REPORT

Preliminary Geotechnical Design Report

I-295 DESERT ROAD BRIDGE REPLACEMENT #5720 (EXIT 20)

FREEPORT, MAINE

MAINEDOT WIN 023627.00

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Table of Contents

1.0 INTRODUCTION 1

2.0 PROJECT BACKGROUND 1

3.0 GEOLOGIC SETTING 1

3.1 Regional Surficial Geology 1

3.2 Regional Bedrock Geology 2

4.0 SUBSURFACE INVESTIGATIONS 2

4.1 Historical Geotechnical Investigation 2

4.2 Geotechnical Borings 3

5.0 LABORATORY TESTING PROGRAM 4

6.0 SUBSURFACE CONDITIONS 5

7.0 GEOTECHNICAL ANALYSES AND RECOMMENDATIONS 6

7.1 Frost Considerations 7

7.2 Seismic Site Classification 7

7.3 Approach Embankment Stability 7

7.4 Settlement 10

7.4.1 Approach Embankments 10

7.4.2 Water Force Main 10

7.5 Proposed Bridge Abutment Lateral Earth Pressure 11

7.6 Proposed Bridge Pile Foundations 12

7.6.1 Pile Design Load 12

7.6.2 Downdrag on Piles 13

7.6.3 Axial Pile Resistance 13

7.6.4 Lateral Pile Response 14

7.6.5 Pile Analysis Summary and Recommendations 16

8.0 CONSTRUCTION CONSIDERATIONS 17

9.0 RECOMMENDATIONS FOR FINAL DESIGN 18

10.0 REPORT AND EXPLORATION LIMITATIONS **19**

TABLES

Table 5-1 Laboratory Testing of Soils 4

Table 5-2: Laboratory Testing of Rocks 5

Table 7-1: Seismic Parameters 7

Table 7-2: Factors of Safety for Static Conditions and Pseudo-Static Seismic Conditions for the Embankments 8

Table 7-3: Factors of Safety for Static Conditions and Pseudo-Static Seismic Conditions for the Longitudinal Abutments (not considering piles). 9

Table 7-4: Estimated Settlement and Duration for Analyzed Bridge Embankment Locations 10

Table 7-5: Estimated Settlement and Duration for Analyzed Water Maine Locations 11

Table 7-6: Lateral Earth Pressure Coefficients 12

Table 7-7: Summary of Strength Limit State Factored Axial Pile Resistance 14

Table 7-8: LPile Lateral Analysis Results - First Iteration 15

Table 7-9: Pile Lateral Analysis Results - Second Iteration 16

Table 7-10: Pile Analysis Summary 16

Table 1: Subsurface Exploration Locations

Table 2: Summary of In Situ Vane Shear Testing Results

Table 3: Summary of Rock Core Quality

Table 4: Summary of Laboratory Soil Index and Classification Test Results

Table 5: Summary of Rock Laboratory Test Results

Table 6: Summary of Soil/Rock Properties Used in LPILE Analysis – Abutment No. 1

Table 7: Summary of Soil/Rock Properties Used in LPILE Analysis –Abutment No. 2

SHEETS

Sheet 1: Site Location

Sheet 2: Boring Location Plan

Sheet 3: Subsurface Profile A-A'

APPENDICES

Appendix A: Boring Logs

Appendix B: Rock Core Photos

Appendix C: Laboratory Test Results

Appendix D: Rock Core Laboratory Test Results

Appendix E-1: Frost Depth Calculations

Appendix E-2: Seismic Site Class

Appendix E-3: Settlement

Appendix E-4: Lateral Earth Pressure

Appendix E-5: Pile Design

Appendix E-6: Bearing Capacity

1.0 INTRODUCTION

This Preliminary Geotechnical Design Report (PGDR) summarizes the results of Golder Associates Inc.'s (Golder's) geotechnical investigation and recommendations for the replacement of Merrill Road Bridge #5720 over I-295 in Freeport, Maine at Exit 20 (also known as Desert Road) (see Sheet 1). This purpose of this report is to present soils and bedrock information for the bridge replacement site obtained from subsurface investigations and laboratory tests; present recommended geotechnical parameters for design and construction; and provide preliminary geotechnical designs for the bridge foundations and approach embankments. We completed our proposed scope of services in accordance with our General Consultant Agreement (GCA) dated December 28, 2016 with modifications on August 20, 2019.

2.0 PROJECT BACKGROUND

The existing Merrill Road Bridge (Desert Road) at I-295 Exit 20 was originally constructed in 1957 and the deck and substructure were widened in 1984. The bridge is a two span, non-continuous, steel girder structure with a composite deck, where each span is 75 feet in length. The bridge is supported mid-span by a multi-column concrete pier and at the ends by full height, cast-in-place, cantilever abutments and wing walls. The original pier and abutments are supported by cast in place concrete piles and the extended portions of the pier and abutments from the 1984 widening to the south are supported on H piles. The base of the pile cap for the pier is at elevation 143.0 feet (about 22 feet below the deck), the base of the west abutment is at elevation 146.0 feet (about 25 feet below grade) and the base of the east abutment is at elevation 144.5 feet (about 25 feet below grade). The bridge deck is 42.75 feet wide and the pavement surface is 39 feet wide, allowing for one eastbound travel lane and two westbound travel lanes. I-295 passes under the bridge with a 14.67-foot clearance (a 15-foot minimum clearance over I-295 is required for the replacement bridge) and there are currently two travel lanes and one shoulder in both the northbound and southbound directions of I-295. Sheet 2 illustrates the existing bridge and site topography.

We understand that MaineDOT is considering replacement of the Merrill Road Bridge with an integral abutment bridge that will increase bridge clearance over I-295 to the 15 foot minimum standard. Furthermore, proposed Abutment No .1 (west abutment) and Abutment No. 2 (east abutment) will be moved back into the existing embankment a minimum of 12 feet to allow future widening of I-295 to accommodate a potential future third travel lane. Vertical and horizontal alignment modifications are anticipated to meet current design criteria, facilitate the widening of the I-295 roadway, and allow maintenance of traffic (MOT) during construction. These alignment modifications will impact approach embankment configurations and loadings. We understand that MaineDOT has contracted with HNTB to design the bridge. Our design calculations were made using guidance provided in correspondence between MaineDOT, Golder, and HNTB regarding preliminary bridge alignments, loadings, and cross-sections[1]. Design criteria will be discussed in detail in the relevant report section below.

3.0 GEOLOGIC SETTING

3.1 Regional Surficial Geology

The proposed bridge replacement site is located in southern-central Maine within the Seaboard Lowland Section of the New England Physiographic Province.[2] Regional surficial geologic mapping indicates the surficial soils

[1] Hodgdon, Steven (HNTB). "Re: Freeport I-295 Exit 20 and Exit 22 - Request for Sections and Design Info." Message to Melissa Landon (Golder Associates). June 22, 2020. E-mail

[2] Fenneman, N.M., and Johnson, D.W., 1946. Physiographic Divisions of the Conterminous U.S., U.S. Geological Survey, 1 sheet, scale 1:7,000,000.

consist of Holocene (Recent) wetland/saltwater marsh deposits overlying Pleistocene Presumpscot Formation fine grained sediments, which overlie Pleistocene glacial till deposits. The wetland/saltwater marsh deposits consist of peat, clay, silt, and sand deposited in low-lying areas adjacent to tidal inlets, tidal channels, and tidal flats. The Presumpscot Formation consists of fine-grained marine mud (silt and clay with local sandy beds and lenses), locally with marine fossils and dropstones, deposited in deeper, quieter water during marine submergence of the coastal lowland. The till consists of a light to dark gray, heterogeneous, non-sorted to poorly sorted mixture of clay, silt, sand, pebbles, cobbles, and boulders, rarely stratified and deposited directly by glacial ice. The till consists of two varieties: a basal (or lodgment) till, fine grained and very dense; and an overlying ablation (or melt-out) till, coarser grained, stony and relatively loose.[3,4,5] Regional mapping indicates the overburden thickness ranges between 5 feet and 200 feet below ground surface in the Yarmouth-Freeport area.[6]

3.2 Regional Bedrock Geology

Regional bedrock geologic mapping indicates the bedrock beneath the site consists of the Silurian-Ordovician Vassalboro Group, undifferentiated, previously mapped as the Hutchins Corner Formation.[7,8,9] The lithology consists of light to medium gray, fine- to medium-grained, plagioclase-quartz-biotite granofels and gneiss, interlayered with subordinate amounts of greenish gray, fine-grained, calc-silicate granofels or medium gray, medium-grained biotite schist. Layer thickness ranges from 1 to 4 inches, and pegmatite lenses, boudins and sills are common. This formation is interpreted to have been initially deposited as sediments within a marine basin, which subsequently underwent diagenesis to form sedimentary rocks. This formation was then metamorphosed by heat and pressure under miles of younger rocks, forming a layered foliation, and then underwent ductile deformation by several tectonic events dating back to at least Devonian time starting with the Acadian orogeny. This compressional stress created additional foliation textures (low greenschist to upper amphibolite facies), and at least three-fold sets. This in turn was followed by post-metamorphic brittle deformation forming numerous northeast trending thrust faults and joints, with the emplacement of non-metamorphosed discordant pegmatite dikes and layer diabase dikes during the Mesozoic Era.[7,8] Within the Yarmouth-Freeport area, metamorphic compositional layering within the Vassalboro Group strikes northeast-southwest, and dips gently to the southeast.

4.0 SUBSURFACE INVESTIGATIONS

4.1 Historical Geotechnical Investigation

Historical drawings and reports provided by MaineDOT for the Merrill Road Bridge indicate that three borings were conducted in 1983: MTB-1-83 through MTB-3-83 (see Sheet 2). These borings indicate native overburden in the area generally consists of a surficial layer of very loose to medium gray-brown silty sand approximately 5 feet to 8 feet thick underlain by a layer of stiff gray-brown sandy silty clay approximately 5 feet thick, a layer of medium

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3 Retelle, M.J., 1999. Surficial Geology of the Yarmouth Quadrangle, Maine. Maine Geological Survey, Open-File No. 99-105, 1 sheet, scale 1:24,000.
4 Retelle, M.J., 1999. Surficial Geology of the Yarmouth 7.5-minute Quadrangle, Cumberland County, Maine. Maine Geological Survey Open-File 99-136, 8 p.
5 Prescott, G.C., Jr., 1977. Ground-Water Favorability and Surficial Geology of the Windham-Freeport Area, Maine. U.S. Geological Survey, Hydrologic Investigations Atlas HA-564, 1 sheet, scale 1:62,500.
6 Tolman, S.S., 2010. Overburden Thickness in the Portland 30x60-minute Quadrangle, Maine. Maine Geological Survey, Open-File No. 10-65, 1 sheet, scale 1:125,000.
7 Berry, H.N., IV, and Hussey, A.M., II, 1998. Bedrock Geology of the Portland 1:100,000 Quadrangle, Maine and New Hampshire. Maine Geological Survey, Open-File No. 98-1, 1 sheet, scale 1:100,000.
8 Hussey, A.M., II, 1985. The Bedrock Geology of the Bath and Portland 2° Map Sheets, Maine. Maine Geological Survey, Open-File No. 85-87, 82 p., 2 sheets, scale 1:250,000.
9 West, D.P., Jr. and Hussey, A.M., II, 2017. Bedrock Geology of the Yarmouth Quadrangle, Maine. Maine Geological Survey, Open-File No. 17-11, 1 sheet, scale 1:24,000.

to stiff gray sandy silty clay ranging in thickness from 1 feet to 10 feet, a medium to dense gray silty sand and gravel ranging in thickness from 1 feet to 15 feet, and bedrock. The overburden thickness is significantly greater at the existing west abutment (near proposed Abutment No. 1) compared to the east abutment (near proposed Abutment No. 2) with the overall depth to bedrock ranging from 35 feet at the existing west abutment to 15 feet at the existing east abutment.

4.2 Geotechnical Borings

Golder completed six (6) test borings (BB-FDR-101 through BB-FDR-106) between December 10 and December 19, 2019 within the eastbound lane of Desert Road at the roadway and bridge elevation (2 borings), and south of the bridge and embankments at the I-295 elevation (4 borings). The field program included Standard Penetration Testing (SPT) testing and sampling of predominantly coarse-grained materials and in-situ vane shear testing in the clay strata to measure undrained shear strength. Golder geotechnical engineers or geologists monitored drilling activities, selected sampling intervals, logged subsurface conditions encountered, and obtained soil samples and rock core for use in visual description and subsequent laboratory testing and classification. The as-drilled boring locations were surveyed by MaineDOT following completion of the drilling program. Boring location coordinates and ground surface elevations are summarized in Table 1, and boring locations with respect to existing site features are illustrated in Sheet 2.

Borings were completed by New England Boring Contractors (NEBC) of Hermon, Maine using a Mobile B-53 track-mounted rig. NEBC drilled the borings using solid-stem augers followed by the cased wash method, in which the boring was advanced by driving casing in 5-foot lengths and the soil in the casing was washed out with a roller bit and water to the depth where samples were subsequently collected or field vane shear tests were subsequently performed. All borings were advanced to the bedrock surface after which 10.0 to 10.5 feet of rock core was collected.

Standard Penetration Testing (SPT) was performed using a calibrated automatic hammer system and standard 2-inch split spoon sampler in accordance with American Society for Testing and Materials (ASTM) D1586 for all borings. Sampling was conducted at approximately 5-foot intervals for all borings. Split spoons were driven 24 inches by a 140-pound hammer dropped 30 inches, and the number of hammer blows required to advance the split spoon sampler through each 6-inch increment was recorded. Measured, uncorrected N-values, calculated as the sum of the hammer blows to advance the sampler during the 6-inch to 18-inch interval, are provided in the boring logs in Appendix A. A hammer efficiency factor of 0.914 (provided by NEBC) was used to convert the measured N-values to N_{60} values for further calculations. Soil samples were collected and stored in jars.

In-situ vane shear tests were conducted in cohesive soils using a Golder-owned Geonor rectangular vane set. For each test or pair of tests, the borehole was advanced to either 4 inches or 1 foot above the desired test depth, and the vane was then advanced the remaining distance by hand, using dedicated vane rods. Upon reaching the desired test depth, the vane was rotated with a torque wrench at a rate of approximately 90 degrees per minute to obtain the peak torque, used to estimate the peak undrained shear strength. The vane was then rotated rapidly at the same depth through 10 revolutions to remold the soil. Thereafter, the vane was again rotated with a torque wrench at a rate of approximately 90 degrees per minute to obtain the residual torque, used to estimate the residual undrained shear strength. Results from vane shear testing, including raw torque readings and calculated uncorrected undrained shear strengths, are summarized in Table 2, and are provided in the boring logs in Appendix A. Due to the stiffness of the glaciomarine clay layer, we were unsuccessful in collecting undisturbed samples during the subsurface investigation for laboratory determination of consolidation parameters.

For each boring, 10.0 feet to 10.5 feet of rock core was collected using NQ-size (1-⅞ inch inside diameter) diamond-tipped core barrels in all borings following refusal of either the casing or rollercone bit to advance. Rock core samples were placed in wooden boxes and transported to the Golder office. Total Core Recovery (TCR), calculated Rock Quality Designation (RQD), and coring rates were recorded for each core run and are provided in the boring logs in Appendix A. A detailed summary of rock quality parameters for the recovered rock core is presented in Table 3, and photographs of the rock core are presented in Appendix B.

Details of the sampling methods used, field data obtained, and soil and rock conditions encountered during the investigation are presented on the boring logs provided in Appendix A. Soils were field characterized in accordance with ASTM D2488. Bedrock lithology was field characterized and the descriptions were revised in the office. A description of the boring log symbols and terms used for the soil and rock descriptions precedes the boring logs.

5.0 LABORATORY TESTING PROGRAM

Laboratory testing on both collected soil samples and rock core was performed by GeoTesting Express (GTX) of Acton, Massachusetts in accordance with applicable American Society for Testing Materials (ASTM) and American Association of State Highway Transportation Officials (AASHTO) testing procedures. Geotechnical laboratory tests were performed on SPT split spoon soil samples representative of each soil type collected from the borings to assist in soil classification. Geotechnical rock core tests were performed on representative rock core samples from borings BB-FDR-102, -104, -105, and -106. The types and numbers of each of the laboratory tests conducted on soil samples and rock core are summarized in Table 5-1 and Table 5-2. Measured index and classification test results from soil samples are summarized in Table 4. Measured rock core properties are summarized in Table 5. Soil testing results are included on the boring logs in Appendix A. Complete laboratory testing results are provided in Appendices C (soil testing) and D (rock core testing).

Table 5-1 Laboratory Testing of Soils

Soil Laboratory Test	Test Standard	No. Tests Completed
Grainsize (sieve)	ASTM D6913 AASHTO T 88	10
Grainsize (sieve and hydrometer)	ASTM D7928 AASHTO T 88	3
Water Content	ASTM D2216 AASHTO T 265	11
Atterberg Limits with Natural Water Content	ASTM D4318 AASHTO T 89/90	8
Organic Content	ASTM D2974 AASHTO T 267	1

Table 5-2: Laboratory Testing of Rocks

Rock Core Laboratory Test	Test Standard	No. Tests Completed
Compressive Strength	ASTM D7012 Methods C and D	4

6.0 SUBSURFACE CONDITIONS

Soils encountered at the borings were found to generally include fill materials placed during construction of the bridge and roadway, naturally occurring silt and clay associated with the Presumpscot Formation, and sand and gravel interpreted as glacial till over bedrock. Detailed descriptions of the soil and bedrock conditions encountered at the borings are provided in the boring logs in Appendix A. The following descriptions summarize the major stratigraphic units from the existing ground surface to depth.

Asphalt Pavement: Asphalt pavement thicknesses observed in borings BB-FDR-103 and BB-FDR-101 was 5-inches and 8-inches thick, respectively.

Topsoil: Topsoil thickness observed in borings BB-FDR-102, BB-FDR-105, and BB-FDR-106 ranged from 0.3 feet to 2 feet.

Fill: Fill was encountered in borings BB-FDR-101, BB-FDR-102, BB-FDR-103, and BB-FDR-105. The layer was observed to be between 3.4 feet and 25.3 feet thick and start between elevation 142.8 feet and 167.5 feet. Generally, the thickness of the fill is greater behind the bridge abutments. The depth of fill ranges from 0.3 feet to 0.7 feet. The fill consisted of fine to coarse sand with little to trace gravel fractions, some to trace silt fractions and fine to coarse sandy GRAVEL with trace silt. Laboratory classifications generally described the layer as SM, SW-SM, SP, or GW-GM (USCS classification) and A-3, A-2-4, A-1-a, or A-1-b (AASHTO classification). N_{60}-values for the fill, corrected for hammer efficiency, ranged from 5 to 67, where N_{60}-values generally decreased with increasing depth within the fill. The fill layer transitions to the Presumpscot Formation silty or sandy clay and silt between 4 feet and 26 feet below ground surface (bgs).

Presumpscot Formation: Presumpscot silty or sandy clay and silt was encountered in all borings except BB-FDR-103. The layer was observed to be between 3.3 feet and 14.0 feet thick and started between elevation 139.4 feet and 145.6 feet. The Presumpscot silty or sandy clay and silt consisted of clay with gravel fractions ranging up to "trace", sand fractions ranging from "sandy" to "trace", and silt fractions ranging up to "silty", and SILT with some sand. Laboratory classifications generally described the layer as CL, CL-ML, or ML (USCS classification) and A-6 or A-4 (AASHTO classification). Field vane test (FVT) data yielded an undrained shear strength (s_u) value of 3,379 pounds per square foot (psf). Where field vane tests were not conducted, SPT N_{60}-values ranged from 0 to 18 (very soft to very stiff). The clay layer transitions to a layer of sand and gravel at 5.3 feet, 9.0 feet, and 34.0 feet bgs for BB-FDR-106, BB-FDR-104, and BB-FDR-101, respectively, and to bedrock at 16.1 feet and 18.0 feet bgs for BB-FDR-102 and BB-FDR-105, respectively.

Sand and Gravel: A layer of sand and gravel (glacial till) was encountered in borings BB-FDR-101, BB-FDR-103, BB-FDR-104, and BB-FDR-106. The layer was observed to be between 1.3 feet and 5.8 feet thick and starts between elevation 134.2 feet and 151.4 feet. The glacial till consisted of fine to coarse SAND with little to trace gravel and silt fractions ranging from "silty" to "some" and gravel with little sand and some silt/clay. Laboratory classifications generally described the layer as GC-GM or SM (USCS classification) and A-2-4 or A-4 (AASHTO classification). SPT N_{60}-values ranged from 11 to refusal. The glacial till layer transitions to bedrock at 6.7 feet, 10.3 feet, 20.8 feet, and 36.6 feet bgs for BB-FDR-106, BB-FDR-104, BB-FDR-103, and BB-FDR-101, respectively.

Bedrock: The bedrock surface was encountered in all borings and varied in elevation by approximately 31 feet across the site. The deepest bedrock surface (elevation 114.6) was encountered in historical boring MTB-3-83, which is located on the southern corner of the existing west abutment. The shallowest bedrock was encountered in the borings located in the northeastern and southwestern corners of the exploration area. Bedrock was encountered at elevation 145.6 feet in BB-FDR-103 behind the east abutment and elevation 140.9 in BB-FDR-106 beyond the southern toe of the existing northwestern embankment. In general, bedrock appears to slope down towards the low point at MTB-3-83 from each side of the site. Therefore, on the northwestern approach side, the bedrock is sloping down from southwest to northeast towards MTB-3-83; and on the southeastern approach side, the bedrock is sloping down from northeast to southwest towards MTB-3-83.

Between 10.0 feet and 10.5 feet of rock core was collected in each boring in up to 5-foot runs. The predominant bedrock lithology encountered was gray, coarse-grained, strongly foliated, fresh to slightly weathered gneiss, interpreted to be part of the Vassalboro Formation. The RQD (rock quality designation) ranged from fair (60%) to excellent (100%), and the estimated RMR (rock mass rating) ranged from 65 to 85. The four unconfined compressive strength tests conducted on rock core samples yielded unconfined compression strength values ranging from 1141 ksf to 2903 ksf. Table 3 provides detailed information about the recovery, rock quality designation (RQD), rock mass rating (RMR), and descriptions of lithology, rock mass, and discontinuities. Table 5 provides detailed information on rock core engineering properties.

Groundwater: Groundwater level measurements were measured in all borings (except for BB-FDR-102 because of ice) upon completion of the boreholes and prior to removal of the casing. Groundwater elevations measured in the borings were between 139.5 feet and 155.6 feet. Groundwater levels shown on the subsurface profile (Sheet 3) were interpreted based on these water level measurements.

7.0 GEOTECHNICAL ANALYSES AND RECOMMENDATIONS

Golder used the geotechnical data collected during the field phase of the project to develop design parameters for the major design elements of the new bridge. These parameters were based on correlations of SPT N_{60} values and measured FVT undrained shear strengths and were used for the subsequent preliminary geotechnical designs.

In a June 22, 2020 email[10] to Golder, HNTB provided two preliminary bridge alignments that accommodated three-lanes of traffic, sidewalks, and an optional pedestrian/bicycle path for both a northern and southern shift of the bridge alignment from the existing alignment. For each of these alignment shifts, HNTB provided two cross-sections for the approach embankments nearest the preliminary proposed Abutment No. 1 (at proposed Stations 60+00 and 60+25) and preliminary proposed Abutment No. 2 (at proposed Stations 62+50 and 62+75). Golder investigated each of these embankment cross-sections in conjunction with the interpreted subsurface profile (Sheet 3, parallel to the existing bridge alignment).

Our analysis found that the southern shift of the bridge alignment that included the optional pedestrian/bicycle path yielded the greatest height and extent of new embankment fill and loading. The thickest layer of glaciomarine clay, likely to govern stability and settlement, is present nearest the location of preliminary proposed Abutment

[10] Hodgdon, Steven (HNTB). "Re: Freeport I-295 Exit 20 and Exit 22 - Request for Sections and Design Info." Message to Melissa Landon (Golder Associates, Inc.). 22 June 2020. Email.

No. 1, and thus our preliminary analyses of settlement and stability were conducted using the embankment cross-section at HNTB Station 60+25 and the corresponding subsurface conditions from Sheet 3. We additionally calculated settlement and stability for the embankment cross-section at Station 62+50 nearest preliminary proposed Abutment No. 2 to evaluate possible differential settlement. The geometry used for lateral earth pressure and pile analyses was from preliminary abutment locations provided by HNTB that are near to, but not exactly at, Station 62+50 and Station 60+25.

7.1 Frost Considerations

Golder estimated an average frost penetration depth for roadway and bridge abutment and pier infrastructure fills using the MaineDOT Bridge Design Guide. The site has an air design freezing index of approximately 1,300 °F-degree days. For roadway and bridge abutment and pier coarse-grained fills with an approximate 11% water content, we estimate a frost penetration depth of 6.2 feet at the site. Refer to the depth of frost penetration calculations in Appendix E.

7.2 Seismic Site Classification

Golder analyzed the seismic site classification following the criteria in Article 3.10.3.1 of AASHTO (2020)[11] at the existing approach embankment near proposed Abutment No. 1 and proposed pier. Results concluded both locations are of Site Class D. Golder recommends a Site Class of D be used for design. Golder determined the site's Seismic Zone as per Article 3.10.6 of AASHTO (2020). The one-second period response acceleration at the site is less than 0.15g; therefore, the site is in Seismic Zone 1. Table 7-1 provides additional seismic parameters determined using AASHTO (2020) for the site. Refer to the full methodology of the analysis, calculations, and locations in Appendix E.

Table 7-1: Seismic Parameters

PGA	A_s (1/s)	S_{DS} (1/s)	S_{D1} (1/s)	Site Class	Seismic Zone
0.080	0.128	0.256	0.103	D	1

7.3 Approach Embankment Stability

Stability was evaluated for the proposed widest alignment shift to the south at Station 60+25 nearest proposed Abutment No. 1 and at Station 62+50 nearest proposed Abutment No. 2 using the two-dimensional limit equilibrium modeling software *Slide2* by Rocscience[12]. Analyses were performed for post-construction static and pseudo-static seismic load conditions transverse to the roadway centerline for both the southern and northern embankment slopes associated with the abutments and longitudinal to the face of both Abutment No. 1 and Abutment No. 2. These analyses incorporated material design parameters estimated from SPT N_{60} values. As outlined in the Maine Bridge Design Guide Section 5.9.2, minimum allowable design factors of safety (FS) of 1.3 and 1.5 for static conditions were used in assessing satisfactory transverse embankment geometries and longitudinal abutment geometries, respectively, for the given subsurface conditions. A minimum allowable design factor of safety of 1.0 was used in assessing pseudo-static seismic conditions for both the transverse

[11] American Association of State Highway and Transportation Officials (2020). "AASHTO LRFD Bridge Design Specifications," 9th edition, Washington, DC.
[12] RocScience Slide Software Package Version 9.005, build date May 6, 2020

embankments and longitudinal abutment geometries. The cross-sections were analyzed using the Spencer and Bishop simplified methods with an auto refine search for circular surfaces, along with the Spencer method and a Cuckoo search with surface altering optimization for noncircular surfaces. The noncircular surfaces were determined to govern the potential failure surfaces. Refer to the full methodology of the analysis, calculations, and locations of critical slip surfaces in Appendix E.

For all locations, we analyzed the lowest factor of safety for two cases: 1) the lowest factor of safety for a deep-seated failure surface in the native glaciomarine soil, and 2) the lowest factor of safety potential failure surface for the overall slope geometry. Results are summarized in Table 7-2 for both static conditions and pseudo-static seismic conditions for the transverse embankment geometries. Results are summarized in Table 7-3 for both static and pseudo-static seismic conditions for the longitudinal abutment geometries. Longitudinal abutment geometries did not include the proposed piles to bedrock, as pile size and spacing were unknown at the time of the analysis. As a result, this analysis is likely conservative and stability should be reevaluated in final design once the pile layout is known.

Table 7-2: Factors of Safety for Static Conditions and Pseudo-Static Seismic Conditions for the Embankments

Condition	Location	Slope	Lowest Factor of Safety[1] (Spencer Method)	
			Non-Circular Failure Surface in Fill	Non-Circular Deep-Seated Failure Surface in Glaciomarine Deposit
Static	Abutment No. 1 Embankment (Station 60+25)	North	1.27	2.36
		South	1.28	2.47
	Abutment No. 2 Embankment (Station 62+50)	North	1.28	2.33
		South	1.27	2.15
Pseudo-Static Seismic	Abutment No. 1 Embankment (Station 60+25)	North	1.09	2.00
		South	1.10	2.11
	Abutment No. 2 Embankment (Station 62+50)	North	1.10	1.98
		South	1.09	1.84

1. Minimum factors of safety are 1.3 for static analyses and 1.0 for pseudo-static seismic analyses.

For the transverse embankment geometries, deep seated potential failure surfaces in the native glaciomarine soil yield a lower bound FS of 2.15 under static conditions and 1.84 under pseudo-static seismic conditions. These factors of safety meet required design FS values. For the transverse embankment geometries, overall potential failure surfaces that are limited to the existing and new fills yield a lower bound FS of 1.27 under static conditions and 1.09 under pseudo-static seismic conditions. For this scenario, the FS under static conditions does not meet the required value of 1.3, however does meet the required value of 1.0 for pseudo-static seismic conditions.

The potential failure surface that does not meet FS ≥ 1.3 under static conditions is shallow and contained entirely within the new embankment fill indicating potential surficial sloughing failure for the transverse embankment. The embankment fill modeled was standard granular borrow with material properties of 125 pounds per cubic foot (pcf) unit weight and 32° friction angle as provided in the MaineDOT Bridge Design Guide[13]. The model geometry is also based on HNTB's specification of a 2 horizontal to 1 vertical (2H:1V) slope along the entire embankment, which does not account for shallower slopes that may be constructed as per MaineDOT Standard Specifications. While the modeled embankment slope angle of 26.5° is less than angle of internal friction of the embankment fill material (32°), the fill angle of internal friction is not great enough to result in a FS ≥ 1.3. The factor of safety for these potential surficial failure areas can likely be increased if the slope has an established protective vegetation or riprap layer, by increasing the required compactive effort for placed fill, decreasing the embankment slope angle, or using an embankment fill material with greater frictional resistance (i.e., angle of internal friction). Based on discussions with MaineDOT, we understand that slopes constructed with material and methods commensurate with MaineDOT Standard Specifications have historically performed adequately.

Table 7-3: Factors of Safety for Static Conditions and Pseudo-Static Seismic Conditions for the Longitudinal Section Through the Abutments (not considering piles).

Condition	Location	Lowest Factor of Safety[1] (Spencer Method)	
		Non-Circular Failure Surface in Fill	Non-Circular Deep Seated Failure Surface in Glaciomarine Deposit
Static	Abutment No. 1	1.29	2.15
	Abutment No. 2	1.31	1.95
Pseudo-Static Seismic	Abutment No. 1	1.15	1.87
	Abutment No. 2	1.17	1.72

1. Minimum factors of safety are 1.5 for static analyses and 1.0 for pseudo-static seismic analyses.

For the longitudinal abutment geometries, deep seated potential failure surfaces in the native glaciomarine soil yield a lower bound FS of 1.95 under static conditions and 1.72 under pseudo-static seismic conditions. These factors of safety meet required design FS values (> 1.5). For the longitudinal abutment geometries, overall potential failure surfaces that are limited to the existing and new fills yield a lower bound FS of 1.29 under static conditions and 1.15 under pseudo-static seismic conditions. For this scenario, the FS under static conditions does not meet the required FS ≥ 1.5, however does meet the required FS > 1.0 for pseudo-static conditions. Failure surfaces for this scenario pass below the abutment wall and are not considered acceptable. Additional analyses

[13] Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.

will need to be conducted in final design. Additional analyses may include evaluating the impact of the abutment piles in improving the overall stability.

7.4 Settlement

Golder analyzed settlement for the proposed widest alignment shift to the south at Station 60+25 (near proposed Abutment No. 1) and Station 62+50 (near proposed Abutment No. 2) for a point directly below the proposed guardrail along the edge of the eastbound traffic lane of the proposed embankment, as well as near the toe of the existing embankment where a force water main is presently located. The loads imposed by the proposed embankment fill were distributed through the subsurface soils using Boussinesq stress distribution theory for the given embankment geometry. The subsurface soils were then discretized into layers less than 10 feet thick, and the effective stress, imposed stresses, and other soil parameters were used to evaluate settlement within each layer using one-dimensional consolidation theory for cohesive soil and the Hough method and correlations to N_{60} for cohesionless soil. An estimate of the time required for the cohesive subsurface soils to reach 95% consolidation under the imposed loads was also evaluated for each location based on an analysis of the material as a single layer using Terzaghi's theory. Due to the stiffness of the glaciomarine clay layer, we were unsuccessful in collecting undisturbed samples during the subsurface investigation for laboratory determination of consolidation parameters. Thus, Golder used its knowledge of Presumpscot Formation glaciomarine soil properties from southern coastal Maine and engineering judgement to estimate compressibility and coefficient of consolidation parameters used in the analysis. Refer to the full methodology of the analysis, material properties, and calculations in Appendix E.

7.4.1 Approach Embankments

We evaluated settlement for the proposed widest alignment shift to the south at both Station 60+25 and Station 62+50 for a point directly below the proposed guardrail along the edge of the eastbound traffic lane of the proposed embankment. The total, immediate, and consolidation settlement values, as well as the estimated time for 95% consolidation settlement to occur for the two locations are presented in Table 7-4.

Table 7-4: Estimated Settlement and Duration for Analyzed Bridge Embankment Locations

Estimated Settlement Conditions	Proposed Southern Shift Station 60+25	Proposed Southern Shift Station 62+50
Total Settlement (inches)	1.4	1.3
Immediate Settlement (inches)	0.7	0.6
Consolidation Settlement (inches)	0.7	0.7
Time for 95% Consolidation Settlement (days)	97	32

7.4.2 Water Force Main

We also evaluated settlement for the proposed widest alignment shift to the south at both Station 60+25 and Station 62+50 for a point directly below the location of the force water main near the toe of the existing embankment slope (see Sheet 2). Only the location of the water main was provided on the base mapping provided by HNTB. We used Maine Water Company[14] references to provide minimum specifications on water main dimensions, trench depth, and properties of materials used above and below the pipe to create a stratigraphy for estimating settlement in the stratigraphic profile. The total, immediate, and consolidation settlement values, as well as the estimated time for 95% consolidation settlement to occur for the two locations are presented in Table 7-5.

Table 7-5: Estimated Settlement and Duration for Analyzed Water Maine Locations

Estimated Settlement *Conditions*	**Proposed Southern Shift Station 60+25**	**Proposed Southern Shift Station 62+50**
Total Settlement (inches)	< 0.2	< 0.1
Immediate Settlement (inches)	< 0.1	< 0.1
Consolidation Settlement (inches)	0.1	not applicable
Time for 95% Consolidation Settlement (days)	10	not applicable

7.5 Proposed Bridge Abutment Lateral Earth Pressure

HNTB provided Golder with approximate abutment dimensions in an email dated June 22, 2020. Abutment dimensions will be 92 feet in width perpendicular to the roadway alignment centerline, have an average height of 12 feet, and have an average thickness of 3.0 to 3.5 feet. Golder used these dimensions to analyze lateral earth pressure.

Both integral abutments should be designed to resist lateral earth pressures along the entire 12-foot-high abutment faces. Under longitudinal expansion, the abutments will be subject to passive earth pressure. Under longitudinal contraction, the abutments will be subject to active earth pressure. Per AASHTO (2020) Table C3.11.1-1, MassDOT (2020) LRFD Bridge Manual[15] Figure 3.10.8-1, the abutment height provided, and the anticipated maximum thermal expansion of 0.7 inches, we determined that the maximum wall rotation of 0.005 is less than 0.02 that MassDOT (2020) specifies is required to develop full passive earth pressure. Thus, the MassDOT (2020) LRFD Bridge Manual was used to determine the passive earth pressure coefficient, which is less than the maximum passive earth pressure coefficient recommended in AASHTO (2020).The Rankine earth pressure coefficient was used to determine active lateral earth pressure (assuming level backfill and no frictional interaction between the abutment wall and the backfill). Analysis and calculated parameters are presented in Table 7-6.

[14] Maine Water Company (2017). "Water Main Installation Instructions," https://www.mainewater.com/media/1260/water-main-installation-instructions_2017-01-09.pdf

[15] MassDOT LRFD Bridge Manual - Part 1, January 2020 Revision (https://www.mass.gov/doc/chapter-3-lrfd-bridge-design-guidelines/download)

Table 7-6: Lateral Earth Pressure Coefficients

Abutment Earth Pressure Parameters	Value
Granular Backfill Unit Weight, γ (pcf)	125
Granular Backfill Friction Angle, ϕ (˚)	32
Passive Earth Pressure Coefficient, K_p	3.93
Active Earth Pressure Coefficient, K_a	0.31

Golder estimates that the resultant force generated from passive earth pressure, the governing scenario, is an unfactored load of 35,370 pounds per foot of abutment length and acts at elevations 162.0 feet at Abutment No. 1 and 161.0 feet at Abutment No. 2. These values assume the fill is free draining (i.e., no water pressure is allowed to build up behind the abutment walls). Refer to the full methodology of the analysis, material properties, and calculations in Appendix E.

7.6 Proposed Bridge Pile Foundations

The new bridge abutments and pier are proposed to be founded on piles driven to the bedrock surface. To provide recommendations on pile design, we analyzed the pile design loads, downdrag, axial resistance, axial and lateral loads, and driveability, as described in the subsections below.

7.6.1 Pile Design Load

Golder received the following design loading and deflection guidance from HNTB in an email dated June 22, 2020.

- Typical [axial] factored pile loads for the abutments and pier will be governed by the Strength I case and are expected to be between 350 kips and 450 kips depending on pile spacing.
- Service loads [for the abutments] are expected to be between 250 kips and 300 kips per pile depending on pile spacing.
- Downdrag loads were not assumed in these values.
- Maximum thermal movements are estimated to be 0.7 inches in either direction at the abutments.
- Pier piles should be proportioned to have similar design loads as abutments; however, axial loads that are 20% greater may be considered if this is more cost-effective.

Golder analyzed HP 14x89 piles at the request of MaineDOT and HNTB for an abutment width of 92 feet. HP 12x74 piles were also analyzed at Abutment No. 2 only based on discussions with MaineDOT and HNTB. Based on our review of the interpreted subsurface profile and preliminary plans, Abutment No. 1 can be supported by piles approximately 34.0 feet in length, the pier can be supported by piles approximately 16.0 feet in length, and Abutment No. 2 can be supported on piles approximately 16.3 feet in length. Our analyses were performed assuming average interpreted depth to bedrock as shown on the subsurface profile for each abutment and pier location. However, as discussed in Section 6.0, the borings show that bedrock elevation varies at the site both parallel and transverse to the roadway alignment. Over the width of the abutments and pier, there may be bedrock elevation differences of between approximately 5 feet (near pier) and approximately 10 to 15 feet (near abutments).

The following sections describe the downdrag analyses completed for abutment piles expected to be influenced by embankment settlement, axial resistance analyses, lateral response of the abutment piles due to thermal movement of the superstructure (no lateral loading was provided for the pier piles), and driveability analyses for the abutment and pier piles.

7.6.2 Downdrag on Piles

In accordance with AASHTO LRFD Article 3.11.8, downdrag loads on piles can be assumed to be fully developed when settlement at the soils surrounding the piles is 0.4 inches or greater. The settlement analysis at Abutment No. 1 and Abutment No. 2 indicates downdrag loads will be imposed on the piles along the existing fill and glaciomarine clay soil layers at the abutment locations. While settlements at the abutment pile locations are expected to vary along the length of the abutments, Golder calculated a maximum value as described in Section 7.3 and used this maximum value for design. Settlement is not expected at the pier location because of the lack of embankment fills, and thus downdrag does not apply for that location.

Downdrag loads were calculated in accordance with methods described in AASHTO LRFD Article 10.7.3.7. The factored maximum downdrag loads at Abutment No. 1 for an HP 14x89 pile are 148 kips and 112 kips per pile for the Strength I and Service I load cases, respectively. The factored maximum downdrag loads at Abutment No. 2 for an HP 14x89 pile are 46 kips and 36 kips per pile for the Strength I and Service I load cases, respectively. The factored maximum downdrag loads at Abutment No. 2 for an HP 12x72 pile are 39 kips and 30 kips per pile for the Strength I and Service I load cases, respectively. Downdrag load factors of 1.40 (Strength I) and 1.00 (Service I) were used for the glaciomarine soils based on the alpha Tomlinson Method in accordance with AASHTO LRFD Table 3.4.1-2. Downdrag load factors of 1.10 (Strength I) and 1.00 (Service I) were used for the existing fill soils based on the Oregon DOT Geotechnical Design Manual[16].

7.6.3 Axial Pile Resistance

Golder analyzed the nominal structural and geotechnical pile resistance of HP 14x89 piles at Abutment No. 1 and No. 2 and the pier and HP 12 x74 piles at Abutment No. 2 only, following the design procedures outlined in the MaineDOT Bridge Design Guide (2018) and AASHTO (2020). Since the piles will be driven to hard rock, the nominal resistance of the piles will be controlled by the structural limit state in accordance with AASHTO Article 10.7.3.2.3. These calculations are provided in Appendix E.

The factored pile structural resistance, P_r, was calculated for the piles using the results of the LPILE analysis outlined in Section 7.6.4 and resistance factors of $\phi_c = 0.70$ for combined axial and bending loading and $\phi_c = 0.50$ for axial compression in the lower segment of the pile based on the potential for hard driving conditions.

As outlined in AASHTO Article 10.7.3.2.3, the nominal axial geotechnical resistance of piles driven to point bearing on hard rock should not exceed the nominal structural resistance values obtained from AASHTO Article 6.9.4.1 with a resistance factor ϕ_c, of 0.50, for severe driving conditions applied. As such, the controlling geotechnical pile resistance is equal to the structural resistance.

Drivability analyses were performed to determine the pile resistance that might be achieved at Abutments No. 1 and No. 2 and the pier considering available diesel hammers. Nominal drivability resistances were determined based on a maximum driving stress of 45 ksi and a limiting driving criterion of 12 bpi (due to a slight risk of piles

[16] Oregon Department of Transportation, Geo-Environmental Section. Geotechnical Design Manual: Chapter 8 – Foundations, Version 2.1. Dated May 6, 2019.

walking out of position before reaching the maximum allowable driving criterion of 15 bpi). The drivability resistances were calculated using the resistance factor, ϕ_{dyn}, of 0.65, for a single pile in axial compression when dynamic testing is performed as specified in AASHTO Table 10.5.5.2.3-1. Drivability controls and the recommended governing resistances for pile design are the resistances provided in the right column "Governing Axial Pile Resistance (kips)" in Table 7-7.

The maximum applied factored axial pile loads should not exceed the governing factored axial pile resistances shown in Table 7-7.

Table 7-7: Summary of Strength Limit State Factored Axial Pile Resistance

Abutment	Pile Section	Structural Resistance $\phi_c = 0.50$ (kips)	Controlling Geotechnical Resistance $\phi_c = 0.50$ (kips)	Drivability Resistance[1,2] $\phi_{dyn} = 0.65$ (kips)	Governing Axial Pile Resistance (kips)
Abutment No. 1	HP 14x89	653	653	390	390
Abutment No. 2	HP 14x89	653	653	455	455
Abutment No. 2	HP 12x74	545	545	325	325
Pier	HP 14x89	653	653	450	450

1. Factored axial load required to limit blow counts of between 3 and 12 blows per inch while limiting the driving stresses to below 45 ksi. 12 blows per inch was used due to the risk of piles walking as a result of sloping bedrock.
2. Drivability resistance based on a Delmag D30 hammer. Refer to Appendix E for fuel setting recommendations.

7.6.4 Lateral Pile Response

Lateral response of the abutment piles was evaluated using LPile[17] analysis software. The input parameters were developed based on layer response models defined by the software, laboratory test results, correlations to soil properties determined from the field investigations, correlations to soil properties identified in the FB-MultiPier user manual[18], and standard properties provided in the MaineDOT Bridge Design Guide. The input parameters are summarized in Table 6 for Abutment No. 1, Table 7 for Abutment No. 2, and in Appendix E. The piles were modeled for lateral response in the weak axis assuming factored pile loads, 0.7 inches of lateral movement from thermal bridge expansion provided by HNTB and MaineDOT- and HNTB-requested pile sizes of HP 14x89 and HP 12x74 (Abutment No. 2 only). This assumes an applied axial load of 522 kips for Abutment No. 1 and 496 or 436 kips for Abutment No. 2 with HP 14x89 or HP 12x74 piles, respectively. The pile head to abutment connection was assumed to be fixed.

[17] Ensoft Inc. (2019). LPile, version 2019.11.03.

[18] Bridge Software Institute. "FB-MultiPier Soil Parameter Table (US Customary Units)." Accessed on 4/10/2020. <https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf>

For Abutment No. 1, Golder analyzed a pile length of 34.0 feet. For the HP 14x89 piles, the analysis indicates a maximum moment of 200 kip-feet in the piles occurs at the top of the pile under the Strength I load case and maximum lateral deflection of the pile cap of 0.7 inches with no lateral deflection at the pile tip.

For Abutment No. 2, Golder analyzed a pile length of 16.3 feet using both an HP 14x89 pile and an HP 12x74 pile. For the HP 14x89 piles, the analysis indicates a maximum moment of 200 kip-feet occurs at the top of the pile under the Strength I load case and maximum lateral deflection of the pile cap of 0.7 inches and <0.1 inches of lateral movement occurring at the pile tip. For the HP 12x74 piles, the analysis indicates a maximum moment of 139 kip-feet occurs at the top of the pile under the Strength I load case and maximum lateral deflection of the pile cap of 0.7 inches and < 0.1 inches of lateral movement occurring at the pile tip. For both pile sections, the ratio of the shear and axial forces acting at the pile tip was compared to the factored friction coefficient at the bedrock/pile interface in order to verify the assumption of a pinned support at the base of the pile. Results of this analysis indicated the chosen pile section can be considered pinned at the bottom of the pile. This analysis should be revisited during final design when final loads are known. Results of the LPile first iteration analyses are summarized in Table 7-8.

Table 7-8: LPile Lateral Analysis Results - First Iteration

Location	Axial Load Analyzed (kips)	Lateral Thermal Deflection (in)	LPILE Moment at Pile Head (in-kips)	Plastic Hinge Moment (in-kips)	Plastic Hinge Forms
Abutment No. 1 (HP 14x89)	522	0.7	2,396	1,433	Yes
Abutment No. 2 (HP 14x89)	496	0.7	2,395	1,533	Yes
Abutment No. 2 (HP 12x74)	436	0.7	1,669	1,045	Yes

Since a plastic hinge forms at both locations, we assume that the pile head will enter plastic deformation. We then performed a second iteration using the same displacement and axial load conditions as the first iteration, but set the pile head moment equal to the plastic hinge moment predicted in the first iteration. Results of the LPile second iteration analyses are summarized in Table 7-9 and indicate that the demand ratio for combined bending is less than 1.0 in Segment 2, and therefore also shows that with the exception of the plastic hinge location, the pile remains within the elastic range over the remainder of its length and is stable against buckling. The results also indicate the nominal structural resistance in Segment 1 is still sufficient to support the loads analyzed.

Table 7-9: Pile Lateral Analysis Results - Second Iteration

Location	Axial Load Analyzed (kips)	Lateral Thermal Deflection (in)	Plastic Hinge Moment at Pile Head (in-kips)	Structural Resistance Demand Ratio, Segment 1	Combined Bending Demand Ratio, Segment 2
Abutment No. 1 (HP 14x89)	522	0.7	1,433	0.60	0.91
Abutment No. 2 (HP 14x89)	496	0.7	1,533	0.57	0.86
Abutment No. 2 (HP 12x74)	436	0.7	1,045	0.60	0.96

Lateral loading was not evaluated at the pier during our preliminary analysis as the foundation size and pile layout had not yet been established.

7.6.5 Pile Analysis Summary and Recommendations

We recommend the abutment piles be driven to, and seated on, bedrock and oriented with the weak axis parallel to the centerline of bearing. The pier piles should be driven to, and seated on, bedrock and oriented with the strong axis parallel to the centerline of bearing. Drivability controls and should be the recommended governing resistances for pile design. The recommended maximum factored loads for design of HP 14X89 piles at Abutments No. 1 and No. 2 and the pier and HP 12x 74 piles at Abutment No. 2 including estimated downdrag loads, pile lengths, and pile orientation are provided in Table 7-10.

Table 7-10: Pile Analysis Summary.

Location	Recommended Nominal Pile Resistance (kips)	Recommended Max. Factored Axial Load Per Pile (kips) [1]	Estimated Factored Downdrag Load (kips)[2]	Estimated Pile Length (feet)	Pile Orientation
Abutment No. 1 (HP 14x89)	600	390	148	34.0 (driven to bedrock)	Weak-axis
Abutment No. 2 (HP 14x89)	700	455	46	16.3 (driven to bedrock)	Weak-axis
Abutment No. 2 (HP 12x74)	500	325	39	16.3 (driven to bedrock)	Weak-axis
Pier (HP 14x89)	692	450	N/A	16.0 (driven to bedrock)	Strong-axis

1. Using a resistance factor of 0.65 for driving criteria established by dynamic testing of at least two piles per site condition, but no less than 2% of the production piles
2. Downdrag loads factored for Strength I.

8.0 CONSTRUCTION CONSIDERATIONS

All areas proposed for embankment fill placement or footing construction should be cleared, grubbed, and stripped of existing vegetation, pavement, and topsoil. During the grubbing and stripping process, unsuitable materials exposed at the subgrade level, such as wood, logs, tree stumps, organic silt, peat, soft clay, debris fill, or other materials that may compress, decay or collapse should be removed. Subgrade surfaces for embankments should be prepared in accordance with MaineDOT Standard Specifications and Standard Details.

Structural Fill materials and placement methods for abutment construction should meet the requirements of MaineDOT Standard Specifications.

If wet subgrade conditions are encountered in abutment and embankment fill areas, wet and disturbed subgrade material should be excavated and replaced with an appropriate size stone and covered with a geotextile (both as per MaineDOT Standard Specifications) to allow proper compaction of overlying fills. If seepages persist over a broad subgrade area, provisions should be made to allow for positive drainage beneath and within the new abutments and embankment fills. Positive drainage could be provided by a column of crushed stone wrapped in filter fabric that daylights beyond the new toe of slope.

The estimated pile lengths provided in this report were based on an average interpreted depth to bedrock at the abutment and pier locations. However, borings show that bedrock elevation varies at the site both parallel and transverse to the roadway alignment. Over the width of the abutments and pier, there may be bedrock elevation differences of between approximately 5 feet (near pier) and approximately 10 to 15 feet (near abutments) that should be anticipated. We recommend a prefabricated pile tip be used in accordance with MaineDOT Standard Specification 501.048 to account for the sloping bedrock and that the number of pile driving blow counts be limited to 12 blows per inch.

To establish the actual pile lengths needed to develop the required axial resistance of the driven piles, we recommend implementing a field-verification program consisting of a wave equation analysis and dynamic testing with signal matching including the following:

- Prior to the beginning of pile driving, a wave equation analysis should be conducted on the contractor’s proposed driving system to ensure the hammer is capable of driving the piles to the required capacities without overstressing the piles to the required penetration depths and within a reasonable number of hammer blow counts, typically 3 to 12 blows per inch at end of driving (EOD). If more than one pile section is used at the site, each different pile section should have its own separate wave equation analysis completed.
- Dynamic testing in accordance with MaineDOT specified procedures should be used to establish the driving criteria at the beginning of production pile driving. Two percent of the production piles or a minimum of one pile per substructure shall be subject to dynamic testing. We recommend the first production pile for each structure be tested. Dynamic testing and field inspection should include verification of hammer stroke or bounce chamber pressures and hammer blows throughout the pile driving operations.
- We recommend that MaineDOT's typical refusal criteria of 10 blows per 0.5 inches be implemented to reduce structural damage to the piles. The Engineer should review all dynamic pile testing results before the piles are accepted.
- While piles will be driven to the bedrock surface, there is some concern for sloping bedrock. We recommend that the re-strike criteria for substructures where relaxation effects may occur as described in MaineDOT

Standard Specifications be met. We recommend the restrike test is performed on the first production pile installed on sloping rock to evaluate the potential to waive additional restrike tests or establish a testing program to mitigate possible risk of piles "walking" along the bedrock surface.

- Signal matching analysis of the dynamic test data using methods described by Rausche et al. (1972)[19] should be conducted to determine pile bearing resistance.

9.0 RECOMMENDATIONS FOR FINAL DESIGN

The analyses performed were for the bridge alignment shift to the south. This allowed Golder to use the stratigraphy determined using the borings from the westbound lane of the existing bridge and the borings located south of the bridge in the analyses. This stratigraphy showed sloping bedrock (southwest to northeast near the Abutment No. 1 location, northeast to southwest near the Abutment No. 2 and pier locations). Due to the sloping bedrock across the site, we recommend that additional geotechnical investigations be performed during final design to better define the bedrock elevations at the proposed abutment and pier locations. Our analysis assumed pile lengths ranging from 16 to 34 feet. If bedrock is encountered shallower than anticipated, an alternative foundation design may need to be considered. We believe that a series of bedrock probes at the proposed foundation locations would provide the most reliable bedrock elevation data. However, geophysical methods may be considered as an alternate method for delineation of the bedrock surface.

Our analyses were performed based on preliminary proposed roadway alignments and grades, as well as estimates of loads and geometries for abutments and foundations. The analyses discussed in Section 7.0 should be revisited when alignments, grades, bridge widths, abutment and pier loads, abutment deflections, and embankment, abutment, and pier geometries of the proposed bridge are known to identify if preliminary analyses are sufficient or if they need to be reanalyzed for the updated conditions. Based on the results of our preliminary geotechnical analyses, the following analyses and conditions may need to be considered for final design:

- Pier foundation analyses should be updated to include anticipated design pile spacing, axial and lateral loads, and footing and pile cap geometry and dimensions. This includes reevaluation of axial resistance, lateral resistance, and driveability. Based on this loading, final pile spacing and footing dimensions should be established for the pier. Once the loads and geometries are further refined, a lateral load analysis should be conducted to verify the lateral movements are acceptable and that combined stresses are within tolerable limits.

- Abutment pile analyses should be updated to include anticipated design axial and lateral loads and pile cap geometries. This includes reevaluation of axial resistance, lateral resistance, and driveability.

  - If the down drag loads are deemed excessive by the design team, options to provide an adequate pile design related to settlement and downdrag at this location may include the following:

    1) Reduce the load per pile by increasing the number of piles to accommodate the calculated downdrag loading.

    2) Pre-load the abutment areas to limit the amount of post-pile installation settlement and downdrag induced loading on the piles.

[19] Rausche, F., F. Moses, and G. G. Goble. 1972. "Soil Resistance Predictions from Pile Dynamics," *Journal of the Soil Mechanics and Foundation Division*. American Society of Civil Engineers, Reston, VA, Vol. 98, No. SM9, pp. 917–937.

3) Choose a pile with a greater capacity.

4) Provide a sleeve around the pile or coating on the pile to limit skin friction and reduce downdrag loads.

5) Obtain additional geotechnical data in the glaciomarine clay later to better define the consolidation parameters and clay thickness assumed in the analysis.

- Our preliminary findings at Abutment No. 2 indicate that the ratio of the shear and axial forces acting at the pile tip compared to the factored friction resistance at the bedrock/pile interface supports an assumption of a pinned support at the base of the pile. However, this analysis should be revisited during final design when final loads are known, bedrock elevations have been verified and the pile size has been chosen.
- Our analysis is for maximum driving blow counts of 12 bpi because of a slight risk of piles walking out of position before reaching the maximum allowable driving criterion of 15 bpi. This limits the axial capacity of the piles at the abutments. We recommend future analyses better evaluate "walking" risk and use of the maximum allowable driving criterion of 15 bpi.

- Perform additional longitudinal slope stability analyses at the abutments to evaluate the impact of the abutment piles in improving the overall stability. Additional design alternatives may be required to achieve a factor of safety greater than or equal to 1.5.
- Evaluate design alternatives that produce a slope stability factor of safety of greater than or equal to 1.3 for embankment fills by incorporating one or more of the stabilization methods outlined in Section 7.2.
- Stability and settlement analyses should be updated based on the updated alignment location and width of the roadway that incorporates the designed number of lanes, median width, numbers and width of sidewalks and pedestrian/cycling paths. While we performed our analyses for the proposed alignment and widest roadway (and hence largest embankment fills) with the aim to capture the most heavily loaded new scenario, settlement and stability may change once the final geometry and location are analyzed.
- Impacts to the existing bridge structure and approach embankments due to construction of the new bridge. We recommend the potential effects of stress relief from excavation and construction of new abutments and embankments adjacent to the existing bridge be investigated to determine if countermeasures to mitigate possible insufficient stability are needed.

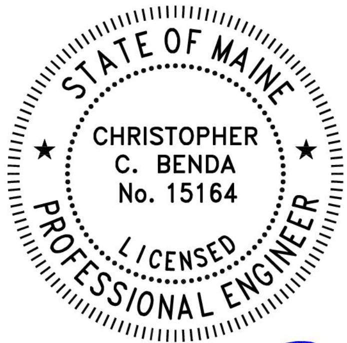
10.0 REPORT AND EXPLORATION LIMITATIONS

This Preliminary Geotechnical Design Report was prepared for the exclusive use of MaineDOT and HNTB for specific application to the proposed bridge replacement at I-295 Exit 20 in Freeport, Maine. We conducted our evaluations and compiled our recommendations in accordance with generally accepted soil and foundation engineering practices in this geographical area and under similar time and financial constraints. Golder makes no other warranty, either express or implied. If changes in the nature, design, or location of the proposed project are planned, Golder should be notified to review the appropriateness of our conclusions and recommendations, and to modify the recommendations as appropriate to reflect the changes in design. In addition, Golder should review the final plans and specifications to evaluate compliance with these recommendations.

Our analyses and recommendations are based, in part, on information obtained from the referenced subsurface explorations completed at the discrete locations described in the report. Variations in the nature and extent of subsurface conditions between explorations should be expected. Golder should be notified if conditions encountered during construction vary from those described in this report so that we may re-evaluate, and if necessary, revise the recommendations made in this report.

The professional services provided by Golder for this project include only the geotechnical aspects of the subsurface conditions at this site. The presence or implications of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this report and have not been investigated or addressed.

Signature Page



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https://golderassociates.sharepoint.com/sites/111584/project files/6 deliverables/final report/freeport merrill rd bridge final pgdr 19126013 2020dec17 v4.docx

Table 1: Subsurface Exploration Locations
Preliminary Geotechnical Design Report
I-295 Merrill Road Bridge Replacement #5720 (Exit 20)
Freeport, Maine
MaineDOT WIN 023627.00

Test Boring Designation[1]	As-Drilled Locations[2,3]		Existing Ground Surface Elevation[3] (ft)	Boring Depth[4] (ft)	Comments[4,5]
	Northing (ft)	Easting (ft)			
BB-FDR-101	368277.8	1051175.0	168.2	46.9	Bedrock at 131.6 ft elevation (36.6 ft bgs)
BB-FDR-102	368209.0	1051295.0	145.7	26.1	Bedrock at 129.6 ft elevation (16.1 ft bgs)
BB-FDR-103	368148.1	1051405.4	166.4	30.8	Bedrock at 145.6 ft elevation (20.8 ft bgs)
BB-FDR-104	368053.2	1051377.0	144.4	20.5	Bedrock at 134.1 ft elevation (10.3 ft bgs)
BB-FDR-105	368127.6	1051255.7	143.4	28.5	Bedrock at 125.4 ft elevation (18.0 ft bgs)
BB-FDR-106	368185.1	1051146.0	147.6	16.8	Bedrock at 140.9 ft elevation (6.7 ft bgs)

Notes:
1. Borings BB-FDR-101 through BB-FDR-106 were performed by New England Boring Contractors from December 10 to 19, 2019.
2. Test boring locations are shown in Sheet 2 entitled "Boring Location Plan".
3. As-drilled locations and elevations are derived from survey files "Freeport 23627.00 Boring Data 2020-01-06.zip" received by Golder on January 6, 2019 from MaineDOT.
4. Boring logs presented in Appendix A.
5. ft = feet
6. bgs = below ground surface

Prepared By: SKB
Checked By: KAR
Reviewed By: JRS

Table 2: Summary of In Situ Vane Shear Testing Results
Preliminary Geotechnical Design Report
I-295 Merrill Road Bridge Replacement #5720 (Exit 20)
Freeport, Maine
MaineDOT WIN 023627.00

Test Boring Designation[1]	Ground Surface Elevation[2] (ft)	Test Number	Test Depth Below Ground Surface (ft)	Approximate Test Elevation (ft)	Vane Size[3] (mm x mm)	Undisturbed[4]		Remolded[4]		S_t[5]	Comments
						Vane Reading Torque	s_u from Vane Reading (psf)	Vane Reading Torque	s_u from Vane Reading (psf)		
BB-FDR-104	144.4	V1	3.7 - 3.8	140.7 - 140.6	16 x 32 mm	21.5 in-lbs	3379	12.5 in-lbs	1965	1.72	
BB-FDR-104	144.4	MV	5.7 - 5.8	138.7 - 138.6	16 x 32 mm	- -	- -	- -	- -	- -	Could not push down into borehole
BB-FDR-105	143.4	MV	5.6 - 6.0	137.8 - 137.4	55 x 110 mm	- -	- -	- -	- -	- -	Could not push down into borehole
BB-FDR-105	143.4	MV	9.6 - 10.0	133.8 - 133.4	55 x 110 mm	- -	- -	- -	- -	- -	Could not push down into borehole
BB-FDR-105	143.4	MV	11.6 - 12.0	131.8 - 131.4	55 x 110 mm	- -	- -	- -	- -	- -	Could not push down into borehole

Notes:
1. Test boring locations are shown in Sheet 2 entitled "Boring Location Plan".
2. As-drilled elevations are derived from survey files "Freeport 23627.00 Boring Data 2020-01-06.zip" received by Golder on January 6, 2019 from MaineDOT. ft = feet
3. Geonor Rectangular Vane, mm = millimeters
4. s_u and $s_{u(remolded)}$ are the calculated peak and remolded undrained shear strengths. Conversion from torque readings to shear strength based on data from Geonor. For the 55 x 110 mm vane, 1 ft-lb = 44.65 psf. For the 16 x 32 mm vane, 1 in-lbs = 157.18 psf.
5. Sensitivity, S_t, is calculated as $s_u/s_{u(remolded)}$
6. Vane shear test results are shown on the boring logs in Appendix A.
7. psf = pounds per square foot, in-lbs = inch-pounds.
8. - - = no data
9. MV = missed vane

Prepared By: SKB
Checked By: KAR
Reviewed By: JRS

Table 3: Summary of Rock Core Quality
Preliminary Geotechnical Design Report
I-295 Merrill Road Bridge Replacement #5720 (Exit 20)
Freeport, Maine
MaineDOT WIN 023627.00

Test Boring Designation	Core Size (in)	Existing Ground Surface Elevation[1] (ft)	Run No.	Run Midpoint Depth Below Bedrock Surface (ft)	Run Depth Below Ground Surface (ft) Start	Run Depth Below Ground Surface (ft) End	Run Depth Below Ground Surface (ft) Midpoint	Run Length (ft)	TCR[2] Length (ft)	TCR[2] %	RQD[3] Length (ft)	RQD[3] %	RQD[3] Designation	Physical Rock Parameters Weathering[4]	Physical Rock Parameters Estimated Field Strength[4]	Physical Rock Parameters Rock Mass Rating [RMR][5]	Lithologic, Rock Mass and Discontinuity Description[6]
BB-FDR-101	NQ (1.875)	168.2	R1	2.5	36.6	41.6	39.1	5.0	3.6	72%	3.0	60%	Fair	Fresh (W1)	Very Strong (R5)	71	36.6-41.6 ft: Grey, coarse grained, strongly foliated, fresh (W1), very strong (R5), GNEISS; discontinuities steep (55°-75°) and parallel to foliation, closely spaced (0.2-0.8 ft) [VASSALBORO FORMATION].
			R2	7.2	41.6	45.9	43.8	4.3	4.9	114%[7]	4.8	100%	Excellent	Fresh (W1)	Very Strong (R5)	76	41.6-45.9 ft: Blue/grey, coarse grained, strongly foliated, fresh (W1), very strong (R5), GNEISS-with PEGMATITE veins; discontinuities steep (70°-80°) and parallel to foliation, very closely to moderately closely spaced (0.1-1.5 ft) [VASSALBORO FORMATION].
			R3	9.8	45.9	46.9	46.4	1.0	1.2	117%[7]	1.2	100%	Excellent	Fresh (W1)	Very Strong (R5)	77	45.9-46.9 ft: Blue/grey, coarse grained, strongly foliated, fresh (W1), very strong (R5), GNEISS; discontinuities near vertical (85°) and parallel to foliation, closely spaced (0.4 ft) [VASSALBORO FORMATION].
BB-FDR-102	NQ (1.875)	145.7	R1	2.5	16.1	21.1	18.6	5.0	4.5	90%	4.3	86%	Good	Fresh (W1)	Very Strong (R5)	76	16.1-21.1 ft: Blue/grey, coarse grained, strongly foliated, fresh (W1), very strong (R5), GNEISS; discontinuities steep (65°-85°) and parallel to foliation, closely to moderately closely spaced (0.5-1.6 ft) [VASSALBORO FORMATION].
			R2	7.5	21.1	26.1	23.6	5.0	5.0	100%	5.0	100%	Excellent	Fresh (W1)	Very Strong (R5)	85	21.1-26.1 ft: Blue/grey, coarse grained, strongly foliated, fresh (W1), very strong (R5), GNEISS; no natural fractures [VASSALBORO FORMATION].
BB-FDR-103	NQ (1.875)	166.4	R1	Cobbles	11.0	16.0	13.5	5.0	0.0	0%	0.0	0%	Very Poor	No Recovery	No Recovery	N/A	11.0-16.0 ft: Large cobbles recovered in rock core run.
			R2	2.5	21.0	25.6	23.3	4.6	4.4	96%	3.4	74%	Fair	Fresh (W1)	Very Strong (R5)	74	21.0-25.6 ft: Grey, coarse grained, strongly foliated, fresh (W1), very strong (R5), GNEISS; discontinuities steep (85°) and parallel to foliation, closely to moderately closely spaced (0.5-2.7 ft) [VASSALBORO FORMATION].
			R3	7.4	25.6	30.8	28.2	5.2	5.1	98%	4.1	79%	Good	Fresh (W1)	Very Strong (R5)	72	25.6-30.8 ft: Grey, coarse grained, strongly foliated, fresh (W1), very strong (R5), GNEISS; discontinuities steep (55°-85°) and parallel to foliation, closely to moderately closely spaced (0.3-1.3 ft) [VASSALBORO FORMATION].
BB-FDR-104	NQ (1.875)	144.4	R1	2.7	10.5	15.5	13.0	5.0	4.8	96%	3.6	72%	Fair	Fresh (W1)	Strong (R4)	68	10.5-15.5 ft: Grey, medium-grained, moderately to strongly foliated, fresh (W1), strong (R4), SCHIST; intermingled with white and black, coarse-grained, moderately to strongly foliated, fresh (W1), very strong (R5), GNEISS; discontinuities have shallow dips (20°-30°), very close to moderately closely spaced (0.1- 1.3 ft) [VASSALBORO FORMATION].
			R2	7.7	15.5	20.5	18.0	5.0	4.4	88%	3.5	70%	Fair	Fresh (W1)	Very Strong (R5)	68	15.5-20.5 ft: Top 3.2 ft: Grey, medium-grained, strongly foliated, fresh (W1), very strong (R5), SCHIST;-discontinuities have shallow dips (20°-30°), very close to closely spaced (0.1- 1.0 ft) [VASSALBORO FORMATION]. Bottom 1.2 ft: White and black, coarse-grained, moderately foliated, fresh (W1), very strong (R5), GNEISS; with discontinuities low angle (20°- 30°), very close to closely spaced (0.1- 1.0) [VASSALBORO FORMATION].

Table 3: Summary of Rock Core Quality
Preliminary Geotechnical Design Report
I-295 Merrill Road Bridge Replacement #5720 (Exit 20)
Freeport, Maine
MaineDOT WIN 023627.00

Test Boring Designation	Core Size (in)	Existing Ground Surface Elevation[1] (ft)	Run						TCR [2]		RQD [3]			Physical Rock Parameters			Lithologic, Rock Mass and Discontinuity Description[6]
			No.	Midpoint Depth Below Bedrock Surface (ft)	Depth Below Ground Surface (ft) Start	End	Midpoint	Length (ft)	Length (ft)	%	Length (ft)	%	Designation	Weathering[4]	Estimated Field Strength[4]	Rock Mass Rating [RMR][5]	
BB-FDR-105	NQ (1.875)	143.4	R1	3.0	18.5	23.5	21.0	5.0	4.7	94%	3.9	78%	Good	Fresh (W1) to Slightly Weathered (W2)	Very Strong (R5)	67	18.5-23.5 ft: Grey, medium-grained, moderately foliated, slightly weathered (top 1.4 ft) to fresh (bottom 3.3 ft) (W1-W2), very strong (R5), biotite SCHIST; discontinuities shallow to steeply dipping (20°-80°), very close to closely spaced (0.1- 1.0 ft), slight reddish staining on discontinuity surfaces in top 1.4 ft [VASSALBORO FORMATION].
			R2	8.0	23.5	28.5	26.0	5.0	5.0	100%	3.3	66%	Fair	Fresh (W1)	Very Strong (R5)	65	23.5-28.5 ft: Grey, medium-grained, moderately foliated, fresh (W1), very strong (R5), biotite SCHIST; discontinuities shallow to steeply dipping (10°-60°) with vertical discontinuity at 27.9-28.3 ft bgs, very close to closely spaced (0.1-0.8 ft) [VASSALBORO FORMATION].
BB-FDR-106	NQ (1.875)	147.6	R1	2.6	6.8	11.8	9.3	5.0	4.8	96%	4.8	96%	Excellent	Fresh (W1)	Strong (R4)	78	6.8-11.8 ft: Black and light grey/white, coarse grained, strongly foliated, fresh (W1), strong (R4), GNEISS; discontinuities have shallow dips (10°-20°) and parallel to foliation, close to moderately closely spaced (0.5- 2.5 ft) [VASSALBORO FORMATION].
			R2	7.6	11.8	16.8	14.3	5.0	4.8	96%	4.6	92%	Excellent	Fresh (W1)	Very Strong (R5)	75	11.8-16.8 ft: Black and light grey/white, coarse grained, strongly foliated, fresh (W1), very strong (R5), GNEISS; discontinuities horizontal to shallow (0°-10°) and parallel to foliation, close to moderately closely spaced (0.2-2.3 ft) [VASSALBORO FORMATION].

Notes:
1. As-drilled elevations are derived from survey files "Freeport 23627.00 Boring Data 2020-01-06.zip" received by Golder on January 6, 2019 from MaineDOT.
2. TCR = total core recovery. Total core recovery is the length of core recovered divided by the length of the run.
3. RQD = rock quality designation. RQD is the total length of intact, full diameter core pieces recovered with a length greater than or equal to twice the core diameter (i.e., length of 4 inches) measured along the core axis. The percent RQD is the total length of RQD measured versus the run length. Note that vertical discontinuities are not included in determination of RQD.
4. Weathering and Estimated Field Strength based on Tables II.4 and II.3 (respectively) in Willey, 2004 (based on ISRM, 1981).
5. Rock Mass Rating (RMR) System (Bieniawski, 1989) assigns numerical ratings to six parameters, including the strength of the intact rock, the RQD, the discontinuity spacing, groundwater conditions, and orientation of discontinuities. These ratings are summed to give the RMR value. The rating adjustment for joint orientation was assigned a value of 0; correlation of geologic field mapping data of exposed rock outcrops with the rock core samples and proposed foundation type may allow for a different rating adjustment for joint orientation, and thus a modification to the RMR value shown on this table.
6. Mapped bedrock formation taken from: Berry & Hussey, 1998; Hussey, 1985; and West & Hussey, 2017.
7. In BB-FDR-101, coring recovered a portion of R1 with R2 and a portion of R2 with R3; excess was included in recovery run for TCR and RQD calculations.
8. ft = feet, in = inches

Prepared by: KAR
Checked by: SKB
Reviewed by: JRS

Table 4: Summary of Laboratory Soil Index and Classification Testing Results
Preliminary Geotechnical Design Report
I-295 Merrill Road Bridge Replacement #5720 (Exit 20)
Freeport, Maine
MaineDOT WIN 023627.00

Test Boring Designation[1]	Ground Surface Elevation[2] (ft)	Sample Number[3]	Sample Depth Below Ground Surface (ft)	Approximate Sample Elevation (ft)	Sieve Minus No. 200 (%)	Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Organic Content (%)	AASHTO Soil Classification[4]	USCS Soil Classification[4]
BB-FDR-101	168.2	3D	11.6 - 13.6	156.6 - 154.6	9.5	15.0	- -	- -	- -	- -	- -	A-3 (0)	SW - SM
BB-FDR-101	168.2	5D	21.0 - 23.0	147.2 - 145.2	12.2	16.1	- -	- -	- -	- -	- -	A-2-4 (0)	SM
BB-FDR-101	168.2	6DA	26.0 - 28.0	142.2 - 140.2	93.6	26.0	40	21	19	0.3	- -	A-6 (19)	CL
BB-FDR-101	168.2	7D	31.0 - 33.0	137.2 - 135.2	- -	23.0	24	16	8	0.9	- -	A-4	CL
BB-FDR-101	168.2	8D	36.0 - 36.6	132.2 - 131.6	31.5	18.0	19	13	6	0.8	- -	A-2-4 (0)	GC - GM
BB-FDR-102	145.7	1DA	0.0 - 2.0	145.7 - 143.7	12.1	14.9	- -	- -	- -	- -	- -	A-1-b (0)	SM
BB-FDR-102	145.7	2D	7.0 - 9.0	138.7 - 136.7	- -	22.0	34	21	13	0.1	- -	A-6	CL
BB-FDR-102	145.7	3D	12.0 - 14.0	133.7 - 131.7	- -	22.0	22	15	7	1.0	- -	A-4	CL -ML
BB-FDR-103	166.4	1D	0.5 - 2.5	165.9 - 163.9	7.9	2.1	- -	- -	- -	- -	- -	A-1-a (0)	GW - GM
BB-FDR-103	166.4	2D	5.0 - 7.0	161.4 - 159.4	- -	2.2	- -	- -	- -	- -	- -	- -	- -
BB-FDR-103	166.4	3D	10.0 - 12.0	156.4 - 154.4	27.9	14.2	- -	- -	- -	- -	- -	A-2-4 (0)	SM
BB-FDR-103	166.4	4DB	16.5 - 18.5	149.9 - 147.9	27.5	17.0	- -	- -	- -	- -	2.5	A-2-4 (0)	SM
BB-FDR-103	166.4	5D	20.0 - 20.8	146.4 - 145.6	35.4	- -	- -	- -	- -	- -	- -	A-4 (0)	SM
BB-FDR-104	144.4	1D	0.0 - 2.0	144.4 - 142.4	- -	21.7	- -	- -	- -	- -	- -	- -	- -
BB-FDR-104	144.4	2D	5.5 - 7.5	138.9 - 136.9	- -	28.0	31	17	14	0.8	- -	A-6	CL
BB-FDR-105	143.4	1DB	0.0 - 2.0	143.4 - 141.4	3.7	12.6	- -	- -	- -	- -	- -	A-1-b (0)	SP
BB-FDR-105	143.4	2DB	5.0 - 6.5	138.4 - 136.9	- -	23.0	35	19	16	0.3	- -	A-6	CL
BB-FDR-105	143.4	3DB	10.0 - 12.0	133.4 - 131.4	70.9	22.2	- -	- -	- -	- -	- -	A-4	ML
BB-FDR-105	143.4	4D	15.0 - 17.0	128.4 - 126.4	- -	30.0	27	16	11	1.3	- -	A-6	CL
BB-FDR-106	147.6	1D	0.0 - 2.0	147.6 - 145.6	50.6	- -	- -	- -	- -	- -	- -	A-4 (0)	ML
BB-FDR-106	147.6	2DB	5.0 - 6.8	142.6 - 140.9	35.1	16.7	- -	- -	- -	- -	- -	A-4 (0)	SM

Notes:
1. Test boring locations are shown in Sheet 2 entitled "Boring Location Plan".
2. As-drilled elevations are derived from survey files "Freeport 23627.00 Boring Data 2020-01-06.zip" received by Golder on January 6, 2019 from MaineDOT.
3. Laboratory testing was performed by GeoTesting Express, Inc.
4. AASHTO and USCS symbols assigned based on interpretation of laboratory test results provided to Golder from GeoTesting Express, Inc. on February 5, 2020.
5. Atterberg Limits ASTM D 4318; Grain Size - Sieve & Hydrometer ASTM D 422; Organic Content ASTM D 2974; Moisture Content ASTM D 2216
6. Complete laboratory soil test results are provided in Appendix C.
7. ft = feet

Prepared By: SKB
Checked By: KAR
Reviewed By: JRS

Table 5: Summary Rock Laboratory Test Results
Preliminary Geotechnical Design Report
I-295 Merrill Road Bridge Replacement #5720 (Exit 20)
Freeport, Maine
MaineDOT WIN 023627.00

Test Boring Designation[1]	Existing Ground Surface[2] (ft)	Top of Bedrock Surface Depth[3] (ft)	Top of Bedrock Surface Elevation (ft)	Sample Number[4]	Sample Depth Below Top of Bedrock (ft)	Sample Depth Below Ground Surface (ft)	Sample Elevation (ft)	Unconfined Compressive Strength[5,6] (ksf)	Bulk Density[7] (pcf)
BB-FDR-102	145.7	16.1	129.6	RUN 1	0.3 - 0.6	16.35 - 16.72	129.3 - 128.9	2903	164
BB-FDR-104	144.4	10.3	134.1	RUN 1	1.2 - 1.6	11.50 - 11.87	132.9 - 132.6	1141	159
BB-FDR-105	143.4	18.0	125.4	RUN 2	5.6 - 5.9	23.57 - 23.94	119.9 - 119.5	2279	171
BB-FDR-106	147.6	6.7	140.9	RUN 1	0.1 - 0.4	6.80 - 7.10	140.8 - 140.5	1155	162

Notes:

1. Test boring locations are shown in Sheet 2 entitled "Boring Location Plan".
2. As-drilled elevations are derived from survey files "Freeport 23627.00 Boring Data 2020-01-06.zip" received by Golder on January 6, 2019 from MaineDOT.
3. Top of bedrock surface refers to the interpreted top of competent rock based on roller cone resistance or material observed in rock core barrel. Notes concerning weathered rock, if present, are provided in Appendix A.
4. Laboratory testing was performed by GeoTesting Express, Inc. Test results provided to Golder on February 5, 2020.
5. BB-FDR-102 RUN 1 testing was performed based on ASTM D7012 Method D - Standard Test Methods for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying States of Stress and Temperatures.
6. BB-FDR-104 RUN 1, BB-FDR-105 RUN 2, and BB-FDR-106 RUN 1 testing was performed based ASTM D7012 Method C - Standard Test Methods for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying States of Stress and Temperatures.
7. Density determined by GTX on core samples by measuring dimensions and weight and then calculating.
8. Complete laboratory test results for rock core testing are provided in Appendix D.
9. ft = feet; ksf = kilopounds per square foot; pcf = pounds per cubic foot

Prepared By: SKB
Checked By: KAR
Reviewed By: JRS

Table 6: **Summary of Soil Properties Used in LPile Analysis - Abutment No. 1**
Preliminary Geotechnical Design Report
I-295 Merrill Road Bridge Replacement #5720 (Exit 20)
Freeport, ME
MaineDOT WIN 023627.00

Stratigraphy		Depth Below Base of Abutment (ft)[1]	Layer Thickness (ft)	Lateral Model[6]	Effective Unit Weight (pcf)	Undrained Shear Strength (psf)[2]	ϕ (deg)[2]	Subgrade Modulus (pci)[3]	Major Principal Strain @ 50%[3]	UCS (psi)[2]
Existing Fill (above WT)	Layer 1	0.0 12.7	12.7	Sand (Reese)	125	-	32	124.8	-	-
Existing Fill (below WT)	Layer 2	12.7 16.0	3.3	Sand (Reese)	62.6	-	32	75.5	-	-
Glaciomarine Silty Clay	Layer 3	16.0 27.0	11.0	Stiff Clay w/o Free Water (Reese)	62.6	1600	-	-	0.005	-
Sand and Gravel	Layer 4	27.0 34.0	7.0	Sand (Reese)	62.6	-	37	40.5	-	-
Bedrock	Layer 5	34.0 50.0	16.0	Strong Rock (Vuggy Limestone)	101.6	-	-	-	-	12983

Notes:

1. Golder interpreted subsurface section A-A' (Sheet 3).
2. Golder geotechnical test boring logs (Appendix A.
3. Bridge Software Institute. “FB-MultiPier Soil Parameter Table (US Customary Units).” Accessed July 2020. <https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf>
4. WT = water table
5. ft = feet, pcf = pounds per cubic foot, psf = pounds per square foot, deg = degrees, pci = pounds per cubic inch; psi = pounds per square inch
6. Layer names refer to the LPile lateral model type rather than the actual soil or rock encountered at site.

Prepared by: MLM
Checked by: KAR
Reviewed by: JRS

Table 7: **Summary of Soil Properties Used in LPile Analysis - Abutment No. 2**
Preliminary Geotechnical Design Report
I-295 Merrill Road Bridge Replacement #5720 (Exit 20)
Freeport, ME
MaineDOT WIN 023627.00

Stratigraphy		Depth Below Base of Abutment (ft)[1]	Layer Thickness (ft)	Lateral Model[6]	Effective Unit Weight (pcf)	Undrained Shear Strength (psf)[2]	ϕ (deg)[2]	Subgrade Modulus (pci)[3]	Major Principal Strain @ 50%[3]	UCS (psi)[2]
Existing Fill (above WT)	Layer 1	0.0 9.8	9.8	Sand (Reese)	125	-	32	124.8	-	-
Existing Fill (below WT)	Layer 2	9.8 10.6	0.8	Sand (Reese)	62.6	-	32	75.5	-	-
Glaciomarine Silty Clay	Layer 3	10.6 13.5	2.9	Stiff Clay w/o Free Water (Reese)	62.6	1600	-	-	0.005	-
Sand and Gravel	Layer 4	13.5 16.3	2.8	Sand (Reese)	62.6	-	37	40.5	-	-
Bedrock	Layer 5	16.3 50.0	33.7	Strong Rock (Vuggy Limestone)	101.6	-	-	-	-	12983

Notes:

1. Golder interpreted subsurface section A-A' (Sheet 3).
2. Golder geotechnical test boring logs (Appendix A).
3. Bridge Software Institute. “FB-MultiPier Soil Parameter Table (US Customary Units).” Accessed July 2020. <https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf>
4. WT = water table
5. ft = feet, pcf = pounds per cubic foot, psf = pounds per square foot, deg = degrees, pci = pounds per cubic inch; psi = pounds per square inch
6. Layer names refer to the LPile lateral model type rather than the actual soil or rock encountered at site.

Prepared by: MLM
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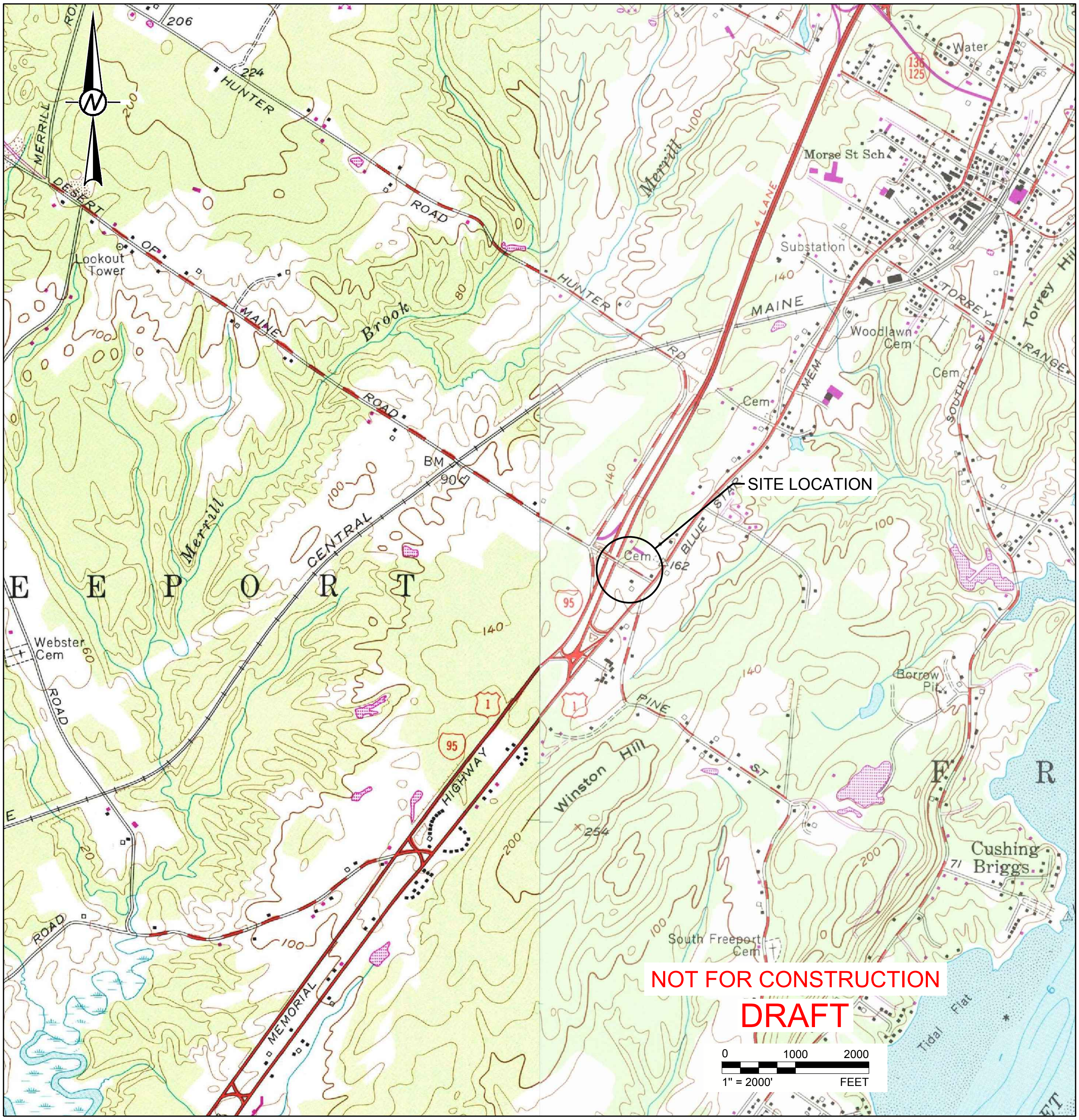


REFERENCE(S)

BASE MAP TAKEN FROM U.S.G.S. 7.5 MINUTE QUADRANGLE OF FREEPORT, MAINE DATED 1957.

CLIENT
MAINE DEPARTMENT OF TRANSPORTATION
16 STATE HOUSE STATION
AUGUSTA, MAINE 04333

PROJECT
I-295 MERRILL ROAD, BRIDGE REPLACEMENT #5720 (EXIT 20)
FREEPORT, MAINE
MAINEDOT WIN 023627.00

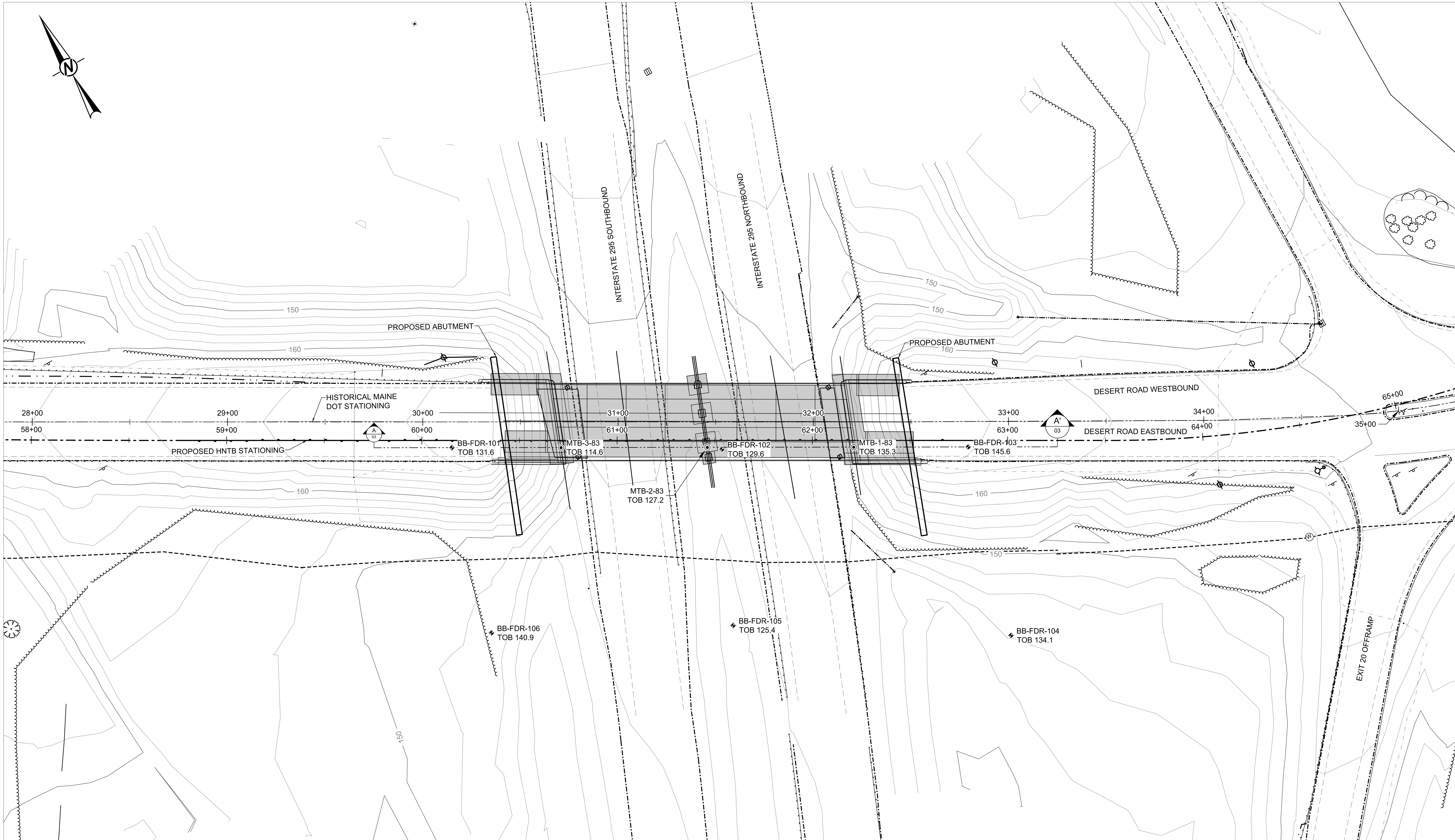
CONSULTANT

GOLDER

YYYY-MM-DD	2020-02-20
DESIGNED	MEL
PREPARED	RWC
REVIEWED	SKB
APPROVED	MCM

TITLE
SITE LOCATION MAP

PROJECT NO.	SUBTITLE	REV.	SHEET
19126013	A	0	1/3



NOTE(S)

1. SEE BORING LOGS (APPENDIX A) FOR DETAILED LITHOLOGIC DESCRIPTIONS.
2. AS DRILLED LOCATIONS OF GOLDER'S 2019 EXPLORATIONS WERE PROVIDED BY MAINEDOT ON JANUARY 6, 2020.
3. HISTORICAL BORING LOCATIONS ARE APPROXIMATE AND BASED ON AS-BUILT DESIGN DRAWINGS FROM 1985 PROVIDED BY MAINEDOT.

REFERENCE(S)

1. BASEMAP ELEMENTS TAKEN FROM MAINE DOT DRAWING TITLED "3DTopo_2019-10-18.dgn" RECEIVED ON JANUARY 2, 2020.
2. SURVEY PROVIDED TO GOLDER ON JANUARY 02, 2020 BY MAINE DOT IN FILE "FREEPORT 23627.00 SURVEY DATA 2019-11-18.zip".
3. AS BUILT JUNE 1985 REPORT STATE MAINE DEPARTMENT OF TRANSPORTATION OF DRAWINGS ELECTRONIC FILE TITLED "5720 FREEPORT 1984".
4. PROPOSED STATIONING PROVIDED TO GOLDER BY HNTB FOR A SOUTHERN SHIFT OF THE BRIDGE.

GOLDER

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
19126013
PIN
023627.00
BRIDGE PLANS
Bridge No. 5720

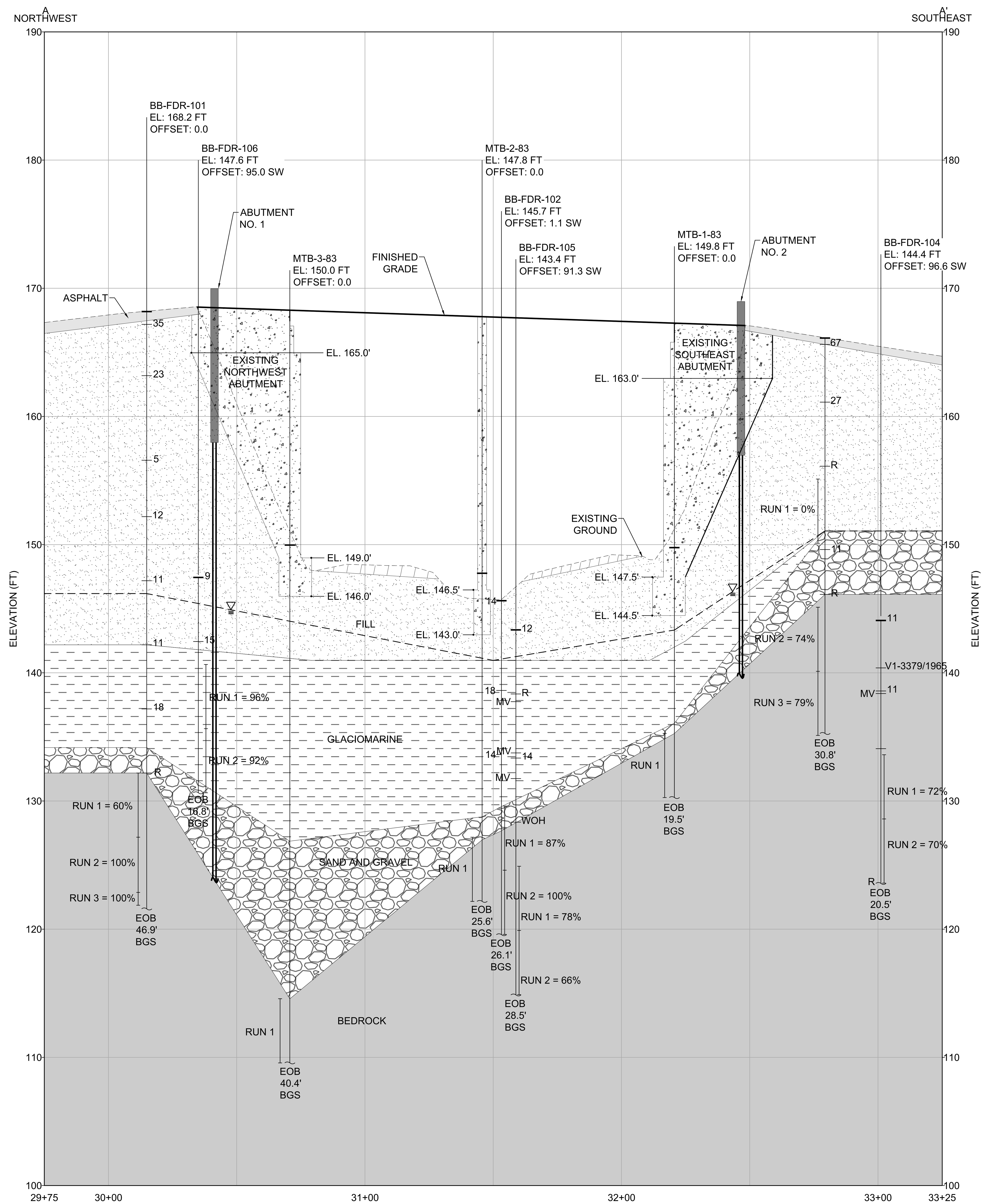
SIGNATURE			
P.E. NUMBER			
DATE			

		BY	DATE
PROJ. MANAGER	MEL		
DESIGN-DETAILED	MEL	AAZ	2020-12-02
CHECKED-REVIEWED			
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

FREEPORT

BORING LOCATION PLAN

SHEET NUMBER
2
OF 3



LEGEND

BB-FDR-104 / EL: 144.4 FT / OFFSET: 96.6 S — BORING LOCATION I.D. / ELEVATION / OFFSET (FT) FROM SECTION LINE

TOP OF BORING

30 — SPT: N60 - VALUE (CORRECTED FOR HAMMER EFFICIENCY)

V1 - 3379/1965 — FIELD VANE SHEAR TEST LOCATION AND UNDRAINED SHEAR STRENGTH (PSF) (PEAK/REMOLDED)

WOR — WEIGHT OF ROD

MV — MISSED VANE

R — REFUSAL

RUN1 - 92% — ROCK CORE RUN NUMBER AND ROD QUALITY DESIGNATION (RQD)

EOB 60.6' BGS — END OF BORING DEPTH BELOW GROUND SURFACE (BGS)

- ASPHALT
- EXISTING ABUTMENT AND FOUNDATION ELEMENTS
- PROPOSED ABUTMENT ELEMENTS
- BROWN TO GRAY, LOOSE TO MEDIUM DENSE, SILTY FINE TO COARSE SAND AND GRAVEL (FILL)
- GRAY, VERY SOFT TO VERY STIFF, LOW PLASTICITY SILTY CLAY (GLACIOMARINE)
- GRAY, VERY LOOSE TO VERY DENSE, FINE TO COARSE SAND, SOME SILT/CLAY, LITTLE GRAVEL (SAND AND GRAVEL)
- LIGHT GRAY, COARSE-GRAINED GNEISS, EXTREMELY STRONG, FRESH TO SLIGHTLY WEATHERED (BEDROCK)
- EXISTING GROUND SURFACE
- INTERPRETED GROUNDWATER SURFACE

REFERENCES:

1. AS DRILLED BORING LOCATION PLAN DERIVED FROM ELECTRONIC FILE NAME: "BOR12-17-19edit.csv" PROVIDED TO GOLDER BY MAINE DEPARTMENT OF TRANSPORTATION ON 01/06/2020.
2. SEE BORING LOGS IN APPENDIX A FOR DETAILED LITHOLOGIC DESCRIPTIONS.
3. SEE LABORATORY REPORTS FOR COMPLETE LABORATORY DATA.
4. GROUNDWATER SURFACE IS INTERPRETED FROM LOCALIZED SURFACE WATER LEVELS AND MEASUREMENTS TAKEN DURING THE SUBSURFACE EXPLORATION PROGRAM.
5. THIS GENERALIZED SUBSURFACE PROFILE IS INTENDED TO CONVEY TRENDS IN SUBSURFACE CONDITIONS. THE BOUNDARIES BETWEEN STRATA ARE APPROXIMATE AND IDEALIZED, AND HAVE BEEN DEVELOPED BASED ON INTERPRETATIONS OF WIDELY SPACED EXPLORATIONS. ACTUAL SOIL AND ROCK TRANSITIONS MAY VARY AND ARE PROBABLY MORE ERRATIC. FOR MORE SPECIFIC INFORMATION, REFER TO BORING LOGS.
6. HISTORICAL BORING LOCATIONS, EXISTING ABUTMENT AND FOUNDATION ELEMENTS, AND SOIL STRATA ARE APPROXIMATE AND HAVE BEEN INTERPRETED FROM ELECTRONIC FILE NAME "5720 FREEPORT 1984" AND "1984 SOILS DATA" PROVIDED TO GOLDER BY MAINE DEPARTMENT OF TRANSPORTATION ON MAY 2, 2019.
7. SOIL STRATA ANALYSIS WITH THE ASPHALT LAYER AND ROADFILL LAYER ARE COMBINED FOR A LAYER THICKNESS OF FIVE FEET BENEATH THE I-295 NORTHBOUND AND SOUTHBOUND LANES.

VERTICAL SCALE 0 5 10 1" = 5' FEET

HORIZONTAL SCALE 0 25 50 1" = 25' FEET

GOLDER

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

19126013

PIN 023627.00

BRIDGE PLANS

Bridge No. 5720

SIGNATURE

P.E. NUMBER

DATE

	BY	DATE
PROJ. MANAGER	MEL	
DESIGN-DETAILED	MEL	
CHECKED-REVIEWED	AAZ	2020-12-02
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

FREEPORT

SUBSURFACE PROFILE A-A'

SHEET NUMBER 3 OF 3

Date:2020-12-02

Username:

Division:

Filename: 19126013_0120_004

APPENDIX A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
COARSE-GRAINED SOILS (more than half of material is larger than No. 200 sieve size)	GRAVELS (more than half of coarse fraction is larger than No. 4 sieve size)	CLEAN GRAVELS (little or no fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines.
			GP	Poorly-graded gravels, gravel sand mixtures, little or no fines.
		GRAVEL WITH FINES (Appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixtures.
			GC	Clayey gravels, gravel-sand-clay mixtures.
	SANDS (more than half of coarse fraction is smaller than No. 4 sieve size)	CLEAN SANDS (little or no fines)	SW	Well-graded sands, gravelly sands, little or no fines
			SP	Poorly-graded sands, gravelly sand, little or no fines.
		SANDS WITH FINES (Appreciable amount of fines)	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures.
FINE-GRAINED SOILS (more than half of material is smaller than No. 200 sieve size)	SILTS AND CLAYS (liquid limit less than 50)		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays of low plasticity.
	SILTS AND CLAYS (liquid limit greater than 50)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts.
	HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils.

Desired Soil Observations (in this order, if applicable):

Color (Munsell color chart)
Moisture (dry, damp, moist, wet)
Density/Consistency (from above right hand side)
Texture (fine, medium, coarse, etc.)
Name (sand, silty sand, clay, etc., including portions - trace, little, etc.)
Gradation (well-graded, poorly-graded, uniform, etc.)
Plasticity (non-plastic, slightly plastic, moderately plastic, highly plastic)
Structure (layering, fractures, cracks, etc.)
Bonding (well, moderately, loosely, etc.,)
Cementation (weak, moderate, or strong)
Geologic Origin (till, marine clay, alluvium, etc.)
Groundwater level

Maine Department of Transportation
Geotechnical Section
Key to Soil and Rock Descriptions and Terms
Field Identification Information

MODIFIED BURMISTER SYSTEM

Descriptive Term	Portion of Total (%)
trace	0 - 10
little	11 - 20
some	21 - 35
adjective (e.g. sandy, clayey)	36 - 50

TERMS DESCRIBING DENSITY/CONSISTENCY

Coarse-grained soils (more than half of material is larger than No. 200 sieve): Includes (1) clean gravels; (2) silty or clayey gravels; and (3) silty, clayey or gravelly sands. Density is rated according to standard penetration resistance (N-value).

Density of Cohesionless Soils	Standard Penetration Resistance N-Value (blows per foot)
Very loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

Fine-grained soils (more than half of material is smaller than No. 200 sieve): Includes (1) inorganic and organic silts and clays; (2) gravelly, sandy or silty clays; and (3) clayey silts. Consistency is rated according to undrained shear strength as indicated.

Consistency of Cohesive soils	SPT N-Value (blows per foot)	Approximate Undrained Shear Strength (psf)	Field Guidelines
Very Soft	WOH, WOR, WOP, <2	0 - 250	Fist easily penetrates
Soft	2 - 4	250 - 500	Thumb easily penetrates
Medium Stiff	5 - 8	500 - 1000	Thumb penetrates with moderate effort
Stiff	9 - 15	1000 - 2000	Indented by thumb with great effort
Very Stiff	16 - 30	2000 - 4000	Indented by thumbnail
Hard	>30	over 4000	Indented by thumbnail with difficulty

Rock Quality Designation (RQD):

$$\text{RQD (\%)} = \frac{\text{sum of the lengths of intact pieces of core* > 4 inches}}{\text{length of core advance}}$$

*Minimum NQ rock core (1.88 in. OD of core)

Correlation of RQD to Rock Mass Quality

Rock Mass Quality	RQD (%)
Very Poor	≤25
Poor	26 - 50
Fair	51 - 75
Good	76 - 90
Excellent	91 - 100

Desired Rock Observations (in this order, if applicable):

Color (Munsell color chart)
Texture (aphanitic, fine-grained, etc.)
Rock Type (granite, schist, sandstone, etc.)
Hardness (very hard, hard, mod. hard, etc.)
Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.)
Geologic discontinuities/jointing:
-dip (horiz - 0-5 deg., low angle - 5-35 deg., mod. dipping - 35-55 deg., steep - 55-85 deg., vertical - 85-90 deg.)
-spacing (very close - <2 inch, close - 2-12 inch, mod. close - 1-3 feet, wide - 3-10 feet, very wide >10 feet)
-tightness (tight, open, or healed)
-infilling (grain size, color, etc.)
Formation (Waterville, Ellsworth, Cape Elizabeth, etc.)
RQD and correlation to rock mass quality (very poor, poor, etc.)
ref: ASTM D6032 and AASHTO Standard Specification for Highway Bridges, 17th Ed. Table 4.4.8.1.2A
Recovery (inch/inch and percentage)
Rock Core Rate (X.X ft - Y.Y ft (min:sec))

Sample Container Labeling Requirements:

WIN	Blow Counts
Bridge Name / Town	Sample Recovery
Boring Number	Date
Sample Number	Personnel Initials
Sample Depth	

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-295 Merrill Road Bridge Replacement #5720 (Exit 20)
Location: Freeport, Maine

Boring No.: BB-FDR-101
WIN: 023627.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	168.2	**Auger ID/OD:**	4 in OD Solid Stem
Operator:	Mike Porter	**Datum:**	NAD83 (2011) Maine 2000 West	**Sampler:**	Standard Split Spoon
Logged By:	Shiv Bhardwaj	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	12/10/19 (8:57);12/11/19 (11:50)	**Drilling Method:**	Solid Stem Auger / Cased Wash	**Core Barrel:**	1-7/8 in - NQ
Boring Location:	N: 368277.8, E: 1051175.0	**Casing ID/OD:**	4 in/4.5 in	**Water Level*:**	12.6 ft on 12/10/19 at 15:41
Hammer Efficiency Factor: 0.914		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample Attempt
V = Field Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Field Vane Shear Test Attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	167.5		Driller notes asphalt thickness of 8 in (ASPHALT). 0.7	
	1D	24/17	1.00 - 3.00	14/12/11/13	23	35				Brown, dry, dense, fine to coarse SAND, little fine gravel, trace silt, well-graded (FILL).	
5	2D	24/16	5.00 - 7.00	8/7/8/11	15	23				Brown, dry, medium dense, fine to coarse SAND, little fine gravel, trace silt, well-graded (FILL).	
							60				
							67			Losing water to formation from 5-8 ft bgs.	
							277				
10							47			Driller notes obstruction from 9.4-9.9 ft bgs. Rock fragments in wash water. Casing bent from encountered hard material.	
	3D	24/10	11.60 - 13.60	1/2/1/2	3	5	44			Brown, wet, loose, fine to coarse SAND, little fine gravel, trace silt, well-graded (FILL).	GTX #539965 WC = 15.0% Fines = 9.5% A-3 (0), SW-SM
							44				
							40				
15							47				
							31				
	4D	24/8	16.00 - 18.00	2/5/3/5	8	12	42			Brown, moist, medium dense, fine to coarse SAND, little silt, well-graded (FILL).	
							58				
							73				
20							56				
							27				
	5D	24/7.5	21.00 - 23.00	4/3/4/7	7	11	40			Brown, moist, medium dense, fine to coarse SAND, little silt, trace fine gravel, well-graded (FILL).	GTX #539966 WC = 16.1% Fines = 12.2% A-2-4 (0), SM
							46				
							102				
25							78				

Remarks:

1. Hammer Efficiency Factor provided by New England Boring Contractors and taken from "SPT Energy Testing - ATV Mobile Drill Rig No. 20 Mobile Hammer NEBC-1" by GZA GeoEnvironmental, dated 7/12/19. 2. As-drilled boring locations and ground surface elevations were provided by MaineDOT. 3. Water Levels: 12.6 ft on 12/10/2019 at 15:41, 23.8 ft on 12/11/19 at 9:46.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-295 Merrill Road Bridge Replacement #5720 (Exit 20)
Location: Freeport, Maine

Boring No.: BB-FDR-101
WIN: 023627.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	168.2	**Auger ID/OD:**	4 in OD Solid Stem
Operator:	Mike Porter	**Datum:**	NAD83 (2011) Maine 2000 West	**Sampler:**	Standard Split Spoon
Logged By:	Shiv Bhardwaj	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	12/10/19 (8:57);12/11/19 (11:50)	**Drilling Method:**	Solid Stem Auger / Cased Wash	**Core Barrel:**	1-7/8 in - NQ
Boring Location:	N: 368277.8, E: 1051175.0	**Casing ID/OD:**	4 in/4.5 in	**Water Level*:**	12.6 ft on 12/10/19 at 15:41
Hammer Efficiency Factor:	0.914	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)
25							21	
	6D	24/14	26.00 - 28.00	5/4/3/6	7	11	31	142.2
							60	
							93	
							131	
30							103	
	7D	24/24	31.00 - 33.00	7/4/8/10	12	18	123	
							127	
							102	
							126	134.2
35							Open	
	8D	7/7	36.00 - 36.58	16/50(1")			NQ	131.6
	R1	60/43.5	36.60 - 41.60	RQD = 60%				
40								
	R2	51.6/59	41.60 - 45.90	RQD = 100%				
45	R3	12/14	45.90 - 46.90	RQD = 100%				121.3
50								

Visual Description and Remarks

Asphalt in wash water from 24-25 ft bgs.

26.0

6DB, Top 6.5 in: Grey, wet, stiff, Sandy CLAY, trace fine gravel, slightly plastic (GLACIOMARINE).
6DA, Bottom 7.5 in: Grey, wet, stiff, CLAY, trace fine to medium sand, moderately plastic. q_p = 5.0 ksf (Pocket Pentrometer), T_v = 500, 600 psf (GLACIOMARINE).
Wash color changes from brown to grey at 26 ft bgs.

Grey, wet, very stiff, CLAY, trace fine sand, slightly plastic (GLACIOMARINE).

34.0

Drill rig shaking at 34 ft bgs.
Grey clay and sand in wash water at 35 ft bgs.

Grey, wet, very dense, GRAVEL, some silt/clay, little fine to coarse sand, slightly plastic (SAND AND GRAVEL).

36.6

Top of Bedrock at Elev. 131.6 ft.

R1: Grey, coarse grained, strongly foliated, fresh (W1), very strong (R5), GNEISS; discontinuities steep (55°-75°) and parallel to foliation, closely spaced (0.2-0.8 ft) [VASSALBORO FORMATION].
Rock Mass Quality = Fair
Core Times (min:sec)
36.6-37.6 ft (2:06)
37.6-38.6 ft (1:47)
38.6-39.6 ft (1:57)
39.6-40.6 ft (1:19)
40.6-41.6 ft (1:30)
73% Recovery
Driller notes void in core run R1

R2: Blue/grey, coarse grained, strongly foliated, fresh (W1), very strong (R5), GNEISS with PEGMATITE veins; discontinuities steep (70°-80°) and parallel to foliation, very closely to moderately closely spaced (0.1-1.5 ft) [VASSALBORO FORMATION].
Rock Mass Quality = Excellent
Core Times (min:sec)
41.6-42.6 ft (3:34)

Laboratory Testing Results/ AASHTO and Unified Class.

6DA:
GTX #539975, 539957
WC = 26%
Fines = 93.6%
LL = 40
PL = 21
PI = 19
LI = 0.3
A-6 (19), CL

7D:
GTX #539958
WC = 23%
LL = 24
PL = 16
PI = 8
LI = 0.9
A-4 , CL

8D:
GTX #539967, 539959
WC = 18%
Fines = 31.5%
LL = 19
PL = 13
PI = 6
LI = 0.8
A-2-4 (0), GC-GM

Remarks:

1. Hammer Efficiency Factor provided by New England Boring Contractors and taken from "SPT Energy Testing - ATV Mobile Drill Rig No. 20 Mobile Hammer NEBC-1" by GZA GeoEnvironmental, dated 7/12/19. 2. As-drilled boring locations and ground surface elevations were provided by MaineDOT. 3. Water Levels: 12.6 ft on 12/10/2019 at 15:41, 23.8 ft on 12/11/19 at 9:46.

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Maine Department of Transportation

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US CUSTOMARY UNITS

Project: I-295 Merrill Road Bridge Replacement #5720 (Exit 20)
Location: Freeport, Maine

Boring No.: BB-FDR-101
WIN: 023627.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	168.2	**Auger ID/OD:**	4 in OD Solid Stem
Operator:	Mike Porter	**Datum:**	NAD83 (2011) Maine 2000 West	**Sampler:**	Standard Split Spoon
Logged By:	Shiv Bhardwaj	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140 lbs/30 in
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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50										42.6-43.6 ft (1:29) 43.6-44.6 ft (1:22) 44.6-45.6 ft (1:47) 45.6-45.9 ft (1:19) 114% Recovery Recovered a portion of R1 with R2. R3: Blue/grey, coarse grained, strongly foliated, fresh (W1), very strong (R5), GNEISS; discontinuities near vertical (85°) and parallel to foliation, closely spaced (0.4 ft) [VASSALBORO FORMATION]. Rock Mass Quality = Excellent Core Times (min:sec) 45.9-46.9 ft (1:42) 117% Recovery. Recovered a portion of R2 with R3.	
55											
										46.9	
										Bottom of Exploration at 46.9 feet below ground surface. Boring backfilled with bentonite grout to 10 ft bgs, bentonite chips from 10-8 ft bgs, some sand, gravel from 8-1 ft bgs and cold patch asphalt from 1 ft bgs to road surface.	
60											
65											
70											
75											

Remarks:

1. Hammer Efficiency Factor provided by New England Boring Contractors and taken from "SPT Energy Testing - ATV Mobile Drill Rig No. 20 Mobile Hammer NEBC-1" by GZA GeoEnvironmental, dated 7/12/19. 2. As-drilled boring locations and ground surface elevations were provided by MaineDOT. 3. Water Levels: 12.6 ft on 12/10/2019 at 15:41, 23.8 ft on 12/11/19 at 9:46.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-295 Merrill Road Bridge Replacement #5720 (Exit 20)	**Location:** Freeport, Maine	**Boring No.:**	BB-FDR-102
		WIN:	023627.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	145.7	**Auger ID/OD:**	N/A
Operator:	Mike Porter	**Datum:**	NAD83 (2011) Maine 2000 West	**Sampler:**	Standard Split Spoon
Logged By:	Shiv Bhardwaj	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	12/11/19 (12:00);12/12/19 (14:14)	**Drilling Method:**	Cased Wash	**Core Barrel:**	1-7/8 in - NQ
Boring Location:	N: 368209.0, E: 1051295.0	**Casing ID/OD:**	4 in/4.5 in	**Water Level*:**	Not Recorded
Hammer Efficiency Factor:	0.914	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample Attempt
V = Field Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Field Vane Shear Test Attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/10	0.00 - 2.00	WOR(3"),1(3")/4/5/6	9	14	Open	145.4		1DB, Top: 3.5 in: brown, moist, stiff, SILT, some fine to coarse sand, some organics, non-plastic (TOPSOIL). 0.3	WC = 14.9% 1DA: GTX #539968 Fines = 12.1% A-1-b (0), SM
										1DA, Bottom 6.5 in: Brown, moist, medium dense, fine to coarse SAND, little silt, trace fine gravel, well-graded (FILL). Brown sand in wash water at 2 ft bgs.	
							23				
							28				
							20				
5							23				
							35	139.7		6.0 Brown clay and gravel in wash water at 6 ft bgs.	
	2D	24/24	7.00 - 9.00	3/5/7/10	12	18	54			Wash water color changes to grey at 7 ft bgs; clay in wash water at 7 ft bgs.	GTX #539993 WC = 22% LL = 34 PL = 21 PI = 13 LI = 0.1 A-6, CL
							80			Grey, dry, very stiff, CLAY, little silt, moderately plastic (GLACIOMARINE). q_p = 8.0, 7.0 ksf (Pocket Pentrometer), T_v = 500, 600, 500 psf.	
							97				
10							105				
							123				
	3D	24/4	12.00 - 14.00	4/6/3/5	9	14	71			Grey, wet, stiff, Silty CLAY, slightly plastic (GLACIOMARINE).	GTX #539961 WC = 22 LL = 22 PL = 15 PI = 7 LI = 1.1 A-4, CL-ML
							62				
							75				
15							Open				
	R1	60/54	16.10 - 21.10	RQD = 87%			NQ	129.6		16.1 Top of Bedrock at Elev. 129.6 ft.	GTX #311185 q_p = 2903 ksf (128.9-129.3 ft)
										R1: Blue/grey, coarse grained, strongly foliated, fresh (W1), very strong (R5), GNEISS; discontinuities steep (65°-85°) and parallel to foliation, closely to moderately closely spaced (0.5-1.6 ft) [VASSALBORO FORMATION]. Rock Mass Quality = Good Core Times (min:sec) 16.1-17.1 ft (3:25) 17.1-18.1 ft (2:07) 18.1-19.1 ft (1:14) 19.1-10.1 ft (1:28) 20.1-21.1 ft (1:42) 90% Recovery	
20											
	R2	60/60	21.10 - 26.10	RQD = 100%							
25											

Remarks:

1. Hammer Efficiency Factor provided by New England Boring Contractors and taken from "SPT Energy Testing - ATV Mobile Drill Rig No. 20 Mobile Hammer NEBC-1" by GZA GeoEnvironmental, dated 7/12/19. 2. As-drilled boring locations and ground surface elevations were provided by MaineDOT. 3. Boring drilled through bridge deck; driller noted 23 ft from bottom of bridge deck to ground surface.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-295 Merrill Road Bridge Replacement #5720 (Exit 20)
Location: Freeport, Maine

Boring No.: BB-FDR-102
WIN: 023627.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	145.7	**Auger ID/OD:**	N/A
Operator:	Mike Porter	**Datum:**	NAD83 (2011) Maine 2000 West	**Sampler:**	Standard Split Spoon
Logged By:	Shiv Bhardwaj	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	12/11/19 (12:00);12/12/19 (14:14)	**Drilling Method:**	Cased Wash	**Core Barrel:**	1-7/8 in - NQ
Boring Location:	N: 368209.0, E: 1051295.0	**Casing ID/OD:**	4 in/4.5 in	**Water Level*:**	Not Recorded
Hammer Efficiency Factor:	0.914	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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MV = Unsuccessful Field Vane Shear Test Attempt
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SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140 lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25								119.6		R2: Blue/grey, coarse grained, strongly foliated, fresh (W1), very strong (R5), GNEISS; no natural fractures [VASSALBORO FORMATION]. Rock Mass Quality = Excellent Core Times (min:sec) 21.1-22.1 ft (1:54) 22.1-23.1 ft (1:40) 23.1-24.1 ft (1:12) 24.1-25.1 ft (1:24) 25.1-26.1 ft (1:45) 100% Recovery 26.1 **Bottom of Exploration at 26.1 feet below ground surface.** Boring backfilled with bentonite grout to 0.5 ft bgs and gravel from 0.5 ft bgs to 0.5 ft above ground surface. Bridge deck patched with steel rebar and quick setting concrete.	
30											
35											
40											
45											
50											

Remarks:

1. Hammer Efficiency Factor provided by New England Boring Contractors and taken from "SPT Energy Testing - ATV Mobile Drill Rig No. 20 Mobile Hammer NEBC-1" by GZA GeoEnvironmental, dated 7/12/19. 2. As-drilled boring locations and ground surface elevations were provided by MaineDOT. 3. Boring drilled through bridge deck; driller noted 23 ft from bottom of bridge deck to ground surface.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-295 Merrill Road Bridge Replacement #5720 (Exit 20)
Location: Freeport, Maine

Boring No.: BB-FDR-103
WIN: 023627.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	166.4	**Auger ID/OD:**	4 in OD Solid Stem
Operator:	Mike Porter	**Datum:**	NAD83 (2011) Maine 2000 West	**Sampler:**	Standard Split Spoon
Logged By:	Shiv Bhardwaj	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	12/12/19 (13:41);12/13/19 (12:19)	**Drilling Method:**	Cased Wash	**Core Barrel:**	1-7/8 in - NQ
Boring Location:	N: 368148.1, E: 1051405.4	**Casing ID/OD:**	4 in/4.5 in	**Water Level*:**	15.4 ft on 12/13/19 at 8:37
Hammer Efficiency Factor:	0.914	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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MV = Unsuccessful Field Vane Shear Test Attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/24	0.50 - 2.50	20/24/20/18	44	67	SSA	166.0		Driller notes asphalt thickness of 5 in (ASPHALT). 0.4 Brown, dry, very dense, fine to coarse Sandy fine to coarse GRAVEL, trace silt, well-graded (FILL). Concrete fragments found in sample.	GTX #539969 WC = 2.1% Fines = 7.9% A-1-a (0), GW-GM
5	2D	24/16	5.00 - 7.00	15/9/9/8	18	27	56			Brown, dry, medium dense, fine to coarse SAND, little fine gravel, well-graded (FILL).	WC = 2.2%
							47			Wash color starts brown.	
							54				
							60				
							68				
10	3D	24/7	10.00 - 12.00	16/14/50(1")			29			Brown, wet, very dense, fine to coarse SAND, some silt, little fine gravel, well-graded (FILL).	GTX #539970 WC = 14.2% Fines = 27.9% A-2-4 (0), SM
	R1	60/0	11.00 - 16.00	RQD = 0%			NQ			Large cobbles recovered in rock core run R1, 11.0-16.0 ft bgs, no recovery.	
15								151.4		15.0 Wash water from 15-20 ft bgs consisted of some gravel, trace amounts of clay, trace amounts of organic matter.	
	4D	24/11.5	16.50 - 18.50	3/4/3/13	7	11	9			Brown, moist, medium dense, fine to coarse SAND, some silt, little fine gravel, trace organics, well-graded (SAND AND GRAVEL).	4D: GTX #539976 WC = 17% Organic Matter = 2.5% Fines = 27.5% A-2-4 (0), SM
							16				
							28				
							31				
20	5D	9/7	20.00 - 20.75	12/50(3")			OPEN	145.6		Grey, wet, hard, Silty fine to coarse SAND, trace fine gravel (SAND AND GRAVEL). 20.8	5D: GTX #539971 Fines = 35.4% A-4 (0), SM
	R2	55.2/53	21.00 - 25.60	RQD = 74%			NQ			Top of Bedrock at Elev. 145.6 ft. R2: Grey, coarse grained, strongly foliated, fresh (W1), very strong (R5), GNEISS; discontinuities steep (85°) and parallel to foliation, closely to moderately closely spaced (0.5-2.7 ft) [VASSALBORO FORMATION].	
25											

Remarks:

1. Hammer Efficiency Factor provided by New England Boring Contractors and taken from "SPT Energy Testing - ATV Mobile Drill Rig No. 20 Mobile Hammer NEBC-1" by GZA GeoEnvironmental, dated 7/12/19. 2. As-drilled boring locations and ground surface elevations were provided by MaineDOT.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-295 Merrill Road Bridge Replacement #5720 (Exit 20)
Location: Freeport, Maine

Boring No.: BB-FDR-103
WIN: 023627.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	166.4	**Auger ID/OD:**	4 in OD Solid Stem
Operator:	Mike Porter	**Datum:**	NAD83 (2011) Maine 2000 West	**Sampler:**	Standard Split Spoon
Logged By:	Shiv Bhardwaj	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	12/12/19 (13:41);12/13/19 (12:19)	**Drilling Method:**	Cased Wash	**Core Barrel:**	1-7/8 in - NQ
Boring Location:	N: 368148.1, E: 1051405.4	**Casing ID/OD:**	4 in/4.5 in	**Water Level*:**	15.4 ft on 12/13/19 at 8:37
Hammer Efficiency Factor:	0.914	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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MV = Unsuccessful Field Vane Shear Test Attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140 lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	R3	62.4/61	25.60 - 30.80	RQD = 79%						Rock Mass Quality = Fair Core Times (min:sec) 21.0-22.0 ft (3:36) 22.0-23.0 ft (2:14) 23.0-24.0 ft (2:06) 24.0-25.0 ft (2:32) 25.0-25.6 ft (2:29) 96% Recovery	
30								135.6		R3: Grey, coarse grained, strongly foliated, fresh (W1), very strong (R5), GNEISS; discontinuities steep (55°-85°) and parallel to foliation, closely to moderately closely spaced (0.3-1.3 ft) [VASSALBORO FORMATION]. Rock Mass Quality = Good Core Times (min:sec) 25.6-26.6 ft (2:28) 26.6-27.6 ft (1:43) 27.6-28.6 ft (2:35) 28.6-29.6 ft (3:39) 29.6-30.8 ft (2:13) 98% Recovery	
35										30.8 **Bottom of Exploration at 30.8 feet below ground surface.** Boring backfilled with bentonite grout to 8 ft bgs, bentonite chips from 8-7.5 ft bgs, gravel from 7.5-0.5 ft bgs, and cold patch asphalt from 0.5 ft bgs to road surface. Bentonite grout was lost to borehole formation at 8 ft bgs.	
40											
45											
50											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-295 Merrill Road Bridge Replacement #5720 (Exit 20)
Location: Freeport, Maine

Boring No.: BB-FDR-104
WIN: 023627.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	144.4	**Auger ID/OD:**	4 in OD Solid Stem
Operator:	Mike Porter	**Datum:**	NAD83 (2011) Maine 2000 West	**Sampler:**	Standard Split Spoon
Logged By:	Shiv Bhardwaj / Karen Roth	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	12/13/19 (13:47);12/16/19 (12:49)	**Drilling Method:**	Solid Stem Auger / Cased Wash	**Core Barrel:**	1-7/8 in - NQ
Boring Location:	N: 368053.2, E: 1051377.0	**Casing ID/OD:**	4 in/4.5 in	**Water Level*:**	0.7 ft on 12/16/19 at 10:39
Hammer Efficiency Factor:	0.914	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample Attempt
V = Field Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Field Vane Shear Test Attempt

R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person

S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected

T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/21.5	0.00 - 2.00	3/4/3/1	7	11	SSA			Brown and grey, dry, stiff, CLAY, trace organics, moderately plastic (GLACIOMARINE). q_p = 4.50, 4.00 ksf (Pocket Penetrometer), T_v = 700, 800 psf.	WC = 21.7%
	V1			S_u=3379/1965psf						16x32 mm vane raw torque readings (3.7 ft bgs): V1: 21.5/12.5 in-lbs	
5	2D	24/24	5.50 - 7.50	2/4/3/4	7	11	1			Brown, wet, stiff, CLAY, little fine sand, moderately plastic (GLACIOMARINE). q_p = 7.0, 7.0 ksf (Pocket Penetrometer), T_v = 1000, 1000 psf.	GTX #539963 WC = 28% LL = 31 PL = 17 PI = 14 LI = 0.8 A-6, CL
	MV			Would Not Push			11				
							12			Failed 16x32 mm vane, would not push to 5.8 ft bgs.	
							16				
							27	135.4		Sand and silt in wash water 7.5-9 ft bgs. 9.0	
10	3D	3/3	10.00 - 10.25	50(3")			OPEN	134.1		Gravel and sand in wash water 9-10 ft bgs.	
	R1	60/58	10.50 - 15.50	RQD = 72%			NQ			Brown to grey, wet, very dense, fine to medium SAND, some silt, little gravel, well-graded (SAND AND GRAVEL). 10.3	
										Top of Bedrock at Elev. 134.1 ft.	
										R1: Grey, medium-grained, moderately to strongly foliated, fresh (W1), strong (R4), SCHIST; intermingled with white and black, coarse-grained, moderately to strongly foliated, fresh (W1), very strong (R5), GNEISS; discontinuities have shallow dips (20°-30°), very close to moderately closely spaced (0.1-1.3 ft) [VASSALBORO FORMATION]. Rock Mass Quality = Fair Rock Core Rate (min:sec) 10.5-11.5 ft (3:05) 11.5-12.5 ft (2:27) 12.5-13.5 ft (2:48) 13.5-14.5 ft (2:28) 14.5-15.5 ft (2:50) 96% Recovery	q_p = 1141 ksf (132.6-132.9 ft)
15	R2	60/53	15.50 - 20.50	RQD = 70%							
20								123.9			
										R2, Top 3.2 ft: Grey, medium-grained, strongly foliated, fresh (W1), very strong (R5), SCHIST; discontinuities have shallow dips (20°-30°), very close to closely spaced (0.1-1.0 ft) [VASSALBORO FORMATION].	
25										R2, Bottom 1.2 ft: White and black, coarse-grained, moderately	

Remarks:

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Boring No.: BB-FDR-104

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: I-295 Merrill Road Bridge Replacement #5720 (Exit 20) Location: Freeport, Maine	Boring No.: BB-FDR-104 WIN: 023627.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	144.4	**Auger ID/OD:**	4 in OD Solid Stem
Operator:	Mike Porter	**Datum:**	NAD83 (2011) Maine 2000 West	**Sampler:**	Standard Split Spoon
Logged By:	Shiv Bhardwaj / Karen Roth	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	12/13/19 (13:47);12/16/19 (12:49)	**Drilling Method:**	Solid Stem Auger / Cased Wash	**Core Barrel:**	1-7/8 in - NQ
Boring Location:	N: 368053.2, E: 1051377.0	**Casing ID/OD:**	4 in/4.5 in	**Water Level*:**	0.7 ft on 12/16/19 at 10:39
Hammer Efficiency Factor:	0.914	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25										foliated, fresh (W1), very strong (R5), GNEISS; with discontinuities low angle (20°- 30°), very close to closely spaced (0.1-1.0 ft) [VASSALBORO FORMATION]. Rock Mass Quality = Fair Rock Core Rate (min:sec) 15.5-16.5 ft (3:04) 16.5-17.5 ft (3:02) 17.5-18.5 ft (2:57) 18.5-19.5 ft (2:28) 19.5-20.5 ft (2:29) 88% Recovery 20.5 **Bottom of Exploration at 20.5 feet below ground surface.** Boring backfilled with cuttings to 9.5 ft bgs then with gravel and bentonite chips to ground surface.	
30											
35											
40											
45											
50											

Remarks:

1. Hammer Efficiency Factor provided by New England Boring Contractors and taken from "SPT Energy Testing - ATV Mobile Drill Rig No. 20 Mobile Hammer NEBC-1" by GZA GeoEnvironmental, dated 7/12/19. 2. As-drilled boring locations and ground surface elevations were provided by MaineDOT.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-295 Merrill Road Bridge Replacement #5720 (Exit 20)
Location: Freeport, Maine

Boring No.: BB-FDR-105
WIN: 023627.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	143.4	**Auger ID/OD:**	4 in OD Solid Stem
Operator:	Mike Porter	**Datum:**	NAD83 (2011) Maine 2000 West	**Sampler:**	Standard Split Spoon
Logged By:	Karen Roth	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	12/19/19 (8:09) ; 12/19/19 (11:53)	**Drilling Method:**	Solid Stem Auger / Cased Wash	**Core Barrel:**	1-7/8 in - NQ
Boring Location:	N: 368127.6, E:1051255.7	**Casing ID/OD:**	4 in/4.5 in; 3 in/3.5 in	**Water Level*:**	3.9 ft on 12/19/19 at 12:26
Hammer Efficiency Factor:	0.914	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample Attempt
V = Field Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Field Vane Shear Test Attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)
0	1D	24/15	0.00 - 2.00	1/2/6/7	8	12	SSA	142.8
								139.4
5	2D	18/15	5.00 - 6.50	8/53/112(6")			29	
	MV			Would Not Push			59	
							76	
							88	
							107	
10	3D	24/17	10.00 - 12.00	2/4/5/4	9	14	OPEN	
	MV			Would Not Push				
	MV			Would Not Push				
15	4D	24/11	15.00 - 17.00	WOH(24")	- -			
								125.4
	R1	60/56	18.50 - 23.50	RQD = 78%			NQ	
20								
	R2	60/60	23.50 - 28.50	RQD = 66%				
25								

Visual Description and Remarks

1DA, Top 7 in: Medium brown, damp (frozen), stiff, SILT, little sand, trace organics (grass roots), slightly plastic (TOPSOIL).
0.6
1DB, Bottom 8 in: Light brown, moist, medium dense, fine to coarse SAND, trace fine gravel, trace silt, poorly-graded (FILL).
4.0
Auger cuttings 4-5 ft bgs are grey CLAY, slightly to moderately plastic.
Driller notes hole is caving at 5 ft bgs.

2DA, Top 9 in: Light brown, wet, very dense, fine to coarse SAND, trace fine gravel, well-graded (GLACIOMARINE).
2DB, Bottom 6 in: Alternating layers 1-2" thick of greyish brown, moist, hard, CLAY, moderately plastic; and light brown, wet, very dense, fine to coarse SAND, trace fine gravel, well-graded (GLACIOMARINE). q_p = 6.5 ksf (Pocket Penetrometer).
Failed 55x110 mm vane, would not push to 6 ft bgs.

Clay and sand in wash water 8-10 ft bgs.

3DA, Top 13 in: Grey, wet, stiff, CLAY, slightly plastic (GLACIOMARINE). q_p = 3.0 ksf (Pocket Penetrometer)
3DB, Bottom 4 in: Grey, wet, stiff, SILT, some fine sand (GLACIOMARINE).
Failed 55x110 mm vane, would not push to 11 ft bgs.
Failed 55x110 mm vane, would not push to 13 ft bgs.
Grey, wet, very soft, CLAY, trace fine gravel in top 2 in, trace fine sand in top 2 in, moderately plastic (GLACIOMARINE). q_p = < 1.0 ksf (Pocket Penetrometer), T_v = 300 psf
18.0
Rig starts chattering at 18 ft bgs; driller notes hard material.
Top of Bedrock at Elev. 125.4 ft.
R1: Grey, medium-grained, moderately foliated, slightly weathered (top 1.4 ft) to fresh (bottom 3.3 ft) (W1-W2), very strong (R5), biotite SCHIST; discontinuities shallow to steeply dipping (20°-80°), very close to closely spaced (0.1-1.0 ft), slight reddish staining on discontinuity surfaces in top 1.4 ft [VASSALBORO FORMATION].
Rock Mass Quality = Good
Rock Core Rate (min:sec)
18.5-19.5 ft (5:10)
19.5-20.5 ft (2:28)
20.5-21.5 ft (3:20)

Laboratory Testing Results/ AASHTO and Unified Class.

1DB:
GTX #539972
WC = 12.6%
Fines = 3.7%
A-1-b (0), SP

2DB:
GTX #539964
WC = 23%
LL = 35
PL = 19
PI = 16
LI = 0.3
A-6, CL

3DB:
GTX #539977
WC = 22.2%
Fines = 70.9%
A-4, ML

GTX #539994
WC = 30%
LL = 27
PL = 16
PI = 11
LI = 1.3
A-6, CL

q_p = 2279 ksf
(119.5-119.9

Remarks:

1. Hammer Efficiency Factor provided by New England Boring Contractors and taken from "SPT Energy Testing - ATV Mobile Drill Rig No. 20 Mobile Hammer NEBC-1" by GZA GeoEnvironmental, dated 7/12/19. 2. As-drilled boring locations and ground surface elevations were provided by MaineDOT.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** I-295 Merrill Road Bridge Replacement #5720 (Exit 20) **Location:** Freeport, Maine	**Boring No.:** BB-FDR-105 **WIN:** 023627.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	143.4	**Auger ID/OD:**	4 in OD Solid Stem
Operator:	Mike Porter	**Datum:**	NAD83 (2011) Maine 2000 West	**Sampler:**	Standard Split Spoon
Logged By:	Karen Roth	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	12/19/19 (8:09) ; 12/19/19 (11:53)	**Drilling Method:**	Solid Stem Auger / Cased Wash	**Core Barrel:**	1-7/8 in - NQ
Boring Location:	N: 368127.6, E:1051255.7	**Casing ID/OD:**	4 in/4.5 in; 3 in/3.5 in	**Water Level*:**	3.9 ft on 12/19/19 at 12:26
Hammer Efficiency Factor:	0.914	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample Attempt
V = Field Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Field Vane Shear Test Attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140 lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25										21.5-22.5 ft (3:00) 22.5-23.5 ft (2:49) 94% Recovery R2: Grey, medium-grained, moderately foliated, fresh (W1), very strong (R5), biotite SCHIST; discontinuities shallow to steeply dipping (10°-60°) with vertical discontinuity at 27.9-28.3 ft bgs, very close to closely spaced (0.1- 0.8 ft) [VASSALBORO FORMATION]. Rock Mass Quality = Fair Rock Core Rate (min:sec) 23.5-24.5 ft (1:26) 24.5-25.5 ft (1:19) 25.5-26.5 ft (2:01) 26.5-28.5 ft (3:23) 100% Recovery	ft)
								114.9		28.5	
30										**Bottom of Exploration at 28.5 feet below ground surface.** Boring backfilled with cuttings to 3.5 ft bgs then with gravel and bentonite chips to ground surface.	
35											
40											
45											
50											

Remarks:

1. Hammer Efficiency Factor provided by New England Boring Contractors and taken from "SPT Energy Testing - ATV Mobile Drill Rig No. 20 Mobile Hammer NEBC-1" by GZA GeoEnvironmental, dated 7/12/19. 2. As-drilled boring locations and ground surface elevations were provided by MaineDOT.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-295 Merrill Road Bridge Replacement #5720 (Exit 20)
Location: Freeport, Maine

Boring No.: BB-FDR-106
WIN: 023627.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	147.6	**Auger ID/OD:**	4 in OD Solid Stem
Operator:	Mike Porter	**Datum:**	NAD83 (2011) Maine 2000 West	**Sampler:**	Standard Split Spoon
Logged By:	Karen Roth	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	12/17/19 (8:06) ; 12/17/19 (9:46)	**Drilling Method:**	Solid Stem Auger / Cased Wash	**Core Barrel:**	1-7/8 in - NQ
Boring Location:	N: 368185.1, E: 1051146.0	**Casing ID/OD:**	4 in /4.5 in	**Water Level*:**	3.8 ft on 12/17/19 at 10:14
Hammer Efficiency Factor:	0.914	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample Attempt
V = Field Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Field Vane Shear Test Attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/16	0.00 - 2.00	WOH/2/4/16	6	9	SSA			Medium brown, damp (frozen), stiff, fine to coarse Sandy SILT, trace fine gravel, trace organics (grass roots), non-plastic (TOPSOIL).	1D: GTX #539973 Fines = 50.6% A-4, ML
								145.6		2.0	
										Auger cuttings 2-5 ft bgs are brown, damp, SILT, little sand, trace gravel, slightly plastic.	
5	2D	21/20	5.00 - 6.75	1/4/6/50(3")	10	15	OPEN	142.3		2DA, Top 3 in: Greyish brown, moist, stiff, SILT, little sand, slightly plastic (GLACIOMARINE). q_p = 3.0 ksf (Pocket Penetrometer) 5.3	2DB: GTX #539974 WC = 16.7% Fines = 35.1% A-4 (0), SM
	R1	60/58	6.80 - 11.80	RQD = 96%			NQ	140.9		2DB, Bottom 17 in: Greyish brown, wet, medium dense, Silty fine to coarse SAND, trace fine gravel, well-graded (SAND AND GRAVEL). 6.7	
										Driller notes 0.3 ft of water in bottom of hole upon refusal. Top of Bedrock at Elev. 140.9 ft. R1: Black and light grey/white, coarse grained, strongly foliated, fresh (W1), strong (R4), GNEISS; discontinuities have shallow dips (10°-20°) and parallel to foliation, close to moderately closely spaced (0.5-2.5 ft) [VASSALBORO FORMATION]. Rock Mass Quality = Excellent Rock Core Rate (min:sec) 6.8-7.8 ft (6:07) 7.8-8.8 ft (3:58) 8.8-9.8 ft (3:39) 9.8-10.8 ft (3:47) 10.8-11.8 ft (4:05) 96% Recovery	q_p = 1155 ksf (140.5-140.8 ft)
10											
	R2	60/58	11.80 - 16.80	RQD = 92%							
15								130.8		R2: Black and light grey/white, coarse grained, strongly foliated, fresh (W1), very strong (R5), GNEISS; discontinuities horizontal to shallow (0°-10°) and parallel to foliation, close to moderately closely spaced (0.2-2.3 ft) [VASSALBORO FORMATION]. Rock Core Rate (min:sec) 11.8-12.8 ft (4:20) 12.8-13.8 ft (4:28) 13.8-14.8 ft (4:12) 14.8-15.8 ft (4:01) 15.8-16.8 ft (4:07) 96% Recovery	
20										16.8	
										Bottom of Exploration at 16.8 feet below ground surface. Boring backfilled with rock cuttings to 14 ft bgs, gravel to 6 ft bgs, soil cuttings to 2.3 ft bgs, and bentonite chips to ground surface.	
25											

Remarks:

1. Hammer Efficiency Factor provided by New England Boring Contractors and taken from "SPT Energy Testing - ATV Mobile Drill Rig No. 20 Mobile Hammer NEBC-1" by GZA GeoEnvironmental, dated 7/12/19. 2. As-drilled boring locations and ground surface elevations were provided by MaineDOT.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

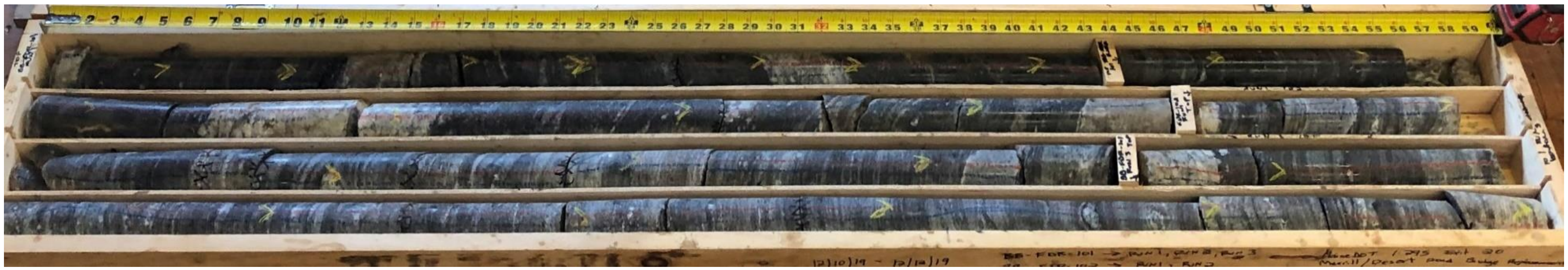
Boring No.: BB-FDR-106

APPENDIX B

Rock Core Photos

APPENDIX B
Rock Core Photos
I-295 MERRILL ROAD BRIDGE REPLACEMENT #5720 (EXIT 20)
FREEPORT, MAINE
MAINEDOT WIN 023627.00

Boring	Date Cored	Run	Depth Below Surface feet	Recovery		RQD	
				Feet	%	Feet	%
BB-FDR-101	12/10/2019	R1	36.6 - 41.6	3.6 / 5.0	73	3.0 / 5.0	60
	12/10/2019	R2	41.6 - 45.9	4.9 / 4.3	114	4.9 / 4.3	100
	12/10/2019	R3	45.9 - 46.9	1.2 / 1.0	117	1.2 / 1.0	100
BB-FDR-102	12/12/2019	R1	16.1 - 21.1	4.5 / 5.0	90	4.3 / 5.0	87
	12/12/2019	R2	21.1 - 26.1	5.0 / 5.0	100	5.0 / 5.0	100



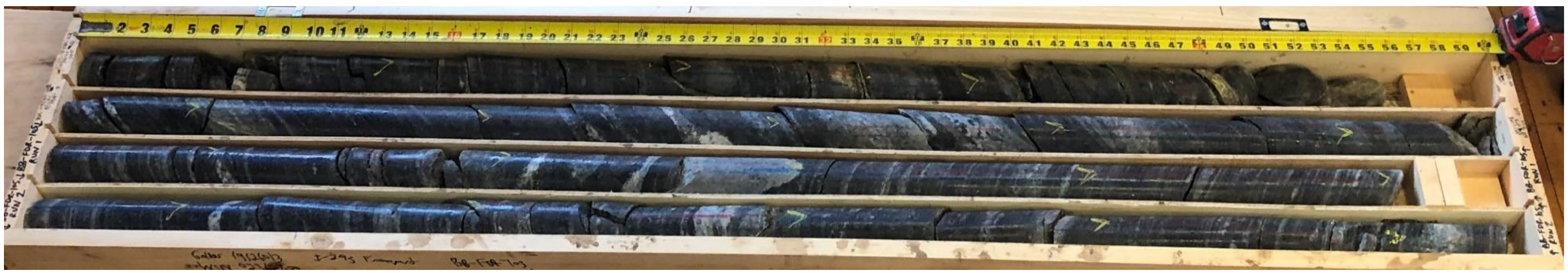
From top to bottom of photo:
Row 1 = BB-FDR-101 Run 1: 36.6 - 41.6 ft bgs and BB-FDR-101 Run 2: 41.6 - 42.8 ft bgs
Row 2 = BB-FDR-101 Run 2: 42.8 - 45.6 ft bgs and BB-FDR-102 Run 1: 16.1 - 17.0 ft bgs
Row 3 = BB-FDR-102 Run 1: 17.0 - 21.1 ft bgs and BB-FDR-101 Run 3: 45.9 - 46.9 ft bgs
Row 4 = BB-FDR-102 Run 2: 21.1 - 26.1 ft bgs

Note:
BB-FDR-101 Run 1 recovery picked up in BB-FDR-101 Run 2; Excess recovery was included in Run 2 for RQD calculation
BB-FDR-101 Run 2 recovery picked up in BB-FDR-101 Run 3; Excess recovery was included in Run 3 for RQD calculation
Rock core was wetted with water for photographs.

GOLDER

APPENDIX B
Rock Core Photos
I-295 MERRILL ROAD BRIDGE REPLACEMENT #5720 (EXIT 20)
FREEPORT, MAINE
MAINEDOT WIN 023627.00

Boring	Date Cored	Run	Depth Below Surface feet	Recovery		RQD	
				Feet	%	Feet	%
BB-FDR-103	12/13/2019	R1	11.0 - 16.0	0.0 / 5.0	0	0.0 / 5.0	0
	12/13/2019	R2	21.0 - 25.6	4.4 / 4.6	96	3.4 / 4.6	74
	12/13/2019	R3	25.6 - 30.8	5.1 / 5.2	98	4.1 / 5.2	79
BB-FDR-105	12/19/2019	R1	18.5 - 23.5	4.7 / 5.0	94	3.9 / 5.0	78
	12/19/2019	R2	23.5 - 28.5	5.0 / 5.0	100	3.3 / 5.0	66



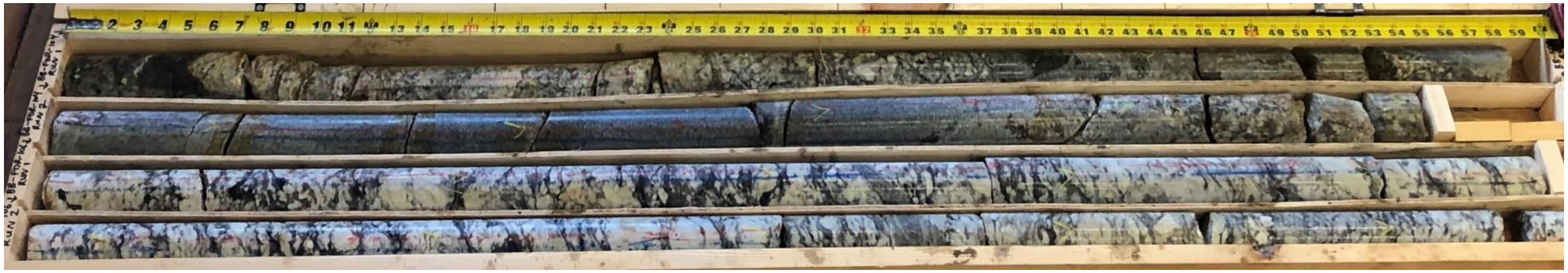
From top to bottom of photo:
Row 1 = BB-FDR-103 Run 2: 21.0 - 25.6 ft bgs
Row 2 = BB-FDR-103 Run 3: 25.6 - 30.8 ft bgs
Row 3 = BB-FDR-105 Run 1: 18.5 - 23.5 ft bgs
Row 4 = BB-FDR-105 Run 2: 23.5 - 28.5 ft bgs

Note:
BB-FDR-103 Run 1 recovered cobbles; Sample not collected. BB-FDR-103 Run 1 did not occur in bedrock.
Rock core was wetted with water for photographs.

GOLDER

APPENDIX B
Rock Core Photos
I-295 MERRILL ROAD BRIDGE REPLACEMENT #5720 (EXIT 20)
FREEPORT, MAINE
MAINEDOT WIN 023627.00

Boring	Date Cored	Run	Depth Below Surface feet			Recovery				RQD			
						Feet			%	Feet			%
BB-FDR-104	12/16/2019	R1	10.5	-	15.5	4.8	/	5.0	96	3.6	/	5.0	72
	12/16/2019	R2	15.5	-	20.5	4.4	/	5.0	88	3.5	/	5.0	70
BB-FDR-106	12/17/2019	R1	6.8	-	11.8	4.8	/	5.0	96	4.8	/	5.0	96
	12/17/2019	R2	11.8	-	16.8	4.8	/	5.0	96	4.6	/	5.0	92



From top to bottom of photo:
Row 1 = BB-FDR-104 Run 1: 10.5 - 15.5 ft bgs
Row 2 = BB-FDR-104 Run 2: 15.5 - 20.5 ft bgs
Row 3 = BB-FDR-106 Run 1: 6.8 - 11.8 ft bgs
Row 4 = BB-FDR-106 Run 2: 11.8 - 16.8 ft bgs

Note:
Rock core was wetted with water for photographs.

GOLDER

APPENDIX C

Laboratory Soil Test Results

MOISTURE CONTENT OF SOIL AND ROCK

GeoTesting EXPRESS

Client:	Golder Associates		
Project:	Merrill Rd Bridge Replace I-295 Ex 20		
Location:	Freeport, ME	Project No:	GTX-311185
Boring ID: ---	Sample Type: ---	Tested By:	GA
Sample ID: ---	Test Date: 02/05/20	Checked By:	jsc
Depth : ---	Test Id: 539987		

Moisture Content of Soil and Rock - ASTM D2216

Boring ID	Sample ID	Depth	Description	Moisture Content,%
BB-FDR-101	3D	11.6-13.6 ft	Moist, dark yellowish brown sand with silt and gravel	15.0
BB-FDR-101	5D	21-23 ft	Moist, yellowish brown silty sand	16.1
BB-FDR-102	1DA	0-2 ft	Moist, yellowish brown silty sand	14.9
BB-FDR-103	1D	0.5-2.5 ft	Moist, yellowish brown gravel with silt and sand	2.1
BB-FDR-103	2D	5-7 ft	Moist, yellowish brown sand with silt	2.2
BB-FDR-103	3D	10-12 ft	Moist, brown clayey sand	14.2
BB-FDR-104	1D	0-2 ft	Moist, dark brown clay with sand	21.7
BB-FDR-105	1DB	0-2 ft	Moist, yellowish brown sand	12.6
BB-FDR-105	3DB	10-12 ft	Moist, dark gray silt with sand	22.2
BB-FDR-106	2DB	5-7 ft	Moist, olive brown clayey sand	16.7

Notes: Temperature of Drying : 110º Celsius

MOISTURE, ASH AND ORGANIC MATTER

Client:	Golder Associates				
Project:	Merrill Rd Bridge Replace I-295 Ex 20				
Location:	Freeport, ME			Project No:	GTX-311185
Boring ID:	BB-FDR-103	Sample Type:	jar	Tested By:	GA
Sample ID:	4DB	Test Date:	02/04/20	Checked By:	jsc
Depth :	16.5-18.5 ft	Test Id:	539988		

Test Comment: ---
Visual Description: Moist, dark olive gray silty sand
Sample Comment: ---

Moisture, Ash, and Organic Matter - ASTM D2974

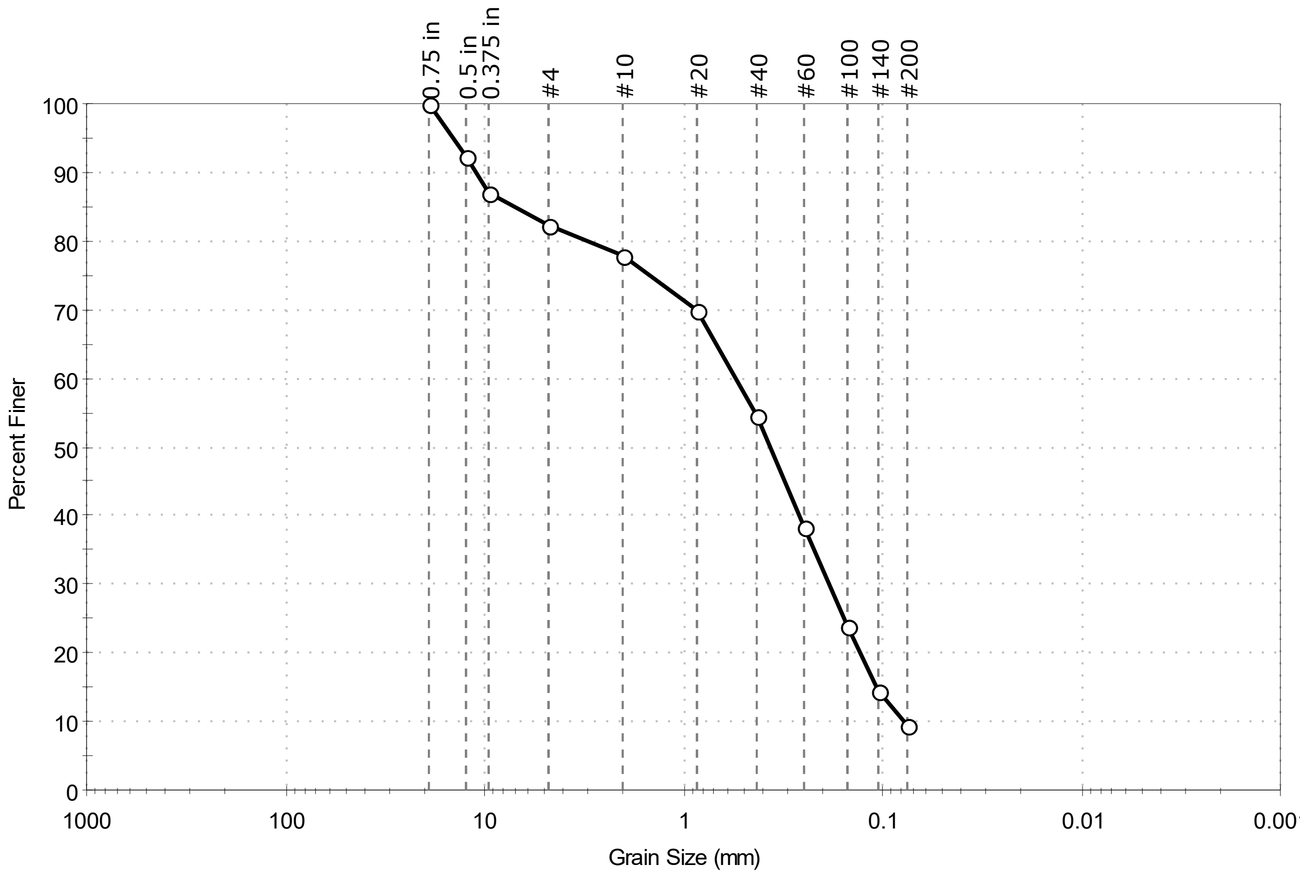
Boring ID	**Sample ID**	**Depth**	**Description**	**Moisture Content,%**	**Ash Content,%**	**Organic Matter,%**
BB-FDR-103	4DB	16.5-18.5 ft	Moist, dark olive gray silty sand	17	97.5	2.5

Notes: Moisture content determined by Method A and reported as a percentage of oven-dried mass; dried to a constant mass at temperature of 105º C
Ash content and organic matter determined by Method C; dried to constant mass at temperature 440º C

PARTICLE SIZE ANALYSIS

Client:	Golder Associates		
Project:	Merrill Rd Bridge Replace I-295 Ex 20		
Location:	Freeport, ME	Project No:	GTX-311185
Boring ID: BB-FDR-101	Sample Type: jar	Tested By:	GA
Sample ID: 3D	Test Date: 02/05/20	Checked By:	jsc
Depth : 11.6-13.6 ft	Test Id: 539965		
Test Comment:	---		
Visual Description:	Moist, dark yellowish brown sand with silt and gravel		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	17.7	72.8	9.5

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	92		
0.375 in	9.50	87		
#4	4.75	82		
#10	2.00	78		
#20	0.85	70		
#40	0.42	55		
#60	0.25	38		
#100	0.15	24		
#140	0.11	14		
#200	0.075	9.5		

Coefficients

D_{85} =7.0706 mm D_{30} =0.1864 mm

D_{60} =0.5424 mm D_{15} =0.1085 mm

D_{50} =0.3655 mm D_{10} =0.0775 mm

C_u =6.999 C_c =0.827

Classification

ASTM N/A

AASHTO Fine Sand (A-3 (1))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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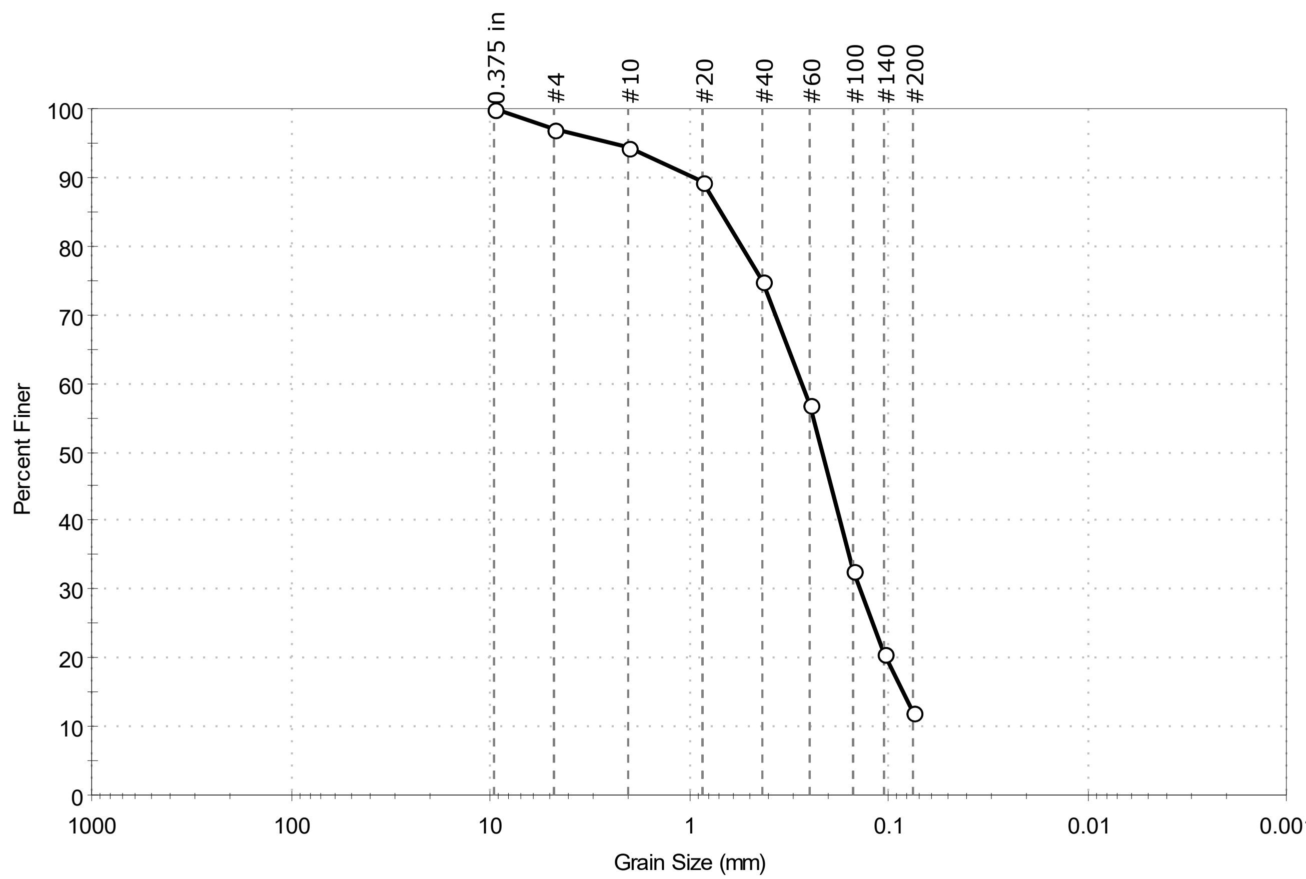
Client:	Golder Associates		
Project:	Merrill Rd Bridge Replace I-295 Ex 20		
Location:	Freeport, ME	Project No:	GTX-311185
Boring ID: BB-FDR-101	Sample Type: jar	Tested By:	GA
Sample ID: 5D	Test Date: 02/05/20	Checked By:	jsc
Depth : 21-23 ft	Test Id: 539966		
Test Comment:	---		
Visual Description:	Moist, yellowish brown silty sand		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	2.9	84.9	12.2

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.375 in	9.50	100		
#4	4.75	97		
#10	2.00	94		
#20	0.85	89		
#40	0.42	75		
#60	0.25	57		
#100	0.15	33		
#140	0.11	21		
#200	0.075	12		

Coefficients

D_{85} =0.6877 mm　D_{30} =0.1383 mm

D_{60} =0.2735 mm　D_{15} =0.0841 mm

D_{50} =0.2157 mm　D_{10} =N/A

C_u =N/A　C_c =N/A

Classification

ASTM　N/A

AASHTO　Silty Gravel and Sand (A-2-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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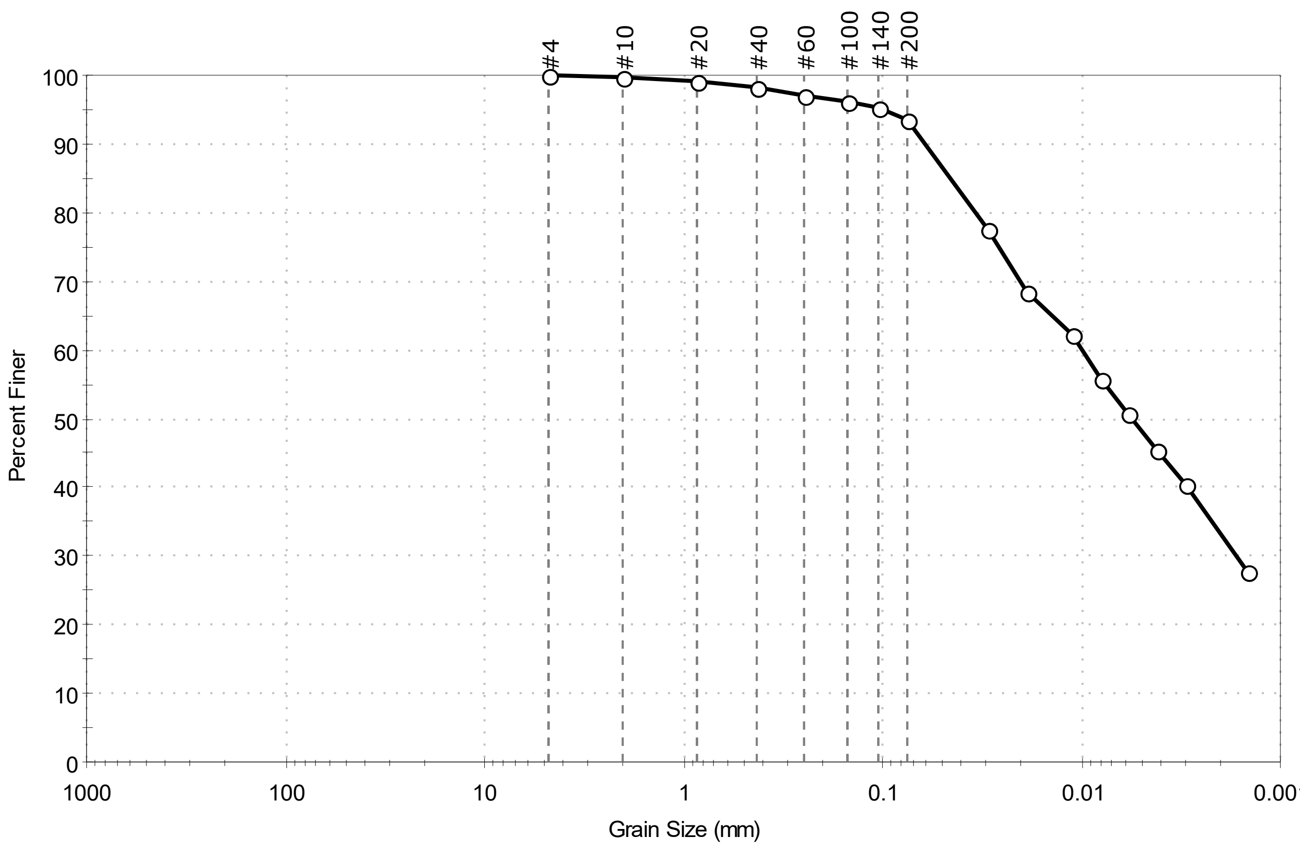
Client:	Golder Associates			
Project:	Merrill Rd Bridge Replace I-295 Ex 20			
Location:	Freeport, ME		Project No:	GTX-311185
Boring ID:	BB-FDR-101	Sample Type: jar	Tested By:	GA
Sample ID:	6DA	Test Date: 02/04/20	Checked By:	jsc
Depth :	26-28 ft	Test Id: 539975		
Test Comment:	---			
Visual Description:	Moist, olive gray clay			
Sample Comment:	---			

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	6.4	93.6

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	100		
#20	0.85	99		
#40	0.42	98		
#60	0.25	97		
#100	0.15	96		
#140	0.11	95		
#200	0.075	94		
Hydrometer	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0296	78		
---	0.0190	68		
---	0.0112	62		
---	0.0081	56		
---	0.0058	51		
---	0.0042	45		
---	0.0030	40		
---	0.0014	28		

Coefficients

D_{85} =0.0453 mm	D_{30} =0.0016 mm
D_{60} =0.0100 mm	D_{15} =N/A
D_{50} =0.0056 mm	D_{10} =N/A
C_u =N/A	C_c =N/A

Classification

ASTM Lean CLAY (CL)

AASHTO Clayey Soils (A-6 (19))

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

Dispersion Device : Apparatus A - Mech Mixer

Dispersion Period : 1 minute

Est. Specific Gravity : 2.65

Separation of Sample: #200 Sieve

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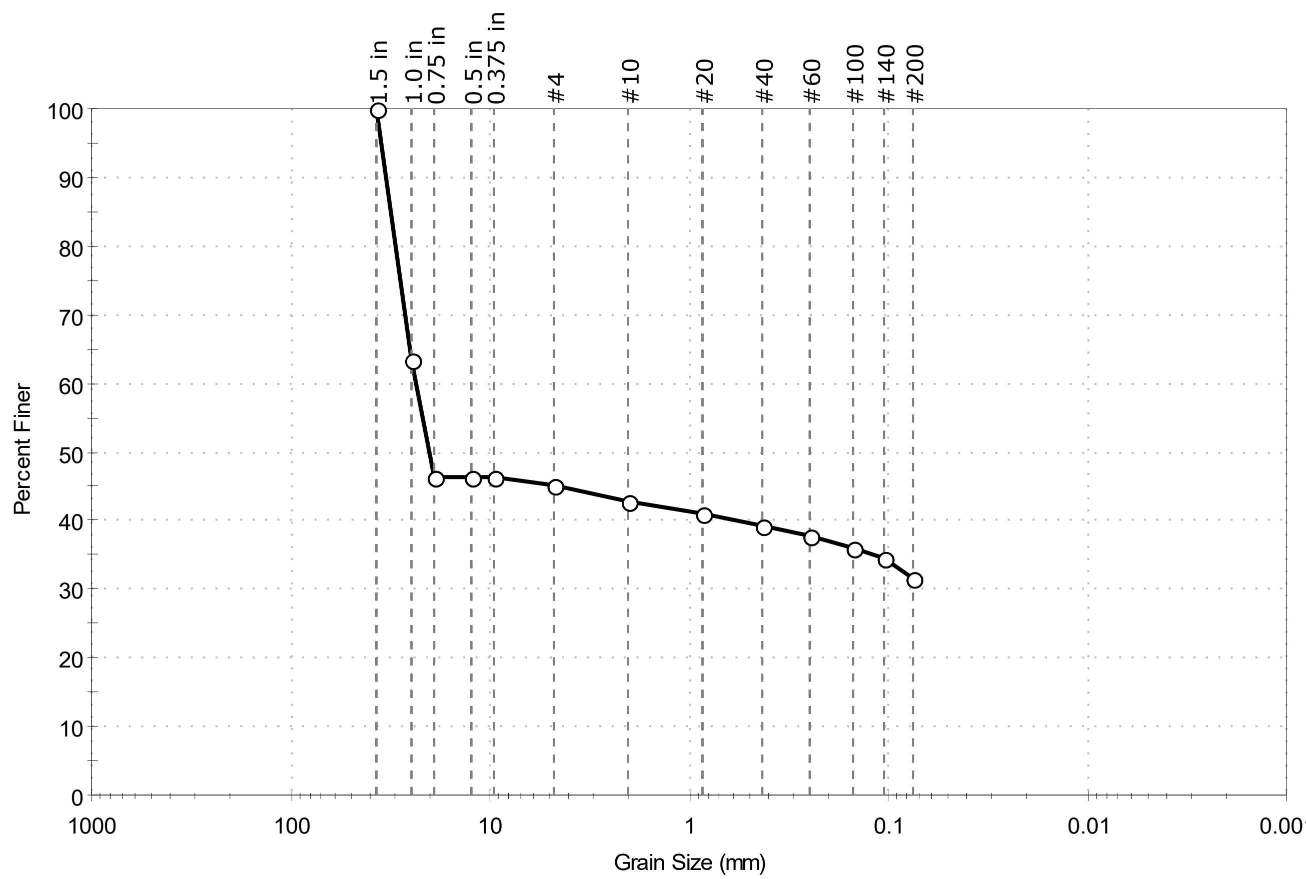
Client:	Golder Associates		
Project:	Merrill Rd Bridge Replace I-295 Ex 20		
Location:	Freeport, ME	Project No:	GTX-311185
Boring ID: BB-FDR-101	Sample Type: jar	Tested By:	GA
Sample ID: 8D	Test Date: 02/04/20	Checked By:	jsc
Depth : 36-38 ft	Test Id: 539967		
Test Comment:	---		
Visual Description:	Moist, dark gray silty clayey gravel		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	55.0	13.5	31.5

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.5 in	37.50	100		
1.0 in	25.00	64		
0.75 in	19.00	46		
0.5 in	12.50	46		
0.375 in	9.50	46		
#4	4.75	45		
#10	2.00	43		
#20	0.85	41		
#40	0.42	39		
#60	0.25	38		
#100	0.15	36		
#140	0.11	34		
#200	0.075	32		

Coefficients

D_{85} =31.7394 mm D_{30} =N/A

D_{60} =23.6293 mm D_{15} =N/A

D_{50} =20.1441 mm D_{10} =N/A

C_u =N/A C_c =N/A

Classification

ASTM   Silty, Clayey GRAVEL (GC-GM)

AASHTO   Silty Gravel and Sand (A-2-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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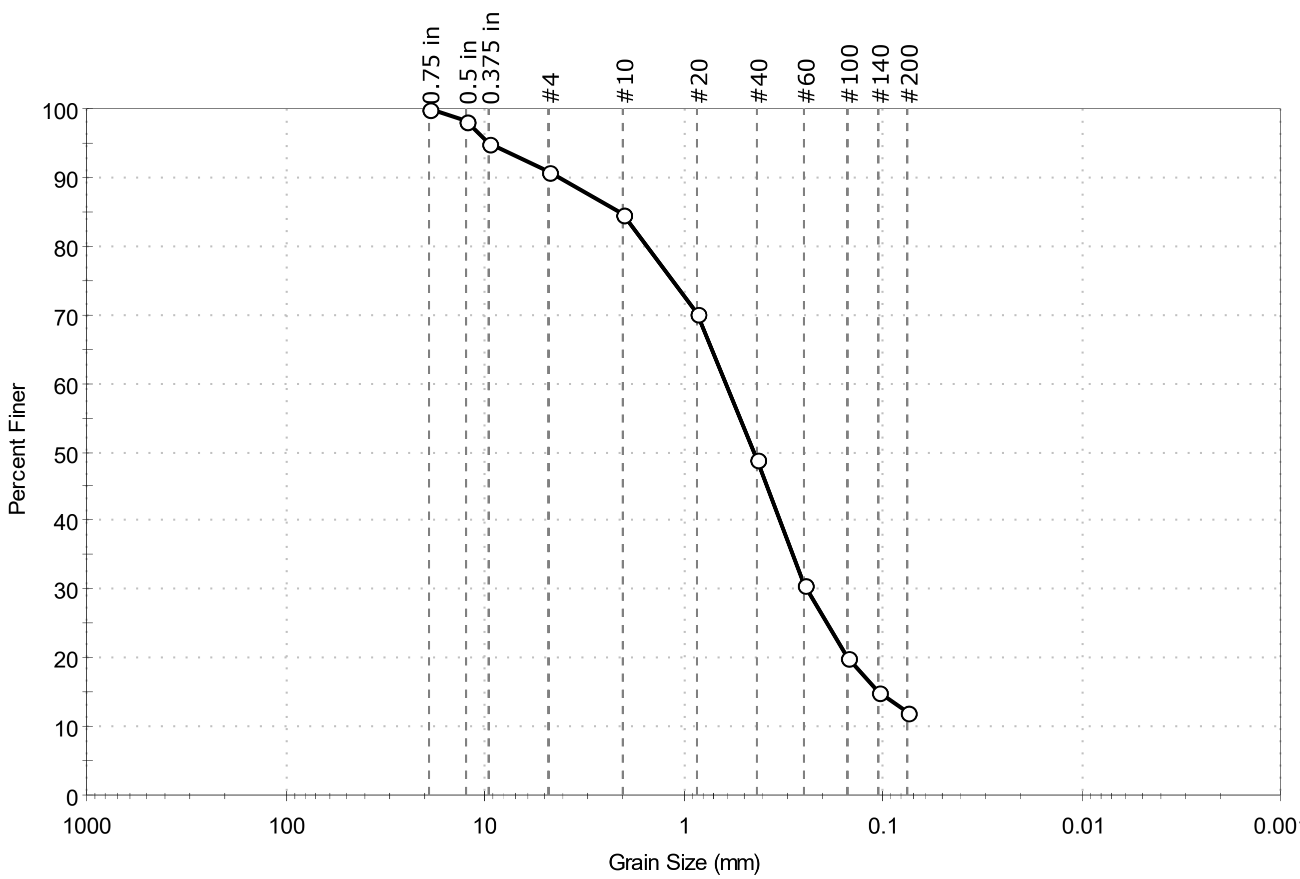
GeoTesting EXPRESS

Client: Golder Associates			
Project: Merrill Rd Bridge Replace I-295 Ex 20			
Location: Freeport, ME		Project No:	GTX-311185
Boring ID: BB-FDR-102	Sample Type: jar	Tested By:	GA
Sample ID: 1DA	Test Date: 02/04/20	Checked By:	jsc
Depth : 0-2 ft	Test Id: 539968		

Test Comment: ---
Visual Description: Moist, yellowish brown silty sand
Sample Comment: ---

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	9.1	78.8	12.1

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	98		
0.375 in	9.50	95		
#4	4.75	91		
#10	2.00	85		
#20	0.85	70		
#40	0.42	49		
#60	0.25	31		
#100	0.15	20		
#140	0.11	15		
#200	0.075	12		

Coefficients

D_{85} =2.1287 mm
D_{30} =0.2411 mm
D_{60} =0.6082 mm
D_{15} =0.1053 mm
D_{50} =0.4383 mm
D_{10} =N/A
C_u =N/A
C_c =N/A

Classification

ASTM Silty SAND (SM)

AASHTO Stone Fragments, Gravel and Sand (A-1-b (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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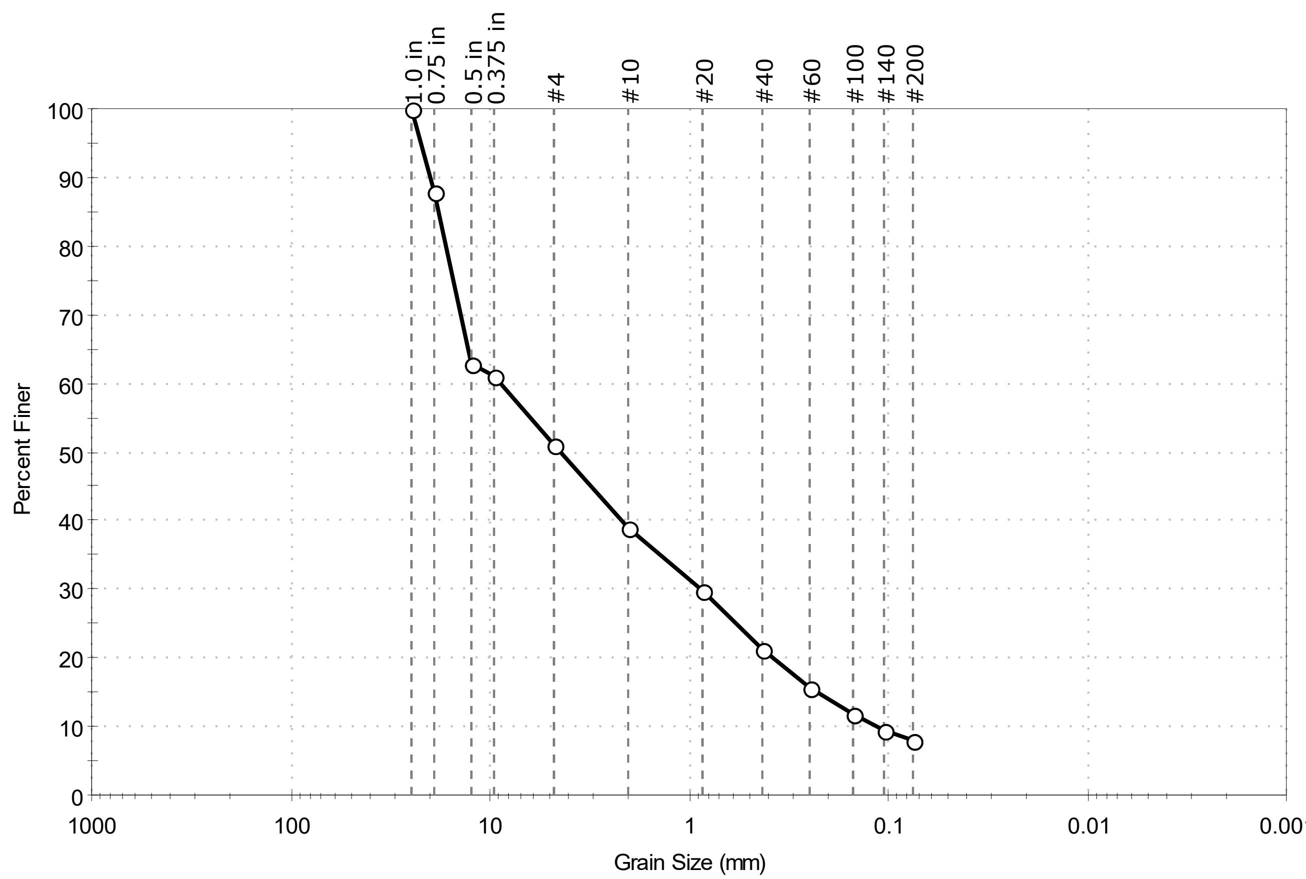
Client:	Golder Associates				
Project:	Merrill Rd Bridge Replace I-295 Ex 20				
Location:	Freeport, ME			Project No:	GTX-311185
Boring ID:	BB-FDR-103	Sample Type:	jar	Tested By:	GA
Sample ID:	1D	Test Date:	02/05/20	Checked By:	jsc
Depth :	0.5-2.5 ft	Test Id:	539969		

Test Comment: ---
Visual Description: Moist, yellowish brown gravel with silt and sand
Sample Comment: ---

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	48.9	43.2	7.9

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.0 in	25.00	100		
0.75 in	19.00	88		
0.5 in	12.50	63		
0.375 in	9.50	61		
#4	4.75	51		
#10	2.00	39		
#20	0.85	30		
#40	0.42	21		
#60	0.25	16		
#100	0.15	12		
#140	0.11	10		
#200	0.075	7.9		

Coefficients

D_{85} =18.1316 mm — D_{30} =0.8772 mm
D_{60} =8.8064 mm — D_{15} =0.2260 mm
D_{50} =4.4017 mm — D_{10} =0.1138 mm
C_u =77.385 — C_c =0.768

Classification

ASTM: Poorly graded GRAVEL with Silt and Sand (GP-GM)

AASHTO: Stone Fragments, Gravel and Sand (A-1-a (1))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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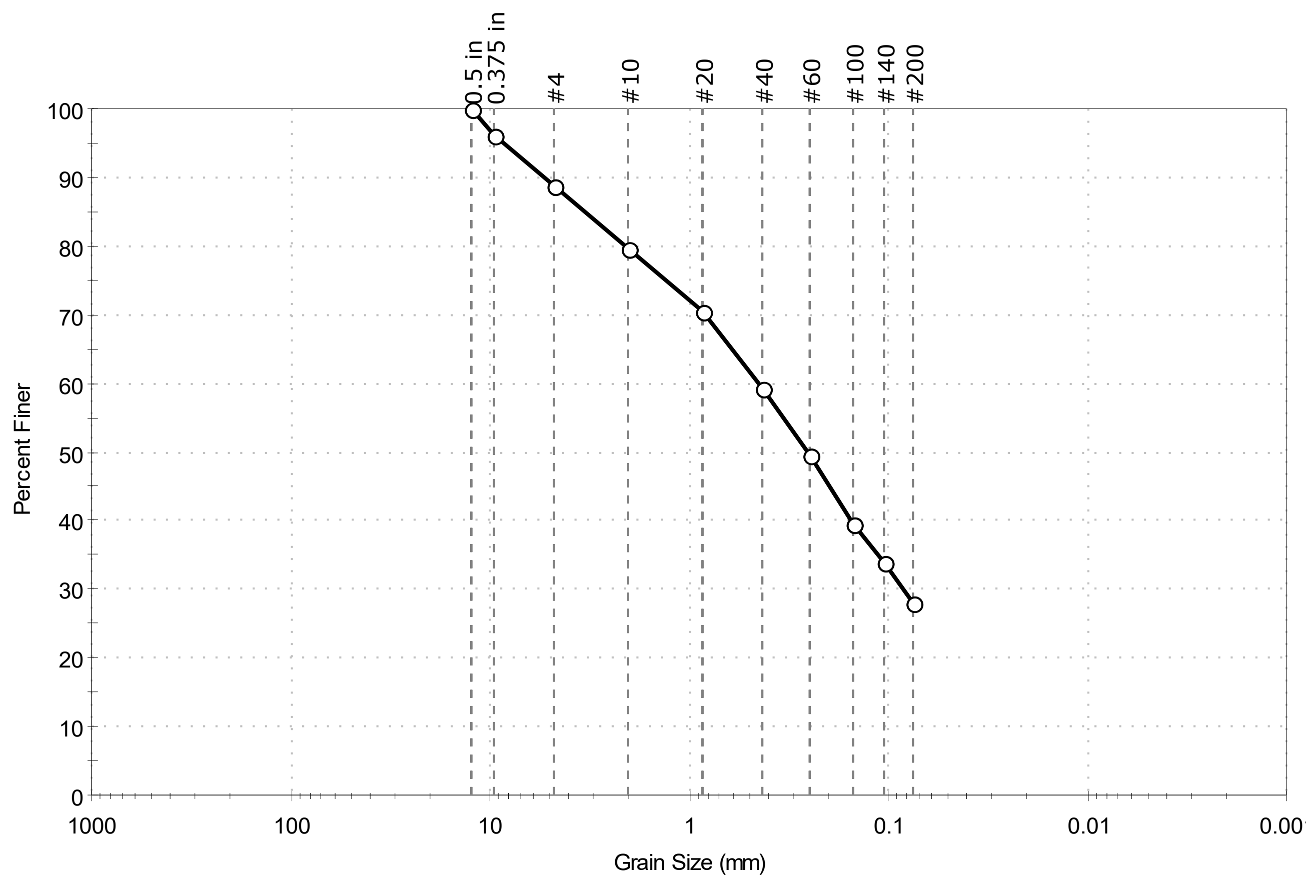
Client:	Golder Associates		
Project:	Merrill Rd Bridge Replace I-295 Ex 20		
Location:	Freeport, ME	Project No:	GTX-311185
Boring ID: BB-FDR-103	Sample Type: jar	Tested By:	GA
Sample ID: 3D	Test Date: 02/04/20	Checked By:	jsc
Depth : 10-12 ft	Test Id: 539970		
Test Comment:	---		
Visual Description:	Moist, brown clayey sand		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	11.2	60.9	27.9

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.5 in	12.50	100		
0.375 in	9.50	96		
#4	4.75	89		
#10	2.00	80		
#20	0.85	71		
#40	0.42	59		
#60	0.25	50		
#100	0.15	40		
#140	0.11	34		
#200	0.075	28		

Coefficients

D_{85} =3.3096 mm D_{30} =0.0845 mm
D_{60} =0.4441 mm D_{15} =N/A
D_{50} =0.2553 mm D_{10} =N/A
C_u =N/A C_c =N/A

Classification

ASTM N/A

AASHTO Silty Gravel and Sand (A-2-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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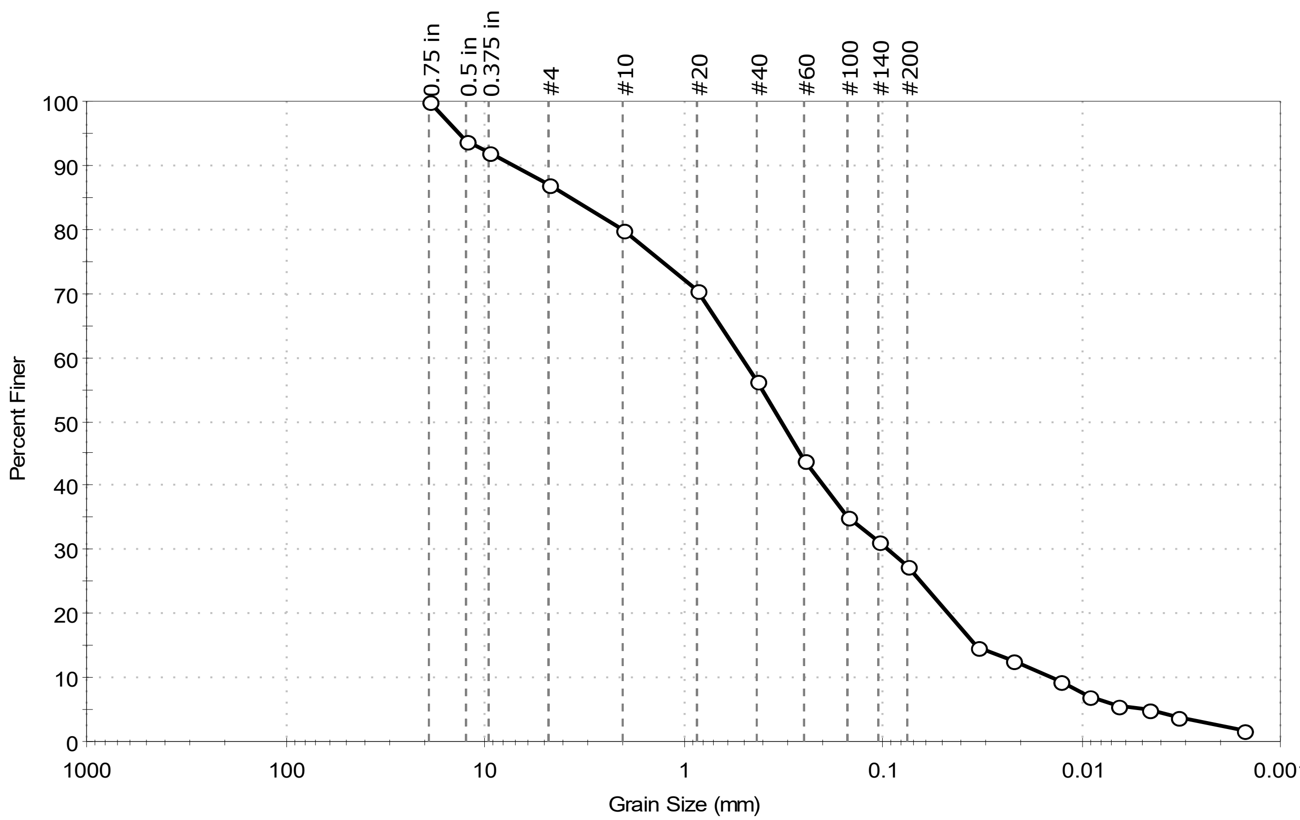
Client:	Golder Associates			
Project:	Merrill Rd Bridge Replace I-295 Ex 20			
Location:	Freeport, ME		Project No:	GTX-311185
Boring ID:	BB-FDR-103	Sample Type: jar	Tested By:	GA
Sample ID:	4DB	Test Date: 02/04/20	Checked By:	jsc
Depth :	16.5-18.5 ft	Test Id: 539976		

Test Comment: ---
Visual Description: Moist, dark olive gray silty sand
Sample Comment: ---

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	13.1	59.4	27.5

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	94		
0.375 in	9.50	92		
#4	4.75	87		
#10	2.00	80		
#20	0.85	70		
#40	0.42	56		
#60	0.25	44		
#100	0.15	35		
#140	0.11	31		
#200	0.075	27		
Hydrometer	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0332	15		
---	0.0223	13		
---	0.0130	10		
---	0.0091	7		
---	0.0066	6		
---	0.0047	5		
---	0.0033	4		
---	0.0016	2		

Coefficients

D_{85} =3.7363 mm	D_{30} =0.0944 mm
D_{60} =0.5102 mm	D_{15} =0.0338 mm
D_{50} =0.3252 mm	D_{10} =0.0140 mm
C_u =36.443	C_c =1.248

Classification

ASTM N/A

AASHTO Silty Gravel and Sand (A-2-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR
Sand/Gravel Hardness : HARD
Dispersion Device : Apparatus A - Mech Mixer
Dispersion Period : 1 minute
Est. Specific Gravity : 2.65
Separation of Sample: #200 Sieve

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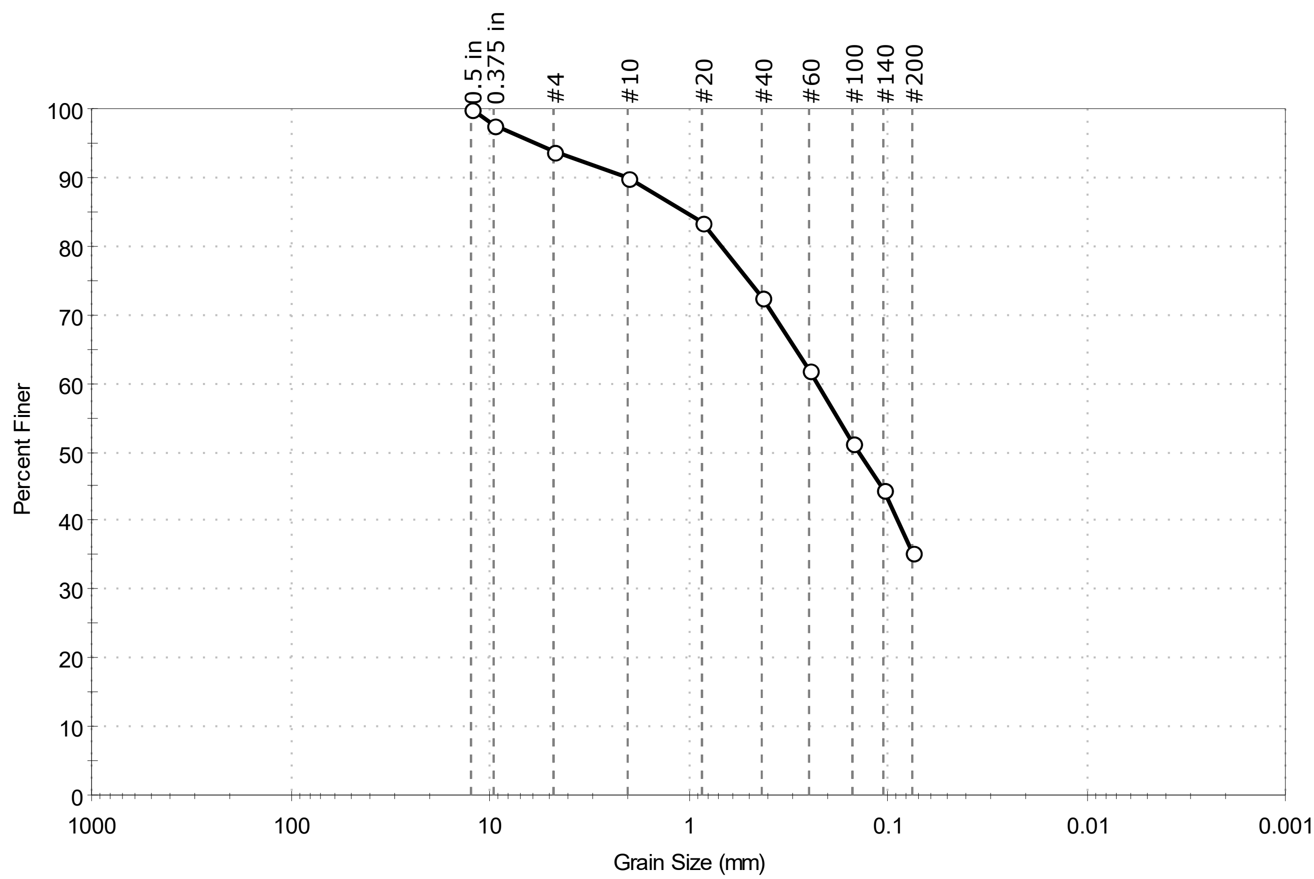
Client:	Golder Associates				
Project:	Merrill Rd Bridge Replace I-295 Ex 20				
Location:	Freeport, ME			Project No:	GTX-311185
Boring ID:	BB-FDR-103	Sample Type:	jar	Tested By:	GA
Sample ID:	5D	Test Date:	02/05/20	Checked By:	jsc
Depth :	20-22 ft	Test Id:	539971		
Test Comment:	---				
Visual Description:	Moist, olive brown silty sand				
Sample Comment:	---				

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	6.1	58.5	35.4

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.5 in	12.50	100		
0.375 in	9.50	98		
#4	4.75	94		
#10	2.00	90		
#20	0.85	83		
#40	0.42	73		
#60	0.25	62		
#100	0.15	51		
#140	0.11	45		
#200	0.075	35		

Coefficients

D_{85} =1.0553 mm D_{30} =N/A

D_{60} =0.2272 mm D_{15} =N/A

D_{50} =0.1392 mm D_{10} =N/A

C_u =N/A C_c =N/A

Classification

ASTM   N/A

AASHTO   Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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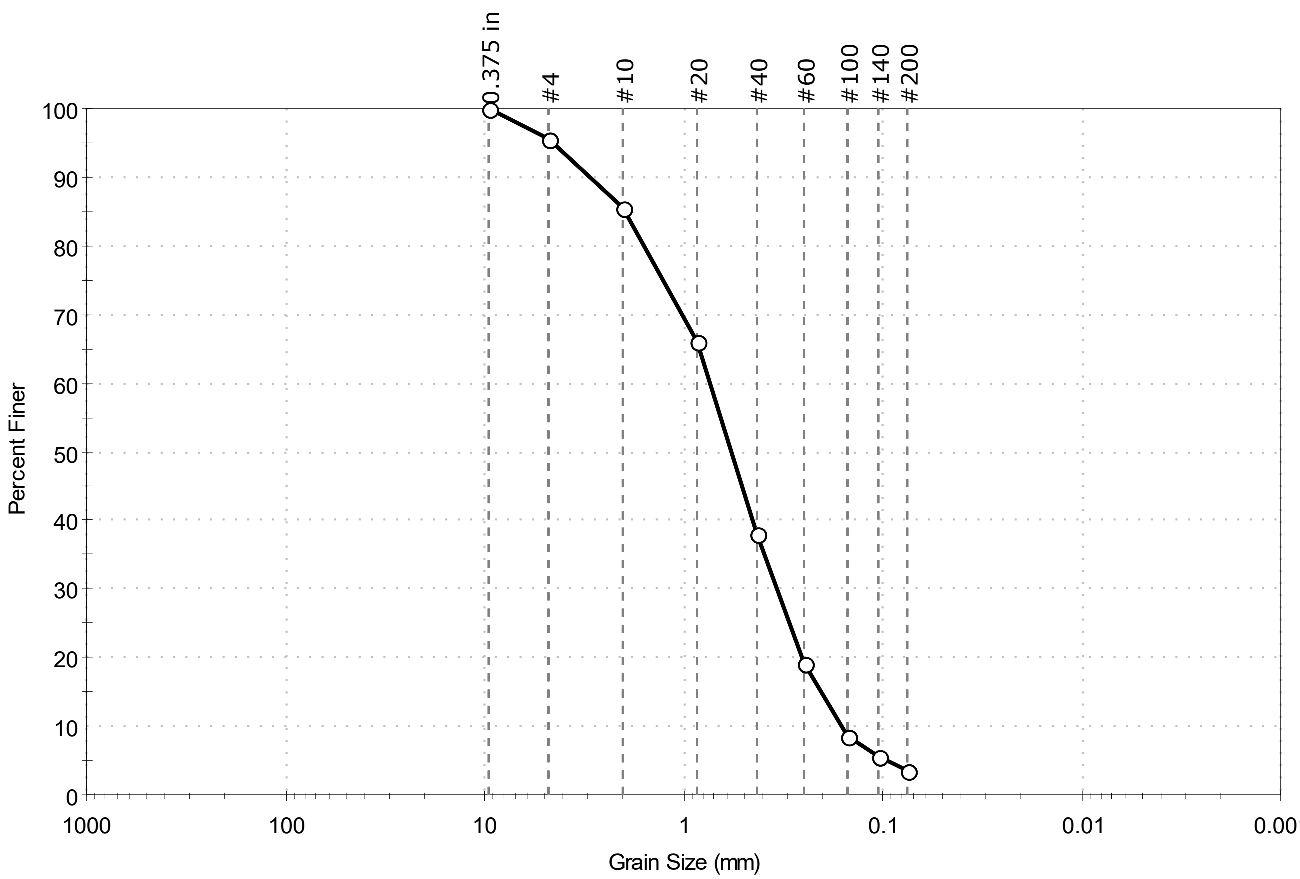
Client:	Golder Associates
Project:	Merrill Rd Bridge Replace I-295 Ex 20
Location:	Freeport, ME
Project No:	GTX-311185
Boring ID:	BB-FDR-105
Sample Type:	jar
Tested By:	GA
Sample ID:	1DB
Test Date:	02/04/20
Checked By:	jsc
Depth :	0-2 ft
Test Id:	539972
Test Comment:	---
Visual Description:	Moist, yellowish brown sand
Sample Comment:	---

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	4.4	91.9	3.7

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.375 in	9.50	100		
#4	4.75	96		
#10	2.00	86		
#20	0.85	66		
#40	0.42	38		
#60	0.25	19		
#100	0.15	8		
#140	0.11	5		
#200	0.075	3.7		

Coefficients

D_{85} =1.9564 mm D_{30} =0.3390 mm

D_{60} =0.7329 mm D_{15} =0.2045 mm

D_{50} =0.5721 mm D_{10} =0.1612 mm

C_u =4.547 C_c =0.973

Classification

ASTM Poorly graded SAND (SP)

AASHTO Stone Fragments, Gravel and Sand (A-1-b (1))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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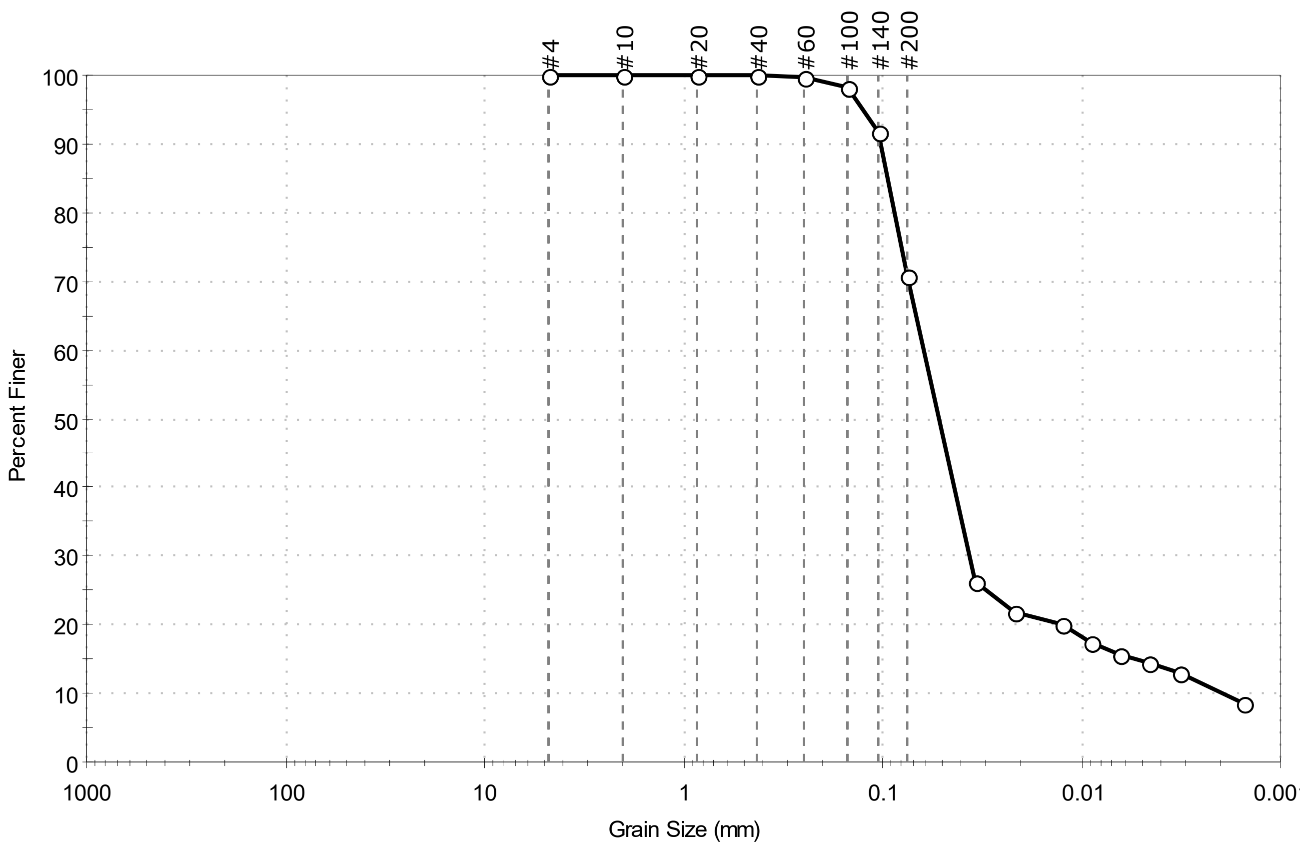
Client:	Golder Associates		
Project:	Merrill Rd Bridge Replace I-295 Ex 20		
Location:	Freeport, ME	Project No:	GTX-311185
Boring ID: BB-FDR-105	Sample Type: jar	Tested By:	GA
Sample ID: 3DB	Test Date: 02/04/20	Checked By:	jsc
Depth : 10-12 ft	Test Id: 539977		

Test Comment: ---
Visual Description: Moist, dark gray silt with sand
Sample Comment: ---

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	29.1	70.9

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	100		
#20	0.85	100		
#40	0.42	100		
#60	0.25	100		
#100	0.15	98		
#140	0.11	92		
#200	0.075	71		
Hydrometer	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0345	26		
---	0.0220	22		
---	0.0127	20		
---	0.0090	18		
---	0.0064	16		
---	0.0046	14		
---	0.0032	13		
---	0.0015	9		

Coefficients

D_{85} =0.0949 mm | D_{30} =0.0367 mm
D_{60} =0.0620 mm | D_{15} =0.0053 mm
D_{50} =0.0521 mm | D_{10} =0.0019 mm
C_u =32.632 | C_c =11.434

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ---
Sand/Gravel Hardness : ---
Dispersion Device : Apparatus A - Mech Mixer
Dispersion Period : 1 minute
Est. Specific Gravity : 2.65
Separation of Sample: #200 Sieve

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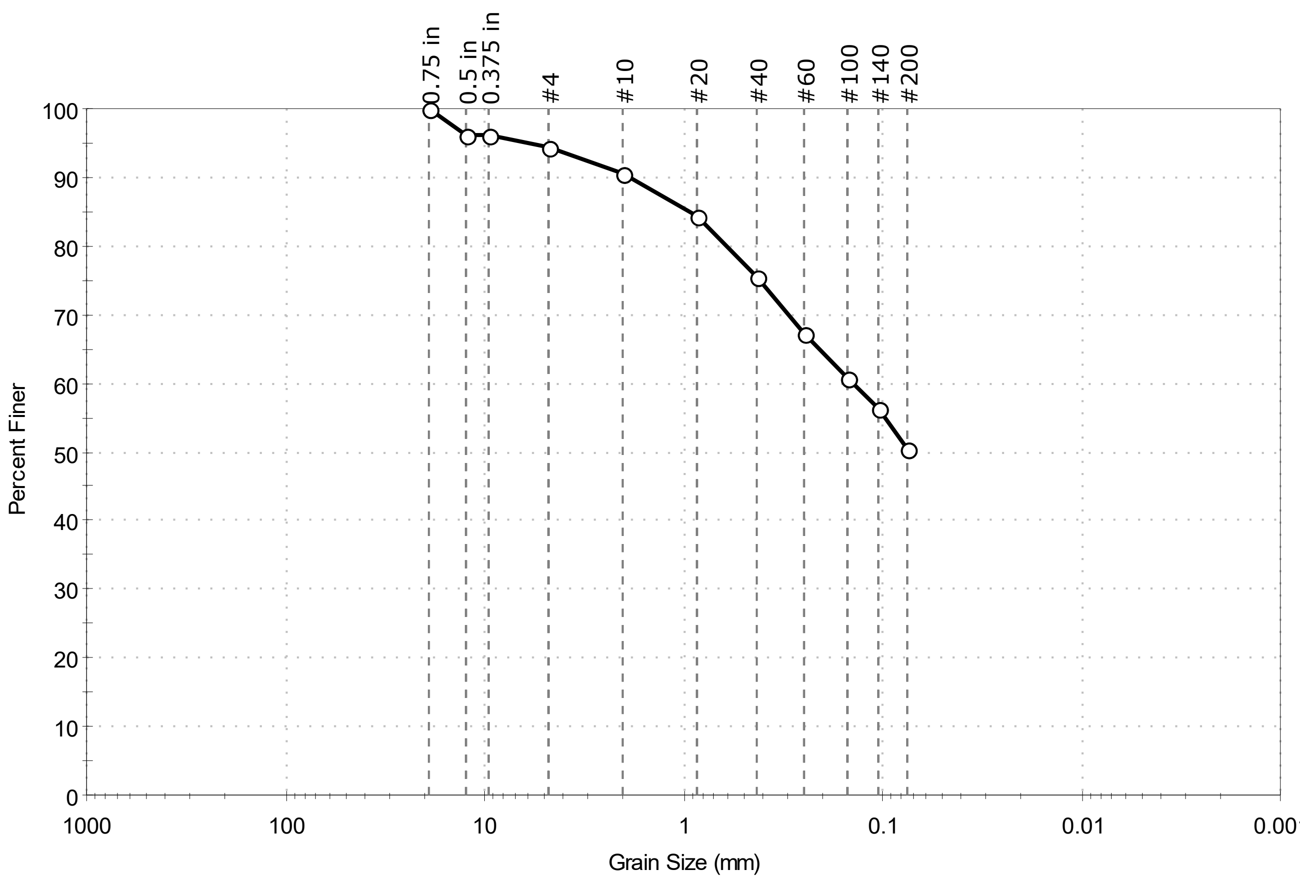
Client:	Golder Associates			
Project:	Merrill Rd Bridge Replace I-295 Ex 20			
Location:	Freeport, ME		Project No:	GTX-311185
Boring ID:	BB-FDR-106	Sample Type: jar	Tested By:	GA
Sample ID:	1D	Test Date: 02/04/20	Checked By:	jsc
Depth :	0-2 ft	Test Id: 539973		
Test Comment:	---			
Visual Description:	Moist, brown sandy clay			
Sample Comment:	---			

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	5.5	43.9	50.6

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	96		
0.375 in	9.50	96		
#4	4.75	94		
#10	2.00	91		
#20	0.85	84		
#40	0.42	76		
#60	0.25	67		
#100	0.15	61		
#140	0.11	56		
#200	0.075	51		

Coefficients

D_{85} =0.9264 mm | D_{30} =N/A

D_{60} =0.1402 mm | D_{15} =N/A

D_{50} =N/A | D_{10} =N/A

C_u =N/A | C_c =N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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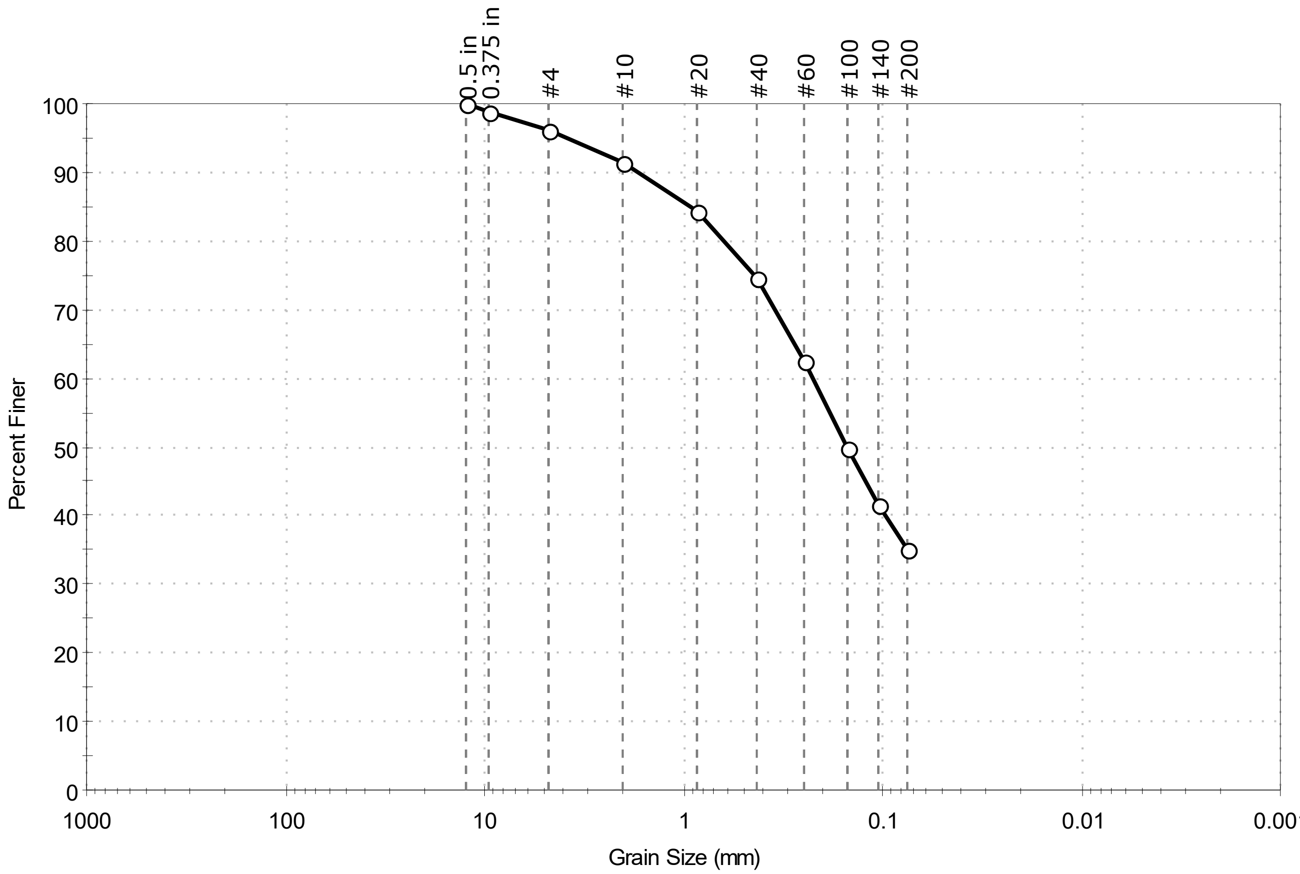
Client:	Golder Associates		
Project:	Merrill Rd Bridge Replace I-295 Ex 20		
Location:	Freeport, ME	Project No:	GTX-311185
Boring ID: BB-FDR-106	Sample Type: jar	Tested By:	GA
Sample ID: 2DB	Test Date: 02/05/20	Checked By:	jsc
Depth : 5-7 ft	Test Id: 539974		
Test Comment:	---		
Visual Description:	Moist, olive brown clayey sand		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	3.9	61.0	35.1

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.5 in	12.50	100		
0.375 in	9.50	99		
#4	4.75	96		
#10	2.00	91		
#20	0.85	84		
#40	0.42	75		
#60	0.25	63		
#100	0.15	50		
#140	0.11	41		
#200	0.075	35		

Coefficients

D_{85} =0.9088 mm D_{30} =N/A

D_{60} =0.2258 mm D_{15} =N/A

D_{50} =0.1506 mm D_{10} =N/A

C_u =N/A C_c =N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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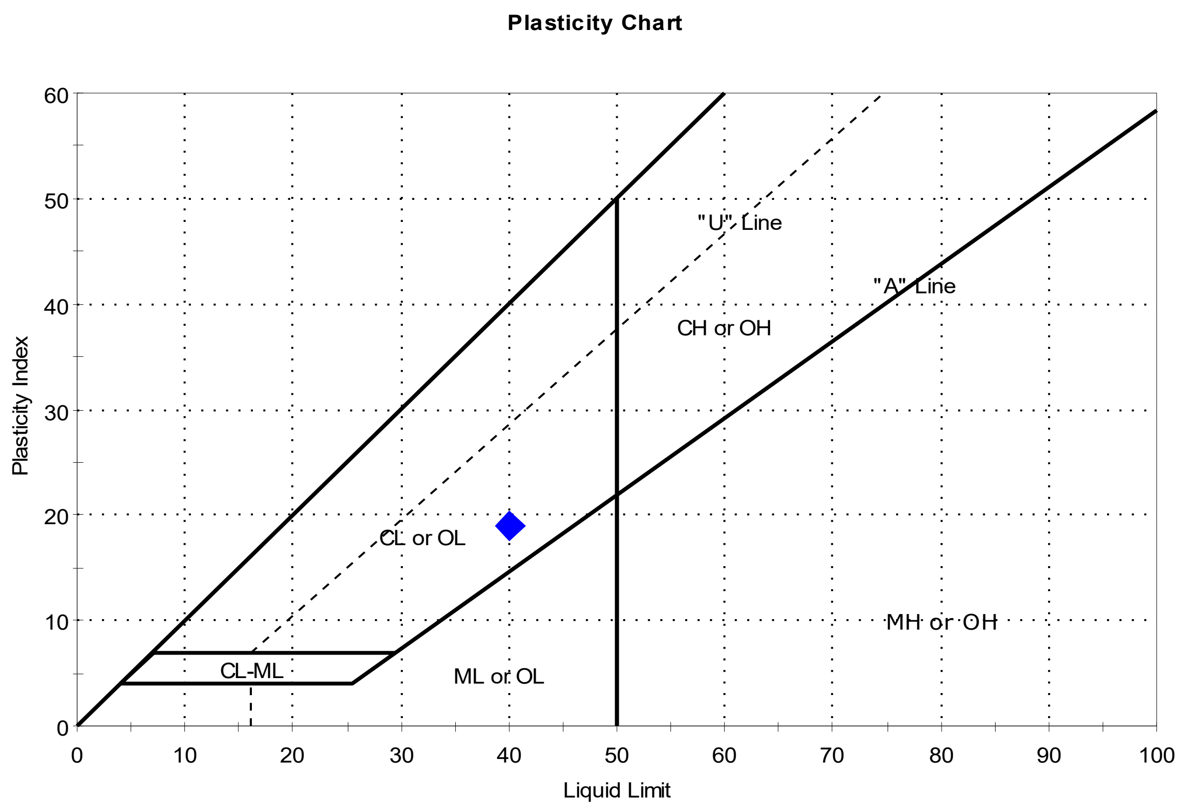
ATTERBERG LIMITS

Client:	Golder Associates				
Project:	Merrill Rd Bridge Replace I-295 Ex 20				
Location:	Freeport, ME			Project No:	GTX-311185
Boring ID:	BB-FDR-101	Sample Type:	jar	Tested By:	GA
Sample ID:	6DA	Test Date:	02/04/20	Checked By:	jsc
Depth :	26-28 ft	Test Id:	539957		
Test Comment:	---				
Visual Description:	Moist, olive gray clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	6DA	B-FDR-10	26-28 ft	26	40	21	19	0.3	Lean CLAY (CL)

Sample Prepared using the WET method

2% Retained on #40 Sieve

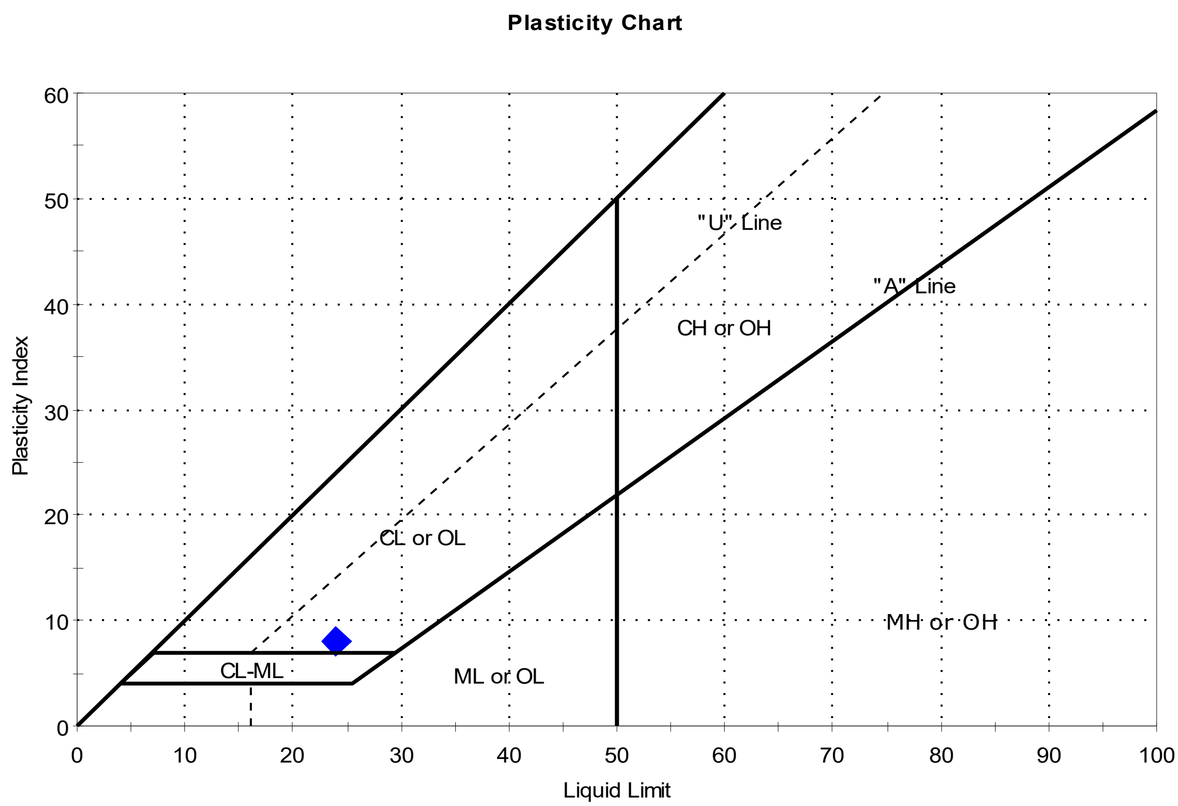
Dry Strength: VERY HIGH

Dilatancy: NONE

Toughness: MEDIUM

Client:	Golder Associates		
Project:	Merrill Rd Bridge Replace I-295 Ex 20		
Location:	Freeport, ME	Project No:	GTX-311185
Boring ID:	BB-FDR-101	Sample Type: jar	Tested By: GA
Sample ID:	7D	Test Date: 02/04/20	Checked By: jsc
Depth :	31-33 ft	Test Id: 539958	
Test Comment:	---		
Visual Description:	Moist, olive clay with sand		
Sample Comment:	---		

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	7D	B-FDR-10	31-33 ft	23	24	16	8	0.9	

Sample Prepared using the WET method

Dry Strength: HIGH

Dilatancy: NONE

Toughness: MEDIUM

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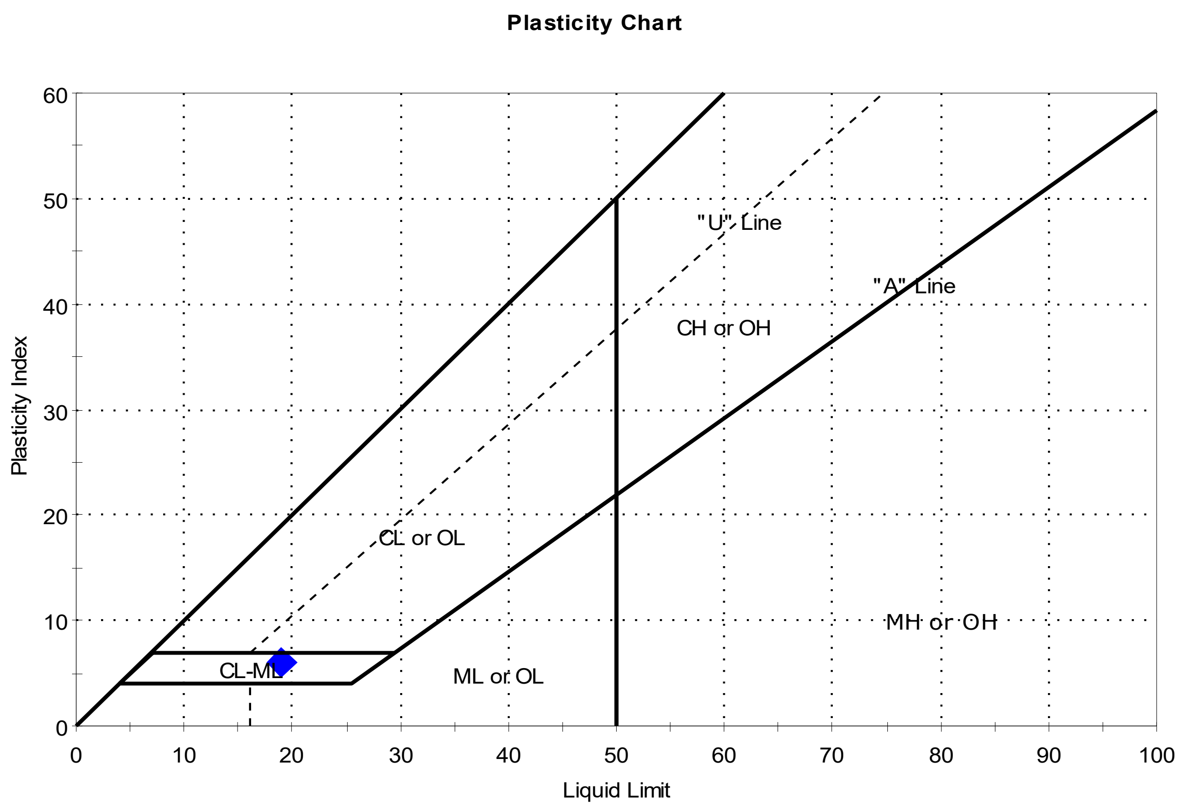
Client:	Golder Associates		
Project:	Merrill Rd Bridge Replace I-295 Ex 20		
Location:	Freeport, ME	Project No:	GTX-311185
Boring ID: BB-FDR-101	Sample Type: jar	Tested By:	GA
Sample ID: 8D	Test Date: 02/04/20	Checked By:	jsc
Depth : 36-38 ft	Test Id: 539959		
Test Comment:	---		
Visual Description:	Moist, dark gray silty clayey gravel		
Sample Comment:	---		

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	8D	B-FDR-10	36-38 ft	18	19	13	6	0.8	Silty, Clayey GRAVEL (GC-GM)

Sample Prepared using the WET method

61% Retained on #40 Sieve

Dry Strength: HIGH

Dilatancy: NONE

Toughness: MEDIUM

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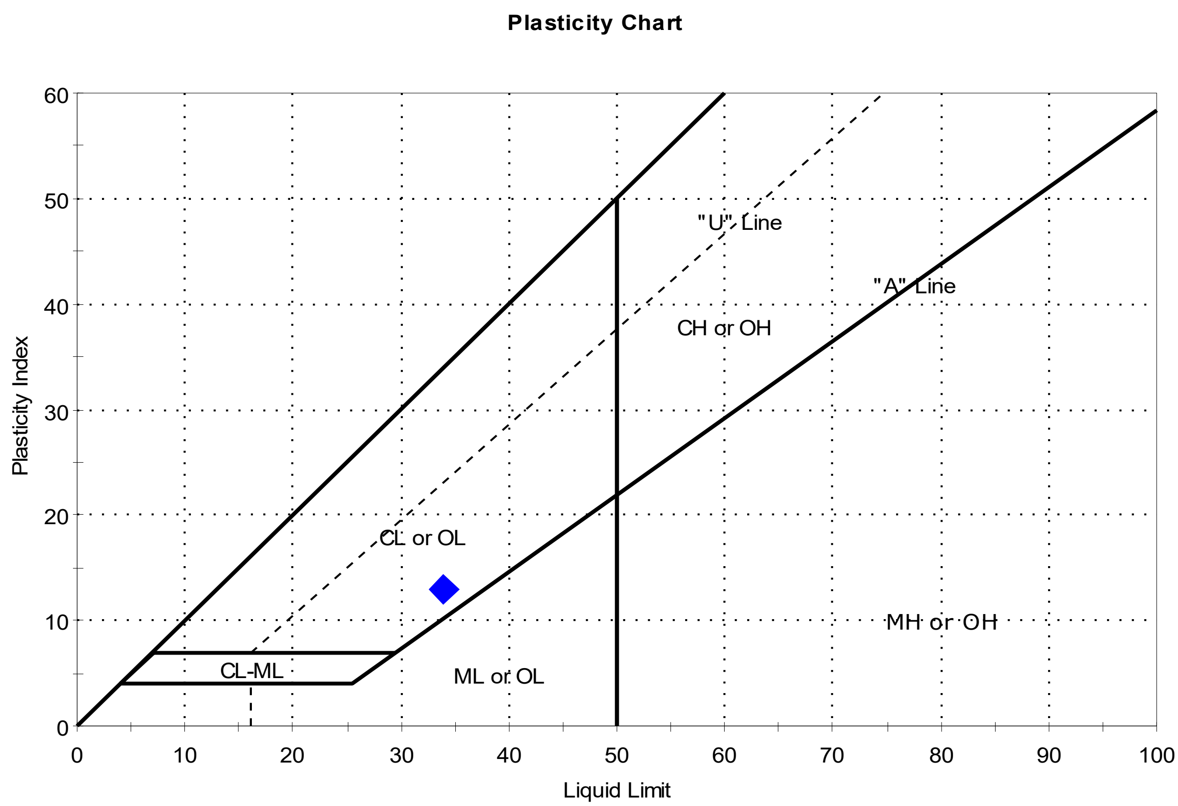
Client:	Golder Associates				
Project:	Merrill Rd Bridge Replace I-295 Ex 20				
Location:	Freeport, ME			Project No:	GTX-311185
Boring ID:	BB-FDR-102	Sample Type:	jar	Tested By:	GA
Sample ID:	2D	Test Date:	02/04/20	Checked By:	jsc
Depth :	7-9 ft	Test Id:	539993		
Test Comment:	---				
Visual Description:	Moist, olive clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



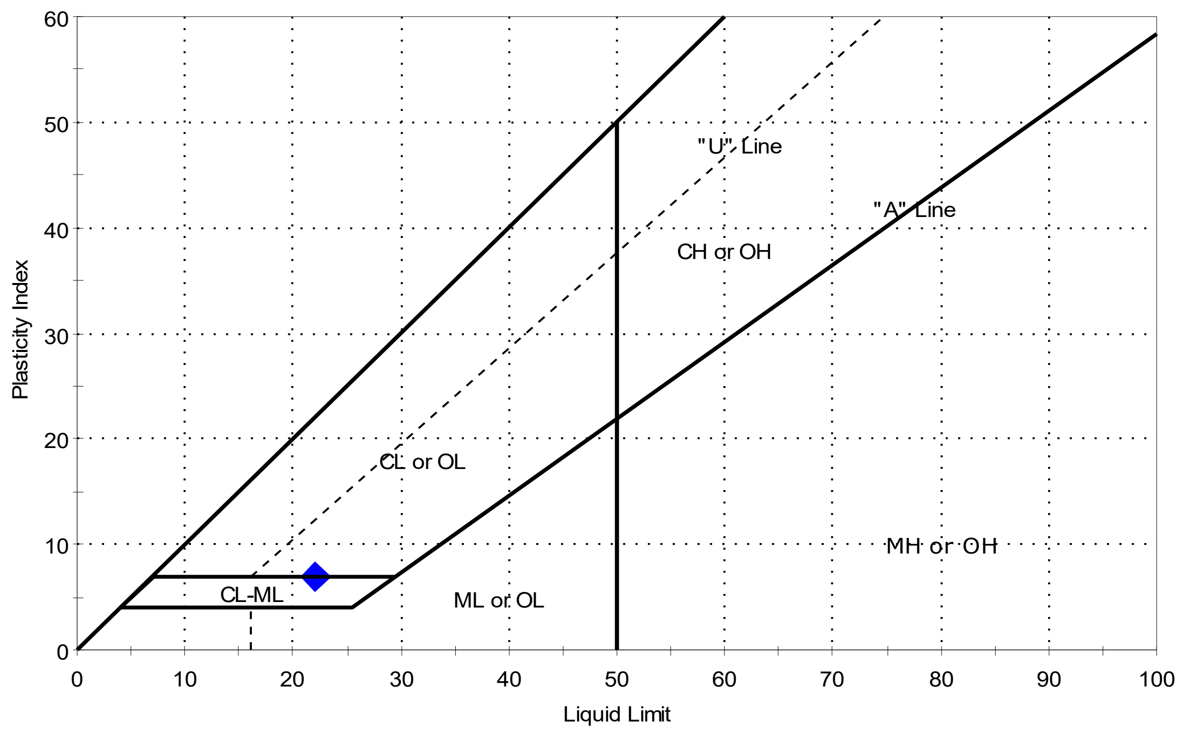
Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	2D	B-FDR-10	7-9 ft	22	34	21	13	0.1	

Sample Prepared using the WET method

Dry Strength: VERY HIGH
Dilatancy: NONE
Toughness: MEDIUM

Client:	Golder Associates
Project:	Merrill Rd Bridge Replace I-295 Ex 20
Location:	Freeport, ME
Project No:	GTX-311185
Boring ID:	BB-FDR-102
Sample Type:	jar
Tested By:	GA
Sample ID:	3D
Test Date:	02/04/20
Checked By:	jsc
Depth :	12-14 ft
Test Id:	539961
Test Comment:	---
Visual Description:	Moist, dark gray silty clay
Sample Comment:	---

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	3D	B-FDR-10	12-14 ft	22	22	15	7	1.1	

Sample Prepared using the WET method

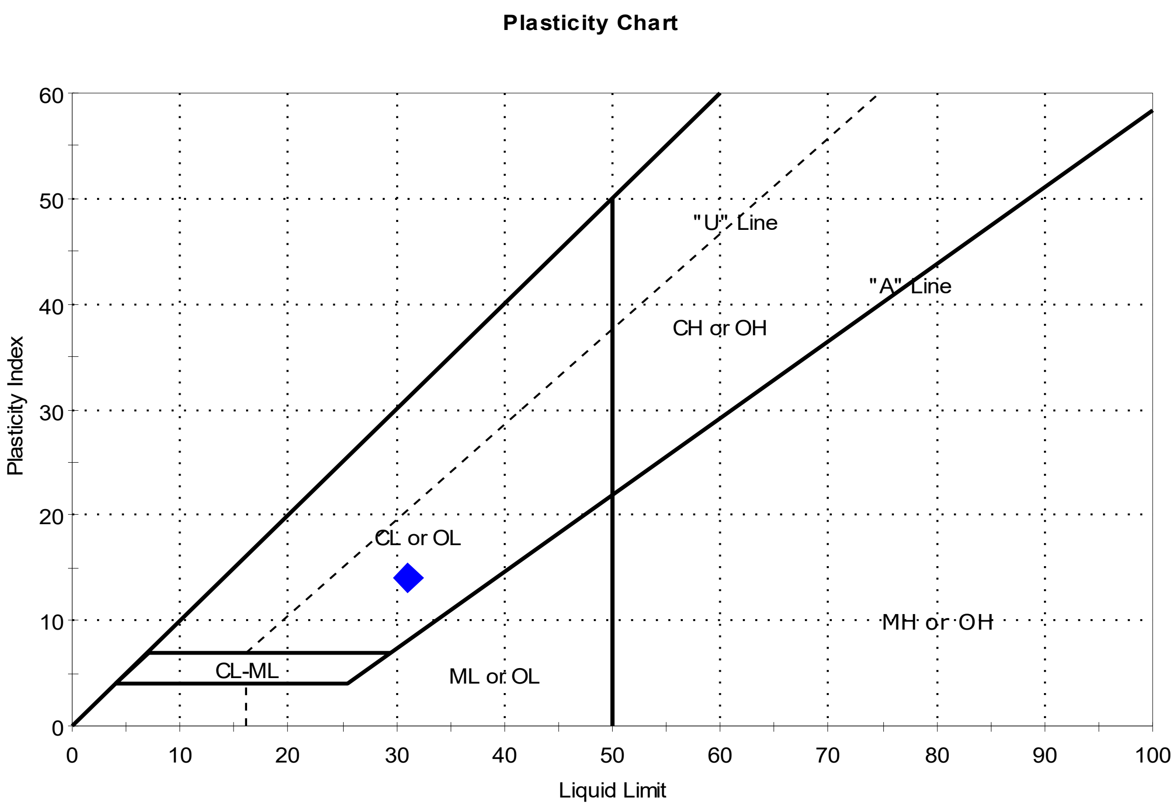
Dry Strength: HIGH

Dilatancy: NONE

Toughness: MEDIUM

Client:	Golder Associates				
Project:	Merrill Rd Bridge Replace I-295 Ex 20				
Location:	Freeport, ME			Project No:	GTX-311185
Boring ID:	BB-FDR-104	Sample Type:	jar	Tested By:	GA
Sample ID:	2D	Test Date:	02/04/20	Checked By:	jsc
Depth :	5.5-7.5 ft	Test Id:	539963		
Test Comment:	---				
Visual Description:	Moist, olive clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	2D	B-FDR-10	5.5-7.5 ft	28	31	17	14	0.8	

Sample Prepared using the WET method

Dry Strength: VERY HIGH

Dilatancy: NONE

Toughness: MEDIUM

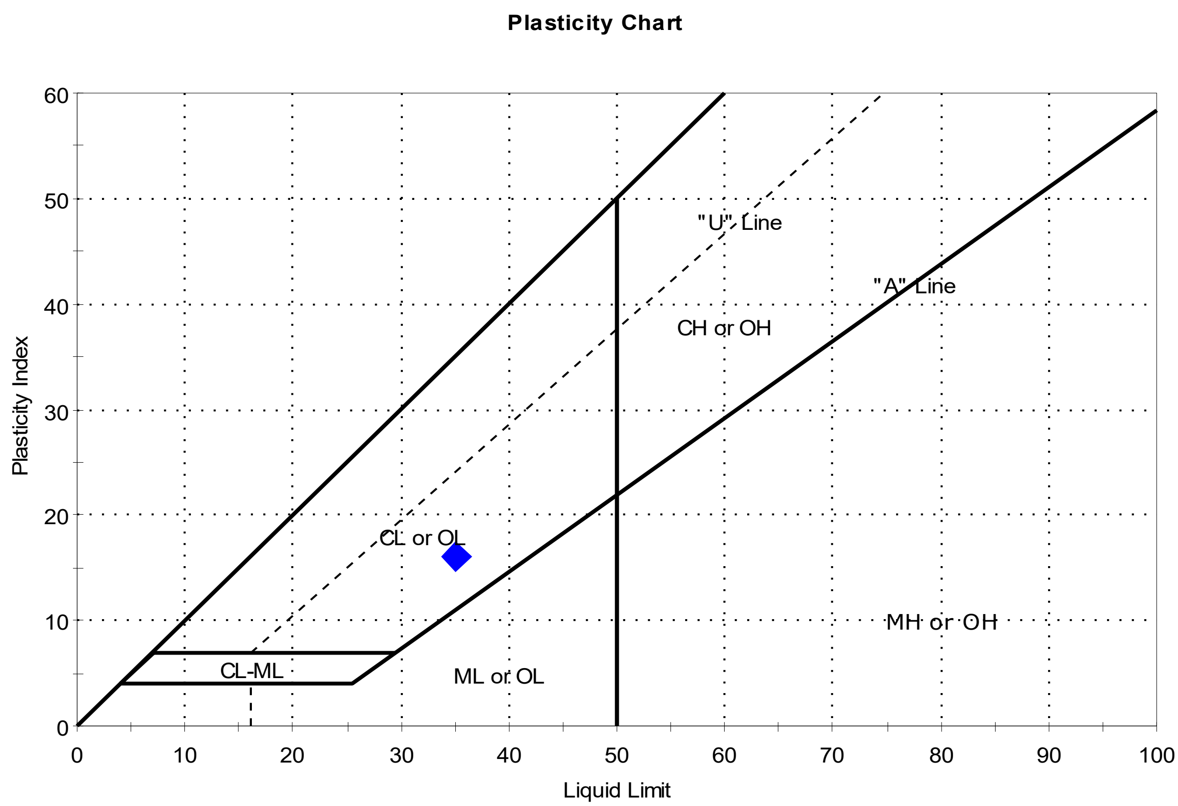
Client:	Golder Associates				
Project:	Merrill Rd Bridge Replace I-295 Ex 20				
Location:	Freeport, ME			Project No:	GTX-311185
Boring ID:	BB-FDR-105	Sample Type:	jar	Tested By:	GA
Sample ID:	2DB	Test Date:	02/04/20	Checked By:	jsc
Depth :	5-7 ft	Test Id:	539964		

Test Comment: ---

Visual Description: Moist, olive clay with sand

Sample Comment: ---

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	2DB	B-FDR-10	5-7 ft	23	35	19	16	0.3	

Sample Prepared using the WET method

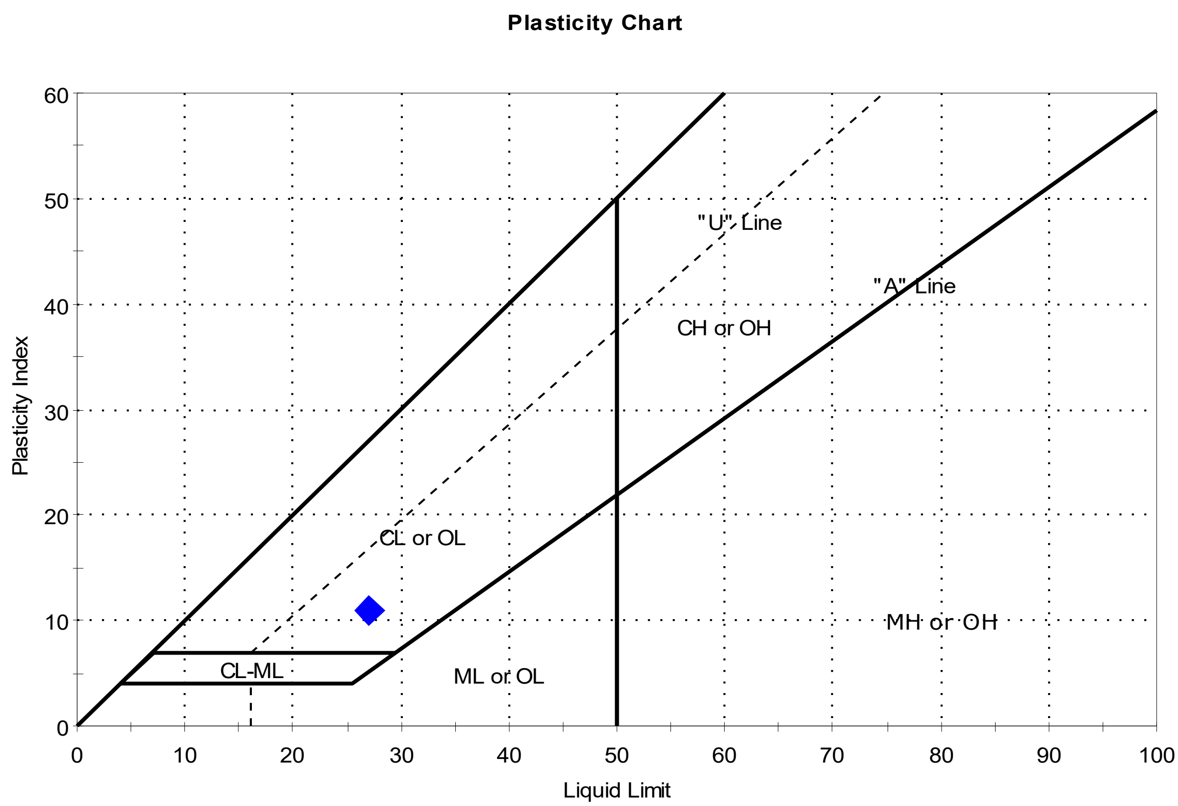
Dry Strength: VERY HIGH

Dilatancy: NONE

Toughness: MEDIUM

Client:	Golder Associates		
Project:	Merrill Rd Bridge Replace I-295 Ex 20		
Location:	Freeport, ME	Project No:	GTX-311185
Boring ID: BB-FDR-105	Sample Type: jar	Tested By:	GA
Sample ID: 4D	Test Date: 02/04/20	Checked By:	jsc
Depth : 15-17 ft	Test Id: 539994		
Test Comment:	---		
Visual Description:	Moist, dark gray clay with sand		
Sample Comment:	---		

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	4D	B-FDR-10	15-17 ft	30	27	16	11	1.3	

Sample Prepared using the WET method

Dry Strength: HIGH

Dilatancy: NONE

Toughness: MEDIUM

APPENDIX D

Rock Core Laboratory Test Results

APPENDIX D
LABORATORY TEST RESULTS FOR ROCK CORE SAMPLES

Presented in the following order

Bulk Density and Compressive Strength

of Rock Core Specimen

Compressive Strength and Elastic Moduli of Rock

Bulk Density and Compressive Strength of Rock Core Specimen

Bulk Density and Compressive Strength of Rock Core Specimens by ASTM D7012 Method C

Boring ID	Sample Number	Depth	Bulk Density, pcf	Compressive strength, psi	Failure Type	Meets ASTM D4543	Note(s)
BB-FDR-104	Run 1	11.50 - 11.87 ft	159	7921	2	Yes	---
BB-FDR-105	Run 2	23.57 - 23.94 ft	171	15826	1	Yes	---
BB-FDR-106	Run 1	6.80 - 7.10 ft	162	8024	3	Yes	---

Notes: Density determined on core samples by measuring dimensions and weight and then calculating.

All specimens tested at the approximate as-received moisture content and at standard laboratory temperature.

The axial load was applied continuously at a stress rate that produced failure in a test time between 2 and 15 minutes.

Failure Type: 1 = Intact Material Failure; 2 = Discontinuity Failure; 3 = Intact Material and Discontinuity Failure
(See attached photographs)

Client:	Golder Associates	Test Date:	1/17/2020
Project Name:	Merrill Rd Bridge Replace I-295 Ex 20	Tested By:	jck
Project Location:	Freeport, ME	Checked By:	smd
GTX #:	311185		
Boring ID:	BB-FDR-104		
Sample ID:	RUN 1		
Depth:	11.50-11.87 ft		
Visual Description:	See photographs		

UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D4543

BULK DENSITY

	1	2	Average		
Specimen Length, in:	4.34	4.34	4.34		
Specimen Diameter, in:	2.00	2.00	2.00		
Specimen Mass, g:	570.34				
Bulk Density, lb/ft^3	159			**Minimum Diameter Tolerence Met?**	**YES**
Length to Diameter Ratio:	2.2			**Length to Diameter Ratio Tolerance Met?**	**YES**

DEVIATION FROM STRAIGHTNESS (Procedure S1)

Maximum gap between side of core and reference surface plate:
Is the maximum gap $\leq$ 0.02 in.? YES

Maximum difference must be < 0.020 in.

Straightness Tolerance Met? **YES**

END FLATNESS AND PARALLELISM (Procedure FP1)

END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00030	-0.00030	-0.00020	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
Diameter 2, in (rotated 90°)	-0.00070	-0.00060	-0.00050	-0.00040	-0.00030	-0.00020	0.00000	0.00000	0.00000	0.00010	0.00020	0.00030	0.00040	0.00050	0.00050

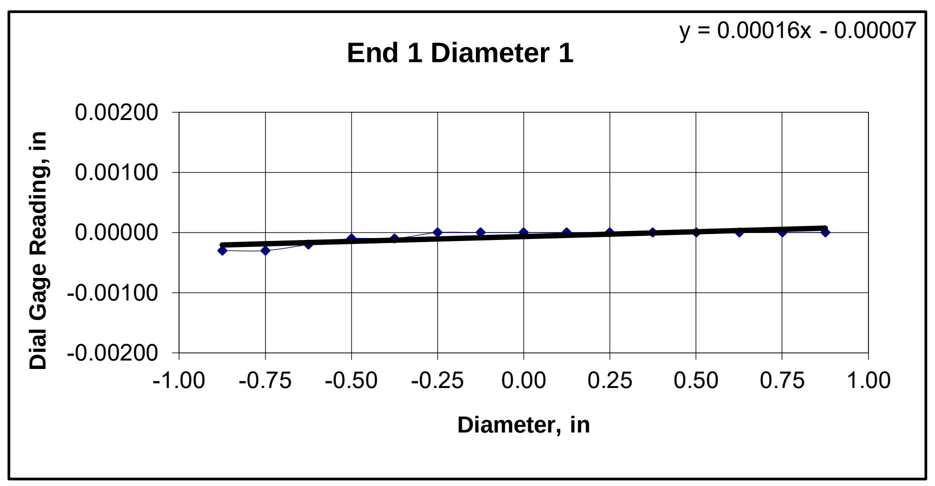
Difference between max and min readings, in:
0° = 0.00030 90° = 0.00120

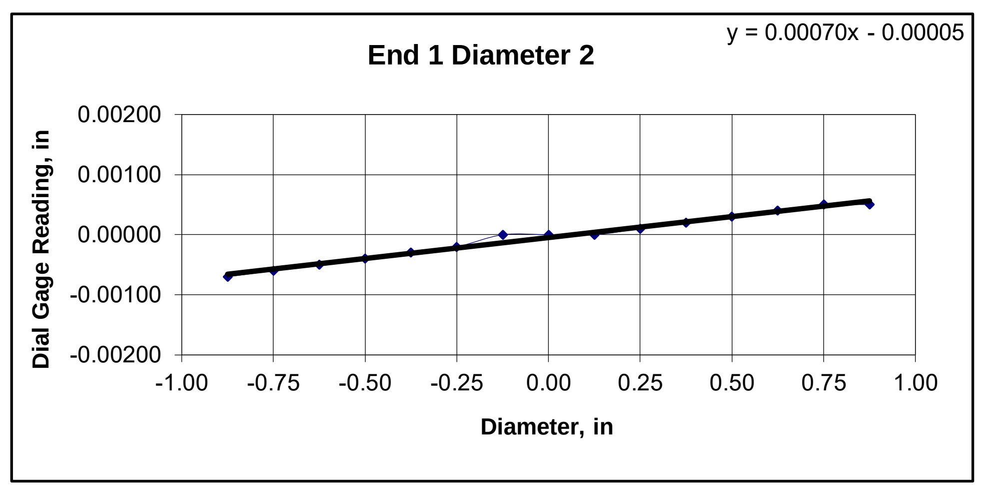
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00020	-0.00020	-0.00020	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
Diameter 2, in (rotated 90°)	-0.00070	-0.00060	-0.00050	-0.00040	-0.00030	-0.00020	0.00000	0.00000	0.00000	0.00010	0.00010	0.00020	0.00030	0.00050	0.00050

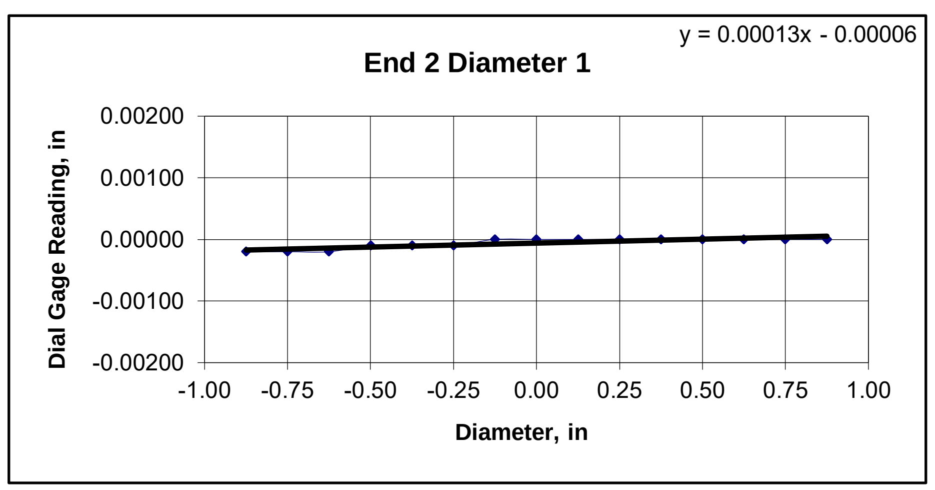
Difference between max and min readings, in:
0° = 0.0002 90° = 0.0012

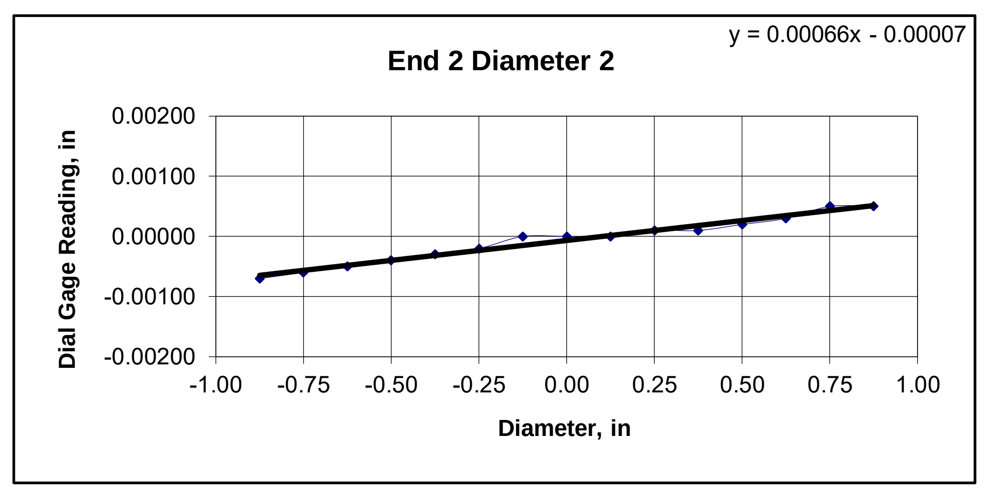
Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00060

Flatness Tolerance Met? **YES**









DIAMETER 1

End 1:	
Slope of Best Fit Line	0.00016
Angle of Best Fit Line:	0.00917
End 2:	
Slope of Best Fit Line	0.00013
Angle of Best Fit Line:	0.00737
Maximum Angular Difference:	0.00180
Parallelism Tolerance Met?	**YES**
Spherically Seated	

DIAMETER 2

End 1:	
Slope of Best Fit Line	0.00070
Angle of Best Fit Line:	0.03994
End 2:	
Slope of Best Fit Line	0.00066
Angle of Best Fit Line:	0.03798
Maximum Angular Difference:	0.00196
Parallelism Tolerance Met?	**YES**
Spherically Seated	

PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00030	2.000	0.00015	0.009	YES
Diameter 2, in (rotated 90°)	0.00120	2.000	0.00060	0.034	YES
END 2					
Diameter 1, in	0.00020	2.000	0.00010	0.006	YES
Diameter 2, in (rotated 90°)	0.00120	2.000	0.00060	0.034	YES

Maximum angle of departure must be $\leq$ 0.25°

Perpendicularity Tolerance Met? **YES**

Client:	Golder Associates
Project Name:	Merrill Rd Bridge Replace I-295 Ex 20
Project Location:	Freeport, ME
GTX #:	311185
Test Date:	1/17/2020
Tested By:	jck
Checked By:	smd
Boring ID:	BB-FDR-104
Sample ID:	RUN 1
Depth, ft:	11.50-11.87

After cutting and grinding

After break

Client:	Golder Associates	Test Date:	1/17/2020
Project Name:	Merrill Rd Bridge Replace I-295 Ex 20	Tested By:	jck
Project Location:	Freeport, ME	Checked By:	smd
GTX #:	311185		
Boring ID:	BB-FDR-105		
Sample ID:	RUN 2		
Depth:	23.57-23.94 ft		
Visual Description:	See photographs		

UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D4543

BULK DENSITY

	1	2	Average
Specimen Length, in:	4.35	4.35	4.35
Specimen Diameter, in:	1.99	2.00	2.00
Specimen Mass, g:	613.09		
Bulk Density, lb/ft^3	171	**Minimum Diameter Tolerence Met?**	YES
Length to Diameter Ratio:	2.2	**Length to Diameter Ratio Tolerance Met?**	YES

DEVIATION FROM STRAIGHTNESS (Procedure S1)

Maximum gap between side of core and reference surface plate:

Is the maximum gap ≤ 0.02 in.? YES

Maximum difference must be < 0.020 in.

Straightness Tolerance Met? YES

END FLATNESS AND PARALLELISM (Procedure FP1)

END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00050	-0.00040	-0.00030	-0.00030	-0.00020	-0.00010	-0.00010	0.00000	-0.00010	-0.00010	0.00000	0.00000	-0.00010	-0.00010	-0.00010
Diameter 2, in (rotated 90°)	0.00040	0.00030	0.00030	0.00020	0.00010	0.00010	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00030	-0.00040	-0.00050	-0.00070

Difference between max and min readings, in:

0° = 0.00050 90° = 0.00110

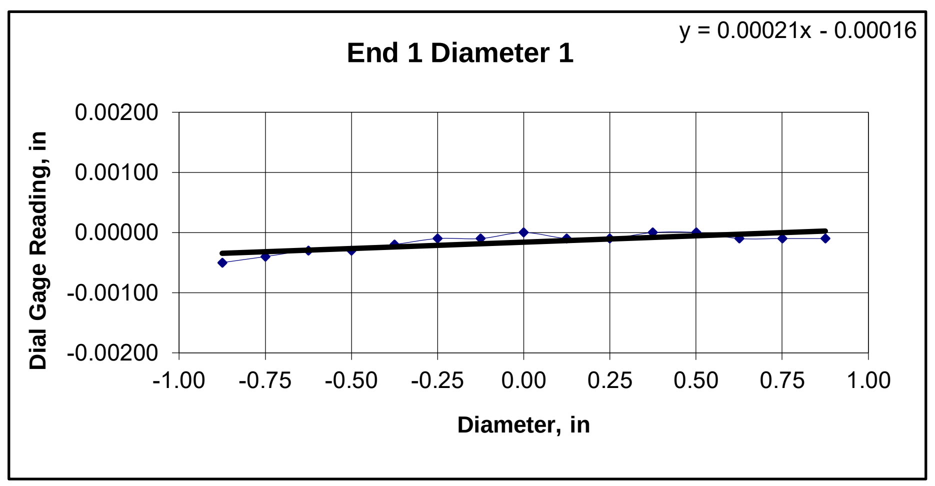
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00050	-0.00040	-0.00030	-0.00030	-0.00020	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010	-0.00010
Diameter 2, in (rotated 90°)	0.00040	0.00040	0.00040	0.00030	0.00030	0.00020	0.00010	0.00000	0.00000	-0.00010	-0.00020	-0.00030	-0.00040	-0.00050	-0.00070

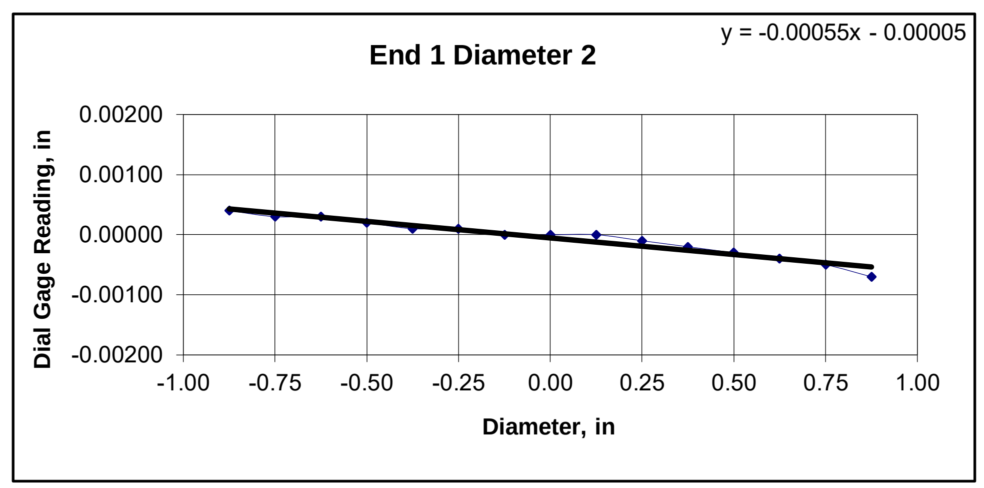
Difference between max and min readings, in:

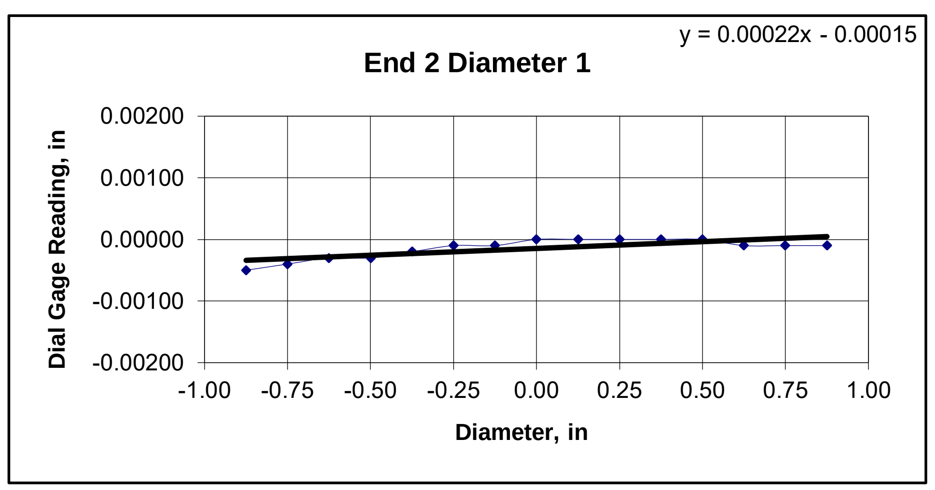
0° = 0.0005 90° = 0.0011

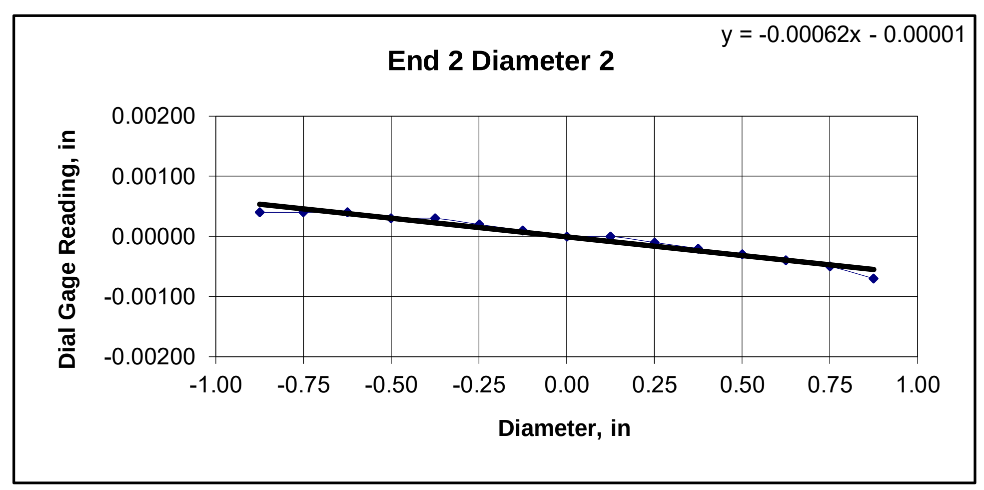
Maximum difference must be < 0.0020 in. Difference = ± 0.00055

Flatness Tolerance Met? YES









DIAMETER 1

End 1:	Slope of Best Fit Line	0.00021
	Angle of Best Fit Line:	0.01211
End 2:	Slope of Best Fit Line	0.00022
	Angle of Best Fit Line:	0.01261
Maximum Angular Difference:		0.00049

Parallelism Tolerance Met? YES

Spherically Seated

DIAMETER 2

End 1:	Slope of Best Fit Line	0.00055
	Angle of Best Fit Line:	0.03159
End 2:	Slope of Best Fit Line	0.00062
	Angle of Best Fit Line:	0.03552
Maximum Angular Difference:		0.00393

Parallelism Tolerance Met? YES

Spherically Seated

PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00050	1.995	0.00025	0.014	YES
Diameter 2, in (rotated 90°)	0.00110	1.995	0.00055	0.032	YES
END 2					
Diameter 1, in	0.00050	1.995	0.00025	0.014	YES
Diameter 2, in (rotated 90°)	0.00110	1.995	0.00055	0.032	YES

Maximum angle of departure must be ≤ 0.25°

Perpendicularity Tolerance Met? YES

Client:	Golder Associates
Project Name:	Merrill Rd Bridge Replace I-295 Ex 20
Project Location:	Freeport, ME
GTX #:	311185
Test Date:	1/17/2020
Tested By:	jck
Checked By:	smd
Boring ID:	BB-FDR-105
Sample ID:	RUN 2
Depth, ft:	23.57-23.94

After cutting and grinding

After break

GeoTesting EXPRESS

Client:	Golder Associates	Test Date:	1/17/2020
Project Name:	Merrill Rd Bridge Replace I-295 Ex 20	Tested By:	jck
Project Location:	Freeport, ME	Checked By:	smd
GTX #:	311185		
Boring ID:	BB-FDR-106		
Sample ID:	RUN 1		
Depth:	6.80-7.10 ft		
Visual Description:	See photographs		

UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D4543

BULK DENSITY

	1	2	Average		
Specimen Length, in:	4.26	4.26	4.26		
Specimen Diameter, in:	2.00	2.00	2.00		
Specimen Mass, g:	570.47				
Bulk Density, lb/ft^3	162	**Minimum Diameter Tolerence Met?**		**YES**	
Length to Diameter Ratio:	2.1	**Length to Diameter Ratio Tolerance Met?**		**YES**	

DEVIATION FROM STRAIGHTNESS (Procedure S1)

Maximum gap between side of core and reference surface plate:
Is the maximum gap $\leq$ 0.02 in.? YES

Maximum difference must be < 0.020 in.

Straightness Tolerance Met? **YES**

END FLATNESS AND PARALLELISM (Procedure FP1)

END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00020
Diameter 2, in (rotated 90°)	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00020

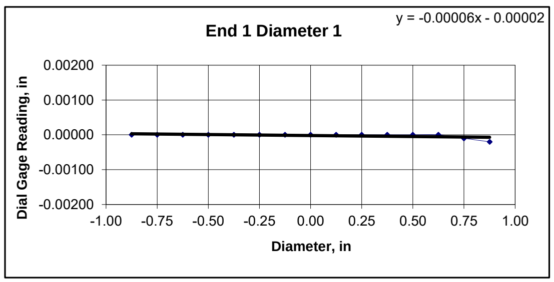
Difference between max and min readings, in:
0° = 0.00020 90° = 0.00020

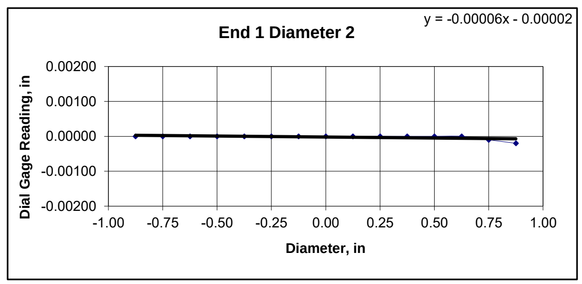
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
Diameter 2, in (rotated 90°)	0.00000	0.00000	0.00000	0.00000	0.00000	0.00010	0.00000	0.00000	0.00000	0.00000	-0.00030	0.00000	0.00000	-0.00010	-0.00010

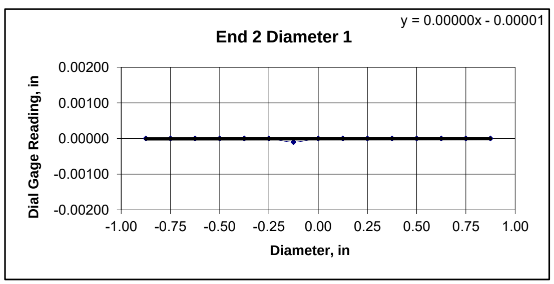
Difference between max and min readings, in:
0° = 0.0001 90° = 0.0004

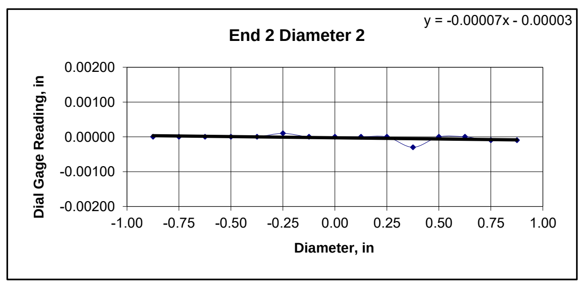
Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00020

Flatness Tolerance Met? **YES**









DIAMETER 1

End 1:	
Slope of Best Fit Line	0.00006
Angle of Best Fit Line:	0.00327
End 2:	
Slope of Best Fit Line	0.00000
Angle of Best Fit Line:	0.00016
Maximum Angular Difference:	0.00311

Parallelism Tolerance Met? **YES**
Spherically Seated

DIAMETER 2

End 1:	
Slope of Best Fit Line	0.00006
Angle of Best Fit Line:	0.00327
End 2:	
Slope of Best Fit Line	0.00007
Angle of Best Fit Line:	0.00393
Maximum Angular Difference:	0.00065

Parallelism Tolerance Met? **YES**
Spherically Seated

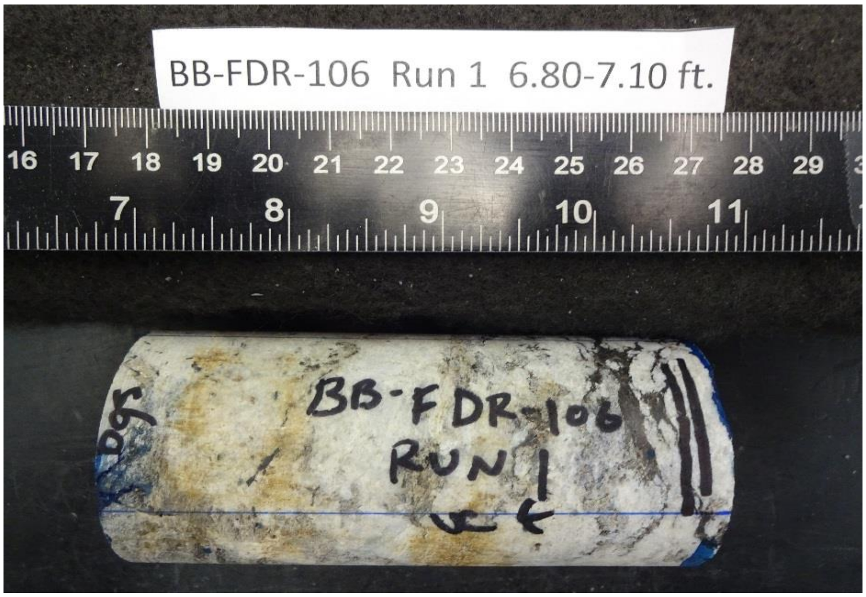
PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00020	2.000	0.00010	0.006	YES
Diameter 2, in (rotated 90°)	0.00020	2.000	0.00010	0.006	YES
END 2					
Diameter 1, in	0.00010	2.000	0.00005	0.003	YES
Diameter 2, in (rotated 90°)	0.00040	2.000	0.00020	0.011	YES

Maximum angle of departure must be $\leq$ 0.25°

Perpendicularity Tolerance Met? **YES**

Client:	Golder Associates
Project Name:	Merrill Rd Bridge Replace I-295 Ex 20
Project Location:	Freeport, ME
GTX #:	311185
Test Date:	1/17/2020
Tested By:	jck
Checked By:	smd
Boring ID:	BB-FDR-106
Sample ID:	RUN 1
Depth, ft:	6.80-7.10



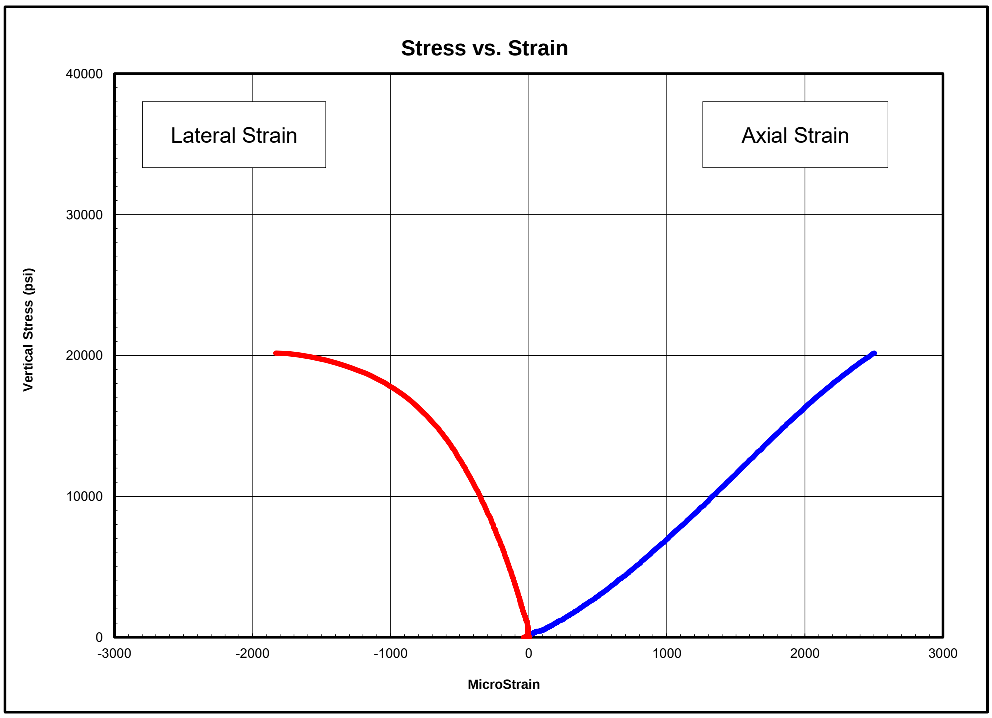
After cutting and grinding

After break

Compressive Strength and Elastic Moduli of Rock

Client:	Golder Associates
Project Name:	Merrill Rd Bridge Replace I-295 Ex 20
Project Location:	Freeport, ME
GTX #:	311185
Test Date:	1/17/2020
Tested By:	jck
Checked By:	jsc
Boring ID:	BB-FDR-102
Sample ID:	RUN 1
Depth, ft:	16.35-16.72
Sample Type:	rock core
Sample Description:	See photographs Intact material and discontinuity failure

Compressive Strength and Elastic Moduli of Rock by ASTM D7012 - Method D



Peak Compressive Stress: 20,161 psi

The strain values recorded within the third stress range for this test produce values of Poisson's Ratio that exceed maximum values found in rocks.

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
2000-7400	7,820,000	0.27
7400-12800	9,420,000	0.48
12800-18100	9,120,000	---

Notes: Test specimen tested at the approximate as-received moisture content and at standard laboratory temperature.
The axial load was applied continuously at a stress rate that produced failure in a test time between 2 and 15 minutes.
Young's Modulus and Poisson's Ratio calculated using the tangent to the line in the stress range listed.
Calculations assume samples are isotropic, which is not necessarily the case.

Client:	Golder Associates	Test Date:	1/17/2020
Project Name:	Merrill Rd Bridge Replace I-295 Ex 20	Tested By:	jck
Project Location:	Freeport, ME	Checked By:	smd
GTX #:	311185		
Boring ID:	BB-FDR-102		
Sample ID:	RUN 1		
Depth:	16.35-16.72 ft		
Visual Description:	See photographs		

UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D4543

BULK DENSITY

	1	2	Average	
Specimen Length, in:	4.39	4.39	4.39	
Specimen Diameter, in:	1.99	2.00	2.00	
Specimen Mass, g:	592.15			
Bulk Density, lb/ft^3	164	**Minimum Diameter Tolerence Met?**		YES
Length to Diameter Ratio:	2.2	**Length to Diameter Ratio Tolerance Met?**		YES

DEVIATION FROM STRAIGHTNESS (Procedure S1)

Maximum gap between side of core and reference surface plate:
Is the maximum gap ≤ 0.02 in.? NO

Maximum difference must be < 0.020 in.

Straightness Tolerance Met? NO

END FLATNESS AND PARALLELISM (Procedure FP1)

END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00010	-0.00030	-0.00040	-0.00050
Diameter 2, in (rotated 90°)	-0.00080	-0.00070	-0.00060	-0.00050	-0.00040	-0.00030	-0.00020	0.00000	0.00000	0.00010	0.00020	0.00030	0.00040	0.00040	0.00060

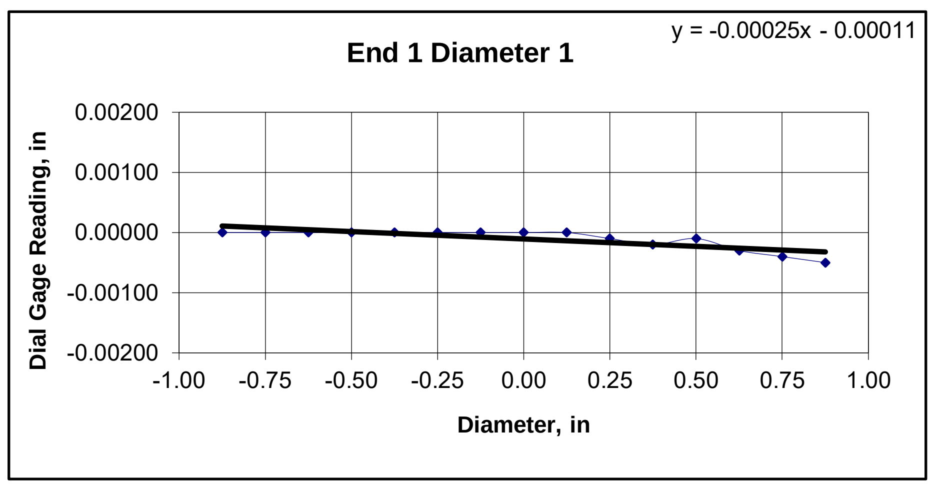
Difference between max and min readings, in:
0° = 0.00050 90° = 0.00140

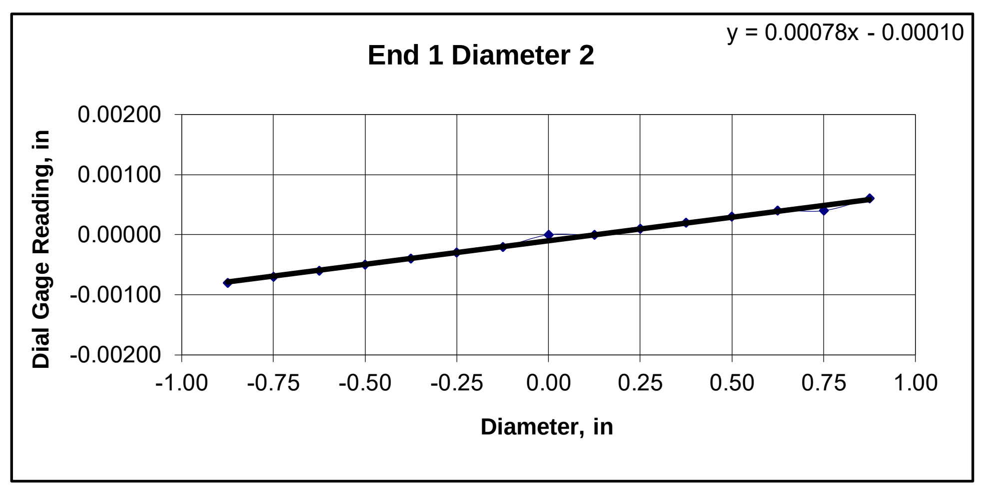
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010	-0.00020	-0.00030	-0.00060
Diameter 2, in (rotated 90°)	-0.00080	-0.00070	-0.00060	-0.00040	-0.00040	-0.00030	-0.00020	0.00000	0.00010	0.00010	0.00020	0.00030	0.00040	0.00050	0.00060

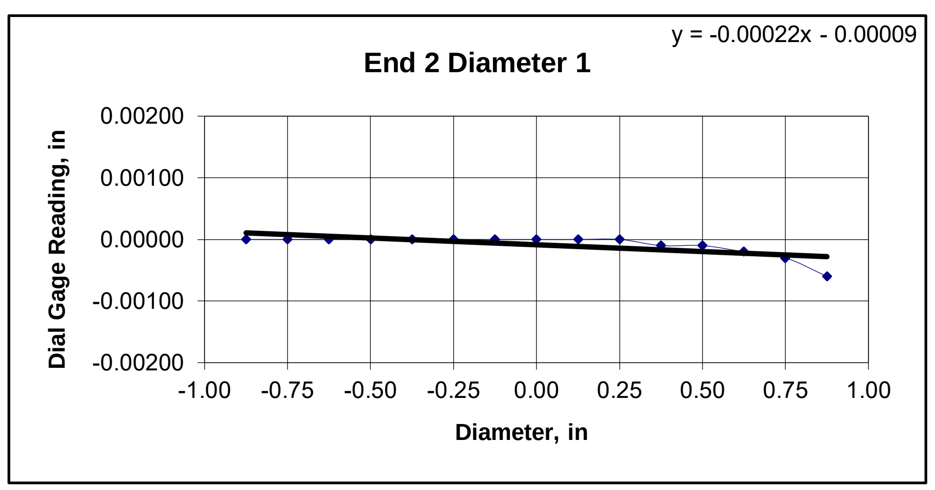
Difference between max and min readings, in:
0° = 0.0006 90° = 0.0014

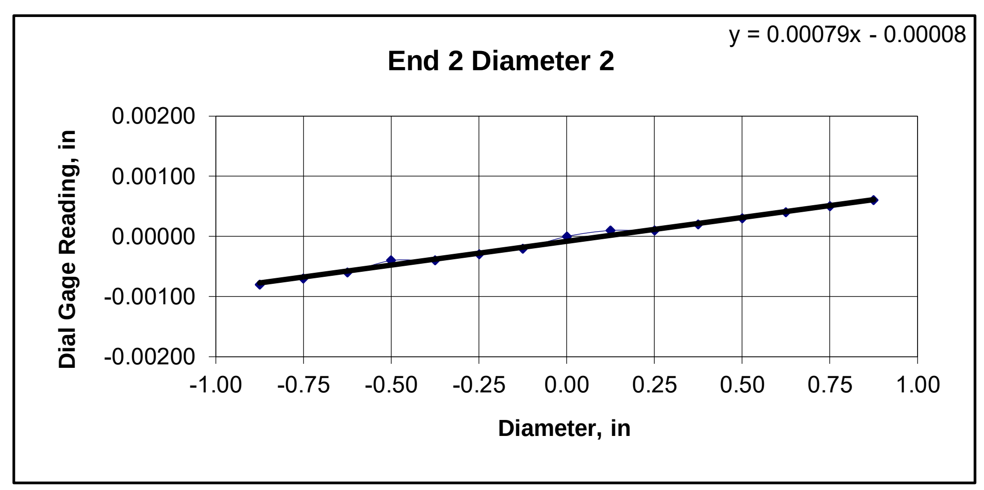
Maximum difference must be < 0.0020 in. Difference = ± 0.00070

Flatness Tolerance Met? YES









DIAMETER 1

End 1:
Slope of Best Fit Line 0.00025
Angle of Best Fit Line: 0.01408

End 2:
Slope of Best Fit Line 0.00022
Angle of Best Fit Line: 0.01261

Maximum Angular Difference: 0.00147

Parallelism Tolerance Met? YES
Spherically Seated

DIAMETER 2

End 1:
Slope of Best Fit Line 0.00078
Angle of Best Fit Line: 0.04485

End 2:
Slope of Best Fit Line 0.00079
Angle of Best Fit Line: 0.04535

Maximum Angular Difference: 0.00049

Parallelism Tolerance Met? YES
Spherically Seated

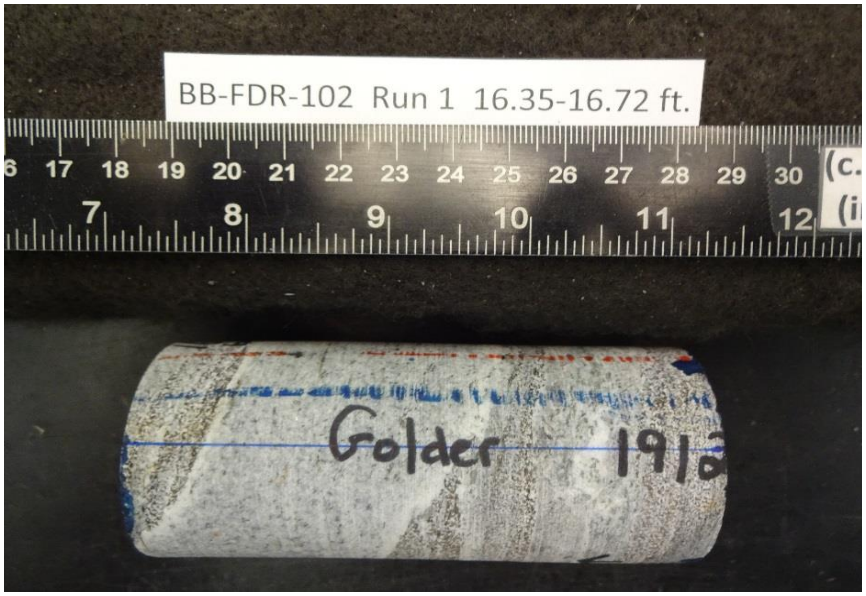
PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00050	1.995	0.00025	0.014	YES
Diameter 2, in (rotated 90°)	0.00140	1.995	0.00070	0.040	YES
END 2					
Diameter 1, in	0.00060	1.995	0.00030	0.017	YES
Diameter 2, in (rotated 90°)	0.00140	1.995	0.00070	0.040	YES

Maximum angle of departure must be ≤ 0.25°

Perpendicularity Tolerance Met? YES

Client:	Golder Associates
Project Name:	Merrill Rd Bridge Replace I-295 Ex 20
Project Location:	Freeport, ME
GTX #:	311185
Test Date:	1/17/2020
Tested By:	jck
Checked By:	smd
Boring ID:	BB-FDR-102
Sample ID:	RUN 1
Depth, ft:	16.35-16.72



After cutting and grinding

After break

APPENDIX E1

Frost Depth

CALCULATIONS

Date:	12/3/2020	**Made by:**	KAR
Project No.:	19126013	**Checked by:**	HTV
Subject:	Depth of Frost Penetration	**Reviewed by:**	MCM
Project Short Title:	MaineDOT I-295 Exit 20 Merrill Road Bridge Replacement No. 5720		

OBJECTIVE
Determine the depth of frost penetration at the proposed bridge site.

REFERENCES
1. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.
2. Golder interpreted subsurface section A-A' (Figure 3, Preliminary Geotechnical Design Report, dated September 2020).
3. Golder geotechnical test boring logs (Appendix A, Preliminary Geotechnical Design Report, dated September 2020).
4. GeoTesting Express laboratory testing results, dated February 5, 2020 (Appendix C, Preliminary Geotechnical Design Report, dated September 2020).

CALCULATION

Follow the procedure in the MaineDOT Bridge Design Guide Section 5.2.1 to determine the depth of frost penetration.

The site location is Freeport, Maine. Using the Maine Design Freezing Index Map (Reference 1, Figure 5-1), the design freezing index at the site location is 1300 degree-days.

Based on Reference 2, the proposed abutments and pier will be founded within the existing fill layer, which is a coarse-grained soil.

Based on References 3 and 4, the average moisture content of the samples tested from the existing fill layer is 11.0%.

From Reference 1, Table 5-1, with design freezing index = 1300, coarse-grained soil, and linear interpolation between w = 10% and w = 20% for w = 11%:

Frost Penetration = 75.0 inches = 6.2 feet

CONCLUSIONS
The depth of frost penetration at the proposed bridge site is estimated to be 6.2 feet. The foundations should be embedded below this depth.

5.2 General

5.2.1 Frost

Any foundation placed on seasonally frozen soils must be embedded below the depth of frost penetration to provide adequate frost protection and to minimize the potential for freeze/thaw movements. Fine-grained soils with low cohesion tend to be most frost susceptible. Soils containing a high percentage of particles smaller than the No. 200 sieve also tend to promote frost penetration.

In order to estimate the depth of frost penetration at a site, Table 5-1 has been developed using the Modified Berggren equation and Figure 5-1 Maine Design Freezing Index Map. The use of Table 5-1 assumes site specific, uniform soil conditions where the Geotechnical Designer has evaluated subsurface conditions. Coarse-grained soils are defined as soils with sand as the major constituent. Fine-grained soils are those having silt and/or clay as the major constituent. If the make-up of the soil is not easily discerned, consult the Geotechnical Designer for assistance. In the event that specific site soil conditions vary, the depth of frost penetration should be calculated by the Geotechnical Designer.

Table 5-1 Depth of Frost Penetration

Design Freezing Index	Frost Penetration (in)					
	Coarse Grained			Fine Grained		
	w=10%	w=20%	w=30%	w=10%	w=20%	w=30%
1000	66.3	55.0	47.5	47.1	40.7	36.9
1100	69.8	57.8	49.8	49.6	42.7	38.7
1200	73.1	60.4	52.0	51.9	44.7	40.5
1300	76.3	63.0	54.3	54.2	46.6	42.2
1400	79.2	65.5	56.4	56.3	48.5	43.9
1500	82.1	67.9	58.4	58.3	50.2	45.4
1600	84.8	70.2	60.3	60.2	51.9	46.9
1700	87.5	72.4	62.2	62.2	53.5	48.4
1800	90.1	74.5	64.0	64.0	55.1	49.8
1900	92.6	76.6	65.7	65.8	56.7	51.1
2000	95.1	78.7	67.5	67.6	58.2	52.5
2100	97.6	80.7	69.2	69.3	59.7	53.8
2200	100.0	82.6	70.8	71.0	61.1	55.1
2300	102.3	84.5	72.4	72.7	62.5	56.4
2400	104.6	86.4	74.0	74.3	63.9	57.6
2500	106.9	88.2	75.6	75.9	65.2	58.8
2600	109.1	89.9	77.1	77.5	66.5	60.0

Notes: 1. w = water content
2. Where the Freezing Index and/or water content is between the presented values, linear interpretation may be used to determine the frost penetration.

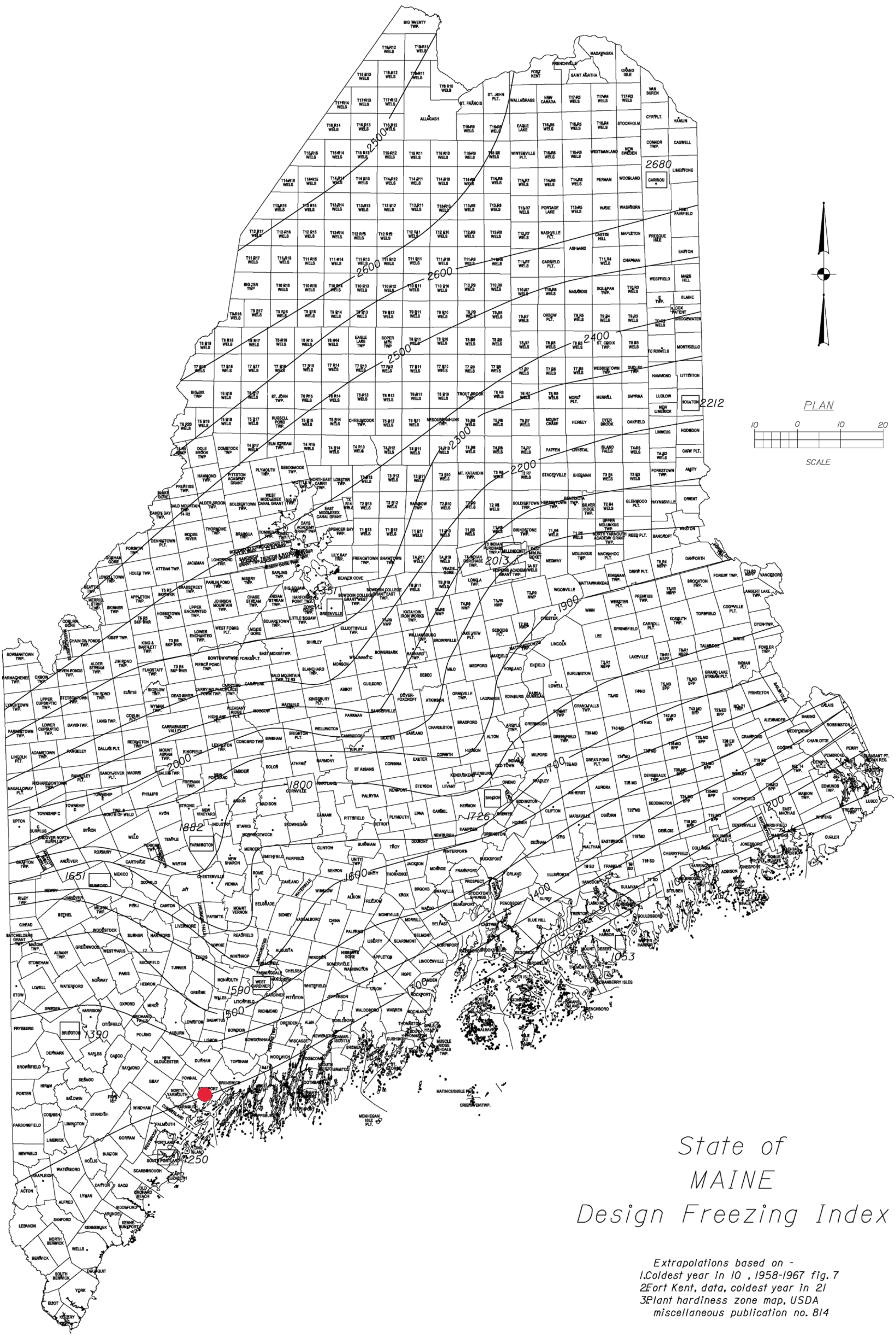


Figure 5-1 Maine Design Freezing Index Map

APPENDIX E2

Seismic Site Class

GOLDER

CALCULATIONS

Date:	7/6/2020	**Made by:**	HTV
Project No.:	19126013	**Checked by:**	KAR
Subject:	Seismic Site Class	**Reviewed by:**	JEL
Project Short Title:	MaineDOT I-295 Exit 20 Merrill Road Bridge Replacement No. 5720		

OBJECTIVE

Determine the seismic site class and acceleration coefficient for the thickest soil deposit encountered at the I-295 Exit 20 Merrill Road Bridge.

METHOD

Use the procedure outlined in AASHTO LRFD Section 3.10.3.1 Method B (Ref. 1) to determine site class.

REFERENCES

1. AASHTO. (2020). AASHTO LRFD Bridge Design Specifications, 9th Ed. American Association of State Highway and Transportation Officials (AASHTO), Washington, D.C.

2. Golder Associates, Inc. (2020). Preliminary Geotechnical Design Report I-295 Merrill Road Bridge Replacement #5720 (Exit 20) - Appendix A: Geotechnical Boring Logs

3. Guertin Elkerton & Associates for Maine Department of Transportation. (2003 with 2018 updates). Bridge Design Guide.

ASSUMPTIONS

1. Use average N_{60}-values (corrected only for hammer efficiencies) in the analysis for Method B. Average N_{60} values were taken as averages from the specific associated boring and layer and are based on values presented in Ref. 2.

2. The strata encountered at BB-FDR-101 and at BB-FDR-105 were used to evaluate seismic site class, since BB-FDR-101 contained the thickest total soil deposit and the thickest fill layer, while BB-FDR-105 contained the thickest clay layer.

CALCULATIONS

Site Class*	
C	$\bar{N} > 50$; $\bar{s}_u > 2.0$ ksf; 1200 ft/s < $\bar{V}_s$ < 2500 ft/s
D	15 < $\bar{N}$ < 50; 1.0 < $\bar{s}_u$ < 2.0 ksf; 600 ft/s < $\bar{V}_s$ < 1200 ft/s
E	$\bar{N} < 15$; $\bar{s}_u < 1.0$ ksf; $\bar{V}_s$ < 600 ft/s
F	Peats, organic clays, high plasticity clay, thick soft/medium stiff clays

* Site class A & B pertain to rock and are not included in this table.

1. Per AASHTO Table C3.10.3.1-1 (Ref 1), use Method B, $\bar{N}$ method:

$$\bar{N} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}}$$

where:

d_i = Thickness of a layer between 0 and 100 ft.

N_i = SPT blow count, N_{60}, of a layer (not to exceed 100 blows/ft). Where refusal is met for a rock layer, N_i should be taken as 100 blows/ft.

The seismic site class is assigned using the $\bar{N}$ method for the full embankment soil height at BB-FDR-101 as:

Date:	7/6/2020	**Made by:**	HTV
Project No.:	19126013	**Checked by:**	KAR
Subject:	Seismic Site Class	**Reviewed by:**	JEL
Project Short Title:	MaineDOT I-295 Exit 20 Merrill Road Bridge Replacement No. 5720		

Layer	N_{60}	Deposit	d_i	d_i/N_i
1	17	Fill	26.0	1.53
2	15	Glaciomarine Silty Clay	8.0	0.53
3	50	Sand/Gravel	2.6	0.05
4	100	Bedrock	63.4	0.63
		Sum:	100	2.75

$\bar{N}$ = 36.38

Site Class: D

2. Per AASHTO Table C3.10.3.1-1 (Ref 1), use Method C, $\bar{s}$u method:

For cohesionless soil layers:

$$\bar{N}_{ch} = \frac{\sum_{i=1}^{m} d_i}{\sum_{i=1}^{m} \frac{d_i}{N_{chi}}}$$

where:
d_i = Thickness of a cohesionless layer between 0 and 100 ft.
N_i = SPT blow count, N_{60}, of a layer (not to exceed 100 blows/ft).

For cohesive soil layers:

$$\bar{s}_u = \frac{\sum_{i=1}^{k} d_i}{\sum_{i=1}^{k} \frac{d_i}{s_{ui}}}$$

where:
d_i = Thickness of a cohesive layer between 0 and 100 ft.
s_{ui} = undrained shear strength for cohesive soil layer (not to exceed 5.0 ksf).

The seismic site class is assigned using the N_{ch} method for the cohesionless soil and rock layers and average s_u method for cohesive soils in the top 100 ft of the soil/rock profile at BB-FDR-105 as:

Cohesionless Soils:

Layer	N_{60}	Deposit	d_i	d_i/N_i
1	12	Fill	4.0	0.33
3	100	Bedrock	82.0	0.82
		Sum:	86	1.15

$\bar{N}_{ch}$ = 74.57

Site Class: C

Date:	7/6/2020	**Made by:**	HTV
Project No.:	19126013	**Checked by:**	KAR
Subject:	Seismic Site Class	**Reviewed by:**	JEL
Project Short Title:	MaineDOT I-295 Exit 20 Merrill Road Bridge Replacement No. 5720		

Cohesive Soils:

Layer	s_u (ksf)	Deposit	d_i	d_i/s_{ui}
2	1.60	Glaciomarine Silty Clay	14.0	8.75
		Sum:	14.0	8.75

$\bar{s}_u$ = 1.60

Site Class: D

According to Ref. 1 Table C3.10.3.1-1 Note, when using Method C, if the site class resulting from $\bar{N}_{ch}$ and $\bar{s}_u$ differ, select the site class that gives the highest site factors and design spectral response in the period range of interest. Based on Ref. 1 Tables 3.10.3.2-2 and 3.10.3.2-3, site factor values are higher with site class D than with site class C for both short-period and long-period range of acceleration spectrum. Thus, site class D is selected.

Site Class: D

3. Seismic Performance Zone, per AASHTO LRFD 9th Ed. (2020)

Date:	7/6/2020	**Made by:**	HTV
Project No.:	19126013	**Checked by:**	KAR
Subject:	Seismic Site Class	**Reviewed by:**	JEL
Project Short Title:	MaineDOT I-295 Exit 20 Merrill Road Bridge Replacement No. 5720		

S_1 = 0.0431 Figure 3.10.2.1-3: 1 second period response acceleration (AASHTO 2020).

Seismic Zone 1 per AASHTO 3.10.6 Table 3.10.6-1		0.15g			
AASHTO Section 3.10.4.2	$SD_1 = F_vS_1$				
AASHTO Table 3.10.3.2-3	Site Class C F_v	1.7	$SD_{1\text{-}C}$ =	0.073	Zone: 1
	Site Class D F_v	2.4	$SD_{1\text{-}D}$ =	0.103	Zone: 1

4. Calculation of Seismic Patameters using AASHTO LRFD 9th Ed. (2020)

Values for PGA, S_s, and S_1 were obtained comparing the the site location to the following geographic charts:

a_1. Using Figure 3.10.2.1-1

*PGA = 0.08

*Horizontal ground coefficient with a 7% probability of exceedence in 75 years

a_2. Using Figure 3.10.2.1-2

*S_s = 0.16

*Horizontal response spectral acceleration at period of 0.2 s with a 7% probability of exceedence in 75 years and 5% critical damping

a_3. Using Figure 3.10.2.1-3

*S_1 = 0.043

*Horizontal response spectral acceleration at a period of 1 s with a 7% probability of exceedence in 75 years and 5% critical damping

b_1. Using Table 3.10.3.2-1

Using a site class value of D, taken from calcuation step 2b and a PGA of 0.08, taken from step $4a_1$

F_{pga} = 1.6

b_2. Using Table 3.10.3.2-2

Using a site class value of D, taken from calcuation step 2b and a S_s of 0.08, taken from step $4a_2$

F_a = 1.6

b_3. Using Table 3.10.3.2-3

Using a site class value of D, taken from calcuation step 2b and a S_1 of 0.04, taken from step $4a_3$

F_v = 2.4

c_1. Using Equation 3.10.4.2-2

$$A_s = F_{pga} * PGA$$

PGA = 0.08

F_{pga} = 1.6

$\boxed{A_s = 0.128}$

c_2. Using Equation 3.10.4.2-3

$$S_{DS} = F_a * S_S$$

S_S = 0.16

F_a = 1.6

$\boxed{S_{DS} = 0.256}$

c_3. Using Equation 3.10.4.2-6

$$S_{D1} = F_v * S_1$$

S_1 = 0.043

F_v = 2.4

$\boxed{S_{D1} = 0.103}$

Date:	7/6/2020	**Made by:**	HTV
Project No.:	19126013	**Checked by:**	KAR
Subject:	Seismic Site Class	**Reviewed by:**	JEL
Project Short Title:	MaineDOT I-295 Exit 20 Merrill Road Bridge Replacement No. 5720		

Seismic Parameters					
PGA[1]	A_s[2]	S_{DS}[3]	S_{D1}[4]	Site Class[5]	Zone[6]
-	1/s	1/s	1/s	-	-
0.080	0.128	0.256	0.103	D	1

[1]Using AASHTO Figure 3.10.2.1-1

[2]Using AASHTO Figure 3.10.2.1-1, Table 3.10.3.2-1, and Equation 3.10.4.2-2

[3]Using AASHTO Figure 3.10.2.1-2, Table 3.10.3.2-2, and Equation 3.10.4.2-3

[4]Using AASHTO Figure 3.10.2.1-3, Table 3.10.3.2-3, and Equation 3.10.4.2-6

[5]Using result of step 2 of calculation package

[6]Using result of step 3 of calculation package

CONCLUSIONS

In the area where the thickest soil deposit over bedrock was encountered (at BB-FDR-101) for the full 100 ft of considered depth of soil and rock including the fill above the I-295 highway elevation, the Seismic Site Class is D.

In the area where the thickest clay deposit was encountered (at BB-FDR-105) for the full 100 ft of considered soil and rock profile, the Seismic Site Class is D.

As per Section 3.10.4.2 of AASHTO (2020), the 1 second period response acceleration at the site is less than 0.15g, therefore the site is in Seismic Performance Zone 1.

APPENDIX E3

Global Stability Analysis

GOLDER

CALCULATIONS

Date:	12/2/2020	**Made by:**	KAR
Project No.:	19126013	**Checked by:**	MEL
Subject:	Global Stability Analysis Abutment No. 1 Embankment	**Reviewed by:**	MCM
Project Short Title:	MaineDOT I-295 Exit 20 Merrill Road Bridge Replacement No. 5720		

OBJECTIVE

Calculate global factor of safety for the Abutment No. 1 proposed bridge approach embankment, assuming the "southern shift" option with the bike path scenario.

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 9th Ed, 2020.
2. Email communication between Golder and HNTB, subject "RE: Freeport I-295 Exit 20 and Exit 22 - Request for Sections and Design Info", dated June 22, 2020.
3. Golder geotechnical test boring logs (Appendix A, Preliminary Geotechnical Design Report, dated September 2020).
4. HNTB for State of Maine Department of Transportation. Merrill Road Bridge Freeport Interstate 295: Desert Road South Cross Sections, dated November 2019.
5. Das, Braja M. 2011. Principles of Foundation Engineering, 7th Edition. Cengage Learning.
6. FHWA. 2017. Geotechnical Engineering Circular No. 5: Geotechnical Site Characterization. Publication No. FHWA NHI-16-072.
7. GeoTesting Express laboratory testing results, dated February 5, 2020 (Appendix C, Preliminary Geotechnical Design Report, dated September 2020).
8. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.
9. Golder calculation titled "Seismic Site Class" (Appendix E, Preliminary Geotechnical Design Report, dated September 2020).
10. Rocscience Slide Software Package Version 2020 9.007 64-bit, build date May 29, 2020.

ATTACHMENTS

1. Slide output figures
2. HNTB plans showing "southern shift" option

ASSUMPTIONS

1. The load applied by the road and traffic for final design conditions is modeled as a 3 ft equivalent load of soil (Reference 1, Table 3.11.6.4-1) based on a 12 ft abutment height (Reference 2). 3 ft x 125 pcf (fill) = 375 psf.
2. A static FS ≥ 1.3 is recommended for embankment final design conditions per Section 5.9.2 in Reference 8. A pseudo-static FS> 1.0 is recommended per Section 3.7.4.1 in Reference 8
3. Circular surfaces were analyzed using the Spencer and Bishop simplified methods and auto refine search. Non-circular surfaces were analyzed using the Spencer method and cuckoo search with surface altering optimization.
4. The existing stratigraphy and water table surface are estimated from samples and groundwater measurements encountered during drilling and provided in Reference 3.
5. The existing grading, proposed grading, and construction design features are taken from Reference 4.
6. Undrained conditions (ϕ = 0) were assumed for the glaciomarine silty clay layer.

CALCULATION

1. Determine input parameters to build the soil model in Slide.

The material parameters selected for use in the Slide models are shown in the table below.

a. The friction angle parameters for the existing fill and sand/gravel layers are based on empirical correlation (Reference 5, Eqn. 2.26) to the average of the N_{60}-values encountered in all borings for each layer (Reference 3).

b. The cohesion parameter for the glaciomarine silty clay layer is based on shear strength measurements made in the field and on empirical correlation (Reference 6, Eqn. 7.19) to the average of the N_{60}-values encountered in all borings for each layer (Reference 3).

Date:	12/2/2020	**Made by:**	KAR
Project No.:	19126013	**Checked by:**	MEL
Subject:	Global Stability Analysis Abutment No. 1 Embankment	**Reviewed by:**	MCM
Project Short Title:	MaineDOT I-295 Exit 20 Merrill Road Bridge Replacement No. 5720		

c. The unit weight parameter for the glaciomarine silty clay layer is calculated from soil moisture contents determined in laboratory testing (Reference 7), assuming 100% saturation. The unit weight parameters for the existing fill and sand/gravel layers are selected based on local engineering experience.

d. The UCS and unit weight parameters for the bedrock are selected based on the average of laboratory test results for all borings (Reference 7). The GSI, m_i, and D parameters for the bedrock are selected based on field descriptions of the rock quality encountered in the borings (Reference 3).

e. The friction angle and unit weight parameters for the construction materials are selected based on MaineDOT standard practice (Reference 8, Table 3-3).

Material Name	Unit Weight (pcf)	Strength Type	Cohesion (psf)	Friction Angle (°)	UCS (psf)	GSI	m_i	D
Existing Fill	125	Mohr-Coulomb	0	32	-	-	-	-
Glaciomarine Silty Clay	125	Mohr-Coulomb	1600	0	-	-	-	-
Sand and Gravel	125	Mohr-Coulomb	0	37	-	-	-	-
Bedrock	164	Generalized Hoek-Brown	-	-	1,869,552	60	28	0
New Fill	125	Mohr-Coulomb	0	32	-	-	-	-
New Subbase	135	Mohr-Coulomb	0	36	-	-	-	-

2. Use the soil layer parameters listed above to analyze the slope stability with Slide.

The soil layer properties above were used to analyze the following scenarios. HNTB provided Golder with both a northern shift and southern shift of the proposed bridge centerline alignment. Following analysis, Golder determined that the southern shift in the alignment would result in the largest height and offset of embankment fills over the existing ground surface, which would likely have the greatest impact in stability. Therefore the southern shift option was analyzed as the critical case. This analysis evaluates the fills at Abutment No. 1 since the proposed fills and the underlying foundation clays are the thickest. Both the northern and the southern fill slopes were analyzed at the Abutment No. 1. The results of the Slide stability analyses are summarized in the following table.

Baseline	Station	Feature	Slope	Lowest Factor of Safety (Spencer Method)			
				NonCircular Failure Surface Through Proposed Fill		NonCircular Failure Surface Through Glaciomarine Deposit	
Southern Shift	60+25	Northwest Embankment	North	1.27	(Fig. A.1)	2.36	(Fig. A.2)
			South	1.28	(Fig. A.3)	2.47	(Fig. A.4)

Circular Surfaces:

Date:	12/2/2020	**Made by:**	KAR
Project No.:	19126013	**Checked by:**	MEL
Subject:	Global Stability Analysis Abutment No. 1 Embankment	**Reviewed by:**	MCM
Project Short Title:	MaineDOT I-295 Exit 20 Merrill Road Bridge Replacement No. 5720		

Circular failure surfaces using Spencer and Bishop simplified methods were also evaluated for each scenario. The circular surfaces that were produced were similar in location and size to the noncircular surfaces and had factors of safety similar in magnitude to the noncircular factors of safety. The circular factors of safety ranged from 1.27 to 1.28 for surfaces through the proposed fill and from 2.45 to 2.57 for surfaces through the glaciomarine deposit.

3. Repeat the Slide analysis with pseudo-static seismic load conditions.

The same scenarios were also analyzed with a horizontal seismic load coefficient of $A_s/2$ = 0.064 (A_s from Reference 9) as recommended in AASHTO (Reference 1) Appendix 11A. The results of the seismic Slide stability analyses are summarized in the following table.

Baseline	Station	Feature	Slope	Lowest Factor of Safety (Spencer Method)	
				NonCircular Failure Surface Through Proposed Fill	NonCircular Failure Surface Through Glaciomarine Deposit
Southern Shift	60+25	Northwest Embankment	North	1.09 (Fig. B.1)	2.00 (Fig. B.2)
			South	1.10 (Fig. B.3)	2.11 (Fig. B.4)

Circular Surfaces:

Circular failure surfaces using Spencer and Bishop simplified methods were also evaluated for each scenario. The circular surfaces that were produced were similar in location and size to the noncircular surfaces and had factors of safety similar in magnitude to the noncircular factors of safety. The circular factors of safety ranged from 1.09 to 1.10 for surfaces through the proposed fill and from 2.06 to 2.19 for surfaces through the glaciomarine deposit.

CONCLUSIONS

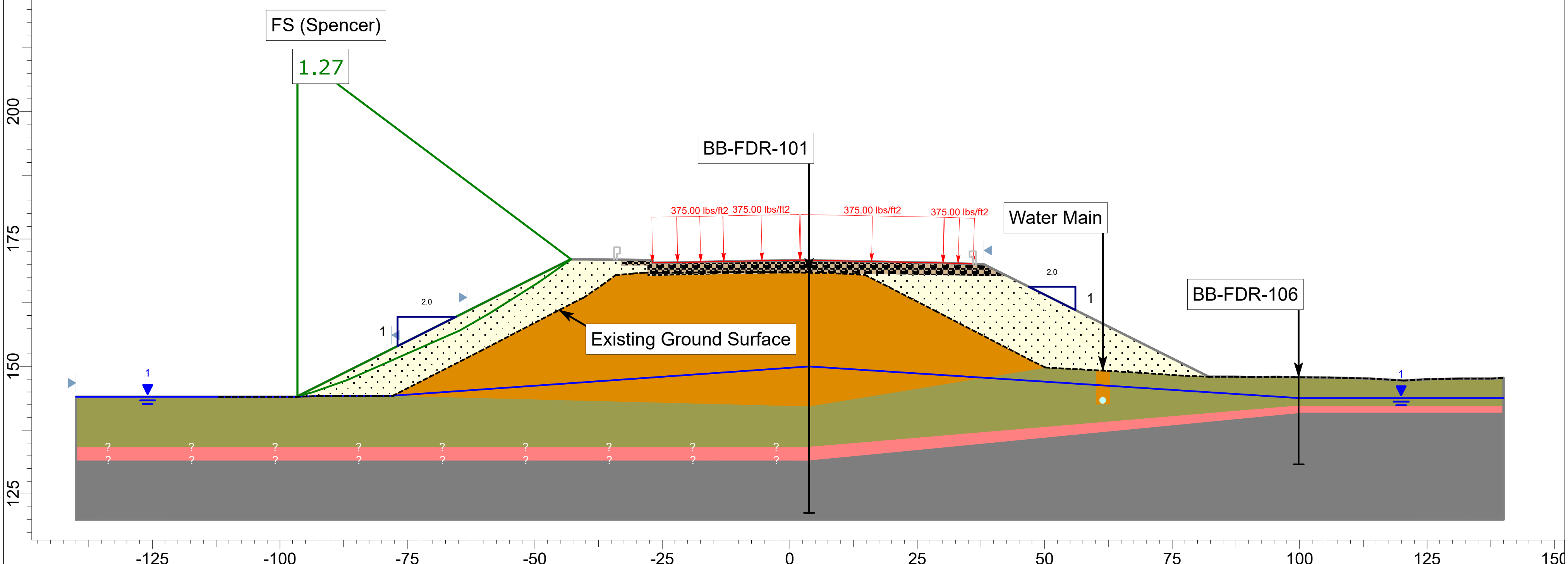
The proposed embankment and slope grading system produces a global stability factor of safety less than the recommended factor of safety of 1.3 for potential surficial slope failures in the embankment fill when using embankment fill engineering parameters recommended in the MaineDOT Bridge Design Guide, and it may require further analysis to stabilize. Failure surfaces wtih FS < 1.3 are surficial in nature. Review of the failure surfaces and material selection should be evaluated in final design.

The analysis of the proposed embankment and slope grading system yields adequate factors of safety (FS > 1.3) for the potential deep seated slope failures in native soils underlying the proposed embankment under static conditions.

The analysis of the proposed embankment and slope grading system yields adequate factors of safety (FS > 1.0) for the potential slope failures in both the native soils underlying the proposed embankment and embankment fills under pseudo-static conditions where seismic loading is applied.

Attachment 1

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	UCS (psf)	GSI	mi	D	Water Surface
Existing Fill		125	Mohr-Coulomb	0	32					Piezometric Line 1
Glaciomarine Silty Clay		125	Mohr-Coulomb	1600	0					Piezometric Line 1
Sand and Gravel		125	Mohr-Coulomb	0	37					Piezometric Line 1
Bedrock		164	Generalized Hoek-Brown			1.86955e+06	60	28	0	Piezometric Line 1
New Fill		125	Mohr-Coulomb	0	32					Piezometric Line 1
New Subbase		135	Mohr-Coulomb	0	36					Piezometric Line 1



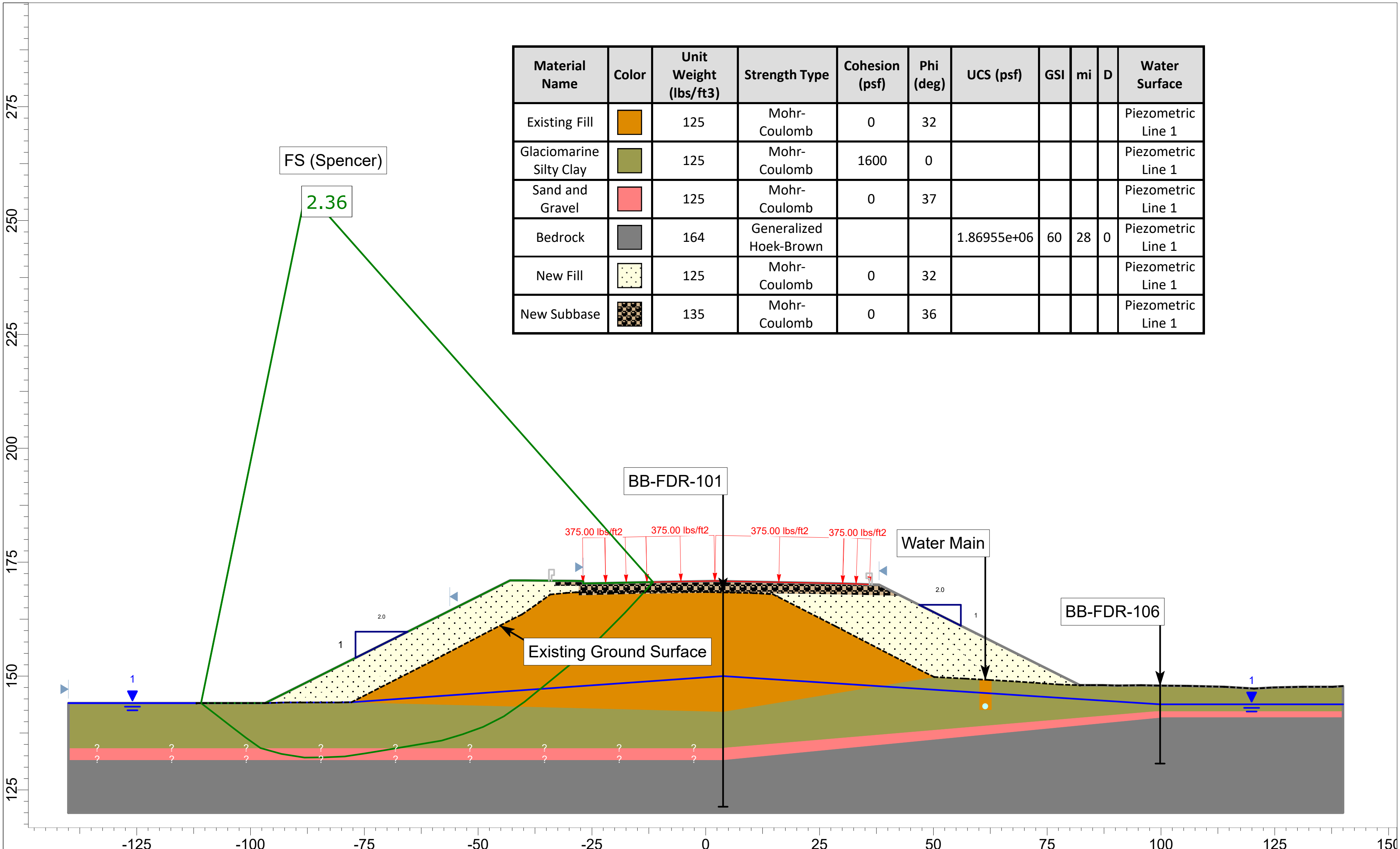
GOLDER

SLIDEINTERPRET 9.007

Project	19126013 MaineDOT I-295 Exit 20 Merrill Road Bridge No. 5720		
Analysis Description	Southern Shift - Station 60+25 North Slope (NonCircular, Cuckoo Search)		
Drawn By KAR	*Checked By* MEL	*Reviewed By* MCM	*Scale* 1:350
Date 7/29/2020	*File Name* Southern Shift 60+25 Freeport Exit 20 19126013.slmd		

Figure A.1

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	UCS (psf)	GSI	mi	D	Water Surface
Existing Fill		125	Mohr-Coulomb	0	32					Piezometric Line 1
Glaciomarine Silty Clay		125	Mohr-Coulomb	1600	0					Piezometric Line 1
Sand and Gravel		125	Mohr-Coulomb	0	37					Piezometric Line 1
Bedrock		164	Generalized Hoek-Brown			1.86955e+06	60	28	0	Piezometric Line 1
New Fill		125	Mohr-Coulomb	0	32					Piezometric Line 1
New Subbase		135	Mohr-Coulomb	0	36					Piezometric Line 1



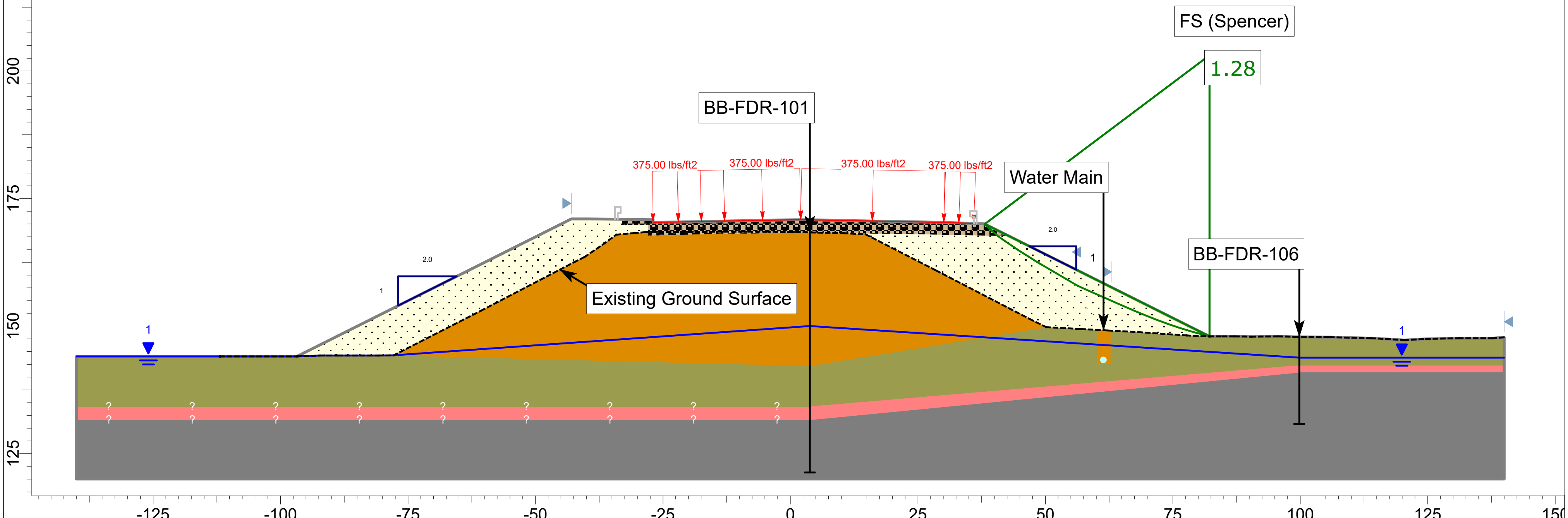
GOLDER

SLIDEINTERPRET 9.007

Project	19126013 MaineDOT I-295 Exit 20 Merrill Road Bridge No. 5720
Analysis Description	Southern Shift - Station 60+25 North Slope (NonCircular, Cuckoo Search)
Drawn By KAR	*Checked By* MEL
Reviewed By MCM	*Scale* 1:350
Date 7/29/2020	*File Name* Southern Shift 60+25 Freeport Exit 20 19126013.slmd

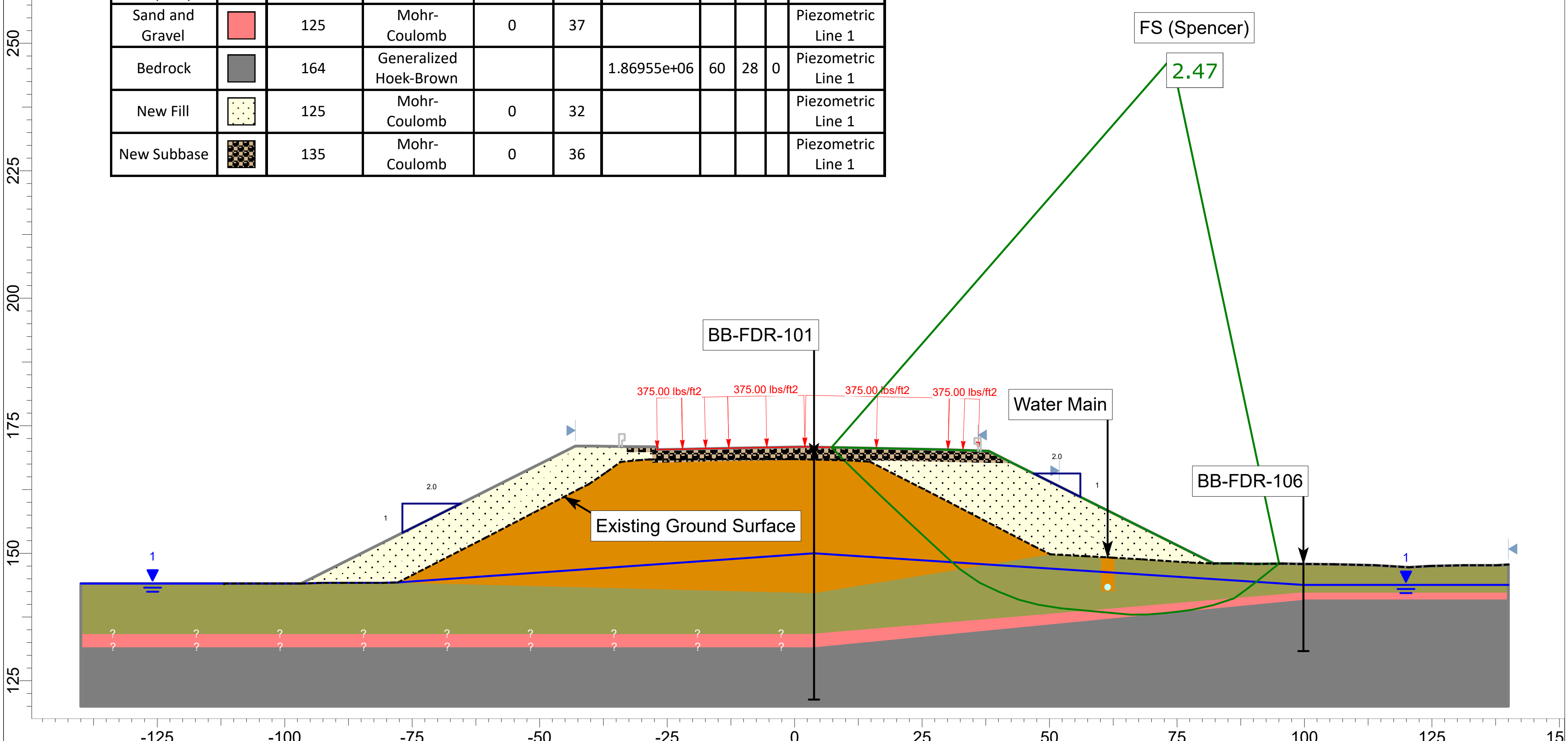
Figure A.2

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	UCS (psf)	GSI	mi	D	Water Surface
Existing Fill		125	Mohr-Coulomb	0	32					Piezometric Line 1
Glaciomarine Silty Clay		125	Mohr-Coulomb	1600	0					Piezometric Line 1
Sand and Gravel		125	Mohr-Coulomb	0	37					Piezometric Line 1
Bedrock		164	Generalized Hoek-Brown			1.86955e+06	60	28	0	Piezometric Line 1
New Fill		125	Mohr-Coulomb	0	32					Piezometric Line 1
New Subbase		135	Mohr-Coulomb	0	36					Piezometric Line 1



GOLDER	*Project* 19126013 MaineDOT I-295 Exit 20 Merrill Road Bridge No. 5720			
	Analysis Description Southern Shift - Station 60+25 South Slope (NonCircular, Cuckoo Search)			
	Drawn By KAR	*Checked By* MEL	*Reviewed By* MCM	*Scale* 1:350
SLIDEINTERPRET 9.007	*Date* 7/29/2020	*File Name* Southern Shift 60+25 Freeport Exit 20 19126013.slmd		**Figure A.3**

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	UCS (psf)	GSI	mi	D	Water Surface
Existing Fill		125	Mohr-Coulomb	0	32					Piezometric Line 1
Glaciomarine Silty Clay		125	Mohr-Coulomb	1600	0					Piezometric Line 1
Sand and Gravel		125	Mohr-Coulomb	0	37					Piezometric Line 1
Bedrock		164	Generalized Hoek-Brown			1.86955e+06	60	28	0	Piezometric Line 1
New Fill		125	Mohr-Coulomb	0	32					Piezometric Line 1
New Subbase		135	Mohr-Coulomb	0	36					Piezometric Line 1



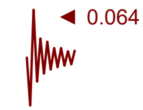
SLIDEINTERPRET 9.007

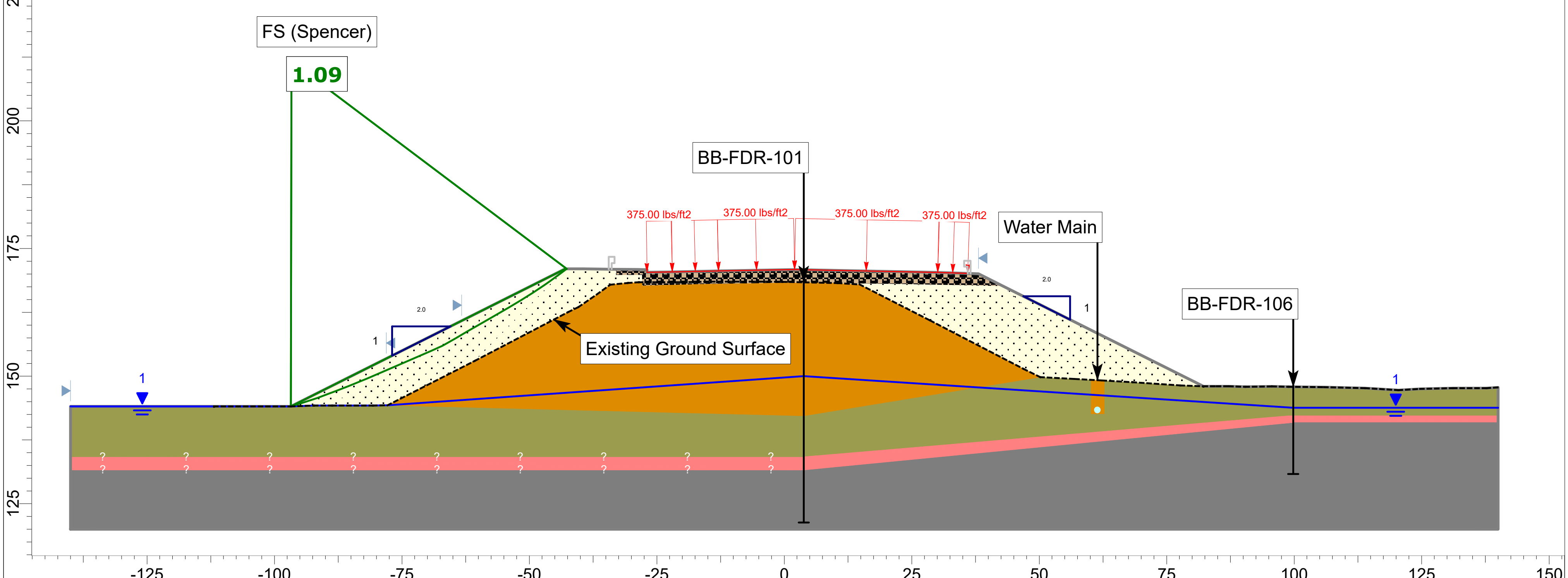
GOLDER

Project	19126013 MaineDOT I-295 Exit 20 Merrill Road Bridge No. 5720
Analysis Description	Southern Shift - Station 60+25 South Slope (NonCircular, Cuckoo Search)
Drawn By KAR	*Checked By* MEL
Reviewed By MCM	*Scale* 1:350
Date 7/29/2020	*File Name* Southern Shift 60+25 Freeport Exit 20 19126013.slmd

Figure A.4

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	UCS (psf)	GSI	mi	D	Water Surface
Existing Fill		125	Mohr-Coulomb	0	32					Piezometric Line 1
Glaciomarine Silty Clay		125	Mohr-Coulomb	1600	0					Piezometric Line 1
Sand and Gravel		125	Mohr-Coulomb	0	37					Piezometric Line 1
Bedrock		164	Generalized Hoek-Brown			1.86955e+06	60	28	0	Piezometric Line 1
New Fill		125	Mohr-Coulomb	0	32					Piezometric Line 1
New Subbase		135	Mohr-Coulomb	0	36					Piezometric Line 1

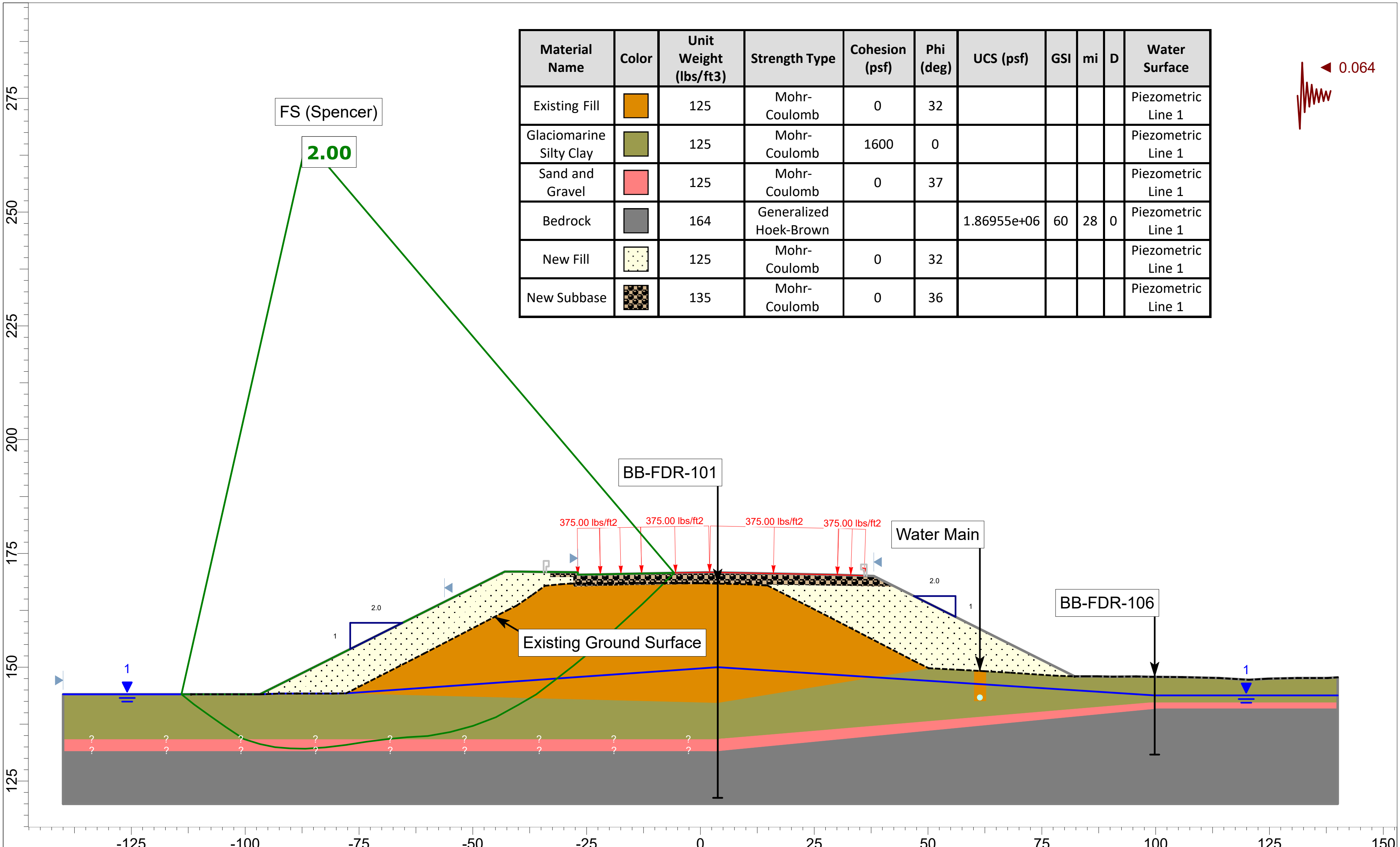




GOLDER	Project	19126013 MaineDOT I-295 Exit 20 Merrill Road Bridge No. 5720	
	Analysis Description	Southern Shift - Station 60+25 North Slope (NonCircular, Cuckoo Search)	
	Drawn By KAR	Checked By MEL	Reviewed By MCM	Scale 1:350
SLIDEINTERPRET 9.010	Date 12/2/2020	File Name Southern Shift 60+25 Freeport Exit 20 19126013 - Seismic.slmd	

Figure B.1

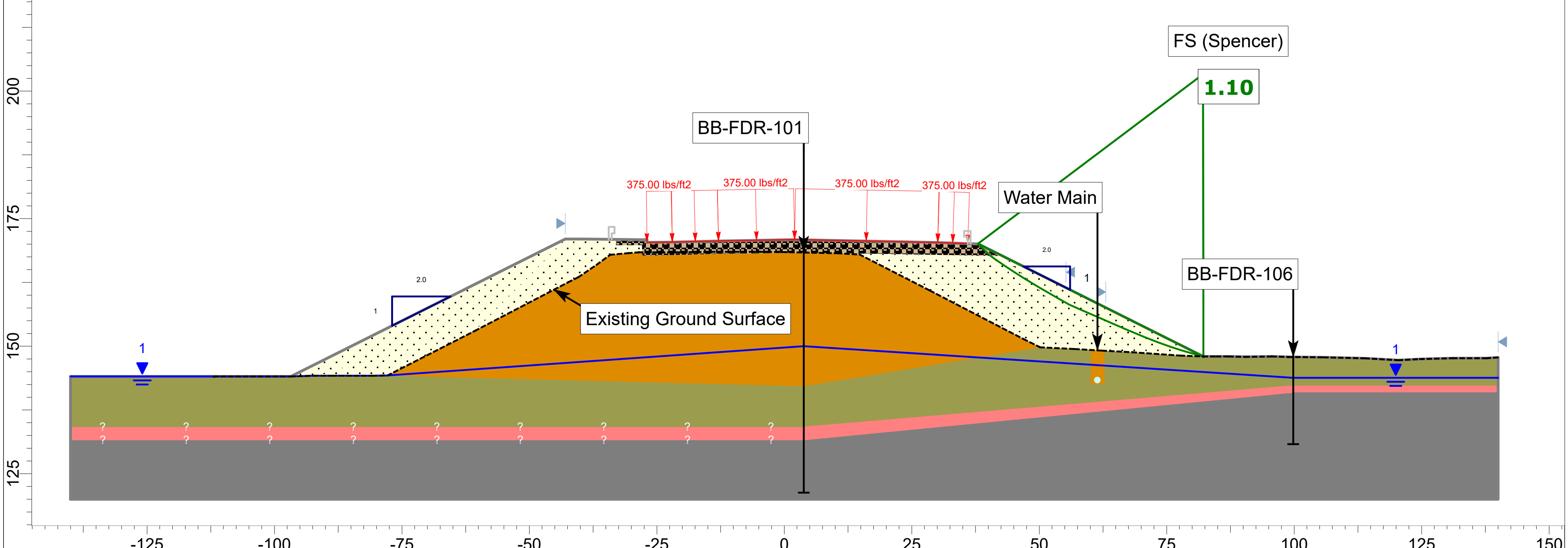
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	UCS (psf)	GSI	mi	D	Water Surface
Existing Fill		125	Mohr-Coulomb	0	32					Piezometric Line 1
Glaciomarine Silty Clay		125	Mohr-Coulomb	1600	0					Piezometric Line 1
Sand and Gravel		125	Mohr-Coulomb	0	37					Piezometric Line 1
Bedrock		164	Generalized Hoek-Brown			1.86955e+06	60	28	0	Piezometric Line 1
New Fill		125	Mohr-Coulomb	0	32					Piezometric Line 1
New Subbase		135	Mohr-Coulomb	0	36					Piezometric Line 1



GOLDER	*Project*	19126013 MaineDOT I-295 Exit 20 Merrill Road Bridge No. 5720				
	Analysis Description	Southern Shift - Station 60+25 North Slope (NonCircular, Cuckoo Search)				
	Drawn By KAR	*Checked By* MEL	*Reviewed By* MCM	*Scale* 1:350	**Figure B.2**	
SLIDEINTERPRET 9.010	*Date* 12/2/2020	*File Name* Southern Shift 60+25 Freeport Exit 20 19126013 - Seismic.slmd				

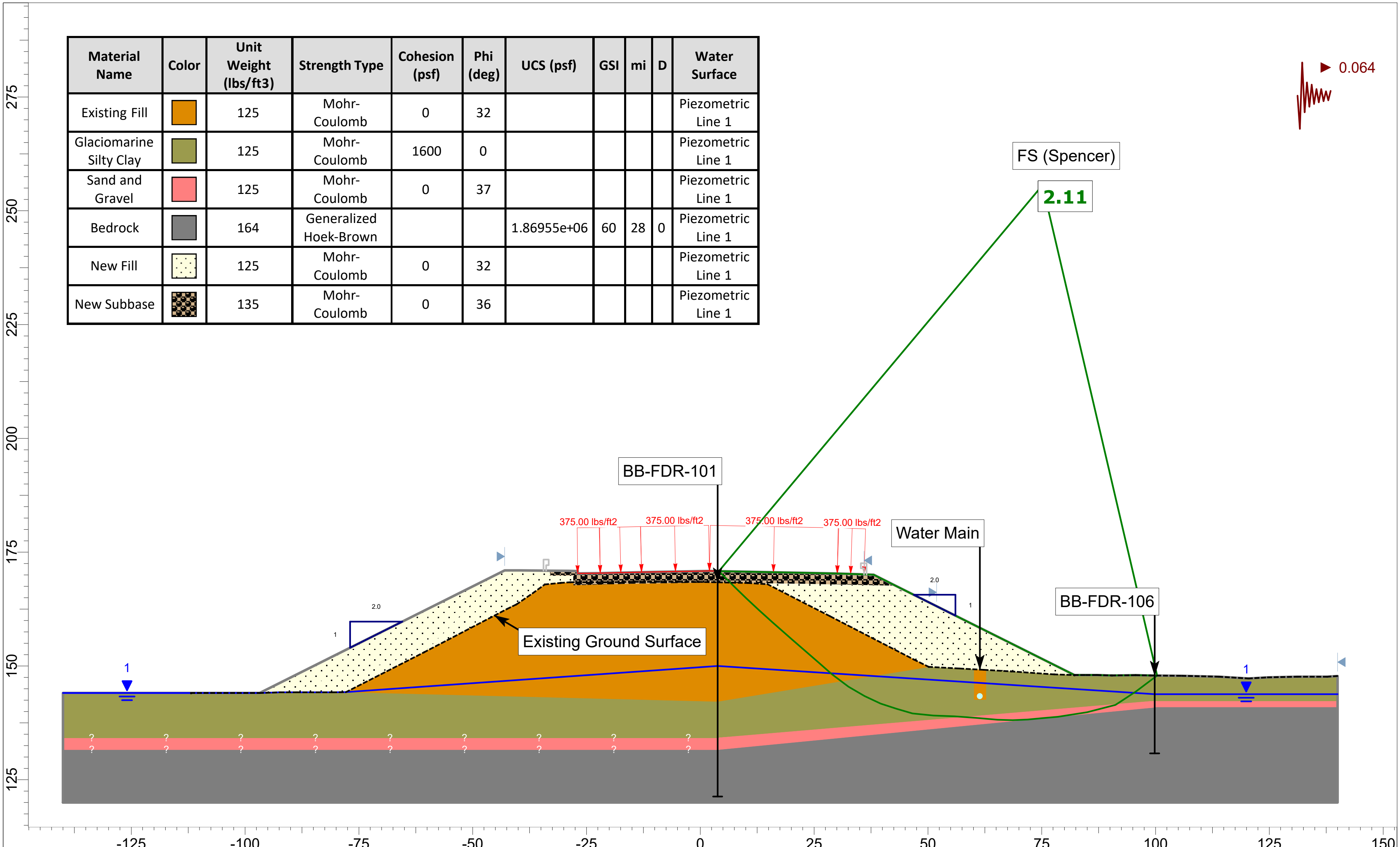
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	UCS (psf)	GSI	mi	D	Water Surface
Existing Fill		125	Mohr-Coulomb	0	32					Piezometric Line 1
Glaciomarine Silty Clay		125	Mohr-Coulomb	1600	0					Piezometric Line 1
Sand and Gravel		125	Mohr-Coulomb	0	37					Piezometric Line 1
Bedrock		164	Generalized Hoek-Brown			1.86955e+06	60	28	0	Piezometric Line 1
New Fill		125	Mohr-Coulomb	0	32					Piezometric Line 1
New Subbase		135	Mohr-Coulomb	0	36					Piezometric Line 1

0.064



GOLDER	Project	19126013 MaineDOT I-295 Exit 20 Merrill Road Bridge No. 5720				
	Analysis Description	Southern Shift - Station 60+25 South Slope (NonCircular, Cuckoo Search)				
	Drawn By KAR	Checked By MEL	Reviewed By MCM	Scale 1:350		Figure B.3
SLIDEINTERPRET 9.010	Date 12/2/2020	File Name Southern Shift 60+25 Freeport Exit 20 19126013 - Seismic.slmd				

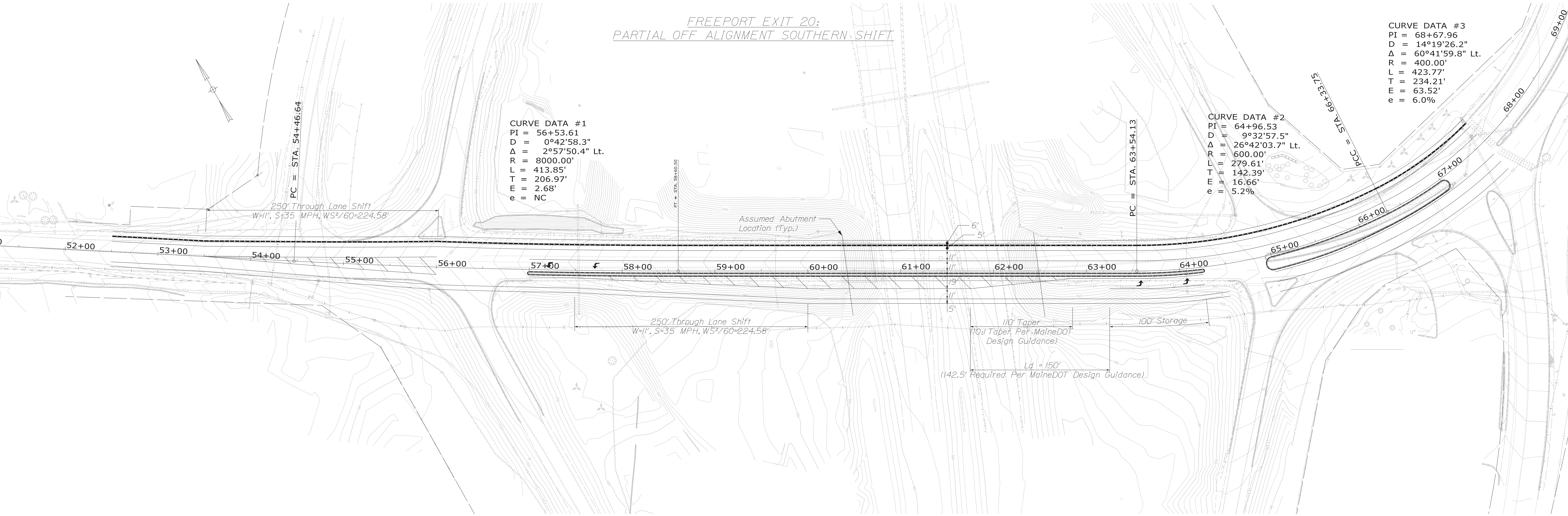
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	UCS (psf)	GSI	mi	D	Water Surface
Existing Fill		125	Mohr-Coulomb	0	32					Piezometric Line 1
Glaciomarine Silty Clay		125	Mohr-Coulomb	1600	0					Piezometric Line 1
Sand and Gravel		125	Mohr-Coulomb	0	37					Piezometric Line 1
Bedrock		164	Generalized Hoek-Brown			1.86955e+06	60	28	0	Piezometric Line 1
New Fill		125	Mohr-Coulomb	0	32					Piezometric Line 1
New Subbase		135	Mohr-Coulomb	0	36					Piezometric Line 1

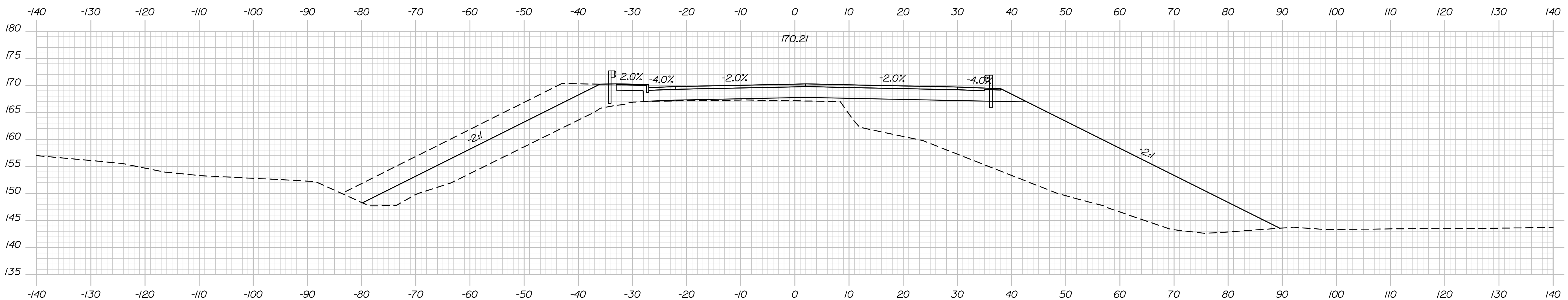


GOLDER	*Project*	19126013 MaineDOT I-295 Exit 20 Merrill Road Bridge No. 5720					
	Analysis Description	Southern Shift - Station 60+25 South Slope (NonCircular, Cuckoo Search)					
	Drawn By	KAR	*Checked By*	MEL	*Reviewed By*	MCM	*Scale* 1:350
SLIDEINTERPRET 9.010	*Date*	12/2/2020	*File Name*	Southern Shift 60+25 Freeport Exit 20 19126013 - Seismic.slmd			

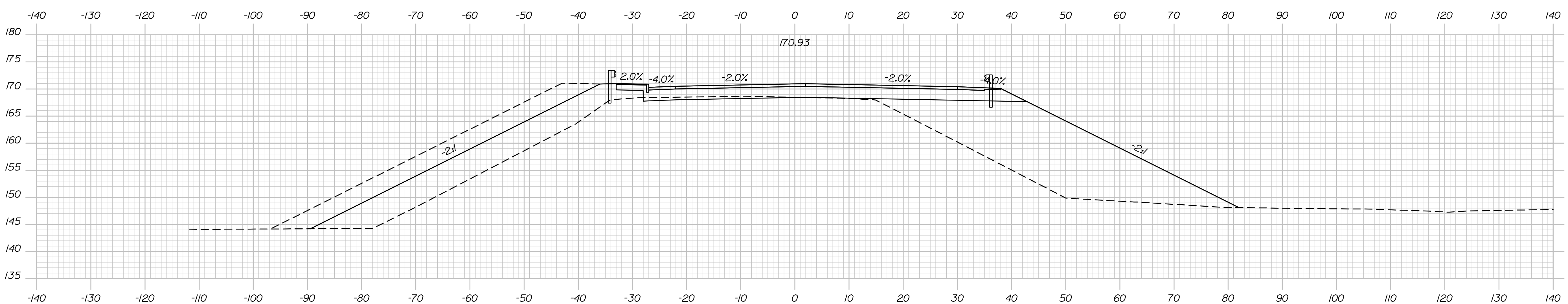
Figure B.4

Attachment 2

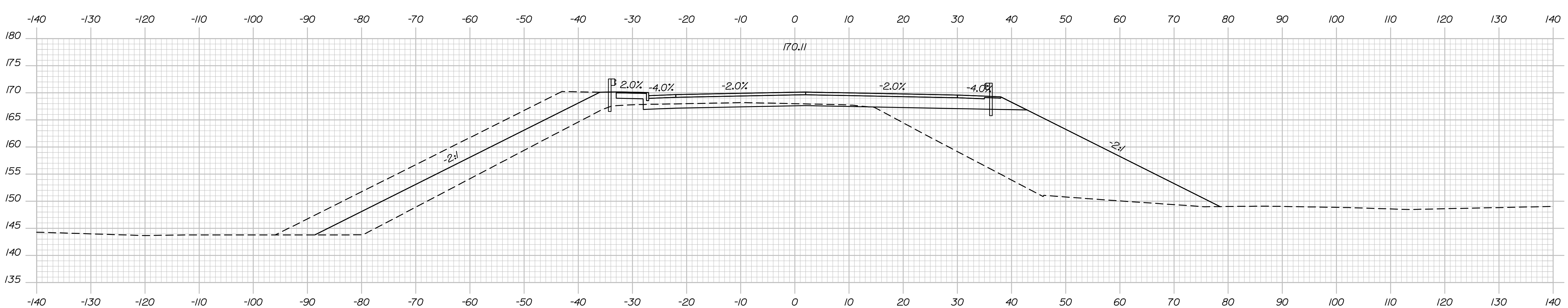




62+50.00



60+25.00



60+00.00

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

HIGHWAY PLANS

WIN
023627.00

Bridge No. 5720

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED		11\19
CHECKED-REVIEWED		11\19
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE
P.E. NUMBER
DATE

MERRILL ROAD BRIDGE
INTERSTATE 295
FREEPORT CUMBERLAND

DESERT ROAD SOUTH
CROSS SECTIONS

SHEET NUMBER
Wor
OF 4

Sta. 60+00.00 to Sta. 62+50.00

GOLDER

CALCULATIONS

Date:	12/2/2020	**Made by:**	KAR
Project No.:	19126013	**Checked by:**	HTV / MEL
Subject:	Global Stability Analysis Abutment No. 2 Embankment	**Reviewed by:**	MCM
Project Short Title:	MaineDOT I-295 Exit 20 Merrill Road Bridge Replacement No. 5720		

OBJECTIVE
Calculate global factor of safety for the southeastern proposed bridge approach embankment, assuming the "southern shift" option with the bike path scenario.

REFERENCES
1. AASHTO LRFD Bridge Design Specifications, 9th Ed, 2020.
2. Email communication between Golder and HNTB, subject "RE: Freeport I-295 Exit 20 and Exit 22 - Request for Sections and Design Info", dated June 22, 2020.
3. Golder geotechnical test boring logs (Appendix A, Preliminary Geotechnical Design Report, dated September 2020).
4. HNTB for State of Maine Department of Transportation. Merrill Road Bridge Freeport Interstate 295: Desert Road South Cross Sections, dated November 2019.
5. Das, Braja M. 2011. Principles of Foundation Engineering, 7th Edition. Cengage Learning.
6. FHWA. 2017. Geotechnical Engineering Circular No. 5: Geotechnical Site Characterization. Publication No. FHWA NHI-16-072.
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8. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.
9. Golder calculation titled "Seismic Site Class" (Appendix E, Preliminary Geotechnical Design Report, dated September 2020).
10. Rocscience Slide Software Package Version 2020 9.007 64-bit, build date May 29, 2020.

ATTACHMENTS
1. Slide output figures
2. HNTB plans showing "southern shift" option

ASSUMPTIONS
1. The load applied by the road and traffic for final design conditions is modeled as a 3 ft equivalent load of soil (Reference 1, Table 3.11.6.4-1) based on a 12 ft abutment height (Reference 2). 3 ft x 125 pcf (fill) = 375 psf.

2. A static FS ≥ 1.3 is recommended for embankment final design conditions per Section 5.9.2 in Reference 8. A pseudo-static FS> 1.0 is recommended per Section 3.7.4.1 in Reference 8

3. Circular surfaces were analyzed using the Spencer and Bishop simplified methods and auto refine search. Non-circular surfaces were analyzed using the Spencer method and cuckoo search with surface altering optimization.

4. The existing stratigraphy and water table surface are estimated from samples and groundwater measurements encountered during drilling and provided in Reference 3.

5. The existing grading, proposed grading, and construction design features are taken from Reference 4.

6. Undrained conditions ($\phi = 0$) were assumed for the glaciomarine silty clay layer.

CALCULATION

1. Determine input parameters to build the soil model in Slide.
The material parameters selected for use in the Slide models are shown in the table below.

Date:	12/2/2020	**Made by:**	KAR
Project No.:	19126013	**Checked by:**	HTV / MEL
Subject:	Global Stability Analysis Abutment No. 2 Embankment	**Reviewed by:**	MCM
Project Short Title:	MaineDOT I-295 Exit 20 Merrill Road Bridge Replacement No. 5720		

a. The friction angle parameters for the existing fill and sand/gravel layers are based on empirical correlation (Reference 5, Eqn. 2.26) to the average of the N_{60}-values encountered in all borings for each layer (Reference 3).

b. The cohesion parameter for the glaciomarine silty clay layer is based on shear strength measurements made in the field and on empirical correlation (Reference 6, Eqn. 7.19) to the average of the N_{60}-values encountered in all borings for each layer (Reference 3).

c. The unit weight parameter for the glaciomarine silty clay layer is calculated from soil moisture contents determined in laboratory testing (Reference 7), assuming 100% saturation. The unit weight parameters for the existing fill and sand/gravel layers are selected based on local engineering experience.

d. The UCS and unit weight parameters for the bedrock are selected based on the average of laboratory test results for all borings (Reference 7). The GSI, m_i, and D parameters for the bedrock are selected based on field descriptions of the rock quality encountered in the borings (Reference 3).

e. The friction angle and unit weight parameters for the construction materials are selected based on MaineDOT standard practice (Reference 8, Table 3-3).

Material Name	Unit Weight (pcf)	Strength Type	Cohesion (psf)	Friction Angle (°)	UCS (psf)	GSI	m_i	D
Existing Fill	125	Mohr-Coulomb	0	32	-	-	-	-
Glaciomarine Silty Clay	125	Mohr-Coulomb	1600	0	-	-	-	-
Sand and Gravel	125	Mohr-Coulomb	0	37	-	-	-	-
Bedrock	164	Generalized Hoek-Brown	-	-	1,869,552	60	28	0
New Fill	125	Mohr-Coulomb	0	32	-	-	-	-
New Subbase	135	Mohr-Coulomb	0	36	-	-	-	-

2. Use the soil layer parameters listed above to analyze the slope stability with Slide.

The soil layer properties above were used to analyze the following scenarios. HNTB provided Golder with both a northern shift and southern shift of the proposed bridge centerline alignment. Following analysis, Golder determined that the southern shift in the alignment would result in the largest height and offset of embankment fills over the existing ground surface, which would likely have the greatest impact in stability. Therefore the southern shift option was analyzed as the critical case. This analysis evaluates the fills at the southeastern abutment. Both the northern and the southern fill slopes were analyzed at the southeastern abutment. The results of the Slide stability analyses are summarized in the following table.

GOLDER

CALCULATIONS

Date:	12/2/2020	**Made by:**	KAR
Project No.:	19126013	**Checked by:**	HTV / MEL
Subject:	Global Stability Analysis Abutment No. 2 Embankment	**Reviewed by:**	MCM
Project Short Title:	MaineDOT I-295 Exit 20 Merrill Road Bridge Replacement No. 5720		

Baseline	Station	Feature	Slope	Lowest Factor of Safety (Spencer Method)			
				NonCircular Failure Surface Through Proposed Fill		NonCircular Failure Surface Through Glaciomarine Deposit	
Southern Shift	62+50	Southeast Embankment	North	1.28	(Fig. A.1)	2.33	(Fig. A.2)
			South	1.27	(Fig. A.3)	2.15	(Fig. A.4)

Circular Surfaces:

Circular failure surfaces using Spencer and Bishop simplified methods were also evaluated for each scenario. The circular surfaces that were produced were similar in location and size to the noncircular surfaces and had factors of safety similar in magnitude to the noncircular factors of safety. The circular factors of safety ranged from 1.27 to 1.29 for surfaces through the proposed fill and from 2.28 to 2.44 for surfaces through the glaciomarine deposit.

3. Repeat the Slide analysis with pseudo-static seismic load conditions.

The same scenarios were also analyzed with a horizontal seismic load coefficient of $A_s/2 = 0.064$ (A_s from Reference 9) as recommended in AASHTO (Reference 1) Appendix 11A. The results of the seismic Slide stability analyses are summarized in the following table.

Baseline	Station	Feature	Slope	Lowest Factor of Safety (Spencer Method)			
				NonCircular Failure Surface Through Proposed Fill		NonCircular Failure Surface Through Glaciomarine Deposit	
Southern Shift	62+50	Southeast Embankment	North	1.10	(Fig. B.1)	1.98	(Fig. B.2)
			South	1.09	(Fig. B.3)	1.84	(Fig. B.4)

Circular Surfaces:

Circular failure surfaces using Spencer and Bishop simplified methods were also evaluated for each scenario. The circular surfaces that were produced were similar in location and size to the noncircular surfaces and had factors of safety similar in magnitude to the noncircular factors of safety. The circular factors of safety ranged from 1.09 to 1.11 for surfaces through the proposed fill and from 1.96 to 2.05 for surfaces through the glaciomarine deposit.

CONCLUSIONS

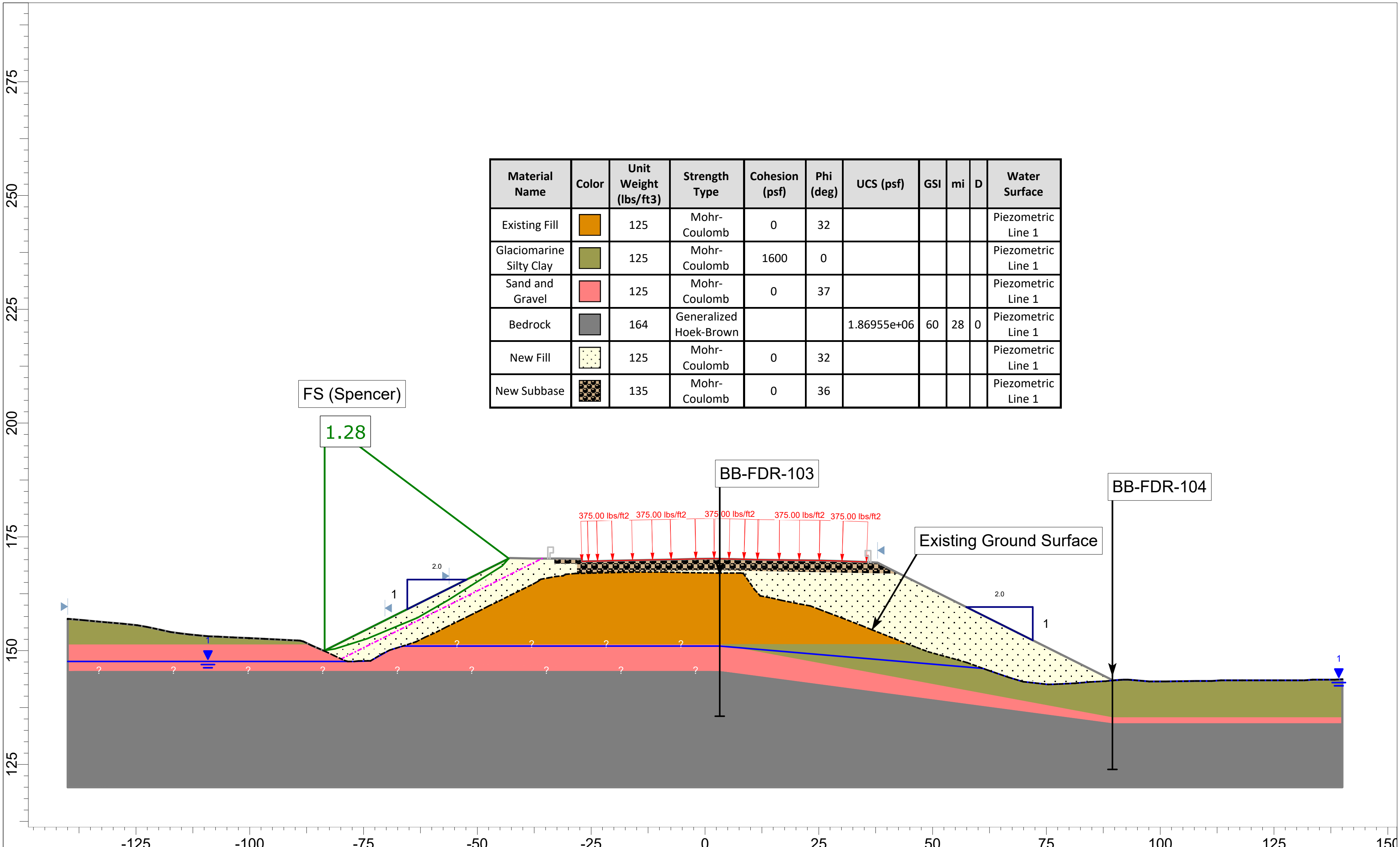
The proposed embankment and slope grading system produces a global stability factor of safety less than the recommended factor of safety of 1.3 for potential surficial slope failures in the embankment fill when using embankment fill engineering parameters recommended in the MaineDOT Bridge Design Guide, and it may require further analysis to stabilize.

The analysis of the proposed embankment and slope grading system yields adequate factors of safety (FS > 1.3) for the potential deep seated slope failures in native soils underlying the proposed embankment under static conditions.

The analysis of the proposed embankment and slope grading system yields adequate factors of safety (FS > 1.0) for the potential slope failures in both the native soils underlying the proposed embankment and embankment fills under pseudo-static conditions where seismic loading is applied.

Attachment 1

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	UCS (psf)	GSI	mi	D	Water Surface
Existing Fill		125	Mohr-Coulomb	0	32					Piezometric Line 1
Glaciomarine Silty Clay		125	Mohr-Coulomb	1600	0					Piezometric Line 1
Sand and Gravel		125	Mohr-Coulomb	0	37					Piezometric Line 1
Bedrock		164	Generalized Hoek-Brown			1.86955e+06	60	28	0	Piezometric Line 1
New Fill		125	Mohr-Coulomb	0	32					Piezometric Line 1
New Subbase		135	Mohr-Coulomb	0	36					Piezometric Line 1

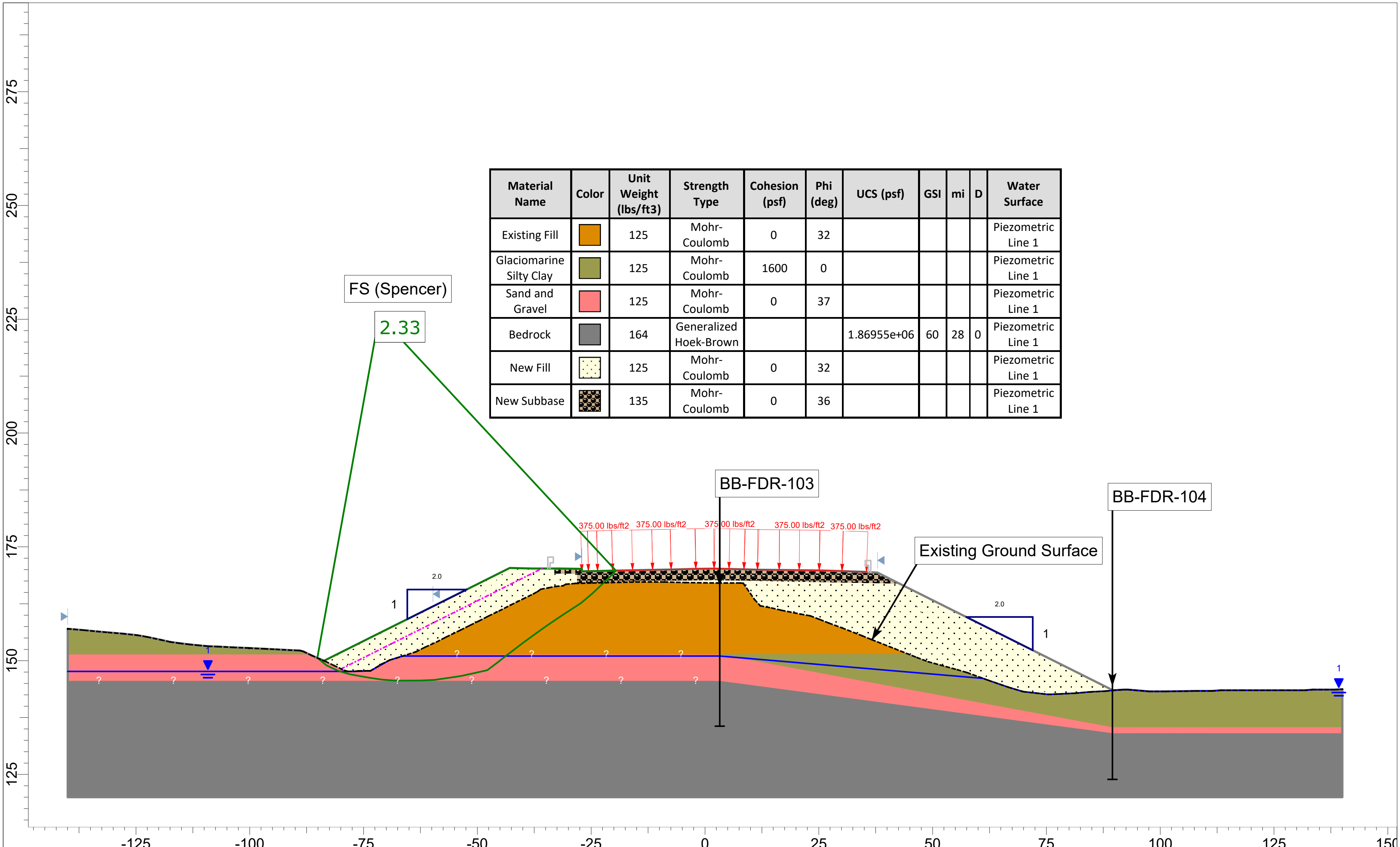


SLIDEINTERPRET 9.007

GOLDER

Project	19126013 MaineDOT I-295 Exit 20 Merrill Road Bridge No. 5720
Analysis Description	Southern Shift - Station 62+50 North Slope (NonCircular, Cuckoo Search)
Drawn By	KAR
Checked By	HTV
Reviewed By	MCM
Scale	1:350
Date	7/7/2020
File Name	Southern Shift 62+50 Freeport Exit 20 19126013.slmd

Figure A.1



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	UCS (psf)	GSI	mi	D	Water Surface
Existing Fill		125	Mohr-Coulomb	0	32					Piezometric Line 1
Glaciomarine Silty Clay		125	Mohr-Coulomb	1600	0					Piezometric Line 1
Sand and Gravel		125	Mohr-Coulomb	0	37					Piezometric Line 1
Bedrock		164	Generalized Hoek-Brown			1.86955e+06	60	28	0	Piezometric Line 1
New Fill		125	Mohr-Coulomb	0	32					Piezometric Line 1
New Subbase		135	Mohr-Coulomb	0	36					Piezometric Line 1

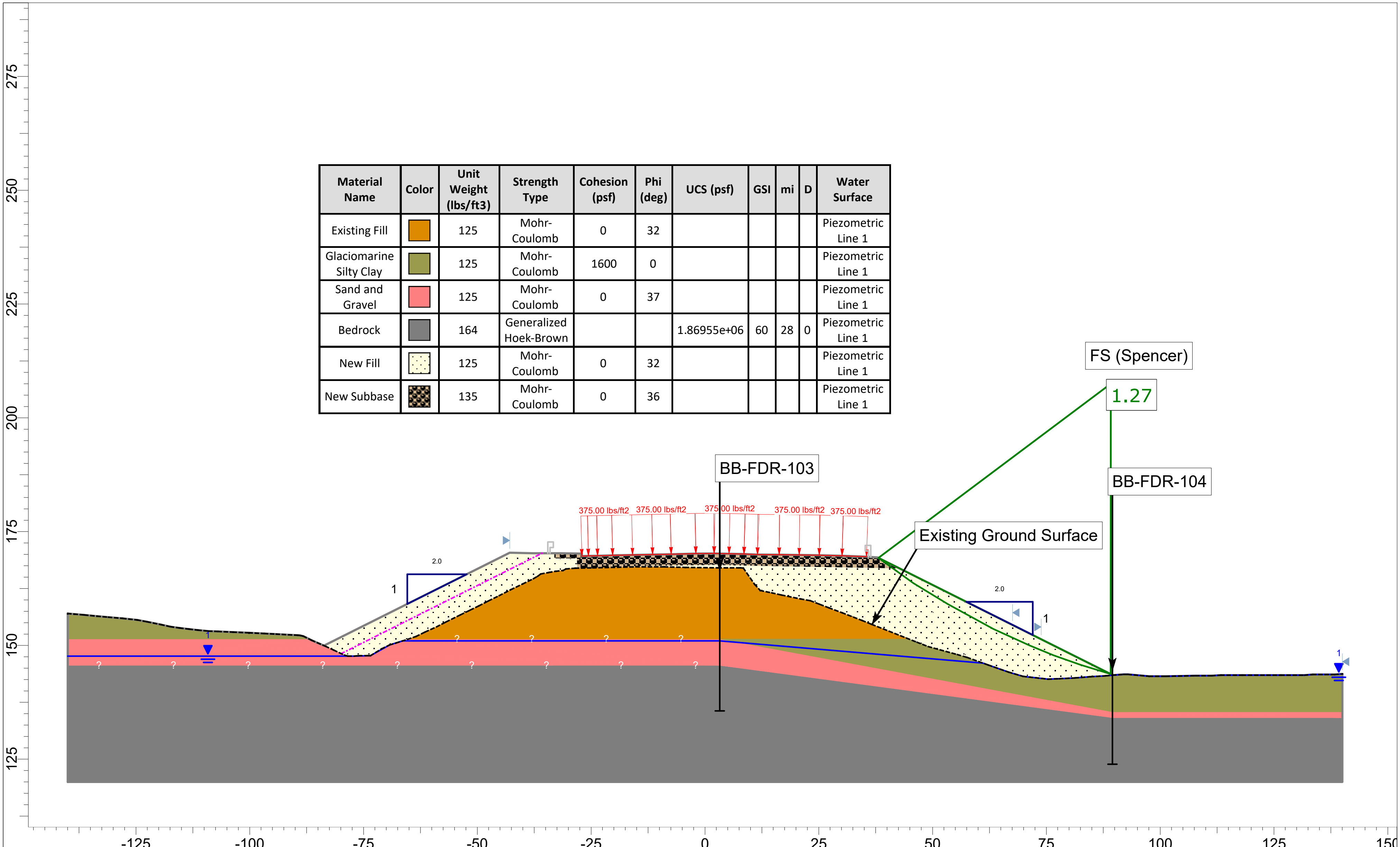
GOLDER

SLIDEINTERPRET 9.007

Project 19126013 MaineDOT I-295 Exit 20 Merrill Road Bridge No. 5720

Analysis Description Southern Shift - Station 62+50 North Slope (NonCircular, Cuckoo Search)

Drawn By KAR	*Checked By* HTV	*Reviewed By* MCM	*Scale* 1:350	**Figure A.2**
Date 7/7/2020	*File Name* Southern Shift 62+50 Freeport Exit 20 19126013.slmd			



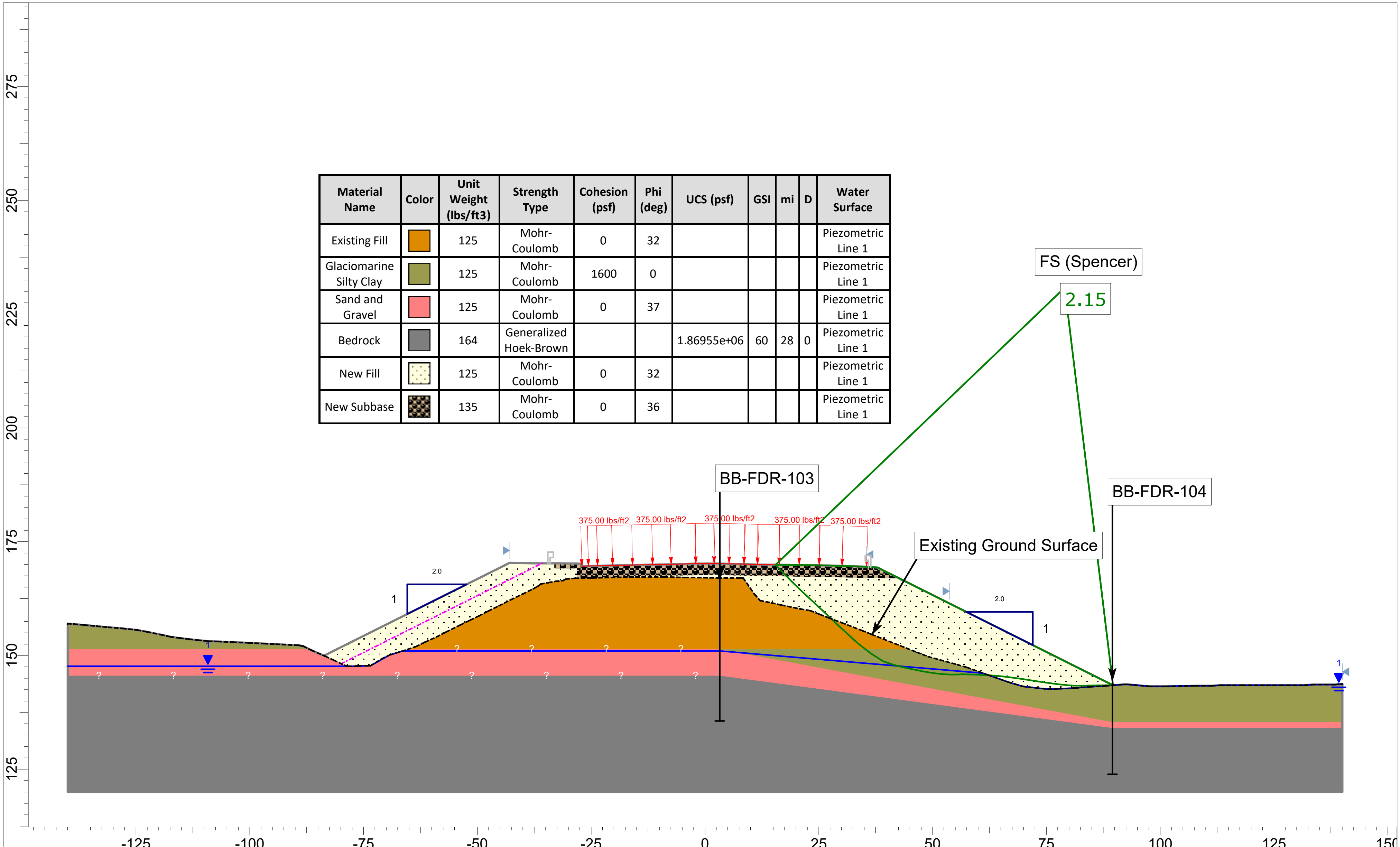
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	UCS (psf)	GSI	mi	D	Water Surface
Existing Fill		125	Mohr-Coulomb	0	32					Piezometric Line 1
Glaciomarine Silty Clay		125	Mohr-Coulomb	1600	0					Piezometric Line 1
Sand and Gravel		125	Mohr-Coulomb	0	37					Piezometric Line 1
Bedrock		164	Generalized Hoek-Brown			1.86955e+06	60	28	0	Piezometric Line 1
New Fill		125	Mohr-Coulomb	0	32					Piezometric Line 1
New Subbase		135	Mohr-Coulomb	0	36					Piezometric Line 1

GOLDER

SLIDEINTERPRET 9.007

Project	19126013 MaineDOT I-295 Exit 20 Merrill Road Bridge No. 5720
Analysis Description	Southern Shift - Station 62+50 South Slope (NonCircular, Cuckoo Search)
Drawn By KAR	*Checked By* HTV
Reviewed By MCM	*Scale* 1:350
Date 7/7/2020	*File Name* Southern Shift 62+50 Freeport Exit 20 19126013.slmd

Figure A.3



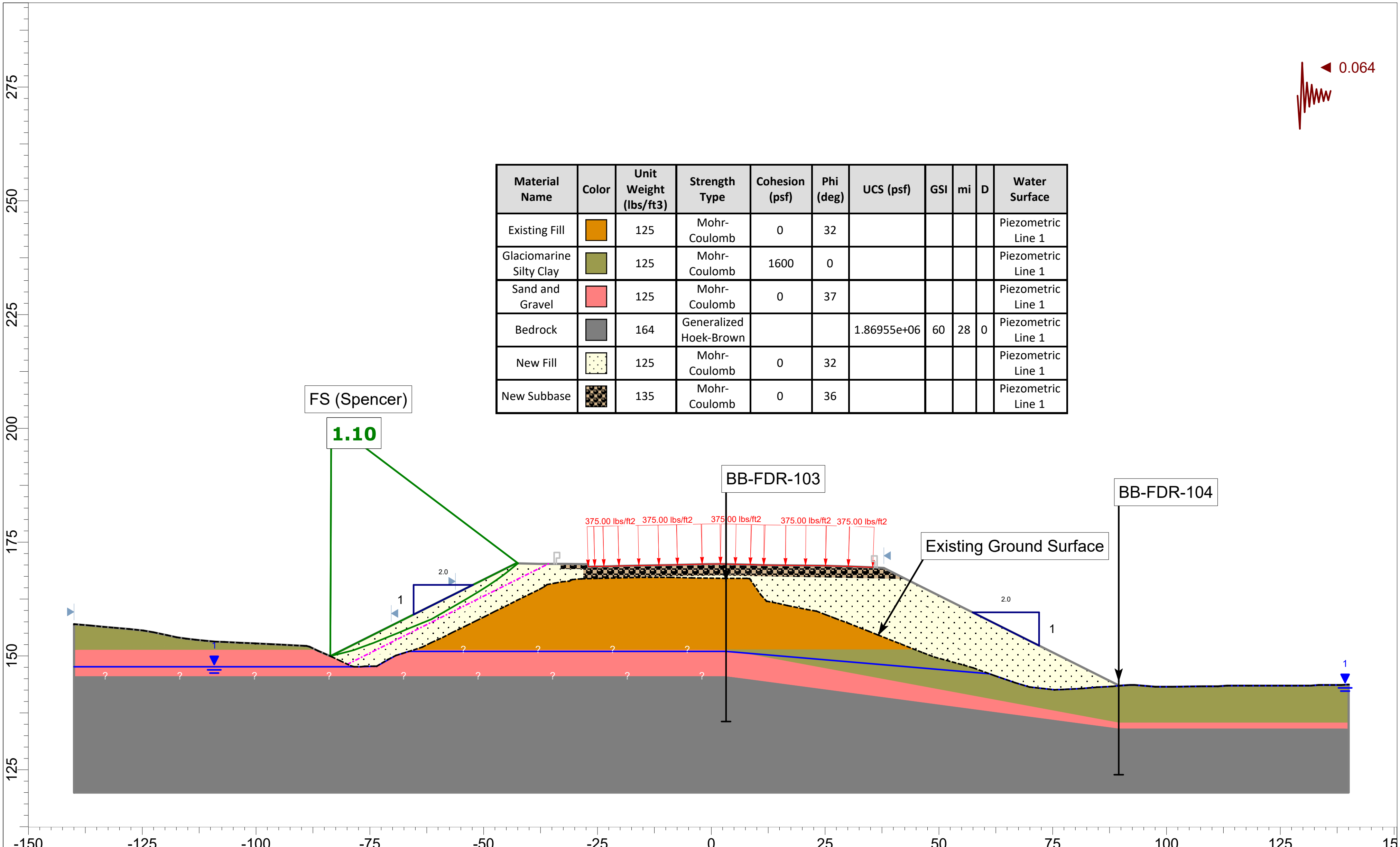
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	UCS (psf)	GSI	mi	D	Water Surface
Existing Fill		125	Mohr-Coulomb	0	32					Piezometric Line 1
Glaciomarine Silty Clay		125	Mohr-Coulomb	1600	0					Piezometric Line 1
Sand and Gravel		125	Mohr-Coulomb	0	37					Piezometric Line 1
Bedrock		164	Generalized Hoek-Brown			1.86955e+06	60	28	0	Piezometric Line 1
New Fill		125	Mohr-Coulomb	0	32					Piezometric Line 1
New Subbase		135	Mohr-Coulomb	0	36					Piezometric Line 1

GOLDER

SLIDEINTERPRET 9.007

Project	19126013 MaineDOT I-295 Exit 20 Merrill Road Bridge No. 5720
Analysis Description	Southern Shift - Station 62+50 South Slope (NonCircular, Cuckoo Search)
Drawn By	KAR
Checked By	HTV
Reviewed By	MCM
Scale	1:350
Date	7/7/2020
File Name	Southern Shift 62+50 Freeport Exit 20 19126013.slmd

Figure A.4



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	UCS (psf)	GSI	mi	D	Water Surface
Existing Fill		125	Mohr-Coulomb	0	32					Piezometric Line 1
Glaciomarine Silty Clay		125	Mohr-Coulomb	1600	0					Piezometric Line 1
Sand and Gravel		125	Mohr-Coulomb	0	37					Piezometric Line 1
Bedrock		164	Generalized Hoek-Brown			1.86955e+06	60	28	0	Piezometric Line 1
New Fill		125	Mohr-Coulomb	0	32					Piezometric Line 1
New Subbase		135	Mohr-Coulomb	0	36					Piezometric Line 1

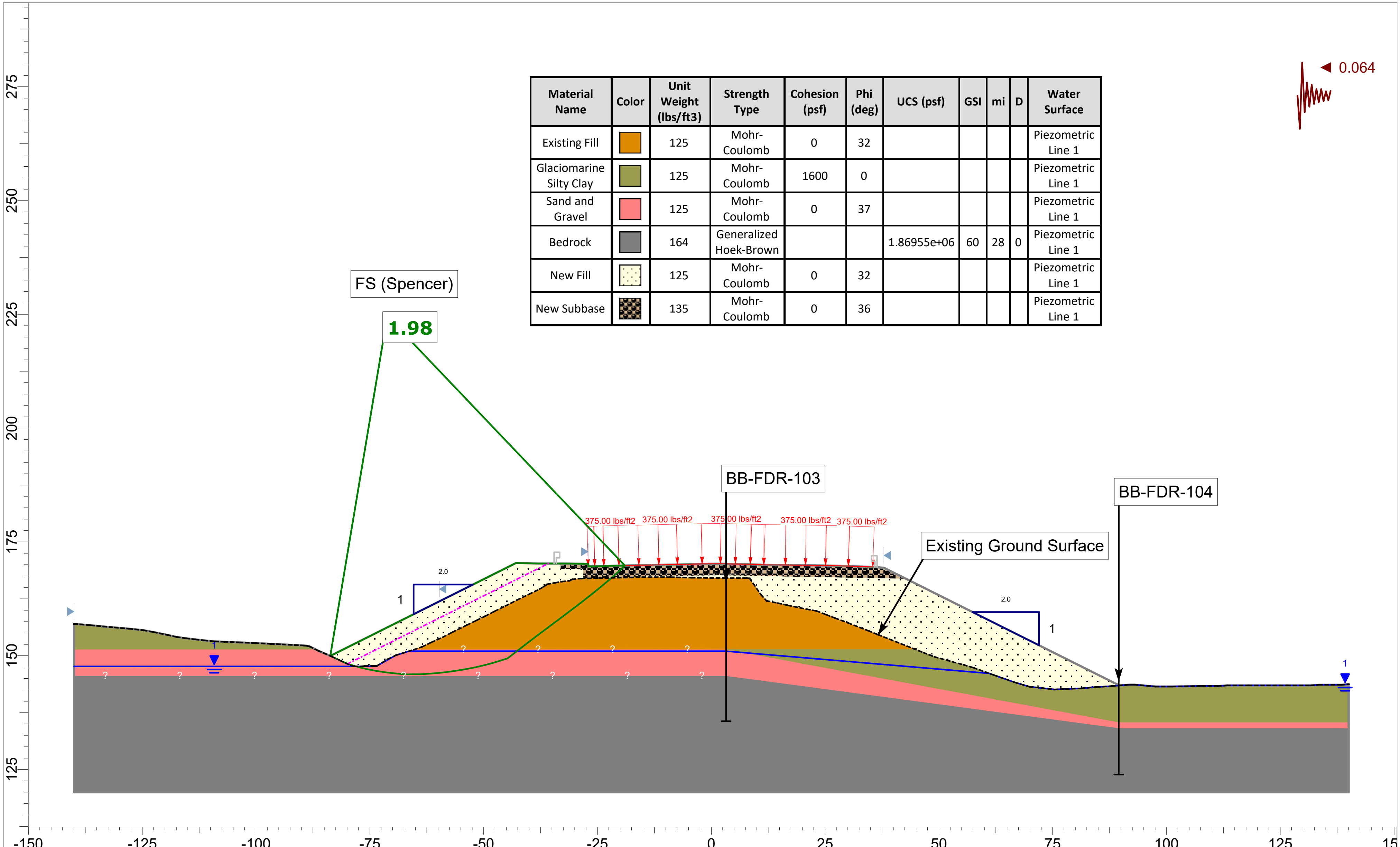
GOLDER

SLIDEINTERPRET 9.010

Project	19126013 MaineDOT I-295 Exit 20 Merrill Road Bridge No. 5720
Analysis Description	Southern Shift - Station 62+50 North Slope (NonCircular, Cuckoo Search)
Drawn By KAR	*Checked By* MEL
Reviewed By MCM	*Scale* 1:350
Date 12/2/2020	*File Name* Southern Shift 62+50 Freeport Exit 20 19126013 - Seismic.slmd

Figure B.1

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	UCS (psf)	GSI	mi	D	Water Surface
Existing Fill		125	Mohr-Coulomb	0	32					Piezometric Line 1
Glaciomarine Silty Clay		125	Mohr-Coulomb	1600	0					Piezometric Line 1
Sand and Gravel		125	Mohr-Coulomb	0	37					Piezometric Line 1
Bedrock		164	Generalized Hoek-Brown			1.86955e+06	60	28	0	Piezometric Line 1
New Fill		125	Mohr-Coulomb	0	32					Piezometric Line 1
New Subbase		135	Mohr-Coulomb	0	36					Piezometric Line 1



GOLDER

SLIDEINTERPRET 9.010

Project 19126013 MaineDOT I-295 Exit 20 Merrill Road Bridge No. 5720

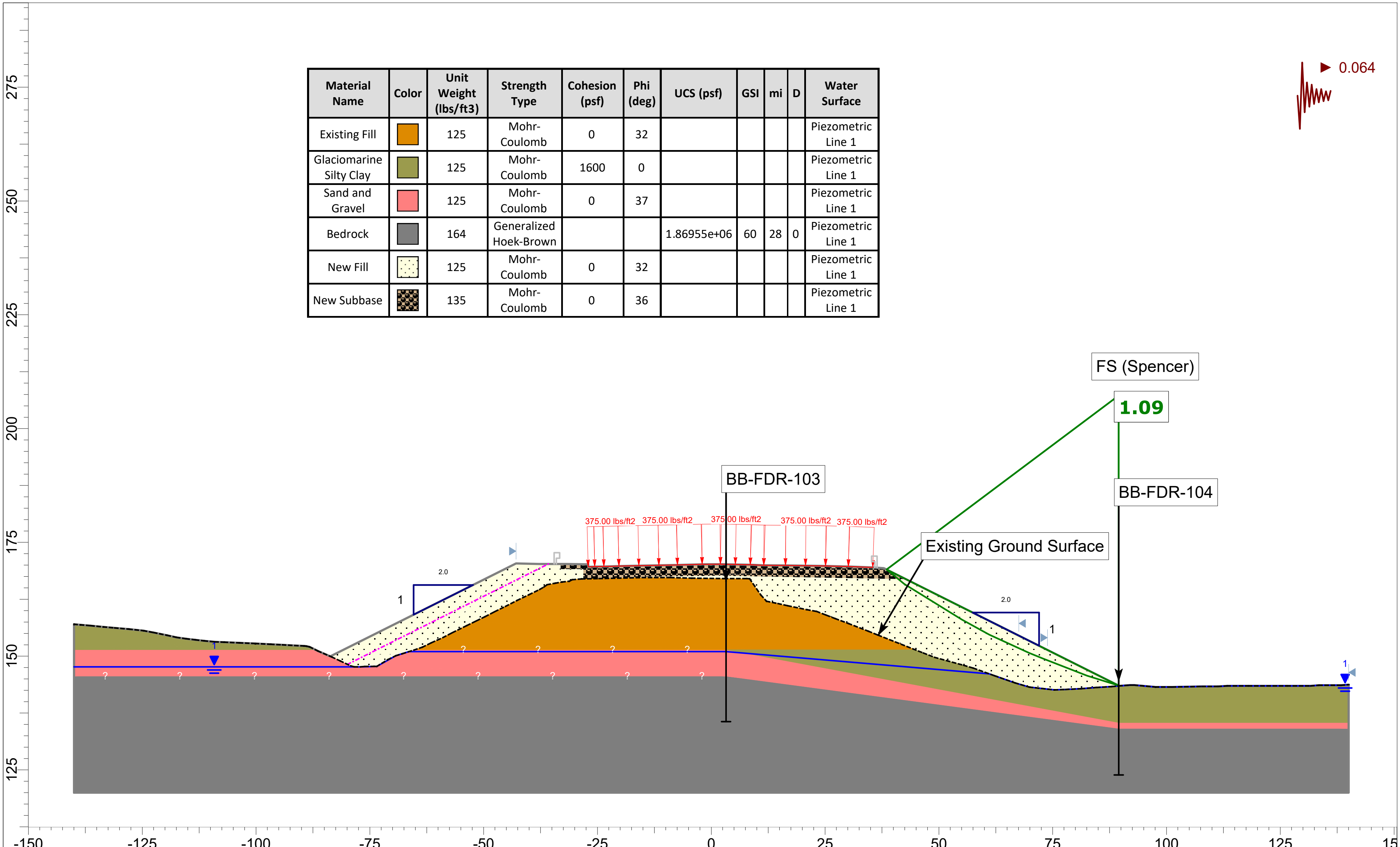
Analysis Description Southern Shift - Station 62+50 North Slope (NonCircular, Cuckoo Search)

Drawn By KAR *Checked By* MEL *Reviewed By* MCM *Scale* 1:350

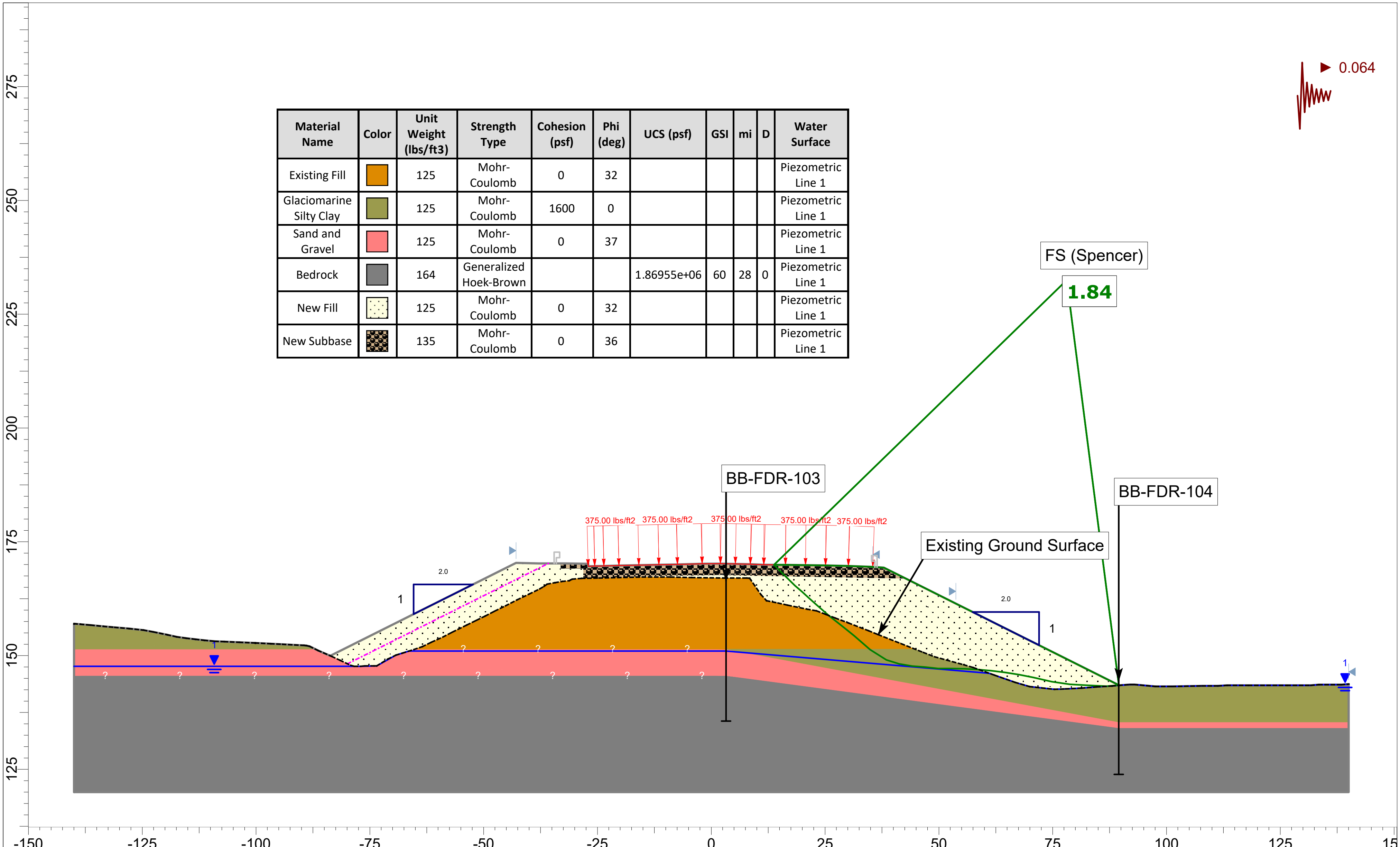
Date 12/2/2020 *File Name* Southern Shift 62+50 Freeport Exit 20 19126013 - Seismic.slmd

Figure B.2

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	UCS (psf)	GSI	mi	D	Water Surface
Existing Fill		125	Mohr-Coulomb	0	32					Piezometric Line 1
Glaciomarine Silty Clay		125	Mohr-Coulomb	1600	0					Piezometric Line 1
Sand and Gravel		125	Mohr-Coulomb	0	37					Piezometric Line 1
Bedrock		164	Generalized Hoek-Brown			1.86955e+06	60	28	0	Piezometric Line 1
New Fill		125	Mohr-Coulomb	0	32					Piezometric Line 1
New Subbase		135	Mohr-Coulomb	0	36					Piezometric Line 1



GOLDER	*Project*	19126013 MaineDOT I-295 Exit 20 Merrill Road Bridge No. 5720		
	Analysis Description	Southern Shift - Station 62+50 South Slope (NonCircular, Cuckoo Search)		
	Drawn By KAR	*Checked By* MEL	*Reviewed By* MCM	*Scale* 1:350
SLIDEINTERPRET 9.010	*Date* 12/2/2020	*File Name* Southern Shift 62+50 Freeport Exit 20 19126013 - Seismic.slmd		**Figure B.3**



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	UCS (psf)	GSI	mi	D	Water Surface
Existing Fill		125	Mohr-Coulomb	0	32					Piezometric Line 1
Glaciomarine Silty Clay		125	Mohr-Coulomb	1600	0					Piezometric Line 1
Sand and Gravel		125	Mohr-Coulomb	0	37					Piezometric Line 1
Bedrock		164	Generalized Hoek-Brown			1.86955e+06	60	28	0	Piezometric Line 1
New Fill		125	Mohr-Coulomb	0	32					Piezometric Line 1
New Subbase		135	Mohr-Coulomb	0	36					Piezometric Line 1

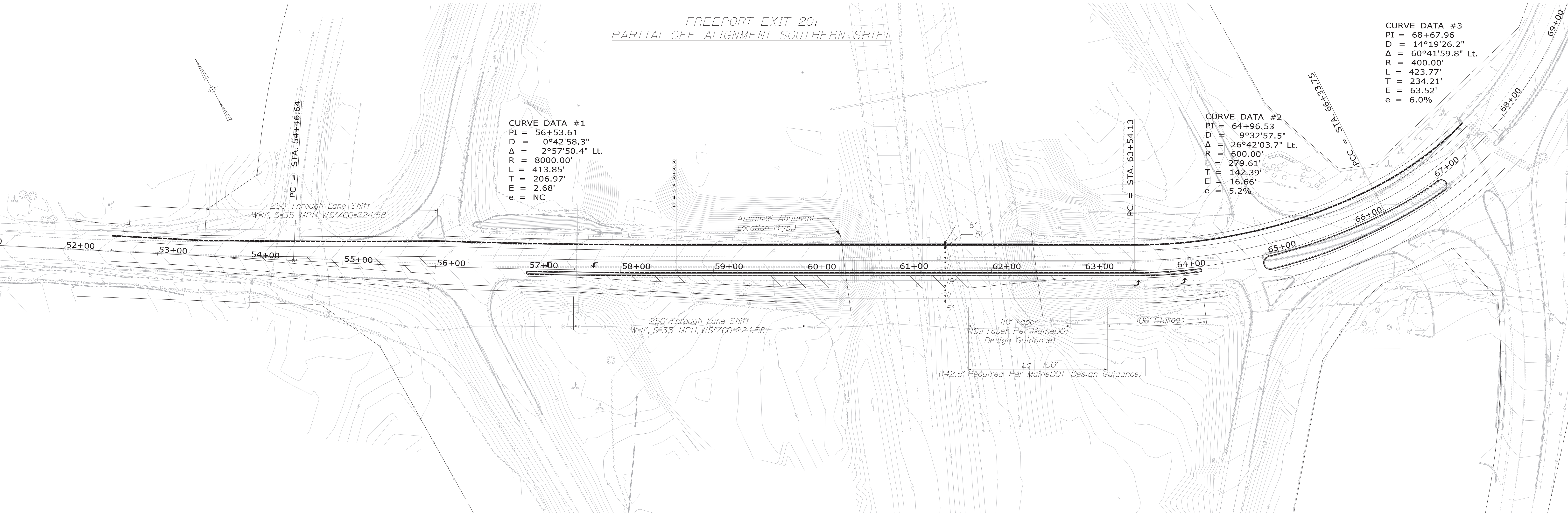
GOLDER

SLIDEINTERPRET 9.010

Project	19126013 MaineDOT I-295 Exit 20 Merrill Road Bridge No. 5720
Analysis Description	Southern Shift - Station 62+50 South Slope (NonCircular, Cuckoo Search)
Drawn By KAR	*Checked By* MEL
Reviewed By MCM	*Scale* 1:350
Date 12/2/2020	*File Name* Southern Shift 62+50 Freeport Exit 20 19126013 - Seismic.slmd

Figure B.4

Attachment 2

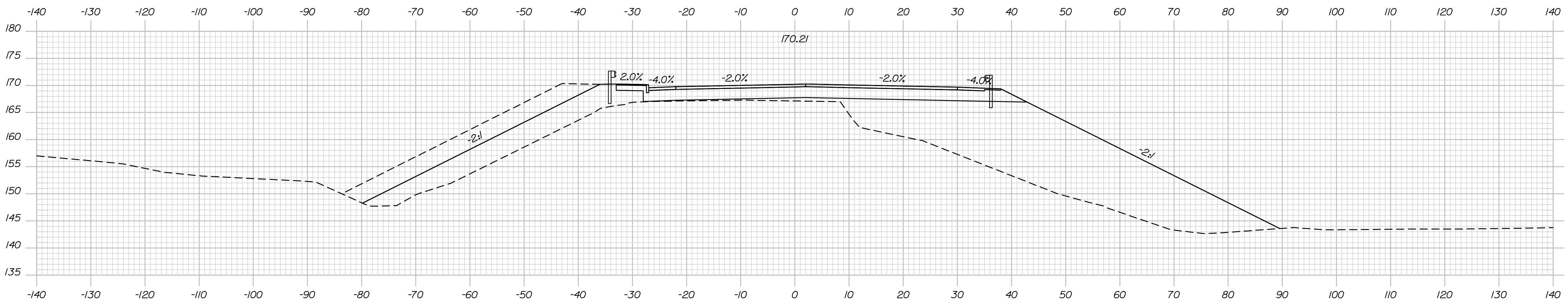


Date:6/22/2020

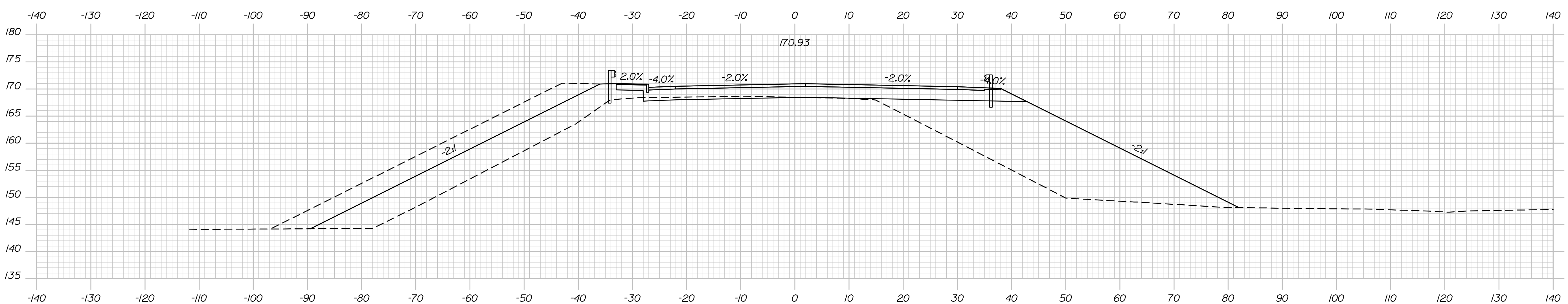
Username:

Division:

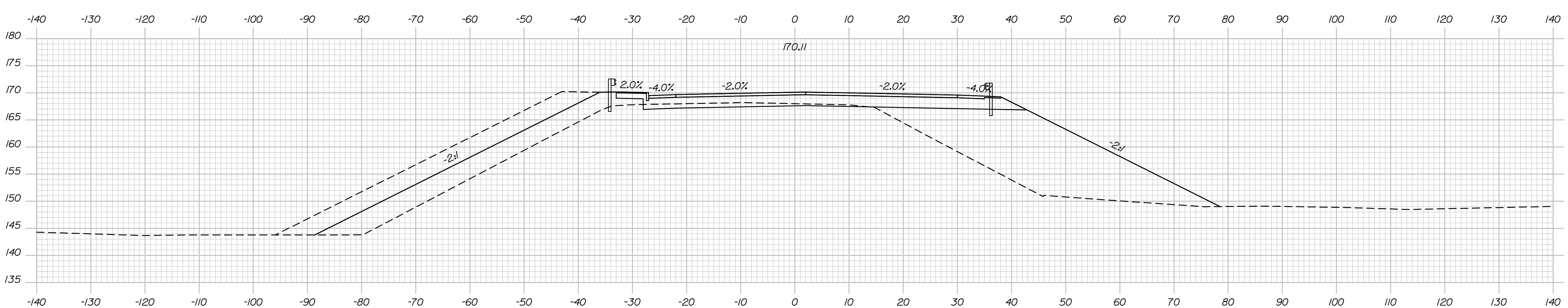
Filename: Working Sections.dgn



62+50.00



60+25.00



60+00.00

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

HIGHWAY PLANS

WIN
023627.00

Bridge No. 5720

SIGNATURE

P.E. NUMBER

DATE

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED		11/19
CHECKED-REVIEWED		11/19
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

MERRILL ROAD BRIDGE
INTERSTATE 295

FREEPORT CUMBERLAND

DESERT ROAD SOUTH
CROSS SECTIONS

SHEET NUMBER

Wor

OF 4

Sta. 60+00.00 to Sta. 62+50.00

GOLDER

CALCULATIONS

Date:	12/3/2020	**Made by:**	KAR
Project No.:	19126013	**Checked by:**	BK
Subject:	Global Stability Analysis Abutments No. 1 and No. 2	**Reviewed by:**	MCM
Project Short Title:	MaineDOT I-295 Exit 20 Merrill Road Bridge Replacement No. 5720		

OBJECTIVE
Calculate global factor of safety for Abutment No. 1 (northwestern) and Abutment No. 2 (southeastern), assuming the "southern shift" option.

REFERENCES
1. AASHTO LRFD Bridge Design Specifications, 9th Ed, 2020.
2. Email communication between Golder and HNTB, subject "RE: Freeport I-295 Exit 20 and Exit 22 - Request for Sections and Design Info", dated June 22, 2020.
3. Golder geotechnical test boring logs (Appendix A, Preliminary Geotechnical Design Report, dated September 2020).
4. HNTB for State of Maine Department of Transportation. Merrill Road Bridge Freeport Interstate 295: Desert Road South Cross Sections, dated November 2019.
5. Das, Braja M. 2011. Principles of Foundation Engineering, 7th Edition. Cengage Learning.
6. FHWA. 2017. Geotechnical Engineering Circular No. 5: Geotechnical Site Characterization. Publication No. FHWA NHI-16-072.
7. GeoTesting Express laboratory testing results, dated February 5, 2020 (Appendix C, Preliminary Geotechnical Design Report, dated September 2020).
8. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.
9. Golder calculation titled "Seismic Site Class" (Appendix E, Preliminary Geotechnical Design Report, dated September 2020).
10. Rocscience Slide Software Package Version 2020 9.010 64-bit, build date Oct 14, 2020.

ATTACHMENTS
1. Slide output figures.
2. HNTB plans showing "southern shift" option.

ASSUMPTIONS
1. The load applied by the road and traffic for final design conditions is modeled as a 3 ft equivalent load of soil (Reference 1, Table 3.11.6.4-1) based on a 12 ft abutment height (Reference 2). 3 ft x 125 pcf (fill) = 375 psf.
2. A static FS ≥ 1.5 is recommended for abutment final design conditions per Section 5.9.2 in Reference 8. A pseudo-static FS> 1.0 is recommended per Section 3.7.4.1 in Reference 8
3. Circular surfaces were analyzed using the Spencer and Bishop simplified methods and auto refine search. Non-circular surfaces were analyzed using the Spencer method and cuckoo search with surface altering optimization.
4. The existing stratigraphy and water table surface are estimated from samples and groundwater measurements encountered during drilling and provided in Reference 3.
5. The existing grading, proposed grading, and construction design features are taken from Reference 4.
6. Undrained conditions ($\phi = 0$) were assumed for the glaciomarine silty clay layer.

CALCULATION

1. Determine input parameters to build the soil model in Slide.
The material parameters selected for use in the Slide models are shown in the table below.

Date:	12/3/2020	**Made by:**	KAR
Project No.:	19126013	**Checked by:**	BK
Subject:	Global Stability Analysis Abutments No. 1 and No. 2	**Reviewed by:**	MCM
Project Short Title:	MaineDOT I-295 Exit 20 Merrill Road Bridge Replacement No. 5720		

a. The friction angle parameters for the existing fill and sand/gravel layers are based on empirical correlation (Reference 5, Eqn. 2.26) to the average of the N_{60}-values encountered in all borings for each layer (Reference 3).

b. The cohesion parameter for the glaciomarine silty clay layer is based on shear strength measurements made in the field and on empirical correlation (Reference 6, Eqn. 7.19) to the average of the N_{60}-values encountered in all borings for each layer (Reference 3).

c. The unit weight parameter for the glaciomarine silty clay layer is calculated from soil moisture contents determined in laboratory testing (Reference 7), assuming 100% saturation. The unit weight parameters for the existing fill and sand/gravel layers are selected based on local engineering experience.

d. The UCS and unit weight parameters for the bedrock are selected based on the average of laboratory test results for all borings (Reference 7). The GSI, m_i, and D parameters for the bedrock are selected based on field descriptions of the rock quality encountered in the borings (Reference 3).

e. The friction angle and unit weight parameters for the construction materials are selected based on MaineDOT standard practice (Reference 8, Table 3-3).

Material Name	Unit Weight (pcf)	Strength Type	Cohesion (psf)	Friction Angle (°)	UCS (psf)	GSI	m_i	D
Existing Fill	125	Mohr-Coulomb	0	32	-	-	-	-
Glaciomarine Silty Clay	125	Mohr-Coulomb	1600	0	-	-	-	-
Sand and Gravel	125	Mohr-Coulomb	0	37	-	-	-	-
Bedrock	164	Generalized Hoek-Brown	-	-	1,869,552	60	28	0
New Fill	125	Mohr-Coulomb	0	32	-	-	-	-
New Subbase	135	Mohr-Coulomb	0	36	-	-	-	-

2. Use the soil layer parameters listed above to analyze the slope stability with Slide.

The soil layer properties above were used to analyze the following scenarios. HNTB provided Golder with both a northern shift and southern shift of the proposed bridge centerline alignment. Following analysis, Golder determined that the southern shift in the alignment would result in the largest height and offset of embankment fills over the existing ground surface, which would likely have the greatest impact in stability. Therefore the southern shift option was analyzed as the critical case. Both Abutment 1 and Abutment 2 were analyzed. This analysis does not account for the additional shear resistance that may be provided by the piles supporting the abutment wall as pile type and spacing were unknown. The results of the Slide stability analyses are summarized in the following table.

Baseline	Interpreted Subsurface Section	Abutment	Lowest Factor of Safety (Spencer Method)	
			NonCircular Failure Surface Through Proposed and Existing Fill	NonCircular Failure Surface Through Glaciomarine Deposit
Southern Shift	A-A'	1	1.29 (Fig. A.1)	2.15 (Fig. A.2)
		2	1.31 (Fig. B.1)	1.95 (Fig. B.2)

Circular Surfaces:

Circular failure surfaces using Spencer and Bishop simplified methods were also evaluated for each scenario. The circular surfaces that were produced were similar in location and size to the noncircular surfaces and had factors of safety similar in magnitude to the noncircular factors of safety. The circular factors of safety ranged from 1.37 to 1.39 for surfaces through the proposed and existing fill and from 2.18 to 2.36 for surfaces through the glaciomarine deposit.

3. Repeat the Slide analysis with pseudo-static seismic load conditions.

The same scenarios were also analyzed with a horizontal seismic load coefficient of $A_s/2 = 0.064$ (A_s from Reference 9) as recommended in AASHTO (Reference 1) Appendix 11A. The results of the seismic Slide stability analyses are summarized in the following table.

Baseline	Interpreted Subsurface Section	Abutment	Lowest Factor of Safety (Spencer Method)	
			NonCircular Failure Surface Through Proposed and Existing Fill	NonCircular Failure Surface Through Glaciomarine Deposit
Southern Shift	A-A'	1	1.15 (Fig. C.1)	1.87 (Fig. C.2)
		2	1.17 (Fig. D.1)	1.72 (Fig. D.2)

Circular Surfaces:

Circular failure surfaces using Spencer and Bishop simplified methods were also evaluated for each seismic scenario. The circular surfaces that were produced were similar in location and size to the noncircular surfaces and had factors of safety similar in magnitude to the noncircular factors of safety. The circular factors of safety ranged from 1.20 to 1.23 for surfaces through the proposed and existing fill and from 1.90 to 2.02 for surfaces through the glaciomarine deposit.

CONCLUSIONS

The proposed abutment and slope grading system produces a global stability factor of safety less than the recommended factor of safety of 1.5 for potential slope failures in the existing and proposed fill when using proposed fill engineering parameters recommended in the MaineDOT Bridge Design Guide section 5.9.2 for slopes with footings. Preliminary analyses indicate failure surfaces passing underneath the abutment wall. Further analysis is required to evaluate the stability of the abutments during final design.

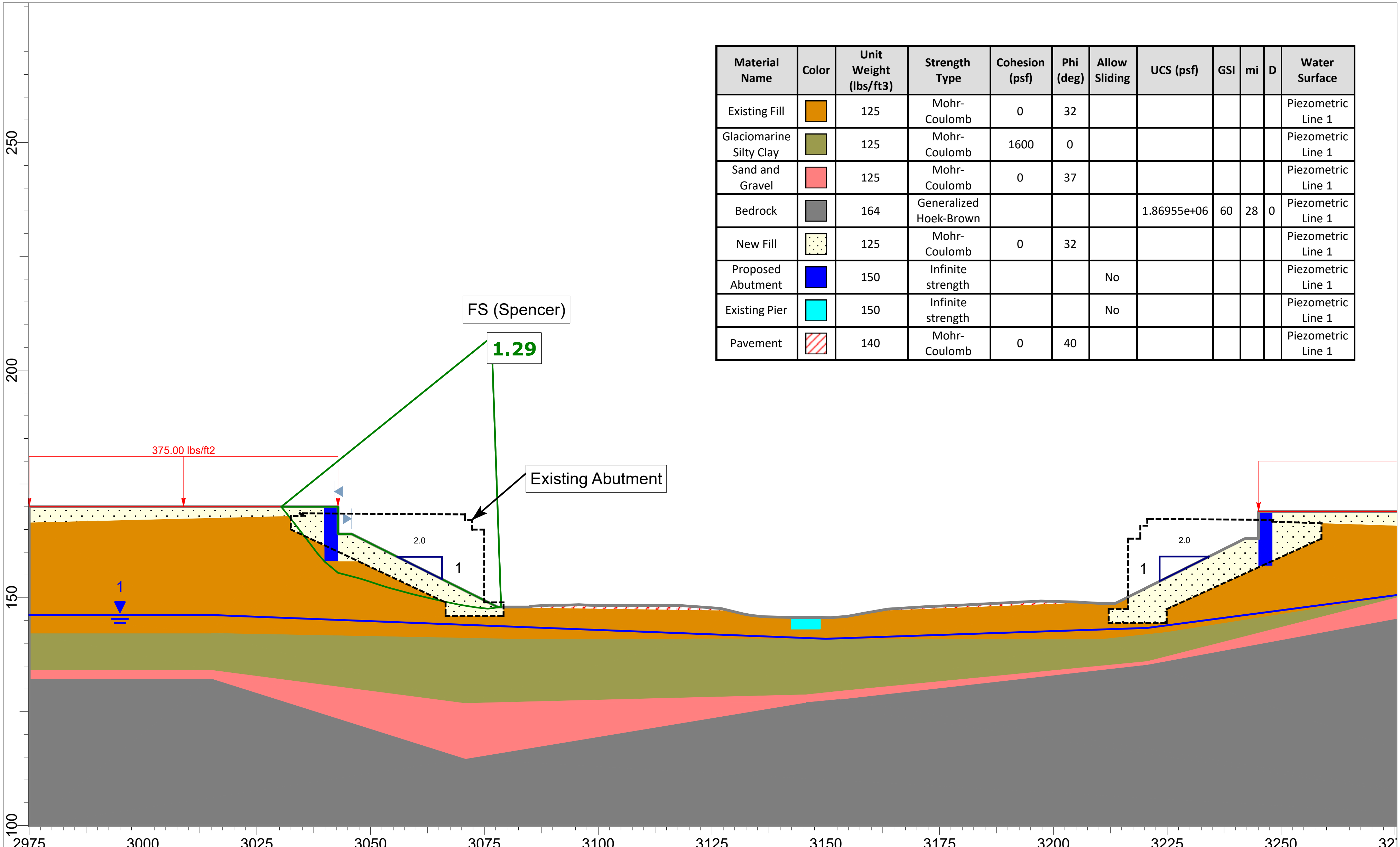
The analysis of the proposed abutment and slope grading system yields adequate factors of safety (FS > 1.5) for the potential deep seated slope failures in native soils underlying the proposed abutmentt under static conditions.

CALCULATIONS

Date:	12/3/2020	**Made by:**	KAR
Project No.:	19126013	**Checked by:**	BK
Subject:	Global Stability Analysis Abutments No. 1 and No. 2	**Reviewed by:**	MCM
Project Short Title:	MaineDOT I-295 Exit 20 Merrill Road Bridge Replacement No. 5720		

The analysis of the proposed abutment and slope grading system yields adequate factors of safety (FS > 1.0) for the potential slope failures in both the native soils underlying the proposed abutment and abutment surface fills under pseudo-static conditions where seismic loading is applied.

Attachment 1



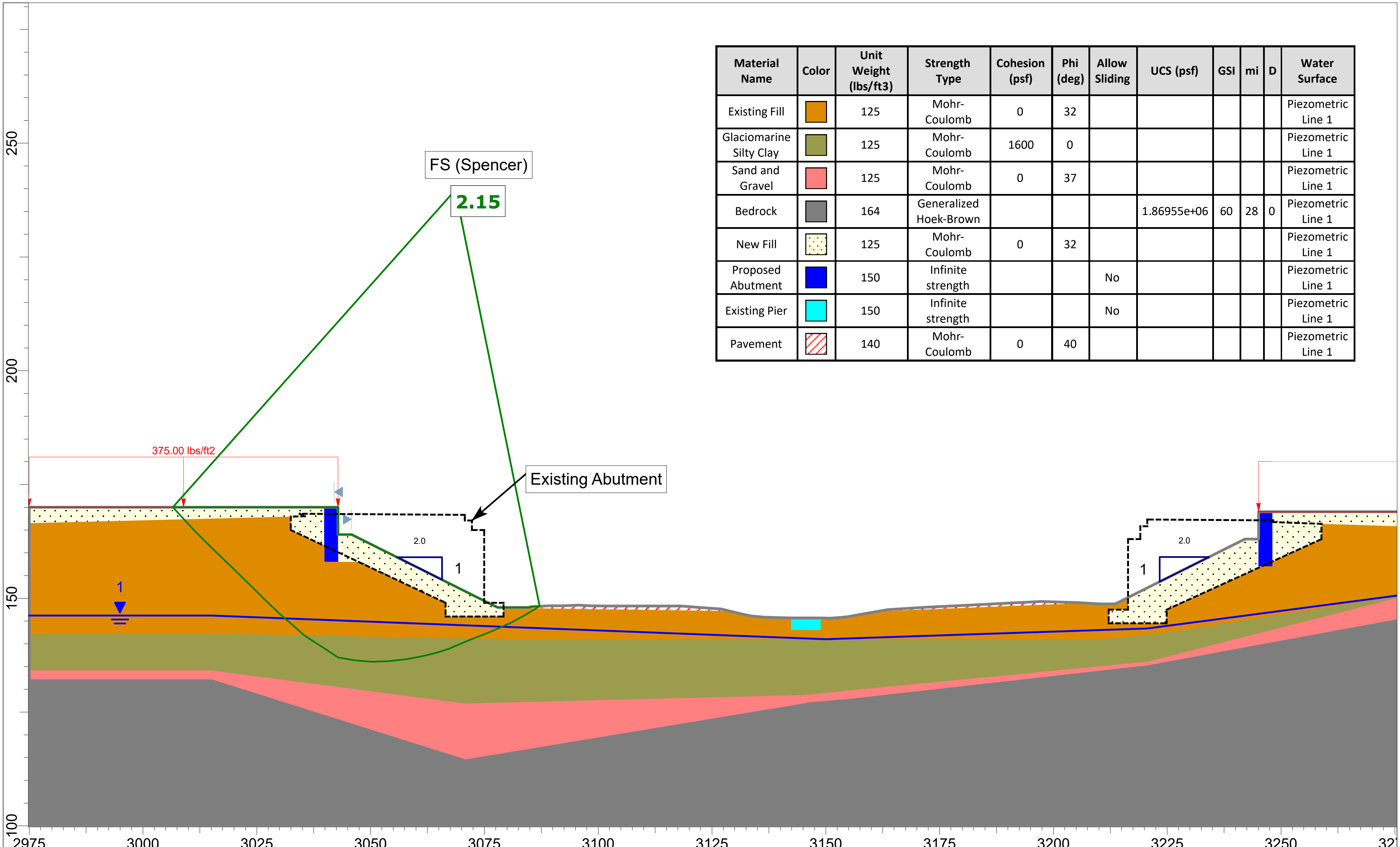
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Allow Sliding	UCS (psf)	GSI	mi	D	Water Surface
Existing Fill		125	Mohr-Coulomb	0	32						Piezometric Line 1
Glaciomarine Silty Clay		125	Mohr-Coulomb	1600	0						Piezometric Line 1
Sand and Gravel		125	Mohr-Coulomb	0	37						Piezometric Line 1
Bedrock		164	Generalized Hoek-Brown				1.86955e+06	60	28	0	Piezometric Line 1
New Fill		125	Mohr-Coulomb	0	32						Piezometric Line 1
Proposed Abutment		150	Infinite strength			No					Piezometric Line 1
Existing Pier		150	Infinite strength			No					Piezometric Line 1
Pavement		140	Mohr-Coulomb	0	40						Piezometric Line 1

GOLDER

SLIDEINTERPRET 9.010

Project	19126013 MaineDOT I-295 Exit 20 Merrill Road Bridge No. 5720
Analysis Description	Southern Shift - Profile A-A', Proposed Abutment 1 (NonCircular, Cuckoo Search)
Drawn By KAR	*Checked By* BK
Reviewed By MCM	*Scale* 1:350
Date 12/3/2020	*File Name* Southern Shift Profile A-A' Freeport Exit 20 19126013.slmd

Figure A.1



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Allow Sliding	UCS (psf)	GSI	mi	D	Water Surface
Existing Fill		125	Mohr-Coulomb	0	32						Piezometric Line 1
Glaciomarine Silty Clay		125	Mohr-Coulomb	1600	0						Piezometric Line 1
Sand and Gravel		125	Mohr-Coulomb	0	37						Piezometric Line 1
Bedrock		164	Generalized Hoek-Brown				1.86955e+06	60	28	0	Piezometric Line 1
New Fill		125	Mohr-Coulomb	0	32						Piezometric Line 1
Proposed Abutment		150	Infinite strength			No					Piezometric Line 1
Existing Pier		150	Infinite strength			No					Piezometric Line 1
Pavement		140	Mohr-Coulomb	0	40						Piezometric Line 1

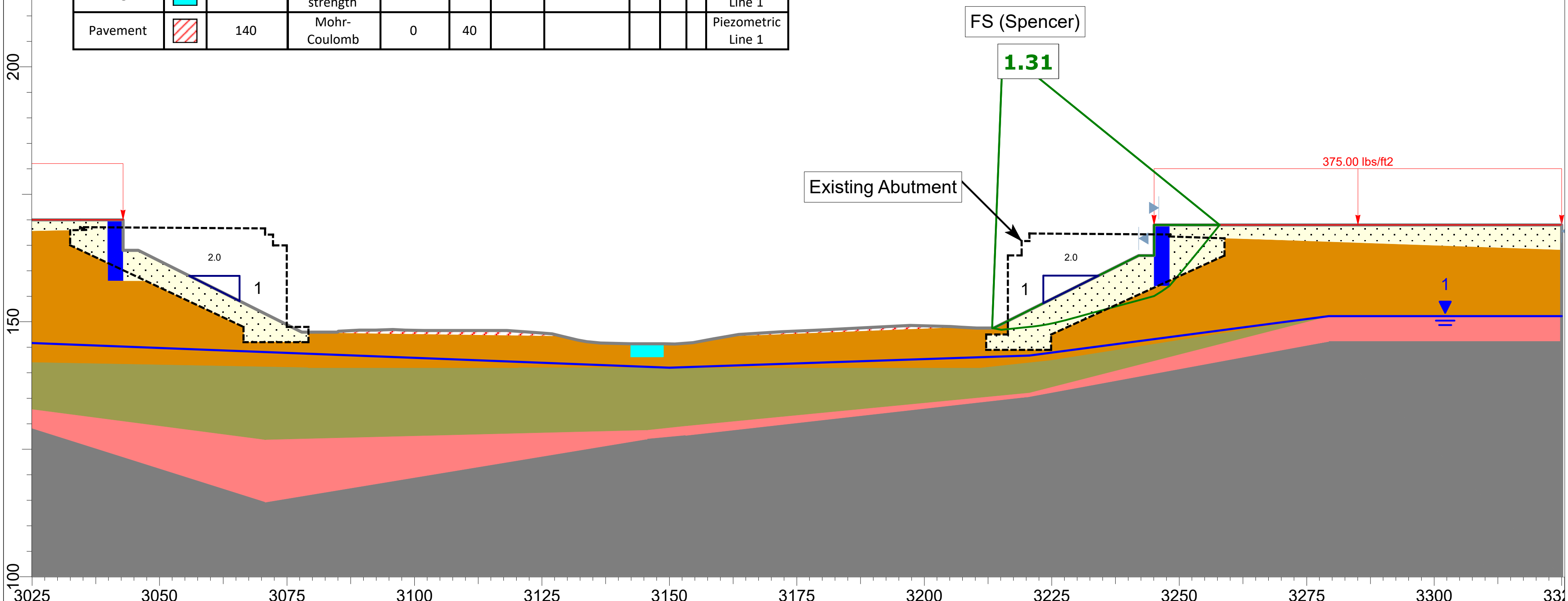
GOLDER

SLIDEINTERPRET 9.010

Project	19126013 MaineDOT I-295 Exit 20 Merrill Road Bridge No. 5720
Analysis Description	Southern Shift - Profile A-A', Proposed Abutment 1 (NonCircular, Cuckoo Search)
Drawn By	KAR
Checked By	BK
Reviewed By	MCM
Scale	1:350
Date	12/3/2020
File Name	Southern Shift Profile A-A' Freeport Exit 20 19126013.slmd

Figure A.2

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Allow Sliding	UCS (psf)	GSI	mi	D	Water Surface
Existing Fill		125	Mohr-Coulomb	0	32						Piezometric Line 1
Glaciomarine Silty Clay		125	Mohr-Coulomb	1600	0						Piezometric Line 1
Sand and Gravel		125	Mohr-Coulomb	0	37						Piezometric Line 1
Bedrock		164	Generalized Hoek-Brown				1.86955e+06	60	28	0	Piezometric Line 1
New Fill		125	Mohr-Coulomb	0	32						Piezometric Line 1
Proposed Abutment		150	Infinite strength			No					Piezometric Line 1
Existing Pier		150	Infinite strength			No					Piezometric Line 1
Pavement		140	Mohr-Coulomb	0	40						Piezometric Line 1



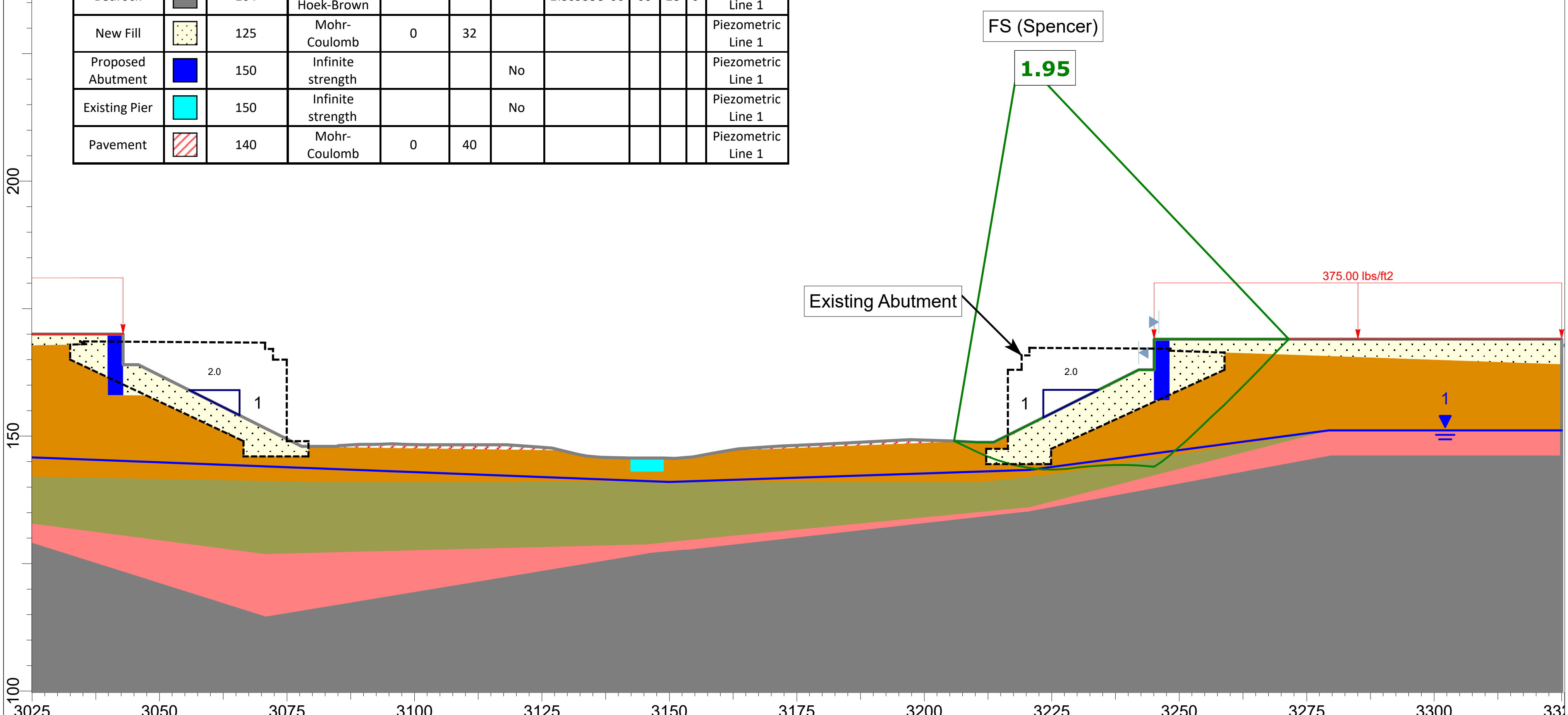
SLIDEINTERPRET 9.010

GOLDER

Project	19126013 MaineDOT I-295 Exit 20 Merrill Road Bridge No. 5720						
Analysis Description	Southern Shift - Profile A-A', Proposed Abutment 2 (NonCircular, Cuckoo Search)						
Drawn By	KAR	*Checked By*	BK	*Reviewed By*	MCM	*Scale*	1:350
Date	12/3/2020	*File Name*	Southern Shift Profile A-A' Freeport Exit 20 19126013.slmd				

Figure B.1

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Allow Sliding	UCS (psf)	GSI	mi	D	Water Surface
Existing Fill		125	Mohr-Coulomb	0	32						Piezometric Line 1
Glaciomarine Silty Clay		125	Mohr-Coulomb	1600	0						Piezometric Line 1
Sand and Gravel		125	Mohr-Coulomb	0	37						Piezometric Line 1
Bedrock		164	Generalized Hoek-Brown				1.86955e+06	60	28	0	Piezometric Line 1
New Fill		125	Mohr-Coulomb	0	32						Piezometric Line 1
Proposed Abutment		150	Infinite strength			No					Piezometric Line 1
Existing Pier		150	Infinite strength			No					Piezometric Line 1
Pavement		140	Mohr-Coulomb	0	40						Piezometric Line 1

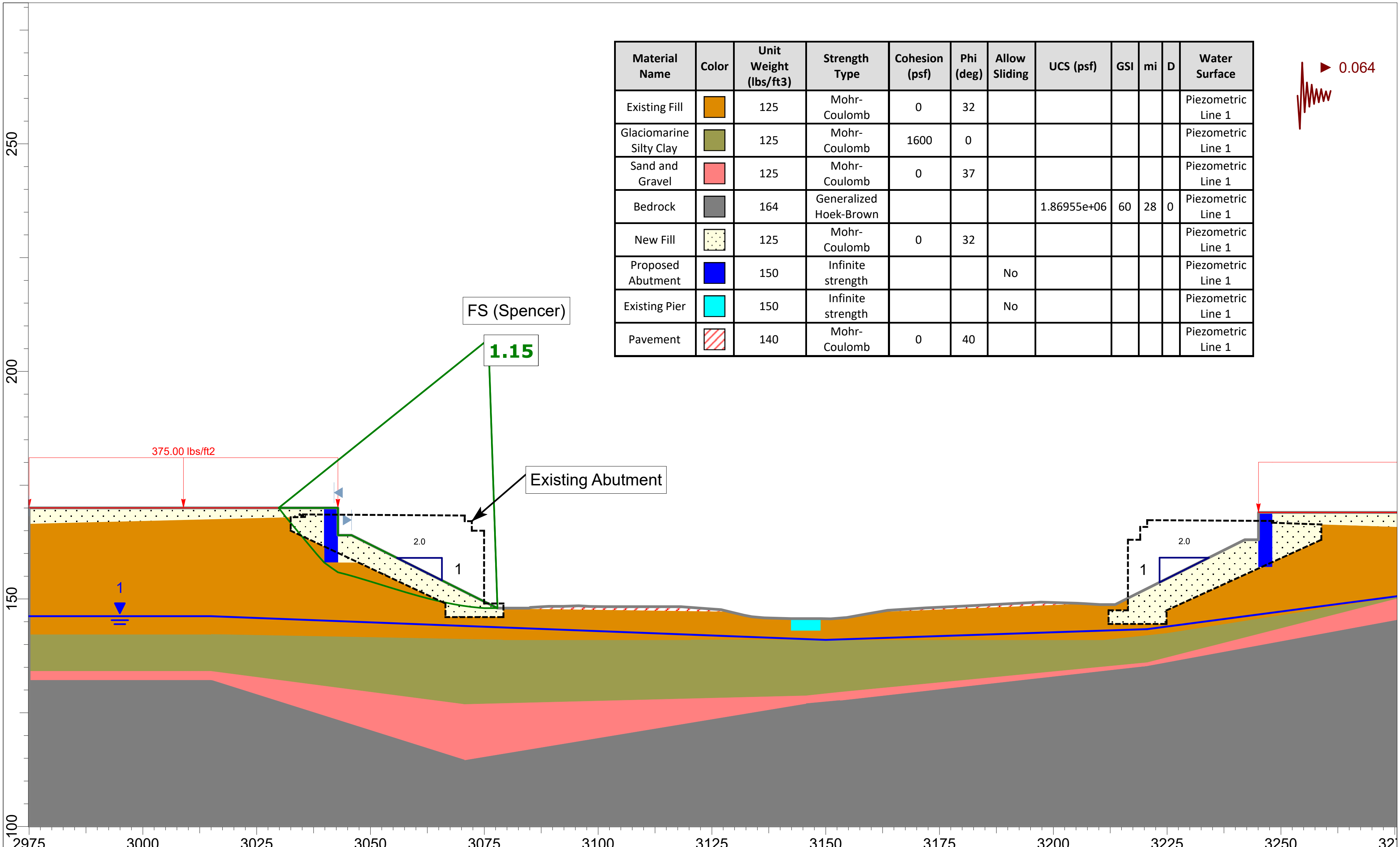


GOLDER

SLIDEINTERPRET 9.010

Project	19126013 MaineDOT I-295 Exit 20 Merrill Road Bridge No. 5720
Analysis Description	Southern Shift - Profile A-A', Proposed Abutment 2 (NonCircular, Cuckoo Search)
Drawn By KAR	*Checked By* BK
Reviewed By MCM	*Scale* 1:350
Date 12/3/2020	*File Name* Southern Shift Profile A-A' Freeport Exit 20 19126013.slmd

Figure B.2



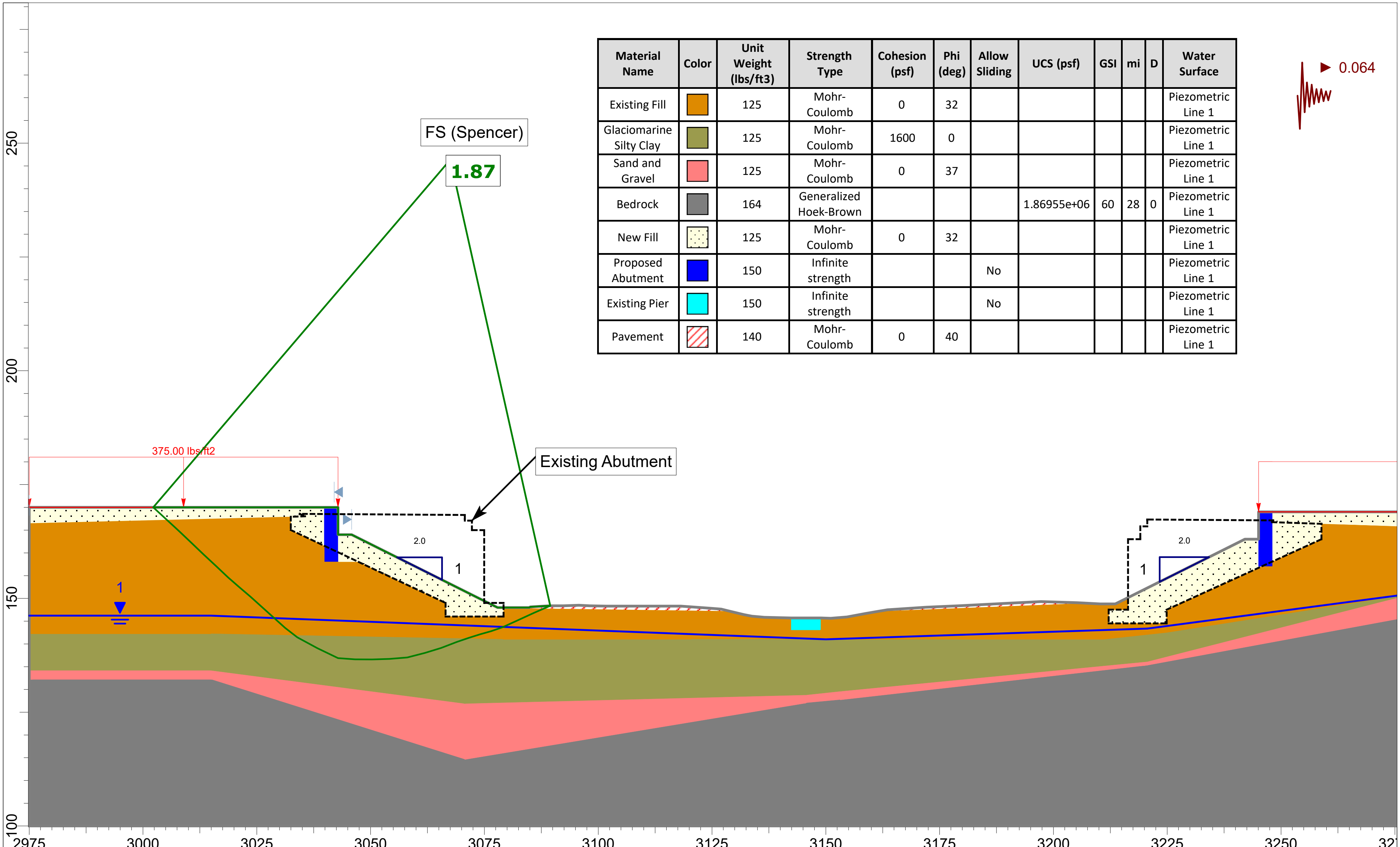
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Allow Sliding	UCS (psf)	GSI	mi	D	Water Surface
Existing Fill		125	Mohr-Coulomb	0	32						Piezometric Line 1
Glaciomarine Silty Clay		125	Mohr-Coulomb	1600	0						Piezometric Line 1
Sand and Gravel		125	Mohr-Coulomb	0	37						Piezometric Line 1
Bedrock		164	Generalized Hoek-Brown				1.86955e+06	60	28	0	Piezometric Line 1
New Fill		125	Mohr-Coulomb	0	32						Piezometric Line 1
Proposed Abutment		150	Infinite strength			No					Piezometric Line 1
Existing Pier		150	Infinite strength			No					Piezometric Line 1
Pavement		140	Mohr-Coulomb	0	40						Piezometric Line 1

GOLDER

SLIDEINTERPRET 9.010

Project	19126013 MaineDOT I-295 Exit 20 Merrill Road Bridge No. 5720
Analysis Description	Southern Shift - Profile A-A', Proposed Abutment 1 (NonCircular, Cuckoo Search)
Drawn By KAR	*Checked By* BK
Reviewed By MCM	*Scale* 1:350
Date 12/7/2020	*File Name* Southern Shift Profile A-A' Freeport Exit 20 19126013.slmd

Figure C.1



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Allow Sliding	UCS (psf)	GSI	mi	D	Water Surface
Existing Fill		125	Mohr-Coulomb	0	32						Piezometric Line 1
Glaciomarine Silty Clay		125	Mohr-Coulomb	1600	0						Piezometric Line 1
Sand and Gravel		125	Mohr-Coulomb	0	37						Piezometric Line 1
Bedrock		164	Generalized Hoek-Brown				1.86955e+06	60	28	0	Piezometric Line 1
New Fill		125	Mohr-Coulomb	0	32						Piezometric Line 1
Proposed Abutment		150	Infinite strength			No					Piezometric Line 1
Existing Pier		150	Infinite strength			No					Piezometric Line 1
Pavement		140	Mohr-Coulomb	0	40						Piezometric Line 1

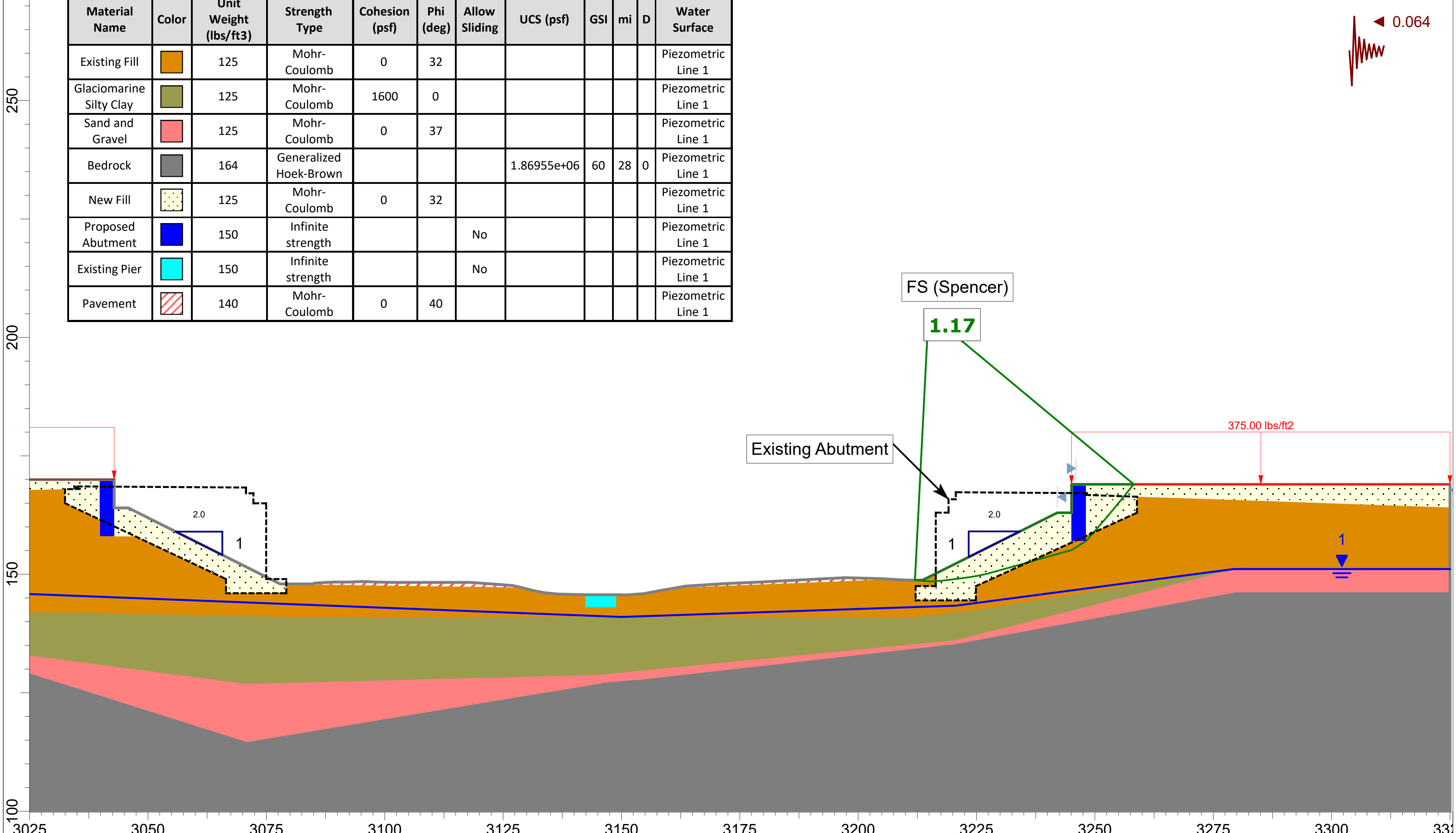
GOLDER

SLIDEINTERPRET 9.010

Project	19126013 MaineDOT I-295 Exit 20 Merrill Road Bridge No. 5720		
Analysis Description	Southern Shift - Profile A-A', Proposed Abutment 1 (NonCircular, Cuckoo Search)		
Drawn By KAR	*Checked By* BK	*Reviewed By* MCM	*Scale* 1:350
Date 12/7/2020	*File Name* Southern Shift Profile A-A' Freeport Exit 20 19126013.slmd		

Figure C.2

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Allow Sliding	UCS (psf)	GSI	mi	D	Water Surface
Existing Fill		125	Mohr-Coulomb	0	32						Piezometric Line 1
Glaciomarine Silty Clay		125	Mohr-Coulomb	1600	0						Piezometric Line 1
Sand and Gravel		125	Mohr-Coulomb	0	37						Piezometric Line 1
Bedrock		164	Generalized Hoek-Brown				1.86955e+06	60	28	0	Piezometric Line 1
New Fill		125	Mohr-Coulomb	0	32						Piezometric Line 1
Proposed Abutment		150	Infinite strength			No					Piezometric Line 1
Existing Pier		150	Infinite strength			No					Piezometric Line 1
Pavement		140	Mohr-Coulomb	0	40						Piezometric Line 1



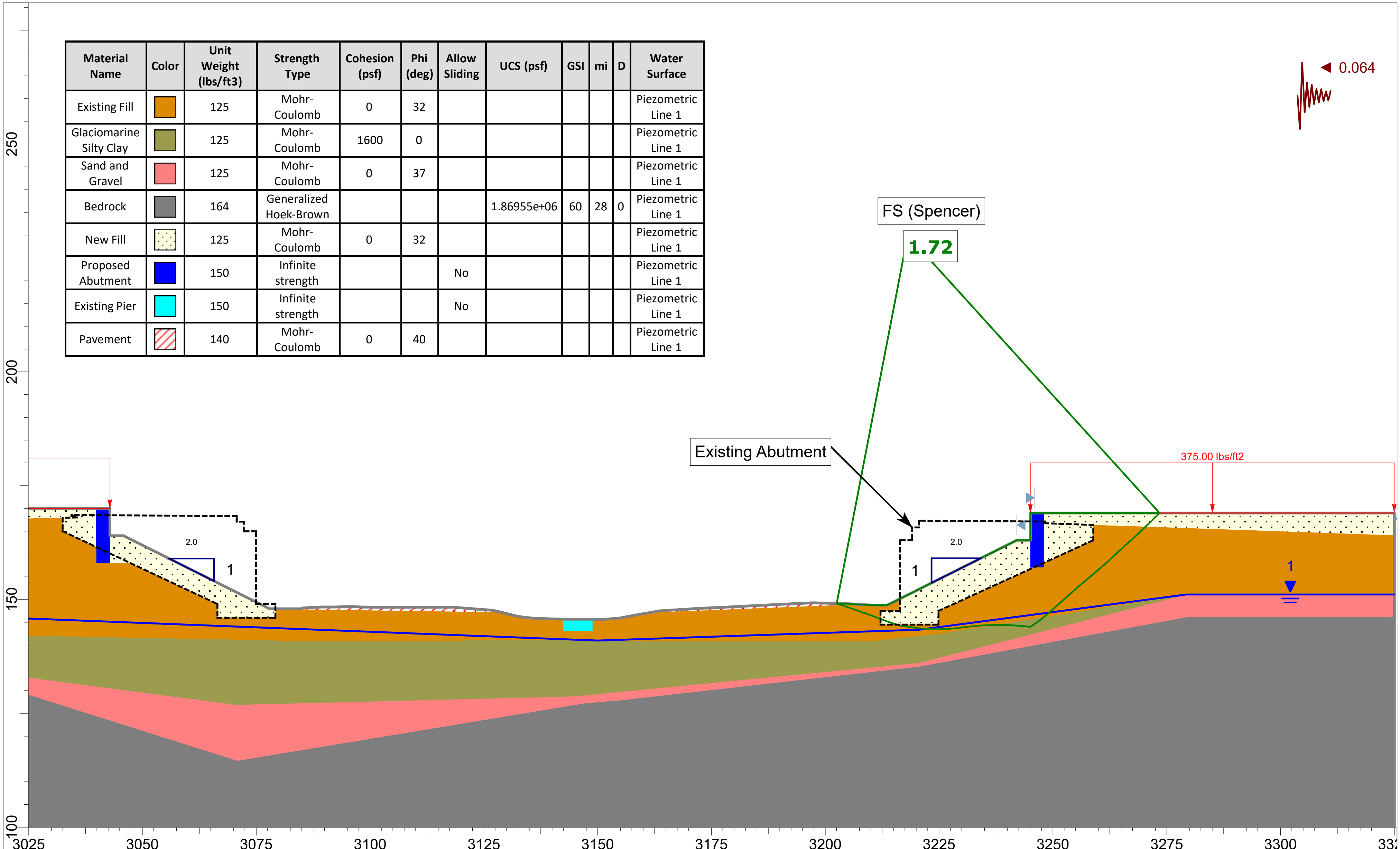
GOLDER

SLIDEINTERPRET 9.010

Project	19126013 MaineDOT I-295 Exit 20 Merrill Road Bridge No. 5720
Analysis Description	Southern Shift - Profile A-A', Proposed Abutment 2 (NonCircular, Cuckoo Search)
Drawn By KAR	*Checked By* BK
Reviewed By MCM	*Scale* 1:350
Date 12/7/2020	*File Name* Southern Shift Profile A-A' Freeport Exit 20 19126013.slmd

Figure D.1

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Allow Sliding	UCS (psf)	GSI	mi	D	Water Surface
Existing Fill		125	Mohr-Coulomb	0	32						Piezometric Line 1
Glaciomarine Silty Clay		125	Mohr-Coulomb	1600	0						Piezometric Line 1
Sand and Gravel		125	Mohr-Coulomb	0	37						Piezometric Line 1
Bedrock		164	Generalized Hoek-Brown				1.86955e+06	60	28	0	Piezometric Line 1
New Fill		125	Mohr-Coulomb	0	32						Piezometric Line 1
Proposed Abutment		150	Infinite strength			No					Piezometric Line 1
Existing Pier		150	Infinite strength			No					Piezometric Line 1
Pavement		140	Mohr-Coulomb	0	40						Piezometric Line 1



GOLDER

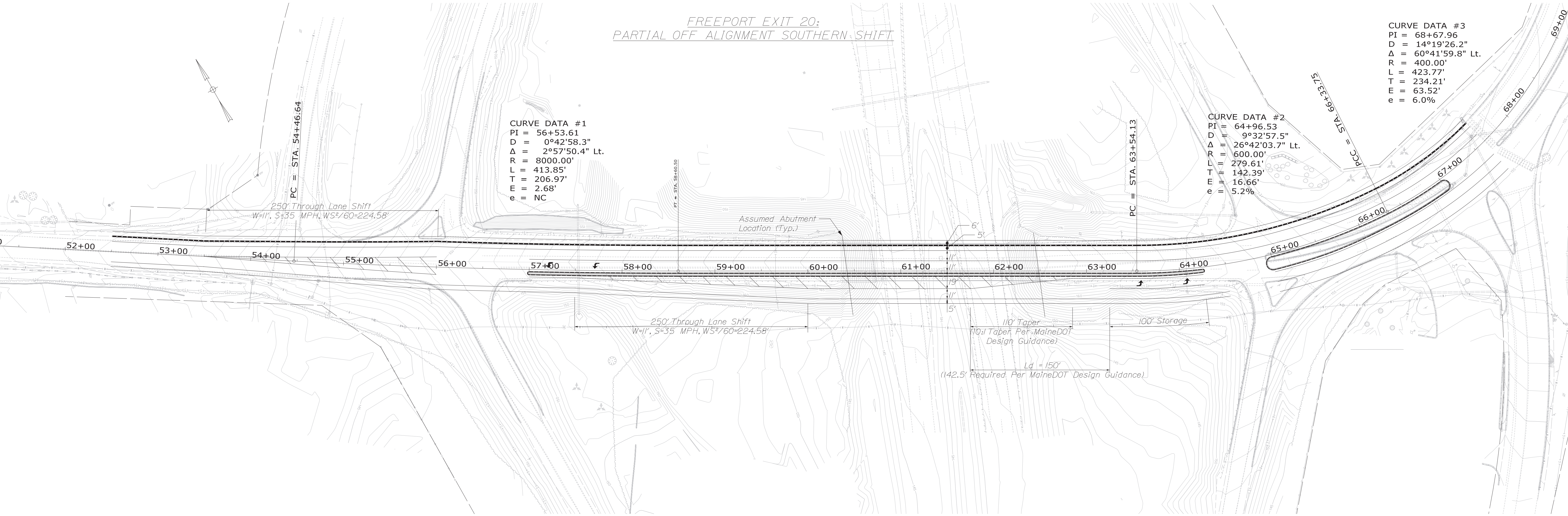
SLIDEINTERPRET 9.010

Project	19126013 MaineDOT I-295 Exit 20 Merrill Road Bridge No. 5720
Analysis Description	Southern Shift - Profile A-A', Proposed Abutment 2 (NonCircular, Cuckoo Search)

Drawn By	KAR	*Checked By*	BK	*Reviewed By*	MCM	*Scale*	1:350
Date	12/7/2020	*File Name*	Southern Shift Profile A-A' Freeport Exit 20 19126013.slmd				

Figure D.2

Attachment 2



APPENDIX E4

Settlement

GOLDER

CALCULATIONS

Date:	7/29/2020	**Made by:**	KAR
Project No.:	19126013	**Checked by:**	MEL
Subject:	Settlement at NW Bridge Embankment - Southern Shift	**Reviewed by:**	MCM
Project Short Title:	MaineDOT I-295 Exit 20 Merrill Road Bridge Replacement No. 5720		

OBJECTIVE

Calculate the primary settlement at the base of the northwestern proposed bridge approach embankment (assuming the "southern shift" option) at a location where embankment fill is thickest (36 feet southwest of southern shift alignment at HNTB Station 60+25).

REFERENCES

1. Golder calculation titled "Global Stability Analysis NW Embankment", dated July 29, 2020 (Appendix E, Preliminary Geotechnical Design Report, dated September 2020).
2. GeoTesting Express laboratory testing results, dated February 5, 2020 (Appendix C, Preliminary Geotechnical Design Report, dated September 2020).
3. Golder geotechnical test boring logs (Appendix A, Preliminary Geotechnical Design Report, dated September 2020).
4. HNTB for State of Maine Department of Transportation. Merrill Road Bridge Freeport Interstate 295: Desert Road South Cross Sections, dated November 2019.
5. Holtz, R.D. and Kovacs, W.D. 1981. An Introduction to Geotechnical Engineering, 1st ed. Prentice Hall, Englewood Cliffs, NJ.
6. FHWA. 2002. Geotechnical Engineering Circular No. 6: Shallow Foundations. Report No. FHWA-SA-02-054.

ASSUMPTIONS

1. The existing stratigraphy and water table surface are estimated from samples and groundwater measurements encountered during drilling and interpreted in Reference 1.

GS elev. = 157.1 ft
WL elev. = 147.9 ft

2. Clay consolidation parameters for the glaciomarine silty clay layer, based on Golder's local engineering experience ($C_{c\varepsilon}$, $C_{r\varepsilon}$, c_v) and calculated (e_0) from soil moisture contents determined in laboratory testing (Reference 2), are:

$C_{c\varepsilon}$ = 0.25
$C_{r\varepsilon}$ = 0.02
e_0 = 0.68
c_v = 120 ft^2/yr

3. N_{60}-values for the existing fill and the sand/gravel layers, based on the average of the N_{60}-values encountered in all borings for each layer (Reference 3), are:

$N_{60,Fill}$ = 23
$N_{60,S/G}$ = 13

4. The proposed top of road grade and base of embankment elevations at the location of analysis are (Reference 4, Section 60+25):

El. road = 170.1 ft
El. base = 157.1 ft

5. Assume that the glaciomarine silty clay is sufficiently overconsolidated and will experience recompression settlement only after loading (based on Golder's local engineering experience).

GOLDER

CALCULATIONS

Date:	7/29/2020	**Made by:**	KAR
Project No.:	19126013	**Checked by:**	MEL
Subject:	Settlement at NW Bridge Embankment - Southern Shift	**Reviewed by:**	MCM
Project Short Title:	MaineDOT I-295 Exit 20 Merrill Road Bridge Replacement No. 5720		

CALCULATION

A. Determine the change in effective stress state within the soil beneath the guardrail on the south side of the proposed roadway to identify if settlement or heave will occur. Calculate the vertical stress increase beneath the embankment at the guardrail location.

The change in effective stress state due to change in stratigraphy is determined at an elevation of 157.1 ft (base of embankment elevation).

Existing Conditions (Reference 1):

Total σ'_v at Elev. 157.1 ft (psf): 0

After Construction (Reference 4, Section 60+25):

Layer	Unit weight (pcf)	Elevation (ft)		Thickness (ft)	Stress Contribution of Layer (psf)
		Top	Bottom		
New Pavement	140	170.1	169.6	0.5	70
New Subbase	135	169.6	167.6	2.0	270
New Fill	125	167.6	157.1	10.5	1313

Calculate the increase or decrease in effective stress as a result of construction.

	σ_v at Elev. 157.1 ft (psf)	σ'_v at Elev. 157.1 ft (psf)	$\Delta\sigma'_v$ at Elev. 157.1 ft (psf)	Result
Existing conditions	0	0	1653	Settlement
After construction	1653	1653		

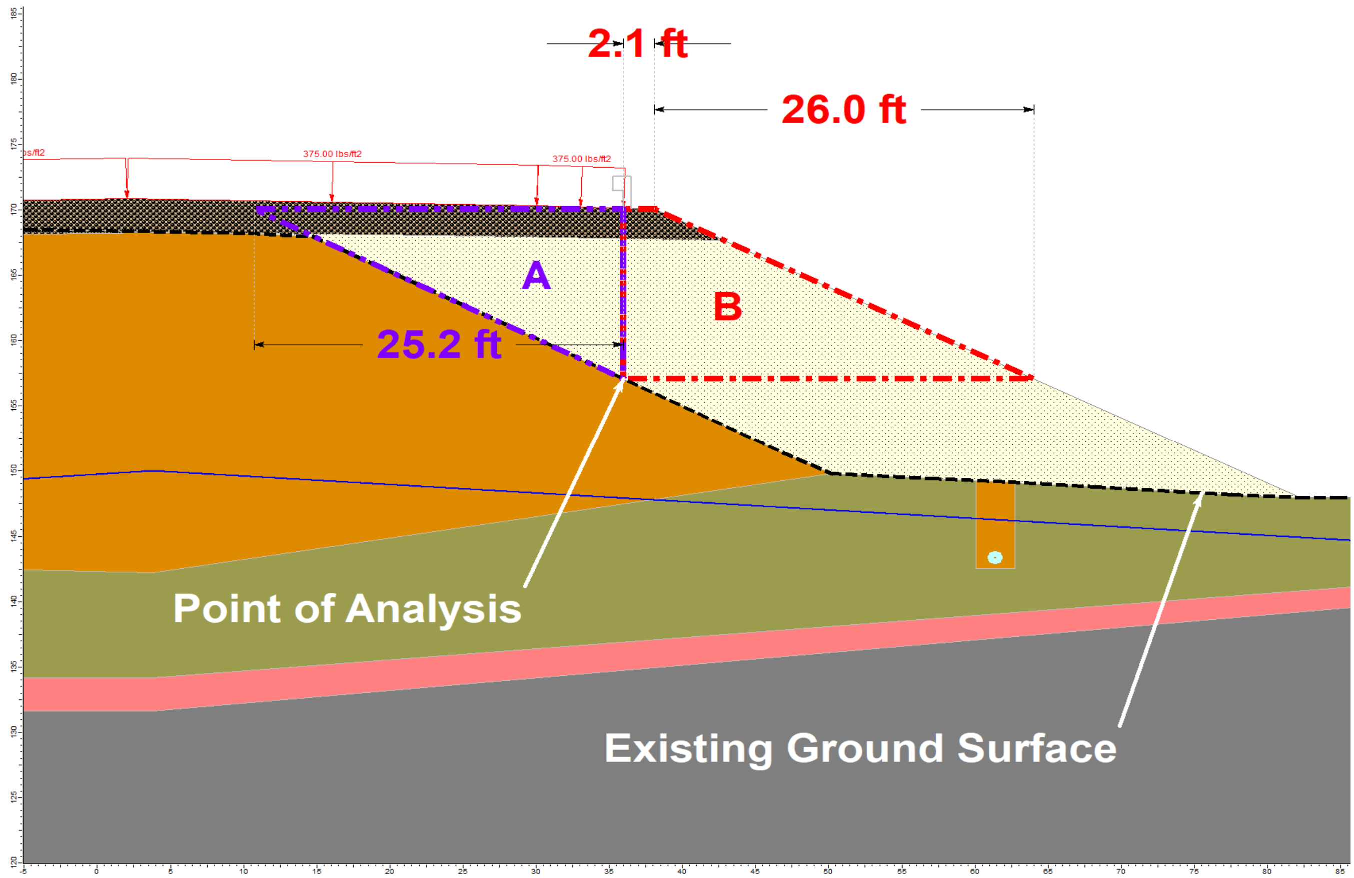
(Water table is below Elev. 157.1 ft)

Subdivide the subsurface soils into layers no larger than 10 ft thick and to a depth of either twice the footing width or to bedrock. Calculate the vertical stress increase at each layer, assuming the Boussinesq stress distribution method for stress under a long trapezoidal embankment. The Boussinesq method was developed for footings but in this case is used for the proposed embankment fill.

CALCULATIONS

Date:	7/29/2020	**Made by:**	KAR
Project No.:	19126013	**Checked by:**	MEL
Subject:	Settlement at NW Bridge Embankment - Southern Shift	**Reviewed by:**	MCM
Project Short Title:	MaineDOT I-295 Exit 20 Merrill Road Bridge Replacement No. 5720		

Assumed Fill Loading (section view):



$$\sigma_z = q_0 \times I$$ Reference 5, Eqn. 8-30

$$a/z$$ Reference 5, Figure 8.23

$$b/z$$ Reference 5, Figure 8.23

where:

	Trapezoid A (+)	Trapezoid B (+)		
σ_z = vertical stress increase, psf				
q_0 = stress applied by footing	1653	1653	psf	(Part A)
a = dimension of embankment slope, ft	25.2	26.0	ft	(Ref. 4)
b = dimension of embankment top, ft	0.0	2.1	ft	(Ref. 4)
z = depth to midpoint of layer, ft				

CALCULATIONS

Date:	7/29/2020	**Made by:**	KAR
Project No.:	19126013	**Checked by:**	MEL
Subject:	Settlement at NW Bridge Embankment - Southern Shift	**Reviewed by:**	MCM
Project Short Title:	MaineDOT I-295 Exit 20 Merrill Road Bridge Replacement No. 5720		

Trapezoid A:

Layer		Depth below footing (ft)	Layer Thickness (ft)	z (ft)	a/z	b/z	I	Stress Increase σ_z (psf)
Existing Fill	1	0-9.6	9.6	4.8	5.3	0.0	0.437	722
Glaciomarine	2	9.6-14.9	5.3	12.3	2.1	0.0	0.350	579
Glaciomarine	3	14.9-20.2	5.3	17.6	1.4	0.0	0.315	521
Sand and Gravel	4	20.2-22.4	2.2	21.3	1.2	0.0	0.273	451

Trapezoid B:

Layer		Depth below footing (ft)	Layer Thickness (ft)	z (ft)	a/z	b/z	I	Stress Increase σ_z (psf)
Existing Fill	1	0-9.6	9.6	4.8	5.4	0.4	0.470	777
Glaciomarine	2	9.6-14.9	5.3	12.3	2.1	0.2	0.410	678
Glaciomarine	3	14.9-20.2	5.3	17.6	1.5	0.1	0.350	579
Sand and Gravel	4	20.2-22.4	2.2	21.3	1.2	0.1	0.305	504

Total Footing (Trapezoid A + Trapezoid B):

Layer		Depth below footing (ft)	Layer Thickness (ft)	z (ft)	Stress Increase (psf)
Existing Fill	1	0-9.6	9.6	4.8	1499
Glaciomarine	2	9.6-14.9	5.3	12.3	1256
Glaciomarine	3	14.9-20.2	5.3	17.6	1099
Sand and Gravel	4	20.2-22.4	2.2	21.3	955

B. Use consolidation theory to estimate settlement of the glaciomarine silty clay layer beneath the embankment (Layers 2 and 3); use the Hough method to estimate settlement of the existing fill and sand/gravel layers beneath the embankment (Layers 1 and 4).

1 Determine σ'_{v0} and C' at layer midpoints for in situ existing soil. Use Ref. 6 Figures 5-18 and 5-19 to obtain corrected N' and C' for Layers 1 and 4, assuming the existing fill is "well graded fine to medium silty SAND" and the sand/gravel is "well graded silty SAND & GRAVEL".

	σ'_{v0} (psf)
Stress due to existing soil above footing	0

Water unit weight (pcf)
62.4

GOLDER

CALCULATIONS

Date:	7/29/2020	**Made by:**	KAR
Project No.:	19126013	**Checked by:**	MEL
Subject:	Settlement at NW Bridge Embankment - Southern Shift	**Reviewed by:**	MCM
Project Short Title:	MaineDOT I-295 Exit 20 Merrill Road Bridge Replacement No. 5720		

Layer		Layer Thickness (ft)	Unit Weight of Layer (pcf)	σ'_{v0} at layer midpoint (psf)	σ'_{v0} at layer midpoint (kPa)	N_{60}	N'	C'
Existing Fill	1	9.6	125	600	28.7	23	42	100
Glaciomarine	2	5.3	125	1341	64.2	Not required for clay consolidation analysis		
Glaciomarine	3	5.3	125	1673	80.1			
Sand and Gravel	4	2.2	125	1907	91.3	13	13	60

2 Calculate the total settlement of the existing fill and sand/gravel (Layers 1 and 4) using the Hough method:

General Equation (Ref. 6, Eqn 5-24)

$$\Delta H_i = H_c \frac{1}{C'} \log\left(\frac{\sigma'_{v0} + \Delta\sigma_v}{\sigma'_{v0}}\right)$$

where:

ΔH_i	settlement in each layer, ft
H_c	initial height of layer i, ft
C'	bearing capacity index from Ref. 6, Figure 5-19
$\Delta\sigma_v$	vertical stress increase, ksf

ΔH_i (Layer 1)	ft	0.052
	in	0.63

ΔH_i (Layer 4)	ft	0.006
	in	0.08

3 Calculate the total settlement of the glaciomarine silty clay (Layers 2 and 3) using the consolidation theory:

General Equation (Ref. 5, Eqn 8-11, 8-16, 8-18b)

$$\Delta H_i = C_c \frac{H_0}{1+e_0} \log\frac{\sigma'_{v0} + \Delta\sigma_v}{\sigma'_{v0}}$$ for normally consolidated clay

$$\Delta H_i = C_r \frac{H_0}{1+e_0} \log\frac{\sigma'_{v0} + \Delta\sigma_v}{\sigma'_{v0}}$$ for overconsolidated clay, $\sigma'_{v0} + \Delta\sigma_v \le \sigma'_p$

$$\Delta H_i = C_r \frac{H_0}{1+e_0} \log\frac{\sigma'_p}{\sigma'_{v0}} + C_c \frac{H_0}{1+e_0} \log\frac{\sigma'_{v0} + \Delta\sigma_v}{\sigma'_p}$$ for overconsolidated clay, $\sigma'_{v0} + \Delta\sigma_v > \sigma'_p$

Date:	7/29/2020	**Made by:**	KAR
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where:

			Layer 2	Layer 3	
H_0	initial height of layer i, ft		5.3	5.3	
$\Delta\sigma_v$	surcharge load, psf		1256	1099	
σ'_{v0}	in situ vertical effective stress, psf		1341	1673	
$\sigma'_{v0} + \Delta\sigma_v$			2597	2772	
σ'_p	preconsolidated stress, psf		N/A	N/A	
C_c	compression index, $C_c = C_{c\varepsilon}(1+e_0)$		0.42	0.42	
C_r	recompression index, $C_r = C_{r\varepsilon}(1+e_0)$		0.03	0.03	
e_0	initial void ratio		0.68	0.68	
	Use equation:		8-16	8-16	(Asmpt. 5)
ΔH_i		ft	0.030	0.023	
		in	0.37	0.28	

Layer	Settlement Based on Calc. Loading Stress ΔH_i (in)
1	0.63
2	0.37
3	0.28
4	0.08
Total Settlement (in)	1.36

C. Determine the time rate of settlement that will occur at the embankment location.

Use a single layer analysis to determine the time for the entire settlement to occur, assuming double drainage conditions based on the soil types above and below the glaciomarine silty clay layer having a higher permeability relative to the clay layer (Reference 3). Subsurface conditions are simplified to a single clay layer with uniform soil properties for the purpose of the calculation.

$$T_v = \frac{c_V t}{H^2_{dr}}$$ (Ref. 5, Eqn 9-5)

$$T_v = \frac{\pi}{4}\left(\frac{U\%}{100}\right)^2$$ For U = 0% to 60% (Ref. 5, Eqn 9-10)

$$T_v = 1.781 - 0.933\log(100 - U\%)$$ For U ≥ 60% (Ref. 5, Eqn 9-11)

$$U_{avg} = \frac{s(t)}{S_c}$$ (Ref. 5, Eqn 9-12)

where:

T_v = time factor (dimensionless)

c_v = coefficient of consolidation (ft^2/day)

t = time (day)

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H_{dr} = drainage path (ft)
U_{avg} = average degree or percent consolidation
s(t) = settlement at any time (in)
s_c = final or ultimate settlement (in)

Based on Reference 1, the clay layer thickness below the footing is: 10.6 ft
With double drainage conditions, H_{dr} = 5.3 ft
The coefficient of consolidation value is assumed based on local engineering experience.
c_v = 120 ft^2/yr = 0.33 ft^2/day

Determine time factor using Reference 5, Eqn 9-11

U_{avg} = 95 (Calculate time to reach 95% consolidation)
T_v = 1.13

Determine time required for settlement to occur using Reference 5, Eqn 9-5

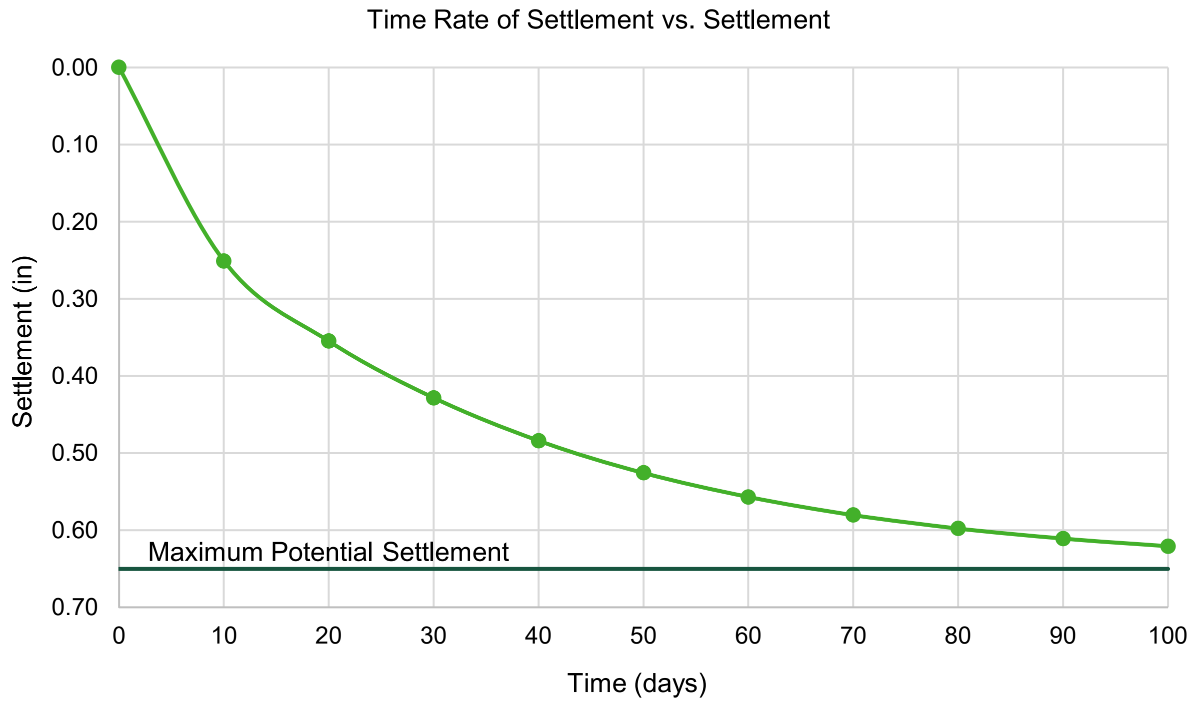
c_v = 0.33 ft^2/day
H_{dr} = 5.3 ft
T_v = 1.13
t = 97 days = 0.27 years (95% consolidation)

s_c (in) = 0.65 (final potential settlement of the clay layer, Part B)

Time (days)	Time (years)	T_v	U_{avg} (%)	s(t) (in)
0	0.00	0.00	0.00	0.00
10	0.03	0.12	38.60	0.25
20	0.05	0.23	54.59	0.35
30	0.08	0.35	65.92	0.43
40	0.11	0.47	74.47	0.48
50	0.14	0.59	80.87	0.53
60	0.16	0.70	85.67	0.56
70	0.19	0.82	89.27	0.58
80	0.22	0.94	91.96	0.60
90	0.25	1.05	93.98	0.61
100	0.27	1.17	95.49	0.62

CALCULATIONS

Date:	7/29/2020	**Made by:**	KAR
Project No.:	19126013	**Checked by:**	MEL
Subject:	Settlement at NW Bridge Embankment - Southern Shift	**Reviewed by:**	MCM
Project Short Title:	MaineDOT I-295 Exit 20 Merrill Road Bridge Replacement No. 5720		



CONCLUSIONS

The primary settlement at the base of the northwestern proposed bridge approach embankment after construction (assuming the "southern shift" option) is estimated to be 1.36 inches. This includes 0.71 inches of immediate settlement and 0.65 inches of consolidation settlement that is estimated to reach 95% consolidation in 97 days.

GOLDER

CALCULATIONS

Date:	7/8/2020	**Made by:**	KAR
Project No.:	19126013	**Checked by:**	MEL
Subject:	Settlement at SE Bridge Embankment - Southern Shift	**Reviewed by:**	MCM
Project Short Title:	MaineDOT I-295 Exit 20 Merrill Road Bridge Replacement No. 5720		

OBJECTIVE

Calculate the primary settlement at the base of the southeastern proposed bridge approach embankment (assuming the "southern shift" option) at a location where embankment fill is thickest (36 feet southwest of southern shift alignment at HNTB Station 62+50).

REFERENCES

1. Golder calculation titled "Global Stability Analysis SE Embankment", dated July 8, 2020 (Appendix E, Preliminary Geotechnical Design Report, dated September 2020).
2. GeoTesting Express laboratory testing results, dated February 5, 2020 (Appendix C, Preliminary Geotechnical Design Report, dated September 2020).
3. Golder geotechnical test boring logs (Appendix A, Preliminary Geotechnical Design Report, dated September 2020).
4. HNTB for State of Maine Department of Transportation. Merrill Road Bridge Freeport Interstate 295: Desert Road South Cross Sections, dated November 2019.
5. Holtz, R.D. and Kovacs, W.D. 1981. An Introduction to Geotechnical Engineering, 1st ed. Prentice Hall, Englewood Cliffs, NJ.
6. FHWA. 2002. Geotechnical Engineering Circular No. 6: Shallow Foundations. Report No. FHWA-SA-02-054.

ASSUMPTIONS

1. The existing stratigraphy and water table surface are estimated from samples and groundwater measurements encountered during drilling and interpreted in Reference 1.

GS elev. = 154.9 ft
WL elev. = 148.2 ft

2. Clay consolidation parameters for the glaciomarine silty clay layer, based on Golder's local engineering experience ($C_{c\varepsilon}$, $C_{r\varepsilon}$, c_v) and calculated (e_0) from soil moisture contents determined in laboratory testing (Reference 2), are:

$C_{c\varepsilon}$ = 0.25
$C_{r\varepsilon}$ = 0.02
e_0 = 0.68
c_v = 120 ft^2/yr

3. N_{60}-values for the existing fill and the sand/gravel layers, based on the average of the N_{60}-values encountered in all borings for each layer (Reference 3), are:

$N_{60,Fill}$ = 23
$N_{60,S/G}$ = 13

4. The proposed top of road grade and base of embankment elevations at the location of analysis are (Reference 4, Section 62+50):

El. road = 169.5 ft
El. base = 154.9 ft

5. Assume that the glaciomarine silty clay is sufficiently overconsolidated and will experience recompression settlement only after loading (based on Golder's local engineering experience).

CALCULATION

A. Determine the change in effective stress state within the soil beneath the guardrail on the south side of the proposed roadway to identify if settlement or heave will occur. Calculate the vertical stress increase beneath the embankment at the guardrail location.

The change in effective stress state due to change in stratigraphy is determined at an elevation of 154.9 ft (base of embankment elevation).

Existing Conditions (Reference 1):

Date:	7/8/2020	**Made by:**	KAR
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Subject:	Settlement at SE Bridge Embankment - Southern Shift	**Reviewed by:**	MCM
Project Short Title:	MaineDOT I-295 Exit 20 Merrill Road Bridge Replacement No. 5720		

Total σ'_v at Elev. 154.9 ft (psf): 0

After Construction (Reference 4, Section 62+50):

Layer	Unit weight (pcf)	Elevation (ft) Top	Elevation (ft) Bottom	Thickness (ft)	Stress Contribution of Layer (psf)
New Pavement	140	169.5	169.0	0.5	70
New Subbase	135	169.0	167.0	2.0	270
New Fill	125	167.0	154.9	12.1	1513

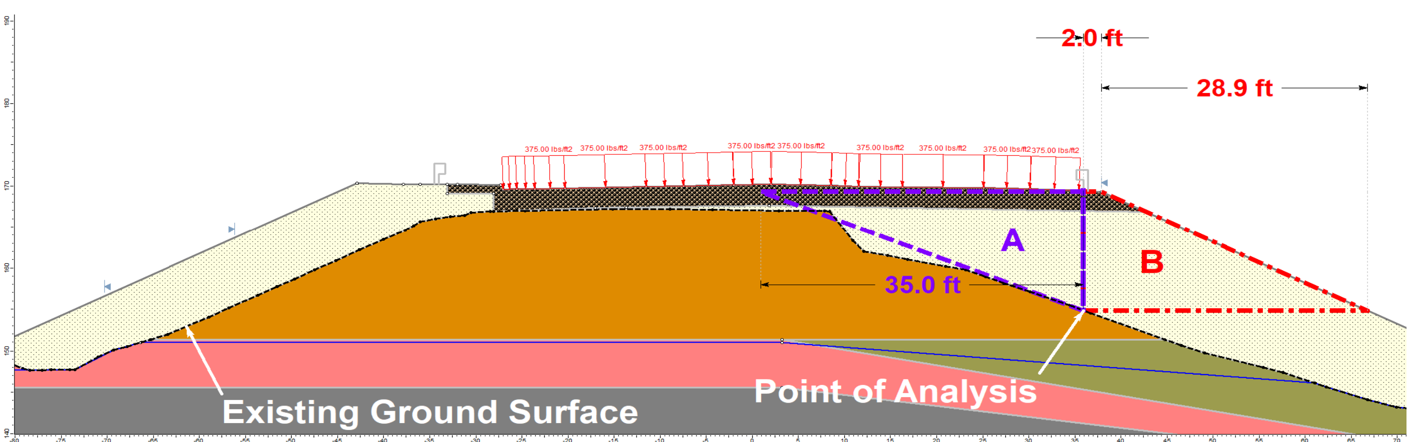
Calculate the increase or decrease in effective stress as a result of construction.

	σ_v at Elev. 154.9 ft (psf)	σ'_v at Elev. 154.9 ft (psf)	$\Delta\sigma'_v$ at Elev. 154.9 ft (psf)	Result
Existing conditions	0	0	1853	Settlement
After construction	1853	1853		

(Water table is below Elev. 154.9 ft)

Subdivide the subsurface soils into layers no larger than 10 ft thick and to a depth of either twice the footing width or to bedrock. Calculate the vertical stress increase at each layer, assuming the Boussinesq stress distribution method for stress under a long trapezoidal embankment. The Boussinesq method was developed for footings but in this case is used for the proposed embankment fill.

Assumed Fill Loading (section view):



$\sigma_z = q_0 \times I$	Reference 5, Eqn. 8-30
a/z	Reference 5, Figure 8.23
b/z	Reference 5, Figure 8.23

where:

	Trapezoid A (+)	Trapezoid B (+)
σ_z = vertical stress increase, psf		

GOLDER

CALCULATIONS

Date:	7/8/2020	**Made by:**	KAR
Project No.:	19126013	**Checked by:**	MEL
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Project Short Title:	MaineDOT I-295 Exit 20 Merrill Road Bridge Replacement No. 5720		

q_0 = stress applied by footing	1853	1853	psf	(Part A)
a = dimension of embankment slope, ft	35.0	28.9	ft	(Ref. 4)
b = dimension of embankment top, ft	0.0	2.0	ft	(Ref. 4)
z = depth to midpoint of layer, ft				

Trapezoid A:

Layer		Depth below footing (ft)	Layer Thickness (ft)	z (ft)	a/z	b/z	I	Stress Increase σ_z (psf)
Existing Fill	1	0-3.5	3.5	1.8	20.0	0.0	0.470	871
Glaciomarine	2	3.5-9.6	6.1	6.6	5.3	0.0	0.438	812
Sand and Gravel	3	9.6-13.7	4.1	11.7	3.0	0.0	0.396	734

Trapezoid B:

Layer		Depth below footing (ft)	Layer Thickness (ft)	z (ft)	a/z	b/z	I	Stress Increase σ_z (psf)
Existing Fill	1	0-3.5	3.5	1.8	16.5	1.1	0.490	908
Glaciomarine	2	3.5-9.6	6.1	6.6	4.4	0.3	0.454	841
Sand and Gravel	3	9.6-13.7	4.1	11.7	2.5	0.2	0.405	750

Total Footing (Trapezoid A + Trapezoid B):

Layer		Depth below footing (ft)	Layer Thickness (ft)	z (ft)	Stress Increase (psf)
Existing Fill	1	0-3.5	3.5	1.8	1779
Glaciomarine	2	3.5-9.6	6.1	6.6	1653
Sand and Gravel	3	9.6-13.7	4.1	11.7	1484

B. Use consolidation theory to estimate settlement of the glaciomarine silty clay layer beneath the embankment (Layer 2); use the Hough method to estimate settlement of the existing fill and sand/gravel layers beneath the embankment (Layers 1 and 3).

1 Determine σ'_{v0} and C' at layer midpoints for in situ existing soil. Use Ref. 6 Figures 5-18 and 5-19 to obtain corrected N' and C' for Layers 1 and 3, assuming the existing fill is "well graded fine to medium silty SAND" and the sand/gravel is "well graded silty SAND & GRAVEL".

Stress due to existing soil above footing	σ'_{v0} (psf)
	0

Water unit weight (pcf)
62.4

CALCULATIONS

Date:	7/8/2020	**Made by:**	KAR
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Project Short Title:	MaineDOT I-295 Exit 20 Merrill Road Bridge Replacement No. 5720		

Layer		Layer Thickness (ft)	Unit Weight of Layer (pcf)	σ'$_{v0}$ at layer midpoint (psf)	σ'$_{v0}$ at layer midpoint (kPa)	N	N'	C'
Existing Fill	1	3.5	125	219	10.5	23	46	110
Glaciomarine	2	6.1	125	819	39.2	Not required for clay consolid.		
Sand and Gravel	3	4.1	125	1147	54.9	13	17	68

2 Calculate the total settlement of the existing fill and sand/gravel (Layers 1 and 3) using the Hough method:

General Equation (Ref. 6, Eqn 5-24)

$$\Delta H_i = H_c \frac{1}{C'} \log\left(\frac{\sigma'_{v0} + \Delta\sigma_v}{\sigma'_{v0}}\right)$$

where:

- ΔH_i settlement in each layer, ft
- H_c initial height of layer i, ft
- C' bearing capacity index from Ref. 6, Figure 5-19
- $\Delta\sigma_v$ vertical stress increase, ksf

ΔH_i (Layer 1)	ft	0.031
	in	0.37

ΔH_i (Layer 3)	ft	0.022
	in	0.26

3 Calculate the total settlement of the glaciomarine silty clay (Layer 2) using the consolidation theory:

General Equation (Ref. 5, Eqn 8-11, 8-16, 8-18b)

$$\Delta H_i = C_c \frac{H_0}{1+e_0} \log \frac{\sigma'_{v0} + \Delta\sigma_v}{\sigma'_{v0}}$$ for normally consolidated clay

$$\Delta H_i = C_r \frac{H_0}{1+e_0} \log \frac{\sigma'_{v0} + \Delta\sigma_v}{\sigma'_{v0}}$$ for overconsolidated clay, $\sigma'_{v0} + \Delta\sigma_v \le \sigma'_p$

$$\Delta H_i = C_r \frac{H_0}{1+e_0} \log \frac{\sigma'_p}{\sigma'_{v0}} + C_c \frac{H_0}{1+e_0} \log \frac{\sigma'_{v0} + \Delta\sigma_v}{\sigma'_p}$$ for overconsolidated clay, $\sigma'_{v0} + \Delta\sigma_v > \sigma'_p$

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where:			Layer 2	
H_0	initial height of layer i, ft		6.1	
$\Delta\sigma_v$	surcharge load, psf		1653	
σ'_{v0}	in situ vertical effective stress, psf		819	
$\sigma'_{v0} + \Delta\sigma_v$			2472	
σ'_p	preconsolidated stress, psf		N/A	
C_c	compression index, $C_c = C_{c\varepsilon}(1+e_0)$		0.42	
C_r	recompression index, $C_r = C_{r\varepsilon}(1+e_0)$		0.03	
e_0	initial void ratio		0.68	
		Use equation:	8-16	(Assumption 5)
ΔH_i		ft	0.059	
		in	0.70	

Layer	Settlement Based on Calc. Loading Stress ΔH_i (in)
1	0.37
2	0.70
3	0.26
Total Settlement (in)	1.33

C. Determine the time rate of settlement that will occur at the embankment location.

Use a single layer analysis to determine the time for the entire settlement to occur, assuming double drainage conditions based on the soil types above and below the glaciomarine silty clay layer having a higher permeability relative to the clay layer (Reference 3). Subsurface conditions are simplified to a single clay layer with uniform soil properties for the purpose of the calculation.

$$T_v = \frac{c_V t}{H^2_{dr}}$$ (Ref. 5, Eqn 9-5)

$$T_v = \frac{\pi}{4}\left(\frac{U\%}{100}\right)^2$$ For U = 0% to 60% (Ref. 5, Eqn 9-10)

$$T_v = 1.781 - 0.933\log(100 - U\%)$$ For U ≥ 60% (Ref. 5, Eqn 9-11)

$$U_{avg} = \frac{s(t)}{S_c}$$ (Ref. 5, Eqn 9-12)

where:

T_v = time factor (dimensionless)
c_v = coefficient of consolidation (ft^2/day)
t = time (day)
H_{dr} = drainage path (ft)
U_{avg} = average degree or percent consolidation
s(t) = settlement at any time (in)
s_c = final or ultimate settlement (in)

Based on Reference 1, the clay layer thickness below the footing is: 6.1 ft
With double drainage conditions, H_{dr} = 3.1 ft

The coefficient of consolidation value is assumed based on local engineering experience.

c_v = 120 ft^2/yr = 0.33 ft^2/day

Determine time factor using Reference 5, Eqn 9-11

U_{avg} = 95 (Calculate time to reach 95% consolidation)
T_v = 1.13

Determine time required for settlement to occur using Reference 5, Eqn 9-5

c_v = 0.33 ft^2/day
H_{dr} = 3.1 ft
T_v = 1.13

t = 32 days = 0.09 years (95% consolidation)

s_c (in) = 0.70 (final potential settlement of the clay layer, Part B)

Time (days)	Time (years)	T_v	U_{avg} (%)	s(t) (in)
0	0.00	0.00	0.00	0.00
5	0.01	0.18	47.43	0.33
10	0.03	0.35	66.11	0.46
15	0.04	0.53	78.09	0.55
20	0.05	0.71	85.83	0.60
25	0.07	0.88	90.84	0.64
30	0.08	1.06	94.08	0.66
35	0.10	1.24	96.17	0.67
40	0.11	1.41	97.52	0.68
45	0.12	1.59	98.40	0.69
50	0.14	1.77	98.97	0.69

GOLDER

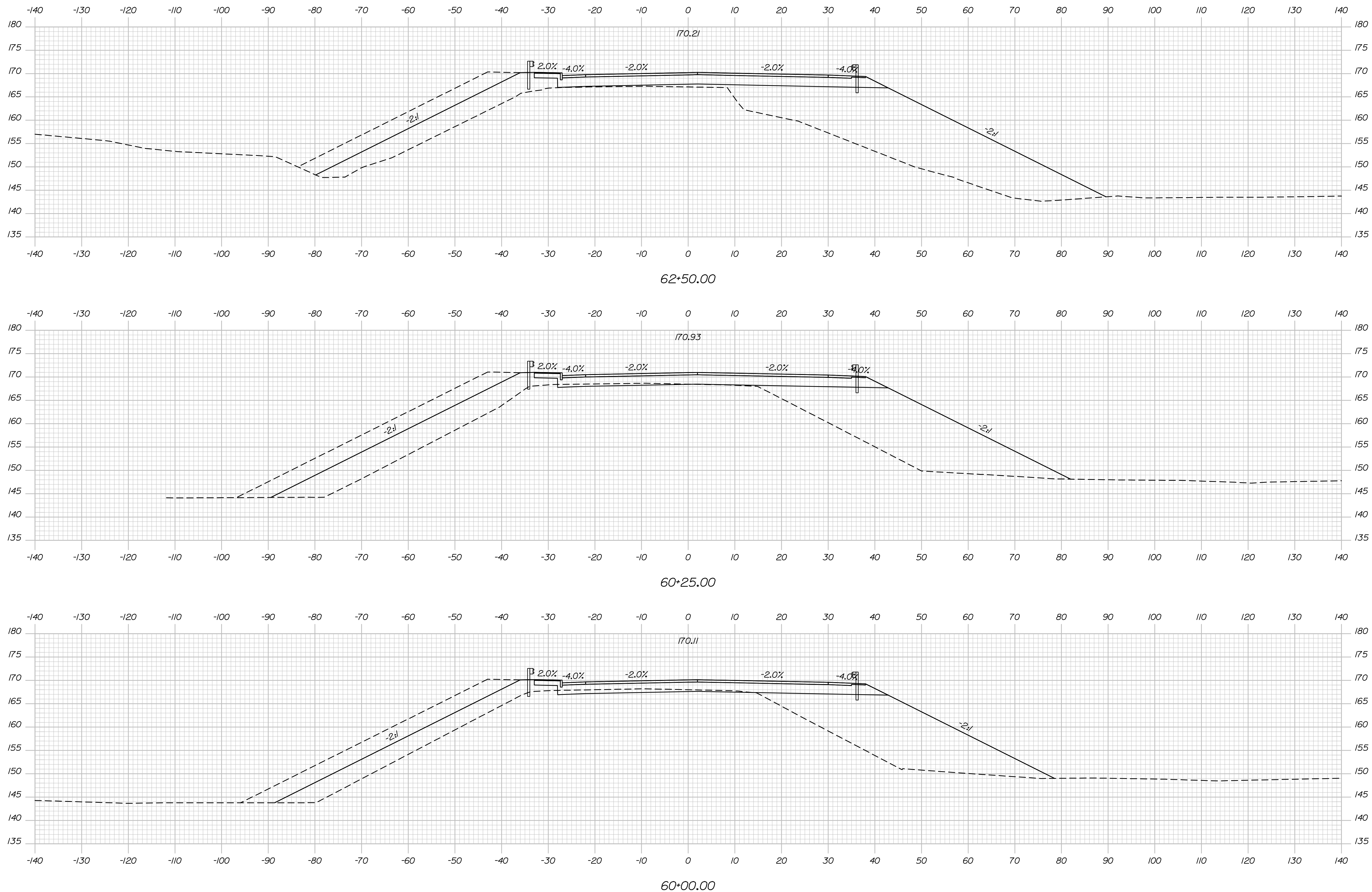
CALCULATIONS

Date:	7/8/2020	**Made by:**	KAR
Project No.:	19126013	**Checked by:**	MEL
Subject:	Settlement at SE Bridge Embankment - Southern Shift	**Reviewed by:**	MCM
Project Short Title:	MaineDOT I-295 Exit 20 Merrill Road Bridge Replacement No. 5720		

CONCLUSIONS

The primary settlement at the base of the southeastern proposed bridge approach embankment after construction (assuming the "southern shift" option) is estimated to be 1.33 inches. This includes 0.63 inches of immediate settlement and 0.70 inches of consolidation settlement that is estimated to reach 95% consolidation in 32 days.

Reference 4



STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

HIGHWAY PLANS

WIN
023627.00

Bridge No. 5720

BY	DATE
	11\19
	11\19

PROJ. MANAGER
DESIGN-DETAILED
CHECKED-REVIEWED
DESIGN2-DETAILED2
DESIGN3-DETAILED3
REVISIONS 1
REVISIONS 2
REVISIONS 3
REVISIONS 4
FIELD CHANGES

SIGNATURE
P.E. NUMBER
DATE

MERRILL ROAD BRIDGE
INTERSTATE 295
FREEPORT
CUMBERLAND

DESERT ROAD SOUTH
CROSS SECTIONS

SHEET NUMBER
Wor
OF 4

Date:6/22/2020
Username:
Division:
Filename: Working Sections.dgn

Sta. 60+00.00 to Sta. 62+50.00

Reference 5

vertical stress is

$$\sigma_z = \frac{2P}{\pi}\,\frac{z^3}{x^4} \tag{8-26}$$

where P = line load, and
$x = (z^2 + r^2)^{1/2}$ (see Fig. 8.20a).
Equations for the horizontal and shear stress are also available.

The next logical step is to integrate a line load over a finite area. Newmark (1935) performed the integration of Eq. 8.26 and derived the following equation for the vertical stress under the corner of a *uniformly loaded rectangular area*:

$$\sigma_z = q_o \frac{1}{4\pi}\left[\frac{2mn(m^2+n^2+1)^{1/2}}{m^2+n^2+1+m^2n^2} \times \frac{(m^2+n^2+2)}{(m^2+n^2+1)} + \arctan\frac{2mn(m^2+n^2+1)^{1/2}}{m^2+n^2+1-m^2n^2}\right] \tag{8-27}$$

where q_o = surface or contact stress,
$m = x/z$, (8-28)
$n = y/z$, and (8-29)
x, y = length and width of the uniformly loaded area, respectively.
The parameters m and n are interchangeable. Fortunately Eq. 8-27 may be rewritten as

$$\sigma_z = q_o I \tag{8-30}$$

where I = an influence value which depends on m and n.

Values of I for various values of m and n are shown in Fig. 8.21.

EXAMPLE 8.18

Given:

The 3 × 4 m rectangular footing of Example 8.17 is loaded uniformly by 117 kPa.

Required:

a. Find the vertical stress under the corner of the footing at a depth of 2 m.

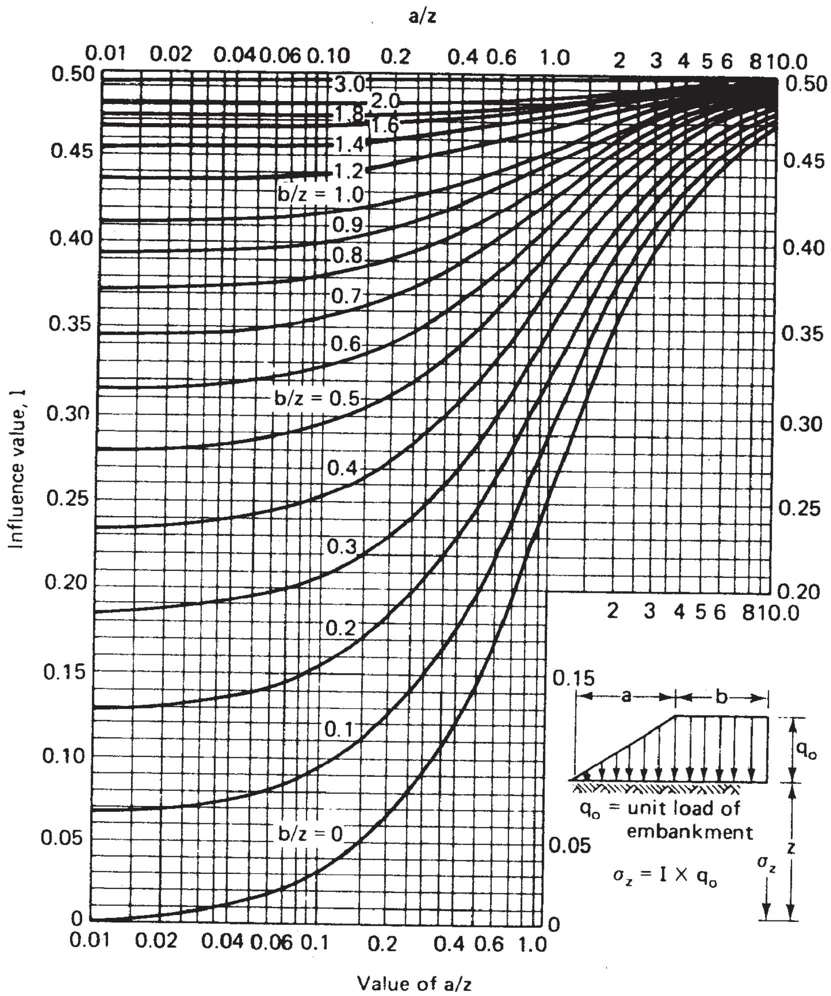


Fig. 8.23 Influence values for vertical stress under a very long embankment; length = ∞ (from U.S. Navy, 1971, after Osterberg, 1957).

A second graphical way is to find Δe over *one* cycle; for example, $\log \frac{1000}{100} = \log 10 = 1$. When this is done, $C_c = \Delta e$. In Fig. Ex. 8.9 the vertical scale is not sufficient for $\Delta \sigma' = 1$ log cycle, but it can be done in two steps, e_a to e_b and e_c to e_d. (To extend the line $\overline{e_a e_b}$ to one full log cycle on the *same* graph, choose e_c at the same pressure as e_b. Then draw the line $\overline{e_c e_d}$ parallel to $\overline{e_a e_b}$. This second line is merely the extension of $\overline{e_a e_b}$ if the graph paper extended lower than shown.) Or,

$$\begin{aligned}\Delta e = C_c &= (e_a - e_b) + (e_c - e_d)\\ &= (0.870 - 0.655) + (0.90 - 0.664)\\ &= 0.215 + 0.236\\ &= 0.451\text{, or same as above}\end{aligned}$$

c. The modified compression index $C_{c\epsilon}$ is

$$C_{c\epsilon} = \frac{C_c}{1 + e_o} = \frac{0.451}{1 + 0.865} = 0.242$$

To calculate consolidation settlement, Eqs. 8-5, 8-6, or 8-7 and 8-8 may be combined with Eq. 8-4. For example, using Eqs. 8-7 and 8-4 we obtain

$$s_c = C_c \frac{H_o}{1 + e_o} \log \frac{\sigma_2'}{\sigma_1'} \tag{8-10}$$

If the soil is normally consolidated, then σ_1' would be equal to the existing vertical overburden stress σ_{vo}', and σ_2' would include the additional stress $\Delta \sigma_v$ applied by the structure, or

$$s_c = C_c \frac{H_o}{1 + e_o} \log \frac{\sigma_{vo}' + \Delta \sigma_v}{\sigma_{vo}'} \tag{8-11}$$

When computing the settlement by means of the percent consolidation versus log effective stress curve, Eq. 8-8 is combined with Eq. 8-4 to get

$$s_c = C_{c\epsilon} H_o \log \frac{\sigma_2'}{\sigma_1'} \tag{8-12}$$

or, analogous to Eq. 8-11, for normally consolidated clays,

$$s_c = C_{c\epsilon} H_o \log \frac{\sigma_{vo}' + \Delta \sigma_v}{\sigma_{vo}'} \tag{8-13}$$

Other similar settlement equations can be derived using a_v and m_v. In this case the average stress for a given stress increment must be used since the compression curves are nonlinear.

Required:

Calculate (a) the recompression index C_r and (b) the modified recompression index $C_{r\epsilon}$.

Solution:

a. The recompression index C_r is found in a similar manner to the C_c (Eq. 8-7). Using the points e and f over 1 log cycle, we find that

$$C_r = e_e - e_f = 0.790 - 0.760 = 0.030$$

b. The modified recompression index $C_{r\epsilon}$ is found from Eq. 8-15.

$$C_{r\epsilon} = \frac{C_r}{1 + e_o} = \frac{0.030}{1 + 0.865} = 0.016$$

Note that neither of these terms has units.

To calculate settlements of overconsolidated clays, Eqs. 8-11 and 8-13 become

$$s_c = C_r \frac{H_o}{1 + e_o} \log \frac{\sigma'_{vo} + \Delta\sigma_v}{\sigma'_{vo}} \tag{8-16}$$

$$s_c = C_{r\epsilon} H_o \log \frac{\sigma'_{vo} + \Delta\sigma_v}{\sigma'_{vo}} \tag{8-17}$$

when $\sigma'_{vo} + \Delta\sigma_v \leq \sigma'_p$. Since C_r is usually much less than C_c, the settlements occurring when $\sigma'_{vo} + \Delta\sigma_v \leq \sigma'_p$ are much less than if the soil were normally consolidated.

If the added stress caused by the structure exceeds the preconsolidation stress, then much larger settlements would be expected. This is because the compressibility of the soil is much greater on the virgin compression curve than on the recompression curve as was shown, for example, in Fig. 8.7. For the case, then, where $\sigma'_{vo} + \Delta\sigma_v > \sigma'_p$ the settlement equation consists of two parts: (1) the change in void ratio or strain on the recompression curve from the original in situ conditions of (e_o, σ'_{vo}) or $(\epsilon_{vo}, \sigma'_{vo})$ to σ'_p; and (2) the change in void ratio or strain on the virgin compression curve from σ'_p to the final conditions of (e_f, σ'_{vf}) or $(\epsilon_{vf}, \sigma'_{vf})$. Note that $\sigma'_{vf} = \sigma'_{vo} + \Delta\sigma_v$. These two parts are shown graphically in Fig. 8.10b. The complete settlement equation then becomes

$$s_c = C_r \frac{H_o}{1 + e_o} \log \frac{\sigma'_{vo} + (\sigma'_p - \sigma'_{vo})}{\sigma'_{vo}} + C_c \frac{H_o}{1 + e_o} \log \frac{\sigma'_p + (\sigma'_{vo} + \Delta\sigma_v - \sigma'_p)}{\sigma'_p} \tag{8-18a}$$

This equation reduces to

$$s_c = C_r \frac{H_o}{1 + e_o} \log \frac{\sigma'_p}{\sigma'_{vo}} + C_c \frac{H_o}{1 + e_o} \log \frac{\sigma'_{vo} + \Delta\sigma_v}{\sigma'_p} \tag{8-18b}$$

In terms of the modified indices, we have

$$s_c = C_{r\epsilon} H_o \log \frac{\sigma'_p}{\sigma'_{vo}} + C_{c\epsilon} H_o \log \frac{\sigma'_{vo} + \Delta\sigma_v}{\sigma'_p} \tag{8-19}$$

Both Eqs. 8-18 and 8-19 give the same results. One could argue that in the right-hand term of Eq. 8-18 the void ratio corresponding to the preconsolidation pressure on the true virgin compression curve should be used. Although this is technically correct, it doesn't make any significant difference in the answer.

Sometimes the degree of overconsolidation varies throughout the compressible layer. You could apply Eq. 8-16 or 8-17 to the part where $\sigma'_{vo} + \Delta\sigma_v < \sigma'_p$ and Eq. 8-18 or 8-19 to the part where $\sigma'_{vo} + \Delta\sigma_v > \sigma'_p$. In practice, however, it is usually easier to simply divide the entire stratum into several layers, apply the appropriate equation to calculate the average settlement for each layer, and then sum up the settlements by Eq. 8-14.

What is the best way to get C_r and $C_{r\epsilon}$ for use in Eqs. 8-16 through 8-19? Because of sample disturbance, the slope of the initial recompression portion of the laboratory consolidation curve (Fig. 8.7) is too steep and would yield values that are too large for these indices. Leonards (1976) offers the reasons why in situ values are generally smaller than those obtained from laboratory measurements: (1) disturbance during sampling, storage, and preparation of test specimens; (2) recompression of gas bubbles in the voids; and (3) errors in test procedures and methods of interpreting test results. This latter item includes the problem of reproducing the in situ state of stress in the specimen. Leonards recommends that the σ'_{vo} be applied to the specimen and that it be innundated and allowed to come to equilibrium for at least 24 hours before starting the incremental loading. Any tendency to swell should be controlled. Then the consolidation test is continued with relatively large load increments. To reproduce as closely as possible the in situ stress state, Leonards recommends that the sample be consolidated to slightly less than the σ'_p and then be allowed to rebound. This is the first cycle shown in Fig. 8.11. If you don't have a good idea of the σ'_p, then consolidate initially to $\sigma'_{vo} + \Delta\sigma_v$ only, which is presumably less than σ'_p. The determination of C_r or $C_{r\epsilon}$ is over the range of $\sigma'_{vo} + \Delta\sigma_v$, as shown in Fig. 8.11. It is common practice to take the average slope of the two curves. From the typical test results shown in Fig. 8.11, you can see that the actual values of the recompression index depend on the stress at which the rebound-reload cycle starts, especially whether it starts at a stress less than or greater than the σ'_p. See the difference in

Equation 9-2 is the *Terzaghi one-dimensional consolidation equation*. It could just as easily be written in three dimensions, but most of the time in engineering practice one-dimensional consolidation is assumed. Basically, the equation is a form of the diffusion equation from mathematical physics. Many physical diffusion phenomena are described by this equation, for example, heat flow in a solid body. The "diffusion constant" for the soil is the c_v. Note that we called the c_v a constant. It really isn't, but we must assume it is, that is, that k, a_v, and e_0 are constants, in order to make the equation linear and easily solvable.

So how do we solve the Terzaghi consolidation equation? Just like we solve all other second-order partial differential equations with constant coefficients. There are a variety of ways; some are mathematically exact; others are only approximate. For example, Harr (1966) presents an approximate solution by using the method of finite differences. Taylor (1948), following Terzaghi (1925), gives a mathematically rigorous solution in terms of a Fourier series expansion, and this is what we do in detail in Appendix B-2. Here we shall just give an outline of the solution. First, the boundary and initial conditions for the case of one-dimensional consolidation are:

1. There is complete drainage at the top and bottom of the compressible layer.
2. The initial *excess* hydrostatic pressure $\Delta u = u_i$ is equal to the applied increment of stress at the boundary, $\Delta\sigma$.

We can write these boundary and initial conditions as follows:

$$\begin{aligned}&\text{When } z = 0 \text{ and when}\\ &z = 2H,\ u = 0\\ &\text{When } t = 0,\ \Delta u = u_i = \Delta\sigma = (\sigma_2' - \sigma_1')\end{aligned}$$

We usually take the thickness of the consolidating layer to be $2H$, so that the *length of the longest drainage path* is equal to H or H_{dr}. Of course at $t = \infty$, $\Delta u = 0$, or complete dissipation of the pore pressure will have occurred.

Terzaghi (1925) was obviously familiar with the early work on heat transfer, and he adapted those closed-form solutions to the consolidation problem. The solution comes out in terms of a Fourier series expansion of the form

$$u = (\sigma_2' - \sigma_1') \sum_{n=0}^{\infty} f_1(Z) f_2(T) \tag{9-4}$$

where Z and T are dimensionless parameters (see also Taylor, 1948). The first term, Z, is a geometry parameter, and it is equal to z/H. The second term, T, is known as the *time factor*, and it is related to the coefficient of consolidation c_v by

$$T = c_v \frac{t}{H_{dr}^2} \tag{9-5}$$

where t = time, and

H_{dr} = length of the longest drainage path.

We have already mentioned that c_v has dimensions of L^2T^{-1} or units of m^2/s (or equivalent).

From Eq. 9-3, the time factor can also be written as

$$T = \frac{k(1 + e_o)}{a_v \rho_w g} \frac{t}{H_{dr}^2} \tag{9-6}$$

Note that t has the same time units as k. That is, if k is in centimetres per second, then t must be in seconds. The drainage path for double drainage would be equal to half the thickness H of the clay layer, or $2H/2 = H_{dr}$. If we had only a singly drained layer, the drainage path would still be H_{dr}, but then it would be equal to the thickness H of the layer.

The progress of consolidation after some time t and at any depth z in the consolidating layer can be related to the void ratio at that time and the final change in void ratio. This relationship is called the *consolidation ratio*, and it is expressed as

$$U_z = \frac{e_1 - e}{e_1 - e_2} \tag{9-7}$$

where e is some intermediate void ratio, as shown on Fig. 9.2. What we are looking at graphically in that figure is the ratio of ordinates corresponding to AB and AC. In terms of stresses and pore pressures, Eq. 9-7 becomes

$$U_z = \frac{\sigma' - \sigma_1'}{\sigma_2' - \sigma_1'} = \frac{\sigma' - \sigma_1'}{\Delta\sigma'} = \frac{u_i - u}{u_i} = 1 - \frac{u}{u_i} \tag{9-8}$$

where σ' and u are intermediate values corresponding to e in Eq. 9-7, and u_i is the initial *excess* pore pressure induced by the applied stress $\Delta\sigma'$. You should satisfy yourself that these equations are correct from the relationships shown in Fig. 9.2 and from $\Delta\sigma' = -\Delta u$. (See also Appendix B-2.)

From Eqs. 9-7 and 9-8, it is evident that U_z is zero at the start of loading, and it gradually increases to 1 (or 100%) as the void ratio decreases from e_1 to e_2. At the same time, of course, as long as the total stress remains constant, the effective stress increases from σ_1' to σ_2' as the excess hydrostatic stress (pore water pressure) dissipates from u_i to zero. The consolidation ratio U_z is sometimes called the *degree or percent consolidation*, and it represents conditions *at a point* in the consolidating layer. It is now possible to put our solution for u in Eq. 9-4 in terms of the

TABLE 9-1

U_{avg}	T
0.1	0.008
0.2	0.031
0.3	0.071
0.4	0.126
0.5	0.197
0.6	0.287
0.7	0.403
0.8	0.567
0.9	0.848
0.95	1.163
1.0	∞

where the initial pore pressure distribution is sinusoidal, half sine, and triangular are presented by Leonards (1962).

Casagrande (1938) and Taylor (1948) provide the following useful approximations:

For $U < 60\%$,

$$T = \frac{\pi}{4}U^2 = \frac{\pi}{4}\left(\frac{U\%}{100}\right)^2 \tag{9-10}$$

For $U > 60\%$,

$$T = 1.781 - 0.933 \log(100 - U\%) \tag{9-11}$$

EXAMPLE 9.3

Given:

$T = 0.05$ for a compressible clay deposit.

Required:

Average degree of consolidation and the percent consolidation at the center and at $z/H = 0.1$.

Solution:

From Table 9-1 and Fig. 9.5, $U_{avg} = 26\%$. Therefore the clay is 26% consolidated, on the average. From Fig. 9.3 you can see that the center of the layer is less than 0.5% consolidated, while at the "10%" depth ($z/H = 0.1$) the clay is 73% consolidated. But, *on the average* throughout the layer, the clay is 26% consolidated.

What does the average consolidation mean in terms of settlements? U_{avg} can be expressed as

$$U_{avg} = \frac{s(t)}{s_c} \tag{9-12}$$

where $s(t)$ is the settlement at any time, and s_c is the final or ultimate consolidation (primary) settlement at $t = \infty$.

EXAMPLE 9.4

Given:

The data of Example 9.3.

Required:

Find the settlement when U_{avg} is 26%, if the final consolidation settlement is 1 m.

Solution:

From Eq. 9-12, $s(t) = U_{avg}(s_c)$. Therefore

$$s(t) = 26\% \text{ (1 m)} = 0.26 \text{ m}$$

EXAMPLE 9.5

Given:

The soil profile and properties of Examples 9.1 and 9.2.

Required:

Compute the time required for the clay layer to settle 0.25 m.

- The D'Appolonia method under-predicted the settlement by an average factor of about 0.77 for the five footings, but the accuracy was generally better than Hough, and the scatter in the predicted settlement values was also less (standard deviation, $s_o = 0.22$).

The data from the five load tests therefore generally corroborate the conclusions from the Gifford et al. and Burland & Burbidge studies.

Therefore, it is currently recommended that the Hough method be used as a primary design and analysis tool. The D'Appolonia or load-settlement methods could be employed as alternates, or when a more accurate estimate of settlement is desired, recognizing that these alternate methods have less history of use in the design of highway bridges. Naturally, if another method is locally or regionally known to provide more reliable estimates of settlement on granular soils, and the method meets the criteria noted above, the geotechnical engineer should exercise judgment and use the best method available.

5.3.4.1 Hough Method

Hough (1959) developed an empirical method for predicting settlements of shallow foundations on cohesionless soils that follows the same approach as that used for calculating consolidation settlement of clay layers. Note that the method is applicable only for normally consolidated cohesionless soils. Cheney and Chassie (2000) recommend that the SPT blowcounts be corrected for overburden pressure before correlating the N-values to the bearing capacity index, C′. An overburden correction by Bazaraa (1967) was recommended by Cheney and Chassie (2000). Since that time, many researchers have studied the effect of overburden stress on the SPT N-value, largely in support of liquefaction hazard assessment procedures. Recent consensus by the 1996 and 1998 National Center for Earthquake Engineering Research (NCEER) (Youd et al., 2001) concluded that the correction proposed by Liao & Whitman (1986) (shown in Figure 5-18) could be used for routine engineering applications. Therefore, the correction by Liao & Whitman is included here as part of the Hough procedure, in particular because it is easy to calculate and can be used without charts in simple computation spreadsheets. The soil is divided into layers, and the change in effective vertical stress at the mid-height of the layer as a result of the applied load is estimated using elastic theory.

The total settlement by the Hough method is calculated as follows:

1. Correct SPT blowcounts for overburden stress using Figure 5-18.

2. Determine bearing capacity index (C′) from Figure 5-19 using corrected SPT blowcounts, N′, determined in Step 1.

3. Subdivide subsurface soil profile into approximately 3-m (10-ft) layers based on stratigraphy to a depth of about three times the footing width.

4. Calculate the effective vertical stress, σ'_{vo}, at the midpoint of each layer and the average bearing capacity index for that layer.

5. Calculate the increase in stress at the midpoint of each layer, $\Delta\sigma_v$, using either Figure 5-9, 5-10 or 5-11, or the 2:1 method (Figure 5-12).

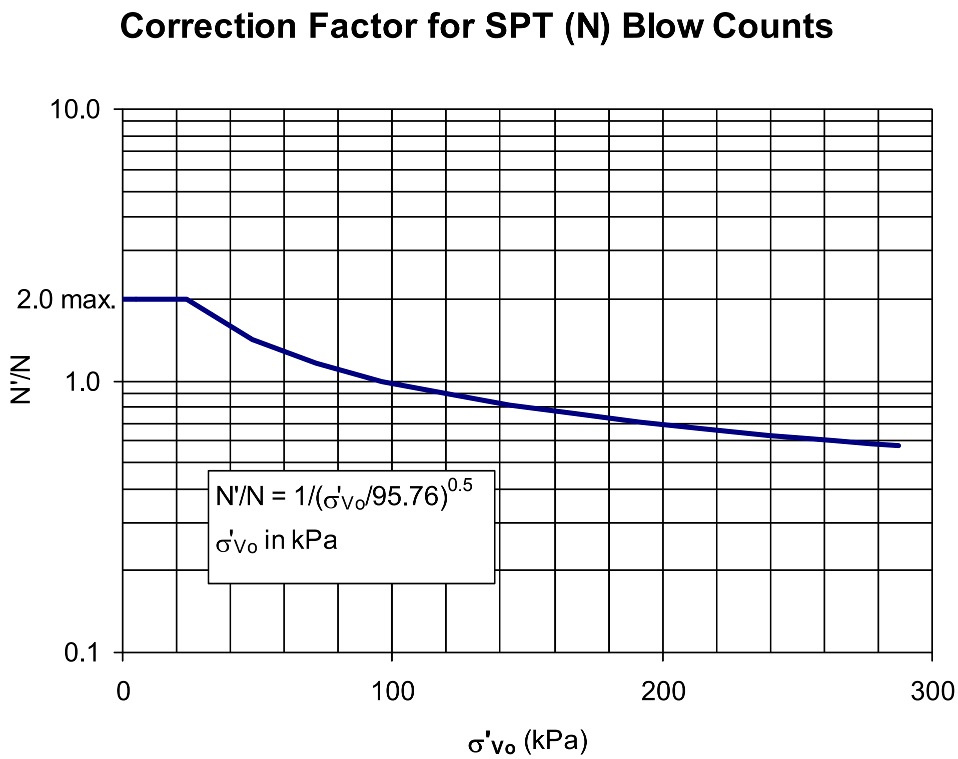


Figure 5-18: Corrected SPT (N) versus Overburden Pressure (after Liao & Whitman, 1986)

6. Calculate the settlement in each layer, ΔH, under the applied load using the following formula:

$$\Delta H \quad = H_o \frac{1}{C'} \log\left(\frac{\sigma'_{vo} + \Delta\sigma_{v_f}}{\sigma'_{vo}}\right) \tag{5-24}$$

7. Sum the incremental settlements to determine the total settlement.

5.3.4.2 D'Appolonia Method

The D'Appolonia method calculates settlement using the following basic equation:

$$\Delta H = \left(\frac{\Delta\sigma_V B_f}{M}\right)\mu_o\mu_1 \tag{5-25}$$

where: ΔH = settlement in sand or sand and gravel
$\Delta\sigma_v$ = applied stress beneath footing
B_f = footing width
μ_O = correction factor for embedment – see Figure 5-20 (dimensionless)
μ_1 = correction factor for thickness of sand layer – see Figure 5-20 (dimensionless)
M = modulus of compressibility of sand – see Figure 5-21

Any consistent set of units can be used for ΔH, $\Delta\sigma_v$, B_f and M.

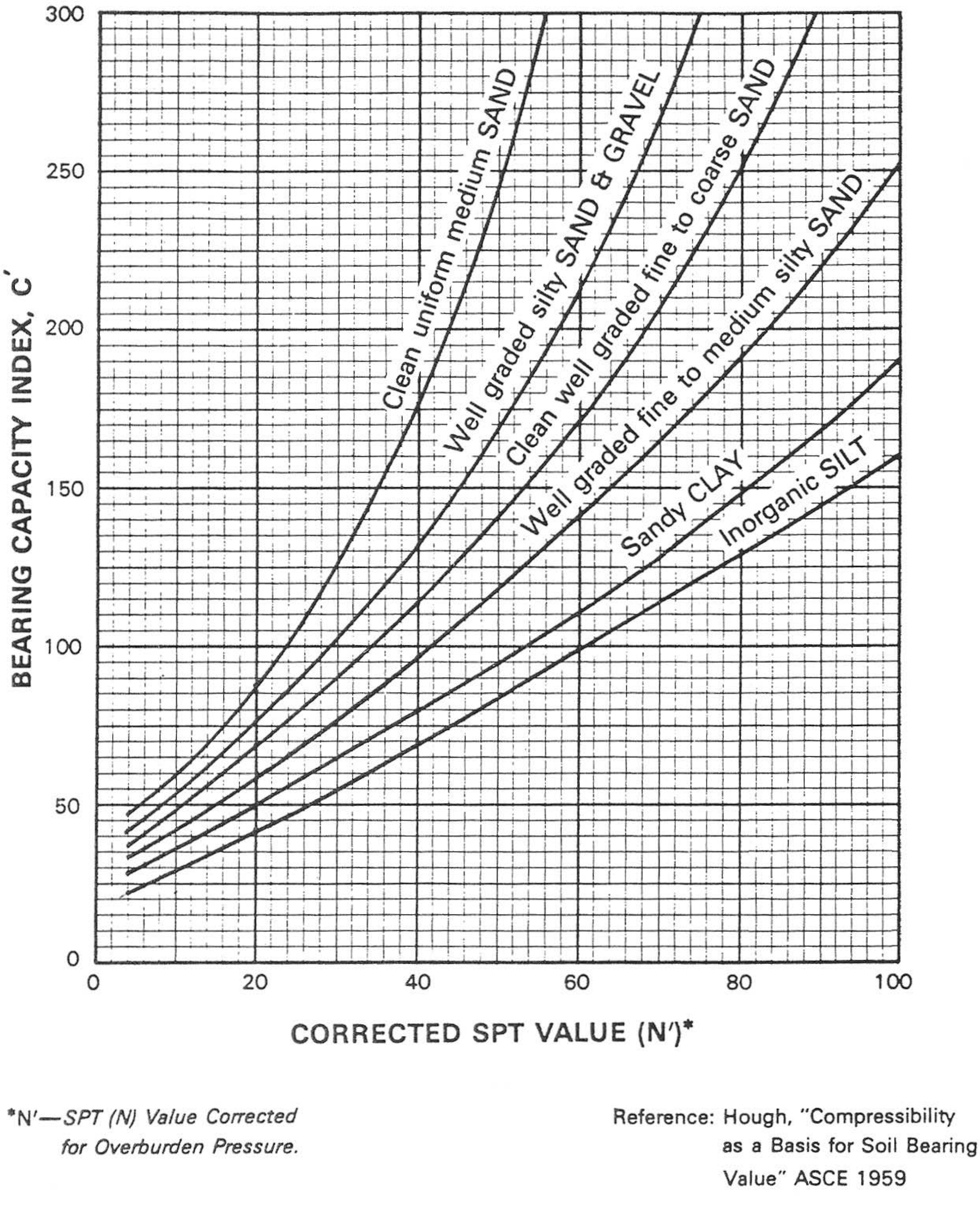


Figure 5-19: Bearing Capacity Index versus Corrected SPT
(Cheney & Chassie, 2000, modified from Hough, 1959)

GOLDER

CALCULATIONS

Date:	7/30/2020	**Made by:**	KAR
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Subject:	Settlement of Water Main at NW Embankment - Southern Shift	**Reviewed by:**	MCM
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OBJECTIVE

Calculate the primary settlement beneath the existing water main after construction of the northwestern proposed bridge approach embankment (assuming the "southern shift" option) at a location where embankment fill is thickest (southern shift alignment at HNTB Station 60+25).

REFERENCES

1. Golder calculation titled "Global Stability Analysis NW Embankment", dated July 29, 2020 (Appendix E, Preliminary Geotechnical Design Report, dated September 2020).
2. Golder boring location plan (Figure 2, Preliminary Geotechnical Design Report, dated September 2020).
3. Golder geotechnical test boring logs (Appendix A, Preliminary Geotechnical Design Report, dated September 2020).
4. HNTB for State of Maine Department of Transportation. Merrill Road Bridge Freeport Interstate 295: Desert Road South Cross Sections, dated November 2019.
5. Holtz, R.D. and Kovacs, W.D. 1981. An Introduction to Geotechnical Engineering, 1st ed. Prentice Hall, Englewood Cliffs, NJ.
6. FHWA. 2002. Geotechnical Engineering Circular No. 6: Shallow Foundations. Report No. FHWA-SA-02-054.
7. Maine Water Company. Water Main Installation Instructions, dated January 9, 2017.
8. Maine Water Company. Water Main Trench Detail. Drawing Number MWC SD-4, dated March 9, 2016.
9. Das, Braja M. 2011. Principles of Foundation Engineering, 7th Edition. Cengage Learning.
10. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.
11. GeoTesting Express laboratory testing results, dated February 5, 2020 (Appendix C, Geotechnical Design Report, dated July 2020).

ASSUMPTIONS

1. The existing stratigraphy and water table surface are estimated from samples and groundwater measurements encountered during drilling and interpreted in Reference 1.	GS elev. =	149.2	ft
	WL elev. =	146.3	ft
2. N_{60}-value for the trench bottom bedding material, based on correlation (Ref. 9, Eqn. 2.26) to the MaineDOT specified friction angle for granular borrow (Ref. 10, Table 3-3), is:	$N_{60,Trench}$ =	16	
3. N_{60}-value for the native sand/gravel layer, based on the average of the N_{60}-values encountered in all borings for the layer (Reference 3) is:	$N_{60,S/G}$ =	13	
4. The proposed top of slope and base of embankment elevations at the location of analysis (Reference 2) are (Reference 4, Section 62+50):	El. top =	158.4	ft
	El. base =	149.2	ft
5. Based on the DigSafe tickets issued for the borings, the utility owner of the water main is Maine Water Company. Minimum pipe diameter, depth of cover, and trench construction details for the water main were assumed based on values provided in Maine Water Company standard specifications (Ref. 7 and 8).	Min. pipe diameter =	8	in
	Depth of cover =	5.5	ft
	Screen above pipe =	12	in
	Trench below pipe =	6	in
6. Clay consolidation parameters for the glaciomarine silty clay layer, based on Golder's local engineering experience ($C_{c\varepsilon}$, $C_{r\varepsilon}$, c_v) and calculated (e_0) from soil moisture contents determined in laboratory testing (Reference 11), are:	$C_{c\varepsilon}$ =	0.25	
	$C_{r\varepsilon}$ =	0.02	
	e_0 =	0.68	
	c_v =	120	ft^2/yr

7. Assume that the glaciomarine silty clay is sufficiently overconsolidated and will experience recompression settlement only after loading (based on Golder's local engineering experience).

CALCULATION

A. Determine the change in effective stress state within the soil beneath the water main to identify if settlement or heave will occur. Calculate the vertical stress increase beneath the water main due to embankment construction.

The change in effective stress state due to change in stratigraphy is determined at the assumed bottom-of-pipe elevation 143.0 ft (existing ground surface minus depth of cover minus pipe diameter). As per References 7 and 8, the existing trench construction above the pipe is assumed to consist of 1 foot of compacted select screen sand overlain by compacted native backfill to finished grade, for a total cover thickness of 5.5 feet.

Existing Conditions (References 7 and 8):

Layer	Unit weight (pcf)	Elevation (ft)		Thickness (ft)	Stress Contribution of Layer (psf)	Effective Stress Contribution of Layer (psf)
		Top	Bottom			
Compacted Native Backfill	130	149.2	144.7	4.5	585.0	485.2
Compacted Select Screen Sand	125	144.7	143.7	1.0	125.0	62.6
Pipe (assume full of water and neglect wall thickness)	62.4	143.7	143	0.7	43.7	43.7

After Construction (Reference 4, Section 60+25):

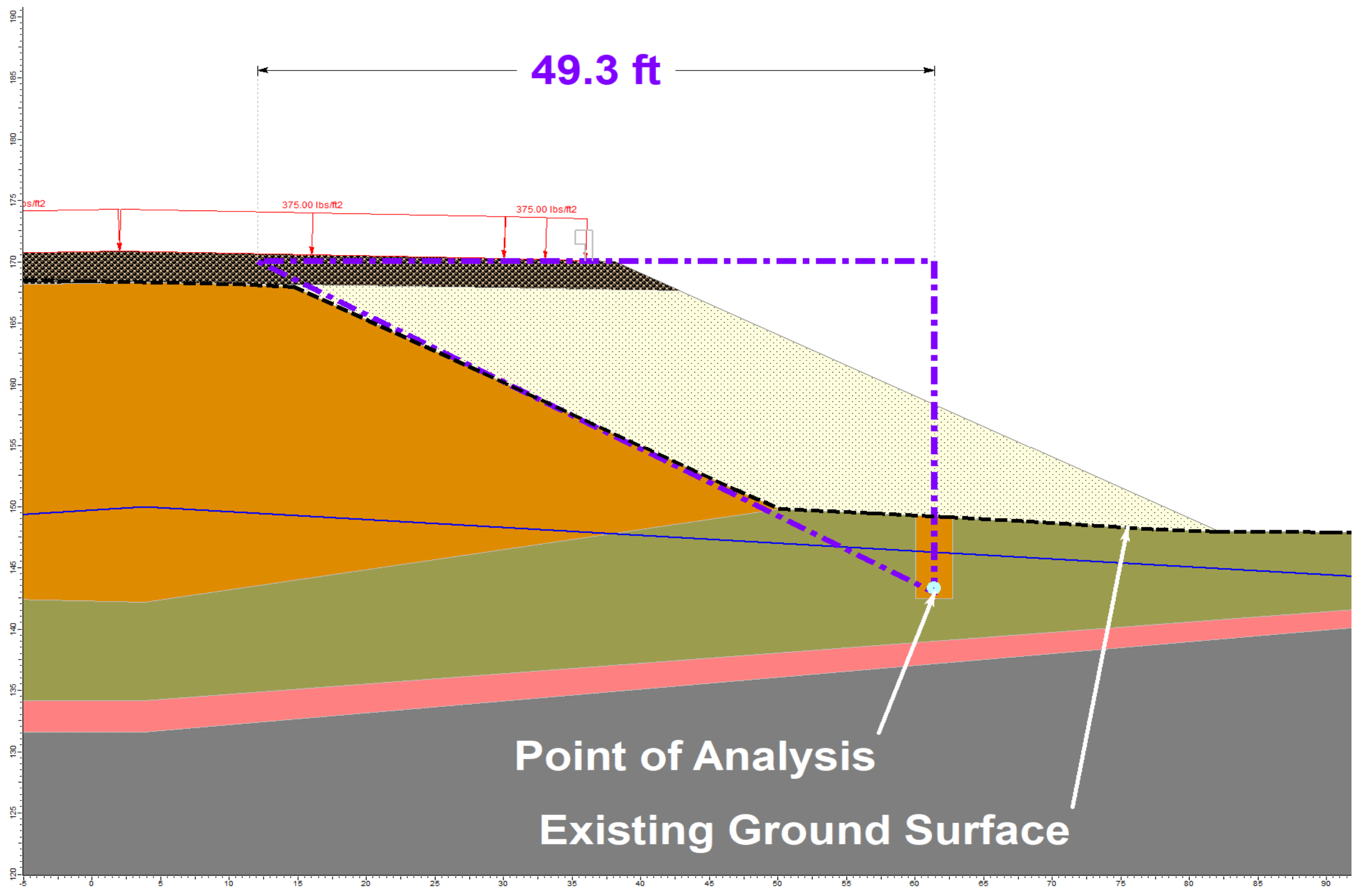
Layer	Unit weight (pcf)	Elevation (ft)		Thickness (ft)	Stress Contribution of Layer (psf)	Effective Stress Contribution of Layer (psf)
		Top	Bottom			
New Fill	125	158.4	149.2	9.2	1150.0	1150.0
Compacted Native Backfill	130	149.2	144.7	4.5	585.0	485.2
Compacted Select Screen Sand	125	144.7	143.7	1.0	125.0	62.6
Pipe (assume full of water and neglect wall thickness)	62.4	143.7	143	0.7	43.7	43.7

Calculate the increase or decrease in effective stress as a result of construction at the ground surface.

	σ_v at Elev. 143.0 ft (psf)	σ'_v at Elev. 143.0 ft (psf)	$\Delta\sigma'_v$ at Elev. 143.0 ft (psf)	Result
Existing conditions	754	591	1150	Settlement
After construction	1904	1741		

Subdivide the subsurface soils into layers no larger than 10 ft thick and to a depth of either twice the footing width or to bedrock. Calculate the vertical stress increase at each layer, assuming the Boussinesq stress distribution method for stress under the corner of a triangular load. The Boussinesq method was developed for footings but in this case is used for the proposed embankment fill.

Assumed Fill Loading (section view):



$$\sigma_z = q_0 \times I$$ Reference 5, Eqn. 8-30

$$m = L/z$$ Reference 5, Figure 8.24

$$n = B/z$$ Reference 5, Figure 8.24

where:

σ_z = vertical stress increase, psf			
q_0 = net stress applied by footing	1150	psf	(Part A)
L = length of the footing, ft	∞	ft	(Ref. 4)
B = width of the footing, ft	49.3	ft	(Ref. 4)
z = depth to midpoint of layer, ft			

Note: This Boussinesq geometry may overestimate the load applied at the top of the pipe from the left; however, the rightmost triangle of fill is not added to account for this overestimation of load.

As per References 7 and 8, the existing trench construction below the pipe is assumed to consist of 6 inches of sand bedding material.

Layer		Depth below bottom of pipe (ft)	Layer Thickness (ft)	z (ft)	m	n	I	Stress Increase σ_z (psf)
Trench Sand	1	0-0.5	0.5	0.3	∞	197.2	0.250	288
Glaciomarine	2	0.5-3.9	3.4	2.2	∞	22.4	0.250	288
Native Sand/Gravel	3	3.9-5.8	1.9	4.9	∞	10.2	0.234	269

B. Use consolidation theory to estimate settlement of the glaciomarine silty clay layer beneath the water main (Layer 2); use the Hough method to estimate settlement of the trench sand and native sand/gravel layers beneath the water main (Layers 1 and 3).

1 Determine σ'_{v0} and C' at layer midpoints for in situ existing soil. Use Ref. 6 Figures 5-18 and 5-19 to obtain corrected N' and C' for Layers 1 and 3, assuming the trench sand is "clean well graded fine to coarse SAND" and the native sand/gravel is "well graded silty SAND & GRAVEL".

Stress due to existing soil above pipe (Part A)	σ'_{v0} (psf)
	591

Water unit weight (pcf)
62.4

Layer		Layer Thickness (ft)	Unit Weight of Layer (pcf)	σ'_{v0} at layer midpoint (psf)	σ'_{v0} at layer midpoint (kPa)	N_{60}	N'	C'
Trench Sand	1	0.5	125	607	29.1	16	29	87
Glaciomarine	2	3.4	125	729	34.9	Not required for clay consolidation		
Native Sand/Gravel	3	1.9	125	895	42.8	13	19	75

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Subject:	Settlement of Water Main at NW Embankment - Southern Shift	**Reviewed by:**	MCM
Project Short Title:	MaineDOT I-295 Exit 20 Merrill Road Bridge Replacement No. 5720		

2 Calculate the total settlement of the trench sand and the native sand/gravel (Layers 1 and 3) using the Hough method:

General Equation (Ref. 6, Eqn 5-24)

$$\Delta H_i = H_c \frac{1}{C'} \log\left(\frac{\sigma'_{v0} + \Delta\sigma_v}{\sigma'_{v0}}\right)$$

where:

ΔH_i settlement in each layer, ft

H_c initial height of layer i, ft

C' bearing capacity index from Ref. 6, Figure 5-19

$\Delta\sigma_v$ vertical stress increase, ksf

ΔH_i (Layer 1)	ft	0.001
	in	0.01

ΔH_i (Layer 3)	ft	0.003
	in	0.03

3 Calculate the total settlement of the glaciomarine silty clay (Layer 2) using the consolidation theory:

General Equation (Ref. 5, Eqn 8-11, 8-16, 8-18b)

$$\Delta H_i = C_c \frac{H_0}{1 + e_0} \log\frac{\sigma'_{v0} + \Delta\sigma_v}{\sigma'_{v0}}$$

for normally consolidated clay

$$\Delta H_i = C_r \frac{H_0}{1 + e_0} \log\frac{\sigma'_{v0} + \Delta\sigma_v}{\sigma'_{v0}}$$

for overconsolidated clay, $\sigma'_{v0} + \Delta\sigma_v \leq \sigma'_p$

$$\Delta H_i = C_r \frac{H_0}{1 + e_0} \log\frac{\sigma'_p}{\sigma'_{v0}} + C_c \frac{H_0}{1 + e_0} \log\frac{\sigma'_{v0} + \Delta\sigma_v}{\sigma'_p}$$

for overconsolidated clay, $\sigma'_{v0} + \Delta\sigma_v > \sigma'_p$

Date:	7/30/2020	**Made by:**	KAR
Project No.:	19126013	**Checked by:**	MEL
Subject:	Settlement of Water Main at NW Embankment - Southern Shift	**Reviewed by:**	MCM
Project Short Title:	MaineDOT I-295 Exit 20 Merrill Road Bridge Replacement No. 5720		

where:			Layer 2	
H_0	initial height of layer i, ft		3.4	
$\Delta\sigma_v$	surcharge load, psf		288	
σ'_{v0}	in situ vertical effective stress, psf		729	
$\sigma'_{v0} + \Delta\sigma_v$			1017	
σ'_p	preconsolidated stress, psf		N/A	
C_c	compression index, $C_c = C_{c\varepsilon}(1+e_0)$		0.42	
C_r	recompression index, $C_r = C_{r\varepsilon}(1+e_0)$		0.03	
e_0	initial void ratio		0.68	
		Use equation:	8-16	(Asmpt. 7)
ΔH_i		ft	0.010	
		in	0.12	

Layer	Settlement Based on Calc. Loading Stress ΔH_i (in)
1	0.01
2	0.12
3	0.03
Total Settlement (in)	0.16

C. Determine the time rate of settlement that will occur at the embankment location.

Use a single layer analysis to determine the time for the entire settlement to occur, assuming double drainage conditions based on the soil types above and below the glaciomarine silty clay layer having a higher permeability relative to the clay layer (Reference 3). Subsurface conditions are simplified to a single clay layer with uniform soil properties for the purpose of the calculation.

$$T_v = \frac{c_V t}{H^2{}_{dr}}$$ (Ref. 5, Eqn 9-5)

$$T_v = \frac{\pi}{4}\left(\frac{U\%}{100}\right)^2$$ For U = 0% to 60% (Ref. 5, Eqn 9-10)

$$T_v = 1.781 - 0.933\log(100 - U\%)$$ For U ≥ 60% (Ref. 5, Eqn 9-11)

$$U_{avg} = \frac{s(t)}{S_c}$$ (Ref. 5, Eqn 9-12)

where:
- T_v = time factor (dimensionless)
- c_v = coefficient of consolidation (ft^2/day)
- t = time (day)
- H_{dr} = drainage path (ft)

U_{avg} = average degree or percent consolidation
$s(t)$ = settlement at any time (in)
s_c = final or ultimate settlement (in)

Based on Reference 1, the clay layer thickness below the footing is: 3.4 ft
With double drainage conditions, H_{dr} = 1.7 ft
The coefficient of consolidation value is assumed based on local engineering experience.
c_v = 120 ft^2/yr = 0.33 ft^2/day

Determine time factor using Reference 5, Eqn 9-11

U_{avg} = 95 (Calculate time to reach 95% consolidation)
T_v = 1.13

Determine time required for settlement to occur using Reference 5, Eqn 9-5

c_v = 0.33 ft^2/day
H_{dr} = 1.7 ft
T_v = 1.13

t = 10 days = 0.03 years (95% consolidation)

s_c (in) = 0.12 (final potential settlement of the clay layer, Part B)

Time (days)	Time (years)	T_v	U_{avg} (%)	s(t) (in)
0	0.00	0.00	0.00	0.00
1	0.00	0.11	38.06	0.05
2	0.01	0.23	53.82	0.06
3	0.01	0.34	65.08	0.08
4	0.01	0.46	73.63	0.09
5	0.01	0.57	80.08	0.10
6	0.02	0.68	84.96	0.10
7	0.02	0.80	88.64	0.11
8	0.02	0.91	91.42	0.11
9	0.02	1.02	93.52	0.11
10	0.03	1.14	95.11	0.11

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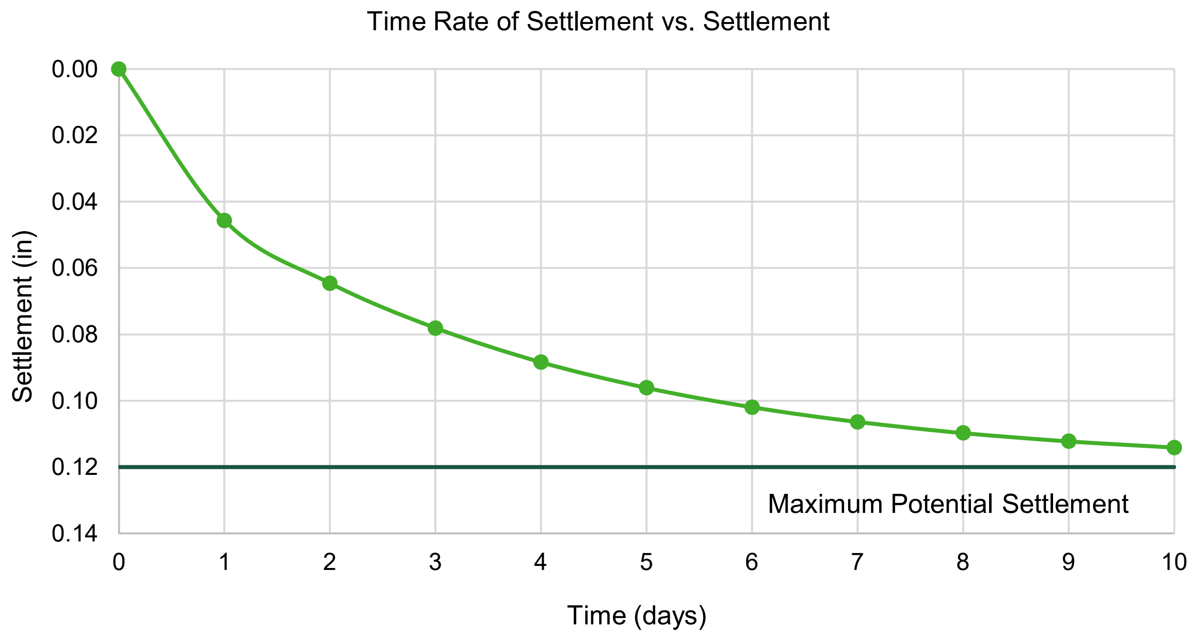
Subject:	Settlement of Water Main at NW Embankment - Southern Shift	**Reviewed by:**	MCM
Project Short Title:	MaineDOT I-295 Exit 20 Merrill Road Bridge Replacement No. 5720		



CONCLUSIONS

The primary settlement beneath the existing water main after construction of the northwestern proposed bridge approach embankment (assuming the "southern shift" option) is estimated to be 0.16 inches. This includes 0.04 inches of immediate settlement and 0.12 inches of consolidation settlement that is estimated to reach 95% consolidation in 10 days.

GOLDER

CALCULATIONS

Date:	7/30/2020	**Made by:**	KAR
Project No.:	19126013	**Checked by:**	MEL
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Project Short Title:	MaineDOT I-295 Exit 20 Merrill Road Bridge Replacement No. 5720		

OBJECTIVE

Calculate the primary settlement beneath the existing water main after construction of the southeastern proposed bridge approach embankment (assuming the "southern shift" option) at a location where embankment fill is thickest (southern shift alignment at HNTB Station 62+50).

REFERENCES

1. Golder calculation titled "Global Stability Analysis SE Embankment", dated July 8, 2020 (Appendix E, Preliminary Geotechnical Design Report, dated September 2020).
2. Golder boring location plan (Figure 2, Preliminary Geotechnical Design Report, dated September 2020).
3. Golder geotechnical test boring logs (Appendix A, Preliminary Geotechnical Design Report, dated September 2020).
4. HNTB for State of Maine Department of Transportation. Merrill Road Bridge Freeport Interstate 295: Desert Road South Cross Sections, dated November 2019.
5. Holtz, R.D. and Kovacs, W.D. 1981. An Introduction to Geotechnical Engineering, 1st ed. Prentice Hall, Englewood Cliffs, NJ.
6. FHWA. 2002. Geotechnical Engineering Circular No. 6: Shallow Foundations. Report No. FHWA-SA-02-054.
7. Maine Water Company. Water Main Installation Instructions, dated January 9, 2017.
8. Maine Water Company. Water Main Trench Detail. Drawing Number MWC SD-4, dated March 9, 2016.
9. Das, Braja M. 2011. Principles of Foundation Engineering, 7th Edition. Cengage Learning.
10. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.

ASSUMPTIONS

1. The existing stratigraphy and water table surface are estimated from samples and groundwater measurements encountered during drilling and interpreted in Reference 1.	GS elev. =	146.6	ft
	WL elev. =	146.2	ft
2. N_{60}-value for the trench bottom bedding material, based on correlation (Ref. 9, Eqn. 2.26) to the MaineDOT specified friction angle for granular borrow (Ref. 10, Table 3-3), is:	$N_{60,Trench}$ =	16	
3. N_{60}-value for the native sand/gravel layer, based on the average of the N_{60}-values encountered in all borings for the layer (Reference 3) is:	$N_{60,S/G}$ =	13	
4. The proposed top of slope and base of embankment elevations at the location of analysis (Reference 2) are (Reference 4, Section 62+50):	El. top =	158.4	ft
	El. base =	146.6	ft
5. Based on the DigSafe tickets issued for the borings, the utility owner of the water main is Maine Water Company. Minimum pipe diameter, depth of cover, and trench construction details for the water main were assumed based on values provided in Maine Water Company standard specifications (Ref. 7 and 8).	Min. pipe diameter =	8	in
	Depth of cover =	5.5	ft
	Screen above pipe =	12	in
	Trench below pipe =	6	in

CALCULATION

A. Determine the change in effective stress state within the soil beneath the water main to identify if settlement or heave will occur. Calculate the vertical stress increase beneath the water main due to embankment construction.

https://golderassociates.sharepoint.com/sites/111584/Project Files/5 Technical Work/600 Analysis/Settlement/WaterMain settlement Southern Shift 19126013 Freeport Exit 20 v2.xlsx

The change in effective stress state due to change in stratigraphy is determined at the assumed bottom-of-pipe elevation 140.4 ft (existing ground surface minus depth of cover minus pipe diameter). As per References 7 and 8, the existing trench construction above the pipe is assumed to consist of 1 foot of compacted select screen sand overlain by compacted native backfill to finished grade, for a total cover thickness of 5.5 feet.

Existing Conditions (References 7 and 8):

Layer	Unit weight (pcf)	Elevation (ft)		Thickness (ft)	Stress Contribution of Layer (psf)	Effective Stress Contribution of Layer (psf)
		Top	Bottom			
Compacted Native Backfill	130	146.6	142.1	4.5	585.0	329.2
Compacted Select Screen Sand	125	142.1	141.1	1.0	125.0	62.6
Pipe (assume full of water and neglect wall thickness)	62.4	141.1	140.4	0.7	43.7	43.7

After Construction (Reference 4, Section 62+50):

Layer	Unit weight (pcf)	Elevation (ft)		Thickness (ft)	Stress Contribution of Layer (psf)	Effective Stress Contribution of Layer (psf)
		Top	Bottom			
New Fill	125	158.4	146.6	11.8	1475.0	1475.0
Compacted Native Backfill	130	146.6	142.1	4.5	585.0	329.2
Compacted Select Screen Sand	125	142.1	141.1	1.0	125.0	62.6
Pipe (assume full of water and neglect wall thickness)	62.4	141.1	140.4	0.7	43.7	43.7

Calculate the increase or decrease in effective stress as a result of construction at the ground surface.

	σ_v at Elev. 140.4 ft (psf)	σ'_v at Elev. 140.4 ft (psf)	$\Delta\sigma'_v$ at Elev. 140.4 ft (psf)	Result
Existing conditions	754	435	1475	Settlement
After construction	2229	1910		

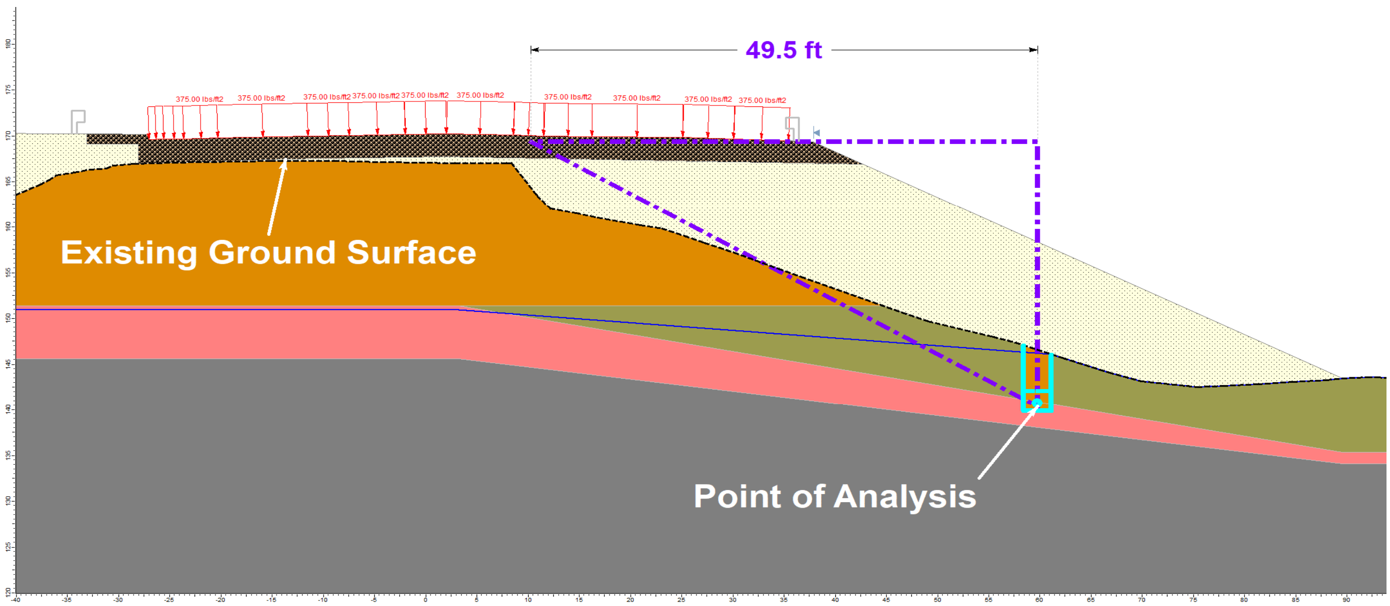
Subdivide the subsurface soils into layers no larger than 10 ft thick and to a depth of either twice the footing width or to bedrock. Calculate the vertical stress increase at each layer, assuming the Boussinesq stress distribution method for stress under the corner of a triangular load. The Boussinesq method was developed for footings but in this case is used for the proposed embankment fill.

GOLDER

CALCULATIONS

Date:	7/30/2020	**Made by:**	KAR
Project No.:	19126013	**Checked by:**	MEL
Subject:	Settlement of Water Main at SE Embankment - Southern Shift	**Reviewed by:**	MCM
Project Short Title:	MaineDOT I-295 Exit 20 Merrill Road Bridge Replacement No. 5720		

Assumed Fill Loading (section view):



$$\sigma_z = q_0 \times I \qquad \text{Reference 5, Eqn. 8-30}$$
$$m = L/z \qquad \text{Reference 5, Figure 8.24}$$
$$n = B/z \qquad \text{Reference 5, Figure 8.24}$$

where:

σ_z = vertical stress increase, psf			
q_0 = net stress applied by footing	1475	psf	(Part A)
L = length of the footing, ft	∞	ft	(Ref. 4)
B = width of the footing, ft	49.5	ft	(Ref. 4)
z = depth to midpoint of layer, ft			

Note: This Boussinesq geometry may overestimate the load applied at the top of the pipe from the left; however, the rightmost triangle of fill is not added to account for this overestimation of load.

As per References 7 and 8, the existing trench construction below the pipe is assumed to consist of 6 inches of sand bedding material.

Layer		Depth below bottom of pipe (ft)	Layer Thickness (ft)	z (ft)	m	n	I	Stress Increase σ_z (psf)
Trench Sand	1	0-0.5	0.5	0.3	∞	198.0	0.250	369
Native Sand/Gravel	2	0.5-2.3	1.8	1.4	∞	35.4	0.250	369

Date:	7/30/2020	**Made by:**	KAR
Project No.:	19126013	**Checked by:**	MEL
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Project Short Title:	MaineDOT I-295 Exit 20 Merrill Road Bridge Replacement No. 5720		

B. Use the Hough method to estimate settlement of the trench sand and native sand/gravel layers beneath the water main.

1 Determine σ'_{v0} and C' at layer midpoints for in situ existing soil. Use Ref. 6 Figures 5-18 and 5-19 to obtain corrected N' and C' for Layers 1 and 2, assuming the trench sand is "clean well graded fine to coarse SAND" and the native sand/gravel is "well graded silty SAND & GRAVEL".

Stress due to existing soil above pipe (Part A)	σ'_{v0} (psf)
	435

Water unit weight (pcf)
62.4

Layer		Layer Thickness (ft)	Unit Weight of Layer (pcf)	σ'_{v0} at layer midpoint (psf)	σ'_{v0} at layer midpoint (kPa)	N_{60}	N'	C'
Trench Sand	1	0.5	125	451	21.6	16	32	95
Native Sand/Gravel	2	1.8	125	523	25.0	13	25	90

2 Calculate the total settlement of the trench sand and the native sand/gravel (Layers 1 and 2) using the Hough method:

General Equation (Ref. 6, Eqn 5-24)

$$\Delta H_i = H_c \frac{1}{C'} \log\left(\frac{\sigma'_{v0} + \Delta\sigma_v}{\sigma'_{v0}}\right)$$

where:

ΔH_i	settlement in each layer, ft
H_c	initial height of layer i, ft
C'	bearing capacity index from Ref. 6, Figure 5-19
$\Delta\sigma_v$	vertical stress increase, ksf

ΔH_i (Layer 1)	ft	0.001
	in	0.02

ΔH_i (Layer 2)	ft	0.005
	in	0.06

	Settlement Based on Calc. Loading Stress
Layer	ΔH_i (in)
1	0.02
2	0.06
Total Settlement (in)	0.08

GOLDER

CALCULATIONS

Date:	7/30/2020	**Made by:**	KAR
Project No.:	19126013	**Checked by:**	MEL
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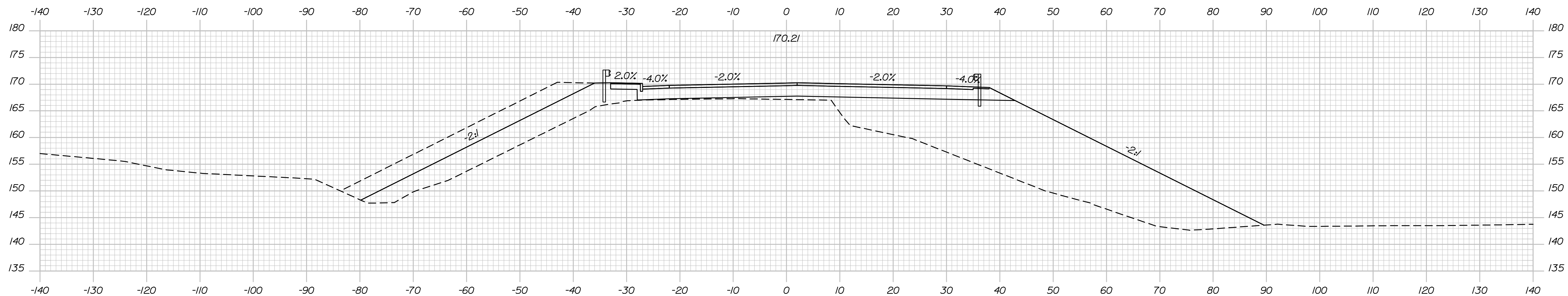
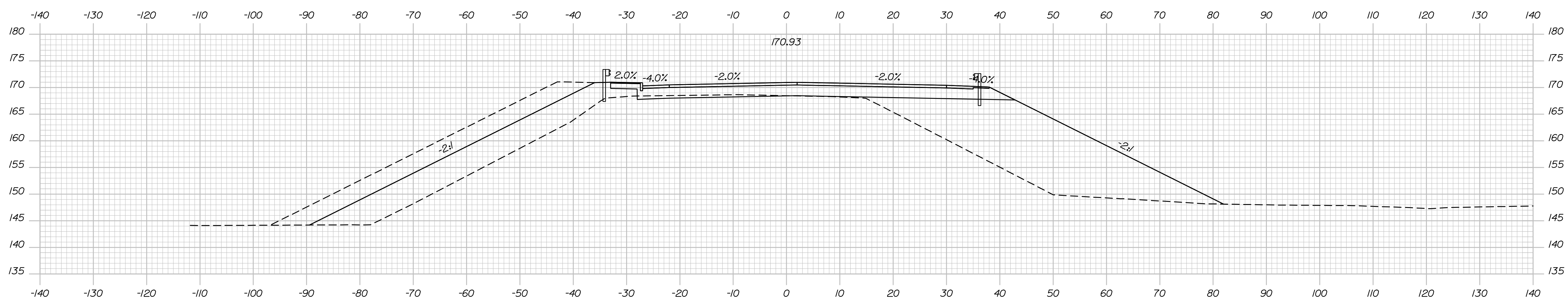
CONCLUSIONS

The primary settlement beneath the existing water main after construction of the southeastern proposed bridge approach embankment (assuming the "southern shift" option) is estimated to be 0.08 inches of immediate settlement.

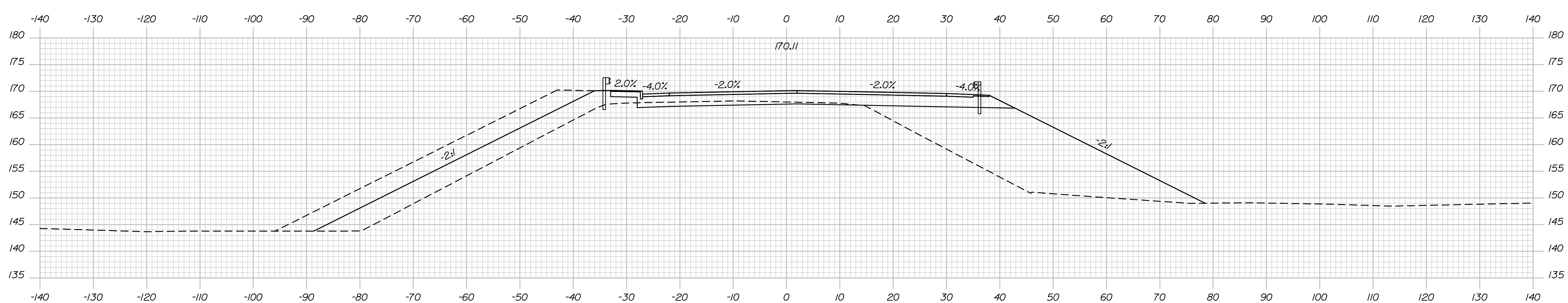
Reference 4



62+50.00



60+25.00



60+00.00

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

HIGHWAY PLANS

WIN
023627.00

Bridge No. 5720

	BY	DATE	SIGNATURE	P.E. NUMBER	DATE
PROJ. MANAGER					
DESIGN-DETAILED		11\19			
CHECKED-REVIEWED		11\19			
DESIGN2-DETAILED2					
DESIGN3-DETAILED3					
REVISIONS 1					
REVISIONS 2					
REVISIONS 3					
REVISIONS 4					
FIELD CHANGES					

MERRILL ROAD BRIDGE
INTERSTATE 295

FREEPORT CUMBERLAND

DESERT ROAD SOUTH
CROSS SECTIONS

SHEET NUMBER

Wor

OF 4

Sta. 60+00.00 to Sta. 62+50.00

Date:6/22/2020

Username:

Division:

Filename: Working Sections.dgn

Reference 5

vertical stress is

$$\sigma_z = \frac{2P}{\pi}\frac{z^3}{x^4} \tag{8-26}$$

where P = line load, and
$x = (z^2 + r^2)^{1/2}$ (see Fig. 8.20a).
Equations for the horizontal and shear stress are also available.

The next logical step is to integrate a line load over a finite area. Newmark (1935) performed the integration of Eq. 8.26 and derived the following equation for the vertical stress under the corner of a *uniformly loaded rectangular area*:

$$\sigma_z = q_o \frac{1}{4\pi}\left[\frac{2mn(m^2+n^2+1)^{1/2}}{m^2+n^2+1+m^2n^2} \times \frac{(m^2+n^2+2)}{(m^2+n^2+1)} + \arctan\frac{2mn(m^2+n^2+1)^{1/2}}{m^2+n^2+1-m^2n^2}\right] \tag{8-27}$$

where q_o = surface or contact stress,
$m = x/z$, (8-28)
$n = y/z$, and (8-29)
x, y = length and width of the uniformly loaded area, respectively.
The parameters m and n are interchangeable. Fortunately Eq. 8-27 may be rewritten as

$$\sigma_z = q_o I \tag{8-30}$$

where I = an influence value which depends on m and n.

Values of I for various values of m and n are shown in Fig. 8.21.

EXAMPLE 8.18

Given:

The 3 × 4 m rectangular footing of Example 8.17 is loaded uniformly by 117 kPa.

Required:

a. Find the vertical stress under the corner of the footing at a depth of 2 m.

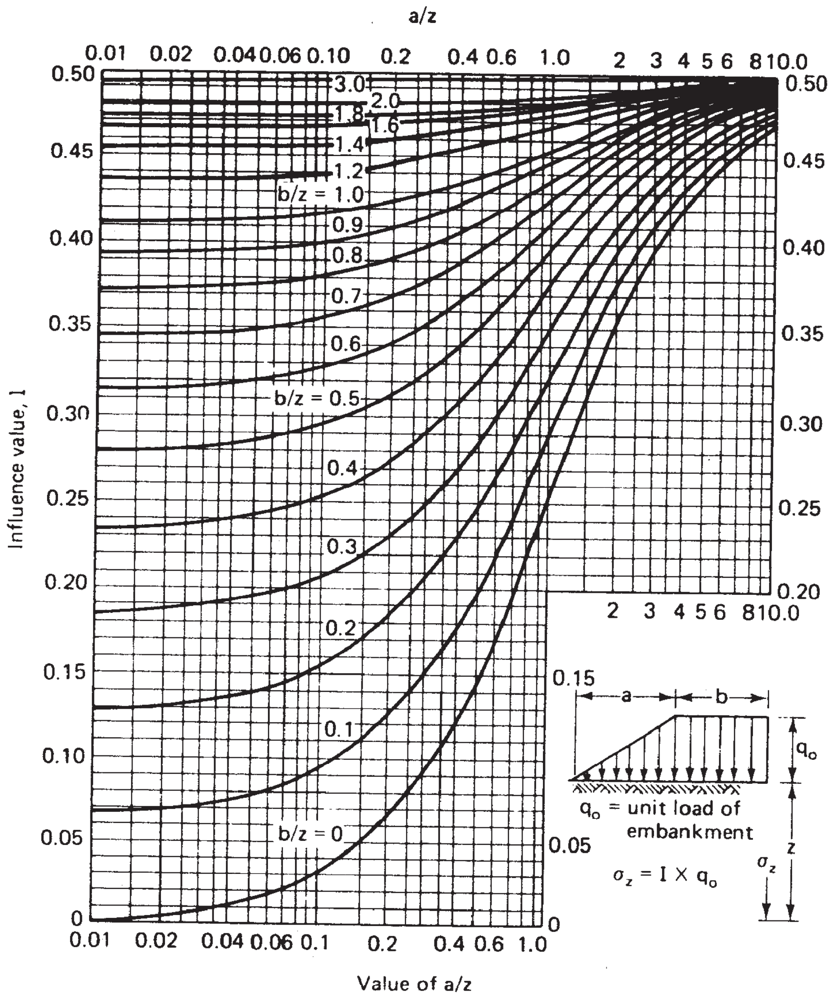


Fig. 8.23 Influence values for vertical stress under a very long embankment; length = ∞ (from U.S. Navy, 1971, after Osterberg, 1957).

A second graphical way is to find Δe over *one* cycle; for example, $\log\frac{1000}{100} = \log 10 = 1$. When this is done, $C_c = \Delta e$. In Fig. Ex. 8.9 the vertical scale is not sufficient for $\Delta\sigma' = 1$ log cycle, but it can be done in two steps, e_a to e_b and e_c to e_d. (To extend the line $\overline{e_a e_b}$ to one full log cycle on the *same* graph, choose e_c at the same pressure as e_b. Then draw the line $\overline{e_c e_d}$ parallel to $\overline{e_a e_b}$. This second line is merely the extension of $\overline{e_a e_b}$ if the graph paper extended lower than shown.) Or,

$$\begin{aligned}\Delta e = C_c &= (e_a - e_b) + (e_c - e_d)\\ &= (0.870 - 0.655) + (0.90 - 0.664)\\ &= 0.215 + 0.236\\ &= 0.451\text{, or same as above}\end{aligned}$$

c. The modified compression index $C_{c\epsilon}$ is

$$C_{c\epsilon} = \frac{C_c}{1 + e_o} = \frac{0.451}{1 + 0.865} = 0.242$$

To calculate consolidation settlement, Eqs. 8-5, 8-6, or 8-7 and 8-8 may be combined with Eq. 8-4. For example, using Eqs. 8-7 and 8-4 we obtain

$$s_c = C_c\frac{H_o}{1 + e_o}\log\frac{\sigma_2'}{\sigma_1'} \tag{8-10}$$

If the soil is normally consolidated, then σ_1' would be equal to the existing vertical overburden stress σ_{vo}', and σ_2' would include the additional stress $\Delta\sigma_v$ applied by the structure, or

$$s_c = C_c\frac{H_o}{1 + e_o}\log\frac{\sigma_{vo}' + \Delta\sigma_v}{\sigma_{vo}'} \tag{8-11}$$

When computing the settlement by means of the percent consolidation versus log effective stress curve, Eq. 8-8 is combined with Eq. 8-4 to get

$$s_c = C_{c\epsilon}H_o\log\frac{\sigma_2'}{\sigma_1'} \tag{8-12}$$

or, analogous to Eq. 8-11, for normally consolidated clays,

$$s_c = C_{c\epsilon}H_o\log\frac{\sigma_{vo}' + \Delta\sigma_v}{\sigma_{vo}'} \tag{8-13}$$

Other similar settlement equations can be derived using a_v and m_v. In this case the average stress for a given stress increment must be used since the compression curves are nonlinear.

Required:

Calculate (a) the recompression index C_r and (b) the modified recompression index $C_{r\epsilon}$.

Solution:

a. The recompression index C_r is found in a similar manner to the C_c (Eq. 8-7). Using the points e and f over 1 log cycle, we find that

$$C_r = e_e - e_f = 0.790 - 0.760 = 0.030$$

b. The modified recompression index $C_{r\epsilon}$ is found from Eq. 8-15.

$$C_{r\epsilon} = \frac{C_r}{1 + e_o} = \frac{0.030}{1 + 0.865} = 0.016$$

Note that neither of these terms has units.

To calculate settlements of overconsolidated clays, Eqs. 8-11 and 8-13 become

$$s_c = C_r \frac{H_o}{1 + e_o} \log \frac{\sigma'_{vo} + \Delta\sigma_v}{\sigma'_{vo}} \tag{8-16}$$

$$s_c = C_{r\epsilon} H_o \log \frac{\sigma'_{vo} + \Delta\sigma_v}{\sigma'_{vo}} \tag{8-17}$$

when $\sigma'_{vo} + \Delta\sigma_v \leq \sigma'_p$. Since C_r is usually much less than C_c, the settlements occurring when $\sigma'_{vo} + \Delta\sigma_v \leq \sigma'_p$ are much less than if the soil were normally consolidated.

If the added stress caused by the structure exceeds the preconsolidation stress, then much larger settlements would be expected. This is because the compressibility of the soil is much greater on the virgin compression curve than on the recompression curve as was shown, for example, in Fig. 8.7. For the case, then, where $\sigma'_{vo} + \Delta\sigma_v > \sigma'_p$ the settlement equation consists of two parts: (1) the change in void ratio or strain on the recompression curve from the original in situ conditions of (e_o, σ'_{vo}) or $(\epsilon_{vo}, \sigma'_{vo})$ to σ'_p; and (2) the change in void ratio or strain on the virgin compression curve from σ'_p to the final conditions of (e_f, σ'_{vf}) or $(\epsilon_{vf}, \sigma'_{vf})$. Note that $\sigma'_{vf} = \sigma'_{vo} + \Delta\sigma_v$. These two parts are shown graphically in Fig. 8.10b. The complete settlement equation then becomes

$$s_c = C_r \frac{H_o}{1 + e_o} \log \frac{\sigma'_{vo} + (\sigma'_p - \sigma'_{vo})}{\sigma'_{vo}} + C_c \frac{H_o}{1 + e_o} \log \frac{\sigma'_p + (\sigma'_{vo} + \Delta\sigma_v - \sigma'_p)}{\sigma'_p} \tag{8-18a}$$

This equation reduces to

$$s_c = C_r \frac{H_o}{1 + e_o} \log \frac{\sigma'_p}{\sigma'_{vo}} + C_c \frac{H_o}{1 + e_o} \log \frac{\sigma'_{vo} + \Delta\sigma_v}{\sigma'_p} \tag{8-18b}$$

In terms of the modified indices, we have

$$s_c = C_{r\epsilon} H_o \log \frac{\sigma'_p}{\sigma'_{vo}} + C_{c\epsilon} H_o \log \frac{\sigma'_{vo} + \Delta\sigma_v}{\sigma'_p} \tag{8-19}$$

Both Eqs. 8-18 and 8-19 give the same results. One could argue that in the right-hand term of Eq. 8-18 the void ratio corresponding to the preconsolidation pressure on the true virgin compression curve should be used. Although this is technically correct, it doesn't make any significant difference in the answer.

Sometimes the degree of overconsolidation varies throughout the compressible layer. You could apply Eq. 8-16 or 8-17 to the part where $\sigma'_{vo} + \Delta\sigma_v < \sigma'_p$ and Eq. 8-18 or 8-19 to the part where $\sigma'_{vo} + \Delta\sigma_v > \sigma'_p$. In practice, however, it is usually easier to simply divide the entire stratum into several layers, apply the appropriate equation to calculate the average settlement for each layer, and then sum up the settlements by Eq. 8-14.

What is the best way to get C_r and $C_{r\epsilon}$ for use in Eqs. 8-16 through 8-19? Because of sample disturbance, the slope of the initial recompression portion of the laboratory consolidation curve (Fig. 8.7) is too steep and would yield values that are too large for these indices. Leonards (1976) offers the reasons why in situ values are generally smaller than those obtained from laboratory measurements: (1) disturbance during sampling, storage, and preparation of test specimens; (2) recompression of gas bubbles in the voids; and (3) errors in test procedures and methods of interpreting test results. This latter item includes the problem of reproducing the in situ state of stress in the specimen. Leonards recommends that the σ'_{vo} be applied to the specimen and that it be innundated and allowed to come to equilibrium for at least 24 hours before starting the incremental loading. Any tendency to swell should be controlled. Then the consolidation test is continued with relatively large load increments. To reproduce as closely as possible the in situ stress state, Leonards recommends that the sample be consolidated to slightly less than the σ'_p and then be allowed to rebound. This is the first cycle shown in Fig. 8.11. If you don't have a good idea of the σ'_p, then consolidate initially to $\sigma'_{vo} + \Delta\sigma_v$ only, which is presumably less than σ'_p. The determination of C_r or $C_{r\epsilon}$ is over the range of $\sigma'_{vo} + \Delta\sigma_v$, as shown in Fig. 8.11. It is common practice to take the average slope of the two curves. From the typical test results shown in Fig. 8.11, you can see that the actual values of the recompression index depend on the stress at which the rebound-reload cycle starts, especially whether it starts at a stress less than or greater than the σ'_p. See the difference in

Equation 9-2 is the *Terzaghi one-dimensional consolidation equation*. It could just as easily be written in three dimensions, but most of the time in engineering practice one-dimensional consolidation is assumed. Basically, the equation is a form of the diffusion equation from mathematical physics. Many physical diffusion phenomena are described by this equation, for example, heat flow in a solid body. The "diffusion constant" for the soil is the c_v. Note that we called the c_v a constant. It really isn't, but we must assume it is, that is, that k, a_v, and e_0 are constants, in order to make the equation linear and easily solvable.

So how do we solve the Terzaghi consolidation equation? Just like we solve all other second-order partial differential equations with constant coefficients. There are a variety of ways; some are mathematically exact; others are only approximate. For example, Harr (1966) presents an approximate solution by using the method of finite differences. Taylor (1948), following Terzaghi (1925), gives a mathematically rigorous solution in terms of a Fourier series expansion, and this is what we do in detail in Appendix B-2. Here we shall just give an outline of the solution. First, the boundary and initial conditions for the case of one-dimensional consolidation are:

1. There is complete drainage at the top and bottom of the compressible layer.
2. The initial *excess* hydrostatic pressure $\Delta u = u_i$ is equal to the applied increment of stress at the boundary, $\Delta\sigma$.

We can write these boundary and initial conditions as follows:

$$\text{When } z = 0 \text{ and when}$$
$$z = 2H, u = 0$$
$$\text{When } t = 0, \Delta u = u_i = \Delta\sigma = (\sigma_2' - \sigma_1')$$

We usually take the thickness of the consolidating layer to be $2H$, so that the *length of the longest drainage path* is equal to H or H_{dr}. Of course at $t = \infty$, $\Delta u = 0$, or complete dissipation of the pore pressure will have occurred.

Terzaghi (1925) was obviously familiar with the early work on heat transfer, and he adapted those closed-form solutions to the consolidation problem. The solution comes out in terms of a Fourier series expansion of the form

$$u = (\sigma_2' - \sigma_1') \sum_{n=0}^{\infty} f_1(Z) f_2(T) \tag{9-4}$$

where Z and T are dimensionless parameters (see also Taylor, 1948). The first term, Z, is a geometry parameter, and it is equal to z/H. The second term, T, is known as the *time factor*, and it is related to the coefficient of consolidation c_v by

$$T = c_v \frac{t}{H_{dr}^2} \tag{9-5}$$

where t = time, and
H_{dr} = length of the longest drainage path.

We have already mentioned that c_v has dimensions of L^2T^{-1} or units of m^2/s (or equivalent).

From Eq. 9-3, the time factor can also be written as

$$T = \frac{k(1 + e_o)}{a_v \rho_w g} \frac{t}{H_{dr}^2} \tag{9-6}$$

Note that t has the same time units as k. That is, if k is in centimetres per second, then t must be in seconds. The drainage path for double drainage would be equal to half the thickness H of the clay layer, or $2H/2 = H_{dr}$. If we had only a singly drained layer, the drainage path would still be H_{dr}, but then it would be equal to the thickness H of the layer.

The progress of consolidation after some time t and at any depth z in the consolidating layer can be related to the void ratio at that time and the final change in void ratio. This relationship is called the *consolidation ratio*, and it is expressed as

$$U_z = \frac{e_1 - e}{e_1 - e_2} \tag{9-7}$$

where e is some intermediate void ratio, as shown on Fig. 9.2. What we are looking at graphically in that figure is the ratio of ordinates corresponding to AB and AC. In terms of stresses and pore pressures, Eq. 9-7 becomes

$$U_z = \frac{\sigma' - \sigma_1'}{\sigma_2' - \sigma_1'} = \frac{\sigma' - \sigma_1'}{\Delta\sigma'} = \frac{u_i - u}{u_i} = 1 - \frac{u}{u_i} \tag{9-8}$$

where σ' and u are intermediate values corresponding to e in Eq. 9-7, and u_i is the initial *excess* pore pressure induced by the applied stress $\Delta\sigma'$. You should satisfy yourself that these equations are correct from the relationships shown in Fig. 9.2 and from $\Delta\sigma' = -\Delta u$. (See also Appendix B-2.)

From Eqs. 9-7 and 9-8, it is evident that U_z is zero at the start of loading, and it gradually increases to 1 (or 100%) as the void ratio decreases from e_1 to e_2. At the same time, of course, as long as the total stress remains constant, the effective stress increases from σ_1' to σ_2' as the excess hydrostatic stress (pore water pressure) dissipates from u_i to zero. The consolidation ratio U_z is sometimes called the *degree or percent consolidation*, and it represents conditions *at a point* in the consolidating layer. It is now possible to put our solution for u in Eq. 9-4 in terms of the

TABLE 9-1

U_{avg}	T
0.1	0.008
0.2	0.031
0.3	0.071
0.4	0.126
0.5	0.197
0.6	0.287
0.7	0.403
0.8	0.567
0.9	0.848
0.95	1.163
1.0	∞

where the initial pore pressure distribution is sinusoidal, half sine, and triangular are presented by Leonards (1962).

Casagrande (1938) and Taylor (1948) provide the following useful approximations:

For $U < 60\%$,

$$T = \frac{\pi}{4} U^2 = \frac{\pi}{4}\left(\frac{U\%}{100}\right)^2 \tag{9-10}$$

For $U > 60\%$,

$$T = 1.781 - 0.933 \log (100 - U\%) \tag{9-11}$$

EXAMPLE 9.3

Given:

$T = 0.05$ for a compressible clay deposit.

Required:

Average degree of consolidation and the percent consolidation at the center and at $z/H = 0.1$.

Solution:

From Table 9-1 and Fig. 9.5, $U_{avg} = 26\%$. Therefore the clay is 26% consolidated, on the average. From Fig. 9.3 you can see that the center of the layer is less than 0.5% consolidated, while at the "10%" depth ($z/H =$ 0.1) the clay is 73% consolidated. But, *on the average* throughout the layer, the clay is 26% consolidated.

What does the average consolidation mean in terms of settlements? U_{avg} can be expressed as

$$U_{avg} = \frac{s(t)}{s_c} \tag{9-12}$$

where $s(t)$ is the settlement at any time, and s_c is the final or ultimate consolidation (primary) settlement at $t = \infty$.

EXAMPLE 9.4

Given:

The data of Example 9.3.

Required:

Find the settlement when U_{avg} is 26%, if the final consolidation settlement is 1 m.

Solution:

From Eq. 9-12, $s(t) = U_{avg}(s_c)$. Therefore

$$s(t) = 26\% \text{ (1 m)} = 0.26 \text{ m}$$

EXAMPLE 9.5

Given:

The soil profile and properties of Examples 9.1 and 9.2.

Required:

Compute the time required for the clay layer to settle 0.25 m.

Reference 6

- The D'Appolonia method under-predicted the settlement by an average factor of about 0.77 for the five footings, but the accuracy was generally better than Hough, and the scatter in the predicted settlement values was also less (standard deviation, s_o = 0.22).

The data from the five load tests therefore generally corroborate the conclusions from the Gifford et al. and Burland & Burbidge studies.

Therefore, it is currently recommended that the Hough method be used as a primary design and analysis tool. The D'Appolonia or load-settlement methods could be employed as alternates, or when a more accurate estimate of settlement is desired, recognizing that these alternate methods have less history of use in the design of highway bridges. Naturally, if another method is locally or regionally known to provide more reliable estimates of settlement on granular soils, and the method meets the criteria noted above, the geotechnical engineer should exercise judgment and use the best method available.

5.3.4.1 Hough Method

Hough (1959) developed an empirical method for predicting settlements of shallow foundations on cohesionless soils that follows the same approach as that used for calculating consolidation settlement of clay layers. Note that the method is applicable only for normally consolidated cohesionless soils. Cheney and Chassie (2000) recommend that the SPT blowcounts be corrected for overburden pressure before correlating the N-values to the bearing capacity index, C′. An overburden correction by Bazaraa (1967) was recommended by Cheney and Chassie (2000). Since that time, many researchers have studied the effect of overburden stress on the SPT N-value, largely in support of liquefaction hazard assessment procedures. Recent consensus by the 1996 and 1998 National Center for Earthquake Engineering Research (NCEER) (Youd et al., 2001) concluded that the correction proposed by Liao & Whitman (1986) (shown in Figure 5-18) could be used for routine engineering applications. Therefore, the correction by Liao & Whitman is included here as part of the Hough procedure, in particular because it is easy to calculate and can be used without charts in simple computation spreadsheets. The soil is divided into layers, and the change in effective vertical stress at the mid-height of the layer as a result of the applied load is estimated using elastic theory.

The total settlement by the Hough method is calculated as follows:

1. Correct SPT blowcounts for overburden stress using Figure 5-18.

2. Determine bearing capacity index (C′) from Figure 5-19 using corrected SPT blowcounts, N′, determined in Step 1.

3. Subdivide subsurface soil profile into approximately 3-m (10-ft) layers based on stratigraphy to a depth of about three times the footing width.

4. Calculate the effective vertical stress, σ'_{vo}, at the midpoint of each layer and the average bearing capacity index for that layer.

5. Calculate the increase in stress at the midpoint of each layer, $\Delta\sigma_v$, using either Figure 5-9, 5-10 or 5-11, or the 2:1 method (Figure 5-12).

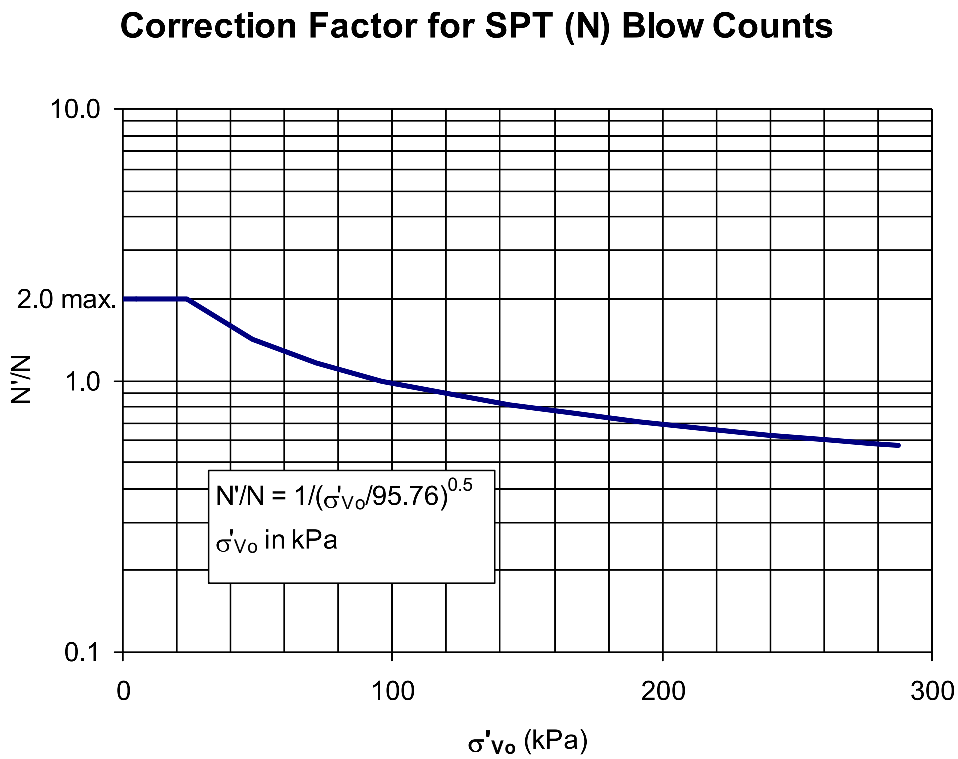


Figure 5-18: Corrected SPT (N) versus Overburden Pressure (after Liao & Whitman, 1986)

6. Calculate the settlement in each layer, ΔH, under the applied load using the following formula:

$$\Delta H \quad = H_o \frac{1}{C'} \log\left(\frac{\sigma'_{vo} + \Delta\sigma_{v_f}}{\sigma'_{vo}}\right) \tag{5-24}$$

7. Sum the incremental settlements to determine the total settlement.

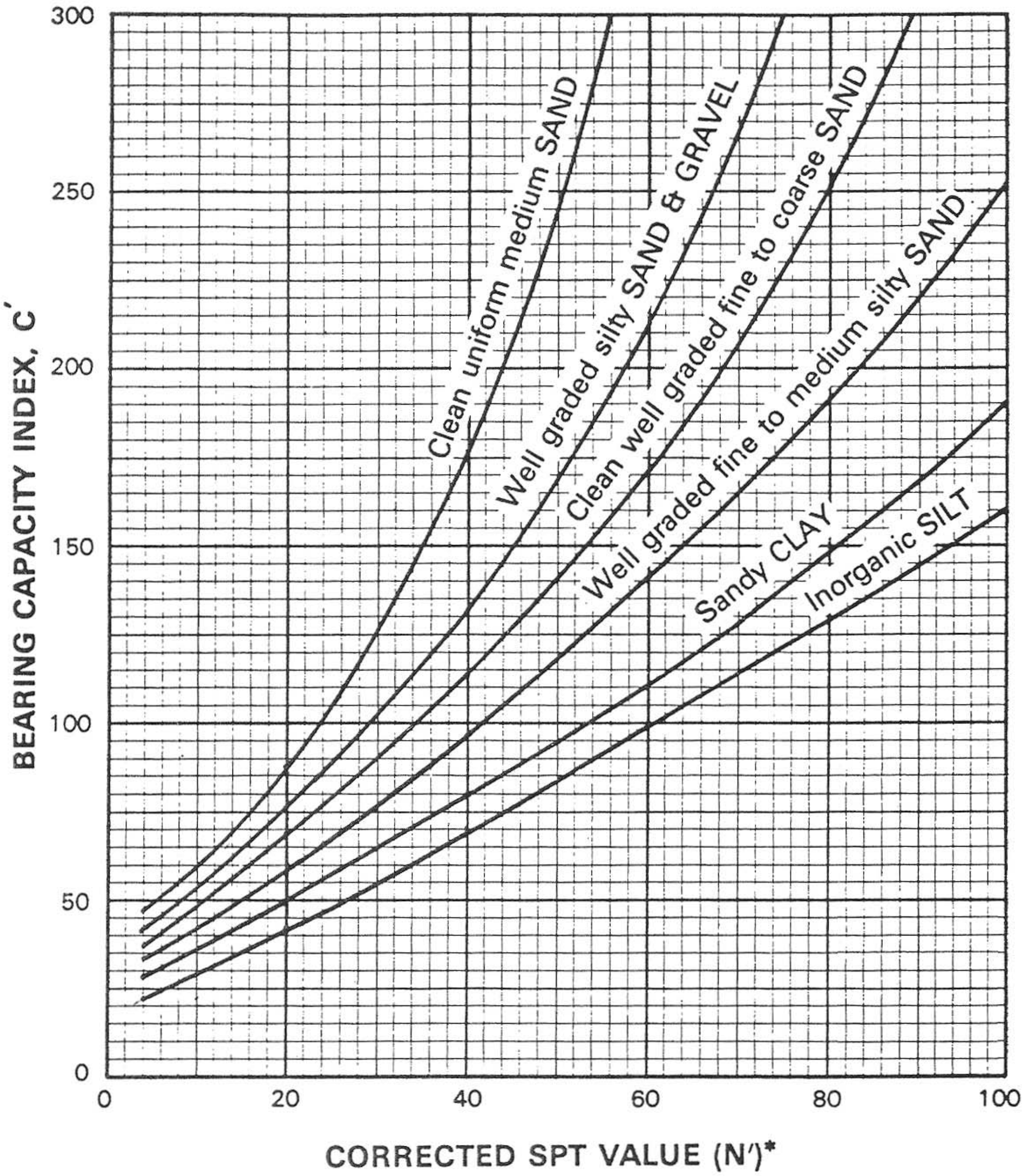
5.3.4.2 D'Appolonia Method

The D'Appolonia method calculates settlement using the following basic equation:

$$\Delta H = \left(\frac{\Delta\sigma_V B_f}{M}\right)\mu_o \mu_1 \tag{5-25}$$

where: ΔH = settlement in sand or sand and gravel
$\Delta\sigma_v$ = applied stress beneath footing
B_f = footing width
μ_O = correction factor for embedment – see Figure 5-20 (dimensionless)
μ_1 = correction factor for thickness of sand layer – see Figure 5-20 (dimensionless)
M = modulus of compressibility of sand – see Figure 5-21

Any consistent set of units can be used for ΔH, $\Delta\sigma_v$, B_f and M.



*N′—*SPT (N) Value Corrected for Overburden Pressure.*

Reference: Hough, "Compressibility as a Basis for Soil Bearing Value" ASCE 1959

Figure 5-19: Bearing Capacity Index versus Corrected SPT
(Cheney & Chassie, 2000, modified from Hough, 1959)

APPENDIX E5

Lateral Earth Pressure

GOLDER

CALCULATIONS

Date:	7/16/2020	**Made by:**	KAR/MEL
Project No.:	19126013	**Checked by:**	CJS/MEL
Subject:	Lateral Earth Pressure	**Reviewed by:**	CCB
Project Short Title:	MaineDOT I-295 Exit 20 Merrill Road Bridge Replacement No. 5720		

OBJECTIVE

Determine lateral earth pressure acting on the proposed bridge abutments, assuming the "southern shift" option with the bike path scenario.

REFERENCES

1. Golder interpreted subsurface section A-A' (Figure 3, Preliminary Geotechnical Design Report, dated September 2020).
2. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.
3. Email communication between Golder and HNTB, subject "RE: Freeport I-295 Exit 20 and Exit 22 - Request for Sections and Design Info", dated June 22, 2020.
4. Email and telephone communication between Golder and Laura Krusinski on July 29 and July 30, 2020 recommending the use of MassDOT passive earth pressure coefficient.
5. MassDOT LRFD Bridge Manual - Part 1, January 2020 Revision (https://www.mass.gov/doc/chapter-3-lrfd-bridge-design-guidelines/download)

ASSUMPTIONS

1. The backfill surface behind the abutments is assumed to be horizontal.
2. The fill is assumed to be free draining (i.e., no water pressure is allowed to build up behind the abutment walls).
3. The elevation of the base of the abutments is assumed to equal 158 feet at the northwestern abutment and 157 feet at the southeastern abutment (Ref. 1).

CALCULATION

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

As per Ref. 2 Section 5.4.2.11, the abutment reinforcement should be designed for the passive earth pressure (P_p) that results on the back face due to bridge thermal expansion. Developing full passive earth pressure requires that wall rotation (the ratio of lateral abutment movement to abutment height) exceeds 0.005. If the calculated rotation is less than that required to develop full passive pressure, the designer may consider using the Rankine passive earth pressure case, which assumes no wall friction. If full passive conditions are not achieved, MaineDOT and Golder discussed that the passive earth pressure coefficient in the Massachusetts DOT LRFD Bridge Design Manual (Ref. 5) would be more realistic as it is based on wall rotation rather than friction angle (Rankine).

Maximum lateral thermal movement =	0.7	inches	=	0.06	feet	(Ref. 3)
Abutment height =	12	feet				(Ref. 3)
Maximum wall rotation =	0.005					

Since the maximum wall rotation is estimated to be 0.005, full passive earth pressure may not develop. Thus, the earth pressure coefficients will be developed using Rankine active and Ref. 5 passive.

Date:	7/16/2020	**Made by:**	KAR/MEL
Project No.:	19126013	**Checked by:**	CJS/MEL
Subject:	Lateral Earth Pressure	**Reviewed by:**	CCB
Project Short Title:	MaineDOT I-295 Exit 20 Merrill Road Bridge Replacement No. 5720		

2. Calculate Rankine active earth pressure coefficient.

$$K_a = \tan^2\left(45° - \frac{\phi}{2}\right)$$ (Ref. 2, page 3-7)

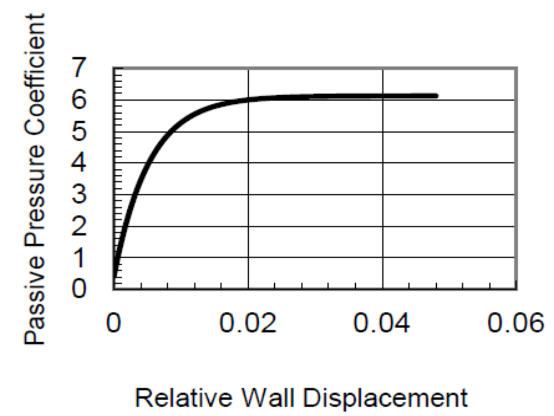
for horizontal backfill surface, where:

ϕ = internal friction angle of fill — ϕ = 32 degrees ("Granular borrow", Ref. 2, Table 3-3)

K_a = 0.31

3. Calculate the earth pressure coefficient under partially passive conditions per MASSDOT LRFD Bridge Manual Figure 3.10.8-1. (Note, full passive conditions are not met as per Calculation 1).

$$K = 0.43 + 5.7[1 - e^{-190(\delta_T/H)}]$$



δ_T/H = 0.005 (relative wall displacement)

K_p = 3.93

4. Determine the UNFACTORED passive pressure P_p acting on the abutments.

$$P_p = \frac{1}{2} \cdot \gamma_{soil} \cdot H_{abut}^2 \cdot k_p$$ (Ref. 2, page 5-51)

where:

γ_{soil} = unit weight of fill — γ_{soil} = 125 pcf ("Granular borrow", Ref. 2, Table 3-3)

H_{abut} = height of the abutment backwall — H_{abut} = 12 feet (Ref. 3)

P_p = 35,370 lbs per foot of abutment width

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

For the northwestern abutment, the load acts at: 162 feet elevation

For the southeastern abutment, the load acts at: 161 feet elevation

GOLDER

CALCULATIONS

Date:	7/16/2020	**Made by:**	KAR/MEL
Project No.:	19126013	**Checked by:**	CJS/MEL
Subject:	Lateral Earth Pressure	**Reviewed by:**	CCB
Project Short Title:	MaineDOT I-295 Exit 20 Merrill Road Bridge Replacement No. 5720		

CONCLUSIONS

For the designer-given wall geometry and thermal lateral movement, the calculated rotation is less than that required to develop full passive pressure, and thus the Rankine passive earth pressure case without wall friction was used. For the recommended soil parameters and given wall geometry, the Rankine passive earth pressure coefficient is K_p = 3.93, which corresponds to an unfactored passive earth force of P_p = 35,370 pounds per linear foot acting at elevations 162 feet and 161 feet on the northwestern and southeastern abutments, respectively. If, during final design, the calculated passive earth pressure forces against the abutments are deemed excessive, incorporating expanded polystyrene foam (EPS) geofoam behind the backwall can be used to mitigate a portion of the load.

APPENDIX E6

Pile Design

GOLDER

CALCULATIONS

Date:	8/26/2020	**Made by:**	KAR
Project No.:	19126013	**Checked by:**	MLM
Subject:	Pile Design at Abutment 1 - Southern Shift	**Reviewed by:**	CCB
Project Title:	MaineDOT I-295 Exit 20 Merrill Road Bridge Replacement No. 5720		

OBJECTIVE

Determine if the proposed HP 14x89 piles will provide adequate support for Abutment 1 (the northwestern integral abutment) based on the anticipated thermal movement and preliminary design loads, assuming the "southern shift" option with the bike path scenario.

METHOD

Use the procedure outlined in AASHTO LRFD (Ref. 1) and the design method provided in the MaineDOT Bridge Design Guide (Ref. 2).

REFERENCES

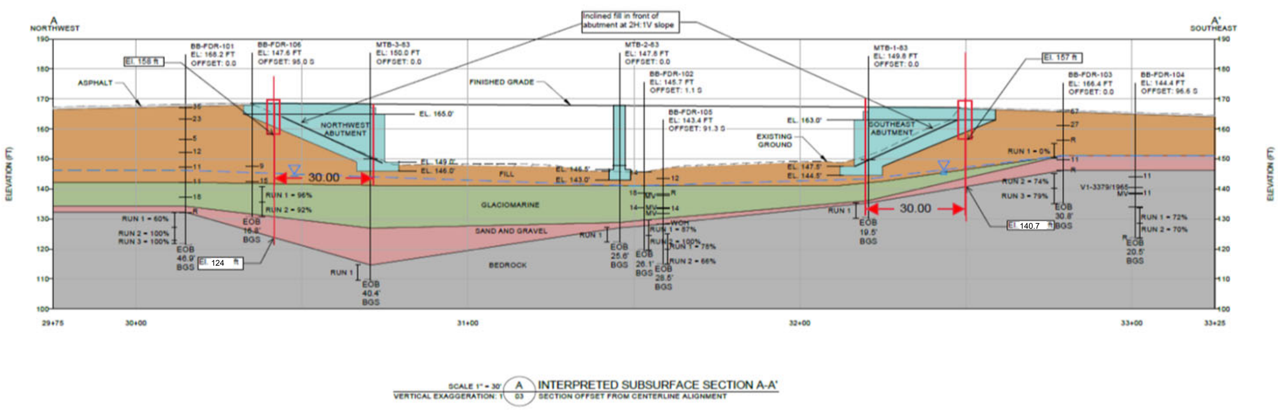
1. AASHTO LRFD Bridge Design Specifications, 9th Ed. 2020.
2. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.
3. Email communication between Golder and HNTB, subject "RE: Freeport I-295 Exit 20 and Exit 22 - Request for Sections and Design Info", dated June 22, 2020.
4. Isenhower, W.M. et al. LPile v2019 Technical Manual: A Program for the Analysis of Deep Foundations Under Lateral Loading. Ensoft, Inc. Dated March 2020.
5. Golder interpreted subsurface section A-A' (Figure 3, Preliminary Geotechnical Design Report, dated September 2020).
6. Golder geotechnical test boring logs (Appendix A, Preliminary Geotechnical Design Report, dated September 2020).
7. Bridge Software Institute. FB-MultiPier Soil Parameter Table (US Customary Units). Accessed July 2020. https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf
8. VTrans Integral Abutment Committee. Integral Abutment Bridge Design Guidelines, 2nd Ed. 2008.
9. AISC Steel Construction Manual, 13th Ed.
10. Golder calculation titled "Settlement at NW Bridge Embankment - Southern Shift" (Appendix E, Preliminary Geotechnical Design Report, dated September 2020).
11. Oregon Department of Transportation, Geo-Environmental Section. Geotechnical Design Manual: Chapter 8 - Foundations, Version 2.1. Dated May 6, 2019.
12. HNTB for State of Maine Department of Transportation. Merrill Road Bridge Freeport Interstate 295: Desert Road South Cross Sections, dated November 2019.

ASSUMPTIONS

1. The selected pile orientation is weak axis bending (Ref. 2, page 5-42).
2. The vertical load is assumed to be evenly distributed.
3. Based on discussions with HNTB, the new northwest abutment will be located approximately 30 ft behind the face of the existing northwest abutment. The post-construction ground surface elevation at the new northwest abutment will be 171 ft (Ref. 12). Assuming 1 ft of pavement atop the abutment plus a 12-ft abutment height (Ref. 3), the top of the piles will be located at elevation 158 ft.

CALCULATIONS

Date:	8/26/2020	**Made by:**	KAR
Project No.:	19126013	**Checked by:**	MLM
Subject:	Pile Design at Abutment 1 - Southern Shift	**Reviewed by:**	CCB
Project Title:	MaineDOT I-295 Exit 20 Merrill Road Bridge Replacement No. 5720		



ATTACHMENTS

1. LPile analysis output for Strength I
2. LPile analysis output for Strength I with Plastic Hinge
3. LPile analysis output for Service I

CALCULATION

1. Determine the downdrag load acting on the piles at the northwestern abutment.

As per Ref. 1 Article 3.11.8, downdrag can be assumed to fully develop if the settlement in the soil layer is 0.4 inches or greater relative to the pile. Since the settlement calculated in the bridge approach embankment approximately 17.5 feet behind the new northwestern abutment was estimated to be 1.36 inches (Ref. 10), it is assumed that downdrag will develop.

Determine the soil layers contributing to downdrag (the deepest layer with settlement ≥ 0.4 inches and all layers above that).

Layer		Layer Thickness in Embankment (ft)	σ'_{v0} at layer midpoint in Embankment (ksf)	Settlement Based on Calculated Loading Stress (in)
Existing Fill	1	9.6	0.600	0.63
Glaciomarine	2	5.3	1.341	0.37
Glaciomarine	3	5.3	1.673	0.28
Sand and Gravel	4	2.2	1.907	0.08
(Ref. 10)			Total Settlement (in):	1.36

Layers 1, 2, and 3 will contribute to the downdrag load (considering 2 and 3 as a single layer).

Use the α-method to calculate the nominal skin resistance for the cohesive soils contributing to downdrag (Ref. 1, Article 10.7.3.8.6b); use the Nordlund/Thurman method to calculate the skin resistance for the cohesionless soils contributing to downdrag (Ref. 1, Article 10.7.3.8.6f).

α-method for Layers 2 and 3, Glaciomarine:

$$q_s = \alpha S_u$$ (Ref. 1, Eqn 10.7.3.8.6b-1)

where:

S_u = 1.600 ksf (based on shear strength measurements made in the field and on empirical correlation to the average of the N_{60}-values encountered in all borings for the layer)

D = 13.83 in = 1.2 ft (Ref. 4, Table 5.6.3, HP 14x89)

D_b = 11 ft (thickness of glaciomarine at abutment, Ref. 5)

α = adhesion factor from Ref. 1 Figure 10.7.3.8.6b-1
Use the plot for "Sands over Stiff Clay" and the curve for "D_b less than 10D"

α = 1.00

q_s = 1.600 ksf

Nordlund/Thurman method for Layer 1, Existing Fill:

$$q_s = K_\delta C_F \sigma'_v \frac{\sin(\delta + \omega)}{\cos \omega}$$ (Ref. 1, Eqn 10.7.3.8.6f-1)

where:

ϕ_f = 32 degrees (based on empirical correlation to the average of the N_{60}-values encountered in all borings for the layer)

V = A_s = 26.1 in^2 = 0.18 ft^3/ft (Soil Displacement, Ref. 4, Table 5.6.3, HP 14x89)

K_δ = 1.01 (interpolation between Ref. 1 Figures 10.7.3.8.6f-2 and 10.7.3.8.6f-3, based on V)

C_F = 0.94 (Correction factor, based on φf and δ/φf)

σ'_v = 1.000 ksf (Ref. 5; fill thickness at abutment is 16 ft)

δ/ϕ_f = 0.81 (Ref. 1, Figure 10.7.3.8.6f-6)

δ = 26 degrees (Ref. 1, Figure 10.7.3.8.6f-6)

ω = 0 degrees (assume pile battering not required as per Step 3)

q_s = 0.414 ksf

Convert nominal skin resistance to nominal axial downdrag load.

As per Ref. 1 Article C10.7.3.8.6b, for H-piles the perimeter or "box" area should generally be used to compute the surface area of the pile side.

GOLDER

CALCULATIONS

Date:	8/26/2020	**Made by:**	KAR
Project No.:	19126013	**Checked by:**	MLM
Subject:	Pile Design at Abutment 1 - Southern Shift	**Reviewed by:**	CCB
Project Title:	MaineDOT I-295 Exit 20 Merrill Road Bridge Replacement No. 5720		

Perimeter of HP 14x89 pile = 57.05 in = 4.75 ft

Layer		Contributing Layer Thickness at Abutm. (ft)	Surface area of pile side (ft^2)	Load (lbs)	Strength I Load Factor	Service I Load Factor
Existing Fill	1	16	76.1	31502	1.10	1.00
Glaciomarine	2	5.3	25.2	40315	1.40	1.00
Glaciomarine	3	5.3	25.2	40315	1.40	1.00

(Ref. 1 Tables 3.4.1-1 and 3.4.1-2; Ref. 11 Table 8.2)

Total Factored Load, Strength I = 147536 lbs per pile
= 147.5 kips per pile

Total Factored Load, Service I = 112133 lbs per pile
= 112.1 kips per pile

According to Ref. 3, typical factored pile loads (Strength I) are expected to be on the order of 350 to 450 kips per pile depending on pile spacing. A downdrag load of 148 kips (Strength I) or 112 kips (Service I) per pile will be added.

2. Select the preliminary pile size.

Determine the factored applied superstructure vertical dead and live load (P_u) distributed to each pile.

Maximum P_u = 598 kips (maximum factored load from Ref. 3 plus downdrag from Step 1)

As part of this analysis loads up to the expected maximum of 598 kips were evaluated, and it was determined that loads higher than 522 kips would require selection of a pile size with an area larger than that provided by HP 14x89. Since the axial loads provided by HNTB in Ref. 3 are preliminary, this analysis was perfomed with P_u = 522 kips, which would correspond to a factored axial load excluding downdrag of 374 kips.

Determine the factored applied superstructure vertical dead and live load (P_u) distributed to each pile.

Design P_u = 522 kips (assumed preliminary factored load)

Select the steel pile strength.

F_y = 50 ksi
E = 29,000 ksi

Date:	8/26/2020	**Made by:**	KAR
Project No.:	19126013	**Checked by:**	MLM
Subject:	Pile Design at Abutment 1 - Southern Shift	**Reviewed by:**	CCB
Project Title:	MaineDOT I-295 Exit 20 Merrill Road Bridge Replacement No. 5720		

Determine resistance factors (Φ_c and Φ_f) for the structural strength in the upper and lower zones of the pile.

ϕ_{cl} = 0.50 for axial resistance in the lower zone of the pile (Ref. 2, page 5-41)
ϕ_{cu} = 0.70 for axial resistance in the upper zone of the pile (Ref. 2, page 5-42)
ϕ_f = 1.00 for flexural resistance in the upper zone of the pile (Ref. 2, page 5-42)

Determine the maximum required nominal axial pile resistance (Ref. 1, Article 6.9.2.1).

$$R_{n.upper} = \frac{P_u}{\phi_{cu}}$$

$R_{n.upper}$ = 746 kips

$$R_{n.lower} = \frac{P_u}{\phi_{cl}}$$

$R_{n.lower}$ = 1044 kips

$$R_n = \max\left(R_{n.upper}, R_{n.lower}\right)$$

R_n = 1044 kips

Use the required nominal axial pile resistance to estimate the required pile area.

$$A_{s.req} = \frac{R_n}{0.80\ Fy}$$ (Ref. 2, page 5-42)

$A_{s.req}$ = 26.1 in^2

Select a pile size with an area of $A_{s.req}$ or greater.

Preferred selection is HP 14x89 based on June 16, 2020 meeting with MaineDOT and HNTB.
Check that preferred selection satisfies pile area requirement:

HP 14x89 A_s = 26.1 in^2 (Ref. 4, Table 5.6.3)
A_s = $A_{s.req}$ OK

3. Use LPile analysis to determine the pile unbraced length and maximum moment at the top of the pile.

The following input parameters were used in the LPile analysis:

Pile Properties

Section type: Steel H Section Weak Axis (Assumption 1)

Length of section:	34	ft	(piles driven to bedrock with no rock socketing)
Flange width, b:	14.695	in	(Ref. 4, Table 5.6.3)
Section depth, d:	13.83	in	(Ref. 4, Table 5.6.3)
Flange thickness, t_f:	0.615	in	(Ref. 4, Table 5.6.3)
Web thickness, t_w:	0.615	in	(Ref. 4, Table 5.6.3)
Pile batter:	Vertical		(pile battering not required)

Pile Loading

Lateral deflection normal to pile axis, y:	0.7	in	(Ref. 3)
Axial load:	522,000	lbs	(Ref. 3)

Soil Layers

Layer	Depth below base of abutment[1]	Lateral Model	Effective Unit Weight (pcf)	Undrained Shear Strength (psf)[2]	Friction Angle (°)[2]	Subgrade Modulus (pci)[3]	Major Principal Strain at 50%[3]	UCS (psi)[2]
Existing Fill (above water table)	0 - 12.7 ft	Sand (Reese)	125	-	32	124.8	-	-
Existing Fill (below water table)	12.7 - 16 ft	Sand (Reese)	62.6	-	32	75.5	-	-
Glaciomarine Silty Clay	16 - 27 ft	Stiff Clay w/o Free Water (Reese)	62.6	1600	-	-	0.005	-
Sand and Gravel	27 - 34 ft	Sand (Reese)	62.6	-	37	40.5	-	-
Bedrock	>34 ft	Strong Rock (Vuggy Limestone)	101.6	-	-	-	-	12983

1) Ref. 5
2) Ref. 6
3) Ref. 7

The full LPile output is provided in Attachment 1.

Obtain the maximum moment at the top of the pile.

$M_{u.Top}$ = 2396 in-kips (LPile)

Obtain the unbraced lengths of the top segment and the second segment of the upper zone of the pile.

$l_{b.top}$ = 4.61 ft (LPile)
$l_{b.top}$ = 55.34 in

$l_{b.2nd}$ = 11.66 ft (LPile)
$l_{b.2nd}$ = 139.89 in

4. Determine if the applied moment on the pile will cause pile head plastic deformation by using the interaction of combined axial and flexural load effects on a single pile.

Determine K values for the top and bottom of the pile and calculate the column slenderness factor (λ) for each segment.

For the top segment (fixed at top and pinned at bottom):

$$\lambda_{top} = \frac{K_{top} l_{b.top}}{r_y} \leq 120$$ (Ref. 1, Article 6.9.3)

$$r_y = \sqrt{I_{yy}/A_s}$$

where:
K_{top} = 1.2 (Ref. 1, Table C4.6.2.5-1)
I_{yy} = 326 in^4 (Ref. 4, Table 5.6.3)
r_y = 3.53 in

λ_{top} = 18.79 OK

For the second segment (pinned at top and bottom):

$$\lambda_{2nd} = \frac{K_{2nd} l_{b.2nd}}{r_y} \leq 120$$ (Ref. 1, Article 6.9.3)

where:
K_{2nd} = 1.0 (Ref. 1, Table C4.6.2.5-1)

λ_{2nd} = 39.58 OK

Calculate the critical elastic buckling resistance, P_e, and the nominal yield resistance, P_o.

Use Ref. 1 Table 6.9.4.1.1-1 to select equation for P_e based on cross-section shape and potential buckling mode.

$$P_e = \frac{\pi^2 E}{\left(\frac{K\, l_b}{r_y}\right)^2} A_s$$ (Ref. 1, Eqn 6.9.4.1.2-1)

$P_{e.top}$ = 21156 kips
$P_{e.2nd}$ = 4768 kips

$$P_o = F_y A_s$$ (Ref. 1, Article 6.9.4.1)

P_o = 1305 kips

Calculate the nominal structural pile resistance, P_n, for both segments of the upper zone of the pile as well as the lower zone of the pile.

Determine P_o/P_e to select equation for P_n as per Ref. 1 Article 6.9.4.1.

$P_o/P_{e.top}$ = 0.06 ≤ 2.25
$P_o/P_{e.2nd}$ = 0.27 ≤ 2.25

thus use Ref. 1 Eqn 6.9.4.1.1-1:

$$P_n = \left[0.658^{\left(\frac{P_o}{P_e}\right)}\right] P_o$$

$P_{n.top}$ = 1272 kips

$P_{n.2nd}$ = 1164 kips

$$P_{n.bottom} = (0.658^{(0)}) \times F_y A_s$$ (0 for a fully braced pile - Ref. 8, Appendix B, Eqn 6-9)

$P_{n.bottom}$ = 1305 kips

Calculate the factored structural pile resistance, P_r, for both segments of the upper zone of the pile as well as the lower zone of the pile.

$$P_{r.top} = \phi_{cu}\, P_{n.top}$$
$P_{r.top}$ = 890.2 kips

$$P_{r.2nd} = \phi_{cu}\, P_{n.2nd}$$
$P_{r.2nd}$ = 814.6 kips

$$P_{r.bottom} = \phi_{cl}\, P_{n.bottom}$$

$P_{r.bottom}$ = 652.5 kips

Compare the ratio of P_u to the structural resistance in the upper portion of the pile – the pile size should be such that the ratio is not less than 0.20.

$$\frac{P_u}{P_{r.top}} = \quad 0.59 \qquad \text{OK}$$

$$\frac{P_u}{P_{r.2nd}} = \quad 0.64 \qquad \text{OK}$$

Since the lower zone of the pile will have virtually no moment, the entire section can carry the required vertical loads. Make sure the applied load will not exceed the resistance of the lower zone.

$$\text{Check}\left(\frac{P_u}{P_{r.bottom}} < 1\right)$$

$$\frac{P_u}{P_{r.bottom}} = \quad 0.80 \qquad \text{OK}$$

Determine the nominal and factored flexural resistance about H-Pile weak axis (LRFD 6.12.2.2).

Slenderness ratio for the flange:

$$\lambda_f = \frac{b_f}{2t_f} \qquad \text{(Ref. 1, Eqn 6.12.2.2.1-3)}$$

$\lambda_f =$ 11.95

Limiting slenderness ratio for a compact flange:

$$\lambda_{pf} = 0.38\sqrt{\frac{E}{F_y}} \qquad \text{(Ref. 1, Eqn 6.12.2.2.1-4)}$$

$\lambda_{pf} =$ 9.15

Limiting slenderness ratio for a noncompact flange:

$$\lambda_{rf} = 0.83\sqrt{\frac{E}{F_y}} \qquad \text{(Ref. 1, Eqn 6.12.2.2.1-5)}$$

$\lambda r_f =$ 19.99

Elastic and plastic section moduli about the weak axis:

$$S_y = \frac{I_{yy}}{b/2}$$

$$Z_y = (b^2 t_f)/2 + 0.25t_w^2(d - 2t_f)$$

S_y = 44.4 in^3

Z_y = 67.6 in^3

Nominal flexural resistance:

$$M_n = M_p = (F_y Z_y) \qquad \text{if } \lambda_f \leq \lambda_{pf}$$ (Ref. 1, Eqn 6.12.2.2.1-1)

$$M_n = \left[1 - \left(1 - \frac{S_y}{Z_y}\right)\left(\frac{\lambda_f - \lambda_{pf}}{0.45\sqrt{\frac{E}{F_y}}}\right)\right] F_y Z_y \qquad \text{if } \lambda_{pf} < \lambda_f \leq \lambda_{rf}$$

(Ref. 1, Eqn 6.12.2.2.1-2)

Since $\lambda_{pf} < \lambda_f \leq \lambda_{rf}$,

M_n = 3080 in-kips

Factored flexural resistance:

ϕ_f = 1.00 (Ref. 2, page 5-42)

$$M_r = \phi_f M_n$$

M_r = 3080 in-kips

Calculate the moment that will cause a plastic hinge at the top of the pile, M_p'(Ref. 2, Article 6.9.2.2).

$$M'_p = \frac{9}{8}\left(1 - \frac{P_u}{P_{r.top}}\right) M_r$$ (Ref. 8, Appendix B, Eqn 6-24)

M_p' = 1433 in-kips = 1433281.6 inch-lb

If the applied moment exceeds the moment that would cause a plastic hinge, it can be assumed that the pile head has entered plastic deformation and therefore the moment that can be applied to the pile head cannot exceed M_p'.

$M_{u.Top}$ = 2396 in-kips (From Step 3)

$M_{u.Top}$ > M_p' Plastic Hinge Forms

5. Run a second LPile analysis with displacement, plastic moment (M_p'), and P_u as load conditions, and calculate new unbraced lengths from the moment vs. depth curve. Then repeat Step 4 with the new unbraced lengths.

$l_{b.top}$ = 3.43 ft (LPile)

$l_{b.top}$ = 41.14 in

$l_{b.2nd}$ = 12.38 ft (LPile)

$l_{b.2nd}$ = 148.52 in

$M_{u.2nd}$ = 910.03 in-kips (LPile)

Since a plastic hinge developed at the pile head, the value of K for the top segment becomes 2.1 (Ref. 2, page 5-43).

K_{top} = 2.1 (Ref. 1, Table C4.6.2.5-1)
K_{2nd} = 1.0 (Ref. 1, Table C4.6.2.5-1)

λ_{top} = 24.45 < 120 OK
λ_{2nd} = 42.02 < 120 OK

$P_{e.top}$ = 12499 kips
$P_{e.2nd}$ = 4230 kips

$P_o/P_{e.top}$ = 0.10 ≤ 2.25 (to select P_n equation)
$P_o/P_{e.2nd}$ = 0.31 ≤ 2.25 (to select P_n equation)

$P_{n.top}$ = 1249 kips
$P_{n.2nd}$ = 1147 kips

$P_{r.top}$ = 874 kips
$P_{r.2nd}$ = 803 kips

$\frac{P_u}{P_{r.top}} =$ 0.60 > 0.20 OK

$\frac{P_u}{P_{r.2nd}} =$ 0.65 > 0.20 OK

Since the pile is appropriately sized, the second segment of the upper zone of the pile needs to be checked with the interaction equation of LRFD Section 6.9.2.2. It is important that this segment of the pile does not form a plastic hinge. A plastic hinge in this segment will cause the pile to fail.

Check: $\frac{P_u}{P_{r.2nd}} + \frac{8}{9}\left(\frac{M_{u.2nd}}{M_r}\right) < 1$ (Ref. 8, Appendix B, Eqn 7-13)

Check: 0.91 < 1 OK

6. Because the piles have weak axis orientation and the flanges resist the shear as opposed to the web, check the maximum shear from the LPile output against the structural shear resistance per AISC G7.

V_u = 34.68 kips (LPile)

AASHTO LRFD does not directly address weak axis shear. This analysis will use the AISC Steel Construction Manual 13th edition (G7) to ensure the pile will not shear under the longitudinal load.

k_v = 1.2 (Ref. 9, Section G2.1)

$$C_v = 1.0 \text{ if } b/t_f \leq 1.1\sqrt{k_v E/F_y}$$ (Ref. 9, Eqn. G2-3)

C_v = 1.0

Both flanges will resist shear forces:

$$A_w = 2b_f t_f$$ (Ref. 8, Appendix B, Eqn 7-17)

A_w = 18.07 in^2

$$V_n = 0.6F_y A_w C_V$$ (Ref. 9, Eqn G2-1)

V_n = 542 kips

$$V_r = \Phi_v V_n$$

ϕ_v = 1.00 (Ref. 1, Article 6.5.4.2)

V_r = 542 kips

Check that the shear resistance is sufficient:

V_u < V_r OK

7. Check that the maximum factored applied pile load does not exceed the factored pile drivability resistance.

While driving the pile, the maximum stress that is permitted in the pile is:

$$\sigma_{dr} = 0.9\Phi_{da}F_y$$ (Ref. 8, Appendix B, Eqn 7-22)

ϕ_{da} = 1.00 (Ref. 1, Article 6.5.4.2)

σ_{dr} = 45 ksi

This translates into an ultimate maximum driving force that can be applied to the pile of:

$$P_0 = \sigma_{dr} A_s$$ (Ref. 8, Appendix B, Eqn 7-23)

P_0 = 1175 kips

Calculate the nominal pile driving resistance (R_{ndr}) from the applied load divided by the resistance factor associated with the pile monitoring method. In this design, the pile will be bearing on rock. The driving criteria will be established by dynamic testing.

Date:	8/26/2020	**Made by:**	KAR
Project No.:	19126013	**Checked by:**	MLM
Subject:	Pile Design at Abutment 1 - Southern Shift	**Reviewed by:**	CCB
Project Title:	MaineDOT I-295 Exit 20 Merrill Road Bridge Replacement No. 5720		

ϕ_{mon} = 0.65 (Ref. 1, Table 10.5.5.2.3-1)

$$R_{ndr} = \frac{P_u}{\phi_{mon}}$$ (Ref. 8, Appendix B, Eqn 7-25)

R_{ndr} = 803 kips

The nominal pile driving resistance (R_{ndr}) should not exceed the nominal structural pile resistance (P_n) or the maximum driving force (P_0) calculated above.

$P_{n.top}$ = 1249 kips (From Step 5)
$P_{n.2nd}$ = 1147 kips (From Step 5)

Check $R_{ndr} < P_n$: OK
Check $R_{ndr} < P_0$: OK

CONCLUSIONS

The results of the analysis indicate that a maximum moment of 2396 in-kips (200 ft-kips) occurs at the top of the pile under the Strength I load case, with a maximum bridge expansion of 0.7 inches. The results indicate that the depth to bedrock is sufficient for driven piles to achieve fixity, and rock socketing is not anticipated to be required at Abutment 1. HP 14x89 piles will provide adequate support for Abutment 1 based on the anticipated thermal movement. A maximum factored axial load (excluding downdrag) of 374 kips should be used with HP 14x89 piles. Additional piles per abutment can be used to reduce the load on each pile; alternatively, downdrag forces can be mitigated to reduce the total load. A drivability analysis will be performed in a separate package.

==

LPile for Windows, Version 2019-11.005

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

==

This copy of LPile is being used by:

KAR
Golder Associates, Inc.

Serial Number of Security Device: 239146533

This copy of LPile is licensed for exclusive use by:

GOLDER, Manchester, NH

Files Used for Analysis

Path to file locations:
\Users\kroth\Documents\Projects\19126013 MaineDOT I-295 Freeport Exit 20 Merrill Rd Bridge\Pile Design\LPile Northwest Abutment\

Name of input data file:
Freeport Exit 20 Northwest Abutment Southern Shift.lp11d

Name of output report file:
Freeport Exit 20 Northwest Abutment Southern Shift.lp11o

Name of plot output file:
Freeport Exit 20 Northwest Abutment Southern Shift.lp11p

Name of runtime message file:
Freeport Exit 20 Northwest Abutment Southern Shift.lp11r

Date and Time of Analysis

Date: August 26, 2020 Time: 9:35:04

Problem Title

Project Name: MaineDOT I-295 Exit 20 Merrill Road Bridge No. 5720
Job Number: 19126013
Client: MaineDOT
Engineer: KAR
Description: Northwest Abutment Pile Design - Strength I

Program Options and Settings

Computational Options:
- Conventional Analysis

Engineering Units Used for Data Input and Computations:
- US Customary System Units (pounds, feet, inches)

Analysis Control Options:
- Maximum number of iterations allowed = 500
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 100.0000 in
- Number of pile increments = 100

Loading Type and Number of Cycles of Loading:
- Static loading specified

- Use of p-y modification factors for p-y curves not selected
- Analysis uses layering correction (Method of Georgiadis)
- No distributed lateral loads are entered
- Loading by lateral soil movements acting on pile not selected
- Input of shear resistance at the pile tip not selected
- Input of moment resistance at the pile tip not selected
- Input of side resistance moment along pile not selected
- Computation of pile-head foundation stiffness matrix not selected
- Push-over analysis of pile not selected
- Buckling analysis of pile not selected

Output Options:
- Output files use decimal points to denote decimal symbols.
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (nodal spacing of output points) = 1
- No p-y curves to be computed and reported for user-specified depths
- Print using wide report formats

Pile Structural Properties and Geometry

Number of pile sections defined = 1
Total length of pile = 34.000 ft
Depth of ground surface below top of pile = 0.0000 ft

Pile diameters used for p-y curve computations are defined using 2 points.

p-y curves are computed using pile diameter values interpolated with depth over the length of the pile. A summary of values of pile diameter vs. depth follows.

Point No.	Depth Below Pile Head feet	Pile Diameter inches
1	0.000	14.6950
2	34.000	14.6950

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is a H weak axis steel pile
Length of section = 34.000000 ft
Pile width = 13.830000 in
Shear capacity of section = 0.0000 lbs

Ground Slope and Pile Batter Angles

Ground Slope Angle = 0.000 degrees
= 0.000 radians

Pile Batter Angle = 0.000 degrees
= 0.000 radians

Soil and Rock Layering Information

The soil profile is modelled using 5 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer = 0.0000 ft
Distance from top of pile to bottom of layer = 12.700000 ft

Effective unit weight at top of layer = 125.000000 pcf
Effective unit weight at bottom of layer = 125.000000 pcf
Friction angle at top of layer = 32.000000 deg.
Friction angle at bottom of layer = 32.000000 deg.
Subgrade k at top of layer = 124.800000 pci
Subgrade k at bottom of layer = 124.800000 pci

Layer 2 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer = 12.700000 ft
Distance from top of pile to bottom of layer = 16.000000 ft
Effective unit weight at top of layer = 62.600000 pcf
Effective unit weight at bottom of layer = 62.600000 pcf
Friction angle at top of layer = 32.000000 deg.
Friction angle at bottom of layer = 32.000000 deg.
Subgrade k at top of layer = 75.500000 pci
Subgrade k at bottom of layer = 75.500000 pci

Layer 3 is stiff clay without free water

Distance from top of pile to top of layer = 16.000000 ft
Distance from top of pile to bottom of layer = 27.000000 ft
Effective unit weight at top of layer = 62.600000 pcf
Effective unit weight at bottom of layer = 62.600000 pcf
Undrained cohesion at top of layer = 1600. psf
Undrained cohesion at bottom of layer = 1600. psf
Epsilon-50 at top of layer = 0.005000
Epsilon-50 at bottom of layer = 0.005000

Layer 4 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer = 27.000000 ft
Distance from top of pile to bottom of layer = 34.000000 ft
Effective unit weight at top of layer = 62.600000 pcf
Effective unit weight at bottom of layer = 62.600000 pcf
Friction angle at top of layer = 37.000000 deg.
Friction angle at bottom of layer = 37.000000 deg.
Subgrade k at top of layer = 40.500000 pci
Subgrade k at bottom of layer = 40.500000 pci

Layer 5 is strong rock (vuggy limestone)

Distance from top of pile to top of layer = 34.000000 ft
Distance from top of pile to bottom of layer = 50.000000 ft
Effective unit weight at top of layer = 101.600000 pcf
Effective unit weight at bottom of layer = 101.600000 pcf
Uniaxial compressive strength at top of layer = 12983. psi
Uniaxial compressive strength at bottom of layer = 12983. psi

(Depth of the lowest soil layer extends 16.000 ft below the pile tip)

Summary of Input Soil Properties

Layer Layer Num.	Soil Type Name (p-y Curve Type)	Layer Depth ft	Effective Unit Wt. pcf	Undrained Cohesion psf	Angle of Friction deg.	Uniaxial qu psi	E50 or krm	kpy pci
1	Sand	0.00	125.0000	--	32.0000	--	--	124.8000
	(Reese, et al.)	12.7000	125.0000	--	32.0000	--	--	124.8000
2	Sand	12.7000	62.6000	--	32.0000	--	--	75.5000
	(Reese, et al.)	16.0000	62.6000	--	32.0000	--	--	75.5000
3	Stiff Clay	16.0000	62.6000	1600.	--	--	0.00500	--
	w/o Free Water	27.0000	62.6000	1600.	--	--	0.00500	--
4	Sand	27.0000	62.6000	--	37.0000	--	--	40.5000
	(Reese, et al.)	34.0000	62.6000	--	37.0000	--	--	40.5000
5	Strong Rock	34.0000	101.6000	--	--	12983.	--	--
	(Vuggy Limestone)	50.0000	101.6000	--	--	12983.	--	--

Static Loading Type

Static loading criteria were used when computing p-y curves for all analyses.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load No.	Load Type	Condition 1	Condition 2	Axial Thrust Force, lbs	Compute Top y vs. Pile Length	Run Analysis
1	5	y = -0.700000 in	S = 0.0000 in/in	522000.	N.A.	Yes

V = shear force applied normal to pile axis
M = bending moment applied to pile head
y = lateral deflection normal to pile axis
S = pile slope relative to original pile batter angle
R = rotational stiffness applied to pile head
Values of top y vs. pile lengths can be computed only for load types with specified shear loading (Load Types 1, 2, and 3).
Thrust force is assumed to be acting axially for all pile batter angles.

Computations of Nominal Moment Capacity and Nonlinear Bending Stiffness
---

Axial thrust force values were determined from pile-head loading conditions

Number of Pile Sections Analyzed = 1

Pile Section No. 1:

Dimensions and Properties of Steel H Weak Axis:

Length of Section	=	34.000000 ft
Flange Width	=	14.695000 in
Section Depth	=	13.830000 in
Flange Thickness	=	0.615000 in
Web Thickness	=	0.615000 in
Yield Stress of Pipe	=	50.000000 ksi
Elastic Modulus	=	29000. ksi
Cross-sectional Area	=	25.823850 sq. in.
Moment of Inertia	=	325.505721 in^4
Elastic Bending Stiffness	=	9439666. kip-in^2
Plastic Modulus, Z	=	67.593889in^3
Plastic Moment Capacity = Fy Z	=	3380.in-kip

Axial Structural Capacities:

Nom. Axial Structural Capacity = Fy As	=	1291.193 kips
Nominal Axial Tensile Capacity	=	-1291.193 kips

Number of Axial Thrust Force Values Determined from Pile-head Loadings = 1

Number	Axial Thrust Force kips
1	522.000

Definition of Run Messages:

Y = part of pipe section has yielded.

Axial Thrust Force = 522.000 kips

Bending Curvature rad/in.	Bending Moment in-kip	Bending Stiffness kip-in2	Depth to N Axis in	Max Total Stress ksi	Run Msg

0.00000404	38.1169076	9438779.	179.9510245	21.0657447	
0.00000808	76.2338151	9438779.	93.6492623	21.9176173	
0.00001211	114.3507227	9438779.	64.8820082	22.7694899	
0.00001615	152.4676303	9438779.	50.4983811	23.6213625	
0.00002019	190.5845379	9438779.	41.8682049	24.4732347	
0.00002423	228.7014454	9438779.	36.1147541	25.3251077	
0.00002827	266.8183530	9438779.	32.0051464	26.1769800	
0.00003231	304.9352606	9438779.	28.9229406	27.0288528	
0.00003634	343.0521682	9438779.	26.5256694	27.8807252	
0.00004038	381.1690757	9438779.	24.6078525	28.7325980	
0.00004442	419.2859833	9438779.	23.0387295	29.5844705	
0.00004846	457.4028909	9438779.	21.7311270	30.4363432	
0.00005250	495.5197985	9438779.	20.6246942	31.2882157	
0.00005654	533.6367060	9438779.	19.6763232	32.1400884	
0.00006057	571.7536136	9438779.	18.8544016	32.9919609	
0.00006461	609.8705212	9438779.	18.1352203	33.8438336	
0.00006865	647.9874287	9438779.	17.5006485	34.6957061	
0.00007269	686.1043363	9438779.	16.9365847	35.5475788	
0.00007673	724.2212439	9438779.	16.4318960	36.3994513	
0.00008077	762.3381515	9438779.	15.9776762	37.2513240	
0.00008480	800.4550590	9438779.	15.5667155	38.1031965	
0.00008884	838.5719666	9438779.	15.1931148	38.9550692	
0.00009288	876.6888742	9438779.	14.8520011	39.8069417	
0.00009692	914.8057818	9438779.	14.5393135	40.6588144	
0.0001010	952.9226893	9438779.	14.2516410	41.5106869	
0.0001050	991.0395969	9438779.	13.9860971	42.3625596	
0.0001090	1029.	9438779.	13.7402231	43.2144321	
0.0001131	1067.	9438779.	13.5119116	44.0663048	
0.0001171	1105.	9438779.	13.2993457	44.9181773	
0.0001211	1144.	9438779.	13.1009508	45.7700500	
0.0001252	1182.	9438779.	12.9153556	46.6219225	
0.0001292	1220.	9438779.	12.7413601	47.4737952	
0.0001333	1258.	9438779.	12.5779098	48.3256677	
0.0001373	1296.	9438779.	12.4240743	49.1775404	
0.0001413	1334.	9438503.	12.2790799	50.0000000	Y
0.0001454	1371.	9428344.	12.1439669	50.0000000	Y
0.0001494	1406.	9409586.	12.0178871	50.0000000	Y
0.0001535	1440.	9383244.	11.9001053	50.0000000	Y
0.0001575	1472.	9349477.	11.7901137	50.0000000	Y
0.0001656	1534.	9266985.	11.5903389	50.0000000	Y
0.0001736	1592.	9169338.	11.4140577	50.0000000	Y
0.0001817	1647.	9061268.	11.2577541	50.0000000	Y
0.0001898	1698.	8946572.	11.1184873	50.0000000	Y
0.0001979	1747.	8828299.	10.9937734	50.0000000	Y
0.0002060	1793.	8707082.	10.8819826	50.0000000	Y
0.0002140	1838.	8586292.	10.7809631	50.0000000	Y
0.0002221	1880.	8465251.	10.6898028	50.0000000	Y
0.0002302	1921.	8346099.	10.6069483	50.0000000	Y
0.0002383	1961.	8229110.	10.5314770	50.0000000	Y
0.0002463	1999.	8114243.	10.4626706	50.0000000	Y
0.0002544	2036.	8001908.	10.3997576	50.0000000	Y
0.0002625	2072.	7892519.	10.3420286	50.0000000	Y
0.0002706	2107.	7786163.	10.2889390	50.0000000	Y
0.0002786	2141.	7682905.	10.2400041	50.0000000	Y

0.0002867	2174.	7582635.	10.1948487	50.0000000	Y
0.0002948	2207.	7485140.	10.1531823	50.0000000	Y
0.0003029	2238.	7390722.	10.1145559	50.0000000	Y
0.0003110	2270.	7299394.	10.0786516	50.0000000	Y
0.0003190	2301.	7211159.	10.0451808	50.0000000	Y
0.0003271	2331.	7125311.	10.0141774	50.0000000	Y
0.0003352	2360.	7042223.	9.9852588	50.0000000	Y
0.0003433	2390.	6962131.	9.9580967	50.0000000	Y
0.0003513	2419.	6884117.	9.9328973	50.0000000	Y
0.0003594	2447.	6808768.	9.9092193	50.0000000	Y
0.0003675	2475.	6734583.	9.8865733	50.0000000	Y
0.0003756	2501.	6659479.	9.8644946	50.0000000	Y
0.0003836	2526.	6584200.	9.8425979	50.0000000	Y
0.0003917	2549.	6507980.	9.8211367	50.0000000	Y
0.0003998	2572.	6432402.	9.7999368	50.0000000	Y
0.0004079	2593.	6357211.	9.7793214	50.0000000	Y
0.0004159	2613.	6282459.	9.7589722	50.0000000	Y
0.0004240	2632.	6208099.	9.7387285	50.0000000	Y
0.0004321	2651.	6134589.	9.7191895	50.0000000	Y
0.0004402	2668.	6062109.	9.6997209	50.0000000	Y
0.0004483	2685.	5990090.	9.6805797	50.0000000	Y
0.0004563	2701.	5919573.	9.6620360	50.0000000	Y
0.0004644	2717.	5849516.	9.6433045	50.0000000	Y
0.0004725	2731.	5780904.	9.6252131	50.0000000	Y
0.0004806	2745.	5712960.	9.6073496	50.0000000	Y
0.0005129	2797.	5453073.	9.5384924	50.0000000	Y
0.0005452	2841.	5210708.	9.4736651	50.0000000	Y
0.0005775	2879.	4985738.	9.4126605	50.0000000	Y
0.0006098	2913.	4776442.	9.3547946	50.0000000	Y
0.0006421	2942.	4582345.	9.3005953	50.0000000	Y
0.0006744	2969.	4401840.	9.2486846	50.0000000	Y
0.0007067	2992.	4233668.	9.1998396	50.0000000	Y
0.0007390	3013.	4077246.	9.1531380	50.0000000	Y
0.0007713	3032.	3931180.	9.1090266	50.0000000	Y
0.0008036	3050.	3794736.	9.0669946	50.0000000	Y
0.0008359	3066.	3667157.	9.0268187	50.0000000	Y
0.0008682	3080.	3547063.	8.9887224	50.0000000	Y
0.0009005	3093.	3434191.	8.9526051	50.0000000	Y

Summary of Results for Nominal Moment Capacity for Section 1

Load No.	Axial Thrust kips	Nominal Moment Capacity in-kips
1	522.0000000000	3093.

Note that the values in the above table are not factored by a strength reduction factor for LRFD.

The value of the strength reduction factor depends on the provisions of the LRFD code being followed.

The above values should be multiplied by the appropriate strength reduction factor to compute ultimate moment capacity according to the LRFD structural design standard being followed.

Layering Correction Equivalent Depths of Soil & Rock Layers

Layer No.	Top of Layer Below Pile Head ft	Equivalent Top Depth Below Grnd Surf ft	Same Layer Type As Layer Above	Layer is Rock or is Below Rock Layer	F0 Integral for Layer lbs	F1 Integral for Layer lbs
1	0.00	0.00	N.A.	No	0.00	208062.
2	12.7000	12.7000	Yes	No	208062.	178840.
3	16.0000	26.1215	No	No	386902.	193974.
4	27.0000	17.5689	No	No	580876.	917051.
5	34.0000	34.0000	No	Yes	N.A.	N.A.

Notes: The F0 integral of Layer n+1 equals the sum of the F0 and F1 integrals for Layer n. Layering correction equivalent depths are computed only for soil types with both shallow-depth and deep-depth expressions for peak lateral load transfer. These soil types are soft and stiff clays, non-liquefied sands, and cemented c-phi soil.

Computed Values of Pile Loading and Deflection
for Lateral Loading for Load Case Number 1

Pile-head conditions are Displacement and Pile-head Rotation (Loading Type 5)
Displacement of pile head = -0.700000 inches
Rotation of pile head = 0.000E+00 radians
Axial load on pile head = 522000.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness lb-in^2	Soil Res. p lb/inch	Soil Spr. Es*h lb/inch	Distrib. Lat. Load lb/inch
0.00	-0.7000	2395738.	-46453.	0.00	74292.	6.95E+09	0.00	0.00	0.00
0.3400	-0.6971	2205169.	-46257.	0.00135	69990.	6.95E+09	40.9313	239.5537	0.00
0.6800	-0.6890	2012523.	-45992.	0.00251	65642.	8.07E+09	89.1584	527.9842	0.00
1.0200	-0.6767	1819194.	-45523.	0.00345	61278.	8.64E+09	140.8805	849.4487	0.00

1.3600	-0.6609	1626379.	-44841.	0.00424	56925.	9.10E+09	193.2696	1193.	0.00
1.7000	-0.6421	1435229.	-43949.	0.00492	52611.	9.39E+09	243.7366	1549.	0.00
2.0400	-0.6207	1246808.	-42855.	0.00550	48358.	9.44E+09	292.7342	1924.	0.00
2.3800	-0.5972	1062112.	-41567.	0.00600	44188.	9.44E+09	338.7402	2314.	0.00
2.7200	-0.5718	882077.	-40099.	0.00642	40125.	9.44E+09	380.8070	2717.	0.00
3.0600	-0.5448	707568.	-38480.	0.00676	36186.	9.44E+09	412.8589	3092.	0.00
3.4000	-0.5166	539282.	-36738.	0.00703	32387.	9.44E+09	441.0330	3483.	0.00
3.7400	-0.4875	377840.	-34894.	0.00723	28743.	9.44E+09	462.7988	3874.	0.00
4.0800	-0.4576	223754.	-32944.	0.00736	25265.	9.44E+09	493.0296	4396.	0.00
4.4200	-0.4274	77670.	-30889.	0.00742	21967.	9.44E+09	514.5732	4912.	0.00
4.7600	-0.3971	-59920.	-28766.	0.00743	21566.	9.44E+09	525.9082	5404.	0.00
5.1000	-0.3668	-188701.	-26581.	0.00737	24473.	9.44E+09	545.0433	6063.	0.00
5.4400	-0.3369	-308235.	-24304.	0.00727	27172.	9.44E+09	571.1854	6918.	0.00
5.7800	-0.3075	-417977.	-21931.	0.00711	29649.	9.44E+09	592.0892	7856.	0.00
6.1200	-0.2789	-517478.	-19484.	0.00691	31895.	9.44E+09	607.3597	8886.	0.00
6.4600	-0.2511	-606392.	-16967.	0.00666	33902.	9.44E+09	626.8296	10183.	0.00
6.8000	-0.2245	-684313.	-14378.	0.00639	35661.	9.44E+09	642.3143	11674.	0.00
7.1400	-0.1990	-750912.	-11737.	0.00608	37164.	9.44E+09	652.2767	13371.	0.00
7.4800	-0.1749	-805962.	-9067.	0.00574	38407.	9.44E+09	656.3321	15310.	0.00
7.8200	-0.1522	-849345.	-6394.	0.00538	39386.	9.44E+09	654.2001	17537.	0.00
8.1600	-0.1310	-881055.	-3742.	0.00501	40102.	9.44E+09	645.6573	20110.	0.00
8.5000	-0.1113	-901206.	-1138.	0.00462	40556.	9.44E+09	630.5334	23105.	0.00
8.8400	-0.09328	-910032.	1390.	0.00423	40756.	9.44E+09	608.7025	26625.	0.00
9.1800	-0.07682	-907887.	3815.	0.00384	40707.	9.44E+09	580.0688	30808.	0.00
9.5200	-0.06196	-895250.	6109.	0.00345	40422.	9.44E+09	544.5402	35856.	0.00
9.8600	-0.04868	-872725.	8244.	0.00307	39914.	9.44E+09	501.9832	42068.	0.00
10.2000	-0.03695	-841040.	10190.	0.00270	39198.	9.44E+09	452.1401	49931.	0.00
10.5400	-0.02669	-801054.	11917.	0.00234	38296.	9.44E+09	394.4645	60301.	0.00
10.8800	-0.01785	-753764.	13315.	0.00200	37228.	9.44E+09	290.7913	66479.	0.00
11.2200	-0.01033	-700939.	14263.	0.00169	36036.	9.44E+09	173.6239	68557.	0.00
11.5600	-0.00406	-644580.	14760.	0.00140	34764.	9.44E+09	70.2060	70634.	0.00
11.9000	0.00109	-586458.	14864.	0.00113	33452.	9.44E+09	-19.3456	72711.	0.00
12.2400	0.00519	-528118.	14630.	8.92E-04	32135.	9.44E+09	-95.1733	74789.	0.00
12.5800	0.00837	-470876.	14114.	6.76E-04	30843.	9.44E+09	-157.6356	76866.	0.00
12.9200	0.01071	-415825.	13537.	4.85E-04	29600.	9.44E+09	-125.3875	47759.	0.00
13.2600	0.01232	-362478.	12979.	3.17E-04	28396.	9.44E+09	-148.0448	49015.	0.00
13.6000	0.01330	-311262.	12343.	1.71E-04	27240.	9.44E+09	-163.8179	50272.	0.00
13.9400	0.01372	-262487.	11655.	4.70E-05	26139.	9.44E+09	-173.2569	51529.	0.00
14.2800	0.01368	-216353.	10941.	-5.65E-05	25098.	9.44E+09	-176.9673	52786.	0.00
14.6200	0.01326	-172967.	10222.	-1.41E-04	24118.	9.44E+09	-175.5991	54043.	0.00
14.9600	0.01253	-132344.	9517.	-2.07E-04	23201.	9.44E+09	-169.8368	55299.	0.00
15.3000	0.01157	-94427.	8843.	-2.56E-04	22345.	9.44E+09	-160.3914	56556.	0.00
15.6400	0.01044	-59093.	8214.	-2.89E-04	21548.	9.44E+09	-147.9950	57813.	0.00
15.9800	0.00921	-26168.	7640.	-3.07E-04	20805.	9.44E+09	-133.3958	59070.	0.00
16.3200	0.00794	4561.	6684.	-3.12E-04	20317.	9.44E+09	-335.1477	172282.	0.00
16.6600	0.00667	29706.	5346.	-3.05E-04	20884.	9.44E+09	-320.8973	196341.	0.00
17.0000	0.00545	49482.	4069.	-2.87E-04	21331.	9.44E+09	-305.1765	228379.	0.00
17.3400	0.00432	64133.	2859.	-2.63E-04	21662.	9.44E+09	-288.0186	271833.	0.00
17.6800	0.00331	73930.	1722.	-2.33E-04	21883.	9.44E+09	-269.4113	332386.	0.00
18.0200	0.00242	79174.	663.5402	-2.00E-04	22001.	9.44E+09	-249.2780	420021.	0.00
18.3600	0.00168	80196.	-308.9493	-1.65E-04	22024.	9.44E+09	-227.4325	553816.	0.00
18.7000	0.00107	77358.	-1188.	-1.31E-04	21960.	9.44E+09	-203.4656	775093.	0.00
19.0400	6.03E-04	71062.	-1963.	-9.94E-05	21818.	9.44E+09	-176.4046	1193665.	0.00
19.3800	2.60E-04	61764.	-2588.	-7.07E-05	21608.	9.44E+09	-130.1119	2040000.	0.00

19.7200	2.64E-05	50243.	-2881.	-4.64E-05	21348.	9.44E+09	-13.2086	2040000.	0.00
20.0600	-1.19E-04	38456.	-2786.	-2.73E-05	21082.	9.44E+09	59.3898	2040000.	0.00
20.4000	-1.96E-04	27623.	-2465.	-1.30E-05	20837.	9.44E+09	98.0772	2040000.	0.00
20.7400	-2.25E-04	18396.	-2036.	-3.05E-06	20629.	9.44E+09	112.4067	2040000.	0.00
21.0800	-2.21E-04	11024.	-1581.	3.31E-06	20463.	9.44E+09	110.5144	2040000.	0.00
21.4200	-1.98E-04	5481.	-1154.	6.88E-06	20338.	9.44E+09	98.9012	2040000.	0.00
21.7600	-1.65E-04	1580.	-783.8214	8.40E-06	20250.	9.44E+09	82.4548	2040000.	0.00
22.1000	-1.29E-04	-950.7393	-483.7979	8.54E-06	20235.	9.44E+09	64.6155	2040000.	0.00
22.4400	-9.52E-05	-2405.	-254.8484	7.81E-06	20268.	9.44E+09	47.6146	2040000.	0.00
22.7800	-6.55E-05	-3064.	-90.9371	6.63E-06	20283.	9.44E+09	32.7341	2040000.	0.00
23.1200	-4.11E-05	-3175.	17.7727	5.28E-06	20286.	9.44E+09	20.5550	2040000.	0.00
23.4600	-2.24E-05	-2941.	82.5031	3.96E-06	20280.	9.44E+09	11.1756	2040000.	0.00
23.8000	-8.78E-06	-2519.	114.2562	2.78E-06	20271.	9.44E+09	4.3896	2040000.	0.00
24.1400	3.51E-07	-2021.	122.8531	1.80E-06	20259.	9.44E+09	-0.1755	2040000.	0.00
24.4800	5.92E-06	-1524.	116.4592	1.04E-06	20248.	9.44E+09	-2.9588	2040000.	0.00
24.8200	8.80E-06	-1075.	101.4504	4.73E-07	20238.	9.44E+09	-4.3985	2040000.	0.00
25.1600	9.78E-06	-697.8955	82.5007	9.04E-08	20230.	9.44E+09	-4.8905	2040000.	0.00
25.5000	9.53E-06	-401.8686	62.7991	-1.47E-07	20223.	9.44E+09	-4.7672	2040000.	0.00
25.8400	8.58E-06	-184.8277	44.3237	-2.74E-07	20218.	9.44E+09	-4.2894	2040000.	0.00
26.1800	7.30E-06	-39.0198	28.1300	-3.23E-07	20215.	9.44E+09	-3.6487	2040000.	0.00
26.5200	5.95E-06	46.0867	14.6207	-3.21E-07	20215.	9.44E+09	-2.9735	2040000.	0.00
26.8600	4.68E-06	81.6520	3.7830	-2.93E-07	20216.	9.44E+09	-2.3390	2040000.	0.00
27.2000	3.55E-06	78.2057	-1.0845	-2.59E-07	20216.	9.44E+09	-0.04697	53934.	0.00
27.5400	2.57E-06	73.9054	-1.2503	-2.26E-07	20216.	9.44E+09	-0.03434	54609.	0.00
27.8800	1.71E-06	68.9654	-1.3676	-1.95E-07	20215.	9.44E+09	-0.02316	55283.	0.00
28.2200	9.74E-07	63.5764	-1.4421	-1.66E-07	20215.	9.44E+09	-0.01336	55957.	0.00
28.5600	3.51E-07	57.9064	-1.4793	-1.40E-07	20215.	9.44E+09	-0.00487	56631.	0.00
28.9000	-1.70E-07	52.1020	-1.4844	-1.16E-07	20215.	9.44E+09	0.00238	57305.	0.00
29.2400	-5.99E-07	46.2893	-1.4622	-9.51E-08	20215.	9.44E+09	0.00851	57979.	0.00
29.5800	-9.46E-07	40.5755	-1.4171	-7.64E-08	20215.	9.44E+09	0.01360	58654.	0.00
29.9200	-1.22E-06	35.0508	-1.3531	-6.00E-08	20215.	9.44E+09	0.01776	59328.	0.00
30.2600	-1.44E-06	29.7895	-1.2738	-4.60E-08	20215.	9.44E+09	0.02111	60002.	0.00
30.6000	-1.60E-06	24.8523	-1.1823	-3.42E-08	20214.	9.44E+09	0.02375	60676.	0.00
30.9400	-1.71E-06	20.2876	-1.0813	-2.44E-08	20214.	9.44E+09	0.02578	61350.	0.00
31.2800	-1.80E-06	16.1333	-0.9729	-1.66E-08	20214.	9.44E+09	0.02731	62024.	0.00
31.6200	-1.85E-06	12.4189	-0.8592	-1.04E-08	20214.	9.44E+09	0.02843	62699.	0.00
31.9600	-1.88E-06	9.1662	-0.7416	-5.73E-09	20214.	9.44E+09	0.02922	63373.	0.00
32.3000	-1.90E-06	6.3915	-0.6213	-2.36E-09	20214.	9.44E+09	0.02977	64047.	0.00
32.6400	-1.90E-06	4.1065	-0.4991	-9.56E-11	20214.	9.44E+09	0.03015	64721.	0.00
32.9800	-1.90E-06	2.3195	-0.3755	1.29E-09	20214.	9.44E+09	0.03041	65395.	0.00
33.3200	-1.89E-06	1.0367	-0.2510	2.02E-09	20214.	9.44E+09	0.03061	66070.	0.00
33.6600	-1.88E-06	0.2624	-0.1258	2.30E-09	20214.	9.44E+09	0.03077	66744.	0.00
34.0000	-1.87E-06	0.00	0.00	2.36E-09	20214.	9.44E+09	0.03092	33709.	0.00

* This analysis computed pile response using nonlinear moment-curvature relationships. Values of total stress due to combined axial and bending stresses are computed only for elastic sections only and do not equal the actual stresses in concrete and steel. Stresses in concrete and steel may be interpolated from the output for nonlinear bending properties relative to the magnitude of bending moment developed in the pile.

Output Summary for Load Case No. 1:

Pile-head deflection = -0.70000000 inches
Computed slope at pile head = 0.000000 radians
Maximum bending moment = 2395738. inch-lbs
Maximum shear force = -46453. lbs
Depth of maximum bending moment = 0.000000 feet below pile head
Depth of maximum shear force = 0.000000 feet below pile head
Number of iterations = 11
Number of zero deflection points = 4

Summary of Pile-head Responses for Conventional Analyses

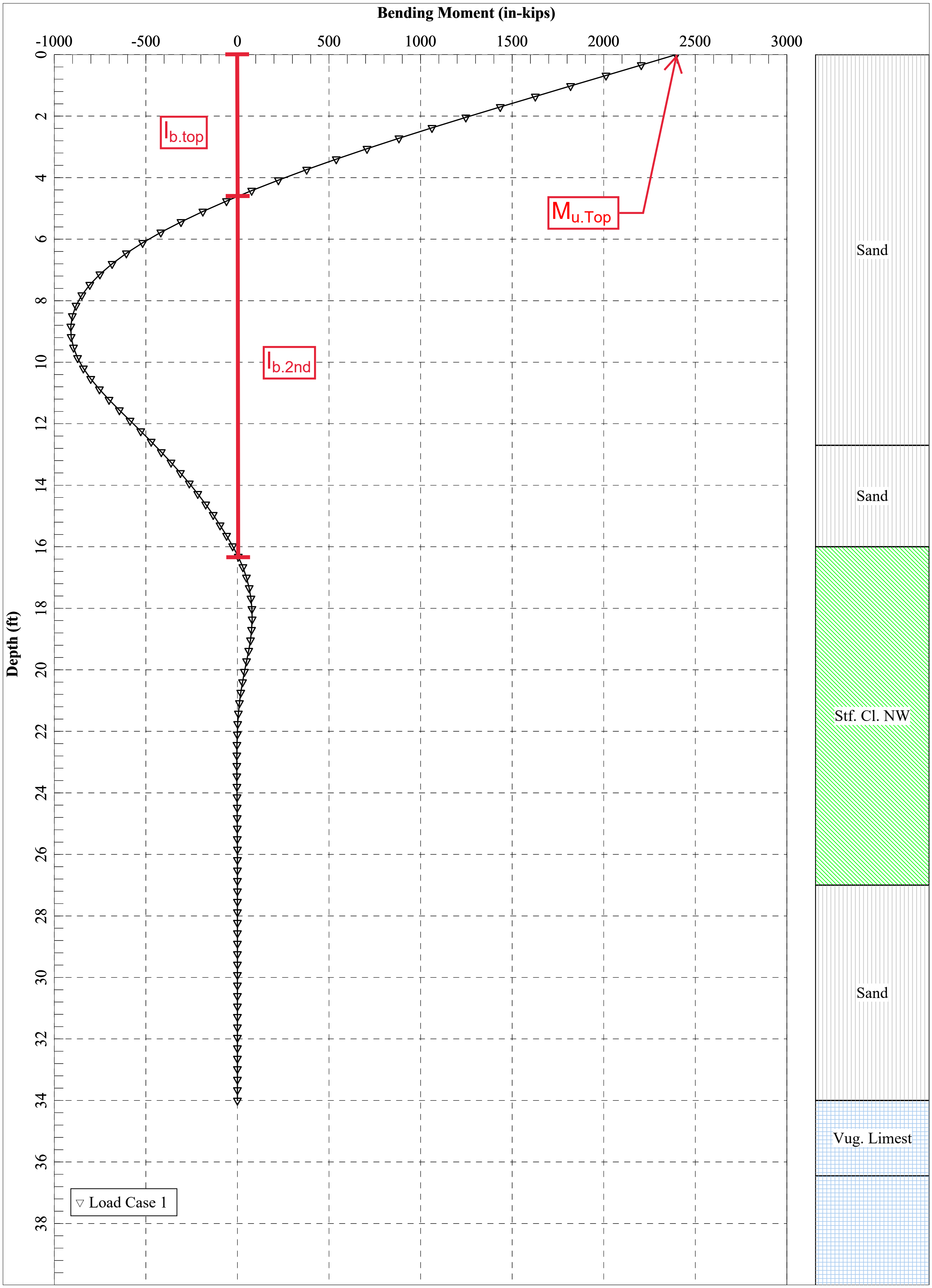
Definitions of Pile-head Loading Conditions:

Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians
Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
Load Type 5: Load 1 = Top Deflection, y, inches, and Load 2 = Slope, S, radians

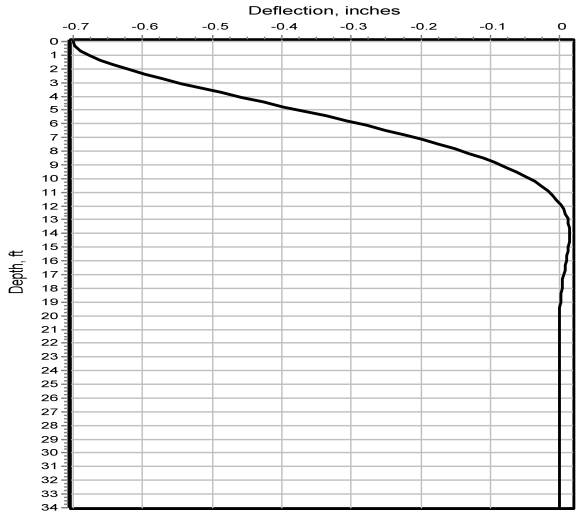
Load Case No.	Load Type 1	Pile-head Load 1	Load Type 2	Pile-head Load 2	Axial Loading lbs	Pile-head Deflection inches	Pile-head Rotation radians	Max Shear in Pile lbs	Max Moment in Pile in-lbs
1	y, in	-0.7000	S, rad	0.00	522000.	-0.7000	0.00	-46453.	2395738.

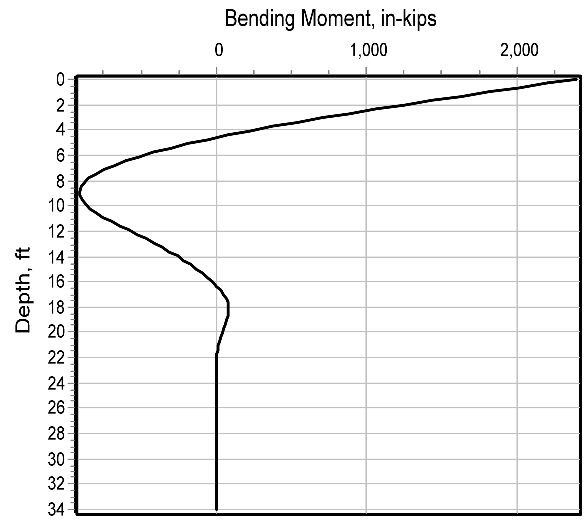
Maximum pile-head deflection = -0.7000000000 inches
Maximum pile-head rotation = 0.0000000000 radians = 0.000000 deg.

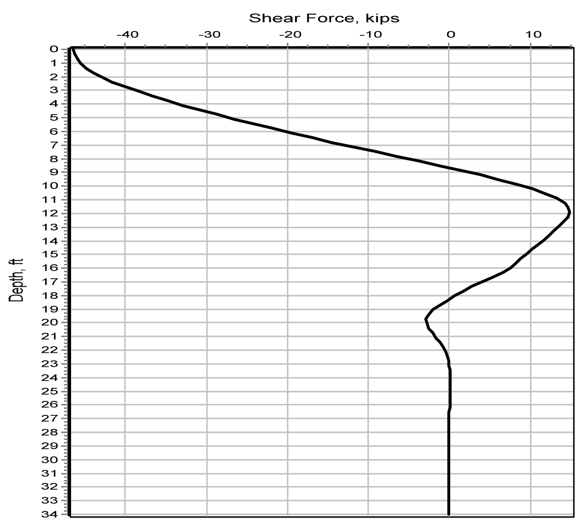
The analysis ended normally.

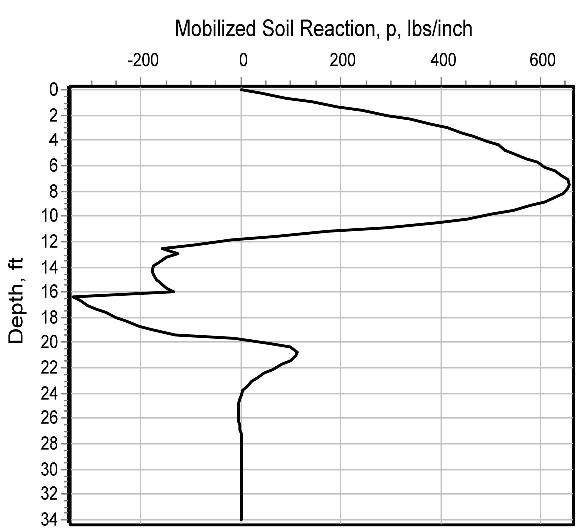


Bending Moment (in-kips)
-1000
-500
0
500
1000
1500
2000
2500
3000
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
$I_{b.top}$
$I_{b.2nd}$
$M_{u.Top}$
Load Case 1
Sand
Sand
Stf. Cl. NW
Sand
Vug. Limest


Deflection, inches
-0.7 -0.6 -0.5 -0.4 -0.3 -0.2 -0.1 0
Depth, ft


Bending Moment, in-kips
0 1,000 2,000
Depth, ft


Shear Force, kips
-40 -30 -20 -10 0 10
Depth, ft


Mobilized Soil Reaction, p, lbs/inch
-200 0 200 400 600
Depth, ft

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LPile for Windows, Version 2019-11.005

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

==

This copy of LPile is being used by:

KAR
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Files Used for Analysis

Path to file locations:
\Users\kroth\Documents\Projects\19126013 MaineDOT I-295 Freeport Exit 20 Merrill Rd Bridge\Pile Design\LPile Northwest Abutment\

Name of input data file:
Freeport Exit 20 Northwest Abutment Southern Shift _ Plastic Hinge.lp11d

Name of output report file:
Freeport Exit 20 Northwest Abutment Southern Shift _ Plastic Hinge.lp11o

Name of plot output file:
Freeport Exit 20 Northwest Abutment Southern Shift _ Plastic Hinge.lp11p

Name of runtime message file:
Freeport Exit 20 Northwest Abutment Southern Shift _ Plastic Hinge.lp11r

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Date and Time of Analysis

Date: August 26, 2020 Time: 9:40:35

Problem Title

Project Name: MaineDOT I-295 Exit 20 Merrill Road Bridge No. 5720
Job Number: 19126013
Client: MaineDOT
Engineer: KAR
Description: Northwest Abutment Pile Design - Strength I (Plastic Hinge)

Program Options and Settings

Computational Options:
- Conventional Analysis

Engineering Units Used for Data Input and Computations:
- US Customary System Units (pounds, feet, inches)

Analysis Control Options:
- Maximum number of iterations allowed = 500
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 100.0000 in
- Number of pile increments = 100

Loading Type and Number of Cycles of Loading:
- Static loading specified

- Use of p-y modification factors for p-y curves not selected
- Analysis uses layering correction (Method of Georgiadis)
- No distributed lateral loads are entered
- Loading by lateral soil movements acting on pile not selected
- Input of shear resistance at the pile tip not selected
- Input of moment resistance at the pile tip not selected
- Input of side resistance moment along pile not selected
- Computation of pile-head foundation stiffness matrix not selected
- Push-over analysis of pile not selected
- Buckling analysis of pile not selected

Output Options:
- Output files use decimal points to denote decimal symbols.
- Values of pile-head deflection, bending moment, shear force, and
  soil reaction are printed for full length of pile.
- Printing Increment (nodal spacing of output points) = 1
- No p-y curves to be computed and reported for user-specified depths
- Print using wide report formats

Pile Structural Properties and Geometry

Number of pile sections defined = 1
Total length of pile = 34.000 ft
Depth of ground surface below top of pile = 0.0000 ft

Pile diameters used for p-y curve computations are defined using 2 points.

p-y curves are computed using pile diameter values interpolated with depth over the length of the pile. A summary of values of pile diameter vs. depth follows.

Point No.	Depth Below Pile Head feet	Pile Diameter inches
1	0.000	14.6950
2	34.000	14.6950

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is a H weak axis steel pile
Length of section = 34.000000 ft
Pile width = 13.830000 in
Shear capacity of section = 0.0000 lbs

Ground Slope and Pile Batter Angles

Ground Slope Angle = 0.000 degrees
= 0.000 radians

Pile Batter Angle = 0.000 degrees
= 0.000 radians

Soil and Rock Layering Information

The soil profile is modelled using 5 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer = 0.0000 ft
Distance from top of pile to bottom of layer = 12.700000 ft

Effective unit weight at top of layer = 125.000000 pcf
Effective unit weight at bottom of layer = 125.000000 pcf
Friction angle at top of layer = 32.000000 deg.
Friction angle at bottom of layer = 32.000000 deg.
Subgrade k at top of layer = 124.800000 pci
Subgrade k at bottom of layer = 124.800000 pci

Layer 2 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer = 12.700000 ft
Distance from top of pile to bottom of layer = 16.000000 ft
Effective unit weight at top of layer = 62.600000 pcf
Effective unit weight at bottom of layer = 62.600000 pcf
Friction angle at top of layer = 32.000000 deg.
Friction angle at bottom of layer = 32.000000 deg.
Subgrade k at top of layer = 75.500000 pci
Subgrade k at bottom of layer = 75.500000 pci

Layer 3 is stiff clay without free water

Distance from top of pile to top of layer = 16.000000 ft
Distance from top of pile to bottom of layer = 27.000000 ft
Effective unit weight at top of layer = 62.600000 pcf
Effective unit weight at bottom of layer = 62.600000 pcf
Undrained cohesion at top of layer = 1600. psf
Undrained cohesion at bottom of layer = 1600. psf
Epsilon-50 at top of layer = 0.005000
Epsilon-50 at bottom of layer = 0.005000

Layer 4 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer = 27.000000 ft
Distance from top of pile to bottom of layer = 34.000000 ft
Effective unit weight at top of layer = 62.600000 pcf
Effective unit weight at bottom of layer = 62.600000 pcf
Friction angle at top of layer = 37.000000 deg.
Friction angle at bottom of layer = 37.000000 deg.
Subgrade k at top of layer = 40.500000 pci
Subgrade k at bottom of layer = 40.500000 pci

Layer 5 is strong rock (vuggy limestone)

Distance from top of pile to top of layer = 34.000000 ft
Distance from top of pile to bottom of layer = 50.000000 ft
Effective unit weight at top of layer = 101.600000 pcf
Effective unit weight at bottom of layer = 101.600000 pcf
Uniaxial compressive strength at top of layer = 12983. psi
Uniaxial compressive strength at bottom of layer = 12983. psi

(Depth of the lowest soil layer extends 16.000 ft below the pile tip)

Summary of Input Soil Properties

Layer Num.	Soil Type Name (p-y Curve Type)	Layer Depth ft	Effective Unit Wt. pcf	Undrained Cohesion psf	Angle of Friction deg.	Uniaxial qu psi	E50 or krm	kpy pci
1	Sand	0.00	125.0000	--	32.0000	--	--	124.8000
	(Reese, et al.)	12.7000	125.0000	--	32.0000	--	--	124.8000
2	Sand	12.7000	62.6000	--	32.0000	--	--	75.5000
	(Reese, et al.)	16.0000	62.6000	--	32.0000	--	--	75.5000
3	Stiff Clay	16.0000	62.6000	1600.	--	--	0.00500	--
	w/o Free Water	27.0000	62.6000	1600.	--	--	0.00500	--
4	Sand	27.0000	62.6000	--	37.0000	--	--	40.5000
	(Reese, et al.)	34.0000	62.6000	--	37.0000	--	--	40.5000
5	Strong Rock	34.0000	101.6000	--	--	12983.	--	--
	(Vuggy Limestone)	50.0000	101.6000	--	--	12983.	--	--

Static Loading Type

Static loading criteria were used when computing p-y curves for all analyses.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 2

Load No.	Load Type	Condition 1	Condition 2	Axial Thrust Force, lbs	Compute Top y vs. Pile Length	Run Analysis
1	5	y = -0.700000 in	S = 0.0000 in/in	522000.	N.A.	Yes
2	4	y = -0.700000 in	M = 1433282. in-lbs	522000.	N.A.	Yes

V = shear force applied normal to pile axis
M = bending moment applied to pile head
y = lateral deflection normal to pile axis
S = pile slope relative to original pile batter angle
R = rotational stiffness applied to pile head
Values of top y vs. pile lengths can be computed only for load types with specified shear loading (Load Types 1, 2, and 3).
Thrust force is assumed to be acting axially for all pile batter angles.

Computations of Nominal Moment Capacity and Nonlinear Bending Stiffness
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Axial thrust force values were determined from pile-head loading conditions

Number of Pile Sections Analyzed = 1

Pile Section No. 1:

Dimensions and Properties of Steel H Weak Axis:

Length of Section	=	34.000000 ft
Flange Width	=	14.695000 in
Section Depth	=	13.830000 in
Flange Thickness	=	0.615000 in
Web Thickness	=	0.615000 in
Yield Stress of Pipe	=	50.000000 ksi
Elastic Modulus	=	29000. ksi
Cross-sectional Area	=	25.823850 sq. in.
Moment of Inertia	=	325.505721 in^4
Elastic Bending Stiffness	=	9439666. kip-in^2
Plastic Modulus, Z	=	67.593889in^3
Plastic Moment Capacity = Fy Z	=	3380.in-kip

Axial Structural Capacities:

Nom. Axial Structural Capacity = Fy As	=	1291.193 kips
Nominal Axial Tensile Capacity	=	-1291.193 kips

Number of Axial Thrust Force Values Determined from Pile-head Loadings = 1

Number	Axial Thrust Force kips
1	522.000

Definition of Run Messages:

Y = part of pipe section has yielded.

Axial Thrust Force = 522.000 kips

Bending Curvature rad/in.	Bending Moment in-kip	Bending Stiffness kip-in2	Depth to N Axis in	Max Total Stress ksi	Run Msg

0.00000404	38.1169076	9438779.	179.9510245	21.0657447	
0.00000808	76.2338151	9438779.	93.6492623	21.9176173	
0.00001211	114.3507227	9438779.	64.8820082	22.7694899	
0.00001615	152.4676303	9438779.	50.4983811	23.6213625	
0.00002019	190.5845379	9438779.	41.8682049	24.4732347	
0.00002423	228.7014454	9438779.	36.1147541	25.3251077	
0.00002827	266.8183530	9438779.	32.0051464	26.1769800	
0.00003231	304.9352606	9438779.	28.9229406	27.0288528	
0.00003634	343.0521682	9438779.	26.5256694	27.8807252	
0.00004038	381.1690757	9438779.	24.6078525	28.7325980	
0.00004442	419.2859833	9438779.	23.0387295	29.5844705	
0.00004846	457.4028909	9438779.	21.7311270	30.4363432	
0.00005250	495.5197985	9438779.	20.6246942	31.2882157	
0.00005654	533.6367060	9438779.	19.6763232	32.1400884	
0.00006057	571.7536136	9438779.	18.8544016	32.9919609	
0.00006461	609.8705212	9438779.	18.1352203	33.8438336	
0.00006865	647.9874287	9438779.	17.5006485	34.6957061	
0.00007269	686.1043363	9438779.	16.9365847	35.5475788	
0.00007673	724.2212439	9438779.	16.4318960	36.3994513	
0.00008077	762.3381515	9438779.	15.9776762	37.2513240	
0.00008480	800.4550590	9438779.	15.5667155	38.1031965	
0.00008884	838.5719666	9438779.	15.1931148	38.9550692	
0.00009288	876.6888742	9438779.	14.8520011	39.8069417	
0.00009692	914.8057818	9438779.	14.5393135	40.6588144	
0.0001010	952.9226893	9438779.	14.2516410	41.5106869	
0.0001050	991.0395969	9438779.	13.9860971	42.3625596	
0.0001090	1029.	9438779.	13.7402231	43.2144321	
0.0001131	1067.	9438779.	13.5119116	44.0663048	
0.0001171	1105.	9438779.	13.2993457	44.9181773	
0.0001211	1144.	9438779.	13.1009508	45.7700500	
0.0001252	1182.	9438779.	12.9153556	46.6219225	
0.0001292	1220.	9438779.	12.7413601	47.4737952	
0.0001333	1258.	9438779.	12.5779098	48.3256677	
0.0001373	1296.	9438779.	12.4240743	49.1775404	
0.0001413	1334.	9438503.	12.2790799	50.0000000	Y
0.0001454	1371.	9428344.	12.1439669	50.0000000	Y
0.0001494	1406.	9409586.	12.0178871	50.0000000	Y
0.0001535	1440.	9383244.	11.9001053	50.0000000	Y
0.0001575	1472.	9349477.	11.7901137	50.0000000	Y
0.0001656	1534.	9266985.	11.5903389	50.0000000	Y
0.0001736	1592.	9169338.	11.4140577	50.0000000	Y
0.0001817	1647.	9061268.	11.2577541	50.0000000	Y
0.0001898	1698.	8946572.	11.1184873	50.0000000	Y
0.0001979	1747.	8828299.	10.9937734	50.0000000	Y
0.0002060	1793.	8707082.	10.8819826	50.0000000	Y
0.0002140	1838.	8586292.	10.7809631	50.0000000	Y
0.0002221	1880.	8465251.	10.6898028	50.0000000	Y
0.0002302	1921.	8346099.	10.6069483	50.0000000	Y
0.0002383	1961.	8229110.	10.5314770	50.0000000	Y
0.0002463	1999.	8114243.	10.4626706	50.0000000	Y
0.0002544	2036.	8001908.	10.3997576	50.0000000	Y
0.0002625	2072.	7892519.	10.3420286	50.0000000	Y
0.0002706	2107.	7786163.	10.2889390	50.0000000	Y

0.0002786	2141.	7682905.	10.2400041	50.0000000	Y
0.0002867	2174.	7582635.	10.1948487	50.0000000	Y
0.0002948	2207.	7485140.	10.1531823	50.0000000	Y
0.0003029	2238.	7390722.	10.1145559	50.0000000	Y
0.0003110	2270.	7299394.	10.0786516	50.0000000	Y
0.0003190	2301.	7211159.	10.0451808	50.0000000	Y
0.0003271	2331.	7125311.	10.0141774	50.0000000	Y
0.0003352	2360.	7042223.	9.9852588	50.0000000	Y
0.0003433	2390.	6962131.	9.9580967	50.0000000	Y
0.0003513	2419.	6884117.	9.9328973	50.0000000	Y
0.0003594	2447.	6808768.	9.9092193	50.0000000	Y
0.0003675	2475.	6734583.	9.8865733	50.0000000	Y
0.0003756	2501.	6659479.	9.8644946	50.0000000	Y
0.0003836	2526.	6584200.	9.8425979	50.0000000	Y
0.0003917	2549.	6507980.	9.8211367	50.0000000	Y
0.0003998	2572.	6432402.	9.7999368	50.0000000	Y
0.0004079	2593.	6357211.	9.7793214	50.0000000	Y
0.0004159	2613.	6282459.	9.7589722	50.0000000	Y
0.0004240	2632.	6208099.	9.7387285	50.0000000	Y
0.0004321	2651.	6134589.	9.7191895	50.0000000	Y
0.0004402	2668.	6062109.	9.6997209	50.0000000	Y
0.0004483	2685.	5990090.	9.6805797	50.0000000	Y
0.0004563	2701.	5919573.	9.6620360	50.0000000	Y
0.0004644	2717.	5849516.	9.6433045	50.0000000	Y
0.0004725	2731.	5780904.	9.6252131	50.0000000	Y
0.0004806	2745.	5712960.	9.6073496	50.0000000	Y
0.0005129	2797.	5453073.	9.5384924	50.0000000	Y
0.0005452	2841.	5210708.	9.4736651	50.0000000	Y
0.0005775	2879.	4985738.	9.4126605	50.0000000	Y
0.0006098	2913.	4776442.	9.3547946	50.0000000	Y
0.0006421	2942.	4582345.	9.3005953	50.0000000	Y
0.0006744	2969.	4401840.	9.2486846	50.0000000	Y
0.0007067	2992.	4233668.	9.1998396	50.0000000	Y
0.0007390	3013.	4077246.	9.1531380	50.0000000	Y
0.0007713	3032.	3931180.	9.1090266	50.0000000	Y
0.0008036	3050.	3794736.	9.0669946	50.0000000	Y
0.0008359	3066.	3667157.	9.0268187	50.0000000	Y
0.0008682	3080.	3547063.	8.9887224	50.0000000	Y
0.0009005	3093.	3434191.	8.9526051	50.0000000	Y

Summary of Results for Nominal Moment Capacity for Section 1

Load No.	Axial Thrust kips	Nominal Moment Capacity in-kips
1	522.0000000000	3093.

Note that the values in the above table are not factored by a strength reduction factor for LRFD.

The value of the strength reduction factor depends on the provisions of the LRFD code being followed.

The above values should be multiplied by the appropriate strength reduction factor to compute ultimate moment capacity according to the LRFD structural design standard being followed.

Layering Correction Equivalent Depths of Soil & Rock Layers

Layer No.	Top of Layer Below Pile Head ft	Equivalent Top Depth Below Grnd Surf ft	Same Layer Type As Layer Above	Layer is Rock or is Below Rock Layer	F0 Integral for Layer lbs	F1 Integral for Layer lbs
1	0.00	0.00	N.A.	No	0.00	208062.
2	12.7000	12.7000	Yes	No	208062.	178840.
3	16.0000	26.1215	No	No	386902.	193974.
4	27.0000	17.5689	No	No	580876.	917051.
5	34.0000	34.0000	No	Yes	N.A.	N.A.

Notes: The F0 integral of Layer n+1 equals the sum of the F0 and F1 integrals for Layer n. Layering correction equivalent depths are computed only for soil types with both shallow-depth and deep-depth expressions for peak lateral load transfer. These soil types are soft and stiff clays, non-liquefied sands, and cemented c-phi soil.

Computed Values of Pile Loading and Deflection
for Lateral Loading for Load Case Number 1

Pile-head conditions are Displacement and Pile-head Rotation (Loading Type 5)
Displacement of pile head = -0.700000 inches
Rotation of pile head = 0.000E+00 radians
Axial load on pile head = 522000.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness lb-in^2	Soil Res. p lb/inch	Soil Spr. Es*h lb/inch	Distrib. Lat. Load lb/inch
0.00	-0.7000	2395738.	-46453.	0.00	74292.	6.95E+09	0.00	0.00	0.00
0.3400	-0.6971	2205169.	-46257.	0.00135	69990.	6.95E+09	40.9313	239.5537	0.00
0.6800	-0.6890	2012523.	-45992.	0.00251	65642.	8.07E+09	89.1584	527.9842	0.00

1.0200	-0.6767	1819194.	-45523.	0.00345	61278.	8.64E+09	140.8805	849.4487	0.00
1.3600	-0.6609	1626379.	-44841.	0.00424	56925.	9.10E+09	193.2696	1193.	0.00
1.7000	-0.6421	1435229.	-43949.	0.00492	52611.	9.39E+09	243.7366	1549.	0.00
2.0400	-0.6207	1246808.	-42855.	0.00550	48358.	9.44E+09	292.7342	1924.	0.00
2.3800	-0.5972	1062112.	-41567.	0.00600	44188.	9.44E+09	338.7402	2314.	0.00
2.7200	-0.5718	882077.	-40099.	0.00642	40125.	9.44E+09	380.8070	2717.	0.00
3.0600	-0.5448	707568.	-38480.	0.00676	36186.	9.44E+09	412.8589	3092.	0.00
3.4000	-0.5166	539282.	-36738.	0.00703	32387.	9.44E+09	441.0330	3483.	0.00
3.7400	-0.4875	377840.	-34894.	0.00723	28743.	9.44E+09	462.7988	3874.	0.00
4.0800	-0.4576	223754.	-32944.	0.00736	25265.	9.44E+09	493.0296	4396.	0.00
4.4200	-0.4274	77670.	-30889.	0.00742	21967.	9.44E+09	514.5732	4912.	0.00
4.7600	-0.3971	-59920.	-28766.	0.00743	21566.	9.44E+09	525.9082	5404.	0.00
5.1000	-0.3668	-188701.	-26581.	0.00737	24473.	9.44E+09	545.0433	6063.	0.00
5.4400	-0.3369	-308235.	-24304.	0.00727	27172.	9.44E+09	571.1854	6918.	0.00
5.7800	-0.3075	-417977.	-21931.	0.00711	29649.	9.44E+09	592.0892	7856.	0.00
6.1200	-0.2789	-517478.	-19484.	0.00691	31895.	9.44E+09	607.3597	8886.	0.00
6.4600	-0.2511	-606392.	-16967.	0.00666	33902.	9.44E+09	626.8296	10183.	0.00
6.8000	-0.2245	-684313.	-14378.	0.00639	35661.	9.44E+09	642.3143	11674.	0.00
7.1400	-0.1990	-750912.	-11737.	0.00608	37164.	9.44E+09	652.2767	13371.	0.00
7.4800	-0.1749	-805962.	-9067.	0.00574	38407.	9.44E+09	656.3321	15310.	0.00
7.8200	-0.1522	-849345.	-6394.	0.00538	39386.	9.44E+09	654.2001	17537.	0.00
8.1600	-0.1310	-881055.	-3742.	0.00501	40102.	9.44E+09	645.6573	20110.	0.00
8.5000	-0.1113	-901206.	-1138.	0.00462	40556.	9.44E+09	630.5334	23105.	0.00
8.8400	-0.09328	-910032.	1390.	0.00423	40756.	9.44E+09	608.7025	26625.	0.00
9.1800	-0.07682	-907887.	3815.	0.00384	40707.	9.44E+09	580.0688	30808.	0.00
9.5200	-0.06196	-895250.	6109.	0.00345	40422.	9.44E+09	544.5402	35856.	0.00
9.8600	-0.04868	-872725.	8244.	0.00307	39914.	9.44E+09	501.9832	42068.	0.00
10.2000	-0.03695	-841040.	10190.	0.00270	39198.	9.44E+09	452.1401	49931.	0.00
10.5400	-0.02669	-801054.	11917.	0.00234	38296.	9.44E+09	394.4645	60301.	0.00
10.8800	-0.01785	-753764.	13315.	0.00200	37228.	9.44E+09	290.7913	66479.	0.00
11.2200	-0.01033	-700939.	14263.	0.00169	36036.	9.44E+09	173.6239	68557.	0.00
11.5600	-0.00406	-644580.	14760.	0.00140	34764.	9.44E+09	70.2060	70634.	0.00
11.9000	0.00109	-586458.	14864.	0.00113	33452.	9.44E+09	-19.3456	72711.	0.00
12.2400	0.00519	-528118.	14630.	8.92E-04	32135.	9.44E+09	-95.1733	74789.	0.00
12.5800	0.00837	-470876.	14114.	6.76E-04	30843.	9.44E+09	-157.6356	76866.	0.00
12.9200	0.01071	-415825.	13537.	4.85E-04	29600.	9.44E+09	-125.3875	47759.	0.00
13.2600	0.01232	-362478.	12979.	3.17E-04	28396.	9.44E+09	-148.0448	49015.	0.00
13.6000	0.01330	-311262.	12343.	1.71E-04	27240.	9.44E+09	-163.8179	50272.	0.00
13.9400	0.01372	-262487.	11655.	4.70E-05	26139.	9.44E+09	-173.2569	51529.	0.00
14.2800	0.01368	-216353.	10941.	-5.65E-05	25098.	9.44E+09	-176.9673	52786.	0.00
14.6200	0.01326	-172967.	10222.	-1.41E-04	24118.	9.44E+09	-175.5991	54043.	0.00
14.9600	0.01253	-132344.	9517.	-2.07E-04	23201.	9.44E+09	-169.8368	55299.	0.00
15.3000	0.01157	-94427.	8843.	-2.56E-04	22345.	9.44E+09	-160.3914	56556.	0.00
15.6400	0.01044	-59093.	8214.	-2.89E-04	21548.	9.44E+09	-147.9950	57813.	0.00
15.9800	0.00921	-26168.	7640.	-3.07E-04	20805.	9.44E+09	-133.3958	59070.	0.00
16.3200	0.00794	4561.	6684.	-3.12E-04	20317.	9.44E+09	-335.1477	172282.	0.00
16.6600	0.00667	29706.	5346.	-3.05E-04	20884.	9.44E+09	-320.8973	196341.	0.00
17.0000	0.00545	49482.	4069.	-2.87E-04	21331.	9.44E+09	-305.1765	228379.	0.00
17.3400	0.00432	64133.	2859.	-2.63E-04	21662.	9.44E+09	-288.0186	271833.	0.00
17.6800	0.00331	73930.	1722.	-2.33E-04	21883.	9.44E+09	-269.4113	332386.	0.00
18.0200	0.00242	79174.	663.5402	-2.00E-04	22001.	9.44E+09	-249.2780	420021.	0.00
18.3600	0.00168	80196.	-308.9493	-1.65E-04	22024.	9.44E+09	-227.4325	553816.	0.00
18.7000	0.00107	77358.	-1188.	-1.31E-04	21960.	9.44E+09	-203.4656	775093.	0.00
19.0400	6.03E-04	71062.	-1963.	-9.94E-05	21818.	9.44E+09	-176.4046	1193665.	0.00

19.3800	2.60E-04	61764.	-2588.	-7.07E-05	21608.	9.44E+09	-130.1119	2040000.	0.00
19.7200	2.64E-05	50243.	-2881.	-4.64E-05	21348.	9.44E+09	-13.2086	2040000.	0.00
20.0600	-1.19E-04	38456.	-2786.	-2.73E-05	21082.	9.44E+09	59.3898	2040000.	0.00
20.4000	-1.96E-04	27623.	-2465.	-1.30E-05	20837.	9.44E+09	98.0772	2040000.	0.00
20.7400	-2.25E-04	18396.	-2036.	-3.05E-06	20629.	9.44E+09	112.4067	2040000.	0.00
21.0800	-2.21E-04	11024.	-1581.	3.31E-06	20463.	9.44E+09	110.5144	2040000.	0.00
21.4200	-1.98E-04	5481.	-1154.	6.88E-06	20338.	9.44E+09	98.9012	2040000.	0.00
21.7600	-1.65E-04	1580.	-783.8214	8.40E-06	20250.	9.44E+09	82.4548	2040000.	0.00
22.1000	-1.29E-04	-950.7393	-483.7979	8.54E-06	20235.	9.44E+09	64.6155	2040000.	0.00
22.4400	-9.52E-05	-2405.	-254.8484	7.81E-06	20268.	9.44E+09	47.6146	2040000.	0.00
22.7800	-6.55E-05	-3064.	-90.9371	6.63E-06	20283.	9.44E+09	32.7341	2040000.	0.00
23.1200	-4.11E-05	-3175.	17.7727	5.28E-06	20286.	9.44E+09	20.5550	2040000.	0.00
23.4600	-2.24E-05	-2941.	82.5031	3.96E-06	20280.	9.44E+09	11.1756	2040000.	0.00
23.8000	-8.78E-06	-2519.	114.2562	2.78E-06	20271.	9.44E+09	4.3896	2040000.	0.00
24.1400	3.51E-07	-2021.	122.8531	1.80E-06	20259.	9.44E+09	-0.1755	2040000.	0.00
24.4800	5.92E-06	-1524.	116.4592	1.04E-06	20248.	9.44E+09	-2.9588	2040000.	0.00
24.8200	8.80E-06	-1075.	101.4504	4.73E-07	20238.	9.44E+09	-4.3985	2040000.	0.00
25.1600	9.78E-06	-697.8955	82.5007	9.04E-08	20230.	9.44E+09	-4.8905	2040000.	0.00
25.5000	9.53E-06	-401.8686	62.7991	-1.47E-07	20223.	9.44E+09	-4.7672	2040000.	0.00
25.8400	8.58E-06	-184.8277	44.3237	-2.74E-07	20218.	9.44E+09	-4.2894	2040000.	0.00
26.1800	7.30E-06	-39.0198	28.1300	-3.23E-07	20215.	9.44E+09	-3.6487	2040000.	0.00
26.5200	5.95E-06	46.0867	14.6207	-3.21E-07	20215.	9.44E+09	-2.9735	2040000.	0.00
26.8600	4.68E-06	81.6520	3.7830	-2.93E-07	20216.	9.44E+09	-2.3390	2040000.	0.00
27.2000	3.55E-06	78.2057	-1.0845	-2.59E-07	20216.	9.44E+09	-0.04697	53934.	0.00
27.5400	2.57E-06	73.9054	-1.2503	-2.26E-07	20216.	9.44E+09	-0.03434	54609.	0.00
27.8800	1.71E-06	68.9654	-1.3676	-1.95E-07	20215.	9.44E+09	-0.02316	55283.	0.00
28.2200	9.74E-07	63.5764	-1.4421	-1.66E-07	20215.	9.44E+09	-0.01336	55957.	0.00
28.5600	3.51E-07	57.9064	-1.4793	-1.40E-07	20215.	9.44E+09	-0.00487	56631.	0.00
28.9000	-1.70E-07	52.1020	-1.4844	-1.16E-07	20215.	9.44E+09	0.00238	57305.	0.00
29.2400	-5.99E-07	46.2893	-1.4622	-9.51E-08	20215.	9.44E+09	0.00851	57979.	0.00
29.5800	-9.46E-07	40.5755	-1.4171	-7.64E-08	20215.	9.44E+09	0.01360	58654.	0.00
29.9200	-1.22E-06	35.0508	-1.3531	-6.00E-08	20215.	9.44E+09	0.01776	59328.	0.00
30.2600	-1.44E-06	29.7895	-1.2738	-4.60E-08	20215.	9.44E+09	0.02111	60002.	0.00
30.6000	-1.60E-06	24.8523	-1.1823	-3.42E-08	20214.	9.44E+09	0.02375	60676.	0.00
30.9400	-1.71E-06	20.2876	-1.0813	-2.44E-08	20214.	9.44E+09	0.02578	61350.	0.00
31.2800	-1.80E-06	16.1333	-0.9729	-1.66E-08	20214.	9.44E+09	0.02731	62024.	0.00
31.6200	-1.85E-06	12.4189	-0.8592	-1.04E-08	20214.	9.44E+09	0.02843	62699.	0.00
31.9600	-1.88E-06	9.1662	-0.7416	-5.73E-09	20214.	9.44E+09	0.02922	63373.	0.00
32.3000	-1.90E-06	6.3915	-0.6213	-2.36E-09	20214.	9.44E+09	0.02977	64047.	0.00
32.6400	-1.90E-06	4.1065	-0.4991	-9.56E-11	20214.	9.44E+09	0.03015	64721.	0.00
32.9800	-1.90E-06	2.3195	-0.3755	1.29E-09	20214.	9.44E+09	0.03041	65395.	0.00
33.3200	-1.89E-06	1.0367	-0.2510	2.02E-09	20214.	9.44E+09	0.03061	66070.	0.00
33.6600	-1.88E-06	0.2624	-0.1258	2.30E-09	20214.	9.44E+09	0.03077	66744.	0.00
34.0000	-1.87E-06	0.00	0.00	2.36E-09	20214.	9.44E+09	0.03092	33709.	0.00

* This analysis computed pile response using nonlinear moment-curvature relationships. Values of total stress due to combined axial and bending stresses are computed only for elastic sections only and do not equal the actual stresses in concrete and steel. Stresses in concrete and steel may be interpolated from the output for nonlinear bending properties relative to the magnitude of bending moment developed in the pile.

Output Summary for Load Case No. 1:

Pile-head deflection = -0.70000000 inches
Computed slope at pile head = 0.000000 radians
Maximum bending moment = 2395738. inch-lbs
Maximum shear force = -46453. lbs
Depth of maximum bending moment = 0.000000 feet below pile head
Depth of maximum shear force = 0.000000 feet below pile head
Number of iterations = 11
Number of zero deflection points = 4

Computed Values of Pile Loading and Deflection
for Lateral Loading for Load Case Number 2

Pile-head conditions are Displacement and Moment (Loading Type 4)
Displacement of pile head = -0.700000 inches
Moment at pile head = 1433282.0 in-lbs
Axial load at pile head = 522000.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness lb-in^2	Soil Res. p lb/inch	Soil Spr. Es*h lb/inch	Distrib. Lat. Load lb/inch
0.00	-0.7000	1433282.	-34682.	0.00440	52567.	9.39E+09	0.00	0.00	0.00
0.3400	-0.6808	1281756.	-34598.	0.00499	49146.	9.39E+09	40.9310	245.2990	0.00
0.6800	-0.6593	1129724.	-34333.	0.00551	45715.	9.44E+09	89.1575	551.7269	0.00
1.0200	-0.6358	978137.	-33863.	0.00596	42293.	9.44E+09	140.8786	903.9701	0.00
1.3600	-0.6106	827994.	-33182.	0.00635	38904.	9.44E+09	193.2667	1291.	0.00
1.7000	-0.5840	680306.	-32290.	0.00668	35570.	9.44E+09	243.7326	1703.	0.00
2.0400	-0.5561	536049.	-31196.	0.00694	32314.	9.44E+09	292.7288	2148.	0.00
2.3800	-0.5273	396172.	-29923.	0.00714	29156.	9.44E+09	331.1499	2562.	0.00
2.7200	-0.4978	261442.	-28511.	0.00729	26115.	9.44E+09	361.2951	2961.	0.00
3.0600	-0.4679	132486.	-26995.	0.00737	23204.	9.44E+09	381.4464	3326.	0.00
3.4000	-0.4377	9757.	-25394.	0.00740	20434.	9.44E+09	403.7704	3764.	0.00
3.7400	-0.4075	-106259.	-23714.	0.00738	22612.	9.44E+09	419.4012	4200.	0.00
4.0800	-0.3774	-215196.	-21958.	0.00731	25071.	9.44E+09	441.7394	4775.	0.00
4.4200	-0.3478	-316581.	-20128.	0.00720	27360.	9.44E+09	455.1855	5340.	0.00
4.7600	-0.3187	-410097.	-18264.	0.00704	29471.	9.44E+09	458.3898	5868.	0.00
5.1000	-0.2903	-495606.	-16371.	0.00684	31401.	9.44E+09	469.8717	6603.	0.00
5.4400	-0.2629	-572837.	-14414.	0.00661	33144.	9.44E+09	488.9909	7590.	0.00
5.7800	-0.2364	-641400.	-12390.	0.00635	34692.	9.44E+09	503.3826	8689.	0.00
6.1200	-0.2110	-700993.	-10319.	0.00606	36037.	9.44E+09	511.6675	9893.	0.00
6.4600	-0.1869	-751424.	-8207.	0.00575	37175.	9.44E+09	523.7464	11432.	0.00
6.8000	-0.1641	-792444.	-6056.	0.00541	38101.	9.44E+09	530.9484	13199.	0.00
7.1400	-0.1427	-823897.	-3885.	0.00506	38811.	9.44E+09	532.8932	15232.	0.00
7.4800	-0.1228	-845720.	-1718.	0.00470	39304.	9.44E+09	529.3317	17586.	0.00
7.8200	-0.1044	-857953.	422.4263	0.00434	39580.	9.44E+09	520.0611	20332.	0.00
8.1600	-0.08743	-860739.	2513.	0.00396	39643.	9.44E+09	504.9178	23563.	0.00
8.5000	-0.07201	-854328.	4530.	0.00359	39498.	9.44E+09	483.7653	27408.	0.00
8.8400	-0.05811	-839077.	6448.	0.00323	39154.	9.44E+09	456.4718	32051.	0.00
9.1800	-0.04568	-815455.	8242.	0.00287	38621.	9.44E+09	422.8715	37768.	0.00

9.5200	-0.03469	-784043.	9886.	0.00252	37912.	9.44E+09	382.6927	45006.	0.00
9.8600	-0.02509	-745539.	11351.	0.00219	37043.	9.44E+09	335.4149	54551.	0.00
10.2000	-0.01680	-700765.	12558.	0.00188	36032.	9.44E+09	256.5594	62324.	0.00
10.5400	-0.00974	-651075.	13395.	0.00159	34910.	9.44E+09	153.7460	64402.	0.00
10.8800	-0.00383	-598227.	13836.	0.00132	33717.	9.44E+09	62.4572	66479.	0.00
11.2200	0.00102	-543788.	13929.	0.00107	32489.	9.44E+09	-17.1192	68557.	0.00
11.5600	0.00491	-489133.	13720.	8.48E-04	31255.	9.44E+09	-85.0336	70634.	0.00
11.9000	0.00794	-435444.	13258.	6.49E-04	30043.	9.44E+09	-141.5390	72711.	0.00
12.2400	0.01020	-383710.	12588.	4.72E-04	28875.	9.44E+09	-187.0532	74789.	0.00
12.5800	0.01179	-334736.	11753.	3.16E-04	27770.	9.44E+09	-222.1220	76866.	0.00
12.9200	0.01279	-289152.	10995.	1.81E-04	26741.	9.44E+09	-149.6588	47759.	0.00
13.2600	0.01327	-245793.	10364.	6.58E-05	25762.	9.44E+09	-159.4277	49015.	0.00
13.6000	0.01332	-204861.	9704.	-3.16E-05	24838.	9.44E+09	-164.1544	50272.	0.00
13.9400	0.01301	-166473.	9034.	-1.12E-04	23972.	9.44E+09	-164.3500	51529.	0.00
14.2800	0.01241	-130668.	8371.	-1.76E-04	23163.	9.44E+09	-160.5565	52786.	0.00
14.6200	0.01158	-97416.	7731.	-2.25E-04	22413.	9.44E+09	-153.3390	54043.	0.00
14.9600	0.01057	-66626.	7126.	-2.61E-04	21718.	9.44E+09	-143.2794	55299.	0.00
15.3000	0.00945	-38160.	6566.	-2.83E-04	21075.	9.44E+09	-130.9717	56556.	0.00
15.6400	0.00826	-11839.	6060.	-2.94E-04	20481.	9.44E+09	-117.0186	57813.	0.00
15.9800	0.00705	12545.	5613.	-2.94E-04	20497.	9.44E+09	-102.0300	59070.	0.00
16.3200	0.00586	35219.	4772.	-2.84E-04	21009.	9.44E+09	-310.4978	216240.	0.00
16.6600	0.00473	52691.	3538.	-2.65E-04	21403.	9.44E+09	-294.3510	253810.	0.00
17.0000	0.00370	65216.	2373.	-2.39E-04	21686.	9.44E+09	-276.7564	305356.	0.00
17.3400	0.00278	73073.	1283.	-2.09E-04	21863.	9.44E+09	-257.6812	378306.	0.00
17.6800	0.00199	76574.	273.4301	-1.77E-04	21942.	9.44E+09	-237.0130	486147.	0.00
18.0200	0.00133	76059.	-647.6425	-1.44E-04	21931.	9.44E+09	-214.4932	655896.	0.00
18.3600	8.14E-04	71903.	-1472.	-1.12E-04	21837.	9.44E+09	-189.5359	950582.	0.00
18.7000	4.20E-04	64526.	-2186.	-8.26E-05	21670.	9.44E+09	-160.6191	1561892.	0.00
19.0400	1.39E-04	54415.	-2656.	-5.69E-05	21442.	9.44E+09	-69.7169	2040000.	0.00
19.3800	-4.47E-05	43095.	-2753.	-3.58E-05	21187.	9.44E+09	22.3680	2040000.	0.00
19.7200	-1.53E-04	32106.	-2551.	-1.96E-05	20939.	9.44E+09	76.4518	2040000.	0.00
20.0600	-2.04E-04	22361.	-2187.	-7.80E-06	20719.	9.44E+09	102.2239	2040000.	0.00
20.4000	-2.17E-04	14297.	-1757.	1.23E-07	20537.	9.44E+09	108.2776	2040000.	0.00
20.7400	-2.03E-04	8022.	-1329.	4.95E-06	20395.	9.44E+09	101.7239	2040000.	0.00
21.0800	-1.76E-04	3434.	-941.5062	7.42E-06	20291.	9.44E+09	88.0958	2040000.	0.00
21.4200	-1.43E-04	308.1693	-616.0531	8.23E-06	20221.	9.44E+09	71.4400	2040000.	0.00
21.7600	-1.09E-04	-1628.	-359.1103	7.95E-06	20251.	9.44E+09	54.5124	2040000.	0.00
22.1000	-7.80E-05	-2656.	-168.3028	7.02E-06	20274.	9.44E+09	39.0207	2040000.	0.00
22.4400	-5.17E-05	-3032.	-35.9234	5.79E-06	20282.	9.44E+09	25.8712	2040000.	0.00
22.7800	-3.08E-05	-2974.	48.2594	4.49E-06	20281.	9.44E+09	15.3949	2040000.	0.00
23.1200	-1.51E-05	-2657.	95.0487	3.28E-06	20274.	9.44E+09	7.5410	2040000.	0.00
23.4600	-4.06E-06	-2212.	114.5737	2.22E-06	20264.	9.44E+09	2.0301	2040000.	0.00
23.8000	3.06E-06	-1732.	115.5934	1.37E-06	20253.	9.44E+09	-1.5302	2040000.	0.00
24.1400	7.13E-06	-1275.	105.2023	7.21E-07	20243.	9.44E+09	-3.5635	2040000.	0.00
24.4800	8.95E-06	-876.1533	88.8082	2.56E-07	20234.	9.44E+09	-4.4728	2040000.	0.00
24.8200	9.22E-06	-551.1866	70.2804	-5.21E-08	20226.	9.44E+09	-4.6094	2040000.	0.00
25.1600	8.52E-06	-302.4430	52.1866	-2.37E-07	20221.	9.44E+09	-4.2601	2040000.	0.00
25.5000	7.29E-06	-124.3357	36.0624	-3.29E-07	20217.	9.44E+09	-3.6440	2040000.	0.00
25.8400	5.84E-06	-6.7728	22.6755	-3.57E-07	20214.	9.44E+09	-2.9182	2040000.	0.00
26.1800	4.37E-06	62.2180	12.2617	-3.45E-07	20215.	9.44E+09	-2.1865	2040000.	0.00
26.5200	3.02E-06	94.7533	4.7213	-3.11E-07	20216.	9.44E+09	-1.5097	2040000.	0.00
26.8600	1.83E-06	102.0701	-0.2280	-2.69E-07	20216.	9.44E+09	-0.9164	2040000.	0.00
27.2000	8.26E-07	94.0375	-2.1198	-2.26E-07	20216.	9.44E+09	-0.01092	53934.	0.00
27.5400	-1.44E-08	85.7364	-2.1417	-1.88E-07	20216.	9.44E+09	1.92E-04	54609.	0.00

27.8800	-7.04E-07	77.3597	-2.1219	-1.52E-07	20216.	9.44E+09	0.00954	55283.	0.00
28.2200	-1.26E-06	69.0704	-2.0673	-1.21E-07	20215.	9.44E+09	0.01724	55957.	0.00
28.5600	-1.69E-06	61.0046	-1.9843	-9.25E-08	20215.	9.44E+09	0.02343	56631.	0.00
28.9000	-2.01E-06	53.2727	-1.8789	-6.78E-08	20215.	9.44E+09	0.02826	57305.	0.00
29.2400	-2.24E-06	45.9620	-1.7562	-4.64E-08	20215.	9.44E+09	0.03185	57979.	0.00
29.5800	-2.39E-06	39.1393	-1.6211	-2.80E-08	20215.	9.44E+09	0.03436	58654.	0.00
29.9200	-2.47E-06	32.8526	-1.4778	-1.24E-08	20215.	9.44E+09	0.03591	59328.	0.00
30.2600	-2.49E-06	27.1334	-1.3298	5.51E-10	20214.	9.44E+09	0.03664	60002.	0.00
30.6000	-2.47E-06	21.9992	-1.1802	1.12E-08	20214.	9.44E+09	0.03666	60676.	0.00
30.9400	-2.40E-06	17.4551	-1.0318	1.97E-08	20214.	9.44E+09	0.03609	61350.	0.00
31.2800	-2.30E-06	13.4957	-0.8867	2.64E-08	20214.	9.44E+09	0.03503	62024.	0.00
31.6200	-2.18E-06	10.1070	-0.7468	3.15E-08	20214.	9.44E+09	0.03358	62699.	0.00
31.9600	-2.05E-06	7.2680	-0.6134	3.52E-08	20214.	9.44E+09	0.03180	63373.	0.00
32.3000	-1.90E-06	4.9517	-0.4877	3.79E-08	20214.	9.44E+09	0.02979	64047.	0.00
32.6400	-1.74E-06	3.1267	-0.3707	3.96E-08	20214.	9.44E+09	0.02758	64721.	0.00
32.9800	-1.57E-06	1.7579	-0.2630	4.07E-08	20214.	9.44E+09	0.02523	65395.	0.00
33.3200	-1.41E-06	0.8074	-0.1651	4.12E-08	20214.	9.44E+09	0.02278	66070.	0.00
33.6600	-1.24E-06	0.2353	-0.07730	4.15E-08	20214.	9.44E+09	0.02024	66744.	0.00
34.0000	-1.07E-06	0.00	0.00	4.15E-08	20214.	9.44E+09	0.01765	33709.	0.00

* This analysis computed pile response using nonlinear moment-curvature relationships. Values of total stress due to combined axial and bending stresses are computed only for elastic sections only and do not equal the actual stresses in concrete and steel. Stresses in concrete and steel may be interpolated from the output for nonlinear bending properties relative to the magnitude of bending moment developed in the pile.

Output Summary for Load Case No. 2:

Pile-head deflection = -0.70000000 inches
Computed slope at pile head = 0.00439566 radians
Maximum bending moment = 1433282. inch-lbs
Maximum shear force = -34682. lbs
Depth of maximum bending moment = 0.000000 feet below pile head
Depth of maximum shear force = 0.000000 feet below pile head
Number of iterations = 11
Number of zero deflection points = 4

Summary of Pile-head Responses for Conventional Analyses

Definitions of Pile-head Loading Conditions:

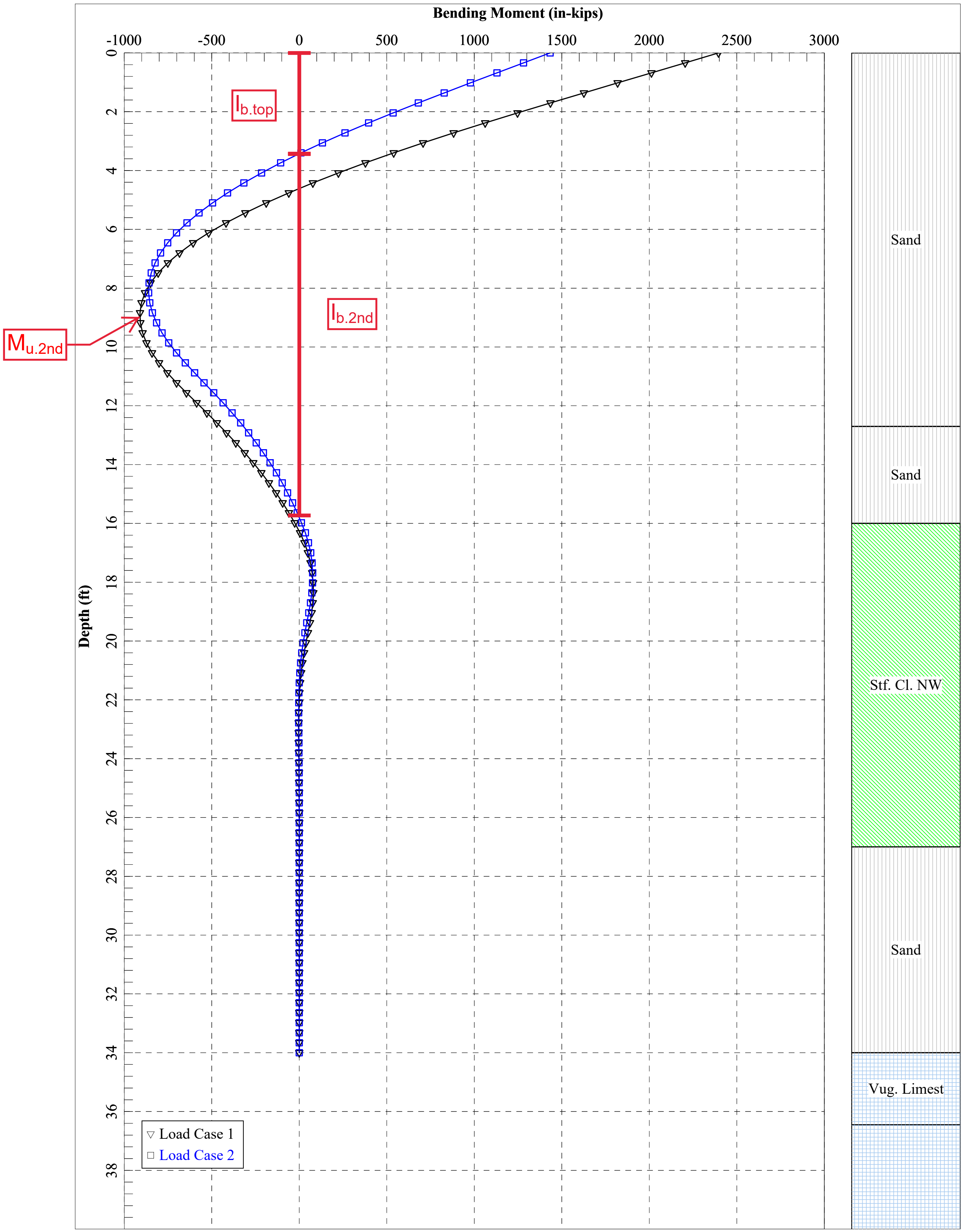
Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians
Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
Load Type 5: Load 1 = Top Deflection, y, inches, and Load 2 = Slope, S, radians

Load Load Load Axial Pile-head Pile-head Max Shear Max Moment

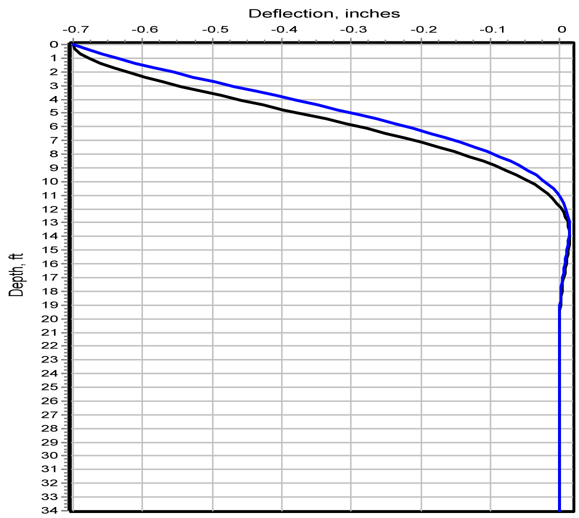
Case No.	Type 1	Pile-head Load 1	Type 2	Pile-head Load 2	Loading lbs	Deflection inches	Rotation radians	in Pile lbs	in Pile in-lbs
1	y, in	-0.7000	S, rad	0.00	522000.	-0.7000	0.00	-46453.	2395738.
2	y, in	-0.7000	M, in-lb	1433282.	522000.	-0.7000	0.00440	-34682.	1433282.

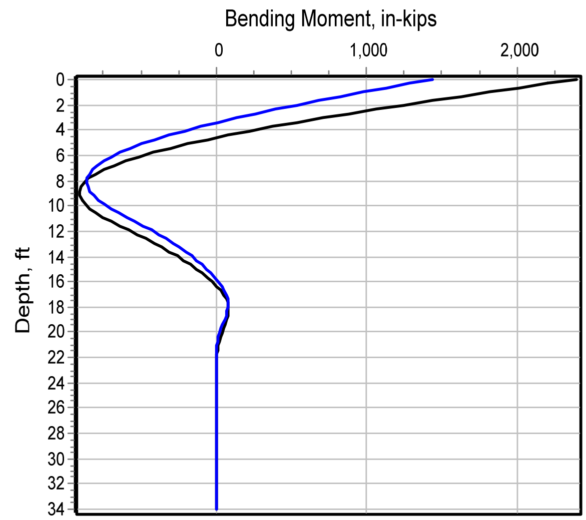
Maximum pile-head deflection = -0.7000000000 inches
Maximum pile-head rotation = 0.0043956616 radians = 0.251853 deg.

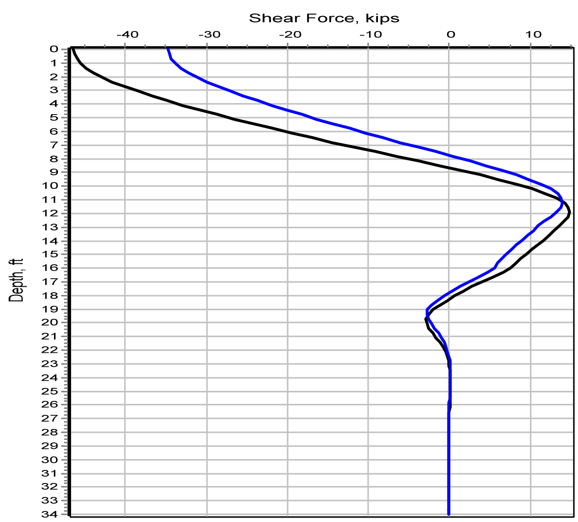
The analysis ended normally.

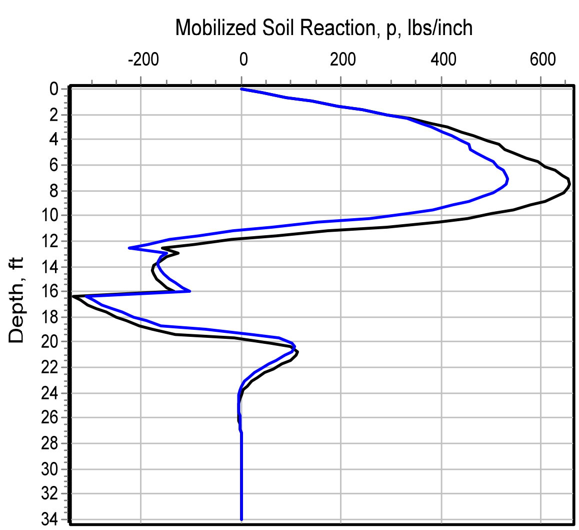


Bending Moment (in-kips)
-1000
-500
0
500
1000
1500
2000
2500
3000
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
$l_{b.top}$
$l_{b.2nd}$
$M_{u.2nd}$
Load Case 1
Load Case 2
Sand
Sand
Stf. Cl. NW
Sand
Vug. Limest


Deflection, inches
-0.7 -0.6 -0.5 -0.4 -0.3 -0.2 -0.1 0
Depth, ft


Bending Moment, in-kips
0 1,000 2,000
Depth, ft


Shear Force, kips
-40 -30 -20 -10 0 10
Depth, ft


Mobilized Soil Reaction, p, lbs/inch
-200 0 200 400 600
Depth, ft

Attachment 3

==

LPile for Windows, Version 2019-11.005

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

==

This copy of LPile is being used by:

KAR
Golder Associates, Inc.

Serial Number of Security Device: 239146533

This copy of LPile is licensed for exclusive use by:

GOLDER, Manchester, NH

Files Used for Analysis

Path to file locations:
\Users\kroth\Documents\Projects\19126013 MaineDOT I-295 Freeport Exit 20 Merrill Rd Bridge\Pile Design\LPile Northwest Abutment\

Name of input data file:
Freeport Exit 20 Northwest Abutment Southern Shift _ Service1.lp11d

Name of output report file:
Freeport Exit 20 Northwest Abutment Southern Shift _ Service1.lp11o

Name of plot output file:
Freeport Exit 20 Northwest Abutment Southern Shift _ Service1.lp11p

Name of runtime message file:
Freeport Exit 20 Northwest Abutment Southern Shift _ Service1.lp11r

----
Date and Time of Analysis

Date: August 26, 2020 Time: 16:52:51

Problem Title

Project Name: MaineDOT I-295 Exit 20 Merrill Road Bridge No. 5720
Job Number: 19126013
Client: MaineDOT
Engineer: KAR
Description: Northwest Abutment Pile Design - Service I

Program Options and Settings

Computational Options:
- Conventional Analysis

Engineering Units Used for Data Input and Computations:
- US Customary System Units (pounds, feet, inches)

Analysis Control Options:
- Maximum number of iterations allowed = 500
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 100.0000 in
- Number of pile increments = 100

Loading Type and Number of Cycles of Loading:
- Static loading specified

- Use of p-y modification factors for p-y curves not selected
- Analysis uses layering correction (Method of Georgiadis)
- No distributed lateral loads are entered
- Loading by lateral soil movements acting on pile not selected
- Input of shear resistance at the pile tip not selected
- Input of moment resistance at the pile tip not selected
- Input of side resistance moment along pile not selected
- Computation of pile-head foundation stiffness matrix not selected
- Push-over analysis of pile not selected
- Buckling analysis of pile not selected

Output Options:
- Output files use decimal points to denote decimal symbols.
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (nodal spacing of output points) = 1
- No p-y curves to be computed and reported for user-specified depths
- Print using wide report formats

Pile Structural Properties and Geometry

Number of pile sections defined = 1
Total length of pile = 34.000 ft
Depth of ground surface below top of pile = 0.0000 ft

Pile diameters used for p-y curve computations are defined using 2 points.

p-y curves are computed using pile diameter values interpolated with depth over the length of the pile. A summary of values of pile diameter vs. depth follows.

Point No.	Depth Below Pile Head feet	Pile Diameter inches
1	0.000	14.6950
2	34.000	14.6950

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is a H weak axis steel pile
Length of section = 34.000000 ft
Pile width = 13.830000 in
Shear capacity of section = 0.0000 lbs

Ground Slope and Pile Batter Angles

Ground Slope Angle = 0.000 degrees
= 0.000 radians

Pile Batter Angle = 0.000 degrees
= 0.000 radians

Soil and Rock Layering Information

The soil profile is modelled using 5 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer = 0.0000 ft
Distance from top of pile to bottom of layer = 12.700000 ft

Effective unit weight at top of layer = 125.000000 pcf
Effective unit weight at bottom of layer = 125.000000 pcf
Friction angle at top of layer = 32.000000 deg.
Friction angle at bottom of layer = 32.000000 deg.
Subgrade k at top of layer = 124.800000 pci
Subgrade k at bottom of layer = 124.800000 pci

Layer 2 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer = 12.700000 ft
Distance from top of pile to bottom of layer = 16.000000 ft
Effective unit weight at top of layer = 62.600000 pcf
Effective unit weight at bottom of layer = 62.600000 pcf
Friction angle at top of layer = 32.000000 deg.
Friction angle at bottom of layer = 32.000000 deg.
Subgrade k at top of layer = 75.500000 pci
Subgrade k at bottom of layer = 75.500000 pci

Layer 3 is stiff clay without free water

Distance from top of pile to top of layer = 16.000000 ft
Distance from top of pile to bottom of layer = 27.000000 ft
Effective unit weight at top of layer = 62.600000 pcf
Effective unit weight at bottom of layer = 62.600000 pcf
Undrained cohesion at top of layer = 1600. psf
Undrained cohesion at bottom of layer = 1600. psf
Epsilon-50 at top of layer = 0.005000
Epsilon-50 at bottom of layer = 0.005000

Layer 4 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer = 27.000000 ft
Distance from top of pile to bottom of layer = 34.000000 ft
Effective unit weight at top of layer = 62.600000 pcf
Effective unit weight at bottom of layer = 62.600000 pcf
Friction angle at top of layer = 37.000000 deg.
Friction angle at bottom of layer = 37.000000 deg.
Subgrade k at top of layer = 40.500000 pci
Subgrade k at bottom of layer = 40.500000 pci

Layer 5 is strong rock (vuggy limestone)

Distance from top of pile to top of layer = 34.000000 ft
Distance from top of pile to bottom of layer = 50.000000 ft
Effective unit weight at top of layer = 101.600000 pcf
Effective unit weight at bottom of layer = 101.600000 pcf
Uniaxial compressive strength at top of layer = 12983. psi
Uniaxial compressive strength at bottom of layer = 12983. psi

(Depth of the lowest soil layer extends 16.000 ft below the pile tip)

Summary of Input Soil Properties

Layer Num.	Soil Type Name (p-y Curve Type)	Layer Depth ft	Effective Unit Wt. pcf	Undrained Cohesion psf	Angle of Friction deg.	Uniaxial qu psi	E50 or krm	kpy pci
1	Sand	0.00	125.0000	--	32.0000	--	--	124.8000
	(Reese, et al.)	12.7000	125.0000	--	32.0000	--	--	124.8000
2	Sand	12.7000	62.6000	--	32.0000	--	--	75.5000
	(Reese, et al.)	16.0000	62.6000	--	32.0000	--	--	75.5000
3	Stiff Clay	16.0000	62.6000	1600.	--	--	0.00500	--
	w/o Free Water	27.0000	62.6000	1600.	--	--	0.00500	--
4	Sand	27.0000	62.6000	--	37.0000	--	--	40.5000
	(Reese, et al.)	34.0000	62.6000	--	37.0000	--	--	40.5000
5	Strong Rock	34.0000	101.6000	--	--	12983.	--	--
	(Vuggy Limestone)	50.0000	101.6000	--	--	12983.	--	--

Static Loading Type

Static loading criteria were used when computing p-y curves for all analyses.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load No.	Load Type	Condition 1	Condition 2	Axial Thrust Force, lbs	Compute Top y vs. Pile Length	Run Analysis
1	5	y = -0.700000 in	S = 0.0000 in/in	412000.	N.A.	Yes

V = shear force applied normal to pile axis
M = bending moment applied to pile head
y = lateral deflection normal to pile axis
S = pile slope relative to original pile batter angle
R = rotational stiffness applied to pile head
Values of top y vs. pile lengths can be computed only for load types with
specified shear loading (Load Types 1, 2, and 3).
Thrust force is assumed to be acting axially for all pile batter angles.

Computations of Nominal Moment Capacity and Nonlinear Bending Stiffness
----

Axial thrust force values were determined from pile-head loading conditions

Number of Pile Sections Analyzed = 1

Pile Section No. 1:

Dimensions and Properties of Steel H Weak Axis:

Length of Section	=	34.000000 ft
Flange Width	=	14.695000 in
Section Depth	=	13.830000 in
Flange Thickness	=	0.615000 in
Web Thickness	=	0.615000 in
Yield Stress of Pipe	=	50.000000 ksi
Elastic Modulus	=	29000. ksi
Cross-sectional Area	=	25.823850 sq. in.
Moment of Inertia	=	325.505721 in^4
Elastic Bending Stiffness	=	9439666. kip-in^2
Plastic Modulus, Z	=	67.593889in^3
Plastic Moment Capacity = Fy Z	=	3380.in-kip

Axial Structural Capacities:

Nom. Axial Structural Capacity = Fy As	=	1291.193 kips
Nominal Axial Tensile Capacity	=	-1291.193 kips

Number of Axial Thrust Force Values Determined from Pile-head Loadings = 1

Number	Axial Thrust Force kips
1	412.000

Definition of Run Messages:

Y = part of pipe section has yielded.

Axial Thrust Force = 412.000 kips

Bending Curvature rad/in.	Bending Moment in-kip	Bending Stiffness kip-in2	Depth to N Axis in	Max Total Stress ksi	Run Msg

0.00000430	40.6230591	9438779.	135.1741499	16.8621262	
0.00000861	81.2461182	9438779.	71.2608249	17.7700086	
0.00001291	121.8691774	9438779.	49.9563833	18.6778911	
0.00001722	162.4922365	9438779.	39.3041625	19.5857735	
0.00002152	203.1152956	9438779.	32.9128300	20.4936560	
0.00002582	243.7383547	9438779.	28.6519416	21.4015384	
0.00003013	284.3614139	9438779.	25.6084500	22.3094208	
0.00003443	324.9844730	9438779.	23.3258312	23.2173033	
0.00003873	365.6075321	9438779.	21.5504611	24.1251857	
0.00004304	406.2305912	9438779.	20.1301650	25.0330681	
0.00004734	446.8536503	9438779.	18.9681045	25.9409506	
0.00005165	487.4767095	9438779.	17.9997208	26.8488330	
0.00005595	528.0997686	9438779.	17.1803192	27.7567154	
0.00006025	568.7228277	9438779.	16.4779750	28.6645980	
0.00006456	609.3458868	9438779.	15.8692767	29.5724803	
0.00006886	649.9689459	9438779.	15.3366656	30.4803628	
0.00007317	690.5920051	9438779.	14.8667147	31.3882452	
0.00007747	731.2150642	9438779.	14.4489805	32.2961276	
0.00008177	771.8381233	9438779.	14.0752184	33.2040100	
0.00008608	812.4611824	9438779.	13.7388325	34.1118926	
0.00009038	853.0842416	9438779.	13.4344833	35.0197749	
0.00009468	893.7073007	9438779.	13.1578023	35.9276574	
0.00009899	934.3303598	9438779.	12.9051804	36.8355399	
0.0001033	974.9534189	9438779.	12.6736104	37.7434222	
0.0001076	1016.	9438779.	12.4605660	38.6513047	
0.0001119	1056.	9438779.	12.2639096	39.5591871	
0.0001162	1097.	9438779.	12.0818204	40.4670696	
0.0001205	1137.	9438779.	11.9127375	41.3749520	
0.0001248	1178.	9438779.	11.7553155	42.2828345	
0.0001291	1219.	9438779.	11.6083883	43.1907168	
0.0001334	1259.	9438779.	11.4709403	44.0985993	
0.0001377	1300.	9438779.	11.3420828	45.0064817	
0.0001420	1341.	9438779.	11.2210348	45.9143642	
0.0001463	1381.	9438779.	11.1071073	46.8222466	
0.0001506	1422.	9438779.	10.9996900	47.7301291	
0.0001549	1462.	9438779.	10.8982403	48.6380115	
0.0001592	1503.	9438779.	10.8022743	49.5458939	
0.0001635	1543.	9435105.	10.7120336	50.0000000	Y
0.0001679	1582.	9422931.	10.6280348	50.0000000	Y
0.0001765	1654.	9375652.	10.4770828	50.0000000	Y
0.0001851	1722.	9306553.	10.3454323	50.0000000	Y
0.0001937	1786.	9221759.	10.2300770	50.0000000	Y
0.0002023	1846.	9125876.	10.1285675	50.0000000	Y
0.0002109	1903.	9022635.	10.0388329	50.0000000	Y
0.0002195	1957.	8914232.	9.9593110	50.0000000	Y
0.0002281	2008.	8802730.	9.8886081	50.0000000	Y
0.0002367	2057.	8690140.	9.8254303	50.0000000	Y
0.0002453	2104.	8577202.	9.7689051	50.0000000	Y
0.0002539	2149.	8464524.	9.7182756	50.0000000	Y
0.0002625	2193.	8353333.	9.6726710	50.0000000	Y
0.0002711	2235.	8243918.	9.6315379	50.0000000	Y
0.0002798	2276.	8136517.	9.5943888	50.0000000	Y
0.0002884	2316.	8031330.	9.5607917	50.0000000	Y
0.0002970	2354.	7928519.	9.5303623	50.0000000	Y

0.0003056	2392.	7828217.	9.5027570	50.0000000	Y
0.0003142	2429.	7730528.	9.4776676	50.0000000	Y
0.0003228	2465.	7635536.	9.4548157	50.0000000	Y
0.0003314	2500.	7543301.	9.4339497	50.0000000	Y
0.0003400	2533.	7449626.	9.4136271	50.0000000	Y
0.0003486	2564.	7353781.	9.3938547	50.0000000	Y
0.0003572	2592.	7257040.	9.3744011	50.0000000	Y
0.0003658	2619.	7160244.	9.3551423	50.0000000	Y
0.0003744	2645.	7063080.	9.3364449	50.0000000	Y
0.0003830	2669.	6966630.	9.3180262	50.0000000	Y
0.0003917	2691.	6871101.	9.2999607	50.0000000	Y
0.0004003	2712.	6776536.	9.2824114	50.0000000	Y
0.0004089	2732.	6682684.	9.2648012	50.0000000	Y
0.0004175	2751.	6590063.	9.2477163	50.0000000	Y
0.0004261	2769.	6499364.	9.2309693	50.0000000	Y
0.0004347	2786.	6409789.	9.2144336	50.0000000	Y
0.0004433	2802.	6321571.	9.1981947	50.0000000	Y
0.0004519	2818.	6235784.	9.1822410	50.0000000	Y
0.0004605	2832.	6150439.	9.1666003	50.0000000	Y
0.0004691	2846.	6067477.	9.1511963	50.0000000	Y
0.0004777	2860.	5986059.	9.1359787	50.0000000	Y
0.0004863	2872.	5905998.	9.1211761	50.0000000	Y
0.0004949	2885.	5827962.	9.1065288	50.0000000	Y
0.0005036	2896.	5751367.	9.0919656	50.0000000	Y
0.0005122	2907.	5676327.	9.0779869	50.0000000	Y
0.0005466	2947.	5391812.	9.0234666	50.0000000	Y
0.0005810	2981.	5131045.	8.9723546	50.0000000	Y
0.0006155	3011.	4891726.	8.9241415	50.0000000	Y
0.0006499	3036.	4672072.	8.8789445	50.0000000	Y
0.0006843	3058.	4469384.	8.8361232	50.0000000	Y
0.0007187	3078.	4282808.	8.7953850	50.0000000	Y
0.0007532	3096.	4110156.	8.7572623	50.0000000	Y
0.0007876	3111.	3950495.	8.7207299	50.0000000	Y
0.0008220	3125.	3802150.	8.6864681	50.0000000	Y
0.0008565	3138.	3664144.	8.6536458	50.0000000	Y
0.0008909	3150.	3535392.	8.6220046	50.0000000	Y
0.0009253	3160.	3415074.	8.5924151	50.0000000	Y
0.0009598	3170.	3302728.	8.5642599	50.0000000	Y

Summary of Results for Nominal Moment Capacity for Section 1

Load No.	Axial Thrust kips	Nominal Moment Capacity in-kips
1	412.0000000000	3170.

Note that the values in the above table are not factored by a strength reduction factor for LRFD.

The value of the strength reduction factor depends on the provisions of the LRFD code being followed.

The above values should be multiplied by the appropriate strength reduction factor to compute ultimate moment capacity according to the LRFD structural design standard being followed.

Layering Correction Equivalent Depths of Soil & Rock Layers

Layer No.	Top of Layer Below Pile Head ft	Equivalent Top Depth Below Grnd Surf ft	Same Layer Type As Layer Above	Layer is Rock or is Below Rock Layer	F0 Integral for Layer lbs	F1 Integral for Layer lbs
1	0.00	0.00	N.A.	No	0.00	208062.
2	12.7000	12.7000	Yes	No	208062.	178840.
3	16.0000	26.1215	No	No	386902.	193974.
4	27.0000	17.5689	No	No	580876.	917051.
5	34.0000	34.0000	No	Yes	N.A.	N.A.

Notes: The F0 integral of Layer n+1 equals the sum of the F0 and F1 integrals for Layer n. Layering correction equivalent depths are computed only for soil types with both shallow-depth and deep-depth expressions for peak lateral load transfer. These soil types are soft and stiff clays, non-liquefied sands, and cemented c-phi soil.

Computed Values of Pile Loading and Deflection
for Lateral Loading for Load Case Number 1

Pile-head conditions are Displacement and Pile-head Rotation (Loading Type 5)
Displacement of pile head = -0.700000 inches
Rotation of pile head = 0.000E+00 radians
Axial load on pile head = 412000.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness lb-in^2	Soil Res. p lb/inch	Soil Spr. Es*h lb/inch	Distrib. Lat. Load lb/inch
0.00	-0.7000	2469278.	-47882.	0.00	71692.	7.62E+09	0.00	0.00	0.00
0.3400	-0.6973	2273238.	-47693.	0.00127	67267.	7.62E+09	40.9313	239.4936	0.00
0.6800	-0.6896	2075834.	-47428.	0.00237	62811.	8.64E+09	89.1585	527.4711	0.00
1.0200	-0.6780	1878268.	-46959.	0.00328	58352.	9.07E+09	140.8805	847.7963	0.00

1.3600	-0.6629	1681625.	-46277.	0.00407	53913.	9.35E+09	193.2697	1190.	0.00
1.7000	-0.6448	1486966.	-45385.	0.00476	49519.	9.44E+09	243.7368	1542.	0.00
2.0400	-0.6241	1295284.	-44291.	0.00536	45192.	9.44E+09	292.7345	1914.	0.00
2.3800	-0.6010	1107534.	-43003.	0.00588	40954.	9.44E+09	338.7405	2299.	0.00
2.7200	-0.5761	924618.	-41535.	0.00632	36825.	9.44E+09	380.8075	2697.	0.00
3.0600	-0.5495	747369.	-39912.	0.00668	32824.	9.44E+09	414.7606	3080.	0.00
3.4000	-0.5216	576481.	-38161.	0.00697	28967.	9.44E+09	443.3731	3468.	0.00
3.7400	-0.4927	412555.	-36307.	0.00718	25267.	9.44E+09	465.6148	3856.	0.00
4.0800	-0.4630	256080.	-34344.	0.00732	21735.	9.44E+09	496.4594	4375.	0.00
4.4200	-0.4329	107683.	-32274.	0.00740	18385.	9.44E+09	518.6589	4888.	0.00
4.7600	-0.4026	-32158.	-30133.	0.00742	16680.	9.44E+09	530.6821	5378.	0.00
5.1000	-0.3724	-163142.	-27927.	0.00738	19637.	9.44E+09	550.5023	6032.	0.00
5.4400	-0.3424	-284844.	-25627.	0.00728	22384.	9.44E+09	577.3147	6879.	0.00
5.7800	-0.3130	-396728.	-23227.	0.00713	24909.	9.44E+09	598.8761	7808.	0.00
6.1200	-0.2842	-498355.	-20751.	0.00694	27203.	9.44E+09	614.7790	8826.	0.00
6.4600	-0.2563	-589386.	-18202.	0.00670	29258.	9.44E+09	634.7859	10104.	0.00
6.8000	-0.2295	-669422.	-15579.	0.00643	31065.	9.44E+09	651.0051	11573.	0.00
7.1400	-0.2038	-738134.	-12901.	0.00613	32616.	9.44E+09	661.8291	13246.	0.00
7.4800	-0.1795	-795293.	-10191.	0.00580	33906.	9.44E+09	666.7582	15155.	0.00
7.8200	-0.1566	-840775.	-7473.	0.00544	34933.	9.44E+09	665.5079	17344.	0.00
8.1600	-0.1351	-874568.	-4773.	0.00507	35695.	9.44E+09	657.8511	19869.	0.00
8.5000	-0.1152	-896775.	-2118.	0.00469	36197.	9.44E+09	643.6159	22801.	0.00
8.8400	-0.09683	-907616.	465.0980	0.00430	36441.	9.44E+09	622.6779	26238.	0.00
9.1800	-0.08009	-907432.	2949.	0.00391	36437.	9.44E+09	594.9475	30309.	0.00
9.5200	-0.06495	-896685.	5306.	0.00352	36195.	9.44E+09	560.3472	35200.	0.00
9.8600	-0.05139	-875959.	7507.	0.00313	35727.	9.44E+09	518.7720	41186.	0.00
10.2000	-0.03938	-845961.	9524.	0.00276	35050.	9.44E+09	470.0197	48698.	0.00
10.5400	-0.02886	-807524.	11327.	0.00240	34182.	9.44E+09	413.6536	58482.	0.00
10.8800	-0.01976	-761614.	12828.	0.00206	33146.	9.44E+09	321.9988	66479.	0.00
11.2200	-0.01201	-709791.	13896.	0.00175	31976.	9.44E+09	201.7836	68557.	0.00
11.5600	-0.00551	-654093.	14502.	0.00145	30719.	9.44E+09	95.3442	70634.	0.00
11.9000	-1.59E-04	-596333.	14703.	0.00118	29415.	9.44E+09	2.8423	72711.	0.00
12.2400	0.00414	-538092.	14554.	9.37E-04	28100.	9.44E+09	-75.8274	74789.	0.00
12.5800	0.00748	-480722.	14112.	7.16E-04	26805.	9.44E+09	-140.9933	76866.	0.00
12.9200	0.00998	-425350.	13586.	5.21E-04	25555.	9.44E+09	-116.8578	47759.	0.00
13.2600	0.01173	-371614.	13060.	3.48E-04	24343.	9.44E+09	-140.9469	49015.	0.00
13.6000	0.01283	-319955.	12450.	1.99E-04	23176.	9.44E+09	-158.0383	50272.	0.00
13.9400	0.01336	-270693.	11783.	7.13E-05	22064.	9.44E+09	-168.6769	51529.	0.00
14.2800	0.01341	-224043.	11085.	-3.56E-05	21011.	9.44E+09	-173.4652	52786.	0.00
14.6200	0.01306	-180118.	10378.	-1.23E-04	20020.	9.44E+09	-173.0520	54043.	0.00
14.9600	0.01240	-138943.	9682.	-1.92E-04	19091.	9.44E+09	-168.1219	55299.	0.00
15.3000	0.01150	-100465.	9014.	-2.44E-04	18222.	9.44E+09	-159.3881	56556.	0.00
15.6400	0.01042	-64567.	8388.	-2.79E-04	17412.	9.44E+09	-147.5857	57813.	0.00
15.9800	0.00922	-31079.	7815.	-3.00E-04	16656.	9.44E+09	-133.4675	59070.	0.00
16.3200	0.00797	209.0527	6858.	-3.07E-04	15959.	9.44E+09	-335.4667	171794.	0.00
16.6600	0.00672	25913.	5518.	-3.01E-04	16539.	9.44E+09	-321.4698	195296.	0.00
17.0000	0.00551	46247.	4238.	-2.85E-04	16998.	9.44E+09	-305.9915	226560.	0.00
17.3400	0.00439	61454.	3024.	-2.62E-04	17341.	9.44E+09	-289.0709	268873.	0.00
17.6800	0.00337	71804.	1882.	-2.33E-04	17575.	9.44E+09	-270.7021	327644.	0.00
18.0200	0.00248	77595.	818.0866	-2.01E-04	17706.	9.44E+09	-250.8182	412303.	0.00
18.3600	0.00173	79155.	-161.2509	-1.67E-04	17741.	9.44E+09	-229.2493	540681.	0.00
18.7000	0.00112	76842.	-1048.	-1.34E-04	17689.	9.44E+09	-205.6216	750762.	0.00
19.0400	6.40E-04	71049.	-1833.	-1.02E-04	17558.	9.44E+09	-179.0605	1140643.	0.00
19.3800	2.89E-04	62225.	-2493.	-7.27E-05	17359.	9.44E+09	-144.4160	2040000.	0.00

19.7200	4.69E-05	50951.	-2835.	-4.83E-05	17104.	9.44E+09	-23.4589	2040000.	0.00
20.0600	-1.05E-04	39249.	-2776.	-2.88E-05	16840.	9.44E+09	52.5692	2040000.	0.00
20.4000	-1.88E-04	28395.	-2477.	-1.42E-05	16595.	9.44E+09	93.9868	2040000.	0.00
20.7400	-2.21E-04	19084.	-2060.	-3.90E-06	16385.	9.44E+09	110.3659	2040000.	0.00
21.0800	-2.20E-04	11596.	-1611.	2.73E-06	16216.	9.44E+09	109.9168	2040000.	0.00
21.4200	-1.98E-04	5930.	-1184.	6.51E-06	16088.	9.44E+09	99.2423	2040000.	0.00
21.7600	-1.67E-04	1911.	-811.7302	8.21E-06	15997.	9.44E+09	83.3389	2040000.	0.00
22.1000	-1.32E-04	-721.6439	-507.5880	8.47E-06	15971.	9.44E+09	65.7504	2040000.	0.00
22.4400	-9.76E-05	-2259.	-273.9088	7.82E-06	16005.	9.44E+09	48.7982	2040000.	0.00
22.7800	-6.77E-05	-2983.	-105.3304	6.69E-06	16022.	9.44E+09	33.8383	2040000.	0.00
23.1200	-4.30E-05	-3141.	7.5778	5.36E-06	16025.	9.44E+09	21.5088	2040000.	0.00
23.4600	-2.39E-05	-2939.	75.8326	4.05E-06	16021.	9.44E+09	11.9494	2040000.	0.00
23.8000	-9.96E-06	-2536.	110.3724	2.87E-06	16011.	9.44E+09	4.9819	2040000.	0.00
24.1400	-5.01E-07	-2048.	121.0467	1.88E-06	16000.	9.44E+09	0.2507	2040000.	0.00
24.4800	5.35E-06	-1555.	116.1023	1.10E-06	15989.	9.44E+09	-2.6744	2040000.	0.00
24.8200	8.46E-06	-1105.	102.0205	5.23E-07	15979.	9.44E+09	-4.2285	2040000.	0.00
25.1600	9.62E-06	-723.9630	83.5849	1.28E-07	15971.	9.44E+09	-4.8086	2040000.	0.00
25.5000	9.50E-06	-422.9116	64.0847	-1.20E-07	15964.	9.44E+09	-4.7503	2040000.	0.00
25.8400	8.64E-06	-200.6284	45.5831	-2.55E-07	15959.	9.44E+09	-4.3191	2040000.	0.00
26.1800	7.42E-06	-50.0969	29.2018	-3.09E-07	15955.	9.44E+09	-3.7110	2040000.	0.00
26.5200	6.12E-06	38.6969	15.3918	-3.11E-07	15955.	9.44E+09	-3.0587	2040000.	0.00
26.8600	4.88E-06	76.5470	4.1735	-2.86E-07	15956.	9.44E+09	-2.4405	2040000.	0.00
27.2000	3.78E-06	73.7162	-0.9070	-2.54E-07	15956.	9.44E+09	-0.04996	53934.	0.00
27.5400	2.81E-06	70.0002	-1.0856	-2.23E-07	15956.	9.44E+09	-0.03759	54609.	0.00
27.8800	1.96E-06	65.6076	-1.2164	-1.94E-07	15956.	9.44E+09	-0.02656	55283.	0.00
28.2200	1.23E-06	60.7252	-1.3050	-1.66E-07	15956.	9.44E+09	-0.01684	55957.	0.00
28.5600	6.03E-07	55.5183	-1.3564	-1.41E-07	15955.	9.44E+09	-0.00837	56631.	0.00
28.9000	7.58E-08	50.1318	-1.3756	-1.18E-07	15955.	9.44E+09	-0.00107	57305.	0.00
29.2400	-3.63E-07	44.6910	-1.3673	-9.79E-08	15955.	9.44E+09	0.00516	57979.	0.00
29.5800	-7.23E-07	39.3037	-1.3356	-7.97E-08	15955.	9.44E+09	0.01039	58654.	0.00
29.9200	-1.01E-06	34.0607	-1.2843	-6.39E-08	15955.	9.44E+09	0.01474	59328.	0.00
30.2600	-1.24E-06	29.0384	-1.2169	-5.02E-08	15955.	9.44E+09	0.01830	60002.	0.00
30.6000	-1.42E-06	24.2995	-1.1364	-3.87E-08	15955.	9.44E+09	0.02117	60676.	0.00
30.9400	-1.56E-06	19.8954	-1.0454	-2.92E-08	15955.	9.44E+09	0.02346	61350.	0.00
31.2800	-1.66E-06	15.8673	-0.9460	-2.14E-08	15955.	9.44E+09	0.02526	62024.	0.00
31.6200	-1.73E-06	12.2480	-0.8401	-1.54E-08	15955.	9.44E+09	0.02666	62699.	0.00
31.9600	-1.79E-06	9.0637	-0.7291	-1.07E-08	15954.	9.44E+09	0.02775	63373.	0.00
32.3000	-1.82E-06	6.3347	-0.6141	-7.42E-09	15954.	9.44E+09	0.02861	64047.	0.00
32.6400	-1.85E-06	4.0773	-0.4960	-5.17E-09	15954.	9.44E+09	0.02930	64721.	0.00
32.9800	-1.86E-06	2.3048	-0.3752	-3.79E-09	15954.	9.44E+09	0.02989	65395.	0.00
33.3200	-1.88E-06	1.0280	-0.2522	-3.07E-09	15954.	9.44E+09	0.03041	66070.	0.00
33.6600	-1.89E-06	0.2568	-0.1271	-2.79E-09	15954.	9.44E+09	0.03091	66744.	0.00
34.0000	-1.90E-06	0.00	0.00	-2.73E-09	15954.	9.44E+09	0.03141	33709.	0.00

* This analysis computed pile response using nonlinear moment-curvature relationships. Values of total stress due to combined axial and bending stresses are computed only for elastic sections only and do not equal the actual stresses in concrete and steel. Stresses in concrete and steel may be interpolated from the output for nonlinear bending properties relative to the magnitude of bending moment developed in the pile.

Output Summary for Load Case No. 1:

Pile-head deflection = -0.70000000 inches
Computed slope at pile head = 0.000000 radians
Maximum bending moment = 2469278. inch-lbs
Maximum shear force = -47882. lbs
Depth of maximum bending moment = 0.000000 feet below pile head
Depth of maximum shear force = 0.000000 feet below pile head
Number of iterations = 11
Number of zero deflection points = 4

Summary of Pile-head Responses for Conventional Analyses

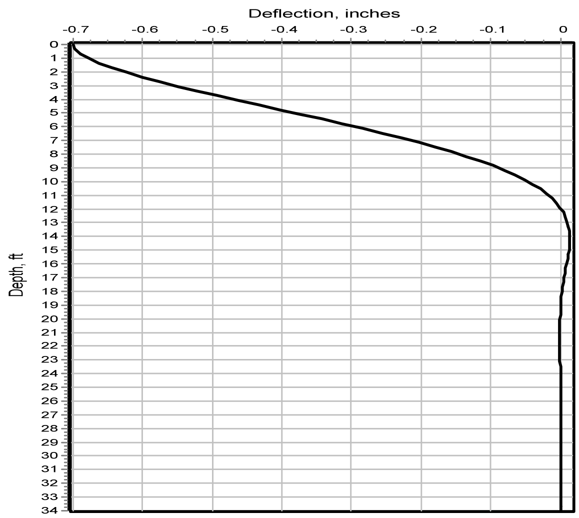
Definitions of Pile-head Loading Conditions:

Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians
Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
Load Type 5: Load 1 = Top Deflection, y, inches, and Load 2 = Slope, S, radians

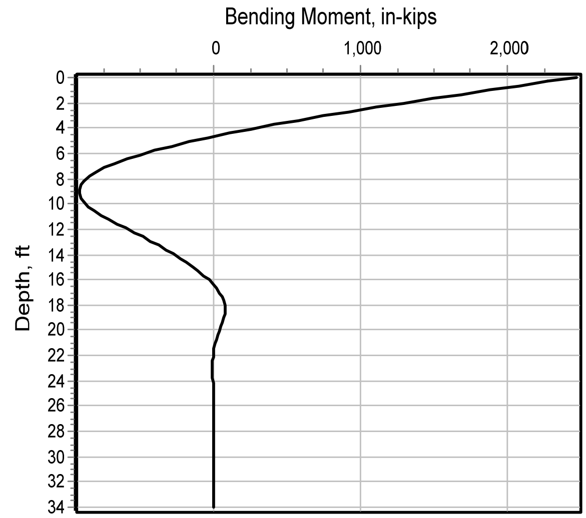
Load Case No.	Load Type 1	Pile-head Load 1	Load Type 2	Pile-head Load 2	Axial Loading lbs	Pile-head Deflection inches	Pile-head Rotation radians	Max Shear in Pile lbs	Max Moment in Pile in-lbs
1	y, in	-0.7000	S, rad	0.00	412000.	-0.7000	0.00	-47882.	2469278.

Maximum pile-head deflection = -0.7000000000 inches
Maximum pile-head rotation = 0.0000000000 radians = 0.000000 deg.

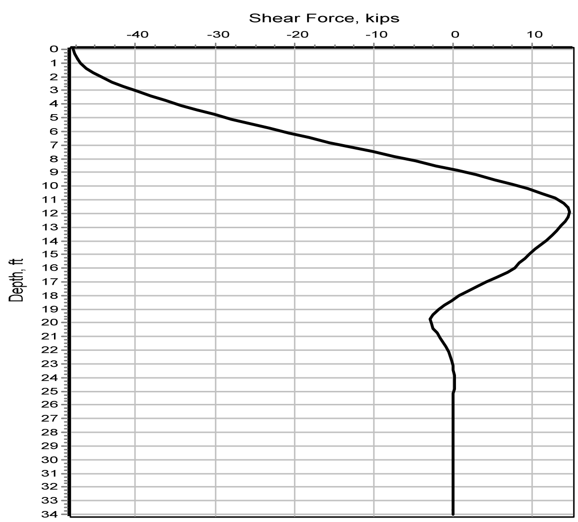
The analysis ended normally.



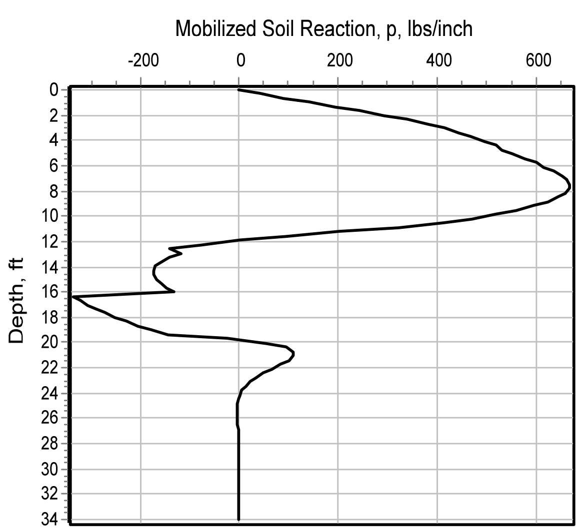
Deflection, inches
-0.7
-0.6
-0.5
-0.4
-0.3
-0.2
-0.1
0
Depth, ft



Bending Moment, in-kips
0
1,000
2,000
Depth, ft



Shear Force, kips
-40
-30
-20
-10
0
10
Depth, ft



Mobilized Soil Reaction, p, lbs/inch
-200
0
200
400
600
Depth, ft

Piles for full integral and integral with hinge abutments shall be designed to resist all vertical superstructure dead and live loads, abutment and pile dead loads, live load girder rotation moments, lateral displacements, live load impact and moments caused by superimposed dead loads and live loads, as appropriate for the type of integral abutment.

Until the behavior of integral abutments with hinged connections to the superstructure is better understood, the pile design criteria for that type of integral abutment may assume that the moment at the top of the pile is zero, and that there is no moment from either the superstructure or earth loads.

The effect of thermal displacements and moments on piles can be investigated by running LPILE® software.

Secondary thermal forces only need be considered for multi-span structures only.

Appropriate load combinations and load factors should be determined per LRFD 3.4.1.

For the strength limit state analysis, design of the piles should consider the factored structural pile resistance, P_r, the factored structural flexural resistance, pile unbraced length, pile moments, the interaction of combined axial and flexural load effects, the structural shear resistance and the factored geotechnical resistance.

For service limit state evaluations, if piles will be driven to practical refusal in bedrock, settlement will not be a concern. However, all designs should consider horizontal movement, overall stability and scour for the design flood event.

B. Resistance Factors for Integral H-Piles

Pile will typically be end bearing on bedrock. For the strength limit state, use the following resistance factors:

- Use $\Phi_c = 0.50$ for axial resistance in compression and subject to severe pile driving condition; this condition should be assumed when analyzing the lower portions of the pile
- Use $\Phi_c = 0.60$ for axial resistance in compression under good driving conditions; this condition should be assumed when analyzing the upper portion of the pile
- For combined axial and flexural resistance in the upper zone of pile, use:

- Φ_c = 0.70 for axial resistance
- Φ_f = 1.00 for flexural resistance

C. Design Steps

The following steps should be followed during design of piles supporting full integral abutments, for the strength limit state:

1. Determine the foundation displacements, and the load effects (P_u and M_u) from the superstructure and substructure designs.

2. If applicable, determine the magnitude of scour.

3. Select preliminary pile size:

   a. Determine the factored applied superstructure vertical dead and live load (P_u) distributed to each pile

   b. Select the steel pile strength
   c. Select pile orientation; typically weak axis bending
   d. Determine resistance factors (Φ_c and Φ_f) for the structural strength in the upper and lower zones of the pile.
   e. Determine the maximum, required nominal axial pile resistance, P_u/Φ_f
   f. Estimate an initial pile area using the approximation
   $$A_s = \frac{Ru}{0.80 \times F_y}$$
   This approximation is based on weak axis bending and an assumed unbraced length of 15 feet based on typical integral abutment pile deflection and moment with depth curves. Select a pile size with an area A_s or greater.

4. Determine the pile unbraced length and maximum moment at the top of the pile by running LPILE® software for the design displacement from Step 1, P_u, and live load rotation

5. Determine if the applied moment on the pile will cause pile head plastic deformation by using the Interaction of combined axial and flexural load effects on a single pile (LRFD 6.9.2.2)

   a. Obtain the unbraced lengths of the top and lower segments of the pile and calculate the column slenderness factor (λ) for each segment. (LRFD 6.9.4.1)
   b. Determine K values for the top and bottom of the pile per LRFD Table C4.6.2.5-1

g. Calculate the nominal and factored structural pile resistance P_n, per LRFD 6.9.4.1 using the λ values
h. Compare the ratio of P_u to the structural resistance in the upper portion of the pile – the pile size should be such that the ratio is not less than 0.20.
i. Determine the nominal and factored flexural resistance about H-Pile weak axis, (LRFD 6.12.2.2)
j. Calculate the moment that will cause a plastic hinge at the top of the pile (M_p')
k. If the applied moment exceeds the moment that would cause a plastic hinge, a plastic hinge forms, and the moment that can be applied cannot exceed that moment (M_p')

6. For fixed head piles, run a second LPILE® analysis with displacement and plastic moment (M_p') as load conditions and P_u, and calculate new unbraced lengths from the moment with depth curve.

   a. Repeat steps 5.a. through 5.d., above
   b. If the pile size is such that the ratio of P_u to structural resistance exceeds 0.2, check the upper zone of the pile with the interaction equation of LRFD 6.9.2.2. If a plastic hinge forms at the top of the pile, the K value of the upper segment (that portion between the top of the pile and the first inflection point on the moment vs. depth curve) changes from 1.2, for a pinned condition, to 2.1, for a free condition at the top. With the new K value repeat Step 5, and check the interaction equation for pile overstress.

7. Because the piles have weak axis orientation and the flanges resist the shear as opposed to the web, check the maximum shear from the LPILE® output to the structural shear resistance per AISC G7.

8. Check that the maximum factored applied pile load does not exceed the factored geotechnical pile resistance or pile drivability resistance (LRFD 10.5.5.2.3 and 10.7.3.13) provided in the Geotechnical Design Report.

5.4.2.5 Pile Length Requirement

A. General Requirements

Piles may be end bearing or friction piles. In order to obtain the pile behavior associated with the equivalent length, piles should be installed 1 to 5 feet beyond the pile length required to achieve fixity. The practical

Reference 4

5.6.3 Steel H-Piles

Section	Weight	Area, A	Depth, d	Flange Width, b	Thickness		I_{xx}	I_{yy}	Compact Section Criteria
					Flange, t_f	Web, t_w			F'_y
	lbs/ft kg/m	**in²** cm²	**in** mm	**in** mm	**in.** mm	**in.** mm	**in⁴** cm⁴	**in⁴** cm⁴	**ksi** MPa
HP 14 HP 360	**117**	**34.4**	**14.21**	**14.885**	**0.805**	**0.805**	**1220**	**443**	**49.4**
	175	222	361	378	20.4	20.4	50800	18400	341
	102	**30**	**14.01**	**14.785**	**0.705**	**0.705**	**1050**	**380**	**38.4**
	153	194	356	376	17.9	17.9	43700	15800	265
	89	**26.1**	**13.83**	**14.695**	**0.615**	**0.615**	**904**	**326**	**29.6**
	133	168	351	373	15.6	15.6	37600	13600	204
	73	**21.4**	**13.61**	**14.585**	**0.505**	**0.505**	**729**	**261**	**20.3**
	109	138	346	370	12.8	12.8	30300	10900	140
HP 13 HP 330	**100**	**29.4**	**13.15**	**13.205**	**0.765**	**0.765**	**886**	**294**	**56.7**
	150	190	334	335	19.4	19.4	36878	12237	391
	87	**25.5**	**12.95**	**13.105**	**0.665**	**0.665**	**755**	**250**	**43.5**
	130	165	329	333	16.9	16.9	31425	10406	300
	73	**21.6**	**12.75**	**13.005**	**0.565**	**0.565**	**630**	**207**	**31.9**
	109	139	324	330	14.4	14.4	26223	8616	220
	60	**17.5**	**12.54**	**12.9**	**0.46**	**0.46**	**503**	**165**	**21.5**
	90	113	319	328	11.7	11.7	20936	6868	148

Section	Weight	Area, A	Depth, d	Flange Width, b	Thickness		I_{xx}	I_{yy}	Compact Section Criteria
					Flange, t_f	Web, t_w			F'_y
	lbs/ft kg/m	**in²** cm²	**in** mm	**in** mm	**in.** mm	**in.** mm	**in⁴** cm⁴	**in⁴** cm⁴	**ksi** MPa
HP 12 HP 310	**84**	**24.6**	**12.28**	**12.295**	**0.685**	**0.685**	**650**	**213**	52.5
	126	159	312	312	17.4	17.4	27100	8870	362
	74	**21.8**	**12.13**	**12.215**	**0.61**	**0.61**	**569**	**186**	**42.1**
	111	141	308	310	15.5	15.5	23700	7740	290
	63	**18.4**	**11.94**	**12.125**	**0.515**	**0.515**	**472**	**153**	**30.5**
	94	119	303	308	13.1	13.1	19600	6370	210
	53	**15.5**	**11.78**	**12.045**	**0.435**	**0.435**	**393**	**127**	**22**
	79	100	299	306	11	11	16400	5290	152
HP10 HP 250	**57**	**16.8**	**9.99**	**10.225**	**0.565**	**0.565**	**294**	**101**	**51.6**
	85	108	254	260	14.4	14.4	12200	4200	356
	42	**12.4**	**9.7**	**10.075**	**0.42**	**0.42**	**210**	**71.7**	**29.4**
	63	80	246	256	10.7	10.7	8740	2980	203
HP 8 HP 200	**36**	**10.6**	**8.02**	**8.155**	**0.445**	**0.445**	**119**	**40.3**	**50.3**
	54	68.4	204	207	11.3	11.3	4950	1680	347

Reference 7

Cohesionless Soil

Soil properties for preliminary design only.

Cohesionless Soil Properties	Symbol	Units	Loose		Medium		Dense	
Total Unit Weight	γ	pcf	90	115	110	130	110	140
Corrected SPT Blow Count	N_{60}		4	10	10	30	30	50
Relative Density	D_r	%	15	35	35	65	65	85
Angle of Internal Friction	ϕ	deg	29	30	30	36	36	41
Coefficient of Lateral Earth Pressure (From Eqn. (1) using ϕ)	K_0		0.51	0.5	0.5	0.41	0.41	0.34
Subgrade Modulus (Below Water Table)	k_{bw}	pci	20	30	30	100	100	160
Subgrade Modulus (Above Water Table)	k_{aw}	pci	20	50	50	165	165	275
Poisson's Ratio	υ		0.20 - 0.40		0.25 - 0.40		0.30 - 0.45	
Young's Modulus (From Eqn. (2) using α =5, p_a =2000psf and N_{60})	E_{em}	psf	40000	100000	100000	300000	300000	500000
Young's Modulus (From Eqn. (2) using α =10, p_a =2000psf and N_{60})	E_{em}	psf	80000	200000	200000	600000	600000	1000000
Young's Modulus (From Eqn. (3) using B = 24in, υ=the minimum of the range, and k_{bw})	E	psf	66360	99530	97200	324000	314500	503190
Young's Modulus (From Eqn. (3) using B = 24in, υ=the minimum of the range, and k_{aw})	E	psf	66360	165890	162000	534600	518920	864860

References :

Ref.[1]

Ref.[2]

Ref.[3]

Ref.[4]

$K_0 = 1 - \sin(\phi)$ (1) Ref.[5]

Ref.[6]

Ref.[6]

Ref.[7]

$E_{em} = p_a * \alpha * N_{60}$ (2) Ref.[8]

$E = k * B * (1 - \upsilon^2)$ (3) Ref.[9]

Notation:

E_{em} = Elastic Modulus based on empirical equation.

Cohesive Soil

Soil properties for preliminary design only.

Cohesive Soil Properties	Symbol	Units	Soft		Medium		Stiff		References :
Total Unit Weight	γ	pcf	100	120	110	130	120	140	Ref.[12]
Corrected SPT Blow Count	N_{60}		2	4	4	8	8	15	Ref. [13]
Unconfined Compressive Strength	q_u	tsf	0.25	0.5	0.5	1	1	2	Ref. [13]
Undrained Shear Strength	C_u	psf	250	500	500	1000	1000	2000	Ref. [14]
Average Undrained Shear Strength		psf	375		750		1500		
Major Principal Strain @ 50%	$\varepsilon 50$		0.02		0.01		0.005		Ref. [15]
Major Principal Strain @ 100%	$\varepsilon 100$		0.06		0.03		0.015		Ref. [16]
Subgrade Modulus (Static Loading)	k	pci	NA		NA		500		Ref. [17]
Subgrade Modulus (Cycling Loading)	k	pci	NA		NA		200		Ref. [17]
Poisson's Ratio	υ		0.4		0.45		0.5		Ref. [18]
Elastic Modulus	E	psi	415	1735	1735	4860	4860	>13890	Ref. [19]
Shear Modulus (From Eqn. (4) using E , and υ)	G	ksi	0.15	0.62	0.60	1.68	1.62	4.63	$G = E/\left(2\left(1+\upsilon\right)\right)$ (4) Ref.[10]
Ultimate Unit End Bearing		ksi	See Fig.2 (For Driven Piles) on pp. 8						
Axial Bearing Failure		kips	Ultimate Unit End Bearing x Tip Area						
Ultimate Unit Skin Friction		psf	See Fig. 3 (For Driven Piles) on pp. 9						

Note: For the input values of vertical failure shear stress and torsional shear stress, the ultimate unit skin friction for a pile or drilled shaft can be used.

GOLDER

CALCULATIONS

Date:	8/26/2020 Rev. 12/9/2020 (section 8 only)	**Made by:**	KAR
Project No.:	19126013	**Checked by:**	MLM
Subject:	Pile Design at Abutment 2 - Southern Shift	**Reviewed by:**	CCB
Project Title:	MaineDOT I-295 Exit 20 Merrill Road Bridge Replacement No. 5720		

OBJECTIVE

Determine if the proposed HP 14x89 piles will provide adequate support for Abutment 2 (the southeastern integral abutment) based on the anticipated thermal movement and preliminary design loads, assuming the "southern shift" option with the bike path scenario.

METHOD

Use the procedure outlined in AASHTO LRFD (Ref. 1) and the design method provided in the MaineDOT Bridge Design Guide (Ref. 2).

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 9th Ed. 2020.
2. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.
3. Email communication between Golder and HNTB, subject "RE: Freeport I-295 Exit 20 and Exit 22 - Request for Sections and Design Info", dated June 22, 2020.
4. Isenhower, W.M. et al. LPile v2019 Technical Manual: A Program for the Analysis of Deep Foundations Under Lateral Loading. Ensoft, Inc. Dated March 2020.
5. Golder interpreted subsurface section A-A' (Figure 3, Preliminary Geotechnical Design Report, dated September 2020).
6. Golder geotechnical test boring logs (Appendix A, Preliminary Geotechnical Design Report, dated September 2020).
7. Bridge Software Institute. FB-MultiPier Soil Parameter Table (US Customary Units). Accessed July 2020. https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf
8. VTrans Integral Abutment Committee. Integral Abutment Bridge Design Guidelines, 2nd Ed. 2008.
9. AISC Steel Construction Manual, 13th Ed.
10. Golder calculation titled "Settlement at SE Bridge Embankment - Southern Shift" (Appendix E, Preliminary Geotechnical Design Report, dated September 2020).
11. Oregon Department of Transportation, Geo-Environmental Section. Geotechnical Design Manual: Chapter 8 - Foundations, Version 2.1. Dated May 6, 2019.
12. HNTB for State of Maine Department of Transportation. Merrill Road Bridge Freeport Interstate 295: Desert Road South Cross Sections, dated November 2019.
13. Golder calculation titled "Pile Driveability - Southern Shift" (Appendix E, Preliminary Geotechnical Design Report, dated September 2020).

ASSUMPTIONS

1. The selected pile orientation is weak axis bending (Ref. 2, page 5-42).
2. The vertical load is assumed to be evenly distributed.
3. Based on discussions with HNTB, the new southeast abutment will be located approximately 30 ft behind the face of the existing southeast abutment. The post-construction ground surface elevation at the new southeast abutment will be 170 ft (Ref. 12). Assuming 1 ft of pavement atop the abutment plus a 12-ft abutment height (Ref. 3), the top of the piles will be located at elevation 157 ft.

GOLDER

CALCULATIONS

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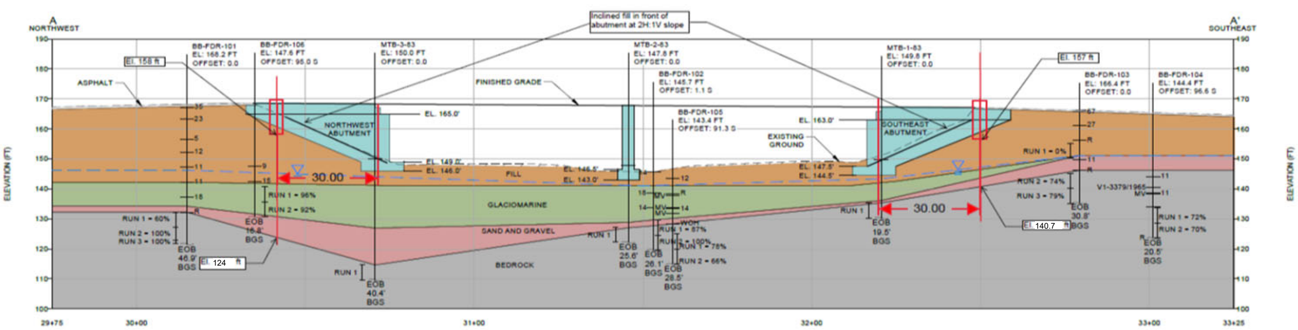

SCALE 1" = 30' A INTERPRETED SUBSURFACE SECTION A-A'
VERTICAL EXAGGERATION: 1 03 SECTION OFFSET FROM CENTERLINE ALIGNMENT

ATTACHMENTS

1. LPile analysis output for Strength I
2. LPile analysis output for Strength I with Plastic Hinge
3. LPile analysis output for Service I

CALCULATION

1. Determine the downdrag load acting on the piles at the southeastern abutment.

As per Ref. 1 Article 3.11.8, downdrag can be assumed to fully develop if the settlement in the soil layer is 0.4 inches or greater relative to the pile. Since the settlement calculated in the bridge approach embankment approximately 1.5 feet behind the new southeastern abutment was estimated to be 1.33 inches (Ref. 10), it is assumed that downdrag will develop.

Determine the soil layers contributing to downdrag (the deepest layer with settlement ≥ 0.4 inches and all layers above that).

Layer		Layer Thickness in Embankment (ft)	σ'_{v0} at layer midpoint in Embankment (ksf)	Settlement Based on Calculated Loading Stress (in)
Existing Fill	1	3.5	0.219	0.37
Glaciomarine	2	6.1	0.819	0.70
Sand and Gravel	3	4.1	1.147	0.26
(Ref. 10)			Total Settlement (in):	1.33

Layers 1 and 2 will contribute to the downdrag load.

Use the α-method to calculate the nominal skin resistance for the cohesive soils contributing to downdrag (Ref. 1, Article 10.7.3.8.6b); use the Nordlund/Thurman method to calculate the skin resistance for the cohesionless soils contributing to downdrag (Ref. 1, Article 10.7.3.8.6f).

α-method for Layer 2, Glaciomarine:

$$q_s = \alpha S_u$$ (Ref. 1, Eqn 10.7.3.8.6b-1)

where:

S_u = 1.600 ksf (based on shear strength measurements made in the field and on empirical correlation to the average of the N_{60}-values encountered in all borings for the layer)

D = 13.83 in = 1.2 ft (Ref. 4, Table 5.6.3, HP 14x89)

D_b = 2.9 ft (thickness of glaciomarine at abutment, Ref. 5)

α = adhesion factor from Ref. 1 Figure 10.7.3.8.6b-1
Use the plot for "Sands over Stiff Clay" and the curve for "D_b less than 10D"

α = 1.00

q_s = 1.600 ksf

Nordlund/Thurman method for Layer 1, Existing Fill:

$$q_s = K_\delta C_F \sigma'_v \frac{\sin(\delta + \omega)}{\cos \omega}$$ (Ref. 1, Eqn 10.7.3.8.6f-1)

where:

ϕ_f = 32 degrees (based on empirical correlation to the average of the N_{60}-values encountered in all borings for the layer)

$V = A_s$ = 26.1 in^2 = 0.18 ft^3/ft (Ref. 4, Table 5.6.3, HP 14x89)

K_δ = 1.01 (interpolation between Ref. 1 Figures 10.7.3.8.6f-2 and 10.7.3.8.6f-3)

C_F = 0.94

σ'_v = 0.663 ksf (Ref. 5; fill thickness at abutment is 10.6 ft)

δ/ϕ_f = 0.81 (Ref. 1, Figure 10.7.3.8.6f-6)

δ = 26 degrees (Ref. 1, Figure 10.7.3.8.6f-6)

ω = 0 degrees (assume pile battering not required as per Step 3)

q_s = 0.274 ksf

Convert nominal skin resistance to nominal axial downdrag load.

As per Ref. 1 Article C10.7.3.8.6b, for H-piles the perimeter or "box" area should generally be used to compute the surface area of the pile side.

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Perimeter of HP 14x89 pile = 57.05 in = 4.75 ft

Layer		Contributing Layer Thickness at Abutm. (ft)	Surface area of pile side (ft^2)	Load (lbs)	Strength I Load Factor	Service I Load Factor
Existing Fill	1	10.6	50.4	13827	1.10	1.00
Glaciomarine	2	2.9	13.8	22059	1.40	1.00

(Ref. 1 Tables 3.4.1-1 and 3.4.1-2; Ref. 11 Table 8.2)

Total Factored Load, Strength I = 46092 lbs per pile
= 46.1 kips per pile

Total Factored Load, Service I = 35886 lbs per pile
= 35.9 kips per pile

According to Ref. 3, typical factored pile loads (Strength I) are expected to be on the order of 350 to 450 kips per pile depending on pile spacing. A downdrag load of 46 kips (Strength I) or 36 kips (Service I) per pile will be added.

2. Select the preliminary pile size.

Determine the factored applied superstructure vertical dead and live load (P_u) distributed to each pile.

P_u = 496 kips (Ref. 3 plus downdrag from Step 1)

Select the steel pile strength.

F_y = 50 ksi
E = 29,000 ksi

Determine resistance factors (Φ_c and Φ_f) for the structural strength in the upper and lower zones of the pile.

ϕ_{cl} = 0.50 for axial resistance in the lower zone of the pile (Ref. 2, page 5-41)
ϕ_{cu} = 0.70 for axial resistance in the upper zone of the pile (Ref. 2, page 5-42)
ϕ_f = 1.00 for flexural resistance in the upper zone of the pile (Ref. 2, page 5-42)

Determine the maximum required nominal axial pile resistance (Ref. 1, Article 6.9.2.1).

$$R_{n.upper} = \frac{P_u}{\phi_{cu}}$$

$R_{n.upper}$ = 709 kips

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$$R_{n.lower} = \frac{P_u}{\phi_{cl}}$$

$R_{n.lower}$ = 992 kips

$$R_n = \max\left(R_{n.upper}, R_{n.lower}\right)$$

R_n = 992 kips

Use the required nominal axial pile resistance to estimate the required pile area.

$$A_{s.req} = \frac{R_n}{0.80\ Fy}$$ (Ref. 2, page 5-42)

$A_{s.req}$ = 24.8 in^2

Select a pile size with an area of $A_{s.req}$ or greater.

Preferred selection is HP 14x89 based on June 16, 2020 meeting with MaineDOT and HNTB.
Check that preferred selection satisfies pile area requirement:

HP 14x89 A_s = 26.1 in^2 (Ref. 4, Table 5.6.3)

A_s > $A_{s.req}$ OK

3. Use LPile analysis to determine the pile unbraced length and maximum moment at the top of the pile.

The following input parameters were used in the LPile analysis:

Pile Properties

Section type:	Steel H Section Weak Axis		(Assumption 1)
Length of section:	16.3	ft	(piles driven to bedrock with no rock socketing)
Flange width, b:	14.695	in	(Ref. 4, Table 5.6.3)
Section depth, d:	13.83	in	(Ref. 4, Table 5.6.3)
Flange thickness, t_f:	0.615	in	(Ref. 4, Table 5.6.3)
Web thickness, t_w:	0.615	in	(Ref. 4, Table 5.6.3)
Pile batter:	Vertical		(pile battering not required)

Pile Loading

Lateral deflection normal to pile axis, y:	0.7	in	(Ref. 3)
Axial load:	496,000	lbs	(Ref. 3)

Soil Layers

Layer	Depth below base of abutment[1]	Lateral Model	Effective Unit Weight (pcf)	Undrained Shear Strength (psf)[2]	Friction Angle (°)[2]	Subgrade Modulus (pci)[3]	Major Principal Strain at 50%[3]	UCS (psi)[2]
Existing Fill (above water table)	0 - 9.8 ft	Sand (Reese)	125	-	32	124.8	-	-
Existing Fill (below water table)	9.8 - 10.6 ft	Sand (Reese)	62.6	-	32	75.5	-	-
Glaciomarine Silty Clay	10.6 - 13.5 ft	Stiff Clay w/o Free Water (Reese)	62.6	1600	-	-	0.005	-
Sand and Gravel	13.5 - 16.3 ft	Sand (Reese)	62.6	-	37	40.5	-	-
Bedrock	>16.3 ft	Strong Rock (Vuggy Limestone)	101.6	-	-	-	-	12983

1) Ref. 5
2) Ref. 6
3) Ref. 7

The full LPile output is provided in Attachment 1.

Obtain the maximum moment at the top of the pile.

$M_{u.Top}$ = 2395 in-kips (LPile)

Obtain the unbraced lengths of the top segment and the second segment of the upper zone of the pile.

$l_{b.top}$ = 4.68 ft (LPile)
$l_{b.top}$ = 56.14 in

$l_{b.2nd}$ = 11.62 ft (LPile)
$l_{b.2nd}$ = 139.46 in

4. Determine if the applied moment on the pile will cause pile head plastic deformation by using the interaction of combined axial and flexural load effects on a single pile.

Determine K values for the top and bottom of the pile and calculate the column slenderness factor (λ) for each segment.

For the top segment (fixed at top and pinned at bottom):

$$\lambda_{top} = \frac{K_{top} l_{b.top}}{r_y} \leq 120$$ (Ref. 1, Article 6.9.3)

$$r_y = \sqrt{I_{yy}/A_s}$$

where:

K_{top} = 1.2 (Ref. 1, Table C4.6.2.5-1)

I_{yy} = 326 in^4 (Ref. 4, Table 5.6.3)

r_y = 3.53 in

λ_{top} = 19.06 OK

For the second segment (pinned at top and bottom):

$$\lambda_{2nd} = \frac{K_{2nd} l_{b.2nd}}{r_y} \leq 120$$ (Ref. 1, Article 6.9.3)

where:

K_{2nd} = 1.0 (Ref. 1, Table C4.6.2.5-1)

λ_{2nd} = 39.46 OK

Calculate the critical elastic buckling resistance, P_e, and the nominal yield resistance, P_o.

Use Ref. 1 Table 6.9.4.1.1-1 to select equation for P_e based on cross-section shape and potential buckling mode.

$$P_e = \frac{\pi^2 E}{\left(\frac{K\, l_b}{r_y}\right)^2} A_s$$ (Ref. 1, Eqn 6.9.4.1.2-1)

$P_{e.top}$ = 20562 kips

$P_{e.2nd}$ = 4797 kips

$$P_o = F_y A_s$$ (Ref. 1, Article 6.9.4.1)

P_o = 1305 kips

Calculate the nominal structural pile resistance, P_n, for both segments of the upper zone of the pile as well as the lower zone of the pile.

Determine P_o/P_e to select equation for P_n as per Ref. 1 Article 6.9.4.1.

$P_o/P_{e.top}$ = 0.06 ≤ 2.25
$P_o/P_{e.2nd}$ = 0.27 ≤ 2.25

thus use Ref. 1 Eqn 6.9.4.1.1-1:

$$P_n = \left[0.658^{\left(\frac{P_o}{P_e}\right)}\right] P_o$$

$P_{n.top}$ = 1271 kips
$P_{n.2nd}$ = 1165 kips

$$P_{n.bottom} = (0.658^{(0)}) \times F_y A_s$$ (0 for a fully braced pile - Ref. 8, Appendix B, Eqn 6-9)

$P_{n.bottom}$ = 1305 kips

Calculate the factored structural pile resistance, P_r, for both segments of the upper zone of the pile as well as the lower zone of the pile.

$$P_{r.top} = \phi_{cu}\, P_{n.top}$$

$P_{r.top}$ = 889.6 kips

$$P_{r.2nd} = \phi_{cu}\, P_{n.2nd}$$

$P_{r.2nd}$ = 815.2 kips

$$P_{r.bottom} = \phi_{cl}\, P_{n.bottom}$$

$P_{r.bottom}$ = 652.5 kips

Compare the ratio of P_u to the structural resistance in the upper portion of the pile – the pile size should be such that the ratio is not less than 0.20.

$$\frac{P_u}{P_{r.top}} = \quad 0.56 \quad \text{OK}$$

$$\frac{P_u}{P_{r.2nd}} = \quad 0.61 \quad \text{OK}$$

Since the lower zone of the pile will have virtually no moment, the entire section can carry the required vertical loads. Make sure the applied load will not exceed the resistance of the lower zone.

$$\text{Check}\left(\frac{P_u}{P_{r.bottom}} < 1\right)$$

$\frac{P_u}{P_{r.bottom}} =$ 0.76 OK

Determine the nominal and factored flexural resistance about H-Pile weak axis (LRFD 6.12.2.2).

Slenderness ratio for the flange:

$$\lambda_f = \frac{b_f}{2t_f}$$ (Ref. 1, Eqn 6.12.2.2.1-3)

$\lambda_f =$ 11.95

Limiting slenderness ratio for a compact flange:

$$\lambda_{pf} = 0.38\sqrt{\frac{E}{F_y}}$$ (Ref. 1, Eqn 6.12.2.2.1-4)

$\lambda_{pf} =$ 9.15

Limiting slenderness ratio for a noncompact flange:

$$\lambda_{rf} = 0.83\sqrt{\frac{E}{F_y}}$$ (Ref. 1, Eqn 6.12.2.2.1-5)

$\lambda r_f =$ 19.99

Elastic and plastic section moduli about the weak axis:

$$S_y = \frac{I_{yy}}{b/2}$$

$$Z_y = (b^2 t_f)/2 + 0.25t_w^2(d - 2t_f)$$

$S_y =$ 44.4 in^3

$Z_y =$ 67.6 in^3

Nominal flexural resistance:

$$M_n = M_p = (F_y Z_y) \qquad \text{if } \lambda_f \le \lambda_{pf}$$ (Ref. 1, Eqn 6.12.2.2.1-1)

$$M_n = \left[1 - \left(1 - \frac{S_y}{Z_y}\right)\left(\frac{\lambda_f - \lambda_{pf}}{0.45\sqrt{\frac{E}{F_y}}}\right)\right] F_y Z_y \qquad \text{if } \lambda_{pf} < \lambda_f \le \lambda_{rf}$$

(Ref. 1, Eqn 6.12.2.2.1-2)

Since $\lambda_{pf} < \lambda_f \leq \lambda_{rf}$,
$M_n =$ 3080 in-kips

Factored flexural resistance:
$\phi_f =$ 1.00 (Ref. 2, page 5-42)
$$M_r = \phi_f M_n$$
$M_r =$ 3080 in-kips

Calculate the moment that will cause a plastic hinge at the top of the pile, M_p'(Ref. 2, Article 6.9.2.2).

$$M'_p = \frac{9}{8}\left(1 - \frac{P_u}{P_{r.top}}\right) M_r$$ (Ref. 8, Appendix B, Eqn 6-24)

$M_p' =$ 1533 in-kips = 1533046.9 inch-lb

If the applied moment exceeds the moment that would cause a plastic hinge, it can be assumed that the pile head has entered plastic deformation and therefore the moment that can be applied to the pile head cannot exceed M_p'.

$M_{u.Top} =$ 2395 in-kips (From Step 3)
$M_{u.Top}$ > M_p' Plastic Hinge Forms

5. Run a second LPile analysis with displacement, plastic moment (M_p'), and P_u as load conditions, and calculate new unbraced lengths from the moment vs. depth curve. Then repeat Step 4 with the new unbraced lengths.

$l_{b.top} =$ 3.62 ft (LPile)
$l_{b.top} =$ 43.42 in

$l_{b.2nd} =$ 12.68 ft (LPile)
$l_{b.2nd} =$ 152.18 in

$M_{u.2nd} =$ 839.06 in-kips (LPile)

Since a plastic hinge developed at the pile head, the value of K for the top segment becomes 2.1 (Ref. 2, page 5-43).

$K_{top} =$ 2.1 (Ref. 1, Table C4.6.2.5-1)
$K_{2nd} =$ 1.0 (Ref. 1, Table C4.6.2.5-1)

$\lambda_{top} =$ 25.80 < 120 OK
$\lambda_{2nd} =$ 43.06 < 120 OK

$P_{e.top} =$ 11223 kips

$P_{e.2nd}$ = 4029 kips

$P_o/P_{e.top}$ = 0.12 ≤ 2.25 (to select P_n equation)
$P_o/P_{e.2nd}$ = 0.32 ≤ 2.25 (to select P_n equation)

$P_{n.top}$ = 1243 kips
$P_{n.2nd}$ = 1140 kips

$P_{r.top}$ = 870 kips
$P_{r.2nd}$ = 798 kips

$$\frac{P_u}{P_{r.top}} = 0.57 > 0.20 \quad \text{OK}$$

$$\frac{P_u}{P_{r.2nd}} = 0.62 > 0.20 \quad \text{OK}$$

Since the pile is appropriately sized, the second segment of the upper zone of the pile needs to be checked with the interaction equation of LRFD Section 6.9.2.2. It is important that this segment of the pile does not form a plastic hinge. A plastic hinge in this segment will cause the pile to fail.

Check: $$\frac{P_u}{P_{r.2nd}} + \frac{8}{9}\left(\frac{M_{u.2nd}}{M_r}\right) < 1$$ (Ref. 8, Appendix B, Eqn 7-13)

Check: 0.86 < 1 OK

6. Because the piles have weak axis orientation and the flanges resist the shear as opposed to the web, check the maximum shear from the LPile output against the structural shear resistance per AISC G7.

V_u = 35.83 kips (LPile)

AASHTO LRFD does not directly address weak axis shear. This analysis will use the AISC Steel Construction Manual 13th edition (G7) to ensure the pile will not shear under the longitudinal load.

k_v = 1.2 (Ref. 9, Section G2.1)

$$C_v = 1.0 \quad \text{if} \quad b/t_f \leq 1.1\sqrt{k_v E/F_y}$$ (Ref. 9, Eqn. G2-3)

C_v = 1.0

Both flanges will resist shear forces:

$$A_w = 2b_f t_f$$ (Ref. 8, Appendix B, Eqn 7-17)

A_w = 18.07 in^2

$$V_n = 0.6F_y A_w C_V$$ (Ref. 9, Eqn G2-1)

V_n = 542 kips

$$V_r = \Phi_v V_n$$

ϕ_v = 1.00 (Ref. 1, Article 6.5.4.2)

V_r = 542 kips

Check that the shear resistance is sufficient:

V_u < V_r OK

7. Check that the maximum factored applied pile load does not exceed the factored pile drivability resistance.

While driving the pile, the maximum stress that is permitted in the pile is:

$$\sigma_{dr} = 0.9\Phi_{da} F_y$$ (Ref. 8, Appendix B, Eqn 7-22)

ϕ_{da} = 1.00 (Ref. 1, Article 6.5.4.2)

σ_{dr} = 45 ksi

This translates into an ultimate maximum driving force that can be applied to the pile of:

$$P_0 = \sigma_{dr} A_s$$ (Ref. 8, Appendix B, Eqn 7-23)

P_0 = 1175 kips

Calculate the nominal pile driving resistance (R_{ndr}) from the applied load divided by the resistance factor associated with the pile monitoring method. In this design, the pile will be bearing on rock. The driving criteria will be established by dynamic testing.

ϕ_{mon} = 0.65 (Ref. 1, Table 10.5.5.2.3-1)

$$R_{ndr} = \frac{P_u}{\phi_{mon}}$$ (Ref. 8, Appendix B, Eqn 7-25)

R_{ndr} = 763 kips

The nominal pile driving resistance (R_{ndr}) should not exceed the nominal structural pile resistance (P_n) or the maximum driving force (P_0) calculated above.

$P_{n.top}$ = 1243 kips (From Step 5)
$P_{n.2nd}$ = 1140 kips (From Step 5)

Check $R_{ndr} < P_n$: OK
Check $R_{ndr} < P_0$: OK

8. Verify the assumption of a pinned support at the base of the pile by comparing the ratio of the shear and axial forces acting at the pile tip to the factored friction coefficient at the bedrock/pile interface.

V_u at pile tip = 0.59 kips (LPile)
ϕ_v = 1.00 (Ref. 1, Article 6.5.4.2)
$V_{factored}$ at pile tip = 0.59 kips

According to Ref. 3, typical factored pile loads (Strength I) are expected to be on the order of 350 to 450 kips per pile depending on pile spacing. Since unfactored loads are not available at this time, the live load is assumed to be 50% of the factored Strength I load. A minimum factored axial pile load of 350 ÷ 2 (to remove the live load) will be used for this analysis. The analysis should be revisited during final design when actual loads are known.

Minimum P = 175 kips

V / P = 0.003

Friction coefficient, μ = 0.40 (Ref. 1, Table C3.11.5.3-1: steel sheet piles against clean gravel, gravel-sand mixtures, well-graded rock fill with spalls)

Resistance factor = 0.5 (per discussion with MaineDOT)
μ * resistance factor = 0.2

If the shear/axial ratio is less than μ multiplied by the resistance factor, then the chosen pile section can be considered pinned.

V / P	<	μ * resistance factor
0.003	<	0.2

The chosen pile section can be considered pinned.

CONCLUSIONS

The results of the analysis indicate that a maximum moment of 2395 in-kips (200 ft-kips) occurs at the top of the pile under the Strength I load case, with a maximum bridge expansion of 0.7 inches. The results indicate that the depth to bedrock is sufficient for driven piles to limit translation of the pile tip to approximately 0.06 inches, and rock socketing is not anticipated to be required at Abutment 2. HP 14x89 piles will provide adequate support for Abutment 2 based on the anticipated thermal movement. The analysis should be revisited during final design when actual loads are known. A drivability analysis will be performed in a separate package.

Attachment 1

==

LPile for Windows, Version 2019-11.005

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

==

This copy of LPile is being used by:

KAR
Golder Associates, Inc.

Serial Number of Security Device: 239146533

This copy of LPile is licensed for exclusive use by:

GOLDER, Manchester, NH

Use of this program by any entity other than GOLDER, Manchester, NH
is a violation of the software license agreement.

Files Used for Analysis

Path to file locations:
\Users\kroth\Documents\Projects\19126013 MaineDOT I-295 Freeport Exit 20 Merrill Rd Bridge\Pile Design\LPile Southeast Abutment\

Name of input data file:
Freeport Exit 20 Southeast Abutment Southern Shift _ No Socket.lp11d

Name of output report file:
Freeport Exit 20 Southeast Abutment Southern Shift _ No Socket.lp11o

Name of plot output file:
Freeport Exit 20 Southeast Abutment Southern Shift _ No Socket.lp11p

Name of runtime message file:
Freeport Exit 20 Southeast Abutment Southern Shift _ No Socket.lp11r

----
Date and Time of Analysis

Date: August 26, 2020 Time: 15:15:18

Problem Title

Project Name: MaineDOT I-295 Exit 20 Merrill Road Bridge No. 5720
Job Number: 19126013
Client: MaineDOT
Engineer: KAR
Description: Southeast Abutment Pile Design - Strength I (No Rock Socket)

Program Options and Settings

Computational Options:
 - Conventional Analysis
Engineering Units Used for Data Input and Computations:
 - US Customary System Units (pounds, feet, inches)

Analysis Control Options:
 - Maximum number of iterations allowed = 500
 - Deflection tolerance for convergence = 1.0000E-05 in
 - Maximum allowable deflection = 100.0000 in
 - Number of pile increments = 100

Loading Type and Number of Cycles of Loading:
 - Static loading specified

 - Use of p-y modification factors for p-y curves not selected
 - Analysis uses layering correction (Method of Georgiadis)
 - No distributed lateral loads are entered
 - Loading by lateral soil movements acting on pile not selected
 - Input of shear resistance at the pile tip not selected
 - Input of moment resistance at the pile tip not selected
 - Input of side resistance moment along pile not selected
 - Computation of pile-head foundation stiffness matrix not selected
 - Push-over analysis of pile not selected
 - Buckling analysis of pile not selected

Output Options:
 - Output files use decimal points to denote decimal symbols.
 - Values of pile-head deflection, bending moment, shear force, and
   soil reaction are printed for full length of pile.
 - Printing Increment (nodal spacing of output points) = 1
 - No p-y curves to be computed and reported for user-specified depths
 - Print using wide report formats

Pile Structural Properties and Geometry

Number of pile sections defined = 1
Total length of pile = 16.300 ft
Depth of ground surface below top of pile = 0.0000 ft

Pile diameters used for p-y curve computations are defined using 2 points.

p-y curves are computed using pile diameter values interpolated with depth over the length of the pile. A summary of values of pile diameter vs. depth follows.

Point No.	Depth Below Pile Head feet	Pile Diameter inches
1	0.000	14.6950
2	16.300	14.6950

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is a H weak axis steel pile
Length of section = 16.300000 ft
Pile width = 13.830000 in
Shear capacity of section = 0.0000 lbs

Ground Slope and Pile Batter Angles

Ground Slope Angle = 0.000 degrees
= 0.000 radians

Pile Batter Angle = 0.000 degrees
= 0.000 radians

Soil and Rock Layering Information

The soil profile is modelled using 5 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer = 0.0000 ft
Distance from top of pile to bottom of layer = 9.800000 ft

Effective unit weight at top of layer = 125.000000 pcf
Effective unit weight at bottom of layer = 125.000000 pcf
Friction angle at top of layer = 32.000000 deg.
Friction angle at bottom of layer = 32.000000 deg.
Subgrade k at top of layer = 124.800000 pci
Subgrade k at bottom of layer = 124.800000 pci

Layer 2 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer = 9.800000 ft
Distance from top of pile to bottom of layer = 10.600000 ft
Effective unit weight at top of layer = 62.600000 pcf
Effective unit weight at bottom of layer = 62.600000 pcf
Friction angle at top of layer = 32.000000 deg.
Friction angle at bottom of layer = 32.000000 deg.
Subgrade k at top of layer = 75.500000 pci
Subgrade k at bottom of layer = 75.500000 pci

Layer 3 is stiff clay without free water

Distance from top of pile to top of layer = 10.600000 ft
Distance from top of pile to bottom of layer = 13.500000 ft
Effective unit weight at top of layer = 62.600000 pcf
Effective unit weight at bottom of layer = 62.600000 pcf
Undrained cohesion at top of layer = 1600. psf
Undrained cohesion at bottom of layer = 1600. psf
Epsilon-50 at top of layer = 0.005000
Epsilon-50 at bottom of layer = 0.005000

Layer 4 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer = 13.500000 ft
Distance from top of pile to bottom of layer = 16.300000 ft
Effective unit weight at top of layer = 62.600000 pcf
Effective unit weight at bottom of layer = 62.600000 pcf
Friction angle at top of layer = 37.000000 deg.
Friction angle at bottom of layer = 37.000000 deg.
Subgrade k at top of layer = 40.500000 pci
Subgrade k at bottom of layer = 40.500000 pci

Layer 5 is strong rock (vuggy limestone)

Distance from top of pile to top of layer = 16.300000 ft
Distance from top of pile to bottom of layer = 50.000000 ft
Effective unit weight at top of layer = 101.600000 pcf
Effective unit weight at bottom of layer = 101.600000 pcf
Uniaxial compressive strength at top of layer = 12983. psi
Uniaxial compressive strength at bottom of layer = 12983. psi

(Depth of the lowest soil layer extends 33.700 ft below the pile tip)

Summary of Input Soil Properties

Layer Num.	Soil Type Name (p-y Curve Type)	Layer Depth ft	Effective Unit Wt. pcf	Undrained Cohesion psf	Angle of Friction deg.	Uniaxial qu psi	E50 or krm	kpy pci
1	Sand	0.00	125.0000	--	32.0000	--	--	124.8000
	(Reese, et al.)	9.8000	125.0000	--	32.0000	--	--	124.8000
2	Sand	9.8000	62.6000	--	32.0000	--	--	75.5000
	(Reese, et al.)	10.6000	62.6000	--	32.0000	--	--	75.5000
3	Stiff Clay	10.6000	62.6000	1600.	--	--	0.00500	--
	w/o Free Water	13.5000	62.6000	1600.	--	--	0.00500	--
4	Sand	13.5000	62.6000	--	37.0000	--	--	40.5000
	(Reese, et al.)	16.3000	62.6000	--	37.0000	--	--	40.5000
5	Strong Rock	16.3000	101.6000	--	--	12983.	--	--
	(Vuggy Limestone)	50.0000	101.6000	--	--	12983.	--	--

Static Loading Type

Static loading criteria were used when computing p-y curves for all analyses.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load No.	Load Type	Condition 1	Condition 2	Axial Thrust Force, lbs	Compute Top y vs. Pile Length	Run Analysis
1	5	y = -0.700000 in	S = 0.0000 in/in	496000.	N.A.	Yes

V = shear force applied normal to pile axis
M = bending moment applied to pile head
y = lateral deflection normal to pile axis
S = pile slope relative to original pile batter angle
R = rotational stiffness applied to pile head
Values of top y vs. pile lengths can be computed only for load types with specified shear loading (Load Types 1, 2, and 3).
Thrust force is assumed to be acting axially for all pile batter angles.

Computations of Nominal Moment Capacity and Nonlinear Bending Stiffness
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Axial thrust force values were determined from pile-head loading conditions

Number of Pile Sections Analyzed = 1

Pile Section No. 1:

Dimensions and Properties of Steel H Weak Axis:

Length of Section	=	16.300000 ft
Flange Width	=	14.695000 in
Section Depth	=	13.830000 in
Flange Thickness	=	0.615000 in
Web Thickness	=	0.615000 in
Yield Stress of Pipe	=	50.000000 ksi
Elastic Modulus	=	29000. ksi
Cross-sectional Area	=	25.823850 sq. in.
Moment of Inertia	=	325.505721 in^4
Elastic Bending Stiffness	=	9439666. kip-in^2
Plastic Modulus, Z	=	67.593889in^3
Plastic Moment Capacity = Fy Z	=	3380.in-kip

Axial Structural Capacities:

Nom. Axial Structural Capacity = Fy As	=	1291.193 kips
Nominal Axial Tensile Capacity	=	-1291.193 kips

Number of Axial Thrust Force Values Determined from Pile-head Loadings = 1

Number	Axial Thrust Force kips
1	496.000

Definition of Run Messages:

Y = part of pipe section has yielded.

Axial Thrust Force = 496.000 kips

Bending Curvature rad/in.	Bending Moment in-kip	Bending Stiffness kip-in2	Depth to N Axis in	Max Total Stress ksi	Run Msg

0.00000410	38.7092707	9438779.	168.8441446	20.0721621	
0.00000820	77.4185413	9438779.	88.0958223	20.9372734	
0.00001230	116.1278120	9438779.	61.1797149	21.8023847	
0.00001640	154.8370827	9438779.	47.7216612	22.6674960	
0.00002051	193.5463533	9438779.	39.6468289	23.5326073	
0.00002461	232.2556240	9438779.	34.2636074	24.3977186	
0.00002871	270.9648947	9438779.	30.4184492	25.2628299	
0.00003281	309.6741653	9438779.	27.5345806	26.1279412	
0.00003691	348.3834360	9438779.	25.2915716	26.9930524	
0.00004101	387.0927067	9438779.	23.4971645	27.8581637	
0.00004511	425.8019773	9438779.	22.0290131	28.7232750	
0.00004921	464.5112480	9438779.	20.8055537	29.5883863	
0.00005331	503.2205187	9438779.	19.7703188	30.4534976	
0.00005742	541.9297893	9438779.	18.8829746	31.3186089	
0.00006152	580.6390600	9438779.	18.1139430	32.1837202	
0.00006562	619.3483307	9438779.	17.4410403	33.0488313	
0.00006972	658.0576013	9438779.	16.8473026	33.9139426	
0.00007382	696.7668720	9438779.	16.3195358	34.7790540	
0.00007792	735.4761427	9438779.	15.8473234	35.6441652	
0.00008202	774.1854133	9438779.	15.4223322	36.5092765	
0.00008612	812.8946840	9438779.	15.0378164	37.3743878	
0.00009022	851.6039547	9438779.	14.6882566	38.2394991	
0.00009433	890.3132253	9438779.	14.3690932	39.1046104	
0.00009843	929.0224960	9438779.	14.0765269	39.9697217	
0.0001025	967.7317667	9438779.	13.8073658	40.8348329	
0.0001066	1006.	9438779.	13.5589094	41.6999442	
0.0001107	1045.	9438779.	13.3288572	42.5650555	
0.0001148	1084.	9438779.	13.1152373	43.4301668	
0.0001189	1123.	9438779.	12.9163498	44.2952781	
0.0001230	1161.	9438779.	12.7307215	45.1603894	
0.0001271	1200.	9438779.	12.5570692	46.0255007	
0.0001312	1239.	9438779.	12.3942701	46.8906120	
0.0001353	1277.	9438779.	12.2413377	47.7557232	
0.0001394	1316.	9438779.	12.0974013	48.6208345	
0.0001435	1355.	9438779.	11.9616898	49.4859458	
0.0001476	1393.	9435630.	11.8340958	50.0000000	Y
0.0001517	1430.	9423189.	11.7151548	50.0000000	Y
0.0001558	1465.	9401682.	11.6043273	50.0000000	Y
0.0001599	1499.	9372940.	11.5008180	50.0000000	Y
0.0001681	1564.	9298986.	11.3131408	50.0000000	Y
0.0001763	1624.	9208132.	11.1479422	50.0000000	Y
0.0001845	1680.	9105392.	11.0018488	50.0000000	Y
0.0001928	1734.	8994798.	10.8720396	50.0000000	Y
0.0002010	1784.	8879596.	10.7561309	50.0000000	Y
0.0002092	1833.	8761920.	10.6522148	50.0000000	Y
0.0002174	1879.	8642614.	10.5589058	50.0000000	Y
0.0002256	1923.	8523842.	10.4746073	50.0000000	Y
0.0002338	1965.	8406198.	10.3982652	50.0000000	Y
0.0002420	2006.	8289664.	10.3291161	50.0000000	Y
0.0002502	2045.	8175461.	10.2661194	50.0000000	Y
0.0002584	2083.	8063777.	10.2086111	50.0000000	Y
0.0002666	2121.	7954765.	10.1560061	50.0000000	Y
0.0002748	2157.	7848546.	10.1077868	50.0000000	Y
0.0002830	2192.	7745213.	10.0634927	50.0000000	Y

0.0002912	2226.	7644839.	10.0227128	50.0000000	Y
0.0002994	2260.	7547473.	9.9850782	50.0000000	Y
0.0003076	2292.	7453149.	9.9502567	50.0000000	Y
0.0003158	2325.	7361353.	9.9181596	50.0000000	Y
0.0003240	2356.	7272323.	9.8884391	50.0000000	Y
0.0003322	2387.	7186286.	9.8607593	50.0000000	Y
0.0003404	2418.	7103106.	9.8349468	50.0000000	Y
0.0003486	2448.	7021963.	9.8111543	50.0000000	Y
0.0003568	2477.	6943724.	9.7888115	50.0000000	Y
0.0003650	2505.	6863933.	9.7667827	50.0000000	Y
0.0003732	2532.	6783309.	9.7452225	50.0000000	Y
0.0003814	2556.	6702787.	9.7239014	50.0000000	Y
0.0003896	2580.	6621989.	9.7032218	50.0000000	Y
0.0003978	2602.	6542068.	9.6827615	50.0000000	Y
0.0004060	2624.	6461906.	9.6627498	50.0000000	Y
0.0004142	2644.	6382452.	9.6428665	50.0000000	Y
0.0004224	2663.	6304050.	9.6235040	50.0000000	Y
0.0004306	2681.	6226470.	9.6045323	50.0000000	Y
0.0004388	2699.	6149752.	9.5854694	50.0000000	Y
0.0004470	2715.	6074095.	9.5672717	50.0000000	Y
0.0004552	2731.	5999452.	9.5490446	50.0000000	Y
0.0004634	2746.	5926182.	9.5310259	50.0000000	Y
0.0004716	2761.	5853963.	9.5136901	50.0000000	Y
0.0004798	2775.	5782700.	9.4962739	50.0000000	Y
0.0004880	2788.	5713420.	9.4792601	50.0000000	Y
0.0005208	2837.	5446310.	9.4137545	50.0000000	Y
0.0005536	2878.	5198257.	9.3520700	50.0000000	Y
0.0005865	2914.	4968699.	9.2942150	50.0000000	Y
0.0006193	2945.	4756136.	9.2394863	50.0000000	Y
0.0006521	2973.	4559738.	9.1877218	50.0000000	Y
0.0006849	2998.	4376891.	9.1388079	50.0000000	Y
0.0007177	3020.	4207432.	9.0920845	50.0000000	Y
0.0007505	3039.	4049624.	9.0481902	50.0000000	Y
0.0007833	3057.	3902770.	9.0061951	50.0000000	Y
0.0008161	3073.	3765724.	8.9663316	50.0000000	Y
0.0008489	3088.	3637422.	8.9284493	50.0000000	Y
0.0008817	3101.	3517110.	8.8919693	50.0000000	Y
0.0009145	3113.	3404384.	8.8571154	50.0000000	Y

Summary of Results for Nominal Moment Capacity for Section 1

Load No.	Axial Thrust kips	Nominal Moment Capacity in-kips
1	496.0000000000	3113.

Note that the values in the above table are not factored by a strength reduction factor for LRFD.

The value of the strength reduction factor depends on the provisions of the LRFD code being followed.

The above values should be multiplied by the appropriate strength reduction factor to compute ultimate moment capacity according to the LRFD structural design standard being followed.

Layering Correction Equivalent Depths of Soil & Rock Layers

Layer No.	Top of Layer Below Pile Head ft	Equivalent Top Depth Below Grnd Surf ft	Same Layer Type As Layer Above	Layer is Rock or is Below Rock Layer	F0 Integral for Layer lbs	F1 Integral for Layer lbs
1	0.00	0.00	N.A.	No	0.00	102493.
2	9.8000	9.8000	Yes	No	102493.	23660.
3	10.6000	11.2547	No	No	126153.	50452.
4	13.5000	10.8890	No	No	176605.	151498.
5	16.3000	16.3000	No	Yes	N.A.	N.A.

Notes: The F0 integral of Layer n+1 equals the sum of the F0 and F1 integrals for Layer n. Layering correction equivalent depths are computed only for soil types with both shallow-depth and deep-depth expressions for peak lateral load transfer. These soil types are soft and stiff clays, non-liquefied sands, and cemented c-phi soil.

Computed Values of Pile Loading and Deflection
for Lateral Loading for Load Case Number 1

Pile-head conditions are Displacement and Pile-head Rotation (Loading Type 5)
Displacement of pile head = -0.700000 inches
Rotation of pile head = 0.000E+00 radians
Axial load on pile head = 496000.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness lb-in^2	Soil Res. p lb/inch	Soil Spr. Es*h lb/inch	Distrib. Lat. Load lb/inch
0.00	-0.7000	2395316.	-46117.	0.00	73275.	7.16E+09	0.00	0.00	0.00
0.1630	-0.6994	2304842.	-46074.	6.42E-04	71233.	7.16E+09	18.4998	51.7411	0.00
0.3260	-0.6975	2213828.	-46018.	0.00124	69179.	7.68E+09	39.0784	109.5891	0.00
0.4890	-0.6945	2122417.	-45920.	0.00178	67115.	7.95E+09	61.3826	172.8751	0.00

0.6520	-0.6905	2030733.	-45777.	0.00228	65046.	8.22E+09	85.0148	240.8168	0.00
0.8150	-0.6856	1938906.	-45586.	0.00275	62973.	8.48E+09	109.4491	312.2651	0.00
0.9780	-0.6798	1847063.	-45348.	0.00318	60900.	8.72E+09	134.4094	386.7593	0.00
1.1410	-0.6731	1755333.	-45061.	0.00358	58829.	8.94E+09	159.5225	463.5400	0.00
1.3040	-0.6658	1663841.	-44724.	0.00395	56764.	9.13E+09	184.6472	542.4927	0.00
1.4670	-0.6577	1572710.	-44338.	0.00429	54707.	9.28E+09	209.5562	623.2331	0.00
1.6300	-0.6490	1482059.	-43905.	0.00461	52661.	9.39E+09	233.6949	704.3637	0.00
1.7930	-0.6396	1392002.	-43425.	0.00491	50628.	9.44E+09	256.7102	785.0135	0.00
1.9560	-0.6297	1302648.	-42900.	0.00519	48611.	9.44E+09	280.4889	871.1991	0.00
2.1190	-0.6193	1214105.	-42328.	0.00545	46613.	9.44E+09	303.9701	960.0138	0.00
2.2820	-0.6084	1126481.	-41712.	0.00569	44635.	9.44E+09	326.1500	1049.	0.00
2.4450	-0.5971	1039878.	-41054.	0.00592	42680.	9.44E+09	346.7462	1136.	0.00
2.6080	-0.5853	954393.	-40355.	0.00613	40750.	9.44E+09	367.4973	1228.	0.00
2.7710	-0.5731	870122.	-39618.	0.00631	38848.	9.44E+09	386.5782	1319.	0.00
2.9340	-0.5606	787155.	-38845.	0.00649	36975.	9.44E+09	403.6889	1409.	0.00
3.0970	-0.5477	705574.	-38041.	0.00664	35134.	9.44E+09	418.7846	1496.	0.00
3.2600	-0.5346	625453.	-37207.	0.00678	33325.	9.44E+09	433.3782	1586.	0.00
3.4230	-0.5212	546865.	-36348.	0.00690	31551.	9.44E+09	445.6801	1673.	0.00
3.5860	-0.5076	469872.	-35466.	0.00701	29813.	9.44E+09	455.4981	1755.	0.00
3.7490	-0.4938	394528.	-34563.	0.00710	28113.	9.44E+09	467.5723	1852.	0.00
3.9120	-0.4798	320893.	-33633.	0.00717	26450.	9.44E+09	483.4460	1971.	0.00
4.0750	-0.4657	249043.	-32674.	0.00723	24829.	9.44E+09	497.5349	2090.	0.00
4.2380	-0.4515	179046.	-31689.	0.00727	23249.	9.44E+09	509.6682	2208.	0.00
4.4010	-0.4373	110964.	-30682.	0.00730	21712.	9.44E+09	519.6766	2325.	0.00
4.5640	-0.4230	44847.	-29658.	0.00732	20219.	9.44E+09	527.3937	2439.	0.00
4.7270	-0.4087	-19261.	-28621.	0.00732	19642.	9.44E+09	532.6553	2550.	0.00
4.8900	-0.3943	-81327.	-27577.	0.00731	21043.	9.44E+09	535.3005	2655.	0.00
5.0530	-0.3801	-141329.	-26516.	0.00729	22397.	9.44E+09	549.8223	2830.	0.00
5.2160	-0.3658	-199198.	-25426.	0.00725	23703.	9.44E+09	564.0411	3016.	0.00
5.3790	-0.3517	-254870.	-24310.	0.00721	24960.	9.44E+09	577.1542	3210.	0.00
5.5420	-0.3376	-308282.	-23170.	0.00715	26166.	9.44E+09	589.1064	3413.	0.00
5.7050	-0.3237	-359379.	-22007.	0.00708	27319.	9.44E+09	599.8474	3624.	0.00
5.8680	-0.3099	-408108.	-20824.	0.00700	28419.	9.44E+09	609.3317	3845.	0.00
6.0310	-0.2963	-454424.	-19624.	0.00691	29465.	9.44E+09	617.5187	4076.	0.00
6.1940	-0.2829	-498286.	-18408.	0.00681	30455.	9.44E+09	626.2839	4330.	0.00
6.3570	-0.2697	-539651.	-17173.	0.00670	31388.	9.44E+09	636.5570	4617.	0.00
6.5200	-0.2567	-578473.	-15919.	0.00659	32265.	9.44E+09	646.0238	4923.	0.00
6.6830	-0.2439	-614707.	-14646.	0.00646	33083.	9.44E+09	654.6929	5250.	0.00
6.8460	-0.2314	-648312.	-13358.	0.00633	33841.	9.44E+09	662.3278	5599.	0.00
7.0090	-0.2191	-679253.	-12057.	0.00620	34540.	9.44E+09	668.7172	5969.	0.00
7.1720	-0.2072	-707499.	-10744.	0.00605	35177.	9.44E+09	673.8212	6362.	0.00
7.3350	-0.1955	-733024.	-9422.	0.00590	35753.	9.44E+09	677.6026	6780.	0.00
7.4980	-0.1841	-755810.	-8094.	0.00575	36268.	9.44E+09	680.0262	7226.	0.00
7.6610	-0.1730	-775842.	-6763.	0.00559	36720.	9.44E+09	681.0593	7701.	0.00
7.8240	-0.1622	-793113.	-5431.	0.00543	37110.	9.44E+09	680.6712	8208.	0.00
7.9870	-0.1518	-807620.	-4102.	0.00526	37437.	9.44E+09	678.8330	8750.	0.00
8.1500	-0.1416	-819367.	-2777.	0.00509	37702.	9.44E+09	675.5175	9329.	0.00
8.3130	-0.1318	-828365.	-1461.	0.00492	37905.	9.44E+09	670.6982	9951.	0.00
8.4760	-0.1224	-834631.	-154.8362	0.00475	38047.	9.44E+09	664.3498	10619.	0.00
8.6390	-0.1133	-838186.	1137.	0.00458	38127.	9.44E+09	656.4466	11338.	0.00
8.8020	-0.1045	-839062.	2412.	0.00440	38147.	9.44E+09	646.9621	12113.	0.00
8.9650	-0.09603	-837294.	3666.	0.00423	38107.	9.44E+09	635.8684	12952.	0.00
9.1280	-0.08793	-832925.	4898.	0.00406	38008.	9.44E+09	623.1341	13862.	0.00
9.2910	-0.08017	-826004.	6102.	0.00388	37852.	9.44E+09	608.7235	14853.	0.00

9.4540	-0.07274	-816589.	7277.	0.00371	37640.	9.44E+09	592.5941	15936.	0.00
9.6170	-0.06564	-804742.	8419.	0.00355	37372.	9.44E+09	574.6941	17126.	0.00
9.7800	-0.05887	-790534.	9524.	0.00338	37051.	9.44E+09	554.9588	18440.	0.00
9.9430	-0.05241	-774044.	10528.	0.00322	36679.	9.44E+09	472.1707	17620.	0.00
10.1060	-0.04628	-755592.	11404.	0.00306	36263.	9.44E+09	423.7137	17909.	0.00
10.2690	-0.04045	-735367.	12187.	0.00291	35806.	9.44E+09	376.2933	18198.	0.00
10.4320	-0.03491	-713555.	12877.	0.00276	35314.	9.44E+09	329.9678	18487.	0.00
10.5950	-0.02967	-690336.	13479.	0.00261	34790.	9.44E+09	284.7844	18776.	0.00
10.7580	-0.02470	-665889.	14169.	0.00247	34238.	9.44E+09	420.8922	33326.	0.00
10.9210	-0.02001	-639699.	14974.	0.00233	33647.	9.44E+09	402.7103	39367.	0.00
11.0840	-0.01557	-611838.	15741.	0.00220	33018.	9.44E+09	381.4768	47911.	0.00
11.2470	-0.01139	-582395.	16462.	0.00208	32353.	9.44E+09	355.7443	61109.	0.00
11.4100	-0.00744	-551475.	17126.	0.00196	31655.	9.44E+09	322.5140	84838.	0.00
11.5730	-0.00371	-519209.	17708.	0.00185	30927.	9.44E+09	273.4106	144217.	0.00
11.7360	-1.91E-04	-485793.	18069.	0.00175	30173.	9.44E+09	95.0213	972292.	0.00
11.8990	0.00313	-451915.	17902.	0.00165	29408.	9.44E+09	-265.1079	165724.	0.00
12.0620	0.00627	-418961.	17334.	0.00156	28664.	9.44E+09	-315.5727	98510.	0.00
12.2250	0.00923	-387130.	16686.	0.00148	27946.	9.44E+09	-347.7602	73671.	0.00
12.3880	0.01204	-356552.	15982.	0.00140	27255.	9.44E+09	-371.6845	60366.	0.00
12.5510	0.01471	-327325.	15236.	0.00133	26596.	9.44E+09	-390.7606	51963.	0.00
12.7140	0.01724	-299526.	14457.	0.00126	25968.	9.44E+09	-406.6119	46127.	0.00
12.8770	0.01965	-273223.	13648.	0.00120	25374.	9.44E+09	-420.1524	41815.	0.00
13.0400	0.02195	-248472.	12815.	0.00115	24816.	9.44E+09	-431.9535	38484.	0.00
13.2030	0.02415	-225324.	11960.	0.00110	24293.	9.44E+09	-442.3990	35824.	0.00
13.3660	0.02626	-203824.	11085.	0.00106	23808.	9.44E+09	-451.7607	33645.	0.00
13.5290	0.02829	-184011.	10461.	0.00102	23361.	9.44E+09	-186.0084	12861.	0.00
13.6920	0.03024	-164872.	10083.	9.81E-04	22929.	9.44E+09	-201.2361	13016.	0.00
13.8550	0.03213	-146470.	9674.	9.48E-04	22513.	9.44E+09	-216.3231	13171.	0.00
14.0180	0.03395	-128867.	9236.	9.20E-04	22116.	9.44E+09	-231.3042	13326.	0.00
14.1810	0.03572	-112122.	8769.	8.95E-04	21738.	9.44E+09	-246.2145	13481.	0.00
14.3440	0.03745	-96297.	8273.	8.73E-04	21381.	9.44E+09	-261.0890	13636.	0.00
14.5070	0.03914	-81452.	7748.	8.55E-04	21046.	9.44E+09	-275.9619	13791.	0.00
14.6700	0.04080	-67645.	7194.	8.39E-04	20734.	9.44E+09	-290.8671	13946.	0.00
14.8330	0.04243	-54939.	6610.	8.27E-04	20447.	9.44E+09	-305.8369	14100.	0.00
14.9960	0.04403	-43391.	5997.	8.17E-04	20186.	9.44E+09	-320.9023	14255.	0.00
15.1590	0.04562	-33062.	5355.	8.09E-04	19953.	9.44E+09	-336.0927	14410.	0.00
15.3220	0.04719	-24013.	4682.	8.03E-04	19749.	9.44E+09	-351.4349	14565.	0.00
15.4850	0.04876	-16303.	3980.	7.99E-04	19575.	9.44E+09	-366.9534	14720.	0.00
15.6480	0.05032	-9994.	3247.	7.96E-04	19433.	9.44E+09	-382.6696	14875.	0.00
15.8110	0.05187	-5147.	2482.	7.94E-04	19323.	9.44E+09	-398.6017	15030.	0.00
15.9740	0.05343	-1824.	1687.	7.94E-04	19248.	9.44E+09	-414.7638	15185.	0.00
16.1370	0.05498	-86.9820	859.6428	7.93E-04	19209.	9.44E+09	-431.1662	15340.	0.00
16.3000	0.05653	0.00	0.00	7.93E-04	19207.	9.44E+09	-447.8141	7748.	0.00

* This analysis computed pile response using nonlinear moment-curvature relationships. Values of total stress due to combined axial and bending stresses are computed only for elastic sections only and do not equal the actual stresses in concrete and steel. Stresses in concrete and steel may be interpolated from the output for nonlinear bending properties relative to the magnitude of bending moment developed in the pile.

Output Summary for Load Case No. 1:

Pile-head deflection = -0.70000000 inches
Computed slope at pile head = 0.000000 radians
Maximum bending moment = 2395316. inch-lbs
Maximum shear force = -46117. lbs
Depth of maximum bending moment = 0.000000 feet below pile head
Depth of maximum shear force = 0.000000 feet below pile head
Number of iterations = 11
Number of zero deflection points = 1

Summary of Pile-head Responses for Conventional Analyses

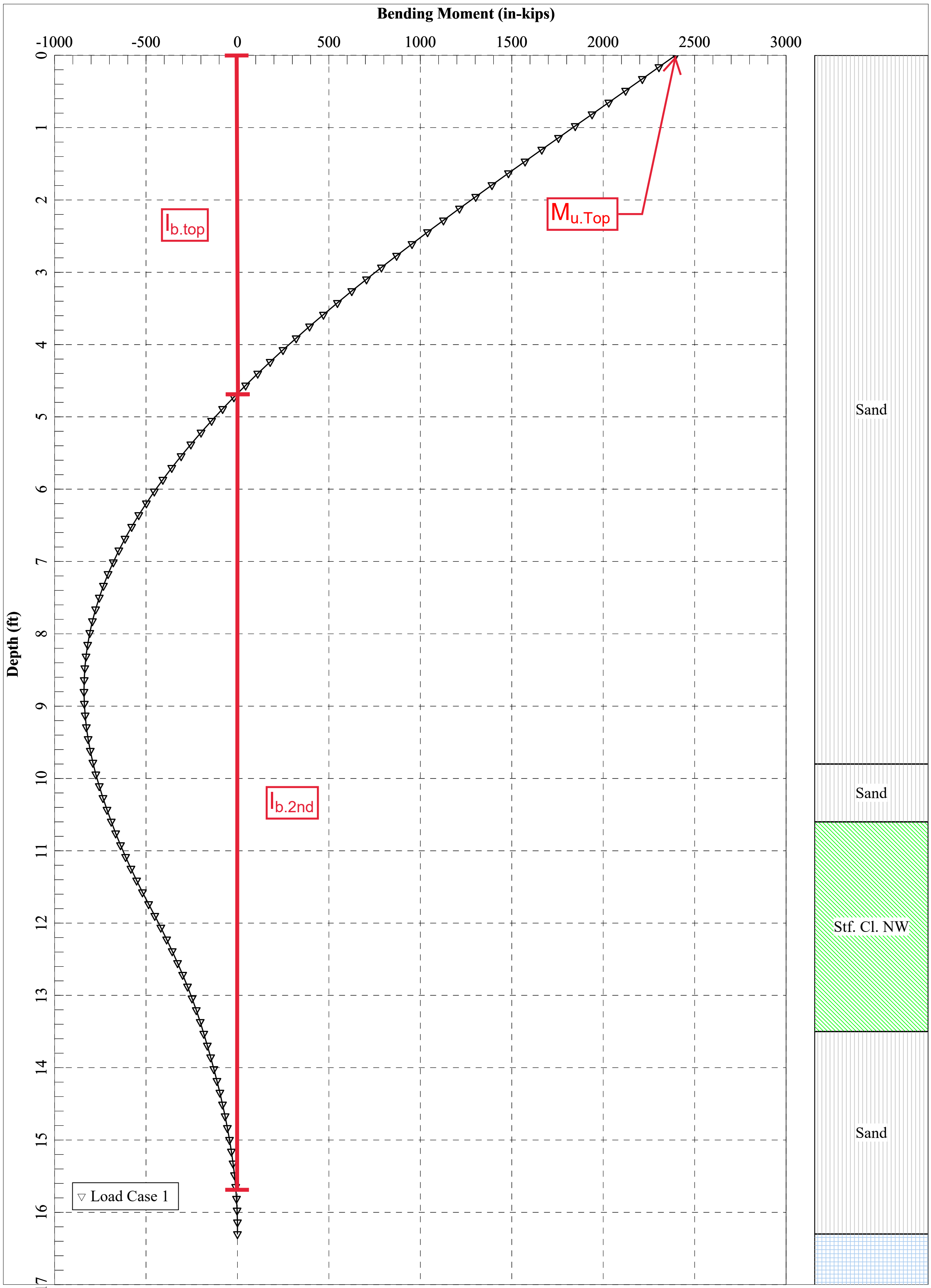
Definitions of Pile-head Loading Conditions:

Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians
Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
Load Type 5: Load 1 = Top Deflection, y, inches, and Load 2 = Slope, S, radians

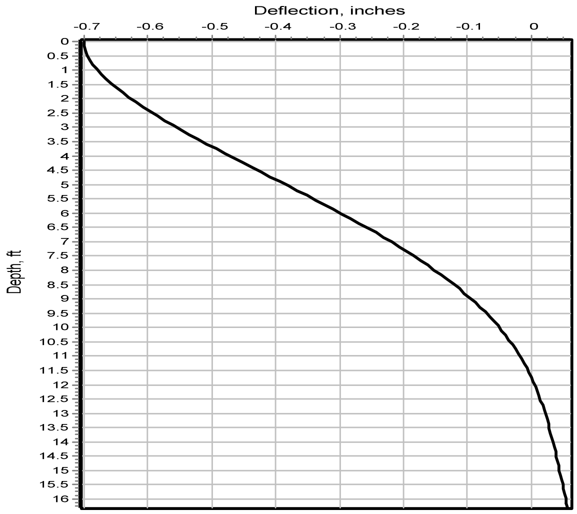
Load Case No.	Load Type 1	Pile-head Load 1	Load Type 2	Pile-head Load 2	Axial Loading lbs	Pile-head Deflection inches	Pile-head Rotation radians	Max Shear in Pile lbs	Max Moment in Pile in-lbs
1	y, in	-0.7000	S, rad	0.00	496000.	-0.7000	0.00	-46117.	2395316.

Maximum pile-head deflection = -0.7000000000 inches
Maximum pile-head rotation = 0.0000000000 radians = 0.000000 deg.

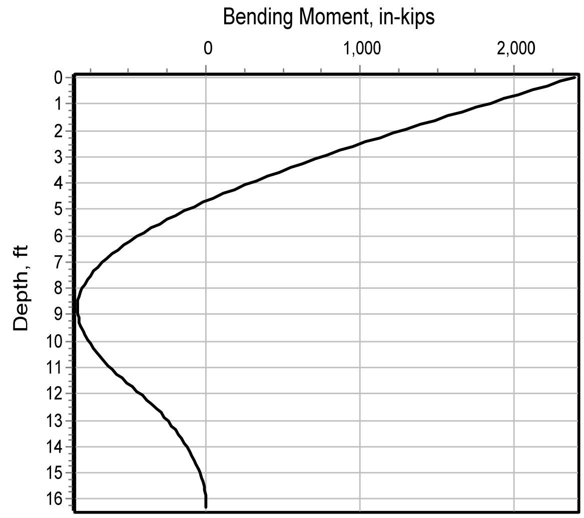
The analysis ended normally.



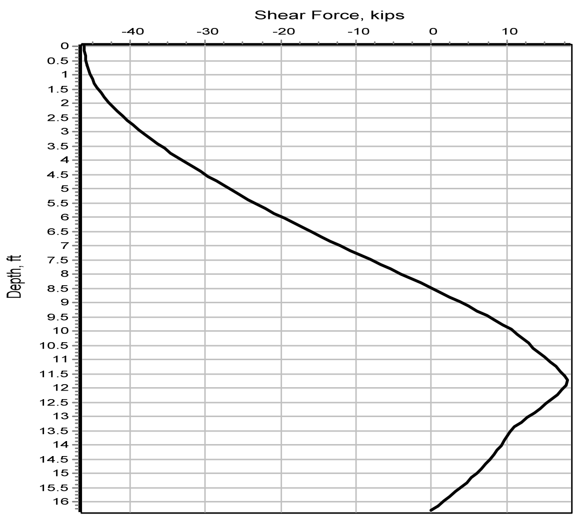
Bending Moment (in-kips)
-1000
-500
0
500
1000
1500
2000
2500
3000
Depth (ft)
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
$I_{b.top}$
$M_{u.Top}$
$I_{b.2nd}$
Load Case 1
Sand
Sand
Stf. Cl. NW
Sand



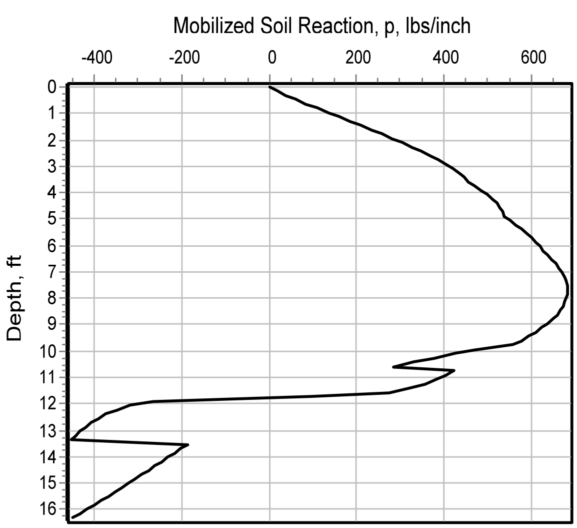
Deflection, inches
-0.7
-0.6
-0.5
-0.4
-0.3
-0.2
-0.1
0
Depth, ft



Bending Moment, in-kips
0
1,000
2,000
Depth, ft



Shear Force, kips
-40
-30
-20
-10
0
10
Depth, ft



Mobilized Soil Reaction, p, lbs/inch
-400
-200
0
200
400
600
Depth, ft

Attachment 2

==

LPile for Windows, Version 2019-11.005

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

==

This copy of LPile is being used by:

KAR
Golder Associates, Inc.

Serial Number of Security Device: 239146533

This copy of LPile is licensed for exclusive use by:

GOLDER, Manchester, NH

Files Used for Analysis

Path to file locations:
\Users\kroth\Documents\Projects\19126013 MaineDOT I-295 Freeport Exit 20 Merrill Rd Bridge\Pile Design\LPile Southeast Abutment\

Name of input data file:
Freeport Exit 20 Southeast Abutment Southern Shift _ No Socket _ Plastic Hinge.lp11d

Name of output report file:
Freeport Exit 20 Southeast Abutment Southern Shift _ No Socket _ Plastic Hinge.lp11o

Name of plot output file:
Freeport Exit 20 Southeast Abutment Southern Shift _ No Socket _ Plastic Hinge.lp11p

Name of runtime message file:
Freeport Exit 20 Southeast Abutment Southern Shift _ No Socket _ Plastic Hinge.lp11r

----
Date and Time of Analysis

Date: August 26, 2020 Time: 15:21:01

Problem Title

Project Name: MaineDOT I-295 Exit 20 Merrill Road Bridge No. 5720
Job Number: 19126013
Client: MaineDOT
Engineer: KAR
Description: Southeast Abutment Pile Design - Strength I (Plastic Hinge)

Program Options and Settings

Computational Options:
- Conventional Analysis

Engineering Units Used for Data Input and Computations:
- US Customary System Units (pounds, feet, inches)

Analysis Control Options:
- Maximum number of iterations allowed = 500
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 100.0000 in
- Number of pile increments = 100

Loading Type and Number of Cycles of Loading:
- Static loading specified

- Use of p-y modification factors for p-y curves not selected
- Analysis uses layering correction (Method of Georgiadis)
- No distributed lateral loads are entered
- Loading by lateral soil movements acting on pile not selected
- Input of shear resistance at the pile tip not selected
- Input of moment resistance at the pile tip not selected
- Input of side resistance moment along pile not selected
- Computation of pile-head foundation stiffness matrix not selected
- Push-over analysis of pile not selected
- Buckling analysis of pile not selected

Output Options:
- Output files use decimal points to denote decimal symbols.
- Values of pile-head deflection, bending moment, shear force, and
  soil reaction are printed for full length of pile.
- Printing Increment (nodal spacing of output points) = 1
- No p-y curves to be computed and reported for user-specified depths
- Print using wide report formats

Pile Structural Properties and Geometry

Number of pile sections defined = 1
Total length of pile = 16.300 ft
Depth of ground surface below top of pile = 0.0000 ft

Pile diameters used for p-y curve computations are defined using 2 points.

p-y curves are computed using pile diameter values interpolated with depth over the length of the pile. A summary of values of pile diameter vs. depth follows.

Point No.	Depth Below Pile Head feet	Pile Diameter inches
1	0.000	14.6950
2	16.300	14.6950

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is a H weak axis steel pile
Length of section = 16.300000 ft
Pile width = 13.830000 in
Shear capacity of section = 0.0000 lbs

Ground Slope and Pile Batter Angles

Ground Slope Angle = 0.000 degrees
= 0.000 radians

Pile Batter Angle = 0.000 degrees
= 0.000 radians

Soil and Rock Layering Information

The soil profile is modelled using 5 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer = 0.0000 ft
Distance from top of pile to bottom of layer = 9.800000 ft

Effective unit weight at top of layer = 125.000000 pcf
Effective unit weight at bottom of layer = 125.000000 pcf
Friction angle at top of layer = 32.000000 deg.
Friction angle at bottom of layer = 32.000000 deg.
Subgrade k at top of layer = 124.800000 pci
Subgrade k at bottom of layer = 124.800000 pci

Layer 2 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer = 9.800000 ft
Distance from top of pile to bottom of layer = 10.600000 ft
Effective unit weight at top of layer = 62.600000 pcf
Effective unit weight at bottom of layer = 62.600000 pcf
Friction angle at top of layer = 32.000000 deg.
Friction angle at bottom of layer = 32.000000 deg.
Subgrade k at top of layer = 75.500000 pci
Subgrade k at bottom of layer = 75.500000 pci

Layer 3 is stiff clay without free water

Distance from top of pile to top of layer = 10.600000 ft
Distance from top of pile to bottom of layer = 13.500000 ft
Effective unit weight at top of layer = 62.600000 pcf
Effective unit weight at bottom of layer = 62.600000 pcf
Undrained cohesion at top of layer = 1600. psf
Undrained cohesion at bottom of layer = 1600. psf
Epsilon-50 at top of layer = 0.005000
Epsilon-50 at bottom of layer = 0.005000

Layer 4 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer = 13.500000 ft
Distance from top of pile to bottom of layer = 16.300000 ft
Effective unit weight at top of layer = 62.600000 pcf
Effective unit weight at bottom of layer = 62.600000 pcf
Friction angle at top of layer = 37.000000 deg.
Friction angle at bottom of layer = 37.000000 deg.
Subgrade k at top of layer = 40.500000 pci
Subgrade k at bottom of layer = 40.500000 pci

Layer 5 is strong rock (vuggy limestone)

Distance from top of pile to top of layer = 16.300000 ft
Distance from top of pile to bottom of layer = 50.000000 ft
Effective unit weight at top of layer = 101.600000 pcf
Effective unit weight at bottom of layer = 101.600000 pcf
Uniaxial compressive strength at top of layer = 12983. psi
Uniaxial compressive strength at bottom of layer = 12983. psi

(Depth of the lowest soil layer extends 33.700 ft below the pile tip)

Summary of Input Soil Properties

Layer Num.	Layer Soil Type Name (p-y Curve Type)	Layer Depth ft	Effective Unit Wt. pcf	Undrained Cohesion psf	Angle of Friction deg.	Uniaxial qu psi	krm	E50 or kpy pci
1	Sand	0.00	125.0000	--	32.0000	--	--	124.8000
	(Reese, et al.)	9.8000	125.0000	--	32.0000	--	--	124.8000
2	Sand	9.8000	62.6000	--	32.0000	--	--	75.5000
	(Reese, et al.)	10.6000	62.6000	--	32.0000	--	--	75.5000
3	Stiff Clay	10.6000	62.6000	1600.	--	--	0.00500	--
	w/o Free Water	13.5000	62.6000	1600.	--	--	0.00500	--
4	Sand	13.5000	62.6000	--	37.0000	--	--	40.5000
	(Reese, et al.)	16.3000	62.6000	--	37.0000	--	--	40.5000
5	Strong Rock	16.3000	101.6000	--	--	12983.	--	--
	(Vuggy Limestone)	50.0000	101.6000	--	--	12983.	--	--

Static Loading Type

Static loading criteria were used when computing p-y curves for all analyses.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 2

Load No.	Load Type	Condition 1	Condition 2	Axial Thrust Force, lbs	Compute Top y vs. Pile Length	Run Analysis
1	5	y = -0.700000 in	S = 0.0000 in/in	496000.	N.A.	Yes
2	4	y = -0.700000 in	M = 1533047. in-lbs	496000.	N.A.	Yes

V = shear force applied normal to pile axis
M = bending moment applied to pile head
y = lateral deflection normal to pile axis
S = pile slope relative to original pile batter angle
R = rotational stiffness applied to pile head
Values of top y vs. pile lengths can be computed only for load types with
specified shear loading (Load Types 1, 2, and 3).
Thrust force is assumed to be acting axially for all pile batter angles.

Computations of Nominal Moment Capacity and Nonlinear Bending Stiffness
----

Axial thrust force values were determined from pile-head loading conditions

Number of Pile Sections Analyzed = 1

Pile Section No. 1:

Dimensions and Properties of Steel H Weak Axis:

Length of Section = 16.300000 ft
Flange Width = 14.695000 in
Section Depth = 13.830000 in
Flange Thickness = 0.615000 in
Web Thickness = 0.615000 in
Yield Stress of Pipe = 50.000000 ksi
Elastic Modulus = 29000. ksi
Cross-sectional Area = 25.823850 sq. in.
Moment of Inertia = 325.505721 in^4
Elastic Bending Stiffness = 9439666. kip-in^2
Plastic Modulus, Z = 67.593889in^3
Plastic Moment Capacity = Fy Z = 3380.in-kip

Axial Structural Capacities:

Nom. Axial Structural Capacity = Fy As = 1291.193 kips
Nominal Axial Tensile Capacity = -1291.193 kips

Number of Axial Thrust Force Values Determined from Pile-head Loadings = 1

Number	Axial Thrust Force kips
1	496.000

Definition of Run Messages:

Y = part of pipe section has yielded.

Axial Thrust Force = 496.000 kips

Bending Curvature rad/in.	Bending Moment in-kip	Bending Stiffness kip-in2	Depth to N Axis in	Max Total Stress ksi	Run Msg

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0.00000410	38.7092707	9438779.	168.8441446	20.0721621	
0.00000820	77.4185413	9438779.	88.0958223	20.9372734	
0.00001230	116.1278120	9438779.	61.1797149	21.8023847	
0.00001640	154.8370827	9438779.	47.7216612	22.6674960	
0.00002051	193.5463533	9438779.	39.6468289	23.5326073	
0.00002461	232.2556240	9438779.	34.2636074	24.3977186	
0.00002871	270.9648947	9438779.	30.4184492	25.2628299	
0.00003281	309.6741653	9438779.	27.5345806	26.1279412	
0.00003691	348.3834360	9438779.	25.2915716	26.9930524	
0.00004101	387.0927067	9438779.	23.4971645	27.8581637	
0.00004511	425.8019773	9438779.	22.0290131	28.7232750	
0.00004921	464.5112480	9438779.	20.8055537	29.5883863	
0.00005331	503.2205187	9438779.	19.7703188	30.4534976	
0.00005742	541.9297893	9438779.	18.8829746	31.3186089	
0.00006152	580.6390600	9438779.	18.1139430	32.1837202	
0.00006562	619.3483307	9438779.	17.4410403	33.0488313	
0.00006972	658.0576013	9438779.	16.8473026	33.9139426	
0.00007382	696.7668720	9438779.	16.3195358	34.7790540	
0.00007792	735.4761427	9438779.	15.8473234	35.6441652	
0.00008202	774.1854133	9438779.	15.4223322	36.5092765	
0.00008612	812.8946840	9438779.	15.0378164	37.3743878	
0.00009022	851.6039547	9438779.	14.6882566	38.2394991	
0.00009433	890.3132253	9438779.	14.3690932	39.1046104	
0.00009843	929.0224960	9438779.	14.0765269	39.9697217	
0.0001025	967.7317667	9438779.	13.8073658	40.8348329	
0.0001066	1006.	9438779.	13.5589094	41.6999442	
0.0001107	1045.	9438779.	13.3288572	42.5650555	
0.0001148	1084.	9438779.	13.1152373	43.4301668	
0.0001189	1123.	9438779.	12.9163498	44.2952781	
0.0001230	1161.	9438779.	12.7307215	45.1603894	
0.0001271	1200.	9438779.	12.5570692	46.0255007	
0.0001312	1239.	9438779.	12.3942701	46.8906120	
0.0001353	1277.	9438779.	12.2413377	47.7557232	
0.0001394	1316.	9438779.	12.0974013	48.6208345	
0.0001435	1355.	9438779.	11.9616898	49.4859458	
0.0001476	1393.	9435630.	11.8340958	50.0000000	Y
0.0001517	1430.	9423189.	11.7151548	50.0000000	Y
0.0001558	1465.	9401682.	11.6043273	50.0000000	Y
0.0001599	1499.	9372940.	11.5008180	50.0000000	Y
0.0001681	1564.	9298986.	11.3131408	50.0000000	Y
0.0001763	1624.	9208132.	11.1479422	50.0000000	Y
0.0001845	1680.	9105392.	11.0018488	50.0000000	Y
0.0001928	1734.	8994798.	10.8720396	50.0000000	Y
0.0002010	1784.	8879596.	10.7561309	50.0000000	Y
0.0002092	1833.	8761920.	10.6522148	50.0000000	Y
0.0002174	1879.	8642614.	10.5589058	50.0000000	Y
0.0002256	1923.	8523842.	10.4746073	50.0000000	Y
0.0002338	1965.	8406198.	10.3982652	50.0000000	Y
0.0002420	2006.	8289664.	10.3291161	50.0000000	Y
0.0002502	2045.	8175461.	10.2661194	50.0000000	Y
0.0002584	2083.	8063777.	10.2086111	50.0000000	Y
0.0002666	2121.	7954765.	10.1560061	50.0000000	Y
0.0002748	2157.	7848546.	10.1077868	50.0000000	Y

0.0002830	2192.	7745213.	10.0634927	50.0000000	Y
0.0002912	2226.	7644839.	10.0227128	50.0000000	Y
0.0002994	2260.	7547473.	9.9850782	50.0000000	Y
0.0003076	2292.	7453149.	9.9502567	50.0000000	Y
0.0003158	2325.	7361353.	9.9181596	50.0000000	Y
0.0003240	2356.	7272323.	9.8884391	50.0000000	Y
0.0003322	2387.	7186286.	9.8607593	50.0000000	Y
0.0003404	2418.	7103106.	9.8349468	50.0000000	Y
0.0003486	2448.	7021963.	9.8111543	50.0000000	Y
0.0003568	2477.	6943724.	9.7888115	50.0000000	Y
0.0003650	2505.	6863933.	9.7667827	50.0000000	Y
0.0003732	2532.	6783309.	9.7452225	50.0000000	Y
0.0003814	2556.	6702787.	9.7239014	50.0000000	Y
0.0003896	2580.	6621989.	9.7032218	50.0000000	Y
0.0003978	2602.	6542068.	9.6827615	50.0000000	Y
0.0004060	2624.	6461906.	9.6627498	50.0000000	Y
0.0004142	2644.	6382452.	9.6428665	50.0000000	Y
0.0004224	2663.	6304050.	9.6235040	50.0000000	Y
0.0004306	2681.	6226470.	9.6045323	50.0000000	Y
0.0004388	2699.	6149752.	9.5854694	50.0000000	Y
0.0004470	2715.	6074095.	9.5672717	50.0000000	Y
0.0004552	2731.	5999452.	9.5490446	50.0000000	Y
0.0004634	2746.	5926182.	9.5310259	50.0000000	Y
0.0004716	2761.	5853963.	9.5136901	50.0000000	Y
0.0004798	2775.	5782700.	9.4962739	50.0000000	Y
0.0004880	2788.	5713420.	9.4792601	50.0000000	Y
0.0005208	2837.	5446310.	9.4137545	50.0000000	Y
0.0005536	2878.	5198257.	9.3520700	50.0000000	Y
0.0005865	2914.	4968699.	9.2942150	50.0000000	Y
0.0006193	2945.	4756136.	9.2394863	50.0000000	Y
0.0006521	2973.	4559738.	9.1877218	50.0000000	Y
0.0006849	2998.	4376891.	9.1388079	50.0000000	Y
0.0007177	3020.	4207432.	9.0920845	50.0000000	Y
0.0007505	3039.	4049624.	9.0481902	50.0000000	Y
0.0007833	3057.	3902770.	9.0061951	50.0000000	Y
0.0008161	3073.	3765724.	8.9663316	50.0000000	Y
0.0008489	3088.	3637422.	8.9284493	50.0000000	Y
0.0008817	3101.	3517110.	8.8919693	50.0000000	Y
0.0009145	3113.	3404384.	8.8571154	50.0000000	Y

Summary of Results for Nominal Moment Capacity for Section 1

Load No.	Axial Thrust kips	Nominal Moment Capacity in-kips
1	496.0000000000	3113.

Note that the values in the above table are not factored by a strength reduction factor for LRFD.

The value of the strength reduction factor depends on the provisions of the LRFD code being followed.

The above values should be multiplied by the appropriate strength reduction factor to compute ultimate moment capacity according to the LRFD structural design standard being followed.

Layering Correction Equivalent Depths of Soil & Rock Layers

Layer No.	Top of Layer Below Pile Head ft	Equivalent Top Depth Below Grnd Surf ft	Same Layer Type As Layer Above	Layer is Rock or is Below Rock Layer	F0 Integral for Layer lbs	F1 Integral for Layer lbs
1	0.00	0.00	N.A.	No	0.00	102493.
2	9.8000	9.8000	Yes	No	102493.	23660.
3	10.6000	11.2547	No	No	126153.	50452.
4	13.5000	10.8890	No	No	176605.	151498.
5	16.3000	16.3000	No	Yes	N.A.	N.A.

Notes: The F0 integral of Layer n+1 equals the sum of the F0 and F1 integrals for Layer n. Layering correction equivalent depths are computed only for soil types with both shallow-depth and deep-depth expressions for peak lateral load transfer. These soil types are soft and stiff clays, non-liquefied sands, and cemented c-phi soil.

Computed Values of Pile Loading and Deflection
for Lateral Loading for Load Case Number 1

Pile-head conditions are Displacement and Pile-head Rotation (Loading Type 5)
Displacement of pile head = -0.700000 inches
Rotation of pile head = 0.000E+00 radians
Axial load on pile head = 496000.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness lb-in^2	Soil Res. p lb/inch	Soil Spr. Es*h lb/inch	Distrib. Lat. Load lb/inch
0.00	-0.7000	2395316.	-46117.	0.00	73275.	7.16E+09	0.00	0.00	0.00
0.1630	-0.6994	2304842.	-46074.	6.42E-04	71233.	7.16E+09	18.4998	51.7411	0.00
0.3260	-0.6975	2213828.	-46018.	0.00124	69179.	7.68E+09	39.0784	109.5891	0.00

0.4890	-0.6945	2122417.	-45920.	0.00178	67115.	7.95E+09	61.3826	172.8751	0.00
0.6520	-0.6905	2030733.	-45777.	0.00228	65046.	8.22E+09	85.0148	240.8168	0.00
0.8150	-0.6856	1938906.	-45586.	0.00275	62973.	8.48E+09	109.4491	312.2651	0.00
0.9780	-0.6798	1847063.	-45348.	0.00318	60900.	8.72E+09	134.4094	386.7593	0.00
1.1410	-0.6731	1755333.	-45061.	0.00358	58829.	8.94E+09	159.5225	463.5400	0.00
1.3040	-0.6658	1663841.	-44724.	0.00395	56764.	9.13E+09	184.6472	542.4927	0.00
1.4670	-0.6577	1572710.	-44338.	0.00429	54707.	9.28E+09	209.5562	623.2331	0.00
1.6300	-0.6490	1482059.	-43905.	0.00461	52661.	9.39E+09	233.6949	704.3637	0.00
1.7930	-0.6396	1392002.	-43425.	0.00491	50628.	9.44E+09	256.7102	785.0135	0.00
1.9560	-0.6297	1302648.	-42900.	0.00519	48611.	9.44E+09	280.4889	871.1991	0.00
2.1190	-0.6193	1214105.	-42328.	0.00545	46613.	9.44E+09	303.9701	960.0138	0.00
2.2820	-0.6084	1126481.	-41712.	0.00569	44635.	9.44E+09	326.1500	1049.	0.00
2.4450	-0.5971	1039878.	-41054.	0.00592	42680.	9.44E+09	346.7462	1136.	0.00
2.6080	-0.5853	954393.	-40355.	0.00613	40750.	9.44E+09	367.4973	1228.	0.00
2.7710	-0.5731	870122.	-39618.	0.00631	38848.	9.44E+09	386.5782	1319.	0.00
2.9340	-0.5606	787155.	-38845.	0.00649	36975.	9.44E+09	403.6889	1409.	0.00
3.0970	-0.5477	705574.	-38041.	0.00664	35134.	9.44E+09	418.7846	1496.	0.00
3.2600	-0.5346	625453.	-37207.	0.00678	33325.	9.44E+09	433.3782	1586.	0.00
3.4230	-0.5212	546865.	-36348.	0.00690	31551.	9.44E+09	445.6801	1673.	0.00
3.5860	-0.5076	469872.	-35466.	0.00701	29813.	9.44E+09	455.4981	1755.	0.00
3.7490	-0.4938	394528.	-34563.	0.00710	28113.	9.44E+09	467.5723	1852.	0.00
3.9120	-0.4798	320893.	-33633.	0.00717	26450.	9.44E+09	483.4460	1971.	0.00
4.0750	-0.4657	249043.	-32674.	0.00723	24829.	9.44E+09	497.5349	2090.	0.00
4.2380	-0.4515	179046.	-31689.	0.00727	23249.	9.44E+09	509.6682	2208.	0.00
4.4010	-0.4373	110964.	-30682.	0.00730	21712.	9.44E+09	519.6766	2325.	0.00
4.5640	-0.4230	44847.	-29658.	0.00732	20219.	9.44E+09	527.3937	2439.	0.00
4.7270	-0.4087	-19261.	-28621.	0.00732	19642.	9.44E+09	532.6553	2550.	0.00
4.8900	-0.3943	-81327.	-27577.	0.00731	21043.	9.44E+09	535.3005	2655.	0.00
5.0530	-0.3801	-141329.	-26516.	0.00729	22397.	9.44E+09	549.8223	2830.	0.00
5.2160	-0.3658	-199198.	-25426.	0.00725	23703.	9.44E+09	564.0411	3016.	0.00
5.3790	-0.3517	-254870.	-24310.	0.00721	24960.	9.44E+09	577.1542	3210.	0.00
5.5420	-0.3376	-308282.	-23170.	0.00715	26166.	9.44E+09	589.1064	3413.	0.00
5.7050	-0.3237	-359379.	-22007.	0.00708	27319.	9.44E+09	599.8474	3624.	0.00
5.8680	-0.3099	-408108.	-20824.	0.00700	28419.	9.44E+09	609.3317	3845.	0.00
6.0310	-0.2963	-454424.	-19624.	0.00691	29465.	9.44E+09	617.5187	4076.	0.00
6.1940	-0.2829	-498286.	-18408.	0.00681	30455.	9.44E+09	626.2839	4330.	0.00
6.3570	-0.2697	-539651.	-17173.	0.00670	31388.	9.44E+09	636.5570	4617.	0.00
6.5200	-0.2567	-578473.	-15919.	0.00659	32265.	9.44E+09	646.0238	4923.	0.00
6.6830	-0.2439	-614707.	-14646.	0.00646	33083.	9.44E+09	654.6929	5250.	0.00
6.8460	-0.2314	-648312.	-13358.	0.00633	33841.	9.44E+09	662.3278	5599.	0.00
7.0090	-0.2191	-679253.	-12057.	0.00620	34540.	9.44E+09	668.7172	5969.	0.00
7.1720	-0.2072	-707499.	-10744.	0.00605	35177.	9.44E+09	673.8212	6362.	0.00
7.3350	-0.1955	-733024.	-9422.	0.00590	35753.	9.44E+09	677.6026	6780.	0.00
7.4980	-0.1841	-755810.	-8094.	0.00575	36268.	9.44E+09	680.0262	7226.	0.00
7.6610	-0.1730	-775842.	-6763.	0.00559	36720.	9.44E+09	681.0593	7701.	0.00
7.8240	-0.1622	-793113.	-5431.	0.00543	37110.	9.44E+09	680.6712	8208.	0.00
7.9870	-0.1518	-807620.	-4102.	0.00526	37437.	9.44E+09	678.8330	8750.	0.00
8.1500	-0.1416	-819367.	-2777.	0.00509	37702.	9.44E+09	675.5175	9329.	0.00
8.3130	-0.1318	-828365.	-1461.	0.00492	37905.	9.44E+09	670.6982	9951.	0.00
8.4760	-0.1224	-834631.	-154.8362	0.00475	38047.	9.44E+09	664.3498	10619.	0.00
8.6390	-0.1133	-838186.	1137.	0.00458	38127.	9.44E+09	656.4466	11338.	0.00
8.8020	-0.1045	-839062.	2412.	0.00440	38147.	9.44E+09	646.9621	12113.	0.00
8.9650	-0.09603	-837294.	3666.	0.00423	38107.	9.44E+09	635.8684	12952.	0.00
9.1280	-0.08793	-832925.	4898.	0.00406	38008.	9.44E+09	623.1341	13862.	0.00

9.2910	-0.08017	-826004.	6102.	0.00388	37852.	9.44E+09	608.7235	14853.	0.00
9.4540	-0.07274	-816589.	7277.	0.00371	37640.	9.44E+09	592.5941	15936.	0.00
9.6170	-0.06564	-804742.	8419.	0.00355	37372.	9.44E+09	574.6941	17126.	0.00
9.7800	-0.05887	-790534.	9524.	0.00338	37051.	9.44E+09	554.9588	18440.	0.00
9.9430	-0.05241	-774044.	10528.	0.00322	36679.	9.44E+09	472.1707	17620.	0.00
10.1060	-0.04628	-755592.	11404.	0.00306	36263.	9.44E+09	423.7137	17909.	0.00
10.2690	-0.04045	-735367.	12187.	0.00291	35806.	9.44E+09	376.2933	18198.	0.00
10.4320	-0.03491	-713555.	12877.	0.00276	35314.	9.44E+09	329.9678	18487.	0.00
10.5950	-0.02967	-690336.	13479.	0.00261	34790.	9.44E+09	284.7844	18776.	0.00
10.7580	-0.02470	-665889.	14169.	0.00247	34238.	9.44E+09	420.8922	33326.	0.00
10.9210	-0.02001	-639699.	14974.	0.00233	33647.	9.44E+09	402.7103	39367.	0.00
11.0840	-0.01557	-611838.	15741.	0.00220	33018.	9.44E+09	381.4768	47911.	0.00
11.2470	-0.01139	-582395.	16462.	0.00208	32353.	9.44E+09	355.7443	61109.	0.00
11.4100	-0.00744	-551475.	17126.	0.00196	31655.	9.44E+09	322.5140	84838.	0.00
11.5730	-0.00371	-519209.	17708.	0.00185	30927.	9.44E+09	273.4106	144217.	0.00
11.7360	-1.91E-04	-485793.	18069.	0.00175	30173.	9.44E+09	95.0213	972292.	0.00
11.8990	0.00313	-451915.	17902.	0.00165	29408.	9.44E+09	-265.1079	165724.	0.00
12.0620	0.00627	-418961.	17334.	0.00156	28664.	9.44E+09	-315.5727	98510.	0.00
12.2250	0.00923	-387130.	16686.	0.00148	27946.	9.44E+09	-347.7602	73671.	0.00
12.3880	0.01204	-356552.	15982.	0.00140	27255.	9.44E+09	-371.6845	60366.	0.00
12.5510	0.01471	-327325.	15236.	0.00133	26596.	9.44E+09	-390.7606	51963.	0.00
12.7140	0.01724	-299526.	14457.	0.00126	25968.	9.44E+09	-406.6119	46127.	0.00
12.8770	0.01965	-273223.	13648.	0.00120	25374.	9.44E+09	-420.1524	41815.	0.00
13.0400	0.02195	-248472.	12815.	0.00115	24816.	9.44E+09	-431.9535	38484.	0.00
13.2030	0.02415	-225324.	11960.	0.00110	24293.	9.44E+09	-442.3990	35824.	0.00
13.3660	0.02626	-203824.	11085.	0.00106	23808.	9.44E+09	-451.7607	33645.	0.00
13.5290	0.02829	-184011.	10461.	0.00102	23361.	9.44E+09	-186.0084	12861.	0.00
13.6920	0.03024	-164872.	10083.	9.81E-04	22929.	9.44E+09	-201.2361	13016.	0.00
13.8550	0.03213	-146470.	9674.	9.48E-04	22513.	9.44E+09	-216.3231	13171.	0.00
14.0180	0.03395	-128867.	9236.	9.20E-04	22116.	9.44E+09	-231.3042	13326.	0.00
14.1810	0.03572	-112122.	8769.	8.95E-04	21738.	9.44E+09	-246.2145	13481.	0.00
14.3440	0.03745	-96297.	8273.	8.73E-04	21381.	9.44E+09	-261.0890	13636.	0.00
14.5070	0.03914	-81452.	7748.	8.55E-04	21046.	9.44E+09	-275.9619	13791.	0.00
14.6700	0.04080	-67645.	7194.	8.39E-04	20734.	9.44E+09	-290.8671	13946.	0.00
14.8330	0.04243	-54939.	6610.	8.27E-04	20447.	9.44E+09	-305.8369	14100.	0.00
14.9960	0.04403	-43391.	5997.	8.17E-04	20186.	9.44E+09	-320.9023	14255.	0.00
15.1590	0.04562	-33062.	5355.	8.09E-04	19953.	9.44E+09	-336.0927	14410.	0.00
15.3220	0.04719	-24013.	4682.	8.03E-04	19749.	9.44E+09	-351.4349	14565.	0.00
15.4850	0.04876	-16303.	3980.	7.99E-04	19575.	9.44E+09	-366.9534	14720.	0.00
15.6480	0.05032	-9994.	3247.	7.96E-04	19433.	9.44E+09	-382.6696	14875.	0.00
15.8110	0.05187	-5147.	2482.	7.94E-04	19323.	9.44E+09	-398.6017	15030.	0.00
15.9740	0.05343	-1824.	1687.	7.94E-04	19248.	9.44E+09	-414.7638	15185.	0.00
16.1370	0.05498	-86.9820	859.6428	7.93E-04	19209.	9.44E+09	-431.1662	15340.	0.00
16.3000	0.05653	0.00	0.00	7.93E-04	19207.	9.44E+09	-447.8141	7748.	0.00

* This analysis computed pile response using nonlinear moment-curvature relationships. Values of total stress due to combined axial and bending stresses are computed only for elastic sections only and do not equal the actual stresses in concrete and steel. Stresses in concrete and steel may be interpolated from the output for nonlinear bending properties relative to the magnitude of bending moment developed in the pile.

Output Summary for Load Case No. 1:

Pile-head deflection = -0.70000000 inches
Computed slope at pile head = 0.000000 radians
Maximum bending moment = 2395316. inch-lbs
Maximum shear force = -46117. lbs
Depth of maximum bending moment = 0.000000 feet below pile head
Depth of maximum shear force = 0.000000 feet below pile head
Number of iterations = 11
Number of zero deflection points = 1

Computed Values of Pile Loading and Deflection
for Lateral Loading for Load Case Number 2

Pile-head conditions are Displacement and Moment (Loading Type 4)
Displacement of pile head = -0.700000 inches
Moment at pile head = 1533047.0 in-lbs
Axial load at pile head = 496000.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness lb-in^2	Soil Res. p lb/inch	Soil Spr. Es*h lb/inch	Distrib. Lat. Load lb/inch
0.00	-0.7000	1533047.	-35829.	0.00389	53812.	9.33E+09	0.00	0.00	0.00
0.1630	-0.6921	1459035.	-35811.	0.00420	52141.	9.33E+09	18.4998	52.2857	0.00
0.3260	-0.6836	1384797.	-35755.	0.00450	50465.	9.44E+09	39.0783	111.8232	0.00
0.4890	-0.6745	1310430.	-35656.	0.00478	48787.	9.44E+09	61.3825	178.0127	0.00
0.6520	-0.6649	1236035.	-35513.	0.00504	47108.	9.44E+09	85.0145	250.1123	0.00
0.8150	-0.6547	1161716.	-35323.	0.00529	45430.	9.44E+09	109.4486	326.9725	0.00
0.9780	-0.6442	1087582.	-35085.	0.00553	43757.	9.44E+09	134.4087	408.1389	0.00
1.1410	-0.6331	1013744.	-34797.	0.00574	42090.	9.44E+09	159.5216	492.8325	0.00
1.3040	-0.6217	940313.	-34460.	0.00595	40432.	9.44E+09	184.6460	580.9491	0.00
1.4670	-0.6099	867399.	-34075.	0.00613	38786.	9.44E+09	209.5547	672.0970	0.00
1.6300	-0.5977	795112.	-33641.	0.00630	37155.	9.44E+09	233.6930	764.7778	0.00
1.7930	-0.5852	723559.	-33162.	0.00646	35540.	9.44E+09	256.7079	858.0314	0.00
1.9560	-0.5724	652843.	-32636.	0.00660	33943.	9.44E+09	280.4862	958.4517	0.00
2.1190	-0.5594	583070.	-32065.	0.00673	32368.	9.44E+09	303.9669	1063.	0.00
2.2820	-0.5461	514341.	-31450.	0.00685	30817.	9.44E+09	324.6228	1163.	0.00
2.4450	-0.5326	446752.	-30799.	0.00695	29291.	9.44E+09	340.6620	1251.	0.00
2.6080	-0.5189	380375.	-30118.	0.00703	27793.	9.44E+09	356.1869	1343.	0.00
2.7710	-0.5051	315286.	-29408.	0.00710	26324.	9.44E+09	369.4139	1431.	0.00
2.9340	-0.4911	251546.	-28675.	0.00716	24885.	9.44E+09	380.0983	1514.	0.00
3.0970	-0.4770	189209.	-27923.	0.00721	23478.	9.44E+09	389.4632	1597.	0.00
3.2600	-0.4629	128325.	-27149.	0.00724	22104.	9.44E+09	401.4518	1696.	0.00
3.4230	-0.4487	68951.	-26354.	0.00726	20763.	9.44E+09	411.1538	1792.	0.00
3.5860	-0.4345	11136.	-25543.	0.00727	19458.	9.44E+09	418.3953	1883.	0.00
3.7490	-0.4203	-45080.	-24716.	0.00727	20225.	9.44E+09	427.5144	1990.	0.00
3.9120	-0.4061	-99652.	-23867.	0.00725	21456.	9.44E+09	439.8969	2119.	0.00
4.0750	-0.3919	-152520.	-22997.	0.00723	22650.	9.44E+09	450.4262	2248.	0.00
4.2380	-0.3778	-203635.	-22107.	0.00719	23804.	9.44E+09	458.9475	2376.	0.00
4.4010	-0.3638	-252953.	-21203.	0.00714	24917.	9.44E+09	465.3084	2502.	0.00

4.5640	-0.3499	-300440.	-20289.	0.00708	25989.	9.44E+09	469.3597	2624.	0.00
4.7270	-0.3361	-346070.	-19370.	0.00702	27019.	9.44E+09	470.9549	2741.	0.00
4.8900	-0.3224	-389829.	-18450.	0.00694	28007.	9.44E+09	469.9513	2851.	0.00
5.0530	-0.3089	-431712.	-17519.	0.00686	28952.	9.44E+09	481.0932	3046.	0.00
5.2160	-0.2956	-471667.	-16568.	0.00676	29854.	9.44E+09	492.0197	3256.	0.00
5.3790	-0.2825	-509646.	-15596.	0.00666	30711.	9.44E+09	501.9334	3476.	0.00
5.5420	-0.2695	-545601.	-14605.	0.00655	31523.	9.44E+09	510.7960	3707.	0.00
5.7050	-0.2568	-579492.	-13598.	0.00643	32288.	9.44E+09	518.5735	3949.	0.00
5.8680	-0.2444	-611283.	-12578.	0.00631	33005.	9.44E+09	525.2361	4204.	0.00
6.0310	-0.2322	-640941.	-11545.	0.00618	33675.	9.44E+09	530.5774	4470.	0.00
6.1940	-0.2202	-668441.	-10501.	0.00605	34295.	9.44E+09	536.4402	4765.	0.00
6.3570	-0.2085	-693753.	-9445.	0.00590	34867.	9.44E+09	543.7608	5101.	0.00
6.5200	-0.1971	-716846.	-8375.	0.00576	35388.	9.44E+09	550.0111	5458.	0.00
6.6830	-0.1860	-737691.	-7294.	0.00561	35859.	9.44E+09	555.1551	5839.	0.00
6.8460	-0.1752	-756263.	-6205.	0.00545	36278.	9.44E+09	559.1594	6244.	0.00
7.0090	-0.1646	-772544.	-5108.	0.00529	36645.	9.44E+09	561.9931	6676.	0.00
7.1720	-0.1544	-786519.	-4007.	0.00513	36961.	9.44E+09	563.6276	7138.	0.00
7.3350	-0.1446	-798180.	-2904.	0.00497	37224.	9.44E+09	564.0369	7631.	0.00
7.4980	-0.1350	-807522.	-1802.	0.00480	37435.	9.44E+09	563.1970	8159.	0.00
7.6610	-0.1258	-814547.	-702.4898	0.00463	37593.	9.44E+09	561.0861	8725.	0.00
7.8240	-0.1169	-819262.	391.6675	0.00446	37700.	9.44E+09	557.6841	9333.	0.00
7.9870	-0.1083	-821679.	1478.	0.00429	37754.	9.44E+09	552.9726	9986.	0.00
8.1500	-0.1001	-821814.	2554.	0.00412	37758.	9.44E+09	546.9341	10689.	0.00
8.3130	-0.09218	-819692.	3616.	0.00395	37710.	9.44E+09	539.5518	11449.	0.00
8.4760	-0.08461	-815341.	4663.	0.00379	37611.	9.44E+09	530.8088	12271.	0.00
8.6390	-0.07737	-808795.	5691.	0.00362	37464.	9.44E+09	520.6875	13163.	0.00
8.8020	-0.07046	-800094.	6699.	0.00345	37267.	9.44E+09	509.1681	14134.	0.00
8.9650	-0.06388	-789284.	7682.	0.00329	37023.	9.44E+09	496.2279	15195.	0.00
9.1280	-0.05761	-776417.	8638.	0.00312	36733.	9.44E+09	481.8388	16359.	0.00
9.2910	-0.05166	-761551.	9565.	0.00296	36397.	9.44E+09	465.9657	17643.	0.00
9.4540	-0.04602	-744749.	10460.	0.00281	36018.	9.44E+09	448.5631	19067.	0.00
9.6170	-0.04068	-726080.	11319.	0.00266	35597.	9.44E+09	429.5710	20657.	0.00
9.7800	-0.03563	-705623.	12139.	0.00251	35135.	9.44E+09	408.9091	22449.	0.00
9.9430	-0.03087	-683459.	12810.	0.00236	34634.	9.44E+09	278.0729	17620.	0.00
10.1060	-0.02638	-660093.	13319.	0.00222	34107.	9.44E+09	241.5796	17909.	0.00
10.2690	-0.02217	-635671.	13757.	0.00209	33556.	9.44E+09	206.2514	18198.	0.00
10.4320	-0.01821	-610332.	14127.	0.00196	32984.	9.44E+09	172.1133	18487.	0.00
10.5950	-0.01450	-584211.	14431.	0.00184	32394.	9.44E+09	139.1808	18776.	0.00
10.7580	-0.01103	-557441.	14903.	0.00172	31790.	9.44E+09	343.7914	60992.	0.00
10.9210	-0.00778	-529243.	15550.	0.00161	31153.	9.44E+09	317.6909	79902.	0.00
11.0840	-0.00474	-499723.	16138.	0.00150	30487.	9.44E+09	283.0070	116702.	0.00
11.2470	-0.00191	-469020.	16637.	0.00140	29794.	9.44E+09	227.0377	232232.	0.00
11.4100	7.29E-04	-437355.	16681.	0.00130	29079.	9.44E+09	-181.6207	487477.	0.00
11.5730	0.00319	-406296.	16246.	0.00122	28378.	9.44E+09	-263.4871	161436.	0.00
11.7360	0.00549	-376163.	15691.	0.00114	27698.	9.44E+09	-304.0171	108287.	0.00
11.8990	0.00764	-347118.	15069.	0.00106	27042.	9.44E+09	-332.0078	85022.	0.00
12.0620	0.00964	-319274.	14400.	9.92E-04	26414.	9.44E+09	-351.8879	71371.	0.00
12.2250	0.01152	-292711.	13696.	9.29E-04	25814.	9.44E+09	-367.8484	62456.	0.00
12.3880	0.01328	-267498.	12964.	8.71E-04	25245.	9.44E+09	-381.1186	56142.	0.00
12.5510	0.01493	-243688.	12207.	8.18E-04	24708.	9.44E+09	-392.4216	51420.	0.00
12.7140	0.01648	-221331.	11430.	7.70E-04	24203.	9.44E+09	-402.2236	47745.	0.00
12.8770	0.01794	-200468.	10635.	7.26E-04	23732.	9.44E+09	-410.8449	44797.	0.00
13.0400	0.01932	-181137.	9824.	6.87E-04	23296.	9.44E+09	-418.5161	42375.	0.00
13.2030	0.02062	-163371.	8998.	6.51E-04	22895.	9.44E+09	-425.4094	40345.	0.00

13.3660	0.02186	-147199.	8160.	6.19E-04	22530.	9.44E+09	-431.6566	38615.	0.00
13.5290	0.02305	-132649.	7590.	5.90E-04	22201.	9.44E+09	-151.5238	12861.	0.00
13.6920	0.02417	-118652.	7284.	5.64E-04	21885.	9.44E+09	-160.8457	13016.	0.00
13.8550	0.02525	-105247.	6961.	5.40E-04	21583.	9.44E+09	-170.0223	13171.	0.00
14.0180	0.02629	-92471.	6619.	5.20E-04	21294.	9.44E+09	-179.0791	13326.	0.00
14.1810	0.02728	-80362.	6260.	5.02E-04	21021.	9.44E+09	-188.0417	13481.	0.00
14.3440	0.02825	-68956.	5884.	4.87E-04	20764.	9.44E+09	-196.9354	13636.	0.00
14.5070	0.02919	-58290.	5490.	4.73E-04	20523.	9.44E+09	-205.7850	13791.	0.00
14.6700	0.03010	-48399.	5079.	4.62E-04	20300.	9.44E+09	-214.6147	13946.	0.00
14.8330	0.03100	-39319.	4650.	4.53E-04	20095.	9.44E+09	-223.4479	14100.	0.00
14.9960	0.03187	-31087.	4204.	4.46E-04	19909.	9.44E+09	-232.3066	14255.	0.00
15.1590	0.03274	-23737.	3741.	4.40E-04	19743.	9.44E+09	-241.2116	14410.	0.00
15.3220	0.03360	-17305.	3261.	4.36E-04	19598.	9.44E+09	-250.1823	14565.	0.00
15.4850	0.03445	-11827.	2763.	4.33E-04	19474.	9.44E+09	-259.2358	14720.	0.00
15.6480	0.03529	-7338.	2247.	4.31E-04	19373.	9.44E+09	-268.3875	14875.	0.00
15.8110	0.03613	-3875.	1712.	4.30E-04	19295.	9.44E+09	-277.6500	15030.	0.00
15.9740	0.03697	-1473.	1160.	4.29E-04	19240.	9.44E+09	-287.0338	15185.	0.00
16.1370	0.03781	-169.4142	589.4762	4.29E-04	19211.	9.44E+09	-296.5459	15340.	0.00
16.3000	0.03865	0.00	0.00	4.29E-04	19207.	9.44E+09	-306.1905	7748.	0.00

* This analysis computed pile response using nonlinear moment-curvature relationships. Values of total stress due to combined axial and bending stresses are computed only for elastic sections only and do not equal the actual stresses in concrete and steel. Stresses in concrete and steel may be interpolated from the output for nonlinear bending properties relative to the magnitude of bending moment developed in the pile.

Output Summary for Load Case No. 2:

Pile-head deflection = -0.70000000 inches
Computed slope at pile head = 0.00389057 radians
Maximum bending moment = 1533047. inch-lbs
Maximum shear force = -35829. lbs
Depth of maximum bending moment = 0.000000 feet below pile head
Depth of maximum shear force = 0.000000 feet below pile head
Number of iterations = 15
Number of zero deflection points = 1

Summary of Pile-head Responses for Conventional Analyses

Definitions of Pile-head Loading Conditions:

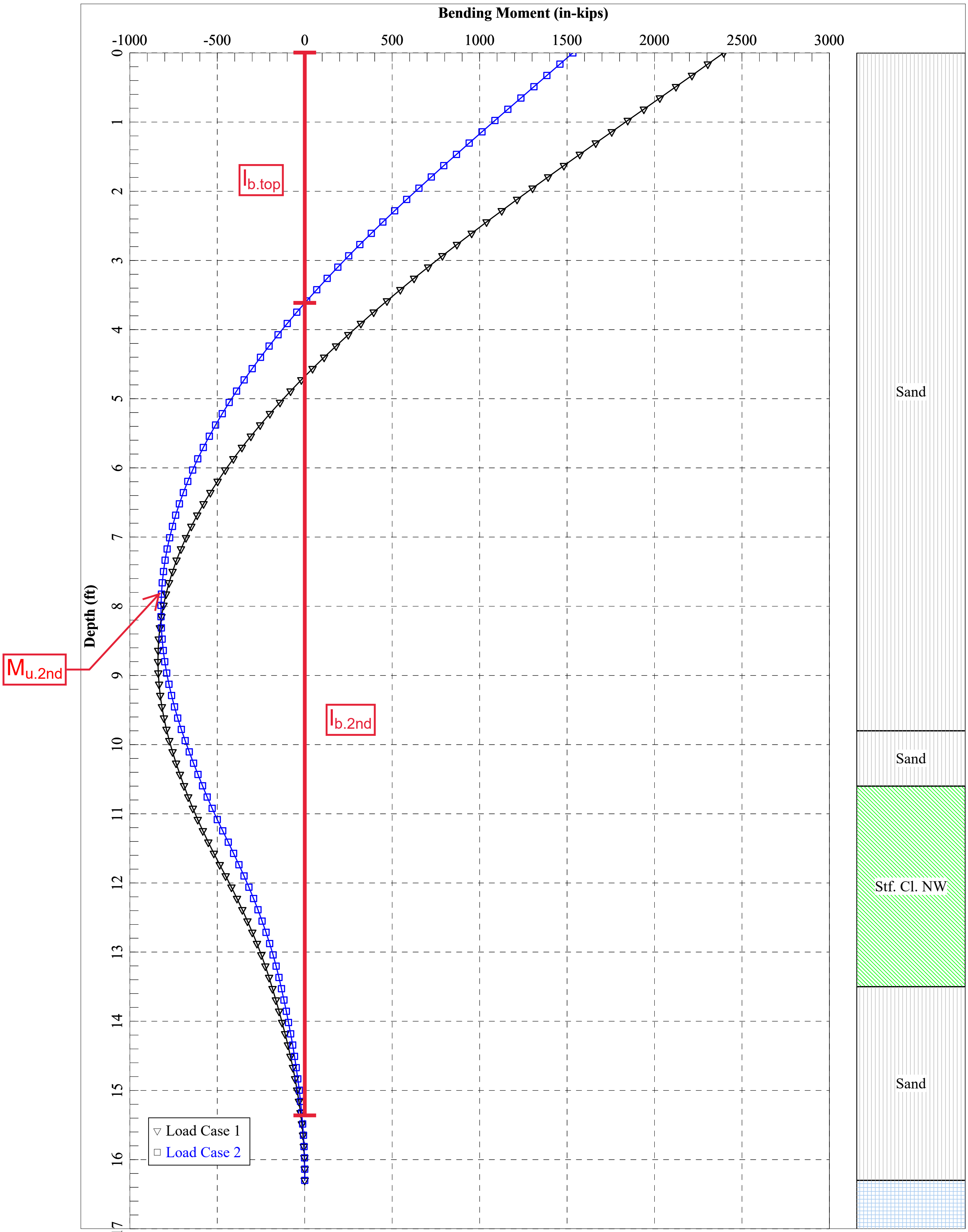
Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians
Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
Load Type 5: Load 1 = Top Deflection, y, inches, and Load 2 = Slope, S, radians

Load Load Load Axial Pile-head Pile-head Max Shear Max Moment

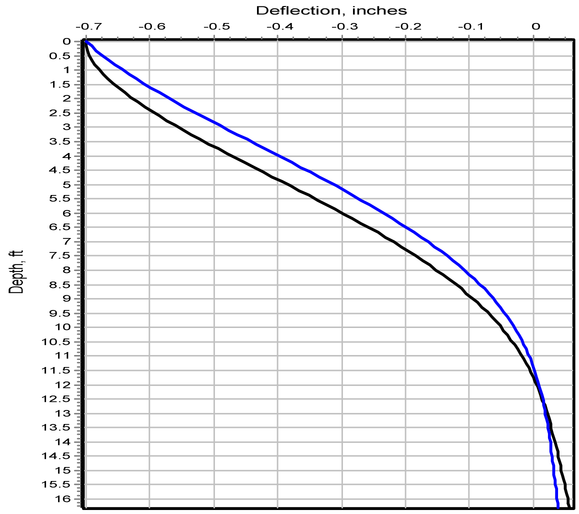
Case No.	Type 1	Pile-head Load 1	Type 2	Pile-head Load 2	Loading lbs	Deflection inches	Rotation radians	in Pile lbs	in Pile in-lbs
1	y, in	-0.7000	S, rad	0.00	496000.	-0.7000	0.00	-46117.	2395316.
2	y, in	-0.7000	M, in-lb	1533047.	496000.	-0.7000	0.00389	-35829.	1533047.

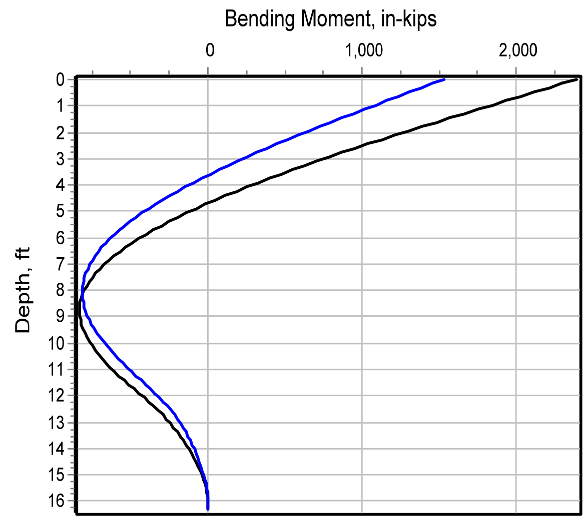
Maximum pile-head deflection = -0.7000000000 inches
Maximum pile-head rotation = 0.0038905675 radians = 0.222913 deg.

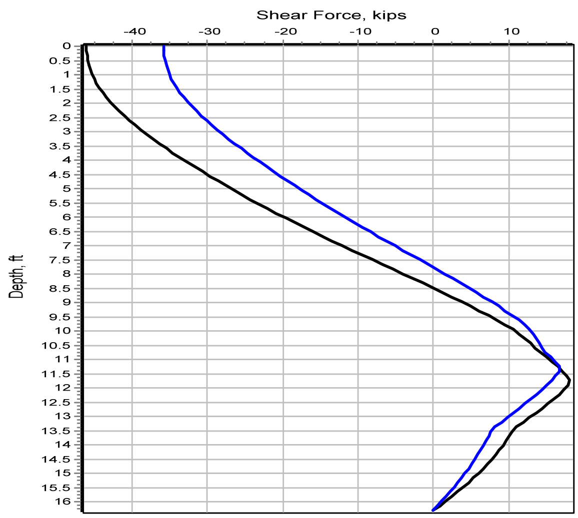
The analysis ended normally.

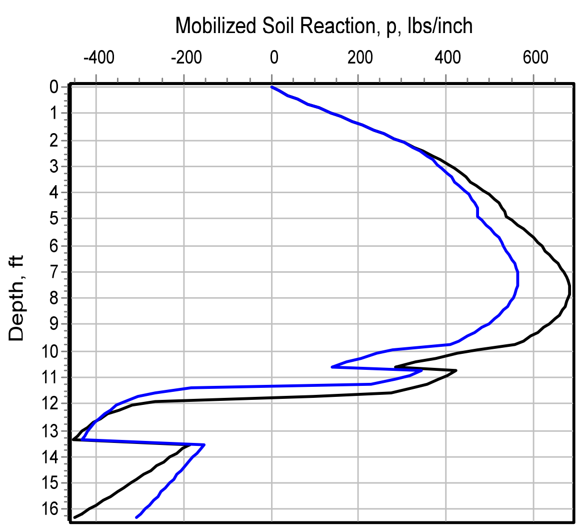


Bending Moment (in-kips)
-1000
-500
0
500
1000
1500
2000
2500
3000
Depth (ft)
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
$I_{b.top}$
$I_{b.2nd}$
$M_{u.2nd}$
Load Case 1
Load Case 2
Sand
Sand
Stf. Cl. NW
Sand


Deflection, inches
-0.7
-0.6
-0.5
-0.4
-0.3
-0.2
-0.1
0
Depth, ft


Bending Moment, in-kips
0
1,000
2,000
Depth, ft


Shear Force, kips
-40
-30
-20
-10
0
10
Depth, ft


Mobilized Soil Reaction, p, lbs/inch
-400
-200
0
200
400
600
Depth, ft

Attachment 3

==

LPile for Windows, Version 2019-11.005

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

==

This copy of LPile is being used by:

KAR
Golder Associates, Inc.

Serial Number of Security Device: 239146533

This copy of LPile is licensed for exclusive use by:

GOLDER, Manchester, NH

Files Used for Analysis

Path to file locations:
\Users\kroth\Documents\Projects\19126013 MaineDOT I-295 Freeport Exit 20 Merrill Rd Bridge\Pile Design\LPile Southeast Abutment\

Name of input data file:
Freeport Exit 20 Southeast Abutment Southern Shift _ No Socket _ Service1.lp11d

Name of output report file:
Freeport Exit 20 Southeast Abutment Southern Shift _ No Socket _ Service1.lp11o

Name of plot output file:
Freeport Exit 20 Southeast Abutment Southern Shift _ No Socket _ Service1.lp11p

Name of runtime message file:
Freeport Exit 20 Southeast Abutment Southern Shift _ No Socket _ Service1.lp11r

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Date and Time of Analysis

Date: August 26, 2020 Time: 15:26:08

Problem Title

Project Name: MaineDOT I-295 Exit 20 Merrill Road Bridge No. 5720
Job Number: 19126013
Client: MaineDOT
Engineer: KAR
Description: Southeast Abutment Pile Design - Service I (No Rock Socket)

Program Options and Settings

Computational Options:
 - Conventional Analysis
Engineering Units Used for Data Input and Computations:
 - US Customary System Units (pounds, feet, inches)

Analysis Control Options:
 - Maximum number of iterations allowed = 500
 - Deflection tolerance for convergence = 1.0000E-05 in
 - Maximum allowable deflection = 100.0000 in
 - Number of pile increments = 100

Loading Type and Number of Cycles of Loading:
 - Static loading specified

 - Use of p-y modification factors for p-y curves not selected
 - Analysis uses layering correction (Method of Georgiadis)
 - No distributed lateral loads are entered
 - Loading by lateral soil movements acting on pile not selected
 - Input of shear resistance at the pile tip not selected
 - Input of moment resistance at the pile tip not selected
 - Input of side resistance moment along pile not selected
 - Computation of pile-head foundation stiffness matrix not selected
 - Push-over analysis of pile not selected
 - Buckling analysis of pile not selected

Output Options:
 - Output files use decimal points to denote decimal symbols.
 - Values of pile-head deflection, bending moment, shear force, and
   soil reaction are printed for full length of pile.
 - Printing Increment (nodal spacing of output points) = 1
 - No p-y curves to be computed and reported for user-specified depths
 - Print using wide report formats

Pile Structural Properties and Geometry

Number of pile sections defined = 1
Total length of pile = 16.300 ft
Depth of ground surface below top of pile = 0.0000 ft

Pile diameters used for p-y curve computations are defined using 2 points.

p-y curves are computed using pile diameter values interpolated with depth over the length of the pile. A summary of values of pile diameter vs. depth follows.

Point No.	Depth Below Pile Head feet	Pile Diameter inches
1	0.000	14.6950
2	16.300	14.6950

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is a H weak axis steel pile
Length of section = 16.300000 ft
Pile width = 13.830000 in
Shear capacity of section = 0.0000 lbs

Ground Slope and Pile Batter Angles

Ground Slope Angle = 0.000 degrees
= 0.000 radians

Pile Batter Angle = 0.000 degrees
= 0.000 radians

Soil and Rock Layering Information

The soil profile is modelled using 5 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer = 0.0000 ft
Distance from top of pile to bottom of layer = 9.800000 ft

Effective unit weight at top of layer = 125.000000 pcf
Effective unit weight at bottom of layer = 125.000000 pcf
Friction angle at top of layer = 32.000000 deg.
Friction angle at bottom of layer = 32.000000 deg.
Subgrade k at top of layer = 124.800000 pci
Subgrade k at bottom of layer = 124.800000 pci

Layer 2 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer = 9.800000 ft
Distance from top of pile to bottom of layer = 10.600000 ft
Effective unit weight at top of layer = 62.600000 pcf
Effective unit weight at bottom of layer = 62.600000 pcf
Friction angle at top of layer = 32.000000 deg.
Friction angle at bottom of layer = 32.000000 deg.
Subgrade k at top of layer = 75.500000 pci
Subgrade k at bottom of layer = 75.500000 pci

Layer 3 is stiff clay without free water

Distance from top of pile to top of layer = 10.600000 ft
Distance from top of pile to bottom of layer = 13.500000 ft
Effective unit weight at top of layer = 62.600000 pcf
Effective unit weight at bottom of layer = 62.600000 pcf
Undrained cohesion at top of layer = 1600. psf
Undrained cohesion at bottom of layer = 1600. psf
Epsilon-50 at top of layer = 0.005000
Epsilon-50 at bottom of layer = 0.005000

Layer 4 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer = 13.500000 ft
Distance from top of pile to bottom of layer = 16.300000 ft
Effective unit weight at top of layer = 62.600000 pcf
Effective unit weight at bottom of layer = 62.600000 pcf
Friction angle at top of layer = 37.000000 deg.
Friction angle at bottom of layer = 37.000000 deg.
Subgrade k at top of layer = 40.500000 pci
Subgrade k at bottom of layer = 40.500000 pci

Layer 5 is strong rock (vuggy limestone)

Distance from top of pile to top of layer = 16.300000 ft
Distance from top of pile to bottom of layer = 50.000000 ft
Effective unit weight at top of layer = 101.600000 pcf
Effective unit weight at bottom of layer = 101.600000 pcf
Uniaxial compressive strength at top of layer = 12983. psi
Uniaxial compressive strength at bottom of layer = 12983. psi

(Depth of the lowest soil layer extends 33.700 ft below the pile tip)

Summary of Input Soil Properties

Layer Layer Num.	Soil Type Name (p-y Curve Type)	Layer Depth ft	Effective Unit Wt. pcf	Undrained Cohesion psf	Angle of Friction deg.	Uniaxial qu psi	E50 or krm	kpy pci
1	Sand	0.00	125.0000	--	32.0000	--	--	124.8000
	(Reese, et al.)	9.8000	125.0000	--	32.0000	--	--	124.8000
2	Sand	9.8000	62.6000	--	32.0000	--	--	75.5000
	(Reese, et al.)	10.6000	62.6000	--	32.0000	--	--	75.5000
3	Stiff Clay	10.6000	62.6000	1600.	--	--	0.00500	--
	w/o Free Water	13.5000	62.6000	1600.	--	--	0.00500	--
4	Sand	13.5000	62.6000	--	37.0000	--	--	40.5000
	(Reese, et al.)	16.3000	62.6000	--	37.0000	--	--	40.5000
5	Strong Rock	16.3000	101.6000	--	--	12983.	--	--
	(Vuggy Limestone)	50.0000	101.6000	--	--	12983.	--	--

Static Loading Type

Static loading criteria were used when computing p-y curves for all analyses.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load No.	Load Type	Condition 1	Condition 2	Axial Thrust Force, lbs	Compute Top y vs. Pile Length	Run Analysis
1	5	y = -0.700000 in	S = 0.0000 in/in	336000.	N.A.	Yes

V = shear force applied normal to pile axis
M = bending moment applied to pile head
y = lateral deflection normal to pile axis
S = pile slope relative to original pile batter angle
R = rotational stiffness applied to pile head
Values of top y vs. pile lengths can be computed only for load types with specified shear loading (Load Types 1, 2, and 3).
Thrust force is assumed to be acting axially for all pile batter angles.

Computations of Nominal Moment Capacity and Nonlinear Bending Stiffness
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Axial thrust force values were determined from pile-head loading conditions

Number of Pile Sections Analyzed = 1

Pile Section No. 1:

Dimensions and Properties of Steel H Weak Axis:

Length of Section	=	16.300000 ft
Flange Width	=	14.695000 in
Section Depth	=	13.830000 in
Flange Thickness	=	0.615000 in
Web Thickness	=	0.615000 in
Yield Stress of Pipe	=	50.000000 ksi
Elastic Modulus	=	29000. ksi
Cross-sectional Area	=	25.823850 sq. in.
Moment of Inertia	=	325.505721 in^4
Elastic Bending Stiffness	=	9439666. kip-in^2
Plastic Modulus, Z	=	67.593889in^3
Plastic Moment Capacity = Fy Z	=	3380.in-kip

Axial Structural Capacities:

Nom. Axial Structural Capacity = Fy As	=	1291.193 kips
Nominal Axial Tensile Capacity	=	-1291.193 kips

Number of Axial Thrust Force Values Determined from Pile-head Loadings = 1

Number	Axial Thrust Force kips
1	336.000

Definition of Run Messages:

Y = part of pipe section has yielded.

Axial Thrust Force = 336.000 kips

Bending Curvature rad/in.	Bending Moment in-kip	Bending Stiffness kip-in2	Depth to N Axis in	Max Total Stress ksi	Run Msg

0.00000449	42.3545820	9438779.	107.3326939	13.9578081	
0.00000897	84.7091640	9438779.	57.3400969	14.9043883	
0.00001346	127.0637460	9438779.	40.6758980	15.8509684	
0.00001795	169.4183280	9438779.	32.3437985	16.7975485	
0.00002244	211.7729100	9438779.	27.3445388	17.7441288	
0.00002692	254.1274921	9438779.	24.0116990	18.6907088	
0.00003141	296.4820741	9438779.	21.6310991	19.6372889	
0.00003590	338.8366561	9438779.	19.8456492	20.5838692	
0.00004039	381.1912381	9438779.	18.4569660	21.5304491	
0.00004487	423.5458201	9438779.	17.3460194	22.4770294	
0.00004936	465.9004021	9438779.	16.4370631	23.4236096	
0.00005385	508.2549841	9438779.	15.6795995	24.3701897	
0.00005833	550.6095661	9438779.	15.0386688	25.3167698	
0.00006282	592.9641481	9438779.	14.4892996	26.2633499	
0.00006731	635.3187301	9438779.	14.0131796	27.2099301	
0.00007180	677.6733122	9438779.	13.5965746	28.1565102	
0.00007628	720.0278942	9438779.	13.2289820	29.1030903	
0.00008077	762.3824762	9438779.	12.9022330	30.0496705	
0.00008526	804.7370582	9438779.	12.6098786	30.9962507	
0.00008975	847.0916402	9438779.	12.3467597	31.9428308	
0.00009423	889.4462222	9438779.	12.1086997	32.8894110	
0.00009872	931.8008042	9438779.	11.8922815	33.8359911	
0.0001032	974.1553862	9438779.	11.6946823	34.7825712	
0.0001077	1017.	9438779.	11.5135497	35.7291513	
0.0001122	1059.	9438779.	11.3469078	36.6757315	
0.0001167	1101.	9438779.	11.1930844	37.6223116	
0.0001212	1144.	9438779.	11.0506553	38.5688918	
0.0001256	1186.	9438779.	10.9183998	39.5154719	
0.0001301	1228.	9438779.	10.7952653	40.4620521	
0.0001346	1271.	9438779.	10.6803398	41.4086322	
0.0001391	1313.	9438779.	10.5728288	42.3552123	
0.0001436	1355.	9438779.	10.4720373	43.3017925	
0.0001481	1398.	9438779.	10.3773544	44.2483726	
0.0001526	1440.	9438779.	10.2882410	45.1949528	
0.0001571	1482.	9438779.	10.2042198	46.1415329	
0.0001615	1525.	9438779.	10.1248665	47.0881131	
0.0001660	1567.	9438779.	10.0498025	48.0346932	
0.0001705	1609.	9438779.	9.9786893	48.9812734	
0.0001750	1652.	9438779.	9.9112229	49.9278535	
0.0001840	1732.	9416303.	9.7903319	50.0000000	Y
0.0001930	1807.	9367531.	9.6861503	50.0000000	Y
0.0002019	1878.	9299387.	9.5960131	50.0000000	Y
0.0002109	1944.	9217036.	9.5177724	50.0000000	Y
0.0002199	2006.	9124681.	9.4496099	50.0000000	Y
0.0002289	2066.	9025739.	9.3899713	50.0000000	Y
0.0002378	2122.	8921671.	9.3378471	50.0000000	Y
0.0002468	2176.	8815137.	9.2920129	50.0000000	Y
0.0002558	2227.	8706859.	9.2517749	50.0000000	Y
0.0002648	2277.	8598710.	9.2161940	50.0000000	Y
0.0002737	2324.	8490570.	9.1849067	50.0000000	Y
0.0002827	2370.	8383302.	9.1573245	50.0000000	Y
0.0002917	2414.	8277623.	9.1329241	50.0000000	Y
0.0003006	2457.	8173779.	9.1113468	50.0000000	Y
0.0003096	2499.	8071974.	9.0922731	50.0000000	Y

0.0003186	2539.	7968037.	9.0742665	50.0000000	Y
0.0003276	2575.	7860509.	9.0565520	50.0000000	Y
0.0003365	2608.	7750686.	9.0391050	50.0000000	Y
0.0003455	2640.	7639662.	9.0220460	50.0000000	Y
0.0003545	2669.	7527838.	9.0055007	50.0000000	Y
0.0003635	2696.	7416424.	8.9892223	50.0000000	Y
0.0003724	2721.	7305745.	8.9730707	50.0000000	Y
0.0003814	2744.	7195317.	8.9573955	50.0000000	Y
0.0003904	2767.	7086484.	8.9419353	50.0000000	Y
0.0003994	2787.	6979420.	8.9267565	50.0000000	Y
0.0004083	2807.	6873951.	8.9121125	50.0000000	Y
0.0004173	2825.	6770399.	8.8974031	50.0000000	Y
0.0004263	2843.	6668133.	8.8831667	50.0000000	Y
0.0004353	2859.	6568659.	8.8690963	50.0000000	Y
0.0004442	2875.	6470687.	8.8554375	50.0000000	Y
0.0004532	2889.	6374810.	8.8417886	50.0000000	Y
0.0004622	2903.	6281493.	8.8286588	50.0000000	Y
0.0004712	2916.	6189407.	8.8155157	50.0000000	Y
0.0004801	2929.	6100231.	8.8026080	50.0000000	Y
0.0004891	2941.	6012342.	8.7900970	50.0000000	Y
0.0004981	2952.	5926871.	8.7777482	50.0000000	Y
0.0005071	2963.	5843455.	8.7653720	50.0000000	Y
0.0005160	2973.	5761679.	8.7536190	50.0000000	Y
0.0005250	2983.	5681830.	8.7418036	50.0000000	Y
0.0005340	2993.	5604308.	8.7301868	50.0000000	Y
0.0005699	3026.	5310494.	8.6856753	50.0000000	Y
0.0006058	3055.	5042890.	8.6439802	50.0000000	Y
0.0006417	3079.	4798775.	8.6049411	50.0000000	Y
0.0006776	3100.	4575826.	8.5682686	50.0000000	Y
0.0007135	3119.	4371622.	8.5334724	50.0000000	Y
0.0007494	3135.	4183730.	8.5007569	50.0000000	Y
0.0007853	3149.	4010645.	8.4695760	50.0000000	Y
0.0008212	3162.	3850650.	8.4401142	50.0000000	Y
0.0008571	3173.	3702658.	8.4119275	50.0000000	Y
0.0008930	3184.	3565198.	8.3856342	50.0000000	Y
0.0009289	3193.	3437556.	8.3605209	50.0000000	Y
0.0009648	3201.	3318185.	8.3361140	50.0000000	Y
0.0010007	3209.	3206769.	8.3134489	50.0000000	Y

Summary of Results for Nominal Moment Capacity for Section 1

Load No.	Axial Thrust kips	Nominal Moment Capacity in-kips
1	336.0000000000	3209.

Note that the values in the above table are not factored by a strength reduction factor for LRFD.

The value of the strength reduction factor depends on the provisions of the LRFD code being followed.

The above values should be multiplied by the appropriate strength reduction factor to compute ultimate moment capacity according to the LRFD structural design standard being followed.

Layering Correction Equivalent Depths of Soil & Rock Layers

Layer No.	Top of Layer Below Pile Head ft	Equivalent Top Depth Below Grnd Surf ft	Same Layer Type As Layer Above	Layer is Rock or is Below Rock Layer	F0 Integral for Layer lbs	F1 Integral for Layer lbs
1	0.00	0.00	N.A.	No	0.00	102493.
2	9.8000	9.8000	Yes	No	102493.	23660.
3	10.6000	11.2547	No	No	126153.	50452.
4	13.5000	10.8890	No	No	176605.	151498.
5	16.3000	16.3000	No	Yes	N.A.	N.A.

Notes: The F0 integral of Layer n+1 equals the sum of the F0 and F1 integrals for Layer n. Layering correction equivalent depths are computed only for soil types with both shallow-depth and deep-depth expressions for peak lateral load transfer. These soil types are soft and stiff clays, non-liquefied sands, and cemented c-phi soil.

Computed Values of Pile Loading and Deflection
for Lateral Loading for Load Case Number 1

Pile-head conditions are Displacement and Pile-head Rotation (Loading Type 5)
Displacement of pile head = -0.700000 inches
Rotation of pile head = 0.000E+00 radians
Axial load on pile head = 336000.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness lb-in^2	Soil Res. p lb/inch	Soil Spr. Es*h lb/inch	Distrib. Lat. Load lb/inch
0.00	-0.7000	2481229.	-47939.	0.00	69019.	8.12E+09	0.00	0.00	0.00
0.1630	-0.6994	2387307.	-47899.	5.87E-04	66899.	8.12E+09	18.4998	51.7370	0.00
0.3260	-0.6977	2293078.	-47842.	0.00114	64772.	8.56E+09	39.0783	109.5553	0.00
0.4890	-0.6950	2198654.	-47744.	0.00164	62640.	8.77E+09	61.3825	172.7620	0.00

0.6520	-0.6913	2104142.	-47601.	0.00212	60507.	8.95E+09	85.0146	240.5536	0.00
0.8150	-0.6867	2009653.	-47411.	0.00256	58374.	9.12E+09	109.4488	311.7636	0.00
0.9780	-0.6812	1915300.	-47172.	0.00298	56244.	9.25E+09	134.4090	385.9180	0.00
1.1410	-0.6750	1821195.	-46885.	0.00338	54120.	9.35E+09	159.5219	462.2497	0.00
1.3040	-0.6680	1727450.	-46548.	0.00374	52004.	9.42E+09	184.6465	540.6396	0.00
1.4670	-0.6604	1634175.	-46163.	0.00409	49899.	9.44E+09	209.5553	620.7040	0.00
1.6300	-0.6520	1541480.	-45729.	0.00442	47806.	9.44E+09	233.6938	701.0544	0.00
1.7930	-0.6431	1449469.	-45250.	0.00473	45729.	9.44E+09	256.7089	780.8310	0.00
1.9560	-0.6335	1358243.	-44724.	0.00502	43670.	9.44E+09	280.4874	866.0204	0.00
2.1190	-0.6234	1267904.	-44153.	0.00530	41631.	9.44E+09	303.9684	953.7263	0.00
2.2820	-0.6128	1178557.	-43536.	0.00555	39614.	9.44E+09	326.1480	1041.	0.00
2.4450	-0.6017	1090296.	-42878.	0.00578	37622.	9.44E+09	346.7440	1127.	0.00
2.6080	-0.5902	1003213.	-42180.	0.00600	35656.	9.44E+09	367.4948	1218.	0.00
2.7710	-0.5782	917400.	-41442.	0.00620	33719.	9.44E+09	386.5754	1308.	0.00
2.9340	-0.5659	832941.	-40669.	0.00638	31813.	9.44E+09	403.6859	1395.	0.00
3.0970	-0.5533	749913.	-39864.	0.00655	29939.	9.44E+09	420.1733	1485.	0.00
3.2600	-0.5403	668391.	-39026.	0.00669	28099.	9.44E+09	435.9266	1578.	0.00
3.4230	-0.5271	588445.	-38162.	0.00682	26294.	9.44E+09	448.4821	1664.	0.00
3.5860	-0.5136	510135.	-37274.	0.00694	24526.	9.44E+09	458.5571	1746.	0.00
3.7490	-0.4999	433510.	-36365.	0.00703	22797.	9.44E+09	470.9255	1842.	0.00
3.9120	-0.4861	358627.	-35428.	0.00712	21106.	9.44E+09	487.1453	1960.	0.00
4.0750	-0.4721	285560.	-34461.	0.00718	19457.	9.44E+09	501.5942	2078.	0.00
4.2380	-0.4580	214372.	-33468.	0.00723	17850.	9.44E+09	514.1001	2196.	0.00
4.4010	-0.4438	145123.	-32452.	0.00727	16287.	9.44E+09	524.4929	2312.	0.00
4.5640	-0.4295	77860.	-31418.	0.00730	14769.	9.44E+09	532.6047	2425.	0.00
4.7270	-0.4153	12625.	-30371.	0.00730	13296.	9.44E+09	538.2703	2535.	0.00
4.8900	-0.4010	-50553.	-29315.	0.00730	14152.	9.44E+09	541.3273	2641.	0.00
5.0530	-0.3867	-111653.	-28242.	0.00728	15532.	9.44E+09	556.2452	2814.	0.00
5.2160	-0.3725	-170610.	-27140.	0.00725	16862.	9.44E+09	570.8612	2998.	0.00
5.3790	-0.3583	-227359.	-26010.	0.00721	18143.	9.44E+09	584.3721	3190.	0.00
5.5420	-0.3443	-281842.	-24855.	0.00716	19373.	9.44E+09	596.7212	3390.	0.00
5.7050	-0.3303	-334003.	-23677.	0.00710	20551.	9.44E+09	607.8566	3600.	0.00
5.8680	-0.3165	-383793.	-22478.	0.00702	21674.	9.44E+09	617.7311	3818.	0.00
6.0310	-0.3028	-431167.	-21261.	0.00694	22744.	9.44E+09	626.3026	4045.	0.00
6.1940	-0.2894	-476087.	-20027.	0.00684	23758.	9.44E+09	635.4308	4295.	0.00
6.3570	-0.2761	-518510.	-18774.	0.00674	24715.	9.44E+09	646.0382	4577.	0.00
6.5200	-0.2630	-558391.	-17501.	0.00663	25616.	9.44E+09	655.8263	4878.	0.00
6.6830	-0.2501	-595687.	-16209.	0.00651	26457.	9.44E+09	664.8038	5199.	0.00
6.8460	-0.2375	-630359.	-14901.	0.00638	27240.	9.44E+09	672.9084	5542.	0.00
7.0090	-0.2252	-662370.	-13578.	0.00625	27963.	9.44E+09	679.8089	5906.	0.00
7.1720	-0.2131	-691689.	-12243.	0.00611	28624.	9.44E+09	685.4295	6292.	0.00
7.3350	-0.2013	-718293.	-10898.	0.00596	29225.	9.44E+09	689.7324	6703.	0.00
7.4980	-0.1897	-742159.	-9546.	0.00581	29764.	9.44E+09	692.6819	7141.	0.00
7.6610	-0.1785	-763275.	-8189.	0.00566	30240.	9.44E+09	694.2446	7606.	0.00
7.8240	-0.1676	-781630.	-6831.	0.00550	30655.	9.44E+09	694.3894	8103.	0.00
7.9870	-0.1570	-797222.	-5474.	0.00533	31007.	9.44E+09	693.0869	8633.	0.00
8.1500	-0.1468	-810054.	-4121.	0.00516	31296.	9.44E+09	690.3094	9200.	0.00
8.3130	-0.1368	-820134.	-2775.	0.00500	31524.	9.44E+09	686.0307	9807.	0.00
8.4760	-0.1272	-827478.	-1439.	0.00483	31690.	9.44E+09	680.2250	10458.	0.00
8.6390	-0.1180	-832107.	-115.8946	0.00465	31794.	9.44E+09	672.8670	11158.	0.00
8.8020	-0.1090	-834048.	1191.	0.00448	31838.	9.44E+09	663.9308	11912.	0.00
8.9650	-0.1004	-833335.	2480.	0.00431	31822.	9.44E+09	653.3893	12727.	0.00
9.1280	-0.09216	-830009.	3746.	0.00414	31747.	9.44E+09	641.2126	13608.	0.00
9.2910	-0.08424	-824117.	4987.	0.00396	31614.	9.44E+09	627.3669	14567.	0.00

9.4540	-0.07666	-815712.	6199.	0.00379	31424.	9.44E+09	611.8127	15611.	0.00
9.6170	-0.06940	-804856.	7378.	0.00363	31179.	9.44E+09	594.5022	16756.	0.00
9.7800	-0.06247	-791615.	8522.	0.00346	30880.	9.44E+09	575.3764	18016.	0.00
9.9430	-0.05586	-776065.	9577.	0.00330	30529.	9.44E+09	503.2124	17620.	0.00
10.1060	-0.04957	-758484.	10513.	0.00314	30132.	9.44E+09	453.8279	17909.	0.00
10.2690	-0.04358	-739064.	11354.	0.00298	29694.	9.44E+09	405.4445	18198.	0.00
10.4320	-0.03789	-717991.	12100.	0.00283	29218.	9.44E+09	358.1242	18487.	0.00
10.5950	-0.03249	-695451.	12756.	0.00269	28709.	9.44E+09	311.9177	18776.	0.00
10.7580	-0.02738	-671623.	13483.	0.00255	28171.	9.44E+09	431.8779	30853.	0.00
10.9210	-0.02254	-646051.	14311.	0.00241	27594.	9.44E+09	414.8966	36008.	0.00
11.0840	-0.01796	-618803.	15104.	0.00228	26979.	9.44E+09	395.3315	43063.	0.00
11.2470	-0.01363	-589959.	15854.	0.00215	26328.	9.44E+09	372.1200	53413.	0.00
11.4100	-0.00954	-559610.	16554.	0.00203	25643.	9.44E+09	343.2553	70404.	0.00
11.5730	-0.00567	-527872.	17187.	0.00192	24927.	9.44E+09	304.0822	104851.	0.00
11.7360	-0.00202	-494899.	17717.	0.00181	24182.	9.44E+09	237.4037	229557.	0.00
11.8990	0.00143	-460950.	17736.	0.00172	23416.	9.44E+09	-217.1117	297731.	0.00
12.0620	0.00469	-427769.	17237.	0.00162	22667.	9.44E+09	-293.2778	122347.	0.00
12.2250	0.00778	-395652.	16625.	0.00154	21942.	9.44E+09	-333.0205	83751.	0.00
12.3880	0.01071	-364755.	15946.	0.00146	21245.	9.44E+09	-360.8025	65917.	0.00
12.5510	0.01349	-335189.	15219.	0.00139	20577.	9.44E+09	-382.2917	55443.	0.00
12.7140	0.01613	-307040.	14455.	0.00132	19942.	9.44E+09	-399.8279	48479.	0.00
12.8770	0.01865	-280378.	13658.	0.00126	19340.	9.44E+09	-414.6292	43480.	0.00
13.0400	0.02106	-255265.	12834.	0.00120	18773.	9.44E+09	-427.4204	39699.	0.00
13.2030	0.02336	-231752.	11987.	0.00115	18242.	9.44E+09	-438.6712	36727.	0.00
13.3660	0.02557	-209886.	11120.	0.00111	17749.	9.44E+09	-448.7057	34322.	0.00
13.5290	0.02770	-189709.	10503.	0.00107	17293.	9.44E+09	-182.1048	12861.	0.00
13.6920	0.02974	-170202.	10131.	0.00103	16853.	9.44E+09	-197.9229	13016.	0.00
13.8550	0.03172	-151429.	9729.	9.96E-04	16429.	9.44E+09	-213.6008	13171.	0.00
14.0180	0.03364	-133453.	9295.	9.66E-04	16024.	9.44E+09	-229.1741	13326.	0.00
14.1810	0.03550	-116335.	8832.	9.40E-04	15637.	9.44E+09	-244.6782	13481.	0.00
14.3440	0.03732	-100138.	8338.	9.18E-04	15272.	9.44E+09	-260.1487	13636.	0.00
14.5070	0.03909	-84922.	7814.	8.99E-04	14928.	9.44E+09	-275.6208	13791.	0.00
14.6700	0.04083	-70749.	7260.	8.83E-04	14608.	9.44E+09	-291.1286	13946.	0.00
14.8330	0.04255	-57681.	6675.	8.69E-04	14313.	9.44E+09	-306.7056	14100.	0.00
14.9960	0.04423	-45778.	6060.	8.59E-04	14045.	9.44E+09	-322.3834	14255.	0.00
15.1590	0.04590	-35102.	5414.	8.50E-04	13804.	9.44E+09	-338.1921	14410.	0.00
15.3220	0.04756	-25715.	4737.	8.44E-04	13592.	9.44E+09	-354.1594	14565.	0.00
15.4850	0.04921	-17680.	4028.	8.39E-04	13410.	9.44E+09	-370.3106	14720.	0.00
15.6480	0.05084	-11059.	3288.	8.36E-04	13261.	9.44E+09	-386.6681	14875.	0.00
15.8110	0.05248	-5917.	2516.	8.35E-04	13145.	9.44E+09	-403.2506	15030.	0.00
15.9740	0.05411	-2316.	1710.	8.34E-04	13063.	9.44E+09	-420.0734	15185.	0.00
16.1370	0.05574	-321.5930	872.0102	8.34E-04	13018.	9.44E+09	-437.1474	15340.	0.00
16.3000	0.05737	0.00	0.00	8.34E-04	13011.	9.44E+09	-454.4786	7748.	0.00

* This analysis computed pile response using nonlinear moment-curvature relationships. Values of total stress due to combined axial and bending stresses are computed only for elastic sections only and do not equal the actual stresses in concrete and steel. Stresses in concrete and steel may be interpolated from the output for nonlinear bending properties relative to the magnitude of bending moment developed in the pile.

Output Summary for Load Case No. 1:

Pile-head deflection = -0.70000000 inches
Computed slope at pile head = 0.000000 radians
Maximum bending moment = 2481229. inch-lbs
Maximum shear force = -47939. lbs
Depth of maximum bending moment = 0.000000 feet below pile head
Depth of maximum shear force = 0.000000 feet below pile head
Number of iterations = 15
Number of zero deflection points = 1

Summary of Pile-head Responses for Conventional Analyses

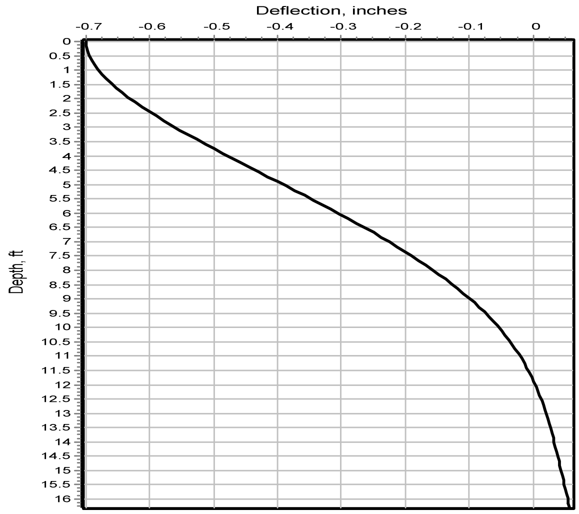
Definitions of Pile-head Loading Conditions:

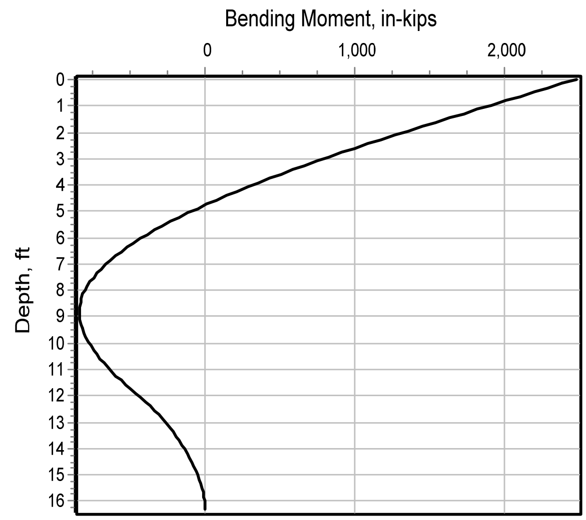
Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians
Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
Load Type 5: Load 1 = Top Deflection, y, inches, and Load 2 = Slope, S, radians

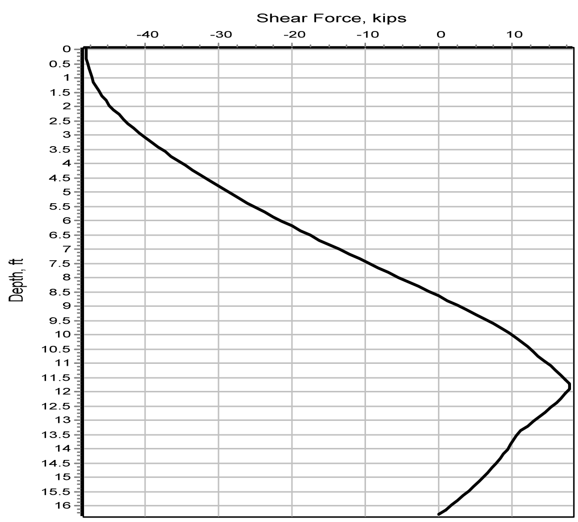
Load Case No.	Load Type 1	Pile-head Load 1	Load Type 2	Pile-head Load 2	Axial Loading lbs	Pile-head Deflection inches	Pile-head Rotation radians	Max Shear in Pile lbs	Max Moment in Pile in-lbs
1	y, in	-0.7000	S, rad	0.00	336000.	-0.7000	0.00	-47939.	2481229.

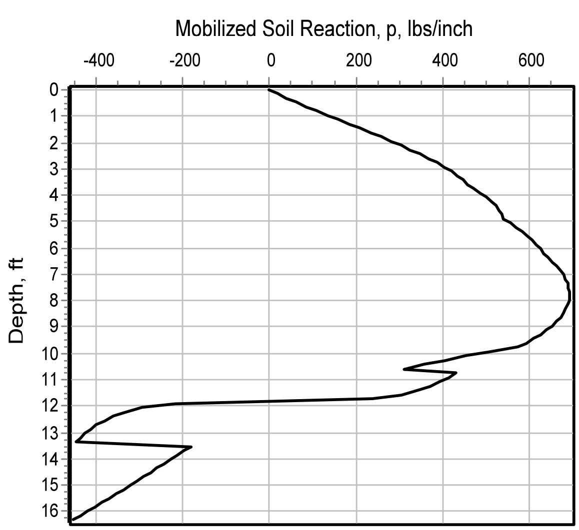
Maximum pile-head deflection = -0.7000000000 inches
Maximum pile-head rotation = 0.0000000000 radians = 0.000000 deg.

The analysis ended normally.


Deflection, inches
Depth, ft


Bending Moment, in-kips
Depth, ft


Shear Force, kips
Depth, ft


Mobilized Soil Reaction, p, lbs/inch
Depth, ft

Piles for full integral and integral with hinge abutments shall be designed to resist all vertical superstructure dead and live loads, abutment and pile dead loads, live load girder rotation moments, lateral displacements, live load impact and moments caused by superimposed dead loads and live loads, as appropriate for the type of integral abutment.

Until the behavior of integral abutments with hinged connections to the superstructure is better understood, the pile design criteria for that type of integral abutment may assume that the moment at the top of the pile is zero, and that there is no moment from either the superstructure or earth loads.

The effect of thermal displacements and moments on piles can be investigated by running LPILE® software.

Secondary thermal forces only need be considered for multi-span structures only.

Appropriate load combinations and load factors should be determined per LRFD 3.4.1.

For the strength limit state analysis, design of the piles should consider the factored structural pile resistance, P_r, the factored structural flexural resistance, pile unbraced length, pile moments, the interaction of combined axial and flexural load effects, the structural shear resistance and the factored geotechnical resistance.

For service limit state evaluations, if piles will be driven to practical refusal in bedrock, settlement will not be a concern. However, all designs should consider horizontal movement, overall stability and scour for the design flood event.

B. Resistance Factors for Integral H-Piles

Pile will typically be end bearing on bedrock. For the strength limit state, use the following resistance factors:

- Use $\Phi_c = 0.50$ for axial resistance in compression and subject to severe pile driving condition; this condition should be assumed when analyzing the lower portions of the pile
- Use $\Phi_c = 0.60$ for axial resistance in compression under good driving conditions; this condition should be assumed when analyzing the upper portion of the pile
- For combined axial and flexural resistance in the upper zone of pile, use:

- Φ_c = 0.70 for axial resistance
- Φ_f = 1.00 for flexural resistance

C. Design Steps

The following steps should be followed during design of piles supporting full integral abutments, for the strength limit state:

1. Determine the foundation displacements, and the load effects (P_u and M_u) from the superstructure and substructure designs.

2. If applicable, determine the magnitude of scour.

3. Select preliminary pile size:

   a. Determine the factored applied superstructure vertical dead and live load (P_u) distributed to each pile

   b. Select the steel pile strength
   c. Select pile orientation; typically weak axis bending
   d. Determine resistance factors (Φ_c and Φ_f) for the structural strength in the upper and lower zones of the pile.
   e. Determine the maximum, required nominal axial pile resistance, P_u/Φ_f
   f. Estimate an initial pile area using the approximation

   $$A_s = \frac{Ru}{0.80 \times F_y}$$

   This approximation is based on weak axis bending and an assumed unbraced length of 15 feet based on typical integral abutment pile deflection and moment with depth curves. Select a pile size with an area A_s or greater.

4. Determine the pile unbraced length and maximum moment at the top of the pile by running LPILE® software for the design displacement from Step 1, P_u, and live load rotation

5. Determine if the applied moment on the pile will cause pile head plastic deformation by using the Interaction of combined axial and flexural load effects on a single pile (LRFD 6.9.2.2)

   a. Obtain the unbraced lengths of the top and lower segments of the pile and calculate the column slenderness factor (λ) for each segment. (LRFD 6.9.4.1)
   b. Determine K values for the top and bottom of the pile per LRFD Table C4.6.2.5-1

g. Calculate the nominal and factored structural pile resistance P_n, per LRFD 6.9.4.1 using the λ values
h. Compare the ratio of P_u to the structural resistance in the upper portion of the pile – the pile size should be such that the ratio is not less than 0.20.
i. Determine the nominal and factored flexural resistance about H-Pile weak axis, (LRFD 6.12.2.2)
j. Calculate the moment that will cause a plastic hinge at the top of the pile (M_p')
k. If the applied moment exceeds the moment that would cause a plastic hinge, a plastic hinge forms, and the moment that can be applied cannot exceed that moment (M_p')

6. For fixed head piles, run a second LPILE® analysis with displacement and plastic moment (M_p') as load conditions and P_u, and calculate new unbraced lengths from the moment with depth curve.

   a. Repeat steps 5.a. through 5.d., above
   b. If the pile size is such that the ratio of P_u to structural resistance exceeds 0.2, check the upper zone of the pile with the interaction equation of LRFD 6.9.2.2. If a plastic hinge forms at the top of the pile, the K value of the upper segment (that portion between the top of the pile and the first inflection point on the moment vs. depth curve) changes from 1.2, for a pinned condition, to 2.1, for a free condition at the top. With the new K value repeat Step 5, and check the interaction equation for pile overstress.

7. Because the piles have weak axis orientation and the flanges resist the shear as opposed to the web, check the maximum shear from the LPILE® output to the structural shear resistance per AISC G7.

8. Check that the maximum factored applied pile load does not exceed the factored geotechnical pile resistance or pile drivability resistance (LRFD 10.5.5.2.3 and 10.7.3.13) provided in the Geotechnical Design Report.

5.4.2.5 Pile Length Requirement

A. General Requirements

Piles may be end bearing or friction piles. In order to obtain the pile behavior associated with the equivalent length, piles should be installed 1 to 5 feet beyond the pile length required to achieve fixity. The practical

5.6.3 Steel H-Piles

Section	Weight lbs/ft kg/m	Area, *A* in^2 cm^2	Depth, *d* in mm	Flange Width, *b* in mm	Thickness: Flange, t_f in. mm	Thickness: Web, t_w in. mm	I_{xx} in^4 cm^4	I_{yy} in^4 cm^4	Compact Section Criteria F'_y ksi MPa
HP 14 HP 360	**117**	**34.4**	**14.21**	**14.885**	**0.805**	**0.805**	**1220**	**443**	**49.4**
	175	222	361	378	20.4	20.4	50800	18400	341
	102	**30**	**14.01**	**14.785**	**0.705**	**0.705**	**1050**	**380**	**38.4**
	153	194	356	376	17.9	17.9	43700	15800	265
	89	**26.1**	**13.83**	**14.695**	**0.615**	**0.615**	**904**	**326**	**29.6**
	133	168	351	373	15.6	15.6	37600	13600	204
	73	**21.4**	**13.61**	**14.585**	**0.505**	**0.505**	**729**	**261**	**20.3**
	109	138	346	370	12.8	12.8	30300	10900	140
HP 13 HP 330	**100**	**29.4**	**13.15**	**13.205**	**0.765**	**0.765**	**886**	**294**	**56.7**
	150	190	334	335	19.4	19.4	36878	12237	391
	87	**25.5**	**12.95**	**13.105**	**0.665**	**0.665**	**755**	**250**	**43.5**
	130	165	329	333	16.9	16.9	31425	10406	300
	73	**21.6**	**12.75**	**13.005**	**0.565**	**0.565**	**630**	**207**	**31.9**
	109	139	324	330	14.4	14.4	26223	8616	220
	60	**17.5**	**12.54**	**12.9**	**0.46**	**0.46**	**503**	**165**	**21.5**
	90	113	319	328	11.7	11.7	20936	6868	148

Section	Weight lbs/ft kg/m	Area, *A* in^2 cm^2	Depth, *d* in mm	Flange Width, *b* in mm	Thickness: Flange, t_f in. mm	Thickness: Web, t_w in. mm	I_{xx} in^4 cm^4	I_{yy} in^4 cm^4	Compact Section Criteria F'_y ksi MPa
HP 12 HP 310	**84**	**24.6**	**12.28**	**12.295**	**0.685**	**0.685**	**650**	**213**	52.5
	126	159	312	312	17.4	17.4	27100	8870	362
	74	**21.8**	**12.13**	**12.215**	**0.61**	**0.61**	**569**	**186**	**42.1**
	111	141	308	310	15.5	15.5	23700	7740	290
	63	**18.4**	**11.94**	**12.125**	**0.515**	**0.515**	**472**	**153**	**30.5**
	94	119	303	308	13.1	13.1	19600	6370	210
	53	**15.5**	**11.78**	**12.045**	**0.435**	**0.435**	**393**	**127**	**22**
	79	100	299	306	11	11	16400	5290	152
HP10 HP 250	**57**	**16.8**	**9.99**	**10.225**	**0.565**	**0.565**	**294**	**101**	**51.6**
	85	108	254	260	14.4	14.4	12200	4200	356
	42	**12.4**	**9.7**	**10.075**	**0.42**	**0.42**	**210**	**71.7**	**29.4**
	63	80	246	256	10.7	10.7	8740	2980	203
HP 8 HP 200	**36**	**10.6**	**8.02**	**8.155**	**0.445**	**0.445**	**119**	**40.3**	**50.3**
	54	68.4	204	207	11.3	11.3	4950	1680	347

Reference 7

Cohesionless Soil

Soil properties for preliminary design only.

Cohesionless Soil Properties	Symbol	Units	Loose		Medium		Dense		References :
Total Unit Weight	γ	pcf	90	115	110	130	110	140	Ref.[1]
Corrected SPT Blow Count	N_{60}		4	10	10	30	30	50	Ref.[2]
Relative Density	D_r	%	15	35	35	65	65	85	Ref.[3]
Angle of Internal Friction	ϕ	deg	29	30	30	36	36	41	Ref.[4]
Coefficient of Lateral Earth Pressure (From Eqn. (1) using ϕ)	K_0		0.51	0.5	0.5	0.41	0.41	0.34	$K_0 = 1-\sin(\phi)$ (1) Ref.[5]
Subgrade Modulus (Below Water Table)	k_{bw}	pci	20	30	30	100	100	160	Ref.[6]
Subgrade Modulus (Above Water Table)	k_{aw}	pci	20	50	50	165	165	275	Ref.[6]
Poisson's Ratio	υ		0.20 - 0.40		0.25 - 0.40		0.30 - 0.45		Ref.[7]
Young's Modulus (From Eqn. (2) using α =5, p_a =2000psf and N_{60})	E_{em}	psf	40000	100000	100000	300000	300000	500000	$E_{em} = p_a * \alpha * N_{60}$ (2) Ref.[8]
Young's Modulus (From Eqn. (2) using α =10, p_a =2000psf and N_{60})	E_{em}	psf	80000	200000	200000	600000	600000	1000000	
Young's Modulus (From Eqn. (3) using B = 24in, υ=the minimum of the range, and k_{bw})	E	psf	66360	99530	97200	324000	314500	503190	$E = k * B * (1-\upsilon^2)$ (3) Ref.[9]
Young's Modulus (From Eqn. (3) using B = 24in, υ=the minimum of the range, and k_{aw})	E	psf	66360	165890	162000	534600	518920	864860	

Notation:

E_{em} = Elastic Modulus based on empirical equation.

Cohesive Soil

Soil properties for preliminary design only.

Cohesive Soil Properties	Symbol	Units	Soft		Medium		Stiff		References :
Total Unit Weight	γ	pcf	100	120	110	130	120	140	Ref.[12]
Corrected SPT Blow Count	N_{60}		2	4	4	8	8	15	Ref. [13]
Unconfined Compressive Strength	q_u	tsf	0.25	0.5	0.5	1	1	2	Ref. [13]
Undrained Shear Strength	C_u	psf	250	500	500	1000	1000	2000	Ref. [14]
Average Undrained Shear Strength		psf	375		750		1500		
Major Principal Strain @ 50%	$\varepsilon 50$		0.02		0.01		0.005		Ref. [15]
Major Principal Strain @ 100%	$\varepsilon 100$		0.06		0.03		0.015		Ref. [16]
Subgrade Modulus (Static Loading)	k	pci	NA		NA		500		Ref. [17]
Subgrade Modulus (Cycling Loading)	k	pci	NA		NA		200		Ref. [17]
Poisson's Ratio	υ		0.4		0.45		0.5		Ref. [18]
Elastic Modulus	E	psi	415	1735	1735	4860	4860	>13890	Ref. [19]
Shear Modulus (From Eqn. (4) using E , and υ)	G	ksi	0.15	0.62	0.60	1.68	1.62	4.63	$G = E/\left(2\left(1+\upsilon\right)\right)$ (4) Ref.[10]
Ultimate Unit End Bearing		ksi	See Fig.2 (For Driven Piles) on pp. 8						
Axial Bearing Failure		kips	Ultimate Unit End Bearing x Tip Area						
Ultimate Unit Skin Friction		psf	See Fig. 3 (For Driven Piles) on pp. 9						

Note: For the input values of vertical failure shear stress and torsional shear stress, the ultimate unit skin friction for a pile or drilled shaft can be used.

GOLDER

CALCULATIONS

Date:	8/26/2020 Rev. 12/9/2020 (section 8 only)	**Made by:**	KAR
Project No.:	19126013	**Checked by:**	MLM
Subject:	Pile Design at Abutment 2 - Southern Shift (HP12x74)	**Reviewed by:**	CCB
Project Title:	MaineDOT I-295 Exit 20 Merrill Road Bridge Replacement No. 5720		

OBJECTIVE

Determine if the proposed HP 12x74 piles will provide adequate support for Abutment 2 (the southeastern integral abutment) based on the anticipated thermal movement and preliminary design loads, assuming the "southern shift" option with the bike path scenario.

METHOD

Use the procedure outlined in AASHTO LRFD (Ref. 1) and the design method provided in the MaineDOT Bridge Design Guide (Ref. 2).

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 9th Ed. 2020.
2. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.
3. Email communication between Golder and HNTB, subject "RE: Freeport I-295 Exit 20 and Exit 22 - Request for Sections and Design Info", dated June 22, 2020.
4. Isenhower, W.M. et al. LPile v2019 Technical Manual: A Program for the Analysis of Deep Foundations Under Lateral Loading. Ensoft, Inc. Dated March 2020.
5. Golder interpreted subsurface section A-A' (Figure 3, Preliminary Geotechnical Design Report, dated September 2020).
6. Golder geotechnical test boring logs (Appendix A, Preliminary Geotechnical Design Report, dated September 2020).
7. Bridge Software Institute. FB-MultiPier Soil Parameter Table (US Customary Units). Accessed July 2020. https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf
8. VTrans Integral Abutment Committee. Integral Abutment Bridge Design Guidelines, 2nd Ed. 2008.
9. AISC Steel Construction Manual, 13th Ed.
10. Golder calculation titled "Settlement at SE Bridge Embankment - Southern Shift" (Appendix E, Preliminary Geotechnical Design Report, dated September 2020).
11. Oregon Department of Transportation, Geo-Environmental Section. Geotechnical Design Manual: Chapter 8 - Foundations, Version 2.1. Dated May 6, 2019.
12. HNTB for State of Maine Department of Transportation. Merrill Road Bridge Freeport Interstate 295: Desert Road South Cross Sections, dated November 2019.
13. Golder calculation titled "Pile Driveability - Southern Shift" (Appendix E, Preliminary Geotechnical Design Report, dated September 2020).

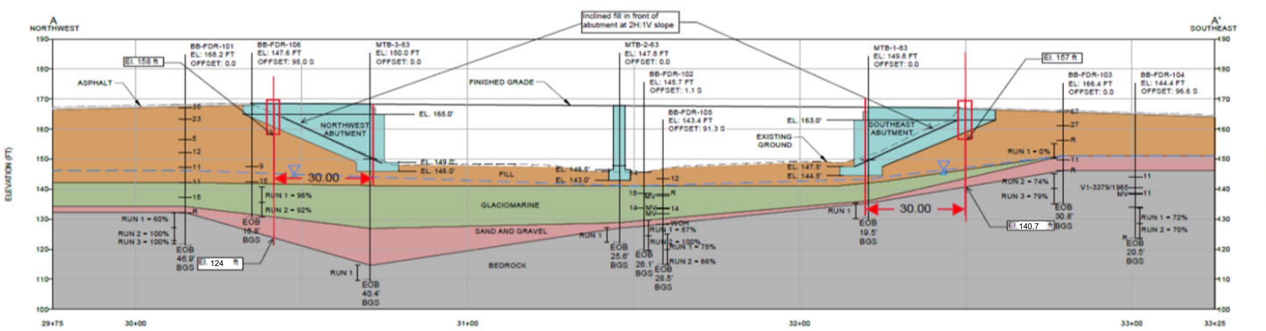
ASSUMPTIONS

1. The selected pile orientation is weak axis bending (Ref. 2, page 5-42).
2. The vertical load is assumed to be evenly distributed.
3. Based on discussions with HNTB, the new southeast abutment will be located approximately 30 ft behind the face of the existing southeast abutment. The post-construction ground surface elevation at the new southeast abutment will be 170 ft (Ref. 12). Assuming 1 ft of pavement atop the abutment plus a 12-ft abutment height (Ref. 3), the top of the piles will be located at elevation 157 ft.

GOLDER

CALCULATIONS

Date:	8/26/2020 Rev. 12/9/2020 (section 8 only)	**Made by:**	KAR
Project No.:	19126013	**Checked by:**	MLM
Subject:	Pile Design at Abutment 2 - Southern Shift (HP12x74)	**Reviewed by:**	CCB
Project Title:	MaineDOT I-295 Exit 20 Merrill Road Bridge Replacement No. 5720		



INTERPRETED SUBSURFACE SECTION A-A'

ATTACHMENTS

1. LPile analysis output for Strength I
2. LPile analysis output for Strength I with Plastic Hinge
3. LPile analysis output for Service I

CALCULATION

1. Determine the downdrag load acting on the piles at the southeastern abutment.

As per Ref. 1 Article 3.11.8, downdrag can be assumed to fully develop if the settlement in the soil layer is 0.4 inches or greater relative to the pile. Since the settlement calculated in the bridge approach embankment approximately 1.5 feet behind the new southeastern abutment was estimated to be 1.33 inches (Ref. 10), it is assumed that downdrag will develop.

Determine the soil layers contributing to downdrag (the deepest layer with settlement ≥ 0.4 inches and all layers above that).

Layer		Layer Thickness in Embankment (ft)	σ'_{v0} at layer midpoint in Embankment (ksf)	Settlement Based on Calculated Loading Stress (in)
Existing Fill	1	3.5	0.219	0.37
Glaciomarine	2	6.1	0.819	0.70
Sand and Gravel	3	4.1	1.147	0.26
(Ref. 10)			Total Settlement (in):	1.33

Layers 1 and 2 will contribute to the downdrag load.

Use the α-method to calculate the nominal skin resistance for the cohesive soils contributing to downdrag (Ref. 1, Article 10.7.3.8.6b); use the Nordlund/Thurman method to calculate the skin resistance for the cohesionless soils contributing to downdrag (Ref. 1, Article 10.7.3.8.6f).

α-method for Layer 2, Glaciomarine:

$$q_s = \alpha S_u$$ (Ref. 1, Eqn 10.7.3.8.6b-1)

where:

S_u = 1.600 ksf (based on shear strength measurements made in the field and on empirical correlation to the average of the N_{60}-values encountered in all borings for the layer)

D = 12.13 in = 1.0 ft (Ref. 4, Table 5.6.3, HP 12x74)

D_b = 2.9 ft (thickness of glaciomarine at abutment, Ref. 5)

α = adhesion factor from Ref. 1 Figure 10.7.3.8.6b-1
Use the plot for "Sands over Stiff Clay" and the curve for "D_b less than 10D"

α = 1.00

q_s = 1.600 ksf

Nordlund/Thurman method for Layer 1, Existing Fill:

$$q_s = K_\delta C_F \sigma'_v \frac{\sin(\delta + \omega)}{\cos \omega}$$ (Ref. 1, Eqn 10.7.3.8.6f-1)

where:

ϕ_f = 32 degrees (based on empirical correlation to the average of the N_{60}-values encountered in all borings for the layer)

V = A_s = 21.8 in^2 = 0.15 ft^3/ft (Ref. 4, Table 5.6.3, HP 12x74)

K_δ = 0.99 (interpolation between Ref. 1 Figures 10.7.3.8.6f-2 and 10.7.3.8.6f-3)

C_F = 0.92

σ'_v = 0.663 ksf (Ref. 5; fill thickness at abutment is 10.6 ft)

δ/ϕ_f = 0.79 (Ref. 1, Figure 10.7.3.8.6f-6)

δ = 25 degrees (Ref. 1, Figure 10.7.3.8.6f-6)

ω = 0 degrees (assume pile battering not required as per Step 3)

q_s = 0.259 ksf

Convert nominal skin resistance to nominal axial downdrag load.

As per Ref. 1 Article C10.7.3.8.6b, for H-piles the perimeter or "box" area should generally be used to compute the surface area of the pile side.

Perimeter of HP 12x74 pile = 48.69 in = 4.06 ft

Layer		Contributing Layer Thickness at Abutm. (ft)	Surface area of pile side (ft^2)	Load (lbs)	Strength I Load Factor	Service I Load Factor
Existing Fill	1	10.6	43.0	11127	1.10	1.00
Glaciomarine	2	2.9	11.8	18827	1.40	1.00

(Ref. 1 Tables 3.4.1-1 and 3.4.1-2; Ref. 11 Table 8.2)

Total Factored Load, Strength I = 38597 lbs per pile
= 38.6 kips per pile

Total Factored Load, Service I = 29954 lbs per pile
= 30.0 kips per pile

According to Ref. 3, typical factored pile loads (Strength I) are expected to be on the order of 350 to 450 kips per pile depending on pile spacing. A downdrag load of 39 kips (Strength I) or 30 kips (Service I) per pile will be added.

2. Select the preliminary pile size.

Determine the factored applied superstructure vertical dead and live load (P_u) distributed to each pile.

Maximum P_u = 489 kips (maximum factored load from Ref. 3 plus downdrag from Step 1)

As part of this analysis loads up to the expected maximum of 489 kips were evaluated, and it was determined that loads higher than 436 kips would require selection of a pile size with an area larger than that provided by HP 12x74. Since the axial loads provided by HNTB in Ref. 3 are preliminary, this analysis was perfomed with P_u = 436 kips, which would correspond to a factored axial load excluding downdrag of 397 kips.

Design P_u = 436 kips (assumed preliminary factored load)

Select the steel pile strength.

F_y = 50 ksi
E = 29,000 ksi

Determine resistance factors (Φ_c and Φ_f) for the structural strength in the upper and lower zones of the pile.

ϕ_{cl} = 0.50 for axial resistance in the lower zone of the pile (Ref. 2, page 5-41)

ϕ_{cu} = 0.70 for axial resistance in the upper zone of the pile (Ref. 2, page 5-42)

ϕ_f = 1.00 for flexural resistance in the upper zone of the pile (Ref. 2, page 5-42)

Determine the maximum required nominal axial pile resistance (Ref. 1, Article 6.9.2.1).

$$R_{n.upper} = \frac{P_u}{\phi_{cu}}$$

$R_{n.upper}$ = 623 kips

$$R_{n.lower} = \frac{P_u}{\phi_{cl}}$$

$R_{n.lower}$ = 872 kips

$$R_n = \max\left(R_{n.upper}, R_{n.lower}\right)$$

R_n = 872 kips

Use the required nominal axial pile resistance to estimate the required pile area.

$$A_{s.req} = \frac{R_n}{0.80\ Fy}$$ (Ref. 2, page 5-42)

$A_{s.req}$ = 21.8 in^2

Select a pile size with an area of $A_{s.req}$ or greater.

Preferred selection is HP 12x74 based on August 21, 2020 call with HNTB.
Check that preferred selection satisfies pile area requirement:

HP 12x74 A_s = 21.8 in^2 (Ref. 4, Table 5.6.3)

A_s = $A_{s.req}$ OK

3. Use LPile analysis to determine the pile unbraced length and maximum moment at the top of the pile.

The following input parameters were used in the LPile analysis:

Pile Properties

Section type:	Steel H Section Weak Axis		(Assumption 1)
Length of section:	16.3	ft	(piles driven to bedrock with no rock socketing)
Flange width, b:	12.215	in	(Ref. 4, Table 5.6.3)
Section depth, d:	12.13	in	(Ref. 4, Table 5.6.3)
Flange thickness, t_f:	0.61	in	(Ref. 4, Table 5.6.3)
Web thickness, t_w:	0.61	in	(Ref. 4, Table 5.6.3)

Pile batter: Vertical (pile battering not required)

Pile Loading

Lateral deflection normal to pile axis, y:	0.7	in	(Ref. 3)
Axial load:	436,000	lbs	(Ref. 3)

Soil Layers

Layer	Depth below base of abutment[1]	Lateral Model	Effective Unit Weight (pcf)	Undrained Shear Strength (psf)[2]	Friction Angle (°)[2]	Subgrade Modulus (pci)[3]	Major Principal Strain at 50%[3]	UCS (psi)[2]
Existing Fill (above water table)	0 - 9.8 ft	Sand (Reese)	125	-	32	124.8	-	-
Existing Fill (below water table)	9.8 - 10.6 ft	Sand (Reese)	62.6	-	32	75.5	-	-
Glaciomarine Silty Clay	10.6 - 13.5 ft	Stiff Clay w/o Free Water (Reese)	62.6	1600	-	-	0.005	-
Sand and Gravel	13.5 - 16.3 ft	Sand (Reese)	62.6	-	37	40.5	-	-
Bedrock	>16.3 ft	Strong Rock (Vuggy Limestone)	101.6	-	-	-	-	12983

1) Ref. 5
2) Ref. 6
3) Ref. 7

The full LPile output is provided in Attachment 1.

Obtain the maximum moment at the top of the pile.

$M_{u.Top}$ = 1669 in-kips (LPile)

Obtain the unbraced lengths of the top segment and the second segment of the upper zone of the pile.

$l_{b.top}$ = 4.16 ft (LPile)
$l_{b.top}$ = 49.87 in

Date:	8/26/2020 Rev. 12/9/2020 (section 8 only)	**Made by:**	KAR
Project No.:	19126013	**Checked by:**	MLM
Subject:	Pile Design at Abutment 2 - Southern Shift (HP12x74)	**Reviewed by:**	CCB
Project Title:	MaineDOT I-295 Exit 20 Merrill Road Bridge Replacement No. 5720		

$l_{b.2nd}$ = 12.14 ft (LPile)
$l_{b.2nd}$ = 145.73 in

4. Determine if the applied moment on the pile will cause pile head plastic deformation by using the interaction of combined axial and flexural load effects on a single pile.

Determine K values for the top and bottom of the pile and calculate the column slenderness factor (λ) for each segment.

For the top segment (fixed at top and pinned at bottom):

$$\lambda_{top} = \frac{K_{top} l_{b.top}}{r_y} \leq 120$$ (Ref. 1, Article 6.9.3)

$$r_y = \sqrt{I_{yy}/A_s}$$

where:

K_{top} = 1.2 (Ref. 1, Table C4.6.2.5-1)
I_{yy} = 186 in^4 (Ref. 4, Table 5.6.3)
r_y = 2.92 in

λ_{top} = 20.49 OK

For the second segment (pinned at top and bottom):

$$\lambda_{2nd} = \frac{K_{2nd} l_{b.2nd}}{r_y} \leq 120$$ (Ref. 1, Article 6.9.3)

where:

K_{2nd} = 1.0 (Ref. 1, Table C4.6.2.5-1)

λ_{2nd} = 49.89 OK

Calculate the critical elastic buckling resistance, P_e, and the nominal yield resistance, P_o.

Use Ref. 1 Table 6.9.4.1.1-1 to select equation for P_e based on cross-section shape and potential buckling mode.

$$P_e = \frac{\pi^2 E}{\left(\frac{K\, l_b}{r_y}\right)^2} A_s$$ (Ref. 1, Eqn 6.9.4.1.2-1)

$P_{e.top}$ = 14868 kips
$P_{e.2nd}$ = 2507 kips

$$P_o = F_y A_s$$ (Ref. 1, Article 6.9.4.1)

$P_o =$ 1090 kips

Calculate the nominal structural pile resistance, P_n, for both segments of the upper zone of the pile as well as the lower zone of the pile.

Determine P_o/P_e to select equation for P_n as per Ref. 1 Article 6.9.4.1.

$P_o/P_{e.top} =$ 0.07 ≤ 2.25
$P_o/P_{e.2nd} =$ 0.43 ≤ 2.25

thus use Ref. 1 Eqn 6.9.4.1.1-1:

$$P_n = \left[0.658^{\left(\frac{P_o}{P_e}\right)}\right] P_o$$

$P_{n.top} =$ 1057 kips
$P_{n.2nd} =$ 909 kips

$$P_{n.bottom} = \left(0.658^{(0)}\right) \times F_y A_s$$ (0 for a fully braced pile - Ref. 8, Appendix B, Eqn 6-9)

$P_{n.bottom} =$ 1090 kips

Calculate the factored structural pile resistance, P_r, for both segments of the upper zone of the pile as well as the lower zone of the pile.

$$P_{r.top} = \phi_{cu} P_{n.top}$$
$P_{r.top} =$ 739.9 kips

$$P_{r.2nd} = \phi_{cu} P_{n.2nd}$$
$P_{r.2nd} =$ 636.0 kips

$$P_{r.bottom} = \phi_{cl} P_{n.bottom}$$
$P_{r.bottom} =$ 545.0 kips

Compare the ratio of P_u to the structural resistance in the upper portion of the pile – the pile size should be such that the ratio is not less than 0.20.

$$\frac{P_u}{P_{r.top}} =$$ 0.59 OK

$$\frac{P_u}{P_{r.2nd}} = \quad 0.69 \quad \text{OK}$$

Since the lower zone of the pile will have virtually no moment, the entire section can carry the required vertical loads. Make sure the applied load will not exceed the resistance of the lower zone.

$$\text{Check}\left(\frac{P_u}{P_{r.bottom}} < 1\right)$$

$$\frac{P_u}{P_{r.bottom}} = \quad 0.80 \quad \text{OK}$$

Determine the nominal and factored flexural resistance about H-Pile weak axis (LRFD 6.12.2.2).

Slenderness ratio for the flange:

$$\lambda_f = \frac{b_f}{2t_f}$$ (Ref. 1, Eqn 6.12.2.2.1-3)

$\lambda_f =$ 10.01

Limiting slenderness ratio for a compact flange:

$$\lambda_{pf} = 0.38\sqrt{\frac{E}{F_y}}$$ (Ref. 1, Eqn 6.12.2.2.1-4)

$\lambda_{pf} =$ 9.15

Limiting slenderness ratio for a noncompact flange:

$$\lambda_{rf} = 0.83\sqrt{\frac{E}{F_y}}$$ (Ref. 1, Eqn 6.12.2.2.1-5)

$\lambda r_f =$ 19.99

Elastic and plastic section moduli about the weak axis:

$$S_y = \frac{I_{yy}}{b/2}$$

$$Z_y = (b^2 t_f)/2 + 0.25t_w^2(d - 2t_f)$$

$S_y =$ 30.5 in^3

$Z_y =$ 46.5 in^3

Nominal flexural resistance:

Date:	8/26/2020 Rev. 12/9/2020 (section 8 only)	**Made by:**	KAR
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Subject:	Pile Design at Abutment 2 - Southern Shift (HP12x74)	**Reviewed by:**	CCB
Project Title:	MaineDOT I-295 Exit 20 Merrill Road Bridge Replacement No. 5720		

$$M_n = M_p = (F_y Z_y) \qquad \text{if } \lambda_f \leq \lambda_{pf}$$ (Ref. 1, Eqn 6.12.2.2.1-1)

$$M_n = \left[1 - \left(1 - \frac{S_y}{Z_y}\right)\left(\frac{\lambda_f - \lambda_{pf}}{0.45\sqrt{\frac{E}{F_y}}}\right)\right] F_y Z_y \qquad \text{if } \lambda_{pf} < \lambda_f \leq \lambda_{rf}$$

(Ref. 1, Eqn 6.12.2.2.1-2)

Since $\lambda_{pf} < \lambda_f \leq \lambda_{rf}$,

$M_n =$ 2262 in-kips

Factored flexural resistance:

$\phi_f =$ 1.00 (Ref. 2, page 5-42)

$$M_r = \phi_f M_n$$

$M_r =$ 2262 in-kips

Calculate the moment that will cause a plastic hinge at the top of the pile, M_p'(Ref. 2, Article 6.9.2.2).

$$M'_p = \frac{9}{8}\left(1 - \frac{P_u}{P_{r.top}}\right) M_r$$ (Ref. 8, Appendix B, Eqn 6-24)

$M_p' =$ 1045 in-kips = 1045449.4 inch-lb

If the applied moment exceeds the moment that would cause a plastic hinge, it can be assumed that the pile head has entered plastic deformation and therefore the moment that can be applied to the pile head cannot exceed M_p'.

$M_{u.Top}$ =	1669	in-kips	(From Step 3)
$M_{u.Top}$	>	M_p'	Plastic Hinge Forms

5. Run a second LPile analysis with displacement, plastic moment (M_p'), and P_u as load conditions, and calculate new unbraced lengths from the moment vs. depth curve. Then repeat Step 4 with the new unbraced lengths.

$l_{b.top}$ =	3.19	ft	(LPile)
$l_{b.top}$ =	38.25	in	
$l_{b.2nd}$ =	13.11	ft	(LPile)
$l_{b.2nd}$ =	157.35	in	
$M_{u.2nd}$ =	641.09	in-kips	(LPile)

Since a plastic hinge developed at the pile head, the value of K for the top segment becomes 2.1 (Ref. 2, page 5-43).

K_{top} = 2.1 (Ref. 1, Table C4.6.2.5-1)
K_{2nd} = 1.0 (Ref. 1, Table C4.6.2.5-1)

λ_{top} = 27.50 < 120 OK
λ_{2nd} = 53.87 < 120 OK

$P_{e.top}$ = 8250 kips
$P_{e.2nd}$ = 2150 kips

$P_o/P_{e.top}$ = 0.13 ≤ 2.25 (to select P_n equation)
$P_o/P_{e.2nd}$ = 0.51 ≤ 2.25 (to select P_n equation)

$P_{n.top}$ = 1031 kips
$P_{n.2nd}$ = 882 kips

$P_{r.top}$ = 722 kips
$P_{r.2nd}$ = 617 kips

$\frac{P_u}{P_{r.top}} =$ 0.60 > 0.20 OK

$\frac{P_u}{P_{r.2nd}} =$ 0.71 > 0.20 OK

Since the pile is appropriately sized, the second segment of the upper zone of the pile needs to be checked with the interaction equation of LRFD Section 6.9.2.2. It is important that this segment of the pile does not form a plastic hinge. A plastic hinge in this segment will cause the pile to fail.

Check: $$\frac{P_u}{P_{r.2nd}} + \frac{8}{9}\left(\frac{M_{u.2nd}}{M_r}\right) < 1$$ (Ref. 8, Appendix B, Eqn 7-13)

Check: 0.96 < 1 OK

6. Because the piles have weak axis orientation and the flanges resist the shear as opposed to the web, check the maximum shear from the LPile output against the structural shear resistance per AISC G7.

V_u = 26.61 kips (LPile)

AASHTO LRFD does not directly address weak axis shear. This analysis will use the AISC Steel Construction Manual 13th edition (G7) to ensure the pile will not shear under the longitudinal load.

k_v = 1.2 (Ref. 9, Section G2.1)

$$C_v = 1.0 \quad \text{if} \quad b/t_f \leq 1.1\sqrt{k_v E/F_y}$$ (Ref. 9, Eqn. G2-3)

C_v = 1.0

Both flanges will resist shear forces:

$$A_w = 2b_f t_f$$ (Ref. 8, Appendix B, Eqn 7-17)

A_w = 14.90 in^2

$$V_n = 0.6F_y A_w C_V$$ (Ref. 9, Eqn G2-1)

V_n = 447 kips

$$V_r = \Phi_v V_n$$

ϕ_v = 1.00 (Ref. 1, Article 6.5.4.2)

V_r = 447 kips

Check that the shear resistance is sufficient:

V_u < V_r OK

7. Check that the maximum factored applied pile load does not exceed the factored pile drivability resistance.

While driving the pile, the maximum stress that is permitted in the pile is:

$$\sigma_{dr} = 0.9\Phi_{da}F_y$$ (Ref. 8, Appendix B, Eqn 7-22)

ϕ_{da} = 1.00 (Ref. 1, Article 6.5.4.2)

σ_{dr} = 45 ksi

This translates into an ultimate maximum driving force that can be applied to the pile of:

$$P_0 = \sigma_{dr}A_s$$ (Ref. 8, Appendix B, Eqn 7-23)

P_0 = 981 kips

Calculate the nominal pile driving resistance (R_{ndr}) from the applied load divided by the resistance factor associated

ϕ_{mon} = 0.65 (Ref. 1, Table 10.5.5.2.3-1)

$$R_{ndr} = \frac{P_u}{\phi_{mon}}$$ (Ref. 8, Appendix B, Eqn 7-25)

R_{ndr} = 671 kips

The nominal pile driving resistance (R_{ndr}) should not exceed the nominal structural pile resistance (P_n) or the

$P_{n.top}$ =	1031	kips	(From Step 5)
$P_{n.2nd}$ =	882	kips	(From Step 5)

Check $R_{ndr} < P_n$: OK
Check $R_{ndr} < P_0$: OK

8. Verify the assumption of a pinned support at the base of the pile by comparing the ratio of the shear and axial forces acting at the pile tip to the factored friction coefficient at the bedrock/pile interface.

V_u at pile tip =	0.11	kips	(LPile)
ϕ_v =	1.00		(Ref. 1, Article 6.5.4.2)
$V_{factored}$ at pile tip =	0.11	kips	

According to Ref. 3, typical factored pile loads (Strength I) are expected to be on the order of 350 to 450 kips per pile depending on pile spacing. Since unfactored loads are not available at this time, the live load is assumed to be 50% of the factored Strength I load. A minimum factored axial pile load of 350 ÷ 2 (to remove the live load) will be used for this analysis. The analysis should be revisited during final design when actual loads are known.

Minimum P = 175 kips

V / P = 0.0006

Friction coefficient, μ =	0.40	(Ref. 1, Table C3.11.5.3-1: steel sheet piles against clean gravel, gravel-sand mixtures, well-graded rock fill with spalls)
Resistance factor =	0.5	(per discussion with MaineDOT)
μ * resistance factor =	0.2	

If the shear/axial ratio is less than μ multiplied by the resistance factor, then the chosen pile section can be considered pinned.

V / P	<	μ * resistance factor
0.0006	<	0.2

The chosen pile section can be considered pinned.

GOLDER

CALCULATIONS

Date:	8/26/2020 Rev. 12/9/2020 (section 8 only)	**Made by:**	KAR
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Subject:	Pile Design at Abutment 2 - Southern Shift (HP12x74)	**Reviewed by:**	CCB
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CONCLUSIONS

The results of the analysis indicate that a maximum moment of 1669 in-kips (139 ft-kips) occurs at the top of the pile under the Strength I load case, with a maximum bridge expansion of 0.7 inches. The results indicate that the depth to bedrock is sufficient for driven piles to limit translation of the pile tip to approximately 0.02 inches, and rock socketing is not anticipated to be required at Abutment 2. HP 12x74 piles will provide adequate support for Abutment 2 based on the anticipated thermal movement. A maximum factored axial load (excluding downdrag) of 397 kips should be used with HP 12x74 piles. Additional piles per abutment can be used to reduce the load on each pile; alternatively, downdrag forces can be mitigated to reduce the total load. The analysis should be revisited during final design when actual loads are known. A drivability analysis will be performed in a separate package.

Attachment 1

==

LPile for Windows, Version 2019-11.005

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

==

This copy of LPile is being used by:

KAR
Golder Associates, Inc.

Serial Number of Security Device: 239146533

This copy of LPile is licensed for exclusive use by:

GOLDER, Manchester, NH

Files Used for Analysis

Path to file locations:
\Users\kroth\Documents\Projects\19126013 MaineDOT I-295 Freeport Exit 20 Merrill Rd Bridge\Pile Design\LPile Southeast Abutment\

Name of input data file:
Freeport Exit 20 Southeast Abutment Southern Shift _ No Socket _ HP12x74.lp11d

Name of output report file:
Freeport Exit 20 Southeast Abutment Southern Shift _ No Socket _ HP12x74.lp11o

Name of plot output file:
Freeport Exit 20 Southeast Abutment Southern Shift _ No Socket _ HP12x74.lp11p

Name of runtime message file:
Freeport Exit 20 Southeast Abutment Southern Shift _ No Socket _ HP12x74.lp11r

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Date and Time of Analysis

Date: August 26, 2020 Time: 16:02:13

Problem Title

Project Name: MaineDOT I-295 Exit 20 Merrill Road Bridge No. 5720
Job Number: 19126013
Client: MaineDOT
Engineer: KAR
Description: Southeast Abutment Pile Design - Strength I (No Rock Socket)

Program Options and Settings

Computational Options:
- Conventional Analysis

Engineering Units Used for Data Input and Computations:
- US Customary System Units (pounds, feet, inches)

Analysis Control Options:
- Maximum number of iterations allowed = 500
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 100.0000 in
- Number of pile increments = 100

Loading Type and Number of Cycles of Loading:
- Static loading specified

- Use of p-y modification factors for p-y curves not selected
- Analysis uses layering correction (Method of Georgiadis)
- No distributed lateral loads are entered
- Loading by lateral soil movements acting on pile not selected
- Input of shear resistance at the pile tip not selected
- Input of moment resistance at the pile tip not selected
- Input of side resistance moment along pile not selected
- Computation of pile-head foundation stiffness matrix not selected
- Push-over analysis of pile not selected
- Buckling analysis of pile not selected

Output Options:
- Output files use decimal points to denote decimal symbols.
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (nodal spacing of output points) = 1
- No p-y curves to be computed and reported for user-specified depths
- Print using wide report formats

Pile Structural Properties and Geometry

Number of pile sections defined = 1
Total length of pile = 16.300 ft
Depth of ground surface below top of pile = 0.0000 ft

Pile diameters used for p-y curve computations are defined using 2 points.

p-y curves are computed using pile diameter values interpolated with depth over the length of the pile. A summary of values of pile diameter vs. depth follows.

Point No.	Depth Below Pile Head feet	Pile Diameter inches
1	0.000	12.2150
2	16.300	12.2150

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is a H weak axis steel pile
Length of section = 16.300000 ft
Pile width = 12.130000 in
Shear capacity of section = 0.0000 lbs

Ground Slope and Pile Batter Angles

Ground Slope Angle = 0.000 degrees
= 0.000 radians

Pile Batter Angle = 0.000 degrees
= 0.000 radians

Soil and Rock Layering Information

The soil profile is modelled using 5 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer = 0.0000 ft
Distance from top of pile to bottom of layer = 9.800000 ft

Effective unit weight at top of layer = 125.000000 pcf
Effective unit weight at bottom of layer = 125.000000 pcf
Friction angle at top of layer = 32.000000 deg.
Friction angle at bottom of layer = 32.000000 deg.
Subgrade k at top of layer = 124.800000 pci
Subgrade k at bottom of layer = 124.800000 pci

Layer 2 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer = 9.800000 ft
Distance from top of pile to bottom of layer = 10.600000 ft
Effective unit weight at top of layer = 62.600000 pcf
Effective unit weight at bottom of layer = 62.600000 pcf
Friction angle at top of layer = 32.000000 deg.
Friction angle at bottom of layer = 32.000000 deg.
Subgrade k at top of layer = 75.500000 pci
Subgrade k at bottom of layer = 75.500000 pci

Layer 3 is stiff clay without free water

Distance from top of pile to top of layer = 10.600000 ft
Distance from top of pile to bottom of layer = 13.500000 ft
Effective unit weight at top of layer = 62.600000 pcf
Effective unit weight at bottom of layer = 62.600000 pcf
Undrained cohesion at top of layer = 1600. psf
Undrained cohesion at bottom of layer = 1600. psf
Epsilon-50 at top of layer = 0.005000
Epsilon-50 at bottom of layer = 0.005000

Layer 4 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer = 13.500000 ft
Distance from top of pile to bottom of layer = 16.300000 ft
Effective unit weight at top of layer = 62.600000 pcf
Effective unit weight at bottom of layer = 62.600000 pcf
Friction angle at top of layer = 37.000000 deg.
Friction angle at bottom of layer = 37.000000 deg.
Subgrade k at top of layer = 40.500000 pci
Subgrade k at bottom of layer = 40.500000 pci

Layer 5 is strong rock (vuggy limestone)

Distance from top of pile to top of layer = 16.300000 ft
Distance from top of pile to bottom of layer = 50.000000 ft
Effective unit weight at top of layer = 101.600000 pcf
Effective unit weight at bottom of layer = 101.600000 pcf
Uniaxial compressive strength at top of layer = 12983. psi
Uniaxial compressive strength at bottom of layer = 12983. psi

(Depth of the lowest soil layer extends 33.700 ft below the pile tip)

Summary of Input Soil Properties

Layer Num.	Soil Type Name (p-y Curve Type)	Layer Depth ft	Effective Unit Wt. pcf	Undrained Cohesion psf	Angle of Friction deg.	Uniaxial qu psi	E50 or krm	kpy pci
1	Sand	0.00	125.0000	--	32.0000	--	--	124.8000
	(Reese, et al.)	9.8000	125.0000	--	32.0000	--	--	124.8000
2	Sand	9.8000	62.6000	--	32.0000	--	--	75.5000
	(Reese, et al.)	10.6000	62.6000	--	32.0000	--	--	75.5000
3	Stiff Clay	10.6000	62.6000	1600.	--	--	0.00500	--
	w/o Free Water	13.5000	62.6000	1600.	--	--	0.00500	--
4	Sand	13.5000	62.6000	--	37.0000	--	--	40.5000
	(Reese, et al.)	16.3000	62.6000	--	37.0000	--	--	40.5000
5	Strong Rock	16.3000	101.6000	--	--	12983.	--	--
	(Vuggy Limestone)	50.0000	101.6000	--	--	12983.	--	--

Static Loading Type

Static loading criteria were used when computing p-y curves for all analyses.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load No.	Load Type	Condition 1	Condition 2	Axial Thrust Force, lbs	Compute Top y vs. Pile Length	Run Analysis
1	5	y = -0.700000 in	S = 0.0000 in/in	436000.	N.A.	Yes

V = shear force applied normal to pile axis
M = bending moment applied to pile head
y = lateral deflection normal to pile axis
S = pile slope relative to original pile batter angle
R = rotational stiffness applied to pile head
Values of top y vs. pile lengths can be computed only for load types with
specified shear loading (Load Types 1, 2, and 3).
Thrust force is assumed to be acting axially for all pile batter angles.

Computations of Nominal Moment Capacity and Nonlinear Bending Stiffness
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Axial thrust force values were determined from pile-head loading conditions

Number of Pile Sections Analyzed = 1

Pile Section No. 1:

Dimensions and Properties of Steel H Weak Axis:

Length of Section	=	16.300000 ft
Flange Width	=	12.215000 in
Section Depth	=	12.130000 in
Flange Thickness	=	0.610000 in
Web Thickness	=	0.610000 in
Yield Stress of Pipe	=	50.000000 ksi
Elastic Modulus	=	29000. ksi
Cross-sectional Area	=	21.557400 sq. in.
Moment of Inertia	=	185.499357 in^4
Elastic Bending Stiffness	=	5379481. kip-in^2
Plastic Modulus, Z	=	46.522801in^3
Plastic Moment Capacity = Fy Z	=	2326.in-kip

Axial Structural Capacities:

Nom. Axial Structural Capacity = Fy As	=	1077.870 kips
Nominal Axial Tensile Capacity	=	-1077.870 kips

Number of Axial Thrust Force Values Determined from Pile-head Loadings = 1

Number	Axial Thrust Force kips
1	436.000

Definition of Run Messages:

Y = part of pipe section has yielded.

Axial Thrust Force = 436.000 kips

Bending Curvature rad/in.	Bending Moment in-kip	Bending Stiffness kip-in2	Depth to N Axis in	Max Total Stress ksi	Run Msg

0.00000460	24.7641803	5379425.	157.6044790	21.0322803	
0.00000921	49.5283606	5379425.	81.8559895	21.8394869	
0.00001381	74.2925408	5379425.	56.6064930	22.6466941	
0.00001841	99.0567211	5379425.	43.9817447	23.4539005	
0.00002302	123.8209014	5379425.	36.4068958	24.2611075	
0.00002762	148.5850817	5379425.	31.3569965	25.0683141	
0.00003222	173.3492619	5379425.	27.7499256	25.8755211	
0.00003683	198.1134422	5379425.	25.0446224	26.6827278	
0.00004143	222.8776225	5379425.	22.9404977	27.4899347	
0.00004603	247.6418028	5379425.	21.2571979	28.2971416	
0.00005064	272.4059830	5379425.	19.8799526	29.1043484	
0.00005524	297.1701633	5379425.	18.7322482	29.9115551	
0.00005985	321.9343436	5379425.	17.7611138	30.7187620	
0.00006445	346.6985239	5379425.	16.9287128	31.5259687	
0.00006905	371.4627041	5379425.	16.2072986	32.3331756	
0.00007366	396.2268844	5379425.	15.5760612	33.1403823	
0.00007826	420.9910647	5379425.	15.0190870	33.9475892	
0.00008286	445.7552450	5379425.	14.5239988	34.7547959	
0.00008747	470.5194253	5379425.	14.0810252	35.5620028	
0.00009207	495.2836055	5379425.	13.6823489	36.3692095	
0.00009667	520.0477858	5379425.	13.3216419	37.1764163	
0.0001013	544.8119661	5379425.	12.9937263	37.9836232	
0.0001059	569.5761464	5379425.	12.6943252	38.7908299	
0.0001105	594.3403266	5379425.	12.4198741	39.5980367	
0.0001151	619.1045069	5379425.	12.1673792	40.4052435	
0.0001197	643.8686872	5379425.	11.9343069	41.2124504	
0.0001243	668.6328675	5379425.	11.7184992	42.0196571	
0.0001289	693.3970477	5379425.	11.5181064	42.8268640	
0.0001335	718.1612280	5379425.	11.3315338	43.6340708	
0.0001381	742.9254083	5379425.	11.1573993	44.4412776	
0.0001427	767.6895886	5379425.	10.9944993	45.2484844	
0.0001473	792.4537688	5379425.	10.8417806	46.0556912	
0.0001519	817.2179491	5379425.	10.6983175	46.8628980	
0.0001565	841.9821294	5379425.	10.5632935	47.6701048	
0.0001611	866.7463097	5379425.	10.4359851	48.4773116	
0.0001657	891.5104899	5379425.	10.3157494	49.2845184	
0.0001703	916.1914453	5378936.	10.2021422	50.0000000	Y
0.0001749	939.9675850	5373301.	10.0958906	50.0000000	Y
0.0001795	962.8854746	5363174.	9.9963908	50.0000000	Y
0.0001887	1006.	5331505.	9.8155851	50.0000000	Y
0.0001980	1047.	5287852.	9.6559832	50.0000000	Y
0.0002072	1085.	5236170.	9.5142392	50.0000000	Y
0.0002164	1121.	5178944.	9.3877622	50.0000000	Y
0.0002256	1154.	5117793.	9.2745034	50.0000000	Y
0.0002348	1187.	5054156.	9.1726884	50.0000000	Y
0.0002440	1217.	4989683.	9.0806156	50.0000000	Y
0.0002532	1247.	4924125.	8.9974326	50.0000000	Y
0.0002624	1275.	4859017.	8.9217253	50.0000000	Y
0.0002716	1302.	4794604.	8.8526670	50.0000000	Y
0.0002808	1328.	4730675.	8.7897181	50.0000000	Y
0.0002900	1354.	4667968.	8.7320095	50.0000000	Y
0.0002992	1378.	4606594.	8.6789939	50.0000000	Y
0.0003084	1402.	4546645.	8.6301875	50.0000000	Y
0.0003176	1426.	4488193.	8.5851602	50.0000000	Y

0.0003268	1448.	4431296.	8.5435286	50.0000000	Y
0.0003361	1471.	4376000.	8.5049490	50.0000000	Y
0.0003453	1492.	4322241.	8.4691624	50.0000000	Y
0.0003545	1514.	4269832.	8.4360127	50.0000000	Y
0.0003637	1534.	4219044.	8.4051152	50.0000000	Y
0.0003729	1555.	4169890.	8.3762382	50.0000000	Y
0.0003821	1575.	4122214.	8.3492663	50.0000000	Y
0.0003913	1595.	4075746.	8.3241818	50.0000000	Y
0.0004005	1614.	4030892.	8.3005713	50.0000000	Y
0.0004097	1634.	3987337.	8.2784658	50.0000000	Y
0.0004189	1653.	3944998.	8.2577803	50.0000000	Y
0.0004281	1671.	3904188.	8.2381726	50.0000000	Y
0.0004373	1690.	3864238.	8.2199632	50.0000000	Y
0.0004465	1708.	3823957.	8.2017414	50.0000000	Y
0.0004557	1724.	3783095.	8.1841046	50.0000000	Y
0.0004650	1740.	3742550.	8.1665596	50.0000000	Y
0.0004742	1755.	3701770.	8.1496775	50.0000000	Y
0.0004834	1770.	3660898.	8.1326296	50.0000000	Y
0.0004926	1783.	3620213.	8.1160861	50.0000000	Y
0.0005018	1796.	3580090.	8.0998955	50.0000000	Y
0.0005110	1809.	3540015.	8.0836547	50.0000000	Y
0.0005202	1821.	3500249.	8.0679778	50.0000000	Y
0.0005294	1832.	3461215.	8.0524865	50.0000000	Y
0.0005386	1843.	3422321.	8.0370205	50.0000000	Y
0.0005478	1854.	3384145.	8.0221876	50.0000000	Y
0.0005846	1892.	3236136.	7.9641966	50.0000000	Y
0.0006215	1925.	3097157.	7.9097244	50.0000000	Y
0.0006583	1953.	2966890.	7.8581756	50.0000000	Y
0.0006951	1978.	2845523.	7.8095143	50.0000000	Y
0.0007320	2000.	2732246.	7.7635826	50.0000000	Y
0.0007688	2019.	2626805.	7.7202072	50.0000000	Y
0.0008056	2037.	2528315.	7.6787989	50.0000000	Y
0.0008424	2052.	2436277.	7.6395664	50.0000000	Y
0.0008793	2067.	2350280.	7.6022418	50.0000000	Y
0.0009161	2079.	2269852.	7.5665744	50.0000000	Y
0.0009529	2091.	2194393.	7.5329185	50.0000000	Y
0.0009898	2102.	2123432.	7.5004584	50.0000000	Y
0.0010266	2111.	2056748.	7.4693295	50.0000000	Y
0.0010634	2120.	1993729.	7.4408820	50.0000000	Y
0.0011002	2128.	1934219.	7.4145964	50.0000000	Y

Summary of Results for Nominal Moment Capacity for Section 1

Load No.	Axial Thrust kips	Nominal Moment Capacity in-kips
1	436.0000000000	2128.

Note that the values in the above table are not factored by a strength reduction factor for LRFD.

The value of the strength reduction factor depends on the provisions of the LRFD code being followed.

The above values should be multiplied by the appropriate strength reduction factor to compute ultimate moment capacity according to the LRFD structural design standard being followed.

Layering Correction Equivalent Depths of Soil & Rock Layers

Layer No.	Top of Layer Below Pile Head ft	Equivalent Top Depth Below Grnd Surf ft	Same Layer Type As Layer Above	Layer is Rock or is Below Rock Layer	F0 Integral for Layer lbs	F1 Integral for Layer lbs
1	0.00	0.00	N.A.	No	0.00	97275.
2	9.8000	9.7999	Yes	No	97275.	23123.
3	10.6000	11.7450	No	No	120398.	42508.
4	13.5000	10.7310	No	No	162906.	145406.
5	16.3000	16.3000	No	Yes	N.A.	N.A.

Notes: The F0 integral of Layer n+1 equals the sum of the F0 and F1 integrals for Layer n. Layering correction equivalent depths are computed only for soil types with both shallow-depth and deep-depth expressions for peak lateral load transfer. These soil types are soft and stiff clays, non-liquefied sands, and cemented c-phi soil.

Computed Values of Pile Loading and Deflection
for Lateral Loading for Load Case Number 1

Pile-head conditions are Displacement and Pile-head Rotation (Loading Type 5)
Displacement of pile head = -0.700000 inches
Rotation of pile head = 0.000E+00 radians
Axial load on pile head = 436000.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness lb-in^2	Soil Res. p lb/inch	Soil Spr. Es*h lb/inch	Distrib. Lat. Load lb/inch
0.00	-0.7000	1669287.	-34938.	0.00	75186.	3.91E+09	0.00	0.00	0.00
0.1630	-0.6992	1600636.	-34900.	8.18E-04	72925.	3.91E+09	15.5610	43.5327	0.00

0.3260	-0.6968	1531362.	-34853.	0.00157	70645.	4.23E+09	33.1291	92.9973	0.00
0.4890	-0.6930	1461610.	-34769.	0.00225	68348.	4.40E+09	52.2795	147.5531	0.00
0.6520	-0.6880	1391503.	-34647.	0.00287	66040.	4.57E+09	72.4163	205.8848	0.00
0.8150	-0.6818	1321166.	-34485.	0.00344	63724.	4.75E+09	93.1327	267.1931	0.00
0.9780	-0.6745	1250722.	-34283.	0.00397	61405.	4.91E+09	113.9963	330.5747	0.00
1.1410	-0.6663	1180289.	-34039.	0.00444	59086.	5.07E+09	134.8802	395.9748	0.00
1.3040	-0.6571	1109983.	-33755.	0.00488	56771.	5.20E+09	155.2816	462.2054	0.00
1.4670	-0.6472	1039915.	-33433.	0.00528	54464.	5.29E+09	174.6923	527.9790	0.00
1.6300	-0.6365	970188.	-33072.	0.00565	52168.	5.36E+09	194.3056	597.1333	0.00
1.7930	-0.6251	900902.	-32673.	0.00599	49887.	5.38E+09	213.6902	668.6785	0.00
1.9560	-0.6130	832154.	-32237.	0.00631	47623.	5.38E+09	231.7075	739.2953	0.00
2.1190	-0.6004	764035.	-31767.	0.00660	45381.	5.38E+09	248.8657	810.7428	0.00
2.2820	-0.5872	696631.	-31265.	0.00686	43161.	5.38E+09	265.1122	883.0434	0.00
2.4450	-0.5736	630025.	-30732.	0.00710	40968.	5.38E+09	279.4282	952.9069	0.00
2.6080	-0.5595	564293.	-30171.	0.00732	38804.	5.38E+09	293.9263	1028.	0.00
2.7710	-0.5449	499511.	-29580.	0.00751	36671.	5.38E+09	310.6229	1115.	0.00
2.9340	-0.5301	435762.	-28958.	0.00768	34572.	5.38E+09	325.8852	1203.	0.00
3.0970	-0.5149	373124.	-28304.	0.00783	32510.	5.38E+09	341.9902	1299.	0.00
3.2600	-0.4994	311680.	-27614.	0.00795	30487.	5.38E+09	364.2081	1426.	0.00
3.4230	-0.4838	251532.	-26880.	0.00806	28507.	5.38E+09	386.1685	1561.	0.00
3.5860	-0.4679	192784.	-26103.	0.00814	26572.	5.38E+09	407.7742	1705.	0.00
3.7490	-0.4519	135536.	-25289.	0.00820	24688.	5.38E+09	424.9763	1839.	0.00
3.9120	-0.4358	79872.	-24449.	0.00824	22855.	5.38E+09	433.9736	1948.	0.00
4.0750	-0.4197	25843.	-23594.	0.00826	21076.	5.38E+09	440.6251	2053.	0.00
4.2380	-0.4036	-26507.	-22716.	0.00826	21098.	5.38E+09	456.9856	2215.	0.00
4.4010	-0.3874	-77101.	-21807.	0.00824	22764.	5.38E+09	472.2220	2384.	0.00
4.5640	-0.3713	-125865.	-20870.	0.00820	24369.	5.38E+09	486.2447	2561.	0.00
4.7270	-0.3553	-172729.	-19906.	0.00815	25912.	5.38E+09	498.9711	2747.	0.00
4.8900	-0.3395	-217630.	-18919.	0.00807	27390.	5.38E+09	510.3258	2941.	0.00
5.0530	-0.3237	-260512.	-17911.	0.00799	28802.	5.38E+09	520.2405	3143.	0.00
5.2160	-0.3082	-301322.	-16883.	0.00789	30146.	5.38E+09	531.2577	3371.	0.00
5.3790	-0.2929	-340006.	-15833.	0.00777	31420.	5.38E+09	541.9615	3619.	0.00
5.5420	-0.2778	-376512.	-14763.	0.00764	32622.	5.38E+09	551.6519	3884.	0.00
5.7050	-0.2630	-410790.	-13676.	0.00750	33750.	5.38E+09	560.3319	4167.	0.00
5.8680	-0.2485	-442797.	-12572.	0.00734	34804.	5.38E+09	568.0125	4471.	0.00
6.0310	-0.2343	-472493.	-11455.	0.00717	35782.	5.38E+09	574.7128	4798.	0.00
6.1940	-0.2204	-499844.	-10325.	0.00700	36682.	5.38E+09	580.4594	5151.	0.00
6.3570	-0.2069	-524819.	-9185.	0.00681	37505.	5.38E+09	585.2866	5532.	0.00
6.5200	-0.1938	-547393.	-8036.	0.00662	38248.	5.38E+09	589.0647	5946.	0.00
6.6830	-0.1810	-567543.	-6882.	0.00641	38911.	5.38E+09	591.3766	6389.	0.00
6.8460	-0.1687	-585254.	-5724.	0.00620	39494.	5.38E+09	592.1707	6866.	0.00
7.0090	-0.1568	-600518.	-4567.	0.00599	39997.	5.38E+09	591.4184	7379.	0.00
7.1720	-0.1453	-613333.	-3412.	0.00577	40419.	5.38E+09	589.0962	7931.	0.00
7.3350	-0.1342	-623704.	-2264.	0.00554	40760.	5.38E+09	585.1847	8528.	0.00
7.4980	-0.1236	-631643.	-1125.	0.00531	41022.	5.38E+09	579.6691	9174.	0.00
7.6610	-0.1134	-637168.	2.1445	0.00508	41204.	5.38E+09	572.5385	9873.	0.00
7.8240	-0.1037	-640306.	1113.	0.00485	41307.	5.38E+09	563.7858	10633.	0.00
7.9870	-0.09445	-641087.	2206.	0.00462	41333.	5.38E+09	553.4071	11461.	0.00
8.1500	-0.08564	-639553.	3277.	0.00439	41282.	5.38E+09	541.4010	12365.	0.00
8.3130	-0.07729	-635748.	4322.	0.00415	41157.	5.38E+09	527.7677	13356.	0.00
8.4760	-0.06939	-629728.	5340.	0.00392	40959.	5.38E+09	512.5077	14446.	0.00
8.6390	-0.06194	-621551.	6326.	0.00370	40689.	5.38E+09	495.6207	15651.	0.00
8.8020	-0.05493	-611286.	7277.	0.00347	40351.	5.38E+09	477.1028	16988.	0.00
8.9650	-0.04836	-599005.	8191.	0.00325	39947.	5.38E+09	456.9443	18482.	0.00

9.1280	-0.04221	-584791.	9063.	0.00304	39479.	5.38E+09	435.1254	20163.	0.00
9.2910	-0.03648	-568730.	9891.	0.00283	38950.	5.38E+09	411.6107	22070.	0.00
9.4540	-0.03115	-550918.	10672.	0.00262	38364.	5.38E+09	386.3414	24257.	0.00
9.6170	-0.02622	-531458.	11401.	0.00243	37723.	5.38E+09	359.2228	26801.	0.00
9.7800	-0.02166	-510458.	12062.	0.00224	37032.	5.38E+09	317.2327	28649.	0.00
9.9430	-0.01746	-488086.	12526.	0.00206	36295.	5.38E+09	157.3273	17620.	0.00
10.1060	-0.01362	-464961.	12802.	0.00188	35534.	5.38E+09	124.6784	17909.	0.00
10.2690	-0.01010	-441215.	13016.	0.00172	34752.	5.38E+09	93.9698	18198.	0.00
10.4320	-0.00690	-416972.	13172.	0.00156	33954.	5.38E+09	65.1883	18487.	0.00
10.5950	-0.00399	-392351.	13273.	0.00141	33143.	5.38E+09	38.3075	18776.	0.00
10.7580	-0.00136	-367461.	13493.	0.00128	32324.	5.38E+09	186.8680	268105.	0.00
10.9210	0.00100	-341743.	13505.	0.00115	31477.	5.38E+09	-174.9874	341335.	0.00
11.0840	0.00313	-316588.	13107.	0.00103	30649.	5.38E+09	-231.4996	144864.	0.00
11.2470	0.00502	-292220.	12626.	9.17E-04	29846.	5.38E+09	-260.4514	101409.	0.00
11.4100	0.00671	-268759.	12098.	8.15E-04	29074.	5.38E+09	-279.9438	81560.	0.00
11.5730	0.00821	-246285.	11536.	7.22E-04	28334.	5.38E+09	-294.3560	70107.	0.00
11.7360	0.00954	-224861.	10949.	6.36E-04	27629.	5.38E+09	-305.5266	62667.	0.00
11.8990	0.01070	-204536.	10343.	5.58E-04	26959.	5.38E+09	-314.4250	57478.	0.00
12.0620	0.01172	-185351.	9721.	4.87E-04	26328.	5.38E+09	-321.6343	53686.	0.00
12.2250	0.01260	-167338.	9086.	4.23E-04	25735.	5.38E+09	-327.5374	50826.	0.00
12.3880	0.01337	-150527.	8441.	3.65E-04	25181.	5.38E+09	-332.4013	48621.	0.00
12.5510	0.01403	-134942.	7786.	3.13E-04	24668.	5.38E+09	-336.4215	46893.	0.00
12.7140	0.01460	-120601.	7125.	2.67E-04	24196.	5.38E+09	-339.7461	45525.	0.00
12.8770	0.01508	-107523.	6458.	2.25E-04	23765.	5.38E+09	-342.4902	44436.	0.00
13.0400	0.01548	-95722.	5786.	1.88E-04	23377.	5.38E+09	-344.7452	43566.	0.00
13.2030	0.01581	-85210.	5110.	1.55E-04	23031.	5.38E+09	-346.5845	42873.	0.00
13.3660	0.01609	-75997.	4430.	1.26E-04	22727.	5.38E+09	-348.0679	42324.	0.00
13.5290	0.01631	-68093.	3985.	9.98E-05	22467.	5.38E+09	-107.2095	12861.	0.00
13.6920	0.01648	-60578.	3773.	7.64E-05	22220.	5.38E+09	-109.6395	13016.	0.00
13.8550	0.01660	-53464.	3556.	5.57E-05	21985.	5.38E+09	-111.8064	13171.	0.00
14.0180	0.01669	-46760.	3336.	3.75E-05	21765.	5.38E+09	-113.7346	13326.	0.00
14.1810	0.01675	-40478.	3112.	2.16E-05	21558.	5.38E+09	-115.4478	13481.	0.00
14.3440	0.01678	-34624.	2884.	7.97E-06	21365.	5.38E+09	-116.9693	13636.	0.00
14.5070	0.01678	-29208.	2654.	-3.63E-06	21187.	5.38E+09	-118.3216	13791.	0.00
14.6700	0.01676	-24235.	2422.	-1.34E-05	21023.	5.38E+09	-119.5263	13946.	0.00
14.8330	0.01673	-19711.	2187.	-2.13E-05	20874.	5.38E+09	-120.6040	14100.	0.00
14.9960	0.01668	-15644.	1950.	-2.78E-05	20740.	5.38E+09	-121.5740	14255.	0.00
15.1590	0.01662	-12036.	1711.	-3.28E-05	20621.	5.38E+09	-122.4543	14410.	0.00
15.3220	0.01655	-8893.	1471.	-3.66E-05	20518.	5.38E+09	-123.2614	14565.	0.00
15.4850	0.01648	-6219.	1229.	-3.94E-05	20430.	5.38E+09	-124.0101	14720.	0.00
15.6480	0.01640	-4017.	985.9050	-4.12E-05	20357.	5.38E+09	-124.7133	14875.	0.00
15.8110	0.01632	-2292.	741.3119	-4.24E-05	20301.	5.38E+09	-125.3820	15030.	0.00
15.9740	0.01623	-1045.	495.4359	-4.30E-05	20259.	5.38E+09	-126.0250	15185.	0.00
16.1370	0.01615	-280.3322	248.3209	-4.32E-05	20234.	5.38E+09	-126.6489	15340.	0.00
16.3000	0.01606	0.00	0.00	-4.33E-05	20225.	5.38E+09	-127.2579	7748.	0.00

* This analysis computed pile response using nonlinear moment-curvature relationships. Values of total stress due to combined axial and bending stresses are computed only for elastic sections only and do not equal the actual stresses in concrete and steel. Stresses in concrete and steel may be interpolated from the output for nonlinear bending properties relative to the magnitude of bending moment developed in the pile.

Output Summary for Load Case No. 1:

Pile-head deflection = -0.70000000 inches
Computed slope at pile head = 0.000000 radians
Maximum bending moment = 1669287. inch-lbs
Maximum shear force = -34938. lbs
Depth of maximum bending moment = 0.000000 feet below pile head
Depth of maximum shear force = 0.000000 feet below pile head
Number of iterations = 15
Number of zero deflection points = 1

Summary of Pile-head Responses for Conventional Analyses

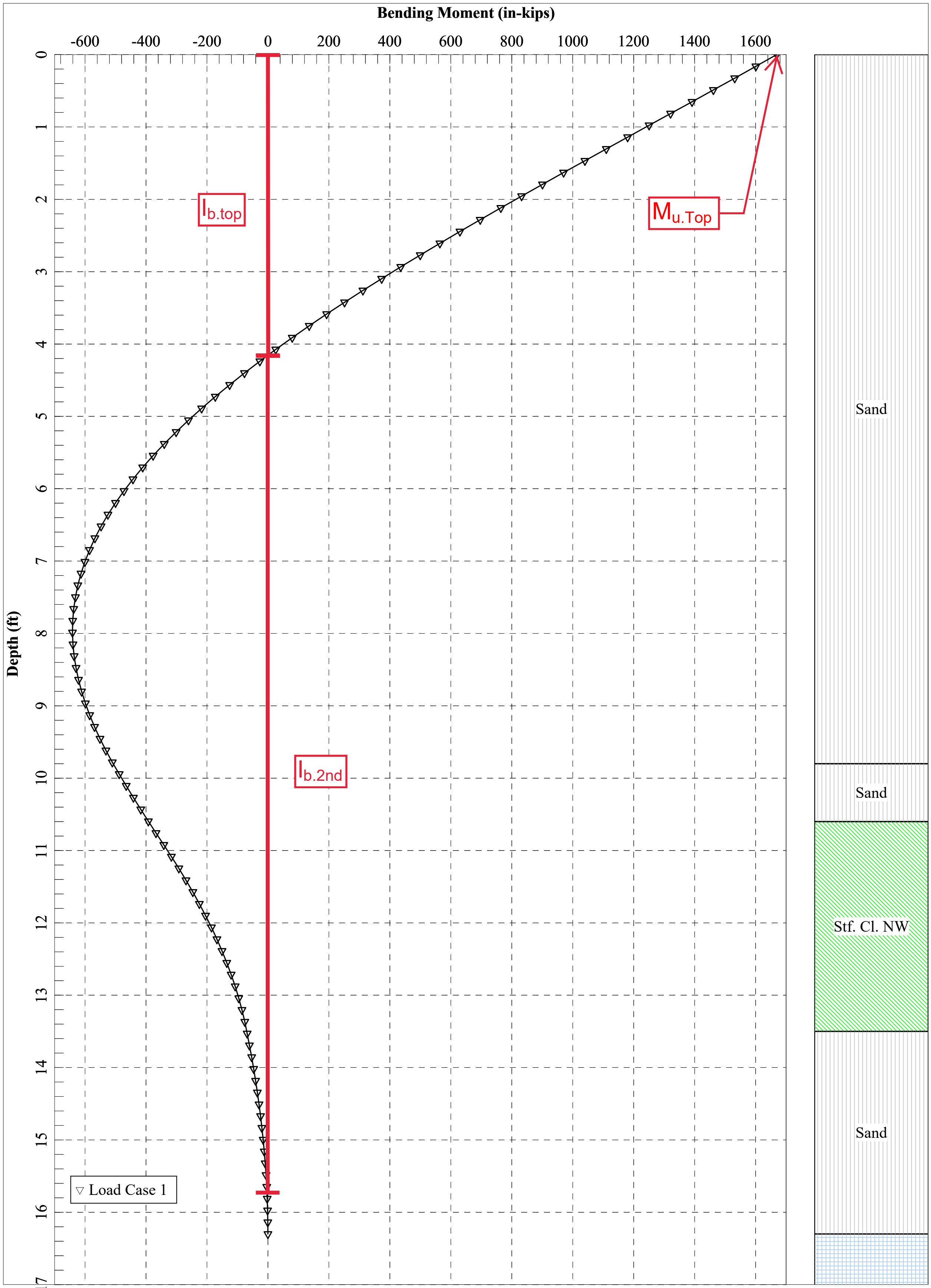
Definitions of Pile-head Loading Conditions:

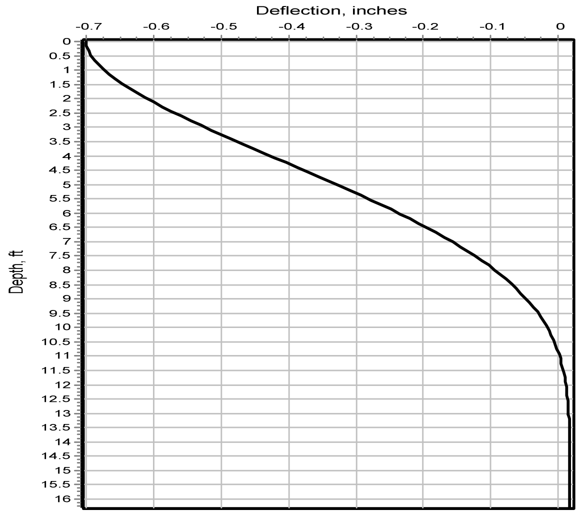
Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians
Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
Load Type 5: Load 1 = Top Deflection, y, inches, and Load 2 = Slope, S, radians

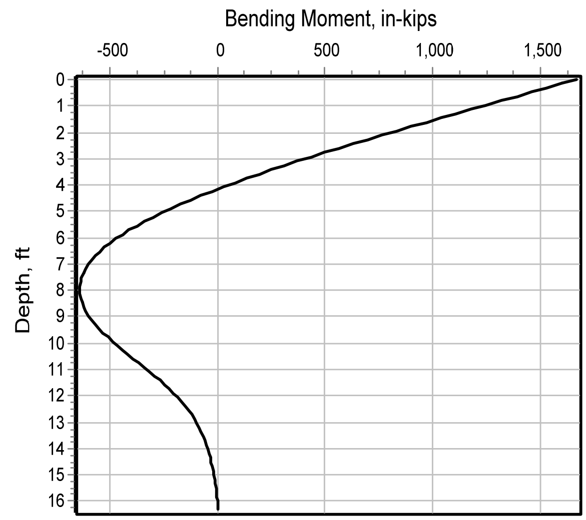
Load Case No.	Load Type 1	Pile-head Load 1	Load Type 2	Pile-head Load 2	Axial Loading lbs	Pile-head Deflection inches	Pile-head Rotation radians	Max Shear in Pile lbs	Max Moment in Pile in-lbs
1	y, in	-0.7000	S, rad	0.00	436000.	-0.7000	0.00	-34938.	1669287.

Maximum pile-head deflection = -0.7000000000 inches
Maximum pile-head rotation = 0.0000000000 radians = 0.000000 deg.

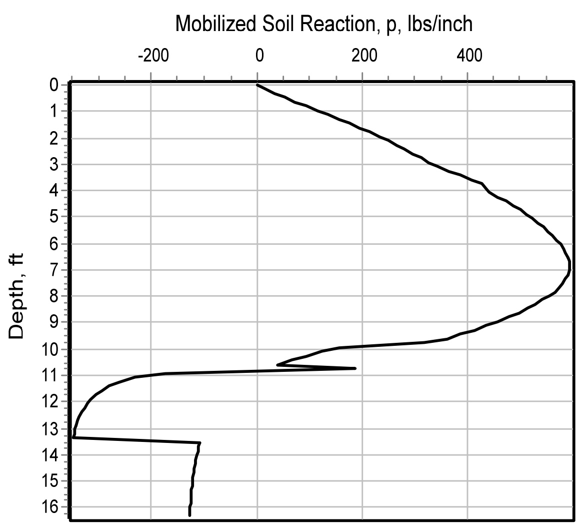
The analysis ended normally.


Bending Moment (in-kips)
-600
-400
-200
0
200
400
600
800
1000
1200
1400
1600
Depth (ft)
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
$l_{b.top}$
$M_{u.Top}$
$l_{b.2nd}$
Load Case 1
Sand
Sand
Stf. Cl. NW
Sand


Deflection, inches
-0.7
-0.6
-0.5
-0.4
-0.3
-0.2
-0.1
0
Depth, ft


Bending Moment, in-kips
-500
0
500
1,000
1,500
Depth, ft


Shear Force, kips
-30
-20
-10
0
10
Depth, ft


Mobilized Soil Reaction, p, lbs/inch
-200
0
200
400
Depth, ft

Attachment 2

==

LPile for Windows, Version 2019-11.005

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

==

This copy of LPile is being used by:

KAR
Golder Associates, Inc.

Serial Number of Security Device: 239146533

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is a violation of the software license agreement.

Files Used for Analysis

Path to file locations:
\Users\kroth\Documents\Projects\19126013 MaineDOT I-295 Freeport Exit 20 Merrill Rd Bridge\Pile Design\LPile Southeast Abutment\

Name of input data file:
Freeport Exit 20 Southeast Abutment Southern Shift _ No Socket _ HP12x74 _ Plastic Hinge.lp11d

Name of output report file:
Freeport Exit 20 Southeast Abutment Southern Shift _ No Socket _ HP12x74 _ Plastic Hinge.lp11o

Name of plot output file:
Freeport Exit 20 Southeast Abutment Southern Shift _ No Socket _ HP12x74 _ Plastic Hinge.lp11p

Name of runtime message file:
Freeport Exit 20 Southeast Abutment Southern Shift _ No Socket _ HP12x74 _ Plastic Hinge.lp11r

----
Date and Time of Analysis

Date: August 26, 2020 Time: 16:23:39

Problem Title

Project Name: MaineDOT I-295 Exit 20 Merrill Road Bridge No. 5720
Job Number: 19126013
Client: MaineDOT
Engineer: KAR
Description: Southeast Abutment Pile Design - Strength I (Plastic Hinge)

Program Options and Settings

Computational Options:
 - Conventional Analysis
Engineering Units Used for Data Input and Computations:
 - US Customary System Units (pounds, feet, inches)

Analysis Control Options:
 - Maximum number of iterations allowed = 500
 - Deflection tolerance for convergence = 1.0000E-05 in
 - Maximum allowable deflection = 100.0000 in
 - Number of pile increments = 100

Loading Type and Number of Cycles of Loading:
 - Static loading specified

 - Use of p-y modification factors for p-y curves not selected
 - Analysis uses layering correction (Method of Georgiadis)
 - No distributed lateral loads are entered
 - Loading by lateral soil movements acting on pile not selected
 - Input of shear resistance at the pile tip not selected
 - Input of moment resistance at the pile tip not selected
 - Input of side resistance moment along pile not selected
 - Computation of pile-head foundation stiffness matrix not selected
 - Push-over analysis of pile not selected
 - Buckling analysis of pile not selected

Output Options:
 - Output files use decimal points to denote decimal symbols.
 - Values of pile-head deflection, bending moment, shear force, and
   soil reaction are printed for full length of pile.
 - Printing Increment (nodal spacing of output points) = 1
 - No p-y curves to be computed and reported for user-specified depths
 - Print using wide report formats

Pile Structural Properties and Geometry

Number of pile sections defined = 1
Total length of pile = 16.300 ft
Depth of ground surface below top of pile = 0.0000 ft

Pile diameters used for p-y curve computations are defined using 2 points.

p-y curves are computed using pile diameter values interpolated with depth over the length of the pile. A summary of values of pile diameter vs. depth follows.

Point No.	Depth Below Pile Head feet	Pile Diameter inches
1	0.000	12.2150
2	16.300	12.2150

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is a H weak axis steel pile
Length of section = 16.300000 ft
Pile width = 12.130000 in
Shear capacity of section = 0.0000 lbs

Ground Slope and Pile Batter Angles

Ground Slope Angle = 0.000 degrees
= 0.000 radians

Pile Batter Angle = 0.000 degrees
= 0.000 radians

Soil and Rock Layering Information

The soil profile is modelled using 5 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer = 0.0000 ft
Distance from top of pile to bottom of layer = 9.800000 ft

Effective unit weight at top of layer = 125.000000 pcf
Effective unit weight at bottom of layer = 125.000000 pcf
Friction angle at top of layer = 32.000000 deg.
Friction angle at bottom of layer = 32.000000 deg.
Subgrade k at top of layer = 124.800000 pci
Subgrade k at bottom of layer = 124.800000 pci

Layer 2 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer = 9.800000 ft
Distance from top of pile to bottom of layer = 10.600000 ft
Effective unit weight at top of layer = 62.600000 pcf
Effective unit weight at bottom of layer = 62.600000 pcf
Friction angle at top of layer = 32.000000 deg.
Friction angle at bottom of layer = 32.000000 deg.
Subgrade k at top of layer = 75.500000 pci
Subgrade k at bottom of layer = 75.500000 pci

Layer 3 is stiff clay without free water

Distance from top of pile to top of layer = 10.600000 ft
Distance from top of pile to bottom of layer = 13.500000 ft
Effective unit weight at top of layer = 62.600000 pcf
Effective unit weight at bottom of layer = 62.600000 pcf
Undrained cohesion at top of layer = 1600. psf
Undrained cohesion at bottom of layer = 1600. psf
Epsilon-50 at top of layer = 0.005000
Epsilon-50 at bottom of layer = 0.005000

Layer 4 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer = 13.500000 ft
Distance from top of pile to bottom of layer = 16.300000 ft
Effective unit weight at top of layer = 62.600000 pcf
Effective unit weight at bottom of layer = 62.600000 pcf
Friction angle at top of layer = 37.000000 deg.
Friction angle at bottom of layer = 37.000000 deg.
Subgrade k at top of layer = 40.500000 pci
Subgrade k at bottom of layer = 40.500000 pci

Layer 5 is strong rock (vuggy limestone)

Distance from top of pile to top of layer = 16.300000 ft
Distance from top of pile to bottom of layer = 50.000000 ft
Effective unit weight at top of layer = 101.600000 pcf
Effective unit weight at bottom of layer = 101.600000 pcf
Uniaxial compressive strength at top of layer = 12983. psi
Uniaxial compressive strength at bottom of layer = 12983. psi

(Depth of the lowest soil layer extends 33.700 ft below the pile tip)

Summary of Input Soil Properties

Layer Num.	Soil Type Name (p-y Curve Type)	Layer Depth ft	Effective Unit Wt. pcf	Undrained Cohesion psf	Angle of Friction deg.	Uniaxial qu psi	E50 or krm	kpy pci
1	Sand	0.00	125.0000	--	32.0000	--	--	124.8000
	(Reese, et al.)	9.8000	125.0000	--	32.0000	--	--	124.8000
2	Sand	9.8000	62.6000	--	32.0000	--	--	75.5000
	(Reese, et al.)	10.6000	62.6000	--	32.0000	--	--	75.5000
3	Stiff Clay	10.6000	62.6000	1600.	--	--	0.00500	--
	w/o Free Water	13.5000	62.6000	1600.	--	--	0.00500	--
4	Sand	13.5000	62.6000	--	37.0000	--	--	40.5000
	(Reese, et al.)	16.3000	62.6000	--	37.0000	--	--	40.5000
5	Strong Rock	16.3000	101.6000	--	--	12983.	--	--
	(Vuggy Limestone)	50.0000	101.6000	--	--	12983.	--	--

Static Loading Type

Static loading criteria were used when computing p-y curves for all analyses.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 2

Load No.	Load Type	Condition 1	Condition 2	Axial Thrust Force, lbs	Compute Top y vs. Pile Length	Run Analysis
1	5	y = -0.700000 in	S = 0.0000 in/in	436000.	N.A.	Yes
2	4	y = -0.700000 in	M = 1045449. in-lbs	436000.	N.A.	Yes

V = shear force applied normal to pile axis
M = bending moment applied to pile head
y = lateral deflection normal to pile axis
S = pile slope relative to original pile batter angle
R = rotational stiffness applied to pile head
Values of top y vs. pile lengths can be computed only for load types with
specified shear loading (Load Types 1, 2, and 3).
Thrust force is assumed to be acting axially for all pile batter angles.

Computations of Nominal Moment Capacity and Nonlinear Bending Stiffness
----

Axial thrust force values were determined from pile-head loading conditions

Number of Pile Sections Analyzed = 1

Pile Section No. 1:

Dimensions and Properties of Steel H Weak Axis:

Length of Section	=	16.300000 ft
Flange Width	=	12.215000 in
Section Depth	=	12.130000 in
Flange Thickness	=	0.610000 in
Web Thickness	=	0.610000 in
Yield Stress of Pipe	=	50.000000 ksi
Elastic Modulus	=	29000. ksi
Cross-sectional Area	=	21.557400 sq. in.
Moment of Inertia	=	185.499357 in^4
Elastic Bending Stiffness	=	5379481. kip-in^2
Plastic Modulus, Z	=	46.522801in^3
Plastic Moment Capacity = Fy Z	=	2326.in-kip

Axial Structural Capacities:

Nom. Axial Structural Capacity = Fy As	=	1077.870 kips
Nominal Axial Tensile Capacity	=	-1077.870 kips

Number of Axial Thrust Force Values Determined from Pile-head Loadings = 1

Number	Axial Thrust Force kips
1	436.000

Definition of Run Messages:

Y = part of pipe section has yielded.

Axial Thrust Force = 436.000 kips

Bending Curvature rad/in.	Bending Moment in-kip	Bending Stiffness kip-in2	Depth to N Axis in	Max Total Stress ksi	Run Msg

------------- ------------- ------------- ------------- ------------- ---

0.00000460	24.7641803	5379425.	157.6044790	21.0322803	
0.00000921	49.5283606	5379425.	81.8559895	21.8394869	
0.00001381	74.2925408	5379425.	56.6064930	22.6466941	
0.00001841	99.0567211	5379425.	43.9817447	23.4539005	
0.00002302	123.8209014	5379425.	36.4068958	24.2611075	
0.00002762	148.5850817	5379425.	31.3569965	25.0683141	
0.00003222	173.3492619	5379425.	27.7499256	25.8755211	
0.00003683	198.1134422	5379425.	25.0446224	26.6827278	
0.00004143	222.8776225	5379425.	22.9404977	27.4899347	
0.00004603	247.6418028	5379425.	21.2571979	28.2971416	
0.00005064	272.4059830	5379425.	19.8799526	29.1043484	
0.00005524	297.1701633	5379425.	18.7322482	29.9115551	
0.00005985	321.9343436	5379425.	17.7611138	30.7187620	
0.00006445	346.6985239	5379425.	16.9287128	31.5259687	
0.00006905	371.4627041	5379425.	16.2072986	32.3331756	
0.00007366	396.2268844	5379425.	15.5760612	33.1403823	
0.00007826	420.9910647	5379425.	15.0190870	33.9475892	
0.00008286	445.7552450	5379425.	14.5239988	34.7547959	
0.00008747	470.5194253	5379425.	14.0810252	35.5620028	
0.00009207	495.2836055	5379425.	13.6823489	36.3692095	
0.00009667	520.0477858	5379425.	13.3216419	37.1764163	
0.0001013	544.8119661	5379425.	12.9937263	37.9836232	
0.0001059	569.5761464	5379425.	12.6943252	38.7908299	
0.0001105	594.3403266	5379425.	12.4198741	39.5980367	
0.0001151	619.1045069	5379425.	12.1673792	40.4052435	
0.0001197	643.8686872	5379425.	11.9343069	41.2124504	
0.0001243	668.6328675	5379425.	11.7184992	42.0196571	
0.0001289	693.3970477	5379425.	11.5181064	42.8268640	
0.0001335	718.1612280	5379425.	11.3315338	43.6340708	
0.0001381	742.9254083	5379425.	11.1573993	44.4412776	
0.0001427	767.6895886	5379425.	10.9944993	45.2484844	
0.0001473	792.4537688	5379425.	10.8417806	46.0556912	
0.0001519	817.2179491	5379425.	10.6983175	46.8628980	
0.0001565	841.9821294	5379425.	10.5632935	47.6701048	
0.0001611	866.7463097	5379425.	10.4359851	48.4773116	
0.0001657	891.5104899	5379425.	10.3157494	49.2845184	
0.0001703	916.1914453	5378936.	10.2021422	50.0000000	Y
0.0001749	939.9675850	5373301.	10.0958906	50.0000000	Y
0.0001795	962.8854746	5363174.	9.9963908	50.0000000	Y
0.0001887	1006.	5331505.	9.8155851	50.0000000	Y
0.0001980	1047.	5287852.	9.6559832	50.0000000	Y
0.0002072	1085.	5236170.	9.5142392	50.0000000	Y
0.0002164	1121.	5178944.	9.3877622	50.0000000	Y
0.0002256	1154.	5117793.	9.2745034	50.0000000	Y
0.0002348	1187.	5054156.	9.1726884	50.0000000	Y
0.0002440	1217.	4989683.	9.0806156	50.0000000	Y
0.0002532	1247.	4924125.	8.9974326	50.0000000	Y
0.0002624	1275.	4859017.	8.9217253	50.0000000	Y
0.0002716	1302.	4794604.	8.8526670	50.0000000	Y
0.0002808	1328.	4730675.	8.7897181	50.0000000	Y
0.0002900	1354.	4667968.	8.7320095	50.0000000	Y
0.0002992	1378.	4606594.	8.6789939	50.0000000	Y
0.0003084	1402.	4546645.	8.6301875	50.0000000	Y

0.0003176	1426.	4488193.	8.5851602	50.0000000	Y
0.0003268	1448.	4431296.	8.5435286	50.0000000	Y
0.0003361	1471.	4376000.	8.5049490	50.0000000	Y
0.0003453	1492.	4322241.	8.4691624	50.0000000	Y
0.0003545	1514.	4269832.	8.4360127	50.0000000	Y
0.0003637	1534.	4219044.	8.4051152	50.0000000	Y
0.0003729	1555.	4169890.	8.3762382	50.0000000	Y
0.0003821	1575.	4122214.	8.3492663	50.0000000	Y
0.0003913	1595.	4075746.	8.3241818	50.0000000	Y
0.0004005	1614.	4030892.	8.3005713	50.0000000	Y
0.0004097	1634.	3987337.	8.2784658	50.0000000	Y
0.0004189	1653.	3944998.	8.2577803	50.0000000	Y
0.0004281	1671.	3904188.	8.2381726	50.0000000	Y
0.0004373	1690.	3864238.	8.2199632	50.0000000	Y
0.0004465	1708.	3823957.	8.2017414	50.0000000	Y
0.0004557	1724.	3783095.	8.1841046	50.0000000	Y
0.0004650	1740.	3742550.	8.1665596	50.0000000	Y
0.0004742	1755.	3701770.	8.1496775	50.0000000	Y
0.0004834	1770.	3660898.	8.1326296	50.0000000	Y
0.0004926	1783.	3620213.	8.1160861	50.0000000	Y
0.0005018	1796.	3580090.	8.0998955	50.0000000	Y
0.0005110	1809.	3540015.	8.0836547	50.0000000	Y
0.0005202	1821.	3500249.	8.0679778	50.0000000	Y
0.0005294	1832.	3461215.	8.0524865	50.0000000	Y
0.0005386	1843.	3422321.	8.0370205	50.0000000	Y
0.0005478	1854.	3384145.	8.0221876	50.0000000	Y
0.0005846	1892.	3236136.	7.9641966	50.0000000	Y
0.0006215	1925.	3097157.	7.9097244	50.0000000	Y
0.0006583	1953.	2966890.	7.8581756	50.0000000	Y
0.0006951	1978.	2845523.	7.8095143	50.0000000	Y
0.0007320	2000.	2732246.	7.7635826	50.0000000	Y
0.0007688	2019.	2626805.	7.7202072	50.0000000	Y
0.0008056	2037.	2528315.	7.6787989	50.0000000	Y
0.0008424	2052.	2436277.	7.6395664	50.0000000	Y
0.0008793	2067.	2350280.	7.6022418	50.0000000	Y
0.0009161	2079.	2269852.	7.5665744	50.0000000	Y
0.0009529	2091.	2194393.	7.5329185	50.0000000	Y
0.0009898	2102.	2123432.	7.5004584	50.0000000	Y
0.0010266	2111.	2056748.	7.4693295	50.0000000	Y
0.0010634	2120.	1993729.	7.4408820	50.0000000	Y
0.0011002	2128.	1934219.	7.4145964	50.0000000	Y

Summary of Results for Nominal Moment Capacity for Section 1

Load No.	Axial Thrust kips	Nominal Moment Capacity in-kips
1	436.0000000000	2128.

Note that the values in the above table are not factored by a strength reduction factor for LRFD.

The value of the strength reduction factor depends on the provisions of the LRFD code being followed.

The above values should be multiplied by the appropriate strength reduction factor to compute ultimate moment capacity according to the LRFD structural design standard being followed.

Layering Correction Equivalent Depths of Soil & Rock Layers

Layer No.	Top of Layer Below Pile Head ft	Equivalent Top Depth Below Grnd Surf ft	Same Layer Type As Layer Above	Layer is Rock or is Below Rock Layer	F0 Integral for Layer lbs	F1 Integral for Layer lbs
1	0.00	0.00	N.A.	No	0.00	97275.
2	9.8000	9.7999	Yes	No	97275.	23123.
3	10.6000	11.7450	No	No	120398.	42508.
4	13.5000	10.7310	No	No	162906.	145406.
5	16.3000	16.3000	No	Yes	N.A.	N.A.

Notes: The F0 integral of Layer n+1 equals the sum of the F0 and F1 integrals for Layer n. Layering correction equivalent depths are computed only for soil types with both shallow-depth and deep-depth expressions for peak lateral load transfer. These soil types are soft and stiff clays, non-liquefied sands, and cemented c-phi soil.

Computed Values of Pile Loading and Deflection
for Lateral Loading for Load Case Number 1

Pile-head conditions are Displacement and Pile-head Rotation (Loading Type 5)
Displacement of pile head = -0.700000 inches
Rotation of pile head = 0.000E+00 radians
Axial load on pile head = 436000.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness lb-in^2	Soil Res. p lb/inch	Soil Spr. Es*h lb/inch	Distrib. Lat. Load lb/inch
0.00	-0.7000	1669287.	-34938.	0.00	75186.	3.91E+09	0.00	0.00	0.00

0.1630	-0.6992	1600636.	-34900.	8.18E-04	72925.	3.91E+09	15.5610	43.5327	0.00
0.3260	-0.6968	1531362.	-34853.	0.00157	70645.	4.23E+09	33.1291	92.9973	0.00
0.4890	-0.6930	1461610.	-34769.	0.00225	68348.	4.40E+09	52.2795	147.5531	0.00
0.6520	-0.6880	1391503.	-34647.	0.00287	66040.	4.57E+09	72.4163	205.8848	0.00
0.8150	-0.6818	1321166.	-34485.	0.00344	63724.	4.75E+09	93.1327	267.1931	0.00
0.9780	-0.6745	1250722.	-34283.	0.00397	61405.	4.91E+09	113.9963	330.5747	0.00
1.1410	-0.6663	1180289.	-34039.	0.00444	59086.	5.07E+09	134.8802	395.9748	0.00
1.3040	-0.6571	1109983.	-33755.	0.00488	56771.	5.20E+09	155.2816	462.2054	0.00
1.4670	-0.6472	1039915.	-33433.	0.00528	54464.	5.29E+09	174.6923	527.9790	0.00
1.6300	-0.6365	970188.	-33072.	0.00565	52168.	5.36E+09	194.3056	597.1333	0.00
1.7930	-0.6251	900902.	-32673.	0.00599	49887.	5.38E+09	213.6902	668.6785	0.00
1.9560	-0.6130	832154.	-32237.	0.00631	47623.	5.38E+09	231.7075	739.2953	0.00
2.1190	-0.6004	764035.	-31767.	0.00660	45381.	5.38E+09	248.8657	810.7428	0.00
2.2820	-0.5872	696631.	-31265.	0.00686	43161.	5.38E+09	265.1122	883.0434	0.00
2.4450	-0.5736	630025.	-30732.	0.00710	40968.	5.38E+09	279.4282	952.9069	0.00
2.6080	-0.5595	564293.	-30171.	0.00732	38804.	5.38E+09	293.9263	1028.	0.00
2.7710	-0.5449	499511.	-29580.	0.00751	36671.	5.38E+09	310.6229	1115.	0.00
2.9340	-0.5301	435762.	-28958.	0.00768	34572.	5.38E+09	325.8852	1203.	0.00
3.0970	-0.5149	373124.	-28304.	0.00783	32510.	5.38E+09	341.9902	1299.	0.00
3.2600	-0.4994	311680.	-27614.	0.00795	30487.	5.38E+09	364.2081	1426.	0.00
3.4230	-0.4838	251532.	-26880.	0.00806	28507.	5.38E+09	386.1685	1561.	0.00
3.5860	-0.4679	192784.	-26103.	0.00814	26572.	5.38E+09	407.7742	1705.	0.00
3.7490	-0.4519	135536.	-25289.	0.00820	24688.	5.38E+09	424.9763	1839.	0.00
3.9120	-0.4358	79872.	-24449.	0.00824	22855.	5.38E+09	433.9736	1948.	0.00
4.0750	-0.4197	25843.	-23594.	0.00826	21076.	5.38E+09	440.6251	2053.	0.00
4.2380	-0.4036	-26507.	-22716.	0.00826	21098.	5.38E+09	456.9856	2215.	0.00
4.4010	-0.3874	-77101.	-21807.	0.00824	22764.	5.38E+09	472.2220	2384.	0.00
4.5640	-0.3713	-125865.	-20870.	0.00820	24369.	5.38E+09	486.2447	2561.	0.00
4.7270	-0.3553	-172729.	-19906.	0.00815	25912.	5.38E+09	498.9711	2747.	0.00
4.8900	-0.3395	-217630.	-18919.	0.00807	27390.	5.38E+09	510.3258	2941.	0.00
5.0530	-0.3237	-260512.	-17911.	0.00799	28802.	5.38E+09	520.2405	3143.	0.00
5.2160	-0.3082	-301322.	-16883.	0.00789	30146.	5.38E+09	531.2577	3371.	0.00
5.3790	-0.2929	-340006.	-15833.	0.00777	31420.	5.38E+09	541.9615	3619.	0.00
5.5420	-0.2778	-376512.	-14763.	0.00764	32622.	5.38E+09	551.6519	3884.	0.00
5.7050	-0.2630	-410790.	-13676.	0.00750	33750.	5.38E+09	560.3319	4167.	0.00
5.8680	-0.2485	-442797.	-12572.	0.00734	34804.	5.38E+09	568.0125	4471.	0.00
6.0310	-0.2343	-472493.	-11455.	0.00717	35782.	5.38E+09	574.7128	4798.	0.00
6.1940	-0.2204	-499844.	-10325.	0.00700	36682.	5.38E+09	580.4594	5151.	0.00
6.3570	-0.2069	-524819.	-9185.	0.00681	37505.	5.38E+09	585.2866	5532.	0.00
6.5200	-0.1938	-547393.	-8036.	0.00662	38248.	5.38E+09	589.0647	5946.	0.00
6.6830	-0.1810	-567543.	-6882.	0.00641	38911.	5.38E+09	591.3766	6389.	0.00
6.8460	-0.1687	-585254.	-5724.	0.00620	39494.	5.38E+09	592.1707	6866.	0.00
7.0090	-0.1568	-600518.	-4567.	0.00599	39997.	5.38E+09	591.4184	7379.	0.00
7.1720	-0.1453	-613333.	-3412.	0.00577	40419.	5.38E+09	589.0962	7931.	0.00
7.3350	-0.1342	-623704.	-2264.	0.00554	40760.	5.38E+09	585.1847	8528.	0.00
7.4980	-0.1236	-631643.	-1125.	0.00531	41022.	5.38E+09	579.6691	9174.	0.00
7.6610	-0.1134	-637168.	2.1445	0.00508	41204.	5.38E+09	572.5385	9873.	0.00
7.8240	-0.1037	-640306.	1113.	0.00485	41307.	5.38E+09	563.7858	10633.	0.00
7.9870	-0.09445	-641087.	2206.	0.00462	41333.	5.38E+09	553.4071	11461.	0.00
8.1500	-0.08564	-639553.	3277.	0.00439	41282.	5.38E+09	541.4010	12365.	0.00
8.3130	-0.07729	-635748.	4322.	0.00415	41157.	5.38E+09	527.7677	13356.	0.00
8.4760	-0.06939	-629728.	5340.	0.00392	40959.	5.38E+09	512.5077	14446.	0.00
8.6390	-0.06194	-621551.	6326.	0.00370	40689.	5.38E+09	495.6207	15651.	0.00
8.8020	-0.05493	-611286.	7277.	0.00347	40351.	5.38E+09	477.1028	16988.	0.00

8.9650	-0.04836	-599005.	8191.	0.00325	39947.	5.38E+09	456.9443	18482.	0.00
9.1280	-0.04221	-584791.	9063.	0.00304	39479.	5.38E+09	435.1254	20163.	0.00
9.2910	-0.03648	-568730.	9891.	0.00283	38950.	5.38E+09	411.6107	22070.	0.00
9.4540	-0.03115	-550918.	10672.	0.00262	38364.	5.38E+09	386.3414	24257.	0.00
9.6170	-0.02622	-531458.	11401.	0.00243	37723.	5.38E+09	359.2228	26801.	0.00
9.7800	-0.02166	-510458.	12062.	0.00224	37032.	5.38E+09	317.2327	28649.	0.00
9.9430	-0.01746	-488086.	12526.	0.00206	36295.	5.38E+09	157.3273	17620.	0.00
10.1060	-0.01362	-464961.	12802.	0.00188	35534.	5.38E+09	124.6784	17909.	0.00
10.2690	-0.01010	-441215.	13016.	0.00172	34752.	5.38E+09	93.9698	18198.	0.00
10.4320	-0.00690	-416972.	13172.	0.00156	33954.	5.38E+09	65.1883	18487.	0.00
10.5950	-0.00399	-392351.	13273.	0.00141	33143.	5.38E+09	38.3075	18776.	0.00
10.7580	-0.00136	-367461.	13493.	0.00128	32324.	5.38E+09	186.8680	268105.	0.00
10.9210	0.00100	-341743.	13505.	0.00115	31477.	5.38E+09	-174.9874	341335.	0.00
11.0840	0.00313	-316588.	13107.	0.00103	30649.	5.38E+09	-231.4996	144864.	0.00
11.2470	0.00502	-292220.	12626.	9.17E-04	29846.	5.38E+09	-260.4514	101409.	0.00
11.4100	0.00671	-268759.	12098.	8.15E-04	29074.	5.38E+09	-279.9438	81560.	0.00
11.5730	0.00821	-246285.	11536.	7.22E-04	28334.	5.38E+09	-294.3560	70107.	0.00
11.7360	0.00954	-224861.	10949.	6.36E-04	27629.	5.38E+09	-305.5266	62667.	0.00
11.8990	0.01070	-204536.	10343.	5.58E-04	26959.	5.38E+09	-314.4250	57478.	0.00
12.0620	0.01172	-185351.	9721.	4.87E-04	26328.	5.38E+09	-321.6343	53686.	0.00
12.2250	0.01260	-167338.	9086.	4.23E-04	25735.	5.38E+09	-327.5374	50826.	0.00
12.3880	0.01337	-150527.	8441.	3.65E-04	25181.	5.38E+09	-332.4013	48621.	0.00
12.5510	0.01403	-134942.	7786.	3.13E-04	24668.	5.38E+09	-336.4215	46893.	0.00
12.7140	0.01460	-120601.	7125.	2.67E-04	24196.	5.38E+09	-339.7461	45525.	0.00
12.8770	0.01508	-107523.	6458.	2.25E-04	23765.	5.38E+09	-342.4902	44436.	0.00
13.0400	0.01548	-95722.	5786.	1.88E-04	23377.	5.38E+09	-344.7452	43566.	0.00
13.2030	0.01581	-85210.	5110.	1.55E-04	23031.	5.38E+09	-346.5845	42873.	0.00
13.3660	0.01609	-75997.	4430.	1.26E-04	22727.	5.38E+09	-348.0679	42324.	0.00
13.5290	0.01631	-68093.	3985.	9.98E-05	22467.	5.38E+09	-107.2095	12861.	0.00
13.6920	0.01648	-60578.	3773.	7.64E-05	22220.	5.38E+09	-109.6395	13016.	0.00
13.8550	0.01660	-53464.	3556.	5.57E-05	21985.	5.38E+09	-111.8064	13171.	0.00
14.0180	0.01669	-46760.	3336.	3.75E-05	21765.	5.38E+09	-113.7346	13326.	0.00
14.1810	0.01675	-40478.	3112.	2.16E-05	21558.	5.38E+09	-115.4478	13481.	0.00
14.3440	0.01678	-34624.	2884.	7.97E-06	21365.	5.38E+09	-116.9693	13636.	0.00
14.5070	0.01678	-29208.	2654.	-3.63E-06	21187.	5.38E+09	-118.3216	13791.	0.00
14.6700	0.01676	-24235.	2422.	-1.34E-05	21023.	5.38E+09	-119.5263	13946.	0.00
14.8330	0.01673	-19711.	2187.	-2.13E-05	20874.	5.38E+09	-120.6040	14100.	0.00
14.9960	0.01668	-15644.	1950.	-2.78E-05	20740.	5.38E+09	-121.5740	14255.	0.00
15.1590	0.01662	-12036.	1711.	-3.28E-05	20621.	5.38E+09	-122.4543	14410.	0.00
15.3220	0.01655	-8893.	1471.	-3.66E-05	20518.	5.38E+09	-123.2614	14565.	0.00
15.4850	0.01648	-6219.	1229.	-3.94E-05	20430.	5.38E+09	-124.0101	14720.	0.00
15.6480	0.01640	-4017.	985.9050	-4.12E-05	20357.	5.38E+09	-124.7133	14875.	0.00
15.8110	0.01632	-2292.	741.3119	-4.24E-05	20301.	5.38E+09	-125.3820	15030.	0.00
15.9740	0.01623	-1045.	495.4359	-4.30E-05	20259.	5.38E+09	-126.0250	15185.	0.00
16.1370	0.01615	-280.3322	248.3209	-4.32E-05	20234.	5.38E+09	-126.6489	15340.	0.00
16.3000	0.01606	0.00	0.00	-4.33E-05	20225.	5.38E+09	-127.2579	7748.	0.00

* This analysis computed pile response using nonlinear moment-curvature relationships. Values of total stress due to combined axial and bending stresses are computed only for elastic sections only and do not equal the actual stresses in concrete and steel. Stresses in concrete and steel may be interpolated from the output for nonlinear bending properties relative to the magnitude of bending moment developed in the pile.

Output Summary for Load Case No. 1:

Pile-head deflection = -0.70000000 inches
Computed slope at pile head = 0.000000 radians
Maximum bending moment = 1669287. inch-lbs
Maximum shear force = -34938. lbs
Depth of maximum bending moment = 0.000000 feet below pile head
Depth of maximum shear force = 0.000000 feet below pile head
Number of iterations = 15
Number of zero deflection points = 1

Computed Values of Pile Loading and Deflection
for Lateral Loading for Load Case Number 2

Pile-head conditions are Displacement and Moment (Loading Type 4)
Displacement of pile head = -0.700000 inches
Moment at pile head = 1045449.0 in-lbs
Axial load at pile head = 436000.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness lb-in^2	Soil Res. p lb/inch	Soil Spr. Es*h lb/inch	Distrib. Lat. Load lb/inch
0.00	-0.7000	1045449.	-26609.	0.00460	54646.	5.29E+09	0.00	0.00	0.00
0.1630	-0.6906	989316.	-26594.	0.00497	52798.	5.29E+09	15.5610	44.0719	0.00
0.3260	-0.6805	932930.	-26546.	0.00533	50941.	5.37E+09	33.1290	95.2188	0.00
0.4890	-0.6698	876381.	-26463.	0.00566	49080.	5.38E+09	52.2793	152.6724	0.00
0.6520	-0.6584	819760.	-26341.	0.00596	47215.	5.38E+09	72.4161	215.1317	0.00
0.8150	-0.6465	763163.	-26179.	0.00625	45352.	5.38E+09	93.1325	281.7931	0.00
0.9780	-0.6340	706685.	-25976.	0.00652	43492.	5.38E+09	113.9960	351.7215	0.00
1.1410	-0.6210	650424.	-25733.	0.00677	41640.	5.38E+09	134.8798	424.8708	0.00
1.3040	-0.6075	594477.	-25449.	0.00699	39798.	5.38E+09	155.2811	499.9773	0.00
1.4670	-0.5936	538940.	-25126.	0.00720	37969.	5.38E+09	174.6917	575.6364	0.00
1.6300	-0.5793	483905.	-24766.	0.00738	36157.	5.38E+09	194.3050	656.0383	0.00
1.7930	-0.5647	429462.	-24367.	0.00755	34365.	5.38E+09	213.6895	740.1605	0.00
1.9560	-0.5498	375705.	-23931.	0.00770	32595.	5.38E+09	231.7068	824.3493	0.00
2.1190	-0.5346	322717.	-23461.	0.00782	30850.	5.38E+09	248.8648	910.5481	0.00
2.2820	-0.5192	270581.	-22958.	0.00793	29134.	5.38E+09	265.1112	998.7966	0.00
2.4450	-0.5036	219375.	-22426.	0.00802	27448.	5.38E+09	279.4272	1085.	0.00
2.6080	-0.4878	169171.	-21865.	0.00809	25795.	5.38E+09	293.9252	1179.	0.00
2.7710	-0.4719	120039.	-21274.	0.00814	24177.	5.38E+09	310.6218	1287.	0.00
2.9340	-0.4559	72058.	-20652.	0.00818	22598.	5.38E+09	325.0110	1394.	0.00
3.0970	-0.4399	25298.	-20008.	0.00820	21058.	5.38E+09	333.8689	1484.	0.00
3.2600	-0.4239	-20193.	-19341.	0.00820	20890.	5.38E+09	347.3524	1603.	0.00
3.4230	-0.4079	-64348.	-18651.	0.00818	22344.	5.38E+09	359.0019	1722.	0.00
3.5860	-0.3919	-107110.	-17939.	0.00815	23752.	5.38E+09	368.6165	1840.	0.00
3.7490	-0.3760	-148428.	-17211.	0.00810	25112.	5.38E+09	375.9988	1956.	0.00
3.9120	-0.3602	-188262.	-16470.	0.00804	26424.	5.38E+09	380.9550	2069.	0.00
4.0750	-0.3445	-226580.	-15723.	0.00797	27685.	5.38E+09	383.5422	2178.	0.00

4.2380	-0.3290	-263360.	-14960.	0.00788	28896.	5.38E+09	396.0926	2355.	0.00
4.4010	-0.3137	-298543.	-14174.	0.00778	30054.	5.38E+09	407.5702	2541.	0.00
4.5640	-0.2986	-332074.	-13367.	0.00766	31158.	5.38E+09	417.9098	2738.	0.00
4.7270	-0.2837	-363903.	-12541.	0.00754	32206.	5.38E+09	427.0524	2944.	0.00
4.8900	-0.2691	-393986.	-11698.	0.00740	33197.	5.38E+09	434.9460	3162.	0.00
5.0530	-0.2548	-422282.	-10840.	0.00725	34129.	5.38E+09	441.5452	3390.	0.00
5.2160	-0.2407	-448758.	-9969.	0.00709	35000.	5.38E+09	449.6810	3654.	0.00
5.3790	-0.2270	-473375.	-9081.	0.00692	35811.	5.38E+09	457.7887	3944.	0.00
5.5420	-0.2137	-496093.	-8179.	0.00675	36559.	5.38E+09	465.1110	4258.	0.00
5.7050	-0.2006	-516878.	-7263.	0.00656	37243.	5.38E+09	471.6548	4598.	0.00
5.8680	-0.1880	-535698.	-6335.	0.00637	37863.	5.38E+09	477.1178	4965.	0.00
6.0310	-0.1757	-552526.	-5397.	0.00617	38417.	5.38E+09	481.3291	5358.	0.00
6.1940	-0.1638	-567342.	-4453.	0.00597	38905.	5.38E+09	484.2505	5782.	0.00
6.3570	-0.1523	-580129.	-3504.	0.00576	39326.	5.38E+09	485.8483	6238.	0.00
6.5200	-0.1413	-590877.	-2554.	0.00555	39679.	5.38E+09	486.0929	6730.	0.00
6.6830	-0.1306	-599583.	-1604.	0.00533	39966.	5.38E+09	484.9591	7261.	0.00
6.8460	-0.1204	-606246.	-657.8153	0.00511	40186.	5.38E+09	482.4256	7836.	0.00
7.0090	-0.1106	-610877.	281.9458	0.00489	40338.	5.38E+09	478.4754	8459.	0.00
7.1720	-0.1013	-613487.	1213.	0.00467	40424.	5.38E+09	473.0955	9136.	0.00
7.3350	-0.09238	-614097.	2131.	0.00445	40444.	5.38E+09	466.2763	9873.	0.00
7.4980	-0.08390	-612732.	3035.	0.00422	40399.	5.38E+09	458.0117	10678.	0.00
7.6610	-0.07586	-609426.	3922.	0.00400	40290.	5.38E+09	448.2981	11560.	0.00
7.8240	-0.06825	-604215.	4788.	0.00378	40119.	5.38E+09	437.1342	12528.	0.00
7.9870	-0.06107	-597144.	5630.	0.00356	39886.	5.38E+09	424.5197	13597.	0.00
8.1500	-0.05431	-588264.	6447.	0.00335	39593.	5.38E+09	410.4542	14782.	0.00
8.3130	-0.04798	-577631.	7235.	0.00313	39243.	5.38E+09	394.9353	16101.	0.00
8.4760	-0.04205	-565308.	7990.	0.00293	38838.	5.38E+09	377.9567	17580.	0.00
8.6390	-0.03653	-551364.	8712.	0.00272	38379.	5.38E+09	359.5044	19249.	0.00
8.8020	-0.03140	-535873.	9395.	0.00253	37869.	5.38E+09	339.5526	21152.	0.00
8.9650	-0.02665	-518917.	10038.	0.00233	37310.	5.38E+09	318.0564	23344.	0.00
9.1280	-0.02227	-500583.	10638.	0.00215	36707.	5.38E+09	294.9413	25905.	0.00
9.2910	-0.01825	-480966.	11175.	0.00197	36061.	5.38E+09	253.8670	27216.	0.00
9.4540	-0.01456	-460228.	11625.	0.00180	35378.	5.38E+09	206.1829	27694.	0.00
9.6170	-0.01121	-438559.	11984.	0.00164	34664.	5.38E+09	161.4152	28171.	0.00
9.7800	-0.00816	-416136.	12259.	0.00148	33926.	5.38E+09	119.5779	28649.	0.00
9.9430	-0.00542	-393126.	12424.	0.00133	33169.	5.38E+09	48.7979	17620.	0.00
10.1060	-0.00295	-369808.	12498.	0.00119	32401.	5.38E+09	27.0035	17909.	0.00
10.2690	-7.45E-04	-346272.	12531.	0.00106	31626.	5.38E+09	6.9273	18198.	0.00
10.4320	0.00121	-322602.	12527.	9.43E-04	30847.	5.38E+09	-11.4725	18487.	0.00
10.5950	0.00294	-298876.	12488.	8.30E-04	30065.	5.38E+09	-28.2483	18776.	0.00
10.7580	0.00446	-275165.	12213.	7.25E-04	29285.	5.38E+09	-252.8306	110902.	0.00
10.9210	0.00578	-252337.	11702.	6.29E-04	28533.	5.38E+09	-269.6887	91266.	0.00
11.0840	0.00692	-230461.	11162.	5.42E-04	27813.	5.38E+09	-282.0674	79716.	0.00
11.2470	0.00790	-209594.	10601.	4.62E-04	27126.	5.38E+09	-291.5059	72189.	0.00
11.4100	0.00873	-189777.	10024.	3.89E-04	26473.	5.38E+09	-298.8427	66982.	0.00
11.5730	0.00942	-171044.	9434.	3.23E-04	25857.	5.38E+09	-304.5930	63246.	0.00
11.7360	0.00999	-153424.	8833.	2.64E-04	25276.	5.38E+09	-309.1002	60510.	0.00
11.8990	0.01045	-136938.	8225.	2.12E-04	24734.	5.38E+09	-312.6074	58489.	0.00
12.0620	0.01082	-121607.	7611.	1.65E-04	24229.	5.38E+09	-315.2944	57001.	0.00
12.2250	0.01110	-107444.	6993.	1.23E-04	23763.	5.38E+09	-317.2990	55923.	0.00
12.3880	0.01130	-94461.	6371.	8.62E-05	23335.	5.38E+09	-318.7295	55170.	0.00
12.5510	0.01144	-82669.	5746.	5.40E-05	22947.	5.38E+09	-319.6730	54680.	0.00
12.7140	0.01151	-72074.	5120.	2.59E-05	22598.	5.38E+09	-320.2006	54408.	0.00
12.8770	0.01154	-62681.	4494.	1.38E-06	22289.	5.38E+09	-320.3707	54319.	0.00

13.0400	0.01152	-54495.	3868.	-1.99E-05	22019.	5.38E+09	-320.2312	54387.	0.00
13.2030	0.01146	-47518.	3242.	-3.85E-05	21790.	5.38E+09	-319.8217	54594.	0.00
13.3660	0.01137	-41749.	2617.	-5.47E-05	21600.	5.38E+09	-319.1737	54925.	0.00
13.5290	0.01124	-37188.	2232.	-6.91E-05	21449.	5.38E+09	-73.9340	12861.	0.00
13.6920	0.01110	-32899.	2088.	-8.18E-05	21308.	5.38E+09	-73.8381	13016.	0.00
13.8550	0.01092	-28882.	1943.	-9.30E-05	21176.	5.38E+09	-73.5610	13171.	0.00
14.0180	0.01073	-25138.	1800.	-1.03E-04	21053.	5.38E+09	-73.1169	13326.	0.00
14.1810	0.01052	-21665.	1658.	-1.11E-04	20938.	5.38E+09	-72.5190	13481.	0.00
14.3440	0.01030	-18463.	1516.	-1.19E-04	20833.	5.38E+09	-71.7805	13636.	0.00
14.5070	0.01006	-15530.	1377.	-1.25E-04	20736.	5.38E+09	-70.9136	13791.	0.00
14.6700	0.00981	-12864.	1239.	-1.30E-04	20649.	5.38E+09	-69.9302	13946.	0.00
14.8330	0.00955	-10461.	1103.	-1.34E-04	20570.	5.38E+09	-68.8413	14100.	0.00
14.9960	0.00928	-8318.	969.9336	-1.38E-04	20499.	5.38E+09	-67.6571	14255.	0.00
15.1590	0.00901	-6432.	838.8383	-1.40E-04	20437.	5.38E+09	-66.3872	14410.	0.00
15.3220	0.00873	-4798.	710.3024	-1.42E-04	20383.	5.38E+09	-65.0401	14565.	0.00
15.4850	0.00845	-3410.	584.4695	-1.44E-04	20337.	5.38E+09	-63.6234	14720.	0.00
15.6480	0.00817	-2266.	461.4691	-1.45E-04	20300.	5.38E+09	-62.1439	14875.	0.00
15.8110	0.00789	-1358.	341.4184	-1.46E-04	20270.	5.38E+09	-60.6073	15030.	0.00
15.9740	0.00760	-681.7940	224.4248	-1.46E-04	20248.	5.38E+09	-59.0181	15185.	0.00
16.1370	0.00732	-231.1669	110.5875	-1.46E-04	20233.	5.38E+09	-57.3799	15340.	0.00
16.3000	0.00703	0.00	0.00	-1.46E-04	20225.	5.38E+09	-55.6952	7748.	0.00

* This analysis computed pile response using nonlinear moment-curvature relationships. Values of total stress due to combined axial and bending stresses are computed only for elastic sections only and do not equal the actual stresses in concrete and steel. Stresses in concrete and steel may be interpolated from the output for nonlinear bending properties relative to the magnitude of bending moment developed in the pile.

Output Summary for Load Case No. 2:

Pile-head deflection = -0.70000000 inches
Computed slope at pile head = 0.00459775 radians
Maximum bending moment = 1045449. inch-lbs
Maximum shear force = -26609. lbs
Depth of maximum bending moment = 0.000000 feet below pile head
Depth of maximum shear force = 0.000000 feet below pile head
Number of iterations = 14
Number of zero deflection points = 1

Summary of Pile-head Responses for Conventional Analyses

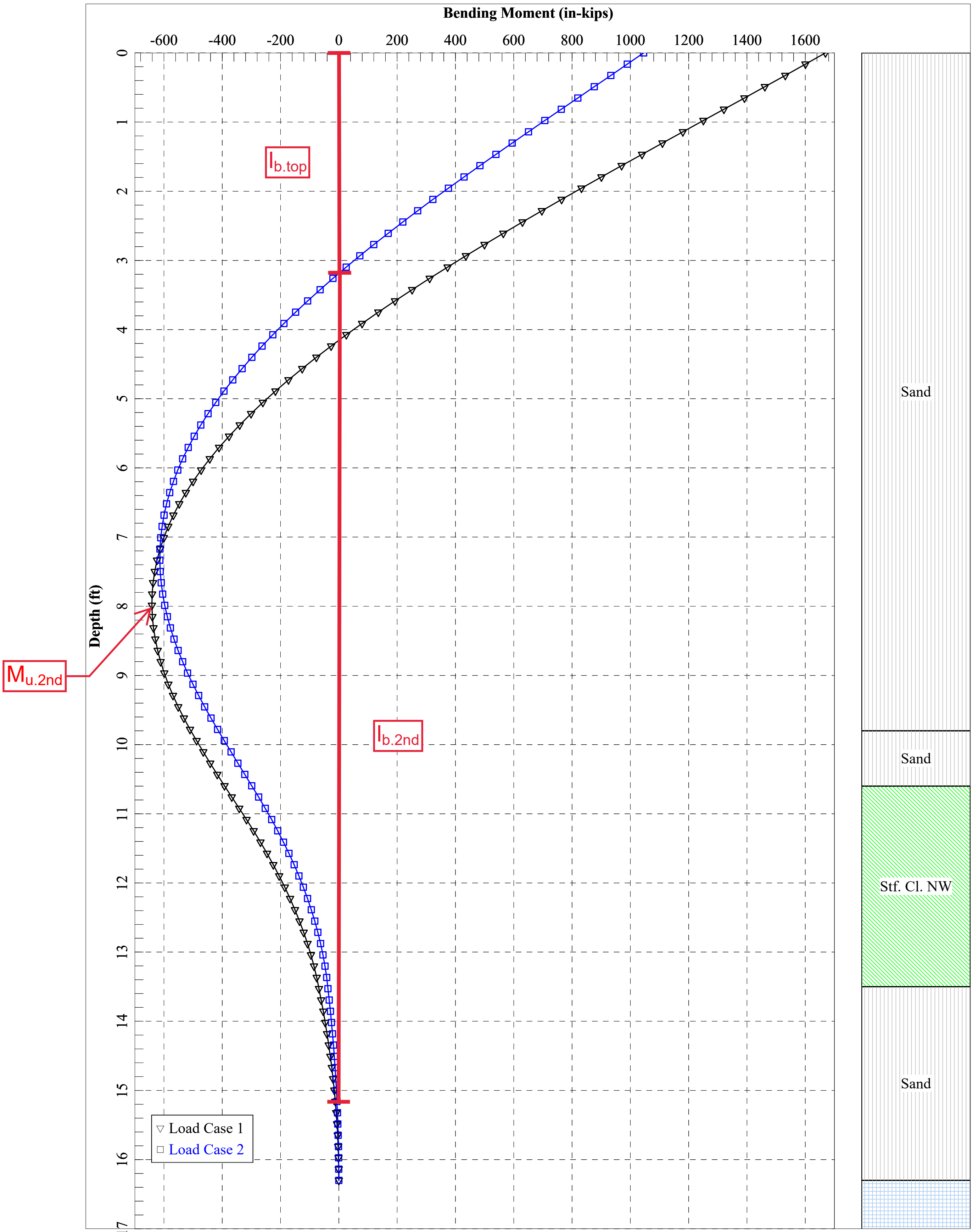
Definitions of Pile-head Loading Conditions:

Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians
Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
Load Type 5: Load 1 = Top Deflection, y, inches, and Load 2 = Slope, S, radians

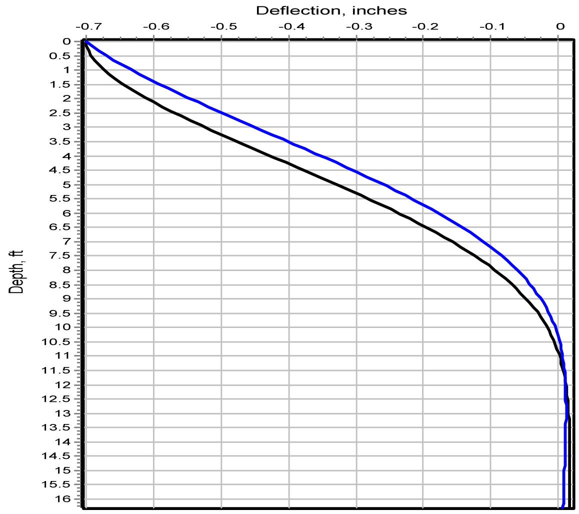
```
Load Load              Load               Axial    Pile-head  Pile-head  Max Shear Max Moment
Case Type   Pile-head    Type     Pile-head   Loading  Deflection  Rotation    in Pile    in Pile
 No.  1      Load 1       2        Load 2       lbs      inches     radians      lbs      in-lbs
---- ----- ---------- ---------- ---------- ---------- ---------- ---------- ---------- ----------
  1  y, in    -0.7000   S, rad         0.00    436000.    -0.7000       0.00    -34938.   1669287.
  2  y, in    -0.7000  M, in-lb   1045449.    436000.    -0.7000    0.00460    -26609.   1045449.
```

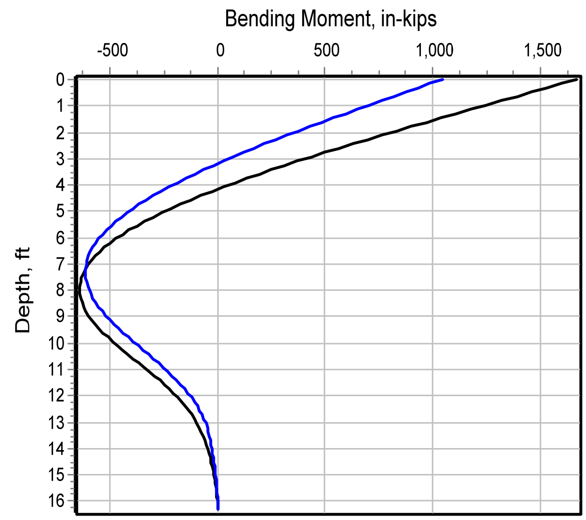
Maximum pile-head deflection = -0.7000000000 inches
Maximum pile-head rotation   = 0.0045977490 radians = 0.263432 deg.

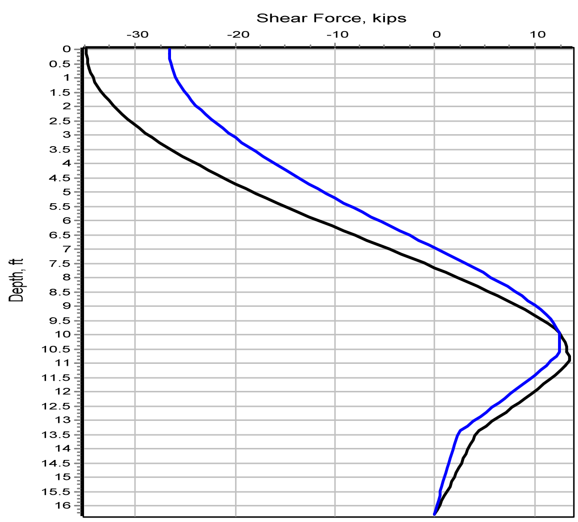
The analysis ended normally.

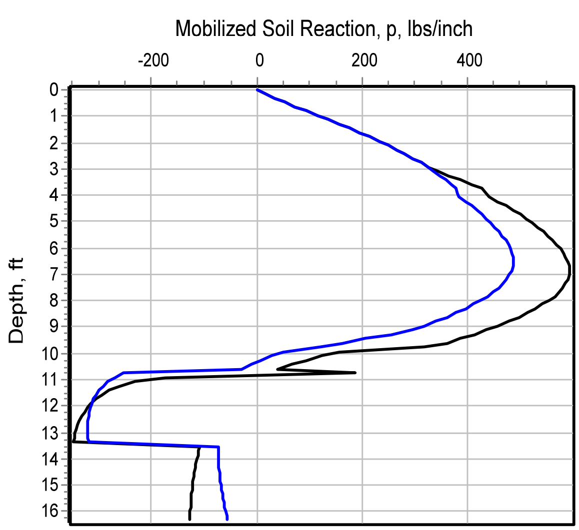


Bending Moment (in-kips)
-600
-400
-200
0
200
400
600
800
1000
1200
1400
1600
Depth (ft)
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
$I_{b.top}$
$I_{b.2nd}$
$M_{u.2nd}$
Load Case 1
Load Case 2
Sand
Sand
Stf. Cl. NW
Sand


Deflection, inches
Depth, ft


Bending Moment, in-kips
Depth, ft


Shear Force, kips
Depth, ft


Mobilized Soil Reaction, p, lbs/inch
Depth, ft

Attachment 3

==

LPile for Windows, Version 2019-11.005

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

==

This copy of LPile is being used by:

KAR
Golder Associates, Inc.

Serial Number of Security Device: 239146533

This copy of LPile is licensed for exclusive use by:

GOLDER, Manchester, NH

Files Used for Analysis

Path to file locations:
\Users\kroth\Documents\Projects\19126013 MaineDOT I-295 Freeport Exit 20 Merrill Rd Bridge\Pile Design\LPile Southeast Abutment\

Name of input data file:
Freeport Exit 20 Southeast Abutment Southern Shift _ No Socket _ HP12x74 _ Service1.lp11d

Name of output report file:
Freeport Exit 20 Southeast Abutment Southern Shift _ No Socket _ HP12x74 _ Service1.lp11o

Name of plot output file:
Freeport Exit 20 Southeast Abutment Southern Shift _ No Socket _ HP12x74 _ Service1.lp11p

Name of runtime message file:
Freeport Exit 20 Southeast Abutment Southern Shift _ No Socket _ HP12x74 _ Service1.lp11r

----
Date and Time of Analysis

Date: August 26, 2020 Time: 16:28:10

Problem Title

Project Name: MaineDOT I-295 Exit 20 Merrill Road Bridge No. 5720
Job Number: 19126013
Client: MaineDOT
Engineer: KAR
Description: Southeast Abutment Pile Design - Service I (No Rock Socket)

Program Options and Settings

Computational Options:
 - Conventional Analysis
Engineering Units Used for Data Input and Computations:
 - US Customary System Units (pounds, feet, inches)

Analysis Control Options:
 - Maximum number of iterations allowed = 500
 - Deflection tolerance for convergence = 1.0000E-05 in
 - Maximum allowable deflection = 100.0000 in
 - Number of pile increments = 100

Loading Type and Number of Cycles of Loading:
 - Static loading specified

 - Use of p-y modification factors for p-y curves not selected
 - Analysis uses layering correction (Method of Georgiadis)
 - No distributed lateral loads are entered
 - Loading by lateral soil movements acting on pile not selected
 - Input of shear resistance at the pile tip not selected
 - Input of moment resistance at the pile tip not selected
 - Input of side resistance moment along pile not selected
 - Computation of pile-head foundation stiffness matrix not selected
 - Push-over analysis of pile not selected
 - Buckling analysis of pile not selected

Output Options:
 - Output files use decimal points to denote decimal symbols.
 - Values of pile-head deflection, bending moment, shear force, and
   soil reaction are printed for full length of pile.
 - Printing Increment (nodal spacing of output points) = 1
 - No p-y curves to be computed and reported for user-specified depths
 - Print using wide report formats

Pile Structural Properties and Geometry

Number of pile sections defined = 1
Total length of pile = 16.300 ft
Depth of ground surface below top of pile = 0.0000 ft

Pile diameters used for p-y curve computations are defined using 2 points.

p-y curves are computed using pile diameter values interpolated with depth over the length of the pile. A summary of values of pile diameter vs. depth follows.

Point No.	Depth Below Pile Head feet	Pile Diameter inches
1	0.000	12.2150
2	16.300	12.2150

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is a H weak axis steel pile
Length of section = 16.300000 ft
Pile width = 12.130000 in
Shear capacity of section = 0.0000 lbs

Ground Slope and Pile Batter Angles

Ground Slope Angle = 0.000 degrees
= 0.000 radians

Pile Batter Angle = 0.000 degrees
= 0.000 radians

Soil and Rock Layering Information

The soil profile is modelled using 5 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer = 0.0000 ft
Distance from top of pile to bottom of layer = 9.800000 ft

Effective unit weight at top of layer = 125.000000 pcf
Effective unit weight at bottom of layer = 125.000000 pcf
Friction angle at top of layer = 32.000000 deg.
Friction angle at bottom of layer = 32.000000 deg.
Subgrade k at top of layer = 124.800000 pci
Subgrade k at bottom of layer = 124.800000 pci

Layer 2 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer = 9.800000 ft
Distance from top of pile to bottom of layer = 10.600000 ft
Effective unit weight at top of layer = 62.600000 pcf
Effective unit weight at bottom of layer = 62.600000 pcf
Friction angle at top of layer = 32.000000 deg.
Friction angle at bottom of layer = 32.000000 deg.
Subgrade k at top of layer = 75.500000 pci
Subgrade k at bottom of layer = 75.500000 pci

Layer 3 is stiff clay without free water

Distance from top of pile to top of layer = 10.600000 ft
Distance from top of pile to bottom of layer = 13.500000 ft
Effective unit weight at top of layer = 62.600000 pcf
Effective unit weight at bottom of layer = 62.600000 pcf
Undrained cohesion at top of layer = 1600. psf
Undrained cohesion at bottom of layer = 1600. psf
Epsilon-50 at top of layer = 0.005000
Epsilon-50 at bottom of layer = 0.005000

Layer 4 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer = 13.500000 ft
Distance from top of pile to bottom of layer = 16.300000 ft
Effective unit weight at top of layer = 62.600000 pcf
Effective unit weight at bottom of layer = 62.600000 pcf
Friction angle at top of layer = 37.000000 deg.
Friction angle at bottom of layer = 37.000000 deg.
Subgrade k at top of layer = 40.500000 pci
Subgrade k at bottom of layer = 40.500000 pci

Layer 5 is strong rock (vuggy limestone)

Distance from top of pile to top of layer = 16.300000 ft
Distance from top of pile to bottom of layer = 50.000000 ft
Effective unit weight at top of layer = 101.600000 pcf
Effective unit weight at bottom of layer = 101.600000 pcf
Uniaxial compressive strength at top of layer = 12983. psi
Uniaxial compressive strength at bottom of layer = 12983. psi

(Depth of the lowest soil layer extends 33.700 ft below the pile tip)

Summary of Input Soil Properties

Layer Num.	Soil Type Name (p-y Curve Type)	Layer Depth ft	Effective Unit Wt. pcf	Undrained Cohesion psf	Angle of Friction deg.	Uniaxial qu psi	E50 or krm	kpy pci
1	Sand	0.00	125.0000	--	32.0000	--	--	124.8000
	(Reese, et al.)	9.8000	125.0000	--	32.0000	--	--	124.8000
2	Sand	9.8000	62.6000	--	32.0000	--	--	75.5000
	(Reese, et al.)	10.6000	62.6000	--	32.0000	--	--	75.5000
3	Stiff Clay	10.6000	62.6000	1600.	--	--	0.00500	--
	w/o Free Water	13.5000	62.6000	1600.	--	--	0.00500	--
4	Sand	13.5000	62.6000	--	37.0000	--	--	40.5000
	(Reese, et al.)	16.3000	62.6000	--	37.0000	--	--	40.5000
5	Strong Rock	16.3000	101.6000	--	--	12983.	--	--
	(Vuggy Limestone)	50.0000	101.6000	--	--	12983.	--	--

Static Loading Type

Static loading criteria were used when computing p-y curves for all analyses.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load No.	Load Type	Condition 1	Condition 2	Axial Thrust Force, lbs	Compute Top y vs. Pile Length	Run Analysis
1	5	y = -0.700000 in	S = 0.0000 in/in	330000.	N.A.	Yes

V = shear force applied normal to pile axis
M = bending moment applied to pile head
y = lateral deflection normal to pile axis
S = pile slope relative to original pile batter angle
R = rotational stiffness applied to pile head
Values of top y vs. pile lengths can be computed only for load types with
specified shear loading (Load Types 1, 2, and 3).
Thrust force is assumed to be acting axially for all pile batter angles.

Computations of Nominal Moment Capacity and Nonlinear Bending Stiffness

---

Axial thrust force values were determined from pile-head loading conditions

Number of Pile Sections Analyzed = 1

Pile Section No. 1:

Dimensions and Properties of Steel H Weak Axis:

Length of Section	=	16.300000 ft
Flange Width	=	12.215000 in
Section Depth	=	12.130000 in
Flange Thickness	=	0.610000 in
Web Thickness	=	0.610000 in
Yield Stress of Pipe	=	50.000000 ksi
Elastic Modulus	=	29000. ksi
Cross-sectional Area	=	21.557400 sq. in.
Moment of Inertia	=	185.499357 in^4
Elastic Bending Stiffness	=	5379481. kip-in^2
Plastic Modulus, Z	=	46.522801in^3
Plastic Moment Capacity = Fy Z	=	2326.in-kip

Axial Structural Capacities:

Nom. Axial Structural Capacity = Fy As	=	1077.870 kips
Nominal Axial Tensile Capacity	=	-1077.870 kips

Number of Axial Thrust Force Values Determined from Pile-head Loadings = 1

Number	Axial Thrust Force kips
1	330.000

Definition of Run Messages:

Y = part of pipe section has yielded.

Axial Thrust Force = 330.000 kips

Bending Curvature rad/in.	Bending Moment in-kip	Bending Stiffness kip-in2	Depth to N Axis in	Max Total Stress ksi	Run Msg

0.00000495	26.6440446	5379425.	112.6824721	16.1764509	
0.00000991	53.2880893	5379425.	59.3949860	17.0449332	
0.00001486	79.9321339	5379425.	41.6324907	17.9134158	
0.00001981	106.5761785	5379425.	32.7512430	18.7818979	
0.00002476	133.2202232	5379425.	27.4224944	19.6503804	
0.00002972	159.8642678	5379425.	23.8699953	20.5188627	
0.00003467	186.5083124	5379425.	21.3324960	21.3873452	
0.00003962	213.1523571	5379425.	19.4293715	22.2558274	
0.00004458	239.7964017	5379425.	17.9491636	23.1243099	
0.00004953	266.4404463	5379425.	16.7649972	23.9927922	
0.00005448	293.0844910	5379425.	15.7961338	24.8612746	
0.00005944	319.7285356	5379425.	14.9887477	25.7297569	
0.00006439	346.3725802	5379425.	14.3055748	26.5982394	
0.00006934	373.0166248	5379425.	13.7199980	27.4667217	
0.00007429	399.6606695	5379425.	13.2124981	28.3352041	
0.00007925	426.3047141	5379425.	12.7684358	29.2036864	
0.00008420	452.9487587	5379425.	12.3766160	30.0721689	
0.00008915	479.5928034	5379425.	12.0283318	30.9406512	
0.00009411	506.2368480	5379425.	11.7167091	31.8091336	
0.00009906	532.8808926	5379425.	11.4362486	32.6776159	
0.0001040	559.5249373	5379425.	11.1824987	33.5460983	
0.0001090	586.1689819	5379425.	10.9518169	34.4145807	
0.0001139	612.8130265	5379425.	10.7411944	35.2830631	
0.0001189	639.4570712	5379425.	10.5481238	36.1515454	
0.0001238	666.1011158	5379425.	10.3704989	37.0200278	
0.0001288	692.7451604	5379425.	10.2065374	37.8885102	
0.0001337	719.3892051	5379425.	10.0547212	38.7569926	
0.0001387	746.0332497	5379425.	9.9137490	39.6254749	
0.0001436	772.6772943	5379425.	9.7824990	40.4939573	
0.0001486	799.3213390	5379425.	9.6599991	41.3624397	
0.0001535	825.9653836	5379425.	9.5454023	42.2309221	
0.0001585	852.6094282	5379425.	9.4379679	43.0994044	
0.0001634	879.2534729	5379425.	9.3370446	43.9678868	
0.0001684	905.8975175	5379425.	9.2420580	44.8363692	
0.0001734	932.5415621	5379425.	9.1524992	45.7048516	
0.0001783	959.1856068	5379425.	9.0679159	46.5733339	
0.0001833	985.8296514	5379425.	8.9879047	47.4418163	
0.0001882	1012.	5379425.	8.9121045	48.3102987	
0.0001932	1039.	5379425.	8.8401916	49.1787811	
0.0002031	1091.	5374439.	8.7082133	50.0000000	Y
0.0002130	1140.	5353431.	8.5929911	50.0000000	Y
0.0002229	1186.	5320248.	8.4920061	50.0000000	Y
0.0002328	1229.	5278009.	8.4031523	50.0000000	Y
0.0002427	1269.	5229256.	8.3246512	50.0000000	Y
0.0002526	1307.	5176060.	8.2549861	50.0000000	Y
0.0002625	1344.	5119592.	8.1930352	50.0000000	Y
0.0002724	1379.	5061014.	8.1377729	50.0000000	Y
0.0002823	1412.	5001332.	8.0882862	50.0000000	Y
0.0002922	1444.	4941296.	8.0438131	50.0000000	Y
0.0003021	1475.	4880835.	8.0039596	50.0000000	Y
0.0003120	1504.	4820882.	7.9679835	50.0000000	Y
0.0003219	1533.	4761632.	7.9354673	50.0000000	Y
0.0003318	1561.	4703248.	7.9060417	50.0000000	Y
0.0003418	1588.	4645866.	7.8793779	50.0000000	Y

0.0003517	1614.	4589602.	7.8551828	50.0000000	Y
0.0003616	1640.	4534551.	7.8331929	50.0000000	Y
0.0003715	1664.	4480791.	7.8131711	50.0000000	Y
0.0003814	1689.	4428386.	7.7949030	50.0000000	Y
0.0003913	1713.	4377386.	7.7781938	50.0000000	Y
0.0004012	1735.	4325545.	7.7621955	50.0000000	Y
0.0004111	1756.	4272511.	7.7465435	50.0000000	Y
0.0004210	1776.	4218940.	7.7310853	50.0000000	Y
0.0004309	1795.	4165058.	7.7158778	50.0000000	Y
0.0004408	1812.	4110858.	7.7011201	50.0000000	Y
0.0004507	1829.	4057106.	7.6864867	50.0000000	Y
0.0004606	1844.	4003506.	7.6723279	50.0000000	Y
0.0004705	1859.	3950183.	7.6581701	50.0000000	Y
0.0004804	1872.	3897329.	7.6443387	50.0000000	Y
0.0004903	1886.	3845366.	7.6307789	50.0000000	Y
0.0005002	1898.	3794044.	7.6175679	50.0000000	Y
0.0005102	1910.	3743341.	7.6042778	50.0000000	Y
0.0005201	1921.	3693475.	7.5915616	50.0000000	Y
0.0005300	1932.	3644611.	7.5788184	50.0000000	Y
0.0005399	1942.	3596288.	7.5664029	50.0000000	Y
0.0005498	1951.	3549235.	7.5543248	50.0000000	Y
0.0005597	1960.	3502741.	7.5420272	50.0000000	Y
0.0005696	1969.	3457287.	7.5304541	50.0000000	Y
0.0005795	1978.	3412506.	7.5186928	50.0000000	Y
0.0005894	1986.	3369041.	7.5072201	50.0000000	Y
0.0006290	2015.	3202701.	7.4633561	50.0000000	Y
0.0006686	2039.	3049466.	7.4219168	50.0000000	Y
0.0007083	2060.	2908808.	7.3830618	50.0000000	Y
0.0007479	2078.	2778991.	7.3464529	50.0000000	Y
0.0007875	2094.	2659506.	7.3119575	50.0000000	Y
0.0008271	2109.	2549298.	7.2791395	50.0000000	Y
0.0008668	2121.	2447138.	7.2483558	50.0000000	Y
0.0009064	2132.	2352539.	7.2187732	50.0000000	Y
0.0009460	2142.	2264490.	7.1909974	50.0000000	Y
0.0009856	2151.	2182632.	7.1643357	50.0000000	Y
0.0010253	2160.	2106297.	7.1391090	50.0000000	Y
0.0010649	2167.	2035011.	7.1152802	50.0000000	Y
0.0011045	2174.	1968060.	7.0920351	50.0000000	Y
0.0011441	2180.	1905399.	7.0699748	50.0000000	Y
0.0011838	2186.	1846475.	7.0491920	50.0000000	Y

Summary of Results for Nominal Moment Capacity for Section 1

Load No.	Axial Thrust kips	Nominal Moment Capacity in-kips
1	330.0000000000	2186.

Note that the values in the above table are not factored by a strength reduction factor for LRFD.

The value of the strength reduction factor depends on the provisions of the LRFD code being followed.

The above values should be multiplied by the appropriate strength reduction factor to compute ultimate moment capacity according to the LRFD structural design standard being followed.

Layering Correction Equivalent Depths of Soil & Rock Layers

Layer No.	Top of Layer Below Pile Head ft	Equivalent Top Depth Below Grnd Surf ft	Same Layer Type As Layer Above	Layer is Rock or is Below Rock Layer	F0 Integral for Layer lbs	F1 Integral for Layer lbs
1	0.00	0.00	N.A.	No	0.00	97275.
2	9.8000	9.7999	Yes	No	97275.	23123.
3	10.6000	11.7450	No	No	120398.	42508.
4	13.5000	10.7310	No	No	162906.	145406.
5	16.3000	16.3000	No	Yes	N.A.	N.A.

Notes: The F0 integral of Layer n+1 equals the sum of the F0 and F1 integrals for Layer n. Layering correction equivalent depths are computed only for soil types with both shallow-depth and deep-depth expressions for peak lateral load transfer. These soil types are soft and stiff clays, non-liquefied sands, and cemented c-phi soil.

Computed Values of Pile Loading and Deflection
for Lateral Loading for Load Case Number 1

Pile-head conditions are Displacement and Pile-head Rotation (Loading Type 5)
Displacement of pile head = -0.700000 inches
Rotation of pile head = 0.000E+00 radians
Axial load on pile head = 330000.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness lb-in^2	Soil Res. p lb/inch	Soil Spr. Es*h lb/inch	Distrib. Lat. Load lb/inch
0.00	-0.7000	1727625.	-36296.	0.00	72189.	4.34E+09	0.00	0.00	0.00
0.1630	-0.6992	1656418.	-36261.	7.62E-04	69845.	4.34E+09	15.5610	43.5293	0.00

0.3260	-0.6970	1584789.	-36213.	0.00147	67487.	4.65E+09	33.1291	92.9681	0.00
0.4890	-0.6935	1512857.	-36130.	0.00211	65118.	4.80E+09	52.2795	147.4543	0.00
0.6520	-0.6888	1440727.	-36008.	0.00270	62743.	4.95E+09	72.4164	205.6526	0.00
0.8150	-0.6829	1368506.	-35846.	0.00325	60366.	5.08E+09	93.1329	266.7475	0.00
0.9780	-0.6760	1296302.	-35643.	0.00376	57988.	5.19E+09	113.9965	329.8243	0.00
1.1410	-0.6682	1224218.	-35400.	0.00423	55615.	5.28E+09	134.8805	394.8201	0.00
1.3040	-0.6595	1152358.	-35116.	0.00467	53249.	5.34E+09	155.2820	460.5465	0.00
1.4670	-0.6500	1080819.	-34793.	0.00507	50894.	5.38E+09	174.6928	525.7223	0.00
1.6300	-0.6397	1009695.	-34432.	0.00545	48552.	5.38E+09	194.3063	594.1728	0.00
1.7930	-0.6286	939078.	-34033.	0.00581	46227.	5.38E+09	213.6910	664.9138	0.00
1.9560	-0.6169	869057.	-33598.	0.00614	43921.	5.38E+09	231.7084	734.6457	0.00
2.1190	-0.6046	799719.	-33128.	0.00644	41638.	5.38E+09	248.8667	805.1182	0.00
2.2820	-0.5917	731146.	-32625.	0.00672	39381.	5.38E+09	265.1134	876.3523	0.00
2.4450	-0.5783	663415.	-32093.	0.00697	37151.	5.38E+09	279.4296	945.0831	0.00
2.6080	-0.5644	596598.	-31532.	0.00720	34951.	5.38E+09	293.9278	1019.	0.00
2.7710	-0.5501	530765.	-30941.	0.00741	32783.	5.38E+09	310.6247	1104.	0.00
2.9340	-0.5355	465996.	-30318.	0.00759	30651.	5.38E+09	325.8872	1190.	0.00
3.0970	-0.5205	402365.	-29665.	0.00775	28556.	5.38E+09	341.9924	1285.	0.00
3.2600	-0.5052	339947.	-28974.	0.00788	26501.	5.38E+09	364.2106	1410.	0.00
3.4230	-0.4896	278843.	-28240.	0.00799	24489.	5.38E+09	386.1713	1543.	0.00
3.5860	-0.4739	219152.	-27464.	0.00808	22523.	5.38E+09	407.7774	1683.	0.00
3.7490	-0.4580	160969.	-26646.	0.00815	20608.	5.38E+09	428.8951	1832.	0.00
3.9120	-0.4420	104389.	-25797.	0.00820	18745.	5.38E+09	438.2854	1940.	0.00
4.0750	-0.4259	49461.	-24933.	0.00823	16936.	5.38E+09	445.3419	2045.	0.00
4.2380	-0.4098	-3774.	-24046.	0.00824	15432.	5.38E+09	462.0964	2206.	0.00
4.4010	-0.3937	-55241.	-23127.	0.00823	17127.	5.38E+09	477.7321	2374.	0.00
4.5640	-0.3776	-104867.	-22178.	0.00820	18761.	5.38E+09	492.1576	2549.	0.00
4.7270	-0.3616	-152585.	-21203.	0.00815	20332.	5.38E+09	505.2882	2733.	0.00
4.8900	-0.3457	-198334.	-20203.	0.00809	21838.	5.38E+09	517.0462	2925.	0.00
5.0530	-0.3300	-242058.	-19181.	0.00801	23278.	5.38E+09	527.3613	3126.	0.00
5.2160	-0.3144	-283708.	-18139.	0.00791	24649.	5.38E+09	538.7494	3352.	0.00
5.3790	-0.2990	-323231.	-17074.	0.00780	25950.	5.38E+09	549.8068	3596.	0.00
5.5420	-0.2839	-360573.	-15989.	0.00768	27180.	5.38E+09	559.8384	3857.	0.00
5.7050	-0.2690	-395690.	-14885.	0.00754	28336.	5.38E+09	568.8449	4136.	0.00
5.8680	-0.2544	-428537.	-13764.	0.00739	29417.	5.38E+09	576.8355	4435.	0.00
6.0310	-0.2401	-459076.	-12629.	0.00723	30423.	5.38E+09	583.8272	4756.	0.00
6.1940	-0.2261	-487274.	-11482.	0.00706	31351.	5.38E+09	589.8447	5102.	0.00
6.3570	-0.2125	-513101.	-10323.	0.00687	32202.	5.38E+09	594.9205	5476.	0.00
6.5200	-0.1992	-536532.	-9155.	0.00668	32973.	5.38E+09	599.0606	5881.	0.00
6.6830	-0.1863	-557544.	-7981.	0.00648	33665.	5.38E+09	601.8469	6317.	0.00
6.8460	-0.1739	-576123.	-6802.	0.00628	34277.	5.38E+09	603.1153	6785.	0.00
7.0090	-0.1618	-592260.	-5623.	0.00607	34808.	5.38E+09	602.8363	7288.	0.00
7.1720	-0.1501	-605951.	-4445.	0.00585	35259.	5.38E+09	600.9849	7830.	0.00
7.3350	-0.1389	-617200.	-3273.	0.00563	35629.	5.38E+09	597.5405	8414.	0.00
7.4980	-0.1281	-626019.	-2109.	0.00540	35919.	5.38E+09	592.4872	9045.	0.00
7.6610	-0.1178	-632423.	-957.0255	0.00517	36130.	5.38E+09	585.8130	9729.	0.00
7.8240	-0.1079	-636438.	180.7044	0.00494	36262.	5.38E+09	577.5099	10470.	0.00
7.9870	-0.09845	-638094.	1301.	0.00471	36317.	5.38E+09	567.5731	11276.	0.00
8.1500	-0.08947	-637429.	2399.	0.00448	36295.	5.38E+09	556.0005	12155.	0.00
8.3130	-0.08094	-634487.	3474.	0.00425	36198.	5.38E+09	542.7921	13117.	0.00
8.4760	-0.07286	-629319.	4521.	0.00402	36028.	5.38E+09	527.9485	14173.	0.00
8.6390	-0.06523	-621984.	5538.	0.00379	35787.	5.38E+09	511.4699	15337.	0.00
8.8020	-0.05804	-612546.	6521.	0.00356	35476.	5.38E+09	493.3540	16626.	0.00
8.9650	-0.05129	-601077.	7466.	0.00334	35098.	5.38E+09	473.5934	18061.	0.00

9.1280	-0.04496	-587654.	8372.	0.00313	34656.	5.38E+09	452.1723	19670.	0.00
9.2910	-0.03906	-572364.	9233.	0.00292	34153.	5.38E+09	429.0616	21488.	0.00
9.4540	-0.03356	-555298.	10048.	0.00271	33591.	5.38E+09	404.2120	23562.	0.00
9.6170	-0.02845	-536555.	10813.	0.00251	32974.	5.38E+09	377.5438	25958.	0.00
9.7800	-0.02373	-516242.	11522.	0.00232	32305.	5.38E+09	347.4924	28649.	0.00
9.9430	-0.01937	-494477.	12033.	0.00214	31588.	5.38E+09	174.4765	17620.	0.00
10.1060	-0.01536	-471930.	12341.	0.00196	30846.	5.38E+09	140.6644	17909.	0.00
10.2690	-0.01169	-448733.	12585.	0.00179	30082.	5.38E+09	108.7921	18198.	0.00
10.4320	-0.00834	-425015.	12768.	0.00164	29301.	5.38E+09	78.8523	18487.	0.00
10.5950	-0.00529	-400896.	12895.	0.00149	28507.	5.38E+09	50.8245	18776.	0.00
10.7580	-0.00253	-376488.	13159.	0.00134	27704.	5.38E+09	218.7543	169013.	0.00
10.9210	-3.63E-05	-351154.	13390.	0.00121	26870.	5.38E+09	18.1733	978000.	0.00
11.0840	0.00221	-325669.	13201.	0.00109	26030.	5.38E+09	-212.3229	187987.	0.00
11.2470	0.00422	-300919.	12749.	9.75E-04	25216.	5.38E+09	-249.3922	115509.	0.00
11.4100	0.00602	-277053.	12239.	8.70E-04	24430.	5.38E+09	-272.4442	88476.	0.00
11.5730	0.00763	-254165.	11690.	7.73E-04	23676.	5.38E+09	-288.9503	74114.	0.00
11.7360	0.00905	-232322.	11112.	6.85E-04	22957.	5.38E+09	-301.5405	65187.	0.00
11.8990	0.01030	-211579.	10513.	6.04E-04	22274.	5.38E+09	-311.4873	59124.	0.00
12.0620	0.01141	-191977.	9895.	5.31E-04	21629.	5.38E+09	-319.5169	54768.	0.00
12.2250	0.01238	-173553.	9264.	4.64E-04	21022.	5.38E+09	-326.0892	51516.	0.00
12.3880	0.01323	-156336.	8621.	4.04E-04	20455.	5.38E+09	-331.5167	49022.	0.00
12.5510	0.01396	-140350.	7968.	3.50E-04	19929.	5.38E+09	-336.0239	47072.	0.00
12.7140	0.01460	-125617.	7307.	3.02E-04	19444.	5.38E+09	-339.7778	45526.	0.00
12.8770	0.01514	-112155.	6639.	2.59E-04	19001.	5.38E+09	-342.9071	44289.	0.00
13.0400	0.01561	-99978.	5966.	2.20E-04	18600.	5.38E+09	-345.5127	43292.	0.00
13.2030	0.01601	-89100.	5288.	1.86E-04	18242.	5.38E+09	-347.6754	42488.	0.00
13.3660	0.01634	-79531.	4606.	1.55E-04	17926.	5.38E+09	-349.4602	41838.	0.00
13.5290	0.01661	-71280.	4158.	1.28E-04	17655.	5.38E+09	-109.2324	12861.	0.00
13.6920	0.01684	-63430.	3941.	1.03E-04	17396.	5.38E+09	-112.0429	13016.	0.00
13.8550	0.01702	-55995.	3720.	8.16E-05	17152.	5.38E+09	-114.5852	13171.	0.00
14.0180	0.01716	-48984.	3493.	6.25E-05	16921.	5.38E+09	-116.8846	13326.	0.00
14.1810	0.01726	-42409.	3263.	4.59E-05	16704.	5.38E+09	-118.9661	13481.	0.00
14.3440	0.01734	-36280.	3028.	3.16E-05	16502.	5.38E+09	-120.8538	13636.	0.00
14.5070	0.01739	-30604.	2790.	1.94E-05	16316.	5.38E+09	-122.5715	13791.	0.00
14.6700	0.01741	-25390.	2549.	9.23E-06	16144.	5.38E+09	-124.1418	13946.	0.00
14.8330	0.01742	-20645.	2305.	8.58E-07	15988.	5.38E+09	-125.5862	14100.	0.00
14.9960	0.01742	-16375.	2058.	-5.87E-06	15847.	5.38E+09	-126.9250	14255.	0.00
15.1590	0.01740	-12588.	1808.	-1.11E-05	15722.	5.38E+09	-128.1770	14410.	0.00
15.3220	0.01737	-9288.	1556.	-1.51E-05	15614.	5.38E+09	-129.3597	14565.	0.00
15.4850	0.01734	-6480.	1302.	-1.80E-05	15521.	5.38E+09	-130.4885	14720.	0.00
15.6480	0.01730	-4171.	1046.	-1.99E-05	15445.	5.38E+09	-131.5771	14875.	0.00
15.8110	0.01726	-2363.	787.4131	-2.11E-05	15386.	5.38E+09	-132.6369	15030.	0.00
15.9740	0.01722	-1063.	526.9579	-2.17E-05	15343.	5.38E+09	-133.6773	15185.	0.00
16.1370	0.01718	-273.8514	264.4800	-2.20E-05	15317.	5.38E+09	-134.7050	15340.	0.00
16.3000	0.01713	0.00	0.00	-2.20E-05	15308.	5.38E+09	-135.7244	7748.	0.00

* This analysis computed pile response using nonlinear moment-curvature relationships. Values of total stress due to combined axial and bending stresses are computed only for elastic sections only and do not equal the actual stresses in concrete and steel. Stresses in concrete and steel may be interpolated from the output for nonlinear bending properties relative to the magnitude of bending moment developed in the pile.

Output Summary for Load Case No. 1:

Pile-head deflection = -0.70000000 inches
Computed slope at pile head = 0.000000 radians
Maximum bending moment = 1727625. inch-lbs
Maximum shear force = -36296. lbs
Depth of maximum bending moment = 0.000000 feet below pile head
Depth of maximum shear force = 0.000000 feet below pile head
Number of iterations = 12
Number of zero deflection points = 1

Summary of Pile-head Responses for Conventional Analyses

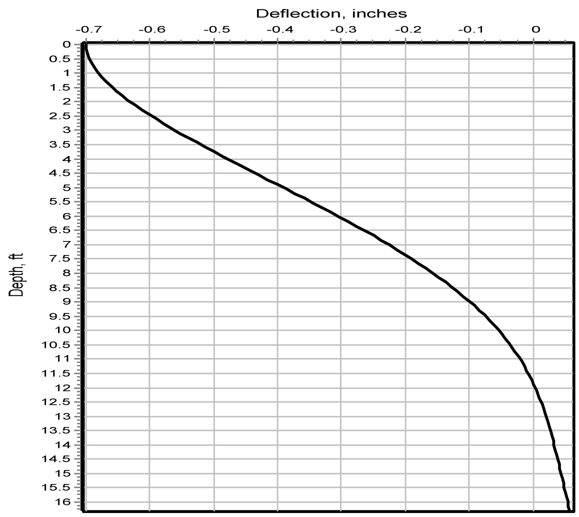
Definitions of Pile-head Loading Conditions:

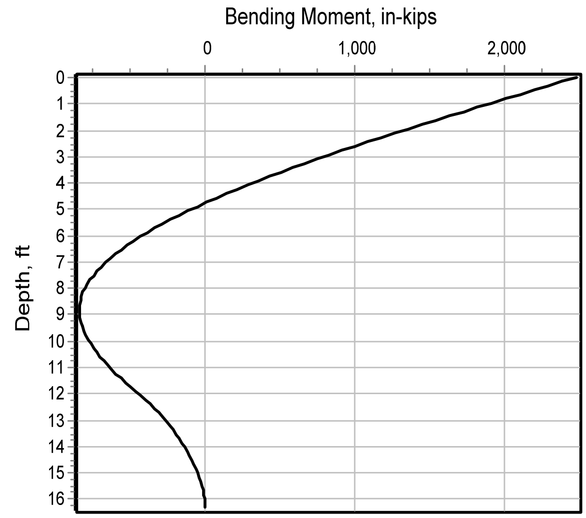
Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians
Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
Load Type 5: Load 1 = Top Deflection, y, inches, and Load 2 = Slope, S, radians

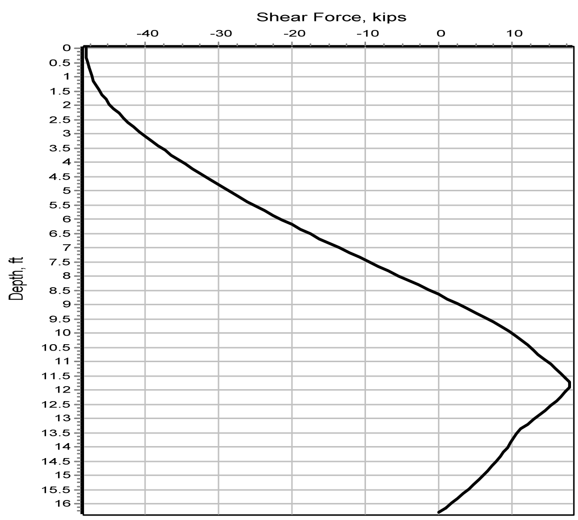
Load Case No.	Load Type 1	Pile-head Load 1	Load Type 2	Pile-head Load 2	Axial Loading lbs	Pile-head Deflection inches	Pile-head Rotation radians	Max Shear in Pile lbs	Max Moment in Pile in-lbs
1	y, in	-0.7000	S, rad	0.00	330000.	-0.7000	0.00	-36296.	1727625.

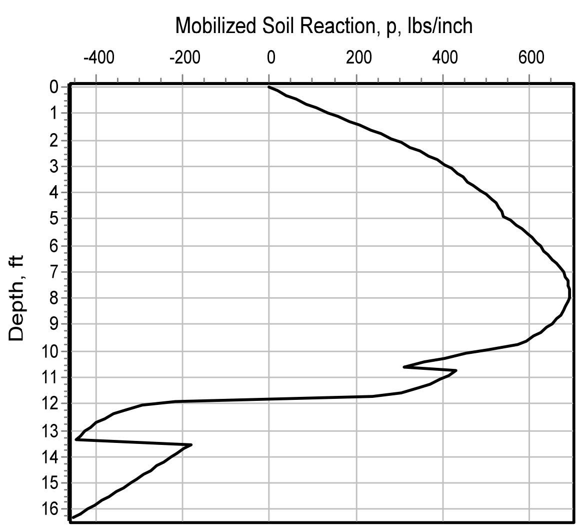
Maximum pile-head deflection = -0.7000000000 inches
Maximum pile-head rotation = 0.0000000000 radians = 0.000000 deg.

The analysis ended normally.


Deflection, inches
-0.7
-0.6
-0.5
-0.4
-0.3
-0.2
-0.1
0
Depth, ft


Bending Moment, in-kips
0
1,000
2,000
Depth, ft


Shear Force, kips
-40
-30
-20
-10
0
10
Depth, ft


Mobilized Soil Reaction, p, lbs/inch
-400
-200
0
200
400
600
Depth, ft

Piles for full integral and integral with hinge abutments shall be designed to resist all vertical superstructure dead and live loads, abutment and pile dead loads, live load girder rotation moments, lateral displacements, live load impact and moments caused by superimposed dead loads and live loads, as appropriate for the type of integral abutment.

Until the behavior of integral abutments with hinged connections to the superstructure is better understood, the pile design criteria for that type of integral abutment may assume that the moment at the top of the pile is zero, and that there is no moment from either the superstructure or earth loads.

The effect of thermal displacements and moments on piles can be investigated by running LPILE® software.

Secondary thermal forces only need be considered for multi-span structures only.

Appropriate load combinations and load factors should be determined per LRFD 3.4.1.

For the strength limit state analysis, design of the piles should consider the factored structural pile resistance, P_r, the factored structural flexural resistance, pile unbraced length, pile moments, the interaction of combined axial and flexural load effects, the structural shear resistance and the factored geotechnical resistance.

For service limit state evaluations, if piles will be driven to practical refusal in bedrock, settlement will not be a concern. However, all designs should consider horizontal movement, overall stability and scour for the design flood event.

B. Resistance Factors for Integral H-Piles

Pile will typically be end bearing on bedrock. For the strength limit state, use the following resistance factors:

- Use Φ_c = 0.50 for axial resistance in compression and subject to severe pile driving condition; this condition should be assumed when analyzing the lower portions of the pile
- Use Φ_c = 0.60 for axial resistance in compression under good driving conditions; this condition should be assumed when analyzing the upper portion of the pile
- For combined axial and flexural resistance in the upper zone of pile, use:

- Φ_c = 0.70 for axial resistance
- Φ_f = 1.00 for flexural resistance

C. Design Steps

The following steps should be followed during design of piles supporting full integral abutments, for the strength limit state:

1. Determine the foundation displacements, and the load effects (P_u and M_u) from the superstructure and substructure designs.

2. If applicable, determine the magnitude of scour.

3. Select preliminary pile size:

   a. Determine the factored applied superstructure vertical dead and live load (P_u) distributed to each pile

   b. Select the steel pile strength
   c. Select pile orientation; typically weak axis bending
   d. Determine resistance factors (Φ_c and Φ_f) for the structural strength in the upper and lower zones of the pile.
   e. Determine the maximum, required nominal axial pile resistance, P_u/Φ_f
   f. Estimate an initial pile area using the approximation

   $$A_s = \frac{Ru}{0.80 \times F_y}$$

   This approximation is based on weak axis bending and an assumed unbraced length of 15 feet based on typical integral abutment pile deflection and moment with depth curves. Select a pile size with an area A_s or greater.

4. Determine the pile unbraced length and maximum moment at the top of the pile by running LPILE® software for the design displacement from Step 1, P_u, and live load rotation

5. Determine if the applied moment on the pile will cause pile head plastic deformation by using the Interaction of combined axial and flexural load effects on a single pile (LRFD 6.9.2.2)

   a. Obtain the unbraced lengths of the top and lower segments of the pile and calculate the column slenderness factor (λ) for each segment. (LRFD 6.9.4.1)
   b. Determine K values for the top and bottom of the pile per LRFD Table C4.6.2.5-1

g. Calculate the nominal and factored structural pile resistance P_n, per LRFD 6.9.4.1 using the λ values
h. Compare the ratio of P_u to the structural resistance in the upper portion of the pile – the pile size should be such that the ratio is not less than 0.20.
i. Determine the nominal and factored flexural resistance about H-Pile weak axis, (LRFD 6.12.2.2)
j. Calculate the moment that will cause a plastic hinge at the top of the pile (M_p')
k. If the applied moment exceeds the moment that would cause a plastic hinge, a plastic hinge forms, and the moment that can be applied cannot exceed that moment (M_p')

6. For fixed head piles, run a second LPILE® analysis with displacement and plastic moment (M_p') as load conditions and P_u, and calculate new unbraced lengths from the moment with depth curve.

   a. Repeat steps 5.a. through 5.d., above
   b. If the pile size is such that the ratio of P_u to structural resistance exceeds 0.2, check the upper zone of the pile with the interaction equation of LRFD 6.9.2.2. If a plastic hinge forms at the top of the pile, the K value of the upper segment (that portion between the top of the pile and the first inflection point on the moment vs. depth curve) changes from 1.2, for a pinned condition, to 2.1, for a free condition at the top. With the new K value repeat Step 5, and check the interaction equation for pile overstress.

7. Because the piles have weak axis orientation and the flanges resist the shear as opposed to the web, check the maximum shear from the LPILE® output to the structural shear resistance per AISC G7.

8. Check that the maximum factored applied pile load does not exceed the factored geotechnical pile resistance or pile drivability resistance (LRFD 10.5.5.2.3 and 10.7.3.13) provided in the Geotechnical Design Report.

5.4.2.5 Pile Length Requirement

A. General Requirements

Piles may be end bearing or friction piles. In order to obtain the pile behavior associated with the equivalent length, piles should be installed 1 to 5 feet beyond the pile length required to achieve fixity. The practical

Reference 4

5.6.3 Steel H-Piles

Section	Weight lbs/ft kg/m	Area, A in^2 cm^2	Depth, d in mm	Flange Width, b in mm	Thickness Flange, t_f in. mm	Thickness Web, t_w in. mm	I_{xx} in^4 cm^4	I_{yy} in^4 cm^4	Compact Section Criteria F'_y ksi MPa
HP 14 HP 360	117	34.4	14.21	14.885	0.805	0.805	1220	443	49.4
	175	222	361	378	20.4	20.4	50800	18400	341
	102	30	14.01	14.785	0.705	0.705	1050	380	38.4
	153	194	356	376	17.9	17.9	43700	15800	265
	89	26.1	13.83	14.695	0.615	0.615	904	326	29.6
	133	168	351	373	15.6	15.6	37600	13600	204
	73	21.4	13.61	14.585	0.505	0.505	729	261	20.3
	109	138	346	370	12.8	12.8	30300	10900	140
HP 13 HP 330	100	29.4	13.15	13.205	0.765	0.765	886	294	56.7
	150	190	334	335	19.4	19.4	36878	12237	391
	87	25.5	12.95	13.105	0.665	0.665	755	250	43.5
	130	165	329	333	16.9	16.9	31425	10406	300
	73	21.6	12.75	13.005	0.565	0.565	630	207	31.9
	109	139	324	330	14.4	14.4	26223	8616	220
	60	17.5	12.54	12.9	0.46	0.46	503	165	21.5
	90	113	319	328	11.7	11.7	20936	6868	148

Section	Weight lbs/ft kg/m	Area, A in^2 cm^2	Depth, d in mm	Flange Width, b in mm	Thickness Flange, t_f in. mm	Thickness Web, t_w in. mm	I_{xx} in^4 cm^4	I_{yy} in^4 cm^4	Compact Section Criteria F'_y ksi MPa
HP 12 HP 310	84	24.6	12.28	12.295	0.685	0.685	650	213	52.5
	126	159	312	312	17.4	17.4	27100	8870	362
	74	21.8	12.13	12.215	0.61	0.61	569	186	42.1
	111	141	308	310	15.5	15.5	23700	7740	290
	63	18.4	11.94	12.125	0.515	0.515	472	153	30.5
	94	119	303	308	13.1	13.1	19600	6370	210
	53	15.5	11.78	12.045	0.435	0.435	393	127	22
	79	100	299	306	11	11	16400	5290	152
HP10 HP 250	57	16.8	9.99	10.225	0.565	0.565	294	101	51.6
	85	108	254	260	14.4	14.4	12200	4200	356
	42	12.4	9.7	10.075	0.42	0.42	210	71.7	29.4
	63	80	246	256	10.7	10.7	8740	2980	203
HP 8 HP 200	36	10.6	8.02	8.155	0.445	0.445	119	40.3	50.3
	54	68.4	204	207	11.3	11.3	4950	1680	347

Reference 7

Cohesionless Soil

Soil properties for preliminary design only.

Cohesionless Soil Properties	Symbol	Units	Loose		Medium		Dense		References :
Total Unit Weight	γ	pcf	90	115	110	130	110	140	Ref.[1]
Corrected SPT Blow Count	N_{60}		4	10	10	30	30	50	Ref.[2]
Relative Density	D_r	%	15	35	35	65	65	85	Ref.[3]
Angle of Internal Friction	ϕ	deg	29	30	30	36	36	41	Ref.[4]
Coefficient of Lateral Earth Pressure (From Eqn. (1) using ϕ)	K_0		0.51	0.5	0.5	0.41	0.41	0.34	$K_0 = 1-\sin(\phi)$ (1) Ref.[5]
Subgrade Modulus (Below Water Table)	k_{bw}	pci	20	30	30	100	100	160	Ref.[6]
Subgrade Modulus (Above Water Table)	k_{aw}	pci	20	50	50	165	165	275	Ref.[6]
Poisson's Ratio	υ		0.20 - 0.40		0.25 - 0.40		0.30 - 0.45		Ref.[7]
Young's Modulus (From Eqn. (2) using α =5, p_a =2000psf and N_{60})	E_{em}	psf	40000	100000	100000	300000	300000	500000	$E_{em} = p_a * \alpha * N_{60}$ (2) Ref.[8]
Young's Modulus (From Eqn. (2) using α =10, p_a =2000psf and N_{60})	E_{em}	psf	80000	200000	200000	600000	600000	1000000	
Young's Modulus (From Eqn. (3) using B = 24in, υ=the minimum of the range, and k_{bw})	E	psf	66360	99530	97200	324000	314500	503190	$E = k * B * (1-\upsilon^2)$ (3) Ref.[9]
Young's Modulus (From Eqn. (3) using B = 24in, υ=the minimum of the range, and k_{aw})	E	psf	66360	165890	162000	534600	518920	864860	

Notation:

E_{em} = Elastic Modulus based on empirical equation.

Cohesive Soil

Soil properties for preliminary design only.

Cohesive Soil Properties	Symbol	Units	Soft		Medium		Stiff		References :
Total Unit Weight	γ	pcf	100	120	110	130	120	140	Ref.[12]
Corrected SPT Blow Count	N_{60}		2	4	4	8	8	15	Ref. [13]
Unconfined Compressive Strength	q_u	tsf	0.25	0.5	0.5	1	1	2	Ref. [13]
Undrained Shear Strength	C_u	psf	250	500	500	1000	1000	2000	Ref. [14]
Average Undrained Shear Strength		psf	375		750		1500		
Major Principal Strain @ 50%	$\varepsilon 50$		0.02		0.01		0.005		Ref. [15]
Major Principal Strain @ 100%	$\varepsilon 100$		0.06		0.03		0.015		Ref. [16]
Subgrade Modulus (Static Loading)	k	pci	NA		NA		500		Ref. [17]
Subgrade Modulus (Cycling Loading)	k	pci	NA		NA		200		Ref. [17]
Poisson's Ratio	υ		0.4		0.45		0.5		Ref. [18]
Elastic Modulus	E	psi	415	1735	1735	4860	4860	>13890	Ref. [19]
Shear Modulus (From Eqn. (4) using E, and υ)	G	ksi	0.15	0.62	0.60	1.68	1.62	4.63	$G = E/(2(1+\upsilon))$ (4) Ref.[10]
Ultimate Unit End Bearing		ksi	See Fig.2 (For Driven Piles) on pp. 8						
Axial Bearing Failure		kips	Ultimate Unit End Bearing x Tip Area						
Ultimate Unit Skin Friction		psf	See Fig. 3 (For Driven Piles) on pp. 9						

Note: For the input values of vertical failure shear stress and torsional shear stress, the ultimate unit skin friction for a pile or drilled shaft can be used.

CALCULATIONS

Date:	8/18/2020	**Made by:**	MLM
Project No.:	19126013	**Checked by:**	KAR
Subject:	Pile Design at Center Pier - Southern Shift	**Reviewed by:**	CCB
Project Title:	MaineDOT I-295 Exit 20 Merrill Road Bridge Replacement No. 5720		

OBJECTIVE

Determine pile design requirements at the proposed bridge pier, assuming the "southern shift" option with the bike path scenario.

METHOD

Use the procedure outlined in AASHTO LRFD (Ref. 1) and the design method provided in the MaineDOT Bridge Design Guide (Ref. 2).

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 9th Ed. 2020.
2. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.
3. Email communication between Golder and HNTB, subject "RE: Freeport I-295 Exit 20 and Exit 22 - Request for Sections and Design Info", dated June 22, 2020.
4. Isenhower, W.M. et al. LPile v2019 Technical Manual: A Program for the Analysis of Deep Foundations Under Lateral Loading. Ensoft, Inc. Dated March 2020.
5. Golder interpreted subsurface section A-A' (Figure 3, Preliminary Geotechnical Design Report, dated September 2020).
6. Golder geotechnical test boring logs (Appendix A, Preliminary Geotechnical Design Report, dated September 2020).
7. Bridge Software Institute. FB-MultiPier Soil Parameter Table (US Customary Units). Accessed July 2020. https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf
8. VTrans Integral Abutment Committee. Integral Abutment Bridge Design Guidelines, 2nd Ed. 2008.
9. AISC Steel Construction Manual, 13th Ed.

ASSUMPTIONS

1. The selected pile orientation is strong axis bending, due to expected greater resistance needed at the pier.
2. The vertical load is assumed to be evenly distributed. All thermal movement will be taken up by the abutments.
3. The spacing between the two rows of piles at the pier is assumed to be >5B and thus reduction due to pile group interaction is not necessary (Ref. 1, Article 10.7.2.4).

CALCULATION

1. Select the preliminary pile size.

Determine the factored applied superstructure vertical dead and live load (P_u) distributed to each pile.

P_u = 450 kips (Ref. 3)

Select the steel pile strength.

F_y = 50 ksi
E = 29,000 ksi

CALCULATIONS

Date:	8/18/2020	**Made by:**	MLM
Project No.:	19126013	**Checked by:**	KAR
Subject:	Pile Design at Center Pier - Southern Shift	**Reviewed by:**	CCB
Project Title:	MaineDOT I-295 Exit 20 Merrill Road Bridge Replacement No. 5720		

Determine resistance factors (Φ_c and Φ_f) for the structural strength in the upper and lower zones of the pile.

ϕ_{cl} = 0.50 for axial resistance in the lower zone of the pile (Ref. 2, page 5-41)
ϕ_{cu} = 0.70 for axial resistance in the upper zone of the pile (Ref. 2, page 5-42)
ϕ_f = 1.00 for flexural resistance in the upper zone of the pile (Ref. 2, page 5-42)

Determine the maximum required nominal axial pile resistance (Ref. 1, Article 6.9.2.1).

$$R_{n.upper} = \frac{P_u}{\phi_{cu}}$$

$R_{n.upper}$ = 643 kips

$$R_{n.lower} = \frac{P_u}{\phi_{cl}}$$

$R_{n.lower}$ = 900 kips

$$R_n = \max\left(R_{n.upper}, R_{n.lower}\right)$$

R_n = 900 kips

Use the required nominal axial pile resistance to estimate the required pile area.

$$A_{s.req} = \frac{R_n}{0.80\ Fy}$$ (Ref. 2, page 5-42)

$A_{s.req}$ = 22.5 in^2

Select a pile size with an area of $A_{s.req}$ or greater.

Preferred selection is HP 14x89 based on June 16, 2020 meeting with MaineDOT and HNTB.
Check that preferred selection satisfies pile area requirement:

HP 14x89 A_s = 26.1 in^2 (Ref. 4, Table 5.6.3)
A_s > $A_{s.req}$ OK

2. Determine the nominal pile driving resistance.

While driving the pile, the maximum stress that is permitted in the pile is:

$$\sigma_{dr} = 0.9\Phi_{da}F_y$$ (Ref. 8, Appendix B, Eqn 7-22)

ϕ_{da} = 1.00 (Ref. 1, Article 6.5.4.2)
σ_{dr} = 45 ksi

Calculate the nominal pile driving resistance (R_{ndr}) from the applied load divided by the resistance factor associated with the pile monitoring method. The driving criteria will be established by dynamic testing.

ϕ_{mon} = 0.65 (Ref. 1, Table 10.5.5.2.3-1)

$$R_{ndr} = \frac{P_u}{\phi_{mon}}$$ (Ref. 8, Appendix B, Eqn 7-25)

R_{ndr} = 692 kips

CONCLUSIONS

For preliminary design purposes, a maximum factored load of 450 kips is recommended. Based on this loading, final pile spacing and footing dimensions should be established for the pier. At this time, only lateral thermal movement loads were provided which will be taken up by the abutments so a lateral analysis was not included. Once the loads are further refined, a lateral load analysis should be conducted to verify the lateral movements are acceptable and that combined stresses are within tolerable limits. A wave equation analysis will be completed in a separate calculation package to determine if the piles can be driven to the required nominal resistance without overstressing the piles.

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