GOLDER

Preliminary Geotechnical Design Report

I-295 MALLET DRIVE BRIDGE REPLACEMENT #5721 (EXIT 22)

FREEPORT, MAINE

MAINEDOT WIN 021726.00

Submitted to:

Ms. Laura Krusinski, PE

Senior Geotechnical Engineer
Maine Department of Transportation
Bridge Division
16 State House Station
Augusta, Maine 04333-0016

Submitted by:

Golder Associates Inc.

670 North Commercial Street, Suite 103

Manchester, NH 03101

+1 603 668-0880

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1.0 INTRODUCTION

This Preliminary Geotechnical Design Report (PGDR) summarizes the results of Golder Associates Inc.'s (Golder's) geotechnical investigation for the replacement of Mallet Drive Bridge #5721 over I-295 in Freeport, Maine at Exit 22 (see Sheet 1). The 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 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 and October 31, 2019.

2.0 PROJECT BACKGROUND

The existing Mallet Drive Bridge at I-295 Exit 22 was originally constructed in 1957 and is a two span, non-continuous steel girder structure with a composite deck, where each span is 70 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 existing pier and east abutment are supported by spread footings on bedrock, and the existing west abutment is supported on cast-in-place concrete piles between 10 and 19 feet in length. The base of the pile cap for the pier is at elevation 142.5 feet, the base of the west abutment is at elevation 145.0 feet, and the base of the east abutment is at elevation 143.5 feet. The bridge deck is 48.5 feet wide and the pavement surface is 40 feet wide, allowing for two travel lanes. I-295 passes under the bridge with 14.75 feet of vertical clearance (a 16 foot minimum clearance over I-295 is required for the replacement bridge) and has two travel lanes and one exit lane in both the northbound and southbound directions. Sheet 2 illustrates the existing bridge and site topography.

We understand that MaineDOT is considering replacement of the Mallet Drive Bridge (Exit 22) with an integral abutment bridge that will increase bridge clearance over I-295 to the 16 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 30 feet to allow for future widening of I-295 to accommodate a third travel lane and shoulder. We understand that MaineDOT has contracted with HNTB to design the bridge. 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 completed our design calculations 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 consist of Holocene (Recent) wetland/saltwater marsh deposits overlying Pleistocene Presumpscot Formation fine

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1 Hodgdon, Stephen (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.

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 Data

Historical drawings and reports provided by MaineDOT for the Mallet Drive Bridge do not indicate the native overburden material type. Construction records from near proposed Abutment No. 2 and the proposed center pier indicate bedrock is shallow and overburden soils are on the order of 10 feet thick or less. Pile installation records indicate overburden may extend from 10 feet and 19 feet below grade near proposed Abutment No. 1.

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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.

4.2 Geotechnical Borings

Golder completed six (6) test borings (BB-FMD-101 through BB-FMD-106) between December 15 and December 23, 2019 along Mallet Drive at the roadway and bridge elevation (4 borings) and south of the bridge abutment embankments (2 borings). The field program included Standard Penetration Test (SPT) 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 classification and subsequent laboratory testing. 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 truck-mounted rig (BB-FMD-101, -102, -103, and -105), a Mobile B-53 track-mounted rig (BB-FMD-104), and a SIMCO 2400 track-mounted rig (BB-FMD-106). 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 feet to 20 feet of rock core was collected.

Standard Penetration Testing (SPT) was performed using a calibrated automatic hammer system (for BB-FMD-101 through -105) or a safety hammer with rope and cathead (for BB-FMD-106) and a 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. Hammer efficiency factors (provided by NEBC) of 0.896 (for BB-FMD-101, -102, -103, and -105), 0.842 (for BB-FMD-104), and 0.600 (for BB-FMD-106) were 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 feet to 20 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 inAppendix 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 is also provided in Appendix A.

5.0 LABORATORY TESTING PROGRAM

Laboratory testing of soil and rock core samples 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-FMD-101, -102, -104, 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 also 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	6
Grainsize (sieve and hydrometer)	ASTM D7928 AASHTO T 88	6
Water Content	ASTM D2216 AASHTO T 265	7
Atterberg Limits with Natural Water Content	ASTM D4318 AASHTO T 89/90	7

Table 5-2: Laboratory Testing of Rock

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 on 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 thickness observed in borings BB-FMD-101 and BB-FMD-103 ranged from 5 inches to 7 inches.

Topsoil: Topsoil thickness observed in borings BB-FMD-104 and BB-FMD-106 was 2 feet.

Fill: Fill was encountered in borings BB-FMD-101, -102, -103, and -105. The layer was observed to be between 9.3 feet and 17.1 feet thick and start between elevation 142.5 feet and 165.6 feet. The fill consisted of fine to coarse SAND with "trace" gravel fractions and silt fractions ranging from “silty” to "trace". Laboratory classifications generally described the layer as SM, SW-SM, or SP-SM (USCS classification) and A-2-4 or A-1-b (AASHTO classification). N_{60}-values for the fill, corrected for hammer efficiency, ranged from 0 to refusal, 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 at 14.6 feet and 17.5 feet below ground surface (bgs) for borings BB-FMD-103 and -101, respectively, and to bedrock at 9.3 feet and 13.0 feet bgs for borings BB-FMD-105 and -102, respectively.

Presumpscot Formation: Presumpscot silty or sandy clay and silt was encountered in borings BB-FMD-101, -103, -104, and -106. It was not encountered in BB-FMD-102 and -105 because native soil was likely removed for placement of the pier foundations on bedrock. The layer was observed to be between 2.2 feet and 23.5 feet thick and start between elevation 127.2 feet and 151.6 feet. The Presumpscot silty or sandy clay and silt consists of: clay with up to "trace" gravel fractions, sand fractions ranging from “sandy” to “trace”, and silt fractions ranging from “silty” to “trace”. The primary layer has sublayers of fine to coarse sand with gravel fractions ranging from “gravelly” to “trace” and silt/clay fractions ranging from “some” to “trace”. Laboratory classifications generally described the layer as CL with sublayers of SM or SP-SM (USCS classification) and A-6, A-7-6, or A-4 with sublayers of A-2-4 or A-1-a (AASHTO classification). Field vane test (FVT) data yielded undrained shear strength (s_u) values between 1250 pounds per square foot (psf) and >4181 psf. Where field vane tests were not conducted, SPT N_{60}-values ranged from 6 to 90 (medium stiff to hard). The clay and silt layer transitions to sand and gravel at 4.2 feet, 14.0 feet and 41.0 feet bgs in BB-FMD-104, -106, and -101, respectively, and to bedrock at 19.0 feet bgs in BB-FMD-103.

Sand and Gravel: Sand and gravel (glacial till) was encountered in borings BB-FMD-101, -104, and -106 and was observed to be between 1.8 feet and 2.8 feet thick. The glacial till consists of fine to coarse sand with up to "trace" gravel fractions and silt fractions ranging from “silty” to “trace”. Laboratory classifications generally described the layer as SM (USCS classification) and A-2-4 or A-4 (AASHTO classification). SPT N_{60}-values ranged from 13 to refusal. The sand and gravel layer transitions to bedrock at 6.3 feet, 15.8 feet, and 43.8 feet for BB-FDR-104, -106, and -101, respectively.

Bedrock: The bedrock surface was encountered in all borings. The bedrock surface elevation varied approximately 35 feet across the site and generally slopes down from the northeast (El. 147.2 at BB-FMD-103) to southwest (El. 113.4 at BB-FMD-106). Rock core intervals of 10 feet to 20 were collected in each boring in up to 5-foot runs. The predominant bedrock lithology encountered was blue/gray, coarse-grained, strongly foliated, fresh to moderately weathered gneiss, interpreted to be part of the Vassalboro Formation. The RQD (rock quality designation) ranged from poor (33%) to excellent (100%), and the estimated RMR (rock mass rating) ranged from 58 to 80. The four unconfined compressive strength tests conducted on rock core samples yielded unconfined compression strength (UCS) values ranging from 358 ksf to 3279 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, and Table 5 provides detailed information on rock core engineering properties.

Groundwater: Groundwater level measurements in BB-FMD-102, -104, -105, -106 were measured upon completion of the boreholes and prior to removal of the casing. Groundwater elevations measured in the borings were between 126.1 feet and 140.1 feet. Groundwater levels shown on the subsurface cross-section (Sheet 3) were interpreted based on these water level meter 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 29+50 and 29+75) and preliminary proposed Abutment No. 2 (at proposed Stations 32+25 and 32+50). Golder evaluated each of these embankment cross-sections in conjunction with the interpreted subsurface cross-section (Sheet 3, parallel to the existing bridge alignment).

Our analysis found that the southern shift of the bridge alignment ("phasing south") 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 preliminary proposed Abutment No. 1 location. Thus, our preliminary analyses of settlement and stability are conducted using the embankment cross-section at Sta. 29+75 and corresponding subsurface conditions. The geometry used for lateral earth pressure and pile analyses was from preliminary proposed abutment locations provided by HNTB that are near to, but not exactly at, Station 29+75 and Station 32+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 estimated 10% water content, we estimate a frost penetration depth of 6.4 feet at the site. Refer to the depth of frost penetration calculations in Appendix E.

[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.

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 both the existing approach embankment and the toe of the slope nearest the proposed Abutment No. 1. Results concluded the existing embankment is Site Class C and the proposed embankment toe is 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	0.096	D	1

7.3 Approach Embankment Stability

Stability was evaluated for the proposed widest alignment shift to the south at Station 29+75 nearest proposed Abutment No. 1 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 Abutment No. 1 southern and northern approach embankment slopes and longitudinal to the face of Abutment No. 1. These analyses incorporated the 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 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.

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. These 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. The longitudinal abutment evaluation 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.

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[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

For the transverse embankment geometries, deep seated potential failure surfaces in the native glaciomarine soil yield a lower bound FS of 1.56 under static conditions and 1.33 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 1.26 under static conditions and 1.08 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.

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

<table>
<tr><th rowspan="2">Condition</th><th rowspan="2">Scenario</th><th rowspan="2">Slope</th><th rowspan="2">Assumed Extent of Clay Beneath Existing Embankment[2]</th><th colspan="2">Lowest Factor of Safety[1] (Spencer Method)</th></tr>
<tr><th>Non-Circular Failure Surface through Fill</th><th>Non-Circular Deep-Seated Failure Surface through Glaciomarine Clay</th></tr>
<tr><td rowspan="4">Static</td><td rowspan="4">Abutment No. 1 Embankment (Station 29+75)</td><td rowspan="2">North</td><td>Small</td><td rowspan="2">1.30</td><td>2.40</td></tr>
<tr><td>Large</td><td>2.40</td></tr>
<tr><td rowspan="2">South</td><td>Small</td><td rowspan="2">1.26</td><td>1.56</td></tr>
<tr><td>Large</td><td>1.93</td></tr>
<tr><td rowspan="4">Pseudo-Static Seismic</td><td rowspan="4">Abutment No. 1 Embankment (Station 29+75)</td><td rowspan="2">North</td><td>Small</td><td rowspan="2">1.12</td><td>1.84</td></tr>
<tr><td>Large</td><td>1.84</td></tr>
<tr><td rowspan="2">South</td><td>Small</td><td rowspan="2">1.08</td><td>1.33</td></tr>
<tr><td>Large</td><td>1.67</td></tr>
</table>

1. Minimum factors of safety are 1.3 for static analyses and 1.0 for pseudo-static seismic analyses.
2. The glaciomarine clay was split into stiff and very stiff layers based on interpreted cohesion, and two different zones of the very stiff, compressed glaciomarine clay underneath the existing embankment, termed the "small" and "large" zones, were analyzed.

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

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

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 Abutment (not considering piles).

Condition	Lowest Factor of Safety[1] (Spencer Method)	
	Non-Circular Failure Surface through Fill	Non-Circular Failure Surface through Glaciomarine Clay
Static	1.35	2.14
Pseudo-Static Seismic	1.21	1.88

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 2.14 under static conditions and 1.88 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 1.35 under static conditions and 1.21 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 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 Approach Embankment Settlement

Golder analyzed settlement for the proposed widest alignment shift to the south at Station 29+75 (near proposed Abutment No. 1) and Station 32+25 (near proposed Abutment No. 2) for a point directly below the guardrail along the edge of the proposed eastbound traffic. 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.

We evaluated settlement for the proposed widest alignment shift to the south at both Station 29+75 and Station 32+25 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 are tabulated for the two locations in Table 7-4:

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

Estimated Settlement Conditions	Proposed Southern Shift Station 29+75	Proposed Southern Shift Station 32+25
Total Settlement (inches)	1.9	1.2
Immediate Settlement (inches)	0.4	0.3
Consolidation Settlement (inches)	1.5	0.9
Time for 95% Consolidation Settlement (days)	585 (1.6 years)	55

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 81 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 at the proposed abutments.

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[14] 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 presented in Table 7-5:

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

Table 7-5: Lateral Earth Pressure Coefficients

Abutment Earth Pressure Parameter	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 elevation 155.0 feet for Abutment No. 1 and 159.0 feet for 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 are proposed to be founded on piles driven to the bedrock surface. The new bridge pier is proposed to be founded on a spread footing bearing on bedrock. To provide recommendations on pile design, we analyzed the pile design loads, downdrag, axial resistance, axial and lateral loads, and driveability. To provide recommendations on spread footing design, we analyzed bearing capacity. These analyses are 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.

Golder analyzed HP 14x89 piles at the request of MaineDOT and HNTB for an abutment width of 81 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 will be supported by piles approximately 34.0 feet in length, the pier will be supported by a spread footing on bedrock, and Abutment No. 2 will be supported on piles approximately 12.0 feet in length. Our analyses were performed assuming average interpreted depth to bedrock as shown on the subsurface profile for each abutment location. However, as discussed in Section 6.0, the borings show that bedrock elevation varies by approximately 34 feet across the site and generally slopes down from the northeast (El. 147.2 at BB-FMD-103) to southwest (El. 113.4 at BB-FMD-106). Over the footprint of the abutments, there may be differences in top of bedrock elevation of up to approximately 5 to 10 feet.

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 superstructure, and driveability analyses for the abutment piles.

7.6.2 Downdrag on Piles

In accordance with AASHTO Article 3.11.8, downdrag loads on piles can be assumed to be fully developed when settlements at the soils surrounding the piles is 0.4 inches or greater. Therefore, the settlement analysis at both Abutment No. 1 and No. 2 indicates downdrag loads will be imposed on the piles along the existing fill and glaciomarine clay soils 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.

Downdrag loads were calculated in accordance with methods described in AASHTO Article 10.7.3.7. The factored maximum downdrag loads at Abutment No. 1 for an HP14x89 pile are 354 kips and 255 kips per pile for the Strength I and Service I load cases, respectively. The factored maximum downdrag loads at Abutment No. 2 for an HP14x89 pile are 156 kips and 114 kips per pile for the Strength I and Service I load cases, respectively. The factored maximum downdrag loads at Abutment No. 2 for an HP12x74 pile are 132 kips and 97 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 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[15].

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 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 ϕ_c = 0.70 for combined axial and bending loading and ϕ_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 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 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

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

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 7.6.

The maximum applied factored axial pile loads should not exceed the governing factored axial pile resistances shown in Table 7.6.

Table 7-6: 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	358	358
Abutment No. 2	HP 14x89	653	653	423	423
Abutment No. 2	HP 12x74	545	545	325	325

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[16] 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[17], 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 the 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). Our analyses assume an applied axial load of 522 kips for both abutments using HP 14x89 piles, and an applied axial load of 436 kips for Abutment No. 2 using HP 12x74 piles. The pile head to abutment connection was assumed to be fixed.

For Abutment No.1, Golder analyzed a pile length of 34.0 feet. For the HP 14x89 piles, the analysis indicated a maximum moment of 207 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 12.0 feet using both an HP 14x89 pile and an HP 12x74 pile. For the HP 14x89 piles, the analysis indicates a maximum moment of 226 kip-feet 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

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

[17] 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>

movement occurring at the pile tip. For the HP 12x74 piles, the analysis indicates a maximum moment of 157 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, and < 0.1 inches of lateral movement occurs 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 7-7.

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 7-8 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 7-7: 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	2,489	1,436	Yes
Abutment No. 2 (HP 14x89)	522	0.7	2,708	1,438	Yes
Abutment No. 2 (HP 12x74)	436	0.7	1,878	1,052	Yes

Table 7-8: 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	1,436	0.59	0.93
Abutment No. 2 (HP 14x89)	522	0.7	1,438	0.59	0.92
Abutment No. 2 (HP 12x74)	436	0.7	1,052	0.60	0.97

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. 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 HP 12x 74 piles at Abutment No. 2 including estimated downdrag loads, pile lengths, and pile orientation are provided in Table 7-9.

Table 7-9: Pile Analysis Summary

Location	Recommended Nominal Pile Resistance (kips)	Recommended Max. Factored Axial Load Per Pile (kips) [1]	Estimated Factored Downdrag Load[2] (kips)	Estimated Pile Length (feet)	Pile Orientation
Abutment No. 1 (HP 14x89)	550	358	354	34.0 (driven to bedrock)	Weak-axis
Abutment No. 2 (HP 14x89)	650	423	156	12.0 (driven to bedrock)	Weak-axis
Abutment No. 2 (HP 12x74)	500	325	132	12.0 (driven to bedrock)	Weak-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.

7.7 Bridge Pier Foundation

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

- The pier substructure will be supported by a spread footing that is 9 feet to 10 feet wide by 60 feet long.
- The pier substructure sub-footing will be 2 feet to 4 feet larger in each dimension depending on excavation support.
- The maximum factored bearing pressure is assumed to be between 20 ksf and 28 ksf depending on the final foundation size.

Bedrock at the proposed pier location is between 10 feet and 13 feet bgs (133.2 feet and 130.1 feet elevation) at borings BB-FMD-105 and BB-FMD-102, respectively (see §6). Calculations and input material properties were based on AASHTO LRFD (2020), the MaineDOT Bridge Design Guide, Wyllie (1999)[18], rock core quality (Table 3), laboratory testing results (Appendix C), and empirical correlations to rock properties.

Golder calculated the bearing capacity for the smallest dimension footing specified by HNTB, specifically 62 feet long and 11 feet wide with no embedment into bedrock. The base of the pier footing will be located at least 11 feet below the ground surface, which is below the frost penetration depth (Section 7.1). A sloping bedrock surface of approximately 8° to the north was also used. The factored bearing resistance for the strength limit state is 68 ksf, using a bearing resistance factor of 0.45 as per AASHTO LRFD Table 10.5.5.2.2-1. The factored presumptive

[18] Wyllie, D.C. (1999). Foundations on Rock, 2nd Ed. E&FN Spon, NY.

bearing resistance for the service limit state (limiting settlement to 1 inch) is 70 ksf using a resistance factor of 1.0 as per AASHTO LRFD Section 10.5.5.1. These results are presented in Table 7-10. In no instance shall the factored bearing stress exceed the nominal resistance of the footing concrete (0.3 f'_c). No footing shall be less than 2 feet wide regardless of the applied bearing pressure or bearing material. These resistances exceed the maximum factored bearing pressure of 28 ksf given by HNTB. Refer to the full methodology of the analysis, material properties, and calculations in Appendix E.

Table 7-10: Summary of center pier factored bearing resistance.

Strength		Service	
Resistance Factor	Factored Resistance (ksf)	Resistance Factor	Factored Resistance (ksf)
0.45	68	1.0	70

Assuming the cast-in-place concrete footing or sub-footing bears on bedrock, a sliding coefficient (tan δ) of 0.70 is recommended per AASHTO LRFD Table C3.11.5.3-1 provided the bedrock subgrade is prepared under dry conditions and can be visually inspected and the slope of the bedrock is less steep than 4V:1H in any direction. A resistance factor of 0.80 is recommended for sliding per MaineDOT Bridge Design Guide Table 5-3.

8.0 CONSTRUCTION CONSIDERATIONS

All areas proposed for embankment fill placement and 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 maintain 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 bedrock at the pier location is estimated to be sloping upward to the northeast at an approximate slope of 8° based on an approximately 3-foot elevation difference between BB-FMD-102 and -105. The bedrock surface should be free of all loose soil and weathered bedrock material. The bearing surface should be consistently at the same elevation to mitigate the potential for out of plane sliding on bedrock. Bedrock slopes that exceed 4H:1V (14°) should be benched to create level steps or a completely level subgrade. For bedrock slopes between 4H:1

and 6H:1V (9.5°), dowels into bedrock should be considered as recommended in the MaineDOT Bridge Design Guide.

The estimated piles lengths provided in this report were based on an average interpreted depth to bedrock at the abutment locations. However, borings show that bedrock elevation varies at the site both parallel and transverse to the roadway alignment. Bedrock elevation differences of approximately 4 feet sloping downward to the south between BB-FMD-101 and -106 near the northwest abutment and approximately 9 feet sloping downward to the southwest between BB-FMD-103 and -104 at the southeast abutment 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 existing bridge and the borings located south of the bridge in the analyses. This stratigraphy showed sloping bedrock (from the northeast down to southwest near the abutment

[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.

locations and southwest to northeast near the pier location). Due to the potential for sloping bedrock at the abutment locations, we recommend that additional geotechnical investigations be performed during final design to better define the bedrock elevations at the proposed abutment locations. Our analysis assumed pile lengths ranging from 12 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 abutment 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:

- 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 downdrag 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.
    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 as 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 22 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

STATE OF MAINE
CHRISTOPHER
C. BENDA
No. 15164
LICENSED
PROFESSIONAL ENGINEER

Golder Associates Inc.

Melissa E Landon, PhD, PE
Senior Project Engineer

Christopher C. Benda, PE
Practice Leader

MEL/MCM/CCB/pba

https://golderassociates.sharepoint.com/sites/114599/project files/6 deliverables/final report/freeport mallet dr bridge final pgdr 2020dec16 v4.docx

Table 1: Subsurface Exploration Locations
Preliminary Geotechnical Design Report
I-295 Mallet Drive Bridge Replacement #5721 (Exit 22)
Freeport, Maine
MaineDOT WIN 21726.00

Test Boring Designation[1]	As-Drilled Locations[2,3]		Existing Ground Surface Elevation[3] (feet)	Boring Depth (feet)	Comments[4,5]
	Northing (feet)	Easting (feet)			
BB-FMD-101	376042.7	1054596.1	161.0	54.0	Bedrock at 117.2 ft elevation (43.8 ft bgs)
BB-FMD-102	375943.7	1054683.2	143.1	24.3	Bedrock at 130.1 ft elevation (13.0 ft bgs)
BB-FMD-103	375852.0	1054767.6	166.2	29.5	Bedrock at 147.2 ft elevation (19.0 ft bgs)
BB-FMD-104	375754.3	1054696.2	144.6	16.3	Bedrock at 138.3 ft elevation (6.3 ft bgs)
BB-FMD-105	375925.3	1054665.6	142.5	19.3	Bedrock at 133.2 ft elevation (9.3 ft bgs)
BB-FMD-106	375933.5	1054554.5	129.2	35.8	Bedrock at 113.4 ft elevation (15.8 ft bgs)

Notes:
1. Borings BB-FMD-101 through BB-FMD-106 were performed by New England Boring Contractors from December 15, 2019 to December 23, 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 21726.00 Boring Data 2020-01-06" received by Golder on January 6, 2019 from MaineDOT.
4. Boring logs presented in Appendix A.
5. bgs = Below ground surface

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

GOLDER

Table 2: Summary of In Situ Vane Shear Testing Results
Preliminary Geotechnical Design Report
I-295 Mallet Drive Bridge Replacement #5721 (Exit 22)
Freeport, Maine
MaineDOT WIN 21726.00

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		Remolded		S_t[5]	Comments
						Vane Reading Torque	s_u from Vane Reading (psf)	Vane Reading Torque	s_u from Vane Reading (psf)		
BB-FMD-101	161.0	V1	33.2 - 33.3	127.8 - 127.7	16 x 32	> 26.6 in-lbs	> 4181	- -	- -	- -	Reached maximum torque without failure
BB-FMD-101	161.0	V2	35.2 - 35.3	125.8 - 125.7	16 x 32	> 26.6 in-lbs	> 4181	- -	- -	- -	Reached maximum torque without failure
BB-FMD-101	161.0	V3	37.2 - 37.3	123.8 - 123.7	16 x 32	16 in-lbs	2515	4 in-lbs	629	4	
BB-FMD-101	161.0	MV	39.6 - 40.0	121.4 - 121.0	65 x 130	- -	- -	- -	- -	- -	Could not push down into borehole
BB-FMD-106	129.2	MV	6.6 - 7.0	122.6 - 122.2	55 x 110	- -	- -	- -	- -	- -	Could not push down into borehole
BB-FMD-106	129.2	MV	8.6 - 9.0	120.6 - 120.2	55 x 110	- -	- -	- -	- -	- -	Could not push down into borehole
BB-FMD-106	129.2	V3	10.6 - 11.0	118.6 - 118.2	55 x 110	42 ft-lbs	1875	6 ft-lbs	268	7	
BB-FMD-106	129.2	V4	11.6 - 12.0	117.6 - 117.2	55 x 110	28 ft-lbs	1250	5 ft-lbs	223	5.6	
BB-FMD-106	129.2	MV	12.6 - 13.0	116.6 - 116.2	55 x 110	- -	- -	- -	- -	- -	Could not push down into borehole
BB-FMD-106	129.2	MV	14.6 - 15.0	114.6 - 114.2	55 x 110	- -	- -	- -	- -	- -	Could not push down into borehole

Notes:
1. Test boring locations are shown in Sheet 2 entitled "Boring Location Plan".
2. As-drilled locations and elevations are derived from survey files "Freeport 21726.00 Boring Data 2020-01-06" received by Golder on January 6, 2019 from MaineDOT.
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.
8. MV = missed vane. in-lbs = inch-pounds. ft-lbs = foot-pounds.

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

Table 3: Summary of Rock Core Quality
Preliminary Geotechnical Design Report
I-295 Mallet Drive Bridge Replacement #5721 (Exit 22)
Freeport, Maine
MaineDOT WIN 21726.00

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	End	Midpoint	Run Length (ft)	TCR[2] Length (ft)	TCR[2] %	RQD[3] Length (ft)	RQD[3] %	RQD[3] Designation	Weathering[4]	Estimated Field Strength[4]	Rock Mass Rating [RMR][5]	Lithologic, Rock Mass and Discontinuity Description[6]
BB-FMD-101	NQ (1.875)	161.0	R1	2.7	44.0	49.0	46.5	5.0	5.0	100%	4.6	92%	Excellent	Fresh (W1)	Very Strong (R5)	73	44.0-49.0 ft: Grey, coarse grained, strongly foliated, fresh (W1), very strong (R5), GNEISS; discontinuities steep (80°- 90°) and parallel to foliation, very close to moderately closely spaced (0.1-1.1 ft) [VASSALBORO FORMATION].
			R2	7.7	49.0	54.0	51.5	5.0	5.0	100%	4.3	86%	Good	Fresh (W1)	Weak (R2)	63	49.0-54.0 ft: Grey, coarse grained, strongly foliated, fresh (W1), weak (R2), GNEISS; discontinuities steep (85°-90°) and parallel to foliation, closely spaced (0.3-0.9 ft) [VASSALBORO FORMATION].
BB-FMD-102	NQ (1.875)	143.1	R1	2.2	14.0	16.3	15.2	2.3	2.0	87%	1.3	57%	Fair	Slightly Weathered (W2)	Strong (R4)	59	14.0-16.3 ft: Grey, coarse grained, strongly foliated, slightly weathered (W2), strong (R4), GNEISS; discontinuities steep (75°) and parallel to foliation, closely spaced (0.1-0.6 ft) [VASSALBORO FORMATION].
			R2	5.3	16.3	20.3	18.3	4.0	3.5	88%	2.9	73%	Fair	Slightly Weathered (W2)	Very Strong (R5)	69	16.3-20.3 ft: Grey, coarse grained, strongly foliated, slightly weathered (W2), very strong (R5), GNEISS; discontinuities moderately dipping to steep (45°-85°) and parallel to foliation, close to moderately closely spaced (0.1-0.6 ft) [VASSALBORO FORMATION].
			R3	9.3	20.3	24.3	22.3	4.0	3.3	83%	1.3	33%	Poor	Moderately Weathered (W3)	Very Strong (R5)	58	20.3-24.3 ft: Grey, coarse grained, strongly foliated, moderately weathered (W3), very strong (R5), GNEISS; discontinuities steep to vertical (75°-90°) and parallel to foliation, close to moderately closely spaced (0.1-0.6 ft) [VASSALBORO FORMATION].
BB-FMD-103	NQ (1.875)	166.2	R1	1.5	19.0	22.0	20.5	3.0	2.9	97%	1.1	37%	Poor	Slightly Weathered (W2)	Very Strong (R5)	60	19.0-22.0 ft: Blue/grey, coarse grained, strongly foliated, slightly weathered (W2), very strong (R5), GNEISS; discontinuities low (5°-35°) and parallel to foliation, very closely spaced [VASSALBORO FORMATION].
			R2	5.5	22.0	27.0	24.5	5.0	4.9	98%	4.8	96%	Excellent	Slightly Weathered (W2)	Very Strong (R5)	76	22.0-27.0 ft: Blue/grey, coarse grained, strongly foliated, slightly weathered (W2), very strong (R5), GNEISS; discontinuities steep (75°-90°) and parallel to foliation, closely spaced (0.2-2.0 ft) [VASSALBORO FORMATION].
			R3	9.3	27.0	29.5	28.3	2.5	2.2	88%	2.2	88%	Good	Fresh (W1)	Very Strong (R5)	79	27.0-29.5 ft: Blue/grey, coarse grained, strongly foliated, fresh (W1), very strong (R5), GNEISS; no discontinuities [VASSALBORO FORMATION].
BB-FMD-104	NQ (1.875)	144.6	R1	2.5	6.3	11.3	8.8	5.0	4.9	98%	4.2	84%	Good	Fresh (W1)	Strong (R4)	71	6.3-11.3 ft: Blue/grey, coarse grained, strongly foliated, fresh (W1), strong (R4), GNEISS; discontinuities very steep to vertical (85°-90°) and parallel to foliation, very closely to moderately closely spaced (0.1-2.4 ft) [VASSALBORO FORMATION].
			R2	7.5	11.3	16.3	13.8	5.0	5.0	100%	4.8	96%	Excellent	Fresh (W1)	Very Strong (R5)	76	11.3-16.3 ft: Blue/grey, coarse grained, strongly foliated, fresh (W1), very strong (R5), GNEISS; discontinuities very steep (80°-85°) and parallel to foliation, closely to moderately closely spaced (0.2-1.3 ft) [VASSALBORO FORMATION].

Table 3: Summary of Rock Core Quality
Preliminary Geotechnical Design Report
I-295 Mallet Drive Bridge Replacement #5721 (Exit 22)
Freeport, Maine
MaineDOT WIN 21726.00

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	End	Midpoint	Run Length (ft)	TCR[2] Length (ft)	TCR[2] %	RQD[3] Length (ft)	RQD[3] %	RQD[3] Designation	Physical Rock Parameters Weathering[4]	Estimated Field Strength[4]	Rock Mass Rating [RMR][5]	Lithologic, Rock Mass and Discontinuity Description[6]
BB-FMD-105	NQ (1.875)	142.5	R1	2.5	9.3	14.3	11.8	5.0	5.0	100%	3.3	66%	Fair	Fresh (W1)	Very Strong (R5)	68	9.3-14.3 ft: Grey, coarse grained, strongly foliated, fresh (W1), very strong (R5), GNEISS; discontinuities moderately to very steep (55°-85°) and parallel to foliation, closely spaced (0.1-0.8 ft) [VASSALBORO FORMATION].
			R2	7.5	14.3	19.3	16.8	5.0	5.0	100%	4.0	80%	Good	Fresh (W1)	Very Strong (R5)	72	14.3-19.3 ft: Grey, coarse grained, strongly foliated, fresh (W1), very strong (R5), GNEISS; discontinuities very steep (50°-85°) and parallel to foliation, very closely to closely spaced (0.1-1.0 ft) [VASSALBORO FORMATION].
BB-FMD-106	NQ (1.875)	129.2	R1	2.5	15.8	20.8	18.3	5.0	4.9	98%	3.5	70%	Fair	Fresh (W1)	Very Strong (R5)	70	15.8-20.8 ft: Blue/grey, coarse grained, strongly foliated, fresh (W1), very strong (R5), GNEISS; discontinuities very steep to vertical (80°-90°) and parallel to foliation, very closely to closely spaced (0.1-1.7 ft) [VASSALBORO FORMATION].
			R2	7.0	20.8	24.8	22.8	4.0	3.6	90%	2.3	58%	Fair	Fresh (W1)	Very Strong (R5)	67	20.8-24.8 ft: Grey, coarse grained, strongly foliated, fresh (W1), very strong (R5), GNEISS; intermingled with grey and blue, coarse-grained, strongly foliated, fresh (W1), very strong (R5), PEGMATITE; with moderately steep to vertical discontinuities (40°-90°), very close to closely spaced (0.1-0.5 ft) [VASSALBORO FORMATION].
			R3	10.6	24.8	28.0	26.4	3.2	3.2	100%	1.6	50%	Poor	Fresh (W1)	Very Strong (R5)	65	24.8-28.0 ft: Blue/grey, coarse grained, strongly foliated, fresh (W1), very strong (R5), GNEISS; discontinuities very steep to vertical (85°-90°) and parallel to foliation, very closely to closely spaced (0.1-1.1 ft) [VASSALBORO FORMATION].
			R4	14.2	28.0	32.0	30.0	4.0	3.9	98%	1.9	48%	Poor	Fresh (W1)	Very Strong (R5)	64	28.0-32.0 ft: Grey, coarse grained, strongly foliated, fresh (W1), very strong (R5), GNEISS; intermingled with grey and blue, coarse-grained, strongly foliated, fresh (W1), extremely strong (R6), PEGMATITE; with very steep to vertical discontinuities (80°-90°), very close to closely spaced (0.1-0.8 ft) [VASSALBORO FORMATION].
			R5	18.1	32.0	35.8	33.9	3.8	3.8	100%	3.8	100%	Excellent	Fresh (W1)	Very Strong (R5)	80	32.0-35.8 ft: Blue/grey, fine grained, strongly foliated, fresh (W1), very strong (R5), PEGMATITE; discontinuities moderately to very steep (55°-85°) and parallel to foliation, closely to moderately closely spaced (0.4-2.9 ft) [VASSALBORO FORMATION].

Notes:
1. As-drilled locations and elevations are derived from survey files "Freeport 21726.00 Boring Data 2020-01-06" 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 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

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

Table 4: Summary of Laboratory Soil Index and Classification Testing Results
Geotechnical Design Report
I-295 Mallet Drive Bridge Replacement #5721 (Exit 22)
Freeport, Maine
MaineDOT WIN 21726.00

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[4]	Plastic Limit[4]	Plasticity Index[4]	Liquidity Index[4]	AASHTO Soil Classification[5]	USCS Soil Classification[5]
BB-FMD-101	161.0	2D	4.0 - 6.0	157.0 - 155.0	7.3	- -	- -	- -	- -	- -	A-1-b (0)	SP - SM
BB-FMD-101	161.0	3D	9.0 - 11.0	152.0 - 150.0	- -	6.0	- -	- -	- -	- -	- -	- -
BB-FMD-101	161.0	5DB	18.0 - 20.0	143.0 - 141.0	32.6	17.8	- -	- -	- -	- -	A-2-4 (0)	SM
BB-FMD-101	161.0	6DA	20.0 - 22.0	141.0 - 139.0	8.6	- -	- -	- -	- -	- -	A-1-a (0)	SP - SM
BB-FMD-101	161.0	7D	24.0 - 26.0	137.0 - 135.0	32.8	14.0	NP	NP	NP	NP	A-2-4 (0)	SM
BB-FMD-101	161.0	10D	33.0 - 35.0	128.0 - 126.0	- -	30.5	- -	- -	- -	- -	- -	- -
BB-FMD-101	161.0	12D	37.0 - 39.0	124.0 - 122.0	- -	30.0	43	23	20	0.4	A-7-6	CL
BB-FMD-101	161.0	13D	39.0 - 41.0	122.0 - 120.0	70.4	32.0	23	15	8	2.1	A-4 (3)	CL
BB-FMD-101	161.0	14D	41.0 - 43.0	120.0 - 118.0	25.6	- -	- -	- -	- -	- -	A-2-4 (0)	SM
BB-FMD-102	143.1	2D	4.0 - 6.0	139.1 - 137.1	- -	15.6	- -	- -	- -	- -	- -	- -
BB-FMD-103	166.2	2D	4.0 - 6.0	162.2 - 160.2	- -	3.0	- -	- -	- -	- -	- -	- -
BB-FMD-103	166.2	4DA	14.0 - 16.0	152.2 - 150.2	- -	26.0	34	18	16	0.5	A-6	CL
BB-FMD-103	166.2	4DB	14.0 - 16.0	152.2 - 150.2	11.4	- -	- -	- -	- -	- -	A-1-b (0)	SW - SM
BB-FMD-104	144.6	2D	4.0 - 6.0	140.6 - 138.6	47.7	22.0	NP	NP	NP	NP	A-4 (0)	SM
BB-FMD-105	142.5	2D	3.0 - 5.0	139.5 - 137.5	21.2	- -	- -	- -	- -	- -	A-2-4 (0)	SM
BB-FMD-106	129.2	2D	4.0 - 6.0	125.2 - 123.2	99.5	26.1	- -	- -	- -	- -	- -	- -
BB-FMD-106	129.2	4D	8.0 - 10.0	121.2 - 119.2	97.4	31.0	45	21	24	0.4	A-7-6 (26)	CL
BB-FMD-106	129.2	6D	12.0 - 14.0	117.2 - 115.2	- -	33.0	38	19	19	0.7	A-6	CL
BB-FMD-106	129.2	7DB	14.0 - 15.8	115.2 - 113.4	42.6	23.4	- -	- -	- -	- -	A-4 (0)	SM

Notes:
1. Test boring locations are shown in Sheet 2 entitled "Boring Location Plan".
2. As-drilled locations and elevations are derived from survey files "Freeport 21726.00 Boring Data 2020-01-06" received by Golder on January 6, 2019 from MaineDOT.
3. Laboratory testing was performed by GeoTesting Express, Inc.
4. NP = non-plastic
5. AASHTO and USCS symbols assigned based on interpretation of laboratory test results provided to Golder from GeoTesting Express, Inc. on February 4, 2020.
6. Atterberg Limits ASTM D 4318; Grain Size - Sieve & Hydrometer ASTM D 422; Moisture Content ASTM D 2216
7. Complete laboratory soil test results are provided in Appendix C.
8. ft = feet

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

Table 5: Summary Rock Laboratory Test Results
Preliminary Geotechnical Design Report
I-295 Mallet Drive Bridge Replacement #5721 (Exit 22)
Freeport, Maine
MaineDOT WIN 21726.00

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-FMD-101	161.0	43.8	117.2	RUN 2	6.5 - 6.8	50.26 - 50.63	110.7 - 110.4	358	169
BB-FMD-102	143.1	13.0	130.1	RUN 1	1.2 - 1.4	14.19 - 14.40	128.9 - 128.7	1495	172
BB-FMD-104	144.6	6.3	138.3	RUN 1	0.5 - 0.9	6.84 - 7.21	137.8 - 137.4	2127	171
BB-FMD-106	129.2	15.8	113.4	RUN 1	10.3 - 10.7	26.11 - 26.48	103.1 - 102.7	3279	162

Notes:

1. Test boring locations are shown in Sheet 2 entitled "Boring Location Plan".
2. Borings BB-FMD-101 through BB-FMD-106 were performed by New England Boring Contractors from December 15, 2019 to December 23, 2019.
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. (GTX). Test results provided to Golder on February 4, 2020.
5. BB-FMD-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-101 RUN 2, BB-FDR-104 RUN 1, 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 Mallet Drive Bridge Replacement #5721 (Exit 22)
Freeport, ME
MaineDOT WIN 021726.00

Stratigraphy		Depth Below Base of Abutment (ft)[1]	Layer Thickness (ft)	Lateral Model[4]	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	Layer 1	0.0 9.6	9.6	Sand (Reese)	125	-	33	165	-	-
Glaciomarine Silty Clay (above WT)	Layer 2	9.6 11.7	2.1	Stiff Clay with Free Water (Reese)	115	3500	-	500	0.005	-
Glaciomarine Silty Clay (below WT)	Layer 3	11.7 34.0	22.3	Stiff Clay w/o Free Water (Reese)	52.6	3500	-	-	0.005	-
Bedrock	Layer 4	34.0 50.0	16.0	Strong Rock (Vuggy Limestone)	106.6	-	-	-	-	12604

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. "Vuggy Limestone" refers to the LPile lateral model rather than the rock type encountered at site.
5. ft = feet; pcf = pounds per cubic foot; pci = pounds per cubic inch; psi = pounds per square inch
6. WT = water table

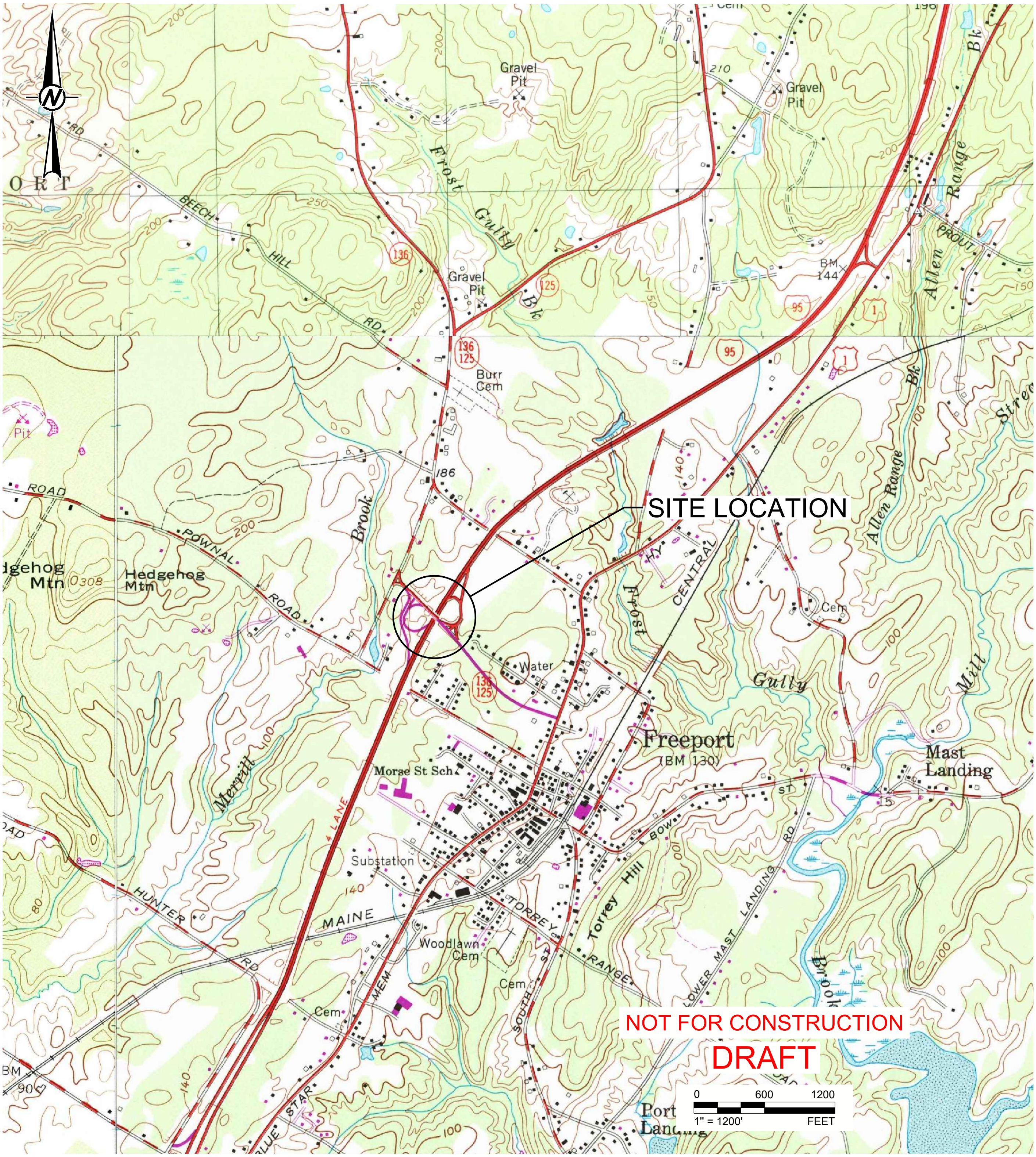
Prepared by: MLM
Checked by: KAR
Reviewed by: JRS

Table 7: Summary of Soil Properties Used in LPile Analysis - Abutment No. 2
Preliminary Geotechnical Design Report
I-295 Mallet Drive Bridge Replacement #5721 (Exit 22)
Freeport, ME
MaineDOT WIN 021726.00

Stratigraphy		Depth Below Base of Abutment (ft)[1]	Layer Thickness (ft)	Lateral Model[4]	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	Layer 1	0.0	6.0	Sand (Reese)	125	-	33	165	-	-
		6.0								
Glaciomarine Silty Clay (above WT)	Layer 2	6.0	4.0	Stiff Clay with Free Water (Reese)	115	3500	-	500	0.005	-
		10.0								
Glaciomarine Silty Clay (below WT)	Layer 3	10.0	2.0	Stiff Clay w/o Free Water (Reese)	52.6	3500	-	-	0.005	-
		12.0								
Bedrock	Layer 4	12.0	38.0	Strong Rock (Vuggy Limestone)	106.6	-	-	-	-	2486
		50.0								

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. "Vuggy Limestone" refers to the LPile lateral model rather than the rock type encountered at site.
5. WT = water table
6. ft = feet; pcf = pounds per cubic foot; pci = pounds per cubic inch; psi - pounds per square inch

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



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 MALLET DRIVE BRIDGE REPLACEMENT #5721 (EXIT 22)
FREEPORT, MAINE
MAINEDOT WIN 021726.00

CONSULTANT

GOLDER

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

TITLE

SITE LOCATION PLAN

PROJECT NO.	SUBTITLE	REV.	SHEET
19129538	A	0	1/3

Last Edited By: azumbado Date: 2020-12-16 Time:2:52:34 PM | Printed By: AZumbado Date: 2020-12-16 Time:3:02:05 PM
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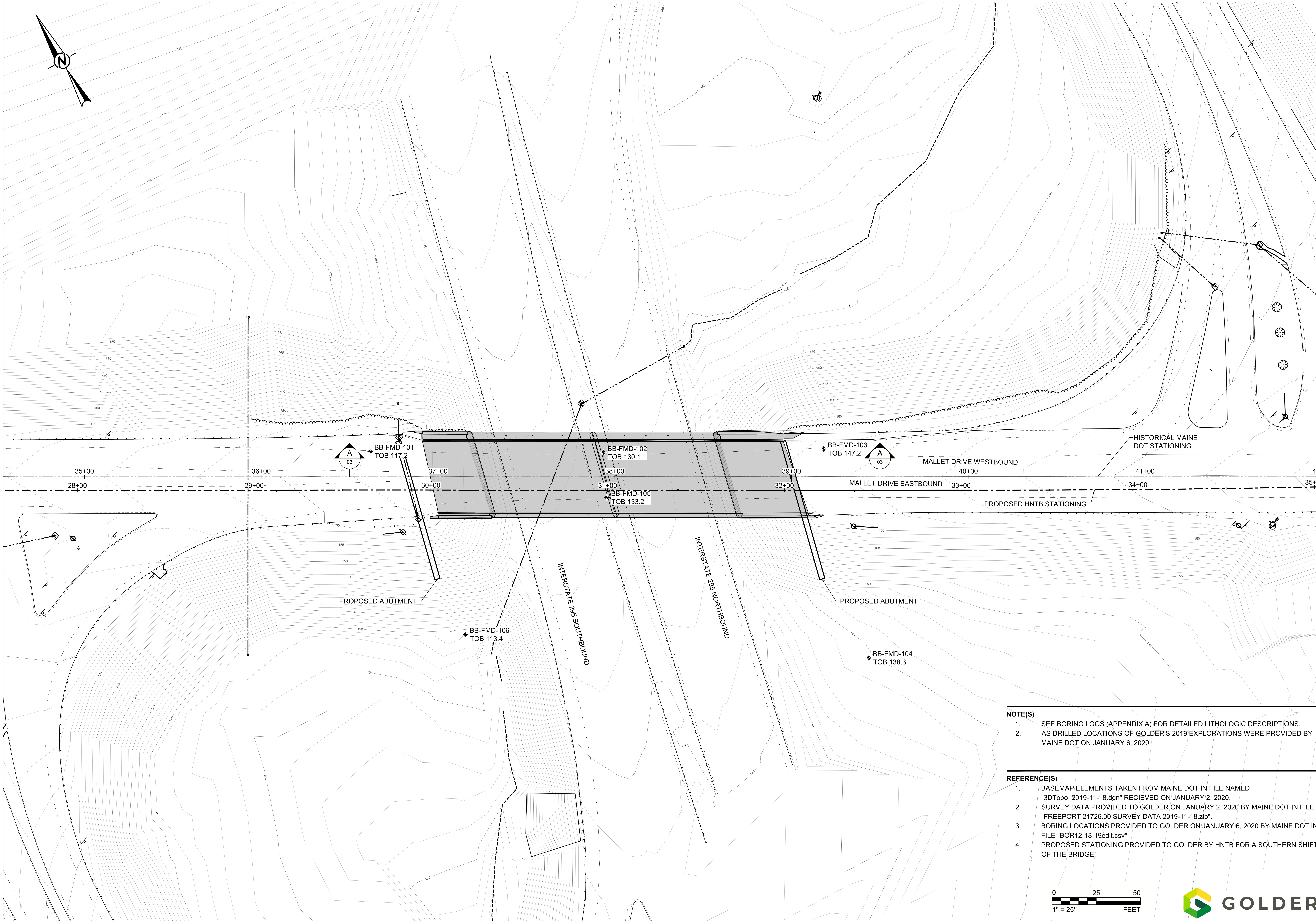
1 in



NOTE(S)

1. SEE BORING LOGS (APPENDIX A) FOR DETAILED LITHOLOGIC DESCRIPTIONS.
2. AS DRILLED LOCATIONS OF GOLDER'S 2019 EXPLORATIONS WERE PROVIDED BY MAINE DOT ON JANUARY 6, 2020.

REFERENCE(S)

1. BASEMAP ELEMENTS TAKEN FROM MAINE DOT IN FILE NAMED "3DTopo_2019-11-18.dgn" RECIEVED ON JANUARY 2, 2020.
2. SURVEY DATA PROVIDED TO GOLDER ON JANUARY 2, 2020 BY MAINE DOT IN FILE "FREEPORT 21726.00 SURVEY DATA 2019-11-18.zip".
3. BORING LOCATIONS PROVIDED TO GOLDER ON JANUARY 6, 2020 BY MAINE DOT IN FILE "BOR12-18-19edit.csv".
4. PROPOSED STATIONING PROVIDED TO GOLDER BY HNTB FOR A SOUTHERN SHIFT OF THE BRIDGE.

GOLDER

Date:2020-12-02
Username:
Division:
Filename: 19129538_0002_003

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

19129538

PIN 021726.00

BRIDGE PLANS

Bridge No. 5721

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

SIGNATURE

P.E. NUMBER

DATE

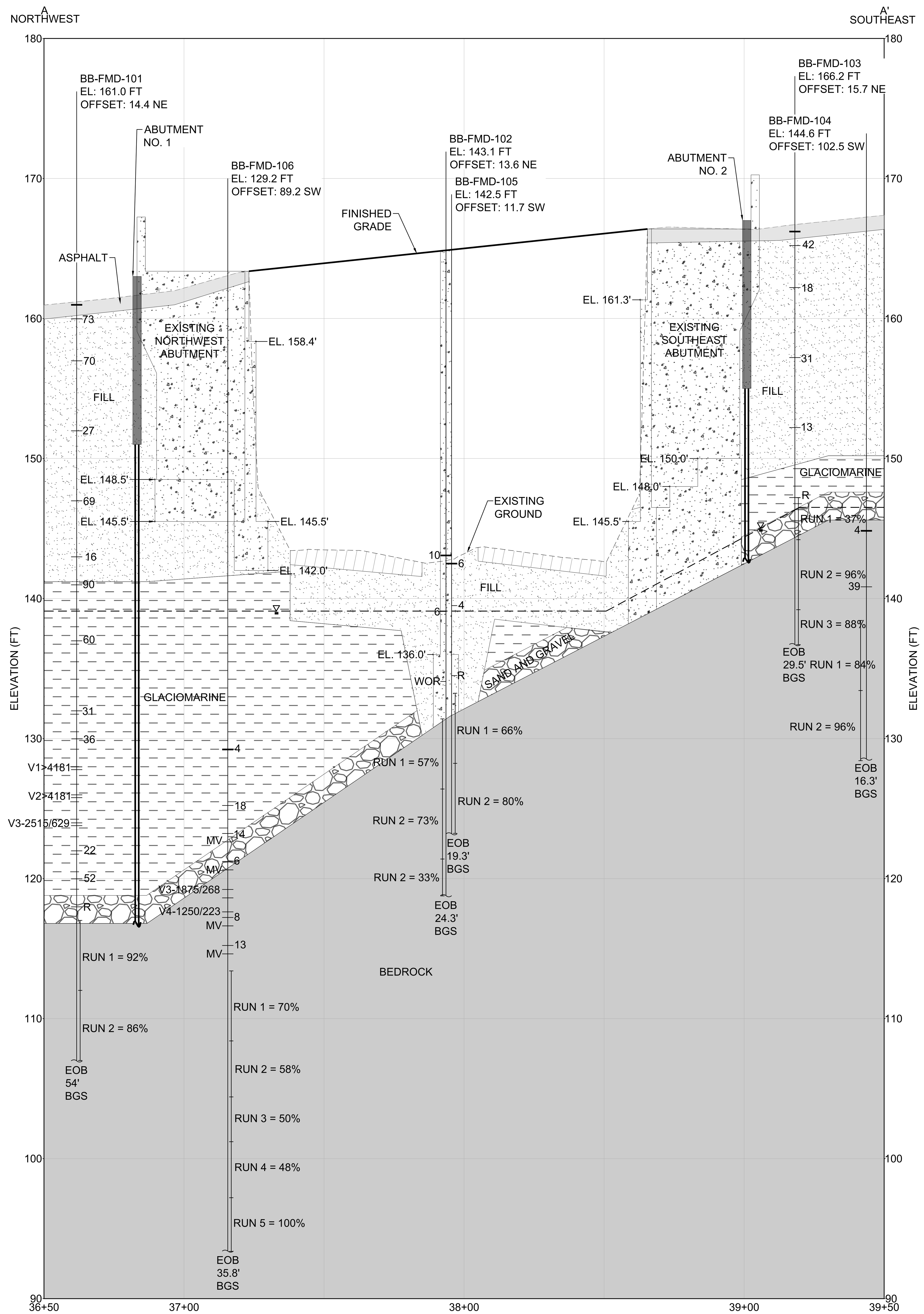
FREEPORT

BORING LOCATION PLAN

SHEET NUMBER

2

OF 3



LEGEND

BB-FMD-101 EL: 161.0 FT OFFSET: 14.4 N	BORING LOCATION I.D. ELEVATION OFFSET (FT) FROM SECTION LINE
	TOP OF BORING
30	SPT: N60 - VALUE (CORRECTED FOR HAMMER EFFICIENCY)
V1 - 2515/629	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 ROCK QUALITY DESIGNATION (RQD)
EOB 54' BGS	END OF BORING DEPTH FEET 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)
	BROWN, MEDIUM DENSE TO VERY DENSE, FINE TO COARSE SAND, SOME SILT, TRACE GRAVEL (SAND AND GRAVEL)
	LIGHT GRAY, COARSE-GRAINED GNEISS, EXTREMELY STRONG, FRESH TO SLIGHTLY WEATHERED (BEDROCK)
	INTERPRETED GROUNDWATER SURFACE
	EXISTING GROUND SURFACE

REFERENCES:

1. AS DRILLED BORING LOCATION PLAN DERIVED FROM ELECTRONIC FILE NAME: "BOR12-18-19edit.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. ABUTMENT AND PIER DETAILS INTERPRETED FROM ELECTRONIC FILE NAME "5721 FREEPORT 1956" PROVIDED TO GOLDER ON 8/22/2019.
7. FOR SOIL STRATA ANALYSIS THE ASPHALT LAYER AND ROADFILL LAYER ARE COMBINED FOR A LAYER THICKNESS OF FIVE FEET.

0 5 10
1" = 5' FEET

0 25 50
1" = 25' FEET

GOLDER

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

19129538

PIN
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BRIDGE PLANS

Bridge No. 5721

		BY	DATE
PROJ. MANAGER	MEL		
DESIGN-DETAILED	MEL	AAZ	2020-12-02
CHECKED-REVIEWED			
DESIGN2-DETAILED2			
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FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

FREEPORT

SUBSURFACE PROFILE A-A'

SHEET NUMBER

3

OF 3

Date:2020-12-02

Username:

Division:

Filename: 19129538_0002_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):

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

*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 MALLET DRIVE BRIDGE REPLACEMENT #5721 (EXIT 22)
Location: FREEPORT, MAINE

Boring No.: BB-FMD-101
WIN: 21726.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	161.0	**Auger ID/OD:**	4 in OD Solid Stem
Operator:	Brad Enos	**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:	12/16/19 (1:46), 12/17/19 (2:28)	**Drilling Method:**	Solid Stem Auger / Cased Wash	**Core Barrel:**	1-7/8 in - NQ
Boring Location:	N: 376042.7, E:1054596.1	**Casing ID/OD:**	4 in/4.5 in	**Water Level*:**	Not Recorded
Hammer Efficiency Factor:	0.896	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:

D = Split Spoon Sample	R = Rock Core Sample	S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)	T_v = Pocket Torvane Shear Strength (psf)
MD = Unsuccessful Split Spoon Sample Attempt	SSA = Solid Stem Auger	$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)	WC = Water Content, percent
U = Thin Wall Tube Sample	HSA = Hollow Stem Auger	q_p = Unconfined Compressive Strength (ksf)	LL = Liquid Limit
MU = Unsuccessful Thin Wall Tube Sample Attempt	RC = Roller Cone	N-uncorrected = Raw Field SPT N-value	PL = Plastic Limit
V = Field Vane Shear Test, PP = Pocket Penetrometer	WOH = Weight of 140lb. Hammer	Hammer Efficiency Factor = Rig Specific Annual Calibration Value	PI = Plasticity Index
MV = Unsuccessful Field Vane Shear Test Attempt	WOR/C = Weight of Rods or Casing	N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency	G = Grain Size Analysis
	WO1P = Weight of One Person	N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected	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	160.6		Driller notes asphalt thickness of 5 in (ASPHALT). 0.4	
	1D	24/9	1.00 - 3.00	27/27/22/16	49	73				Brown, dry, very dense, fine to coarse SAND, well-graded (FILL).	
	2D	24/18	4.00 - 6.00	30/29/18/16	47	70				Brown, dry, very dense, gravelly fine to coarse SAND, trace silt, poorly-graded (FILL).	GTX #540011 Fines = 7.3% A-1-b (0), SP-SM
5											
	3D	24/14	9.00 - 11.00	7/9/9/8	18	27				Brown, dry, medium dense, fine to coarse SAND, trace coarse gravel, trace silt, well-graded (FILL).	WC = 6.0%
10											
							106				
							122				
							203				
	4D	24/10	14.00 - 16.00	17/23/23/19	46	69	56			Brown, moist, very dense, fine to coarse SAND, trace silt, well-graded (FILL).	
15							52				
							55				
							94	143.5		17.5 Driller notes clay layer starting at 17. 5 ft bgs.	
	5D	24/12	18.00 - 20.00	13/6/5/6	11	16	51			5DA, Top 5 in: Brown/grey, wet, very stiff, Silty CLAY, moderately plastic, (GLACIOMARINE). q_p = 4.0, 5.0 ksf. (Pocket Penetrometer), T_v = 200, 300 psf. 5DB, Bottom 7 in: Brown, wet, medium dense, fine to coarse SAND, some silt clay, trace fine gravel, well-graded, non-plastic (GLACIOMARINE).	5DB: GTX #540010 WC = 17.8% Fines = 32.6% A-2-4 (0), SM
							33				
20	6D	24/14	20.00 - 22.00	30/42/18/8	60	90	100			6DA, Top 12 in: Brown, wet, very dense, fine Gravelly fine to coarse SAND, trace silt, poorly-graded (GLACIOMARINE). 6DB, Bottom 2 in: Brown/grey, wet, hard, CLAY, trace sand, moderately plastic, (GLACIOMARINE). q_p = 2.0 ksf (Pocket Penetrometer).	6DA: GTX #540009 Fines = 8.6% A-1-a (0), SP-SM
							76				
							95				
							100				
25	7D	24/18	24.00 - 26.00	5/23/17/16	40	60	101			7DA: Grey, moist, very dense, SAND, some silt, trace gravel, poorly-	7DA: GTX #540003

Remarks:

1. Hammer Efficiency Factor provided by New England Boring Contractors and taken from "SPT Energy Submittal_Drill Rig No. NEBCD-24" by GZA GeoEnvironmental, dated 7/12/19. 2. As-drilled boring locations and ground surface elevations were provided by MaineDOT. 3. "-" Sample interval disturbed by vane test before sampling; blow counts not representative of undisturbed material.

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-FMD-101

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-295 MALLET DRIVE BRIDGE REPLACEMENT #5721 (EXIT 22)
Location: FREEPORT, MAINE

Boring No.: BB-FMD-101
WIN: 21726.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	161.0	**Auger ID/OD:**	4 in OD Solid Stem
Operator:	Brad Enos	**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:	12/16/19 (1:46), 12/17/19 (2:28)	**Drilling Method:**	Solid Stem Auger / Cased Wash	**Core Barrel:**	1-7/8 in - NQ
Boring Location:	N: 376042.7, E:1054596.1	**Casing ID/OD:**	4 in/4.5 in	**Water Level*:**	Not Recorded
Hammer Efficiency Factor:	0.896	**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

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							87	
							89	
							74	
							123	
	8D	24/20	29.00 - 31.00	2/9/12/19	21	31	Open	
30								
	9D	24/14	31.00 - 33.00	12/12/12/14	24	36		
	10D	24/24	33.00 - 35.00	4/6/7/8	-			
	~~V1~~			~~S_u > 4181 psf~~				
35	11D	24/24	35.00 - 37.00	2/5/3/5	-			
	~~V2~~			~~S_u > 4181 psf~~				
	12D	24/24	37.00 - 39.00	WOH(18")/3	-			
	~~V3~~			~~S_u=2515/629 psf~~				
	13D	24/21	39.00 - 41.00	2/4/11/22	15	22		
40	MV			Would Not Push				
	14D	24/12	41.00 - 43.00	8/17/18/19	35	52		120.0
	15D	0/0	43.00 - 43.00	50(0")				117.2
	R1	60/60	44.00 - 49.00	RQD = 92%			NX	
45								
	R2	60/60	49.00 - 54.00	RQD = 86%				
50								

Visual Description and Remarks

graded (GLACIOMARINE).
7DB: Grey, moist, hard, CLAY, some fine to coarse sand, moderately plastic, (GLACIOMARINE). q_p = 8.0, 8.0 ksf (Pocket Penetrometer), T_v = 1100, 700, 900 psf.

8D: Grey, wet, hard, CLAY, some fine to coarse sand, moderately plastic (GLACIOMARINE). q_p = 10.0, 10.0 ksf (Pocket Penetrometer), T_v = 700, 800 psf.

9D: Grey, wet, hard, CLAY, some fine to coarse sand, moderately plastic (GLACIOMARINE). q_p = 2.0 2.0 ksf(Pocket Penetrometer), T_v = 1400 psf.

10D: Wet, grey, hard, CLAY, some fine to coarse sand, moderately plastic (GLACIOMARINE). q_p = 6.0, 6.0 ksf (Pocket Penetrometer), T_v = 1000, 900, psf.
16x32 mm vane raw torque readings:
V1: Reached 26.6 in-lbs without failure.
11D: Grey, wet, hard, CLAY, some fine to coarse sand, moderately plastic (GLACIOMARINE). q_p = 1.0 ksf (Pocket Penetrometer), T_v = 1000, 1100 psf.
16x32 mm vane raw torque readings:
V2: Reached 26.6 in-lbs without failure.
12D: Grey, wet, very stiff, CLAY, some fine to coarse sand, trace fine gravel, moderately plastic (GLACIOMARINE). q_p = 1.0 ksf (Pocket Penetrometer), T_v = 700, 300 psf.
16x32 mm vane raw torque readings:
V3: 16/4 in-lbs
13D: Grey, wet, very stiff, CLAY, some fine to coarse sand, moderately plastic (GLACIOMARINE).
Failed 65x130 mm vane, would not push to 40.0 ft bgs.
41.0
14D: Brown, wet, very dense, fine to coarse SAND, some silt, trace fine gravel, non-plastic (SAND AND GRAVEL).
No recovery; Refusal
43.8
Top of Bedrock at Elev. 117.2 ft.
R1: Grey, coarse grained, strongly foliated, fresh (W1), very strong (R5), GNEISS; discontinuities steep angle (80°- 90°) and parallel to foliation, very close to moderately closely spaced (0.1 - 1.1 ft) [VASSALBORO FORMATION].
Rock Core Rate (min:sec):
44.0-45.0 ft (3:31)
45.0-46.0 ft (3:05)
46.0-47.0 ft (2:29)

Laboratory Testing Results/ AASHTO and Unified Class.

WC = 14%
Fines = 32.8%
A-2-4 (0), SM

WC = 30.5%

12D:
GTX #539996
WC = 30%
LL = 43
PL = 23
PI = 20
LI = 0.3
A-7-6, CL
13D:
GTX #540002, 539995
WC = 32%
Fines = 70.4%
LL = 23
PL = 15
PI = 8
LI = 2.1
A-4 (3), CL
14D:
GTX #540008
Fines = 25.6%
A-2-4 (0), SM
q_p = 358 ksf

Remarks:

1. Hammer Efficiency Factor provided by New England Boring Contractors and taken from "SPT Energy Submittal_Drill Rig No. NEBCD-24" by GZA GeoEnvironmental, dated 7/12/19. 2. As-drilled boring locations and ground surface elevations were provided by MaineDOT. 3. "-" Sample interval disturbed by vane test before sampling; blow counts not representative of undisturbed material.

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 MALLET DRIVE BRIDGE REPLACEMENT #5721 (EXIT 22)
Location: FREEPORT, MAINE

Boring No.: BB-FMD-101
WIN: 21726.00

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Operator:	Brad Enos	**Datum:**	NAD83 (2011) Maine 2000 West	**Sampler:**	Standard Split Spoon
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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										47.0-48.0 ft (3:04) 48.0-49.0' ft (2:42) 100% Recovery R2: Grey, coarse grained, strongly foliated, fresh (W1), weak (R2), GNEISS; discontinuities steep angle (85° - 90°) and parallel to foliation, closely spaced (0.3 - 0.9 ft) [VASSALBORO FORMATION]. Rock Core Rate (min:sec): 49.0-50.0 ft (3:01) 50.0 -51.0 ft (2:26) 51.0 -52.0 ft (2:38) 52.0 -53.0 ft (3:35) 53.0 -54.0 ft (1:39) 100% Recovery	
								107.0		54.0	
55										**Bottom of Exploration at 54.0 feet below ground surface.** Boring backfilled with bentonite grout to 3 ft bgs, gravel from 3 ft bgs to 0.5 ft bgs, and cold patch asphalt to road surface.	
60											
65											
70											
75											

Remarks:

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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: I-295 MALLET DRIVE BRIDGE REPLACEMENT #5721 (EXIT 22) Location: FREEPORT, MAINE	Boring No.: BB-FMD-102 WIN: 21726.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	143.1	**Auger ID/OD:**	N/A
Operator:	Brad Enos	**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:	12/17/19 (2:55), 12/19/19 (0:21)	**Drilling Method:**	Cased Wash	**Core Barrel:**	1-7/8 in - NQ
Boring Location:	N: 375943.7, E: 1054683.2	**Casing ID/OD:**	4 in/4.5 in	**Water Level*:**	6.91 ft on 12/18/19 at 21:23
Hammer Efficiency Factor:	0.896	**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	2/4/3/2	7	10	Open			Brown, dry, loose, fine to coarse SAND, well-graded (FILL).	
	2D	24/4	4.00 - 6.00	3/3/1(12")	4	6	3			Brown, wet, loose, Silty fine SAND, well-graded (FILL).	WC = 15.6%
5							8				
							8				
							43				
							31				
	3D	24/3	9.00 - 11.00	WOR(24")	- -		15			Brown, wet, very loose, fine to coarse SAND, trace fine gravel, well-graded (FILL).	
10							14				
							20				
							144				
							Open	130.1		13.0 Top of Bedrock at Elev. 130.1 ft.	
	R1	27.6/24	14.00 - 16.30	RQD = 57%			NQ			R1: Grey, coarse grained, strongly foliated, slightly weathered (W2), strong (R4), GNEISS; discontinuities steep angle (75°) and parallel to foliation, closely spaced (0.1 - 0.6 ft) [VASSALBORO FORMATION]. Rock Core Rate (min:sec): 14.0-15.0 ft (3:32) 15.0-16.0 ft (2:40) 16.0-16.3 ft (1:43) 87% Recovery	GTX #311186 q_p = 1495 ksf
15											
	R2	48/42	16.30 - 20.30	RQD = 73%						R2: Grey, coarse grained, strongly foliated, slightly weathered (W2), very strong (R5), GNEISS; discontinuities moderately dipping to steep (45° - 85°) and parallel to foliation, close to moderately closely spaced (0.1 - 0.6 ft) [VASSALBORO FORMATION]. Rock Core Rate (min:sec): 16.3-17.3 ft (2:10) 17.3-18.3 ft (2:14) 18.3-19.3 ft (2:29) 19.3-20.3 ft (3:01)	
20	R3	48/39	20.30 - 24.30	RQD = 33%							
25								118.8			

Remarks:

1. Hammer Efficiency Factor provided by New England Boring Contractors and taken from "SPT Energy Submittal_Drill Rig No. NEBCD-24" 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 MALLET DRIVE BRIDGE REPLACEMENT #5721 (EXIT 22) **Location:** FREEPORT, MAINE	**Boring No.:** BB-FMD-102 **WIN:** 21726.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	143.1	**Auger ID/OD:**	N/A
Operator:	Brad Enos	**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:	12/17/19 (2:55), 12/19/19 (0:21)	**Drilling Method:**	Cased Wash	**Core Barrel:**	1-7/8 in - NQ
Boring Location:	N: 375943.7, E: 1054683.2	**Casing ID/OD:**	4 in/4.5 in	**Water Level*:**	6.91 ft on 12/18/19 at 21:23
Hammer Efficiency Factor:	0.896	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample Attempt
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V = Field Vane Shear Test, PP = Pocket Penetrometer
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										88% Recovery R3: Grey, coarse grained, strongly foliated, moderately weathered (W3), very strong (R5), GNEISS; discontinuities steep dipping to vertical (75° - 90°) and parallel to foliation, close to moderately closely spaced (0.1 - 0.6 ft) [VASSALBORO FORMATION]. Rock Core Rate (min:sec): 20.3-21.3 ft (3:02) 21.3-22.3 ft (1:46) 22.3-23.3 ft (1:36) 23.3-24.3 ft (2:46) 83% Recovery 24.3 **Bottom of Exploration at 24.3 feet below ground surface.** Boring backfilled with bentonite grout to ground surface.	
30											
35											
40											
45											
50											

Remarks:

1. Hammer Efficiency Factor provided by New England Boring Contractors and taken from "SPT Energy Submittal_Drill Rig No. NEBCD-24" 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 MALLET DRIVE BRIDGE REPLACEMENT #5721 (EXIT 22) **Location:** FREEPORT, MAINE	**Boring No.:** BB-FMD-103 **WIN:** 21726.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	166.2	**Auger ID/OD:**	4 in OD Solid Stem
Operator:	Brad Enos	**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:	12/15/19 (21:25), 12/16/19 (1:36)	**Drilling Method:**	Solid Stem Auger / Cased Wash	**Core Barrel:**	1-7/8 in - NQ
Boring Location:	N: 375852.0, E: 1054767.6	**Casing ID/OD:**	4 in/4.5 in	**Water Level*:**	Not Recorded
Hammer Efficiency Factor:	0.896	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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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	165.6		Driller notes asphalt thickness of 7 in (ASPHALT). 0.6	
	1D	24/15	1.00 - 3.00	16/13/15/13	28	42				Brown, dry, dense, fine to coarse SAND, trace fine gravel, well-graded (FILL).	
	2D	24/18	4.00 - 6.00	5/6/6/9	12	18				Brown, dry, medium dense, fine to coarse SAND, trace fine gravel, trace clay, well-graded (FILL).	WC = 3.0%
5											
	3D	24/19	9.00 - 11.00	4/12/9/6	21	31	87			Brown, dry, dense, fine to coarse SAND, trace fine gravel, trace clay, well-graded, non-plastic (FILL).	
10							58				
							73				
							230				
							56			Brown fine to coarse sand in wash from 0.6-13.5 ft bgs.	
	4D	24/12	14.00 - 16.00	6/5/4/6	9	13	59	151.6		4DB, Top 7 in: Brown, moist, medium dense, fine to coarse SAND, little fine gravel, little clay, well-graded (FILL). 14.6	4DB: GTX #540004 Fines = 11.4% A-1-b (0), SW-SM
15							37			4DA, Bottom 5 in: Brown, moist, stiff, fine to coarse Sandy CLAY, well-graded, moderately plastic (GLACIOMARINE). q_p = 5.0, 5.0 ksf (Pocket Penetrometer), T_v = 400, 400 psf Blocks of clay in wash water from 14-19 ft bgs.	4DA: GTX #540001 WC = 26% LL = 34 PL = 18 PI = 16 LI = 0.5 A-6, CL
							208				
							147				
							OPEN			19.0	
	R1 ~~5D~~	36/35 ~~0/0~~	19.00 - 22.00 ~~19.00 - 19.00~~	RQD = 37% ~~50(0")~~			NQ	147.2		Top of Bedrock at Elev. 147.2 ft. R1: Blue/grey, coarse grained, strongly foliated, slightly weathered (W2), very strong (R5), GNEISS; discontinuities low angle (5°-35°) and parallel to foliation, very closely spaced [VASSALBORO FORMATION]. Rock Core Rate (min:sec): 19.0-20.0 ft (3:40) 20.0-21.0 ft (1:58) 21.0-22.0 ft (2:46) 97% Recovery	
20											
	R2	60/59	22.00 - 27.00	RQD = 96%							
25											

Remarks:

1. Hammer Efficiency Factor provided by New England Boring Contractors and taken from "SPT Energy Submittal_Drill Rig No. NEBCD-24" 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 MALLET DRIVE BRIDGE REPLACEMENT #5721 (EXIT 22) Location: FREEPORT, MAINE	Boring No.: BB-FMD-103 WIN: 21726.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	166.2	**Auger ID/OD:**	4 in OD Solid Stem
Operator:	Brad Enos	**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:	12/15/19 (21:25), 12/16/19 (1:36)	**Drilling Method:**	Solid Stem Auger / Cased Wash	**Core Barrel:**	1-7/8 in - NQ
Boring Location:	N: 375852.0, E: 1054767.6	**Casing ID/OD:**	4 in/4.5 in	**Water Level*:**	Not Recorded
Hammer Efficiency Factor:	0.896	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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SSA = Solid Stem Auger
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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										R2: Blue/grey, coarse grained, strongly foliated, slightly weathered (W2), very strong (R5), GNEISS; discontinuities steep angle (75°-90°) and parallel to foliation, closely spaced (0.2 - 2.0 ft) [VASSALBORO FORMATION]. Rock Core Rate (min:sec): 22.0-23.0 ft (3:11) 23.0-24.0 ft (2:14) 24.0-25.0 ft (2:21) 25.0-26.0 ft (2:16) 26.0-27.0 ft (2:43) 98% Recovery	
	R3	30/26.5	27.00 - 29.50	RQD = 88%				136.7		R3: Blue/grey, coarse grained, strongly foliated, fresh (W1), very strong (R5), GNEISS; no discontinuities [VASSALBORO FORMATION]. Rock Core Rate (min:sec): 27.0-28.0 ft (2:06) 28.0-29.5 ft (2:10) 88% Recovery 29.5	
30										**Bottom of Exploration at 29.5 feet below ground surface.** Boring backfilled with bentonite grout to 0.5 ft bgs with cuttings and cold patch asphalt to road surface.	
35											
40											
45											
50											

Remarks:

1. Hammer Efficiency Factor provided by New England Boring Contractors and taken from "SPT Energy Submittal_Drill Rig No. NEBCD-24" by GZA GeoEnvironmental, dated 7/12/19. 2. As-drilled boring locations and ground surface elevations were provided by MaineDOT.

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Boring No.: BB-FMD-103

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-295 MALLET DRIVE BRIDGE REPLACEMENT #5721 (EXIT 22)
Location: FREEPORT, MAINE

Boring No.: BB-FMD-104
WIN: 21726.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	144.6	**Auger ID/OD:**	4 in OD Solid Stem
Operator:	Brad Enos	**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:	12/23/19 (7:54), 12/23/19 (9:16)	**Drilling Method:**	Solid Stem Auger	**Core Barrel:**	1-7/8 in - NQ
Boring Location:	N: 375754.3, E:1054696.2	**Casing ID/OD:**	3 in/3.5 in	**Water Level*:**	4.5 ft on 12/23/19 at 8:07
Hammer Efficiency Factor:	0.842	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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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 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	24/6.5	0.00 - 2.00	1/1/2/2	3	4	SSA			Brown, moist, soft, SILT, trace coarse gravel, trace organics, non-plastic (TOPSOIL).	
								142.6		2.0	
	2D	24/24	4.00 - 6.00	4/16/12/22	28	39		140.4		2DC, Top 2 in: Brown, moist, hard, CLAY, trace silt, slightly plastic (GLACIOMARINE).	
5										4.2	2DA/B: GTX #540005 Fines = 47.7% A-4 (0), SM
	R1	60/59	6.30 - 11.30	RQD = 84%			NQ	138.3		2DA and 2DB, Bottom 22 in: Brown, moist, dense, Silty fine to medium SAND, trace fine gravel, poorly-graded (SAND AND GRAVEL).	
										6.3	q_p = 2127 ksf
										Top of Bedrock at Elev. 138.3 ft. R1: Blue/grey, coarse grained, strongly foliated, fresh (W1), strong (R4), GNEISS; discontinuities vertical angle (85°-90°) and parallel to foliation, very closely to moderately closely spaced (0.1 - 2.4 ft) [VASSALBORO FORMATION]. Rock Core Rate (min:sec): 6.3-7.3 ft (2:49) 7.3-8.3 ft (2:24) 8.3-9.3 ft (2:45) 9.3-10.3 ft (3.15) 10.3-11.3 ft (3.02) 98% Recovery	
10											
	R2	60/60	11.30 - 16.30	RQD = 96%							
15										R2: Blue/grey, coarse grained, strongly foliated, fresh (W1), very strong (R5), GNEISS; discontinuities steep angle (80°-85°) and parallel to foliation, closely to moderately closely spaced (0. 2 - 1.3 ft) [VASSALBORO FORMATION]. Rock Core Rate (min:sec): 11.3-12.3 ft (3:29) 12.3-13.3 ft (2:59) 13.3-14.3 ft (2:16) 14.3-15.3 ft(2:14) 15.3-16.3 ft (2:41) 100% Recovery	
								128.3			
20										16.3	
										Bottom of Exploration at 16.3 feet below ground surface. Boring backfilled with soil cuttings and bentonite chips to ground surface.	
25											

Remarks:

1. Hammer Efficiency Factor provided by New England Boring Contractors and taken from "SPT Energy Submittal_Drill Rig No. 23" 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 MALLET DRIVE BRIDGE REPLACEMENT #5721 (EXIT 22)
Location: FREEPORT, MAINE

Boring No.: BB-FMD-105
WIN: 21726.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	142.5	**Auger ID/OD:**	N/A
Operator:	Brad Enos	**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:	12/19/19 (0:52), 12/19/19 (4:15)	**Drilling Method:**	Cased Wash	**Core Barrel:**	1-7/8 in - NQ
Boring Location:	N: 375925.3, E: 1054665.6	**Casing ID/OD:**	4 in/4.5 in	**Water Level*:**	3.41 ft on 12/19/19 at 4:00
Hammer Efficiency Factor:	0.896	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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V = Field Vane Shear Test, PP = Pocket Penetrometer
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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	24/7	0.00 - 2.00	1/2/2/1	4	6	7			Brown, dry, loose, fine to coarse SAND, some silt, trace fine gravel, well-graded (FILL).	
							10				
							15				
	2D	24/4	3.00 - 5.00	2/2/1(12")	3	4	2			Brown, wet, very loose, fine to coarse SAND, some silt, trace fine gravel, well-graded (FILL).	GTX #540012 Fines = 21.2% A-2-4 (0), SM
							9				
5							7				
							12				
							19				
	3D	16/4	8.00 - 9.33	1/1/50(4")	- -					Brown, wet, very dense, fine to coarse SAND, some silt, well-graded (FILL).	
	R1	60/60	9.30 - 14.30	RQD = 66%			NQ	133.2		9.3 Top of Bedrock Elev. 133.2 ft. R1: Grey, coarse grained, strongly foliated, fresh (W1), very strong (R5), GNEISS; discontinuities steep angle (55°-85°) and parallel to foliation, closely spaced (0.1 - 0.8') [VASSALBORO FORMATION]. Rock Core Rate (min:sec): 9.3-10.3 ft (3:34) 10.3-11.3 ft (3:08) 11.3 12.3 ft (3:07) 12.3-13.3 ft (2:37) 13.3-14.3 ft (3:05) 100% Recovery	
10											
15	R2	60/60	14.30 - 19.30	RQD = 80%						R2: Grey, coarse grained, strongly foliated, fresh (W1), very strong (R5), GNEISS; discontinuities steep angle (50°-85°) and parallel to foliation, very closely to closely spaced (0.1 - 1 ft) [VASSALBORO FORMATION]. Rock Core Rate (min:sec): 14.3-15.3 ft (1:56) 15.3-16.3 ft (1:57) 16.3 17.3 ft (1:57) 17.3-18.3 ft(1.53) 18.3-19.3 ft (2.17) 100% Recovery	
20								123.2		19.3 **Bottom of Exploration at 19.3 feet below ground surface.** Boring backfilled with gravel to ground surface.	
25											

Remarks:

1. Hammer Efficiency Factor provided by New England Boring Contractors and taken from "SPT Energy Submittal_Drill Rig No. NEBCD-24" 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-FMD-105

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-295 MALLET DRIVE BRIDGE REPLACEMENT #5721 (EXIT 22)
Location: FREEPORT, MAINE

Boring No.: BB-FMD-106
WIN: 21726.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	129.2	**Auger ID/OD:**	4 in OD Solid Stem
Operator:	Brad Enos	**Datum:**	NAD83 (2011) Maine 2000 West	**Sampler:**	Standard Split Spoon
Logged By:	Shiv Bhardwaj	**Rig Type:**	SIMCO 2400	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	12/23/19 (10:35); 12/23/19 (16:51)	**Drilling Method:**	Solid Stem Auger / Cased Wash	**Core Barrel:**	1-7/8 in - NQ
Boring Location:	N: 375933.5, E: 1054554.5	**Casing ID/OD:**	3 in/3.5 in	**Water Level*:**	3.1 ft on 12/23/2019 at 16:30
Hammer Efficiency Factor: 0.600		**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	24/12	0.00 - 2.00	1/2/2/2	4	4	SSA			Brown, moist, soft, SILT, some fine sand, little organics, non-plastic (TOPSOIL).	
								127.2		2.0	
	2D	24/24	4.00 - 6.00	5/8/10/12	18	18				2D: Grey, dry, very stiff, CLAY, moderately plastic (GLACIOMARINE). q_p = 8.5 ksf (Pocket Penetrometer), T_v = 800, 1000, 1000 psf	GTX #540006 WC = 26.1% Fines = 99.5%
5											
	3D	24/19	6.00 - 8.00	6/7/7/8	14	14	25			3D: Grey, wet, stiff, CLAY, moderately plastic (GLACIOMARINE). q_p = 5.0, 5.0 ksf (Pocket Penetrometer), T_v = 900, 1000 psf.	
	MV			Would Not Push			19			Failed 55x110 mm vane, would not push to 7.0 ft bgs.	
	4D	24/7.5	8.00 - 10.00	2/3/3/2	6	6	20			4D: Grey, wet, medium stiff, CLAY, trace sand, moderately plastic (GLACIOMARINE). q_p = 2.0 2.0 ksf (Pocket Penetrometer), T_v = 300, 700 psf.	4D: GTX #540007, 539999 WC = 31% Fines = 97.4% LL = 45 PL = 21 PI = 24 LI = 0.4 A-7-6 (26), CL
	MV			Would Not Push			24			Failed 55x110 mm vane, would not push to 9.0 ft bgs	
10	5D	24/24	10.00 - 12.00	2/1(12")/2	-		Open			5D: Grey, wet, stiff, CLAY, moderately plastic (GLACIOMARINE). q_p = 1.0 ksf (Pocket Penetrometer), T_v = 500, 500 psf.	
	V3			S_u=1875/268psf						55x110 mm vane raw torque readings: V3: 42/6 ft-lbs	
	V4			S_u=1250/223psf						V4: 28/5 ft-lbs	
	6D	24/5	12.00 - 14.00	1/3/5/5	8	8				6D: Grey, wet, medium stiff, CLAY, moderately plastic (GLACIOMARINE). q_p < 1.0 ksf (Pocket Penetrometer), T_v = 200 psf.	6D: GTX #540000 WC = 33% LL = 38 PL = 19 PI = 19 LI = 0.7 A-6, CL
	MV			Would Not Push				115.2		Failed 55x110 mm vane, would not push to 13.0 ft bgs 14.0	
	7D	22/15	14.00 - 15.83	WOR(5"),1(1")/5/8/50(4")	13	13				7DB, Top 7 in: Brown, wet, medium dense, Silty fine to coarse SAND, well-graded (SAND AND GRAVEL). 7DA, Bottom 8 in: Brown, wet, medium dense, fine to medium SAND, trace fine gravel, trace silt, well-graded (SAND AND GRAVEL).	7DB: GTX #540013 WC = 23.4% Fines = 42.6% A-4 (0), SM
15	MV			Would Not Push			NQ	113.4		Failed 55x110 mm vane, would not push to 15.0 ft bgs 15.8	
	R1	60/59	15.80 - 20.80	RQD = 70%						Top of Bedrock Elev. 113.4 ft. R1: Blue/grey, coarse grained, strongly foliated, fresh (W1), very strong (R5), GNEISS; discontinuities vertical angle (80°-90°) and parallel to foliation, very closely to closely spaced (0.1-1.7 ft) [VASSALBORO FORMATION]. Rock Core Rate (min:sec): 15.8-16.8 ft (3:41) 16.8-17.8 ft (2:51) 17.8-18.8 ft (1:45) 18.8-19.8 ft (2:06)	q_p = 3279 ksf
20	R2	48/43	20.80 - 24.80	RQD = 58%							
25	R3	38.4/38	24.80 - 28.00	RQD = 50%							

Remarks:

1. As-drilled boring locations and ground surface elevations were provided by MaineDOT. 2. "-" Sample interval disturbed by vane test before sampling; blow counts not representative of undisturbed material.

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* 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 MALLET DRIVE BRIDGE REPLACEMENT #5721 (EXIT 22)
Location: FREEPORT, MAINE

Boring No.: BB-FMD-106
WIN: 21726.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	129.2	**Auger ID/OD:**	4 in OD Solid Stem
Operator:	Brad Enos	**Datum:**	NAD83 (2011) Maine 2000 West	**Sampler:**	Standard Split Spoon
Logged By:	Shiv Bhardwaj	**Rig Type:**	SIMCO 2400	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	12/23/19 (10:35); 12/23/19 (16:51)	**Drilling Method:**	Solid Stem Auger / Cased Wash	**Core Barrel:**	1-7/8 in - NQ
Boring Location:	N: 375933.5, E: 1054554.5	**Casing ID/OD:**	3 in/3.5 in	**Water Level*:**	3.1 ft on 12/23/2019 at 16:30
Hammer Efficiency Factor:	0.600	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
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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										19.8-20.8 ft (1:58) 98% Recovery R2: Grey, coarse grained, strongly foliated, fresh (W1), very strong (R5), GNEISS; intermingled with grey and blue, coarse-grained, strongly foliated, fresh (W1), very strong (R5), PEGMATITE; with moderately to vertical discontinuities angle (40° - 90°), very close to closely spaced (0.1'-0.5 ft) [VASSALBORO FORMATION]. Rock Core Rate (min:sec): 20.8-21.8 ft (1:24) 21.8-22.8 ft (3:27) 22.8-23.8 ft (8:01) 23.8-24.8 ft (20:06) 90% Recovery	
	R4	48/47	28.00 - 32.00	RQD = 48%							
30											
	R5	46/46	32.00 - 35.83	RQD = 100%						R3: Blue/grey, coarse grained, strongly foliated, fresh (W1), very strong (R5), GNEISS; discontinuities vertical angle (85°-90°) and parallel to foliation, very closely to closely spaced (0.1-1.1 ft) [VASSALBORO FORMATION]. Rock Core Rate (min:sec): 24.8-25.8 ft (6:05) 25.8-26.8 ft (4:20) 26.8-27.8 ft (7:26) 27.8-28.0 ft (1:22) 100% Recovery	
35								93.4			
40										R4: Grey, coarse grained, strongly foliated, fresh (W1), very strong (R5), GNEISS; intermingled with grey and blue, coarse-grained, strongly foliated, fresh (W1), extremely strong (R6), PEGMATITE; with vertical discontinuities angle (80° - 90°), very close to closely spaced (0.1 - 0.8 ft) [VASSALBORO FORMATION]. Rock Core Rate (min:sec): 28.0-29.0 ft (3:42) 29.0-30.0 ft (3:10) 30.0-31.0 ft (2:55) 31.0-32.0 ft (4:15) 98% Recovery	
45										R5: Blue/grey, fine grained, strongly foliated, fresh (W1), very strong (R5), PEGMATITE; discontinuities steep angle (55°-85°) and parallel to foliation, closely to moderately closely spaced (0.4 - 2.9 ft) [VASSALBORO FORMATION]. Rock Core Rate (min:sec): 32.0-33.0 ft (5:22) 33.0-34.0 ft (7:04)	
50											

Remarks:

1. As-drilled boring locations and ground surface elevations were provided by MaineDOT. 2. "-" Sample interval disturbed by vane test before sampling; blow counts not representative of undisturbed material.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-295 MALLET DRIVE BRIDGE REPLACEMENT #5721 (EXIT 22)
Location: FREEPORT, MAINE

Boring No.: BB-FMD-106
WIN: 21726.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	129.2	**Auger ID/OD:**	4 in OD Solid Stem
Operator:	Brad Enos	**Datum:**	NAD83 (2011) Maine 2000 West	**Sampler:**	Standard Split Spoon
Logged By:	Shiv Bhardwaj	**Rig Type:**	SIMCO 2400	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	12/23/19 (10:35); 12/23/19 (16:51)	**Drilling Method:**	Solid Stem Auger / Cased Wash	**Core Barrel:**	1-7/8 in - NQ
Boring Location:	N: 375933.5, E: 1054554.5	**Casing ID/OD:**	3 in/3.5 in	**Water Level*:**	3.1 ft on 12/23/2019 at 16:30
Hammer Efficiency Factor:	0.600	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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SSA = Solid Stem Auger
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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.
50										34.0-35.0 ft (11:54) 35.0-35.8 ft (9:34) 100% Recovery	
										35.8	
										Bottom of Exploration at 35.8 feet below ground surface. Boring backfilled with soil cuttings, gravel and bentonite chips to ground surface.	
55											
60											
65											
70											
75											

Remarks:

1. As-drilled boring locations and ground surface elevations were provided by MaineDOT. 2. "-" Sample interval disturbed by vane test before sampling; blow counts not representative of undisturbed material.

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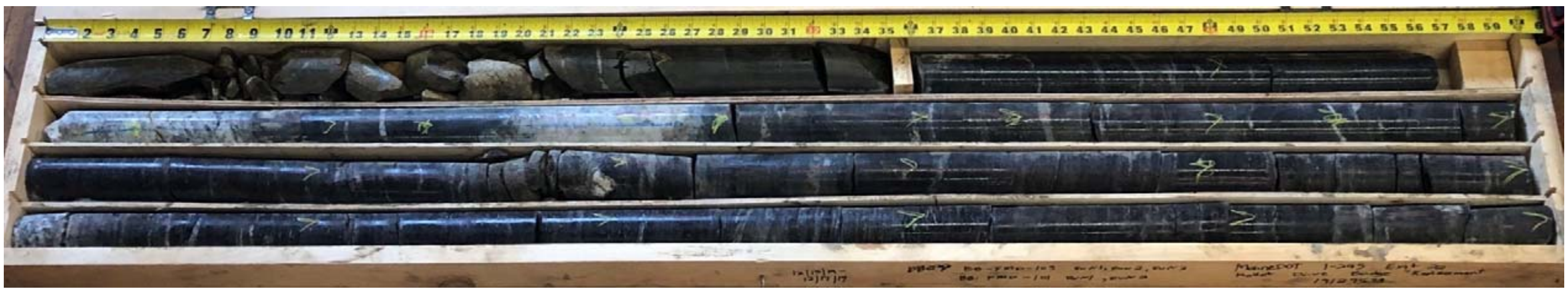
APPENDIX B

Rock Core Photos

APPENDIX B
Rock Core Photos
I-295 MALLET DRIVE BRIDGE REPLACEMENT #5721 (EXIT 22)
FREEPORT, MAINE
MAINEDOT WIN 21726.00

Boring	Date Cored	Run	Depth Below Surface feet	Recovery		RQD	
				Feet	%	Feet	%
BB-FMD-103	12/15/2019	R1	19.0 - 22.0	2.9 / 3.0	97	1.1 / 3.0	37
	12/16/2019	R2	22.0 - 27.0	4.9 / 5.0	98	4.8 / 5.0	96
	12/16/2019	R3	27.0 - 29.5	2.2 / 2.5	88	2.2 / 2.5	88
BB-FMD-101	12/17/2019	R1	44.0 - 49.0	5.0 / 5.0	100	4.6 / 5.0	92
	12/17/2019	R2	49.0 - 54.0	5.0 / 5.0	100	4.3 / 5.0	86



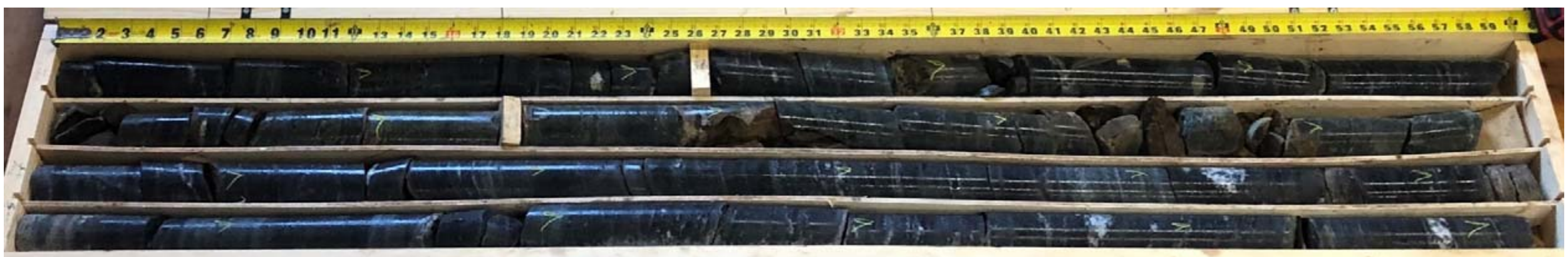
From top to bottom of photo:
Row 1 = BB-FMD-103 Run 1: 19.0 - 22.0 ft bgs and BB-FMD-103 Run 3: 27.0 - 29.5 ft bgs
Row 2 = BB-FMD-103 Run 2: 22.0 - 27.0 ft bgs
Row 3 = BB-FDR-101 Run 1: 44.0 - 49.0 ft bgs
Row 4 = BB-FDR-101 Run 2: 49.0 - 54.0 ft bgs

Note:
Rock core was wetted with water for photographs.

GOLDER

APPENDIX B
Rock Core Photos
I-295 MALLET DRIVE BRIDGE REPLACEMENT #5721 (EXIT 22)
FREEPORT, MAINE
MAINEDOT WIN 21726.00

Boring	Date Cored	Run	Depth Below Surface feet	Recovery		RQD	
				Feet	%	Feet	%
BB-FMD-102	12/18/2019	R1	14.0 - 16.3	2.0 / 2.3	87	1.3 / 2.3	58
	12/18/2019	R2	16.3 - 20.3	3.5 / 4.0	88	2.9 / 4.0	73
	12/18/2019	R3	20.3 - 24.3	3.3 / 4.0	81	1.3 / 4.0	33
BB-FMD-105	12/19/2019	R1	9.3 - 14.3	5.0 / 5.0	100	3.3 / 5.0	66
	12/19/2019	R2	14.3 - 19.3	5.0 / 5.0	100	4.0 / 5.0	80



From top to bottom of photo:
Row 1 = BB-FMD-102 Run 1: 14.0 - 16.3 ft bgs and BB-FMD-102 Run 2: 16.3 - 19.1 ft bgs
Row 2 = BB-FMD-102 Run 2: 19.1 - 20.3 ft bgs and BB-FMD-102 Run 3: 20.3 - 24.3 ft bgs
Row 3 = BB-FMD-105 Run 1: 9.3 - 14.3 ft bgs
Row 4 = BB-FMD-105 Run 2: 14.3 - 19.3 ft bgs

Note:
Rock core was wetted with water for photographs.

GOLDER

APPENDIX B
Rock Core Photos
I-295 MALLET DRIVE BRIDGE REPLACEMENT #5721 (EXIT 22)
FREEPORT, MAINE
MAINEDOT WIN 21726.00

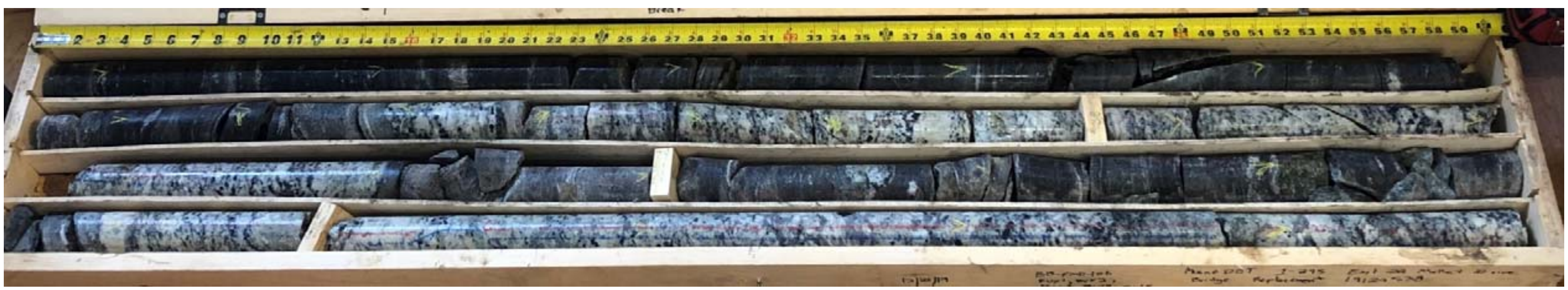
Boring	Date Cored	Run	Depth Below Surface feet	Recovery		RQD	
				Feet	%	Feet	%
BB-FDM-104	12/23/2019	R1	6.3 - 11.3	4.9 / 5.0	98	4.2 / 5.0	84
	12/23/2019	R2	11.3 - 16.3	5.0 / 5.0	100	4.8 / 5.0	96

From top to bottom of photo:
Row 1 = BB-FMD-104 Run 1: 6.3 - 11.3 ft bgs
Row 2 = BB-FDR-104 Run 2: 11.3 - 16.3 ft bgs

Note:
Rock core was wetted with water for photographs.

APPENDIX B
Rock Core Photos
I-295 MALLET DRIVE BRIDGE REPLACEMENT #5721 (EXIT 22)
FREEPORT, MAINE
MAINEDOT WIN 21726.00

Boring	Date Cored	Run	Depth Below Surface feet	Recovery		RQD	
				Feet	%	Feet	%
BB-FMD-106	12/23/2019	R1	15.8 - 20.8	4.9 / 5.0	98	3.5 / 5.0	70
	12/23/2019	R2	20.8 - 24.8	3.6 / 4.0	90	2.3 / 4.0	58
	12/23/2019	R3	24.8 - 28.0	3.2 / 3.2	100	1.6 / 3.2	50
	12/23/2019	R4	28.0 - 32.0	3.9 / 4.0	98	1.9 / 4.0	48
	12/23/2019	R5	32.0 - 35.8	3.8 / 3.8	100	3.8 / 3.8	100



From top to bottom of photo:
Row 1 = BB-FMD-106 Run 1: 15.8 - 20.8 ft bgs
Row 2 = BB-FMD-106 Run 2: 20.8 - 24.8 ft bgs and BB-FMD-106 Run 3: 24.8 - 26.1 ft bgs
Row 3 = BB-FMD-106 Run 3: 26.1 - 28.0 ft bgs and BB-FMD-106 Run 4: 28.0 - 30.7 ft bgs
Row 4 = BB-FMD-106 Run 4: 30.7 - 32.0 ft bgs and BB-FMD-106 Run 5: 32.0 - 35.8 ft bgs

Note:
Rock core was wetted with water for photographs.

GOLDER

APPENDIX C

Laboratory Test Results

MOISTURE CONTENT OF SOIL AND ROCK

Client:	Golder Associates				
Project:	Mallett Dr. Bridge Replace. I-295 Ex 22				
Location:	Freeport, ME			Project No:	GTX-311186
Boring ID:	---	Sample Type:	---	Tested By:	GA
Sample ID:	---	Test Date:	01/28/20	Checked By:	jsc
Depth :	---	Test Id:	540020		

Moisture Content of Soil and Rock - ASTM D2216

Boring ID	Sample ID	Depth	Description	Moisture Content,%
BB-FMD-101	3D	9-11 ft	Moist, yellowish brown silty sand with gravel	6.0
BB-FMD-101	5DB	18-20 ft	Moist, reddish gray clayey sand	17.8
BB-FMD-101	10D	33-35 ft	Moist, gray clay with sand	30.5
BB-FMD-102	2D	4-6 ft	Moist, olive brown silty sand	15.6
BB-FMD-103	2D	4-6 ft	Moist, grayish brown sand with silt	3.0
BB-FMD-106	2D	4-6 ft	Moist, olive gray clay	26.1
BB-FMD-106	7DB	14-15.8 ft	Moist, grayish brown silty sand	23.4

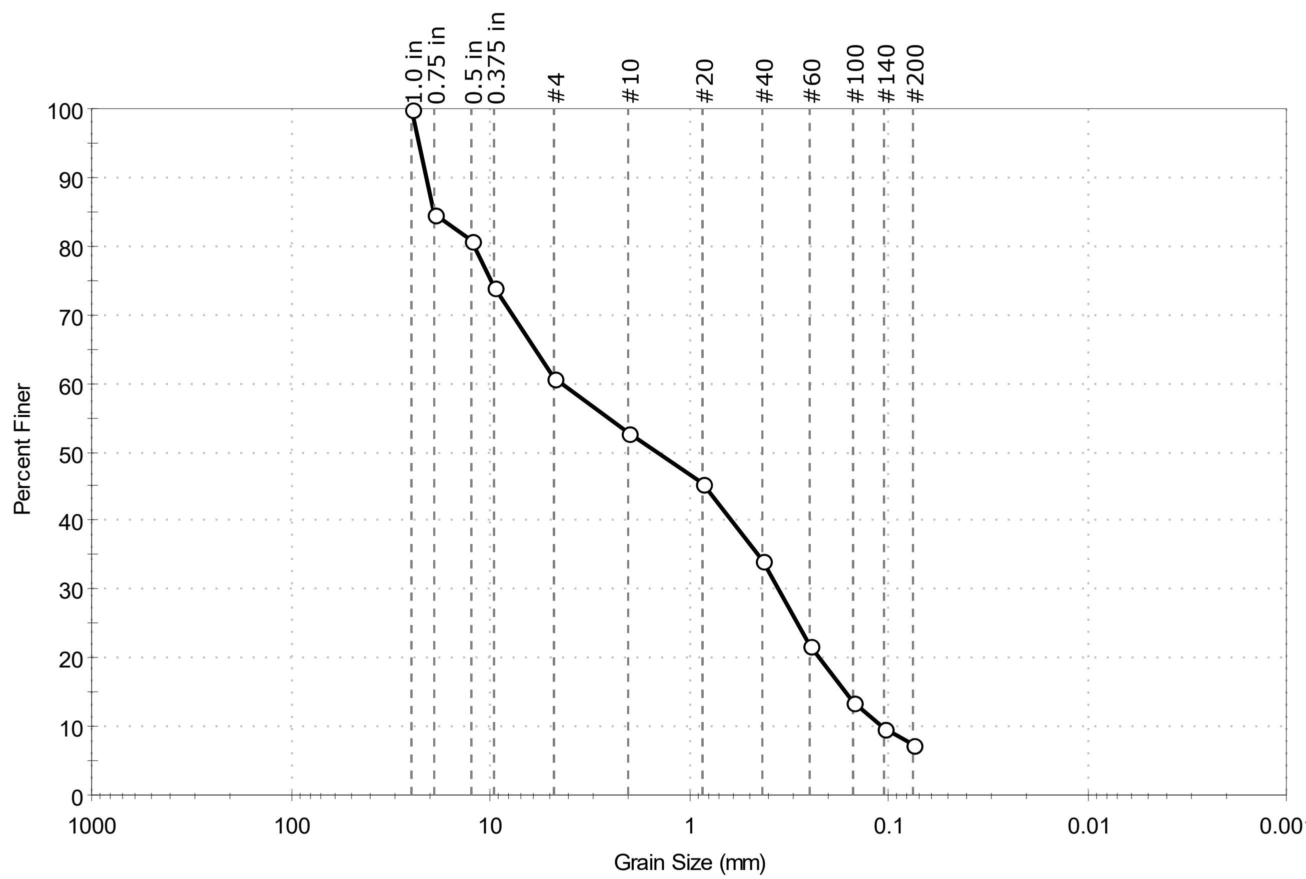
Notes: Temperature of Drying : 110º Celsius

PARTICLE SIZE ANALYSIS

GeoTesting EXPRESS

Client:	Golder Associates		
Project:	Mallett Dr. Bridge Replace. I-295 Ex 22		
Location:	Freeport, ME	Project No:	GTX-311186
Boring ID:	BB-FMD-101	Sample Type: jar	Tested By: GA
Sample ID:	2D	Test Date: 01/28/20	Checked By: jsc
Depth :	4-6 ft	Test Id: 540011	
Test Comment:	---		
Visual Description:	Moist, reddish yellow sand with silt and gravel		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	39.3	53.4	7.3

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.0 in	25.00	100		
0.75 in	19.00	85		
0.5 in	12.50	81		
0.375 in	9.50	74		
#4	4.75	61		
#10	2.00	53		
#20	0.85	46		
#40	0.42	34		
#60	0.25	22		
#100	0.15	14		
#140	0.11	10		
#200	0.075	7.3		

Coefficients

D_{85} =19.1393 mm D_{30} =0.3535 mm

D_{60} =4.3971 mm D_{15} =0.1633 mm

D_{50} =1.4275 mm D_{10} =0.1078 mm

C_u =40.789 C_c =0.264

Classification

ASTM N/A

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

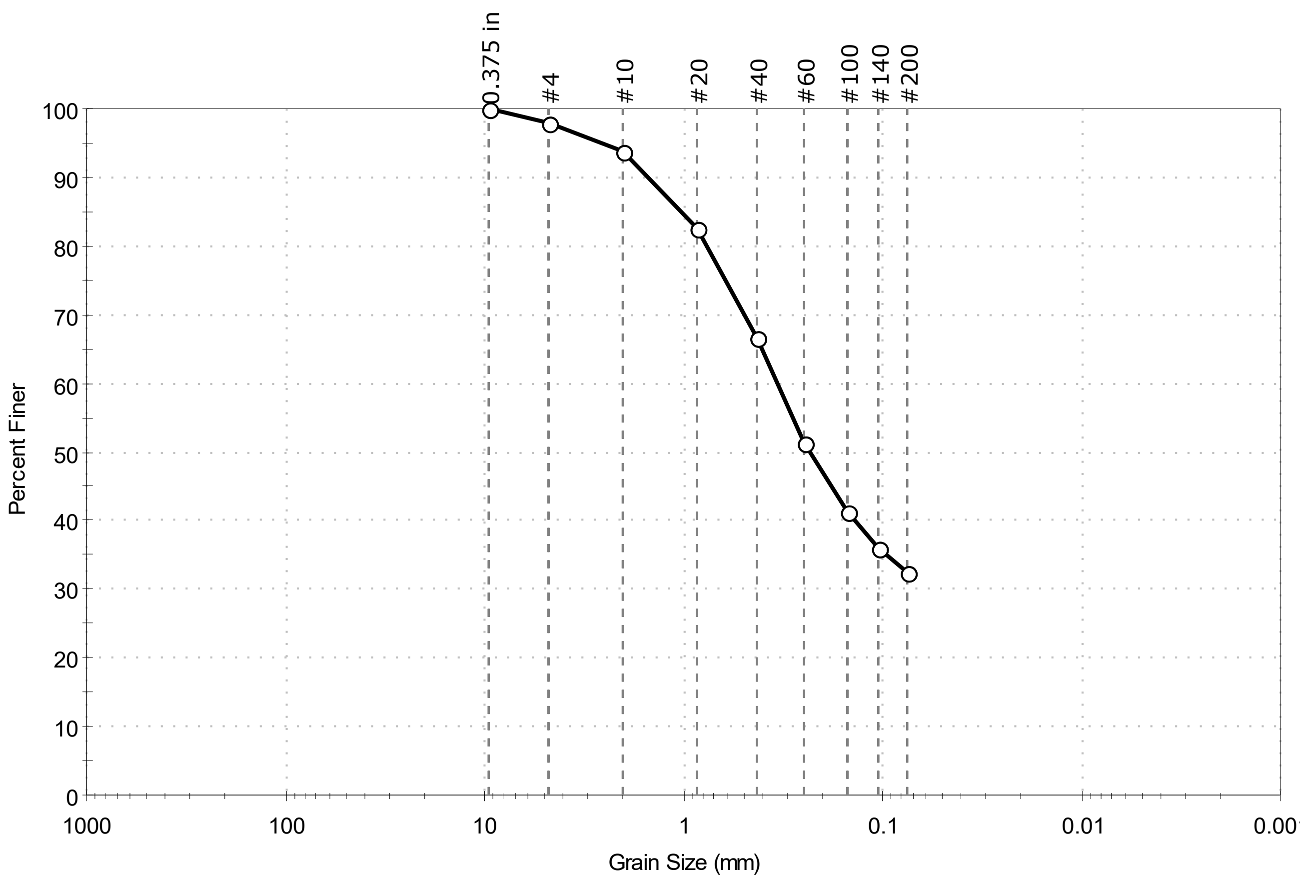
Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Client:	Golder Associates				
Project:	Mallett Dr. Bridge Replace. I-295 Ex 22				
Location:	Freeport, ME			Project No:	GTX-311186
Boring ID:	BB-FMD-101	Sample Type:	jar	Tested By:	GA
Sample ID:	5DB	Test Date:	01/28/20	Checked By:	jsc
Depth :	18-20 ft	Test Id:	540010		
Test Comment:	---				
Visual Description:	Moist, reddish gray clayey sand				
Sample Comment:	---				

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	2.1	65.3	32.6

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.375 in	9.50	100		
#4	4.75	98		
#10	2.00	94		
#20	0.85	82		
#40	0.42	67		
#60	0.25	51		
#100	0.15	41		
#140	0.11	36		
#200	0.075	33		

Coefficients

D_{85} =1.0291 mm D_{30} =N/A
D_{60} =0.3369 mm D_{15} =N/A
D_{50} =0.2330 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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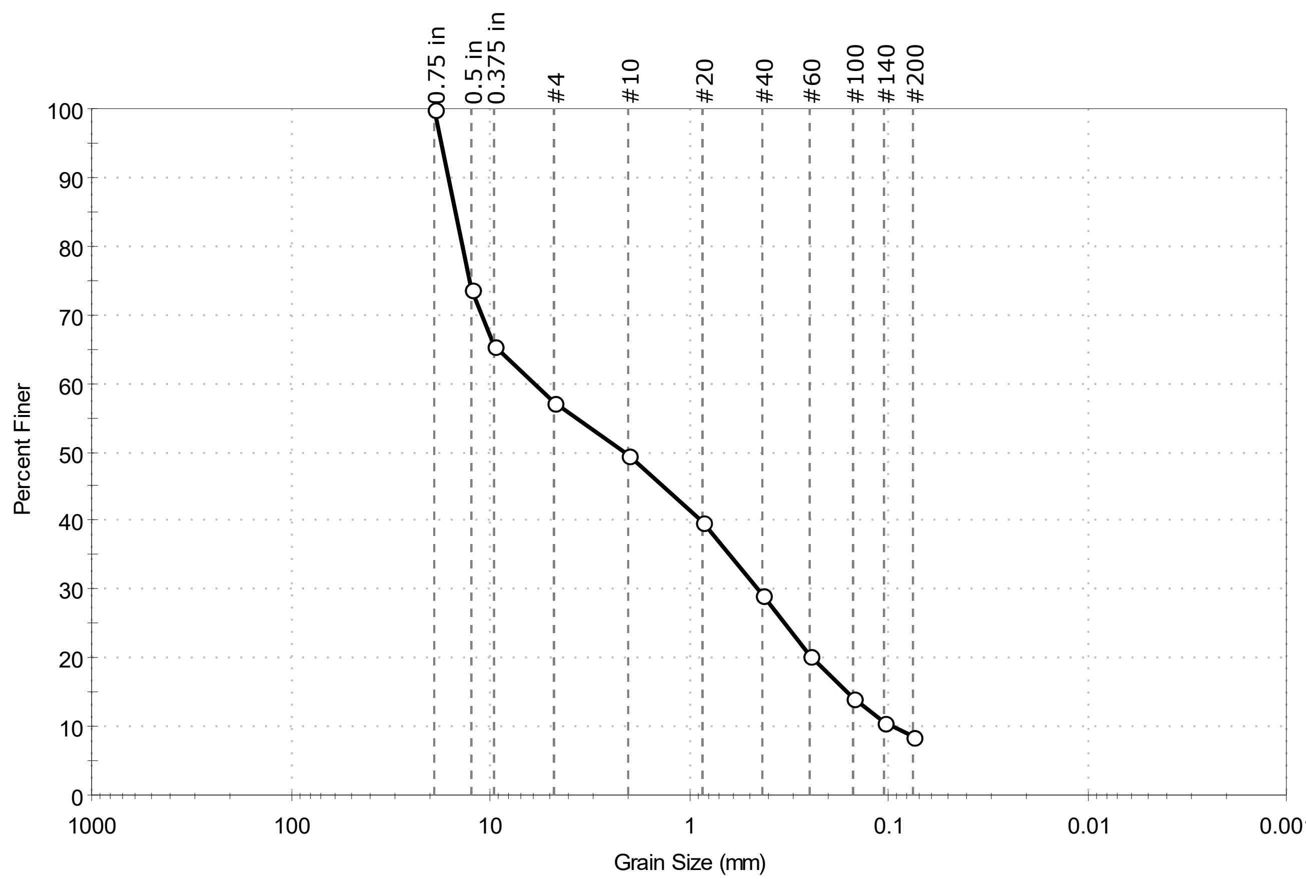
Client: Golder Associates
Project: Mallett Dr. Bridge Replace. I-295 Ex 22
Location: Freeport, ME — Project No: GTX-311186
Boring ID: BB-FMD-101 — Sample Type: jar — Tested By: GA
Sample ID: 6DA — Test Date: 01/28/20 — Checked By: jsc
Depth : 20-22 ft — Test Id: 540009
Test Comment: ---
Visual Description: Moist, reddish gray sand with silt and gravel
Sample Comment: ---

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	42.9	48.5	8.6

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	74		
0.375 in	9.50	66		
#4	4.75	57		
#10	2.00	49		
#20	0.85	40		
#40	0.42	29		
#60	0.25	20		
#100	0.15	14		
#140	0.11	11		
#200	0.075	8.6		

Coefficients

D_{85} =14.9769 mm — D_{30} =0.4439 mm
D_{60} =6.0248 mm — D_{15} =0.1605 mm
D_{50} =2.1176 mm — D_{10} =0.0939 mm
C_u =64.162 — C_c =0.348

Classification

ASTM N/A

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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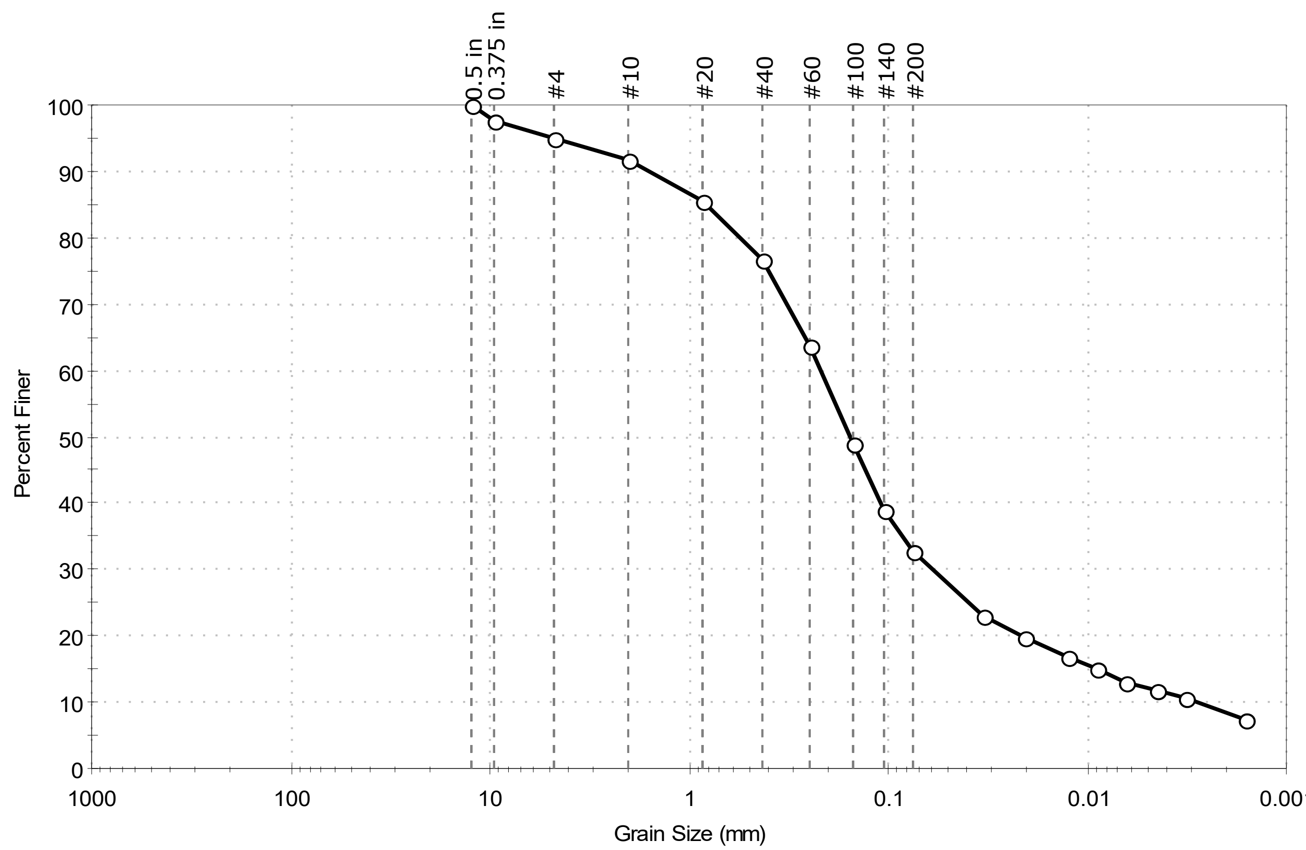
GeoTesting EXPRESS

Client:	Golder Associates				
Project:	Mallett Dr. Bridge Replace. I-295 Ex 22				
Location:	Freeport, ME			Project No:	GTX-311186
Boring ID:	BB-FMD-101	Sample Type:	jar	Tested By:	GA
Sample ID:	7D	Test Date:	01/27/20	Checked By:	jsc
Depth :	24-26 ft	Test Id:	540003		

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

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	4.9	62.3	32.8

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.5 in	12.50	100		
0.375 in	9.50	97		
#4	4.75	95		
#10	2.00	92		
#20	0.85	85		
#40	0.42	77		
#60	0.25	64		
#100	0.15	49		
#140	0.11	39		
#200	0.075	33		
Hydrometer	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0334	23		
---	0.0209	20		
---	0.0124	17		
---	0.0089	15		
---	0.0064	13		
---	0.0045	12		
---	0.0032	11		
---	0.0016	7		

Coefficients

D_{85} =0.8219 mm D_{30} =0.0599 mm
D_{60} =0.2193 mm D_{15} =0.0091 mm
D_{50} =0.1559 mm D_{10} =0.0028 mm
C_u =78.321 C_c =5.843

Classification

ASTM   Silty SAND (SM)

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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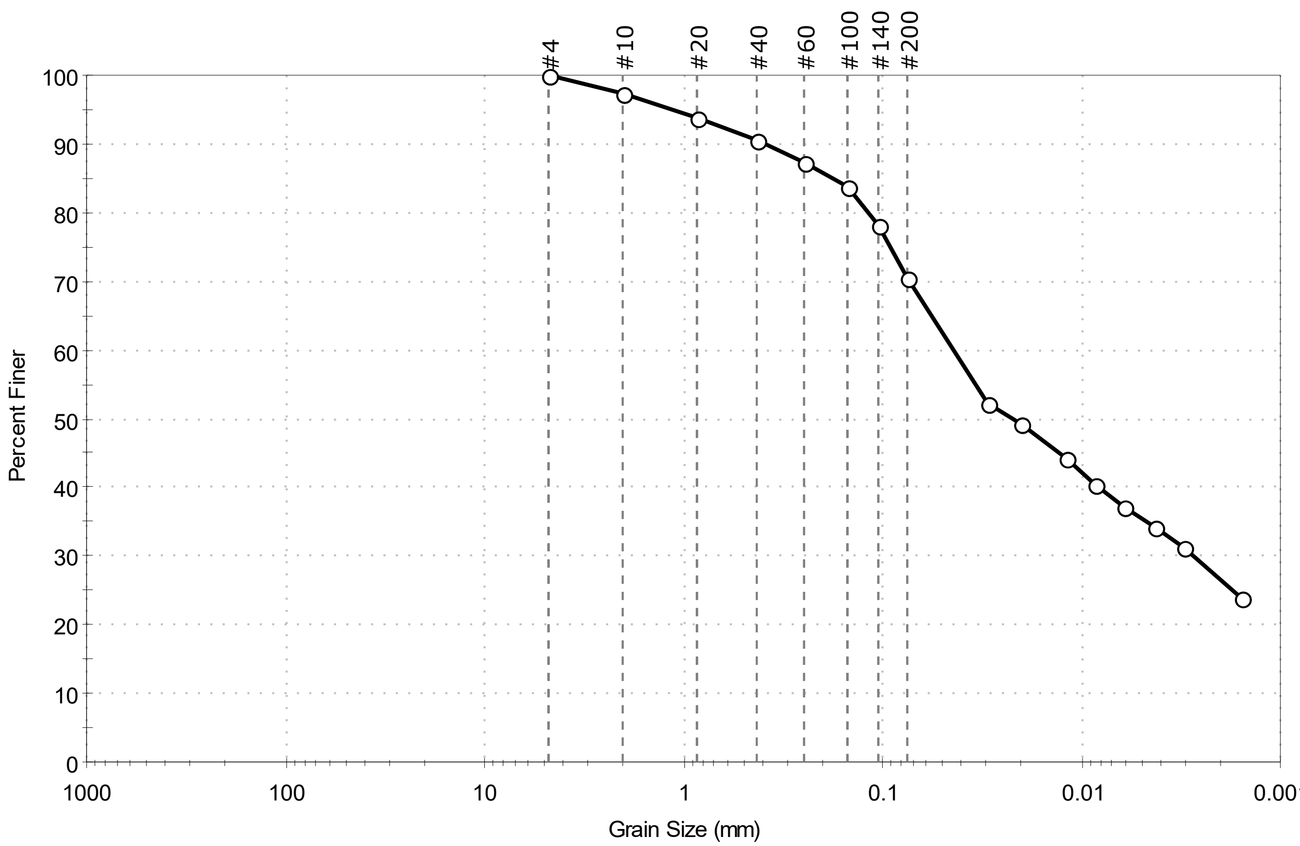
GeoTesting EXPRESS

Client:	Golder Associates			
Project:	Mallett Dr. Bridge Replace. I-295 Ex 22			
Location:	Freeport, ME		Project No:	GTX-311186
Boring ID:	BB-FMD-101	Sample Type: jar	Tested By:	GA
Sample ID:	13D	Test Date: 01/27/20	Checked By:	jsc
Depth :	39-41 ft	Test Id: 540002		
Test Comment:	---			
Visual Description:	Moist, olive gray clay with sand			
Sample Comment:	---			

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	29.6	70.4

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	97		
#20	0.85	94		
#40	0.42	91		
#60	0.25	87		
#100	0.15	84		
#140	0.11	78		
#200	0.075	70		
Hydrometer	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0299	52		
---	0.0202	49		
---	0.0118	44		
---	0.0085	40		
---	0.0061	37		
---	0.0043	34		
---	0.0031	31		
---	0.0016	24		

Coefficients

D_{85} = 0.1823 mm　　D_{30} = 0.0027 mm

D_{60} = 0.0442 mm　　D_{15} = N/A

D_{50} = 0.0221 mm　　D_{10} = N/A

C_u = N/A　　C_c = N/A

Classification

ASTM　Lean CLAY with Sand (CL)

AASHTO　Silty Soils (A-4 (3))

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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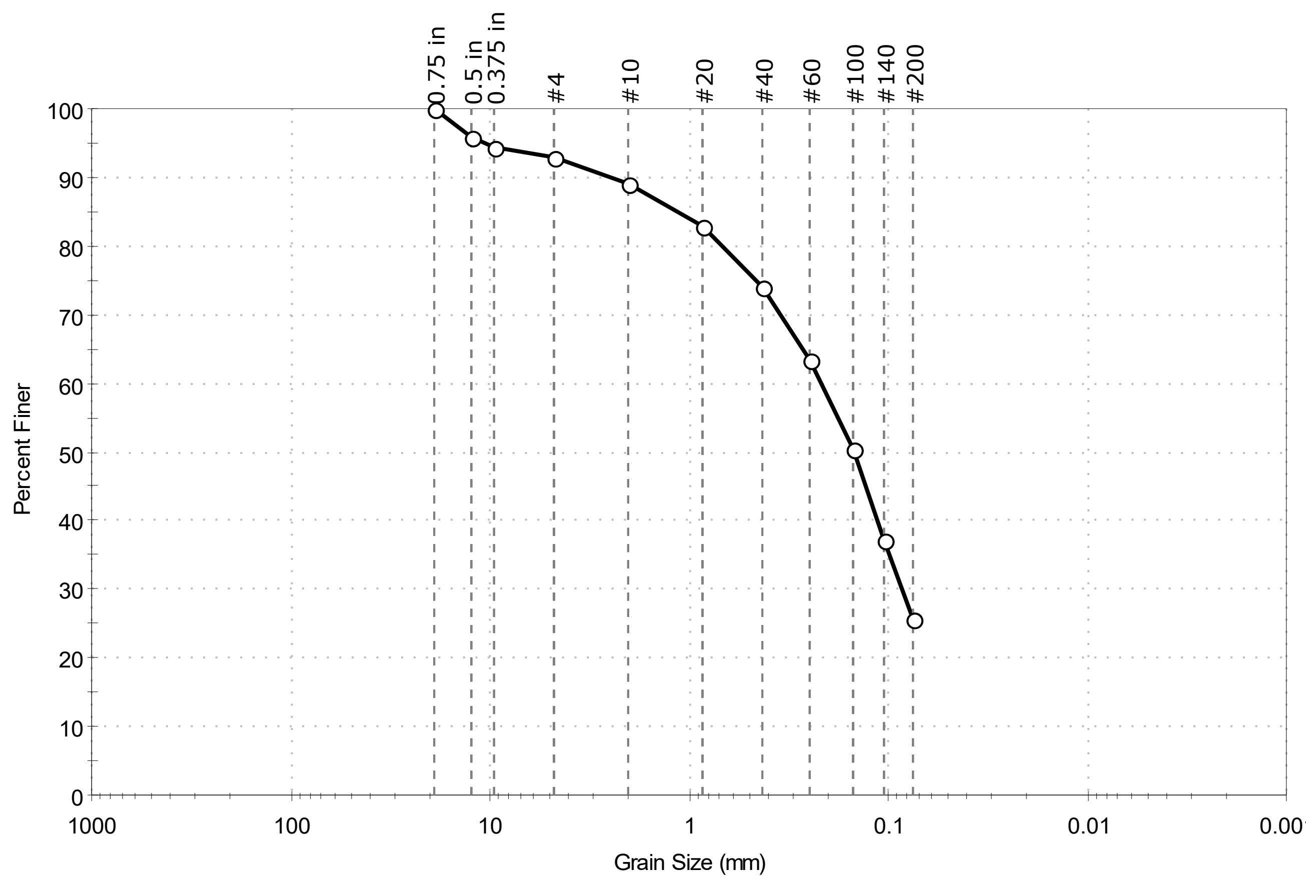
Client:	Golder Associates		
Project:	Mallett Dr. Bridge Replace. I-295 Ex 22		
Location:	Freeport, ME	Project No:	GTX-311186
Boring ID:	BB-FMD-101	Sample Type: jar	Tested By: GA
Sample ID:	14D	Test Date: 01/28/20	Checked By: jsc
Depth :	41-43 ft	Test Id: 540008	
Test Comment:	---		
Visual Description:	Moist, yellowish brown silty sand		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	7.1	67.3	25.6

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	95		
#4	4.75	93		
#10	2.00	89		
#20	0.85	83		
#40	0.42	74		
#60	0.25	63		
#100	0.15	50		
#140	0.11	37		
#200	0.075	26		

Coefficients

D_{85} =1.1455 mm | D_{30} =0.0857 mm
D_{60} =0.2190 mm | D_{15} =N/A
D_{50} =0.1488 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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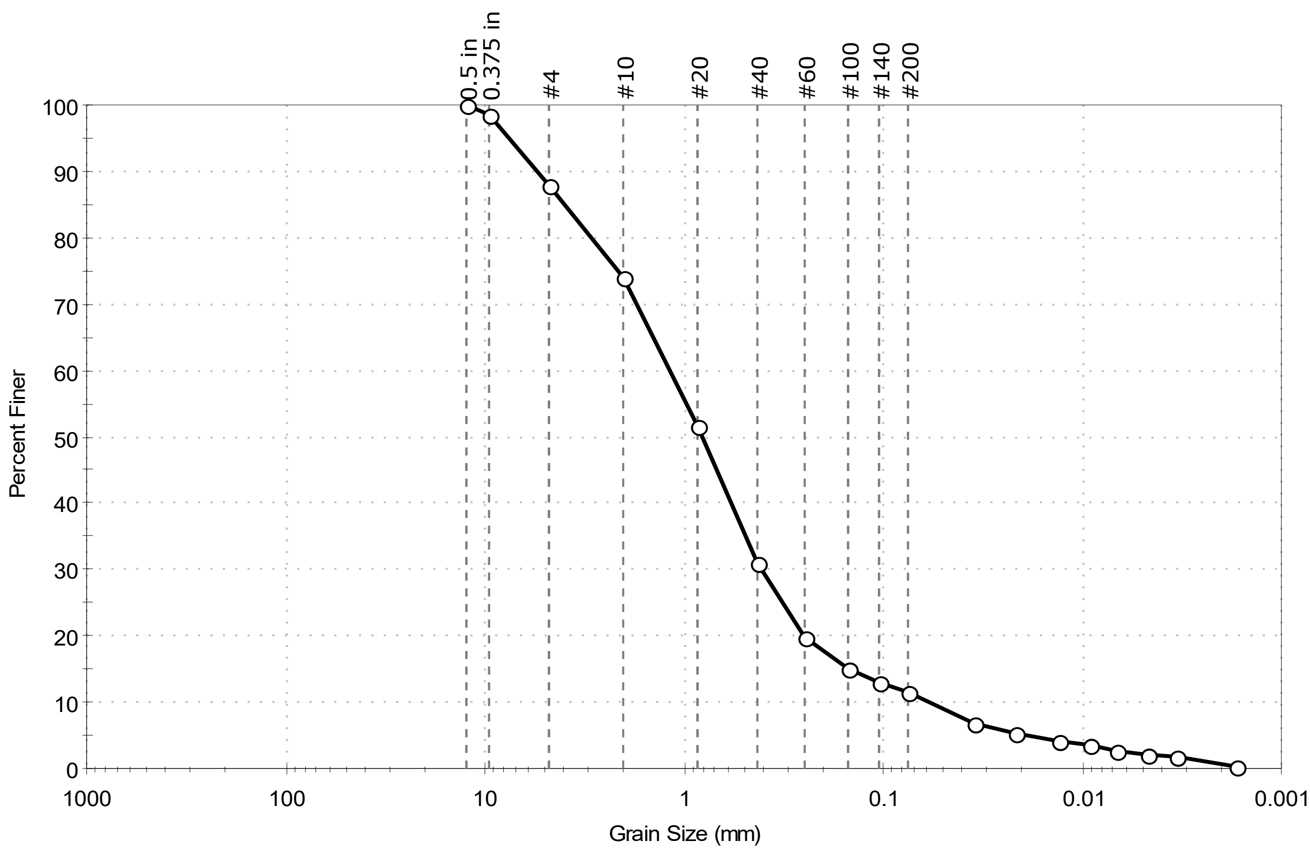
GeoTesting EXPRESS

Client:	Golder Associates		
Project:	Mallett Dr. Bridge Replace. I-295 Ex 22		
Location:	Freeport, ME	Project No:	GTX-311186
Boring ID: BB-FMD-103	Sample Type: jar	Tested By:	GA
Sample ID: 4DB	Test Date: 01/24/20	Checked By:	jsc
Depth : 14-16 ft	Test Id: 540004		
Test Comment:	---		
Visual Description:	Moist, brown sand with silt		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	12.1	76.5	11.4

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	88		
#10	2.00	74		
#20	0.85	52		
#40	0.42	31		
#60	0.25	20		
#100	0.15	15		
#140	0.11	13		
#200	0.075	11		
Hydrometer	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0346	7		
---	0.0218	5		
---	0.0132	4		
---	0.0093	3		
---	0.0067	3		
---	0.0047	2		
---	0.0033	2		
---	0.0017	0		

Coefficients

D_{85} =3.9604 mm
D_{30} =0.4065 mm
D_{60} =1.1642 mm
D_{15} =0.1471 mm
D_{50} =0.8018 mm
D_{10} =0.0596 mm
C_u =19.534
C_c =2.381

Classification

ASTM N/A

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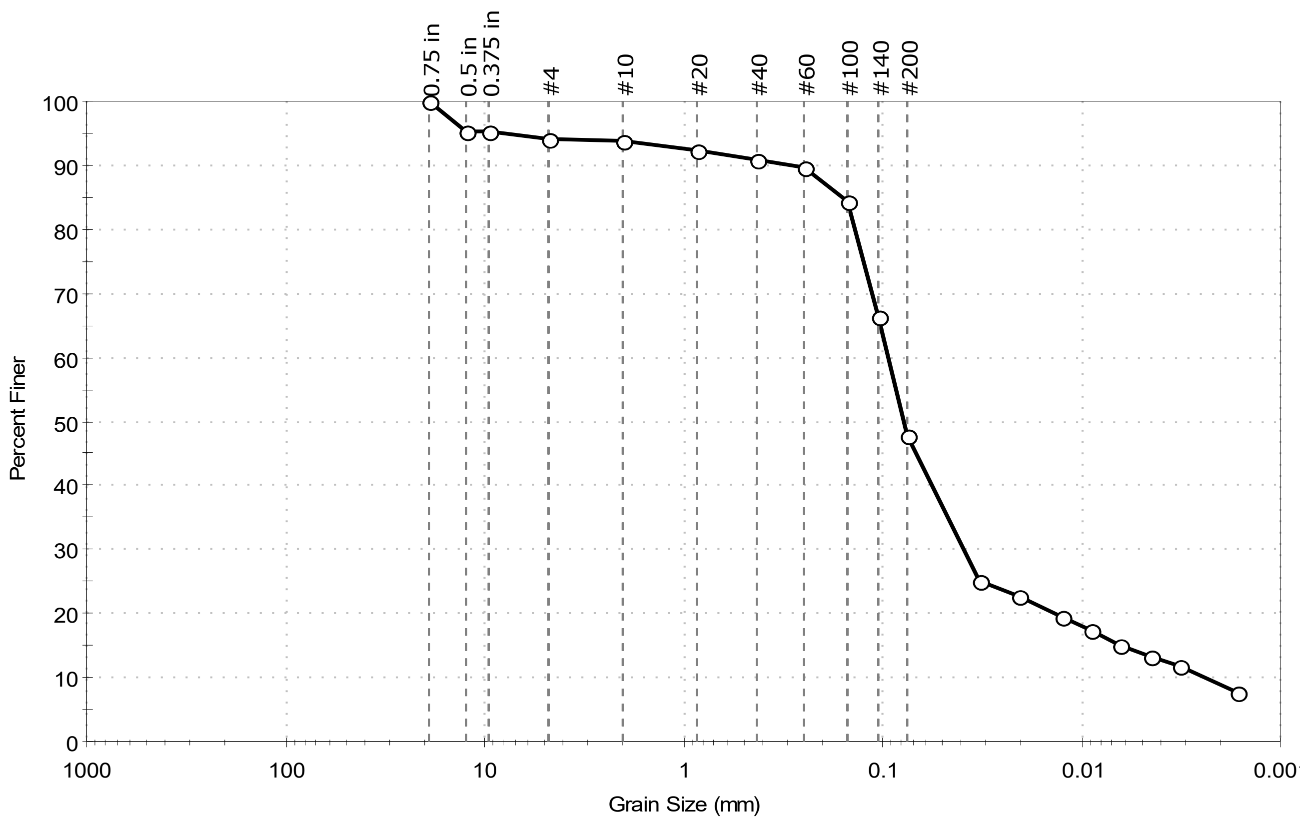
GeoTesting EXPRESS

Client: Golder Associates			
Project: Mallett Dr. Bridge Replace. I-295 Ex 22			
Location: Freeport, ME		Project No:	GTX-311186
Boring ID: BB-FMD-104	Sample Type: jar	Tested By:	GA
Sample ID: 2D	Test Date: 01/27/20	Checked By:	jsc
Depth : 4-6 ft	Test Id: 540005		
Test Comment: ---			
Visual Description: Moist, olive silty sand			
Sample Comment: ---			

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	5.9	46.4	47.7

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	95		
0.375 in	9.50	95		
#4	4.75	94		
#10	2.00	94		
#20	0.85	92		
#40	0.42	91		
#60	0.25	90		
#100	0.15	84		
#140	0.11	66		
#200	0.075	48		
Hydrometer	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0325	25		
---	0.0208	23		
---	0.0125	20		
---	0.0089	17		
---	0.0064	15		
---	0.0045	13		
---	0.0032	12		
---	0.0016	8		

Coefficients

D_{85} =0.1606 mm D_{30} =0.0391 mm

D_{60} =0.0943 mm D_{15} =0.0065 mm

D_{50} =0.0783 mm D_{10} =0.0024 mm

C_u =39.292 C_c =6.755

Classification

ASTM Silty SAND (SM)

AASHTO Silty Soils (A-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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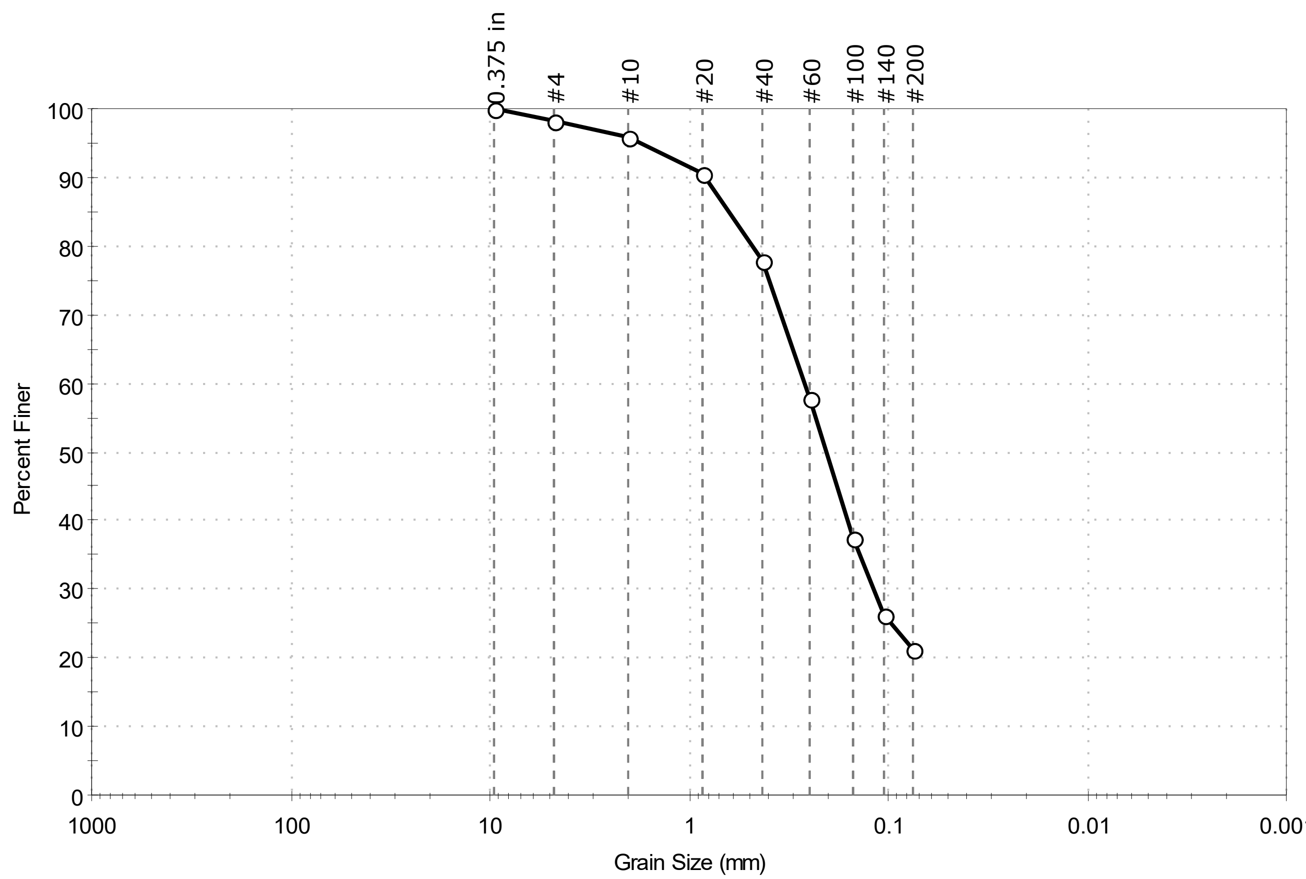
GeoTesting EXPRESS

Client:	Golder Associates				
Project:	Mallett Dr. Bridge Replace. I-295 Ex 22				
Location:	Freeport, ME			Project No:	GTX-311186
Boring ID:	BB-FMD-105	Sample Type:	jar	Tested By:	GA
Sample ID:	2D	Test Date:	01/28/20	Checked By:	jsc
Depth :	3-5 ft	Test Id:	540012		
Test Comment:	---				
Visual Description:	Moist, brown silty sand				
Sample Comment:	---				

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	1.7	77.1	21.2

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.375 in	9.50	100		
#4	4.75	98		
#10	2.00	96		
#20	0.85	90		
#40	0.42	78		
#60	0.25	58		
#100	0.15	37		
#140	0.11	26		
#200	0.075	21		

Coefficients

D_{85} =0.6269 mm D_{30} =0.1191 mm

D_{60} =0.2643 mm D_{15} =N/A

D_{50} =0.2055 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 : ---

Sand/Gravel Hardness : ---

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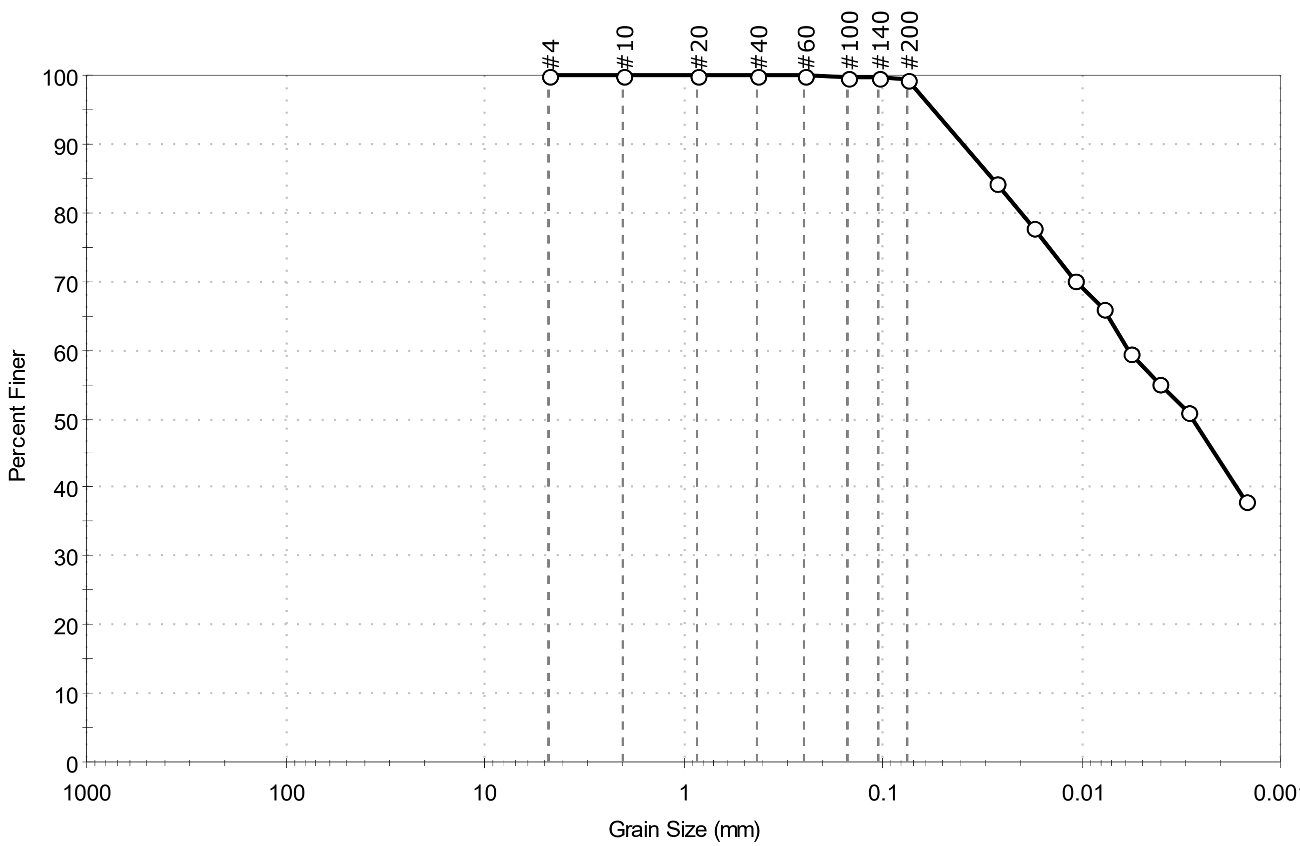
Client:	Golder Associates		
Project:	Mallett Dr. Bridge Replace. I-295 Ex 22		
Location:	Freeport, ME	Project No:	GTX-311186
Boring ID: BB-FMD-106	Sample Type: jar	Tested By:	GA
Sample ID: 2D	Test Date: 01/27/20	Checked By:	jsc
Depth : 4-6 ft	Test Id: 540006		
Test Comment:	---		
Visual Description:	Moist, olive gray clay		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	0.5	99.5

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	100		
#140	0.11	100		
#200	0.075	99		
Hydrometer	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0272	84		
---	0.0176	78		
---	0.0110	70		
---	0.0078	66		
---	0.0057	60		
---	0.0041	55		
---	0.0029	51		
---	0.0015	38		

Coefficients

D_{85} =0.0285 mm D_{30} =N/A
D_{60} =0.0058 mm D_{15} =N/A
D_{50} =0.0028 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 : ---
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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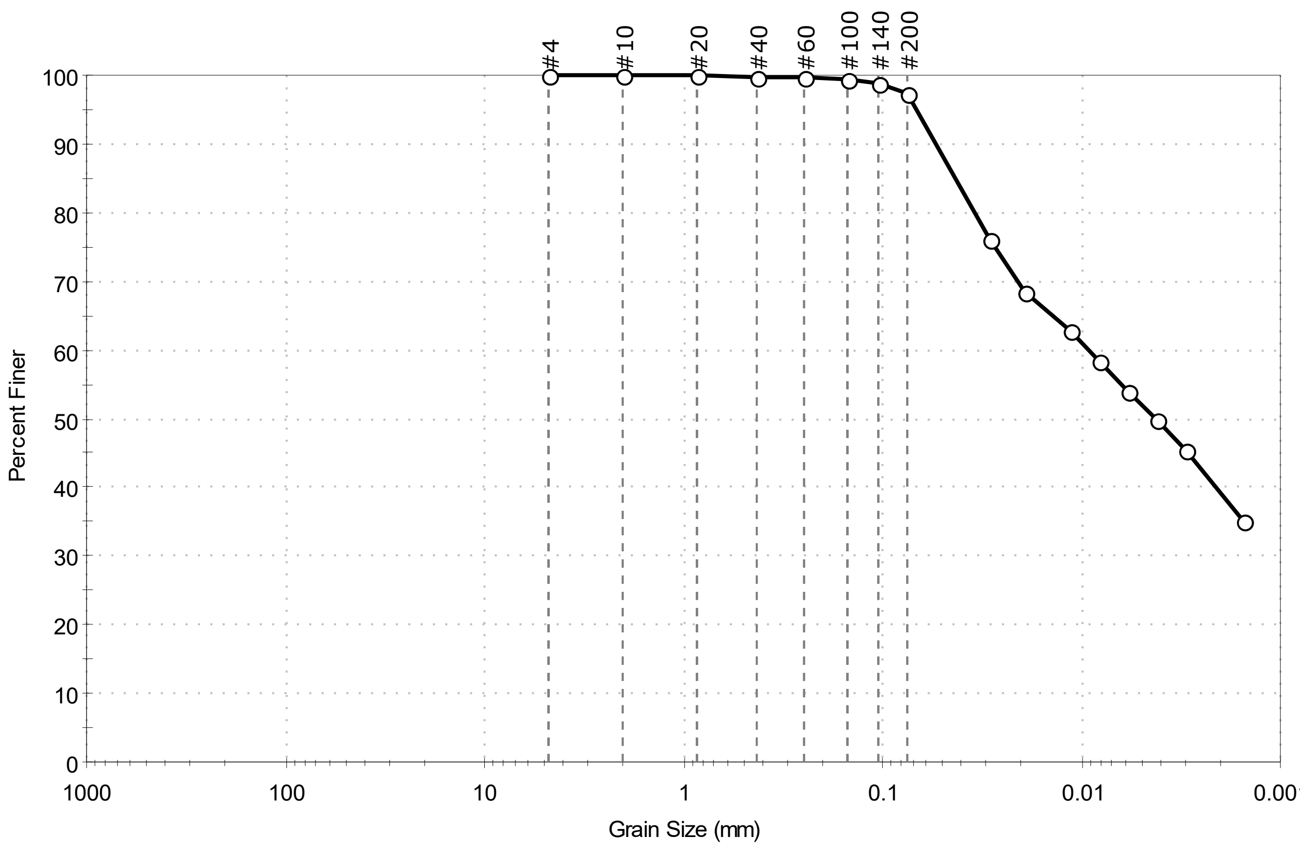
GeoTesting EXPRESS

Client:	Golder Associates				
Project:	Mallett Dr. Bridge Replace. I-295 Ex 22				
Location:	Freeport, ME			Project No:	GTX-311186
Boring ID:	BB-FMD-106	Sample Type:	jar	Tested By:	GA
Sample ID:	4D	Test Date:	01/27/20	Checked By:	jsc
Depth :	8-10 ft	Test Id:	540007		
Test Comment:	---				
Visual Description:	Moist, olive gray clay				
Sample Comment:	---				

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	2.6	97.4

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	99		
#140	0.11	99		
#200	0.075	97		
Hydrometer	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0290	76		
---	0.0191	68		
---	0.0113	63		
---	0.0081	59		
---	0.0058	54		
---	0.0042	50		
---	0.0030	45		
---	0.0015	35		

Coefficients

D_{85} =0.0432 mm D_{30} =N/A

D_{60} =0.0091 mm D_{15} =N/A

D_{50} =0.0043 mm D_{10} =N/A

C_u =N/A C_c =N/A

Classification

ASTM Lean CLAY (CL)

AASHTO Clayey Soils (A-7-6 (26))

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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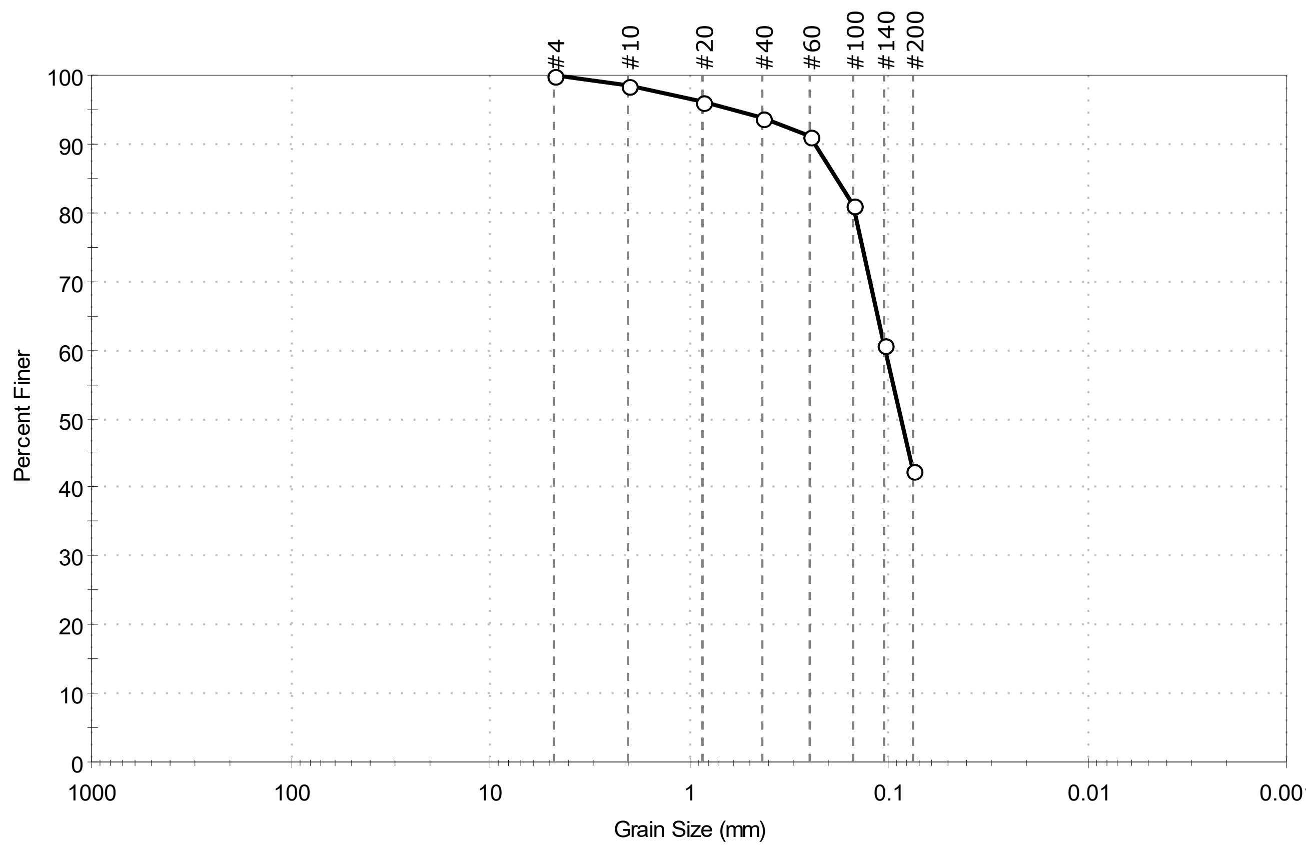
Client:	Golder Associates		
Project:	Mallett Dr. Bridge Replace. I-295 Ex 22		
Location:	Freeport, ME	Project No:	GTX-311186

Boring ID:	BB-FMD-106	Sample Type:	jar	Tested By:	GA
Sample ID:	7DB	Test Date:	01/28/20	Checked By:	jsc
Depth :	14-15.8 ft	Test Id:	540013		

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

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	57.4	42.6

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	98		
#20	0.85	96		
#40	0.42	94		
#60	0.25	91		
#100	0.15	81		
#140	0.11	61		
#200	0.075	43		

Coefficients

D_{85} =0.1838 mm | D_{30} =N/A
D_{60} =0.1046 mm | D_{15} =N/A
D_{50} =0.0864 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 : ---

Sand/Gravel Hardness : ---

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ATTERBERG LIMITS

Client:	Golder Associates				
Project:	Mallett Dr. Bridge Replace. I-295 Ex 22				
Location:	Freeport, ME			Project No:	GTX-311186
Boring ID:	BB-FMD-101	Sample Type:	jar	Tested By:	GA
Sample ID:	7D	Test Date:	01/24/20	Checked By:	jsc
Depth :	24-26 ft	Test Id:	539997		

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

Atterberg Limits - ASTM D4318

Sample Determined to be non-plastic

Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	7D	B-FMD-10	24-26 ft	14	n/a	n/a	n/a	n/a	Silty SAND (SM)

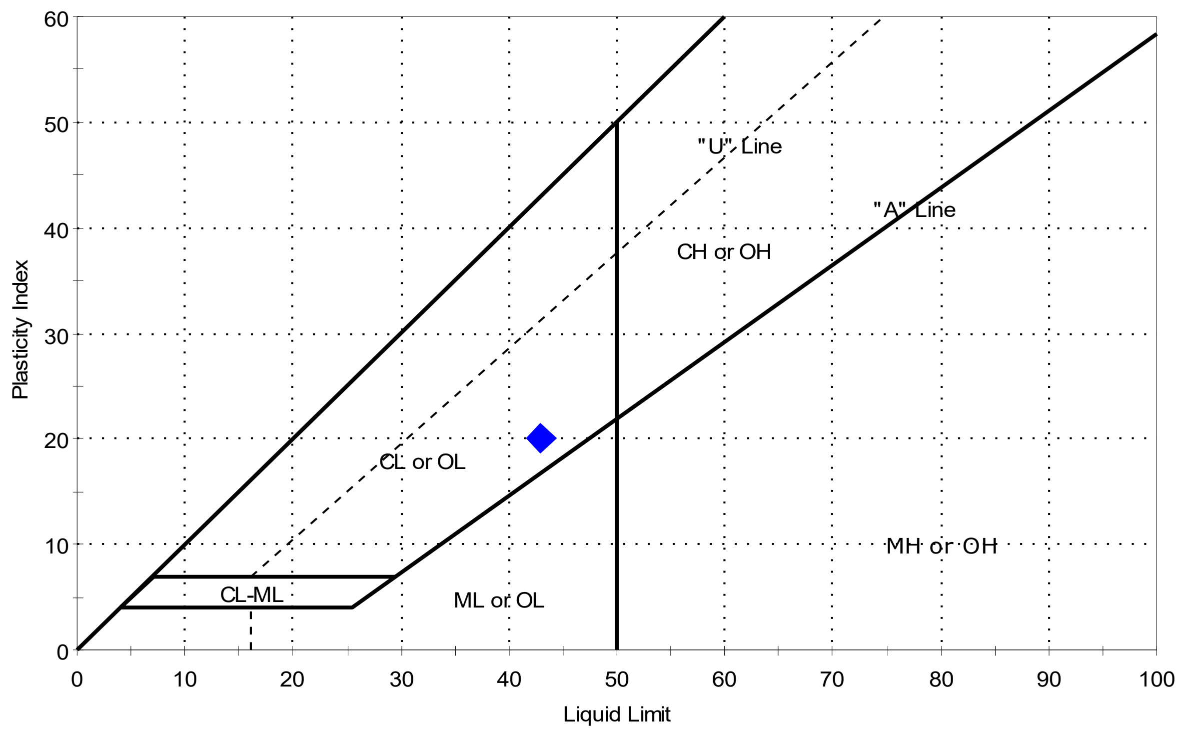
23% Retained on #40 Sieve
Dry Strength: NONE
Dilatancy: RAPID
Toughness: n/a
The sample was determined to be Non-Plastic

GeoTesting EXPRESS

Client: Golder Associates
Project: Mallett Dr. Bridge Replace. I-295 Ex 22
Location: Freeport, ME
Project No: GTX-311186
Boring ID: BB-FMD-101
Sample Type: jar
Tested By: GA
Sample ID: 12D
Test Date: 01/24/20
Checked By: jsc
Depth : 37-39 ft
Test Id: 539996
Test Comment: ---
Visual Description: Moist, olive gray clay
Sample Comment: ---

Atterberg Limits - ASTM D4318

Plasticity Chart



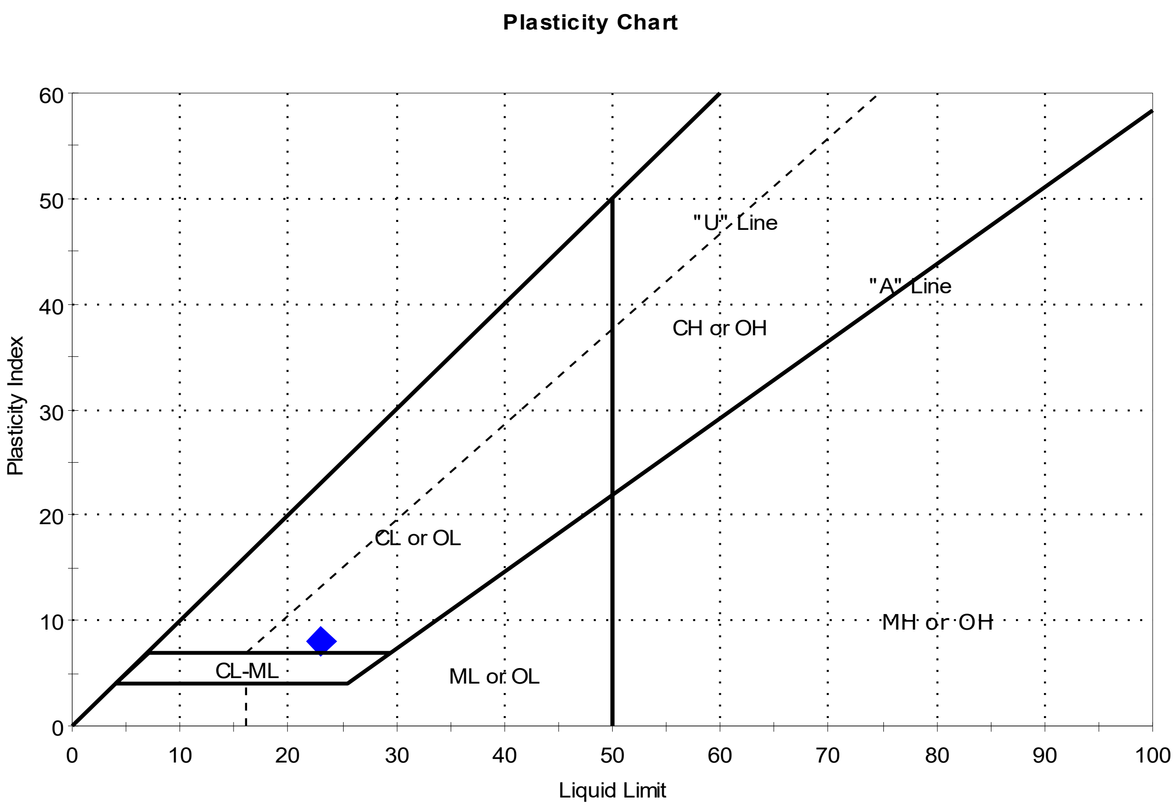
Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	12D	B-FMD-10	37-39 ft	30	43	23	20	0.3	

Sample Prepared using the WET method

Dry Strength: VERY HIGH
Dilatancy: NONE
Toughness: MEDIUM

Client: Golder Associates			
Project: Mallett Dr. Bridge Replace. I-295 Ex 22			
Location: Freeport, ME		Project No:	GTX-311186
Boring ID: BB-FMD-101	Sample Type: jar	Tested By:	GA
Sample ID: 13D	Test Date: 02/03/20	Checked By:	jsc
Depth : 39-41 ft	Test Id: 539995		
Test Comment: ---			
Visual Description: Moist, olive 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
◆	13D	B-FMD-10	39-41 ft	32	23	15	8	2.1	Lean CLAY with Sand (CL)

Sample Prepared using the WET method

9% Retained on #40 Sieve

Dry Strength: MEDIUM

Dilatancy: SLOW

Toughness: MEDIUM

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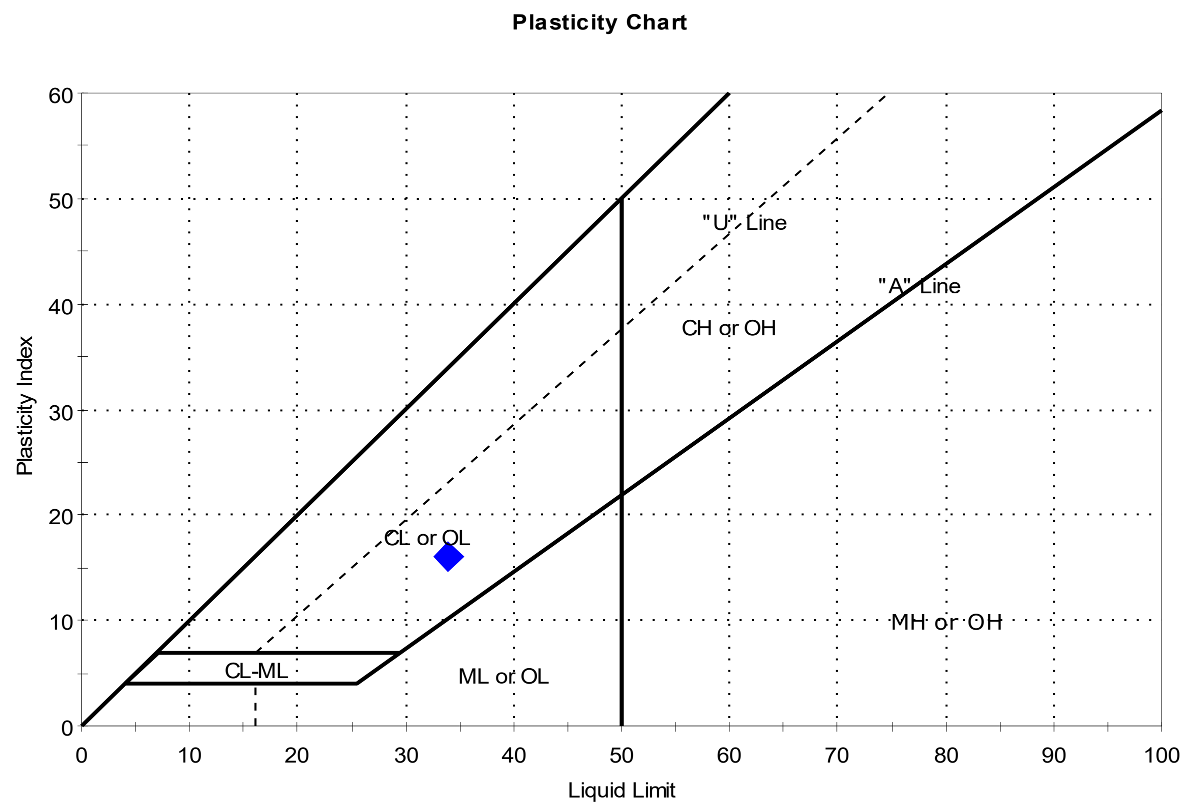
GeoTesting EXPRESS

Client:	Golder Associates				
Project:	Mallett Dr. Bridge Replace. I-295 Ex 22				
Location:	Freeport, ME			Project No:	GTX-311186
Boring ID:	BB-FMD-103	Sample Type:	jar	Tested By:	GA
Sample ID:	4DA	Test Date:	01/24/20	Checked By:	jsc
Depth :	14-16 ft	Test Id:	540001		
Test Comment:	---				
Visual Description:	Moist, olive gray sandy 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
◆	4DA	B-FMD-10	14-16 ft	26	34	18	16	0.5	

Sample Prepared using the WET method

Dry Strength: VERY HIGH

Dilatancy: NONE

Toughness: MEDIUM

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Client:	Golder Associates				
Project:	Mallett Dr. Bridge Replace. I-295 Ex 22				
Location:	Freeport, ME			Project No:	GTX-311186
Boring ID:	BB-FMD-104	Sample Type:	jar	Tested By:	GA
Sample ID:	2D	Test Date:	01/31/20	Checked By:	jsc
Depth :	4-6 ft	Test Id:	539998		

Test Comment: ---

Visual Description: Moist, olive silty sand

Sample Comment: ---

Atterberg Limits - ASTM D4318

Sample Determined to be non-plastic

Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	2D	B-FMD-10	4-6 ft	22	n/a	n/a	n/a	n/a	Silty SAND (SM)

9% Retained on #40 Sieve

Dry Strength: NONE

Dilatancy: RAPID

Toughness: n/a

The sample was determined to be Non-Plastic

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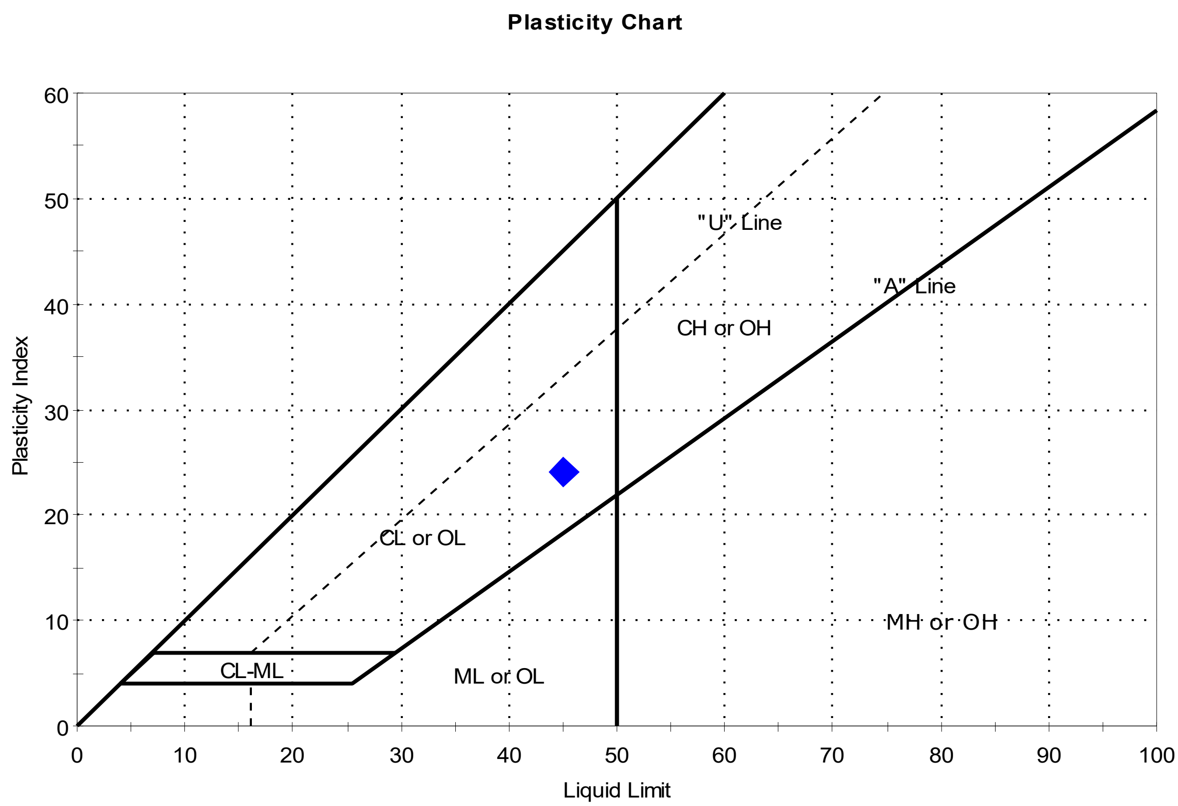
Client:	Golder Associates		
Project:	Mallett Dr. Bridge Replace. I-295 Ex 22		
Location:	Freeport, ME	Project No:	GTX-311186
Boring ID: BB-FMD-106	Sample Type: jar	Tested By:	GA
Sample ID: 4D	Test Date: 01/24/20	Checked By:	jsc
Depth : 8-10 ft	Test Id: 539999		
Test Comment:	---		
Visual Description:	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
◆	4D	B-FMD-10	8-10 ft	31	45	21	24	0.4	Lean CLAY (CL)

Sample Prepared using the WET method

0% Retained on #40 Sieve

Dry Strength: VERY HIGH

Dilatancy: NONE

Toughness: MEDIUM

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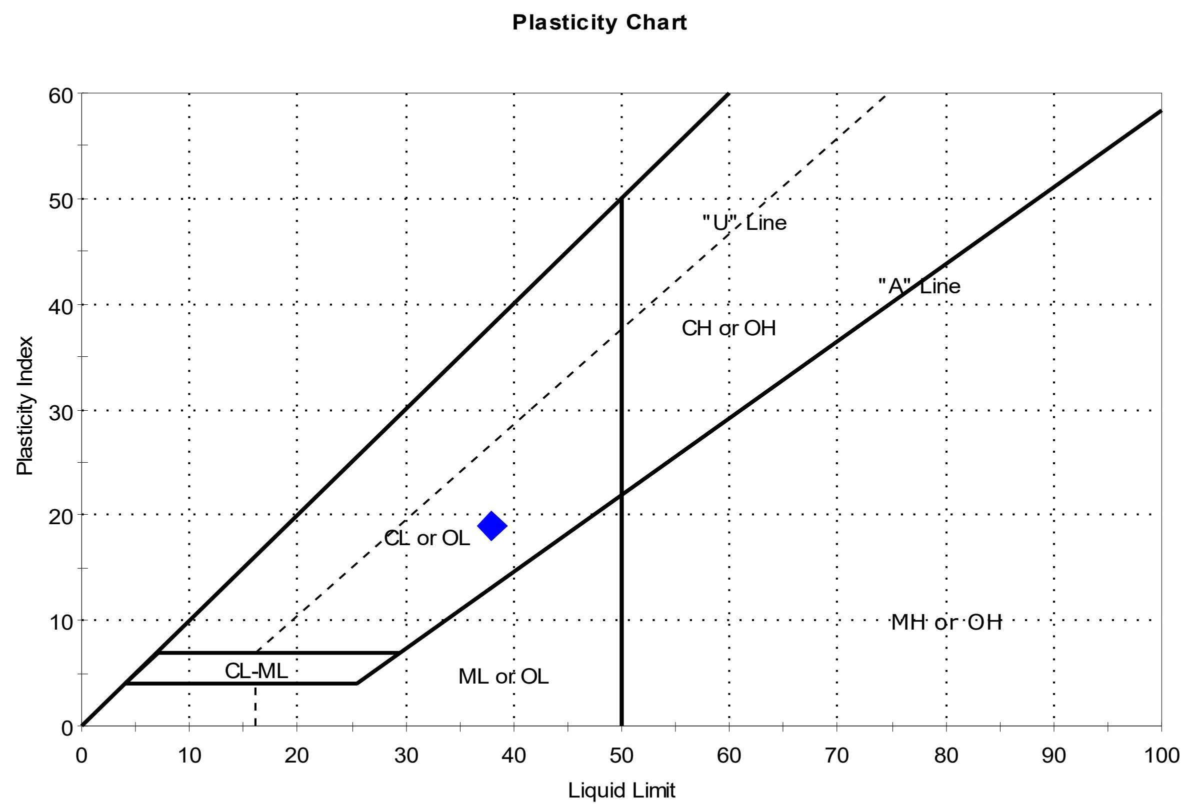
Client:	Golder Associates				
Project:	Mallett Dr. Bridge Replace. I-295 Ex 22				
Location:	Freeport, ME			Project No:	GTX-311186
Boring ID:	BB-FMD-106	Sample Type:	jar	Tested By:	GA
Sample ID:	6D	Test Date:	01/27/20	Checked By:	jsc
Depth :	12-14 ft	Test Id:	540000		
Test Comment:	---				
Visual Description:	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
◆	6D	B-FMD-10	12-14 ft	33	38	19	19	0.7	

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 Specimens

Compressive Strength and Elastic Moduli of Rock

Bulk Density and Compressive Strength of Rock Core Specimens

Client:	Golder Associates		
Project:	Mallett Dr. Bridge Replace. I-295 Ex 22		
Location:	Freeport, ME	Project No:	GTX-311186
Boring ID: ---	Sample Type: ---	Tested By:	tlm
Sample ID: ---	Test Date: 01/17/20	Checked By:	smd
Depth : ---	Test Id: 540024		

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-FMD-101	RUN 2	50.26 - 50.63 ft	169	2486	2	Yes	---
BB-FMD-104	RUN 1	6.84 - 7.21 ft	171	14774	3	Yes	---
BB-FMD-106	RUN 1	26.11 - 26.48 ft	162	22769	1	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:	Mallett Dr. Bridge Replace I-295 EX 22	Tested By:	jck
Project Location:	Freeport, ME	Checked By:	smd
GTX #:	311186		
Boring ID:	BB-FMD-101		
Sample ID:	RUN 2		
Depth:	50.26-50.63 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:	2.00	1.99	2.00	
Specimen Mass, g:	610.12			
Bulk Density, lb/ft^3	169	**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	-0.00080	-0.00070	-0.00050	-0.00050	-0.00040	-0.00030	-0.00020	0.00000	0.00010	0.00020	0.00030	0.00040	0.00050	0.00060	0.00080
Diameter 2, in (rotated 90°)	-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	0.00000

Difference between max and min readings, in:

0° = 0.00160 90° = 0.00030

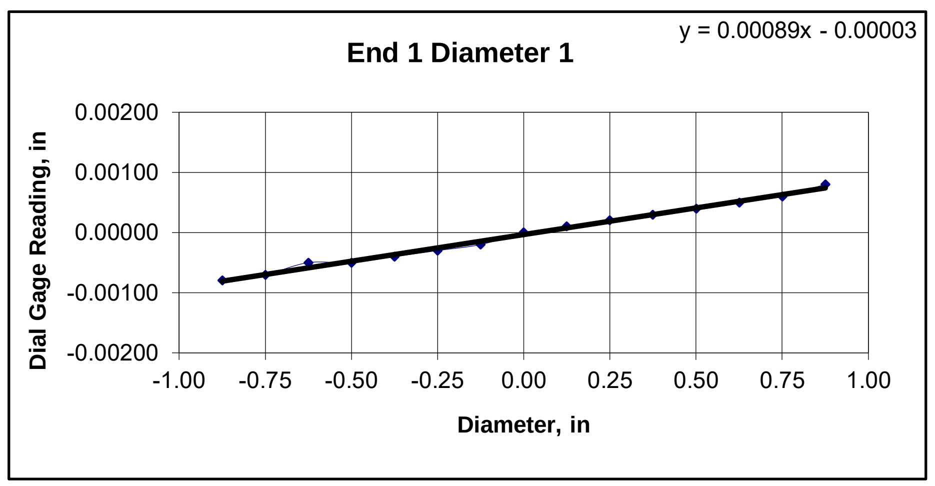
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00070	-0.00080	-0.00060	-0.00040	-0.00040	-0.00030	-0.00020	0.00000	0.00010	0.00020	0.00030	0.00050	0.00050	0.00060	0.00070
Diameter 2, in (rotated 90°)	-0.00030	-0.00020	-0.00010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

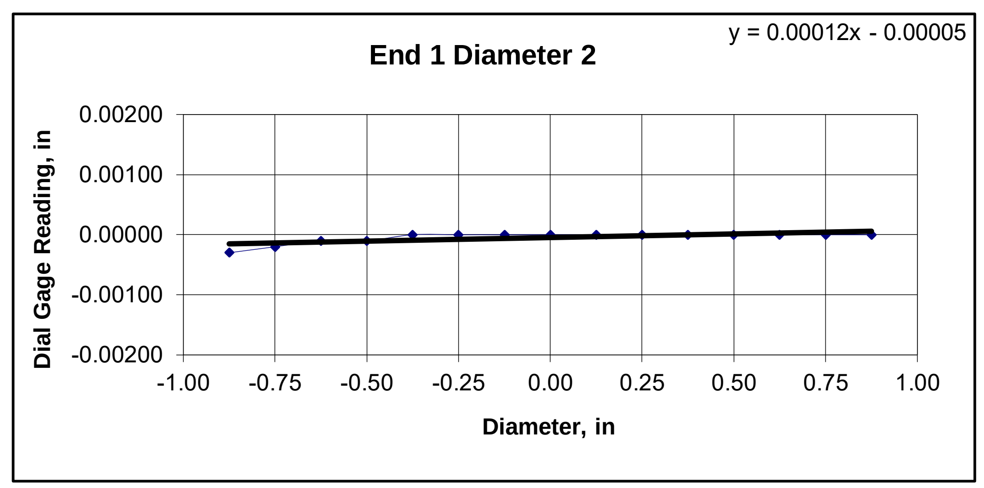
Difference between max and min readings, in:

0° = 0.0015 90° = 0.0003

Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00080

Flatness Tolerance Met? **YES**





DIAMETER 1

End 1:

Slope of Best Fit Line	0.00089
Angle of Best Fit Line:	0.05075

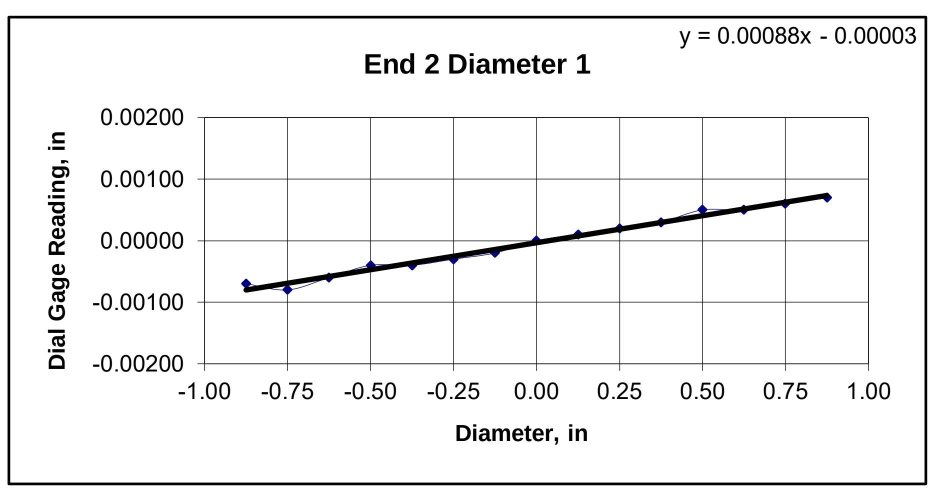
End 2:

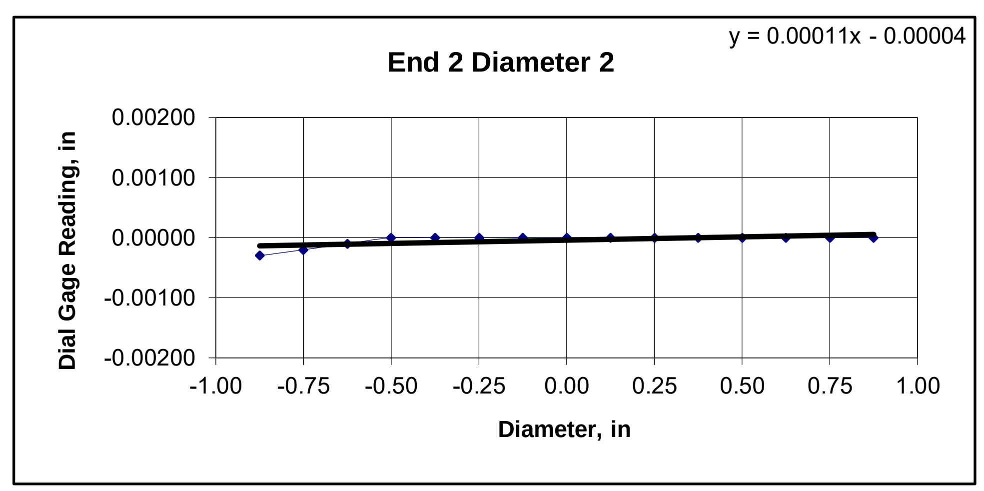
Slope of Best Fit Line	0.00088
Angle of Best Fit Line:	0.05026

Maximum Angular Difference: 0.00049

Parallelism Tolerance Met? **YES**

Spherically Seated





DIAMETER 2

End 1:

Slope of Best Fit Line	0.00012
Angle of Best Fit Line:	0.00688

End 2:

Slope of Best Fit Line	0.00011
Angle of Best Fit Line:	0.00622

Maximum Angular Difference: 0.00065

Parallelism Tolerance Met? **YES**

Spherically Seated

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

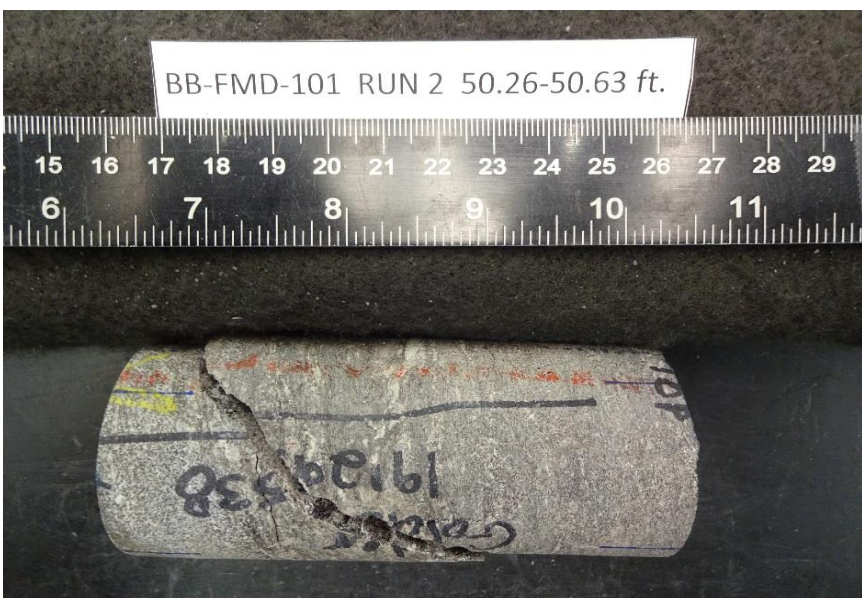
END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00160	1.995	0.00080	0.046	YES
Diameter 2, in (rotated 90°)	0.00030	1.995	0.00015	0.009	YES
END 2					
Diameter 1, in	0.00150	1.995	0.00075	0.043	YES
Diameter 2, in (rotated 90°)	0.00030	1.995	0.00015	0.009	YES

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

Perpendicularity Tolerance Met? **YES**

Client:	Golder Associates
Project Name:	Mallett Dr. Bridge Replace I-295 EX 22
Project Location:	Freeport, ME
GTX #:	311186
Test Date:	1/17/2020
Tested By:	jck
Checked By:	smd
Boring ID:	BB-FMD-101
Sample ID:	RUN 2
Depth, ft:	50.26-50.63

After cutting and grinding



After break

GeoTesting EXPRESS

Client:	Golder Associates	Test Date:	1/17/2020
Project Name:	Mallett Dr. Bridge Replace I-295 EX 22	Tested By:	jck
Project Location:	Freeport, ME	Checked By:	smd
GTX #:	311186		
Boring ID:	BB-FMD-104		
Sample ID:	RUN 1		
Depth:	6.84-7.21 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:	4.33	4.32	4.33
Specimen Diameter, in:	1.99	1.99	1.99
Specimen Mass, g:	604.21		
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 $\leq$ 0.02 in.? YES

Maximum difference must be < 0.020 in.
Straightness Tolerance Met? YES

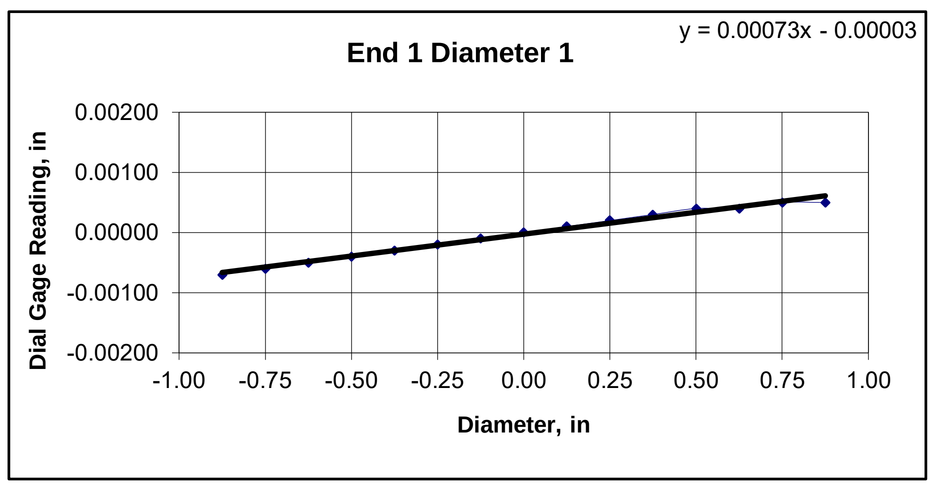
END FLATNESS AND PARALLELISM (Procedure FP1)

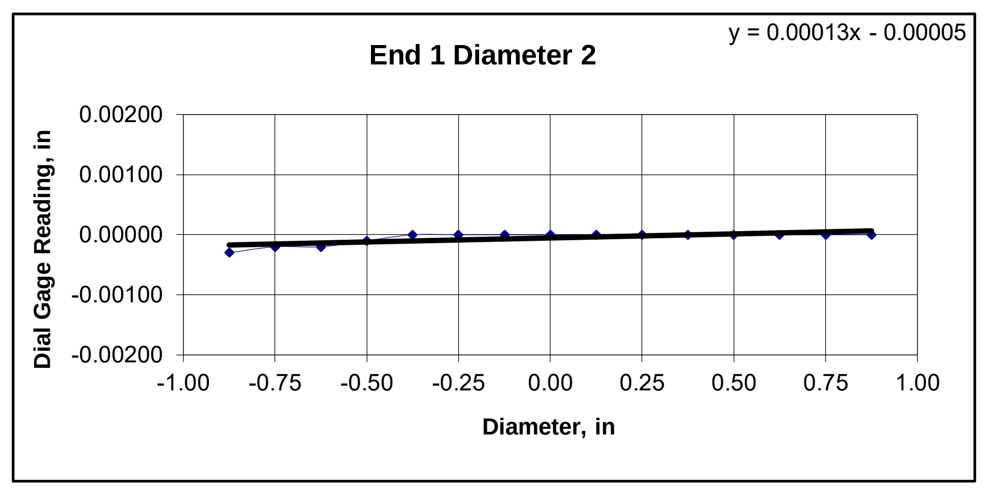
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00070	-0.00060	-0.00050	-0.00040	-0.00030	-0.00020	-0.00010	0.00000	0.00010	0.00020	0.00030	0.00040	0.00040	0.00050	0.00050
Diameter 2, in (rotated 90°)	-0.00030	-0.00020	-0.00020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

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

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	-0.00070	-0.00060	-0.00050	-0.00040	-0.00030	-0.00020	-0.00010	0.00000	0.00010	0.00020	0.00030	0.00030	0.00040	0.00050	0.00050
Diameter 2, in (rotated 90°)	-0.00030	-0.00020	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000

Difference between max and min readings, in:
0° = 0.0012 90° = 0.0003
Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00060
Flatness Tolerance Met? YES





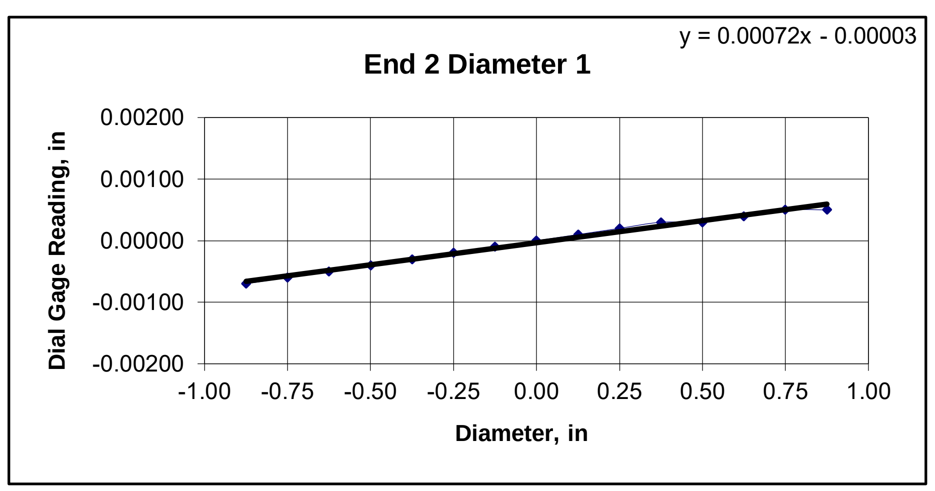
DIAMETER 1

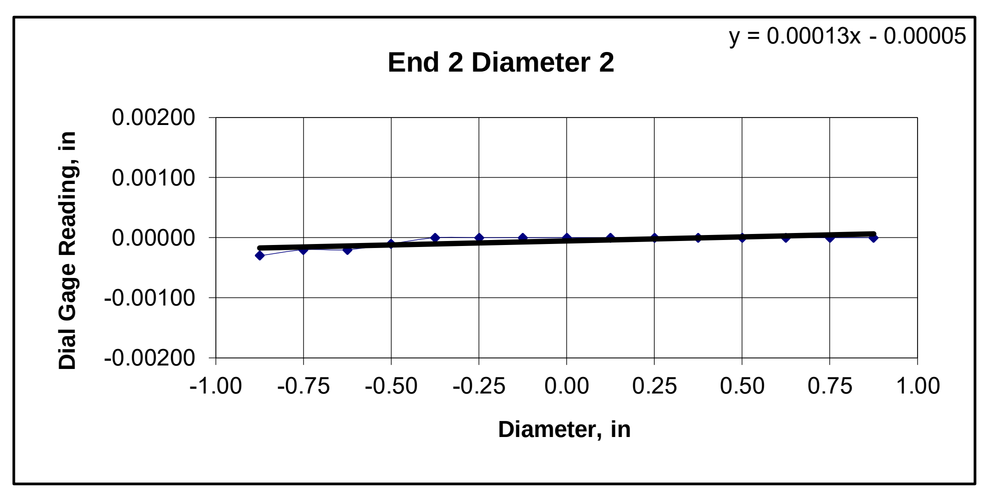
End 1:
Slope of Best Fit Line 0.00073
Angle of Best Fit Line: 0.04174

End 2:
Slope of Best Fit Line 0.00072
Angle of Best Fit Line: 0.04109

Maximum Angular Difference: 0.00065

Parallelism Tolerance Met? YES
Spherically Seated





DIAMETER 2

End 1:
Slope of Best Fit Line 0.00013
Angle of Best Fit Line: 0.00769

End 2:
Slope of Best Fit Line 0.00013
Angle of Best Fit Line: 0.00769

Maximum Angular Difference: 0.00000

Parallelism Tolerance Met? YES
Spherically Seated

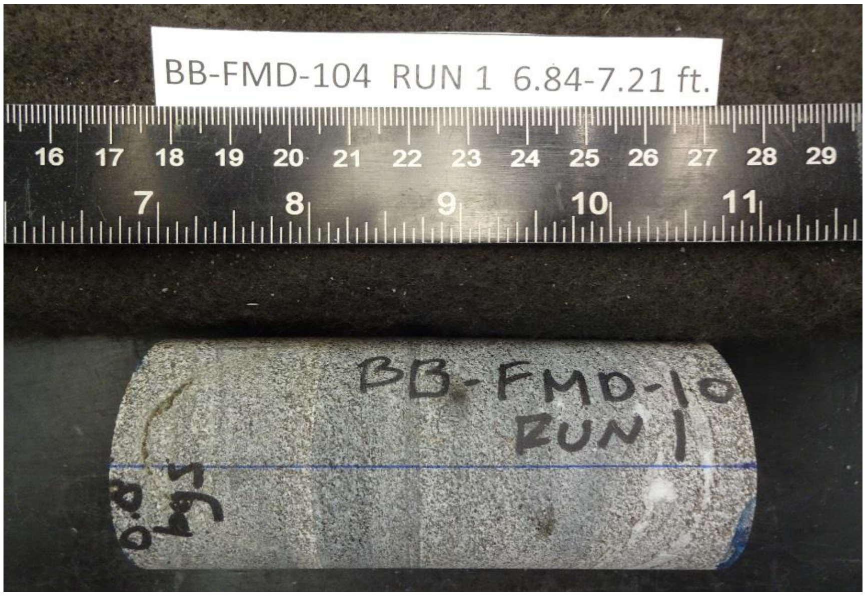
PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00120	1.990	0.00060	0.035	YES
Diameter 2, in (rotated 90°)	0.00030	1.990	0.00015	0.009	YES
END 2					
Diameter 1, in	0.00120	1.990	0.00060	0.035	YES
Diameter 2, in (rotated 90°)	0.00030	1.990	0.00015	0.009	YES

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

Perpendicularity Tolerance Met? YES

Client:	Golder Associates
Project Name:	Mallett Dr. Bridge Replace I-295 EX 22
Project Location:	Freeport, ME
GTX #:	311186
Test Date:	1/17/2020
Tested By:	jck
Checked By:	smd
Boring ID:	BB-FMD-104
Sample ID:	RUN 1
Depth, ft:	6.84-7.21



After cutting and grinding

After break

Client:	Golder Associates	Test Date:	1/17/2020
Project Name:	Mallett Dr. Bridge Replace I-295 EX 22	Tested By:	jck
Project Location:	Freeport, ME	Checked By:	smd
GTX #:	311186		
Boring ID:	BB-FMD-106		
Sample ID:	RUN 1		
Depth:	26.11-26.48 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:	4.48	4.48	4.48
Specimen Diameter, in:	1.99	2.00	2.00
Specimen Mass, g:	595.66		
Bulk Density, lb/ft^3	162	**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**

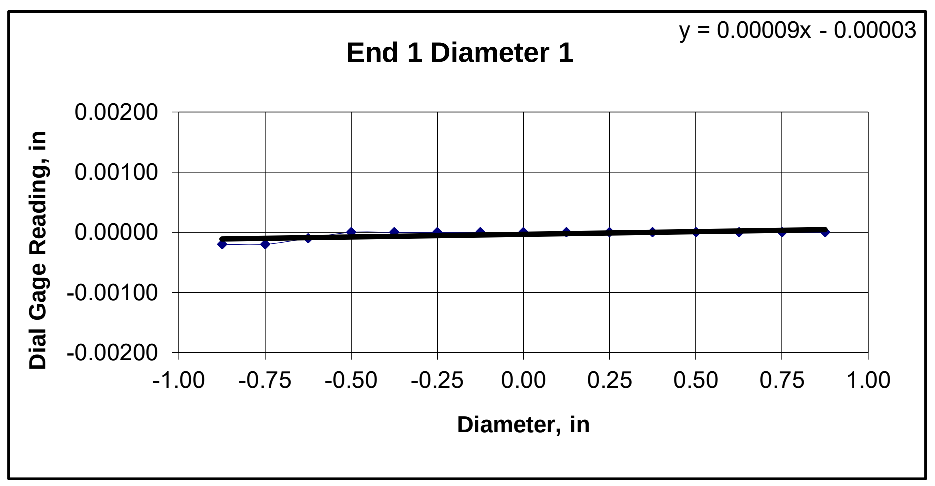
END FLATNESS AND PARALLELISM (Procedure FP1)

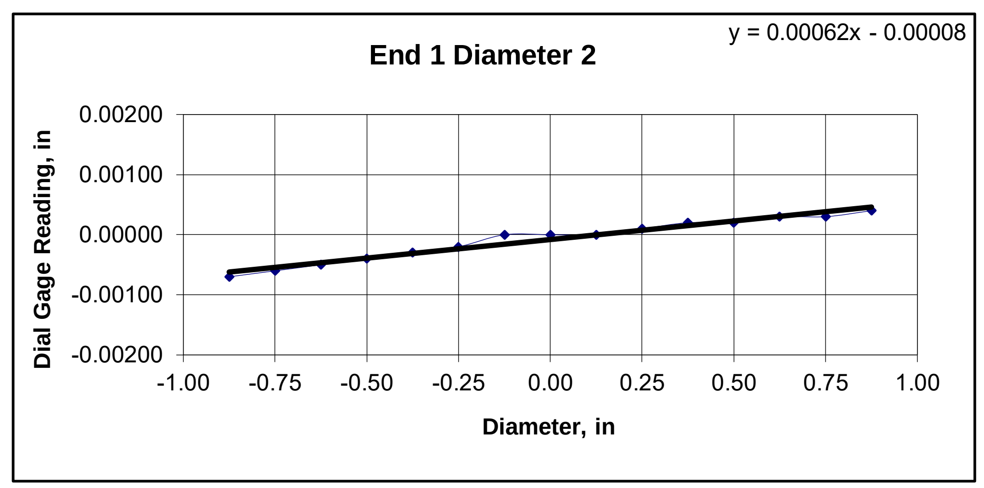
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00020	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
Diameter 2, in (rotated 90°)	-0.00070	-0.00060	-0.00050	-0.00040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

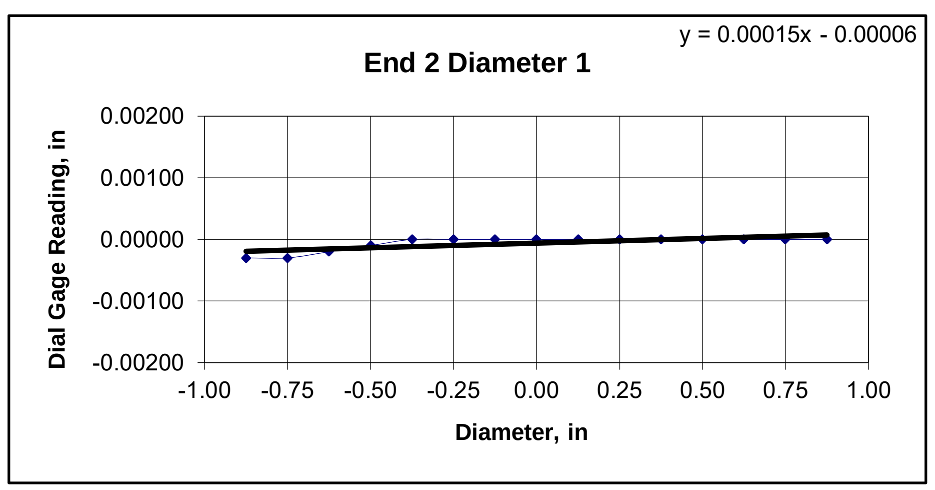
Difference between max and min readings, in:
0° = 0.00020 90° = 0.00110

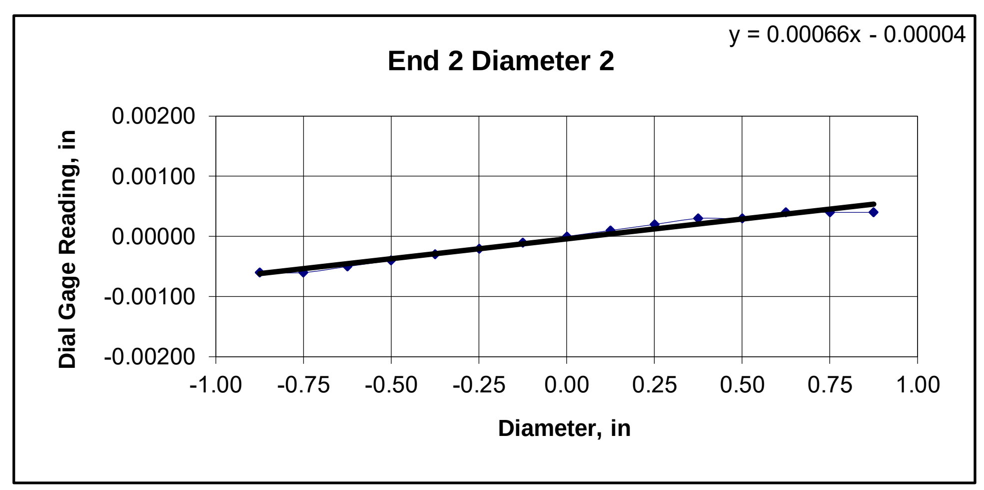
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00030	-0.00030	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
Diameter 2, in (rotated 90°)	-0.00060	-0.00060	-0.00050	-0.00040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

Difference between max and min readings, in:
0° = 0.0003 90° = 0.001
Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00055
Flatness Tolerance Met? **YES**









DIAMETER 1

End 1:	Slope of Best Fit Line	0.00009
	Angle of Best Fit Line:	0.00507
End 2:	Slope of Best Fit Line	0.00015
	Angle of Best Fit Line:	0.00868
Maximum Angular Difference:		0.00360

Parallelism Tolerance Met? **YES**
Spherically Seated

DIAMETER 2

End 1:	Slope of Best Fit Line	0.00062
	Angle of Best Fit Line:	0.03536
End 2:	Slope of Best Fit Line	0.00066
	Angle of Best Fit Line:	0.03782
Maximum Angular Difference:		0.00246

Parallelism Tolerance Met? **YES**
Spherically Seated

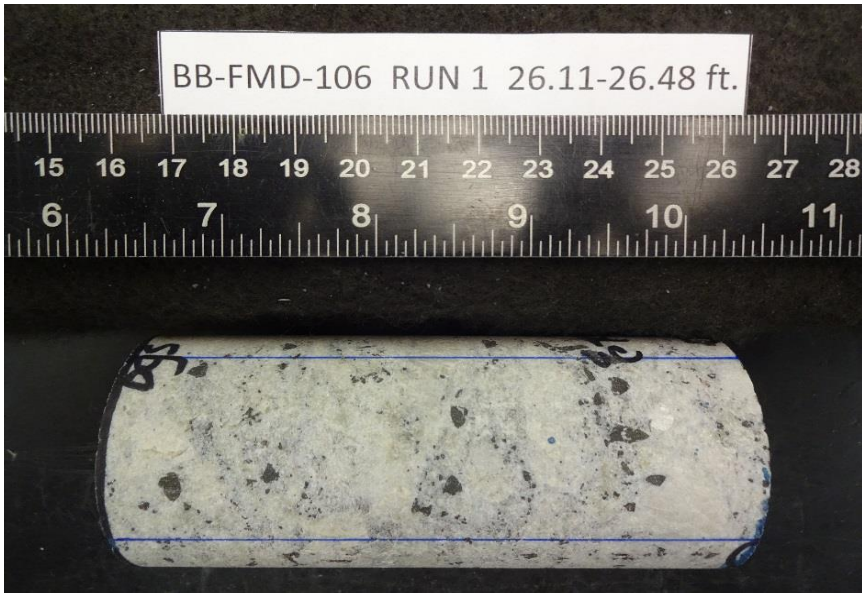
PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00020	1.995	0.00010	0.006	YES
Diameter 2, in (rotated 90°)	0.00110	1.995	0.00055	0.032	YES
END 2					
Diameter 1, in	0.00030	1.995	0.00015	0.009	YES
Diameter 2, in (rotated 90°)	0.00100	1.995	0.00050	0.029	YES

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

Perpendicularity Tolerance Met? **YES**

Client:	Golder Associates
Project Name:	Mallett Dr. Bridge Replace I-295 EX 22
Project Location:	Freeport, ME
GTX #:	311186
Test Date:	1/17/2020
Tested By:	jck
Checked By:	smd
Boring ID:	BB-FMD-106
Sample ID:	RUN 1
Depth, ft:	26.11-26.48



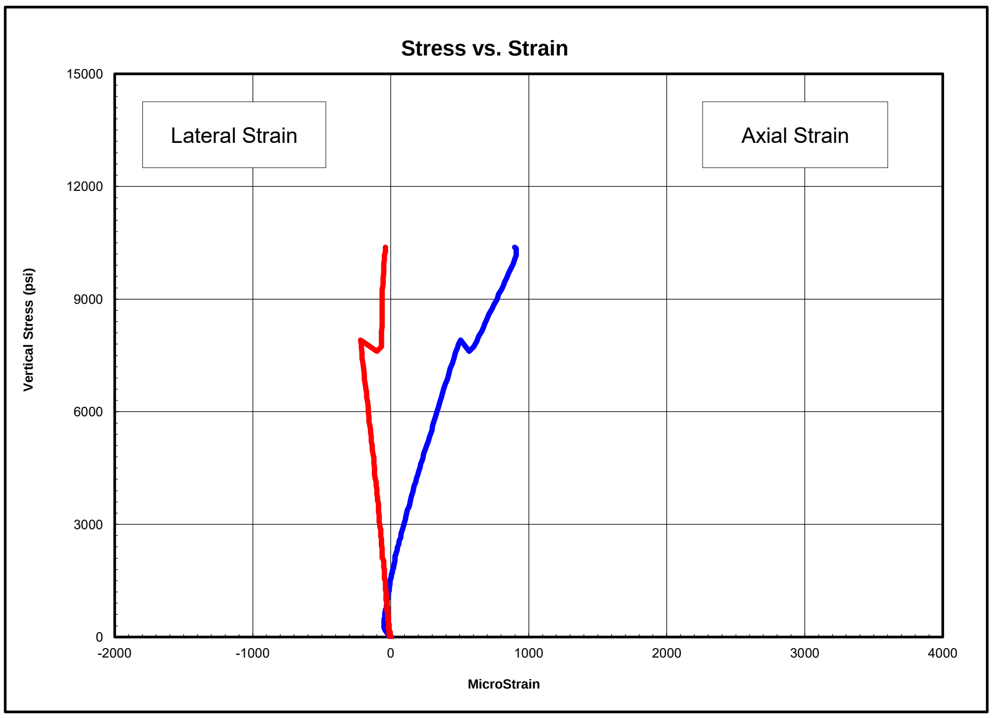
After cutting and grinding

After break

Compressive Strength and Elastic Moduli of Rock

Client:	Golder Associates
Project Name:	Mallett Dr. Bridge Replace. I-295 EX 22
Project Location:	Freeport, ME
GTX #:	311186
Test Date:	1/17/2020
Tested By:	jck
Checked By:	jsc
Boring ID:	BB-FMD-102
Sample ID:	RUN 1
Depth, ft:	14.19-14.40
Sample Type:	rock core
Sample Description:	See photographs Discontinuity failure

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



Peak Compressive Stress: 10,381 psi

The strain gauges picked up initial failure within the specimen and then continued reading until total failure occured. Young's Modulus and Poisson's Ratio within the third stress range could not be determined.

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
1000-3800	15,600,000	0.39
3800-6600	12,000,000	0.34
6600-9300	---	---

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.

GeoTesting EXPRESS

Client:	Golder Associates	Test Date:	1/17/2020
Project Name:	Mallett Dr. Bridge Replace I-295 EX 22	Tested By:	jck
Project Location:	Freeport, ME	Checked By:	smd
GTX #:	311186		
Boring ID:	BB-FMD-102		
Sample ID:	RUN 1		
Depth:	14.19-14.40 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:	4.17	4.17	4.17
Specimen Diameter, in:	1.99	1.99	1.99
Specimen Mass, g:	586.52		
Bulk Density, lb/ft^3	172	**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.? NO

Maximum difference must be < 0.020 in.
Straightness Tolerance Met? NO

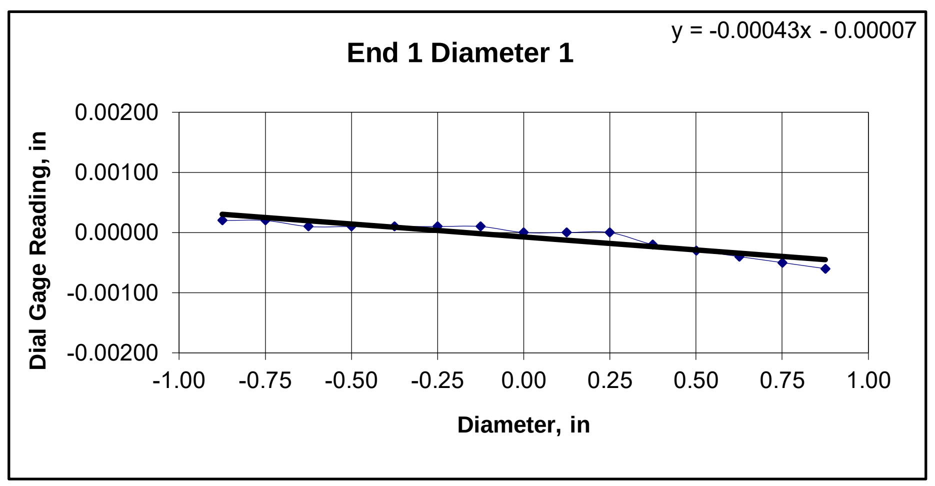
END FLATNESS AND PARALLELISM (Procedure FP1)

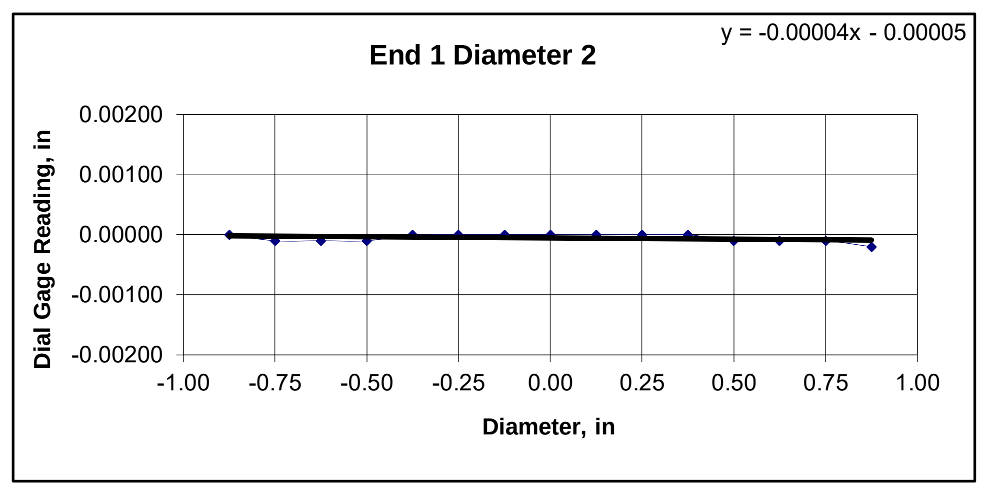
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00020	0.00020	0.00010	0.00010	0.00010	0.00010	0.00010	0.00000	0.00000	0.00000	-0.00020	-0.00030	-0.00040	-0.00050	-0.00060
Diameter 2, in (rotated 90°)	0.00000	-0.00010	-0.00010	-0.00010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

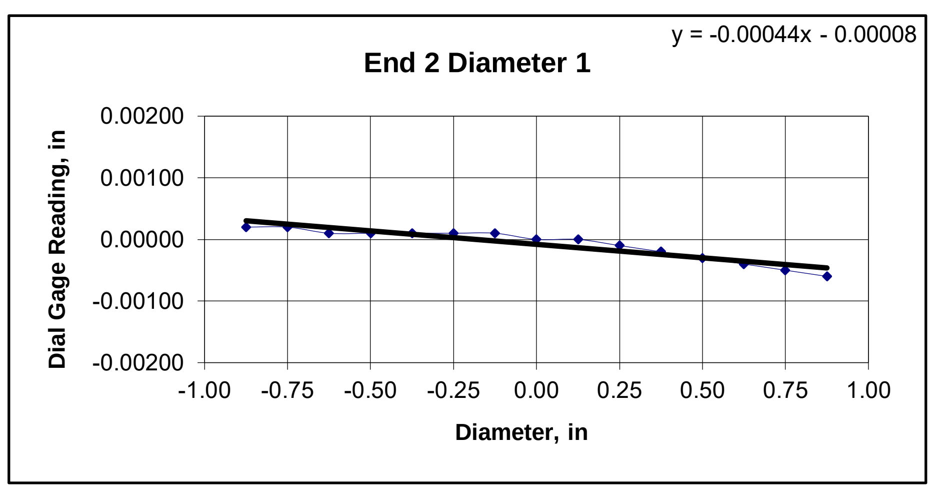
Difference between max and min readings, in:
0° = 0.00080 90° = 0.00020

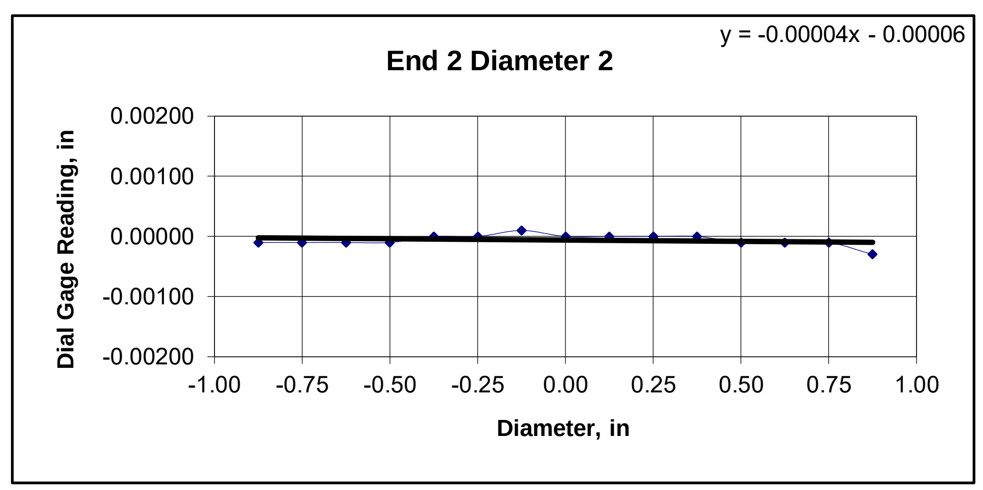
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00020	0.00020	0.00010	0.00010	0.00010	0.00010	0.00010	0.00000	0.00000	-0.00010	-0.00020	-0.00030	-0.00040	-0.00050	-0.00060
Diameter 2, in (rotated 90°)	-0.00010	-0.00010	-0.00010	-0.00010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

Difference between max and min readings, in:
0° = 0.0008 90° = 0.0004
Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00040
Flatness Tolerance Met? YES









DIAMETER 1

End 1:
Slope of Best Fit Line 0.00043
Angle of Best Fit Line: 0.02472

End 2:
Slope of Best Fit Line 0.00044
Angle of Best Fit Line: 0.02505

Maximum Angular Difference: 0.00033

Parallelism Tolerance Met? YES
Spherically Seated

DIAMETER 2

End 1:
Slope of Best Fit Line 0.00004
Angle of Best Fit Line: 0.00229

End 2:
Slope of Best Fit Line 0.00004
Angle of Best Fit Line: 0.00246

Maximum Angular Difference: 0.00016

Parallelism Tolerance Met? YES
Spherically Seated

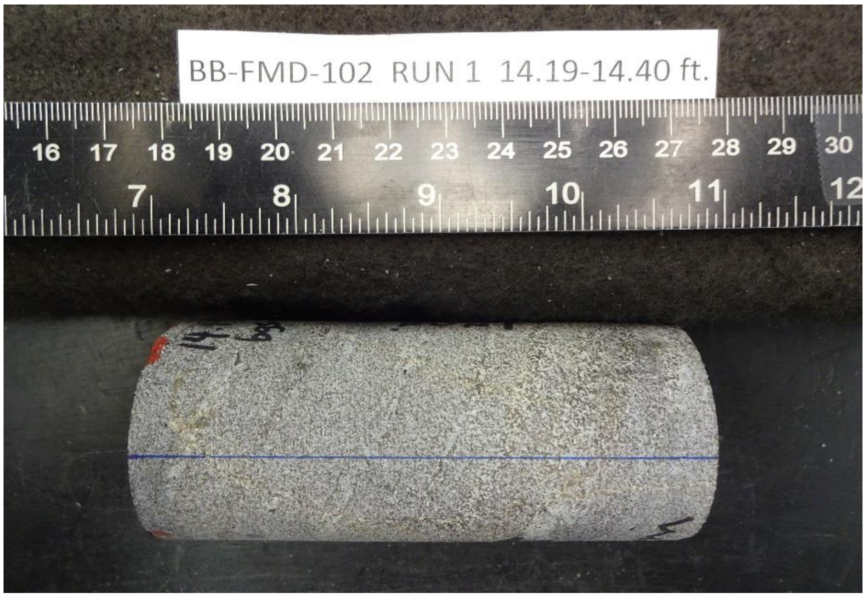
PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00080	1.990	0.00040	0.023	YES
Diameter 2, in (rotated 90°)	0.00020	1.990	0.00010	0.006	YES
END 2					
Diameter 1, in	0.00080	1.990	0.00040	0.023	YES
Diameter 2, in (rotated 90°)	0.00040	1.990	0.00020	0.012	YES

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

Perpendicularity Tolerance Met? YES

Client:	Golder Associates
Project Name:	Mallett Dr. Bridge Replace I-295 EX 22
Project Location:	Freeport, ME
GTX #:	311186
Test Date:	1/17/2020
Tested By:	jck
Checked By:	smd
Boring ID:	BB-FMD-102
Sample ID:	RUN 1
Depth, ft:	14.19-14.40



After cutting and grinding

After break

APPENDIX E1

Frost Depth

GOLDER

CALCULATIONS

Date:	12/3/2020	**Made by:**	KAR
Project No.:	19129538	**Checked by:**	HTV
Subject:	Depth of Frost Penetration	**Reviewed by:**	MCM
Project Short Title:	MaineDOT I-295 Exit 22 Mallet Drive Bridge Replacement No. 5721		

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 17, 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 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 8.2%. Assume water content will be 10% in place.

From Reference 1, Table 5-1, with design freezing index = 1300, coarse-grained soil, and assuming the in-place water content is w = 10%:

Frost Penetration = 76.3 inches = 6.4 feet

CONCLUSIONS

The depth of frost penetration at the proposed bridge site is estimated to be 6.4 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.

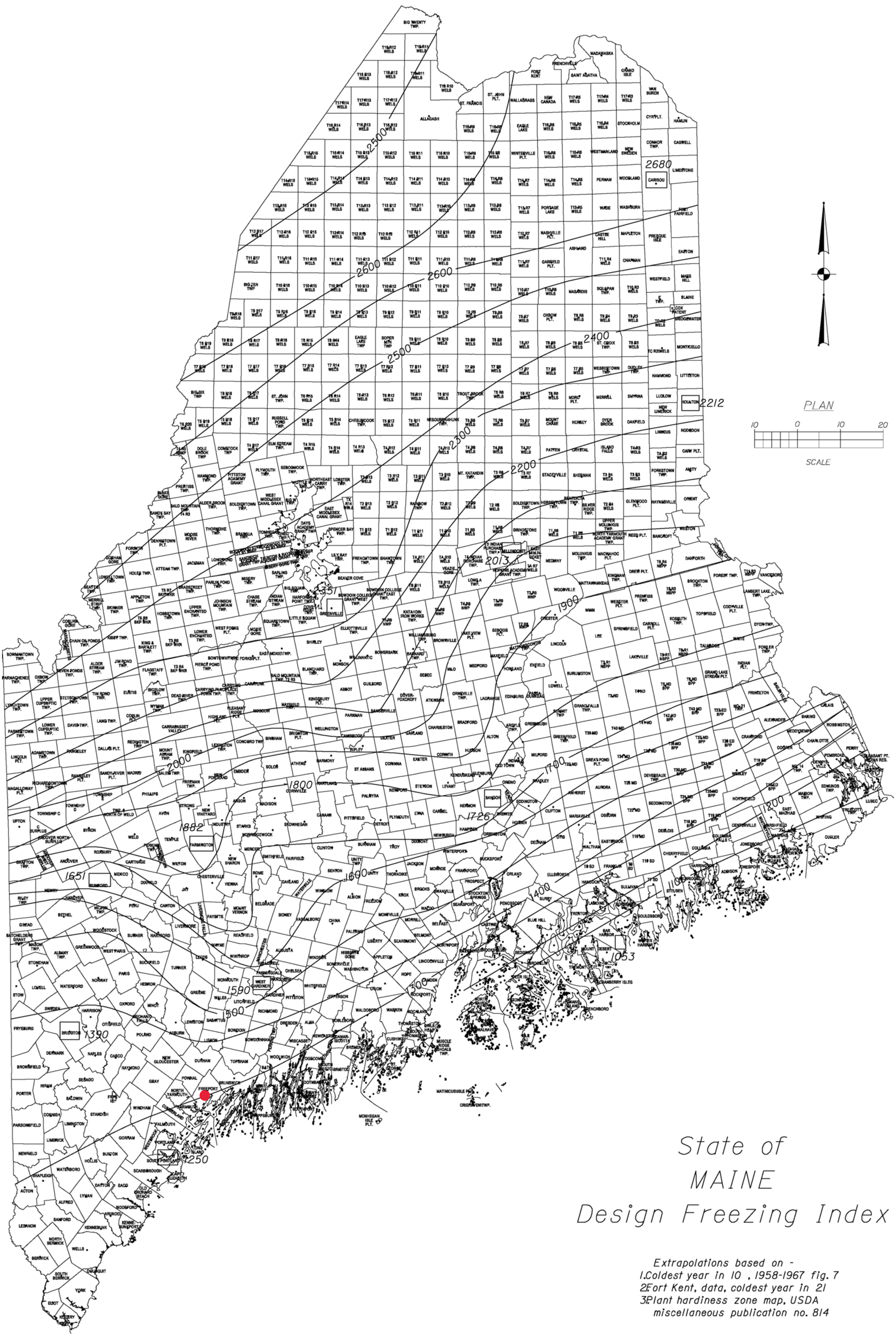


Figure 5-1 Maine Design Freezing Index Map

APPENDIX E2

Seismic Site Class

CALCULATIONS

Date:	7/6/2020	**Made by:**	HTV
Project No.:	19129538	**Checked by:**	KAR
Subject:	Seismic Site Class	**Reviewed by:**	JEL
Project Short Title:	I-295 Mallet Drive Bridge Replacement #5721 (Exit 22)		

OBJECTIVE

Determine the seismic site class, acceleration coefficient, and other seismic parameters for the thickest soil deposit encountered at the I-295 Exit 22 Mallet Drive Bridge.

METHOD

Use the procedure outlined in AASHTO LRFD Section 3.10.3.1 Methods B and C (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 Mallet Drive Bridge Replacement #5721 (Exit 22) - 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-FMD-101 and at BB-FMD-106 were used to evaluate seismic site class, since BB-FMD-101 contained the thickest total soil deposit, the thickest fill layer, and the thickest clay layer, while BB-FMD-106 contained the lowest interpreted s_u.

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:
di = 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 height 100 ft of the soil/rock profile at BB-FMD-101 as:

GOLDER

CALCULATIONS

Date:	7/6/2020	**Made by:**	HTV
Project No.:	19129538	**Checked by:**	KAR
Subject:	Seismic Site Class	**Reviewed by:**	JEL
Project Short Title:	I-295 Mallet Drive Bridge Replacement #5721 (Exit 22)		

Layer	N_{60}	Deposit	d_i	d_i/N_i
1	60	Fill	17.5	0.29
2	33	Glaciomarine	23.5	0.71
3	51	Sand and Gravel	2.8	0.05
4	100	Bedrock	56.2	0.56
		Sum:	100	1.62

$\overline{N}$ = 61.70

Site Class: C

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

For cohesionless soil layers:

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

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).

a. The seismic site class is assigned using the average 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 full embankment height soil profile at BB-FMD-101 as:

Cohesionless Soils:

Layer	N_{60}	Deposit	d_i	d_i/N_i
1	60	Fill	17.5	0.29
3	51	Sand and Gravel	2.8	0.05
4	100	Bedrock	56.2	0.56
		Sum:	76.5	0.91

$\overline{N}_{ch}$ = 84.20

Site Class: C

GOLDER

CALCULATIONS

Date:	7/6/2020	**Made by:**	HTV
Project No.:	19129538	**Checked by:**	KAR
Subject:	Seismic Site Class	**Reviewed by:**	JEL
Project Short Title:	I-295 Mallet Drive Bridge Replacement #5721 (Exit 22)		

Cohesive Soils:

Layer	s_u (ksf)	Deposit	d_i	d_i/s_{ui}
2	3.50	Glaciomarine	23.5	6.71
		Sum:	23.5	6.71

$\bar{s}_u$ = 3.50

Site Class: C

b. The seismic site class is assigned using the average 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-FMD-106 as:

Cohesionless Soils:

Layer	N_{60}	Deposit	d_i	d_i/N_i
2	13	Sand and Gravel	1.8	0.14
3	100	Bedrock	84.2	0.84
		Sum:	86.0	0.98

$\bar{N}_{ch}$ = 87.71

Site Class: C

Cohesive Soils:

Layer	s_u (ksf)	Deposit	d_i	d_i/s_{ui}
1	1.35	Glaciomarine	14.0	10.37
		Sum:	14.0	10.37

$\bar{s}_u$ = 1.35

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.

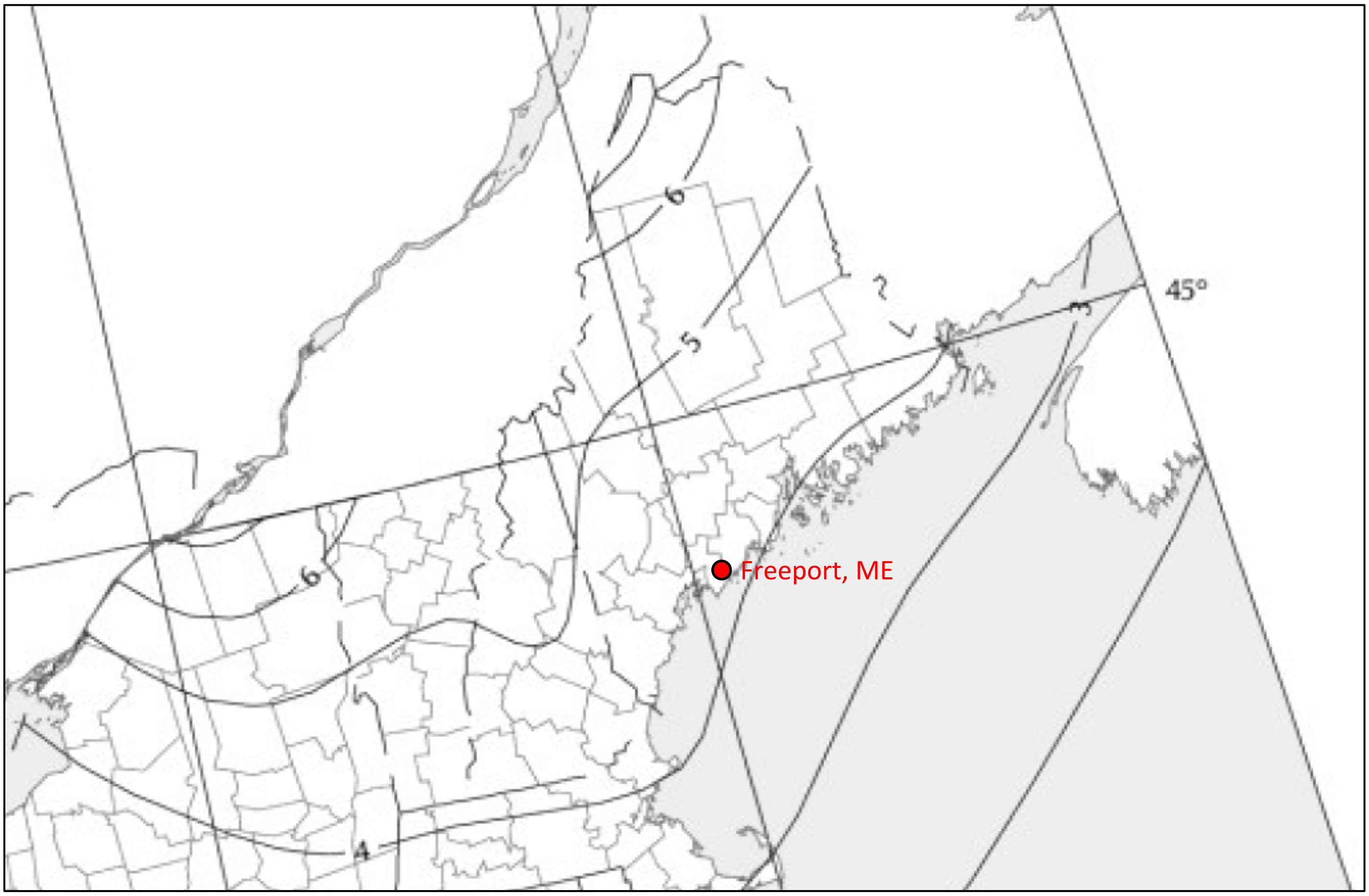
Site Class: D

GOLDER

CALCULATIONS

Date:	7/6/2020	**Made by:**	HTV
Project No.:	19129538	**Checked by:**	KAR
Subject:	Seismic Site Class	**Reviewed by:**	JEL
Project Short Title:	I-295 Mallet Drive Bridge Replacement #5721 (Exit 22)		

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



$S_1 =$ 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 Parameters 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	a_2. Using Figure 3.10.2.1-2	a_3. Using Figure 3.10.2.1-3
*PGA = 0.08	*S_s = 0.16	*S_1 = 0.043

GOLDER

CALCULATIONS

Date:	7/6/2020	**Made by:**	HTV
Project No.:	19129538	**Checked by:**	KAR
Subject:	Seismic Site Class	**Reviewed by:**	JEL
Project Short Title:	I-295 Mallet Drive Bridge Replacement #5721 (Exit 22)		

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

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

*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

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

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-FMD-101), for the full 100 ft considered depth of soil and rock including the fill above the I-295 highway elevation, Seismic Site Class is C.

In the area where the lowest interpreted S_u was encountered (at BB-FMD-106), for the full 100 ft considered soil and rock profile, the Seismic Site Class is D. Note: This condition is similar to where the new embankment is proposed adjacent to the northwestern abutment. The final s_u upon consolidation of the cohesive soil will likely increases follow embankment construction and result in a long term change of seismic site class.

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.

Note: Very loose granular soil at BB-FMD-102 and BB-FMD-105 was excluded from the above seismic site classification analysis based on these soils being localized backfill beside existing pier foundations and not soils at planned foundation locations.

APPENDIX E3

Global Stability Analysis

GOLDER

CALCULATIONS

Date:	12/2/2020	**Made by:**	HTV / KAR
Project No.:	19129538	**Checked by:**	KAR / MEL
Subject:	Global Stability Analysis: Approach Embankment	**Reviewed by:**	MCM
Project Short Title:	I-295 Mallet Drive Bridge Replacement no 5721		

OBJECTIVE

Calculate global factor of safety for the Abutment No. 1 proposed bridge approach embankment, assuming the "Phasing South" 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 23, 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. Approach Road Bridge Freeport Interstate 295: Mallet Drive 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 17, 2020 (Appendix C, Preliminary Geotechnical Design Report, dated September 2020).
8. Golder summary of rock laboratory test results (Table 5, Preliminary Geotechnical Design Report, dated September 2020).
9. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.
10. Golder calculation titled "Seismic Site Class" (Appendix E, Preliminary Geotechnical Design Report, dated September 2020).
11. Rocscience Slide Software Package Version 2020 9.007 64-bit, build date May 29, 2020.

ATTACHMENTS

1. Slide output figures
2. HNTB plans showing "phasing south" 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 9. A pseudo-static FS > 1.0 is recommended per Section 3.7.4.1 in Reference 9.
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 clay layers.

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 parameters for the glaciomarine layers are 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 layers 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 8). 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 9, 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	33	-	-	-	-
Glaciomarine (Stiff)	115	Undrained	1350	0				
Glaciomarine (Very Stiff)	115	Undrained	3500	0	-	-	-	-
Sand and Gravel	125	Mohr-Coulomb	0	36	-	-	-	-
Bedrock	169	Generalized Hoek-Brown	-	-	1,815,000	77	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. At the time of this analysis, MaineDOT/HNTB planned to construct the phasing south option. Therefore the phasing south option was analyzed as the critical case. This analysis evaluates the fills at the proposed Abutment No. 1 approach embankment since the proposed fills and the underlying foundation clays are the thickest. The glaciomarine clay was split into stiff and very stiff layers based on interpreted cohesion, and two different zones of the very stiff, compressed glaciomarine clay underneath the existing embankment, termed the "small" and "large" zones, were analyzed. The results of the Slide stability analyses are summarized in the following table.

GOLDER

CALCULATIONS

Date:	12/2/2020	**Made by:**	HTV / KAR
Project No.:	19129538	**Checked by:**	KAR / MEL
Subject:	Global Stability Analysis: Approach Embankment	**Reviewed by:**	MCM
Project Short Title:	I-295 Mallet Drive Bridge Replacement no 5721		

Baseline	Station	Feature	Slope	Strong Clay Zone	Lowest Factor of Safety (Spencer Method)	
					NonCircular Failure Surface Through Proposed Fill	NonCircular Failure Surface Through Glaciomarine Deposit
Phasing South	29+75	Abutment No. 1 Approach Embankment	North	Small	1.30 (Fig. A.1)	2.40 (Fig. A.2)
				Large		2.40 (Fig. A.3)
			South	Small	1.26 (Fig. A.4)	1.56 (Fig. A.5)
				Large		1.93 (Fig. A.6)

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 1.25 to 1.32 for surfaces through the proposed fill and from 1.59 to 2.82 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 10) 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	Strong Clay Zone	Lowest Factor of Safety (Spencer Method)	
					NonCircular Failure Surface Through Proposed Fill	NonCircular Failure Surface Through Glaciomarine Deposit
Phasing South	29+75	Abutment No. 1 Approach Embankment	North	Small	1.12 (Fig. B.1)	1.84 (Fig. B.2)
				Large		1.84 (Fig. B.3)
			South	Small	1.08 (Fig. B.4)	1.33 (Fig. B.5)
				Large		1.67 (Fig. B.6)

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 1.08 to 1.14 for surfaces through the proposed fill and from 1.35 to 2.45 for surfaces through the glaciomarine deposit.

CONCLUSIONS

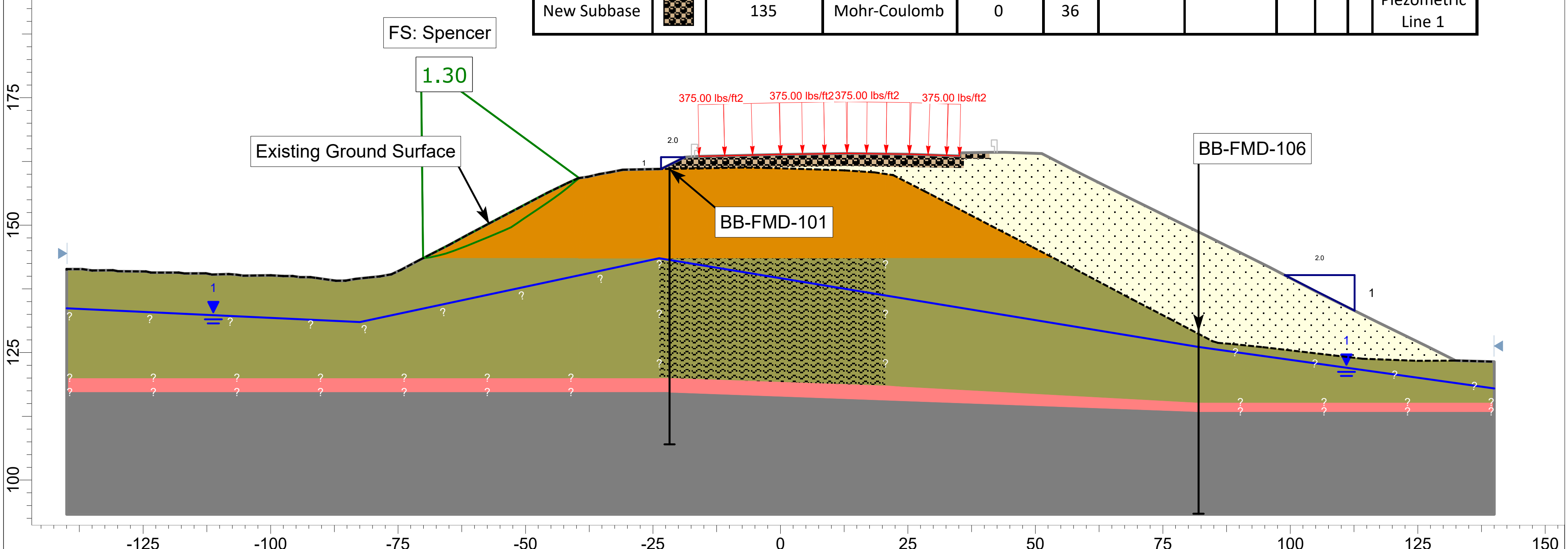
The proposed embankment and slope grading system produces a stability factor of safety less than the recommended factor of safety of 1.3 for potential surficial slope failures in the embankment fill when using embankment fill engineering parameters recommended in the MaineDOT Bridge Design Guide. Failure 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.

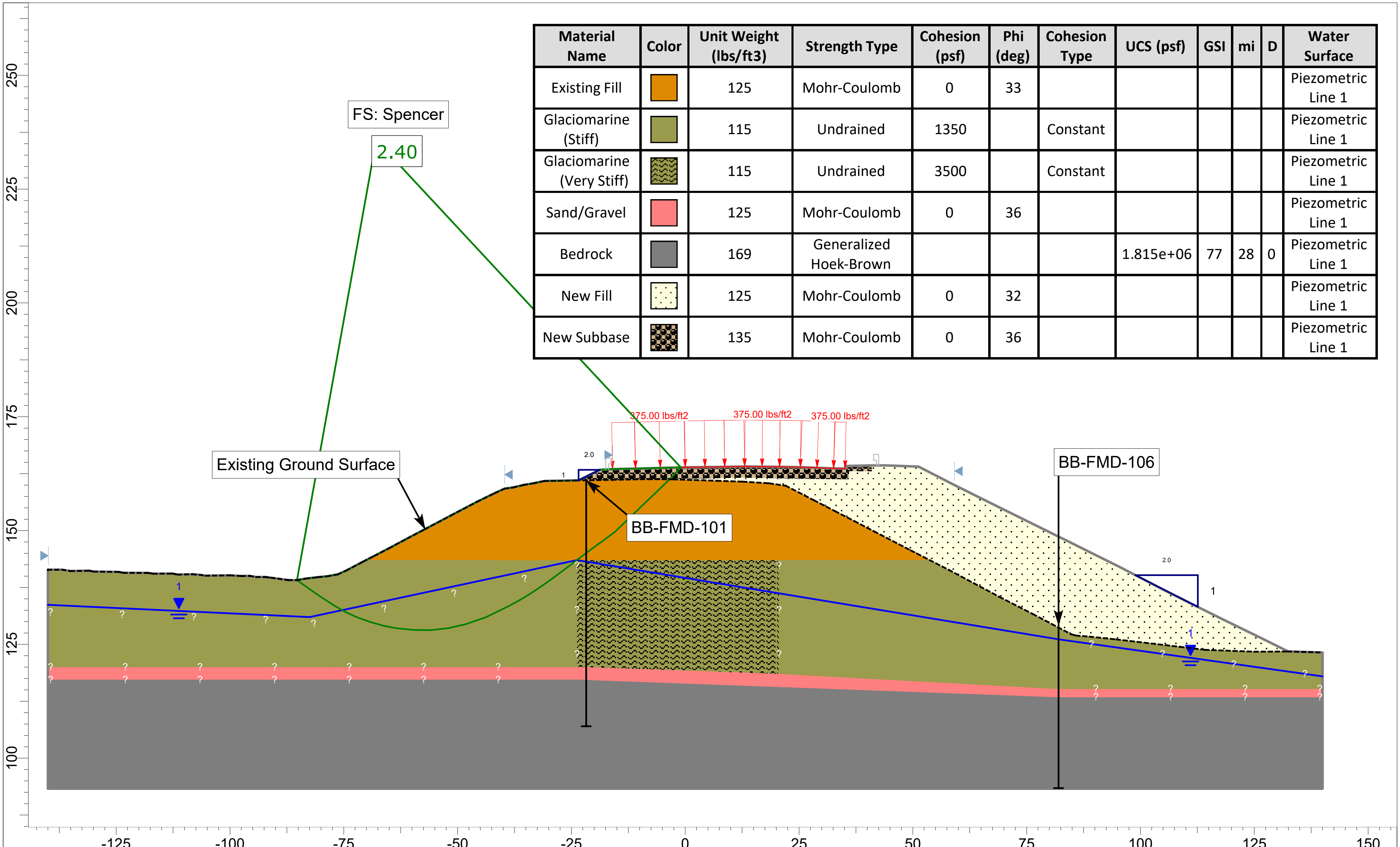
The analysis of the proposed embankment and slope grading system yields adequate factors of safety (FS > 1.0) for the potential slope failures in both the native soils underlying the proposed embankment and embankment fills under pseudo-static conditions where seismic loading is applied.

Attachment 1

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	UCS (psf)	GSI	mi	D	Water Surface
Existing Fill		125	Mohr-Coulomb	0	33						Piezometric Line 1
Glaciomarine (Stiff)		115	Undrained	1350		Constant					Piezometric Line 1
Glaciomarine (Very Stiff)		115	Undrained	3500		Constant					Piezometric Line 1
Sand/Gravel		125	Mohr-Coulomb	0	36						Piezometric Line 1
Bedrock		169	Generalized Hoek-Brown				1.815e+06	77	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*	19129538 I-295 MaineDOT Mallet Drive Bridge no 5721					
	Analysis Description	Phasing South - Station 29+75 North Slope (NonCircular, Cuckoo Search)					
	Drawn By	HTV / KAR	*Checked By*	KAR	*Reviewed By* MCM	*Scale* 1:350	**Figure A.1**
SLIDEINTERPRET 9.007	*Date*	7/7/2020	*File Name*	Mallet Drive Slide 29+75 _ KAR edits.slmd			



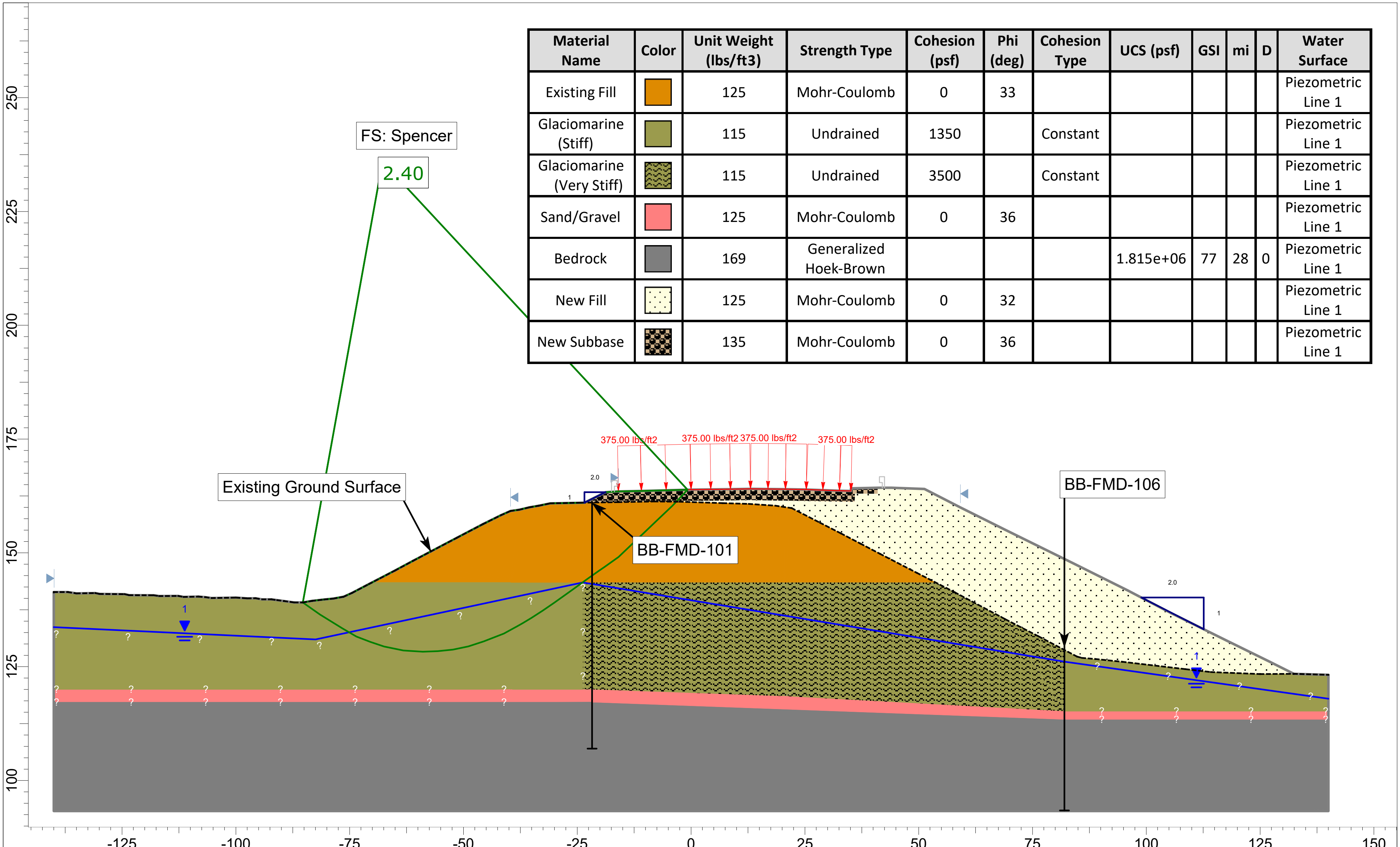
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	UCS (psf)	GSI	mi	D	Water Surface
Existing Fill		125	Mohr-Coulomb	0	33						Piezometric Line 1
Glaciomarine (Stiff)		115	Undrained	1350		Constant					Piezometric Line 1
Glaciomarine (Very Stiff)		115	Undrained	3500		Constant					Piezometric Line 1
Sand/Gravel		125	Mohr-Coulomb	0	36						Piezometric Line 1
Bedrock		169	Generalized Hoek-Brown				1.815e+06	77	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	19129538 I-295 MaineDOT Mallet Drive Bridge no 5721
Analysis Description	Phasing South - Station 29+75 North Slope (NonCircular, Cuckoo Search)
Drawn By	HTV / KAR
Checked By	KAR
Reviewed By	MCM
Scale	1:350
Date	7/7/2020
File Name	Mallet Drive Slide 29+75 _ KAR edits.slmd

Figure A.2



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	UCS (psf)	GSI	mi	D	Water Surface
Existing Fill		125	Mohr-Coulomb	0	33						Piezometric Line 1
Glaciomarine (Stiff)		115	Undrained	1350		Constant					Piezometric Line 1
Glaciomarine (Very Stiff)		115	Undrained	3500		Constant					Piezometric Line 1
Sand/Gravel		125	Mohr-Coulomb	0	36						Piezometric Line 1
Bedrock		169	Generalized Hoek-Brown				1.815e+06	77	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

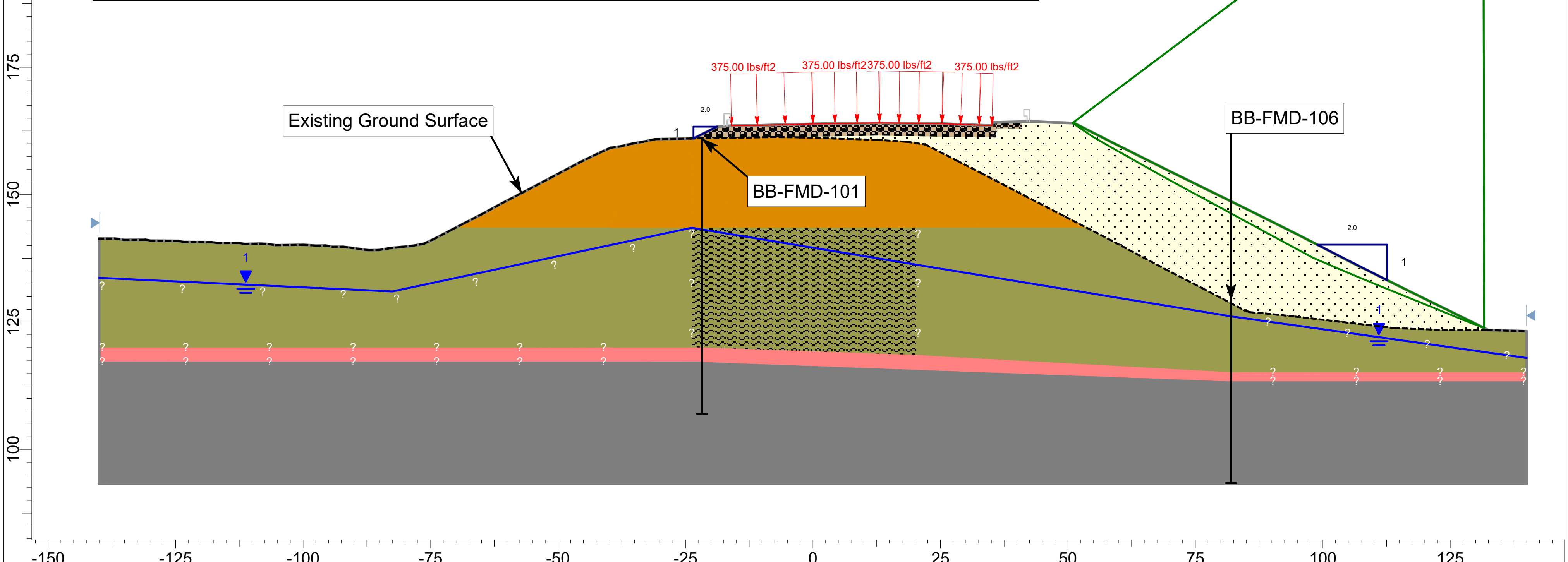
GOLDER

SLIDEINTERPRET 9.007

Project	19129538 I-295 MaineDOT Mallet Drive Bridge no 5721
Analysis Description	Phasing South - Station 29+75 North Slope (NonCircular, Cuckoo Search)
Drawn By	HTV / KAR
Checked By	KAR
Reviewed By	MCM
Scale	1:350
Date	7/7/2020
File Name	Mallet Drive Slide 29+75 _ KAR edits.slmd

Figure A.3

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	UCS (psf)	GSI	mi	D	Water Surface
Existing Fill		125	Mohr-Coulomb	0	33						Piezometric Line 1
Glaciomarine (Stiff)		115	Undrained	1350		Constant					Piezometric Line 1
Glaciomarine (Very Stiff)		115	Undrained	3500		Constant					Piezometric Line 1
Sand/Gravel		125	Mohr-Coulomb	0	36						Piezometric Line 1
Bedrock		169	Generalized Hoek-Brown				1.815e+06	77	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



Project: 19129538 I-295 MaineDOT Mallet Drive Bridge no 5721

Analysis Description: Phasing South - Station 29+75 South Slope (NonCircular, Cuckoo Search)

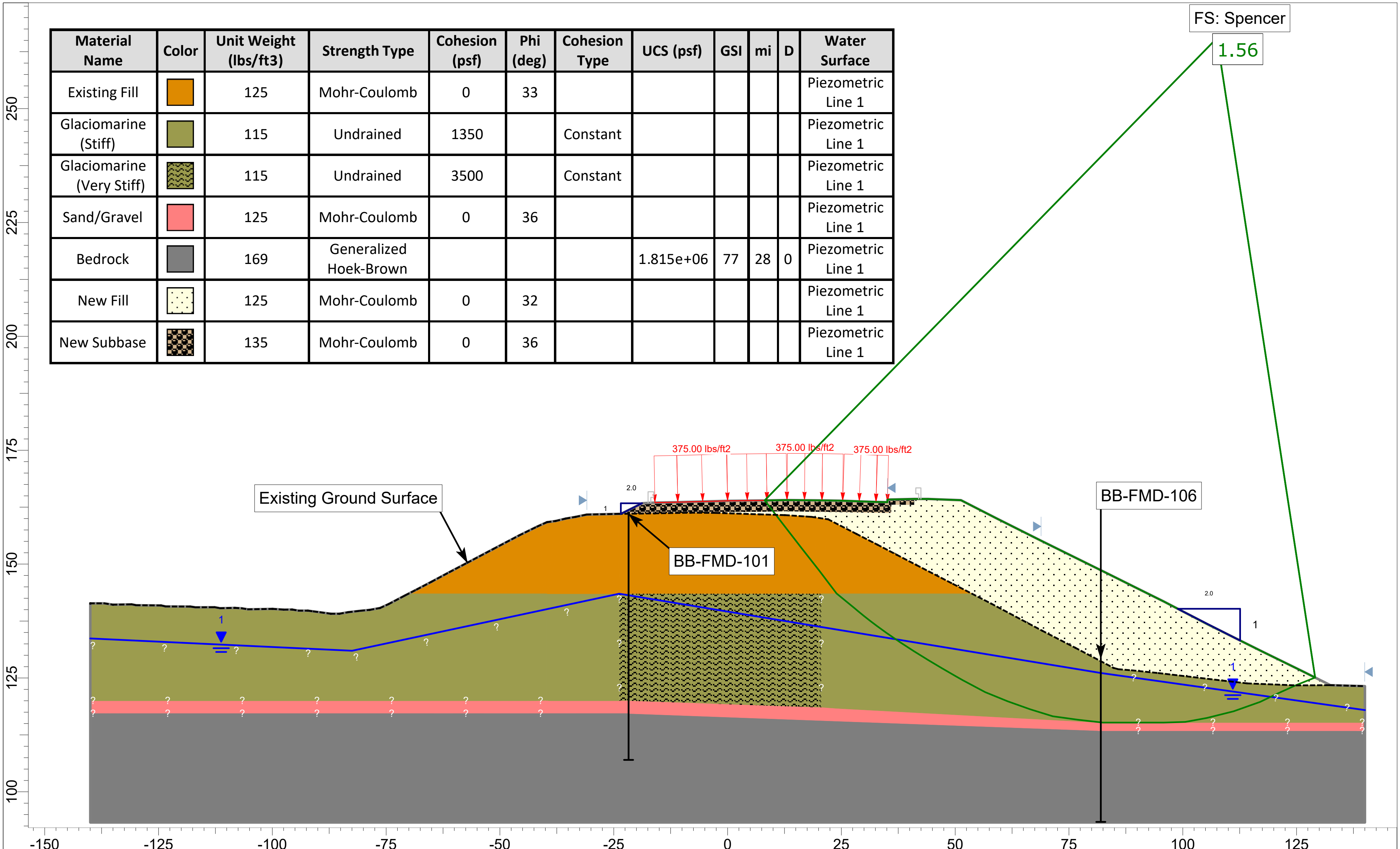
Drawn By	HTV	Checked By	KAR	Reviewed By	MCM	Scale	1:350
Date	7/7/2020	File Name	Mallet Drive Slide 29+75 _ KAR edits.slmd				

GOLDER

SLIDEINTERPRET 9.007

Figure A.4

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	UCS (psf)	GSI	mi	D	Water Surface
Existing Fill		125	Mohr-Coulomb	0	33						Piezometric Line 1
Glaciomarine (Stiff)		115	Undrained	1350		Constant					Piezometric Line 1
Glaciomarine (Very Stiff)		115	Undrained	3500		Constant					Piezometric Line 1
Sand/Gravel		125	Mohr-Coulomb	0	36						Piezometric Line 1
Bedrock		169	Generalized Hoek-Brown				1.815e+06	77	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



Project	19129538 I-295 MaineDOT Mallet Drive Bridge no 5721
Analysis Description	Phasing South - Station 29+75 South Slope (NonCircular, Cuckoo Search)
Drawn By	HTV / KAR
Checked By	KAR
Reviewed By	MCM
Scale	1:350
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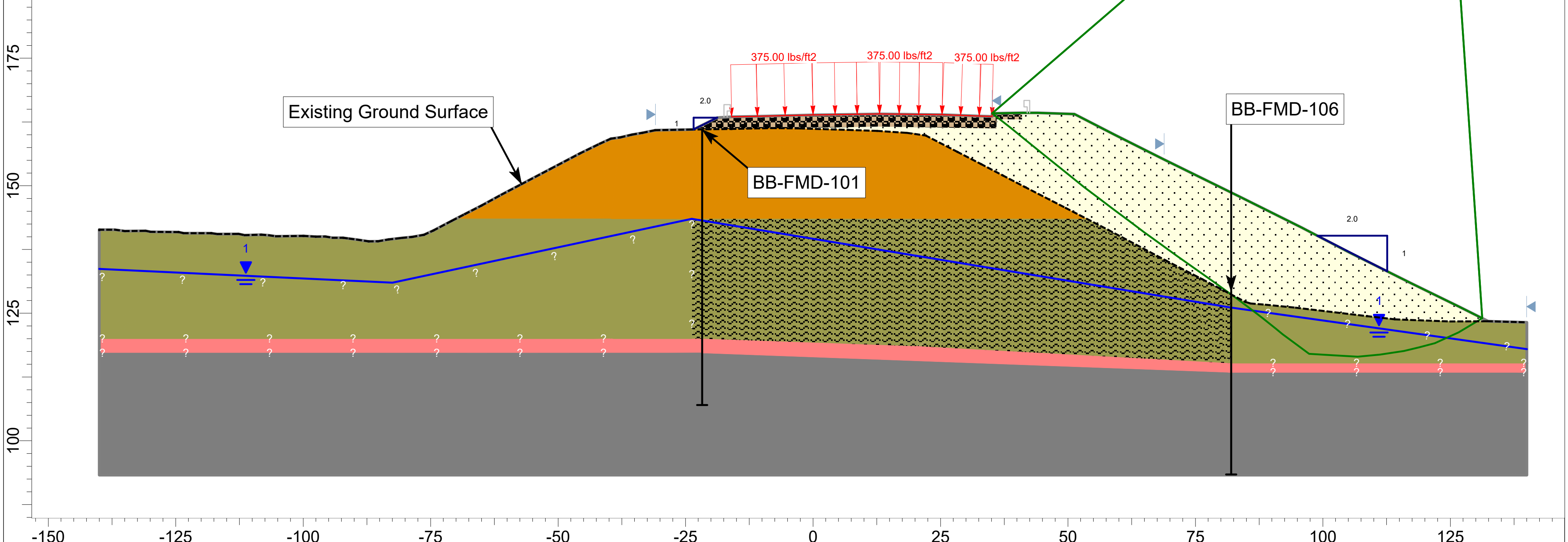
Figure A.5

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	UCS (psf)	GSI	mi	D	Water Surface
Existing Fill		125	Mohr-Coulomb	0	33						Piezometric Line 1
Glaciomarine (Stiff)		115	Undrained	1350		Constant					Piezometric Line 1
Glaciomarine (Very Stiff)		115	Undrained	3500		Constant					Piezometric Line 1
Sand/Gravel		125	Mohr-Coulomb	0	36						Piezometric Line 1
Bedrock		169	Generalized Hoek-Brown				1.815e+06	77	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



Project	19129538 I-295 MaineDOT Mallet Drive Bridge no 5721
Analysis Description	Phasing South - Station 29+75 South Slope (NonCircular, Cuckoo Search)
Drawn By HTV / KAR	*Checked By* KAR
Reviewed By MCM	*Scale* 1:350
Date 7/7/2020	*File Name* Mallet Drive Slide 29+75 _ KAR edits.slmd

Figure A.6

GOLDER

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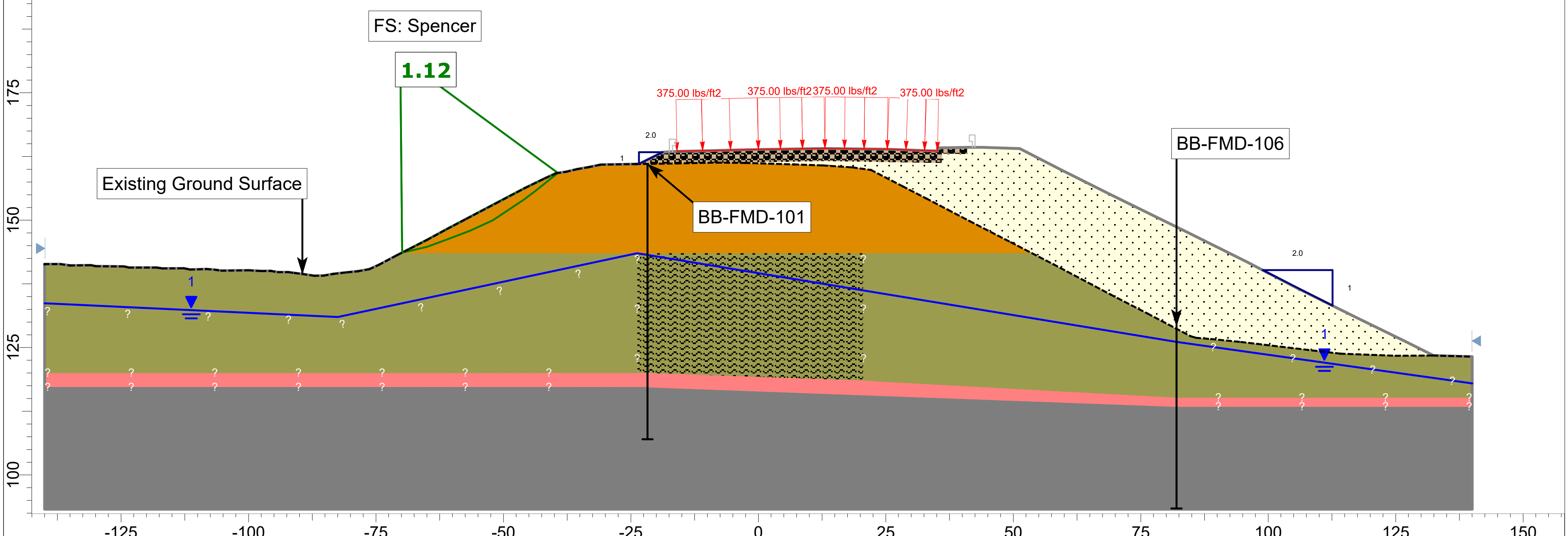
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	UCS (psf)	GSI	mi	D	Water Surface
Existing Fill		125	Mohr-Coulomb	0	33						Piezometric Line 1
Glaciomarine (Weak)		115	Undrained	1350		Constant					Piezometric Line 1
Glaciomarine (Strong)		115	Undrained	3500		Constant					Piezometric Line 1
Sand/Gravel		125	Mohr-Coulomb	0	36						Piezometric Line 1
Bedrock		169	Generalized Hoek-Brown				1.815e+06	77	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* 19129538 I-295 MaineDOT Mallet Drive Bridge no 5721
	Analysis Description Phasing South - Station 29+75 North Slope (NonCircular, Cuckoo Search)
	Drawn By KAR · *Checked By* MEL · *Reviewed By* MCM · *Scale* 1:350
	Date 12/2/2020 · *File Name* Mallet Drive Slide 29+75 _ KAR edits _ seismic.slmd

Figure B.1

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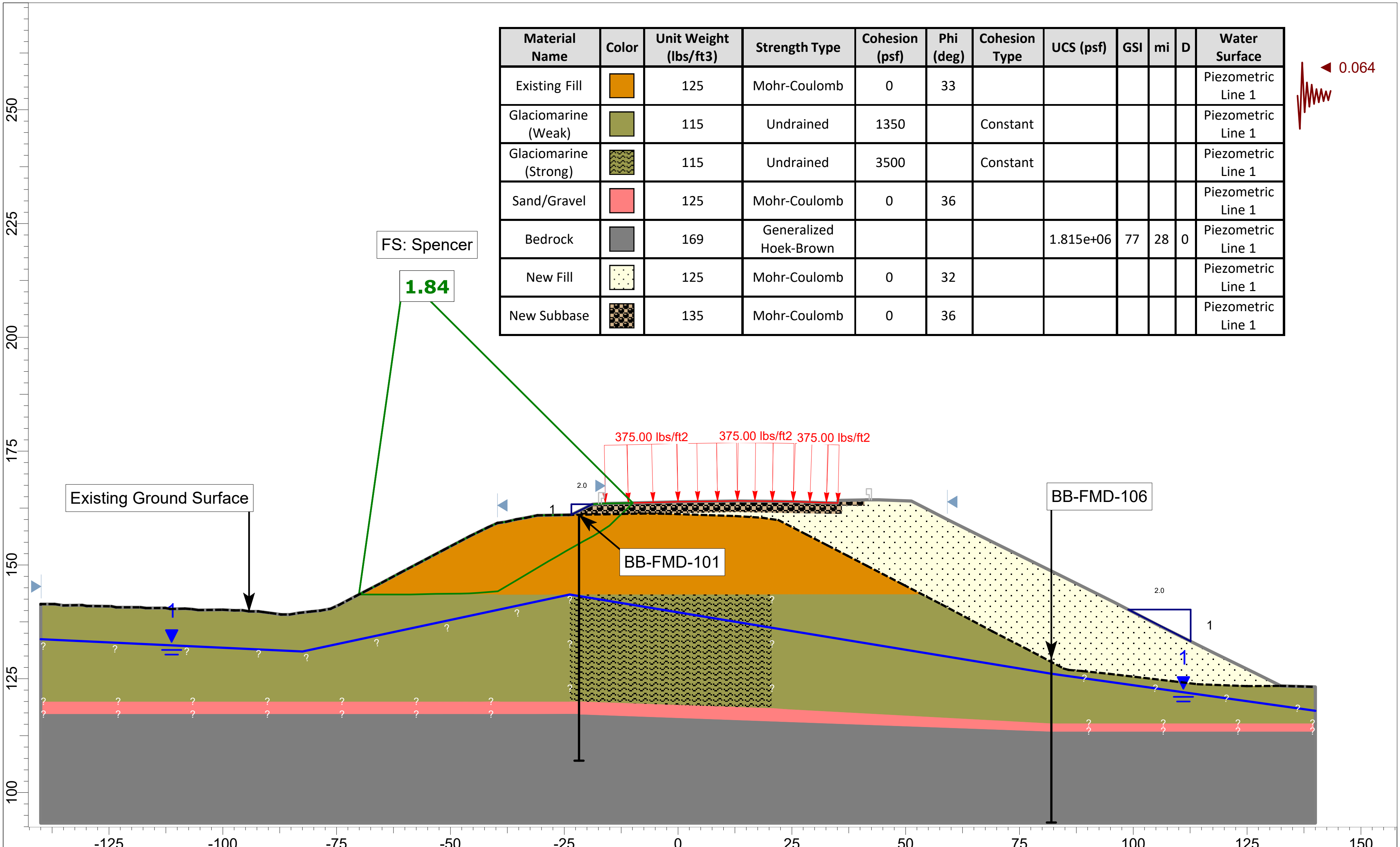

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	UCS (psf)	GSI	mi	D	Water Surface
Existing Fill		125	Mohr-Coulomb	0	33						Piezometric Line 1
Glaciomarine (Weak)		115	Undrained	1350		Constant					Piezometric Line 1
Glaciomarine (Strong)		115	Undrained	3500		Constant					Piezometric Line 1
Sand/Gravel		125	Mohr-Coulomb	0	36						Piezometric Line 1
Bedrock		169	Generalized Hoek-Brown				1.815e+06	77	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

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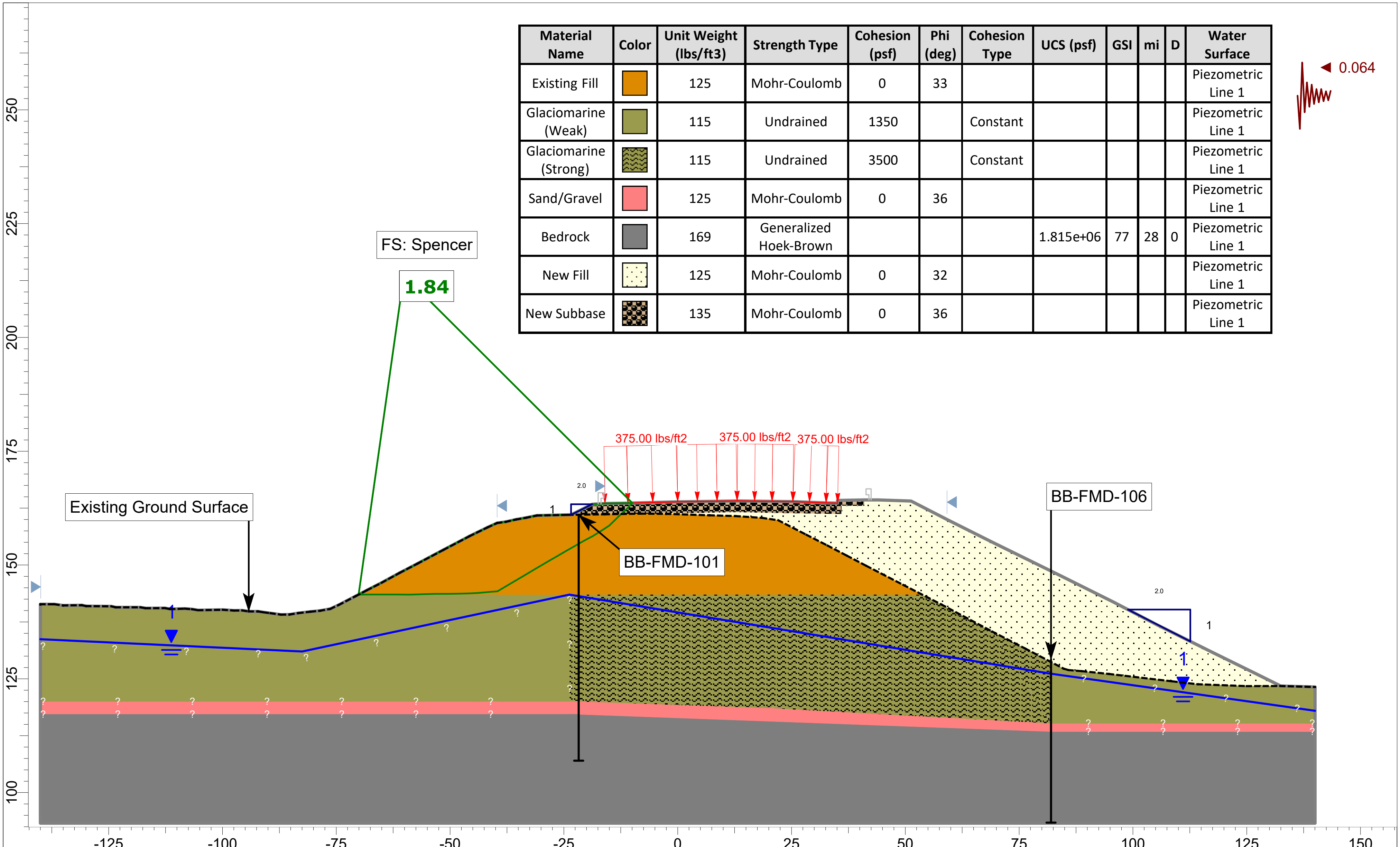
Project	19129538 I-295 MaineDOT Mallet Drive Bridge no 5721
Analysis Description	Phasing South - Station 29+75 North Slope (NonCircular, Cuckoo Search)
Drawn By KAR	*Checked By* MEL
Reviewed By MCM	*Scale* 1:350
Date 12/2/2020	*File Name* Mallet Drive Slide 29+75 _ KAR edits _ seismic.slmd

Figure B.2



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	UCS (psf)	GSI	mi	D	Water Surface
Existing Fill		125	Mohr-Coulomb	0	33						Piezometric Line 1
Glaciomarine (Weak)		115	Undrained	1350		Constant					Piezometric Line 1
Glaciomarine (Strong)		115	Undrained	3500		Constant					Piezometric Line 1
Sand/Gravel		125	Mohr-Coulomb	0	36						Piezometric Line 1
Bedrock		169	Generalized Hoek-Brown				1.815e+06	77	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

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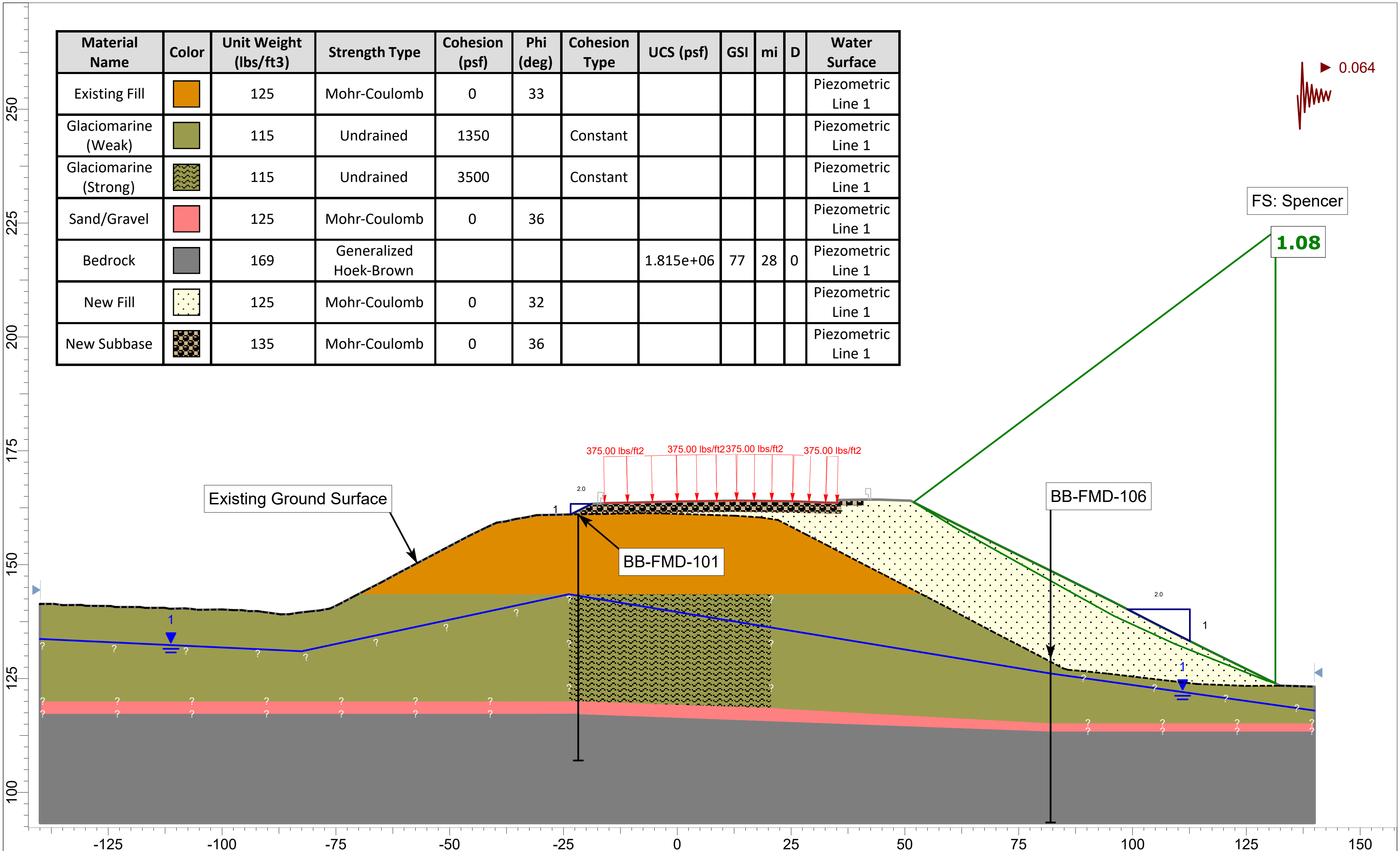
Project	19129538 I-295 MaineDOT Mallet Drive Bridge no 5721
Analysis Description	Phasing South - Station 29+75 North Slope (NonCircular, Cuckoo Search)
Drawn By	KAR
Checked By	MEL
Reviewed By	MCM
Scale	1:350
Date	12/2/2020
File Name	Mallet Drive Slide 29+75 _ KAR edits _ seismic.slmd

Figure B.3

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	UCS (psf)	GSI	mi	D	Water Surface
Existing Fill		125	Mohr-Coulomb	0	33						Piezometric Line 1
Glaciomarine (Weak)		115	Undrained	1350		Constant					Piezometric Line 1
Glaciomarine (Strong)		115	Undrained	3500		Constant					Piezometric Line 1
Sand/Gravel		125	Mohr-Coulomb	0	36						Piezometric Line 1
Bedrock		169	Generalized Hoek-Brown				1.815e+06	77	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



Project 19129538 I-295 MaineDOT Mallet Drive Bridge no 5721

Analysis Description Phasing South - Station 29+75 South Slope (NonCircular, Cuckoo Search)

Drawn By HTV	*Checked By* KAR	*Reviewed By* MCM	*Scale* 1:350
Date 12/2/2020	*File Name* Mallet Drive Slide 29+75 _ KAR edits _ seismic.slmd		

Figure B.4

GOLDER

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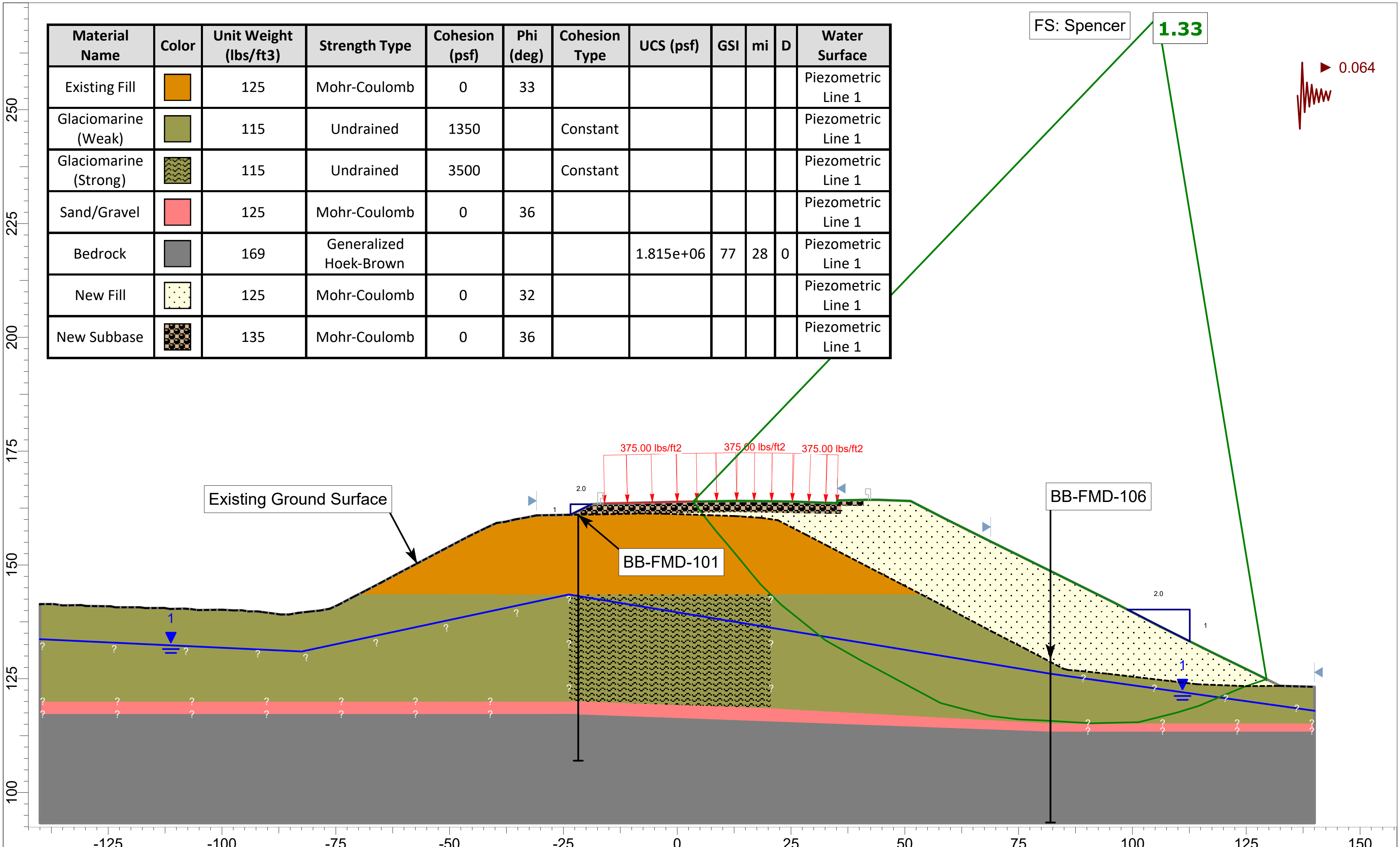


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	UCS (psf)	GSI	mi	D	Water Surface
Existing Fill		125	Mohr-Coulomb	0	33						Piezometric Line 1
Glaciomarine (Weak)		115	Undrained	1350		Constant					Piezometric Line 1
Glaciomarine (Strong)		115	Undrained	3500		Constant					Piezometric Line 1
Sand/Gravel		125	Mohr-Coulomb	0	36						Piezometric Line 1
Bedrock		169	Generalized Hoek-Brown				1.815e+06	77	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 19129538 I-295 MaineDOT Mallet Drive Bridge no 5721

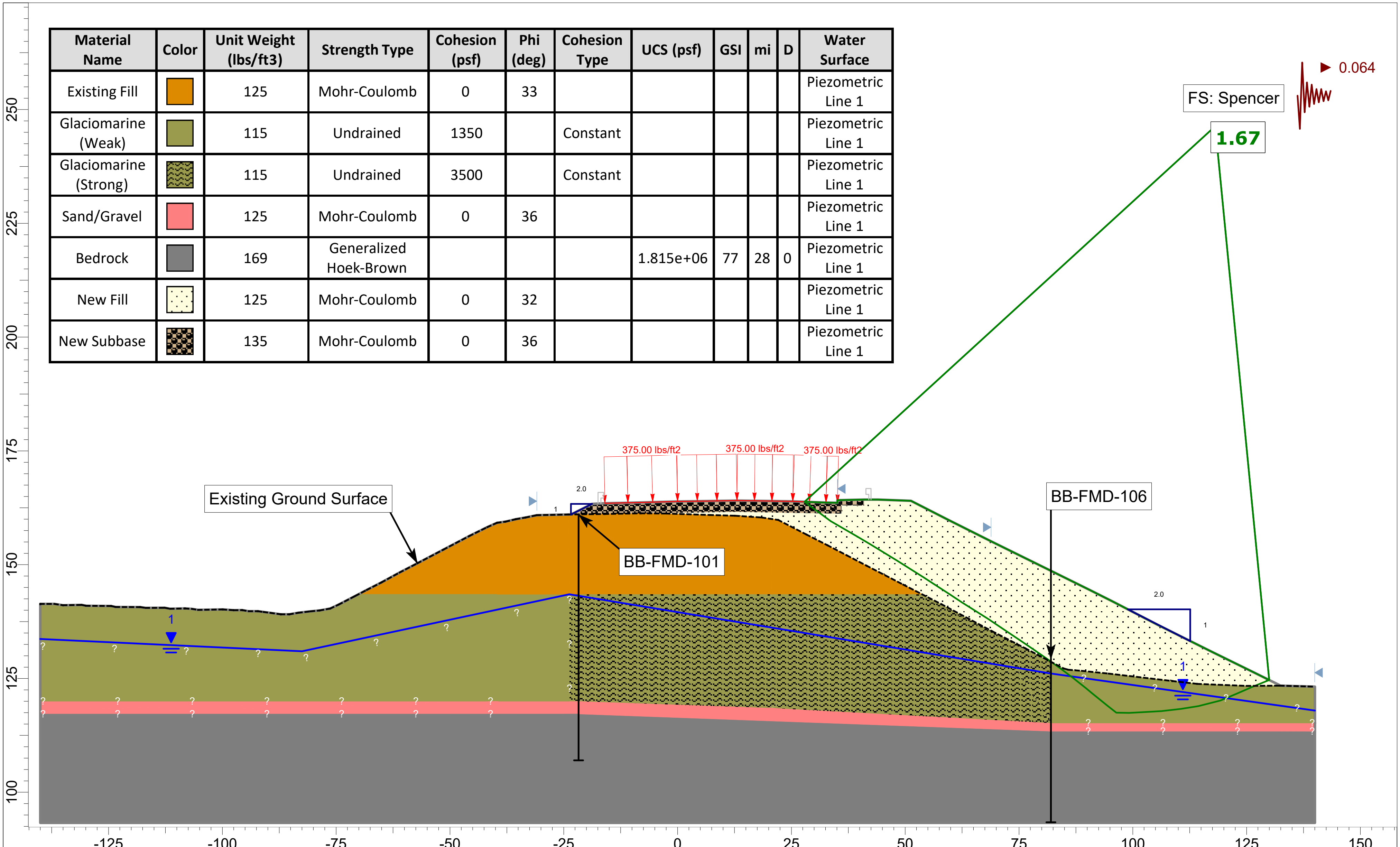
Analysis Description Phasing South - Station 29+75 South Slope (NonCircular, Cuckoo Search)

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

Date 12/2/2020 | *File Name* Mallet Drive Slide 29+75 _ KAR edits _ seismic.slmd

Figure B.5

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	UCS (psf)	GSI	mi	D	Water Surface
Existing Fill		125	Mohr-Coulomb	0	33						Piezometric Line 1
Glaciomarine (Weak)		115	Undrained	1350		Constant					Piezometric Line 1
Glaciomarine (Strong)		115	Undrained	3500		Constant					Piezometric Line 1
Sand/Gravel		125	Mohr-Coulomb	0	36						Piezometric Line 1
Bedrock		169	Generalized Hoek-Brown				1.815e+06	77	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



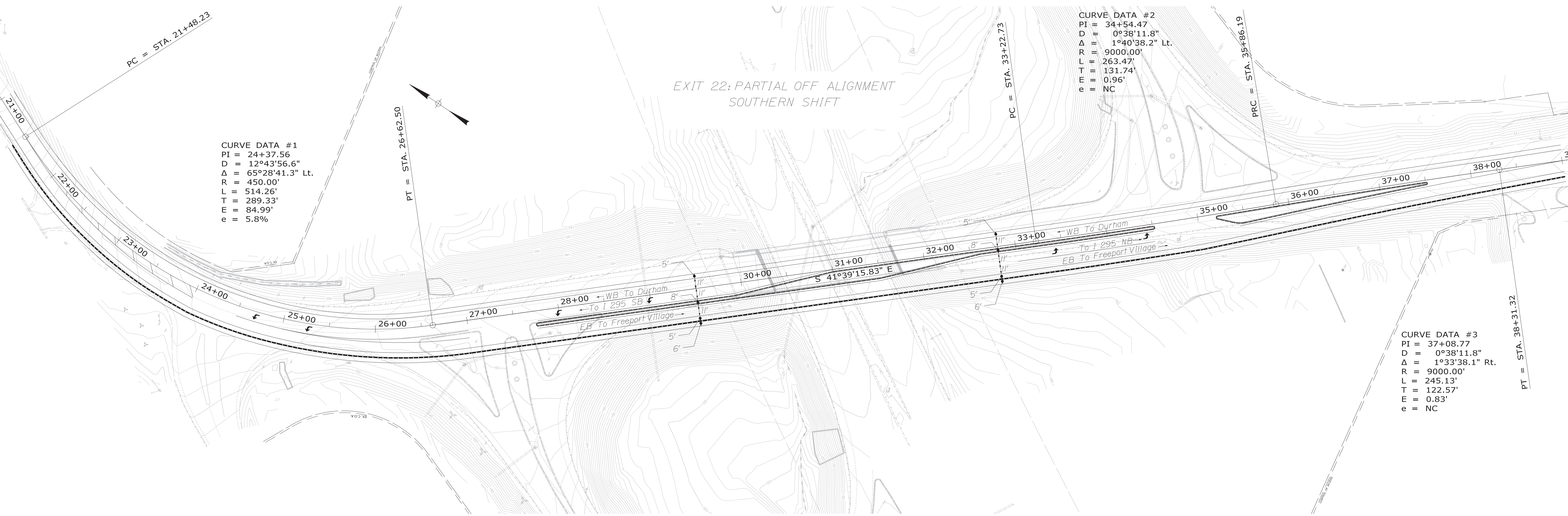
Project	19129538 I-295 MaineDOT Mallet Drive Bridge no 5721
Analysis Description	Phasing South - Station 29+75 South Slope (NonCircular, Cuckoo Search)
Drawn By KAR	*Checked By* MEL
Reviewed By MCM	*Scale* 1:350
Date 12/2/2020	*File Name* Mallet Drive Slide 29+75 _ KAR edits _ seismic.slmd

GOLDER

SLIDEINTERPRET 9.010

Figure B.6

Attachment 2



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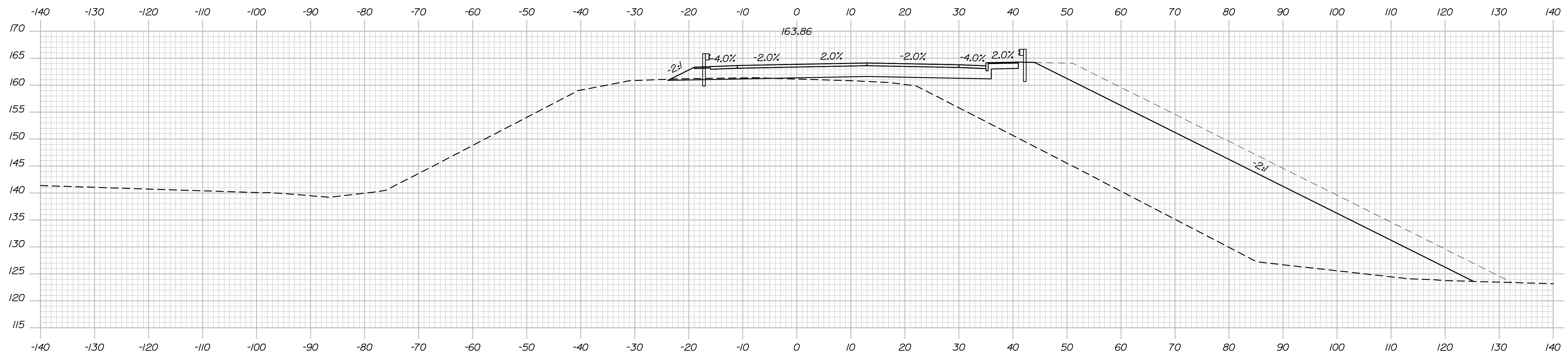

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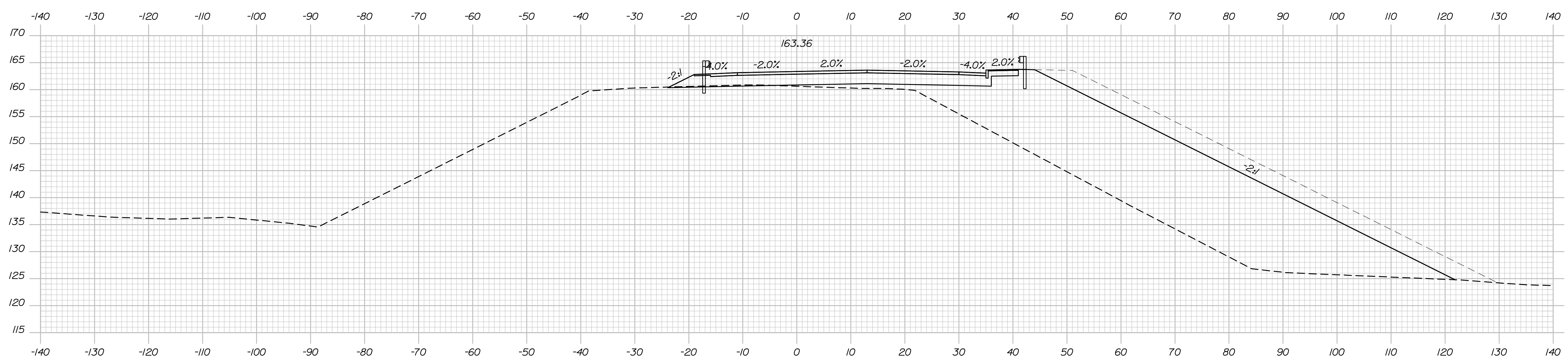

29+50.00

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

HIGHWAY PLANS

WIN
021726.00

Bridge No. 5721

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				

APPROACH ROAD BRIDGE
INTERSTATE 295

FREEPORT CUMBERLAND

MALLET DRIVE SOUTH
CROSS SECTIONS

SHEET NUMBER

Wor

OF 4

Sta. 29+50.00 to Sta. 29+75.00

GOLDER

CALCULATIONS

Date:	12/3/2020	**Made by:**	KAR
Project No.:	19129538	**Checked by:**	BK
Subject:	Global Stability Analysis Abutment No. 1	**Reviewed by:**	MCM
Project Short Title:	I-295 Mallet Drive Bridge Replacement no 5721		

OBJECTIVE

Calculate global factor of safety for Abutment No.1 (the northwestern proposed abutment), assuming the "Phasing South" 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 23, 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. Approach Road Bridge Freeport Interstate 295: Mallet Drive 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 17, 2020 (Appendix C, Preliminary Geotechnical Design Report, dated September 2020).
8. Golder summary of rock laboratory test results (Table 5, Preliminary Geotechnical Design Report, dated September 2020).
9. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.
10. Grading schematic provided by HNTB, file name "19129538 Freeport I-295 Exit 22 Southeast (Abutment No. 2) Abutment Cross-Section_HNTB_edits.pdf", dated August 19, 2020.
11. Golder calculation titled "Seismic Site Class" (Appendix E, Preliminary Geotechnical Design Report, dated September 2020).
12. Rocscience Slide Software Package Version 2020 9.010 64-bit, build date Oct 14, 2020.

ATTACHMENTS

1. Slide output figures
2. HNTB plans showing "phasing south" 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 9. A pseudo-static FS > 1.0 is recommended per Section 3.7.4.1 in Reference 9.
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 References 4 and 10.
6. Undrained conditions ($\phi = 0$) were assumed for the clay layers.
7. This conservative preliminary analysis did not account for the pile foundations proposed to support the abutment as the pile size and spacing were not defined at the time of this analysis.

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
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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 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 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 8). 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 9, 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	33	-	-	-	-
Glaciomarine	115	Undrained	1350	0				
Sand and Gravel	125	Mohr-Coulomb	0	36	-	-	-	-
Bedrock	169	Generalized Hoek-Brown	-	-	1,815,000	77	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. At the time of this analysis, MaineDOT/HNTB planned to construct the phasing south option. Therefore the phasing south option was analyzed as the critical case. This analysis evaluates the fills at Abutment No. 1 since the underlying foundation clays are the thickest at that location. 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	
Phasing South	A-A'	1	1.35	(Fig. A.1)	2.14	(Fig. A.2)

Circular Surfaces:

CALCULATIONS

Date:	12/3/2020	**Made by:**	KAR
Project No.:	19129538	**Checked by:**	BK
Subject:	Global Stability Analysis Abutment No. 1	**Reviewed by:**	MCM
Project Short Title:	I-295 Mallet Drive Bridge Replacement no 5721		

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 were 1.43 for surfaces through the proposed fill and ranged from 2.27 to 2.38 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 11) 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	1.21 (Fig. B.1)	1.88 (Fig. B.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 1.26 to 1.27 for surfaces through the proposed and existing fill and from 1.98 to 2.04 for surfaces through the glaciomarine deposit.

CONCLUSIONS

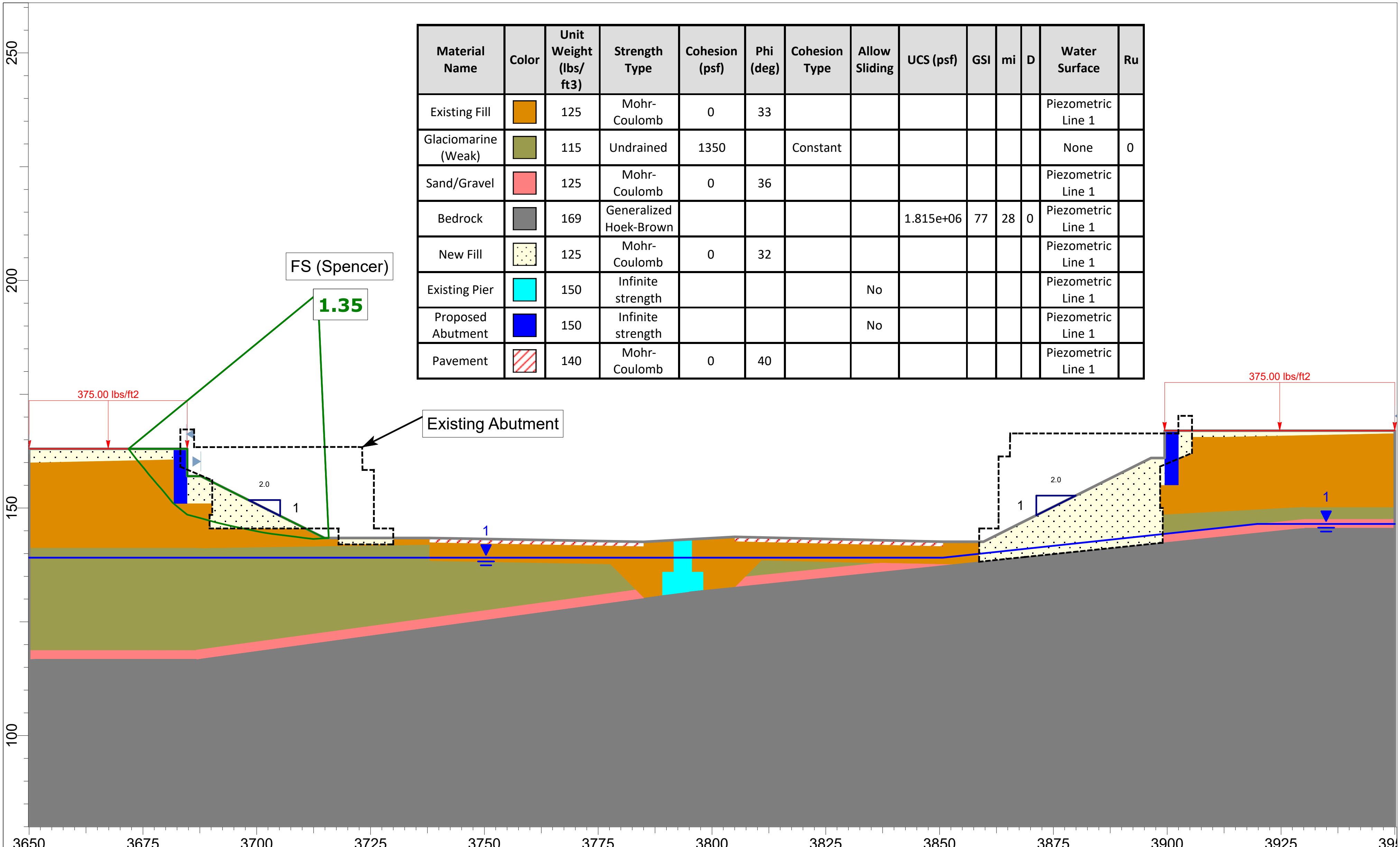
The proposed abutment and slope grading system produces a global stability factor of safety less than the recommended factor of safety of 1.5 for potential slope failures in the existing and proposed fill when using proposed fill engineering parameters recommended in the MaineDOT Bridge Design Guide 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.

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)	Cohesion Type	Allow Sliding	UCS (psf)	GSI	mi	D	Water Surface	Ru
Existing Fill		125	Mohr-Coulomb	0	33							Piezometric Line 1	
Glaciomarine (Weak)		115	Undrained	1350		Constant						None	0
Sand/Gravel		125	Mohr-Coulomb	0	36							Piezometric Line 1	
Bedrock		169	Generalized Hoek-Brown					1.815e+06	77	28	0	Piezometric Line 1	
New Fill		125	Mohr-Coulomb	0	32							Piezometric Line 1	
Existing Pier		150	Infinite strength				No					Piezometric Line 1	
Proposed Abutment		150	Infinite strength				No					Piezometric Line 1	
Pavement		140	Mohr-Coulomb	0	40							Piezometric Line 1	

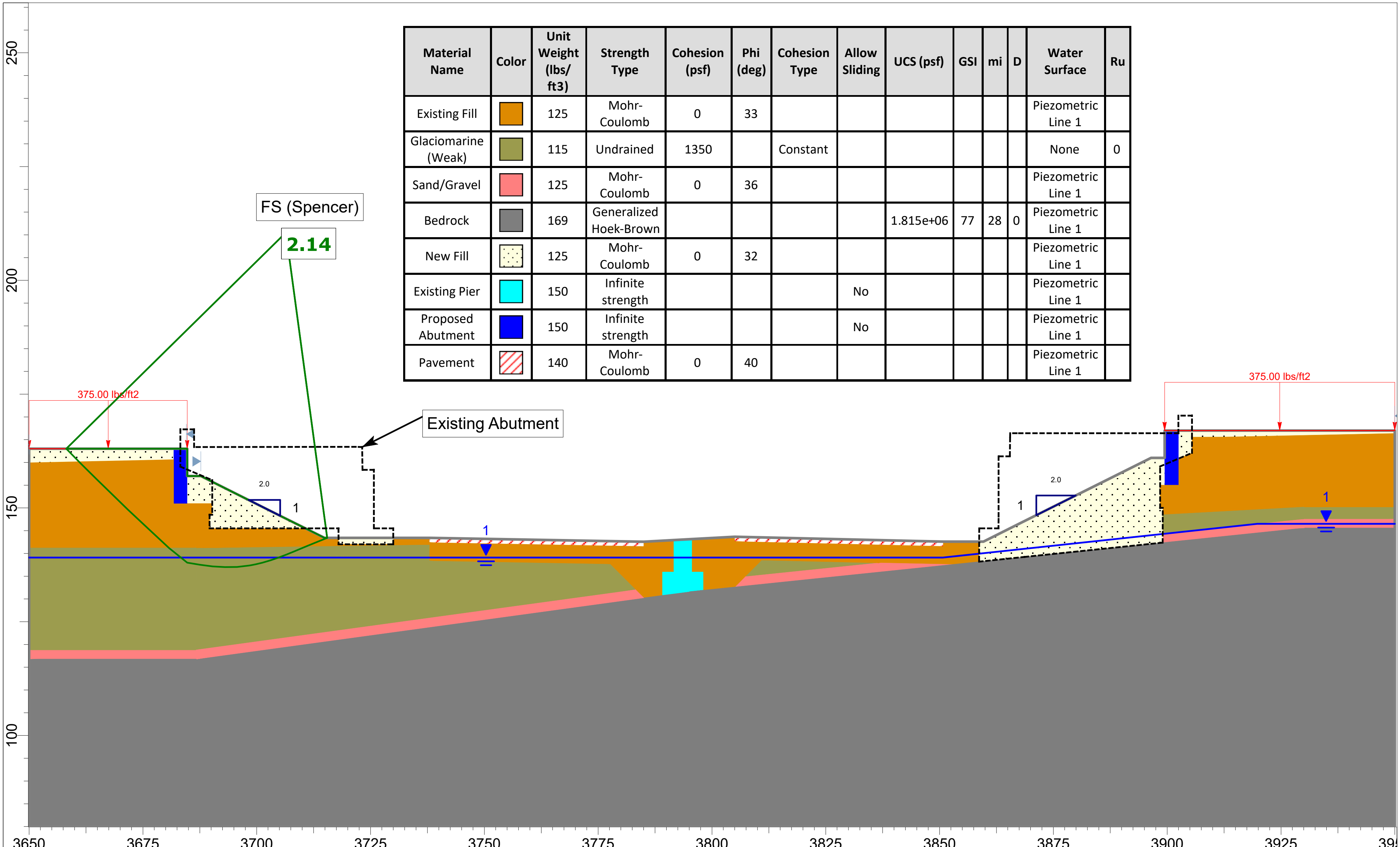


SLIDEINTERPRET 9.010

GOLDER

Project	19129538 I-295 MaineDOT Mallet Drive Bridge no 5721
Analysis Description	Phasing South - 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* Profile A-A' Mallet Drive Freeport Exit 22 19129538.slmd

Figure A.1



Material Name	Color	Unit Weight (lbs/ ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Allow Sliding	UCS (psf)	GSI	mi	D	Water Surface	Ru
Existing Fill		125	Mohr-Coulomb	0	33							Piezometric Line 1	
Glaciomarine (Weak)		115	Undrained	1350		Constant						None	0
Sand/Gravel		125	Mohr-Coulomb	0	36							Piezometric Line 1	
Bedrock		169	Generalized Hoek-Brown					1.815e+06	77	28	0	Piezometric Line 1	
New Fill		125	Mohr-Coulomb	0	32							Piezometric Line 1	
Existing Pier		150	Infinite strength				No					Piezometric Line 1	
Proposed Abutment		150	Infinite strength				No					Piezometric Line 1	
Pavement		140	Mohr-Coulomb	0	40							Piezometric Line 1	

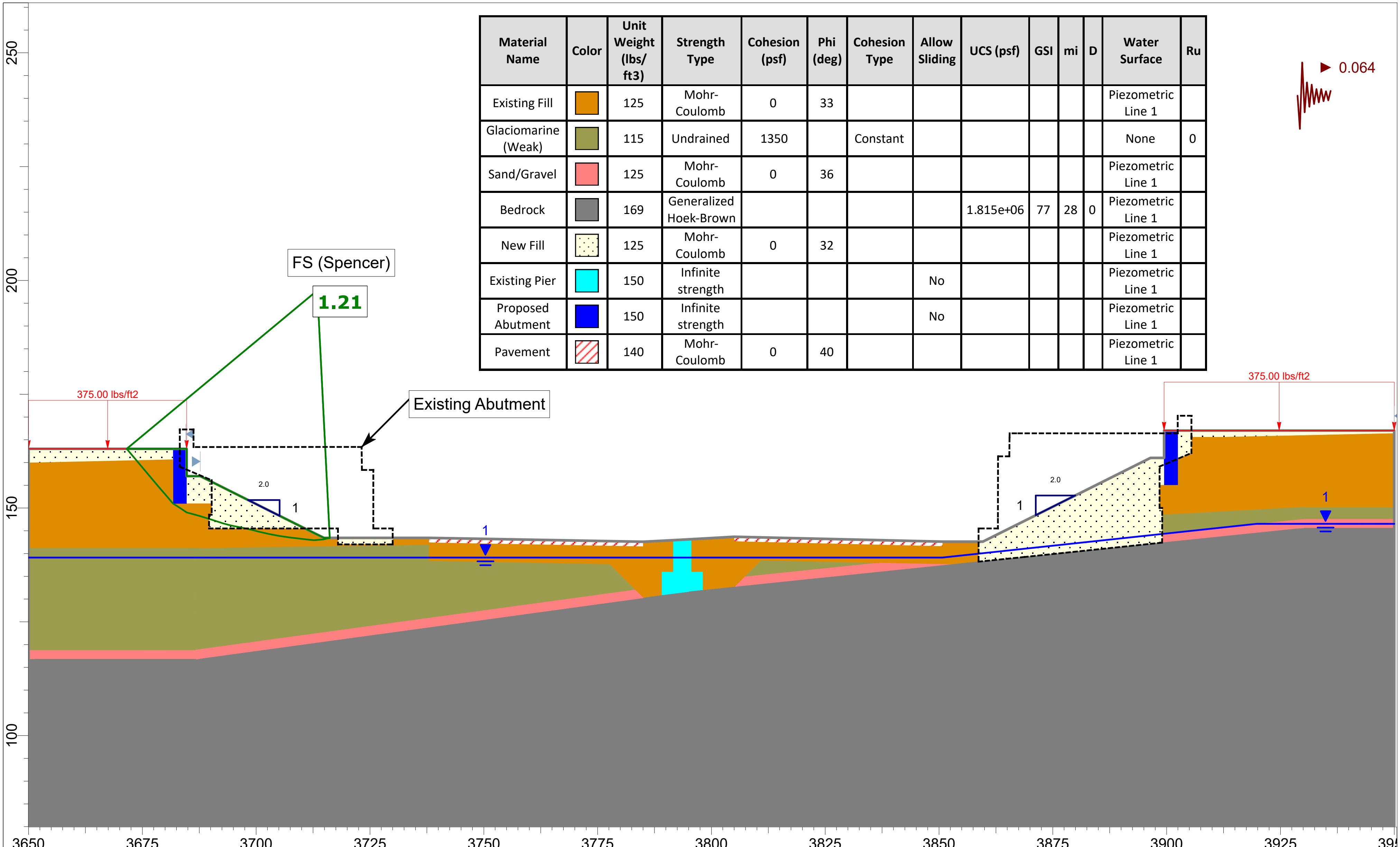
GOLDER

SLIDEINTERPRET 9.010

Project	19129538 I-295 MaineDOT Mallet Drive Bridge no 5721
Analysis Description	Phasing South - 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* Profile A-A' Mallet Drive Freeport Exit 22 19129538.slmd

Figure A.2

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Allow Sliding	UCS (psf)	GSI	mi	D	Water Surface	Ru
Existing Fill		125	Mohr-Coulomb	0	33							Piezometric Line 1	
Glaciomarine (Weak)		115	Undrained	1350		Constant						None	0
Sand/Gravel		125	Mohr-Coulomb	0	36							Piezometric Line 1	
Bedrock		169	Generalized Hoek-Brown					1.815e+06	77	28	0	Piezometric Line 1	
New Fill		125	Mohr-Coulomb	0	32							Piezometric Line 1	
Existing Pier		150	Infinite strength				No					Piezometric Line 1	
Proposed Abutment		150	Infinite strength				No					Piezometric Line 1	
Pavement		140	Mohr-Coulomb	0	40							Piezometric Line 1	



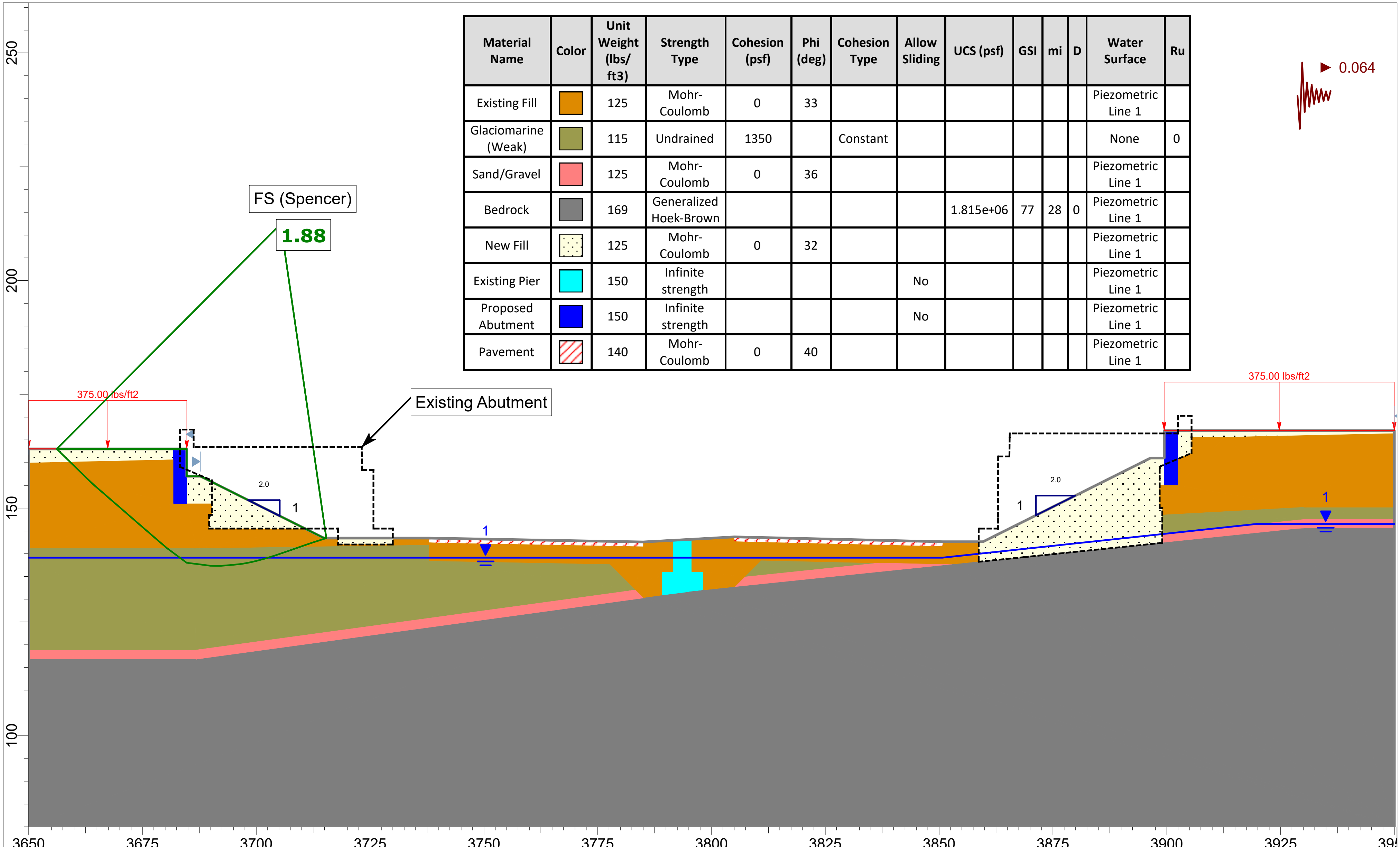
GOLDER

SLIDEINTERPRET 9.010

Project	19129538 I-295 MaineDOT Mallet Drive Bridge no 5721
Analysis Description	Phasing South - 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* Profile A-A' Mallet Drive Freeport Exit 22 19129538.slmd

Figure B.1

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Allow Sliding	UCS (psf)	GSI	mi	D	Water Surface	Ru
Existing Fill		125	Mohr-Coulomb	0	33							Piezometric Line 1	
Glaciomarine (Weak)		115	Undrained	1350		Constant						None	0
Sand/Gravel		125	Mohr-Coulomb	0	36							Piezometric Line 1	
Bedrock		169	Generalized Hoek-Brown					1.815e+06	77	28	0	Piezometric Line 1	
New Fill		125	Mohr-Coulomb	0	32							Piezometric Line 1	
Existing Pier		150	Infinite strength				No					Piezometric Line 1	
Proposed Abutment		150	Infinite strength				No					Piezometric Line 1	
Pavement		140	Mohr-Coulomb	0	40							Piezometric Line 1	



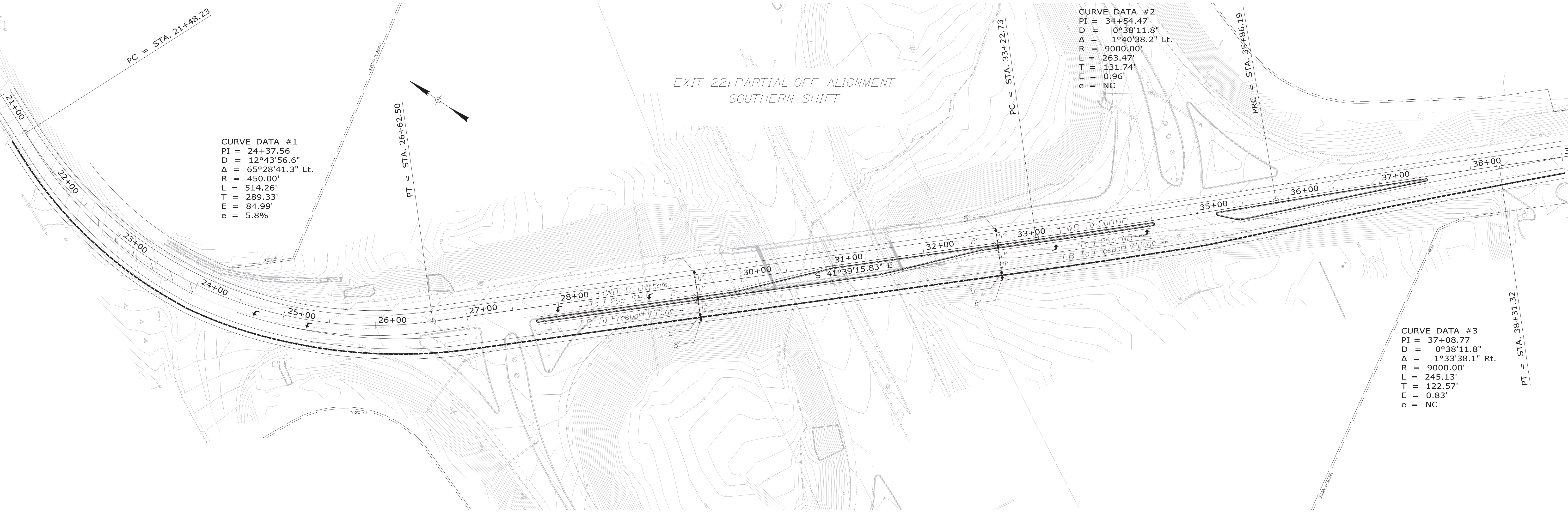
Project	19129538 I-295 MaineDOT Mallet Drive Bridge no 5721
Analysis Description	Phasing South - Profile A-A', Proposed Abutment 1 (NonCircular, Cuckoo Search)
Drawn By: KAR	Checked By: BK
Reviewed By: MCM	Scale: 1:350
Date: 12/7/2020	File Name: Profile A-A' Mallet Drive Freeport Exit 22 19129538.slmd

GOLDER

SLIDEINTERPRET 9.010

Figure B.2

Attachment 2



APPENDIX E4

Settlement

GOLDER

CALCULATIONS

Date:	7/8/2020	**Made by:**	KAR
Project No.:	19129538	**Checked by:**	MEL
Subject:	Settlement at Abutment No. 1 Bridge Embankment - Phasing South	**Reviewed by:**	MCM
Project Short Title:	MaineDOT I-295 Exit 22 Mallet Drive Bridge Replacement No. 5721		

OBJECTIVE

Calculate the primary settlement at the base of the northwestern proposed bridge approach embankment (assuming the "phasing south" option) at a location where embankment fill is thickest (42 feet southwest of phasing south alignment at HNTB Station 29+75).

REFERENCES

1. Golder calculation titled "Global Stability Analysis", dated July 8, 2020 (Appendix E, Preliminary Geotechnical Design Report, dated September 2020).
2. GeoTesting Express laboratory testing results, dated February 4, 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. Approach Road Bridge Freeport Interstate 295: Mallet Drive 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. = 149.5 ft
WL elev. = 132.7 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 = 1.00
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:

$N_{60,Fill}$ = 30
$N_{60,S/G}$ = 35

4. The proposed top of road grade and base of embankment elevations at the location of analysis are (Reference 4, Section 29+75):

El. road = 164.3 ft
El. base = 149.5 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 149.5 ft (base of embankment elevation).

Existing Conditions (Reference 1):

Total σ'_v at Elev. 149.5 ft (psf): $\boxed{0}$

Date:	7/8/2020	**Made by:**	KAR
Project No.:	19129538	**Checked by:**	MEL
Subject:	Settlement at Abutment No. 1 Bridge Embankment - Phasing South	**Reviewed by:**	MCM
Project Short Title:	MaineDOT I-295 Exit 22 Mallet Drive Bridge Replacement No. 5721		

After Construction (Reference 4, Section 29+75):

Layer	Unit weight (pcf)	Elevation (ft)		Thickness (ft)	Stress Contribution of Layer (psf)
		Top	Bottom		
New Fill	125	164.3	149.5	14.8	1850

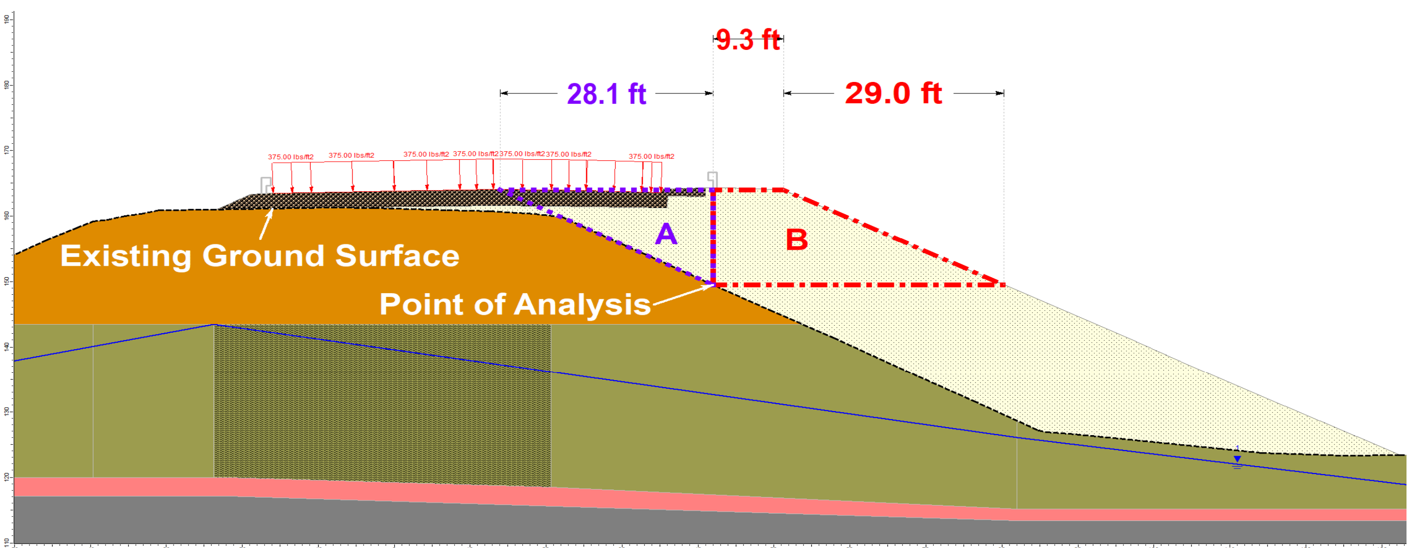
Calculate the increase or decrease in effective stress as a result of construction.

	σ_v at Elev. 149.5 ft (psf)	σ'_v at Elev. 149.5 ft (psf)	$\Delta\sigma'_v$ at Elev. 149.5 ft (psf)	Result
Existing conditions	0	0	1850	Settlement
After construction	1850	1850		

(Water table is below Elev. 149.5 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				
q_0 = stress applied by fill loading	1850	1850	psf	(Part A)
a = dimension of embankment slope, ft	28.1	29.0	ft	(Ref. 4)
b = dimension of embankment top, ft	0.0	9.3	ft	(Ref. 4)
z = depth to midpoint of layer, ft				

GOLDER

CALCULATIONS

Date:	7/8/2020	**Made by:**	KAR
Project No.:	19129538	**Checked by:**	MEL
Subject:	Settlement at Abutment No. 1 Bridge Embankment - Phasing South	**Reviewed by:**	MCM
Project Short Title:	MaineDOT I-295 Exit 22 Mallet Drive Bridge Replacement No. 5721		

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	0-6	6.0	3.0	9.4	0.0	0.468	866
Glaciomarine	2	6-16	10.0	11.0	2.6	0.0	0.380	703
Glaciomarine	3	16-26	10.0	21.0	1.3	0.0	0.285	527
Glaciomarine	4	26-32.1	6.1	29.1	1.0	0.0	0.250	463
Sand and Gravel	5	32.1-34.6	2.5	33.4	0.8	0.0	0.217	401

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	0-6	6.0	3.0	9.7	3.1	0.500	925
Glaciomarine	2	6-16	10.0	11.0	2.6	0.8	0.467	864
Glaciomarine	3	16-26	10.0	21.0	1.4	0.4	0.400	740
Glaciomarine	4	26-32.1	6.1	29.1	1.0	0.3	0.350	648
Sand and Gravel	5	32.1-34.6	2.5	33.4	0.9	0.3	0.340	629

Total Footing (Trapezoid A + Trapezoid B):

Layer		Depth below footing (ft)	Layer Thickness (ft)	z (ft)	Stress Increase (psf)
Existing Fill	1	0-6	6.0	3.0	1791
Glaciomarine	2	6-16	10.0	11.0	1567
Glaciomarine	3	16-26	10.0	21.0	1267
Glaciomarine	4	26-32.1	6.1	29.1	1110
Sand and Gravel	5	32.1-34.6	2.5	33.4	1030

B. Use consolidation theory to estimate settlement of the glaciomarine silty clay layers beneath the embankment (Layers 2-4); use the Hough method to estimate settlement of the existing fill and sand/gravel layers beneath the embankment (Layers 1 and 5).

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 5, 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 22 Mallet Drive Bridge Replacement No. 5721		

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	6.0	125	375	18.0	30	60	140
Glaciomarine	2	10.0	115	1325	63.4	Not required for clay consolidation analysis		
Glaciomarine	3	10.0	115	2213	105.9			
Glaciomarine	4	6.1	115	2636	126.2			
Sand and Gravel	5	2.5	125	2875	137.6	35	29	100

2 Calculate the total settlement of the existing fill and sand/gravel (Layers 1 and 5) 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	0.033
	in	0.39

ΔH_i (Layer 5)	ft	0.003
	in	0.04

3 Calculate the total settlement of the glaciomarine silty clay (Layers 2-4) 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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Subject:	Settlement at Abutment No. 1 Bridge Embankment - Phasing South	**Reviewed by:**	MCM
Project Short Title:	MaineDOT I-295 Exit 22 Mallet Drive Bridge Replacement No. 5721		

where:			Layer 2	Layer 3	Layer 4
H_0	initial height of layer i, ft		10.0	10.0	6.1
$\Delta\sigma_v$	surcharge load, psf		1567	1267	1110
σ'_{v0}	in situ vertical effective stress, psf		1325	2213	2636
$\sigma'_{v0} + \Delta\sigma_v$			2892	3480	3746
σ'_p	preconsolidated stress, psf		N/A	N/A	N/A
C_c	compression index, $C_c = C_{c\varepsilon}(1+e_0)$		0.50	0.50	0.50
C_r	recompression index, $C_r = C_{r\varepsilon}(1+e_0)$		0.04	0.04	0.04
e_0	initial void ratio		1.00	1.00	1.00
	Use equation (Assumption 5):		8-16	8-16	8-16
ΔH_i		ft	0.068	0.039	0.019
		in	0.81	0.47	0.22

	Settlement Based on Calc. Loading Stress
Layer	ΔH_i (in)
1	0.39
2	0.81
3	0.47
4	0.22
5	0.04
Total Settlement (in)	1.93

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 having a higher permeability relative to the clay (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)

Date:	7/8/2020	**Made by:**	KAR
Project No.:	19129538	**Checked by:**	MEL
Subject:	Settlement at Abutment No. 1 Bridge Embankment - Phasing South	**Reviewed by:**	MCM
Project Short Title:	MaineDOT I-295 Exit 22 Mallet Drive Bridge Replacement No. 5721		

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: 26.1 ft
With double drainage conditions, H_{dr} = 13.1 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} = 13.1 ft
T_v = 1.13

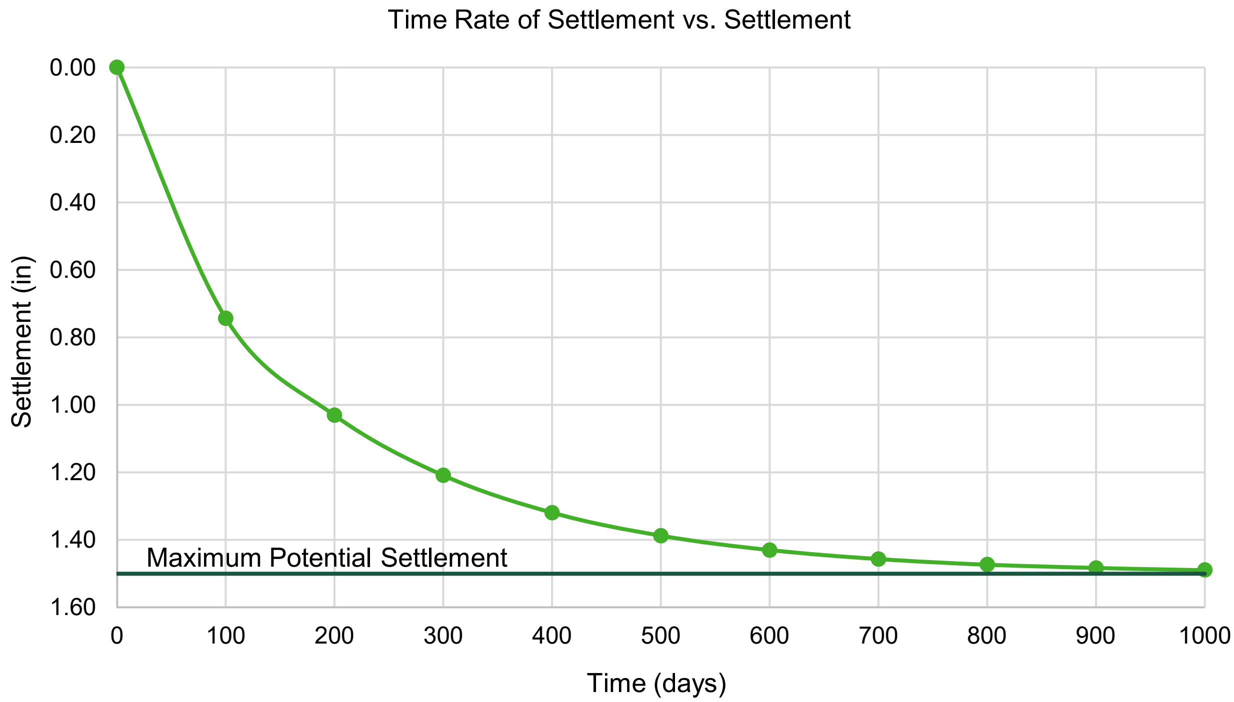
t =	585	days =	1.60	years	(95% consolidation)

s_c (in) = 1.50 (final potential settlement of clay layers, Part B)

Time (days)	Time (years)	T_v	U_{avg} (%)	s(t) (in)
0	0.00	0.00	0.00	0.00
100	0.27	0.19	49.58	0.74
200	0.55	0.39	68.73	1.03
300	0.82	0.58	80.58	1.21
400	1.10	0.77	87.94	1.32
500	1.37	0.97	92.51	1.39
600	1.64	1.16	95.35	1.43
700	1.92	1.35	97.11	1.46
800	2.19	1.54	98.21	1.47
900	2.46	1.74	98.89	1.48
1000	2.74	1.93	99.31	1.49

CALCULATIONS

Date:	7/8/2020	**Made by:**	KAR
Project No.:	19129538	**Checked by:**	MEL
Subject:	Settlement at Abutment No. 1 Bridge Embankment - Phasing South	**Reviewed by:**	MCM
Project Short Title:	MaineDOT I-295 Exit 22 Mallet Drive Bridge Replacement No. 5721		



CONCLUSIONS

The primary settlement at the base of the northwestern proposed bridge approach embankment after construction (assuming the "phasing south" option) is estimated to be 1.93 inches. This includes 0.43 inches of immediate settlement and 1.50 inches of consolidation settlement that is estimated to reach 95% consolidation in 1.6 years.

CALCULATIONS

Date:	8/20/2020	**Made by:**	KAR
Project No.:	19129538	**Checked by:**	MLM
Subject:	Settlement at Abutment No. 2 Bridge Embankment - Phasing South	**Reviewed by:**	MCM
Project Short Title:	MaineDOT I-295 Exit 22 Mallet Drive Bridge Replacement No. 5721		

OBJECTIVE

Calculate the primary settlement at the base of the southeastern proposed bridge approach embankment (assuming the "phasing south" option) at a location where embankment fill is thickest (42 feet southwest of phasing south alignment at HNTB Station 32+25).

REFERENCES

1. Golder interpreted subsurface section A-A' (Figure 3, Preliminary Geotechnical Design Report, dated September
2. GeoTesting Express laboratory testing results, dated February 4, 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. Approach Road Bridge Freeport Interstate 295: Mallet Drive 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.

ATTACHMENTS

1. Interpreted subsurface profile along Station 32+25

ASSUMPTIONS

1. The existing stratigraphy and water table surface are estimated from samples and groundwater measurements encountered during drilling (Reference 3) and interpreted in Attachment 1.

GS elev. = 154.5 ft
WL elev. = 143.0 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 = 1.00
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:

$N_{60,Fill}$ = 30
$N_{60,S/G}$ = 35

4. The proposed top of road grade and base of embankment elevations at the location of analysis are (Reference 4, Section 32+25):

El. road = 168.5 ft
El. base = 154.5 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 154.5 ft (base of embankment elevation).

CALCULATIONS

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Project Short Title:	MaineDOT I-295 Exit 22 Mallet Drive Bridge Replacement No. 5721		

Existing Conditions (Reference 1):

Total σ'_v at Elev. 154.5 ft (psf): 0

After Construction (Reference 4, Section 32+25):

Layer	Unit weight (pcf)	Elevation (ft)		Thickness (ft)	Stress Contribution of Layer (psf)
		Top	Bottom		
New Fill	125	168.5	154.5	14.0	1750

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

	σ_v at Elev. 154.5 ft (psf)	σ'_v at Elev. 154.5 ft (psf)	$\Delta\sigma'_v$ at Elev. 154.5 ft (psf)	Result
Existing conditions	0	0	1750	Settlement
After construction	1750	1750		

(Water table is below Elev. 154.5 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. A section view of the assumed fill loading is depicted in Attachment 1.

$\sigma_z = q_0 \times I$ — 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 fill loading	1750	1750	psf	(Part A)
a = dimension of embankment slope, ft	27.5	27.5	ft	(Attachment 1)
b = dimension of embankment top, ft	0.0	9.0	ft	(Attachment 1)
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	3.0	1.5	18.3	0.0	0.470	823
Glaciomarine	2	3-11	8.0	7.0	3.9	0.0	0.420	735
Sand and Gravel	3	11-12.5	1.5	11.8	2.3	0.0	0.360	630

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	3.0	1.5	18.3	6.0	0.500	875

Glaciomarine	2	3-11	8.0	7.0	3.9	1.3	0.490	858
Sand and Gravel	3	11-12.5	1.5	11.8	2.3	0.8	0.465	814

Total Footing (Trapezoid A + Trapezoid B):

Layer		Depth below footing (ft)	Layer Thickness (ft)	z (ft)	Stress Increase (psf)
Existing Fill	1	0-3	3.0	1.5	1698
Glaciomarine	2	3-11	8.0	7.0	1593
Sand and Gravel	3	11-12.5	1.5	11.8	1444

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

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	3.0	125	188	9.0	30	60	140
Glaciomarine	2	8.0	115	835	40.0	Not required for clay consol.		
Sand and Gravel	3	1.5	125	1373	65.7	35	42	137

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	0.021
	in	0.26

ΔH_i (Layer 3)	ft	0.003
	in	0.04

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, $\sigma'_{v0} + \Delta\sigma_v > \sigma'_p$

where:

			Layer 2
H_0	initial height of layer i, ft		8.0
$\Delta\sigma_v$	surcharge load, psf		1593
σ'_{v0}	in situ vertical effective stress, psf		835
$\sigma'_{v0} + \Delta\sigma_v$			2428
σ'_p	preconsolidated stress, psf		N/A
C_c	compression index, $C_c = C_{c\varepsilon}(1+e_0)$		0.50
C_r	recompression index, $C_r = C_{r\varepsilon}(1+e_0)$		0.04
e_0	initial void ratio		1.00
	Use equation (Assumption 5):		8-16
ΔH_i		ft	0.074
		in	0.89

	Settlement Based on Calc. Loading Stress
Layer	ΔH_i (in)
1	0.26
2	0.89
3	0.04
Total Settlement (in)	1.19

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 having a higher permeability relative to the clay (Reference 3).

$$T_v = \frac{c_V t}{H^2_{dr}}$$ (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: 8.0 ft
With double drainage conditions, H_{dr} = 4.0 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} = 4.0 ft
T_v = 1.13

t =	55	days =	0.15	years	(95% consolidation)

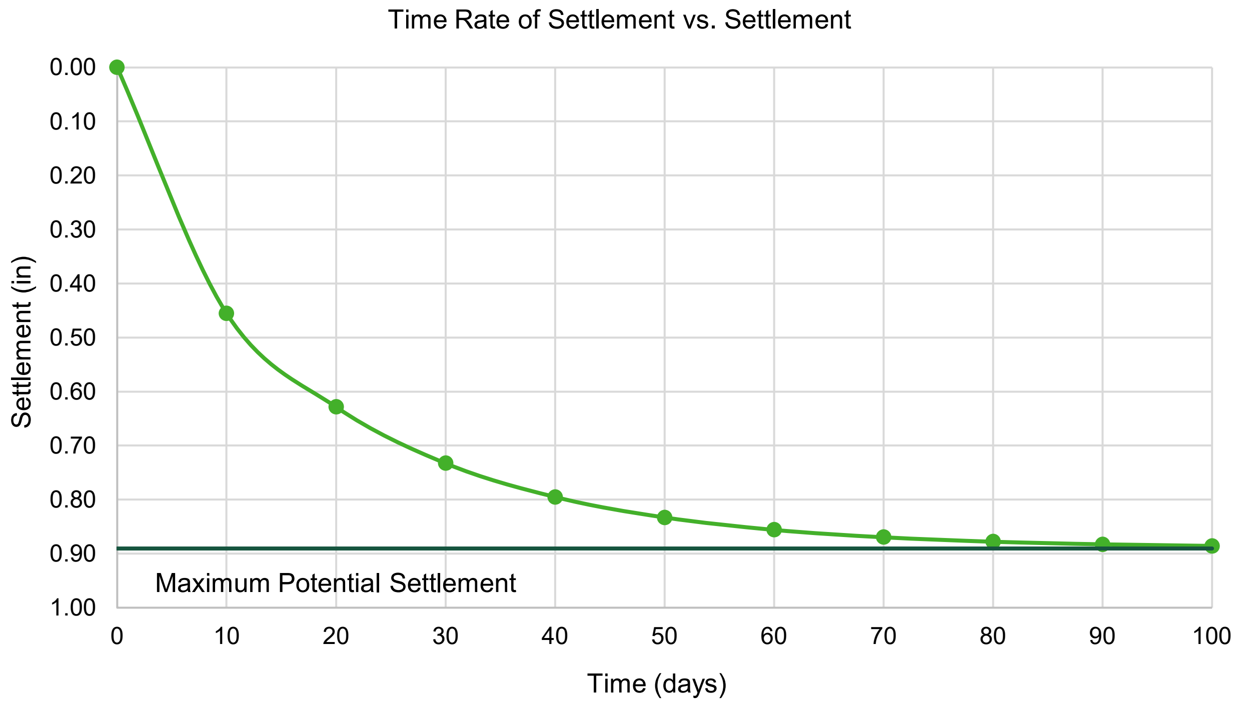
s_c (in) = 0.89 (final potential settlement of clay layer, Part B)

GOLDER

CALCULATIONS

Date:	8/20/2020	**Made by:**	KAR
Project No.:	19129538	**Checked by:**	MLM
Subject:	Settlement at Abutment No. 2 Bridge Embankment - Phasing South	**Reviewed by:**	MCM
Project Short Title:	MaineDOT I-295 Exit 22 Mallet Drive Bridge Replacement No. 5721		

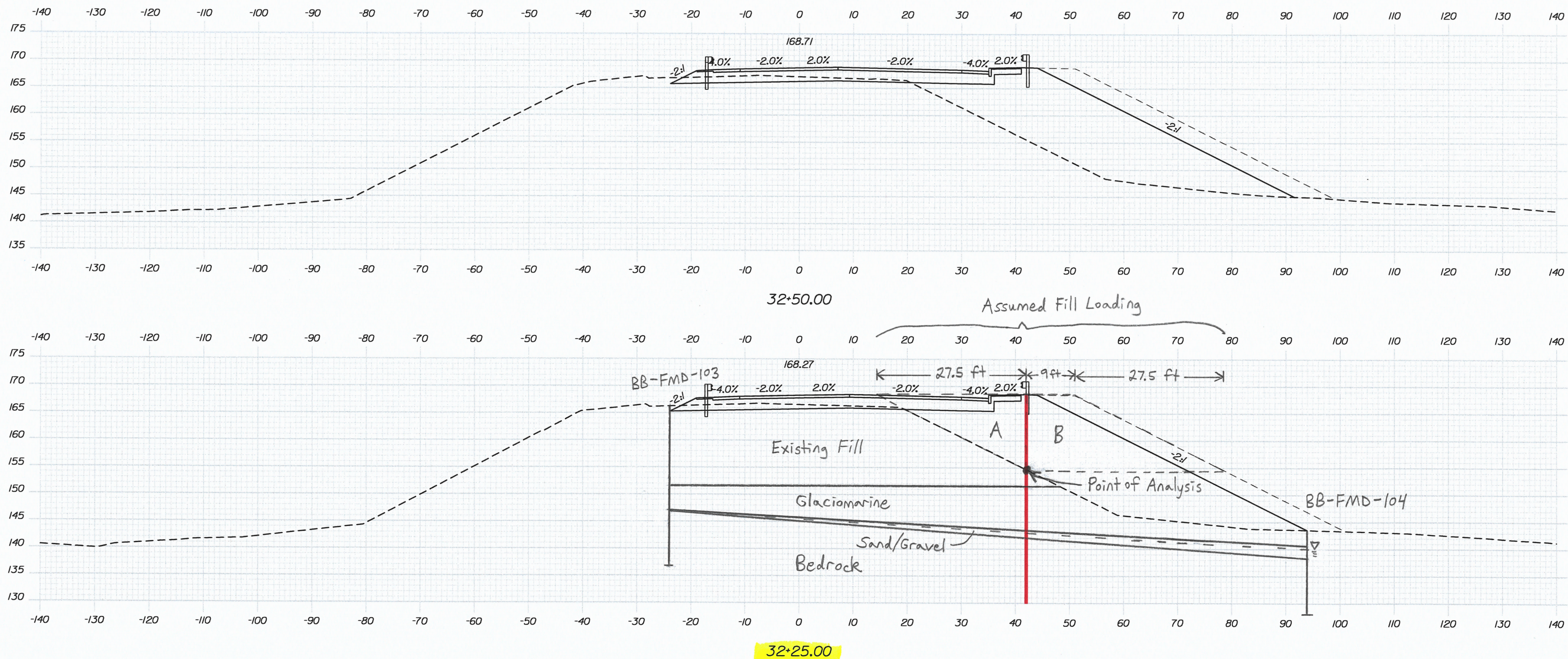
Time (days)	Time (years)	T_v	U_{avg} (%)	s(t) (in)
0	0.00	0.00	0.00	0.00
10	0.03	0.21	51.15	0.46
20	0.05	0.41	70.59	0.63
30	0.08	0.62	82.29	0.73
40	0.11	0.82	89.34	0.80
50	0.14	1.03	93.58	0.83
60	0.16	1.23	96.13	0.86
70	0.19	1.44	97.67	0.87
80	0.22	1.64	98.60	0.88
90	0.25	1.85	99.16	0.88
100	0.27	2.05	99.49	0.89



CONCLUSIONS

The primary settlement at the base of the southeastern proposed bridge approach embankment after construction (assuming the "phasing south" option) is estimated to be 1.19 inches. This includes 0.30 inches of immediate settlement and 0.89 inches of consolidation settlement that is estimated to reach 95% consolidation in 55 days.

Attachment 1



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Division:
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STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

WIN
021726.00

Bridge No. 5721

HIGHWAY PLANS

SIGNATURE

P.E. NUMBER

DATE

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

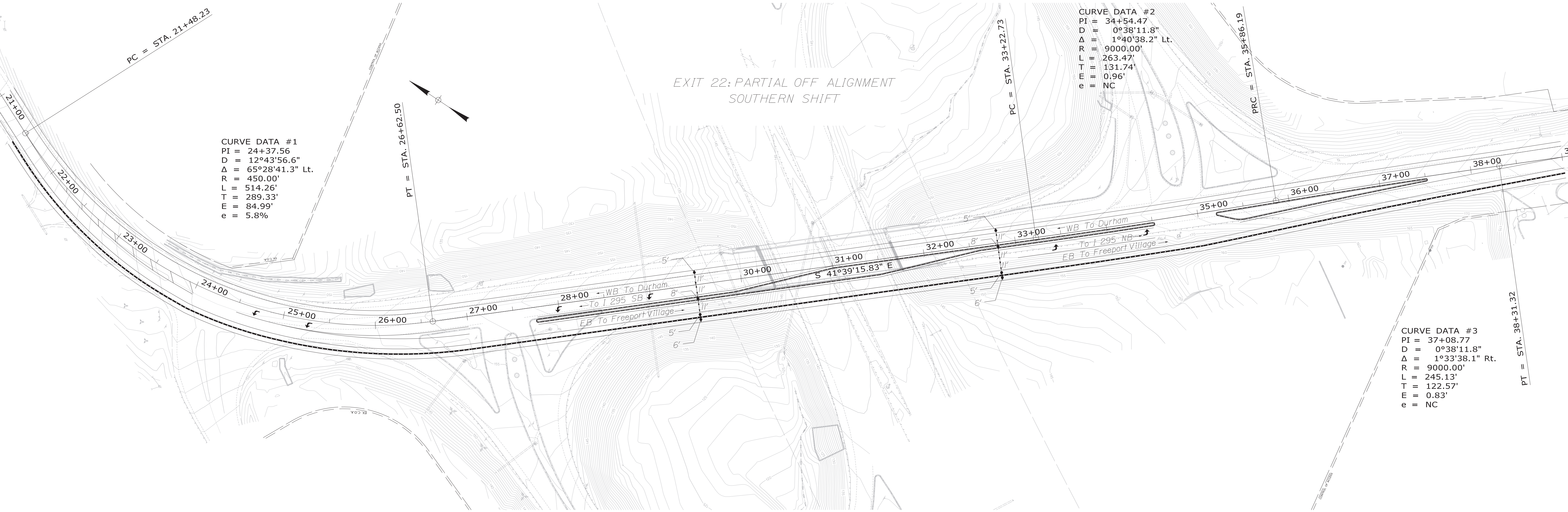
BY
DATE
11\19
11\19

FREEPORT
CUMBERLAND
APPROACH ROAD BRIDGE
INTERSTATE 295
MALLET DRIVE SOUTH
CROSS SECTIONS

SHEET NUMBER
Wor
OF 4

Sta. 32+25.00 to Sta. 32+50.00

Reference 4



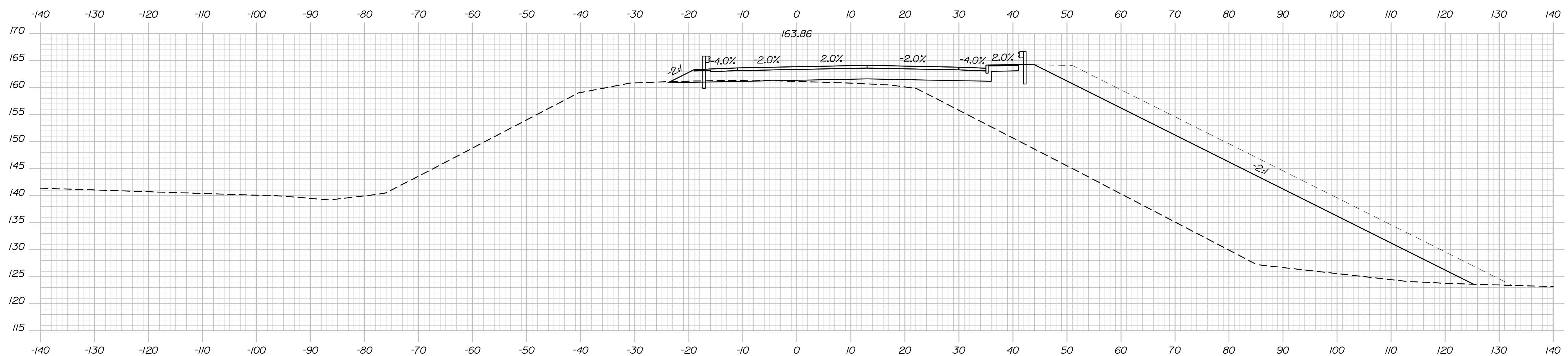
EXIT 22: PARTIAL OFF ALIGNMENT
SOUTHERN SHIFT
PC = STA. 21+48.23
CURVE DATA #1
PI = 24+37.56
D = 12°43'56.6"
Δ = 65°28'41.3" Lt.
R = 450.00'
L = 514.26'
T = 289.33'
E = 84.99'
e = 5.8%
PT = STA. 26+62.50
PC = STA. 33+22.73
CURVE DATA #2
PI = 34+54.47
D = 0°38'11.8"
Δ = 1°40'38.2" Lt.
R = 9000.00'
L = 263.47'
T = 131.74'
E = 0.96'
e = NC
PRC = STA. 35+86.19
CURVE DATA #3
PI = 37+08.77
D = 0°38'11.8"
Δ = 1°33'38.1" Rt.
R = 9000.00'
L = 245.13'
T = 122.57'
E = 0.83'
e = NC
PT = STA. 38+31.32
S 41°39'15.83" E
WB To Durham
To I 295 SB
EB To Freeport Village
To I 295 NB
21+00
22+00
23+00
24+00
25+00
26+00
27+00
28+00
30+00
31+00
32+00
33+00
35+00
36+00
37+00
38+00

Date:6/23/2020

Username:

Division:

Filename: Working Sections.dgn



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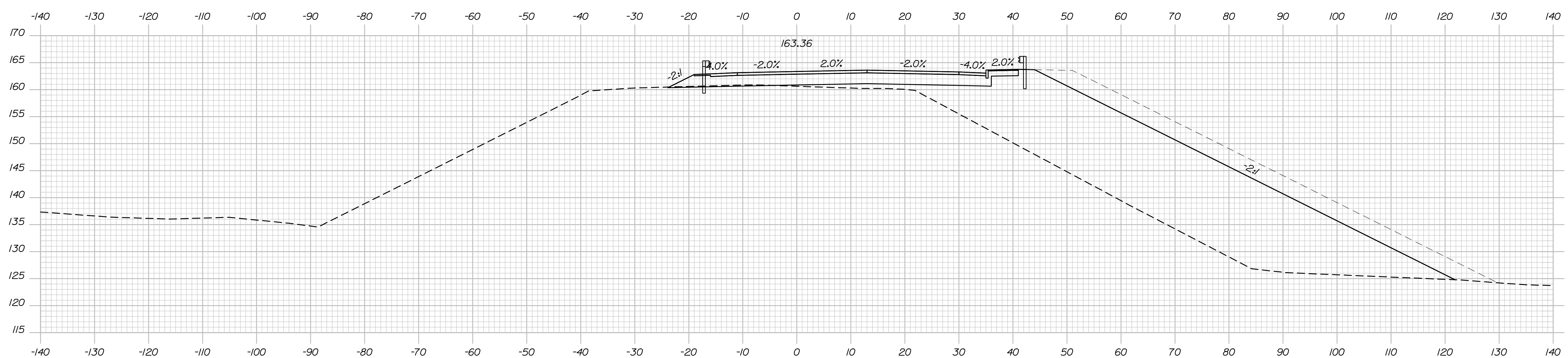


29+50.00

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

WIN
021726.00

Bridge No. 5721

HIGHWAY PLANS

BRIDGE PLANS

	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

APPROACH ROAD BRIDGE
INTERSTATE 295

FREEPORT CUMBERLAND

MALLET DRIVE SOUTH
CROSS SECTIONS

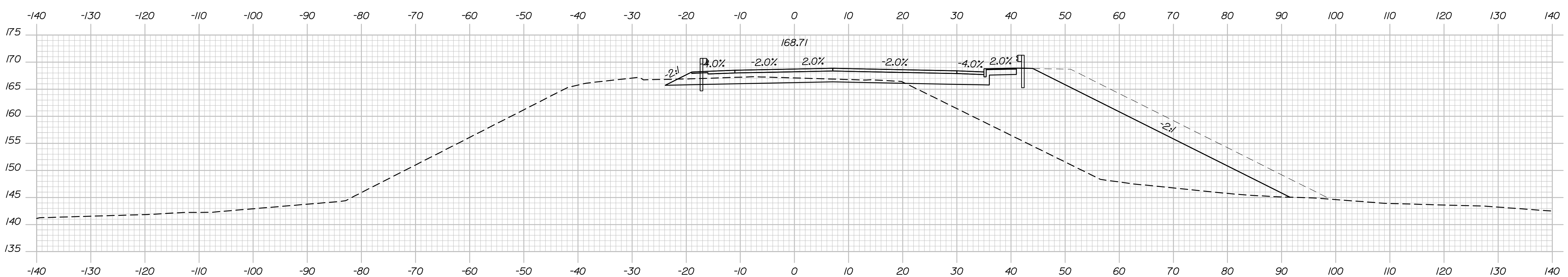
SHEET NUMBER

Wor

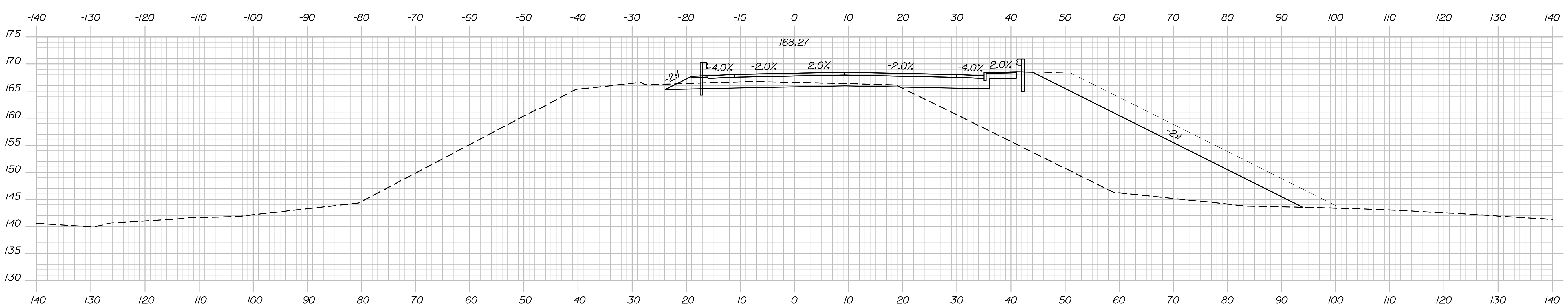
OF 4

Sta. 29+50.00 to Sta. 29+75.00

Date:6/23/2020
Username:
Division:
Filename: Working Sections.dgn



32+50.00



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STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

WIN
021726.00
Bridge No. 5721
HIGHWAY PLANS

	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

APPROACH ROAD BRIDGE
INTERSTATE 295
FREEPORT
CUMBERLAND

MALLET DRIVE SOUTH
CROSS SECTIONS

SHEET NUMBER
Wor
OF 4

Sta. 32+25.00 to Sta. 32+50.00

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.

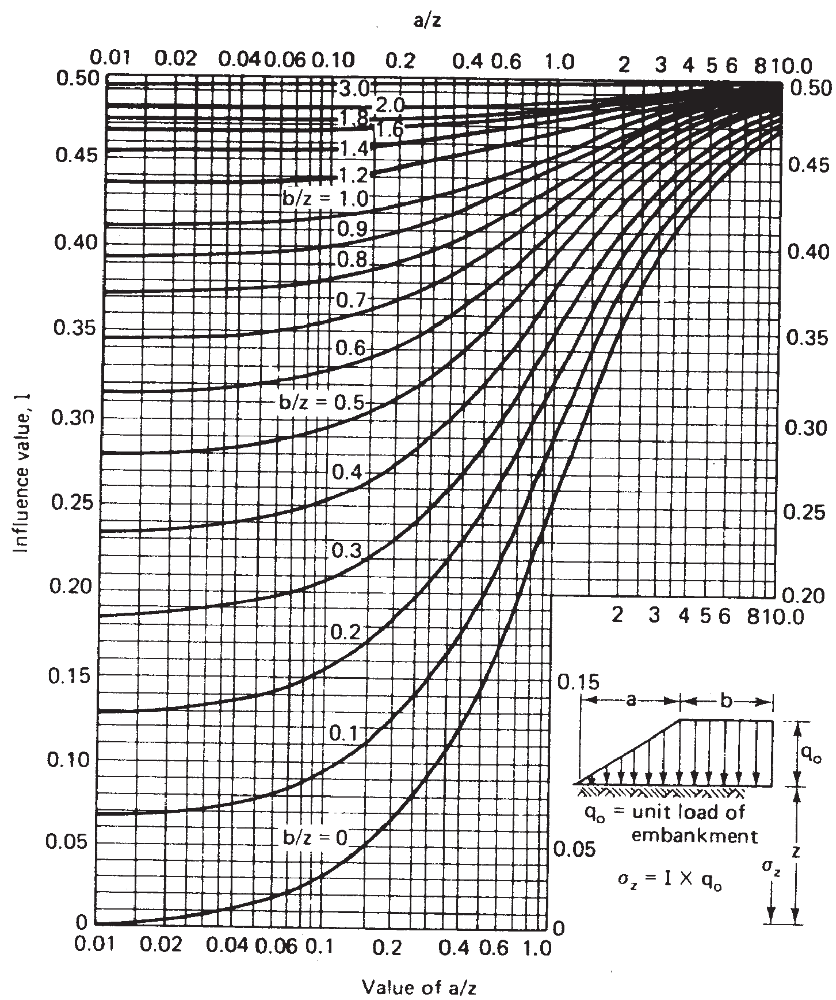


Fig. 8.23 Influence values for vertical stress under a very long embankment; length = ∞ (from U.S. Navy, 1971, after Osterberg, 1957).

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.

- 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).

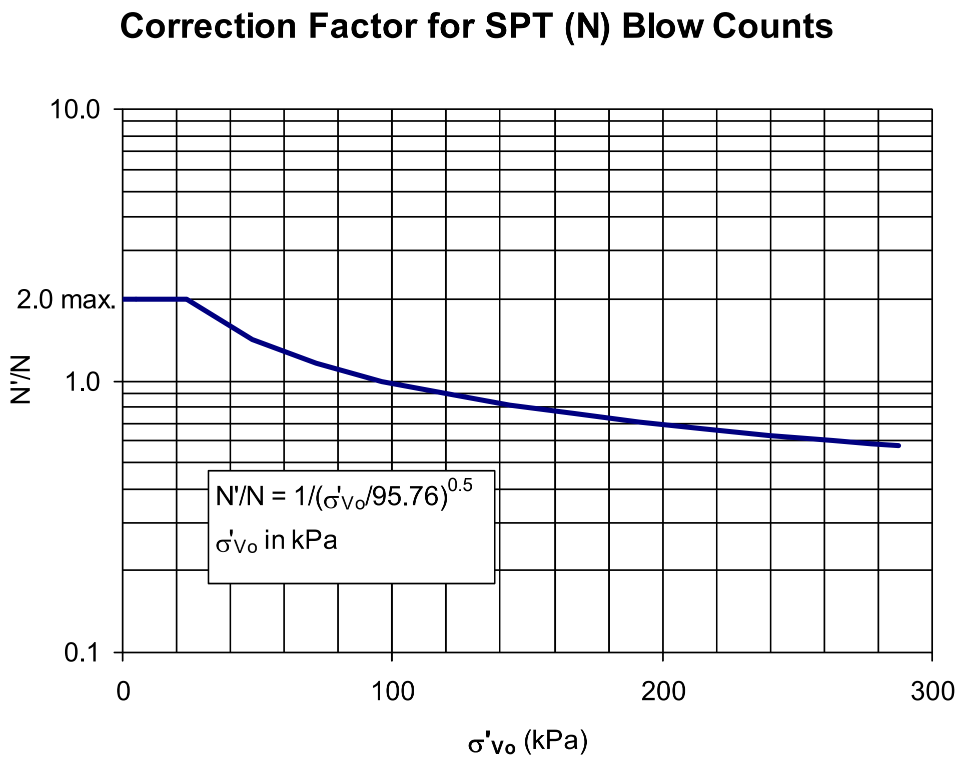


Figure 5-18: Corrected SPT (N) versus Overburden Pressure (after Liao & Whitman, 1986)

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.

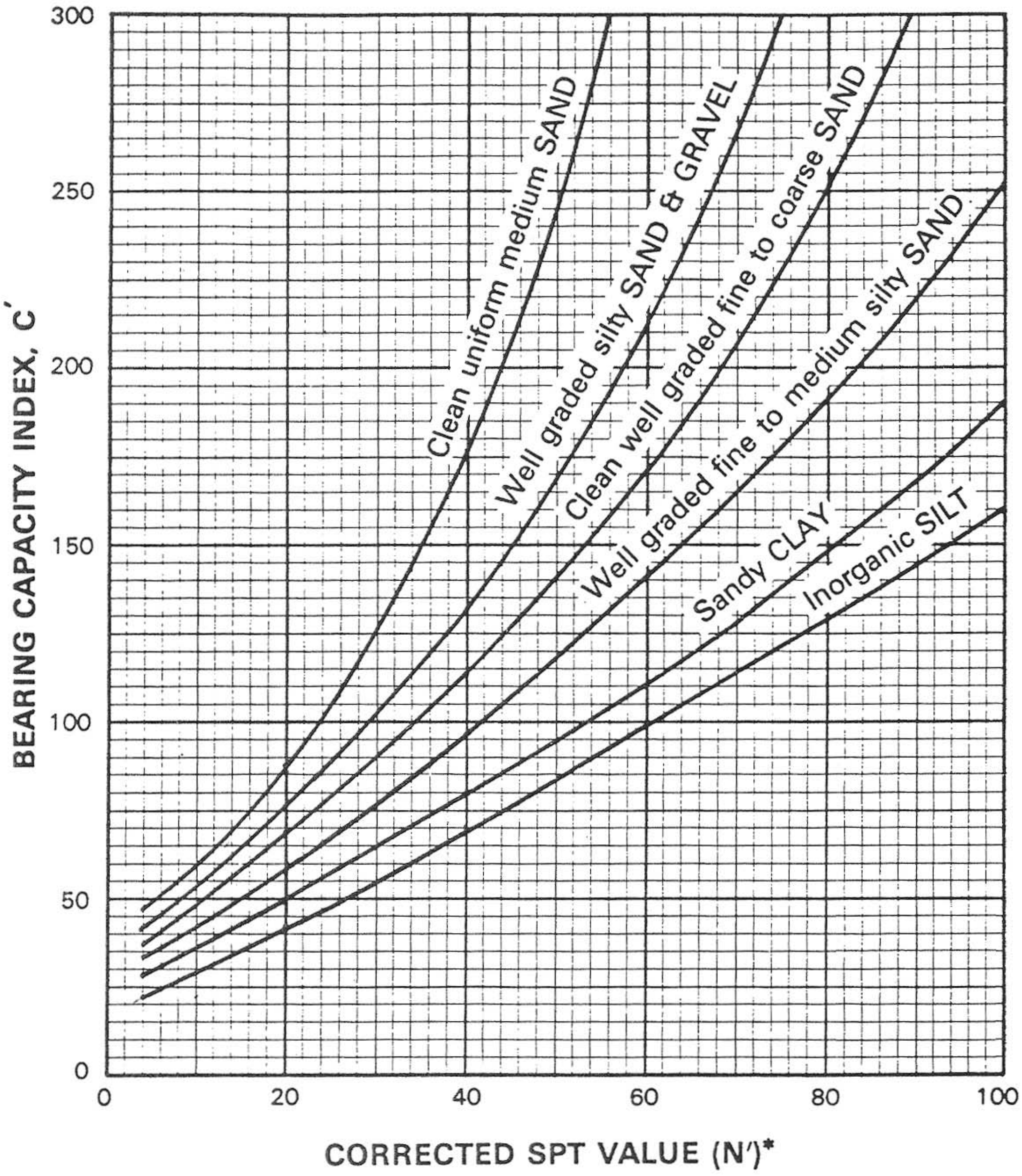
5.3.4.2 D'Appolonia Method

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
Project No.:	19129538	**Checked by:**	CJS/MEL
Subject:	Lateral Earth Pressure	**Reviewed by:**	CCB
Project Short Title:	MaineDOT I-295 Exit 22 Mallet Drive Bridge Replacement No. 5721		

OBJECTIVE

Determine lateral earth pressure acting on the proposed bridge abutments, assuming the "phasing south" 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 23, 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 151 feet at the northwestern abutment and 155 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.

2. Calculate the active and passive earth pressure coefficients using Rankine theory.

$$K_a = \tan^2\left(45° - \frac{\phi}{2}\right)$$ (Ref. 2, page 3-7)

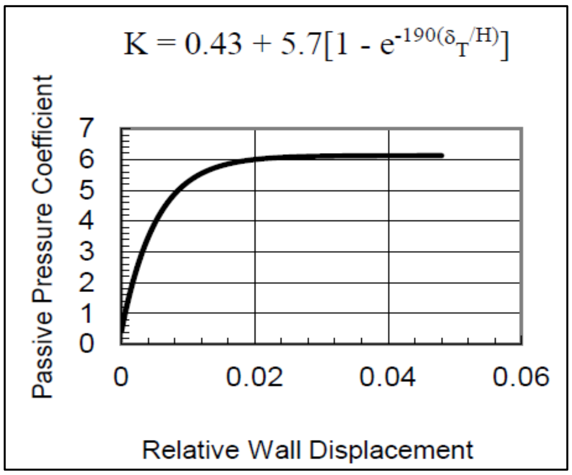
$$K_a = \tan^2\left(45° - \frac{\phi}{2}\right)$$ (Ref. 2, page 3-7)

for horizontal backfill surface, where:

ϕ = internal friction angle of fill　　ϕ = 32　degrees ("Granular borrow", Ref. 2, Table 3-3)

K_a = 0.307

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).



δ_T/H = 0.005　(relative wall displacement)

K_p = 3.93

3. 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:　155　feet elevation

For the southeastern abutment, the load acts at:　159　feet elevation

CONCLUSIONS

Date:	7/16/2020	**Made by:**	KAR
Project No.:	19129538	**Checked by:**	CJS/MEL
Subject:	Lateral Earth Pressure	**Reviewed by:**	CCB
Project Short Title:	MaineDOT I-295 Exit 22 Mallet Drive Bridge Replacement No. 5721		

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 155 feet and 159 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.:	19129538	**Checked by:**	MLM
Subject:	Pile Design at Abutment 1 - Phasing South	**Reviewed by:**	CCB
Project Title:	MaineDOT I-295 Exit 22 Mallet Drive Bridge Replacement No. 5721		

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 "phasing south" 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

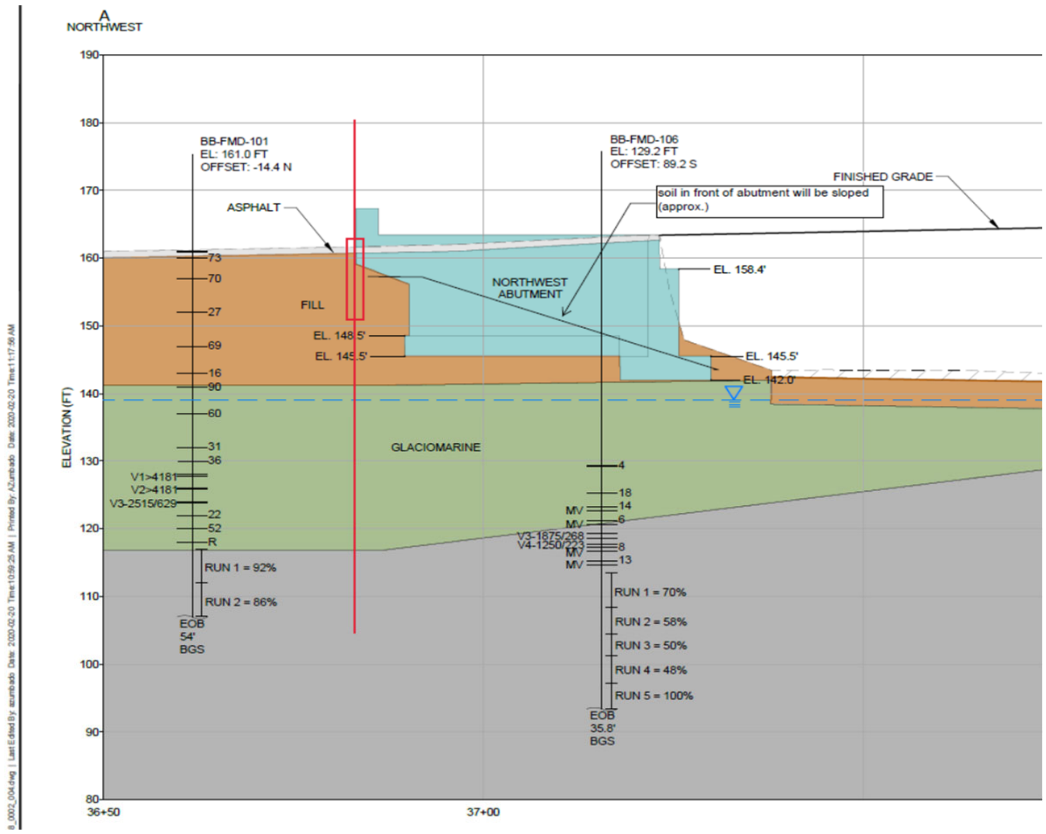
1. AASHTO LRFD Bridge Design Specifications, 9th Ed. 2020.
2. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.
3. Email communication between Golder and HNTB, subject "RE: Freeport I-295 Exit 20 and Exit 22 - Request for Sections and Design Info", dated June 23, 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
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 Bridge Embankment - Phasing South" (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. Approach Road Bridge Freeport Interstate 295: Mallet Drive 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 40 ft behind the face of the existing northwest abutment. The post-construction ground surface elevation at the new northwest abutment will be 164 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 151 ft.

GOLDER

CALCULATIONS

Date:	8/26/2020	**Made by:**	KAR
Project No.:	19129538	**Checked by:**	MLM
Subject:	Pile Design at Abutment 1 - Phasing South	**Reviewed by:**	CCB
Project Title:	MaineDOT I-295 Exit 22 Mallet Drive Bridge Replacement No. 5721		



ATTACHMENTS

1. LPile analysis output for Strength I
2. LPile analysis output for Strength I with Plastic Hinge

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 13 feet behind the new northwestern abutment was estimated to be 1.93 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).

CALCULATIONS

Date:	8/26/2020	**Made by:**	KAR
Project No.:	19129538	**Checked by:**	MLM
Subject:	Pile Design at Abutment 1 - Phasing South	**Reviewed by:**	CCB
Project Title:	MaineDOT I-295 Exit 22 Mallet Drive Bridge Replacement No. 5721		

Layer		Layer Thickness in Embankment (ft)	σ'_{v0} at layer midpoint in Embankment (ksf)	Settlement Based on Calculated Loading Stress (in)
Existing Fill	1	6.0	0.375	0.39
Glaciomarine	2	10.0	1.325	0.81
Glaciomarine	3	10.0	2.213	0.47
Glaciomarine	4	6.1	2.636	0.22
Sand and Gravel	5	2.5	2.875	0.04
(Ref. 10)			Total Settlement (in):	1.93

Layers 1, 2, and 3 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 Layers 2 and 3, Glaciomarine:

$$q_s = \alpha S_u$$ (Ref. 1, Eqn 10.7.3.8.6b-1)

where:

S_u = 3.500 ksf (based on shear strength measurements made in the field and on empirical correlation to the average of the N_{60}-values encountered in BB-FMD-101 for the layer)

D = 13.83 in = 1.2 ft (Ref. 4, Table 5.6.3, HP 14x89)

D_b = 24.4 ft (pile length 34 ft minus fill thickness at abutment 9.6 ft)

α = adhesion factor from Ref. 1 Figure 10.7.3.8.6b-1
Use the plot for "Sands over Stiff Clay" and interpolate between the curves for "D_b = 20D" and "D_b = greater than 40D"

α = 0.73

q_s = 2.553 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 = 33 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 = 1.07 (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 (Ref. 1, Figure 10.7.3.8.6f-5)
σ'_v = 0.600 ksf (Ref. 5; fill thickness at abutment 9.6 ft)
δ/ϕ_f = 0.81 (Ref. 1, Figure 10.7.3.8.6f-6)
δ = 27 degrees (Ref. 1, Figure 10.7.3.8.6f-6)
ω = 0 degrees (assume pile battering not required as per Step 3)

q_s = 0.272 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 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	9.6	45.6	12427	1.10	1.00
Glaciomarine	2 and 3	20.0	95.1	242772	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 = 353550 lbs per pile
= 353.6 kips per pile

Total Factored Load, Service I = 255199 lbs per pile
= 255.2 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 354 kips (Strength I) or 255 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 = 804 kips (maximum factored load from Ref. 3 plus downdrag from Step 1)

As part of this analysis loads up to the expected maximum of 804 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 168 kips.

Design P_u = 522 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).

$$R_{n.upper} = \frac{P_u}{\phi_{cu}}$$

$R_{n.upper}$ = 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:

GOLDER

CALCULATIONS

Date:	8/26/2020	**Made by:**	KAR
Project No.:	19129538	**Checked by:**	MLM
Subject:	Pile Design at Abutment 1 - Phasing South	**Reviewed by:**	CCB
Project Title:	MaineDOT I-295 Exit 22 Mallet Drive Bridge Replacement No. 5721		

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	(Step 2)

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	0 - 9.6 ft	Sand (Reese)	125	-	33	165	-	-
Glaciomarine Silty Clay (above water table)	9.6 - 11.7 ft	Stiff Clay with Free Water (Reese)	115	3500	-	500	0.005	-
Glaciomarine Silty Clay (below water table)	11.7 - 34 ft	Stiff Clay w/o Free Water (Reese)	52.6	3500	-	-	0.005	-
Bedrock	>34 ft	Strong Rock (Vuggy Limestone)	106.6	-	-	-	-	12604

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.

$M_{u.Top}$ = 2489 in-kips (LPile)

Obtain the unbraced lengths of the top segment and the second segment of the upper zone of the pile.

$l_{b.top}$ = 4.49 ft (LPile)
$l_{b.top}$ = 53.90 in

$l_{b.2nd}$ = 10.25 ft (LPile)
$l_{b.2nd}$ = 122.99 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} = 18.30 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} = 34.80 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)

$P_{e.top}$ = 22308 kips
$P_{e.2nd}$ = 6168 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
$P_o/P_{e.2nd}$ = 0.21 ≤ 2.25

thus use Ref. 1 Eqn 6.9.4.1.1-1:

$$P_n = \left[0.658^{\left(\frac{P_o}{P_e}\right)}\right] P_o$$

$P_{n.top}$ = 1273 kips
$P_{n.2nd}$ = 1194 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)

$P_{n.bottom}$ = 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.

$$P_{r.top} = \phi_{cu} P_{n.top}$$

$P_{r.top}$ = 891.4 kips

$$P_{r.2nd} = \phi_{cu} P_{n.2nd}$$

$P_{r.2nd}$ = 836.1 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.

$$\frac{P_u}{P_{r.top}} = \quad 0.59 \qquad \text{OK}$$

$$\frac{P_u}{P_{r.2nd}} = \quad 0.62 \qquad \text{OK}$$

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:

$$\lambda_f = \frac{b_f}{2t_f}$$ (Ref. 1, Eqn 6.12.2.2.1-3)

λ_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)

λ_{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:

$$S_y = \frac{I_{yy}}{b/2}$$

$$Z_y = (b^2 t_f)/2 + 0.25 t_w^2 (d - 2t_f)$$

$S_y =$ 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 $\lambda_{pf} < \lambda_f \le \lambda_{rf}$,
$M_n =$ 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)

$M_p' =$ 1436 in-kips = 1435989.3 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'.

$M_{u.Top} =$ 2489 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.

$l_{b.top}$ =	3.28	ft		(LPile)
$l_{b.top}$ =	39.32	in		
$l_{b.2nd}$ =	10.94	ft		(LPile)
$l_{b.2nd}$ =	131.24	in		
$M_{u.2nd}$ =	1025.61	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} =	23.36	<	120	OK
λ_{2nd} =	37.14	<	120	OK
$P_{e.top}$ =	13685	kips		
$P_{e.2nd}$ =	5417	kips		
$P_o/P_{e.top}$ =	0.10	≤	2.25	(to select P_n equation)
$P_o/P_{e.2nd}$ =	0.24	≤	2.25	(to select P_n equation)
$P_{n.top}$ =	1254	kips		
$P_{n.2nd}$ =	1180	kips		
$P_{r.top}$ =	878	kips		
$P_{r.2nd}$ =	826	kips		
$\frac{P_u}{P_{r.top}} =$	0.59	>	0.20	OK
$\frac{P_u}{P_{r.2nd}} =$	0.63	>	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: 0.93 < 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 = 36.18 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)

$$C_v = 1.0 \text{ if } b/t_f \le 1.1\sqrt{k_v E/F_y}$$ (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.

$\phi_{mon} =$ 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 exceed neither the nominal structural pile resistance (P_n) nor the maximum driving force (P_0) calculated above.

$P_{n.top} =$ 1254 kips (From Step 4)
$P_{n.2nd} =$ 1180 kips (From Step 4)

Check $R_{ndr} < P_n$: OK
Check $R_{ndr} < P_0$: OK

CONCLUSIONS

The results of the analysis indicate that a maximum moment of 2489 in-kips (207 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 168 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
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Files Used for Analysis

Path to file locations:
\Users\kroth\Documents\Projects\19129538 MaineDOT I-295 Freeport Exit 22 Mallet Dr Bridge\Pile Design\LPile Northwest Abutment\

Name of input data file:
Freeport Exit 22 Northwest Abutment Phasing South.lp11d

Name of output report file:
Freeport Exit 22 Northwest Abutment Phasing South.lp11o

Name of plot output file:
Freeport Exit 22 Northwest Abutment Phasing South.lp11p

Name of runtime message file:
Freeport Exit 22 Northwest Abutment Phasing South.lp11r

Date and Time of Analysis

Date: August 26, 2020 Time: 13:35:34

Problem Title

Project Name: MaineDOT I-295 Exit 22 Mallet Drive Bridge No. 5721
Job Number: 19129538
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 4 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 = 9.600000 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 = 33.000000 deg.
Friction angle at bottom of layer = 33.000000 deg.
Subgrade k at top of layer = 165.000000 pci
Subgrade k at bottom of layer = 165.000000 pci

Layer 2 is stiff clay with water-induced erosion

Distance from top of pile to top of layer = 9.600000 ft
Distance from top of pile to bottom of layer = 11.700000 ft
Effective unit weight at top of layer = 115.000000 pcf
Effective unit weight at bottom of layer = 115.000000 pcf
Undrained cohesion at top of layer = 3500. psf
Undrained cohesion at bottom of layer = 3500. psf
Epsilon-50 at top of layer = 0.005000
Epsilon-50 at bottom of layer = 0.005000
Subgrade k at top of layer = 500.000000 pci
Subgrade k at bottom of layer = 500.000000 pci

Layer 3 is stiff clay without free water

Distance from top of pile to top of layer = 11.700000 ft
Distance from top of pile to bottom of layer = 34.000000 ft
Effective unit weight at top of layer = 52.600000 pcf
Effective unit weight at bottom of layer = 52.600000 pcf
Undrained cohesion at top of layer = 3500. psf
Undrained cohesion at bottom of layer = 3500. psf
Epsilon-50 at top of layer = 0.005000
Epsilon-50 at bottom of layer = 0.005000

Layer 4 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 = 106.600000 pcf
Effective unit weight at bottom of layer = 106.600000 pcf
Uniaxial compressive strength at top of layer = 12604. psi
Uniaxial compressive strength at bottom of layer = 12604. psi

(Depth of the lowest soil layer extends 16.000 ft below the pile tip)

Summary of Input Soil Properties

Layer Soil Type Layer Effective Undrained Angle of Uniaxial E50
Layer Name Depth Unit Wt. Cohesion Friction qu or kpy

Num.	(p-y Curve Type)	ft	pcf	psf	deg.	psi	krm	pci
1	Sand	0.00	125.0000	--	33.0000	--	--	165.0000
	(Reese, et al.)	9.6000	125.0000	--	33.0000	--	--	165.0000
2	Stiff Clay	9.6000	115.0000	3500.	--	--	0.00500	500.0000
	with Free Water	11.7000	115.0000	3500.	--	--	0.00500	500.0000
3	Stiff Clay	11.7000	52.6000	3500.	--	--	0.00500	--
	w/o Free Water	34.0000	52.6000	3500.	--	--	0.00500	--
4	Strong Rock	34.0000	106.6000	--	--	12604.	--	--
	(Vuggy Limestone)	50.0000	106.6000	--	--	12604.	--	--

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	105092.
2	9.6000	125.1421	No	No	105092.	8681.
3	11.7000	6.0989	No	No	113774.	806108.
4	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	2488591.	-49340.	0.00	76388.	6.69E+09	0.00	0.00	0.00
0.3400	-0.6969	2286161.	-49131.	0.00145	71818.	6.69E+09	43.3123	253.5694	0.00
0.6800	-0.6881	2081485.	-48849.	0.00269	67198.	7.86E+09	94.7986	562.0735	0.00
1.0200	-0.6749	1876086.	-48349.	0.00368	62562.	8.48E+09	150.3223	908.6928	0.00
1.3600	-0.6581	1671267.	-47621.	0.00451	57939.	9.01E+09	206.7812	1282.	0.00
1.7000	-0.6381	1468277.	-46666.	0.00521	53357.	9.35E+09	261.3350	1671.	0.00
2.0400	-0.6155	1268273.	-45491.	0.00581	48842.	9.44E+09	314.4153	2084.	0.00
2.3800	-0.5907	1072335.	-44107.	0.00631	44419.	9.44E+09	364.3509	2516.	0.00
2.7200	-0.5640	881476.	-42527.	0.00673	40111.	9.44E+09	410.0926	2966.	0.00
3.0600	-0.5358	696631.	-40790.	0.00708	35939.	9.44E+09	441.0852	3359.	0.00
3.4000	-0.5063	518488.	-38930.	0.00734	31917.	9.44E+09	470.6036	3792.	0.00
3.7400	-0.4759	347701.	-36965.	0.00753	28062.	9.44E+09	492.9598	4226.	0.00
4.0800	-0.4449	184801.	-34890.	0.00764	24385.	9.44E+09	523.9294	4805.	0.00
4.4200	-0.4135	30451.	-32709.	0.00769	20901.	9.44E+09	545.1728	5379.	0.00
4.7600	-0.3822	-114851.	-30465.	0.00767	22806.	9.44E+09	555.0276	5926.	0.00
5.1000	-0.3510	-250808.	-28164.	0.00759	25875.	9.44E+09	573.0531	6662.	0.00
5.4400	-0.3202	-376996.	-25774.	0.00745	28724.	9.44E+09	598.4364	7625.	0.00

5.7800	-0.2901	-492874.	-23292.	0.00727	31339.	9.44E+09	617.9167	8689.	0.00
6.1200	-0.2609	-598012.	-20744.	0.00703	33713.	9.44E+09	631.1072	9868.	0.00
6.4600	-0.2328	-692095.	-18134.	0.00675	35836.	9.44E+09	648.7405	11371.	0.00
6.8000	-0.2058	-774740.	-15462.	0.00643	37702.	9.44E+09	660.6770	13096.	0.00
7.1400	-0.1803	-845675.	-12756.	0.00608	39303.	9.44E+09	666.1050	15076.	0.00
7.4800	-0.1562	-904743.	-10041.	0.00571	40636.	9.44E+09	664.6719	17363.	0.00
7.8200	-0.1337	-951914.	-7347.	0.00530	41701.	9.44E+09	656.1037	20021.	0.00
8.1600	-0.1129	-987287.	-4702.	0.00489	42499.	9.44E+09	640.2098	23136.	0.00
8.5000	-0.09384	-1011093.	-2138.	0.00445	43037.	9.44E+09	616.8867	26821.	0.00
8.8400	-0.07656	-1023700.	316.5915	0.00401	43321.	9.44E+09	586.1212	31236.	0.00
9.1800	-0.06108	-1025607.	2630.	0.00357	43364.	9.44E+09	547.9915	36602.	0.00
9.5200	-0.04742	-1017449.	4774.	0.00313	43180.	9.44E+09	502.6670	43250.	0.00
9.8600	-0.03555	-999985.	8586.	0.00269	42786.	9.44E+09	1366.	156820.	0.00
10.2000	-0.02544	-958857.	13732.	0.00227	41858.	9.44E+09	1156.	185368.	0.00
10.5400	-0.01702	-897605.	18018.	0.00187	40475.	9.44E+09	945.4758	226593.	0.00
10.8800	-0.01019	-819789.	21304.	0.00150	38719.	9.44E+09	665.2449	266342.	0.00
11.2200	-0.00480	-730143.	23321.	0.00116	36695.	9.44E+09	323.3351	274666.	0.00
11.5600	-7.03E-04	-634443.	24080.	8.68E-04	34535.	9.44E+09	48.7579	282989.	0.00
11.9000	0.00228	-537347.	23474.	6.15E-04	32343.	9.44E+09	-345.8285	619366.	0.00
12.2400	0.00431	-445513.	21921.	4.02E-04	30270.	9.44E+09	-415.5694	393256.	0.00
12.5800	0.00556	-360187.	20147.	2.28E-04	28344.	9.44E+09	-453.6928	332974.	0.00
12.9200	0.00617	-282082.	18249.	8.91E-05	26581.	9.44E+09	-476.9034	315276.	0.00
13.2600	0.00629	-211656.	16276.	-1.76E-05	24991.	9.44E+09	-490.3792	318256.	0.00
13.6000	0.00603	-149198.	14263.	-9.56E-05	23582.	9.44E+09	-496.4241	335984.	0.00
13.9400	0.00551	-94866.	12238.	-1.48E-04	22355.	9.44E+09	-496.2421	367664.	0.00
14.2800	0.00482	-48707.	10225.	-1.79E-04	21313.	9.44E+09	-490.5102	415368.	0.00
14.6200	0.00404	-10669.	8246.	-1.92E-04	20455.	9.44E+09	-479.6099	483946.	0.00
14.9600	0.00325	19395.	6321.	-1.90E-04	20652.	9.44E+09	-463.7292	582162.	0.00
15.3000	0.00249	41722.	4472.	-1.77E-04	21156.	9.44E+09	-442.9027	725511.	0.00
15.6400	0.00181	56637.	2717.	-1.56E-04	21492.	9.44E+09	-417.0046	942574.	0.00
15.9800	0.00122	64559.	1080.	-1.30E-04	21671.	9.44E+09	-385.6779	1290612.	0.00
16.3200	7.47E-04	66002.	-416.9987	-1.01E-04	21704.	9.44E+09	-348.1056	1900527.	0.00
16.6600	3.92E-04	61589.	-1744.	-7.38E-05	21604.	9.44E+09	-302.2360	3147578.	0.00
17.0000	1.45E-04	52088.	-2647.	-4.93E-05	21390.	9.44E+09	-140.5430	3958616.	0.00
17.3400	-1.02E-05	40199.	-2913.	-2.93E-05	21121.	9.44E+09	10.0781	4030738.	0.00
17.6800	-9.44E-05	28441.	-2699.	-1.45E-05	20856.	9.44E+09	94.8872	4102832.	0.00
18.0200	-1.28E-04	18237.	-2237.	-4.39E-06	20626.	9.44E+09	131.3428	4174898.	0.00
18.3600	-1.30E-04	10202.	-1693.	1.76E-06	20444.	9.44E+09	135.5192	4246938.	0.00
18.7000	-1.14E-04	4414.	-1170.	4.91E-06	20314.	9.44E+09	120.7127	4318954.	0.00
19.0400	-9.01E-05	631.2373	-726.3161	6.00E-06	20228.	9.44E+09	96.9572	4390946.	0.00
19.3800	-6.50E-05	-1538.	-383.4139	5.81E-06	20249.	9.44E+09	71.1320	4462500.	0.00
19.7200	-4.27E-05	-2522.	-143.0488	4.93E-06	20271.	9.44E+09	46.6940	4462500.	0.00
20.0600	-2.48E-05	-2726.	7.5341	3.80E-06	20275.	9.44E+09	27.1211	4462500.	0.00
20.4000	-1.17E-05	-2477.	88.9885	2.67E-06	20270.	9.44E+09	12.8075	4462500.	0.00
20.7400	-2.99E-06	-2012.	121.7902	1.70E-06	20259.	9.44E+09	3.2717	4462500.	0.00
21.0800	2.18E-06	-1490.	123.6019	9.45E-07	20248.	9.44E+09	-2.3836	4462500.	0.00
21.4200	4.72E-06	-1007.	108.2046	4.05E-07	20237.	9.44E+09	-5.1641	4462500.	0.00
21.7600	5.49E-06	-609.0772	85.4258	5.61E-08	20228.	9.44E+09	-6.0019	4462500.	0.00
22.1000	5.18E-06	-310.3342	61.6257	-1.43E-07	20221.	9.44E+09	-5.6648	4462500.	0.00
22.4400	4.32E-06	-105.6040	40.4222	-2.33E-07	20216.	9.44E+09	-4.7291	4462500.	0.00
22.7800	3.28E-06	20.5014	23.4520	-2.51E-07	20214.	9.44E+09	-3.5896	4462500.	0.00
23.1200	2.28E-06	86.8334	11.0501	-2.28E-07	20216.	9.44E+09	-2.4897	4462500.	0.00
23.4600	1.42E-06	111.6404	2.7941	-1.85E-07	20216.	9.44E+09	-1.5573	4462500.	0.00
23.8000	7.68E-07	110.4204	-2.0971	-1.37E-07	20216.	9.44E+09	-0.8403	4462500.	0.00

24.1400	3.07E-07	95.1108	-4.4973	-9.24E-08	20216.	9.44E+09	-0.3362	4462500.	0.00
24.4800	1.43E-08	74.1163	-5.2152	-5.58E-08	20216.	9.44E+09	-0.01567	4462500.	0.00
24.8200	-1.48E-07	52.7929	-4.9167	-2.84E-08	20215.	9.44E+09	0.1620	4462500.	0.00
25.1600	-2.17E-07	34.1167	-4.1014	-9.61E-09	20215.	9.44E+09	0.2377	4462500.	0.00
25.5000	-2.26E-07	19.3665	-3.1111	1.95E-09	20214.	9.44E+09	0.2477	4462500.	0.00
25.8400	-2.01E-07	8.7219	-2.1563	8.02E-09	20214.	9.44E+09	0.2203	4462500.	0.00
26.1800	-1.61E-07	1.7368	-1.3476	1.03E-08	20214.	9.44E+09	0.1761	4462500.	0.00
26.5200	-1.18E-07	-2.3183	-0.7261	1.02E-08	20214.	9.44E+09	0.1286	4462500.	0.00
26.8600	-7.81E-08	-4.2313	-0.2895	8.74E-09	20214.	9.44E+09	0.08546	4462500.	0.00
27.2000	-4.62E-08	-4.7177	-0.01204	6.81E-09	20214.	9.44E+09	0.05054	4462500.	0.00
27.5400	-2.26E-08	-4.3585	0.1415	4.85E-09	20214.	9.44E+09	0.02471	4462500.	0.00
27.8800	-6.67E-09	-3.5838	0.2068	3.13E-09	20214.	9.44E+09	0.00730	4462500.	0.00
28.2200	2.93E-09	-2.6845	0.2151	1.77E-09	20214.	9.44E+09	-0.00321	4462500.	0.00
28.5600	7.80E-09	-1.8361	0.1912	7.97E-10	20214.	9.44E+09	-0.00854	4462500.	0.00
28.9000	9.44E-09	-1.1281	0.1527	1.56E-10	20214.	9.44E+09	-0.01032	4462500.	0.00
29.2400	9.08E-09	-0.5908	0.1114	-2.15E-10	20214.	9.44E+09	-0.00993	4462500.	0.00
29.5800	7.68E-09	-0.2184	0.07398	-3.90E-10	20214.	9.44E+09	-0.00840	4462500.	0.00
29.9200	5.90E-09	0.01449	0.04369	-4.34E-10	20214.	9.44E+09	-0.00645	4462500.	0.00
30.2600	4.14E-09	0.1400	0.02131	-4.01E-10	20214.	9.44E+09	-0.00452	4462500.	0.00
30.6000	2.62E-09	0.1901	0.00623	-3.30E-10	20214.	9.44E+09	-0.00287	4462500.	0.00
30.9400	1.45E-09	0.1922	-0.00286	-2.47E-10	20214.	9.44E+09	-0.00158	4462500.	0.00
31.2800	6.09E-10	0.1678	-0.00745	-1.69E-10	20214.	9.44E+09	-6.66E-04	4462500.	0.00
31.6200	6.72E-11	0.1322	-0.00895	-1.04E-10	20214.	9.44E+09	-7.34E-05	4462500.	0.00
31.9600	-2.42E-10	0.09518	-0.00857	-5.51E-11	20214.	9.44E+09	2.64E-04	4462500.	0.00
32.3000	-3.83E-10	0.06251	-0.00717	-2.11E-11	20214.	9.44E+09	4.19E-04	4462500.	0.00
32.6400	-4.14E-10	0.03675	-0.00540	0.00	20214.	9.44E+09	4.52E-04	4462500.	0.00
32.9800	-3.80E-10	0.01848	-0.00363	1.23E-11	20214.	9.44E+09	4.15E-04	4462500.	0.00
33.3200	-3.13E-10	0.00711	-0.00208	1.79E-11	20214.	9.44E+09	3.42E-04	4462500.	0.00
33.6600	-2.34E-10	0.00143	-8.61E-04	1.97E-11	20214.	9.44E+09	2.56E-04	4462500.	0.00
34.0000	-1.52E-10	0.00	0.00	2.00E-11	20214.	9.44E+09	1.66E-04	2231250.	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 = 2488591. inch-lbs
Maximum shear force = -49340. 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 = 6

Summary of Pile-head Responses for Conventional Analyses

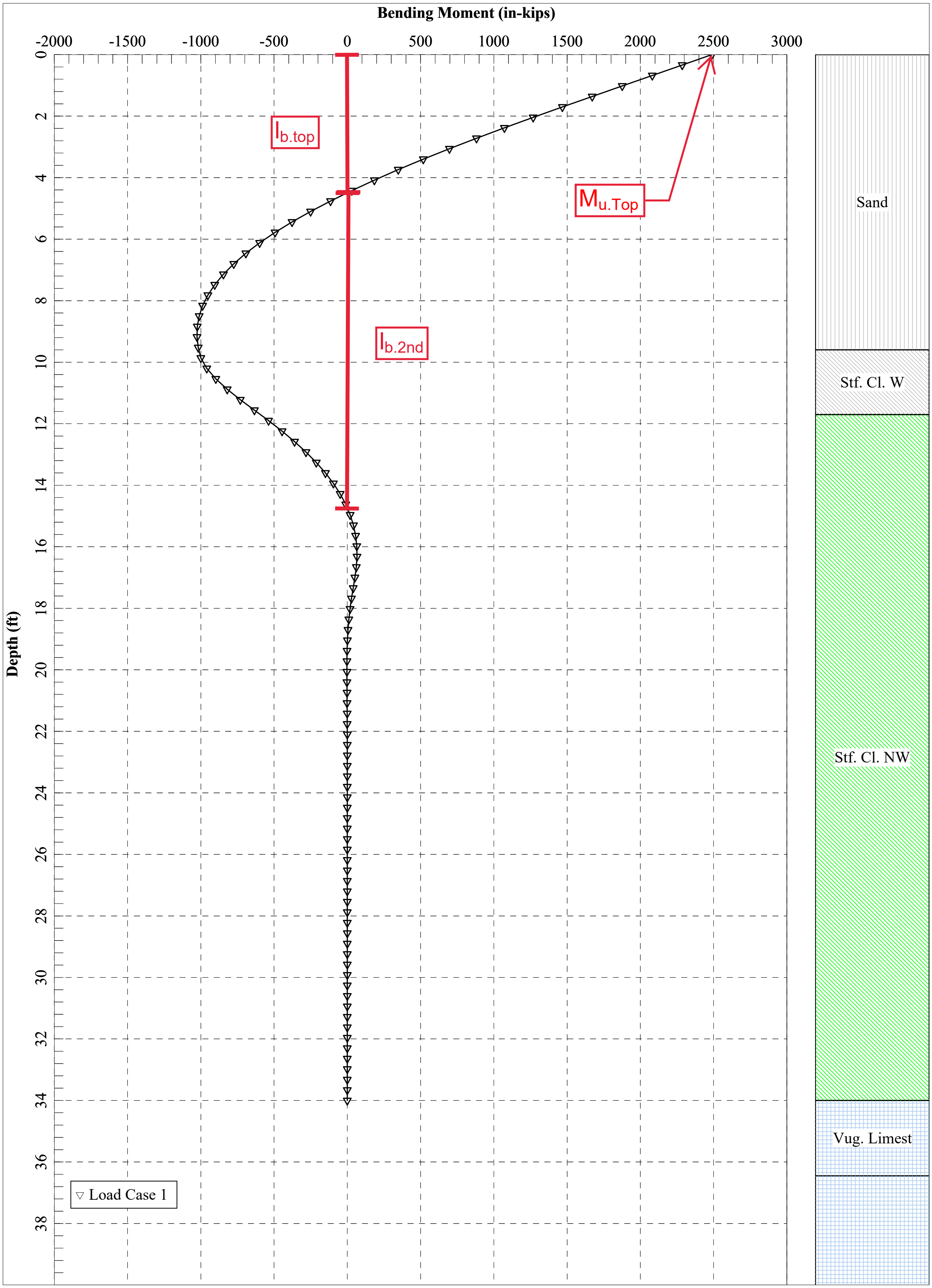
Definitions of Pile-head Loading Conditions:

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

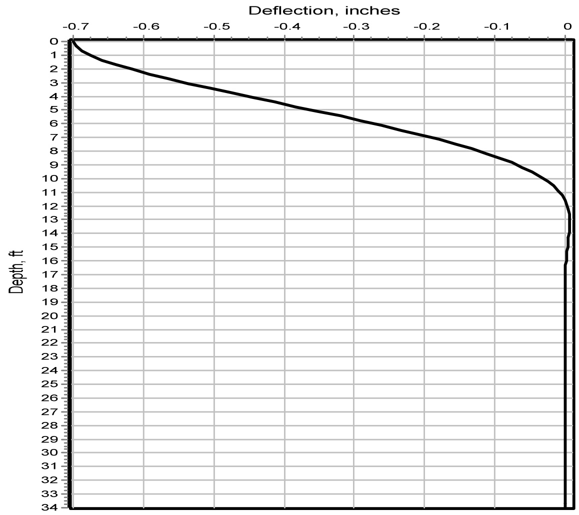
Load Case No.	Load Type 1	Pile-head Load 1	Load Type 2	Pile-head Load 2	Axial Loading lbs	Pile-head Deflection inches	Pile-head Rotation radians	Max Shear in Pile lbs	Max Moment in Pile in-lbs
1	y, in	-0.7000	S, rad	0.00	522000.	-0.7000	0.00	-49340.	2488591.

Maximum pile-head deflection = -0.7000000000 inches
Maximum pile-head rotation = 0.0000000000 radians = 0.000000 deg.

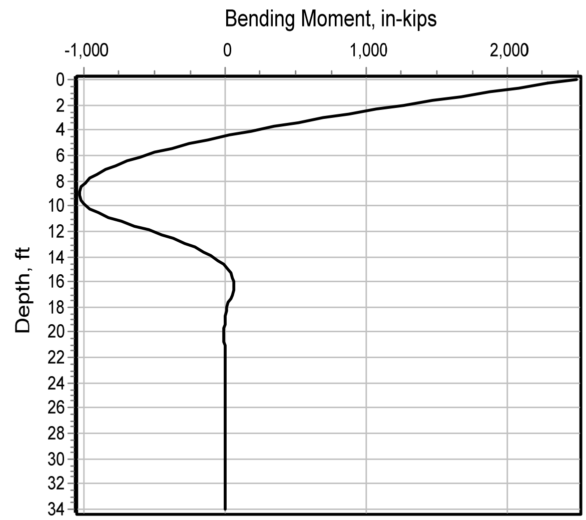
The analysis ended normally.



Bending Moment (in-kips)
-2000
-1500
-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
$l_{b.top}$
$M_{u.Top}$
$l_{b.2nd}$
▽ Load Case 1
Sand
Stf. Cl. W
Stf. Cl. NW
Vug. Limest

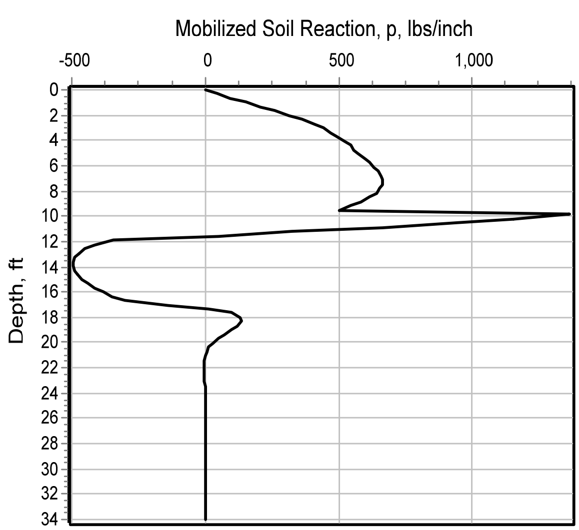


Deflection, inches
-0.7 -0.6 -0.5 -0.4 -0.3 -0.2 -0.1 0
Depth, ft



Bending Moment, in-kips
-1,000 0 1,000 2,000
Depth, ft

Shear Force, kips
-40 -30 -20 -10 0 10 20
Depth, ft



Mobilized Soil Reaction, p, lbs/inch
-500 0 500 1,000
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\19129538 MaineDOT I-295 Freeport Exit 22 Mallet Dr Bridge\Pile Design\LPile Northwest Abutment\

Name of input data file:
Freeport Exit 22 Northwest Abutment Phasing South _ Plastic Hinge.lp11d

Name of output report file:
Freeport Exit 22 Northwest Abutment Phasing South _ Plastic Hinge.lp11o

Name of plot output file:
Freeport Exit 22 Northwest Abutment Phasing South _ Plastic Hinge.lp11p

Name of runtime message file:
Freeport Exit 22 Northwest Abutment Phasing South _ Plastic Hinge.lp11r

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Date and Time of Analysis

Date: August 26, 2020 Time: 13:49:25

Problem Title

Project Name: MaineDOT I-295 Exit 22 Mallet Drive Bridge No. 5721
Job Number: 19129538
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 4 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 = 9.600000 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 = 33.000000 deg.
Friction angle at bottom of layer = 33.000000 deg.
Subgrade k at top of layer = 165.000000 pci
Subgrade k at bottom of layer = 165.000000 pci

Layer 2 is stiff clay with water-induced erosion

Distance from top of pile to top of layer = 9.600000 ft
Distance from top of pile to bottom of layer = 11.700000 ft
Effective unit weight at top of layer = 115.000000 pcf
Effective unit weight at bottom of layer = 115.000000 pcf
Undrained cohesion at top of layer = 3500. psf
Undrained cohesion at bottom of layer = 3500. psf
Epsilon-50 at top of layer = 0.005000
Epsilon-50 at bottom of layer = 0.005000
Subgrade k at top of layer = 500.000000 pci
Subgrade k at bottom of layer = 500.000000 pci

Layer 3 is stiff clay without free water

Distance from top of pile to top of layer = 11.700000 ft
Distance from top of pile to bottom of layer = 34.000000 ft
Effective unit weight at top of layer = 52.600000 pcf
Effective unit weight at bottom of layer = 52.600000 pcf
Undrained cohesion at top of layer = 3500. psf
Undrained cohesion at bottom of layer = 3500. psf
Epsilon-50 at top of layer = 0.005000
Epsilon-50 at bottom of layer = 0.005000

Layer 4 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 = 106.600000 pcf
Effective unit weight at bottom of layer = 106.600000 pcf
Uniaxial compressive strength at top of layer = 12604. psi
Uniaxial compressive strength at bottom of layer = 12604. psi

(Depth of the lowest soil layer extends 16.000 ft below the pile tip)

Summary of Input Soil Properties

Layer Soil Type Layer Effective Undrained Angle of Uniaxial E50
Layer Name Depth Unit Wt. Cohesion Friction qu or kpy

Num.	(p-y Curve Type)	ft	pcf	psf	deg.	psi	krm	pci
1	Sand	0.00	125.0000	--	33.0000	--	--	165.0000
	(Reese, et al.)	9.6000	125.0000	--	33.0000	--	--	165.0000
2	Stiff Clay	9.6000	115.0000	3500.	--	--	0.00500	500.0000
	with Free Water	11.7000	115.0000	3500.	--	--	0.00500	500.0000
3	Stiff Clay	11.7000	52.6000	3500.	--	--	0.00500	--
	w/o Free Water	34.0000	52.6000	3500.	--	--	0.00500	--
4	Strong Rock	34.0000	106.6000	--	--	12604.	--	--
	(Vuggy Limestone)	50.0000	106.6000	--	--	12604.	--	--

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 = 1435989. 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	105092.
2	9.6000	125.1421	No	No	105092.	8681.
3	11.7000	6.0989	No	No	113774.	806108.
4	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	2488591.	-49340.	0.00	76388.	6.69E+09	0.00	0.00	0.00
0.3400	-0.6969	2286161.	-49131.	0.00145	71818.	6.69E+09	43.3123	253.5694	0.00
0.6800	-0.6881	2081485.	-48849.	0.00269	67198.	7.86E+09	94.7986	562.0735	0.00
1.0200	-0.6749	1876086.	-48349.	0.00368	62562.	8.48E+09	150.3223	908.6928	0.00
1.3600	-0.6581	1671267.	-47621.	0.00451	57939.	9.01E+09	206.7812	1282.	0.00
1.7000	-0.6381	1468277.	-46666.	0.00521	53357.	9.35E+09	261.3350	1671.	0.00
2.0400	-0.6155	1268273.	-45491.	0.00581	48842.	9.44E+09	314.4153	2084.	0.00
2.3800	-0.5907	1072335.	-44107.	0.00631	44419.	9.44E+09	364.3509	2516.	0.00
2.7200	-0.5640	881476.	-42527.	0.00673	40111.	9.44E+09	410.0926	2966.	0.00
3.0600	-0.5358	696631.	-40790.	0.00708	35939.	9.44E+09	441.0852	3359.	0.00
3.4000	-0.5063	518488.	-38930.	0.00734	31917.	9.44E+09	470.6036	3792.	0.00
3.7400	-0.4759	347701.	-36965.	0.00753	28062.	9.44E+09	492.9598	4226.	0.00
4.0800	-0.4449	184801.	-34890.	0.00764	24385.	9.44E+09	523.9294	4805.	0.00
4.4200	-0.4135	30451.	-32709.	0.00769	20901.	9.44E+09	545.1728	5379.	0.00
4.7600	-0.3822	-114851.	-30465.	0.00767	22806.	9.44E+09	555.0276	5926.	0.00
5.1000	-0.3510	-250808.	-28164.	0.00759	25875.	9.44E+09	573.0531	6662.	0.00

5.4400	-0.3202	-376996.	-25774.	0.00745	28724.	9.44E+09	598.4364	7625.	0.00
5.7800	-0.2901	-492874.	-23292.	0.00727	31339.	9.44E+09	617.9167	8689.	0.00
6.1200	-0.2609	-598012.	-20744.	0.00703	33713.	9.44E+09	631.1072	9868.	0.00
6.4600	-0.2328	-692095.	-18134.	0.00675	35836.	9.44E+09	648.7405	11371.	0.00
6.8000	-0.2058	-774740.	-15462.	0.00643	37702.	9.44E+09	660.6770	13096.	0.00
7.1400	-0.1803	-845675.	-12756.	0.00608	39303.	9.44E+09	666.1050	15076.	0.00
7.4800	-0.1562	-904743.	-10041.	0.00571	40636.	9.44E+09	664.6719	17363.	0.00
7.8200	-0.1337	-951914.	-7347.	0.00530	41701.	9.44E+09	656.1037	20021.	0.00
8.1600	-0.1129	-987287.	-4702.	0.00489	42499.	9.44E+09	640.2098	23136.	0.00
8.5000	-0.09384	-1011093.	-2138.	0.00445	43037.	9.44E+09	616.8867	26821.	0.00
8.8400	-0.07656	-1023700.	316.5915	0.00401	43321.	9.44E+09	586.1212	31236.	0.00
9.1800	-0.06108	-1025607.	2630.	0.00357	43364.	9.44E+09	547.9915	36602.	0.00
9.5200	-0.04742	-1017449.	4774.	0.00313	43180.	9.44E+09	502.6670	43250.	0.00
9.8600	-0.03555	-999985.	8586.	0.00269	42786.	9.44E+09	1366.	156820.	0.00
10.2000	-0.02544	-958857.	13732.	0.00227	41858.	9.44E+09	1156.	185368.	0.00
10.5400	-0.01702	-897605.	18018.	0.00187	40475.	9.44E+09	945.4758	226593.	0.00
10.8800	-0.01019	-819789.	21304.	0.00150	38719.	9.44E+09	665.2449	266342.	0.00
11.2200	-0.00480	-730143.	23321.	0.00116	36695.	9.44E+09	323.3351	274666.	0.00
11.5600	-7.03E-04	-634443.	24080.	8.68E-04	34535.	9.44E+09	48.7579	282989.	0.00
11.9000	0.00228	-537347.	23474.	6.15E-04	32343.	9.44E+09	-345.8285	619366.	0.00
12.2400	0.00431	-445513.	21921.	4.02E-04	30270.	9.44E+09	-415.5694	393256.	0.00
12.5800	0.00556	-360187.	20147.	2.28E-04	28344.	9.44E+09	-453.6928	332974.	0.00
12.9200	0.00617	-282082.	18249.	8.91E-05	26581.	9.44E+09	-476.9034	315276.	0.00
13.2600	0.00629	-211656.	16276.	-1.76E-05	24991.	9.44E+09	-490.3792	318256.	0.00
13.6000	0.00603	-149198.	14263.	-9.56E-05	23582.	9.44E+09	-496.4241	335984.	0.00
13.9400	0.00551	-94866.	12238.	-1.48E-04	22355.	9.44E+09	-496.2421	367664.	0.00
14.2800	0.00482	-48707.	10225.	-1.79E-04	21313.	9.44E+09	-490.5102	415368.	0.00
14.6200	0.00404	-10669.	8246.	-1.92E-04	20455.	9.44E+09	-479.6099	483946.	0.00
14.9600	0.00325	19395.	6321.	-1.90E-04	20652.	9.44E+09	-463.7292	582162.	0.00
15.3000	0.00249	41722.	4472.	-1.77E-04	21156.	9.44E+09	-442.9027	725511.	0.00
15.6400	0.00181	56637.	2717.	-1.56E-04	21492.	9.44E+09	-417.0046	942574.	0.00
15.9800	0.00122	64559.	1080.	-1.30E-04	21671.	9.44E+09	-385.6779	1290612.	0.00
16.3200	7.47E-04	66002.	-416.9987	-1.01E-04	21704.	9.44E+09	-348.1056	1900527.	0.00
16.6600	3.92E-04	61589.	-1744.	-7.38E-05	21604.	9.44E+09	-302.2360	3147578.	0.00
17.0000	1.45E-04	52088.	-2647.	-4.93E-05	21390.	9.44E+09	-140.5430	3958616.	0.00
17.3400	-1.02E-05	40199.	-2913.	-2.93E-05	21121.	9.44E+09	10.0781	4030738.	0.00
17.6800	-9.44E-05	28441.	-2699.	-1.45E-05	20856.	9.44E+09	94.8872	4102832.	0.00
18.0200	-1.28E-04	18237.	-2237.	-4.39E-06	20626.	9.44E+09	131.3428	4174898.	0.00
18.3600	-1.30E-04	10202.	-1693.	1.76E-06	20444.	9.44E+09	135.5192	4246938.	0.00
18.7000	-1.14E-04	4414.	-1170.	4.91E-06	20314.	9.44E+09	120.7127	4318954.	0.00
19.0400	-9.01E-05	631.2373	-726.3161	6.00E-06	20228.	9.44E+09	96.9572	4390946.	0.00
19.3800	-6.50E-05	-1538.	-383.4139	5.81E-06	20249.	9.44E+09	71.1320	4462500.	0.00
19.7200	-4.27E-05	-2522.	-143.0488	4.93E-06	20271.	9.44E+09	46.6940	4462500.	0.00
20.0600	-2.48E-05	-2726.	7.5341	3.80E-06	20275.	9.44E+09	27.1211	4462500.	0.00
20.4000	-1.17E-05	-2477.	88.9885	2.67E-06	20270.	9.44E+09	12.8075	4462500.	0.00
20.7400	-2.99E-06	-2012.	121.7902	1.70E-06	20259.	9.44E+09	3.2717	4462500.	0.00
21.0800	2.18E-06	-1490.	123.6019	9.45E-07	20248.	9.44E+09	-2.3836	4462500.	0.00
21.4200	4.72E-06	-1007.	108.2046	4.05E-07	20237.	9.44E+09	-5.1641	4462500.	0.00
21.7600	5.49E-06	-609.0772	85.4258	5.61E-08	20228.	9.44E+09	-6.0019	4462500.	0.00
22.1000	5.18E-06	-310.3342	61.6257	-1.43E-07	20221.	9.44E+09	-5.6648	4462500.	0.00
22.4400	4.32E-06	-105.6040	40.4222	-2.33E-07	20216.	9.44E+09	-4.7291	4462500.	0.00
22.7800	3.28E-06	20.5014	23.4520	-2.51E-07	20214.	9.44E+09	-3.5896	4462500.	0.00
23.1200	2.28E-06	86.8334	11.0501	-2.28E-07	20216.	9.44E+09	-2.4897	4462500.	0.00
23.4600	1.42E-06	111.6404	2.7941	-1.85E-07	20216.	9.44E+09	-1.5573	4462500.	0.00

23.8000	7.68E-07	110.4204	-2.0971	-1.37E-07	20216.	9.44E+09	-0.8403	4462500.	0.00
24.1400	3.07E-07	95.1108	-4.4973	-9.24E-08	20216.	9.44E+09	-0.3362	4462500.	0.00
24.4800	1.43E-08	74.1163	-5.2152	-5.58E-08	20216.	9.44E+09	-0.01567	4462500.	0.00
24.8200	-1.48E-07	52.7929	-4.9167	-2.84E-08	20215.	9.44E+09	0.1620	4462500.	0.00
25.1600	-2.17E-07	34.1167	-4.1014	-9.61E-09	20215.	9.44E+09	0.2377	4462500.	0.00
25.5000	-2.26E-07	19.3665	-3.1111	1.95E-09	20214.	9.44E+09	0.2477	4462500.	0.00
25.8400	-2.01E-07	8.7219	-2.1563	8.02E-09	20214.	9.44E+09	0.2203	4462500.	0.00
26.1800	-1.61E-07	1.7368	-1.3476	1.03E-08	20214.	9.44E+09	0.1761	4462500.	0.00
26.5200	-1.18E-07	-2.3183	-0.7261	1.02E-08	20214.	9.44E+09	0.1286	4462500.	0.00
26.8600	-7.81E-08	-4.2313	-0.2895	8.74E-09	20214.	9.44E+09	0.08546	4462500.	0.00
27.2000	-4.62E-08	-4.7177	-0.01204	6.81E-09	20214.	9.44E+09	0.05054	4462500.	0.00
27.5400	-2.26E-08	-4.3585	0.1415	4.85E-09	20214.	9.44E+09	0.02471	4462500.	0.00
27.8800	-6.67E-09	-3.5838	0.2068	3.13E-09	20214.	9.44E+09	0.00730	4462500.	0.00
28.2200	2.93E-09	-2.6845	0.2151	1.77E-09	20214.	9.44E+09	-0.00321	4462500.	0.00
28.5600	7.80E-09	-1.8361	0.1912	7.97E-10	20214.	9.44E+09	-0.00854	4462500.	0.00
28.9000	9.44E-09	-1.1281	0.1527	1.56E-10	20214.	9.44E+09	-0.01032	4462500.	0.00
29.2400	9.08E-09	-0.5908	0.1114	-2.15E-10	20214.	9.44E+09	-0.00993	4462500.	0.00
29.5800	7.68E-09	-0.2184	0.07398	-3.90E-10	20214.	9.44E+09	-0.00840	4462500.	0.00
29.9200	5.90E-09	0.01449	0.04369	-4.34E-10	20214.	9.44E+09	-0.00645	4462500.	0.00
30.2600	4.14E-09	0.1400	0.02131	-4.01E-10	20214.	9.44E+09	-0.00452	4462500.	0.00
30.6000	2.62E-09	0.1901	0.00623	-3.30E-10	20214.	9.44E+09	-0.00287	4462500.	0.00
30.9400	1.45E-09	0.1922	-0.00286	-2.47E-10	20214.	9.44E+09	-0.00158	4462500.	0.00
31.2800	6.09E-10	0.1678	-0.00745	-1.69E-10	20214.	9.44E+09	-6.66E-04	4462500.	0.00
31.6200	6.72E-11	0.1322	-0.00895	-1.04E-10	20214.	9.44E+09	-7.34E-05	4462500.	0.00
31.9600	-2.42E-10	0.09518	-0.00857	-5.51E-11	20214.	9.44E+09	2.64E-04	4462500.	0.00
32.3000	-3.83E-10	0.06251	-0.00717	-2.11E-11	20214.	9.44E+09	4.19E-04	4462500.	0.00
32.6400	-4.14E-10	0.03675	-0.00540	0.00	20214.	9.44E+09	4.52E-04	4462500.	0.00
32.9800	-3.80E-10	0.01848	-0.00363	1.23E-11	20214.	9.44E+09	4.15E-04	4462500.	0.00
33.3200	-3.13E-10	0.00711	-0.00208	1.79E-11	20214.	9.44E+09	3.42E-04	4462500.	0.00
33.6600	-2.34E-10	0.00143	-8.61E-04	1.97E-11	20214.	9.44E+09	2.56E-04	4462500.	0.00
34.0000	-1.52E-10	0.00	0.00	2.00E-11	20214.	9.44E+09	1.66E-04	2231250.	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 = 2488591. inch-lbs
Maximum shear force = -49340. 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 = 6

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 = 1435989.0 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	1435989.	-36182.	0.00472	52628.	9.39E+09	0.00	0.00	0.00
0.3400	-0.6795	1277648.	-36094.	0.00531	49054.	9.39E+09	43.3122	260.0775	0.00
0.6800	-0.6567	1118845.	-35812.	0.00583	45469.	9.44E+09	94.7985	589.0016	0.00
1.0200	-0.6319	960591.	-35312.	0.00628	41897.	9.44E+09	150.3220	970.5963	0.00
1.3600	-0.6054	803954.	-34583.	0.00666	38361.	9.44E+09	206.7809	1394.	0.00
1.7000	-0.5775	650019.	-33628.	0.00698	34886.	9.44E+09	261.3345	1846.	0.00
2.0400	-0.5485	499836.	-32455.	0.00722	31496.	9.44E+09	313.6758	2333.	0.00
2.3800	-0.5186	354415.	-31095.	0.00741	28214.	9.44E+09	353.1864	2779.	0.00
2.7200	-0.4881	214547.	-29589.	0.00753	25057.	9.44E+09	385.2276	3220.	0.00
3.0600	-0.4571	80893.	-27973.	0.00759	22040.	9.44E+09	406.4890	3628.	0.00
3.4000	-0.4261	-46068.	-26268.	0.00760	21254.	9.44E+09	429.7585	4115.	0.00
3.7400	-0.3951	-165833.	-24482.	0.00756	23957.	9.44E+09	445.6390	4602.	0.00
4.0800	-0.3644	-278026.	-22617.	0.00746	26490.	9.44E+09	468.3274	5243.	0.00
4.4200	-0.3342	-382168.	-20680.	0.00732	28840.	9.44E+09	481.2081	5874.	0.00
4.7600	-0.3047	-477948.	-18714.	0.00713	31002.	9.44E+09	482.8279	6465.	0.00
5.1000	-0.2760	-565250.	-16722.	0.00691	32973.	9.44E+09	493.3145	7292.	0.00
5.4400	-0.2483	-643821.	-14671.	0.00665	34747.	9.44E+09	512.0314	8412.	0.00
5.7800	-0.2218	-713274.	-12556.	0.00635	36314.	9.44E+09	525.0590	9658.	0.00
6.1200	-0.1965	-773331.	-10401.	0.00603	37670.	9.44E+09	530.9431	11023.	0.00
6.4600	-0.1726	-823838.	-8215.	0.00569	38810.	9.44E+09	540.8555	12786.	0.00
6.8000	-0.1501	-864583.	-5999.	0.00532	39730.	9.44E+09	545.3061	14820.	0.00
7.1400	-0.1292	-895455.	-3777.	0.00494	40427.	9.44E+09	543.9247	17180.	0.00
7.4800	-0.1098	-916448.	-1573.	0.00455	40900.	9.44E+09	536.5014	19934.	0.00
7.8200	-0.09206	-927666.	588.1326	0.00415	41154.	9.44E+09	522.8989	23175.	0.00
8.1600	-0.07594	-929326.	2681.	0.00375	41191.	9.44E+09	503.0522	27026.	0.00
8.5000	-0.06147	-921756.	4680.	0.00335	41020.	9.44E+09	476.9674	31660.	0.00
8.8400	-0.04862	-905398.	6561.	0.00295	40651.	9.44E+09	444.7154	37321.	0.00
9.1800	-0.03736	-880804.	8297.	0.00257	40096.	9.44E+09	406.4193	44379.	0.00
9.5200	-0.02766	-848633.	9865.	0.00219	39370.	9.44E+09	362.2275	53422.	0.00
9.8600	-0.01946	-809652.	12666.	0.00184	38490.	9.44E+09	1011.	211940.	0.00
10.2000	-0.01269	-753096.	16312.	0.00150	37213.	9.44E+09	776.3959	249696.	0.00
10.5400	-0.00724	-682923.	18830.	0.00119	35629.	9.44E+09	457.8121	258019.	0.00
10.8800	-0.00300	-604500.	20163.	9.09E-04	33859.	9.44E+09	195.6290	266342.	0.00
11.2200	1.80E-04	-522265.	20538.	6.66E-04	32003.	9.44E+09	-12.0931	274666.	0.00
11.5600	0.00243	-439749.	20168.	4.58E-04	30140.	9.44E+09	-168.8891	282989.	0.00
11.9000	0.00391	-359641.	19017.	2.85E-04	28332.	9.44E+09	-395.5717	412270.	0.00
12.2400	0.00476	-285786.	17341.	1.45E-04	26665.	9.44E+09	-425.8866	365026.	0.00
12.5800	0.00510	-218757.	15566.	3.64E-05	25152.	9.44E+09	-444.0377	355109.	0.00
12.9200	0.00506	-158919.	13735.	-4.52E-05	23801.	9.44E+09	-453.7697	366071.	0.00
13.2600	0.00473	-106488.	11877.	-1.03E-04	22618.	9.44E+09	-456.8384	393822.	0.00
13.6000	0.00422	-61564.	10019.	-1.39E-04	21604.	9.44E+09	-454.1804	439065.	0.00

13.9400	0.00360	-24144.	8182.	-1.57E-04	20759.	9.44E+09	-446.3196	505900.	0.00
14.2800	0.00294	5870.	6387.	-1.61E-04	20346.	9.44E+09	-433.5358	602472.	0.00
14.6200	0.00228	28660.	4654.	-1.54E-04	20861.	9.44E+09	-415.9363	743410.	0.00
14.9600	0.00168	44501.	3003.	-1.38E-04	21218.	9.44E+09	-393.4717	955515.	0.00
15.3000	0.00116	53751.	1454.	-1.17E-04	21427.	9.44E+09	-365.8930	1291455.	0.00
15.6400	7.27E-04	56860.	28.7118	-9.29E-05	21497.	9.44E+09	-332.5901	1867634.	0.00
15.9800	3.97E-04	54381.	-1246.	-6.89E-05	21441.	9.44E+09	-292.0485	2997774.	0.00
16.3200	1.64E-04	46990.	-2155.	-4.70E-05	21275.	9.44E+09	-153.5981	3814276.	0.00
16.6600	1.40E-05	36999.	-2495.	-2.88E-05	21049.	9.44E+09	-13.3253	3886462.	0.00
17.0000	-7.11E-05	26752.	-2382.	-1.51E-05	20818.	9.44E+09	68.9544	3958616.	0.00
17.3400	-1.09E-04	17628.	-2021.	-5.47E-06	20612.	9.44E+09	107.6303	4030738.	0.00
17.6800	-1.16E-04	10280.	-1564.	5.58E-07	20446.	9.44E+09	116.3805	4102832.	0.00
18.0200	-1.04E-04	4860.	-1109.	3.83E-06	20324.	9.44E+09	106.8176	4174898.	0.00
18.3600	-8.45E-05	1213.	-711.8820	5.14E-06	20241.	9.44E+09	87.9320	4246938.	0.00
18.7000	-6.24E-05	-971.0651	-397.7016	5.20E-06	20236.	9.44E+09	66.0780	4318954.	0.00
19.0400	-4.21E-05	-2054.	-170.5139	4.54E-06	20260.	9.44E+09	45.2884	4390946.	0.00
19.3800	-2.54E-05	-2382.	-21.5331	3.58E-06	20268.	9.44E+09	27.7414	4462500.	0.00
19.7200	-1.28E-05	-2245.	63.7228	2.58E-06	20265.	9.44E+09	14.0507	4462500.	0.00
20.0600	-4.29E-06	-1873.	101.9562	1.69E-06	20256.	9.44E+09	4.6911	4462500.	0.00
20.4000	9.65E-07	-1421.	109.3721	9.81E-07	20246.	9.44E+09	-1.0559	4462500.	0.00
20.7400	3.71E-06	-984.5273	98.9302	4.61E-07	20236.	9.44E+09	-4.0627	4462500.	0.00
21.0800	4.73E-06	-615.2354	80.0946	1.15E-07	20228.	9.44E+09	-5.1704	4462500.	0.00
21.4200	4.65E-06	-331.4459	59.1606	-8.93E-08	20221.	9.44E+09	-5.0914	4462500.	0.00
21.7600	4.00E-06	-132.1044	39.8533	-1.90E-07	20217.	9.44E+09	-4.3730	4462500.	0.00
22.1000	3.11E-06	-5.4357	23.9969	-2.19E-07	20214.	9.44E+09	-3.3998	4462500.	0.00
22.4400	2.21E-06	64.6440	12.1326	-2.06E-07	20215.	9.44E+09	-2.4161	4462500.	0.00
22.7800	1.42E-06	94.4453	4.0273	-1.72E-07	20216.	9.44E+09	-1.5571	4462500.	0.00
23.1200	8.05E-07	98.2401	-0.9448	-1.30E-07	20216.	9.44E+09	-0.8802	4462500.	0.00
23.4600	3.59E-07	87.2915	-3.5421	-9.03E-08	20216.	9.44E+09	-0.3929	4462500.	0.00
23.8000	6.76E-08	69.7217	-4.4945	-5.64E-08	20215.	9.44E+09	-0.07398	4462500.	0.00
24.1400	-1.01E-07	50.8562	-4.4201	-3.03E-08	20215.	9.44E+09	0.1105	4462500.	0.00
24.4800	-1.80E-07	33.7827	-3.7933	-1.20E-08	20215.	9.44E+09	0.1968	4462500.	0.00
24.8200	-1.99E-07	19.9544	-2.9471	-4.35E-10	20214.	9.44E+09	0.2180	4462500.	0.00
25.1600	-1.83E-07	9.7366	-2.0929	5.98E-09	20214.	9.44E+09	0.2007	4462500.	0.00
25.5000	-1.50E-07	2.8507	-1.3477	8.70E-09	20214.	9.44E+09	0.1646	4462500.	0.00
25.8400	-1.12E-07	-1.2978	-0.7610	9.04E-09	20214.	9.44E+09	0.1230	4462500.	0.00
26.1800	-7.67E-08	-3.3972	-0.3388	8.02E-09	20214.	9.44E+09	0.08394	4462500.	0.00
26.5200	-4.70E-08	-4.0964	-0.06268	6.40E-09	20214.	9.44E+09	0.05140	4462500.	0.00
26.8600	-2.45E-08	-3.9360	0.09681	4.67E-09	20214.	9.44E+09	0.02678	4462500.	0.00
27.2000	-8.91E-09	-3.3263	0.1713	3.10E-09	20214.	9.44E+09	0.00974	4462500.	0.00
27.5400	8.05E-10	-2.5514	0.1894	1.83E-09	20214.	9.44E+09	-8.80E-04	4462500.	0.00
27.8800	6.01E-09	-1.7888	0.1742	8.90E-10	20214.	9.44E+09	-0.00658	4462500.	0.00
28.2200	8.07E-09	-1.1340	0.1427	2.59E-10	20214.	9.44E+09	-0.00883	4462500.	0.00
28.5600	8.13E-09	-0.6252	0.1066	-1.22E-10	20214.	9.44E+09	-0.00889	4462500.	0.00
28.9000	7.08E-09	-0.2637	0.07267	-3.14E-10	20214.	9.44E+09	-0.00774	4462500.	0.00
29.2400	5.57E-09	-0.03081	0.04446	-3.77E-10	20214.	9.44E+09	-0.00609	4462500.	0.00
29.5800	4.00E-09	0.1007	0.02312	-3.62E-10	20214.	9.44E+09	-0.00437	4462500.	0.00
29.9200	2.61E-09	0.1594	0.00836	-3.06E-10	20214.	9.44E+09	-0.00286	4462500.	0.00
30.2600	1.50E-09	0.1703	-8.16E-04	-2.35E-10	20214.	9.44E+09	-0.00164	4462500.	0.00
30.6000	6.96E-10	0.1537	-0.00572	-1.65E-10	20214.	9.44E+09	-7.61E-04	4462500.	0.00
30.9400	1.60E-10	0.1243	-0.00763	-1.05E-10	20214.	9.44E+09	-1.75E-04	4462500.	0.00
31.2800	-1.58E-10	0.09187	-0.00764	-5.79E-11	20214.	9.44E+09	1.72E-04	4462500.	0.00
31.6200	-3.13E-10	0.06223	-0.00659	-2.46E-11	20214.	9.44E+09	3.42E-04	4462500.	0.00
31.9600	-3.58E-10	0.03822	-0.00509	-2.86E-12	20214.	9.44E+09	3.92E-04	4462500.	0.00

32.3000	-3.36E-10	0.02069	-0.00354	9.88E-12	20214.	9.44E+09	3.67E-04	4462500.	0.00
32.6400	-2.77E-10	0.00926	-0.00217	1.63E-11	20214.	9.44E+09	3.03E-04	4462500.	0.00
32.9800	-2.03E-10	0.00288	-0.00110	1.90E-11	20214.	9.44E+09	2.21E-04	4462500.	0.00
33.3200	-1.23E-10	1.73E-04	-3.79E-04	1.96E-11	20214.	9.44E+09	1.34E-04	4462500.	0.00
33.6600	-4.23E-11	-2.99E-04	-1.09E-05	1.96E-11	20214.	9.44E+09	4.63E-05	4462500.	0.00
34.0000	3.74E-11	0.00	0.00	1.95E-11	20214.	9.44E+09	-4.09E-05	2231250.	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 = 0.00472066 radians
Maximum bending moment = 1435989. inch-lbs
Maximum shear force = -36182. 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 = 7

Summary of Pile-head Responses for Conventional Analyses

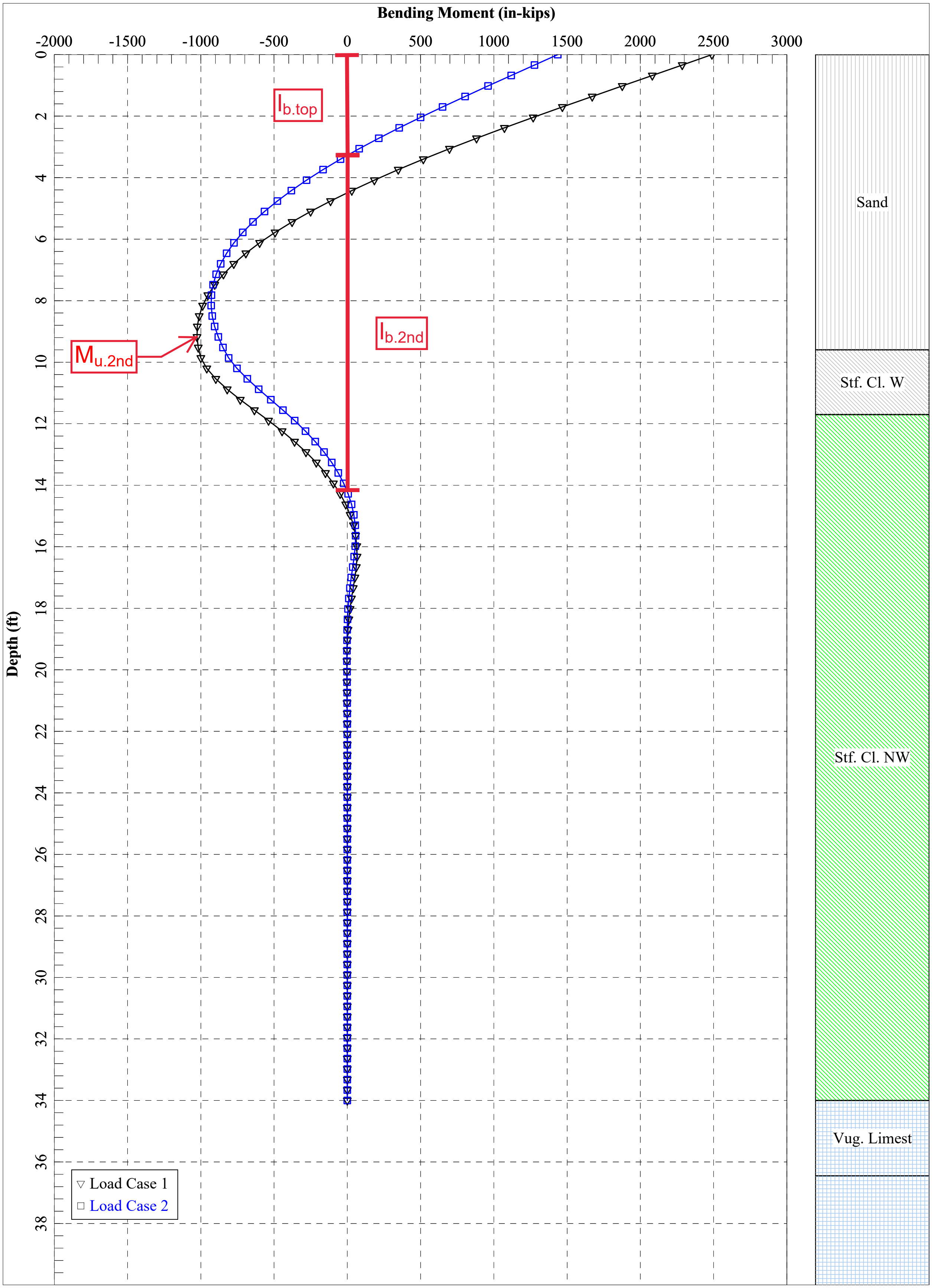
Definitions of Pile-head Loading Conditions:

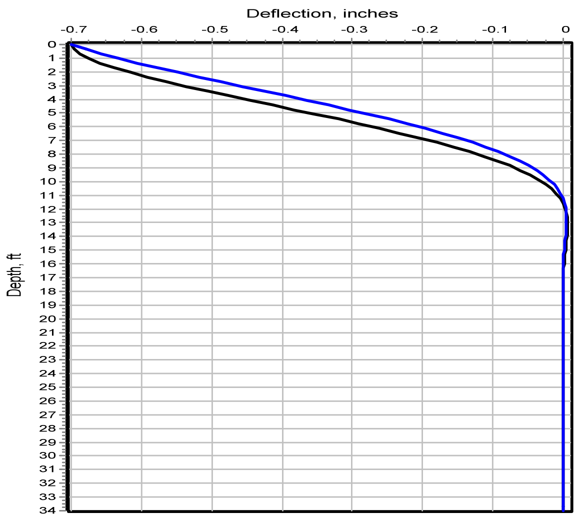
Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians
Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
Load Type 5: Load 1 = Top Deflection, y, inches, and Load 2 = Slope, S, radians

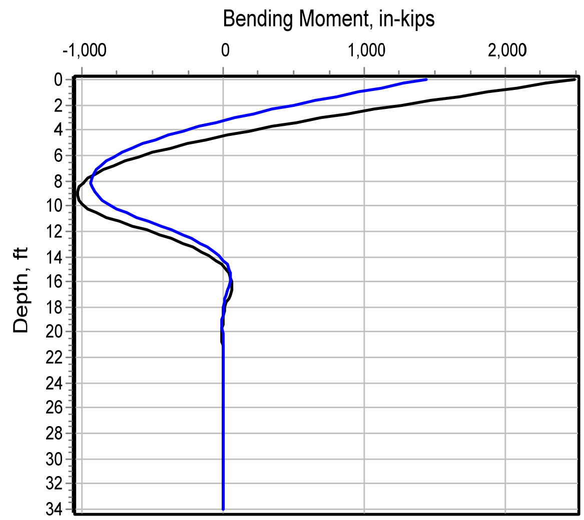
Load Case No.	Load Type 1	Pile-head Load 1	Load Type 2	Pile-head Load 2	Axial Loading lbs	Pile-head Deflection inches	Pile-head Rotation radians	Max Shear in Pile lbs	Max Moment in Pile in-lbs
1	y, in	-0.7000	S, rad	0.00	522000.	-0.7000	0.00	-49340.	2488591.
2	y, in	-0.7000	M, in-lb	1435989.	522000.	-0.7000	0.00472	-36182.	1435989.

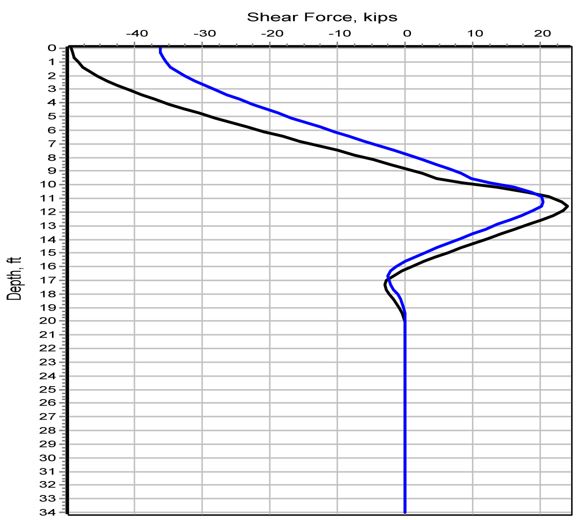
Maximum pile-head deflection = -0.7000000000 inches
Maximum pile-head rotation = 0.0047206572 radians = 0.270474 deg.

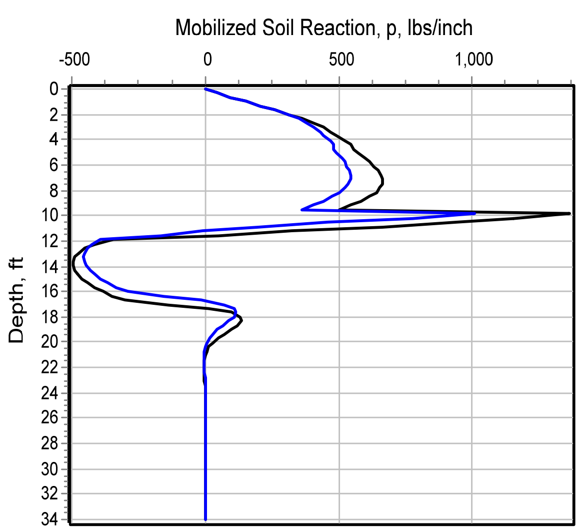
The analysis ended normally.


Bending Moment (in-kips)
-2000
-1500
-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
Ib.top
Ib.2nd
Mu.2nd
Load Case 1
Load Case 2
Sand
Stf. Cl. W
Stf. Cl. NW
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
-1,000 0 1,000 2,000
Depth, ft


Shear Force, kips
-40 -30 -20 -10 0 10 20
Depth, ft


Mobilized Soil Reaction, p, lbs/inch
-500 0 500 1,000
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

5.6.3 Steel H-Piles

Section	Weight lbs/ft kg/m	Area, *A* in² cm²	Depth, *d* in mm	Flange Width, *b* in mm	Thickness Flange, t_f in. mm	 Web, t_w in. mm	I_{xx} in⁴ cm⁴	I_{yy} in⁴ cm⁴	Compact Section Criteria F'_y ksi MPa
HP 14 HP 360	**117** 175	**34.4** 222	**14.21** 361	**14.885** 378	**0.805** 20.4	**0.805** 20.4	**1220** 50800	**443** 18400	**49.4** 341
	102 153	**30** 194	**14.01** 356	**14.785** 376	**0.705** 17.9	**0.705** 17.9	**1050** 43700	**380** 15800	**38.4** 265
	89 133	**26.1** 168	**13.83** 351	**14.695** 373	**0.615** 15.6	**0.615** 15.6	**904** 37600	**326** 13600	**29.6** 204
	73 109	**21.4** 138	**13.61** 346	**14.585** 370	**0.505** 12.8	**0.505** 12.8	**729** 30300	**261** 10900	**20.3** 140
HP 13 HP 330	**100** 150	**29.4** 190	**13.15** 334	**13.205** 335	**0.765** 19.4	**0.765** 19.4	**886** 36878	**294** 12237	**56.7** 391
	87 130	**25.5** 165	**12.95** 329	**13.105** 333	**0.665** 16.9	**0.665** 16.9	**755** 31425	**250** 10406	**43.5** 300
	73 109	**21.6** 139	**12.75** 324	**13.005** 330	**0.565** 14.4	**0.565** 14.4	**630** 26223	**207** 8616	**31.9** 220
	60 90	**17.5** 113	**12.54** 319	**12.9** 328	**0.46** 11.7	**0.46** 11.7	**503** 20936	**165** 6868	**21.5** 148

Section	Weight lbs/ft kg/m	Area, *A* in² cm²	Depth, *d* in mm	Flange Width, *b* in mm	Thickness Flange, t_f in. mm	 Web, t_w in. mm	I_{xx} in⁴ cm⁴	I_{yy} in⁴ cm⁴	Compact Section Criteria F'_y ksi MPa
HP 12 HP 310	**84** 126	**24.6** 159	**12.28** 312	**12.295** 312	**0.685** 17.4	**0.685** 17.4	**650** 27100	**213** 8870	52.5 362
	74 111	**21.8** 141	**12.13** 308	**12.215** 310	**0.61** 15.5	**0.61** 15.5	**569** 23700	**186** 7740	**42.1** 290
	63 94	**18.4** 119	**11.94** 303	**12.125** 308	**0.515** 13.1	**0.515** 13.1	**472** 19600	**153** 6370	**30.5** 210
	53 79	**15.5** 100	**11.78** 299	**12.045** 306	**0.435** 11	**0.435** 11	**393** 16400	**127** 5290	**22** 152
HP10 HP 250	**57** 85	**16.8** 108	**9.99** 254	**10.225** 260	**0.565** 14.4	**0.565** 14.4	**294** 12200	**101** 4200	**51.6** 356
	42 63	**12.4** 80	**9.7** 246	**10.075** 256	**0.42** 10.7	**0.42** 10.7	**210** 8740	**71.7** 2980	**29.4** 203
HP 8 HP 200	**36** 54	**10.6** 68.4	**8.02** 204	**8.155** 207	**0.445** 11.3	**0.445** 11.3	**119** 4950	**40.3** 1680	**50.3** 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.

CALCULATIONS

Date:	8/20/2020 Rev. 12/9/2020 (section 8 only)	**Made by:**	KAR
Project No.:	19129538	**Checked by:**	MLM
Subject:	Pile Design at Abutment 2 - Phasing South	**Reviewed by:**	CCB
Project Title:	MaineDOT I-295 Exit 22 Mallet Drive Bridge Replacement No. 5721		

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 "phasing south" 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 23, 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 Southeast Bridge Embankment - Phasing South" (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. Approach Road Bridge Freeport Interstate 295: Mallet Drive South Cross Sections, dated November 2019.
13. Golder calculation titled "Pile Driveability - Phasing South" (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 40 ft behind the face of the existing southeast abutment. The post-construction ground surface elevation at the new southeast abutment will be 168 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 155 ft.

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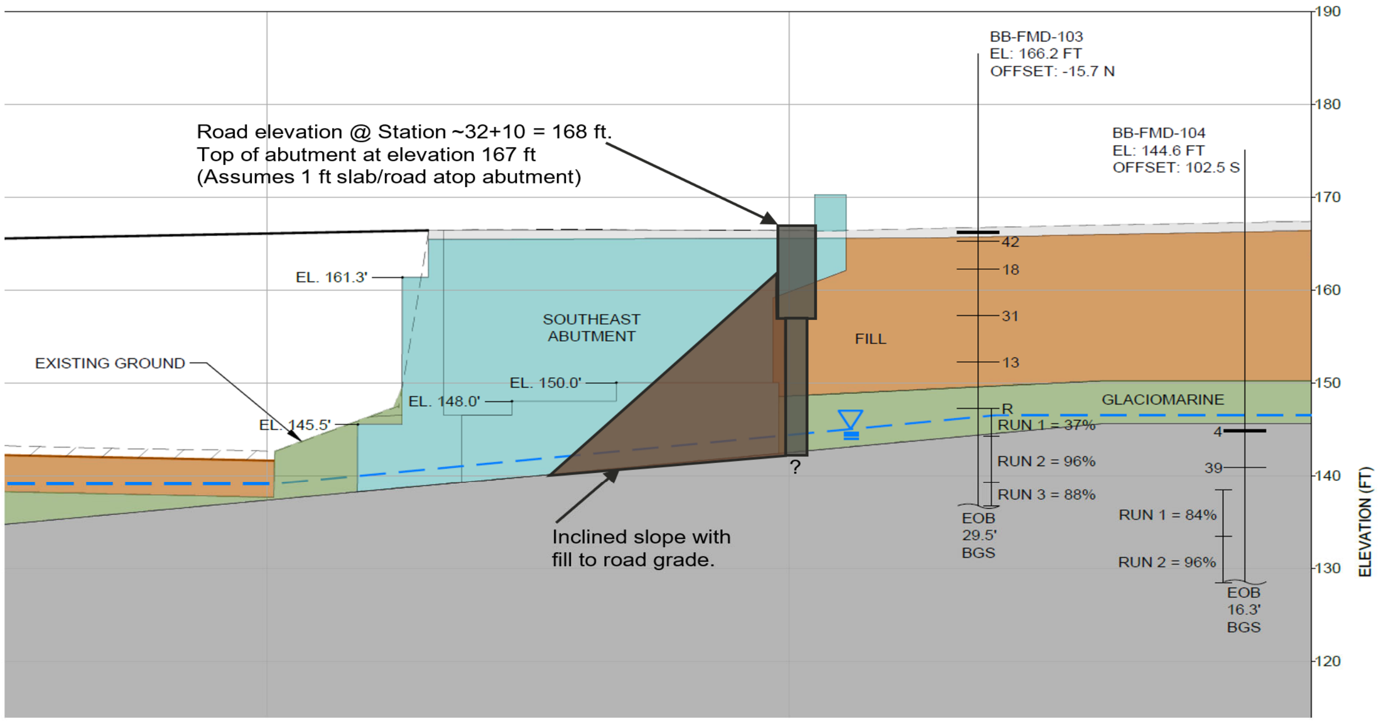

ATTACHMENTS

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 20 feet behind the new southeastern abutment was estimated to be 1.19 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	3.0	0.188	0.26

Glaciomarine	2	8.0	0.835	0.89
Sand and Gravel	3	1.5	1.373	0.04
(Ref. 10)			Total Settlement (in):	1.19

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 = 3.500 ksf (based on shear strength measurements made in the field and on empirical correlation to the average of the N_{60}-values encountered in BB-FMD-101 for the layer)

D = 13.83 in = 1.2 ft (Ref. 4, Table 5.6.3, HP 14x89)

D_b = 6.0 ft (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 = 3.500 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 = 33 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 = 1.07 (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 (Ref. 1, Figure 10.7.3.8.6f-5)

σ'_v = 1.125 ksf (Ref. 5)

δ/ϕ_f = 0.81 (Ref. 1, Figure 10.7.3.8.6f-6)

δ = 27 degrees (Ref. 1, Figure 10.7.3.8.6f-6)

ω = 0 degrees (assume pile battering not required as per Step 3)

q_s = 0.511 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 14x89 pile = 57.05 in = 4.75 ft

Layer		Layer Thickness at Abutment (ft)	Surface area of pile side (ft^2)	Load (lbs)	Strength I Load Factor	Service I Load Factor
Existing Fill	1	6.0	28.5	14563	1.10	1.00
Glaciomarine	2	6.0	28.5	99838	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 = 155792 lbs per pile
= 155.8 kips per pile

Total Factored Load, Service I = 114400 lbs per pile
= 114.4 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 156 kips (Strength I) or 114 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 = 606 kips (maximum factored load from Ref. 3 plus downdrag from Step 1)

As part of this analysis loads up to the expected maximum of 606 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 366 kips.

Design P_u = 522 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).

$$R_{n.upper} = \frac{P_u}{\phi_{cu}}$$

$R_{n.upper}$ = 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 (Assumption 1)

GOLDER

CALCULATIONS

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Length of section:	12	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	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)	Friction Angle (°)	Subgrade Modulus (pci)[3]	Major Principal Strain at 50%	UCS (psi)[2]
Existing Fill	0 - 6 ft	Sand (Reese)	125	-	33	165	-	-
Glaciomarine Silty Clay (above water table)	6 - 10 ft	Stiff Clay with Free Water (Reese)	115	3500	-	500	0.005	-
Glaciomarine Silty Clay (below water table)	10 - 12 ft	Stiff Clay w/o Free Water (Reese)	52.6	3500	-	-	0.005	-
Bedrock	>12 ft	Strong Rock (Vuggy Limestone)	106.6	-	-	-	-	2486

1) Ref. 5
2) Ref. 6. Using lowest UCS value from laboratory test results due to low RQD encountered in boring BB-FMD-103 closest to southeastern abutment.
3) Ref. 7

The full LPile output is provided in Attachment 1.

Obtain the maximum moment at the top of the pile.

$M_{u.Top} =$ 2708 in-kips (LPile)

Obtain the unbraced lengths of the top segment and the second segment of the upper zone of the pile.

$l_{b.top}$ =	4.40	ft	(LPile)
$l_{b.top}$ =	52.79	in	
$l_{b.2nd}$ =	7.19	ft	(LPile)
$l_{b.2nd}$ =	86.31	in	

4. Determine if the applied moment on the pile will cause pile head plastic deformation by using the

Determine K values for the top and bottom of the pile and calculate the column slenderness factor (λ) for each

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} =	17.92	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} =	24.42	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)

$P_{e.top}$ = 23253 kips

$P_{e.2nd}$ = 12526 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

$P_o/P_{e.2nd}$ = 0.10 ≤ 2.25

thus use Ref. 1 Eqn 6.9.4.1.1-1:

$$P_n = \left[0.658^{\left(\frac{P_o}{P_e}\right)}\right] P_o$$

$P_{n.top}$ = 1275 kips

$P_{n.2nd}$ = 1249 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)

$P_{n.bottom}$ = 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.

$$P_{r.top} = \phi_{cu} P_{n.top}$$

$P_{r.top}$ = 892.3 kips

$$P_{r.2nd} = \phi_{cu} P_{n.2nd}$$

$P_{r.2nd}$ = 874.5 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.

$$\frac{P_u}{P_{r.top}} =$$ 0.59 OK

$$\frac{P_u}{P_{r.2nd}} = \quad 0.60 \qquad \text{OK}$$

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:

$$\lambda_f = \frac{b_f}{2t_f}$$ (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 $\lambda_{pf} < \lambda_f \le \lambda_{rf}$,

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' = 1438 in-kips = 1438007.1 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'.

$M_{u.Top}$ = 2708 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.

$l_{b.top}$ = 2.99 ft (LPile)

$l_{b.top}$ = 35.83 in

$l_{b.2nd}$ = 8.76 ft (LPile)

$l_{b.2nd}$ = 105.08 in

$M_{u.2nd}$ = 1064.70 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} = 21.29 < 120 OK
λ_{2nd} = 29.73 < 120 OK

$P_{e.top}$ = 16478 kips
$P_{e.2nd}$ = 8450 kips

$P_o/P_{e.top}$ = 0.08 ≤ 2.25 (to select P_n equation)
$P_o/P_{e.2nd}$ = 0.15 ≤ 2.25 (to select P_n equation)

$P_{n.top}$ = 1262 kips
$P_{n.2nd}$ = 1223 kips

$P_{r.top}$ = 884 kips
$P_{r.2nd}$ = 856 kips

$\frac{P_u}{P_{r.top}} =$ 0.59 > 0.20 OK

$\frac{P_u}{P_{r.2nd}} =$ 0.61 > 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: 0.92 < 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 = 38.77 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

$$V_r = \Phi_v V_n$$

$\phi_v =$ 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)

$\phi_{da} =$ 1.00 (Ref. 1, Article 6.5.4.2)

$\sigma_{dr} =$ 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.

$\phi_{mon} =$ 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 exceed neither the nominal structural pile resistance (P_n) nor the maximum driving force (P_0) calculated above.

$P_{n.top}$ = 1262 kips (From Step 4)
$P_{n.2nd}$ = 1223 kips (From Step 4)

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 = 1.17 kips (LPile)
ϕ_v = 1.00 (Ref. 1, Article 6.5.4.2)
$V_{factored}$ at pile tip = 1.17 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 = 0.007

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
0.007	<	0.2

The chosen pile section can be considered pinned.

Date:	8/20/2020 Rev. 12/9/2020 (section 8 only)	**Made by:**	KAR
Project No.:	19129538	**Checked by:**	MLM
Subject:	Pile Design at Abutment 2 - Phasing South	**Reviewed by:**	CCB
Project Title:	MaineDOT I-295 Exit 22 Mallet Drive Bridge Replacement No. 5721		

CONCLUSIONS

The results of the analysis indicate that a maximum moment of 2708 in-kips (226 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 0.1 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. A maximum factored axial load (excluding downdrag) of 366 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. 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

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Files Used for Analysis

Path to file locations:
\Users\kroth\Documents\Projects\19129538 MaineDOT I-295 Freeport Exit 22 Mallet Dr Bridge\Pile Design\LPile Southeast Abutment\

Name of input data file:
Freeport Exit 22 Southeast Abutment Phasing South R2 _ No Socket.lp11d

Name of output report file:
Freeport Exit 22 Southeast Abutment Phasing South R2 _ No Socket.lp11o

Name of plot output file:
Freeport Exit 22 Southeast Abutment Phasing South R2 _ No Socket.lp11p

Name of runtime message file:
Freeport Exit 22 Southeast Abutment Phasing South R2 _ No Socket.lp11r

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Date and Time of Analysis

Date: August 20, 2020 Time: 16:54:47

Problem Title

Project Name: MaineDOT I-295 Exit 22 Mallet Drive Bridge No. 5721
Job Number: 19129538
Client: MaineDOT
Engineer: KAR
Description: Southeast 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 = 12.000 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	12.000	14.6950

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is a H weak axis steel pile
Length of section = 12.000000 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 4 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 = 6.000000 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 = 33.000000 deg.
Friction angle at bottom of layer = 33.000000 deg.
Subgrade k at top of layer = 165.000000 pci
Subgrade k at bottom of layer = 165.000000 pci

Layer 2 is stiff clay with water-induced erosion

Distance from top of pile to top of layer = 6.000000 ft
Distance from top of pile to bottom of layer = 10.000000 ft
Effective unit weight at top of layer = 115.000000 pcf
Effective unit weight at bottom of layer = 115.000000 pcf
Undrained cohesion at top of layer = 3500. psf
Undrained cohesion at bottom of layer = 3500. psf
Epsilon-50 at top of layer = 0.005000
Epsilon-50 at bottom of layer = 0.005000
Subgrade k at top of layer = 500.000000 pci
Subgrade k at bottom of layer = 500.000000 pci

Layer 3 is stiff clay without free water

Distance from top of pile to top of layer = 10.000000 ft
Distance from top of pile to bottom of layer = 12.000000 ft
Effective unit weight at top of layer = 52.600000 pcf
Effective unit weight at bottom of layer = 52.600000 pcf
Undrained cohesion at top of layer = 3500. psf
Undrained cohesion at bottom of layer = 3500. psf
Epsilon-50 at top of layer = 0.005000
Epsilon-50 at bottom of layer = 0.005000

Layer 4 is strong rock (vuggy limestone)

Distance from top of pile to top of layer = 12.000000 ft
Distance from top of pile to bottom of layer = 50.000000 ft
Effective unit weight at top of layer = 106.600000 pcf
Effective unit weight at bottom of layer = 106.600000 pcf
Uniaxial compressive strength at top of layer = 2486. psi
Uniaxial compressive strength at bottom of layer = 2486. psi

(Depth of the lowest soil layer extends 38.000 ft below the pile tip)

Summary of Input Soil Properties

Layer Soil Type Layer Effective Undrained Angle of Uniaxial E50
Layer Name Depth Unit Wt. Cohesion Friction qu or kpy

Num.	(p-y Curve Type)	ft	pcf	psf	deg.	psi	krm	pci
1	Sand	0.00	125.0000	--	33.0000	--	--	165.0000
	(Reese, et al.)	6.0000	125.0000	--	33.0000	--	--	165.0000
2	Stiff Clay	6.0000	115.0000	3500.	--	--	0.00500	500.0000
	with Free Water	10.0000	115.0000	3500.	--	--	0.00500	500.0000
3	Stiff Clay	10.0000	52.6000	3500.	--	--	0.00500	--
	w/o Free Water	12.0000	52.6000	3500.	--	--	0.00500	--
4	Strong Rock	12.0000	106.6000	--	--	2486.	--	--
	(Vuggy Limestone)	50.0000	106.6000	--	--	2486.	--	--

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

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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 = 12.000000 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	32287.
2	6.0000	69.3676	No	No	32287.	16536.
3	10.0000	3.0916	No	No	48823.	41184.
4	12.0000	12.0000	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	2708304.	-53758.	0.00	81347.	5.88E+09	0.00	0.00	0.00
0.1200	-0.6995	2630665.	-53733.	6.53E-04	79595.	5.88E+09	14.1242	29.0753	0.00
0.2400	-0.6981	2552571.	-53702.	0.00126	77832.	6.50E+09	29.5593	60.9716	0.00
0.3600	-0.6959	2474112.	-53647.	0.00181	76061.	6.74E+09	46.1515	95.4998	0.00
0.4800	-0.6929	2395352.	-53568.	0.00232	74283.	6.95E+09	63.7471	132.4771	0.00
0.6000	-0.6892	2316351.	-53463.	0.00280	72500.	7.17E+09	82.1921	171.7250	0.00
0.7200	-0.6849	2237170.	-53331.	0.00325	70713.	7.39E+09	101.1854	212.7557	0.00
0.8400	-0.6799	2157872.	-53171.	0.00367	68923.	7.63E+09	120.6325	255.5086	0.00
0.9600	-0.6743	2078518.	-52983.	0.00406	67131.	7.87E+09	140.3838	299.8040	0.00
1.0800	-0.6682	1999169.	-52767.	0.00443	65340.	8.11E+09	160.2769	345.4267	0.00
1.2000	-0.6615	1919886.	-52522.	0.00478	63551.	8.35E+09	180.1492	392.1513	0.00
1.3200	-0.6544	1840727.	-52248.	0.00510	61764.	8.58E+09	200.1416	440.4077	0.00
1.4400	-0.6468	1761752.	-51945.	0.00539	59981.	8.79E+09	219.9599	489.6746	0.00
1.5600	-0.6389	1683015.	-51615.	0.00567	58204.	8.98E+09	239.3824	539.5657	0.00
1.6800	-0.6305	1604572.	-51256.	0.00593	56433.	9.14E+09	258.2552	589.8271	0.00
1.8000	-0.6218	1526474.	-50871.	0.00618	54670.	9.28E+09	276.4245	640.1861	0.00
1.9200	-0.6127	1448772.	-50460.	0.00641	52916.	9.37E+09	295.3666	694.1805	0.00

2.0400	-0.6033	1371514.	-50021.	0.00663	51172.	9.43E+09	314.4172	750.4547	0.00
2.1600	-0.5936	1294751.	-49555.	0.00683	49440.	9.44E+09	332.7989	807.2967	0.00
2.2800	-0.5836	1218530.	-49063.	0.00702	47719.	9.44E+09	350.3888	864.4938	0.00
2.4000	-0.5734	1142896.	-48546.	0.00720	46012.	9.44E+09	367.0638	921.8137	0.00
2.5200	-0.5629	1067891.	-48006.	0.00737	44319.	9.44E+09	383.6183	981.3485	0.00
2.6400	-0.5522	993560.	-47442.	0.00753	42641.	9.44E+09	399.8432	1043.	0.00
2.7600	-0.5412	919944.	-46858.	0.00767	40979.	9.44E+09	411.1032	1094.	0.00
2.8800	-0.5301	847075.	-46259.	0.00781	39335.	9.44E+09	420.3534	1142.	0.00
3.0000	-0.5187	774980.	-45648.	0.00793	37707.	9.44E+09	427.9337	1188.	0.00
3.1200	-0.5072	703684.	-45026.	0.00804	36098.	9.44E+09	436.4328	1239.	0.00
3.2400	-0.4956	633212.	-44391.	0.00815	34507.	9.44E+09	446.2624	1297.	0.00
3.3600	-0.4838	563593.	-43742.	0.00824	32936.	9.44E+09	454.6583	1353.	0.00
3.4800	-0.4719	494852.	-43082.	0.00832	31384.	9.44E+09	461.5371	1409.	0.00
3.6000	-0.4598	427011.	-42414.	0.00839	29853.	9.44E+09	466.8178	1462.	0.00
3.7200	-0.4477	360090.	-41737.	0.00845	28342.	9.44E+09	473.4754	1523.	0.00
3.8400	-0.4355	294109.	-41048.	0.00850	26853.	9.44E+09	483.6897	1599.	0.00
3.9600	-0.4232	229097.	-40345.	0.00854	25385.	9.44E+09	492.7078	1676.	0.00
4.0800	-0.4109	165080.	-39630.	0.00857	23940.	9.44E+09	500.4499	1754.	0.00
4.2000	-0.3985	102083.	-38904.	0.00859	22518.	9.44E+09	506.8373	1831.	0.00
4.3200	-0.3862	40124.	-38171.	0.00860	21120.	9.44E+09	511.7922	1908.	0.00
4.4400	-0.3738	-20778.	-37431.	0.00860	20683.	9.44E+09	515.2381	1985.	0.00
4.5600	-0.3614	-80609.	-36688.	0.00859	22033.	9.44E+09	517.0997	2060.	0.00
4.6800	-0.3490	-139358.	-35943.	0.00858	23360.	9.44E+09	517.3030	2134.	0.00
4.8000	-0.3367	-197019.	-35200.	0.00855	24661.	9.44E+09	515.7757	2206.	0.00
4.9200	-0.3244	-253588.	-34458.	0.00852	25938.	9.44E+09	514.5925	2284.	0.00
5.0400	-0.3122	-309060.	-33711.	0.00847	27190.	9.44E+09	521.9248	2408.	0.00
5.1600	-0.3000	-363415.	-32955.	0.00842	28417.	9.44E+09	528.4494	2537.	0.00
5.2800	-0.2879	-416632.	-32190.	0.00836	29618.	9.44E+09	534.1416	2672.	0.00
5.4000	-0.2759	-468694.	-31417.	0.00830	30794.	9.44E+09	538.9789	2813.	0.00
5.5200	-0.2640	-519585.	-30638.	0.00822	31942.	9.44E+09	542.9411	2961.	0.00
5.6400	-0.2522	-569290.	-29854.	0.00814	33064.	9.44E+09	546.0103	3117.	0.00
5.7600	-0.2406	-617798.	-29067.	0.00805	34159.	9.44E+09	548.1500	3281.	0.00
5.8800	-0.2291	-665098.	-28277.	0.00795	35227.	9.44E+09	549.1137	3452.	0.00
6.0000	-0.2177	-711184.	-26310.	0.00784	36267.	9.44E+09	2183.	14437.	0.00
6.1200	-0.2065	-752661.	-23161.	0.00773	37203.	9.44E+09	2190.	15276.	0.00
6.2400	-0.1954	-789511.	-20004.	0.00761	38035.	9.44E+09	2194.	16166.	0.00
6.3600	-0.1846	-821720.	-16845.	0.00749	38762.	9.44E+09	2193.	17115.	0.00
6.4800	-0.1739	-849287.	-13690.	0.00736	39384.	9.44E+09	2189.	18128.	0.00
6.6000	-0.1633	-872218.	-10545.	0.00723	39902.	9.44E+09	2179.	19212.	0.00
6.7200	-0.1530	-890530.	-7417.	0.00710	40315.	9.44E+09	2166.	20377.	0.00
6.8400	-0.1429	-904249.	-4312.	0.00696	40625.	9.44E+09	2147.	21634.	0.00
6.9600	-0.1330	-913413.	-1237.	0.00682	40832.	9.44E+09	2124.	22995.	0.00
7.0800	-0.1233	-918068.	1800.	0.00668	40937.	9.44E+09	2095.	24477.	0.00
7.2000	-0.1137	-918274.	4793.	0.00654	40942.	9.44E+09	2061.	26099.	0.00
7.3200	-0.1044	-914100.	7733.	0.00640	40847.	9.44E+09	2022.	27885.	0.00
7.4400	-0.09530	-905628.	10612.	0.00626	40656.	9.44E+09	1977.	29869.	0.00
7.5600	-0.08638	-892954.	13421.	0.00613	40370.	9.44E+09	1925.	32089.	0.00
7.6800	-0.07765	-876186.	16150.	0.00599	39992.	9.44E+09	1866.	34601.	0.00
7.8000	-0.06912	-855449.	18789.	0.00586	39524.	9.44E+09	1799.	37476.	0.00
7.9200	-0.06078	-830883.	21324.	0.00573	38969.	9.44E+09	1722.	40812.	0.00
8.0400	-0.05261	-802651.	23741.	0.00561	38332.	9.44E+09	1635.	44739.	0.00
8.1600	-0.04463	-770937.	26020.	0.00549	37616.	9.44E+09	1530.	49372.	0.00
8.2800	-0.03681	-735961.	28123.	0.00537	36826.	9.44E+09	1391.	54392.	0.00
8.4000	-0.02916	-698018.	30015.	0.00526	35970.	9.44E+09	1238.	61115.	0.00

8.5200	-0.02166	-657429.	31674.	0.00516	35054.	9.44E+09	1067.	70913.	0.00
8.6400	-0.01430	-614552.	32976.	0.00506	34086.	9.44E+09	741.4043	74650.	0.00
8.7600	-0.00708	-570068.	33777.	0.00497	33082.	9.44E+09	372.1524	75686.	0.00
8.8800	1.55E-05	-524747.	34045.	0.00489	32059.	9.44E+09	-0.8254	76723.	0.00
9.0000	0.00700	-479367.	33772.	0.00481	31034.	9.44E+09	-377.7962	77760.	0.00
9.1200	0.01387	-434716.	32954.	0.00474	30027.	9.44E+09	-759.0566	78797.	0.00
9.2400	0.02065	-391589.	31657.	0.00468	29053.	9.44E+09	-1041.	72621.	0.00
9.3600	0.02735	-350576.	30045.	0.00462	28127.	9.44E+09	-1198.	63110.	0.00
9.4800	0.03396	-312009.	28220.	0.00457	27257.	9.44E+09	-1336.	56629.	0.00
9.6000	0.04051	-276175.	26208.	0.00453	26448.	9.44E+09	-1459.	51851.	0.00
9.7200	0.04700	-243334.	24032.	0.00449	25707.	9.44E+09	-1564.	47917.	0.00
9.8400	0.05343	-213708.	21722.	0.00445	25038.	9.44E+09	-1644.	44310.	0.00
9.9600	0.05982	-187467.	19305.	0.00442	24445.	9.44E+09	-1713.	41235.	0.00
10.0800	0.06617	-164757.	17633.	0.00439	23933.	9.44E+09	-609.3328	13261.	0.00
10.2000	0.07248	-143291.	16740.	0.00437	23448.	9.44E+09	-630.7809	12533.	0.00
10.3200	0.07875	-123117.	15817.	0.00435	22993.	9.44E+09	-651.5826	11914.	0.00
10.4400	0.08501	-104280.	14864.	0.00433	22568.	9.44E+09	-671.8491	11381.	0.00
10.5600	0.09123	-86824.	13882.	0.00432	22174.	9.44E+09	-691.6679	10917.	0.00
10.6800	0.09744	-70792.	12872.	0.00431	21812.	9.44E+09	-711.1092	10509.	0.00
10.8000	0.1036	-56226.	11834.	0.00430	21483.	9.44E+09	-730.2297	10146.	0.00
10.9200	0.1098	-43169.	10769.	0.00429	21188.	9.44E+09	-749.0762	9822.	0.00
11.0400	0.1160	-31660.	9677.	0.00428	20929.	9.44E+09	-767.6875	9531.	0.00
11.1600	0.1222	-21739.	8558.	0.00428	20705.	9.44E+09	-786.0959	9267.	0.00
11.2800	0.1283	-13445.	7413.	0.00428	20517.	9.44E+09	-804.3286	9026.	0.00
11.4000	0.1345	-6818.	6242.	0.00428	20368.	9.44E+09	-822.4087	8807.	0.00
11.5200	0.1406	-1896.	5045.	0.00427	20257.	9.44E+09	-840.3555	8605.	0.00
11.6400	0.1468	1285.	3822.	0.00427	20243.	9.44E+09	-858.1857	8419.	0.00
11.7600	0.1529	2685.	2573.	0.00427	20274.	9.44E+09	-875.9131	8247.	0.00
11.8800	0.1591	2269.	1299.	0.00428	20265.	9.44E+09	-893.5495	8088.	0.00
12.0000	0.1653	0.00	0.00	0.00428	20214.	9.44E+09	-911.1047	3970.	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 = 2708304. inch-lbs
Maximum shear force = -53758. 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 = 17
Number of zero deflection points = 1

Summary of Pile-head Responses for Conventional Analyses

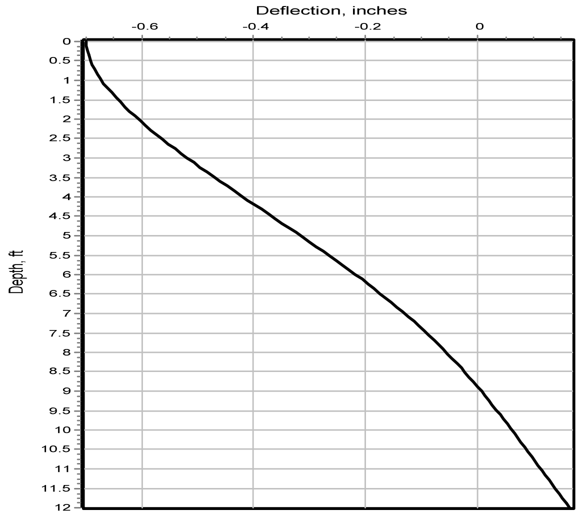
Definitions of Pile-head Loading Conditions:

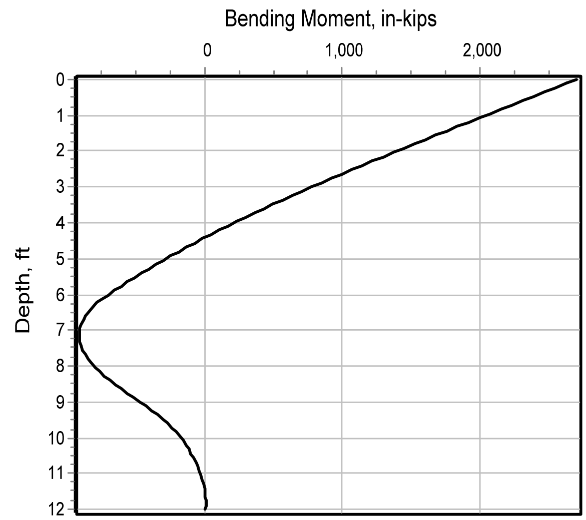
Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians
Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
Load Type 5: Load 1 = Top Deflection, y, inches, and Load 2 = Slope, S, radians

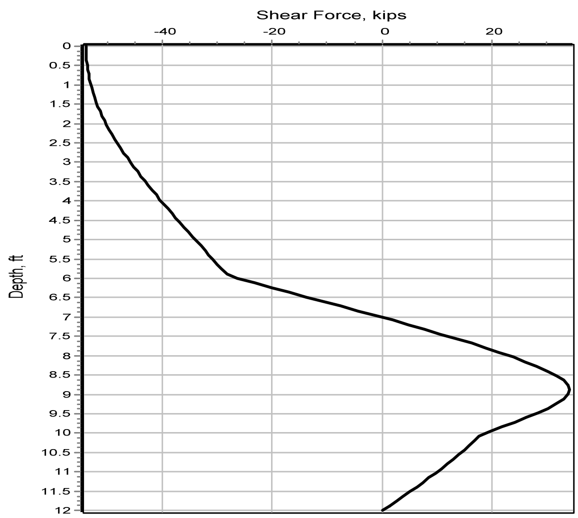
Load Case No.	Load Type 1	Pile-head Load 1	Load Type 2	Pile-head Load 2	Axial Loading lbs	Pile-head Deflection inches	Pile-head Rotation radians	Max Shear in Pile lbs	Max Moment in Pile in-lbs
1	y, in	-0.7000	S, rad	0.00	522000.	-0.7000	0.00	-53758.	2708304.

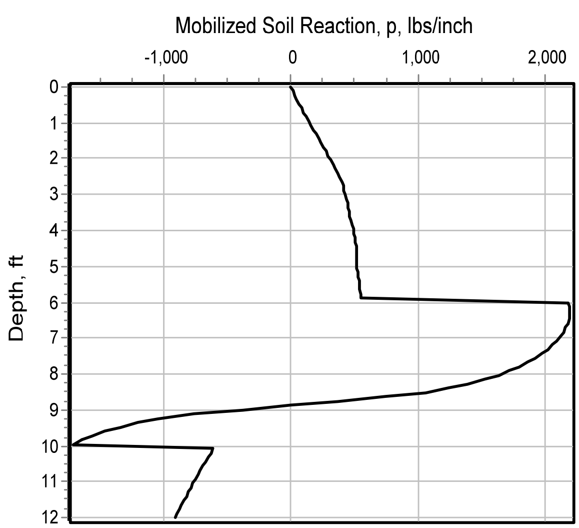
Maximum pile-head deflection = -0.7000000000 inches
Maximum pile-head rotation = 0.0000000000 radians = 0.000000 deg.

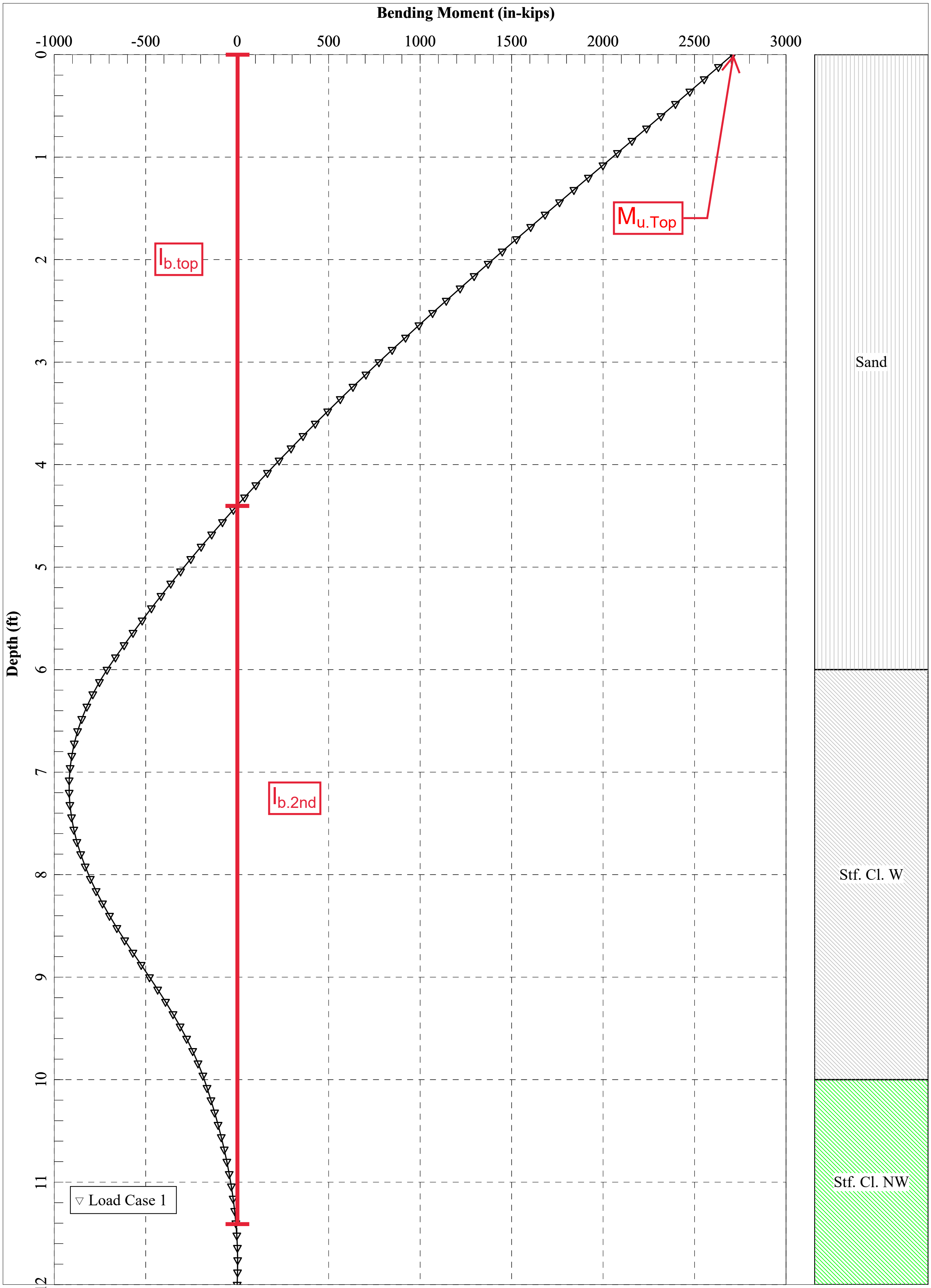
The analysis ended normally.


Deflection, inches
-0.6
-0.4
-0.2
0
Depth, ft
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
5.5
6
6.5
7
7.5
8
8.5
9
9.5
10
10.5
11
11.5
12


Bending Moment, in-kips
0
1,000
2,000
Depth, ft
0
1
2
3
4
5
6
7
8
9
10
11
12


Shear Force, kips
-40
-20
0
20
Depth, ft
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
5.5
6
6.5
7
7.5
8
8.5
9
9.5
10
10.5
11
11.5
12


Mobilized Soil Reaction, p, lbs/inch
-1,000
0
1,000
2,000
Depth, ft
0
1
2
3
4
5
6
7
8
9
10
11
12


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
$M_{u.Top}$
$l_{b.top}$
$l_{b.2nd}$
Load Case 1
Sand
Stf. Cl. W
Stf. Cl. NW

===

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:

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Files Used for Analysis

Path to file locations:
\Users\kroth\Documents\Projects\19129538 MaineDOT I-295 Freeport Exit 22 Mallet Dr Bridge\Pile Design\LPile Southeast Abutment\

Name of input data file:
Freeport Exit 22 Southeast Abutment Phasing South R2 _ No Socket _ Plastic Hinge.lp11d

Name of output report file:
Freeport Exit 22 Southeast Abutment Phasing South R2 _ No Socket _ Plastic Hinge.lp11o

Name of plot output file:
Freeport Exit 22 Southeast Abutment Phasing South R2 _ No Socket _ Plastic Hinge.lp11p

Name of runtime message file:
Freeport Exit 22 Southeast Abutment Phasing South R2 _ No Socket _ Plastic Hinge.lp11r

----
Date and Time of Analysis

Date: August 20, 2020 Time: 16:57:03

Problem Title

Project Name: MaineDOT I-295 Exit 22 Mallet Drive Bridge No. 5721
Job Number: 19129538
Client: MaineDOT
Engineer: KAR
Description: Southeast 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 = 12.000 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	12.000	14.6950

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is a H weak axis steel pile
Length of section = 12.000000 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 4 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 = 6.000000 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 = 33.000000 deg.
Friction angle at bottom of layer = 33.000000 deg.
Subgrade k at top of layer = 165.000000 pci
Subgrade k at bottom of layer = 165.000000 pci

Layer 2 is stiff clay with water-induced erosion

Distance from top of pile to top of layer = 6.000000 ft
Distance from top of pile to bottom of layer = 10.000000 ft
Effective unit weight at top of layer = 115.000000 pcf
Effective unit weight at bottom of layer = 115.000000 pcf
Undrained cohesion at top of layer = 3500. psf
Undrained cohesion at bottom of layer = 3500. psf
Epsilon-50 at top of layer = 0.005000
Epsilon-50 at bottom of layer = 0.005000
Subgrade k at top of layer = 500.000000 pci
Subgrade k at bottom of layer = 500.000000 pci

Layer 3 is stiff clay without free water

Distance from top of pile to top of layer = 10.000000 ft
Distance from top of pile to bottom of layer = 12.000000 ft
Effective unit weight at top of layer = 52.600000 pcf
Effective unit weight at bottom of layer = 52.600000 pcf
Undrained cohesion at top of layer = 3500. psf
Undrained cohesion at bottom of layer = 3500. psf
Epsilon-50 at top of layer = 0.005000
Epsilon-50 at bottom of layer = 0.005000

Layer 4 is strong rock (vuggy limestone)

Distance from top of pile to top of layer = 12.000000 ft
Distance from top of pile to bottom of layer = 50.000000 ft
Effective unit weight at top of layer = 106.600000 pcf
Effective unit weight at bottom of layer = 106.600000 pcf
Uniaxial compressive strength at top of layer = 2486. psi
Uniaxial compressive strength at bottom of layer = 2486. psi

(Depth of the lowest soil layer extends 38.000 ft below the pile tip)

Summary of Input Soil Properties

Layer	Soil Type	Layer	Effective	Undrained	Angle of	Uniaxial	E50
Layer	Name	Depth	Unit Wt.	Cohesion	Friction	qu	or kpy

Num.	(p-y Curve Type)	ft	pcf	psf	deg.	psi	krm	pci
1	Sand	0.00	125.0000	--	33.0000	--	--	165.0000
	(Reese, et al.)	6.0000	125.0000	--	33.0000	--	--	165.0000
2	Stiff Clay	6.0000	115.0000	3500.	--	--	0.00500	500.0000
	with Free Water	10.0000	115.0000	3500.	--	--	0.00500	500.0000
3	Stiff Clay	10.0000	52.6000	3500.	--	--	0.00500	--
	w/o Free Water	12.0000	52.6000	3500.	--	--	0.00500	--
4	Strong Rock	12.0000	106.6000	--	--	2486.	--	--
	(Vuggy Limestone)	50.0000	106.6000	--	--	2486.	--	--

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 = 1438007. 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
----

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	=	12.000000 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	32287.
2	6.0000	69.3676	No	No	32287.	16536.
3	10.0000	3.0916	No	No	48823.	41184.
4	12.0000	12.0000	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	2708304.	-53758.	0.00	81347.	5.88E+09	0.00	0.00	0.00
0.1200	-0.6995	2630665.	-53733.	6.53E-04	79595.	5.88E+09	14.1242	29.0753	0.00
0.2400	-0.6981	2552571.	-53702.	0.00126	77832.	6.50E+09	29.5593	60.9716	0.00
0.3600	-0.6959	2474112.	-53647.	0.00181	76061.	6.74E+09	46.1515	95.4998	0.00
0.4800	-0.6929	2395352.	-53568.	0.00232	74283.	6.95E+09	63.7471	132.4771	0.00
0.6000	-0.6892	2316351.	-53463.	0.00280	72500.	7.17E+09	82.1921	171.7250	0.00
0.7200	-0.6849	2237170.	-53331.	0.00325	70713.	7.39E+09	101.1854	212.7557	0.00
0.8400	-0.6799	2157872.	-53171.	0.00367	68923.	7.63E+09	120.6325	255.5086	0.00
0.9600	-0.6743	2078518.	-52983.	0.00406	67131.	7.87E+09	140.3838	299.8040	0.00
1.0800	-0.6682	1999169.	-52767.	0.00443	65340.	8.11E+09	160.2769	345.4267	0.00
1.2000	-0.6615	1919886.	-52522.	0.00478	63551.	8.35E+09	180.1492	392.1513	0.00
1.3200	-0.6544	1840727.	-52248.	0.00510	61764.	8.58E+09	200.1416	440.4077	0.00
1.4400	-0.6468	1761752.	-51945.	0.00539	59981.	8.79E+09	219.9599	489.6746	0.00
1.5600	-0.6389	1683015.	-51615.	0.00567	58204.	8.98E+09	239.3824	539.5657	0.00
1.6800	-0.6305	1604572.	-51256.	0.00593	56433.	9.14E+09	258.2552	589.8271	0.00
1.8000	-0.6218	1526474.	-50871.	0.00618	54670.	9.28E+09	276.4245	640.1861	0.00

1.9200	-0.6127	1448772.	-50460.	0.00641	52916.	9.37E+09	295.3666	694.1805	0.00
2.0400	-0.6033	1371514.	-50021.	0.00663	51172.	9.43E+09	314.4172	750.4547	0.00
2.1600	-0.5936	1294751.	-49555.	0.00683	49440.	9.44E+09	332.7989	807.2967	0.00
2.2800	-0.5836	1218530.	-49063.	0.00702	47719.	9.44E+09	350.3888	864.4938	0.00
2.4000	-0.5734	1142896.	-48546.	0.00720	46012.	9.44E+09	367.0638	921.8137	0.00
2.5200	-0.5629	1067891.	-48006.	0.00737	44319.	9.44E+09	383.6183	981.3485	0.00
2.6400	-0.5522	993560.	-47442.	0.00753	42641.	9.44E+09	399.8432	1043.	0.00
2.7600	-0.5412	919944.	-46858.	0.00767	40979.	9.44E+09	411.1032	1094.	0.00
2.8800	-0.5301	847075.	-46259.	0.00781	39335.	9.44E+09	420.3534	1142.	0.00
3.0000	-0.5187	774980.	-45648.	0.00793	37707.	9.44E+09	427.9337	1188.	0.00
3.1200	-0.5072	703684.	-45026.	0.00804	36098.	9.44E+09	436.4328	1239.	0.00
3.2400	-0.4956	633212.	-44391.	0.00815	34507.	9.44E+09	446.2624	1297.	0.00
3.3600	-0.4838	563593.	-43742.	0.00824	32936.	9.44E+09	454.6583	1353.	0.00
3.4800	-0.4719	494852.	-43082.	0.00832	31384.	9.44E+09	461.5371	1409.	0.00
3.6000	-0.4598	427011.	-42414.	0.00839	29853.	9.44E+09	466.8178	1462.	0.00
3.7200	-0.4477	360090.	-41737.	0.00845	28342.	9.44E+09	473.4754	1523.	0.00
3.8400	-0.4355	294109.	-41048.	0.00850	26853.	9.44E+09	483.6897	1599.	0.00
3.9600	-0.4232	229097.	-40345.	0.00854	25385.	9.44E+09	492.7078	1676.	0.00
4.0800	-0.4109	165080.	-39630.	0.00857	23940.	9.44E+09	500.4499	1754.	0.00
4.2000	-0.3985	102083.	-38904.	0.00859	22518.	9.44E+09	506.8373	1831.	0.00
4.3200	-0.3862	40124.	-38171.	0.00860	21120.	9.44E+09	511.7922	1908.	0.00
4.4400	-0.3738	-20778.	-37431.	0.00860	20683.	9.44E+09	515.2381	1985.	0.00
4.5600	-0.3614	-80609.	-36688.	0.00859	22033.	9.44E+09	517.0997	2060.	0.00
4.6800	-0.3490	-139358.	-35943.	0.00858	23360.	9.44E+09	517.3030	2134.	0.00
4.8000	-0.3367	-197019.	-35200.	0.00855	24661.	9.44E+09	515.7757	2206.	0.00
4.9200	-0.3244	-253588.	-34458.	0.00852	25938.	9.44E+09	514.5925	2284.	0.00
5.0400	-0.3122	-309060.	-33711.	0.00847	27190.	9.44E+09	521.9248	2408.	0.00
5.1600	-0.3000	-363415.	-32955.	0.00842	28417.	9.44E+09	528.4494	2537.	0.00
5.2800	-0.2879	-416632.	-32190.	0.00836	29618.	9.44E+09	534.1416	2672.	0.00
5.4000	-0.2759	-468694.	-31417.	0.00830	30794.	9.44E+09	538.9789	2813.	0.00
5.5200	-0.2640	-519585.	-30638.	0.00822	31942.	9.44E+09	542.9411	2961.	0.00
5.6400	-0.2522	-569290.	-29854.	0.00814	33064.	9.44E+09	546.0103	3117.	0.00
5.7600	-0.2406	-617798.	-29067.	0.00805	34159.	9.44E+09	548.1500	3281.	0.00
5.8800	-0.2291	-665098.	-28277.	0.00795	35227.	9.44E+09	549.1137	3452.	0.00
6.0000	-0.2177	-711184.	-26310.	0.00784	36267.	9.44E+09	2183.	14437.	0.00
6.1200	-0.2065	-752661.	-23161.	0.00773	37203.	9.44E+09	2190.	15276.	0.00
6.2400	-0.1954	-789511.	-20004.	0.00761	38035.	9.44E+09	2194.	16166.	0.00
6.3600	-0.1846	-821720.	-16845.	0.00749	38762.	9.44E+09	2193.	17115.	0.00
6.4800	-0.1739	-849287.	-13690.	0.00736	39384.	9.44E+09	2189.	18128.	0.00
6.6000	-0.1633	-872218.	-10545.	0.00723	39902.	9.44E+09	2179.	19212.	0.00
6.7200	-0.1530	-890530.	-7417.	0.00710	40315.	9.44E+09	2166.	20377.	0.00
6.8400	-0.1429	-904249.	-4312.	0.00696	40625.	9.44E+09	2147.	21634.	0.00
6.9600	-0.1330	-913413.	-1237.	0.00682	40832.	9.44E+09	2124.	22995.	0.00
7.0800	-0.1233	-918068.	1800.	0.00668	40937.	9.44E+09	2095.	24477.	0.00
7.2000	-0.1137	-918274.	4793.	0.00654	40942.	9.44E+09	2061.	26099.	0.00
7.3200	-0.1044	-914100.	7733.	0.00640	40847.	9.44E+09	2022.	27885.	0.00
7.4400	-0.09530	-905628.	10612.	0.00626	40656.	9.44E+09	1977.	29869.	0.00
7.5600	-0.08638	-892954.	13421.	0.00613	40370.	9.44E+09	1925.	32089.	0.00
7.6800	-0.07765	-876186.	16150.	0.00599	39992.	9.44E+09	1866.	34601.	0.00
7.8000	-0.06912	-855449.	18789.	0.00586	39524.	9.44E+09	1799.	37476.	0.00
7.9200	-0.06078	-830883.	21324.	0.00573	38969.	9.44E+09	1722.	40812.	0.00
8.0400	-0.05261	-802651.	23741.	0.00561	38332.	9.44E+09	1635.	44739.	0.00
8.1600	-0.04463	-770937.	26020.	0.00549	37616.	9.44E+09	1530.	49372.	0.00
8.2800	-0.03681	-735961.	28123.	0.00537	36826.	9.44E+09	1391.	54392.	0.00

8.4000	-0.02916	-698018.	30015.	0.00526	35970.	9.44E+09	1238.	61115.	0.00
8.5200	-0.02166	-657429.	31674.	0.00516	35054.	9.44E+09	1067.	70913.	0.00
8.6400	-0.01430	-614552.	32976.	0.00506	34086.	9.44E+09	741.4043	74650.	0.00
8.7600	-0.00708	-570068.	33777.	0.00497	33082.	9.44E+09	372.1524	75686.	0.00
8.8800	1.55E-05	-524747.	34045.	0.00489	32059.	9.44E+09	-0.8254	76723.	0.00
9.0000	0.00700	-479367.	33772.	0.00481	31034.	9.44E+09	-377.7962	77760.	0.00
9.1200	0.01387	-434716.	32954.	0.00474	30027.	9.44E+09	-759.0566	78797.	0.00
9.2400	0.02065	-391589.	31657.	0.00468	29053.	9.44E+09	-1041.	72621.	0.00
9.3600	0.02735	-350576.	30045.	0.00462	28127.	9.44E+09	-1198.	63110.	0.00
9.4800	0.03396	-312009.	28220.	0.00457	27257.	9.44E+09	-1336.	56629.	0.00
9.6000	0.04051	-276175.	26208.	0.00453	26448.	9.44E+09	-1459.	51851.	0.00
9.7200	0.04700	-243334.	24032.	0.00449	25707.	9.44E+09	-1564.	47917.	0.00
9.8400	0.05343	-213708.	21722.	0.00445	25038.	9.44E+09	-1644.	44310.	0.00
9.9600	0.05982	-187467.	19305.	0.00442	24445.	9.44E+09	-1713.	41235.	0.00
10.0800	0.06617	-164757.	17633.	0.00439	23933.	9.44E+09	-609.3328	13261.	0.00
10.2000	0.07248	-143291.	16740.	0.00437	23448.	9.44E+09	-630.7809	12533.	0.00
10.3200	0.07875	-123117.	15817.	0.00435	22993.	9.44E+09	-651.5826	11914.	0.00
10.4400	0.08501	-104280.	14864.	0.00433	22568.	9.44E+09	-671.8491	11381.	0.00
10.5600	0.09123	-86824.	13882.	0.00432	22174.	9.44E+09	-691.6679	10917.	0.00
10.6800	0.09744	-70792.	12872.	0.00431	21812.	9.44E+09	-711.1092	10509.	0.00
10.8000	0.1036	-56226.	11834.	0.00430	21483.	9.44E+09	-730.2297	10146.	0.00
10.9200	0.1098	-43169.	10769.	0.00429	21188.	9.44E+09	-749.0762	9822.	0.00
11.0400	0.1160	-31660.	9677.	0.00428	20929.	9.44E+09	-767.6875	9531.	0.00
11.1600	0.1222	-21739.	8558.	0.00428	20705.	9.44E+09	-786.0959	9267.	0.00
11.2800	0.1283	-13445.	7413.	0.00428	20517.	9.44E+09	-804.3286	9026.	0.00
11.4000	0.1345	-6818.	6242.	0.00428	20368.	9.44E+09	-822.4087	8807.	0.00
11.5200	0.1406	-1896.	5045.	0.00427	20257.	9.44E+09	-840.3555	8605.	0.00
11.6400	0.1468	1285.	3822.	0.00427	20243.	9.44E+09	-858.1857	8419.	0.00
11.7600	0.1529	2685.	2573.	0.00427	20274.	9.44E+09	-875.9131	8247.	0.00
11.8800	0.1591	2269.	1299.	0.00428	20265.	9.44E+09	-893.5495	8088.	0.00
12.0000	0.1653	0.00	0.00	0.00428	20214.	9.44E+09	-911.1047	3970.	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 = 2708304. inch-lbs
Maximum shear force = -53758. 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 = 17
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 = 1438007.0 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	1438007.	-38765.	0.00581	52673.	9.38E+09	0.00	0.00	0.00
0.1200	-0.6915	1377733.	-38755.	0.00603	51313.	9.38E+09	14.1242	29.4139	0.00
0.2400	-0.6826	1317330.	-38723.	0.00624	49949.	9.44E+09	29.5593	62.3544	0.00
0.3600	-0.6735	1256837.	-38669.	0.00643	48584.	9.44E+09	46.1514	98.6739	0.00
0.4800	-0.6641	1196295.	-38589.	0.00662	47217.	9.44E+09	63.7469	138.2229	0.00
0.6000	-0.6544	1135749.	-38484.	0.00680	45851.	9.44E+09	82.1919	180.8485	0.00
0.7200	-0.6445	1075242.	-38352.	0.00697	44485.	9.44E+09	101.1851	226.0636	0.00
0.8400	-0.6344	1014822.	-38193.	0.00712	43121.	9.44E+09	120.6320	273.8224	0.00
0.9600	-0.6240	954536.	-38005.	0.00728	41760.	9.44E+09	140.3832	323.9518	0.00
1.0800	-0.6134	894432.	-37788.	0.00742	40403.	9.44E+09	160.2762	376.2369	0.00
1.2000	-0.6027	834557.	-37543.	0.00755	39052.	9.44E+09	180.1483	430.4478	0.00
1.3200	-0.5917	774960.	-37269.	0.00767	37707.	9.44E+09	200.1405	487.0759	0.00
1.4400	-0.5806	715690.	-36967.	0.00778	36369.	9.44E+09	219.9587	545.5701	0.00
1.5600	-0.5693	656793.	-36636.	0.00789	35039.	9.44E+09	239.3809	605.5172	0.00
1.6800	-0.5578	598317.	-36278.	0.00798	33719.	9.44E+09	258.2536	666.6436	0.00
1.8000	-0.5463	540309.	-35894.	0.00807	32410.	9.44E+09	275.2237	725.4888	0.00
1.9200	-0.5346	482809.	-35486.	0.00815	31112.	9.44E+09	290.9202	783.6231	0.00
2.0400	-0.5228	425857.	-35056.	0.00822	29827.	9.44E+09	306.2334	843.4707	0.00
2.1600	-0.5109	369491.	-34605.	0.00828	28554.	9.44E+09	320.3938	902.9964	0.00
2.2800	-0.4990	313747.	-34134.	0.00833	27296.	9.44E+09	333.2884	961.8604	0.00
2.4000	-0.4869	258658.	-33646.	0.00838	26052.	9.44E+09	344.8090	1020.	0.00
2.5200	-0.4748	204255.	-33142.	0.00841	24824.	9.44E+09	355.7706	1079.	0.00
2.6400	-0.4627	150565.	-32622.	0.00844	23613.	9.44E+09	365.9895	1139.	0.00
2.7600	-0.4505	97618.	-32089.	0.00846	22417.	9.44E+09	374.7976	1198.	0.00
2.8800	-0.4384	45436.	-31544.	0.00847	21239.	9.44E+09	382.1209	1255.	0.00
3.0000	-0.4262	-5958.	-30989.	0.00847	20348.	9.44E+09	387.8901	1311.	0.00
3.1200	-0.4140	-56547.	-30426.	0.00847	21490.	9.44E+09	394.2720	1372.	0.00
3.2400	-0.4018	-106313.	-29853.	0.00845	22614.	9.44E+09	401.5769	1439.	0.00
3.3600	-0.3896	-155233.	-29271.	0.00843	23718.	9.44E+09	407.4803	1506.	0.00
3.4800	-0.3775	-203291.	-28681.	0.00841	24803.	9.44E+09	411.9108	1571.	0.00
3.6000	-0.3654	-250471.	-28085.	0.00837	25868.	9.44E+09	414.7992	1635.	0.00
3.7200	-0.3534	-296762.	-27485.	0.00833	26913.	9.44E+09	418.7788	1707.	0.00
3.8400	-0.3414	-342151.	-26877.	0.00828	27937.	9.44E+09	425.7630	1796.	0.00
3.9600	-0.3295	-386618.	-26260.	0.00823	28941.	9.44E+09	431.5444	1886.	0.00
4.0800	-0.3177	-430146.	-25635.	0.00816	29923.	9.44E+09	436.0557	1976.	0.00
4.2000	-0.3060	-472720.	-25005.	0.00809	30884.	9.44E+09	439.2310	2067.	0.00
4.3200	-0.2944	-514329.	-24371.	0.00802	31824.	9.44E+09	441.0058	2157.	0.00
4.4400	-0.2829	-554964.	-23736.	0.00794	32741.	9.44E+09	441.3170	2246.	0.00
4.5600	-0.2715	-594621.	-23101.	0.00785	33636.	9.44E+09	440.1030	2334.	0.00
4.6800	-0.2603	-633297.	-22470.	0.00776	34509.	9.44E+09	437.3038	2419.	0.00
4.8000	-0.2492	-670994.	-21843.	0.00766	35360.	9.44E+09	432.8614	2501.	0.00

4.9200	-0.2383	-707716.	-21223.	0.00755	36189.	9.44E+09	428.8652	2592.	0.00
5.0400	-0.2275	-743468.	-20602.	0.00744	36996.	9.44E+09	433.4815	2744.	0.00
5.1600	-0.2168	-778236.	-19975.	0.00732	37781.	9.44E+09	437.1755	2903.	0.00
5.2800	-0.2064	-812007.	-19343.	0.00720	38543.	9.44E+09	439.9076	3070.	0.00
5.4000	-0.1961	-844774.	-18709.	0.00708	39283.	9.44E+09	441.6400	3243.	0.00
5.5200	-0.1860	-876528.	-18072.	0.00695	39999.	9.44E+09	442.3372	3425.	0.00
5.6400	-0.1761	-907264.	-17435.	0.00681	40693.	9.44E+09	441.9663	3614.	0.00
5.7600	-0.1664	-936979.	-16800.	0.00667	41364.	9.44E+09	440.4972	3813.	0.00
5.8800	-0.1569	-965674.	-16168.	0.00652	42012.	9.44E+09	437.9029	4020.	0.00
6.0000	-0.1476	-993350.	-14300.	0.00637	42636.	9.44E+09	2156.	21038.	0.00
6.1200	-0.1385	-1016441.	-11209.	0.00622	43158.	9.44E+09	2137.	22219.	0.00
6.2400	-0.1297	-1034984.	-8147.	0.00606	43576.	9.44E+09	2114.	23482.	0.00
6.3600	-0.1210	-1049023.	-5122.	0.00591	43893.	9.44E+09	2088.	24837.	0.00
6.4800	-0.1127	-1058613.	-2138.	0.00574	44109.	9.44E+09	2057.	26294.	0.00
6.6000	-0.1045	-1063816.	799.6726	0.00558	44227.	9.44E+09	2022.	27867.	0.00
6.7200	-0.09658	-1064704.	3684.	0.00542	44247.	9.44E+09	1983.	29573.	0.00
6.8400	-0.08889	-1061356.	6509.	0.00526	44171.	9.44E+09	1940.	31431.	0.00
6.9600	-0.08143	-1053864.	9268.	0.00510	44002.	9.44E+09	1892.	33464.	0.00
7.0800	-0.07421	-1042326.	11956.	0.00494	43742.	9.44E+09	1840.	35704.	0.00
7.2000	-0.06721	-1026854.	14564.	0.00478	43393.	9.44E+09	1782.	38185.	0.00
7.3200	-0.06044	-1007568.	17085.	0.00462	42957.	9.44E+09	1719.	40956.	0.00
7.4400	-0.05390	-984602.	19510.	0.00447	42439.	9.44E+09	1649.	44069.	0.00
7.5600	-0.04756	-958103.	21829.	0.00432	41841.	9.44E+09	1571.	47576.	0.00
7.6800	-0.04144	-928235.	24023.	0.00418	41167.	9.44E+09	1475.	51262.	0.00
7.8000	-0.03553	-895201.	26068.	0.00404	40421.	9.44E+09	1366.	55367.	0.00
7.9200	-0.02980	-859233.	27953.	0.00391	39609.	9.44E+09	1251.	60447.	0.00
8.0400	-0.02427	-820572.	29666.	0.00378	38736.	9.44E+09	1129.	66981.	0.00
8.1600	-0.01892	-779475.	31146.	0.00366	37809.	9.44E+09	926.3527	70502.	0.00
8.2800	-0.01374	-736368.	32305.	0.00354	36836.	9.44E+09	682.6039	71539.	0.00
8.4000	-0.00872	-691761.	33113.	0.00343	35829.	9.44E+09	439.5484	72576.	0.00
8.5200	-0.00385	-646164.	33571.	0.00333	34799.	9.44E+09	197.0346	73613.	0.00
8.6400	8.71E-04	-600083.	33680.	0.00324	33759.	9.44E+09	-45.1285	74650.	0.00
8.7600	0.00546	-554028.	33441.	0.00315	32720.	9.44E+09	-287.1663	75686.	0.00
8.8800	0.00993	-508504.	32853.	0.00307	31692.	9.44E+09	-529.3332	76723.	0.00
9.0000	0.01429	-464020.	31916.	0.00299	30688.	9.44E+09	-771.9064	77760.	0.00
9.1200	0.01855	-421083.	30650.	0.00292	29719.	9.44E+09	-987.2032	76625.	0.00
9.2400	0.02272	-380145.	29153.	0.00286	28795.	9.44E+09	-1092.	69245.	0.00
9.3600	0.02680	-341429.	27512.	0.00281	27921.	9.44E+09	-1186.	63753.	0.00
9.4800	0.03081	-305134.	25742.	0.00276	27102.	9.44E+09	-1272.	59462.	0.00
9.6000	0.03475	-271441.	23853.	0.00272	26341.	9.44E+09	-1351.	55990.	0.00
9.7200	0.03863	-240519.	21855.	0.00268	25643.	9.44E+09	-1424.	53103.	0.00
9.8400	0.04245	-212523.	19754.	0.00264	25011.	9.44E+09	-1493.	50653.	0.00
9.9600	0.04623	-187598.	17560.	0.00261	24448.	9.44E+09	-1553.	48384.	0.00
10.0800	0.04997	-165874.	16033.	0.00258	23958.	9.44E+09	-568.0638	16369.	0.00
10.2000	0.05368	-145308.	15203.	0.00256	23494.	9.44E+09	-585.1825	15699.	0.00
10.3200	0.05735	-125940.	14348.	0.00254	23057.	9.44E+09	-601.9270	15115.	0.00
10.4400	0.06099	-107804.	13469.	0.00252	22647.	9.44E+09	-618.3573	14600.	0.00
10.5600	0.06461	-90939.	12567.	0.00251	22267.	9.44E+09	-634.5225	14142.	0.00
10.6800	0.06821	-75379.	11642.	0.00249	21915.	9.44E+09	-650.4632	13732.	0.00
10.8000	0.07179	-61159.	10694.	0.00248	21594.	9.44E+09	-666.2133	13363.	0.00
10.9200	0.07536	-48314.	9723.	0.00248	21304.	9.44E+09	-681.8012	13028.	0.00
11.0400	0.07892	-36877.	8731.	0.00247	21046.	9.44E+09	-697.2511	12722.	0.00
11.1600	0.08247	-26882.	7715.	0.00246	20821.	9.44E+09	-712.5832	12442.	0.00
11.2800	0.08602	-18361.	6678.	0.00246	20628.	9.44E+09	-727.8150	12184.	0.00

11.4000	0.08956	-11347.	5619.	0.00246	20470.	9.44E+09	-742.9611	11946.	0.00
11.5200	0.09309	-5872.	4539.	0.00246	20346.	9.44E+09	-758.0338	11725.	0.00
11.6400	0.09663	-1969.	3436.	0.00246	20258.	9.44E+09	-773.0438	11520.	0.00
11.7600	0.1002	331.8434	2312.	0.00246	20221.	9.44E+09	-787.9997	11328.	0.00
11.8800	0.1037	998.4346	1167.	0.00246	20236.	9.44E+09	-802.9086	11149.	0.00
12.0000	0.1072	0.00	0.00	0.00246	20214.	9.44E+09	-817.7763	5490.	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 = 0.00581317 radians
Maximum bending moment = 1438007. inch-lbs
Maximum shear force = -38765. 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

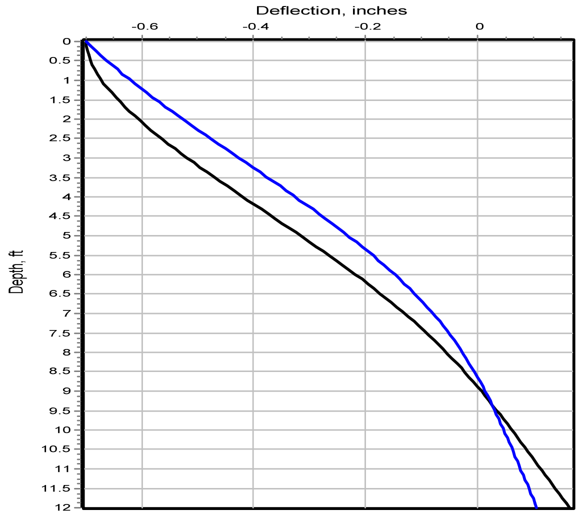
Definitions of Pile-head Loading Conditions:

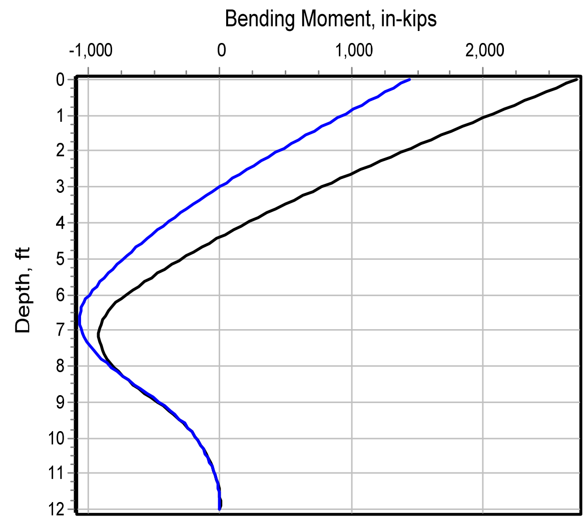
Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians
Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
Load Type 5: Load 1 = Top Deflection, y, inches, and Load 2 = Slope, S, radians

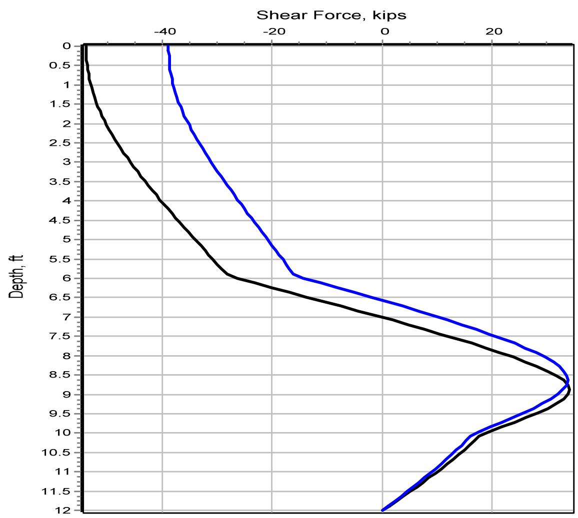
Load Case No.	Load Type 1	Pile-head Load 1	Load Type 2	Pile-head Load 2	Axial Loading lbs	Pile-head Deflection inches	Pile-head Rotation radians	Max Shear in Pile lbs	Max Moment in Pile in-lbs
1	y, in	-0.7000	S, rad	0.00	522000.	-0.7000	0.00	-53758.	2708304.
2	y, in	-0.7000	M, in-lb	1438007.	522000.	-0.7000	0.00581	-38765.	1438007.

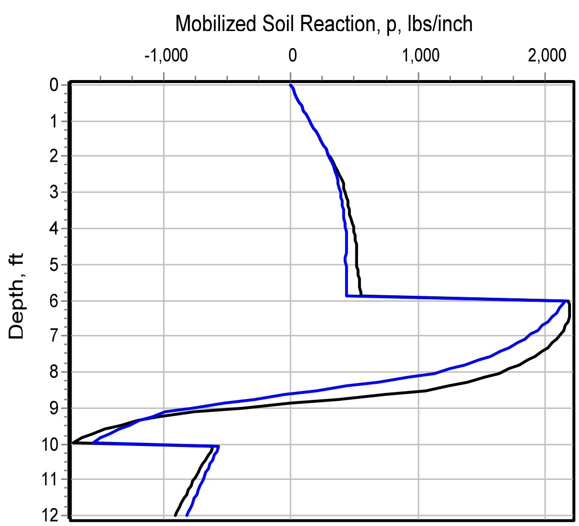
Maximum pile-head deflection = -0.7000000000 inches
Maximum pile-head rotation = 0.0058131672 radians = 0.333070 deg.

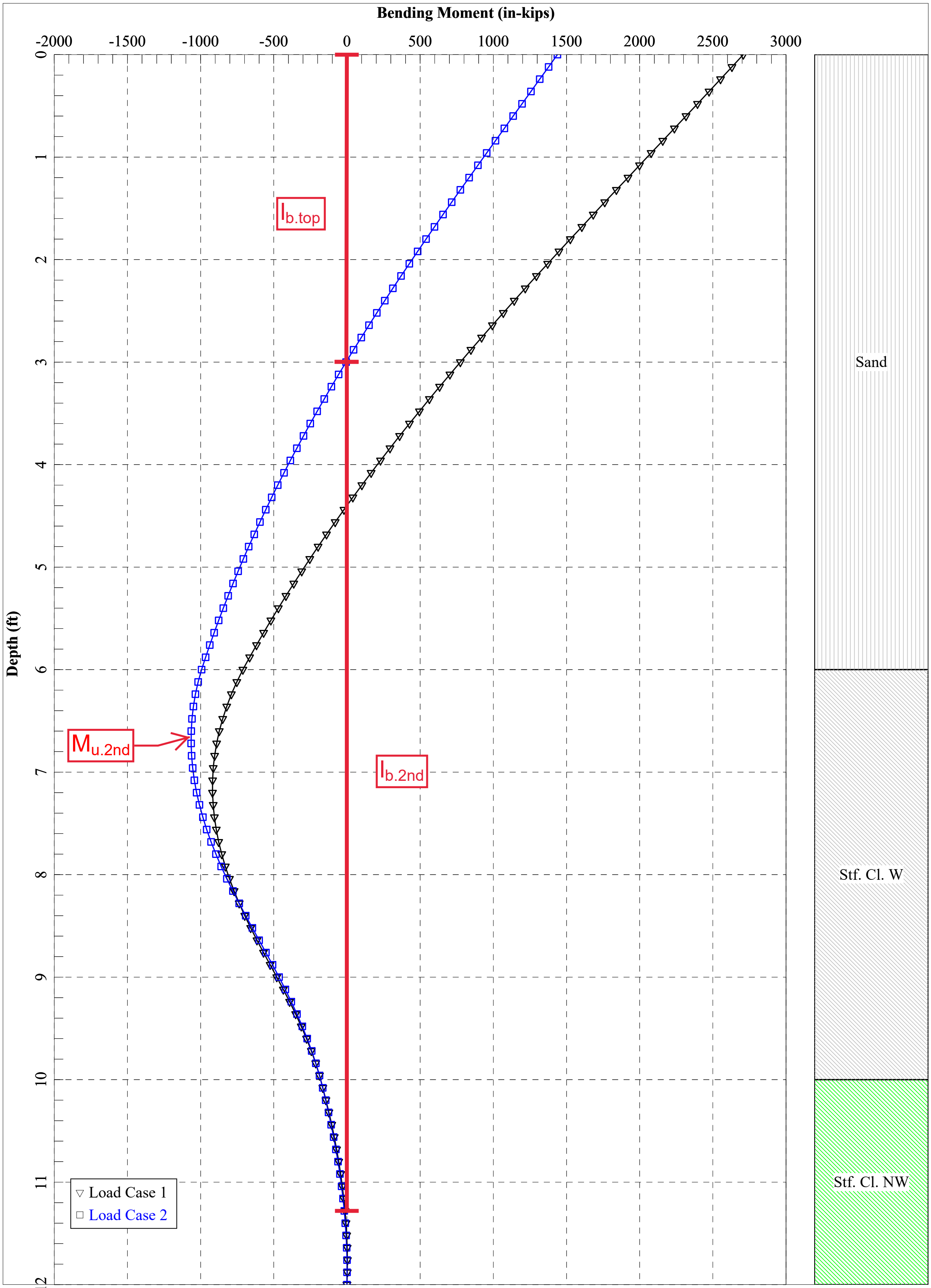
The analysis ended normally.


Deflection, inches
-0.6
-0.4
-0.2
0
Depth, ft


Bending Moment, in-kips
-1,000
0
1,000
2,000
Depth, ft


Shear Force, kips
-40
-20
0
20
Depth, ft


Mobilized Soil Reaction, p, lbs/inch
-1,000
0
1,000
2,000
Depth, ft


Bending Moment (in-kips)
-2000
-1500
-1000
-500
0
500
1000
1500
2000
2500
3000
Depth (ft)
0
1
2
3
4
5
6
7
8
9
10
11
12
Ib.top
Mu.2nd
Ib.2nd
Load Case 1
Load Case 2
Sand
Stf. Cl. W
Stf. Cl. NW

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\19129538 MaineDOT I-295 Freeport Exit 22 Mallet Dr Bridge\Pile Design\LPile Southeast Abutment\

Name of input data file:
Freeport Exit 22 Southeast Abutment Phasing South R2 _ No Socket _ Service1.lp11d

Name of output report file:
Freeport Exit 22 Southeast Abutment Phasing South R2 _ No Socket _ Service1.lp11o

Name of plot output file:
Freeport Exit 22 Southeast Abutment Phasing South R2 _ No Socket _ Service1.lp11p

Name of runtime message file:
Freeport Exit 22 Southeast Abutment Phasing South R2 _ No Socket _ Service1.lp11r

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Date and Time of Analysis

Date: August 21, 2020   Time: 10:58:29

Problem Title

Project Name: MaineDOT I-295 Exit 22 Mallet Drive Bridge No. 5721
Job Number: 19129538
Client: MaineDOT
Engineer: KAR
Description: Southeast 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 = 12.000 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	12.000	14.6950

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is a H weak axis steel pile
Length of section = 12.000000 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 4 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 = 6.000000 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 = 33.000000 deg.
Friction angle at bottom of layer = 33.000000 deg.
Subgrade k at top of layer = 165.000000 pci
Subgrade k at bottom of layer = 165.000000 pci

Layer 2 is stiff clay with water-induced erosion

Distance from top of pile to top of layer = 6.000000 ft
Distance from top of pile to bottom of layer = 10.000000 ft
Effective unit weight at top of layer = 115.000000 pcf
Effective unit weight at bottom of layer = 115.000000 pcf
Undrained cohesion at top of layer = 3500. psf
Undrained cohesion at bottom of layer = 3500. psf
Epsilon-50 at top of layer = 0.005000
Epsilon-50 at bottom of layer = 0.005000
Subgrade k at top of layer = 500.000000 pci
Subgrade k at bottom of layer = 500.000000 pci

Layer 3 is stiff clay without free water

Distance from top of pile to top of layer = 10.000000 ft
Distance from top of pile to bottom of layer = 12.000000 ft
Effective unit weight at top of layer = 52.600000 pcf
Effective unit weight at bottom of layer = 52.600000 pcf
Undrained cohesion at top of layer = 3500. psf
Undrained cohesion at bottom of layer = 3500. psf
Epsilon-50 at top of layer = 0.005000
Epsilon-50 at bottom of layer = 0.005000

Layer 4 is strong rock (vuggy limestone)

Distance from top of pile to top of layer = 12.000000 ft
Distance from top of pile to bottom of layer = 50.000000 ft
Effective unit weight at top of layer = 106.600000 pcf
Effective unit weight at bottom of layer = 106.600000 pcf
Uniaxial compressive strength at top of layer = 2486. psi
Uniaxial compressive strength at bottom of layer = 2486. psi

(Depth of the lowest soil layer extends 38.000 ft below the pile tip)

Summary of Input Soil Properties

Layer Soil Type Layer Effective Undrained Angle of Uniaxial E50
Layer Name Depth Unit Wt. Cohesion Friction qu or kpy

Num.	(p-y Curve Type)	ft	pcf	psf	deg.	psi	krm	pci
1	Sand	0.00	125.0000	--	33.0000	--	--	165.0000
	(Reese, et al.)	6.0000	125.0000	--	33.0000	--	--	165.0000
2	Stiff Clay	6.0000	115.0000	3500.	--	--	0.00500	500.0000
	with Free Water	10.0000	115.0000	3500.	--	--	0.00500	500.0000
3	Stiff Clay	10.0000	52.6000	3500.	--	--	0.00500	--
	w/o Free Water	12.0000	52.6000	3500.	--	--	0.00500	--
4	Strong Rock	12.0000	106.6000	--	--	2486.	--	--
	(Vuggy Limestone)	50.0000	106.6000	--	--	2486.	--	--

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	414000.	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	=	12.000000 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	414.000

Definition of Run Messages:

Y = part of pipe section has yielded.

Axial Thrust Force = 414.000 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	40.5774927	9438779.	135.9389070	16.9385557	
0.00000860	81.1549855	9438779.	71.6432035	17.8454198	
0.00001290	121.7324782	9438779.	50.2113023	18.7522838	
0.00001720	162.3099709	9438779.	39.4953518	19.6591481	
0.00002150	202.8874636	9438779.	33.0657814	20.5660118	
0.00002579	243.4649564	9438779.	28.7794012	21.4728760	
0.00003009	284.0424491	9438779.	25.7177010	22.3797401	
0.00003439	324.6199418	9438779.	23.4214259	23.2866043	
0.00003869	365.1974346	9438779.	21.6354341	24.1934683	
0.00004299	405.7749273	9438779.	20.2066407	25.1003323	
0.00004729	446.3524200	9438779.	19.0376279	26.0071965	
0.00005159	486.9299128	9438779.	18.0634506	26.9140605	

0.00005589	527.5074055	9438779.	17.2391467	27.8209245	
0.00006019	568.0848982	9438779.	16.5326005	28.7277887	
0.00006449	608.6623909	9438779.	15.9202605	29.6346526	
0.00006878	649.2398837	9438779.	15.3844629	30.5415168	
0.00007308	689.8173764	9438779.	14.9117004	31.4483809	
0.00007738	730.3948691	9438779.	14.4914671	32.3552449	
0.00008168	770.9723619	9438779.	14.1154688	33.2621090	
0.00008598	811.5498546	9438779.	13.7770704	34.1689730	
0.00009028	852.1273473	9438779.	13.4709003	35.0758372	
0.00009458	892.7048401	9438779.	13.1925640	35.9827012	
0.00009888	933.2823328	9438779.	12.9384307	36.8895653	
0.0001032	973.8598255	9438779.	12.7054753	37.7964294	
0.0001075	1014.	9438779.	12.4911563	38.7032934	
0.0001118	1055.	9438779.	12.2933233	39.6101575	
0.0001161	1096.	9438779.	12.1101447	40.5170216	
0.0001204	1136.	9438779.	11.9400503	41.4238857	
0.0001247	1177.	9438779.	11.7816864	42.3307497	
0.0001290	1217.	9438779.	11.6338802	43.2376138	
0.0001333	1258.	9438779.	11.4956099	44.1444779	
0.0001376	1298.	9438779.	11.3659815	45.0513420	
0.0001419	1339.	9438779.	11.2442093	45.9582061	
0.0001462	1380.	9438779.	11.1296002	46.8650702	
0.0001505	1420.	9438779.	11.0215402	47.7719342	
0.0001548	1461.	9438779.	10.9194835	48.6787983	
0.0001591	1501.	9438779.	10.8229434	49.5856623	
0.0001634	1541.	9434787.	10.7322172	50.0000000	Y
0.0001677	1580.	9422313.	10.6477586	50.0000000	Y
0.0001763	1652.	9374196.	10.4960139	50.0000000	Y
0.0001849	1720.	9304622.	10.3635947	50.0000000	Y
0.0001935	1784.	9219403.	10.2475420	50.0000000	Y
0.0002021	1843.	9123139.	10.1453978	50.0000000	Y
0.0002107	1900.	9019556.	10.0550835	50.0000000	Y
0.0002192	1954.	8911058.	9.9749770	50.0000000	Y
0.0002278	2005.	8799267.	9.9037868	50.0000000	Y
0.0002364	2054.	8686609.	9.8401082	50.0000000	Y
0.0002450	2101.	8573425.	9.7831693	50.0000000	Y
0.0002536	2146.	8460699.	9.7321063	50.0000000	Y
0.0002622	2190.	8349466.	9.6860975	50.0000000	Y
0.0002708	2232.	8240013.	9.6445867	50.0000000	Y
0.0002794	2273.	8132579.	9.6070839	50.0000000	Y
0.0002880	2312.	8027362.	9.5731549	50.0000000	Y
0.0002966	2351.	7924525.	9.5424137	50.0000000	Y
0.0003052	2388.	7824199.	9.5145150	50.0000000	Y
0.0003138	2425.	7726491.	9.4891488	50.0000000	Y
0.0003224	2461.	7631481.	9.4660359	50.0000000	Y
0.0003310	2496.	7539232.	9.4449231	50.0000000	Y
0.0003396	2529.	7446059.	9.4245139	50.0000000	Y
0.0003482	2560.	7350799.	9.4046186	50.0000000	Y
0.0003568	2589.	7254499.	9.3850985	50.0000000	Y
0.0003654	2616.	7158118.	9.3657835	50.0000000	Y
0.0003740	2641.	7061452.	9.3469885	50.0000000	Y
0.0003826	2665.	6965368.	9.3285314	50.0000000	Y
0.0003912	2688.	6870288.	9.3103830	50.0000000	Y
0.0003998	2709.	6775718.	9.2926905	50.0000000	Y

0.0004084	2729.	6682205.	9.2749826	50.0000000	Y
0.0004170	2748.	6589885.	9.2578894	50.0000000	Y
0.0004256	2766.	6499563.	9.2410852	50.0000000	Y
0.0004342	2783.	6409893.	9.2244056	50.0000000	Y
0.0004428	2799.	6322028.	9.2081205	50.0000000	Y
0.0004514	2815.	6236328.	9.1921149	50.0000000	Y
0.0004600	2829.	6151132.	9.1763587	50.0000000	Y
0.0004686	2844.	6068485.	9.1609232	50.0000000	Y
0.0004772	2857.	5986967.	9.1455902	50.0000000	Y
0.0004858	2870.	5907201.	9.1307652	50.0000000	Y
0.0004944	2882.	5829145.	9.1159511	50.0000000	Y
0.0005030	2894.	5752751.	9.1014353	50.0000000	Y
0.0005116	2905.	5677689.	9.0872975	50.0000000	Y
0.0005460	2945.	5393556.	9.0325367	50.0000000	Y
0.0005804	2979.	5133115.	8.9812363	50.0000000	Y
0.0006148	3009.	4893849.	8.9327336	50.0000000	Y
0.0006492	3034.	4674360.	8.8872660	50.0000000	Y
0.0006835	3057.	4471837.	8.8443265	50.0000000	Y
0.0007179	3077.	4285280.	8.8034040	50.0000000	Y
0.0007523	3094.	4112684.	8.7650530	50.0000000	Y
0.0007867	3110.	3953072.	8.7283160	50.0000000	Y
0.0008211	3124.	3804733.	8.6939416	50.0000000	Y
0.0008555	3137.	3666622.	8.6608188	50.0000000	Y
0.0008899	3148.	3537879.	8.6291007	50.0000000	Y
0.0009243	3159.	3417596.	8.5993787	50.0000000	Y
0.0009587	3169.	3305159.	8.5709736	50.0000000	Y

Summary of Results for Nominal Moment Capacity for Section 1

Load No.	Axial Thrust kips	Nominal Moment Capacity in-kips
1	414.0000000000	3169.

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	32287.
2	6.0000	69.3676	No	No	32287.	16536.
3	10.0000	3.0916	No	No	48823.	41184.
4	12.0000	12.0000	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 = 414000.0 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	2807961.	-55674.	0.00	79415.	6.27E+09	0.00	0.00	0.00
0.1200	-0.6995	2727619.	-55650.	6.36E-04	77601.	6.27E+09	14.1242	29.0748	0.00
0.2400	-0.6982	2646931.	-55618.	0.00122	75780.	7.04E+09	29.5593	60.9673	0.00
0.3600	-0.6960	2565983.	-55564.	0.00174	73953.	7.33E+09	46.1516	95.4832	0.00
0.4800	-0.6931	2484829.	-55485.	0.00223	72121.	7.57E+09	63.7472	132.4334	0.00
0.6000	-0.6896	2403526.	-55380.	0.00269	70285.	7.78E+09	82.1923	171.6328	0.00
0.7200	-0.6854	2322128.	-55248.	0.00312	68448.	8.00E+09	101.1857	212.5875	0.00
0.8400	-0.6806	2240691.	-55088.	0.00353	66610.	8.22E+09	120.6328	255.2312	0.00
0.9600	-0.6752	2159269.	-54900.	0.00391	64772.	8.43E+09	140.3842	299.3792	0.00
1.0800	-0.6693	2077919.	-54684.	0.00427	62936.	8.63E+09	160.2774	344.8121	0.00
1.2000	-0.6630	1996695.	-54438.	0.00460	61102.	8.82E+09	180.1498	391.3015	0.00
1.3200	-0.6561	1915649.	-54165.	0.00492	59273.	8.99E+09	200.1423	439.2736	0.00
1.4400	-0.6488	1834836.	-53862.	0.00522	57449.	9.14E+09	219.9608	488.2063	0.00
1.5600	-0.6411	1754306.	-53531.	0.00550	55631.	9.26E+09	239.3834	537.7136	0.00
1.6800	-0.6330	1674110.	-53173.	0.00576	53821.	9.35E+09	258.2563	587.5428	0.00
1.8000	-0.6245	1594296.	-52788.	0.00601	52019.	9.41E+09	276.4258	637.4234	0.00
1.9200	-0.6156	1514910.	-52376.	0.00625	50227.	9.44E+09	295.3680	690.8785	0.00

2.0400	-0.6065	1435998.	-51937.	0.00648	48446.	9.44E+09	314.4188	746.5575	0.00
2.1600	-0.5970	1357608.	-51471.	0.00669	46676.	9.44E+09	332.8007	802.7574	0.00
2.2800	-0.5872	1279785.	-50979.	0.00689	44920.	9.44E+09	350.3908	859.2671	0.00
2.4000	-0.5771	1202571.	-50463.	0.00708	43177.	9.44E+09	367.0659	915.8557	0.00
2.5200	-0.5668	1126009.	-49922.	0.00726	41449.	9.44E+09	383.6207	974.6011	0.00
2.6400	-0.5562	1050141.	-49358.	0.00742	39736.	9.44E+09	399.8458	1035.	0.00
2.7600	-0.5454	975006.	-48773.	0.00758	38040.	9.44E+09	412.7845	1090.	0.00
2.8800	-0.5344	900638.	-48172.	0.00772	36361.	9.44E+09	422.1580	1138.	0.00
3.0000	-0.5232	827064.	-47559.	0.00785	34701.	9.44E+09	429.8572	1183.	0.00
3.1200	-0.5118	754306.	-46933.	0.00797	33058.	9.44E+09	438.4911	1234.	0.00
3.2400	-0.5002	682388.	-46295.	0.00808	31435.	9.44E+09	448.4772	1291.	0.00
3.3600	-0.4885	611338.	-45643.	0.00818	29831.	9.44E+09	457.0301	1347.	0.00
3.4800	-0.4767	541181.	-44980.	0.00827	28248.	9.44E+09	464.0655	1402.	0.00
3.6000	-0.4647	471936.	-44307.	0.00835	26685.	9.44E+09	469.5018	1455.	0.00
3.7200	-0.4526	403623.	-43626.	0.00841	25142.	9.44E+09	476.3319	1515.	0.00
3.8400	-0.4405	336260.	-42933.	0.00847	23622.	9.44E+09	486.7500	1591.	0.00
3.9600	-0.4282	269876.	-42225.	0.00852	22123.	9.44E+09	495.9752	1668.	0.00
4.0800	-0.4159	204496.	-41506.	0.00855	20648.	9.44E+09	503.9271	1745.	0.00
4.2000	-0.4036	140142.	-40775.	0.00858	19195.	9.44E+09	510.5263	1822.	0.00
4.3200	-0.3912	76834.	-40036.	0.00860	17766.	9.44E+09	515.6945	1898.	0.00
4.4400	-0.3788	14588.	-39291.	0.00860	16361.	9.44E+09	519.3543	1974.	0.00
4.5600	-0.3664	-46582.	-38542.	0.00860	17083.	9.44E+09	521.4297	2049.	0.00
4.6800	-0.3541	-106666.	-37791.	0.00859	18439.	9.44E+09	521.8463	2122.	0.00
4.8000	-0.3417	-165659.	-37040.	0.00857	19771.	9.44E+09	520.5308	2194.	0.00
4.9200	-0.3294	-223557.	-36291.	0.00854	21078.	9.44E+09	519.5549	2271.	0.00
5.0400	-0.3171	-280358.	-35538.	0.00850	22360.	9.44E+09	527.0806	2393.	0.00
5.1600	-0.3049	-336040.	-34774.	0.00845	23617.	9.44E+09	533.7940	2521.	0.00
5.2800	-0.2928	-390584.	-34001.	0.00840	24848.	9.44E+09	539.6699	2654.	0.00
5.4000	-0.2807	-443974.	-33220.	0.00833	26053.	9.44E+09	544.6852	2794.	0.00
5.5200	-0.2688	-496195.	-32433.	0.00826	27232.	9.44E+09	548.8189	2940.	0.00
5.6400	-0.2569	-547232.	-31640.	0.00818	28384.	9.44E+09	552.0527	3094.	0.00
5.7600	-0.2452	-597074.	-30844.	0.00810	29509.	9.44E+09	554.3704	3256.	0.00
5.8800	-0.2336	-645713.	-30044.	0.00800	30607.	9.44E+09	555.6089	3425.	0.00
6.0000	-0.2222	-693141.	-28076.	0.00790	31678.	9.44E+09	2179.	14120.	0.00
6.1200	-0.2109	-735988.	-24932.	0.00779	32645.	9.44E+09	2188.	14940.	0.00
6.2400	-0.1997	-774232.	-21778.	0.00767	33508.	9.44E+09	2193.	15811.	0.00
6.3600	-0.1888	-807858.	-18619.	0.00755	34267.	9.44E+09	2194.	16738.	0.00
6.4800	-0.1780	-836860.	-15461.	0.00743	34922.	9.44E+09	2191.	17727.	0.00
6.6000	-0.1674	-861243.	-12312.	0.00730	35472.	9.44E+09	2184.	18785.	0.00
6.7200	-0.1570	-881019.	-9176.	0.00717	35919.	9.44E+09	2171.	19920.	0.00
6.8400	-0.1467	-896213.	-6061.	0.00703	36262.	9.44E+09	2155.	21144.	0.00
6.9600	-0.1367	-906858.	-2974.	0.00689	36502.	9.44E+09	2133.	22467.	0.00
7.0800	-0.1269	-912996.	78.3846	0.00675	36640.	9.44E+09	2106.	23904.	0.00
7.2000	-0.1173	-914684.	3089.	0.00661	36678.	9.44E+09	2075.	25475.	0.00
7.3200	-0.1078	-911987.	6049.	0.00647	36618.	9.44E+09	2037.	27203.	0.00
7.4400	-0.09862	-904982.	8952.	0.00634	36459.	9.44E+09	1994.	29115.	0.00
7.5600	-0.08960	-893759.	11788.	0.00620	36206.	9.44E+09	1944.	31251.	0.00
7.6800	-0.08077	-878424.	14547.	0.00606	35860.	9.44E+09	1888.	33659.	0.00
7.8000	-0.07214	-859094.	17220.	0.00593	35424.	9.44E+09	1824.	36406.	0.00
7.9200	-0.06369	-835903.	19793.	0.00580	34900.	9.44E+09	1751.	39579.	0.00
8.0400	-0.05543	-809007.	22253.	0.00568	34293.	9.44E+09	1667.	43300.	0.00
8.1600	-0.04734	-778582.	24583.	0.00556	33606.	9.44E+09	1569.	47712.	0.00
8.2800	-0.03943	-744833.	26748.	0.00544	32844.	9.44E+09	1439.	52559.	0.00
8.4000	-0.03168	-708032.	28713.	0.00533	32014.	9.44E+09	1290.	58637.	0.00

8.5200	-0.02408	-668492.	30452.	0.00522	31121.	9.44E+09	1125.	67251.	0.00
8.6400	-0.01663	-626559.	31882.	0.00512	30175.	9.44E+09	862.3028	74650.	0.00
8.7600	-0.00932	-582781.	32856.	0.00503	29187.	9.44E+09	490.0465	75686.	0.00
8.8800	-0.00214	-537934.	33291.	0.00495	28174.	9.44E+09	114.0847	76723.	0.00
9.0000	0.00492	-492801.	33182.	0.00487	27155.	9.44E+09	-265.8381	77760.	0.00
9.1200	0.01188	-448175.	32522.	0.00480	26148.	9.44E+09	-650.0090	78797.	0.00
9.2400	0.01874	-404856.	31340.	0.00473	25170.	9.44E+09	-991.9990	76241.	0.00
9.3600	0.02550	-363557.	29792.	0.00467	24238.	9.44E+09	-1157.	65347.	0.00
9.4800	0.03219	-324625.	28023.	0.00462	23359.	9.44E+09	-1300.	58164.	0.00
9.6000	0.03881	-288360.	26059.	0.00457	22541.	9.44E+09	-1428.	52974.	0.00
9.7200	0.04536	-255029.	23921.	0.00453	21788.	9.44E+09	-1541.	48916.	0.00
9.8400	0.05186	-224871.	21641.	0.00450	21108.	9.44E+09	-1626.	45140.	0.00
9.9600	0.05831	-198063.	19248.	0.00446	20502.	9.44E+09	-1698.	41920.	0.00
10.0800	0.06472	-174757.	17590.	0.00443	19976.	9.44E+09	-605.9674	13483.	0.00
10.2000	0.07108	-152692.	16702.	0.00441	19478.	9.44E+09	-627.7286	12717.	0.00
10.3200	0.07742	-131914.	15782.	0.00439	19009.	9.44E+09	-648.7980	12068.	0.00
10.4400	0.08372	-112470.	14833.	0.00437	18570.	9.44E+09	-669.2965	11512.	0.00
10.5600	0.09000	-94404.	13855.	0.00435	18163.	9.44E+09	-689.3187	11029.	0.00
10.6800	0.09626	-77758.	12848.	0.00434	17787.	9.44E+09	-708.9399	10606.	0.00
10.8000	0.1025	-62575.	11814.	0.00433	17444.	9.44E+09	-728.2213	10231.	0.00
10.9200	0.1087	-48897.	10751.	0.00432	17135.	9.44E+09	-747.2126	9896.	0.00
11.0400	0.1149	-36763.	9662.	0.00431	16862.	9.44E+09	-765.9553	9596.	0.00
11.1600	0.1212	-26215.	8546.	0.00431	16623.	9.44E+09	-784.4837	9324.	0.00
11.2800	0.1274	-17291.	7403.	0.00431	16422.	9.44E+09	-802.8268	9077.	0.00
11.4000	0.1336	-10030.	6234.	0.00430	16258.	9.44E+09	-821.0090	8852.	0.00
11.5200	0.1398	-4470.	5038.	0.00430	16133.	9.44E+09	-839.0508	8645.	0.00
11.6400	0.1459	-650.1893	3817.	0.00430	16046.	9.44E+09	-856.9698	8455.	0.00
11.7600	0.1521	1393.	2570.	0.00430	16063.	9.44E+09	-874.7807	8279.	0.00
11.8800	0.1583	1622.	1298.	0.00430	16068.	9.44E+09	-892.4962	8117.	0.00
12.0000	0.1645	0.00	0.00	0.00430	16032.	9.44E+09	-910.1267	3983.	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 = 2807961. inch-lbs
Maximum shear force = -55674. 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 = 17
Number of zero deflection points = 1

Summary of Pile-head Responses for Conventional Analyses

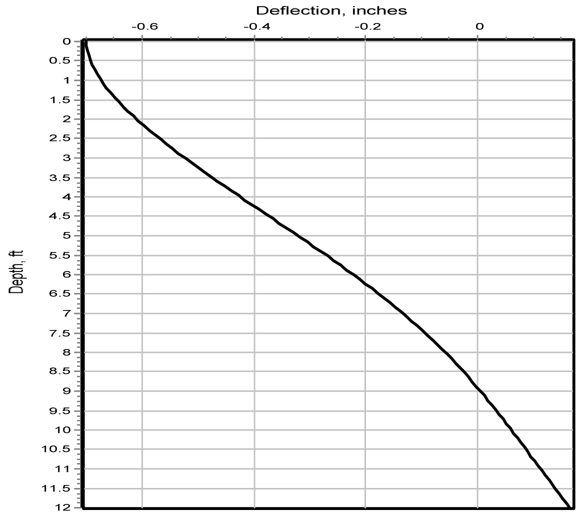
Definitions of Pile-head Loading Conditions:

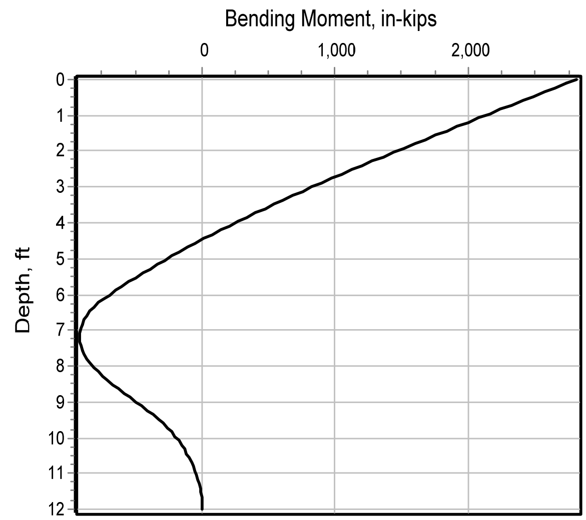
Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians
Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
Load Type 5: Load 1 = Top Deflection, y, inches, and Load 2 = Slope, S, radians

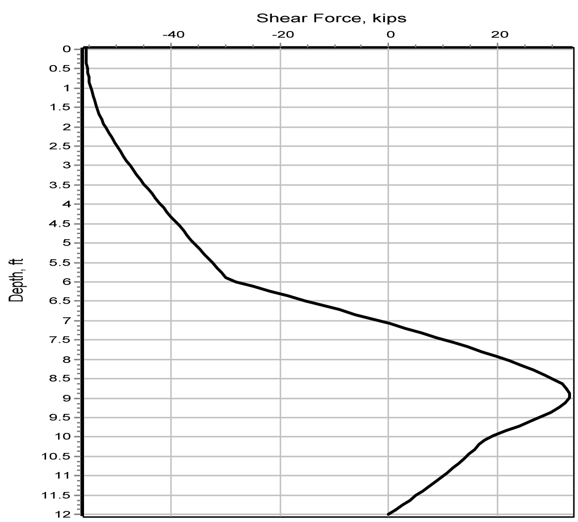
Load Case No.	Load Type 1	Pile-head Load 1	Load Type 2	Pile-head Load 2	Axial Loading lbs	Pile-head Deflection inches	Pile-head Rotation radians	Max Shear in Pile lbs	Max Moment in Pile in-lbs
1	y, in	-0.7000	S, rad	0.00	414000.	-0.7000	0.00	-55674.	2807961.

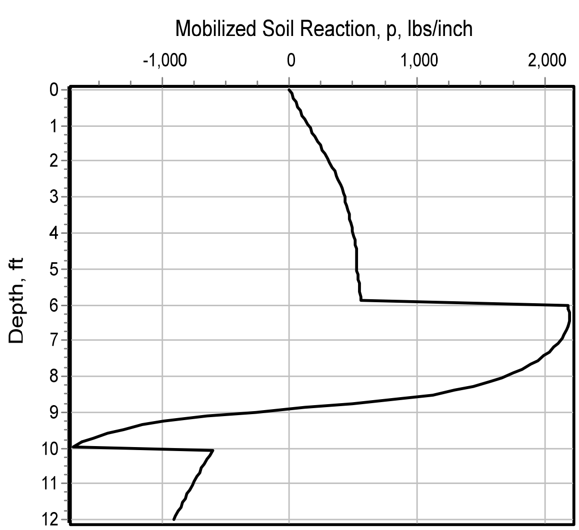
Maximum pile-head deflection = -0.7000000000 inches
Maximum pile-head rotation = 0.0000000000 radians = 0.000000 deg.

The analysis ended normally.


Deflection, inches
-0.6
-0.4
-0.2
0
Depth, ft


Bending Moment, in-kips
0
1,000
2,000
Depth, ft


Shear Force, kips
-40
-20
0
20
Depth, ft


Mobilized Soil Reaction, p, lbs/inch
-1,000
0
1,000
2,000
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 in² cm²	Depth, d in mm	Flange Width, b in mm	Thickness: Flange, t_f in. mm	Thickness: Web, t_w in. mm	I_{xx} in⁴ cm⁴	I_{yy} in⁴ cm⁴	Compact Section Criteria F'_y ksi MPa
HP 14 HP 360	**117** 175	**34.4** 222	**14.21** 361	**14.885** 378	**0.805** 20.4	**0.805** 20.4	**1220** 50800	**443** 18400	**49.4** 341
	102 153	**30** 194	**14.01** 356	**14.785** 376	**0.705** 17.9	**0.705** 17.9	**1050** 43700	**380** 15800	**38.4** 265
	89 133	**26.1** 168	**13.83** 351	**14.695** 373	**0.615** 15.6	**0.615** 15.6	**904** 37600	**326** 13600	**29.6** 204
	73 109	**21.4** 138	**13.61** 346	**14.585** 370	**0.505** 12.8	**0.505** 12.8	**729** 30300	**261** 10900	**20.3** 140
HP 13 HP 330	**100** 150	**29.4** 190	**13.15** 334	**13.205** 335	**0.765** 19.4	**0.765** 19.4	**886** 36878	**294** 12237	**56.7** 391
	87 130	**25.5** 165	**12.95** 329	**13.105** 333	**0.665** 16.9	**0.665** 16.9	**755** 31425	**250** 10406	**43.5** 300
	73 109	**21.6** 139	**12.75** 324	**13.005** 330	**0.565** 14.4	**0.565** 14.4	**630** 26223	**207** 8616	**31.9** 220
	60 90	**17.5** 113	**12.54** 319	**12.9** 328	**0.46** 11.7	**0.46** 11.7	**503** 20936	**165** 6868	**21.5** 148

Section	Weight lbs/ft kg/m	Area, A in² cm²	Depth, d in mm	Flange Width, b in mm	Thickness: Flange, t_f in. mm	Thickness: Web, t_w in. mm	I_{xx} in⁴ cm⁴	I_{yy} in⁴ cm⁴	Compact Section Criteria F'_y ksi MPa
HP 12 HP 310	**84** 126	**24.6** 159	**12.28** 312	**12.295** 312	**0.685** 17.4	**0.685** 17.4	**650** 27100	**213** 8870	52.5 362
	74 111	**21.8** 141	**12.13** 308	**12.215** 310	**0.61** 15.5	**0.61** 15.5	**569** 23700	**186** 7740	**42.1** 290
	63 94	**18.4** 119	**11.94** 303	**12.125** 308	**0.515** 13.1	**0.515** 13.1	**472** 19600	**153** 6370	**30.5** 210
	53 79	**15.5** 100	**11.78** 299	**12.045** 306	**0.435** 11	**0.435** 11	**393** 16400	**127** 5290	**22** 152
HP10 HP 250	**57** 85	**16.8** 108	**9.99** 254	**10.225** 260	**0.565** 14.4	**0.565** 14.4	**294** 12200	**101** 4200	**51.6** 356
	42 63	**12.4** 80	**9.7** 246	**10.075** 256	**0.42** 10.7	**0.42** 10.7	**210** 8740	**71.7** 2980	**29.4** 203
HP 8 HP 200	**36** 54	**10.6** 68.4	**8.02** 204	**8.155** 207	**0.445** 11.3	**0.445** 11.3	**119** 4950	**40.3** 1680	**50.3** 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.

CALCULATIONS

Date:	8/21/2020 Rev. 12/9/2020 (section 8 only)	**Made by:**	KAR
Project No.:	19129538	**Checked by:**	MLM
Subject:	Pile Design at Abutment 2 - Phasing South (HP12x74)	**Reviewed by:**	CCB
Project Title:	MaineDOT I-295 Exit 22 Mallet Drive Bridge Replacement No. 5721		

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 "phasing south" 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

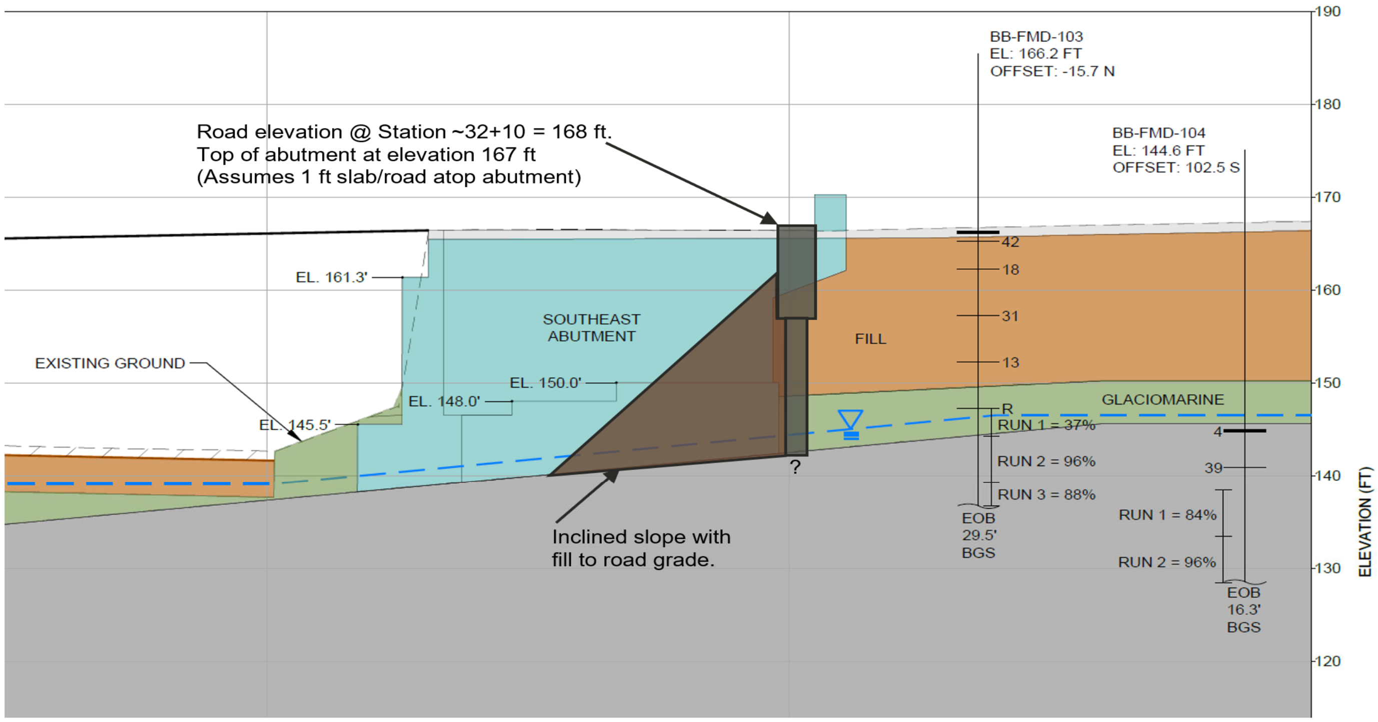
1. AASHTO LRFD Bridge Design Specifications, 9th Ed. 2020.
2. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.
3. Email communication between Golder and HNTB, subject "RE: Freeport I-295 Exit 20 and Exit 22 - Request for Sections and Design Info", dated June 23, 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 Southeast Bridge Embankment - Phasing South" (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. Approach Road Bridge Freeport Interstate 295: Mallet Drive South Cross Sections, dated November 2019.
13. Golder calculation titled "Pile Driveability - Phasing South" (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 40 ft behind the face of the existing southeast abutment. The post-construction ground surface elevation at the new southeast abutment will be 168 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 155 ft.

CALCULATIONS

Date:	8/21/2020 Rev. 12/9/2020 (section 8 only)	**Made by:**	KAR
Project No.:	19129538	**Checked by:**	MLM
Subject:	Pile Design at Abutment 2 - Phasing South (HP12x74)	**Reviewed by:**	CCB
Project Title:	MaineDOT I-295 Exit 22 Mallet Drive Bridge Replacement No. 5721		



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 20 feet behind the new southeastern abutment was estimated to be 1.19 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	3.0	0.188	0.26

Glaciomarine	2	8.0	0.835	0.89
Sand and Gravel	3	1.5	1.373	0.04
(Ref. 10)			Total Settlement (in):	1.19

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 = 3.500 ksf (based on shear strength measurements made in the field and on empirical correlation to the average of the N_{60}-values encountered in BB-FMD-101 for the layer)

D = 12.13 in = 1.0 ft (Ref. 4, Table 5.6.3, HP 12x74)

D_b = 6.0 ft (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 = 3.500 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 = 33 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 (Soil Displacement: Ref. 4, Table 5.6.3, HP 12x74)

K_δ = 1.06 (interpolation between Ref. 1 Figures 10.7.3.8.6f-2 and 10.7.3.8.6f-3, based on V)

C_F = 0.91 (Ref. 1, Figure 10.7.3.8.6f-5)

σ'_v = 1.125 ksf (Ref. 5)

δ/ϕ_f = 0.79 (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 = 0.476 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		Layer Thickness at Abutment (ft)	Surface area of pile side (ft^2)	Load (lbs)	Strength I Load Factor	Service I Load Factor
Existing Fill	1	6.0	24.3	11582	1.10	1.00
Glaciomarine	2	6.0	24.3	85208	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 = 132030 lbs per pile
= 132.0 kips per pile

Total Factored Load, Service I = 96789 lbs per pile
= 96.8 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 132 kips (Strength I) or 97 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 = 582 kips (maximum factored load from Ref. 3 plus downdrag from Step 1)

As part of this analysis loads up to the expected maximum of 582 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 304 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 (Assumption 1)

Length of section:	12	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	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)	Friction Angle (°)	Subgrade Modulus (pci)[3]	Major Principal Strain at 50%	UCS (psi)[2]
Existing Fill	0 - 6 ft	Sand (Reese)	125	-	33	165	-	-
Glaciomarine Silty Clay (above water table)	6 - 10 ft	Stiff Clay with Free Water (Reese)	115	3500	-	500	0.005	-
Glaciomarine Silty Clay (below water table)	10 - 12 ft	Stiff Clay w/o Free Water (Reese)	52.6	3500	-	-	0.005	-
Bedrock	>12 ft	Strong Rock (Vuggy Limestone)	106.6	-	-	-	-	2486

1) Ref. 5
2) Ref. 6. Using lowest UCS value from laboratory test results due to low RQD encountered in boring BB-FMD-103 closest to southeastern abutment.
3) Ref. 7

The full LPile output is provided in Attachment 1.

Obtain the maximum moment at the top of the pile.

$M_{u.Top}$ = 1878 in-kips (LPile)

Obtain the unbraced lengths of the top segment and the second segment of the upper zone of the pile.

$l_{b.top}$ = 3.86 ft (LPile)
$l_{b.top}$ = 46.32 in

$l_{b.2nd}$ = 7.97 ft (LPile)
$l_{b.2nd}$ = 95.69 in

4. Determine if the applied moment on the pile will cause pile head plastic deformation by using the

Determine K values for the top and bottom of the pile and calculate the column slenderness factor (λ) for each

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} = 19.03 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} = 32.76 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)

$P_{e.top}$ = 17228 kips
$P_{e.2nd}$ = 5814 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.

$P_o/P_{e.top}$ = 0.06 ≤ 2.25
$P_o/P_{e.2nd}$ = 0.19 ≤ 2.25

thus use Ref. 1 Eqn 6.9.4.1.1-1:

$$P_n = \left[0.658^{\left(\frac{P_o}{P_e}\right)}\right] P_o$$

$P_{n.top}$ = 1062 kips
$P_{n.2nd}$ = 1008 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.

$$P_{r.top} = \phi_{cu} P_{n.top}$$
$P_{r.top}$ = 743.1 kips

$$P_{r.2nd} = \phi_{cu} P_{n.2nd}$$
$P_{r.2nd}$ = 705.4 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

$$\frac{P_u}{P_{r.2nd}} = \quad 0.62 \quad \text{OK}$$

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:

$$\lambda_f = \frac{b_f}{2t_f} \qquad \text{(Ref. 1, Eqn 6.12.2.2.1-3)}$$

$\lambda_f =$ 10.01

Limiting slenderness ratio for a compact flange:

$$\lambda_{pf} = 0.38\sqrt{\frac{E}{F_y}} \qquad \text{(Ref. 1, Eqn 6.12.2.2.1-4)}$$

$\lambda_{pf} =$ 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:

$$S_y = \frac{I_{yy}}{b/2}$$

$$Z_y = (b^2 t_f)/2 + 0.25 t_w^2 (d - 2t_f)$$

$S_y =$ 30.5 in^3

$Z_y =$ 46.5 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 $\lambda_{pf} < \lambda_f \le \lambda_{rf}$,

M_n = 2262 in-kips

Factored flexural resistance:

ϕ_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' = 1052 in-kips = 1051741 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'.

$M_{u.Top}$ = 1878 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.

$l_{b.top}$ = 2.75 ft (LPile)

$l_{b.top}$ = 33.02 in

$l_{b.2nd}$ = 9.25 ft (LPile)

$l_{b.2nd}$ = 110.98 in

$M_{u.2nd}$ = 855.53 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} = 23.74 < 120 OK

λ_{2nd} = 38.00 < 120 OK

$P_{e.top}$ = 11074 kips

$P_{e.2nd}$ = 4322 kips

$P_o/P_{e.top}$ = 0.10 ≤ 2.25 (to select P_n equation)

$P_o/P_{e.2nd}$ = 0.25 ≤ 2.25 (to select P_n equation)

$P_{n.top}$ = 1046 kips

$P_{n.2nd}$ = 981 kips

$P_{r.top}$ = 732 kips

$P_{r.2nd}$ = 687 kips

$$\frac{P_u}{P_{r.top}} = \quad 0.60 > 0.20 \quad \text{OK}$$

$$\frac{P_u}{P_{r.2nd}} = \quad 0.64 > 0.20 \quad \text{OK}$$

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: 0.97 < 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 = 30.12 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 = 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 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} = 671 kips

The nominal pile driving resistance (R_{ndr}) should exceed neither the nominal structural pile resistance (P_n) nor the maximum driving force (P_0) calculated above.

$P_{n.top}$ = 1046 kips (From Step 4)
$P_{n.2nd}$ = 981 kips (From Step 4)

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 = 0.84 kips (LPile)
ϕ_v = 1.00 (Ref. 1, Article 6.5.4.2)
$V_{factored}$ at pile tip = 0.84 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 (factored load to meet driveability requirements, Ref. 13)

V / P = 0.005

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
0.005 < 0.2

The chosen pile section can be considered pinned.

CONCLUSIONS

The results of the analysis indicate that a maximum moment of 1878 in-kips (157 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 0.08 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 304 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

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\19129538 MaineDOT I-295 Freeport Exit 22 Mallet Dr Bridge\Pile Design\LPile Southeast Abutment\

Name of input data file:
Freeport Exit 22 Southeast Abutment Phasing South R2 _ No Socket _ HP12x74.lp11d

Name of output report file:
Freeport Exit 22 Southeast Abutment Phasing South R2 _ No Socket _ HP12x74.lp11o

Name of plot output file:
Freeport Exit 22 Southeast Abutment Phasing South R2 _ No Socket _ HP12x74.lp11p

Name of runtime message file:
Freeport Exit 22 Southeast Abutment Phasing South R2 _ No Socket _ HP12x74.lp11r

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Date and Time of Analysis

Date: August 21, 2020 Time: 10:47:50

Problem Title

Project Name: MaineDOT I-295 Exit 22 Mallet Drive Bridge No. 5721
Job Number: 19129538
Client: MaineDOT
Engineer: KAR
Description: Southeast 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 = 12.000 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	12.2150
2	12.000	12.2150

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is a H weak axis steel pile
Length of section = 12.000000 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 4 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 = 6.000000 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 = 33.000000 deg.
Friction angle at bottom of layer = 33.000000 deg.
Subgrade k at top of layer = 165.000000 pci
Subgrade k at bottom of layer = 165.000000 pci

Layer 2 is stiff clay with water-induced erosion

Distance from top of pile to top of layer = 6.000000 ft
Distance from top of pile to bottom of layer = 10.000000 ft
Effective unit weight at top of layer = 115.000000 pcf
Effective unit weight at bottom of layer = 115.000000 pcf
Undrained cohesion at top of layer = 3500. psf
Undrained cohesion at bottom of layer = 3500. psf
Epsilon-50 at top of layer = 0.005000
Epsilon-50 at bottom of layer = 0.005000
Subgrade k at top of layer = 500.000000 pci
Subgrade k at bottom of layer = 500.000000 pci

Layer 3 is stiff clay without free water

Distance from top of pile to top of layer = 10.000000 ft
Distance from top of pile to bottom of layer = 12.000000 ft
Effective unit weight at top of layer = 52.600000 pcf
Effective unit weight at bottom of layer = 52.600000 pcf
Undrained cohesion at top of layer = 3500. psf
Undrained cohesion at bottom of layer = 3500. psf
Epsilon-50 at top of layer = 0.005000
Epsilon-50 at bottom of layer = 0.005000

Layer 4 is strong rock (vuggy limestone)

Distance from top of pile to top of layer = 12.000000 ft
Distance from top of pile to bottom of layer = 50.000000 ft
Effective unit weight at top of layer = 106.600000 pcf
Effective unit weight at bottom of layer = 106.600000 pcf
Uniaxial compressive strength at top of layer = 2486. psi
Uniaxial compressive strength at bottom of layer = 2486. psi

(Depth of the lowest soil layer extends 38.000 ft below the pile tip)

Summary of Input Soil Properties

Layer Soil Type Layer Effective Undrained Angle of Uniaxial E50
Layer Name Depth Unit Wt. Cohesion Friction qu or kpy

Num.	(p-y Curve Type)	ft	pcf	psf	deg.	psi	krm	pci
1	Sand	0.00	125.0000	--	33.0000	--	--	165.0000
	(Reese, et al.)	6.0000	125.0000	--	33.0000	--	--	165.0000
2	Stiff Clay	6.0000	115.0000	3500.	--	--	0.00500	500.0000
	with Free Water	10.0000	115.0000	3500.	--	--	0.00500	500.0000
3	Stiff Clay	10.0000	52.6000	3500.	--	--	0.00500	--
	w/o Free Water	12.0000	52.6000	3500.	--	--	0.00500	--
4	Strong Rock	12.0000	106.6000	--	--	2486.	--	--
	(Vuggy Limestone)	50.0000	106.6000	--	--	2486.	--	--

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 = 12.000000 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	28872.
2	6.0000	71.9460	No	No	28872.	13745.
3	10.0000	3.1292	No	No	42617.	36791.
4	12.0000	12.0000	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 = 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	1878122.	-41327.	0.00	82062.	3.29E+09	0.00	0.00	0.00
0.1200	-0.6994	1818374.	-41304.	8.10E-04	80094.	3.29E+09	11.8546	24.4072	0.00
0.2400	-0.6977	1758150.	-41278.	0.00155	78112.	3.69E+09	24.9889	51.5775	0.00
0.3600	-0.6949	1697547.	-41231.	0.00221	76116.	3.85E+09	39.2178	81.2638	0.00
0.4800	-0.6913	1636627.	-41164.	0.00283	74110.	3.98E+09	54.3563	113.2260	0.00
0.6000	-0.6868	1575447.	-41074.	0.00340	72096.	4.12E+09	70.1155	147.0088	0.00
0.7200	-0.6815	1514068.	-40962.	0.00393	70075.	4.27E+09	86.3145	182.3765	0.00
0.8400	-0.6755	1452547.	-40826.	0.00442	68050.	4.42E+09	102.7823	219.1081	0.00
0.9600	-0.6688	1390942.	-40666.	0.00487	66021.	4.57E+09	119.3234	256.9196	0.00
1.0800	-0.6615	1329309.	-40482.	0.00530	63992.	4.73E+09	135.9025	295.8613	0.00
1.2000	-0.6535	1267704.	-40274.	0.00569	61964.	4.88E+09	152.3923	335.7789	0.00
1.3200	-0.6451	1206180.	-40043.	0.00605	59938.	5.01E+09	168.4816	376.0963	0.00
1.4400	-0.6361	1144788.	-39790.	0.00638	57917.	5.13E+09	183.9853	416.4867	0.00
1.5600	-0.6267	1083576.	-39514.	0.00669	55901.	5.24E+09	199.2396	457.7952	0.00
1.6800	-0.6169	1022589.	-39215.	0.00698	53893.	5.31E+09	215.2004	502.3617	0.00

1.8000	-0.6066	961875.	-38894.	0.00724	51894.	5.36E+09	230.5119	547.1933	0.00
1.9200	-0.5960	901477.	-38552.	0.00749	49906.	5.38E+09	245.0259	592.0088	0.00
2.0400	-0.5850	841436.	-38189.	0.00773	47929.	5.38E+09	258.6374	636.6079	0.00
2.1600	-0.5737	781789.	-37807.	0.00794	45965.	5.38E+09	272.4748	683.8637	0.00
2.2800	-0.5622	722576.	-37405.	0.00815	44016.	5.38E+09	285.3221	730.8735	0.00
2.4000	-0.5503	663834.	-36986.	0.00833	42082.	5.38E+09	297.0474	777.3209	0.00
2.5200	-0.5382	605595.	-36551.	0.00850	40164.	5.38E+09	307.5182	822.8526	0.00
2.6400	-0.5258	547893.	-36098.	0.00866	38264.	5.38E+09	320.6069	878.0391	0.00
2.7600	-0.5132	490763.	-35627.	0.00879	36383.	5.38E+09	333.9219	936.9022	0.00
2.8800	-0.5005	434243.	-35137.	0.00892	34522.	5.38E+09	346.4084	996.7150	0.00
3.0000	-0.4875	378369.	-34630.	0.00903	32683.	5.38E+09	357.9767	1057.	0.00
3.1200	-0.4745	323173.	-34104.	0.00912	30865.	5.38E+09	372.6968	1131.	0.00
3.2400	-0.4613	268696.	-33555.	0.00920	29072.	5.38E+09	390.4760	1219.	0.00
3.3600	-0.4480	214983.	-32984.	0.00927	27303.	5.38E+09	402.4225	1294.	0.00
3.4800	-0.4346	162068.	-32398.	0.00932	25561.	5.38E+09	411.4064	1363.	0.00
3.6000	-0.4211	109980.	-31800.	0.00935	23846.	5.38E+09	419.0182	1433.	0.00
3.7200	-0.4077	58742.	-31192.	0.00937	22159.	5.38E+09	425.1589	1502.	0.00
3.8400	-0.3941	8375.	-30576.	0.00938	20501.	5.38E+09	429.7314	1570.	0.00
3.9600	-0.3806	-41101.	-29956.	0.00938	21578.	5.38E+09	432.6403	1637.	0.00
4.0800	-0.3671	-89674.	-29331.	0.00936	23178.	5.38E+09	434.4575	1704.	0.00
4.2000	-0.3537	-137331.	-28699.	0.00933	24747.	5.38E+09	443.9188	1807.	0.00
4.3200	-0.3403	-184044.	-28053.	0.00929	26285.	5.38E+09	452.5578	1915.	0.00
4.4400	-0.3269	-229788.	-27396.	0.00923	27791.	5.38E+09	460.3376	2028.	0.00
4.5600	-0.3137	-274538.	-26728.	0.00917	29264.	5.38E+09	467.2244	2145.	0.00
4.6800	-0.3005	-318274.	-26051.	0.00909	30704.	5.38E+09	473.1877	2267.	0.00
4.8000	-0.2875	-360975.	-25366.	0.00900	32110.	5.38E+09	478.2006	2395.	0.00
4.9200	-0.2746	-402624.	-24675.	0.00889	33481.	5.38E+09	482.2396	2529.	0.00
5.0400	-0.2619	-443205.	-23978.	0.00878	34817.	5.38E+09	485.2848	2668.	0.00
5.1600	-0.2493	-482705.	-23277.	0.00866	36118.	5.38E+09	488.9988	2824.	0.00
5.2800	-0.2370	-521110.	-22569.	0.00852	37382.	5.38E+09	493.1470	2997.	0.00
5.4000	-0.2248	-558405.	-21857.	0.00838	38610.	5.38E+09	496.6256	3181.	0.00
5.5200	-0.2128	-594577.	-21140.	0.00822	39801.	5.38E+09	499.4483	3379.	0.00
5.6400	-0.2011	-629612.	-20419.	0.00806	40955.	5.38E+09	501.6238	3592.	0.00
5.7600	-0.1896	-663502.	-19696.	0.00789	42071.	5.38E+09	502.8965	3819.	0.00
5.8800	-0.1784	-696238.	-18971.	0.00770	43148.	5.38E+09	503.1212	4061.	0.00
6.0000	-0.1674	-727813.	-17297.	0.00751	44188.	5.38E+09	1823.	15674.	0.00
6.1200	-0.1568	-755486.	-14671.	0.00731	45099.	5.38E+09	1824.	16754.	0.00
6.2400	-0.1464	-779251.	-12047.	0.00711	45882.	5.38E+09	1820.	17909.	0.00
6.3600	-0.1363	-799110.	-9432.	0.00690	46535.	5.38E+09	1812.	19147.	0.00
6.4800	-0.1265	-815077.	-6832.	0.00668	47061.	5.38E+09	1799.	20477.	0.00
6.6000	-0.1170	-827177.	-4255.	0.00646	47460.	5.38E+09	1781.	21911.	0.00
6.7200	-0.1079	-835444.	-1707.	0.00624	47732.	5.38E+09	1758.	23463.	0.00
6.8400	-0.09907	-839926.	804.9214	0.00602	47879.	5.38E+09	1730.	25148.	0.00
6.9600	-0.09057	-840680.	3273.	0.00579	47904.	5.38E+09	1697.	26987.	0.00
7.0800	-0.08239	-837772.	5690.	0.00557	47808.	5.38E+09	1660.	29005.	0.00
7.2000	-0.07454	-831282.	8049.	0.00534	47595.	5.38E+09	1617.	31231.	0.00
7.3200	-0.06701	-821300.	10342.	0.00512	47266.	5.38E+09	1568.	33705.	0.00
7.4400	-0.05979	-807928.	12562.	0.00490	46826.	5.38E+09	1515.	36476.	0.00
7.5600	-0.05289	-791279.	14699.	0.00469	46278.	5.38E+09	1455.	39605.	0.00
7.6800	-0.04629	-771481.	16746.	0.00448	45626.	5.38E+09	1388.	43172.	0.00
7.8000	-0.03998	-748676.	18690.	0.00428	44875.	5.38E+09	1312.	47259.	0.00
7.9200	-0.03397	-723024.	20512.	0.00408	44030.	5.38E+09	1218.	51623.	0.00
8.0400	-0.02824	-694725.	22188.	0.00389	43099.	5.38E+09	1110.	56624.	0.00
8.1600	-0.02277	-664007.	23705.	0.00371	42087.	5.38E+09	997.0163	63057.	0.00

8.2800	-0.01756	-631110.	25051.	0.00353	41004.	5.38E+09	872.2317	71539.	0.00
8.4000	-0.01259	-596299.	26136.	0.00337	39858.	5.38E+09	634.4785	72576.	0.00
8.5200	-0.00785	-560071.	26882.	0.00322	38665.	5.38E+09	401.3214	73613.	0.00
8.6400	-0.00333	-522918.	27295.	0.00307	37442.	5.38E+09	172.5329	74650.	0.00
8.7600	9.93E-04	-485318.	27381.	0.00294	36204.	5.38E+09	-52.1734	75686.	0.00
8.8800	0.00513	-447745.	27147.	0.00281	34967.	5.38E+09	-273.1344	76723.	0.00
9.0000	0.00909	-410664.	26597.	0.00270	33746.	5.38E+09	-490.7280	77760.	0.00
9.1200	0.01289	-374531.	25736.	0.00259	32556.	5.38E+09	-705.3636	78797.	0.00
9.2400	0.01655	-339797.	24616.	0.00250	31413.	5.38E+09	-850.0164	73964.	0.00
9.3600	0.02008	-306769.	23330.	0.00241	30325.	5.38E+09	-936.2366	67152.	0.00
9.4800	0.02349	-275631.	21927.	0.00233	29300.	5.38E+09	-1013.	62087.	0.00
9.6000	0.02679	-246546.	20419.	0.00226	28343.	5.38E+09	-1081.	58134.	0.00
9.7200	0.03000	-219663.	18816.	0.00220	27457.	5.38E+09	-1144.	54937.	0.00
9.8400	0.03312	-195115.	17127.	0.00214	26649.	5.38E+09	-1203.	52283.	0.00
9.9600	0.03617	-173029.	15356.	0.00209	25922.	5.38E+09	-1257.	50031.	0.00
10.0800	0.03915	-153518.	14095.	0.00205	25280.	5.38E+09	-495.1066	18211.	0.00
10.2000	0.04207	-135009.	13371.	0.00201	24670.	5.38E+09	-510.8003	17483.	0.00
10.3200	0.04494	-117537.	12624.	0.00198	24095.	5.38E+09	-526.1079	16857.	0.00
10.4400	0.04777	-101135.	11856.	0.00195	23555.	5.38E+09	-541.0980	16312.	0.00
10.5600	0.05055	-85838.	11066.	0.00192	23051.	5.38E+09	-555.8263	15833.	0.00
10.6800	0.05331	-71680.	10255.	0.00190	22585.	5.38E+09	-570.3386	15407.	0.00
10.8000	0.05603	-58692.	9423.	0.00188	22157.	5.38E+09	-584.6730	15026.	0.00
10.9200	0.05873	-46907.	8571.	0.00187	21769.	5.38E+09	-598.8613	14683.	0.00
11.0400	0.06142	-36356.	7699.	0.00186	21422.	5.38E+09	-612.9302	14371.	0.00
11.1600	0.06409	-27069.	6806.	0.00185	21116.	5.38E+09	-626.9022	14086.	0.00
11.2800	0.06675	-19078.	5893.	0.00184	20853.	5.38E+09	-640.7960	13824.	0.00
11.4000	0.06940	-12413.	4961.	0.00184	20634.	5.38E+09	-654.6276	13583.	0.00
11.5200	0.07205	-7102.	4008.	0.00184	20459.	5.38E+09	-668.4098	13359.	0.00
11.6400	0.07469	-3177.	3036.	0.00184	20330.	5.38E+09	-682.1535	13151.	0.00
11.7600	0.07734	-665.6873	2044.	0.00184	20247.	5.38E+09	-695.8671	12957.	0.00
11.8800	0.07998	402.8590	1032.	0.00184	20238.	5.38E+09	-709.5571	12775.	0.00
12.0000	0.08263	0.00	0.00	0.00184	20225.	5.38E+09	-723.2284	6302.	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 = 1878122. inch-lbs
Maximum shear force = -41327. 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 = 20
Number of zero deflection points = 1

Summary of Pile-head Responses for Conventional Analyses

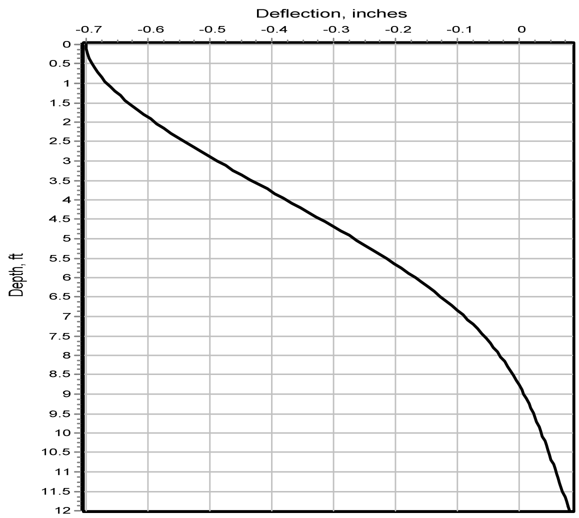
Definitions of Pile-head Loading Conditions:

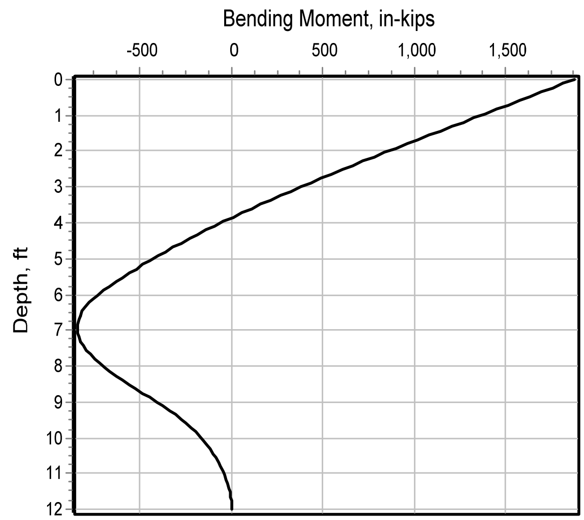
Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians
Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
Load Type 5: Load 1 = Top Deflection, y, inches, and Load 2 = Slope, S, radians

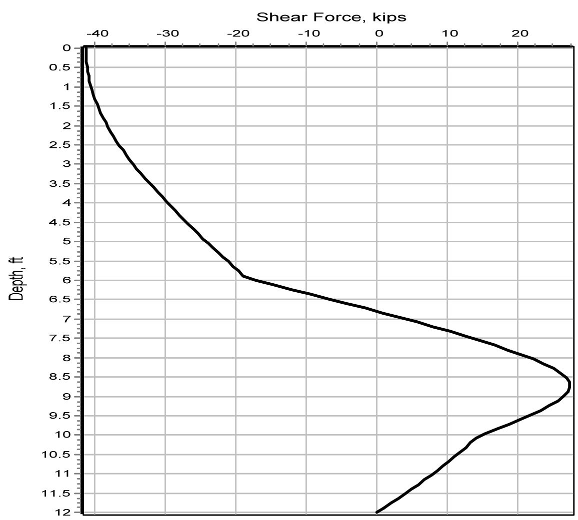
Load Case No.	Load Type 1	Pile-head Load 1	Load Type 2	Pile-head Load 2	Axial Loading lbs	Pile-head Deflection inches	Pile-head Rotation radians	Max Shear in Pile lbs	Max Moment in Pile in-lbs
1	y, in	-0.7000	S, rad	0.00	436000.	-0.7000	0.00	-41327.	1878122.

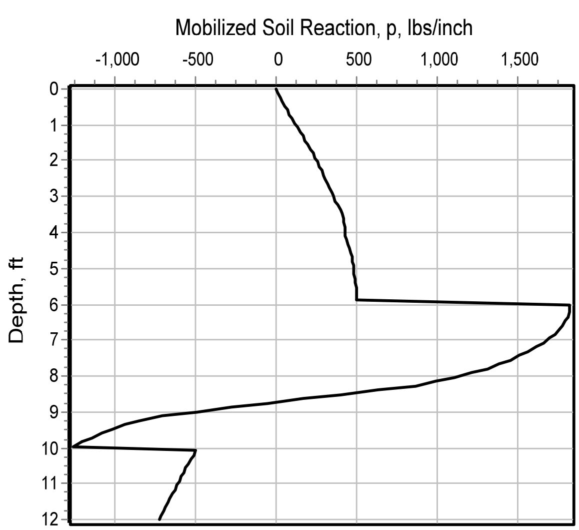
Maximum pile-head deflection = -0.7000000000 inches
Maximum pile-head rotation = 0.0000000000 radians = 0.000000 deg.

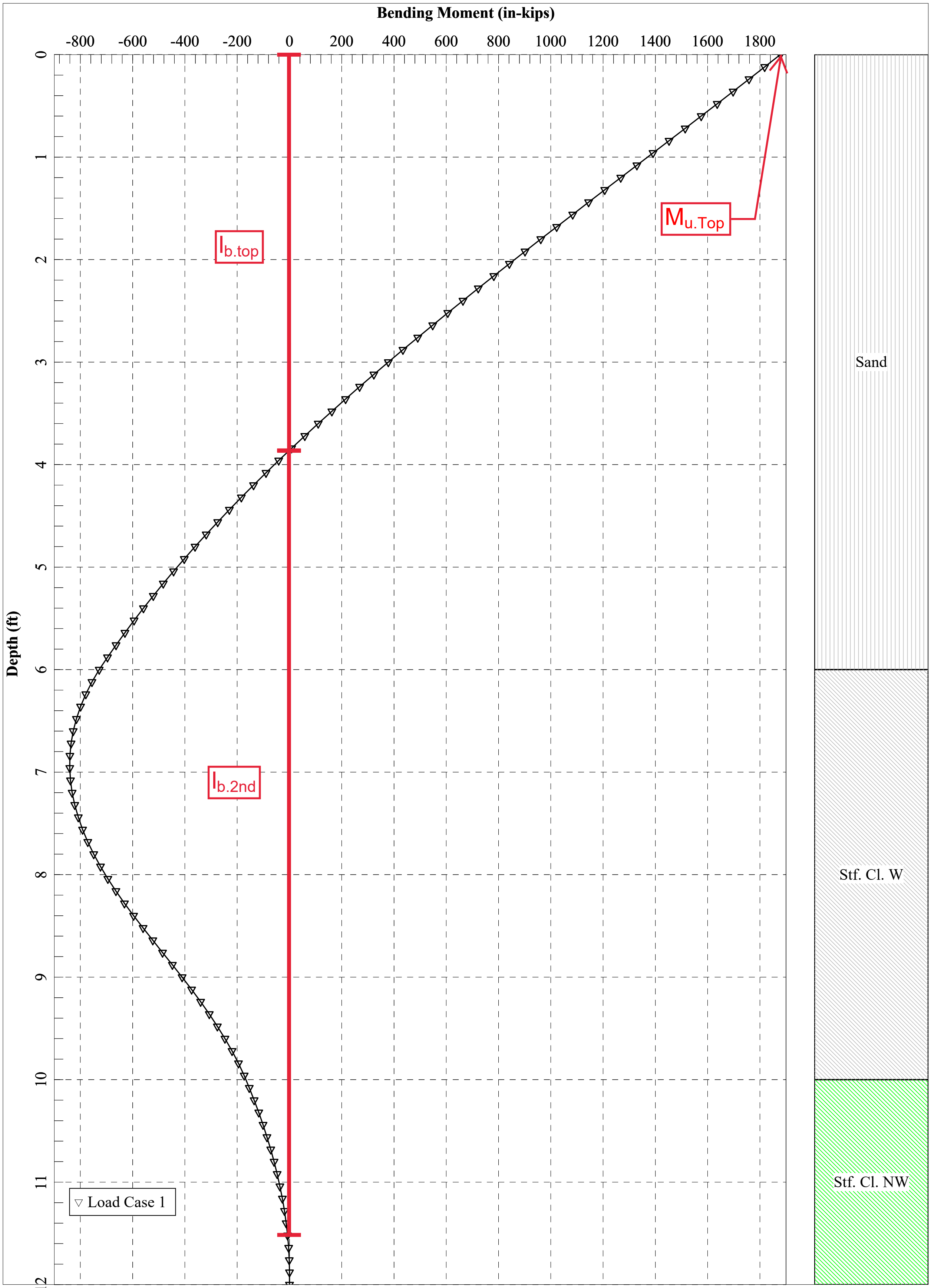
The analysis ended normally.


Deflection, inches
-0.7 -0.6 -0.5 -0.4 -0.3 -0.2 -0.1 0
Depth, ft
0 0.5 1 1.5 2 2.5 3 3.5 4 4.5 5 5.5 6 6.5 7 7.5 8 8.5 9 9.5 10 10.5 11 11.5 12


Bending Moment, in-kips
-500 0 500 1,000 1,500
Depth, ft
0 1 2 3 4 5 6 7 8 9 10 11 12


Shear Force, kips
-40 -30 -20 -10 0 10 20
Depth, ft
0 0.5 1 1.5 2 2.5 3 3.5 4 4.5 5 5.5 6 6.5 7 7.5 8 8.5 9 9.5 10 10.5 11 11.5 12


Mobilized Soil Reaction, p, lbs/inch
-1,000 -500 0 500 1,000 1,500
Depth, ft
0 1 2 3 4 5 6 7 8 9 10 11 12



Bending Moment (in-kips)
-800
-600
-400
-200
0
200
400
600
800
1000
1200
1400
1600
1800
Depth (ft)
0
1
2
3
4
5
6
7
8
9
10
11
12
$M_{u.Top}$
$l_{b.top}$
$l_{b.2nd}$
Load Case 1
Sand
Stf. Cl. W
Stf. Cl. NW

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\19129538 MaineDOT I-295 Freeport Exit 22 Mallet Dr Bridge\Pile Design\LPile Southeast Abutment\

Name of input data file:
Freeport Exit 22 Southeast Abutment Phasing South R2 _ No Socket _ HP12x74 _ Plastic Hinge.lp11d

Name of output report file:
Freeport Exit 22 Southeast Abutment Phasing South R2 _ No Socket _ HP12x74 _ Plastic Hinge.lp11o

Name of plot output file:
Freeport Exit 22 Southeast Abutment Phasing South R2 _ No Socket _ HP12x74 _ Plastic Hinge.lp11p

Name of runtime message file:
Freeport Exit 22 Southeast Abutment Phasing South R2 _ No Socket _ HP12x74 _ Plastic Hinge.lp11r

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Date and Time of Analysis

Date: August 21, 2020 Time: 10:52:37

Problem Title

Project Name: MaineDOT I-295 Exit 22 Mallet Drive Bridge No. 5721
Job Number: 19129538
Client: MaineDOT
Engineer: KAR
Description: Southeast 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 = 12.000 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	12.2150
2	12.000	12.2150

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is a H weak axis steel pile
Length of section = 12.000000 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 4 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 = 6.000000 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 = 33.000000 deg.
Friction angle at bottom of layer = 33.000000 deg.
Subgrade k at top of layer = 165.000000 pci
Subgrade k at bottom of layer = 165.000000 pci

Layer 2 is stiff clay with water-induced erosion

Distance from top of pile to top of layer = 6.000000 ft
Distance from top of pile to bottom of layer = 10.000000 ft
Effective unit weight at top of layer = 115.000000 pcf
Effective unit weight at bottom of layer = 115.000000 pcf
Undrained cohesion at top of layer = 3500. psf
Undrained cohesion at bottom of layer = 3500. psf
Epsilon-50 at top of layer = 0.005000
Epsilon-50 at bottom of layer = 0.005000
Subgrade k at top of layer = 500.000000 pci
Subgrade k at bottom of layer = 500.000000 pci

Layer 3 is stiff clay without free water

Distance from top of pile to top of layer = 10.000000 ft
Distance from top of pile to bottom of layer = 12.000000 ft
Effective unit weight at top of layer = 52.600000 pcf
Effective unit weight at bottom of layer = 52.600000 pcf
Undrained cohesion at top of layer = 3500. psf
Undrained cohesion at bottom of layer = 3500. psf
Epsilon-50 at top of layer = 0.005000
Epsilon-50 at bottom of layer = 0.005000

Layer 4 is strong rock (vuggy limestone)

Distance from top of pile to top of layer = 12.000000 ft
Distance from top of pile to bottom of layer = 50.000000 ft
Effective unit weight at top of layer = 106.600000 pcf
Effective unit weight at bottom of layer = 106.600000 pcf
Uniaxial compressive strength at top of layer = 2486. psi
Uniaxial compressive strength at bottom of layer = 2486. psi

(Depth of the lowest soil layer extends 38.000 ft below the pile tip)

Summary of Input Soil Properties

Layer Soil Type Layer Effective Undrained Angle of Uniaxial E50
Layer Name Depth Unit Wt. Cohesion Friction qu or kpy

Num.	(p-y Curve Type)	ft	pcf	psf	deg.	psi	krm	pci
1	Sand	0.00	125.0000	--	33.0000	--	--	165.0000
	(Reese, et al.)	6.0000	125.0000	--	33.0000	--	--	165.0000
2	Stiff Clay	6.0000	115.0000	3500.	--	--	0.00500	500.0000
	with Free Water	10.0000	115.0000	3500.	--	--	0.00500	500.0000
3	Stiff Clay	10.0000	52.6000	3500.	--	--	0.00500	--
	w/o Free Water	12.0000	52.6000	3500.	--	--	0.00500	--
4	Strong Rock	12.0000	106.6000	--	--	2486.	--	--
	(Vuggy Limestone)	50.0000	106.6000	--	--	2486.	--	--

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 = 1051741. 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 = 12.000000 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	28872.
2	6.0000	71.9460	No	No	28872.	13745.
3	10.0000	3.1292	No	No	42617.	36791.
4	12.0000	12.0000	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 = 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	1878122.	-41327.	0.00	82062.	3.29E+09	0.00	0.00	0.00
0.1200	-0.6994	1818374.	-41304.	8.10E-04	80094.	3.29E+09	11.8546	24.4072	0.00
0.2400	-0.6977	1758150.	-41278.	0.00155	78112.	3.69E+09	24.9889	51.5775	0.00
0.3600	-0.6949	1697547.	-41231.	0.00221	76116.	3.85E+09	39.2178	81.2638	0.00
0.4800	-0.6913	1636627.	-41164.	0.00283	74110.	3.98E+09	54.3563	113.2260	0.00
0.6000	-0.6868	1575447.	-41074.	0.00340	72096.	4.12E+09	70.1155	147.0088	0.00
0.7200	-0.6815	1514068.	-40962.	0.00393	70075.	4.27E+09	86.3145	182.3765	0.00
0.8400	-0.6755	1452547.	-40826.	0.00442	68050.	4.42E+09	102.7823	219.1081	0.00
0.9600	-0.6688	1390942.	-40666.	0.00487	66021.	4.57E+09	119.3234	256.9196	0.00
1.0800	-0.6615	1329309.	-40482.	0.00530	63992.	4.73E+09	135.9025	295.8613	0.00
1.2000	-0.6535	1267704.	-40274.	0.00569	61964.	4.88E+09	152.3923	335.7789	0.00
1.3200	-0.6451	1206180.	-40043.	0.00605	59938.	5.01E+09	168.4816	376.0963	0.00
1.4400	-0.6361	1144788.	-39790.	0.00638	57917.	5.13E+09	183.9853	416.4867	0.00
1.5600	-0.6267	1083576.	-39514.	0.00669	55901.	5.24E+09	199.2396	457.7952	0.00

1.6800	-0.6169	1022589.	-39215.	0.00698	53893.	5.31E+09	215.2004	502.3617	0.00
1.8000	-0.6066	961875.	-38894.	0.00724	51894.	5.36E+09	230.5119	547.1933	0.00
1.9200	-0.5960	901477.	-38552.	0.00749	49906.	5.38E+09	245.0259	592.0088	0.00
2.0400	-0.5850	841436.	-38189.	0.00773	47929.	5.38E+09	258.6374	636.6079	0.00
2.1600	-0.5737	781789.	-37807.	0.00794	45965.	5.38E+09	272.4748	683.8637	0.00
2.2800	-0.5622	722576.	-37405.	0.00815	44016.	5.38E+09	285.3221	730.8735	0.00
2.4000	-0.5503	663834.	-36986.	0.00833	42082.	5.38E+09	297.0474	777.3209	0.00
2.5200	-0.5382	605595.	-36551.	0.00850	40164.	5.38E+09	307.5182	822.8526	0.00
2.6400	-0.5258	547893.	-36098.	0.00866	38264.	5.38E+09	320.6069	878.0391	0.00
2.7600	-0.5132	490763.	-35627.	0.00879	36383.	5.38E+09	333.9219	936.9022	0.00
2.8800	-0.5005	434243.	-35137.	0.00892	34522.	5.38E+09	346.4084	996.7150	0.00
3.0000	-0.4875	378369.	-34630.	0.00903	32683.	5.38E+09	357.9767	1057.	0.00
3.1200	-0.4745	323173.	-34104.	0.00912	30865.	5.38E+09	372.6968	1131.	0.00
3.2400	-0.4613	268696.	-33555.	0.00920	29072.	5.38E+09	390.4760	1219.	0.00
3.3600	-0.4480	214983.	-32984.	0.00927	27303.	5.38E+09	402.4225	1294.	0.00
3.4800	-0.4346	162068.	-32398.	0.00932	25561.	5.38E+09	411.4064	1363.	0.00
3.6000	-0.4211	109980.	-31800.	0.00935	23846.	5.38E+09	419.0182	1433.	0.00
3.7200	-0.4077	58742.	-31192.	0.00937	22159.	5.38E+09	425.1589	1502.	0.00
3.8400	-0.3941	8375.	-30576.	0.00938	20501.	5.38E+09	429.7314	1570.	0.00
3.9600	-0.3806	-41101.	-29956.	0.00938	21578.	5.38E+09	432.6403	1637.	0.00
4.0800	-0.3671	-89674.	-29331.	0.00936	23178.	5.38E+09	434.4575	1704.	0.00
4.2000	-0.3537	-137331.	-28699.	0.00933	24747.	5.38E+09	443.9188	1807.	0.00
4.3200	-0.3403	-184044.	-28053.	0.00929	26285.	5.38E+09	452.5578	1915.	0.00
4.4400	-0.3269	-229788.	-27396.	0.00923	27791.	5.38E+09	460.3376	2028.	0.00
4.5600	-0.3137	-274538.	-26728.	0.00917	29264.	5.38E+09	467.2244	2145.	0.00
4.6800	-0.3005	-318274.	-26051.	0.00909	30704.	5.38E+09	473.1877	2267.	0.00
4.8000	-0.2875	-360975.	-25366.	0.00900	32110.	5.38E+09	478.2006	2395.	0.00
4.9200	-0.2746	-402624.	-24675.	0.00889	33481.	5.38E+09	482.2396	2529.	0.00
5.0400	-0.2619	-443205.	-23978.	0.00878	34817.	5.38E+09	485.2848	2668.	0.00
5.1600	-0.2493	-482705.	-23277.	0.00866	36118.	5.38E+09	488.9988	2824.	0.00
5.2800	-0.2370	-521110.	-22569.	0.00852	37382.	5.38E+09	493.1470	2997.	0.00
5.4000	-0.2248	-558405.	-21857.	0.00838	38610.	5.38E+09	496.6256	3181.	0.00
5.5200	-0.2128	-594577.	-21140.	0.00822	39801.	5.38E+09	499.4483	3379.	0.00
5.6400	-0.2011	-629612.	-20419.	0.00806	40955.	5.38E+09	501.6238	3592.	0.00
5.7600	-0.1896	-663502.	-19696.	0.00789	42071.	5.38E+09	502.8965	3819.	0.00
5.8800	-0.1784	-696238.	-18971.	0.00770	43148.	5.38E+09	503.1212	4061.	0.00
6.0000	-0.1674	-727813.	-17297.	0.00751	44188.	5.38E+09	1823.	15674.	0.00
6.1200	-0.1568	-755486.	-14671.	0.00731	45099.	5.38E+09	1824.	16754.	0.00
6.2400	-0.1464	-779251.	-12047.	0.00711	45882.	5.38E+09	1820.	17909.	0.00
6.3600	-0.1363	-799110.	-9432.	0.00690	46535.	5.38E+09	1812.	19147.	0.00
6.4800	-0.1265	-815077.	-6832.	0.00668	47061.	5.38E+09	1799.	20477.	0.00
6.6000	-0.1170	-827177.	-4255.	0.00646	47460.	5.38E+09	1781.	21911.	0.00
6.7200	-0.1079	-835444.	-1707.	0.00624	47732.	5.38E+09	1758.	23463.	0.00
6.8400	-0.09907	-839926.	804.9214	0.00602	47879.	5.38E+09	1730.	25148.	0.00
6.9600	-0.09057	-840680.	3273.	0.00579	47904.	5.38E+09	1697.	26987.	0.00
7.0800	-0.08239	-837772.	5690.	0.00557	47808.	5.38E+09	1660.	29005.	0.00
7.2000	-0.07454	-831282.	8049.	0.00534	47595.	5.38E+09	1617.	31231.	0.00
7.3200	-0.06701	-821300.	10342.	0.00512	47266.	5.38E+09	1568.	33705.	0.00
7.4400	-0.05979	-807928.	12562.	0.00490	46826.	5.38E+09	1515.	36476.	0.00
7.5600	-0.05289	-791279.	14699.	0.00469	46278.	5.38E+09	1455.	39605.	0.00
7.6800	-0.04629	-771481.	16746.	0.00448	45626.	5.38E+09	1388.	43172.	0.00
7.8000	-0.03998	-748676.	18690.	0.00428	44875.	5.38E+09	1312.	47259.	0.00
7.9200	-0.03397	-723024.	20512.	0.00408	44030.	5.38E+09	1218.	51623.	0.00
8.0400	-0.02824	-694725.	22188.	0.00389	43099.	5.38E+09	1110.	56624.	0.00

8.1600	-0.02277	-664007.	23705.	0.00371	42087.	5.38E+09	997.0163	63057.	0.00
8.2800	-0.01756	-631110.	25051.	0.00353	41004.	5.38E+09	872.2317	71539.	0.00
8.4000	-0.01259	-596299.	26136.	0.00337	39858.	5.38E+09	634.4785	72576.	0.00
8.5200	-0.00785	-560071.	26882.	0.00322	38665.	5.38E+09	401.3214	73613.	0.00
8.6400	-0.00333	-522918.	27295.	0.00307	37442.	5.38E+09	172.5329	74650.	0.00
8.7600	9.93E-04	-485318.	27381.	0.00294	36204.	5.38E+09	-52.1734	75686.	0.00
8.8800	0.00513	-447745.	27147.	0.00281	34967.	5.38E+09	-273.1344	76723.	0.00
9.0000	0.00909	-410664.	26597.	0.00270	33746.	5.38E+09	-490.7280	77760.	0.00
9.1200	0.01289	-374531.	25736.	0.00259	32556.	5.38E+09	-705.3636	78797.	0.00
9.2400	0.01655	-339797.	24616.	0.00250	31413.	5.38E+09	-850.0164	73964.	0.00
9.3600	0.02008	-306769.	23330.	0.00241	30325.	5.38E+09	-936.2366	67152.	0.00
9.4800	0.02349	-275631.	21927.	0.00233	29300.	5.38E+09	-1013.	62087.	0.00
9.6000	0.02679	-246546.	20419.	0.00226	28343.	5.38E+09	-1081.	58134.	0.00
9.7200	0.03000	-219663.	18816.	0.00220	27457.	5.38E+09	-1144.	54937.	0.00
9.8400	0.03312	-195115.	17127.	0.00214	26649.	5.38E+09	-1203.	52283.	0.00
9.9600	0.03617	-173029.	15356.	0.00209	25922.	5.38E+09	-1257.	50031.	0.00
10.0800	0.03915	-153518.	14095.	0.00205	25280.	5.38E+09	-495.1066	18211.	0.00
10.2000	0.04207	-135009.	13371.	0.00201	24670.	5.38E+09	-510.8003	17483.	0.00
10.3200	0.04494	-117537.	12624.	0.00198	24095.	5.38E+09	-526.1079	16857.	0.00
10.4400	0.04777	-101135.	11856.	0.00195	23555.	5.38E+09	-541.0980	16312.	0.00
10.5600	0.05055	-85838.	11066.	0.00192	23051.	5.38E+09	-555.8263	15833.	0.00
10.6800	0.05331	-71680.	10255.	0.00190	22585.	5.38E+09	-570.3386	15407.	0.00
10.8000	0.05603	-58692.	9423.	0.00188	22157.	5.38E+09	-584.6730	15026.	0.00
10.9200	0.05873	-46907.	8571.	0.00187	21769.	5.38E+09	-598.8613	14683.	0.00
11.0400	0.06142	-36356.	7699.	0.00186	21422.	5.38E+09	-612.9302	14371.	0.00
11.1600	0.06409	-27069.	6806.	0.00185	21116.	5.38E+09	-626.9022	14086.	0.00
11.2800	0.06675	-19078.	5893.	0.00184	20853.	5.38E+09	-640.7960	13824.	0.00
11.4000	0.06940	-12413.	4961.	0.00184	20634.	5.38E+09	-654.6276	13583.	0.00
11.5200	0.07205	-7102.	4008.	0.00184	20459.	5.38E+09	-668.4098	13359.	0.00
11.6400	0.07469	-3177.	3036.	0.00184	20330.	5.38E+09	-682.1535	13151.	0.00
11.7600	0.07734	-665.6873	2044.	0.00184	20247.	5.38E+09	-695.8671	12957.	0.00
11.8800	0.07998	402.8590	1032.	0.00184	20238.	5.38E+09	-709.5571	12775.	0.00
12.0000	0.08263	0.00	0.00	0.00184	20225.	5.38E+09	-723.2284	6302.	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 = 1878122. inch-lbs
Maximum shear force = -41327. 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 = 20
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 = 1051741.0 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	1051741.	-30117.	0.00615	54853.	5.28E+09	0.00	0.00	0.00
0.1200	-0.6909	1004424.	-30109.	0.00643	53295.	5.28E+09	11.8546	24.7062	0.00
0.2400	-0.6815	956959.	-30082.	0.00669	51733.	5.37E+09	24.9888	52.8016	0.00
0.3600	-0.6717	909385.	-30036.	0.00694	50166.	5.38E+09	39.2177	84.0789	0.00
0.4800	-0.6615	861739.	-29969.	0.00718	48598.	5.38E+09	54.3563	118.3260	0.00
0.6000	-0.6510	814061.	-29879.	0.00740	47028.	5.38E+09	70.1154	155.0940	0.00
0.7200	-0.6402	766392.	-29766.	0.00761	45458.	5.38E+09	86.3144	194.1516	0.00
0.8400	-0.6291	718773.	-29630.	0.00781	43890.	5.38E+09	102.7821	235.2774	0.00
0.9600	-0.6177	671246.	-29470.	0.00800	42326.	5.38E+09	119.3232	278.1776	0.00
1.0800	-0.6060	623854.	-29287.	0.00817	40765.	5.38E+09	135.9023	322.9177	0.00
1.2000	-0.5941	576639.	-29079.	0.00833	39211.	5.38E+09	152.3921	369.3444	0.00
1.3200	-0.5820	529643.	-28848.	0.00848	37663.	5.38E+09	168.4813	416.8354	0.00
1.4400	-0.5697	482907.	-28594.	0.00862	36125.	5.38E+09	183.9850	465.0319	0.00
1.5600	-0.5572	436472.	-28318.	0.00874	34596.	5.38E+09	199.2392	514.8855	0.00
1.6800	-0.5446	390377.	-28020.	0.00885	33078.	5.38E+09	215.2000	569.0710	0.00
1.8000	-0.5317	344662.	-27699.	0.00895	31573.	5.38E+09	230.5114	624.2563	0.00
1.9200	-0.5188	299367.	-27357.	0.00903	30082.	5.38E+09	245.0254	680.1294	0.00
2.0400	-0.5057	254530.	-26994.	0.00911	28605.	5.38E+09	258.6368	736.4627	0.00
2.1600	-0.4925	210186.	-26612.	0.00917	27145.	5.38E+09	272.4742	796.6035	0.00
2.2800	-0.4793	166372.	-26210.	0.00922	25703.	5.38E+09	285.3215	857.2188	0.00
2.4000	-0.4660	123122.	-25791.	0.00926	24279.	5.38E+09	297.0467	917.9391	0.00
2.5200	-0.4526	80467.	-25357.	0.00929	22874.	5.38E+09	305.5454	972.0688	0.00
2.6400	-0.4392	38432.	-24911.	0.00930	21490.	5.38E+09	313.3564	1027.	0.00
2.7600	-0.4258	-2959.	-24455.	0.00931	20323.	5.38E+09	320.7184	1085.	0.00
2.8800	-0.4124	-43685.	-23989.	0.00930	21663.	5.38E+09	326.5822	1140.	0.00
3.0000	-0.3990	-83727.	-23515.	0.00928	22982.	5.38E+09	330.8563	1194.	0.00
3.1200	-0.3857	-123068.	-23034.	0.00926	24277.	5.38E+09	337.2187	1259.	0.00
3.2400	-0.3724	-161690.	-22543.	0.00922	25549.	5.38E+09	345.3655	1336.	0.00
3.3600	-0.3591	-199568.	-22040.	0.00917	26796.	5.38E+09	352.2949	1413.	0.00
3.4800	-0.3460	-236682.	-21529.	0.00911	28018.	5.38E+09	357.9192	1490.	0.00
3.6000	-0.3329	-273014.	-21011.	0.00904	29214.	5.38E+09	362.1523	1567.	0.00
3.7200	-0.3199	-308549.	-20487.	0.00897	30384.	5.38E+09	364.9099	1642.	0.00
3.8400	-0.3071	-343276.	-19961.	0.00888	31527.	5.38E+09	366.1097	1717.	0.00
3.9600	-0.2944	-377186.	-19434.	0.00878	32644.	5.38E+09	365.6716	1789.	0.00
4.0800	-0.2818	-410274.	-18908.	0.00868	33733.	5.38E+09	364.1987	1861.	0.00
4.2000	-0.2694	-442538.	-18379.	0.00856	34795.	5.38E+09	370.6448	1981.	0.00
4.3200	-0.2571	-473959.	-17842.	0.00844	35830.	5.38E+09	376.3586	2108.	0.00
4.4400	-0.2451	-504520.	-17296.	0.00831	36836.	5.38E+09	381.3184	2241.	0.00
4.5600	-0.2332	-534205.	-16744.	0.00817	37814.	5.38E+09	385.5051	2381.	0.00

4.6800	-0.2215	-563002.	-16186.	0.00802	38762.	5.38E+09	388.9031	2528.	0.00
4.8000	-0.2101	-590897.	-15624.	0.00787	39680.	5.38E+09	391.4996	2684.	0.00
4.9200	-0.1989	-617881.	-15059.	0.00771	40569.	5.38E+09	393.2594	2848.	0.00
5.0400	-0.1879	-643946.	-14493.	0.00754	41427.	5.38E+09	393.9496	3019.	0.00
5.1600	-0.1771	-669086.	-13924.	0.00736	42254.	5.38E+09	395.3310	3214.	0.00
5.2800	-0.1667	-693293.	-13354.	0.00718	43052.	5.38E+09	397.0901	3431.	0.00
5.4000	-0.1565	-716561.	-12781.	0.00699	43818.	5.38E+09	397.9730	3663.	0.00
5.5200	-0.1465	-738883.	-12208.	0.00680	44553.	5.38E+09	397.9561	3911.	0.00
5.6400	-0.1369	-760255.	-11636.	0.00660	45256.	5.38E+09	397.0179	4176.	0.00
5.7600	-0.1275	-780677.	-11066.	0.00639	45929.	5.38E+09	395.1397	4461.	0.00
5.8800	-0.1185	-800148.	-10499.	0.00618	46570.	5.38E+09	392.3049	4768.	0.00
6.0000	-0.1097	-818671.	-8947.	0.00596	47180.	5.38E+09	1763.	23134.	0.00
6.1200	-0.1013	-833401.	-6426.	0.00574	47664.	5.38E+09	1738.	24699.	0.00
6.2400	-0.09321	-844386.	-3945.	0.00552	48026.	5.38E+09	1708.	26390.	0.00
6.3600	-0.08543	-851688.	-1509.	0.00529	48267.	5.38E+09	1674.	28223.	0.00
6.4800	-0.07798	-855375.	874.3858	0.00506	48388.	5.38E+09	1636.	30217.	0.00
6.6000	-0.07086	-855525.	3200.	0.00483	48393.	5.38E+09	1594.	32394.	0.00
6.7200	-0.06407	-852225.	5462.	0.00460	48284.	5.38E+09	1547.	34781.	0.00
6.8400	-0.05760	-845574.	7654.	0.00438	48065.	5.38E+09	1496.	37411.	0.00
6.9600	-0.05146	-835677.	9769.	0.00415	47739.	5.38E+09	1441.	40322.	0.00
7.0800	-0.04565	-822652.	11800.	0.00393	47311.	5.38E+09	1381.	43554.	0.00
7.2000	-0.04015	-806626.	13741.	0.00371	46783.	5.38E+09	1314.	47144.	0.00
7.3200	-0.03496	-787738.	15577.	0.00350	46161.	5.38E+09	1235.	50888.	0.00
7.4400	-0.03008	-766157.	17291.	0.00329	45451.	5.38E+09	1146.	54865.	0.00
7.5600	-0.02549	-742071.	18876.	0.00309	44657.	5.38E+09	1055.	59600.	0.00
7.6800	-0.02118	-715672.	20328.	0.00289	43788.	5.38E+09	961.6827	65373.	0.00
7.8000	-0.01716	-687159.	21598.	0.00270	42850.	5.38E+09	802.9072	67392.	0.00
7.9200	-0.01339	-656866.	22634.	0.00252	41852.	5.38E+09	636.4755	68429.	0.00
8.0400	-0.00988	-625142.	23436.	0.00235	40808.	5.38E+09	476.8405	69466.	0.00
8.1600	-0.00662	-592325.	24013.	0.00219	39727.	5.38E+09	323.9504	70502.	0.00
8.2800	-0.00358	-558737.	24374.	0.00204	38621.	5.38E+09	177.6974	71539.	0.00
8.4000	-7.52E-04	-524686.	24529.	0.00189	37500.	5.38E+09	37.9220	72576.	0.00
8.5200	0.00187	-490468.	24487.	0.00176	36374.	5.38E+09	-95.5816	73613.	0.00
8.6400	0.00430	-456366.	24258.	0.00163	35251.	5.38E+09	-223.0602	74650.	0.00
8.7600	0.00656	-422650.	23849.	0.00151	34141.	5.38E+09	-344.7965	75686.	0.00
8.8800	0.00865	-389577.	23269.	0.00140	33052.	5.38E+09	-461.1028	76723.	0.00
9.0000	0.01060	-357396.	22525.	0.00130	31992.	5.38E+09	-572.3157	77760.	0.00
9.1200	0.01240	-326341.	21624.	0.00121	30970.	5.38E+09	-678.7896	78797.	0.00
9.2400	0.01409	-296639.	20573.	0.00113	29992.	5.38E+09	-780.8906	79834.	0.00
9.3600	0.01565	-268506.	19416.	0.00105	29066.	5.38E+09	-826.6710	76057.	0.00
9.4800	0.01711	-242042.	18198.	9.83E-04	28194.	5.38E+09	-864.4373	72734.	0.00
9.6000	0.01848	-217330.	16929.	9.22E-04	27381.	5.38E+09	-898.3572	69988.	0.00
9.7200	0.01977	-194445.	15613.	8.67E-04	26627.	5.38E+09	-929.0761	67674.	0.00
9.8400	0.02098	-173453.	14255.	8.18E-04	25936.	5.38E+09	-957.1035	65693.	0.00
9.9600	0.02212	-154417.	12858.	7.74E-04	25309.	5.38E+09	-982.8494	63972.	0.00
10.0800	0.02321	-137392.	11838.	7.35E-04	24749.	5.38E+09	-434.4599	26957.	0.00
10.2000	0.02424	-121246.	11205.	7.00E-04	24217.	5.38E+09	-445.0490	26439.	0.00
10.3200	0.02522	-106002.	10556.	6.70E-04	23715.	5.38E+09	-455.3971	25998.	0.00
10.4400	0.02617	-91685.	9893.	6.43E-04	23244.	5.38E+09	-465.5451	25619.	0.00
10.5600	0.02708	-78317.	9216.	6.20E-04	22804.	5.38E+09	-475.5285	25290.	0.00
10.6800	0.02795	-65923.	8524.	6.01E-04	22396.	5.38E+09	-485.3779	25003.	0.00
10.8000	0.02881	-54524.	7818.	5.85E-04	22020.	5.38E+09	-495.1201	24750.	0.00
10.9200	0.02964	-44142.	7098.	5.72E-04	21678.	5.38E+09	-504.7785	24525.	0.00
11.0400	0.03045	-34800.	6364.	5.61E-04	21371.	5.38E+09	-514.3734	24322.	0.00

11.1600	0.03125	-26518.	5617.	5.53E-04	21098.	5.38E+09	-523.9227	24139.	0.00
11.2800	0.03205	-19318.	4855.	5.47E-04	20861.	5.38E+09	-533.4415	23970.	0.00
11.4000	0.03283	-13222.	4080.	5.42E-04	20660.	5.38E+09	-542.9428	23815.	0.00
11.5200	0.03361	-8249.	3292.	5.40E-04	20497.	5.38E+09	-552.4372	23670.	0.00
11.6400	0.03438	-4420.	2489.	5.38E-04	20371.	5.38E+09	-561.9335	23534.	0.00
11.7600	0.03516	-1755.	1673.	5.37E-04	20283.	5.38E+09	-571.4382	23405.	0.00
11.8800	0.03593	-275.2278	843.4405	5.37E-04	20234.	5.38E+09	-580.9559	23283.	0.00
12.0000	0.03670	0.00	0.00	5.37E-04	20225.	5.38E+09	-590.4892	11583.	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 = 0.00614540 radians
Maximum bending moment = 1051741. inch-lbs
Maximum shear force = -30117. 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 = 19
Number of zero deflection points = 1

Summary of Pile-head Responses for Conventional Analyses

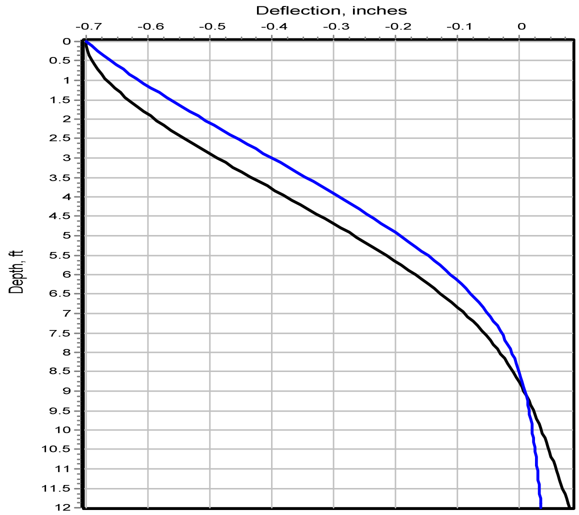
Definitions of Pile-head Loading Conditions:

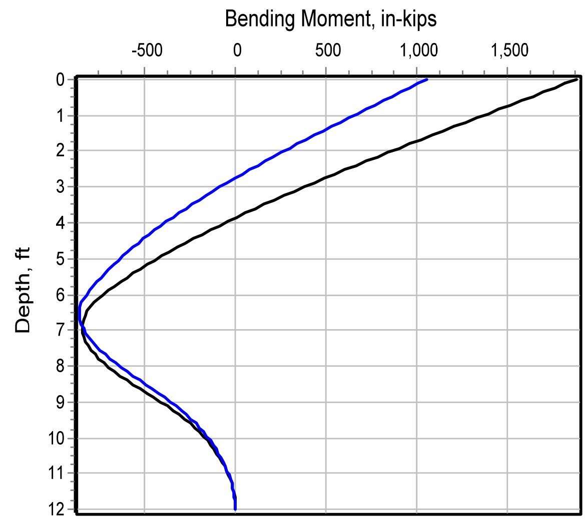
Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians
Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
Load Type 5: Load 1 = Top Deflection, y, inches, and Load 2 = Slope, S, radians

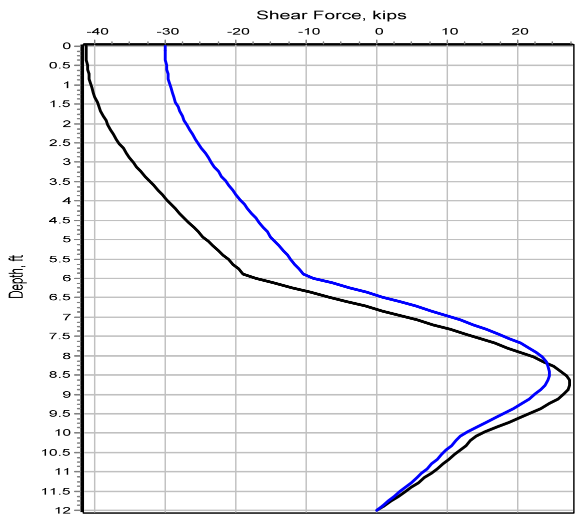
Load Case No.	Load Type 1	Pile-head Load 1	Load Type 2	Pile-head Load 2	Axial Loading lbs	Pile-head Deflection inches	Pile-head Rotation radians	Max Shear in Pile lbs	Max Moment in Pile in-lbs
1	y, in	-0.7000	S, rad	0.00	436000.	-0.7000	0.00	-41327.	1878122.
2	y, in	-0.7000	M, in-lb	1051741.	436000.	-0.7000	0.00615	-30117.	1051741.

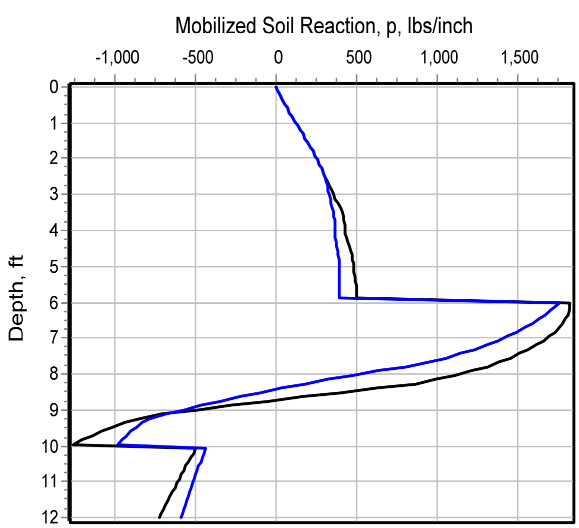
Maximum pile-head deflection = -0.7000000000 inches
Maximum pile-head rotation = 0.0061454038 radians = 0.352106 deg.

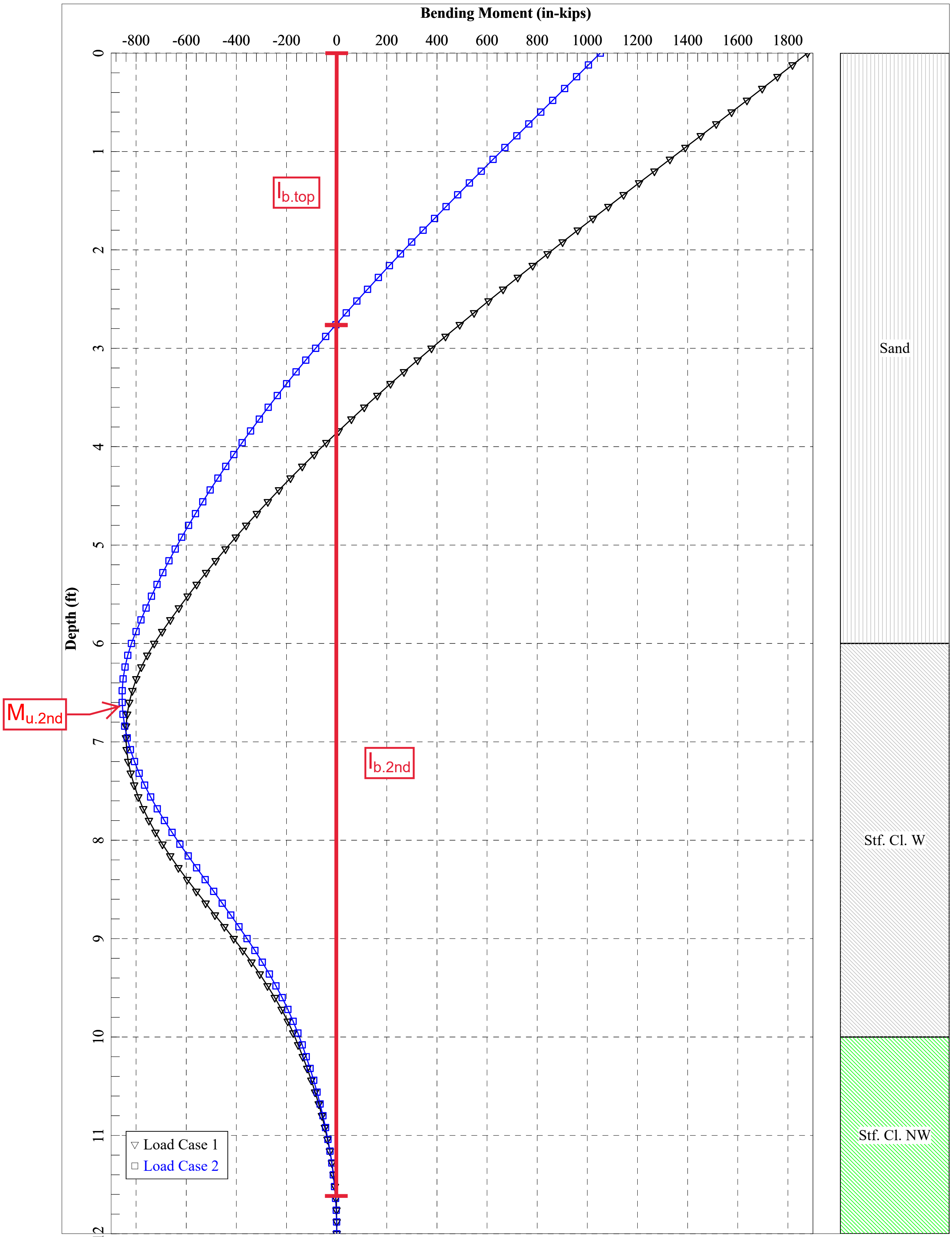
The analysis ended normally.


Deflection, inches
-0.7 -0.6 -0.5 -0.4 -0.3 -0.2 -0.1 0
Depth, ft
0 0.5 1 1.5 2 2.5 3 3.5 4 4.5 5 5.5 6 6.5 7 7.5 8 8.5 9 9.5 10 10.5 11 11.5 12


Bending Moment, in-kips
-500 0 500 1,000 1,500
Depth, ft
0 1 2 3 4 5 6 7 8 9 10 11 12


Shear Force, kips
-40 -30 -20 -10 0 10 20
Depth, ft
0 0.5 1 1.5 2 2.5 3 3.5 4 4.5 5 5.5 6 6.5 7 7.5 8 8.5 9 9.5 10 10.5 11 11.5 12


Mobilized Soil Reaction, p, lbs/inch
-1,000 -500 0 500 1,000 1,500
Depth, ft
0 1 2 3 4 5 6 7 8 9 10 11 12



Bending Moment (in-kips)
-800
-600
-400
-200
0
200
400
600
800
1000
1200
1400
1600
1800
Depth (ft)
0
1
2
3
4
5
6
7
8
9
10
11
12
$I_{b.top}$
$M_{u.2nd}$
$I_{b.2nd}$
Load Case 1
Load Case 2
Sand
Stf. Cl. W
Stf. Cl. NW

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\19129538 MaineDOT I-295 Freeport Exit 22 Mallet Dr Bridge\Pile Design\LPile Southeast Abutment\

Name of input data file:
Freeport Exit 22 Southeast Abutment Phasing South R2 _ No Socket _ HP12x74 _ Service1.lp11d

Name of output report file:
Freeport Exit 22 Southeast Abutment Phasing South R2 _ No Socket _ HP12x74 _ Service1.lp11o

Name of plot output file:
Freeport Exit 22 Southeast Abutment Phasing South R2 _ No Socket _ HP12x74 _ Service1.lp11p

Name of runtime message file:
Freeport Exit 22 Southeast Abutment Phasing South R2 _ No Socket _ HP12x74 _ Service1.lp11r

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Date and Time of Analysis

Date: August 21, 2020 Time: 11:02:31

Problem Title

Project Name: MaineDOT I-295 Exit 22 Mallet Drive Bridge No. 5721
Job Number: 19129538
Client: MaineDOT
Engineer: KAR
Description: Southeast 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 = 12.000 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	12.2150
2	12.000	12.2150

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is a H weak axis steel pile
Length of section = 12.000000 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 4 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 = 6.000000 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 = 33.000000 deg.
Friction angle at bottom of layer = 33.000000 deg.
Subgrade k at top of layer = 165.000000 pci
Subgrade k at bottom of layer = 165.000000 pci

Layer 2 is stiff clay with water-induced erosion

Distance from top of pile to top of layer = 6.000000 ft
Distance from top of pile to bottom of layer = 10.000000 ft
Effective unit weight at top of layer = 115.000000 pcf
Effective unit weight at bottom of layer = 115.000000 pcf
Undrained cohesion at top of layer = 3500. psf
Undrained cohesion at bottom of layer = 3500. psf
Epsilon-50 at top of layer = 0.005000
Epsilon-50 at bottom of layer = 0.005000
Subgrade k at top of layer = 500.000000 pci
Subgrade k at bottom of layer = 500.000000 pci

Layer 3 is stiff clay without free water

Distance from top of pile to top of layer = 10.000000 ft
Distance from top of pile to bottom of layer = 12.000000 ft
Effective unit weight at top of layer = 52.600000 pcf
Effective unit weight at bottom of layer = 52.600000 pcf
Undrained cohesion at top of layer = 3500. psf
Undrained cohesion at bottom of layer = 3500. psf
Epsilon-50 at top of layer = 0.005000
Epsilon-50 at bottom of layer = 0.005000

Layer 4 is strong rock (vuggy limestone)

Distance from top of pile to top of layer = 12.000000 ft
Distance from top of pile to bottom of layer = 50.000000 ft
Effective unit weight at top of layer = 106.600000 pcf
Effective unit weight at bottom of layer = 106.600000 pcf
Uniaxial compressive strength at top of layer = 2486. psi
Uniaxial compressive strength at bottom of layer = 2486. psi

(Depth of the lowest soil layer extends 38.000 ft below the pile tip)

Summary of Input Soil Properties

Layer Soil Type Layer Effective Undrained Angle of Uniaxial E50
Layer Name Depth Unit Wt. Cohesion Friction qu or kpy

Num.	(p-y Curve Type)	ft	pcf	psf	deg.	psi	krm	pci
1	Sand	0.00	125.0000	--	33.0000	--	--	165.0000
	(Reese, et al.)	6.0000	125.0000	--	33.0000	--	--	165.0000
2	Stiff Clay	6.0000	115.0000	3500.	--	--	0.00500	500.0000
	with Free Water	10.0000	115.0000	3500.	--	--	0.00500	500.0000
3	Stiff Clay	10.0000	52.6000	3500.	--	--	0.00500	--
	w/o Free Water	12.0000	52.6000	3500.	--	--	0.00500	--
4	Strong Rock	12.0000	106.6000	--	--	2486.	--	--
	(Vuggy Limestone)	50.0000	106.6000	--	--	2486.	--	--

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	397000.	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 = 12.000000 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	397.000

Definition of Run Messages:

Y = part of pipe section has yielded.

Axial Thrust Force = 397.000 kips

Bending Curvature rad/in.	Bending Moment in-kip	Bending Stiffness kip-in2	Depth to N Axis in	Max Total Stress ksi	Run Msg
0.00000473	25.4558285	5379425.	140.3050882	19.2457016	
0.00000946	50.9116570	5379425.	73.2062941	20.0754534	
0.00001420	76.3674854	5379425.	50.8400294	20.9052047	
0.00001893	101.8233139	5379425.	39.6568971	21.7349566	
0.00002366	127.2791424	5379425.	32.9470176	22.5647080	
0.00002839	152.7349709	5379425.	28.4737647	23.3944595	
0.00003312	178.1907994	5379425.	25.2785840	24.2242110	
0.00003786	203.6466279	5379425.	22.8821985	25.0539625	
0.00004259	229.1024563	5379425.	21.0183431	25.8837143	
0.00004732	254.5582848	5379425.	19.5272588	26.7134658	
0.00005205	280.0141133	5379425.	18.3072807	27.5432175	
0.00005678	305.4699418	5379425.	17.2906324	28.3729690	

0.00006152	330.9257703	5379425.	16.4303914	29.2027206	
0.00006625	356.3815988	5379425.	15.6930420	30.0324722	
0.00007098	381.8374272	5379425.	15.0540059	30.8622238	
0.00007571	407.2932557	5379425.	14.4948493	31.6919753	
0.00008045	432.7490842	5379425.	14.0014758	32.5217269	
0.00008518	458.2049127	5379425.	13.5629216	33.3514786	
0.00008991	483.6607412	5379425.	13.1705310	34.1812301	
0.00009464	509.1165697	5379425.	12.8173794	35.0109817	
0.00009937	534.5723981	5379425.	12.4978613	35.8407333	
0.0001041	560.0282266	5379425.	12.2073904	36.6704849	
0.0001088	585.4840551	5379425.	11.9421777	37.5002365	
0.0001136	610.9398836	5379425.	11.6990662	38.3299881	
0.0001183	636.3957121	5379425.	11.4754035	39.1597397	
0.0001230	661.8515406	5379425.	11.2689457	39.9894912	
0.0001278	687.3073690	5379425.	11.0777810	40.8192428	
0.0001325	712.7631975	5379425.	10.9002710	41.6489944	
0.0001372	738.2190260	5379425.	10.7350030	42.4787460	
0.0001420	763.6748545	5379425.	10.5807529	43.3084976	
0.0001467	789.1306830	5379425.	10.4364545	44.1382492	
0.0001514	814.5865114	5379425.	10.3011746	44.9680008	
0.0001562	840.0423399	5379425.	10.1740936	45.7977524	
0.0001609	865.4981684	5379425.	10.0544879	46.6275040	
0.0001656	890.9539969	5379425.	9.9417168	47.4572556	
0.0001704	916.4098254	5379425.	9.8352108	48.2870072	
0.0001751	941.8656539	5379425.	9.7344618	49.1167587	
0.0001798	967.3214823	5379425.	9.6390155	49.9465103	
0.0001846	991.9584503	5374987.	9.5496410	50.0000000	Y
0.0001940	1039.	5353075.	9.3877125	50.0000000	Y
0.0002035	1082.	5317939.	9.2451910	50.0000000	Y
0.0002129	1123.	5273000.	9.1191944	50.0000000	Y
0.0002224	1161.	5221026.	9.0073145	50.0000000	Y
0.0002319	1197.	5164260.	8.9075189	50.0000000	Y
0.0002413	1232.	5104523.	8.8180762	50.0000000	Y
0.0002508	1265.	5042658.	8.7377322	50.0000000	Y
0.0002603	1296.	4979791.	8.6652577	50.0000000	Y
0.0002697	1326.	4916421.	8.5997464	50.0000000	Y
0.0002792	1355.	4853450.	8.5402189	50.0000000	Y
0.0002887	1383.	4790933.	8.4860989	50.0000000	Y
0.0002981	1410.	4729071.	8.4368202	50.0000000	Y
0.0003076	1436.	4668335.	8.3917490	50.0000000	Y
0.0003170	1461.	4608841.	8.3504460	50.0000000	Y
0.0003265	1486.	4550682.	8.3125212	50.0000000	Y
0.0003360	1510.	4493936.	8.2776261	50.0000000	Y
0.0003454	1533.	4438663.	8.2454486	50.0000000	Y
0.0003549	1556.	4384887.	8.2157199	50.0000000	Y
0.0003644	1579.	4332317.	8.1883634	50.0000000	Y
0.0003738	1600.	4281278.	8.1630010	50.0000000	Y
0.0003833	1622.	4231795.	8.1394215	50.0000000	Y
0.0003928	1643.	4183883.	8.1174320	50.0000000	Y
0.0004022	1664.	4137000.	8.0971844	50.0000000	Y
0.0004117	1684.	4091645.	8.0782422	50.0000000	Y
0.0004212	1705.	4047802.	8.0604652	50.0000000	Y
0.0004306	1724.	4002644.	8.0431365	50.0000000	Y
0.0004401	1742.	3957341.	8.0259279	50.0000000	Y

0.0004495	1758.	3911644.	8.0091748	50.0000000	Y
0.0004590	1775.	3866117.	7.9926575	50.0000000	Y
0.0004685	1790.	3820684.	7.9765762	50.0000000	Y
0.0004779	1804.	3775284.	7.9605253	50.0000000	Y
0.0004874	1818.	3730025.	7.9448817	50.0000000	Y
0.0004969	1831.	3685627.	7.9294002	50.0000000	Y
0.0005063	1844.	3641228.	7.9143993	50.0000000	Y
0.0005158	1856.	3597434.	7.8993844	50.0000000	Y
0.0005253	1867.	3554453.	7.8848342	50.0000000	Y
0.0005347	1878.	3511627.	7.8703899	50.0000000	Y
0.0005442	1888.	3469770.	7.8561392	50.0000000	Y
0.0005537	1898.	3428415.	7.8422366	50.0000000	Y
0.0005631	1908.	3387597.	7.8285114	50.0000000	Y
0.0006010	1942.	3231500.	7.7756558	50.0000000	Y
0.0006388	1972.	3086198.	7.7260392	50.0000000	Y
0.0006767	1997.	2951122.	7.6792137	50.0000000	Y
0.0007145	2019.	2825882.	7.6349831	50.0000000	Y
0.0007524	2039.	2709648.	7.5930986	50.0000000	Y
0.0007903	2056.	2601744.	7.5537398	50.0000000	Y
0.0008281	2072.	2501578.	7.5161353	50.0000000	Y
0.0008660	2085.	2408130.	7.4807469	50.0000000	Y
0.0009038	2098.	2321056.	7.4467853	50.0000000	Y
0.0009417	2109.	2239657.	7.4145379	50.0000000	Y
0.0009795	2119.	2163491.	7.3838416	50.0000000	Y
0.0010174	2129.	2092202.	7.3544458	50.0000000	Y
0.0010553	2137.	2025352.	7.3265314	50.0000000	Y
0.0010931	2145.	1962370.	7.2998800	50.0000000	Y
0.0011310	2152.	1903036.	7.2742035	50.0000000	Y

Summary of Results for Nominal Moment Capacity for Section 1

Load No.	Axial Thrust kips	Nominal Moment Capacity in-kips
1	397.0000000000	2152.

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	28872.
2	6.0000	71.9460	No	No	28872.	13745.
3	10.0000	3.1292	No	No	42617.	36791.
4	12.0000	12.0000	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 = 397000.0 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	1908542.	-41980.	0.00	81254.	3.38E+09	0.00	0.00	0.00
0.1200	-0.6994	1847879.	-41957.	8.00E-04	79257.	3.38E+09	11.8546	24.4070	0.00
0.2400	-0.6977	1786790.	-41931.	0.00153	77245.	3.83E+09	24.9889	51.5754	0.00
0.3600	-0.6950	1725369.	-41885.	0.00218	75223.	4.00E+09	39.2178	81.2559	0.00
0.4800	-0.6914	1663674.	-41817.	0.00278	73192.	4.14E+09	54.3564	113.2049	0.00
0.6000	-0.6870	1601761.	-41728.	0.00334	71153.	4.28E+09	70.1156	146.9641	0.00
0.7200	-0.6818	1539685.	-41615.	0.00386	69110.	4.42E+09	86.3146	182.2950	0.00
0.8400	-0.6759	1477502.	-41479.	0.00434	67062.	4.57E+09	102.7824	218.9737	0.00
0.9600	-0.6693	1415266.	-41319.	0.00479	65013.	4.72E+09	119.3235	256.7142	0.00
1.0800	-0.6621	1353030.	-41135.	0.00520	62964.	4.86E+09	135.9027	295.5646	0.00
1.2000	-0.6543	1290846.	-40928.	0.00559	60917.	4.99E+09	152.3926	335.3693	0.00
1.3200	-0.6460	1228766.	-40697.	0.00595	58873.	5.11E+09	168.4819	375.5518	0.00
1.4400	-0.6372	1166837.	-40443.	0.00628	56834.	5.21E+09	183.9857	415.7855	0.00
1.5600	-0.6279	1105105.	-40167.	0.00660	54801.	5.29E+09	199.2400	456.9143	0.00
1.6800	-0.6182	1043614.	-39868.	0.00689	52777.	5.35E+09	215.2009	501.2731	0.00

1.8000	-0.6081	982409.	-39548.	0.00716	50761.	5.38E+09	230.5125	545.8743	0.00
1.9200	-0.5976	921532.	-39205.	0.00741	48757.	5.38E+09	245.0265	590.4389	0.00
2.0400	-0.5867	861022.	-38842.	0.00765	46765.	5.38E+09	258.6381	634.7678	0.00
2.1600	-0.5755	800916.	-38460.	0.00788	44786.	5.38E+09	272.4755	681.7254	0.00
2.2800	-0.5641	741253.	-38058.	0.00808	42821.	5.38E+09	285.3229	728.4169	0.00
2.4000	-0.5523	682067.	-37639.	0.00827	40873.	5.38E+09	297.0482	774.5270	0.00
2.5200	-0.5402	623394.	-37204.	0.00845	38941.	5.38E+09	307.5192	819.7041	0.00
2.6400	-0.5279	565263.	-36752.	0.00861	37027.	5.38E+09	320.6080	874.4761	0.00
2.7600	-0.5154	507710.	-36280.	0.00875	35132.	5.38E+09	333.9230	932.8835	0.00
2.8800	-0.5027	450771.	-35791.	0.00888	33257.	5.38E+09	346.4096	992.2088	0.00
3.0000	-0.4899	394483.	-35283.	0.00899	31404.	5.38E+09	357.9780	1052.	0.00
3.1200	-0.4769	338876.	-34757.	0.00909	29573.	5.38E+09	372.6982	1125.	0.00
3.2400	-0.4637	283990.	-34208.	0.00917	27766.	5.38E+09	390.4775	1213.	0.00
3.3600	-0.4504	229870.	-33636.	0.00924	25984.	5.38E+09	403.8115	1291.	0.00
3.4800	-0.4371	176552.	-33048.	0.00930	24229.	5.38E+09	412.9113	1360.	0.00
3.6000	-0.4237	124064.	-32448.	0.00934	22501.	5.38E+09	420.6417	1430.	0.00
3.7200	-0.4102	72429.	-31837.	0.00936	20801.	5.38E+09	426.9034	1499.	0.00
3.8400	-0.3967	21668.	-31219.	0.00937	19129.	5.38E+09	431.5988	1567.	0.00
3.9600	-0.3832	-28202.	-30596.	0.00937	19344.	5.38E+09	434.6320	1633.	0.00
4.0800	-0.3697	-77165.	-29968.	0.00936	20957.	5.38E+09	436.5740	1700.	0.00
4.2000	-0.3562	-125212.	-29333.	0.00933	22539.	5.38E+09	446.1542	1803.	0.00
4.3200	-0.3428	-172315.	-28684.	0.00929	24089.	5.38E+09	454.9112	1911.	0.00
4.4400	-0.3295	-218447.	-28023.	0.00924	25608.	5.38E+09	462.8078	2023.	0.00
4.5600	-0.3162	-263587.	-27352.	0.00918	27094.	5.38E+09	469.8097	2139.	0.00
4.6800	-0.3031	-307712.	-26671.	0.00910	28547.	5.38E+09	475.8859	2261.	0.00
4.8000	-0.2900	-350803.	-25982.	0.00901	29966.	5.38E+09	481.0089	2388.	0.00
4.9200	-0.2771	-392843.	-25286.	0.00891	31350.	5.38E+09	485.1546	2521.	0.00
5.0400	-0.2643	-433817.	-24585.	0.00880	32699.	5.38E+09	488.3026	2660.	0.00
5.1600	-0.2518	-473712.	-23880.	0.00868	34013.	5.38E+09	492.1095	2815.	0.00
5.2800	-0.2393	-512514.	-23168.	0.00855	35290.	5.38E+09	496.3411	2986.	0.00
5.4000	-0.2271	-550209.	-22451.	0.00841	36531.	5.38E+09	499.8974	3169.	0.00
5.5200	-0.2151	-586782.	-21729.	0.00825	37736.	5.38E+09	502.7913	3365.	0.00
5.6400	-0.2034	-622224.	-21003.	0.00809	38902.	5.38E+09	505.0401	3576.	0.00
5.7600	-0.1918	-656522.	-20275.	0.00792	40032.	5.38E+09	506.4507	3802.	0.00
5.8800	-0.1806	-689671.	-19545.	0.00774	41123.	5.38E+09	506.8129	4042.	0.00
6.0000	-0.1695	-721662.	-17869.	0.00755	42176.	5.38E+09	1822.	15473.	0.00
6.1200	-0.1588	-749766.	-15244.	0.00735	43102.	5.38E+09	1824.	16539.	0.00
6.2400	-0.1484	-773973.	-12619.	0.00715	43899.	5.38E+09	1821.	17679.	0.00
6.3600	-0.1382	-794284.	-10001.	0.00694	44567.	5.38E+09	1814.	18900.	0.00
6.4800	-0.1284	-810712.	-7398.	0.00673	45108.	5.38E+09	1802.	20212.	0.00
6.6000	-0.1188	-823279.	-4815.	0.00651	45522.	5.38E+09	1785.	21626.	0.00
6.7200	-0.1096	-832020.	-2261.	0.00629	45810.	5.38E+09	1763.	23154.	0.00
6.8400	-0.1007	-836978.	257.9769	0.00606	45973.	5.38E+09	1736.	24813.	0.00
6.9600	-0.09217	-838208.	2735.	0.00584	46014.	5.38E+09	1704.	26622.	0.00
7.0800	-0.08393	-835776.	5162.	0.00561	45934.	5.38E+09	1667.	28605.	0.00
7.2000	-0.07601	-829760.	7533.	0.00539	45735.	5.38E+09	1625.	30791.	0.00
7.3200	-0.06840	-820246.	9839.	0.00517	45422.	5.38E+09	1578.	33218.	0.00
7.4400	-0.06112	-807335.	12073.	0.00495	44997.	5.38E+09	1525.	35933.	0.00
7.5600	-0.05414	-791137.	14227.	0.00474	44464.	5.38E+09	1466.	38996.	0.00
7.6800	-0.04747	-771779.	16291.	0.00453	43826.	5.38E+09	1401.	42484.	0.00
7.8000	-0.04110	-749398.	18254.	0.00433	43090.	5.38E+09	1327.	46483.	0.00
7.9200	-0.03501	-724152.	20100.	0.00413	42258.	5.38E+09	1236.	50849.	0.00
8.0400	-0.02921	-696231.	21803.	0.00394	41339.	5.38E+09	1129.	55673.	0.00
8.1600	-0.02367	-665861.	23348.	0.00376	40339.	5.38E+09	1017.	61842.	0.00

8.2800	-0.01839	-633282.	24725.	0.00358	39267.	5.38E+09	896.0990	70160.	0.00
8.4000	-0.01336	-598748.	25855.	0.00342	38130.	5.38E+09	673.1499	72576.	0.00
8.5200	-0.00855	-562726.	26655.	0.00326	36944.	5.38E+09	437.1294	73613.	0.00
8.6400	-0.00396	-525712.	27117.	0.00312	35725.	5.38E+09	205.4343	74650.	0.00
8.7600	4.23E-04	-488191.	27249.	0.00298	34489.	5.38E+09	-22.2167	75686.	0.00
8.8800	0.00462	-450642.	27056.	0.00285	33253.	5.38E+09	-246.1566	76723.	0.00
9.0000	0.00864	-413534.	26543.	0.00274	32031.	5.38E+09	-466.7604	77760.	0.00
9.1200	0.01251	-377331.	25714.	0.00263	30839.	5.38E+09	-684.4356	78797.	0.00
9.2400	0.01623	-342489.	24615.	0.00254	29692.	5.38E+09	-841.7012	74694.	0.00
9.3600	0.01981	-309340.	23339.	0.00245	28601.	5.38E+09	-930.0870	67596.	0.00
9.4800	0.02328	-278073.	21944.	0.00237	27571.	5.38E+09	-1008.	62360.	0.00
9.6000	0.02664	-248853.	20441.	0.00230	26609.	5.38E+09	-1079.	58294.	0.00
9.7200	0.02991	-221832.	18842.	0.00224	25720.	5.38E+09	-1143.	55021.	0.00
9.8400	0.03308	-197146.	17154.	0.00218	24907.	5.38E+09	-1202.	52311.	0.00
9.9600	0.03619	-174923.	15384.	0.00213	24175.	5.38E+09	-1257.	50018.	0.00
10.0800	0.03922	-155279.	14122.	0.00209	23528.	5.38E+09	-495.3422	18185.	0.00
10.2000	0.04220	-136638.	13397.	0.00205	22915.	5.38E+09	-511.1888	17444.	0.00
10.3200	0.04512	-119036.	12650.	0.00201	22335.	5.38E+09	-526.6370	16807.	0.00
10.4400	0.04800	-102509.	11881.	0.00198	21791.	5.38E+09	-541.7578	16253.	0.00
10.5600	0.05084	-87089.	11090.	0.00196	21283.	5.38E+09	-556.6087	15767.	0.00
10.6800	0.05364	-72810.	10278.	0.00194	20813.	5.38E+09	-571.2368	15335.	0.00
10.8000	0.05642	-59704.	9445.	0.00192	20382.	5.38E+09	-585.6813	14949.	0.00
10.9200	0.05917	-47803.	8591.	0.00191	19990.	5.38E+09	-599.9747	14602.	0.00
11.0400	0.06190	-37140.	7717.	0.00189	19639.	5.38E+09	-614.1446	14286.	0.00
11.1600	0.06462	-27744.	6823.	0.00189	19329.	5.38E+09	-628.2138	13998.	0.00
11.2800	0.06733	-19647.	5908.	0.00188	19063.	5.38E+09	-642.2018	13734.	0.00
11.4000	0.07004	-12878.	4973.	0.00187	18840.	5.38E+09	-656.1246	13491.	0.00
11.5200	0.07273	-7468.	4018.	0.00187	18662.	5.38E+09	-669.9958	13265.	0.00
11.6400	0.07543	-3446.	3043.	0.00187	18529.	5.38E+09	-683.8262	13055.	0.00
11.7600	0.07812	-840.9978	2049.	0.00187	18444.	5.38E+09	-697.6246	12860.	0.00
11.8800	0.08081	317.1026	1034.	0.00187	18426.	5.38E+09	-711.3978	12677.	0.00
12.0000	0.08350	0.00	0.00	0.00187	18416.	5.38E+09	-725.1509	6252.	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 = 1908542. inch-lbs
Maximum shear force = -41980. 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 = 20
Number of zero deflection points = 1

Summary of Pile-head Responses for Conventional Analyses

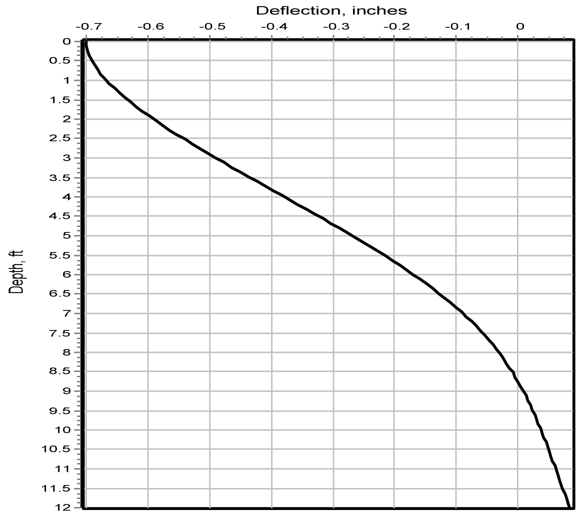
Definitions of Pile-head Loading Conditions:

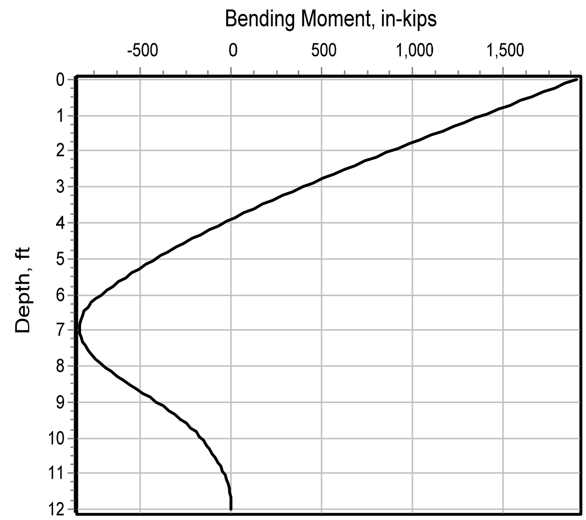
Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians
Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
Load Type 5: Load 1 = Top Deflection, y, inches, and Load 2 = Slope, S, radians

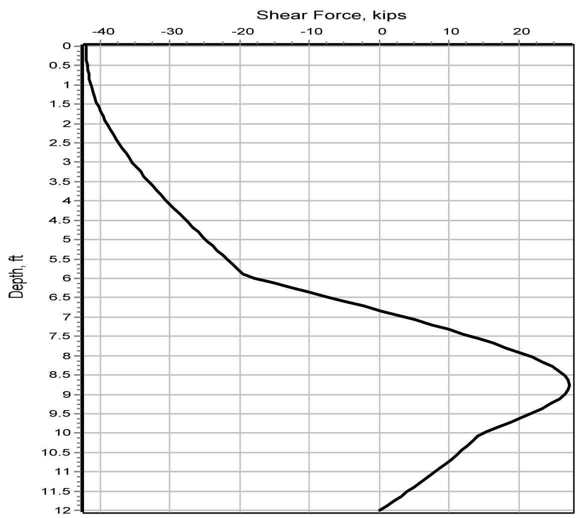
Load Case No.	Load Type 1	Pile-head Load 1	Load Type 2	Pile-head Load 2	Axial Loading lbs	Pile-head Deflection inches	Pile-head Rotation radians	Max Shear in Pile lbs	Max Moment in Pile in-lbs
1	y, in	-0.7000	S, rad	0.00	397000.	-0.7000	0.00	-41980.	1908542.

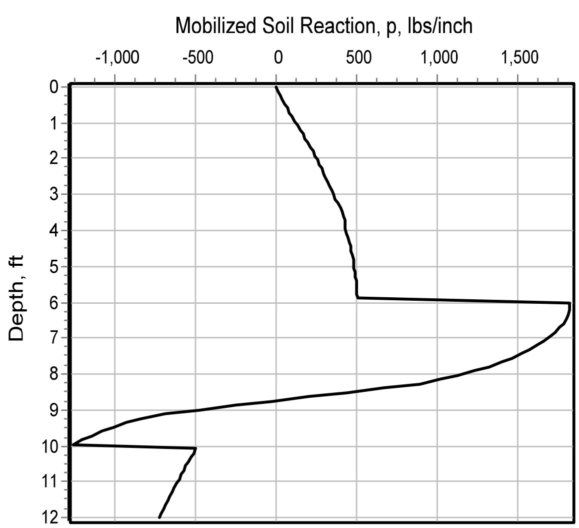
Maximum pile-head deflection = -0.7000000000 inches
Maximum pile-head rotation = 0.0000000000 radians = 0.000000 deg.

The analysis ended normally.


Deflection, inches
-0.7 -0.6 -0.5 -0.4 -0.3 -0.2 -0.1 0
Depth, ft
0 0.5 1 1.5 2 2.5 3 3.5 4 4.5 5 5.5 6 6.5 7 7.5 8 8.5 9 9.5 10 10.5 11 11.5 12


Bending Moment, in-kips
-500 0 500 1,000 1,500
Depth, ft
0 1 2 3 4 5 6 7 8 9 10 11 12


Shear Force, kips
-40 -30 -20 -10 0 10 20
Depth, ft
0 0.5 1 1.5 2 2.5 3 3.5 4 4.5 5 5.5 6 6.5 7 7.5 8 8.5 9 9.5 10 10.5 11 11.5 12


Mobilized Soil Reaction, p, lbs/inch
-1,000 -500 0 500 1,000 1,500
Depth, ft
0 1 2 3 4 5 6 7 8 9 10 11 12

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

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		I_{xx} in^4 cm^4	I_{yy} in^4 cm^4	Compact Section Criteria F'_y ksi MPa
					Flange, t_f in. mm	Web, t_w in. mm			
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		I_{xx} in^4 cm^4	I_{yy} in^4 cm^4	Compact Section Criteria F'_y ksi MPa
					Flange, t_f in. mm	Web, t_w in. mm			
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]

$$K_0 = 1-\sin(\phi) \quad (1)$$ Ref.[5]

Ref.[6]

Ref.[6]

Ref.[7]

$$E_{em} = p_a * \alpha * N_{60} \quad (2)$$ Ref.[8]

$$E = k * B * (1-\upsilon^2) \quad (3)$$ 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.

APPENDIX E7

Bearing Capacity

GOLDER

CALCULATIONS

Date:	12/8/2020	**Made by:**	KAR / MEL
Project No.:	19129538	**Checked by:**	MLM / KAR
Subject:	Bearing Capacity of Pier Spread Footing	**Reviewed by:**	CCB
Project Short Title:	MaineDOT I-295 Exit 22 Mallet Drive Bridge Replacement No. 5721		

OBJECTIVE

Determine nominal and factored bearing resistance of the proposed spread footing on bedrock at the center pier, assuming the "phasing south" option with the bike path scenario.

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 23, 2020.
4. Wyllie, D.C. 1999. Foundations on Rock, 2nd ed. E&FN Spon, NY.
5. Golder interpreted subsurface section A-A' (Figure 3, Preliminary Geotechnical Design Report, dated Sept. 2020).
6. Golder summary of rock core quality (Table 3, Preliminary Geotechnical Design Report, dated September 2020).
7. GeoTesting Express laboratory testing results, dated February 17, 2020 (Appendix C, Preliminary Geotechnical Design Report, dated September 2020).

CALCULATION

A. Determine the bearing resistance at the strength limit state.

As per AASHTO LRFD (Ref. 1) Article 10.6.3.2.2, the nominal bearing resistance of rock should be determined using empirical correlation with the geomechanics RMR system. Since AASHTO LRFD does not directly address bearing resistance on bedrock, this analysis will use Wyllie (1999) Foundations on Rock (Ref. 4) to calculate the unfactored bearing resistance based on correlation to the average RMR value determined for the pier.

Since the footing is anticipated to be on sloping ground (Ref. 5), the procedure in Ref. 4 Section 5.2.5 will be used.

1. Use the average RMR value determined for the bedrock at the pier location to calculate the rock mass friction angle and cohesion.

Rock mass friction angle, ϕ'_i:

$$\phi'_i = \arctan\left[\frac{1}{(4h\cos^2\theta - 1)^{1/2}}\right] \qquad \text{(Ref. 4, Eqn 3.16)}$$

$$h = 1 + \frac{16(m\sigma' + s\sigma_{u(r)})}{3m^2\sigma_{u(r)}} \qquad \text{(Ref. 4, Eqn 3.17)}$$

$$\theta = \frac{1}{3}\left\{90 + \arctan\left[\frac{1}{(h^3 - 1)^{1/2}}\right]\right\} \qquad \text{(Ref. 4, Eqn 3.18)}$$

where:

σ' = vertical effective normal stress on bedrock

γ_{fill} =	125	pcf	(Ref. 2, Table 3-3)
fill thickness at pier location =	11.7	ft	(Ref. 5)
$\sigma' = \gamma_{fill}$ x fill thickness =	1.46	ksf	
RMR = rock mass rating	65		(Ref. 6, average of BB-FMD-102 and -105)
rock type:	Gneiss		(Ref. 6)
m = constant, dependent on rock type and RMR	2.052		(Ref. 4, Table 3.7)
s = constant, dependent on rock type and RMR	0.00293		(Ref. 4, Table 3.7)
$\sigma_{u(r)}$ = unconfined compressive strength of intact rock	1495	ksf	(Ref. 7, BB-FMD-102)

$h =$ 1.01
$\theta =$ 57.4 degrees
$\phi'_i =$ 67.7 degrees

Rock mass cohesion, c_i:

$$c_i = \tau - \sigma' \tan \phi'_i$$ (Ref. 4, Eqn 3.19)

$$\tau = (\cot \phi'_i - \cos \phi'_i) \frac{m\sigma_{u(r)}}{8}$$ (Ref. 4, Eqn 3.15)

$\tau =$ 11.8 ksf
$c_i =$ 8.3 ksf

2. Calculate the nominal bearing resistance.

$$q_n = C_{f1} c N_{cq} + (C_{f2} B \gamma_r / 2) N_{\gamma q}$$ (Ref. 4, Eqn 5.10)

$$N_o = \frac{\gamma_r H}{c}$$ (Ref. 4, Eqn 5.11)

where:

L = footing length	62	ft (Ref. 3, concrete fill subfooting dimension)
B = footing width	11	ft (Ref. 3, concrete fill subfooting dimension)
D = footing embedment depth	0	ft (assumed)
D / B =	0	
L / B =	5.6	
C_{f1} = foundation shape correction factor	1.05	(Ref. 4, Table 5.4, using L / B = 5)
C_{f2} = foundation shape correction factor	0.95	(Ref. 4, Table 5.4, using L / B = 5)
c = rock mass cohesion	8,267	psf (Step 1)
γ_r = rock density	172	pcf (Ref. 7, BB-FMD-102)
Interpreted bedrock elevation at northwest abutment =	117.0	ft (Ref. 5)
Interpreted bedrock elevation at southeast abutment =	143.0	ft (Ref. 5)
H = slope height	26.0	ft (difference in bedrock elev. at abutments)
β = slope angle	7.8	degrees (Ref. 5)
N_o = stability number	0.54	(Ref. 4, Eqn 5.11)
N_{cq} = bearing capacity factor	4.5	(Ref. 4, Figure 5.5)
$N_{\gamma q}$ = bearing capacity factor	220	(Ref. 4, Figure 5.5, using ϕ_i from Step 1)

$q_n =$ 236,773 psf
$=$ 237 ksf

As per AASHTO LRFD (Ref. 1) Article 10.6.2.5.2, if the recommended value of bearing resistance exceeds either the unconfined compressive strength of the rock or the nominal resistance of the concrete, the bearing resistance shall be taken as the lesser of those values. The nominal resistance of concrete shall be taken as $0.3f'_c$.

$f'_c =$ 3500 psi = 504 ksf (assumed)
$q_{n,concrete} = 0.3f'_c =$ 151 ksf

$q_{n,concrete} < q_{n,calculated} < \sigma_{u(r)}$

151 ˋ 237 ˋ 1495 ksf

Thus, use q_n = 151 ksf

3. Calculate the factored bearing resistance and check that it exceeds the applied bearing pressure.

$$q_r = \phi_b q_n$$

where:
ϕ_b = bearing resistance factor 0.45 (Ref. 1, Table 10.5.5.2.2-1, "Footings on rock")

q_r = 68 ksf

Check: $q_r \geq q_{max}$ 68 ksf > 28 ksf OK
(Ref. 3)

B. Determine the bearing resistance at the service limit state.

Use AASHTO LRFD (Ref. 1) Table C10.6.2.5.1-1 to determine the presumptive bearing resistance at the service limit state.

Type of Bearing Material: Foliated metamorphic rock: slate, schist (sound condition allows minor cracks)

Bearing Resistance Recommended Value of Use = 70 ksf

Note: This bearing resistance is settlement limited (1.0 inch as per AASHTO LRFD Section 10.6.2.5.1) and applies only at the service limit state.

Resistance factor for the service limit state: 1.0 (Ref. 1, Section 10.5.5.1)

Factored bearing resistance = 70 ksf

CONCLUSIONS

For the proposed spread footing on bedrock at the center pier (assuming the "phasing south" option), the recommended nominal bearing resistance is 151 ksf for strength and 70 ksf for serviceability. A resistance factor of 0.45 is recommended for use at the strength limit state and a resistance factor of 1.0 is recommended for use at the service limit state. This results in a factored bearing resistance of 68 ksf for strength.

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