	9	3	1	1	1	1
MAXIMUM	0.1583	0.1854	0.1933	-5.0374E-06	1.1750E-03	-7.8965E-04
Pile N.	1	1	7	1	1	1

* PILE TOP REACTIONS, LOCAL *

	AXIAL,LBS	LAT. y,LBS	LAT. z,LBS	MOM x,LBS-IN	MOM y,LBS-IN	MOM z,LBS-IN
	***********	***********	***********	***********	***********	***********
MINIMUM	-6.0216E+04	1.2698E+04	1.7427E+04	-353.35	-7.5487E+05	5.6626E+05
Pile N.	9	1	1	1	7	9
MAXIMUM	2.8341E+05	1.3124E+04	1.7894E+04	-353.35	-7.4980E+05	5.7102E+05
Pile N.	1	9	9	1	3	1

* EFFECTS FOR LATERALLY LOADED PILE *

PILE	DISPL. y-DIR IN	DISPL. z-DIR IN	MOMENT z-DIR LBS-IN	MOMENT y-DIR LBS-IN	SHEAR y-DIR LBS	SHEAR z-DIR LBS	SOIL REACT y-DIR LBS/IN	SOIL REACT z-DIR LBS/IN	TOTAL STRESS LBS/IN**2
******	**********	**********	**********	**********	**********	**********	**********	**********	**********
Min.	-3.0194E-03	-3.5656E-03	-5.7102E+05	-7.5487E+05	-4111.0	-6934.1	-67.947	-91.770	544.32
Pile N.	1	1	1	7	1	1	1	1	6
Max.	0.1854	0.1933	2.7166E+05	5.6026E+05	1.3115E+04	1.7890E+04	166.56	230.69	1.8657E+04
Pile N.	1	7	1	1	9	9	9	9	1

H-pile Structural Capacity Check

H-Pile Capacity Using Corroded Properties (Using AREMA 2019)
(to be confirmed by structural engineer prior to final design)

The **fixed** pile to pile cap connection condition is evaluated.

Pile Size = **HP14x117**

F_y = 50 ksi
E = 29000 ksi

Corroded Properties, Assume 0.0625 inch corrosion (1/16" corrosion)
The entire H-pile will be under groundwater level.

Properties			Corroded Properties		
b_f =	14.9	in	b_f =	14.78	in
d =	14.2	in	d =	14.08	in
t_f=	0.805	in	t_f=	0.68	in
t_w=	0.805	in	t_w=	0.68	in
h=	12.59	in	h=	12.72	in
A =	34.4	in^2	A =	28.74	in^2
I_x=	1220	in^4	I_x=	1018.61	in^4
I_y=	443	in^4	I_y=	365.88	in^4
S_x =	172	in^3	S_x =	144.74	in^3
S_y=	59.5	in^3	S_y=	49.53	in^3
Z_x=	194	in^3	Z_x=	162.06	in^3
Z_y=	91.4	in^3	Z_y=	75.69	in^3
r_x=	5.96	in	r_x=	5.95	in
r_y=	3.59	in	r_y=	3.57	in
c=	7.1	in	c=	7.04	in

Following AREMA, Vol. 2, Chapter 8, Section 4.4.2.6

Allowable Stresses

Axial compression only:	F_a =	12.60	
Bending only:	$F_b = 0.55*F_y$	27.50	ksi
Combined compression and bending:	$F_{ab} = 0.55*F_y$	27.50	ksi

Lateral bracing

Unbraced Length L_x=L_y = 194.4 in 16.2 ft

Design of members for compression - Table 15-1-11

Flexural buckling for major axis

Effective length factor	K_x =	1.0	(fixed-fixed end condition)	
Column slenderness	$K_x * L_x / r_x$ =	32.65	< 5.034/Sqrt[Fy/E]=	121.2
			> 0.629/Sqrt[Fy/E]=	15.1
	F_a =	29.83		

Flexural buckling for minor axis

Effective length factor	K_y =	1.0		
Column slenderness	$K_y * L_y / r_y$ =	54.48	< 5.034/Sqrt[Fy/E]=	121.2
			> 0.629/Sqrt[Fy/E]=	15.1
	F_a =	29.71		

Allowable compressive capacity;	$P_a=F_a*A$=	**362**	kips (under axial load check only)
Allowable bending capacity, X- Axis	$M_{xa} = F_b*S_x$=	**332**	k-ft
Allowable bending capacity, Y- Axis	$M_{ya} = F_b*S_y$=	**113**	k-ft

Design Loads & Moments

		Loading Group III		
Axial Compression Load	P =	**283.4**	kips	Compression, OK
X-Axis Moment	M_x =	**63**	k-ft	X-Axis Bending, OK
Y-Axis Moment	M_y =	**48**	k-ft	Y-Axis Bending, OK

Design of members for combined forces (AREMA, Vol. 2, Chapter 15, Section 1.3.14.1)
Members subjected to both axial compression and bending stresses shall be proportioned to satisfy:

when $\frac{f_a}{F_a} \le 0.15$

$$\frac{f_a}{F_a} + \frac{f_{b1}}{F_{b1}} + \frac{f_{b2}}{F_{b2}} \le 1.0$$

Axial Compression Stress, f_a = 9.86 ksi
X-Axis Bending Stress, f_{bx} = 5.22 ksi
Y-Axis Bending Stress, f_{by} = 11.63 ksi
f_a/F_a = 0.78 >0.15

when $\frac{f_a}{F_a} > 0.15$

$$\frac{f_a}{F_a} + \frac{f_{b1}}{F_{b1}\left[1 - \frac{f_a}{0.514\pi^2 E}\left(\frac{k_1 l_1}{r_1}\right)^2\right]} + \frac{f_{b2}}{F_{b2}\left[1 - \frac{f_a}{0.514\pi^2 E}\left(\frac{k_2 l_2}{r_2}\right)^2\right]} \le 1.0$$

= 1.09

Combined Stressed = 109%
< 125%
OK

Note: 125% allowable stress is based on AREMA Chapter 8, Table 8-2-4.

APILE Pile Analysis Results

```
=========================================================================

                 APILE for Windows, Version 2018.8.5

                  Serial Number :  419704647

                  A Program for Analyzing the Axial Capacity
                  and Short-term Settlement of Driven Piles
                            under Axial Loading.
                (c) Copyright ENSOFT, Inc., 1987-2015
                      All Rights Reserved

=========================================================================


This program is licensed to :

CH2M Hill Inc.
Global License, Global License

Path to file locations      : C:\Users\hada\Desktop\Worked Projects\VT Bridge 132\Geotech Memo\Appendix
F - Geotechnical Analysis\H-pile Group Option\APILE Run\
Name of input data file     : HP14x117.ap8d
Name of output file         : HP14x117.ap8o
Name of plot output file    : HP14x117.ap8p

-------------------------------------------------------------------------
                     Time and Date of Analysis
-------------------------------------------------------------------------

          Date:  December 16, 2021     Time:  13:20:56



1
     ********************
     * INPUT INFORMATION *
     ********************



     VTrans Bridge 132

     DESIGNER   : Da Ha

     JOB NUMBER : E2X88317


     METHOD FOR UNIT LOAD TRANSFERS :

     - API RP 2A (American Petroleum Institute)
       Unfactored Unit Side Friction and Unit Side Resistance are used.



     COMPUTATION METHOD(S) FOR PILE CAPACITY :

     - FHWA (Federal Highway Administration)
     - USACE (U.S. Army Corps of Engineers)
       # Critical Depth Method for Sand:
         10 to 20 Pile Diameter based on the Density
       Use Long Pile Option
     - Revised Lambda
     - API RP 2A (American Petroleum Institute)


     TYPE OF LOADING :
     - COMPRESSION


     PILE TYPE :

     H-Pile/Steel Pile
```

DATA FOR AXIAL STIFFNESS :

- MODULUS OF ELASTICITY = 0.290E+08 PSI
- CROSS SECTION AREA = 208.70 IN2

NONCIRCULAR PILE PROPERTIES :

- TOTAL PILE LENGTH, TL = 56.00 FT.
- BATTER ANGLE = 0.00 DEG
- PILE STICKUP LENGTH, PSL = 0.00 FT.
- ZERO FRICTION LENGTH, ZFL = 0.00 FT.
- PERIMETER OF PILE = 57.80 IN.
- TIP AREA OF PILE = 208.70 IN2
- INCREMENT OF PILE LENGTH
  USED IN COMPUTATION = 1.00 FT.

SOIL INFORMATIONS :

DEPTH FT.	SOIL TYPE	LATERAL EARTH PRESSURE	EFFECTIVE UNIT WEIGHT LB/CF	FRICTION ANGLE DEGREES	BEARING CAPACITY FACTOR
0.00	SAND	0.90	62.60	36.00	50.00
57.00	SAND	0.90	62.60	36.00	50.00
57.00	CLAY	0.00	87.60	0.00	0.00
70.00	CLAY	0.00	87.60	0.00	0.00

MAXIMUM UNIT FRICTION KSF	MAXIMUM UNIT BEARING KSF	UNDISTURB SHEAR STRENGTH KSF	REMOLDED SHEAR STRENGTH KSF	BLOW COUNT	UNIT SKIN FRICTION KSF	UNIT END BEARING KSF
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	288.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	288.00	0.00	0.00	0.00	0.00

* MAXIMUM UNIT FRICTION AND/OR MAXIMUM UNIT BEARING
  WERE SET TO BE 0.10E+08 BECAUSE THE USER DOES NOT
  PLAN TO LIMIT THE COMPUTED DATA.

DEPTH FT.	LRFD FACTOR ON UNIT FRICTION	LRFD FACTOR ON UNIT BEARING
0.00	1.000	1.000
57.00	1.000	1.000
57.00	1.000	1.000
70.00	1.000	1.000

1

```
**********************
* COMPUTATION RESULT *
**********************
```

```
**********************
*  FED. HWY. METHOD  *
**********************
```

PILE PENETRATION FT.	TOTAL SKIN FRICTION KIP	END BEARING KIP	ULTIMATE CAPACITY KIP
0.00	0.0	2.8	2.8
1.00	0.2	5.2	5.4
2.00	0.7	9.3	10.0
3.00	1.6	14.6	16.2
4.00	2.8	19.5	22.3
5.00	4.4	24.4	28.8
6.00	6.3	29.3	35.6
7.00	8.6	34.2	42.8
8.00	11.2	39.1	50.3
9.00	14.2	43.9	58.2
10.00	17.6	48.8	66.4
11.00	21.3	53.7	75.0
12.00	25.3	58.6	83.9
13.00	29.7	63.5	93.2
14.00	34.5	68.3	102.8
15.00	39.5	73.2	112.8
16.00	45.0	78.1	123.1
17.00	50.8	83.0	133.8
18.00	56.9	87.9	144.8
19.00	63.5	92.7	156.2
20.00	70.3	97.6	167.9
21.00	77.5	102.5	180.0
22.00	85.1	107.4	192.5
23.00	93.0	112.3	205.3
24.00	101.2	117.2	218.4
25.00	109.9	122.0	231.9
26.00	118.8	126.9	245.7
27.00	128.1	131.8	259.9
28.00	137.8	136.7	274.5
29.00	147.8	141.6	289.4
30.00	158.2	146.4	304.6
31.00	168.9	151.3	320.2
32.00	180.0	156.2	336.2
33.00	191.4	161.1	352.5
34.00	203.2	166.0	369.2
35.00	215.3	170.8	386.2
36.00	227.8	175.7	403.5
37.00	240.6	180.6	421.2
38.00	253.8	185.5	439.3
39.00	267.3	190.4	457.7
40.00	281.2	195.3	476.5
41.00	295.5	200.1	495.6
42.00	310.1	205.0	515.1
43.00	325.0	209.8	534.9
44.00	340.3	213.9	554.2
45.00	355.9	216.9	572.8
46.00	371.9	218.8	590.7
47.00	388.3	219.7	607.9
48.00	405.0	219.7	624.7
49.00	422.0	219.7	641.7
50.00	439.4	219.7	659.1
51.00	457.2	219.7	676.9
52.00	475.3	219.7	695.0
53.00	493.7	219.7	713.5
54.00	512.5	219.7	732.3
55.00	531.7	450.2	981.9
56.00	551.2	1219.2	1770.4

****
* ARMY CORPS METHOD *
****

PILE PENETRATION FT.	TOTAL SKIN FRICTION KIP	END BEARING KIP	ULTIMATE CAPACITY KIP
0.00	0.0	2.9	2.9
1.00	0.1	5.5	5.6
2.00	0.5	9.7	10.1

3.00	1.0	15.2	16.3
4.00	1.8	20.3	22.2
5.00	2.9	25.4	28.3
6.00	4.2	30.5	34.6
7.00	5.7	35.6	41.2
8.00	7.4	40.6	48.0
9.00	9.4	45.7	55.1
10.00	11.5	50.8	62.4
11.00	14.0	55.9	69.9
12.00	16.6	61.0	77.6
13.00	19.5	66.0	85.6
14.00	22.6	71.1	93.8
15.00	26.0	76.2	102.2
16.00	29.6	81.3	110.9
17.00	33.4	86.4	119.7
18.00	37.4	91.5	128.9
19.00	41.7	96.5	138.2
20.00	46.2	101.6	147.8
21.00	50.9	106.7	157.6
22.00	55.9	111.8	167.7
23.00	61.1	116.8	177.9
24.00	66.5	121.0	187.5
25.00	72.2	124.1	196.3
26.00	78.1	126.1	204.2
27.00	84.2	127.0	211.2
28.00	90.5	127.0	217.6
29.00	97.1	127.0	224.1
30.00	103.9	127.0	231.0
31.00	110.9	127.0	238.0
32.00	118.0	127.0	245.0
33.00	125.1	127.0	252.1
34.00	132.2	127.0	259.2
35.00	139.3	127.0	266.3
36.00	146.4	127.0	273.4
37.00	153.4	127.0	280.5
38.00	160.5	127.0	287.5
39.00	167.6	127.0	294.6
40.00	174.7	127.0	301.7
41.00	181.8	127.0	308.8
42.00	188.8	127.0	315.9
43.00	195.9	127.0	322.9
44.00	203.0	127.0	330.0
45.00	210.1	127.0	337.1
46.00	217.2	127.0	344.2
47.00	224.3	127.0	351.3
48.00	231.3	127.0	358.4
49.00	238.4	127.0	365.4
50.00	245.5	127.0	372.5
51.00	252.6	127.0	379.6
52.00	259.7	127.0	386.7
53.00	266.8	127.0	393.8
54.00	273.8	127.0	400.9
55.00	280.9	363.6	644.5
56.00	288.0	1152.7	1440.7

****
* LAMBDA 2 METHOD   *
****

PILE PENETRATION FT.	TOTAL SKIN FRICTION KIP	END BEARING KIP	ULTIMATE CAPACITY KIP
0.00	0.0	2.6	2.6
1.00	0.2	4.9	5.0
2.00	0.6	8.6	9.2
3.00	1.2	13.6	14.8
4.00	2.0	18.1	20.2
5.00	2.9	22.7	25.6
6.00	4.0	27.2	31.2
7.00	5.2	31.8	37.0
8.00	6.5	36.3	42.8

9.00	8.0	40.8	48.8
10.00	9.5	45.4	54.8
11.00	11.1	49.9	61.0
12.00	12.8	54.4	67.2
13.00	14.5	59.0	73.5
14.00	16.4	63.5	79.9
15.00	18.3	68.0	86.3
16.00	20.3	72.6	92.8
17.00	22.3	77.1	99.4
18.00	24.4	81.7	106.0
19.00	26.5	86.2	112.7
20.00	28.7	90.7	119.4
21.00	30.9	95.3	126.2
22.00	33.2	99.8	133.0
23.00	35.4	104.3	139.8
24.00	37.8	108.9	146.7
25.00	40.1	113.4	153.6
26.00	42.5	117.9	160.5
27.00	44.9	122.5	167.4
28.00	47.4	127.0	174.4
29.00	49.8	131.6	181.4
30.00	52.3	136.1	188.4
31.00	54.8	140.6	195.4
32.00	57.3	145.2	202.5
33.00	59.8	149.7	209.5
34.00	62.4	154.2	216.6
35.00	64.9	158.8	223.7
36.00	67.5	163.3	230.8
37.00	70.0	167.8	237.8
38.00	72.6	172.4	244.9
39.00	75.1	176.9	252.0
40.00	77.9	181.5	259.4
41.00	81.9	186.0	267.9
42.00	85.9	190.5	276.4
43.00	90.1	195.1	285.1
44.00	94.3	199.6	293.9
45.00	98.6	204.1	302.8
46.00	103.1	208.7	311.7
47.00	107.6	213.2	320.8
48.00	112.2	217.7	330.0
49.00	117.0	222.3	339.2
50.00	121.8	226.8	348.6
51.00	126.7	231.4	358.0
52.00	131.7	235.9	367.6
53.00	136.8	240.4	377.2
54.00	142.0	245.0	387.0
55.00	147.3	477.4	624.8
56.00	152.8	1241.7	1394.5

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**********************
*  API RP-2A (2010)  *
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```

PILE PENETRATION FT.	TOTAL SKIN FRICTION KIP	END BEARING KIP	ULTIMATE CAPACITY KIP
0.00	0.0	2.6	2.6
1.00	0.1	4.9	5.0
2.00	0.3	8.6	8.9
3.00	0.7	13.6	14.3
4.00	1.3	18.1	19.4
5.00	2.0	22.7	24.7
6.00	2.9	27.2	30.2
7.00	4.0	31.8	35.7
8.00	5.2	36.3	41.5
9.00	6.6	40.8	47.4
10.00	8.2	45.4	53.5
11.00	9.9	49.9	59.8
12.00	11.7	54.4	66.2
13.00	13.8	59.0	72.8
14.00	16.0	63.5	79.5

15.00	18.3	68.0	86.4
16.00	20.9	72.6	93.5
17.00	23.6	77.1	100.7
18.00	26.4	81.7	108.1
19.00	29.4	86.2	115.6
20.00	32.6	90.7	123.3
21.00	36.0	95.3	131.2
22.00	39.5	99.8	139.3
23.00	43.1	104.3	147.5
24.00	47.0	108.9	155.8
25.00	51.0	113.4	164.4
26.00	55.1	117.9	173.1
27.00	59.4	122.5	181.9
28.00	63.9	127.0	190.9
29.00	68.6	131.6	200.1
30.00	73.4	136.1	209.5
31.00	78.3	140.6	219.0
32.00	83.5	145.2	228.6
33.00	88.8	149.7	238.5
34.00	94.2	154.2	248.5
35.00	99.9	158.8	258.6
36.00	105.7	163.3	269.0
37.00	111.6	167.8	279.5
38.00	117.7	172.4	290.1
39.00	124.0	176.9	300.9
40.00	130.4	181.5	311.9
41.00	137.0	186.0	323.0
42.00	143.8	190.5	334.3
43.00	150.7	195.1	345.8
44.00	157.8	199.6	357.4
45.00	165.1	204.1	369.2
46.00	172.5	208.7	381.2
47.00	180.1	213.2	393.3
48.00	187.8	217.7	405.6
49.00	195.7	222.3	418.0
50.00	203.8	226.8	430.6
51.00	212.1	231.4	443.4
52.00	220.5	235.9	456.3
53.00	229.0	240.4	469.4
54.00	237.7	245.0	482.7
55.00	246.6	477.4	724.1
56.00	255.7	1241.7	1497.4

NOTES:
- AN ASTERISK IS PLACED IN THE END-BEARING COLUMN
  IF THE TIP RESISTANCE IS CONTROLLED BY THE FRICTION
  OF SOIL PLUG INSIDE AN OPEN-ENDED PIPE PILE.

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*************************************************
* COMPUTE LOAD-DISTRIBUTION AND LOAD-SETTLEMENT *
* CURVES FOR AXIAL LOADING                      *
*************************************************
```

T-Z CURVE NO.	NO. OF POINTS	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
1	10	0.0000E+00		
			0.0000E+00	0.0000E+00
			0.0000E+00	0.1000E-01
			0.0000E+00	0.2000E-01
			0.0000E+00	0.4000E-01
			0.0000E+00	0.6000E-01
			0.0000E+00	0.8000E-01
			0.0000E+00	0.9000E-01
			0.0000E+00	0.1000E+00
			0.0000E+00	0.5000E+00
			0.0000E+00	0.2000E+01
2	10	0.2853E+02		
			0.0000E+00	0.0000E+00

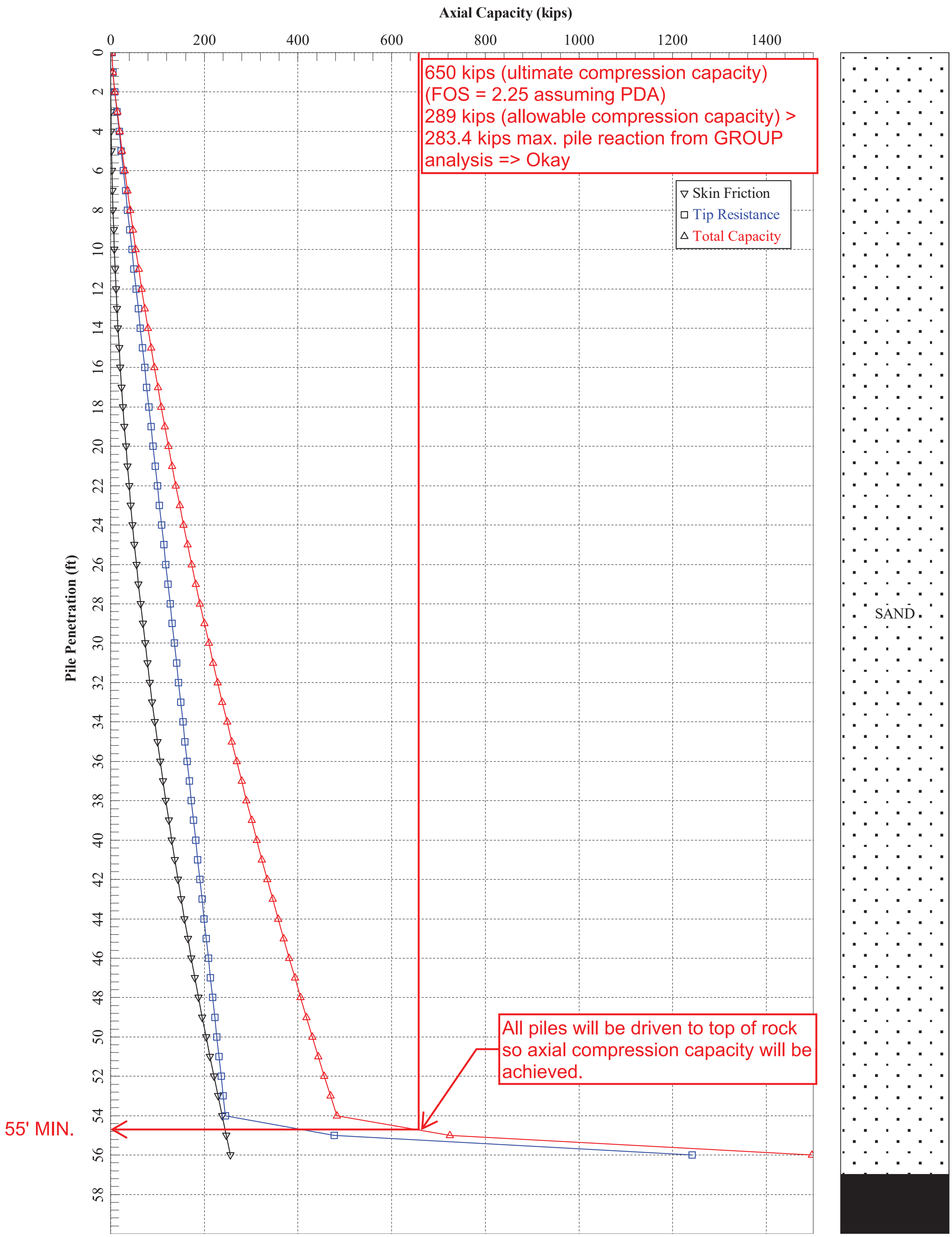
			0.6706E+00	0.1000E-01
			0.1341E+01	0.2000E-01
			0.2682E+01	0.4000E-01
			0.4024E+01	0.6000E-01
			0.5365E+01	0.8000E-01
			0.6035E+01	0.9000E-01
			0.6706E+01	0.1000E+00
			0.6706E+01	0.5000E+00
			0.6706E+01	0.2000E+01
3	10	0.5696E+02		
			0.0000E+00	0.0000E+00
			0.1316E+01	0.1000E-01
			0.2633E+01	0.2000E-01
			0.5266E+01	0.4000E-01
			0.7899E+01	0.6000E-01
			0.1053E+02	0.8000E-01
			0.1185E+02	0.9000E-01
			0.1316E+02	0.1000E+00
			0.1316E+02	0.5000E+00
			0.1316E+02	0.2000E+01
4	10	0.5700E+02		
			0.0000E+00	0.0000E+00
			0.3949E+01	0.2944E-01
			0.6582E+01	0.5703E-01
			0.9874E+01	0.1049E+00
			0.1185E+02	0.1472E+00
			0.1316E+02	0.1840E+00
			0.1185E+02	0.3680E+00
			0.1185E+02	0.5519E+00
			0.1185E+02	0.9199E+00
			0.1185E+02	0.3680E+01
5	10	0.6353E+02		
			0.0000E+00	0.0000E+00
			0.3949E+01	0.2944E-01
			0.6582E+01	0.5703E-01
			0.9874E+01	0.1049E+00
			0.1185E+02	0.1472E+00
			0.1316E+02	0.1840E+00
			0.1185E+02	0.3680E+00
			0.1185E+02	0.5519E+00
			0.1185E+02	0.9199E+00
			0.1185E+02	0.3680E+01
6	10	0.6996E+02		
			0.0000E+00	0.0000E+00
			0.3949E+01	0.2944E-01
			0.6582E+01	0.5703E-01
			0.9874E+01	0.1049E+00
			0.1185E+02	0.1472E+00
			0.1316E+02	0.1840E+00
			0.1185E+02	0.3680E+00
			0.1185E+02	0.5519E+00
			0.1185E+02	0.9199E+00
			0.1185E+02	0.3680E+01

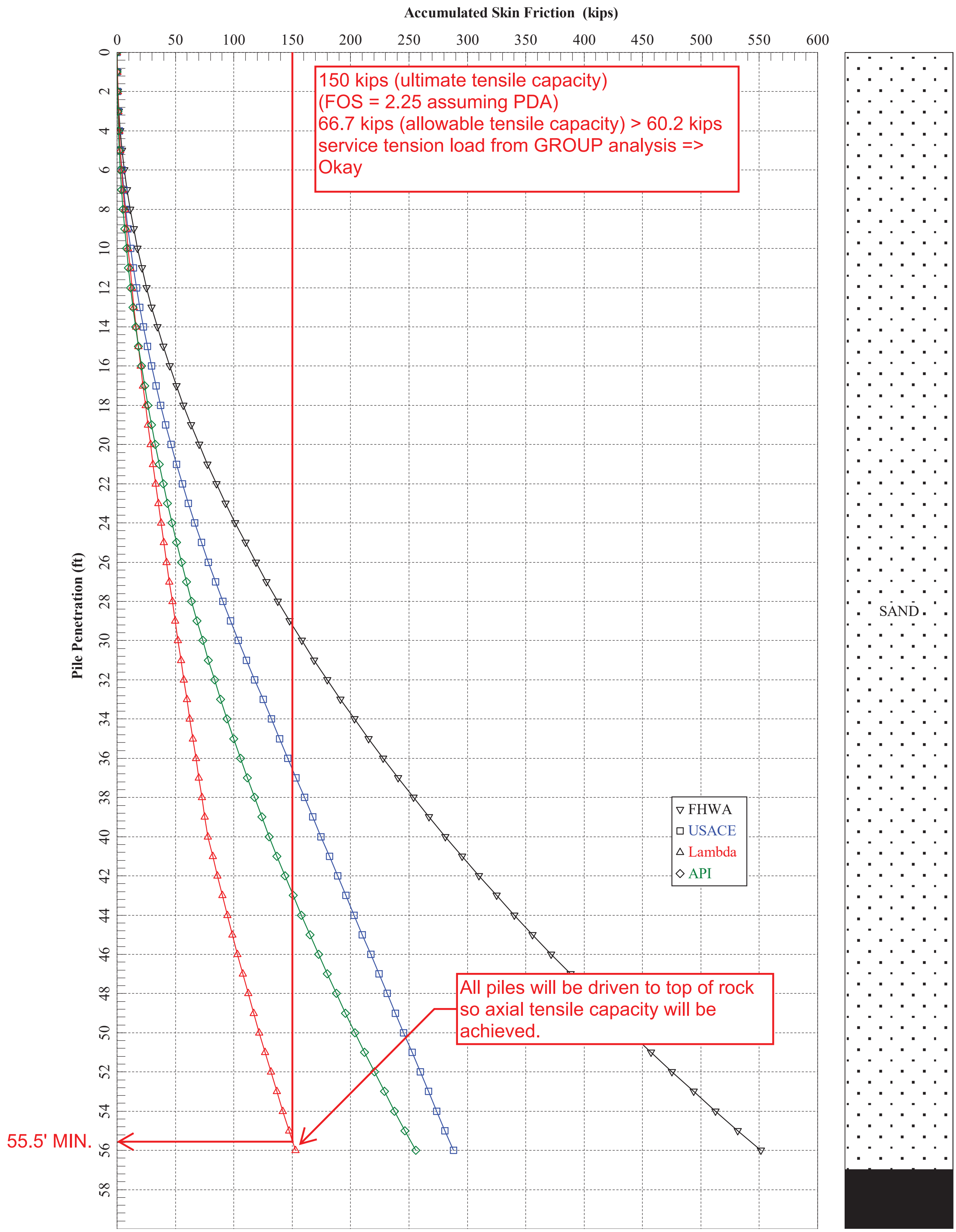
TIP LOAD KIP	TIP MOVEMENT IN.
0.0000E+00	0.0000E+00
0.7761E+02	0.9199E-02
0.1552E+03	0.1840E-01
0.3104E+03	0.3680E-01
0.6209E+03	0.2392E+00
0.9313E+03	0.7727E+00
0.1118E+04	0.1343E+01
0.1242E+04	0.1840E+01
0.1242E+04	0.2760E+01
0.1242E+04	0.3680E+01

LOAD VERSUS SETTLEMENT CURVE

****

TOP LOAD KIP	TOP MOVEMENT IN.	TIP LOAD KIP	TIP MOVEMENT IN.
0.1188E+01	0.2173E-03	0.8436E+00	0.1000E-03
0.1188E+02	0.2173E-02	0.8436E+01	0.1000E-02
0.5939E+02	0.1087E-01	0.4218E+02	0.5000E-02
0.1189E+03	0.2174E-01	0.8436E+02	0.1000E-01
0.4948E+03	0.9816E-01	0.3307E+03	0.5000E-01
0.6628E+03	0.1641E+00	0.4074E+03	0.1000E+00
0.1028E+04	0.6047E+00	0.7726E+03	0.5000E+00
0.1261E+04	0.1131E+01	0.1005E+04	0.1000E+01
0.1497E+04	0.2157E+01	0.1242E+04	0.2000E+01


Axial Capacity (kips)
0
200
400
600
800
1000
1200
1400
Pile Penetration (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
50
52
54
56
58
650 kips (ultimate compression capacity)
(FOS = 2.25 assuming PDA)
289 kips (allowable compression capacity) >
283.4 kips max. pile reaction from GROUP
analysis => Okay
Skin Friction
Tip Resistance
Total Capacity
All piles will be driven to top of rock
so axial compression capacity will be
achieved.
55' MIN.
SAND



Accumulated Skin Friction (kips)
0
50
100
150
200
250
300
350
400
450
500
550
600
Pile Penetration (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
50
52
54
56
58
150 kips (ultimate tensile capacity)
(FOS = 2.25 assuming PDA)
66.7 kips (allowable tensile capacity) > 60.2 kips
service tension load from GROUP analysis =>
Okay
FHWA
USACE
Lambda
API
All piles will be driven to top of rock
so axial tensile capacity will be
achieved.
55.5' MIN.
SAND

c. Live - Horizontal due to surcharge or fluid pressure

d. Centrifugal force

e. Earth pressure

f. Buoyancy

g. Negative skin friction

NOTE: Live Load Impact shall be considered only in Case A of Article 4.2.3 for steel or concrete piles extended above the ground line where they are rigidly connected to the member supporting the superstructure.

4.2.2.3 Secondary Loads and Forces

a. Wind and other lateral forces

b. Ice and Stream flow

c. Longitudinal forces

d. Seismic forces

e. Vessel impact in waterways[1]

4.2.3 LOADS ON PILES (2019)[2]

a. Pile foundations shall be designed using the most restrictive of the following load capacity cases:

(1) Case A: The capacity of an individual pile as a structural member

(2) Case B: The ability of the pile to transfer its load to the ground

(3) Case C: The capacity of the ground to support the load from the pile or pile group

b. When pile foundations are designed for primary and secondary loads in combination as defined in Part 2, the allowable loads may be increased 25% for Load Cases A, B, and C. The number of piles shall not be less than is required for primary forces alone with no increases in allowable stress for Case A. The minimum factor of safety shall be 2.0 for Cases B and C. For group friction piles, the factor of safety for Case C shall not fall below 2.0 for primary and secondary load combinations.

c. If the pile design capacity is not determined by geotechnical investigations, known positive contact with bedrock, or field testing of the pile, the Factor of Safety shall be increased to at least 2.5 times the required design load, and the Engineer shall be notified.

4.2.3.1 Eccentricity of Loads

The maximum design pile load under eccentric loading shall not exceed the allowable load as determined under Section 4.4, Pile Structural Design, with the appropriate factors of safety stipulated in Article 4.2.3. The piles shall be so spaced that the

1 For references see C-23.3.2 Sources of Information, Part 23, Pier Protection Systems at Spans Over Navigable Streams
2 See C - Commentary

eccentric load on the piles, due to primary loads and forces, will be distributed as equally as practicable to the piles in the group. Pile loads due to combinations of primary and secondary loads and forces shall not exceed that permitted by Article 4.2.3.

4.2.3.2 Uplift on Piles

a. In special cases when piles or pile groups are subjected to uplift, and sufficient bond and anchorage are provided between the pile, pile cap and the supported structure, the uplift shall be considered in the design of the pile foundation. The pile foundation shall be designed for uplift considering load capacity Cases A, B, and C of Article 4.2.3. The factor of safety for Cases B and C shall be a minimum of 2.0 for combinations of primary and secondary loads and forces, and a minimum of 3.0 for combinations of secondary loads and forces with dead load alone. The capacity of the pile as a structural member (Case A) shall be based on allowable stresses established in the applicable Parts of the *AREMA Manual for Railway Engineering*: Chapter 7, Timber Structures; Chapter 8, Part 2, Reinforced Concrete Design; or Chapter 15, Steel Structures. The allowable stresses may be increased by 25% for combinations of primary and secondary loads and forces.

b. The ultimate uplift capacity of an individual pile shall be determined by jacking test piles of identical type and dimension to that used in the design, and measuring the pull required per square foot of embedded surface area to raise the pile. When a tension pile group is involved, a group analysis shall also be undertaken. The maximum capacity of a tension pile group shall be considered to be the smaller of (1) the capacity of a single pile multiplied by the number of piles in the group, or (2) the weight of the block of soil contained within the perimeter of the groups, each with a minimum safety factor of 2.0, except as noted in paragraph a.

4.2.3.3 Spacing of Piles

a. Piles shall be spaced to nearly equalize their load consistent with economical design of the footings. The spacing of piles shall depend upon: the type of pile, that is whether friction or end bearing, the pile's structural and crushing strength, and the type of material resisting the pile. Generally, piles should be spaced, center-to-center, at least three times the minimum butt width of the pile. Piles should be spaced far enough apart, or other suitable means used, to prevent heaving or uplifting of adjacent piles during driving.

b. In small groups, the piles may be battered to enlarge the area sustaining the group, thereby increasing the load-carrying capacity of the group without unreasonably increasing the size of the foundation. End-bearing piles may be spaced in accordance with the capacity of the pile and the end-bearing stratum that will carry the design load. When closely spaced friction piles are contemplated, their total group capacity shall be verified by an acceptable geotechnical method which considers the capacity of the engaged soil mass to support the applied pile loads.

c. When determining spacing of piles in granular soils, consideration should be given to the increased difficulty of driving due to the increased soil density that will occur because of soil compaction (packing) or consolidation within the pile group.

4.2.3.4 Batter Piles[1]

a. Piles may be battered to help resist horizontal forces. Primary horizontal forces on pile foundations shall be resisted by batter piles where practicable. Such piles shall be designed to carry horizontal forces combined with their share of the vertical loads. In general, batter should not exceed 3 (horizontal) to 12 (vertical).

b. Secondary horizontal forces on pile foundations may be resisted by the shear and flexural capacity of the vertical piles in conjunction with the development of passive soil pressure, or friction between the soil/foundation interface where these resisting forces can be determined to exist for a particular foundation system. Where these resisting forces are not expected to be reliable over the service life of the structure, batter piles or other dependable means of resisting these forces shall be implemented.

[1] See C - Commentary