REPORT

Supplemental Geotechnical Design Report Part I

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

FREEPORT, MAINE

MAINEDOT WIN 023627.00

Submitted to:

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Table of Contents

TABLES

SHEETS

APPENDICES

Appendix A: Boring Logs

Appendix B: Stability

Appendix C: Settlement

Appendix D: Abutment Lateral Earth Pressure

Appendix E: Abutment 1 Pile Design

Appendix F: Abutment 2 Pile Design

Appendix G: Pier Pile Design

1.0 INTRODUCTION

This Supplemental Geotechnical Design Report (SGDR) Part I summarizes the results of Golder Associates Inc.'s, a member of WSP, (Golder's) supplemental geotechnical design recommendations for the replacement of the Desert Road Bridge #5720 over I-295 in Freeport, Maine at Exit 20 (formerly Merrill Road, see Sheet 1 for the location). This is the first of two supplemental reports associated with the geotechnical design at the site, and specifically pertains to the geotechnical design of the bridge abutments, foundations, and embankments.

Our design calculations and references are made in conjunction with both the HNTB 60% Design Plans[1] and HNTB 98% Design Plans[2]. Golder had previously submitted the Preliminary Geotechnical Design Report (PGDR)[3] on December 21, 2020 which summarized our field activities, field and laboratory data collection, subsurface interpretations, and preliminary geotechnical design for the bridge foundations and embankments. The PGDR[3] serves as the basis for this report. Our work presented herein was completed in accordance with Golder's proposed scope of work[4] for supplemental design and Golder's General Consultant Agreement (GCA) dated June 15, 2020.

Golder's supplemental geotechnical design work is being completed in two stages, and each stage will be documented in a separate report as follows:

- Part I pertains to the supplemental geotechnical subsurface investigation and supplemental design of the bridge abutments, their foundations, and the approach embankments. The supplemental subsurface investigation refined the bedrock profile parallel to the centerline of the proposed abutment locations, updated estimated settlement at the abutments due to the embankment loading, revised the abutment pile analysis, and reanalyzed global stability for both static and pseudo-static loading conditions at both abutments. This effort is the subject of this report.
- Part II pertains to geotechnical foundation designs and recommendations for traffic mast arms and light standards and luminaires within the project development area. This will be the subject of a separate report[5].

1.1 Project Background

The existing Desert Road Bridge (formerly Merrill Road) at I-295 Exit 20 was originally constructed in 1957 and the deck and substructure were widened in 1984. The bridge will be replaced with a two span integral abutment bridge that will increase bridge clearance over I-295 to the 15 foot minimum standard. The proposed abutments will be moved away from I-295 back into the existing embankment, and will be lengthened to the south of the present Desert Road centerline to accommodate widening of Desert Road. These alignment modifications will impact approach embankment configurations and loadings.

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[1] HNTB, May 21, 2021, Freeport, Cumberland County, Merrill Road Bridge over Interstate 295 and Signalized Intersections Exit 20 Interchange: 60% Plans, Filename: Freeport_023627_Exit_20_60%25_Plans.pdf.

[2] HNTB, July 30, 2021, Freeport, Cumberland County, Merrill Road Bridge over Interstate 295 and Signalized Intersections Exit 20 Interchange: 98% Plans, Filename: Exit_20_98%25%20Plans.pdf.

[3] Golder Associates, Inc., December 21, 2020, Preliminary Geotechnical Design Report, I-295 Desert Road Bridge Replacement #5720 (Exit 20), Freeport, Maine, MaineDOT WIN 023627.00.

[4] Golder Associates, Inc., April 9, 2021, Proposal for Phase II Supplemental Geotechnical Design and Analysis: I-295 Desert Road Bridge Replacement #5720 (Exit 20), Freeport, Maine, MaineDOT WIN 023627.00.

[5] Golder Associates, Inc., August 20, 2021, Supplemental Geotechnical Design Report Part II, I-295 Desert Road Bridge Replacement #5720 (Exit 20), Freeport, Maine, MaineDOT WIN 023627.00.

Golder's PGDR details the historical geotechnical investigation findings and Golder's geotechnical subsurface investigation that included in-situ and laboratory testing; presents recommended geotechnical parameters for design and construction; and provides preliminary geotechnical designs for the bridge foundations and approach embankments. The PGDR describes shallow and sloping bedrock encountered at the abutment locations and recommends additional probes to establish the bedrock surface along the proposed abutment centerline. The PGDR additionally recommends abutment pile design, downdrag mitigation strategies for the piles, and engineering analysis and design be performed during final design after the bedrock elevations at the proposed abutment centerlines, specifically Abutment No. 2 with shallow bedrock, are better defined. These recommendations are the basis for the supplemental geotechnical analyses provided in this report.

1.2 Scope of Geotechnical Work

In accordance with the scope of work described in our proposal dated April 9, 2021 and referenced in our Project Contract with MaineDOT dated April 22, 2021, Golder performed the following work for the SGDR Part I:

- Planned, coordinated, and monitored a rock probe field program to establish the bedrock profile along the centerline of proposed Abutment No. 2, which was identified in the PGDR as having shallow sloping bedrock that may have implications for pile design.
- Perform a three-dimensional settlement analysis to better estimate and then mitigate downdrag loading on the abutment piles
- Update the abutment pile analysis, including the geotechnical, structural, and driving resistance, with the revised downdrag and structural loads using the updated abutment locations from the HNTB Design Plans (60%[1] and 98%[2]) and pile lengths based on bedrock elevations from Golder's Interpretive Subsurface Profile (Sheet 5) and Interpretive Subsurface Cross Section (Sheet 6).
- Update the pier pile analysis for the updated loads and pile lengths from the HNTB 98% Design Plans[2] to include a driveability analysis based on bedrock elevations from Golder's Interpretive Subsurface Profile (Sheet 5).
- Update the global stability analyses for both static and pseudo-static loading conditions longitudinal to the abutments and transverse to the Desert Road centerline at the approach embankments based on the HNTB Design Plans (60%[1] and 98%[2]). These analyses include recommendations for changes to proposed geometries and fills to achieve a factor of safety of 1.5 or greater for the abutment stability analyses.

2.0 GEOLOGIC SETTING

2.1 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.[6] Regional surficial geologic mapping indicates the surficial soils consist of Holocene (Recent) wetland/saltwater marsh deposits overlying Pleistocene Presumpscot Formation fine grained sediments, which overlie Pleistocene glacial till deposits. The wetland/saltwater marsh deposits consist of peat, clay, silt, and sand deposited in low-lying areas adjacent to tidal inlets, tidal channels, and tidal flats. The Presumpscot Formation consists of fine-grained marine mud (silt and clay with local sandy beds and lenses),

[6] 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.

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.[7,8,9] Regional mapping indicates the overburden thickness ranges between 5 feet and 200 feet below ground surface in the Yarmouth-Freeport area.[10]

2.2 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.[11,12,13] 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. Within the Yarmouth-Freeport area, metamorphic compositional layering within the Vassalboro Group strikes northeast-southwest, and dips gently to the southeast.

3.0 SUBSURFACE INVESTIGATIONS

3.1 Preliminary Geotechnical Investigation

Golder performed the preliminary geotechnical subsurface investigation as described in in the PGDR[2]. The subsurface investigation included 6 (six) borings (BB-FDR-101 through -106) with two borings near the proposed abutment locations, one boring at the proposed pier location and another boring in the I-295 median approximately 100 feet south of the proposed pier location. For each location, one boring was performed in the southbound lane of Desert Rd and one boring was performed south of the existing embankment to provide information for the proposed bridge shift to the south. These borings were performed in existing fills from the original roadway embankment construction through the in situ glaciomarine and sand and gravel layers to

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7 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.

8 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.

9 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.

10 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.

11 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.

12 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.

13 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.

bedrock. Each boring had 10 feet of rock core drilled as well. Please refer to the PGDR[2] for the methods used, boring logs, and interpretive subsurface stratigraphy. Historical and 100-series borings (BB-FDR-1XX) from the preliminary geotechnical investigation are shown on Sheet 3 and form the basis of the Interpretive Subsurface Profile (shown in Sheet 5 and the Interpretive Subsurface Cross Section shown in Sheet 6 of this report. Boring location, elevation, and bedrock depth for the BB-FDR-100 series borings are summarized in Table 1, and boring logs for these borings are provided in Appendix A.

3.2 Supplemental Investigations

A supplemental boring program was performed at the site to collect geotechnical soil data for the traffic mast arms and light standards and luminaires within the project development area. A detailed description of the boring program field activities, field and laboratory data collection, and subsurface interpretations will be provided in Golder's Supplemental Geotechnical Report Part II[5]. However, some boring data (BB-FDR-206, -207, -208) are used in definition of the bedrock surface and reinterpretation of the soil layering at proposed Abutment No. 2 and have been integrated into the abutment pile design and embankment stability analyses. Sheet 2 through Sheet 4 show the locations of the 200-series supplemental borings (BB-FDR-2XX) with respect to existing and proposed site features. Boring location, elevation, and bedrock depth for the BB-FDR-200 series borings shown in Sheet 5 and Sheet 6 (a subset of the full BB-FDR-200 series) are summarized in Table 1, and boring logs for these borings are provided in Appendix A.

3.3 Supplemental Rock Probes

Golder completed an exploration program to better define the shallow bedrock surface at the proposed location of Abutment No. 2, only. This allowed us to improve our estimate of pile length and design and provide recommendations appropriate for shallow piles on sloping bedrock at this location. Rock probes were not performed at Abutment No. 1 as the PGDR stratigraphy indicates bedrock is sufficiently deep at this location.

Maine Drilling and Blasting, Inc. (MD&B) of Gardiner, Maine completed three (3) bedrock probes (RP-FDR-201, RP-FDR-202, and RP-FDR-203) on June 22, 2021 using an Atlas Copco D7 track-mounted rig, air-track methods, and a 3-inch diameter core bit. A Golder field engineer monitored drilling activities and logged the depth at which bedrock was encountered. The as-drilled rock probe locations were surveyed by MaineDOT following completion of the drilling program. Rock probe location coordinates and ground surface elevations are summarized in Table 1 along with nearby rock, and rock probe locations (RP-FDR-2XX) with respect to existing and proposed site features are illustrated in Sheet 2 through Sheet 4.

4.0 INTERPRETIVE SUBSURFACE PROFILE AND CROSS SECTION

Sheet 5 presents Golder's updated Interpretive Subsurface Profile A-A' along the proposed centerline of Desert Road between Station 58+75 and Station 63+28. Rock probes indicate that the bedrock surface is deeper at the location of proposed Abutment No. 2 by approximately seven (7) feet over the original analysis. Additionally, the stratigraphy has been updated based on findings from borings BB-FDR-206 and BB-FDR -207 west of Abutment No. 1 and BB-FDR-208 east of Abutment No. 2.

Sheet 6 presents Golder's Interpretive Subsurface Cross Section B-B' that illustrates the bedrock depth aligned with the centerline of Abutment No. 2. Interpretive Subsurface Cross Section B-B' was developed to evaluate the impact of possible sloping and shallow bedrock surface on stability of the approach embankment and the pile design, and were used for these analyses.

5.0 GEOTECHNICAL ANALYSES

Golder used the geotechnical data reported in the PGDR to develop design parameters for the major design elements of the proposed bridge and embankment features. Additionally, we used the proposed geometry, elevations, and stations for the proposed bridge abutments and piles and approach embankments from the HNTB 60% Design Plans[1] and 98% Design Plans[2] in the subsequent analyses.

5.1 Stability Analyses

Stability was evaluated for the proposed approach embankments transverse to the Desert Road centerline at Station 60+10 nearest proposed Abutment No. 1 and at Station 62+75 nearest proposed Abutment No. 2 and longitudinal to the proposed abutments along the Desert Road centerline. Golder performed our analyses using the proposed geometry and materials from the HNTB 60% Design Plans[1]. Upon review of the HNTB 98% Design Plans[2], we found no significant changes to the locations, geometry, or elevation of the abutments or the geometry or elevations of the approach embankments, and thus our original analyses based on the HNTB 60% Design Plans[1] are appropriate, and were therefore not updated.

Analyses were performed using the two-dimensional limit equilibrium modeling software *Slide2* by Rocscience[14] for post-construction static and pseudo-static seismic load conditions transverse to the roadway centerline for both the southern and northern embankment slopes associated with the abutments and longitudinal to the face of both Abutment No. 1 and Abutment No. 2. These analyses incorporated material design parameters estimated from SPT N_{60} values. As outlined in the Maine Bridge Design Guide Section 5.9.2, minimum allowable design factors of safety (FS) of 1.3 and 1.5 for static conditions were used in assessing satisfactory transverse embankment geometries and longitudinal abutment geometries, respectively, for the given subsurface conditions. While the Maine Bridge Design Guide Section 5.9.2 provides for a minimum allowable design factors of safety (FS) of 1.0 for pseudo-static seismic conditions, our analysis was based on FHWA (2011)[15] for both the transverse embankments and longitudinal abutment geometries, which requires a minimum FS of 1.1. Our pseudo-static seismic analysis based on FHWA (2011) uses half of the peak ground displacement value provided in the Golder's PGDR[2] or 0.064, assumes an allowable displacement of 1 inch to 2 inches, and evaluates the minimum acceptable FS of 1.1. 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 B.

For all locations, we analyzed the lowest factor of safety for two cases: 1) the lowest factor of safety for a deep-seated failure surface in the native glaciomarine soil, and 2) the lowest factor of safety potential failure surface for the overall slope geometry. Results are summarized in Table 5-1 for both static conditions and pseudo-static seismic conditions for the transverse embankment geometries. Results are summarized in Table 5-2 for both static and pseudo-static seismic conditions for the longitudinal abutment geometries. Longitudinal abutment geometries did not include the proposed piles to bedrock, as Golder does not recommend the piles be relied upon to provide stability of the abutment slopes.

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14 RocScience Slide Software Package Version 9.005, build date May 6, 2020

15 FHWA. 2011. Geotechnical Engineering Circular No. 3 - LRFD Seismic Analysis and Design of Transportation Geotechnical Features and Structural Foundations Reference Manual, Publication No. FHWA-NHI-11-032

For the transverse embankment geometries based on the HNTB 60% Design Plans[1], surfaces extending below the road and through the in situ soil soils, including through the glaciomarine deposit when present, yield a lower bound FS of 2.17 under static conditions and 1.78 under pseudo-static seismic conditions. These factors of safety meet required design FS values for both static and pseudo-static conditions. For the transverse embankment geometries, overall potential failure surfaces that are limited to the existing and proposed fills yield a lower bound FS of 1.25 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 for each of the slopes analyzed, and does not meet the required value of 1.1 for pseudo-static seismic conditions for two of the slopes analyzed.

Table 5-1: Factors of Safety for Static Conditions and Pseudo-Static Seismic Conditions for the Approach Embankments

Condition	Location	Slope	Lowest Factor of Safety[1] (Spencer Method)	
			Non-Circular Failure Surface in Proposed Fill	Non-Circular Deep-Seated Failure Surface Below the Roadway
Static	Abutment No. 1 Embankment (Station 60+10)	North	1.25	2.12
		South	1.28	2.51
	Abutment No. 2 Embankment (Station 62+75)	North	1.29	2.27
		South	1.27	2.17
Pseudo-Static Seismic	Abutment No. 1 Embankment (Station 60+10)	North	1.08	1.78
		South	1.10	2.13
	Abutment No. 2 Embankment (Station 62+75)	North	1.11	1.92
		South	1.09	1.87

1. Minimum factors of safety are 1.3 for static analyses and 1.1 for pseudo-static seismic analyses.

The potential failure surfaces that do not meet FS ≥ 1.3 under static conditions or FS ≥ 1.1 under pseudo-static seismic conditions are 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[16]. The model geometry is also based on HNTB's proposed 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

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

enough to result in a FS ≥ 1.3. The factor of safety for these potential surficial failure areas can likely be increased if the slope has an established protective vegetation or riprap layer, by increasing the required compactive effort for placed fill, decreasing the embankment slope angle, or using an embankment fill material with greater frictional resistance (i.e., angle of internal friction) such as Gravel Borrow with a design angle of internal friction of 36°.

For the longitudinal abutment geometries based on the HNTB 60% Design Plans[1], surfaces extending below the road and through the in situ soil soils, including through the glaciomarine deposit when present, yield a lower bound FS of 2.10 under static conditions and 1.79 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 proposed fills yield a lower bound FS of 1.42 under static conditions and 1.24 under pseudo-static seismic conditions. For this scenario, the FS under static conditions does not meet the required FS ≥ 1.5 for potential failure surfaces that pass below the abutment wall in the proposed and existing fills, however, does meet the required FS > 1.1 for pseudo-static seismic conditions.

Table 5-2: Factors of Safety for Static Conditions and Pseudo-Static Seismic Conditions for the Longitudinal Section Through the Abutments as Designed.

Condition	Location	Lowest Factor of Safety[1] (Spencer Method)	
		Non-Circular Failure Surface in Fill	Non-Circular Deep Seated Failure Surface in Glaciomarine Deposit
Static	Abutment No. 1	1.42	2.10
	Abutment No. 2	1.50	2.15
Pseudo-Static Seismic	Abutment No. 1	1.24	1.79
	Abutment No. 2	1.30	1.87

1. Minimum factors of safety are 1.5 for static analyses and 1.1 for pseudo-static seismic analyses.

Golder revisited the analysis for the longitudinal abutment stability to improve the factor of safety to 1.5 or greater by altering the fill materials and placement area around the abutments and within the slope in front of the abutment while maintaining the overall slope geometry at the Desert Road centerline. The recommended fill changes included:

- A larger volume of rock borrow at the toe of the abutment face slope than proposed, which extends 4 feet in depth and 5 feet in width at the toe of the slope and has a 1.5H:1V backslope between the rock borrow and in situ fill materials;
- Use of gravel borrow as the fill materials below the rock borrow slope face, rather than granular borrow as proposed; and
- Use of a greater volume of gravel borrow in the slope and around the abutment, which extends 4 feet below the base of the abutment, has a horizontal surface forward of the abutment to the back of the rock borrow and a backslope of 1.5H:1V behind the abutment to the roadway.

The geometry for these recommended changes is illustrated in Appendix B, specifically Figure A.4 for Abutment No. 1 and Figure B.4 for Abutment No. 2). Results are summarized in Table 5-3 for both static and pseudo-static seismic conditions for the longitudinal abutment geometries with Golder's proposed fill scenario to improve factor of safety.

For Golder's recommended fill scenario to improve longitudinal abutment stability, surfaces extending below the road and through the in situ soil soils, including through the glaciomarine deposit when present, yield a lower bound FS of 2.07 under static conditions and 1.83 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 proposed fills yield a lower bound FS of 1.53 under static conditions and 1.35 under pseudo-static seismic conditions. For these scenarios, the FS under static conditions meets the required FS ≥ 1.5 for potential failure surfaces and required FS > 1.1 for pseudo-static conditions. The HNTB 98% Design Plans[2] reflect Golder's recommended changes in fill type and geometry longitudinal to the abutments.

Table 5-3: Factors of Safety for Static Conditions and Pseudo-Static Seismic Conditions for the Longitudinal Section Through the Abutments with Recommended Changes to Fills to Meet Required Factors of Safety.

Condition	Location	Lowest Factor of Safety[1] (Spencer Method)	
		Non-Circular Failure Surface in Fill	Non-Circular Deep Seated Failure Surface in Glaciomarine Deposit
Static	Abutment No. 1	1.53	2.07
	Abutment No. 2	1.59	2.15
Pseudo-Static Seismic	Abutment No. 1	1.35	1.83
	Abutment No. 2	1.39	1.88

1. Minimum factors of safety are 1.5 for static analyses and 1.1 for pseudo-static seismic analyses.

5.2 Settlement

Golder evaluated the anticipated total and differential settlement expected at the bridge abutments and at the approach embankments from the proposed improvements using the Rocscience's three-dimensional settlement analysis software *Settle3*[17]. The *Settle3* model incorporates features from the HNTB 60% Design Plans[1] between proposed Desert Road Station 55+50 and Station 65+50, and includes loading from the fill materials placed between the abutment and I-295, approach embankments, and roadway. Upon review of the HNTB 98% Design Plans[2], we found no significant changes in the geometry or elevations of the approach embankments, and thus our original analyses based on the HNTB 60% Design Plans[1] are appropriate, and were therefore not updated.

[17] Rocscience, Inc. *Settle3* Software Package Version 5.001, build date December 19, 2020.

The subsurface stratigraphy was developed based on Interpretive Subsurface Profile A-A' (Sheet 5). Cohesionless soil layers (i.e., proposed fills, existing fill, and sand and gravel) were modeled to have immediate settlement only. The material properties were estimated from the SPT N_{60} values provided in boring logs presented in the Golder PGDR. The cohesive glaciomarine layer was modeled to have consolidation settlement only, immediate settlement and secondary compression was not considered. Due to the stiffness of the glaciomarine clay layer, Golder was unsuccessful in collecting undisturbed samples for consolidation testing during the preliminary geotechnical investigation, thus, we used our knowledge of the Presumpscot Formation 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 and basis of analysis, material properties, and model development in Appendix C.

We evaluated total settlement below the proposed ground surface for the proposed approach embankments along a transverse cross-section at both Station 60+10 and Station 62+75 where the stability analyses were performed. We also evaluated total settlement below Abutment No. 1 and No. 2. The settlement values presented in Table 5-4 include estimated consolidation settlement after 5 years, the time period after which 95% consolidation settlement was estimated to occur. The settlement presented for each abutment is provided to illustrate the settlement of the soil around the abutment piles used in the analysis of downdrag loading.

Table 5-4: Estimated Settlement for Abutment and Approach Embankment Locations

Location	Estimated Total Settlement	Estimated Settlement along the Base of the Abutment
Approach Embankment Station 60+10	max: 2.50 inches	N/A
Approach Embankment Station 62+75	max: 1.52 inches	N/A
Abutment No. 1	N/A	max: 1.39 inches min: 0.38 inches
Abutment No. 2	N/A	max: 0.85 inches min: 0.28 inches

5.3 Proposed Bridge Abutment Lateral Earth Pressure

Per the HNTB 60% Design Plans[1] and HNTB 98% Design Plans[2], the proposed abutment dimensions are 90 feet wide skewed to the roadway alignment centerline, 11 feet tall, and 4.0 feet thick. HNTB also provided Golder with expected abutment movements of 0.8 inches of thermal expansion and 0.18 inches of girder rotation. Golder used these dimensions and abutment movements to analyze lateral earth pressure.

Both integral abutments should be designed to resist lateral earth pressures along the entire 11-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[18] Figure 3.10.8-1, the abutment height provided, the anticipated combined movement of 0.98 inches, we determined that the maximum wall rotation of 0.007 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 identified in AASHTO (2020) when full passive pressures are engaged.

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). Engineering parameters and calculated earth pressure coefficients for the backfill material are presented in Table 5-5.

Golder estimates that the resultant force generated from passive earth pressure, the governing scenario, is an unfactored load of 35,014 pounds per foot of abutment length and acts at elevations 163.7 feet at Abutment No. 1 and 163.1 feet at Abutment No. 2. These values assume the fill is free draining (i.e., no water pressure is allowed to build up behind the abutment walls). Refer to the full methodology of the analysis, material properties, and calculations in Appendix D.

Table 5-5: Lateral Earth Pressure Coefficients

Abutment Earth Pressure Parameters	Value
Granular Backfill Unit Weight, γ (pcf)	125
Granular Backfill Friction Angle, ϕ (˚)	32
Passive Earth Pressure Coefficient, K_p	4.63
Active Earth Pressure Coefficient, K_a	0.31

5.4 Proposed Bridge Abutment Pile Foundations

The new bridge abutments and pier are proposed to be founded on piles driven to the bedrock surface. To provide recommendations on pile design, we analyzed the pile design loads provided by HNTB[19], downdrag loading resulting from settlement of soil around the piles (from embankment loading), axial resistance, lateral loading and resistance, and driveability, as described in the subsections below. Refer to the full methodology of the analysis, material properties, and calculations in Appendix E for Abutment No. 1 and Appendix F for Abutment No. 2.

5.4.1 Pile Design Loading and Deflection

HNTB provided Golder with unfactored loads applied at the bottom of the abutment and the appropriate AASHTO LRFD load factors for the various load cases[19] for 60% design. Golder performed the pile design using the governing factored applied superstructure vertical dead and live load distributed to each of the 9 piles shown in

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

[19] HNTB calculation titled "Freeport Bridges_Loads_Bottom of Footing_flat.pdf", dated May 26, 2021.

the HNTB 60% Design Plans[1], resulting in a load of 357 kips per pile at both abutments. Following pile design, Golder reviewed the 98% design loads provided by HNTB. The Strength I factored vertical load for each abutment was 33 kips less than the 60% design load. The HNTB 98% Design Plans[2] increased the number of piles in each abutment from 9 to 11, resulting in a Strength I factored vertical load of 293 kips per pile at the abutments based on the 60% design loads. Rather than reperform pile design based on the lower load scenario, we present the pile design in subsequent sections based on the Strength I factored vertical load of 357 kips per pile.

HNTB provided Golder with expected lateral movement[20] at each abutment that included a maximum thermal movement of 0.8 inches and horizontal movement due to girder rotation of 0.18 inches. These movements were incorporated into the lateral analysis of the abutment piles.

Golder analyzed HP 14x89 piles at the request of MaineDOT and HNTB for an abutment width of 90 feet. Per Interpretive Profile A-A', Abutment No, 1 is expected to be supported by piles with a length of 32.6 feet. Per Interpretive Subsurface Cross Section B-B', Abutment No, 2 is expected to be supported by piles with a length of 19.0 to 20.5 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 and rotational movements of the superstructure, and driveability analyses for the abutment piles.

5.4.2 Downdrag on Piles

In accordance with AASHTO LRFD Article 3.11.8, downdrag loads on piles can be assumed to be fully developed when settlement at the soils surrounding the piles is 0.4 inches or greater. The settlement analyses at Abutment No. 1 and Abutment No. 2 indicates downdrag loads will be imposed on the piles along the existing fill and glaciomarine clay soil layers. While settlements at the abutment pile locations are expected to vary along the length of the abutments, Golder calculated a maximum downdrag loading value for the pile nearest the southern edge of the abutments where settlement from approach embankment loading was greatest.

The software package APILE[21] was used to calculate shaft resistance contributing to downdrag loads along the length of the pile. Shaft resistance was modeled using the FHWA method for computation of unit load transfers and axial pile capacity to determine the shaft resistance. Specifically, the Nordlund/Thurman Method was used for side resistance in cohesionless soils and the Alpha method with a user-defined alpha value of 1.0 was used for cohesive soils.

Downdrag loads were calculated in accordance with methods described in AASHTO LRFD Article 10.7.3.7. The factored maximum downdrag loads at Abutment No. 1 for an HP 14x89 pile are 106 kips and 96 kips per pile for the Strength I and Service I load cases, respectively. The factored maximum downdrag loads at Abutment No. 2 for an HP 14x89 pile are 76 kips and 69 kips per pile for the Strength I and Service I load cases, respectively. Downdrag load factors of 1.40 (Strength I) and 1.00 (Service I) were used for the glaciomarine soils based on the alpha Tomlinson Method in accordance with AASHTO LRFD Table 3.4.1-2. Downdrag load factors of 1.10 (Strength I) and 1.00 (Service I) were used for the existing fill soils based on the Oregon DOT Geotechnical

[20] HNTB, May 7, 2021. Merrill Road Bridge 60% plans. Abutment 1 Reinforcement Sections Sheet 94 of 113.

[21] Ensoft Inc. (2019). APile, Version 2019.9.6

Design Manual[22]. Downdrag load along the pile increases with depth, and we assume the maximum load occurs at the depth where the remaining settlement is below 0.4 inches.

5.4.3 Axial Pile Resistance

Golder analyzed the nominal structural and geotechnical pile resistance of HP 14x89 piles at Abutment No. 1 and No. 2 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.

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.

Table 5-6: Summary of Strength Limit State Factored Axial Pile Resistance

Abutment	Structural Resistance ϕ_c = 0.50 (kips)	Controlling Geotechnical Resistance ϕ_c = 0.50 (kips)	Drivability Resistance[1,2] ϕ_{dyn} = 0.65 (kips)	Governing Axial Pile Resistance (kips)
Abutment No. 1	653	653	465	465
Abutment No. 2	653	653	435	435
Pier	653	653	414	414

1. Factored axial load required to limit blow counts of between 3 and 15 blows per inch while limiting the driving stresses to below 45 ksi.
2. Drivability resistance based on a Delmag D30 hammer. Refer to Appendix E and Appendix F for fuel setting recommendations.

Drivability analyses were performed using GRLWEAP software[23] to determine the pile resistance that might be achieved at Abutment No. 1 and Abutment 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 15 bpi. The drivability resistances were calculated using the resistance factor, ϕ_{dyn}, of 0.65, for a single pile in axial compression when dynamic testing is performed as specified in AASHTO Table 10.5.5.2.3-1. Drivability controls

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

[23] GRLWEAP Software Package Version 2010-8, Built November 28, 2018.

and the recommended governing resistances for pile design are the resistances provided in the right column "Governing Axial Pile Resistance (kips)" in Table 5-6. The maximum applied factored axial pile loads should not exceed the governing factored axial pile resistances shown in Table 5-6.

5.4.4 Lateral Pile Response

Lateral response of the abutment piles was evaluated using LPile[24] 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[25], and standard properties provided in the MaineDOT Bridge Design Guide. The input parameters are summarized in Table 2 and Appendix E for Abutment No. 1 and Table 3 and Appendix F for Abutment No. 2. The piles were modeled for lateral response in the weak axis assuming factored pile loads, 0.98 inches of combined lateral movement[20] from thermal bridge expansion and girder rotation, and an HP 14x89 pile size. This assumes an applied axial load including pile weight and downdrag of 465 kips for Abutment No. 1 and 435 for Abutment No. 2. The pile head to abutment connection was assumed to be fixed.

For Abutment No. 1, Golder analyzed a pile length of 32.6 feet. For the HP 14x89 piles, the analysis indicates a maximum moment of 242 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 0.98 inches with no lateral deflection at the pile tip. For Abutment No. 2, Golder analyzed a pile length of 19.6 feet. The analysis indicates a maximum moment of 247 kip-feet occurs at the top of the pile under the Strength I load case and maximum lateral deflection of the pile cap of 0.98 inches and no lateral movement occurring at the pile tip. Results of the LPile first iteration analyses are summarized in Table 5-7.

Table 5-7: LPile Lateral Analysis Results - First Iteration

Location	Axial Load Analyzed (kips)	Lateral Deflection[1] (in)	LPILE Moment at Pile Head (in-kips)	Plastic Hinge Moment (in-kips)	Plastic Hinge Forms
Abutment No. 1	465	0.98	2908	1656	Yes
Abutment No. 2	435	0.98	2962	1772	Yes

1. From combined thermal effects and girder rotation as provided by HNTB[20]

Since a plastic hinge forms at both abutments, 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 5-8 and indicate that the demand ratio for combined axial and 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.

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

[25] 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>

The results also indicate the nominal structural resistance in Segment 1 is sufficient to support the loads analyzed.

Table 5-8: LPile Lateral Analysis Results - Second Iteration

Location	Axial Load Analyzed (kips)	Lateral Deflection[1] (in)	Plastic Hinge Moment at Pile Head (in-kips)	Structural Resistance Demand Ratio, Segment 1[2]	Combined Axial and Bending Demand Ratio, Segment 2[3]
Abutment No. 1	465	0.975	1656	0.53	0.94
Abutment No. 2	435	0.975	1772	0.50	0.89

1. From combined thermal effects and girder rotation as provided by HNTB[20]
2. The ratio of the applied axial load (P_u) to the calculated compressive structural pile resistance in the upper portion of the pile ($P_{r.top}$) is recommended to be greater than 0.2 to avoid an unnecessarily large pile, as per the VTrans Integral Abutment Bridge Design Guidelines[26] Section 4.5.2.
3. The combined axial and bending demand ratio should be less than 1.0, as per AASHTO LRFD Article 6.9.2.2.1.

5.4.5 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 including estimated downdrag loads, pile lengths, and pile orientation are provided in Table 5-9.

Table 5-9: Pile Analysis Summary.

Location	Recommended Nominal Pile Resistance (kips)[1]	Recommended Max. Factored Axial Load Per Pile (kips)	Estimated Factored Downdrag Load (kips)[2]	Estimated Pile Length (feet)	Pile Orientation
Abutment No. 1	715	465	106	32.6	Weak
Abutment No. 2	669	435	76	19.6	Weak
Pier	636	414	Assumed Negligible	12.0	Strong

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.

[26] Vermont Agency of Transportation (2008). Integral Abutment Bridge Design Guidelines. Accessed on 8/18/2021 <https://vtrans.vermont.gov/sites/aot/files/highway/documents/structures/SEI-08-004-1.pdf>

5.5 Proposed Pier Pile Foundations

Golder performed an analysis of the pier pile foundations as part of the preliminary geotechnical design. Our updated analyses for use of HP 14x89 piles at the pier are summarized below. Refer to the full methodology of the analysis, material properties, and calculations in Appendix G.

A drivability analysis was performed using GRLWEAP software[23] to determine the pile resistance that might be achieved at the pier considering available diesel hammers. Downdrag loading was assumed to be negligible as the finished grade at the base of the new pier will be similar to the existing. The nominal pile driving resistance was calculated using a resistance factor (ϕ_{dyn}) of 0.65 as specified in AASHTO Table 10.5.5.2.3-1 for a single pile in axial compression when dynamic testing is performed. Based on this resistance factor and the anticipated Strength I factored axial load of 414 kips per pier pile provided by HNTB[27], the recommended nominal pile resistance at the pier is 636 kips. Based on our analysis, available diesel hammers should be capable of driving the piles to the nominal pile driving resistance while staying at a blow count of between 3 bpi and 15 bpi and limiting the driving stresses to below 45 ksi, in accordance with Section 501.042 of the MaineDOT 2020 Standard Specifications. Table 5-6 summarizes the governing factored axial resistance for pier piles and Table 5-9 summarizes the recommended maximum factored loads for design of HP 14X89 piles at the pier, including pile lengths, and pile orientation. We provide construction considerations for pier piles in Section 6.0.

Lateral response of the pier piles was not evaluated, as we assumed the pier piles would be subjected to minimal lateral deflections and the abutments would accommodate the lateral deflection due to thermal expansion/contraction and girder rotation.

6.0 CONSTRUCTION CONSIDERATIONS

All areas proposed for embankment fill placement or footing construction should be cleared, grubbed, and stripped of existing vegetation, pavement, and topsoil. During the grubbing and stripping process, unsuitable materials exposed at the subgrade level, such as wood, logs, tree stumps, organic silt, peat, soft clay, debris fill, or other materials that may compress, decay or collapse should be removed. Subgrade surfaces for embankments should be prepared in accordance with MaineDOT Standard Specifications and Standard Details, specifically Subsection 203.09 related to benching existing slopes before placement of fill materials.

Structural Fill materials and placement methods for abutment construction should meet the requirements of MaineDOT Standard Specifications, specifically Subsection 203.10 through Subsection 203.12.

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

[27] Hodgdon, S. (HNTB). "Freeport Exit 20 and 22 Pier Loading" Message to Melissa Landon (Golder Associates). July 27, 2021. E-mail including an attachment entitled: "Freeport Bridges_Loads_Bottom of Footing_rev.pdf".

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 for each substructure element 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 blows per inch to 15 blows per inch at end of driving (EOD).
- 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 during the initial drive and at the 24-hour (minimum) restrike. Dynamic testing and field inspection should include verification of hammer stroke or bounce chamber pressures and hammer blows throughout the pile driving operations. To facilitate field inspection, the special provisions should include a requirement that the contractor provide the resident with a working Saximeter for the duration of pile testing and production pile driving. The Vermont Agency of Transportation provides the following details in Standard Specification 504.02(e)[28].

  Saximeter. The Contractor shall provide a Saximeter or equivalent device to assist the Inspector in collecting data to monitor the blow count (for all hammer types), the stroke (for open-end diesel hammers only), or the kinetic energy (if the hammer is equipped with proximity switches for measuring impact velocity). The Saximeter shall be completely charged and in sound working order prior to Agency use and shall be available for the duration of the pile driving operation. Pile driving operations shall not be conducted without the use of a Saximeter.

  The Saximeter shall perform the following functions:

  1. Detect hammer blows automatically using sound recognition circuits, or manually with a keypad.
  2. Automatically count blows and determine the blows per minute (BPM) for all impact hammers.
  3. Calculate the stroke for open-end diesel hammers.
  4. Store blow count, penetration, average stroke or BPM data in memory.
  5. Permit viewing of results on built-in screen.
  6. Permit data transfer to PCs or printers.

  For hammers equipped with proximity switches, the Saximeter shall be deployed to acquire hammer impact velocity data by communicating with a transmitter mounted on the hammer and use this information to calculate the hammer's kinetic energy.

[28] Vermont Agency of Transportation (2018). Standard Specifications for Construction. Accessed on 8/18/2021 < https://outside.vermont.gov/agency/VTRANS/external/docs/construction/02ConstrServ/PreContract/2018SpecBook/2018%20Standard%20Specifications%20for%20Construction.pdf>

The Saximeter shall operate on rechargeable batteries, with batteries and charger supplied by the Contractor.

- Piles should be driven to achieve bearing at the top of bedrock. 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 piles should be fitted with standard prefabricated driving shoes meeting MaineDOT Standard Specification 501.048 to reduce potential for damaging the piles during driving and to engage the piles on sloping bedrock.
- The Engineer should review all dynamic pile testing results before the piles are cut off.
- Signal matching analysis of the dynamic test data using methods described by Rausche et al. (1972)[29] should be conducted to determine pile bearing resistance.

Our analyses are presently based on the embankment fills near the abutments being placed after the piles/abutment. This was done to account for the largest potential downdrag loading on the piles.

Since excavations along the existing embankment and for the piers will be adjacent to live traffic, limits on tolerable horizontal and vertical movement of the ground adjacent to the excavation should be provided in the construction documents and the contractor should be required to develop a monitoring plan for implementation prior to start of construction. Requirements for frequency of monitoring and minimum action levels should be outlined in the specification. Action levels would correspond to requiring increased attention to limiting further movements. Limiting movements acceptable to Maine DOT should also be specified requiring the contractor to suspend excavation until they submit an acceptable plan showing how further movements will be restricted.

New piles will be used for the pier. The plans should be clear about the disposition of the existing piles if there is a preference. As-bult plans of the existing bridge should be provided to indicate the locations and types of existing piles which need to either be removed or worked around.

7.0 REPORT AND EXPLORATION LIMITATIONS

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

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

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[29] 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.

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.

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https://golderassociates.sharepoint.com/sites/139980/project files/6 deliverables/sgdr part i 21450908/final/sgdr part 1 final - freeport bridge 5720 win 023627.00 - golder 21450908.docx

Tables

Table 1: Bedrock Depths from Rock Probes and Borings Nearby Abutments and the Pier
Supplemental Geotechnical Design Report Part I
I-295 Desert Road Bridge Replacement #5720 (Exit 20)
Freeport, Maine
MaineDOT WIN 023627.00

Test Boring Designation[1]	As-Drilled Locations[2,3]		Existing Ground Surface Elevation[3] (ft)	Boring Depth[4] (ft)	Bedrock Elevation[4] (ft)	Bedrock Depth[4] (ft)	Nearby Feature
	Stationing	Offset					
BB-FDR-101	60+18.0	3.7 R	168.2	46.9	131.6	36.6	Abutment 1
BB-FDR-102	61+53.7	4.6 R	145.7	26.1	129.6	16.1	Pier
BB-FDR-103	62+79.8	3.3 R	166.4	30.8	145.6	20.8	Abutment 2
BB-FDR-104	63+01.7	99.9 R	144.4	20.5	134.1	10.3	Abutment 2
BB-FDR-105	61+62.1	94.8 R	143.4	28.5	125.4	18.0	Pier
BB-FDR-106	60+38.2	98.6 R	147.6	16.8	140.9	6.7	Abutment 1
MTB-1-83[5]	62+21.1	3.4 R	149.8	19.5	135.3	14.5	Abutment 2
MTB-2-83[5]	61+48.7	3.5 R	147.8	25.6	127.2	20.6	Pier
MTB-3-83[5]	60+73.8	3.6 R	150.0	40.4	114.6	35.4	Abutment 1
BB-FDR-206	58+96.5	6.2 R	164.8	30.5	139.3	25.5	Abutment 1
BB-FDR-207	59+10.9	28.4 L	165.3	26.6	Not encountered		Abutment 1
BB-FDR-208	63+12.5	66.3 R	148.0	12.8	139.6	8.4	Abutment 2
RP-FDR-201	60+22.7	52.6 L	166.8	28.0	138.8	28.0	Abutment 2
RP-FDR-202	60+26.8	25.6 L	166.9	28.0	138.9	28.0	Abutment 2
RP-FDR-203	60+28.3	15.1 L	166.8	27.0	139.8	27.0	Abutment 2

Reference
A. Golder Associates, Inc., December 21, 2020, Preliminary Geotechnical Design Report, I-295 Desert Road Bridge Replacement #5720 (Exit 20), Freeport, Maine, MaineDOT WIN 023627.00. (BB-FDR-1XX and MTB-X-83 boring
B. Golder Associates, Inc., August 20, 2021, Supplemental Geotechnical Design Report Part II, I-295 Desert Road Bridge Replacement #5720 (Exit 20), Freeport, Maine, MaineDOT WIN 023627.00. (BB-FDR-2XX boring information.)

Notes
1. Borings BB-FDR-10X were performed by New England Boring Contractors from December 12 to 16, 2019 as part of preliminary design (Ref. A). Borings BB-FDR-206 through BB-FDR-208 were performed by S.W. Cole on May 12 through 25, 2021. Rock probes RP-FDR-20X were performed by Maine Drilling and Blasting on June 22, 2021.
2. All test boring (BB-FDR-XXX) and rock probe (RP-FDR-XXX) locations are illustrated in Sheet 2 entitled "Boring and Probe Location Plan".
3. As-drilled locations and elevations for BB-FDR-2XX are derived from the survey file received from MaineDOT on June 23, 2021 entitled: 23627 BORE 200 Series Compiled.csv. As-drilled locations and elevations for BB-FDR-1XX are derived from the survey file received from MaineDOT on January 6, 2020 entitled: BOR12-17-19edit.csv.
4. Boring logs provided in Appendix A. Boring logs were not created for MTB-1-83.
5. Information for MTB-1-83, MTB-2-83, and MTB-3-83 regarding location, elevation, depth of boring, and depth to rock has been interpreted from a electronic file named "5720 Freeport 1984" and "1984 soils data" provided to Golder by the Maine Department of Transportation on May 2, 2019.

Prepared By: HTV
Checked By: BK
Reviewed By: CCB

Table 2: Summary of Soil Properties Used in LPile Analysis - Abutment No. 1
Supplemental Geotechnical Design Report Part I
I-295 Desert Road Bridge Replacement #5720 (Exit 20)
Freeport, Maine
MaineDOT WIN 023627.00

Stratigraphy		Depth Below Base of Abutment (ft)[1]	Layer Thickness (ft)	Lateral Model[6]	Effective Unit Weight (pcf)	Undrained Shear Strength (psf)[2]	ϕ (deg)[2]	Subgrade Modulus (pci)[3]	Major Principal Strain @ 50%[3]	UCS (psi)[2]
Existing Fill (above WT)	Layer 1	0.0 14.8	14.8	Sand (Reese)	125	-	32	124.8	-	-
Existing Fill (below WT)	Layer 2	14.8 18.1	3.3	Sand (Reese)	62.6	-	32	75.5	-	-
Glaciomarine Silty Clay	Layer 3	18.1 27.7	9.6	Stiff Clay w/o Free Water (Reese)	62.6	1600	-	-	0.005	-
Sand and Gravel	Layer 4	27.7 32.6	4.9	Sand (Reese)	62.6	-	37	40.5	-	-
Bedrock	Layer 5	32.6 50.0	17.4	Strong Rock (Vuggy Limestone)	101.6	-	-	-	-	12983

Notes:

1. Golder's Interpreted Subsurface Section A-A' (Sheet 5).
2. Golder geotechnical test boring logs (100-series: Appendix A, Preliminary Geotechnical Design Report, dated September 2020, 200-series: Appendix A, Supplemental Geotechnical Design Report Part 2).
3. Bridge Software Institute. “FB-MultiPier Soil Parameter Table (US Customary Units).” Accessed July 2020. <https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf>
4. WT = water table
5. ft = feet, pcf = pounds per cubic foot, psf = pounds per square foot, deg = degrees, pci = pounds per cubic inch; psi = pounds per square inch
6. Layer names refer to the LPile lateral model type rather than the actual soil or rock encountered at site.

Prepared by: MLM
Checked by: MEL
Reviewed by: JEL

Table 3: Summary of Soil Properties Used in LPile Analysis - Abutment No. 2
Supplemental Geotechnical Design Report Part I
I-295 Desert Road Bridge Replacement #5720 (Exit 20)
Freeport, Maine
MaineDOT WIN 023627.00

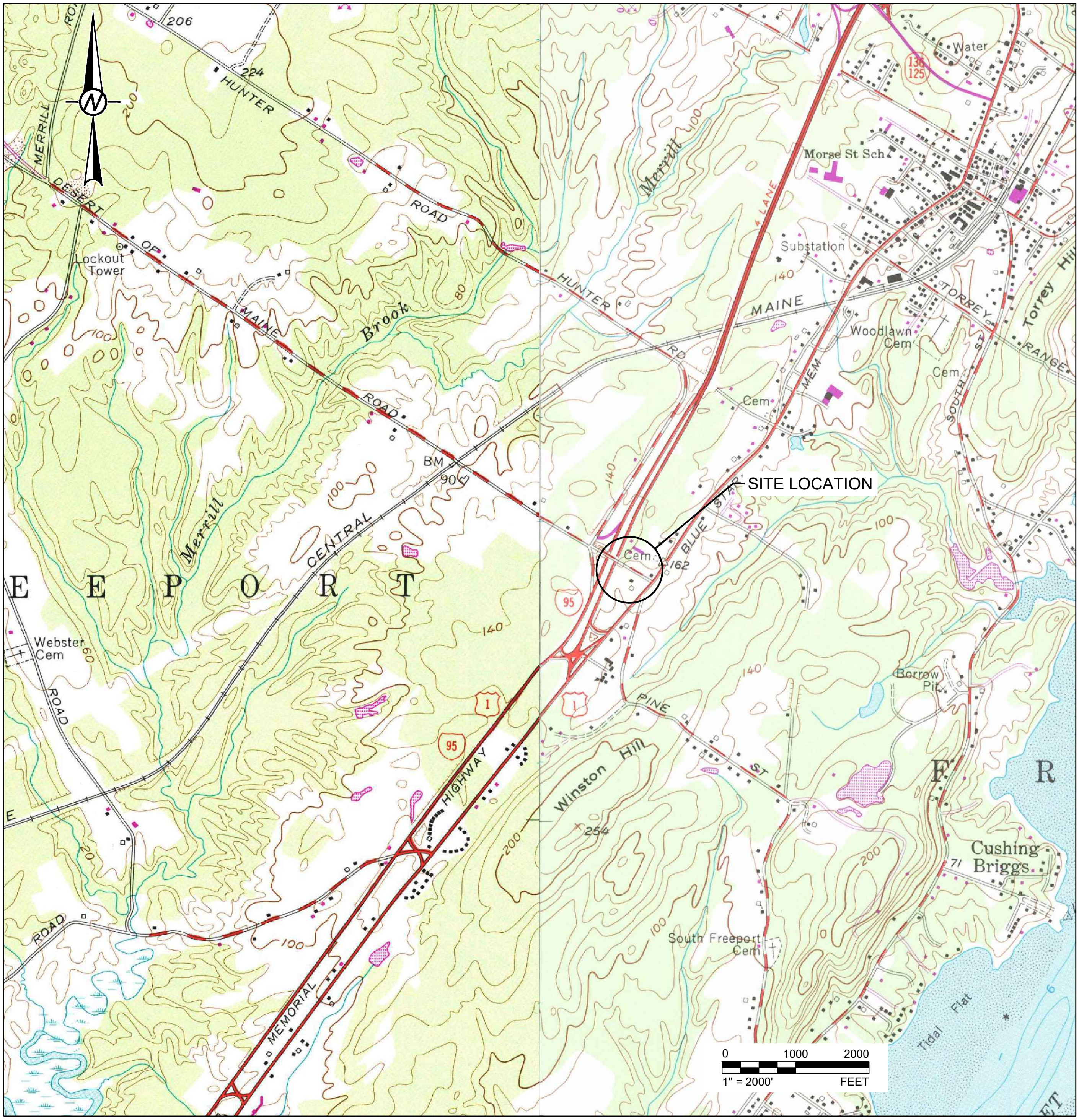
Stratigraphy		Depth Below Base of Abutment (ft)[1]	Layer Thickness (ft)	Lateral Model[6]	Effective Unit Weight (pcf)	Undrained Shear Strength (psf)[2]	ϕ (deg)[2]	Subgrade Modulus (pci)[3]	Major Principal Strain @ 50%[3]	UCS (psi)[2]
Existing Fill (above WT)	Layer 1	0.0 15.3	15.3	Sand (Reese)	125	-	32	124.8	-	-
Glaciomarine Silty Clay (above WT)	Layer 2	15.3 19.0	3.7	Stiff Clay w/ Free Water (Reese)	125	1600		500	0.005	-
Glaciomarine Silty Clay (below WT)	Layer 3	19.0 19.6	0.6	Stiff Clay w/o Free Water (Reese)	62.6	1600	-	-	0.005	-
Bedrock	Layer 5	19.6 50.0	30.4	Strong Rock (Vuggy Limestone)	101.6	-	-	-	-	12983

Notes:

1. Golder's Interpreted Subsurface Section A-A' (Sheet 5) and Interpreted Subsurface Section B-B' (Sheet 6).
2. Golder geotechnical test boring logs (100-series: Appendix A, Preliminary Geotechnical Design Report, dated September 2020, 200-series: Appendix A, Supplemental Geotechnical Design Report Part 2).
3. Bridge Software Institute. “FB-MultiPier Soil Parameter Table (US Customary Units).” Accessed July 2020. <https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf>
4. WT = water table
5. ft = feet, pcf = pounds per cubic foot, psf = pounds per square foot, deg = degrees, pci = pounds per cubic inch; psi = pounds per square inch
6. Layer names refer to the LPile lateral model type rather than the actual soil or rock encountered at site.

Prepared by: MLM
Checked by: MEL
Reviewed by: JEL

Sheets



REFERENCE(S)

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

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

CLIENT

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

PROJECT

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

CONSULTANT

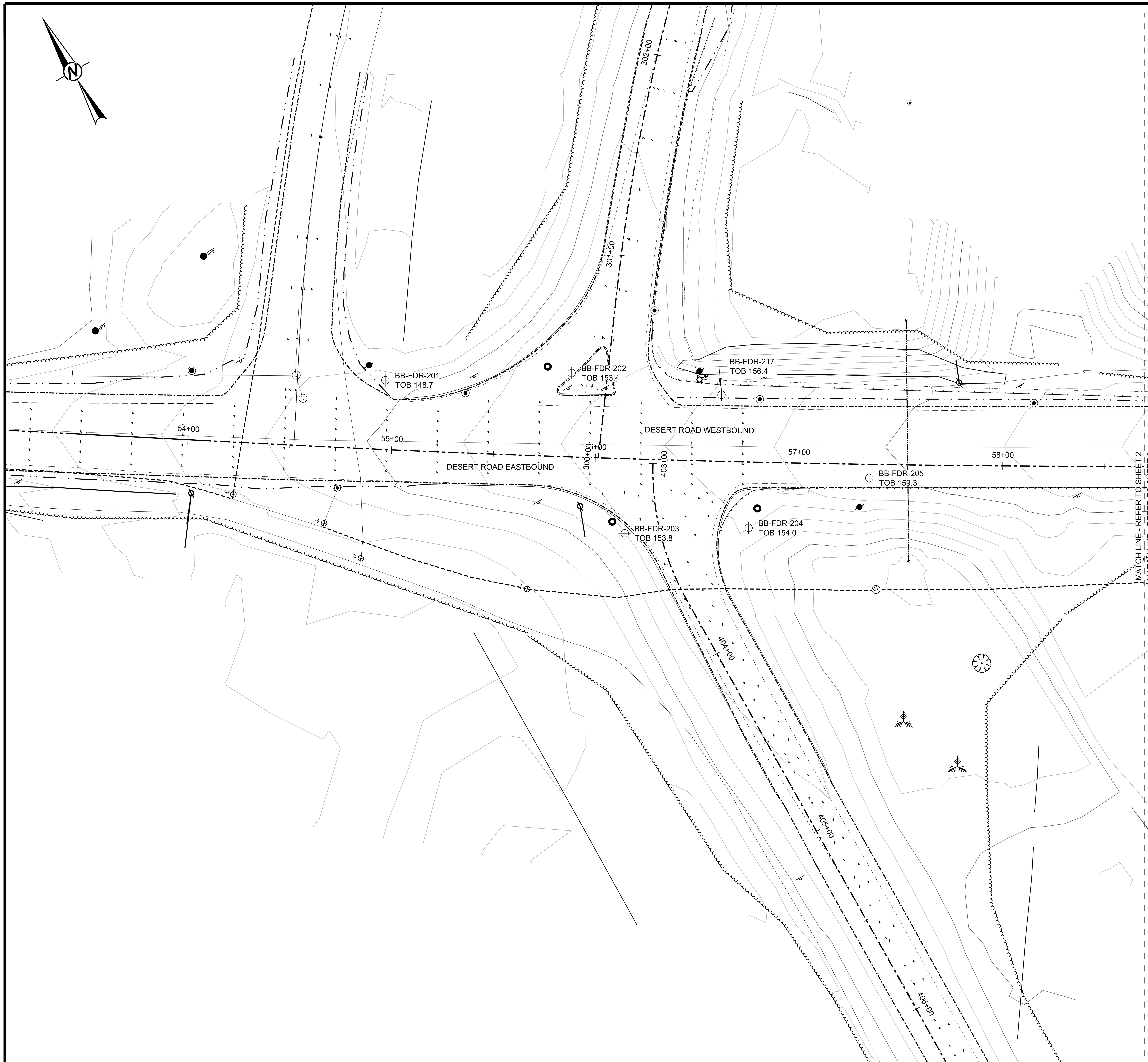
GOLDER
MEMBER OF WSP

YYYY-MM-DD	2021-08-20
DESIGNED	MEL
PREPARED	RWC
REVIEWED	MEL
APPROVED	CCB

TITLE

SITE LOCATION MAP

PROJECT NO.	SUBTITLE	REV.	SHEET
21450908	A	0	1-6



LEGEND

(symbol)	PROPOSED LIGHT STANDARD FOUNDATION
(symbol)	PROPOSED MAST ARM FOUNDATION
MTB-1-83 TOB 135.3	HISTORICAL BORINGS (SEE SHEET 3)
BB-FDR-102 TOB 129.6	COMPLETED 100 SERIES BORINGS (SEE SHEET 3)
BB-FDR-202 TOB 153.4	COMPLETED 200 SERIES BORINGS
TOB XXX.X	ELEVATION OF TOP OF BORING OR PROBE

NOTE(S)

1. AS DRILLED BORING LOCATION PLAN FOR 100-SERIES BORINGS DERIVED FROM ELECTRONIC FILE NAME: "BOR12-17-19edit.csv" PROVIDED TO GOLDER BY MAINE DEPARTMENT OF TRANSPORTATION ON 01/06/2020. AS DRILLED BORING LOCATION PLAN FOR 200-SERIES BORINGS DERIVED FROM ELECTRONIC FILE NAME: "23627 BORE 200 series Compiled.csv" PROVIDED TO GOLDER BY MAINE DEPARTMENT OF TRANSPORTATION ON 06/28/2021.

REFERENCE(S)

1. BASEMAP ELEMENTS TAKEN FROM MAINE DOT DRAWING TITLED "3DTopo_2019-10-18.dgn" RECEIVED ON JANUARY 2, 2020.
2. SURVEY PROVIDED TO GOLDER ON JANUARY 02, 2020 BY MAINE DOT IN FILE "FREEPORT 23627.00 SURVEY DATA 2019-11-18.zip".
3. PROPOSED STATIONING PROVIDED TO GOLDER BY HNTB FOR A SOUTHERN SHIFT OF THE BRIDGE.
4. LOCATIONS FOR MAST ARM FOUNDATIONS AND LIGHT STANDARD FOUNDATIONS DERIVED FROM ELECTRONIC FILE NAME: "Foundation Location Tables.pdf" PROVIDED BY HNTB ON JUNE 29, 2021.
5. GOLDER ASSOCIATES, INC., DECEMBER 21, 2020, PRELIMINARY GEOTECHNICAL DESIGN REPORT, I-295 DESERT ROAD BRIDGE REPLACEMENT #5720 (EXIT 20), FREEPORT, MAINE, MAINEDOT WIN 023627.00.
6. GOLDER ASSOCIATES, INC., AUGUST 20, 2021, SUPPLEMENTAL GEOTECHNICAL DESIGN REPORT PART I, I-295 DESERT ROAD BRIDGE REPLACEMENT #5720 (EXIT 20), FREEPORT, MAINE, MAINEDOT WIN 023627.00.
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8. EXIT RAMP ALIGNMENTS PROVIDED TO GOLDER BY HNTB ON JULY 14, 2021.

0 25 50
1" = 25' FEET

GOLDER
MEMBER OF WSP

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

023627.00

WIN 023627.00

BRIDGE PLANS

Bridge No. 5720

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

SIGNATURE

P.E. NUMBER

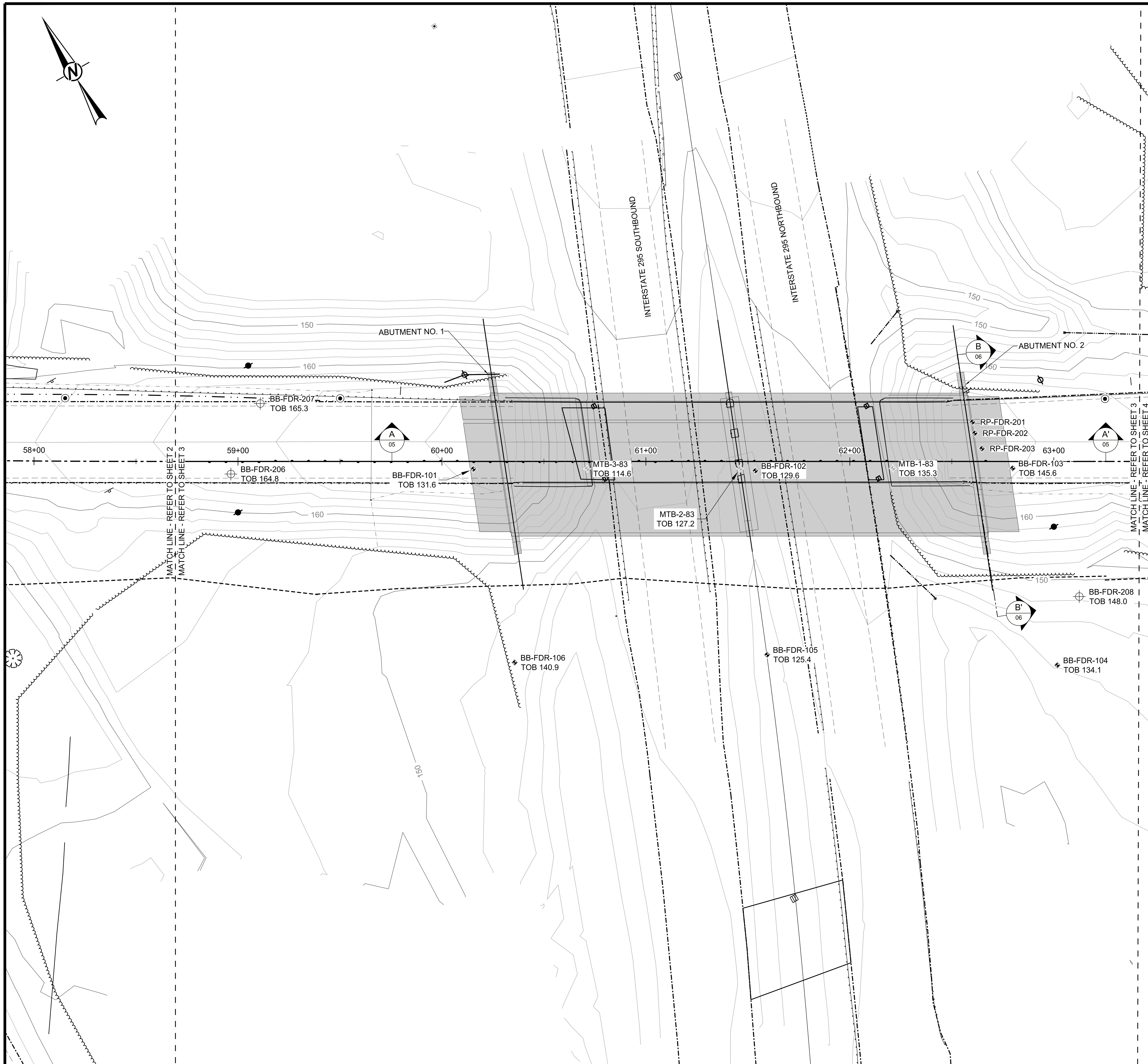
DATE

	MEL	BY	DATE
PROJ. MANAGER			
DESIGN-DETAILED	MEL	AAZ	2021/08/20
CHECKED-REVIEWED	CCB	AAZ	2021/08/20
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

MERRILL ROAD BRIDGE
INTERSTATE 295
FREEPORT CUMBERLAND

BORING LOCATION PLAN

SHEET NUMBER
2
OF 6



LEGEND

RP-FDR-201	COMPLETED ROCK PROBES
MTB-1-83 TOB 135.3	HISTORICAL BORINGS
BB-FDR-102 TOB 129.6	COMPLETED 100 SERIES BORINGS
BB-FDR-202 TOB 153.4	COMPLETED 200 SERIES BORINGS
	PROPOSED LIGHT STANDARD FOUNDATION
	PROPOSED MAST ARM FOUNDATION
TOB XXX.X	ELEVATION OF TOP OF BORING OR PROBE

NOTE(S)

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8. EXIT RAMP ALIGNMENTS PROVIDED TO GOLDER BY HNTB ON JULY 14, 2021.
9. DETAILS OF BRIDGE AND FOUNDATION ELEMENTS TAKEN FROM HNTB, MERRIL ROAD BRIDGE OVER INTERSTATE 295 AND SIGNILIZED INTERSECTIONS, EXIT 20 INTERCHANGE: 98% PS&E, DATED JULY 30 2021.

0 25 50
1" = 25' FEET

GOLDER
MEMBER OF WSP

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

023627.00

WIN
023627.00

BRIDGE PLANS
Bridge No. 5720

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

SIGNATURE

P.E. NUMBER

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	BY	DATE
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DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

MERRILL ROAD BRIDGE
INTERSTATE 295
FREEPORT CUMBERLAND

BORING LOCATION PLAN

SHEET NUMBER
3
OF 6

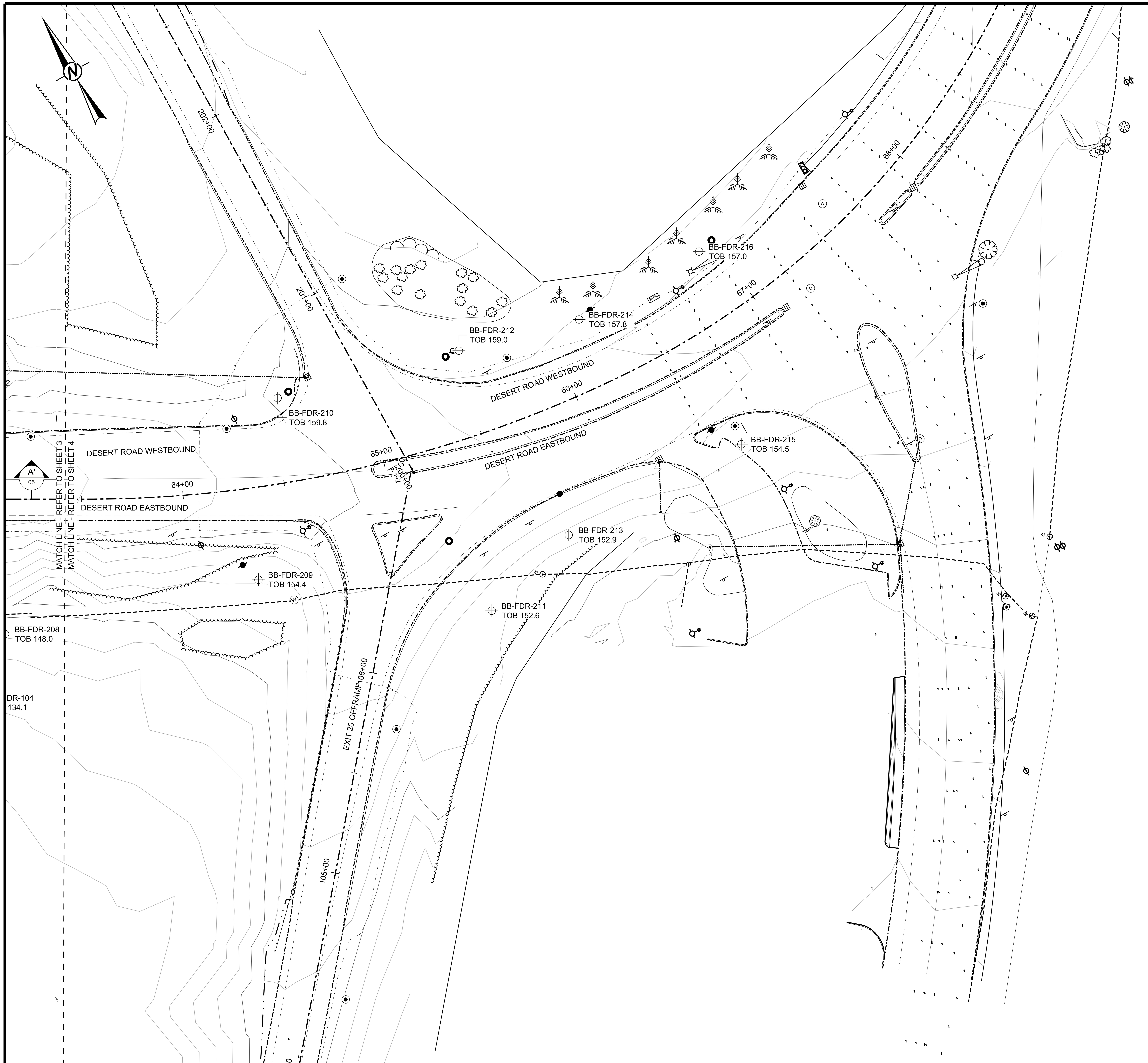
Date:2021-07-01
Username:
Division:
Filename: 21450908_0120_001

Date:2021-07-01

Username:

Division:

Filename: 21450908_0120_001



LEGEND

⊙	PROPOSED LIGHT STANDARD FOUNDATION
○	PROPOSED MAST ARM FOUNDATION
MTB-1-83 TOB 135.3	HISTORICAL BORINGS (SEE SHEET 3)
BB-FDR-102 TOB 129.6	COMPLETED 100 SERIES BORINGS (SEE SHEET 3)
BB-FDR-202 TOB 153.4	COMPLETED 200 SERIES BORINGS
TOB XXX.X	ELEVATION OF TOP OF BORING OR PROBE

NOTE(S)

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REFERENCE(S)

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1" = 25' FEET

GOLDER
MEMBER OF WSP

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

023627.00

WIN 023627.00

BRIDGE PLANS

Bridge No. 5720

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

		BY	DATE
PROJ. MANAGER	MEL		
DESIGN-DETAILED	MEL	AAZ	2021/08/20
CHECKED-REVIEWED	CCB	AAZ	2021/08/20
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

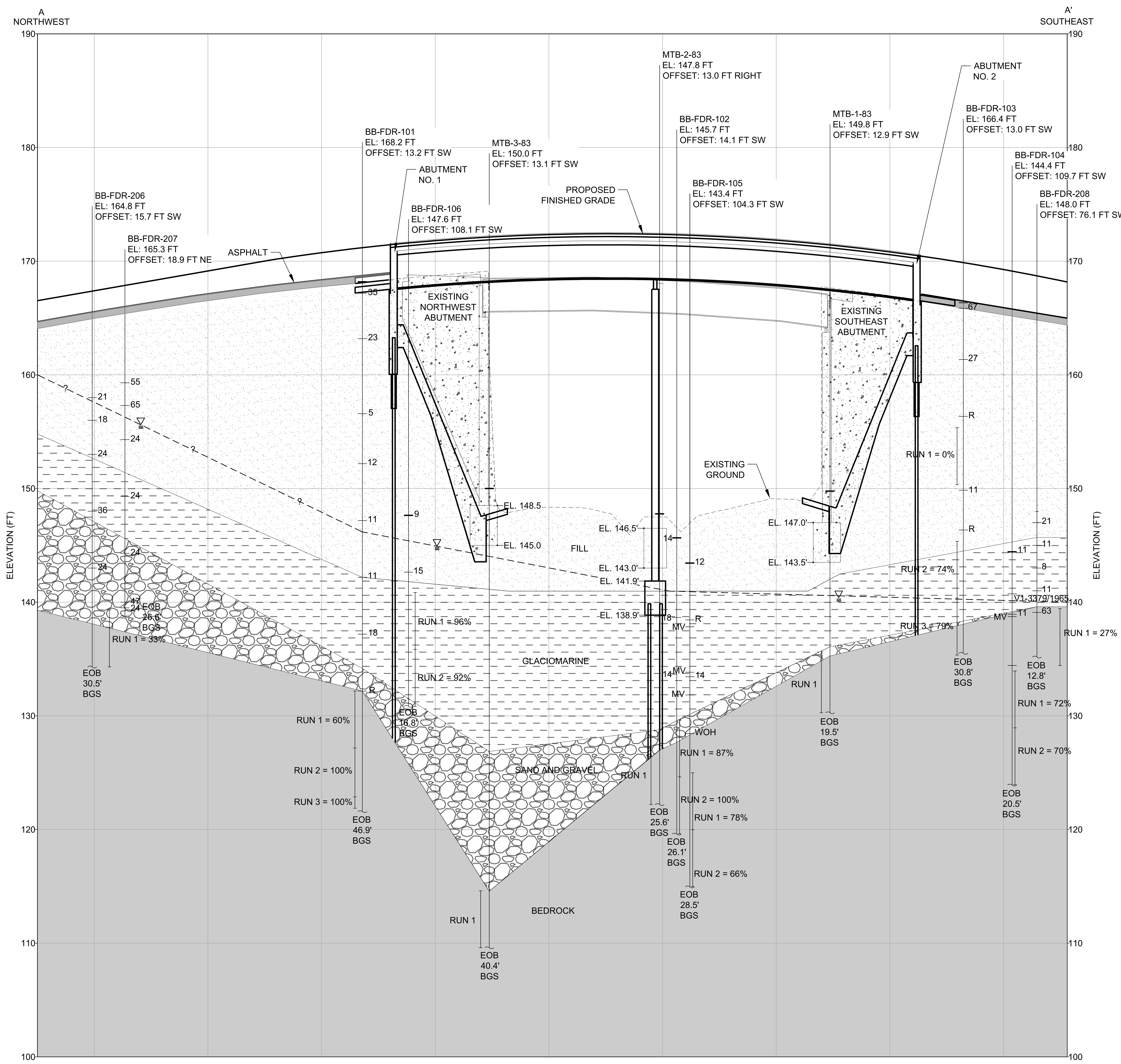
MERRILL ROAD BRIDGE
INTERSTATE 295
FREEPORT CUMBERLAND

BORING LOCATION PLAN

SHEET NUMBER

4

OF 6



LEGEND

BB-FDR-104 EL: 144.4 FT OFFSET: 96.6 S	BORING LOCATION I.D. ELEVATION OFFSET (FT) FROM SECTION LINE
(TOB)	TOP OF BORING OR PROBE
30	SPT: N60 - VALUE (CORRECTED FOR HAMMER EFFICIENCY)
V1 - 3379/1965	FIELD VANE SHEAR TEST LOCATION AND UNDRAINED SHEAR STRENGTH (PSF) (PEAK/REMOLDED)
WOH/WOR	WEIGHT OF HAMMER / WEIGHT OF ROD
MV	MISSED VANE
R	REFUSAL
RUN1 - 92%	ROCK CORE RUN NUMBER AND ROD QUALITY DESIGNATION (RQD)
EOB 60.6' BGS	END OF BORING DEPTH BELOW GROUND SURFACE (BGS)

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

NOTES:

1. AS DRILLED BORING LOCATION PLAN FOR 100-SERIES BORINGS DERIVED FROM ELECTRONIC FILE NAME: "BOR12-17-19edit.csv" PROVIDED TO GOLDER BY MAINE DEPARTMENT OF TRANSPORTATION ON 01/06/2020. AS DRILLED BORING LOCATION PLAN FOR 200-SERIES BORINGS DERIVED FROM ELECTRONIC FILE NAME: "23627 BORE 200 Series Compiled.csv" PROVIDED TO GOLDER BY MAINE DEPARTMENT OF TRANSPORTATION ON 06/28/2021.
2. FOR DETAILED LITHOLOGIC DESCRIPTIONS SEE BORING LOGS IN NOTE 8 APPENDIX A (100-SERIES BORINGS) AND NOTE 9 APPENDIX A (200-SERIES BORINGS).
3. FOR COMPLETE LABORATORY DATA SEE LABORATORY REPORTS IN NOTE 8 (100-SERIES BORINGS) AND NOTE 9 (200-SERIES BORINGS).
4. GROUNDWATER SURFACE IS INTERPRETED FROM LOCALIZED SURFACE WATER LEVELS AND MEASUREMENTS TAKEN DURING THE SUBSURFACE EXPLORATION PROGRAMS. FOR DETAILS ON THE SUBSURFACE EXPLORATION PROGRAMS, SEE NOTE 8 (100-SERIES BORINGS) AND NOTE 9 (200-SERIES BORINGS).
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 IN NOTE 8 APPENDIX A (100-SERIES BORINGS) AND NOTE 9 APPENDIX A (200-SERIES BORINGS).
6. PROPOSED ABUTMENT DETAILS INTERPRETED FROM ELECTRONIC FILE NAME "Freeport_023627_Exit_20_60%25_Plans" PROVIDED TO GOLDER BY HNTB ON JUNE 17, 2021.
7. FOR SOIL STRATA ANALYSIS THE ASPHALT LAYER AND ROADFILL LAYER ARE COMBINED.
8. GOLDER ASSOCIATES, INC., DECEMBER 21, 2020, PRELIMINARY GEOTECHNICAL DESIGN REPORT, I-295 DESERT ROAD BRIDGE REPLACEMENT #5720 (EXIT 20), FREEPORT, MAINE, MAINEDOT WIN 023627.00.
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VERTICAL SCALE 0 5 10 1" = 5' FEET

HORIZONTAL SCALE 0 25 50 1" = 25' FEET

GOLDER
MEMBER OF WSP

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

023627.00

WIN
023627.00

BRIDGE PLANS

Bridge No. 5720

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

SIGNATURE

P.E. NUMBER

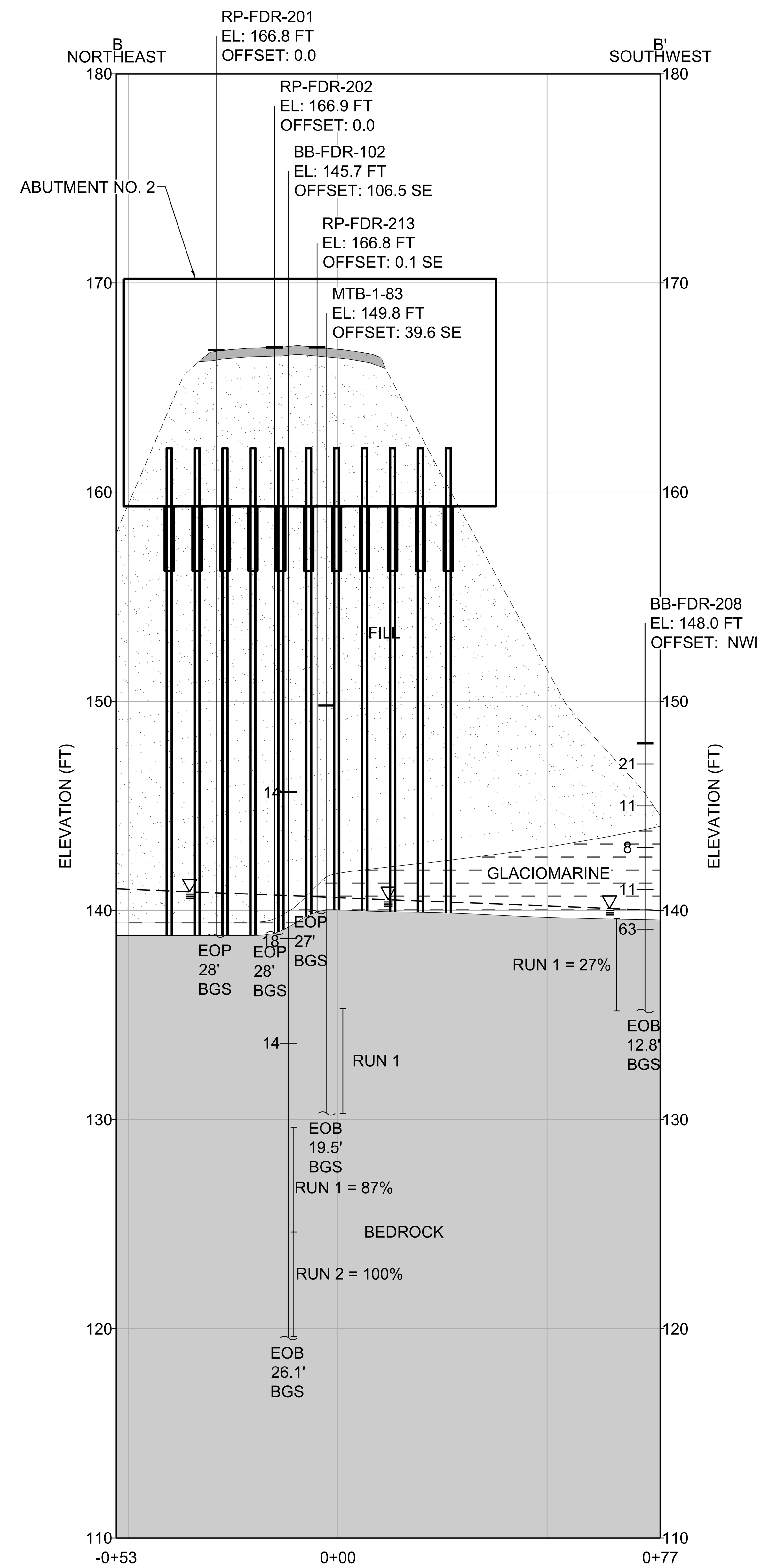
DATE

	BY	DATE
PROJ. MANAGER	MEL	
DESIGN-DETAILED	MEL	AAZ	2021/08/20
CHECKED-REVIEWED	CCB	AAZ	2021/08/20
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

MERRILL ROAD BRIDGE
INTERSTATE 295
FREEPORT CUMBERLAND

INTERPRETIVE SUBSURFACE
PROFILE A-A'

SHEET NUMBER
5
OF 6



LEGEND

RP-FDR-211	ROCK PROBE LOCATION I.D.
BB-FDR-104	BORING LOCATION I.D.
EL: 144.4 FT	ELEVATION
OFFSET: 96.6 S	OFFSET (FT) FROM SECTION LINE
(TOB)	TOP OF BORING OR PROBE
30	SPT: N60 - VALUE (CORRECTED FOR HAMMER EFFICIENCY)
V1 - 3379/1965	FIELD VANE SHEAR TEST LOCATION AND UNDRAINED SHEAR STRENGTH (PSF) (PEAK/REMOLDED)
WOR	WEIGHT OF ROD
MV	MISSED VANE
R	REFUSAL
RUN1 - 92%	ROCK CORE RUN NUMBER AND ROD QUALITY DESIGNATION (RQD)
EOB / EOP 60.6' BGS	END OF BORING / END OF PROBE DEPTH BELOW GROUND SURFACE (BGS)

ASPHALT

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)

LIGHT GRAY, COARSE-GRAINED GNEISS, EXTREMELY STRONG, FRESH TO SLIGHTLY WEATHERED (BEDROCK)

EXISTING GROUND SURFACE

INTERPRETED GROUNDWATER SURFACE

NOTES:

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2. FOR DETAILED LITHOLOGIC DESCRIPTIONS SEE BORING LOGS IN NOTE 8 APPENDIX A (100-SERIES BORINGS) AND NOTE 9 APPENDIX A (200-SERIES BORINGS).
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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 IN NOTE 8 APPENDIX A (100-SERIES BORINGS) AND NOTE 9 APPENDIX A (200-SERIES BORINGS).
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HORIZONTAL SCALE 0 25 50 1" = 25' FEET

GOLDER
MEMBER OF WSP

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

023627.00

WIN 023627.00

Bridge No. 5720

BRIDGE PLANS

	MEL	BY	DATE
PROJ. MANAGER			
DESIGN-DETAILED	KAR	AAZ	2021/08/20
CHECKED-REVIEWED	CCB	AAZ	2021/08/20
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

MERRILL ROAD BRIDGE
INTERSTATE 295
FREEPORT CUMBERLAND

INTERPRETIVE SUBSURFACE
CROSS SECTION B-B'

SHEET NUMBER
6
OF 6

Date:2021-7-2
Username:
Division:
Filename: 21450908_0120_005.dwg

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

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))

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

Sample Container Labeling Requirements:

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

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

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

Boring No.: BB-FDR-101

WIN: 023627.00

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

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

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

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

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

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

Remarks:

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

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

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

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** I-295 Merrill Road Bridge Replacement #5720 (Exit 20) **Location:** Freeport, Maine	**Boring No.:** BB-FDR-101 **WIN:** 023627.00

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

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25							21			Asphalt in wash water from 24-25 ft bgs.	
	6D	24/14	26.00 - 28.00	5/4/3/6	7	11	31	142.2		26.0 6DB, Top 6.5 in: Grey, wet, stiff, Sandy CLAY, trace fine gravel, slightly plastic (GLACIOMARINE). 6DA, Bottom 7.5 in: Grey, wet, stiff, CLAY, trace fine to medium sand, moderately plastic. q_p = 5.0 ksf (Pocket Pentrometer), T_v = 500, 600 psf (GLACIOMARINE). Wash color changes from brown to grey at 26 ft bgs.	6DA: GTX #539975, 539957 WC = 26% Fines = 93.6% LL = 40 PL = 21 PI = 19 LI = 0.3 A-6 (19), CL
							60				
							93				
							131				
30							103				
	7D	24/24	31.00 - 33.00	7/4/8/10	12	18	123			Grey, wet, very stiff, CLAY, trace fine sand, slightly plastic (GLACIOMARINE).	7D: GTX #539958 WC = 23% LL = 24 PL = 16 PI = 8 LI = 0.9 A-4 , CL
							127				
							102				
							126	134.2		34.0 Drill rig shaking at 34 ft bgs.	
35							Open			Grey clay and sand in wash water at 35 ft bgs.	
	8D	7/7	36.00 - 36.58	16/50(1")			~~NQ~~	131.6		Grey, wet, very dense, GRAVEL, some silt/clay, little fine to coarse sand, slightly plastic (SAND AND GRAVEL).	8D: GTX #539967, 539959 WC = 18% Fines = 31.5% LL = 19 PL = 13 PI = 6 LI = 0.8 A-2-4 (0), GC-GM
	R1	60/43.5	36.60 - 41.60	RQD = 60%						36.6 Top of Bedrock at Elev. 131.6 ft. R1: Grey, coarse grained, strongly foliated, fresh (W1), very strong (R5), GNEISS; discontinuities steep (55°-75°) and parallel to foliation, closely spaced (0.2-0.8 ft) [VASSALBORO FORMATION]. Rock Mass Quality = Fair Core Times (min:sec) 36.6-37.6 ft (2:06) 37.6-38.6 ft (1:47) 38.6-39.6 ft (1:57) 39.6-40.6 ft (1:19) 40.6-41.6 ft (1:30) 73% Recovery Driller notes void in core run R1	
40											
	R2	51.6/59	41.60 - 45.90	RQD = 100%						R2: Blue/grey, coarse grained, strongly foliated, fresh (W1), very strong (R5), GNEISS with PEGMATITE veins; discontinuities steep (70°-80°) and parallel to foliation, very closely to moderately closely spaced (0.1-1.5 ft) [VASSALBORO FORMATION]. Rock Mass Quality = Excellent Core Times (min:sec) 41.6-42.6 ft (3:34)	
45	R3	12/14	45.90 - 46.90	RQD = 100%							
								121.3			
50											

Remarks:

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

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

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

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

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

Boring No.: BB-FDR-101

WIN: 023627.00

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

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50										42.6-43.6 ft (1:29) 43.6-44.6 ft (1:22) 44.6-45.6 ft (1:47) 45.6-45.9 ft (1:19) 114% Recovery Recovered a portion of R1 with R2. R3: Blue/grey, coarse grained, strongly foliated, fresh (W1), very strong (R5), GNEISS; discontinuities near vertical (85°) and parallel to foliation, closely spaced (0.4 ft) [VASSALBORO FORMATION]. Rock Mass Quality = Excellent Core Times (min:sec) 45.9-46.9 ft (1:42) 117% Recovery. Recovered a portion of R2 with R3.	
55											
										46.9	
										Bottom of Exploration at 46.9 feet below ground surface. Boring backfilled with bentonite grout to 10 ft bgs, bentonite chips from 10-8 ft bgs, some sand, gravel from 8-1 ft bgs and cold patch asphalt from 1 ft bgs to road surface.	
60											
65											
70											
75											

Remarks:

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

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

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

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** I-295 Merrill Road Bridge Replacement #5720 (Exit 20) **Location:** Freeport, Maine	**Boring No.:** BB-FDR-102 **WIN:** 023627.00

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

Definitions:

D = Split Spoon Sample	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	1D	24/10	0.00 - 2.00	WOR(3"),1(3")/4/5/6	9	14	Open	145.4		1DB, Top: 3.5 in: brown, moist, stiff, SILT, some fine to coarse sand, some organics, non-plastic (TOPSOIL).	WC = 14.9% 1DA:
										0.3 1DA, Bottom 6.5 in: Brown, moist, medium dense, fine to coarse SAND, little silt, trace fine gravel, well-graded (FILL). Brown sand in wash water at 2 ft bgs.	GTX #539968 Fines = 12.1% A-1-b (0), SM
							23				
							28				
							20				
5							23				
							35	139.7		6.0 Brown clay and gravel in wash water at 6 ft bgs.	
	2D	24/24	7.00 - 9.00	3/5/7/10	12	18	54			Wash water color changes to grey at 7 ft bgs; clay in wash water at 7 ft bgs.	GTX #539993 WC = 22% LL = 34 PL = 21 PI = 13 LI = 0.1 A-6, CL
							80				
							97			Grey, dry, very stiff, CLAY, little silt, moderately plastic (GLACIOMARINE). q_p = 8.0, 7.0 ksf (Pocket Pentrometer), T_v = 500, 600, 500 psf.	
10							105				
							123				
	3D	24/4	12.00 - 14.00	4/6/3/5	9	14	71			Grey, wet, stiff, Silty CLAY, slightly plastic (GLACIOMARINE).	GTX #539961 WC = 22 LL = 22 PL = 15 PI = 7 LI = 1.1 A-4, CL-ML
							62				
							75				
15							Open				
	R1	60/54	16.10 - 21.10	RQD = 87%			NQ	129.6		16.1 Top of Bedrock at Elev. 129.6 ft. R1: Blue/grey, coarse grained, strongly foliated, fresh (W1), very strong (R5), GNEISS; discontinuities steep (65°-85°) and parallel to foliation, closely to moderately closely spaced (0.5-1.6 ft) [VASSALBORO FORMATION]. Rock Mass Quality = Good Core Times (min:sec) 16.1-17.1 ft (3:25) 17.1-18.1 ft (2:07) 18.1-19.1 ft (1:14) 19.1-10.1 ft (1:28) 20.1-21.1 ft (1:42) 90% Recovery	GTX #311185 q_p = 2903 ksf (128.9-129.3 ft)
20											
	R2	60/60	21.10 - 26.10	RQD = 100%							
25											

Remarks:

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

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

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

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** I-295 Merrill Road Bridge Replacement #5720 (Exit 20) **Location:** Freeport, Maine	**Boring No.:** BB-FDR-102 **WIN:** 023627.00

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

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

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

Remarks:

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

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

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

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

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

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

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

Remarks:

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

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

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

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

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

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

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

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

Remarks:

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

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

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

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

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

Definitions:

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.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/21.5	0.00 - 2.00	3/4/3/1	7	11	SSA		Brown and grey, dry, stiff, CLAY, trace organics, moderately plastic (GLACIOMARINE). q_p = 4.50, 4.00 ksf (Pocket Penetrometer), T_v = 700, 800 psf.	WC = 21.7%
	V1			S_u=3379/1965psf					16x32 mm vane raw torque readings (3.7 ft bgs): V1: 21.5/12.5 in-lbs	
5	2D	24/24	5.50 - 7.50	2/4/3/4	7	11	1		Brown, wet, stiff, CLAY, little fine sand, moderately plastic (GLACIOMARINE). q_p = 7.0, 7.0 ksf (Pocket Penetrometer), T_v = 1000, 1000 psf.	GTX #539963 WC = 28% LL = 31 PL = 17 PI = 14 LI = 0.8 A-6, CL
	MV			Would Not Push			11		Failed 16x32 mm vane, would not push to 5.8 ft bgs.	
							12			
							16	135.4	Sand and silt in wash water 7.5-9 ft bgs.	
							27		9.0	
10	3D	3/3	10.00 - 10.25	50(3")			OPEN	134.1	Gravel and sand in wash water 9-10 ft bgs.	
	R1	60/58	10.50 - 15.50	RQD = 72%			NQ		Brown to grey, wet, very dense, fine to medium SAND, some silt, little gravel, well-graded (SAND AND GRAVEL). 10.3	
									Top of Bedrock at Elev. 134.1 ft.	q_p = 1141 ksf (132.6-132.9 ft)
									R1: Grey, medium-grained, moderately to strongly foliated, fresh (W1), strong (R4), SCHIST; intermingled with white and black, coarse-grained, moderately to strongly foliated, fresh (W1), very strong (R5), GNEISS; discontinuities have shallow dips (20°-30°), very close to moderately closely spaced (0.1-1.3 ft) [VASSALBORO FORMATION]. Rock Mass Quality = Fair Rock Core Rate (min:sec) 10.5-11.5 ft (3:05) 11.5-12.5 ft (2:27) 12.5-13.5 ft (2:48) 13.5-14.5 ft (2:28) 14.5-15.5 ft (2:50) 96% Recovery	
15	R2	60/53	15.50 - 20.50	RQD = 70%						
20								123.9	R2, Top 3.2 ft: Grey, medium-grained, strongly foliated, fresh (W1), very strong (R5), SCHIST; discontinuities have shallow dips (20°-30°), very close to closely spaced (0.1-1.0 ft) [VASSALBORO FORMATION].	
25									R2, Bottom 1.2 ft: White and black, coarse-grained, moderately	

Remarks:

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

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

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

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

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

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

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

Remarks:

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

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

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

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

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

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/15	0.00 - 2.00	1/2/6/7	8	12	SSA	142.8		1DA, Top 7 in: Medium brown, damp (frozen), stiff, SILT, little sand, trace organics (grass roots), slightly plastic (TOPSOIL). 0.6	
										1DB, Bottom 8 in: Light brown, moist, medium dense, fine to coarse SAND, trace fine gravel, trace silt, poorly-graded (FILL).	1DB: GTX #539972 WC = 12.6% Fines = 3.7% A-1-b (0), SP
								139.4		4.0	
										Auger cuttings 4-5 ft bgs are grey CLAY, slightly to moderately plastic.	
5	2D	18/15	5.00 - 6.50	8/53/112(6")			29			Driller notes hole is caving at 5 ft bgs.	
	MV			Would Not Push			59			2DA, Top 9 in: Light brown, wet, very dense, fine to coarse SAND, trace fine gravel, well-graded (GLACIOMARINE).	2DB: GTX #539964 WC = 23% LL = 35 PL = 19 PI = 16 LI = 0.3 A-6, CL
							76			2DB, Bottom 6 in: Alternating layers 1-2" thick of greyish brown, moist, hard, CLAY, moderately plastic; and light brown, wet, very dense, fine to coarse SAND, trace fine gravel, well-graded	
							88			(GLACIOMARINE). q_p = 6.5 ksf (Pocket Penetrometer).	
							107			Failed 55x110 mm vane, would not push to 6 ft bgs.	
10	3D	24/17	10.00 - 12.00	2/4/5/4	9	14	OPEN			Clay and sand in wash water 8-10 ft bgs.	
	MV			Would Not Push						3DA, Top 13 in: Grey, wet, stiff, CLAY, slightly plastic (GLACIOMARINE). q_p = 3.0 ksf (Pocket Penetrometer)	3DB: GTX #539977 WC = 22.2% Fines = 70.9% A-4, ML
										3DB, Bottom 4 in: Grey, wet, stiff, SILT, some fine sand (GLACIOMARINE).	
	MV			Would Not Push						Failed 55x110 mm vane, would not push to 11 ft bgs.	
										Failed 55x110 mm vane, would not push to 13 ft bgs.	
15	4D	24/11	15.00 - 17.00	WOH(24")	- -					Grey, wet, very soft, CLAY, trace fine gravel in top 2 in, trace fine sand in top 2 in, moderately plastic (GLACIOMARINE). q_p = < 1.0 ksf (Pocket Penetrometer), T_v = 300 psf	GTX #539994 WC = 30% LL = 27 PL = 16 PI = 11 LI = 1.3 A-6, CL
								125.4		18.0	
	R1	60/56	18.50 - 23.50	RQD = 78%			NQ			Rig starts chattering at 18 ft bgs; driller notes hard material. Top of Bedrock at Elev. 125.4 ft.	
20										R1: Grey, medium-grained, moderately foliated, slightly weathered (top 1.4 ft) to fresh (bottom 3.3 ft) (W1-W2), very strong (R5), biotite SCHIST; discontinuities shallow to steeply dipping (20°-80°), very close to closely spaced (0.1-1.0 ft), slight reddish staining on discontinuity surfaces in top 1.4 ft [VASSALBORO FORMATION]. Rock Mass Quality = Good	
	R2	60/60	23.50 - 28.50	RQD = 66%						Rock Core Rate (min:sec) 18.5-19.5 ft (5:10) 19.5-20.5 ft (2:28) 20.5-21.5 ft (3:20)	q_p = 2279 ksf (119.5-119.9
25											

Remarks:

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

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

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

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

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

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample Attempt
V = Field Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Field Vane Shear Test Attempt
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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										21.5-22.5 ft (3:00) 22.5-23.5 ft (2:49) 94% Recovery R2: Grey, medium-grained, moderately foliated, fresh (W1), very strong (R5), biotite SCHIST; discontinuities shallow to steeply dipping (10°-60°) with vertical discontinuity at 27.9-28.3 ft bgs, very close to closely spaced (0.1- 0.8 ft) [VASSALBORO FORMATION]. Rock Mass Quality = Fair Rock Core Rate (min:sec) 23.5-24.5 ft (1:26) 24.5-25.5 ft (1:19) 25.5-26.5 ft (2:01) 26.5-28.5 ft (3:23) 100% Recovery	ft)
								114.9		28.5	
30											
										Bottom of Exploration at 28.5 feet below ground surface. Boring backfilled with cuttings to 3.5 ft bgs then with gravel and bentonite chips to ground surface.	
35											
40											
45											
50											

Remarks:

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

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

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

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

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

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

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample Attempt
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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.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/16	0.00 - 2.00	WOH/2/4/16	6	9	SSA		Medium brown, damp (frozen), stiff, fine to coarse Sandy SILT, trace fine gravel, trace organics (grass roots), non-plastic (TOPSOIL).	1D: GTX #539973 Fines = 50.6% A-4, ML
								145.6	2.0	
									Auger cuttings 2-5 ft bgs are brown, damp, SILT, little sand, trace gravel, slightly plastic.	
5	2D	21/20	5.00 - 6.75	1/4/6/50(3")	10	15	OPEN	142.3	2DA, Top 3 in: Greyish brown, moist, stiff, SILT, little sand, slightly plastic (GLACIOMARINE). q_p = 3.0 ksf (Pocket Penetrometer) 5.3	2DB: GTX #539974 WC = 16.7% Fines = 35.1% A-4 (0), SM
	R1	60/58	6.80 - 11.80	RQD = 96%			NQ	140.9	2DB, Bottom 17 in: Greyish brown, wet, medium dense, Silty fine to coarse SAND, trace fine gravel, well-graded (SAND AND GRAVEL). 6.7	
									Driller notes 0.3 ft of water in bottom of hole upon refusal. Top of Bedrock at Elev. 140.9 ft. R1: Black and light grey/white, coarse grained, strongly foliated, fresh (W1), strong (R4), GNEISS; discontinuities have shallow dips (10°-20°) and parallel to foliation, close to moderately closely spaced (0.5-2.5 ft) [VASSALBORO FORMATION]. Rock Mass Quality = Excellent Rock Core Rate (min:sec) 6.8-7.8 ft (6:07) 7.8-8.8 ft (3:58) 8.8-9.8 ft (3:39) 9.8-10.8 ft (3:47) 10.8-11.8 ft (4:05) 96% Recovery	q_p = 1155 ksf (140.5-140.8 ft)
10										
	R2	60/58	11.80 - 16.80	RQD = 92%						
15										
								130.8	R2: Black and light grey/white, coarse grained, strongly foliated, fresh (W1), very strong (R5), GNEISS; discontinuities horizontal to shallow (0°-10°) and parallel to foliation, close to moderately closely spaced (0.2-2.3 ft) [VASSALBORO FORMATION]. Rock Core Rate (min:sec) 11.8-12.8 ft (4:20) 12.8-13.8 ft (4:28) 13.8-14.8 ft (4:12) 14.8-15.8 ft (4:01) 15.8-16.8 ft (4:07) 96% Recovery 16.8	
20										
									Bottom of Exploration at 16.8 feet below ground surface. Boring backfilled with rock cuttings to 14 ft bgs, gravel to 6 ft bgs, soil cuttings to 2.3 ft bgs, and bentonite chips to ground surface.	
25										

Remarks:

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

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

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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: I-295 Desert Road Bridge Replacement #5720 (Exit 20) Location: Freeport, Maine	Boring No.: BB-FDR-206 WIN: 023627.00

Driller:	SW Cole	**Elevation (ft.)**	164.8 ft	**Auger ID/OD:**	2.5 in
Operator:	J. Layfield	**Datum:**	NAD83 (2011) Maine 2000 West	**Sampler:**	Standard Split Spoon
Logged By:	C. Battistella	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	5/24/21 (11:30) ; 5/24/21 (14:20)	**Drilling Method:**	Pin Auger / Cased Wash	**Core Barrel:**	NQ
Boring Location:	N 368334.079 ft, E 1051070.308 ft	**Casing ID/OD:**	5.5 in	**Water Level*:**	No measurement
Hammer Efficiency Factor:	0.974	**Hammer Type:**	Automatic ☐ Hydraulic ☒ Rope & Cathead ☐		

Definitions:
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MD = Unsuccessful Split Spoon Sample Attempt
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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							DAYL	164.0		Asphalt thickness of 10 in ——0.8	
										Excavated to 5 ft bgs, no sampling No samples from 0 to 5 ft bgs because all disturbed from pot holing Pushing 4 in casing to 4.9 ft bgs washing out to 5 ft bgs	
5	1D	24/10	5.00 - 7.00	8/7/6/7	13	21	10			Brown, wet (from wash out), medium dense, fine to coarse SAND, trace fine gravel, well-graded (FILL)	
							11				
	2D	24/15	7.00 - 9.00	8/6/5/6	11	18	24			Brown, wet (from wash out), medium dense, fine to coarse SAND, trace fine gravel, trace silt, well-graded (FILL)	
							21				
							76				
10	3D	24/10	10.00 - 12.00	7/8/7/7	15	24	28	154.8		——10.0 Brown, wet (from wash out), medium dense, fine to medium Silty SAND, well-graded, layering visible (SAND AND GRAVEL)	
							37				
							70				
							82				
							75				
15	4D	24/12	15.00 - 17.00	8/9/13/18	22	36	34			Brown, wet (from wash out), dense, fine to coarse SAND, trace silt, well-graded (SAND AND GRAVEL)	GTX #620969 WC = 9.8% Fines = 9.2% A-3, SW-SM
							62				
							110				
							81				
							72				
20	5D	24/12	20.00 - 22.00	4/4/11/8	15	24	33			5DA, Top 8 in: Brown, wet (from wash out), medium dense, fine to coarse SAND, trace silt, well-graded (SAND AND GRAVEL) 5DB, Bottom 4 in: Grey, wet (from wash out), medium dense, fine to coarse SAND, little silt/clay, trace fine gravel (SAND AND GRAVEL)	5DB: GTX #620970 WC = 11.0% Fines = 19.7% A-1-b, SM-SC
							87				
							84			Weathered rock, coarse sand, and gravel in wash from 23.9 to 25.5 ft bgs.	
							250 202			Unweathered rock at 25.5 ft bgs	
25											

Remarks:

Hammer calibration obtained from SW Cole calibration report.
DAYL as a advancement method indicates that hole was excavated by non-destructive means to confirm no utilities present.
As-drilled elevations are derived from survey files "23627.00 20 BORE 20o Series Compiled.csv" received by Golder on June 11, 2021 from MaineDOT.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

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

Boring No.: BB-FDR-206
WIN: 023627.00

Driller:	SW Cole	**Elevation (ft.)**	164.8 ft	**Auger ID/OD:**	2.5 in
Operator:	J. Layfield	**Datum:**	NAD83 (2011) Maine 2000 West	**Sampler:**	Standard Split Spoon
Logged By:	C. Battistella	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	5/24/21 (11:30) ; 5/24/21 (14:20)	**Drilling Method:**	Pin Auger / Cased Wash	**Core Barrel:**	NQ
Boring Location:	N 368334.079 ft, E 1051070.308 ft	**Casing ID/OD:**	5.5 in	**Water Level*:**	No measurement
Hammer Efficiency Factor:	0.974	**Hammer Type:**	Automatic ☐ Hydraulic ☒ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	R1	60/58	25.50 - 30.50	RQD = 33%			NQ	139.3		Top of bedrock at elev. 139.3 ft. (25.5) Grey, medium to coarse grained, moderately foliated, GNEISS; slightly weathered (W2), very strong (R5), discontinuities low angle to moderate (10-40 deg), very closely to closely spaced (0.1-0.55 ft), consists of coarse grained quartz bands [VASSALBORO FORMATION]. Rock Mass Quality: Poor Rock Core Rate (min:sec) 25.5-26.5 ft (2:50) 26.5-27.5 ft (2:33) 27.5-28.5 ft (3:24) 28.5-29.5 ft (3:59) 29.5-30.5 ft (4:12) 96% Recovery	
30								134.3		30.5 **Bottom of Exploration at 30.5 feet below ground surface.** Boring backfilled with cuttings to ground surface.	
35											
40											
45											
50											

Remarks:

Hammer calibration obtained from SW Cole calibration report.
DAYL as a advancement method indicates that hole was excavated by non-destructive means to confirm no utilities present.
As-drilled elevations are derived from survey files "23627.00 20 BORE 20o Series Compiled.csv" received by Golder on June 11, 2021 from MaineDOT.

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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** I-295 Desert Road Bridge Replacement #5720 (Exit 20) **Location:** Freeport, Maine	**Boring No.:** BB-FDR-207 **WIN:** 023627.00

Driller:	SW Cole	**Elevation (ft.)**	165.3 ft	**Auger ID/OD:**	2.5 in
Operator:	J. Layfield	**Datum:**	NAD83 (2011) Maine 2000 West	**Sampler:**	Standard Split Spoon
Logged By:	C. Battistella	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	05/25/21 (13:30);05/25/21 (14:45)	**Drilling Method:**	Pin Auger / Cased Wash	**Core Barrel:**	NQ
Boring Location:	N 368357.075 ft, E 1051099.843 ft	**Casing ID/OD:**	5.5 in	**Water Level*:**	No measurement
Hammer Efficiency Factor: 0.974		**Hammer Type:**	Automatic ☐ Hydraulic ☒ Rope & Cathead ☐		

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SSA = Solid Stem Auger
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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							PUSH			Asphalt thickness of 12 in (ASPHALT)	
								164.3		1.0	
										Excavated down to 5 ft bgs, no sampling	
5	1D	24/11	5.00 - 7.00	14/17/17/18	34	55				Brown, moist to damp, very dense, fine to coarse SAND, little silt, trace fine gravel, well-graded (FILL)	GTX #620971 WC = 6.5% Fines = 10.7% A-2-4, SM-SP
	2D	24/10	7.00 - 9.00	22/20/20/20	40	65				Brown, moist, very dense, fine to coarse SAND, little silt, little fine gravel, well-graded (FILL)	
							3				
10	3D	24/8	10.00 - 12.00	6/7/8/4	15	24	37	155.3		10.0 Brown, moist, medium dense, fine to coarse SAND, little fine gravel, well-graded (SAND AND GRAVEL)	
							48				
							59				
							67				
							97				
15	4D	24/8	15.00 - 17.00	8/8/7/5	15	24	OPEN			Brown, moist, medium dense, fine to coarse SAND, little fine gravel, well-graded (SAND AND GRAVEL)	
20	5D	24/8	20.00 - 22.00	19/17/12/4	29	47				Brown and grey, moist, dense, fine to coarse SAND, some fine gravel, trace silt, well-graded, weathered bedrock suspected present (SAND AND GRAVEL)	GTX #620972 WC = 8.3% Fines = 9.6% A-1-b, SM-SC
								140.8		24.5	
25											

Remarks:

Hammer calibration obtained from SW Cole calibration report.
As-drilled elevations are derived from survey files "23627.00 20 BORE 20o Series Compiled.csv" received by Golder on June 11, 2021 from MaineDOT.

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Boring No.: BB-FDR-207

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

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

Boring No.: BB-FDR-207
WIN: 023627.00

Driller:	SW Cole	**Elevation (ft.)**	165.3 ft	**Auger ID/OD:**	2.5 in
Operator:	J. Layfield	**Datum:**	NAD83 (2011) Maine 2000 West	**Sampler:**	Standard Split Spoon
Logged By:	C. Battistella	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	05/25/21 (13:30);05/25/21 (14:45)	**Drilling Method:**	Pin Auger / Cased Wash	**Core Barrel:**	NQ
Boring Location:	N 368357.075 ft, E 1051099.843 ft	**Casing ID/OD:**	5.5 in	**Water Level*:**	No measurement
Hammer Efficiency Factor:	0.974	**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
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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	6D	19.2/18	25.00 - 26.60	4/6/9/50(0.1")	15	24				Clay and other organics present in wash (GLACIOMARINE) 6D: Brown and grey, wet, stiff, SILT, little sand, poorly-graded, low plasticity (GLACIOMARINE)	GTX #620973 WC = 28.0% Fines = 80.0% A-4, ML-CL
								138.7		26.6 **Bottom of Exploration at 26.6 feet below ground surface.** Boring backfilled with cuttings to ground surface.	
30											
35											
40											
45											
50											

Remarks:

Hammer calibration obtained from SW Cole calibration report.
As-drilled elevations are derived from survey files "23627.00 20 BORE 20o Series Compiled.csv" received by Golder on June 11, 2021 from MaineDOT.

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

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

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

Boring No.: BB-FDR-208
WIN: 023627.00

Driller:	SW Cole	**Elevation (ft.)**	148.0 ft	**Auger ID/OD:**	2.5 in
Operator:	J. Layfield	**Datum:**	NAD83 (2011) Maine 2000 West	**Sampler:**	Standard Split Spoon
Logged By:	C. Battistella	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	5/12/21 (13:50) ; 5/13/21 (09:20)	**Drilling Method:**	Pin Auger / Cased Wash	**Core Barrel:**	NQ
Boring Location:	N 368077.192 ft, E 1051402.965 ft	**Casing ID/OD:**	5.5 in	**Water Level*:**	8.0 ft
Hammer Efficiency Factor:	0.974	**Hammer Type:**	Automatic ☐ Hydraulic ☒ Rope & Cathead ☐		

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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	24/11	0.00 - 2.00	3/7/6/2	13	21	PUSH			Brown, moist, medium dense, fine to coarse SAND, little gravel, trace silt, well-graded (FILL)	
	2D	24/20	2.00 - 4.00	4/4/3/4	7	11		145.8		2DA, Top 3 in: Brown, moist, medium dense, fine to coarse SAND, little gravel, trace silt, well-graded (FILL) —2.3— 2DB, Bottom 17 in: Brown, moist, medium stiff, Silty CLAY, some fine to medium sand, low plasticity, q_p = 3.6, 3. 6 ksf (Pocket Penetrometer)(GLACIOMARINE)	
5	3D	24/10	4.00 - 6.00	2/2/3/6	5	8				3D: Grey and brown, moist, soft, Silty CLAY, some fine to medium sand, trace gravel, low plasticity, q_p = 5.1 ksf (Pocket Penetrometer)(GLACIOMARINE)	3D: GTX #620980 WC = 27.0% Fines = 78.9% A-4, CL
	4D	24/22	6.00 - 8.00	4/4/3/6	7	11	51			4D: Grey and brown, moist, medium stiff, Silty CLAY, some fine to medium sand, low plasticity, q_p = 4.1 ksf (Pocket Penetrometer)(GLACIOMARINE)	4D: GTX #620986, 620958 WC = 26.7% Fines = 78.3% A-4, CL LL = 29 PL = 20 PI = 9 LI = 0.7
							172	140.0		—8.0—	
	5D	21/24	8.00 - 9.75	12/13/26/50(3")	39	63	200	139.6		5D: Brown, wet, very dense, fine to coarse SAND, some gravel, little silt, well-graded, highly weathered rock in shoe (SAND AND GRAVEL) —8.4—	
10	~~R1~~	~~53/46~~	~~8.40 - 12.82~~	~~RQD = 27%~~			NQ			Top of bedrock at elev. 139.6 ft. 8.4-8.9 ft: Grey, medium to coarse grained, moderately foliated, GNEISS; slightly weathered (W2), strong (R4), discontinuities low angle to vertical (5-88 deg), very closely spaced (0.15 ft) [VASSALBORO FORMATION]. 8.9-12.2 ft: Dark grey, fine grained, dolomitic LIMESTONE; fresh (W1), extremely strong (R6), discontinuities low angle to steep (5-70 deg), closely spaced (0.2-0.45 ft) [VASSALBORO FORMATION]. Rock Mass Quality: Poor Rock Core Rate (min:sec) 8.4 to 9.4 ft (2:22) 9.4 to 9.8 ft (5:22) 9.8 to 10.8 (4:36) 10.8 to 11.8 (3:43) 11.8 to 12.8 (3:01) 86% Recovery	
								135.2		—12.8—	
15											
20										**Bottom of Exploration at 12.8 feet below ground surface.** Boring backfilled with cuttings to ground surface.	
25											

Remarks:

Hammer calibration obtained from SW Cole calibration report.
As-drilled elevations are derived from survey files "23627.00 20 BORE 20o Series Compiled.csv" received by Golder on June 11, 2021 from 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.

APPENDIX B

Stability

GOLDER

CALCULATIONS

Date:	7/2/2021	**Made by:**	AH
Project No.:	21450908	**Checked by:**	KAR/MEL
Subject:	Global Stability Analysis Abutments No. 1 and No. 2	**Reviewed by:**	JEL
Project Short Title:	MaineDOT I-295 Exit 20 Desert Road Bridge Replacement No. 5720		

OBJECTIVE

Calculate global factor of safety for Abutment No. 1 (northwestern) and Abutment No. 2 (southeastern).

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 9th Ed, 2020.
2. Email communication between Golder and HNTB, subject "RE: Freeport I-295 Exit 20 and Exit 22 - Request for Sections and Design Info", dated June 22, 2020.
3. Golder geotechnical test boring logs for 100-series borings (Appendix A, Preliminary Geotechnical Design Report, dated December 21, 2020).
4. HNTB for State of Maine Department of Transportation. Merrill Road Bridge over Interstate 295 and Signalized Intersections, Exit 20 Interchange: 60% Plans, dated May 21, 2021.
5. Das, Braja M. 2011. Principles of Foundation Engineering, 7th Edition. Cengage Learning.
6. FHWA. 2017. Geotechnical Engineering Circular No. 5: Geotechnical Site Characterization. Publication No. FHWA NHI-16-072.
7. GeoTesting Express laboratory testing results, dated February 5, 2020 (Appendix C, Preliminary Geotechnical Design Report, dated December 21, 2020).
8. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.
9. Golder calculation titled "Seismic Site Class" (Appendix E, Preliminary Geotechnical Design Report, dated December 21, 2020).
10. Rocscience Slide Software Package Version 2020 9.010 64-bit, build date Oct 14, 2020.
11. Golder geotechnical test boring logs for 200-series borings (Supplemental Geotechnical Design Report Part 2, dated July 2020).
12. FHWA. 2011. Geotechnical Engineering Circular No. 3 - LRFD Seismic Analysis and Design of Transportation Geotechnical Features and Structural Foundations Reference Manual, Publication No. FHWA-NHI-11-032

ATTACHMENTS

1. Slide output figures.
2. HNTB 60% plans.

ASSUMPTIONS

1. The load applied by the road and traffic for final design conditions is modeled as a 2 ft equivalent load of soil (Reference 1, Table 3.11.6.4-1) based on a 11 ft abutment height (Reference 2). 2 ft x 125 pcf (fill) = 250 psf.
2. A static FS ≥ 1.5 is recommended for abutment final design conditions per Section 5.9.2 in Reference 8. A pseudo-static FS > 1.1 is recommended in Reference 12.
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 References 3 and 11.
5. The existing grading, proposed grading, and construction design features are taken from Reference 4.
6. Undrained conditions ($\phi = 0$) were assumed for the glaciomarine silty clay layer.

CALCULATION

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

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

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

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

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

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

Material Name	Unit Weight (pcf)	Strength Type	Cohesion (psf)	Friction Angle (°)	UCS (psf)	GSI	m_i	D
Existing Fill	125	Mohr-Coulomb	0	32	-	-	-	-
Glaciomarine Silty Clay	125	Mohr-Coulomb	1600	0	-	-	-	-
Sand and Gravel	125	Mohr-Coulomb	0	37	-	-	-	-
Bedrock	164	Generalized Hoek-Brown	-	-	1,869,552	60	28	0
Granular Borrow	125	Mohr-Coulomb	0	32	-	-	-	-
Gravel Borrow	135	Mohr-Coulomb	0	36	-	-	-	-
Rock Borrow	135	Mohr-Coulomb	0	42	-	-	-	-

2. Use the soil layer parameters listed above to analyze the slope stability with Slide.
The soil layer properties above were used to analyze static scenarios at both Abutment 1 and Abutment 2. Following analysis of the proposed abutment and slope grading system, Golder computed global stability factors of safety less than the recommended factor of safety of 1.5 for potential slope failures in the existing and proposed fill. Golder has presented a recommended slope grading system using gravel and rock borrow materials to produce global stability factors of safety greater than 1.5. The results of the Slide stability analyses are summarized in the following table.

GOLDER

CALCULATIONS

Date:	7/2/2021	**Made by:**	AH
Project No.:	21450908	**Checked by:**	KAR/MEL
Subject:	Global Stability Analysis Abutments No. 1 and No. 2	**Reviewed by:**	JEL
Project Short Title:	MaineDOT I-295 Exit 20 Desert Road Bridge Replacement No. 5720		

Slope Grading System	Interpreted Subsurface Section	Abutment	Lowest Factor of Safety (Spencer Method)			
			NonCircular Failure Surface Through Proposed & Existing Fill		NonCircular Failure Surface Through Glaciomarine Deposit	
Proposed	A-A'	1	1.42	(Fig. A.1)	2.10	(Fig. A.2)
		2	1.50	(Fig. B.1)	2.15	(Fig. B.2)
Recommended	A-A'	1	1.53	(Fig. A.3)	2.07	(Fig. A.4)
		2	1.59	(Fig. B.3)	2.15	(Fig. B.4)

Circular Surfaces:

Circular failure surfaces using Spencer and Bishop simplified methods were also evaluated for each scenario. The circular surfaces that were produced were similar in location and size to the noncircular surfaces and had factors of safety similar in magnitude to the noncircular factors of safety. For the proposed slope geometry, the circular factors of safety ranged from 2.28 to 2.65 for surfaces through the proposed and existing fill and from 2.22 to 2.43 for surfaces extending below the road and through the in situ soil soils, including through the glaciomarine deposit. For Golder's recommended fill geometry, the circular factors of safety ranged from 1.56 to 2.71 for surfaces through the recommended and existing fill and from 2.29 to 2.42 for surfaces extending below the road and through the in situ soil soils, including through the glaciomarine deposit.

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

The same scenarios were also analyzed with a horizontal seismic load coefficient of $A_s/2 = 0.064$ (A_s from Reference 9) as recommended in Reference 12. The results of the seismic Slide stability analyses are summarized in the following table.

Slope Grading System	Interpreted Subsurface Section	Abutment	Lowest Factor of Safety (Spencer Method)			
			NonCircular Failure Surface Through Proposed & Existing Fill		NonCircular Failure Surface Through Glaciomarine Deposit	
Proposed	A-A'	1	1.24	(Fig. C.1)	1.79	(Fig. C.2)
		2	1.30	(Fig. D.1)	1.87	(Fig. D.2)
Recommended	A-A'	1	1.35	(Fig. C.3)	1.83	(Fig. C.4)
		2	1.39	(Fig. D.3)	1.88	(Fig. D.4)

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. For the proposed slope geometry, the circular factors of safety ranged from1.97 to 2.23 for surfaces through the proposed and existing fill and from 1.99 to 2.08 for surfaces extending below the road and through the in situ soil soils, including through the glaciomarine deposit. For Golder's recommended fill geometry, the circular factors of safety ranged from 1.97 to 2.23 for surfaces through the recommended and existing fill and from 1.97 to 2.08 for surfaces extending below the road and through the in situ soil soils, including through the glaciomarine deposit.

GOLDER

CALCULATIONS

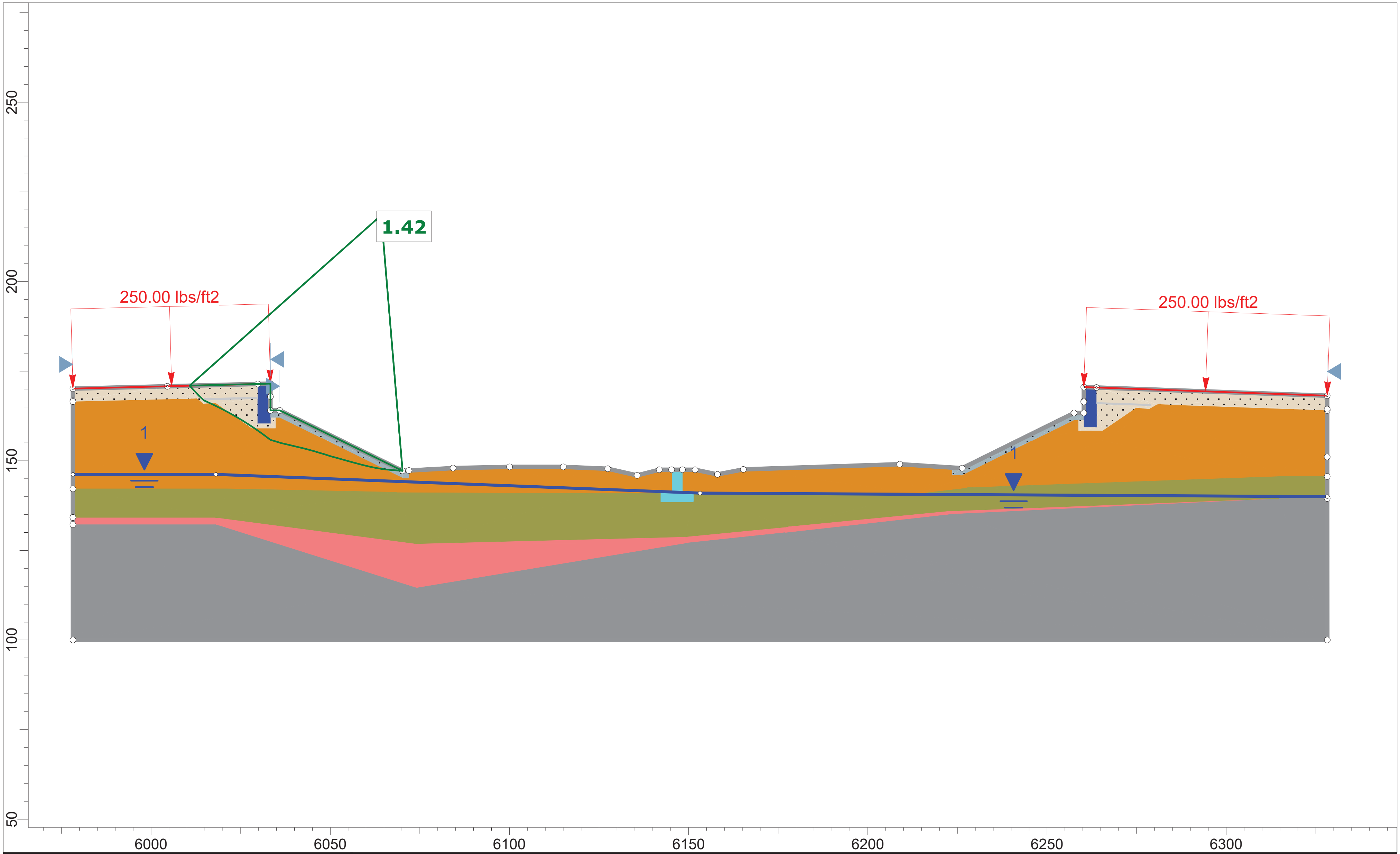
Date:	7/2/2021	**Made by:**	AH
Project No.:	21450908	**Checked by:**	KAR/MEL
Subject:	Global Stability Analysis Abutments No. 1 and No. 2	**Reviewed by:**	JEL
Project Short Title:	MaineDOT I-295 Exit 20 Desert Road Bridge Replacement No. 5720		

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. Golder has provided global stability analysis of the recommended slope grading sytem that yields adequate factors of safety (FS > 1.5).

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 abutment under static conditions.

The analysis of the proposed abutment and slope grading system yields adequate factors of safety (FS > 1.1) 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.



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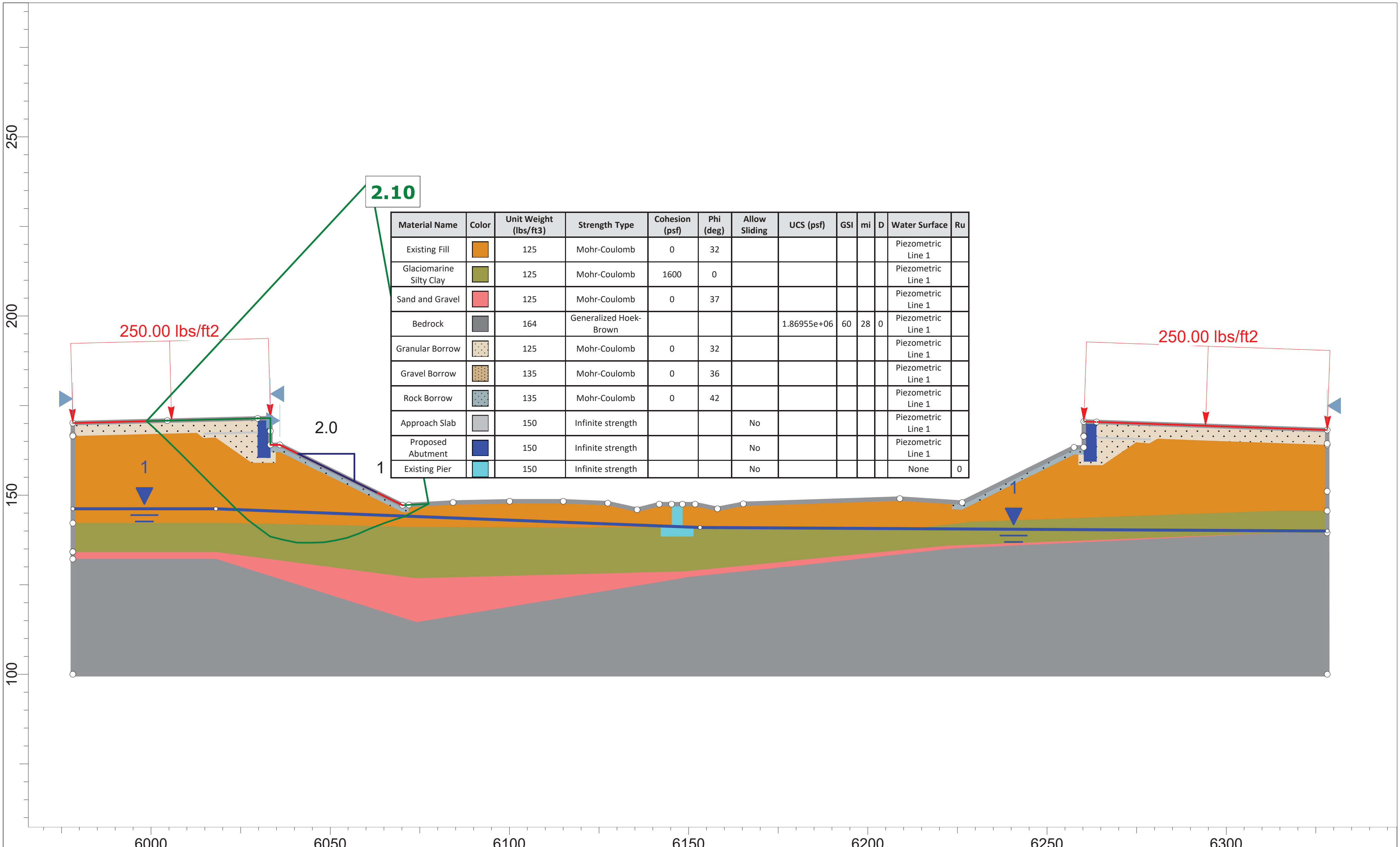
Project	19126013 MaineDOT I-295 Exit 20 Desert Road Bridge No. 5720		
Group	Abutment 1 Proposed	Scenario	**Figure A.1**
Drawn By	KAR/AH/MEL	Company	Golder Associates
Date	7/2/2021	File Name	Desert Rd Profile A-A' Phase 2 V3.slmd



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Allow Sliding	UCS (psf)	GSI	mi	D	Water Surface	Ru
Existing Fill		125	Mohr-Coulomb	0	32						Piezometric Line 1	
Glaciomarine Silty Clay		125	Mohr-Coulomb	1600	0						Piezometric Line 1	
Sand and Gravel		125	Mohr-Coulomb	0	37						Piezometric Line 1	
Bedrock		164	Generalized Hoek-Brown				1.86955e+06	60	28	0	Piezometric Line 1	
Granular Borrow		125	Mohr-Coulomb	0	32						Piezometric Line 1	
Gravel Borrow		135	Mohr-Coulomb	0	36						Piezometric Line 1	
Rock Borrow		135	Mohr-Coulomb	0	42						Piezometric Line 1	
Approach Slab		150	Infinite strength			No					Piezometric Line 1	
Proposed Abutment		150	Infinite strength			No					Piezometric Line 1	
Existing Pier		150	Infinite strength			No					None	0

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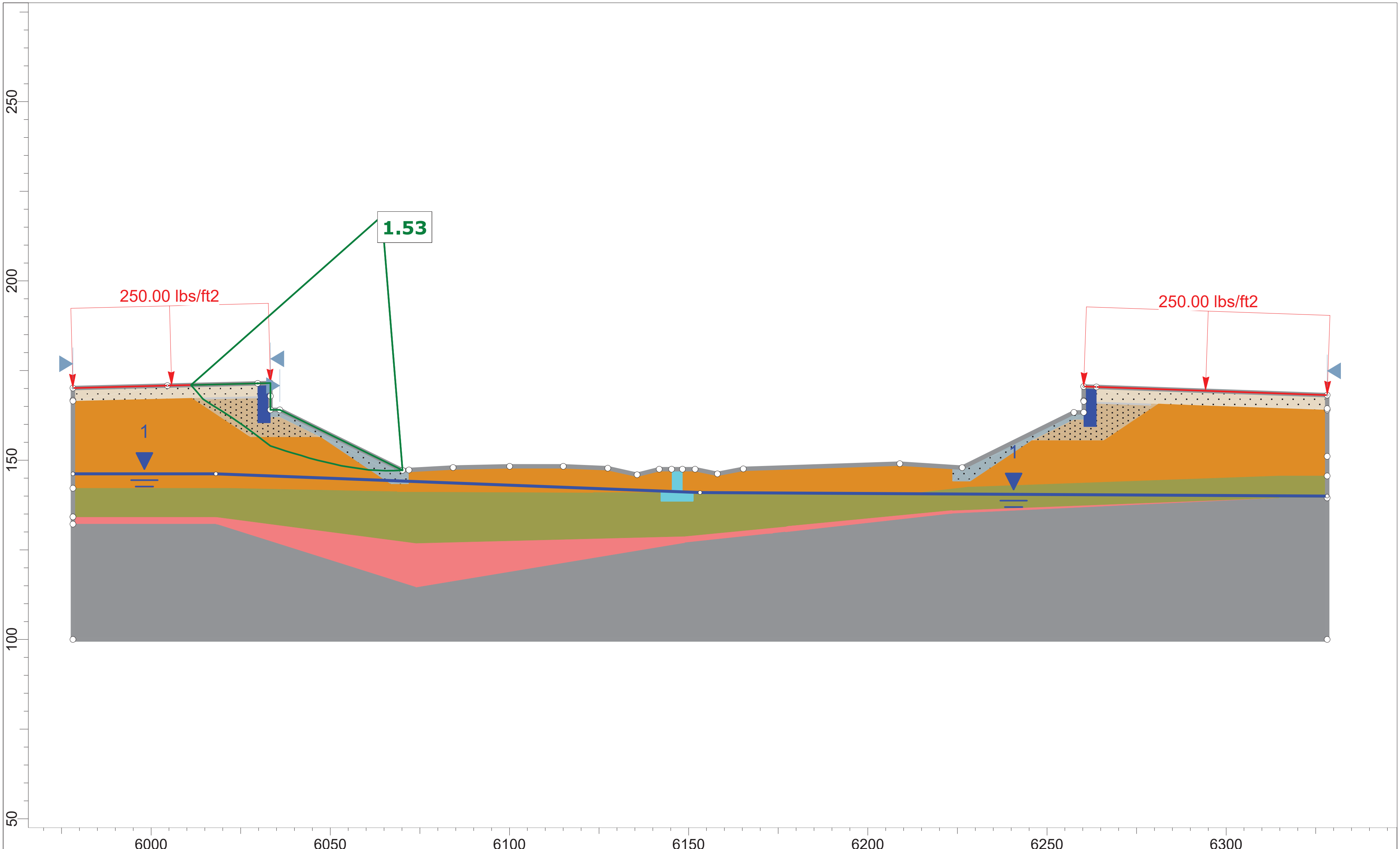
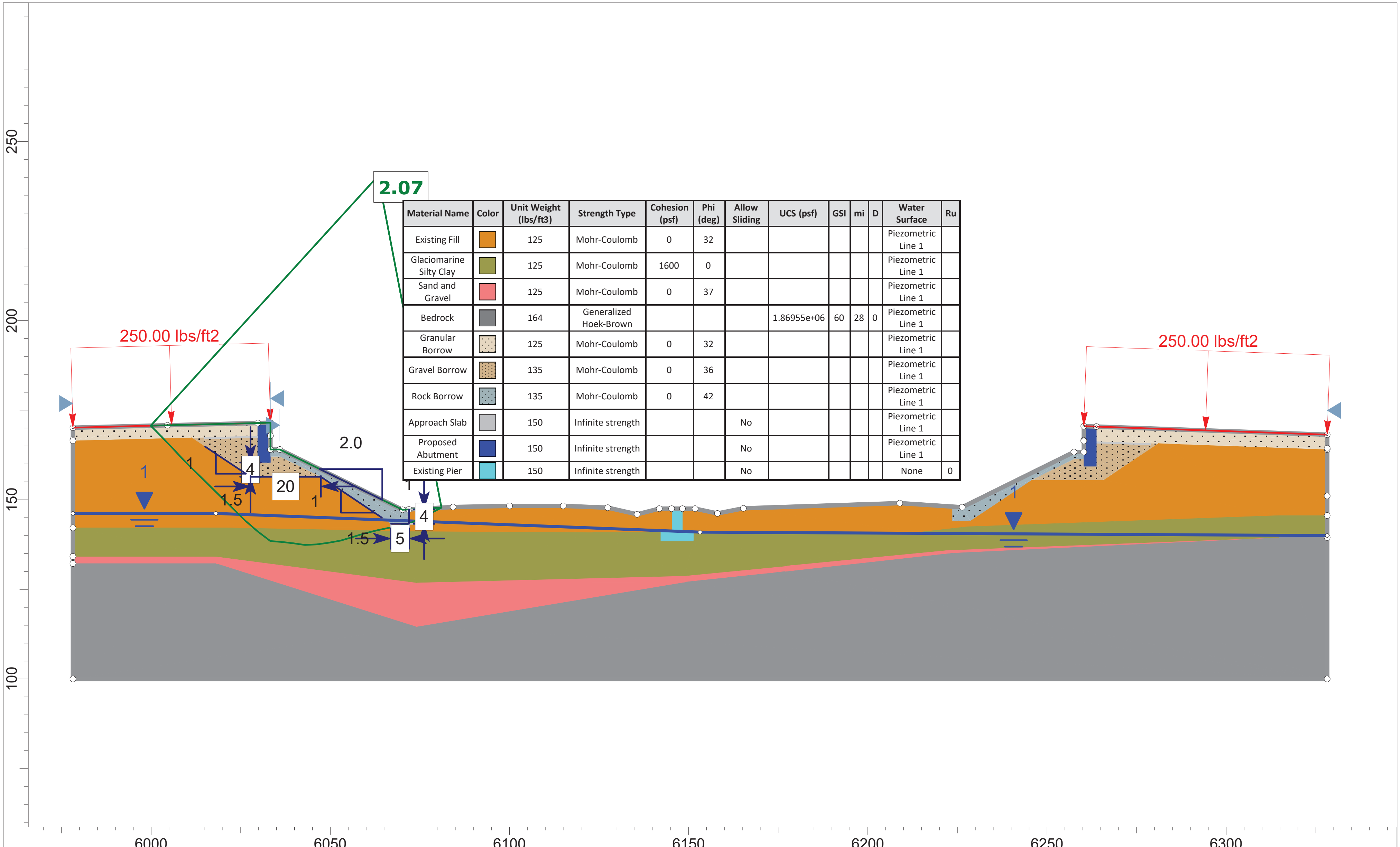
Project	19126013 MaineDOT I-295 Exit 20 Desert Road Bridge No. 5720		
Group	Abutment 1 Proposed	*Scenario*	**Figure A.2**
Drawn By	KAR/AH/MEL	*Company*	Golder Associates
Date	7/2/2021	*File Name*	Desert Rd Profile A-A' Phase 2 V3.slmd



GOLDER MEMBER OF WSP	Project	19126013 MaineDOT I-295 Exit 20 Desert Road Bridge No. 5720	
	Group: Abutment 1 Recommended	Scenario: **Figure A.3**	
	Drawn By: KAR/AH/MEL	Company: Golder Associates	
SLIDEINTERPRET 9.010	Date: 7/2/2021	File Name: Desert Rd Profile A-A' Phase 2 V3.slmd	



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Allow Sliding	UCS (psf)	GSI	mi	D	Water Surface	Ru
Existing Fill		125	Mohr-Coulomb	0	32						Piezometric Line 1	
Glaciomarine Silty Clay		125	Mohr-Coulomb	1600	0						Piezometric Line 1	
Sand and Gravel		125	Mohr-Coulomb	0	37						Piezometric Line 1	
Bedrock		164	Generalized Hoek-Brown				1.86955e+06	60	28	0	Piezometric Line 1	
Granular Borrow		125	Mohr-Coulomb	0	32						Piezometric Line 1	
Gravel Borrow		135	Mohr-Coulomb	0	36						Piezometric Line 1	
Rock Borrow		135	Mohr-Coulomb	0	42						Piezometric Line 1	
Approach Slab		150	Infinite strength			No					Piezometric Line 1	
Proposed Abutment		150	Infinite strength			No					Piezometric Line 1	
Existing Pier		150	Infinite strength			No					None	0

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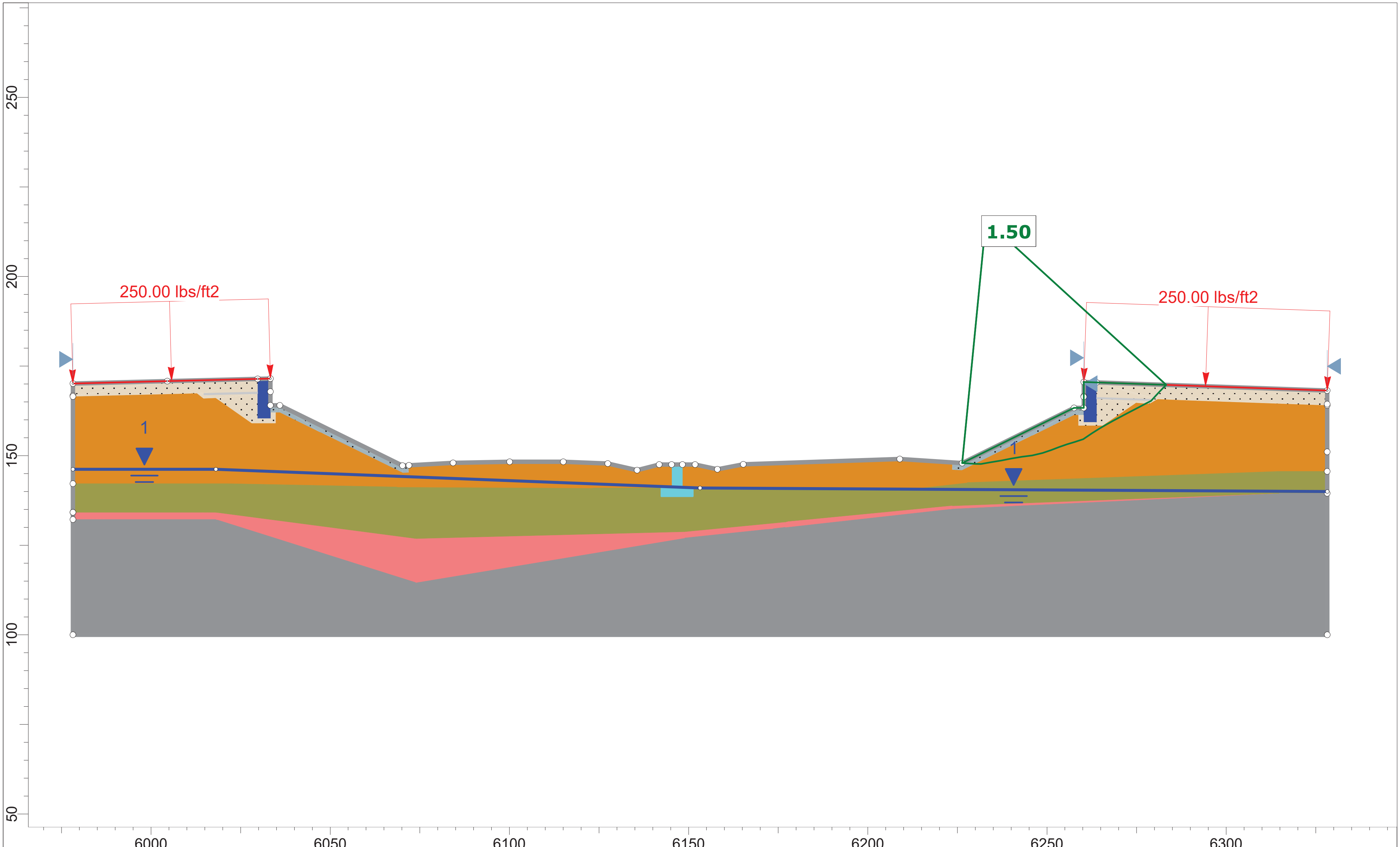
Project	19126013 MaineDOT I-295 Exit 20 Desert Road Bridge No. 5720		
Group	Abutment 1 Recommended	*Scenario*	**Figure A.4**
Drawn By	KAR/AH/MEL	*Company*	Golder Associates
Date	7/2/2021	*File Name*	Desert Rd Profile A-A' Phase 2 V3.slmd



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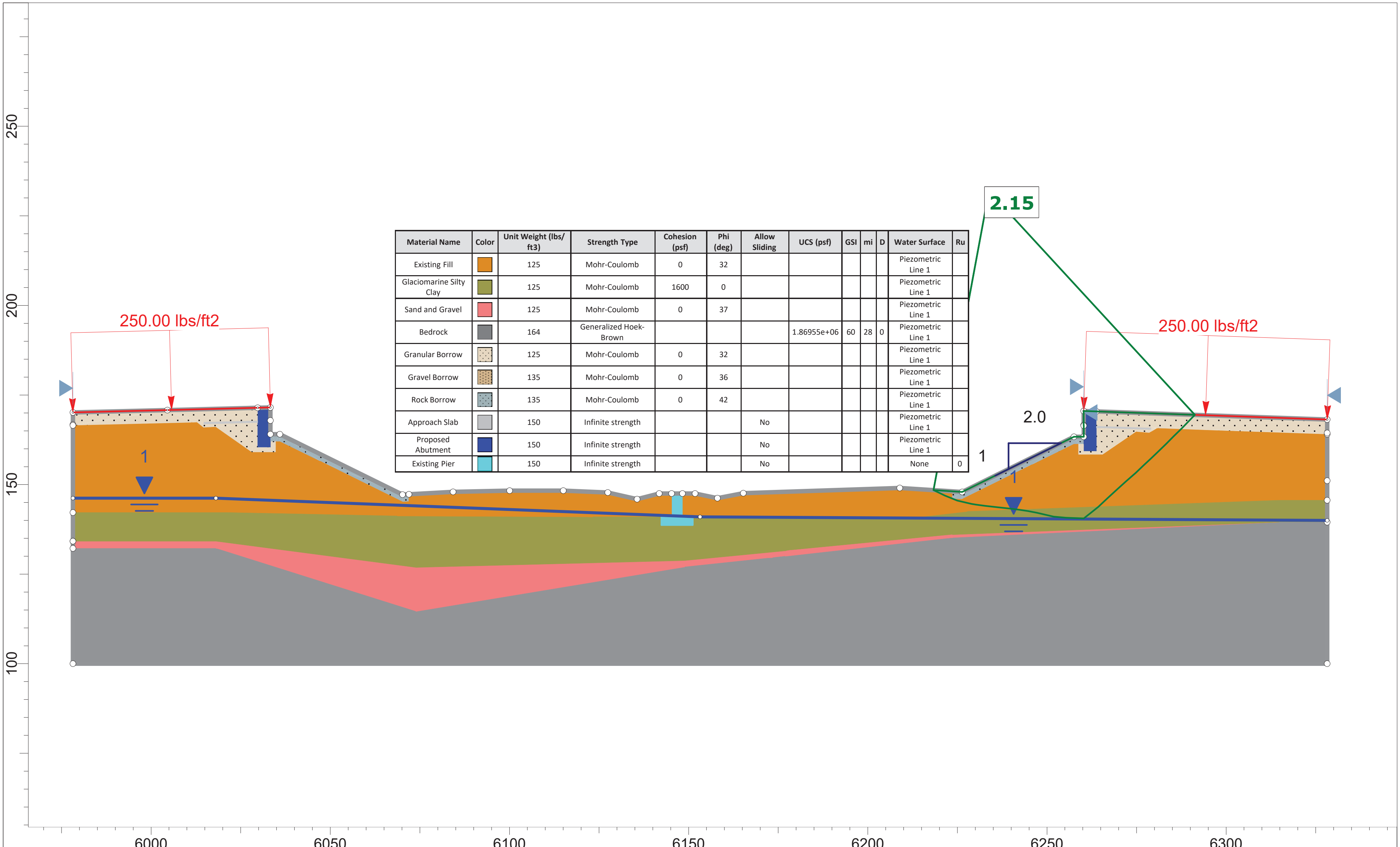
Project	19126013 MaineDOT I-295 Exit 20 Desert Road Bridge No. 5720

Group	Abutment 2 Proposed	*Scenario*	**Figure B.1**
Drawn By	KAR/AH/MEL	*Company*	Golder Associates
Date	7/2/2021	*File Name*	Desert Rd Profile A-A' Phase 2 V3.slmd



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Allow Sliding	UCS (psf)	GSI	mi	D	Water Surface	Ru
Existing Fill		125	Mohr-Coulomb	0	32						Piezometric Line 1	
Glaciomarine Silty Clay		125	Mohr-Coulomb	1600	0						Piezometric Line 1	
Sand and Gravel		125	Mohr-Coulomb	0	37						Piezometric Line 1	
Bedrock		164	Generalized Hoek-Brown				1.86955e+06	60	28	0	Piezometric Line 1	
Granular Borrow		125	Mohr-Coulomb	0	32						Piezometric Line 1	
Gravel Borrow		135	Mohr-Coulomb	0	36						Piezometric Line 1	
Rock Borrow		135	Mohr-Coulomb	0	42						Piezometric Line 1	
Approach Slab		150	Infinite strength			No					Piezometric Line 1	
Proposed Abutment		150	Infinite strength			No					Piezometric Line 1	
Existing Pier		150	Infinite strength			No					None	0

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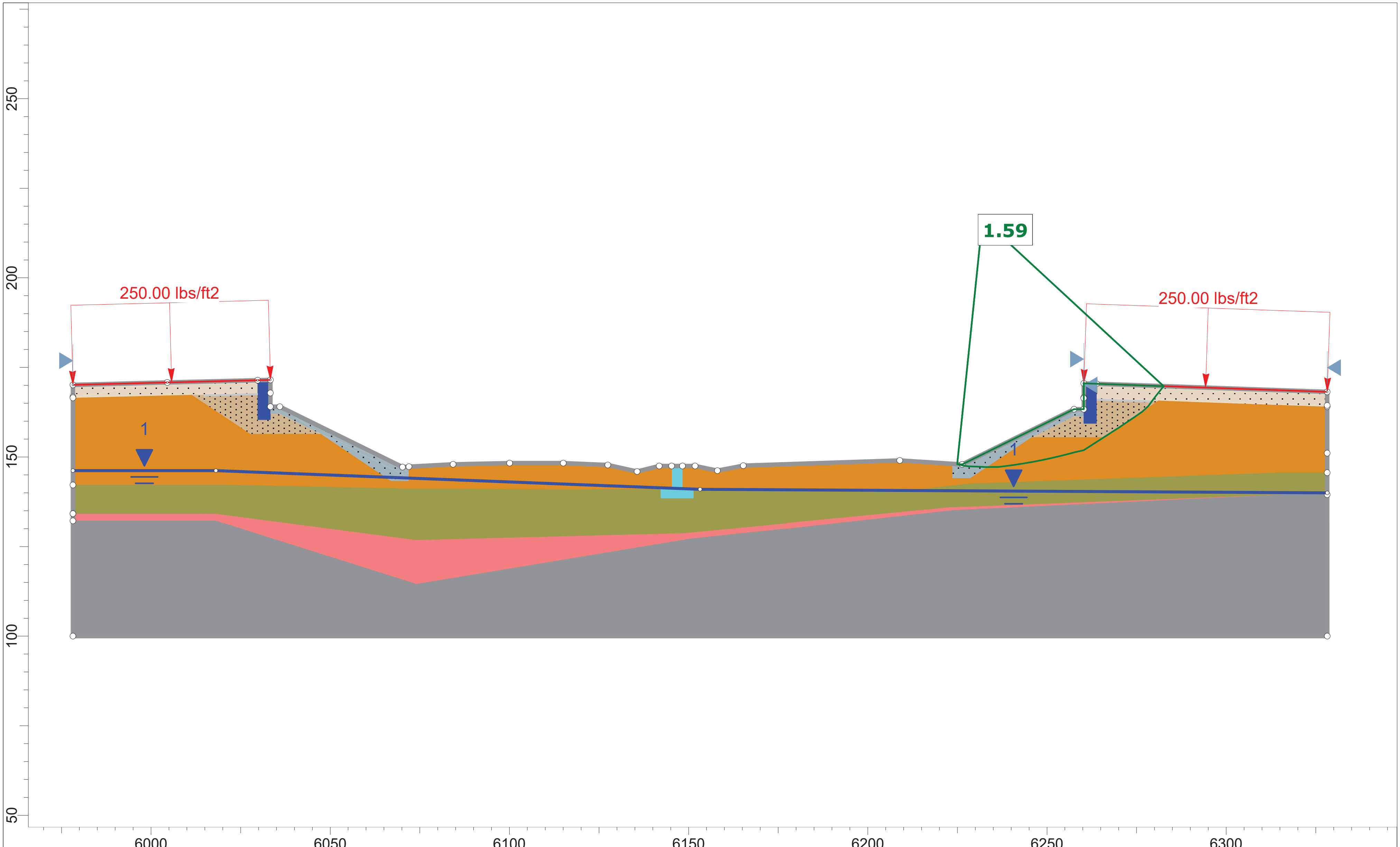
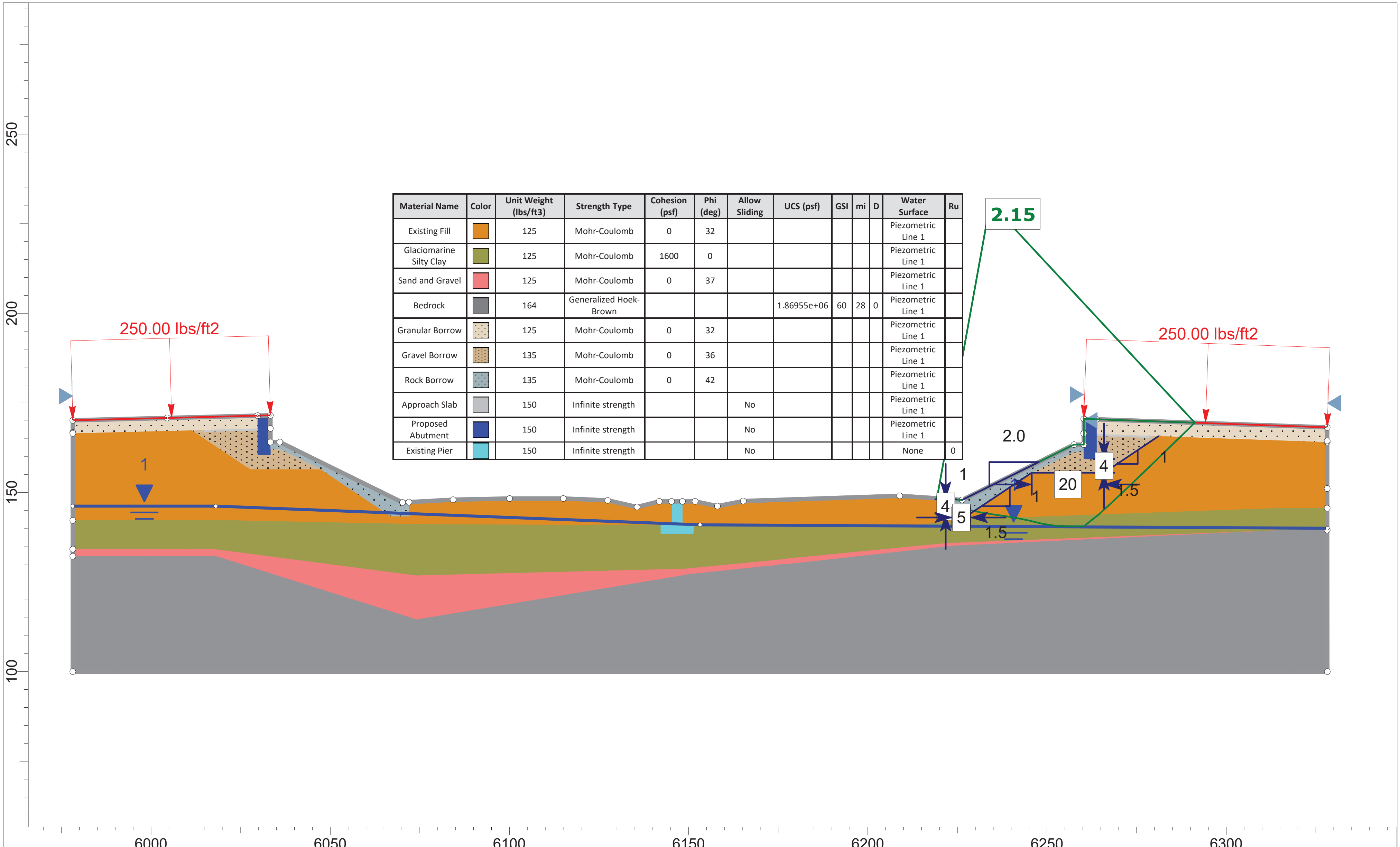
Project	19126013 MaineDOT I-295 Exit 20 Desert Road Bridge No. 5720		
Group	Abutment 2 Proposed	*Scenario*	**Figure B.2**
Drawn By	KAR/AH/MEL	*Company*	Golder Associates
Date	7/2/2021	*File Name*	Desert Rd Profile A-A' Phase 2 V3.slmd



GOLDER MEMBER OF WSP	*Project* 19126013 MaineDOT I-295 Exit 20 Desert Road Bridge No. 5720		
	Group Abutment 2 Recommended	*Scenario* **Figure B.3**	
	Drawn By KAR/AH/MEL	*Company* Golder Associates	
SLIDEINTERPRET 9.010	*Date* 7/2/2021	*File Name* Desert Rd Profile A-A' Phase 2 V3.slmd	



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Allow Sliding	UCS (psf)	GSI	mi	D	Water Surface	Ru
Existing Fill		125	Mohr-Coulomb	0	32						Piezometric Line 1	
Glaciomarine Silty Clay		125	Mohr-Coulomb	1600	0						Piezometric Line 1	
Sand and Gravel		125	Mohr-Coulomb	0	37						Piezometric Line 1	
Bedrock		164	Generalized Hoek-Brown				1.86955e+06	60	28	0	Piezometric Line 1	
Granular Borrow		125	Mohr-Coulomb	0	32						Piezometric Line 1	
Gravel Borrow		135	Mohr-Coulomb	0	36						Piezometric Line 1	
Rock Borrow		135	Mohr-Coulomb	0	42						Piezometric Line 1	
Approach Slab		150	Infinite strength			No					Piezometric Line 1	
Proposed Abutment		150	Infinite strength			No					Piezometric Line 1	
Existing Pier		150	Infinite strength			No					None	0

GOLDER
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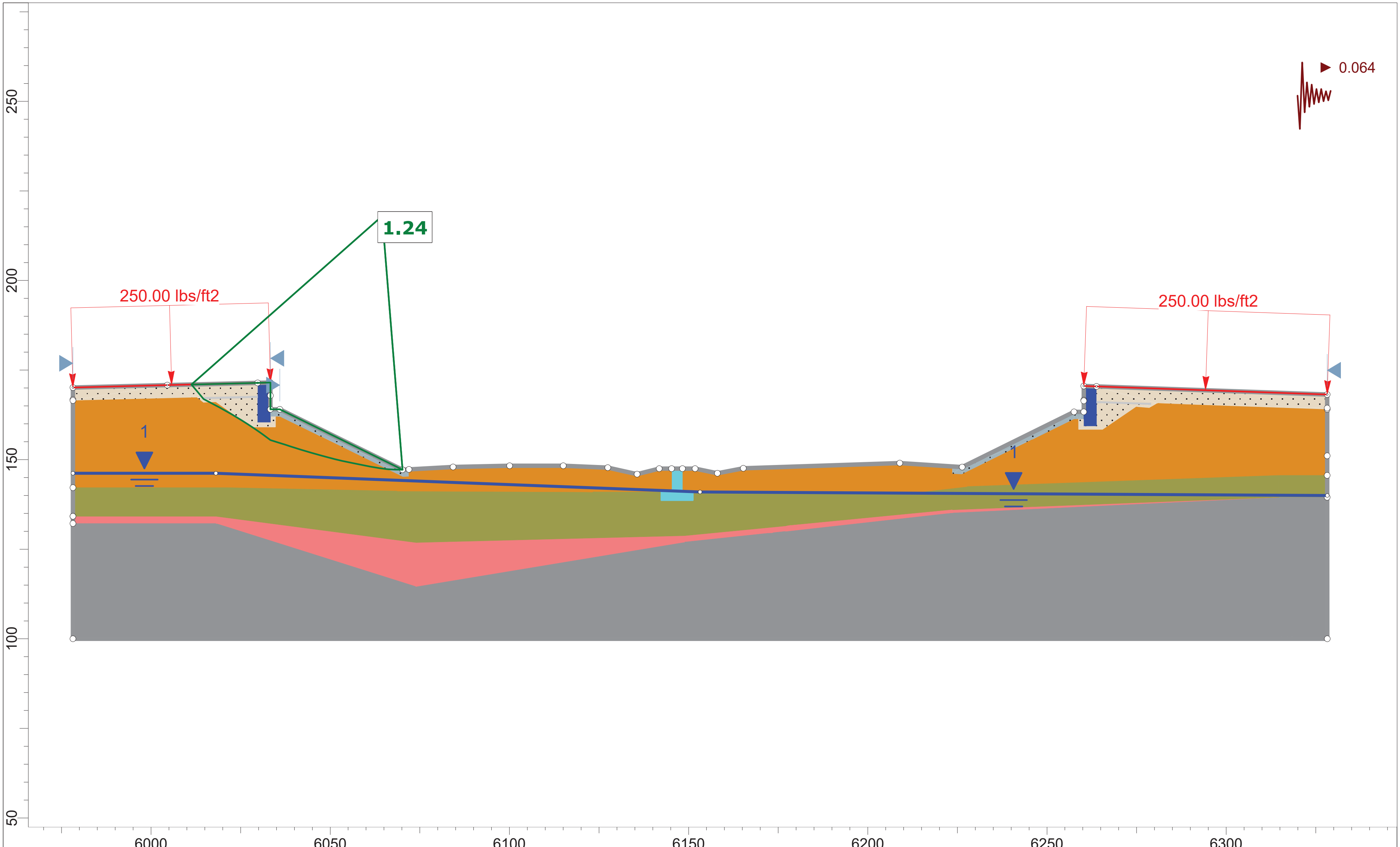
Project	19126013 MaineDOT I-295 Exit 20 Desert Road Bridge No. 5720
Group	Abutment 2 Recommended
Scenario	**Figure B.4**
Drawn By	KAR/AH/MEL
Company	Golder Associates
Date	7/2/2021
File Name	Desert Rd Profile A-A' Phase 2 V3.slmd



Project	19126013 MaineDOT I-295 Exit 20 Desert Road Bridge No. 5720		
Group	Abutment 1 Proposed Seismic	*Scenario*	**Figure C.1**
Drawn By	KAR/AH/MEL	*Company*	Golder Associates
Date	7/2/2021	*File Name*	Desert Rd Profile A-A' Phase 2 V3.slmd

GOLDER
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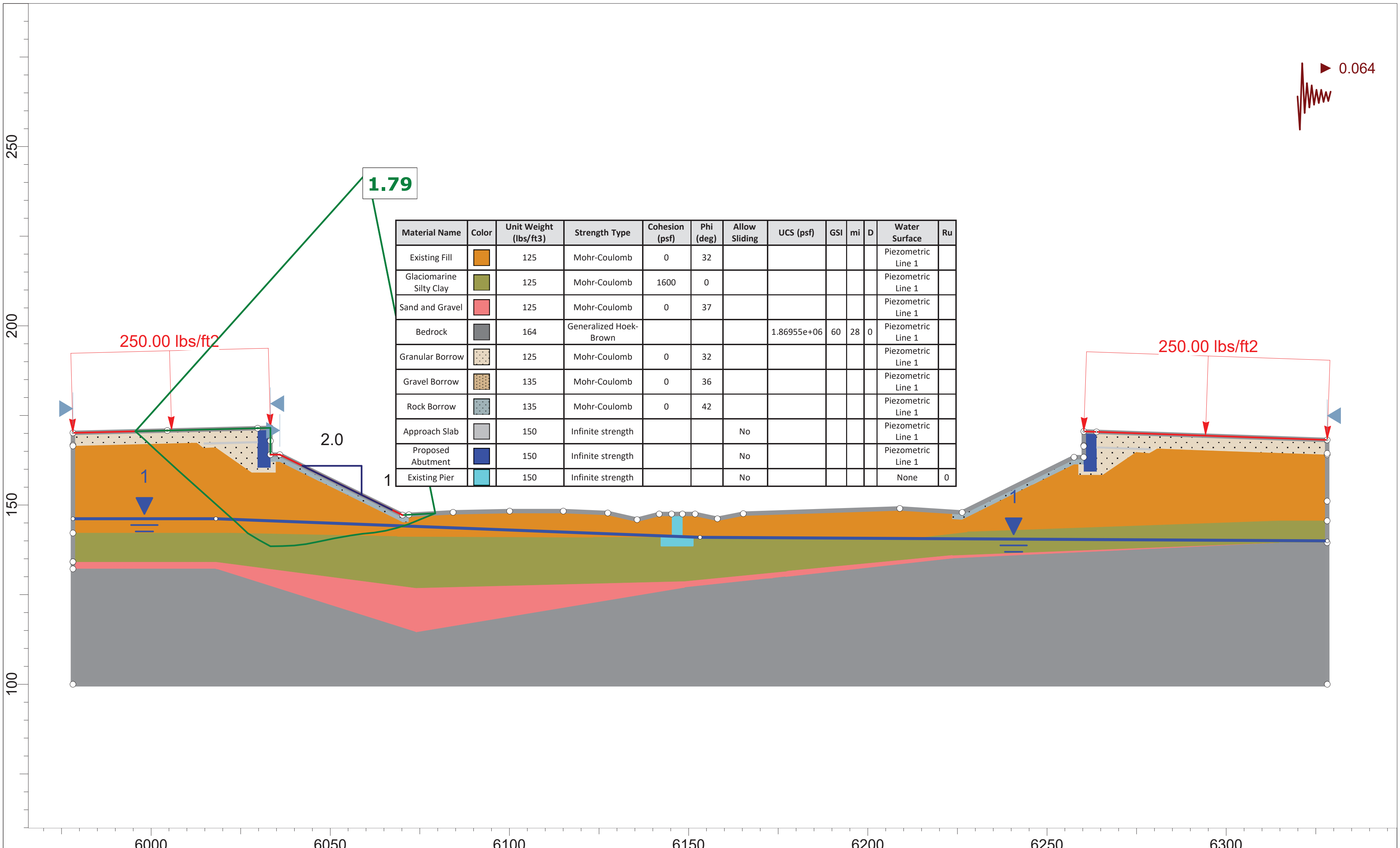
SLIDEINTERPRET 9.010



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Allow Sliding	UCS (psf)	GSI	mi	D	Water Surface	Ru
Existing Fill		125	Mohr-Coulomb	0	32						Piezometric Line 1	
Glaciomarine Silty Clay		125	Mohr-Coulomb	1600	0						Piezometric Line 1	
Sand and Gravel		125	Mohr-Coulomb	0	37						Piezometric Line 1	
Bedrock		164	Generalized Hoek-Brown				1.86955e+06	60	28	0	Piezometric Line 1	
Granular Borrow		125	Mohr-Coulomb	0	32						Piezometric Line 1	
Gravel Borrow		135	Mohr-Coulomb	0	36						Piezometric Line 1	
Rock Borrow		135	Mohr-Coulomb	0	42						Piezometric Line 1	
Approach Slab		150	Infinite strength			No					Piezometric Line 1	
Proposed Abutment		150	Infinite strength			No					Piezometric Line 1	
Existing Pier		150	Infinite strength			No					None	0

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

Group Abutment 1 Proposed Seismic

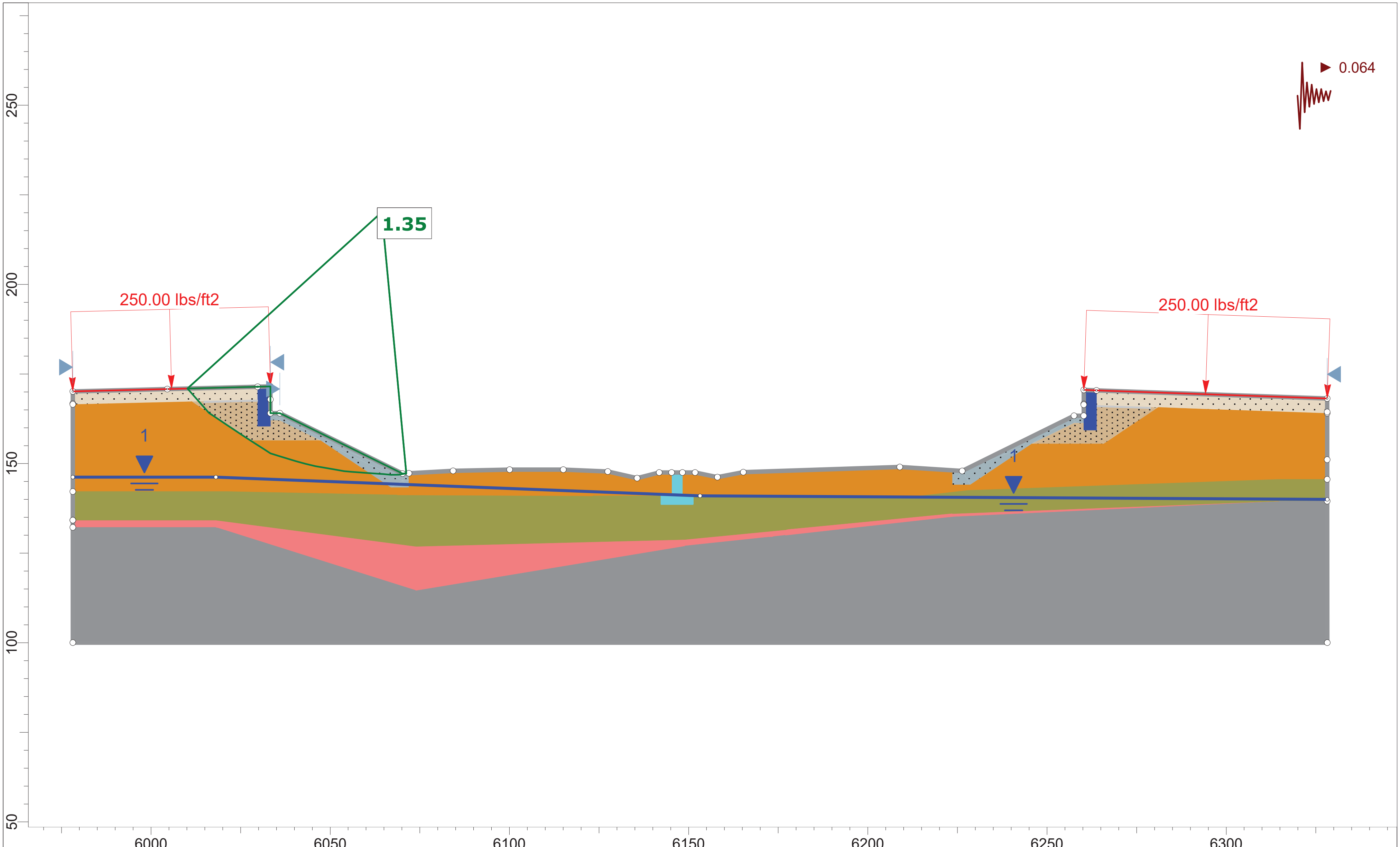
Scenario **Figure C.2**

Drawn By KAR/AH/MEL

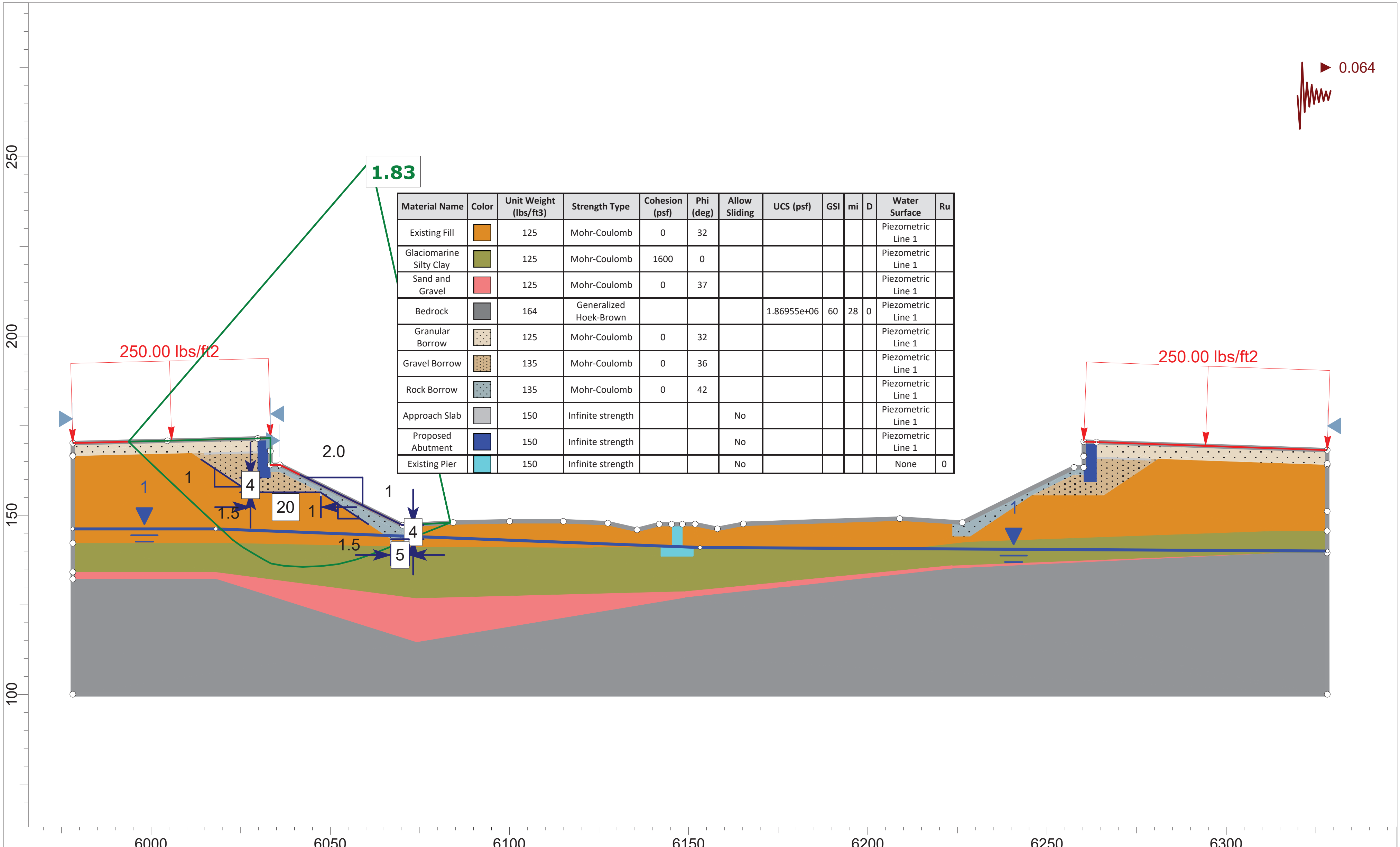
Company Golder Associates

Date 7/2/2021

File Name Desert Rd Profile A-A' Phase 2 V3.slmd



GOLDER MEMBER OF WSP	*Project* 19126013 MaineDOT I-295 Exit 20 Desert Road Bridge No. 5720		
	Group Abutment 1 Recommended Seismic	*Scenario* **Figure C.3**	
	Drawn By KAR/AH/MEL	*Company* Golder Associates	
SLIDEINTERPRET 9.010	*Date* 7/2/2021	*File Name* Desert Rd Profile A-A' Phase 2 V3.slmd	

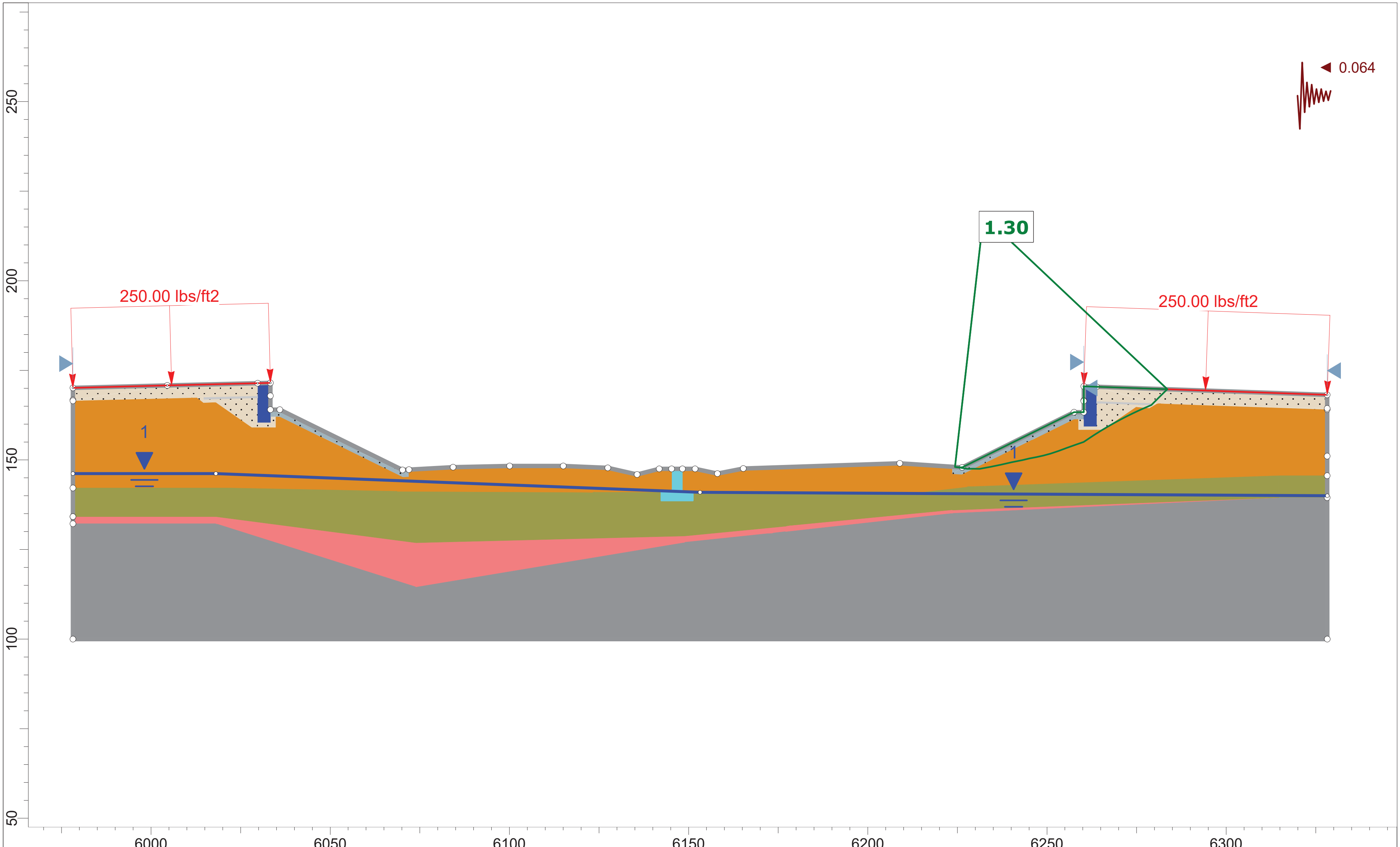


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Allow Sliding	UCS (psf)	GSI	mi	D	Water Surface	Ru
Existing Fill		125	Mohr-Coulomb	0	32						Piezometric Line 1	
Glaciomarine Silty Clay		125	Mohr-Coulomb	1600	0						Piezometric Line 1	
Sand and Gravel		125	Mohr-Coulomb	0	37						Piezometric Line 1	
Bedrock		164	Generalized Hoek-Brown				1.86955e+06	60	28	0	Piezometric Line 1	
Granular Borrow		125	Mohr-Coulomb	0	32						Piezometric Line 1	
Gravel Borrow		135	Mohr-Coulomb	0	36						Piezometric Line 1	
Rock Borrow		135	Mohr-Coulomb	0	42						Piezometric Line 1	
Approach Slab		150	Infinite strength			No					Piezometric Line 1	
Proposed Abutment		150	Infinite strength			No					Piezometric Line 1	
Existing Pier		150	Infinite strength			No					None	0

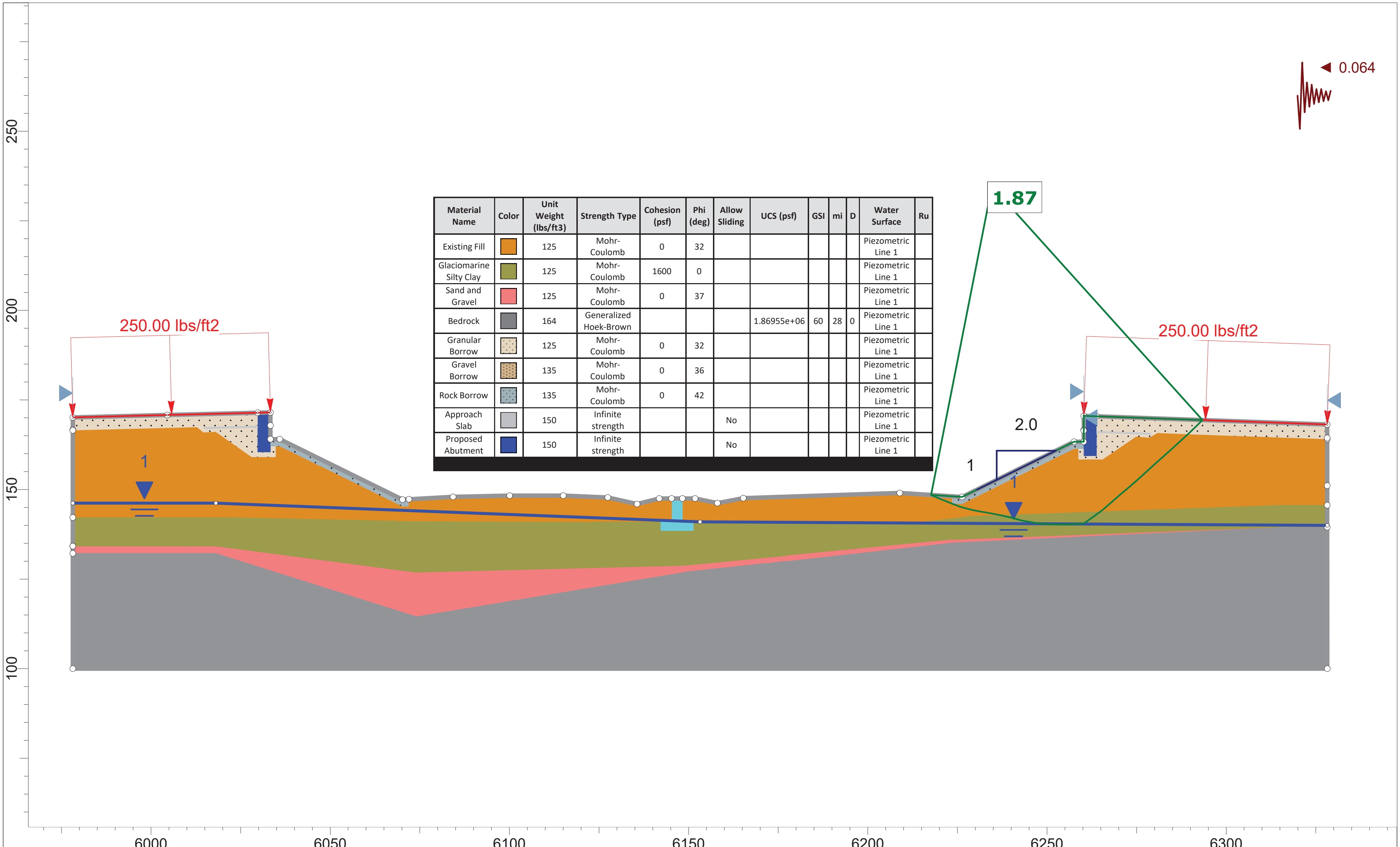
GOLDER
MEMBER OF WSP

SLIDEINTERPRET 9.010

Project	19126013 MaineDOT I-295 Exit 20 Desert Road Bridge No. 5720
Group	Abutment 1 Recommended Seismic
Scenario	**Figure C.4**
Drawn By	KAR/AH/MEL
Company	Golder Associates
Date	7/2/2021
File Name	Desert Rd Profile A-A' Phase 2 V3.slmd



GOLDER MEMBER OF WSP	*Project*	19126013 MaineDOT I-295 Exit 20 Desert Road Bridge No. 5720	
	Group Abutment 2 Proposed Seismic	*Scenario* **Figure D.1**	
	Drawn By KAR/AH/MEL	*Company* Golder Associates	
SLIDEINTERPRET 9.010	*Date* 7/2/2021	*File Name* Desert Rd Profile A-A' Phase 2 V3.slmd	



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Allow Sliding	UCS (psf)	GSI	mi	D	Water Surface	Ru
Existing Fill		125	Mohr-Coulomb	0	32						Piezometric Line 1	
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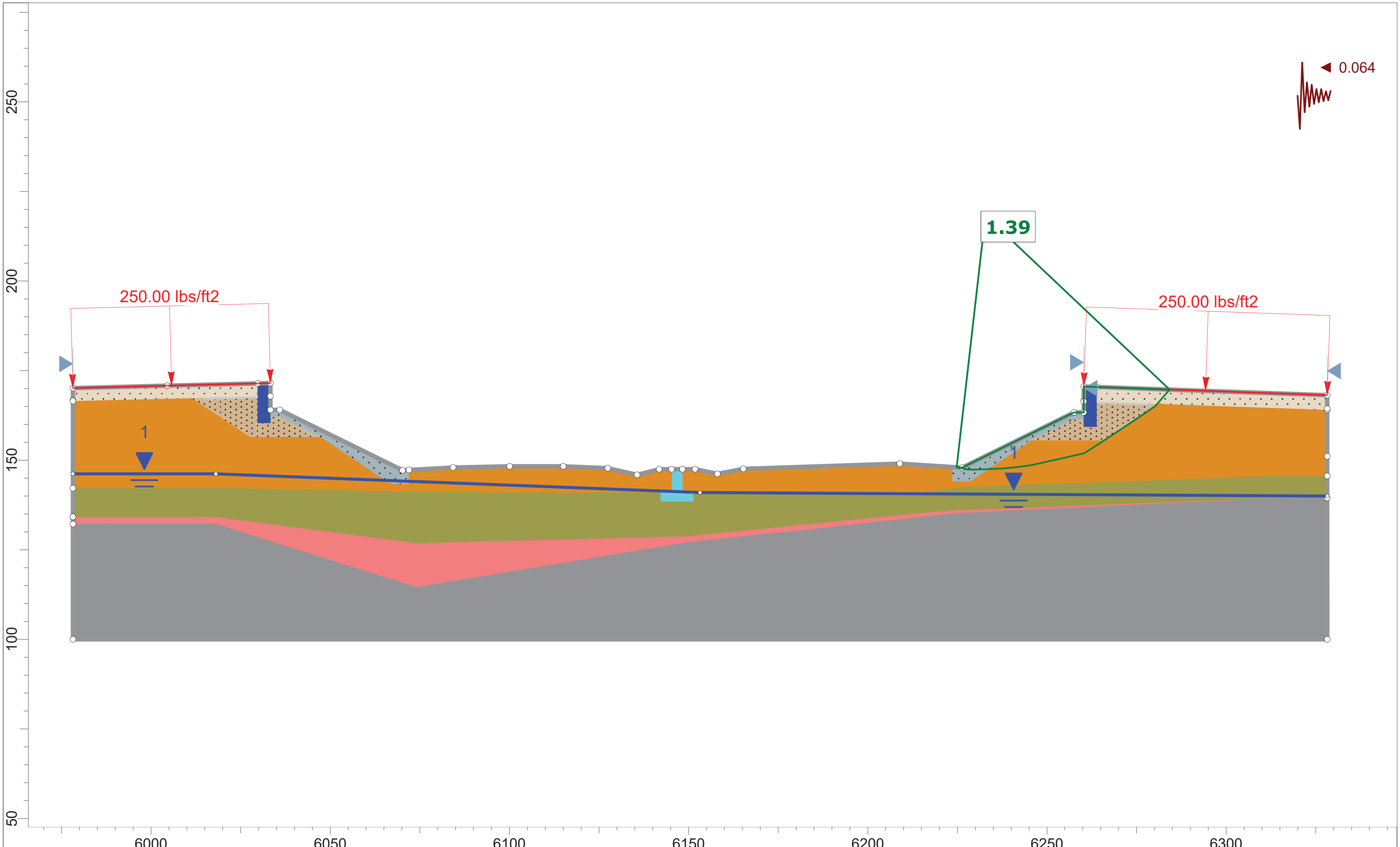
Sand and Gravel		125	Mohr-Coulomb	0	37						Piezometric Line 1	
Bedrock		164	Generalized Hoek-Brown				1.86955e+06	60	28	0	Piezometric Line 1	
Granular Borrow		125	Mohr-Coulomb	0	32						Piezometric Line 1	
Gravel Borrow		135	Mohr-Coulomb	0	36						Piezometric Line 1	
Rock Borrow		135	Mohr-Coulomb	0	42						Piezometric Line 1	
Approach Slab		150	Infinite strength			No					Piezometric Line 1	
Proposed Abutment		150	Infinite strength			No					Piezometric Line 1	

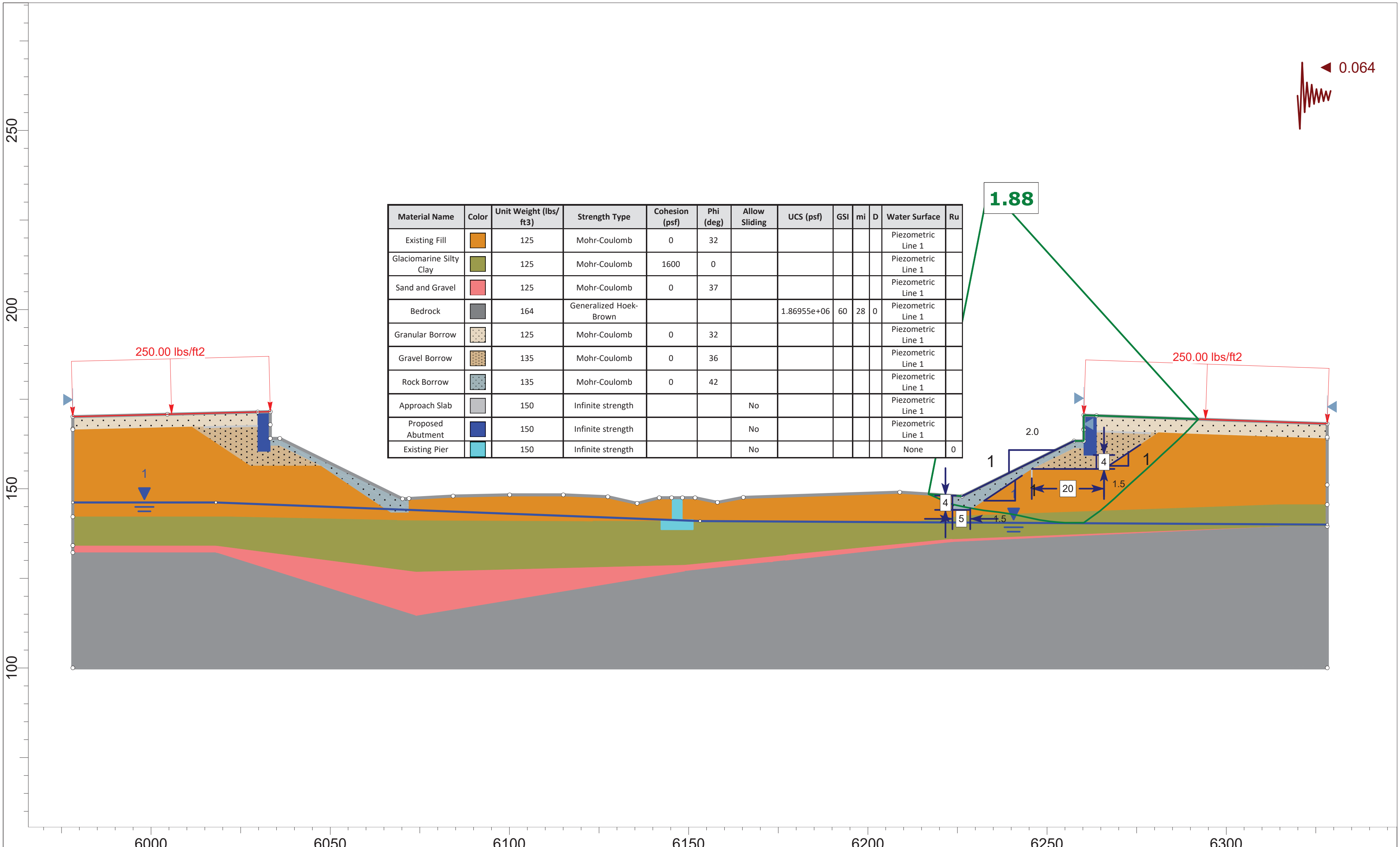
GOLDER
MEMBER OF WSP

SLIDEINTERPRET 9.010

Project	19126013 MaineDOT I-295 Exit 20 Desert Road Bridge No. 5720
Group	Abutment 2 Proposed Seismic
Scenario	**Figure D.2**
Drawn By	KAR/AH/MEL
Company	Golder Associates
Date	7/2/2021
File Name	Desert Rd Profile A-A' Phase 2 V3.slmd



GOLDER MEMBER OF WSP	*Project*	19126013 MaineDOT I-295 Exit 20 Desert Road Bridge No. 5720	
	Group Abutment 2 Recommended Seismic	*Scenario* **Figure D.3**	
	Drawn By KAR/AH/MEL	*Company* Golder Associates	
SLIDEINTERPRET 9.010	*Date* 7/2/2021	*File Name* Desert Rd Profile A-A' Phase 2 V3.slmd	



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Allow Sliding	UCS (psf)	GSI	mi	D	Water Surface	Ru
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Bedrock		164	Generalized Hoek-Brown				1.86955e+06	60	28	0	Piezometric Line 1	
Granular Borrow		125	Mohr-Coulomb	0	32						Piezometric Line 1	
Gravel Borrow		135	Mohr-Coulomb	0	36						Piezometric Line 1	
Rock Borrow		135	Mohr-Coulomb	0	42						Piezometric Line 1	
Approach Slab		150	Infinite strength			No					Piezometric Line 1	
Proposed Abutment		150	Infinite strength			No					Piezometric Line 1	
Existing Pier		150	Infinite strength			No					None	0

GOLDER MEMBER OF WSP	*Project* 19126013 MaineDOT I-295 Exit 20 Desert Road Bridge No. 5720
Group Abutment 2 Recommended Seismic	*Scenario* **Figure D.4**
Drawn By KAR/AH/MEL	*Company* Golder Associates
Date 7/2/2021	*File Name* Desert Rd Profile A-A' Phase 2 V3.slmd

SLIDEINTERPRET 9.010

GOLDER

CALCULATIONS

Date:	7/2/2021	**Made by:**	AH
Project No.:	21450508	**Checked by:**	KAR/MEL
Subject:	Global Stability Analysis Abutment No. 1 Embankment	**Reviewed by:**	JEL
Project Short Title:	MaineDOT I-295 Exit 20 Desert Road Bridge Replacement No. 5720		

OBJECTIVE
Calculate global factor of safety for the Abutment No. 1 proposed bridge approach embankment.

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 9th Ed, 2020.
2. Email communication between Golder and HNTB, subject "RE: Freeport I-295 Exit 20 and Exit 22 - Request for Sections and Design Info", dated June 22, 2020.
3. Golder geotechnical test boring logs for 100-series borings (Appendix A, Preliminary Geotechnical Design Report, dated December 21, 2020).
4. HNTB for State of Maine Department of Transportation. Merrill Road Bridge over Interstate 295 and Signalized Intersections, Exit 20 Interchange: 60% Plans, dated May 21, 2021.
5. Das, Braja M. 2011. Principles of Foundation Engineering, 7th Edition. Cengage Learning.
6. FHWA. 2017. Geotechnical Engineering Circular No. 5: Geotechnical Site Characterization. Publication No. FHWA NHI-16-072.
7. GeoTesting Express laboratory testing results, dated February 5, 2020 (Appendix C, Preliminary Geotechnical Design Report, dated December 21, 2020).
8. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.
9. Golder calculation titled "Seismic Site Class" (Appendix E, Preliminary Geotechnical Design Report, dated December 21, 2020).
10. Rocscience Slide Software Package Version 2020 9.007 64-bit, build date May 29, 2020.
11. FHWA. 2011. Geotechnical Engineering Circular No. 3 - LRFD Seismic Analysis and Design of Transportation Geotechnical Features and Structural Foundations Reference Manual, Publication No. FHWA-NHI-11-032

ATTACHMENTS
1. Slide output figures
2. HNTB 60% Plans

ASSUMPTIONS
1. The load applied by the road and traffic for final design conditions is modeled as a 2 ft equivalent load of soil (Reference 1, Table 3.11.6.4-1) based on a 11 ft abutment height (Reference 2). 2 ft x 125 pcf (fill) = 250 psf.
2. A static FS ≥ 1.3 is recommended for embankment final design conditions per Section 5.9.2 in Reference 8. A pseudo-static FS > 1.1 is recommended in Reference 11.
3. Circular surfaces were analyzed using the Spencer and Bishop simplified methods and auto refine search. Non-circular surfaces were analyzed using the Spencer method and cuckoo search with surface altering optimization.
4. The existing stratigraphy and water table surface are estimated from samples and groundwater measurements encountered during drilling and provided in Reference 3.
5. The existing grading, proposed grading, and construction design features are taken from Reference 4.
6. Undrained conditions ($\phi = 0$) were assumed for the glaciomarine silty clay layer.

GOLDER

CALCULATIONS

Date:	7/2/2021	**Made by:**	AH
Project No.:	21450508	**Checked by:**	KAR/MEL
Subject:	Global Stability Analysis Abutment No. 1 Embankment	**Reviewed by:**	JEL
Project Short Title:	MaineDOT I-295 Exit 20 Desert Road Bridge Replacement No. 5720		

CALCULATION

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

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

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

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

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

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

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

Material Name	Unit Weight (pcf)	Strength Type	Cohesion (psf)	Friction Angle (°)	UCS (psf)	GSI	m_i	D
Existing Fill	125	Mohr-Coulomb	0	32	-	-	-	-
Glaciomarine Silty Clay	125	Mohr-Coulomb	1600	0	-	-	-	-
Sand and Gravel	125	Mohr-Coulomb	0	37	-	-	-	-
Bedrock	164	Generalized Hoek-Brown	-	-	1,869,552	60	28	0
Granular Borrow	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. This analysis evaluates the fills at Abutment No. 1 since the proposed fills and the underlying foundation clays are the thickest. Both the northern and the southern fill slopes were analyzed at the Abutment No. 1. The results of the Slide stability analyses are summarized in the following table.

GOLDER

CALCULATIONS

Date:	7/2/2021	**Made by:**	AH
Project No.:	21450508	**Checked by:**	KAR/MEL
Subject:	Global Stability Analysis Abutment No. 1 Embankment	**Reviewed by:**	JEL
Project Short Title:	MaineDOT I-295 Exit 20 Desert Road Bridge Replacement No. 5720		

Baseline	Station	Feature	Slope	Lowest Factor of Safety (Spencer Method)			
				NonCircular Failure Surface - Proposed Fill		NonCircular Failure Surface - Below Road	
Desert Road	60+10	Northwest Embankment	North	1.25	(Fig. A.1)	2.12	(Fig. A.2)
			South	1.28	(Fig. A.3)	2.51	(Fig. A.4)

Circular Surfaces:
Circular failure surfaces using Spencer and Bishop simplified methods were also evaluated for each scenario. The circular surfaces that were produced were similar in location and size to the noncircular surfaces and had factors of safety similar in magnitude to the noncircular factors of safety. The circular factors of safety ranged from 1.26 to 1.28 for surfaces through the proposed fill and from 2.36 to 2.60 for surfaces extending below the road and through the in situ soil soils, including through the glaciomarine deposit.

3. Repeat the Slide analysis with pseudo-static seismic load conditions.
The same scenarios were also analyzed with a horizontal seismic load coefficient of $A_s/2 = 0.064$ (A_s from Reference 9) as recommended in Reference 11. The results of the seismic Slide stability analyses are summarized in the following table.

Baseline	Station	Feature	Slope	Lowest Factor of Safety (Spencer Method)			
				NonCircular Failure Surface - Proposed Fill		NonCircular Failure Surface - Below Road	
Desert Road	60+10	Northwest Embankment	North	1.08	(Fig. B.1)	1.78	(Fig. B.2)
			South	1.10	(Fig. B.3)	2.13	(Fig. B.4)

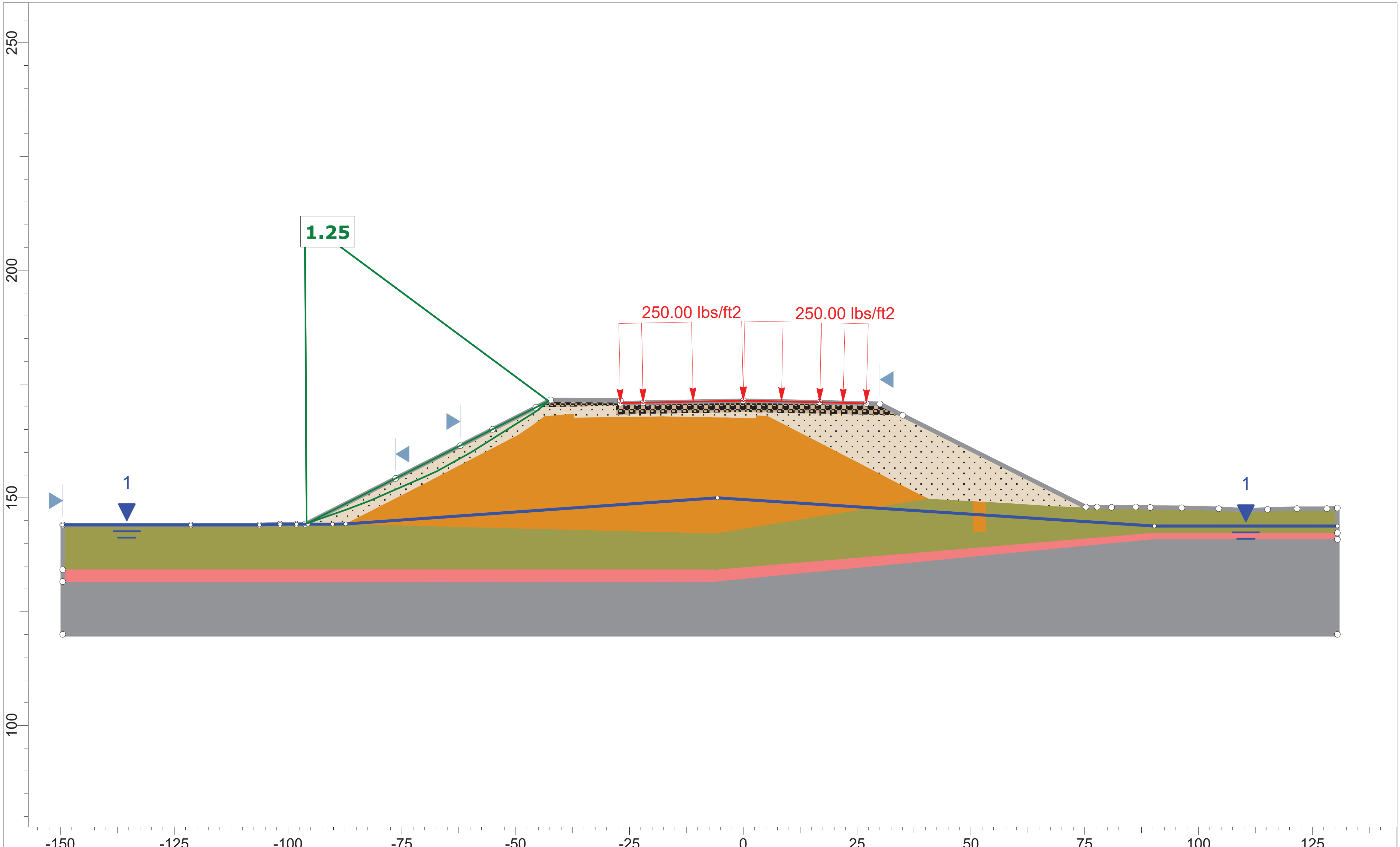
Circular Surfaces:
Circular failure surfaces using Spencer and Bishop simplified methods were also evaluated for each scenario. The circular surfaces that were produced were similar in location and size to the noncircular surfaces and had factors of safety similar in magnitude to the noncircular factors of safety. The circular factors of safety ranged from 1.08 to 1.10 for surfaces through the proposed fill and from 1.94 to 2.21 for surfaces extending below the road and through the in situ soil soils, including through the glaciomarine deposit.

CONCLUSIONS

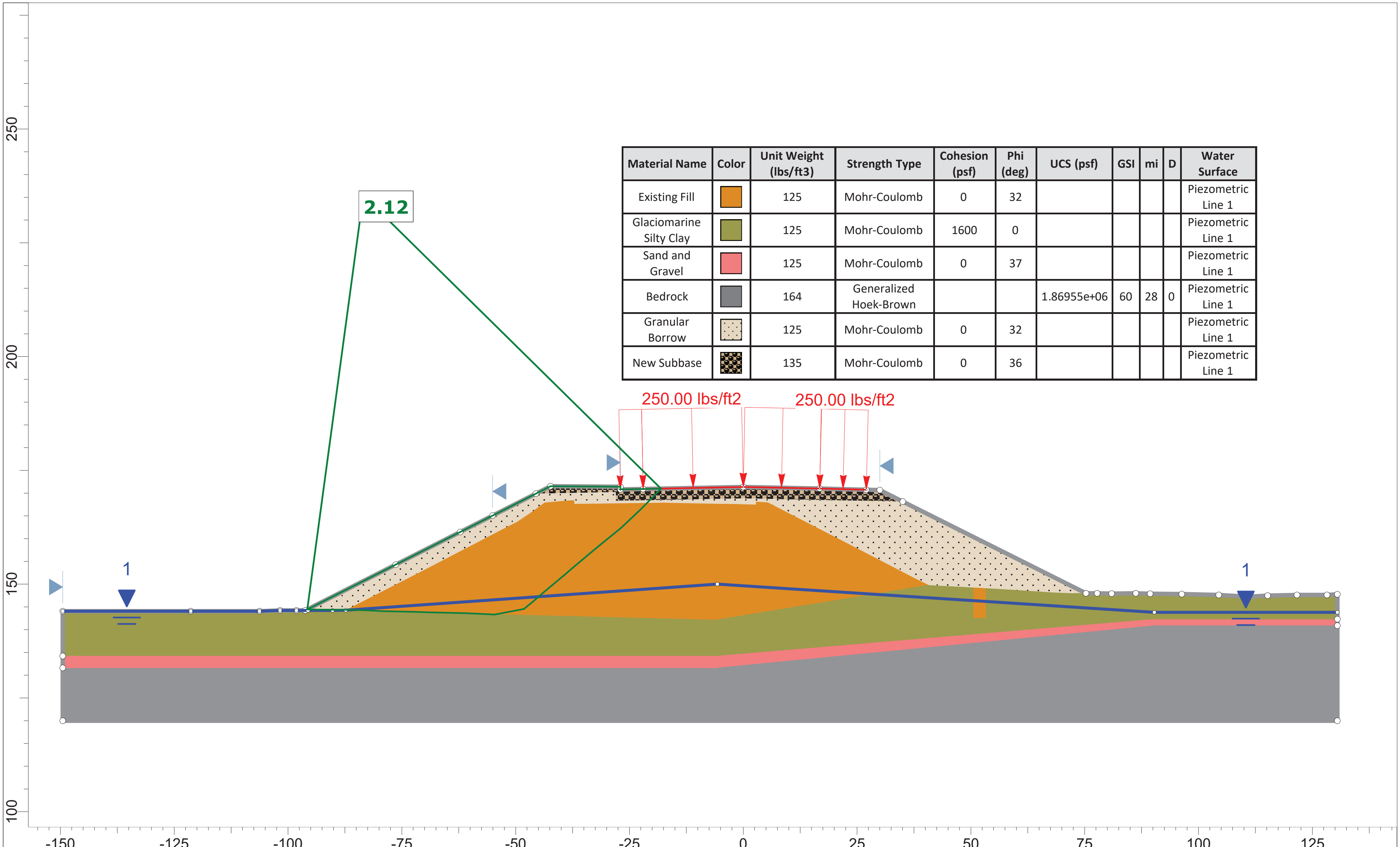
The proposed embankment and slope grading system produces a global stability factor of safety less than the recommended factor of safety of 1.3 for potential surficial slope failures in the embankment fill when using embankment fill engineering parameters recommended in the MaineDOT Bridge Design Guide. Failure surfaces wtih FS < 1.3 are surficial in nature through the embankment slope and not through the in situ soils.

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 > 1.1 based on Ref. 11) 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 with the exception of the existing north slope potential failure surface.



GOLDER MEMBER OF WSP	*Project* 19126013 MaineDOT I-295 Exit 20 Merrill Road Bridge No. 5720		
	Group North Slope Static	*Scenario* **Figure A.1**	
	Drawn By KAR/AH/MEL	*Company* Golder Associates, Inc.	
SLIDEINTERPRET 9.010	*Date* 7/2/2021	*File Name* Desert Rd 60+10 Phase 2 - Seismic.slmd	



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

GOLDER
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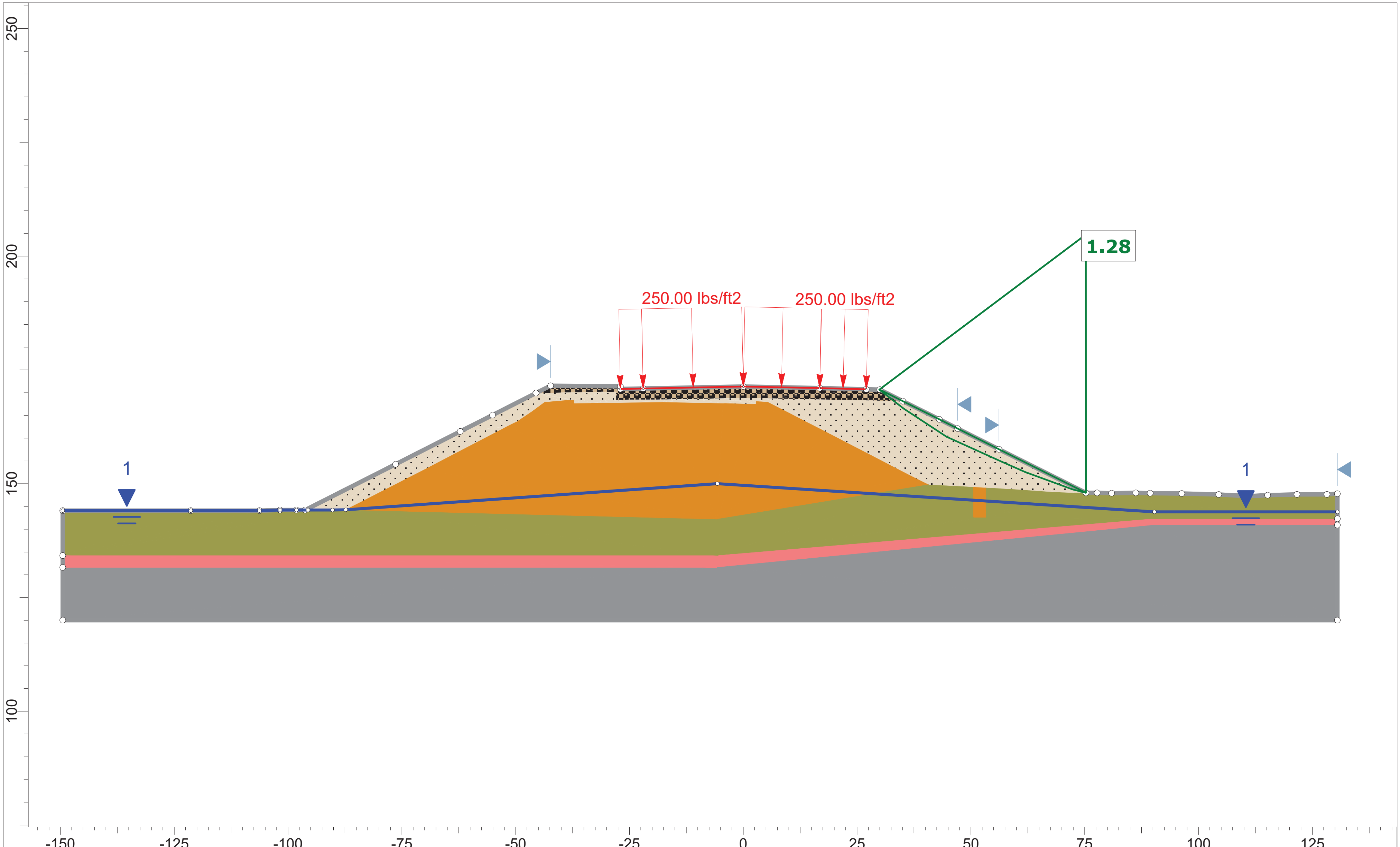
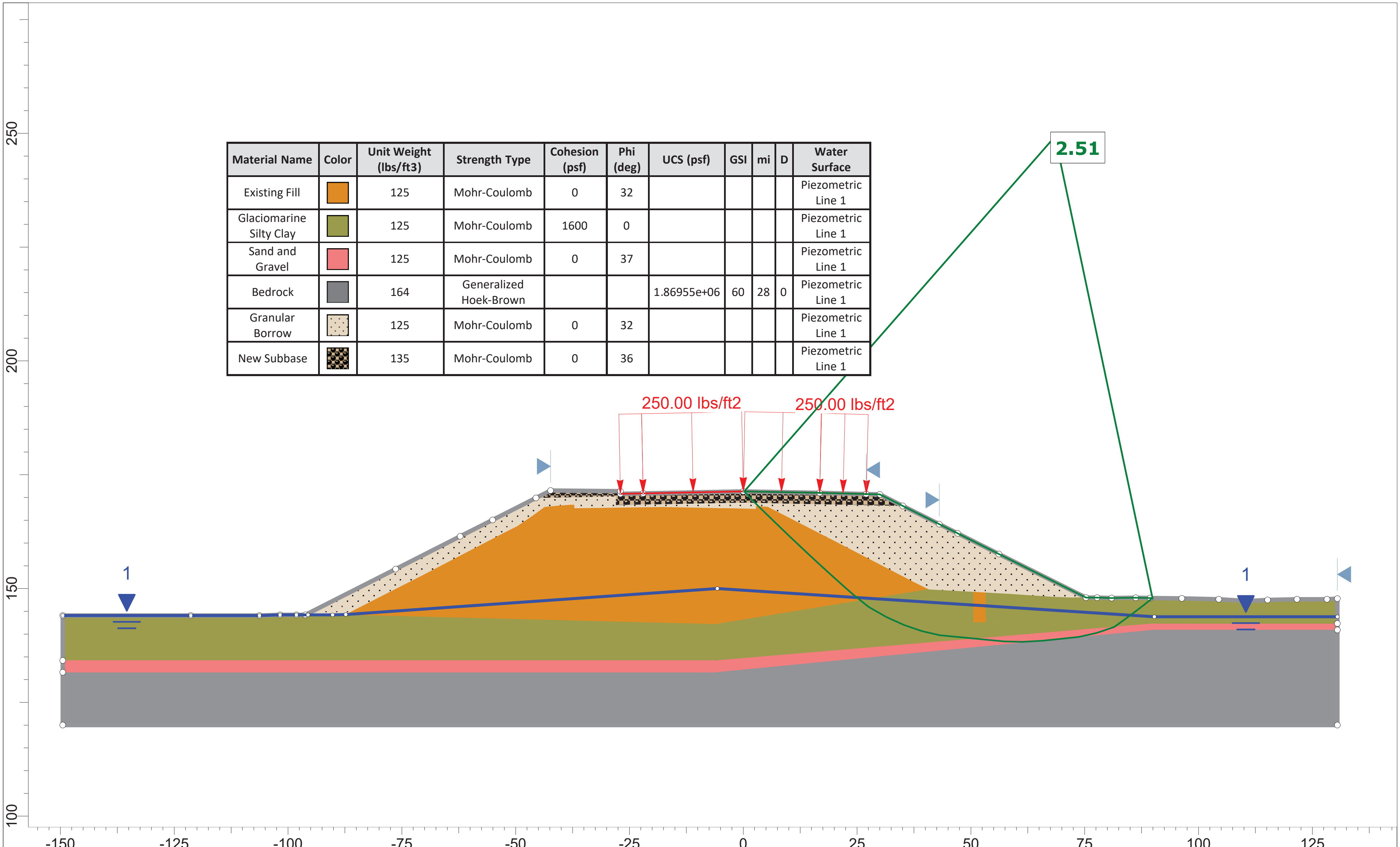
SLIDEINTERPRET 9.010

Project	19126013 MaineDOT I-295 Exit 20 Merrill Road Bridge No. 5720
Group	North Slope Static
Scenario	**Figure A.2**
Drawn By	KAR/AH/MEL
Company	Golder Associates, Inc.
Date	7/2/2021
File Name	Desert Rd 60+10 Phase 2 - Seismic.slmd



GOLDER MEMBER OF WSP	Project	19126013 MaineDOT I-295 Exit 20 Merrill Road Bridge No. 5720	
	Group: South Slope Static	Scenario: **Figure A.3**	
	Drawn By: KAR/AH/MEL	Company: Golder Associates, Inc.	
SLIDEINTERPRET 9.010	Date: 7/2/2021	File Name: Desert Rd 60+10 Phase 2 - Seismic.slmd	



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

GOLDER
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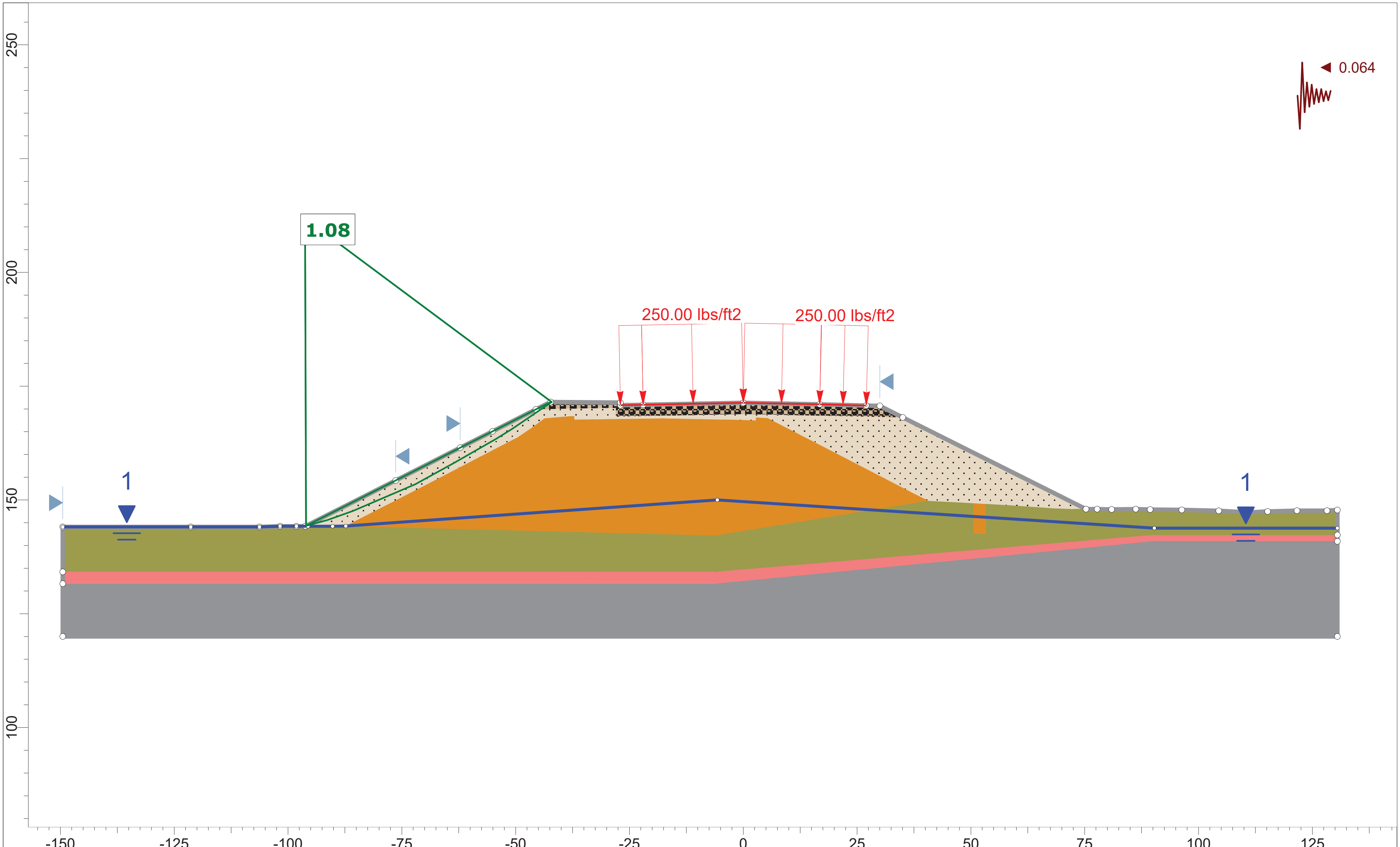
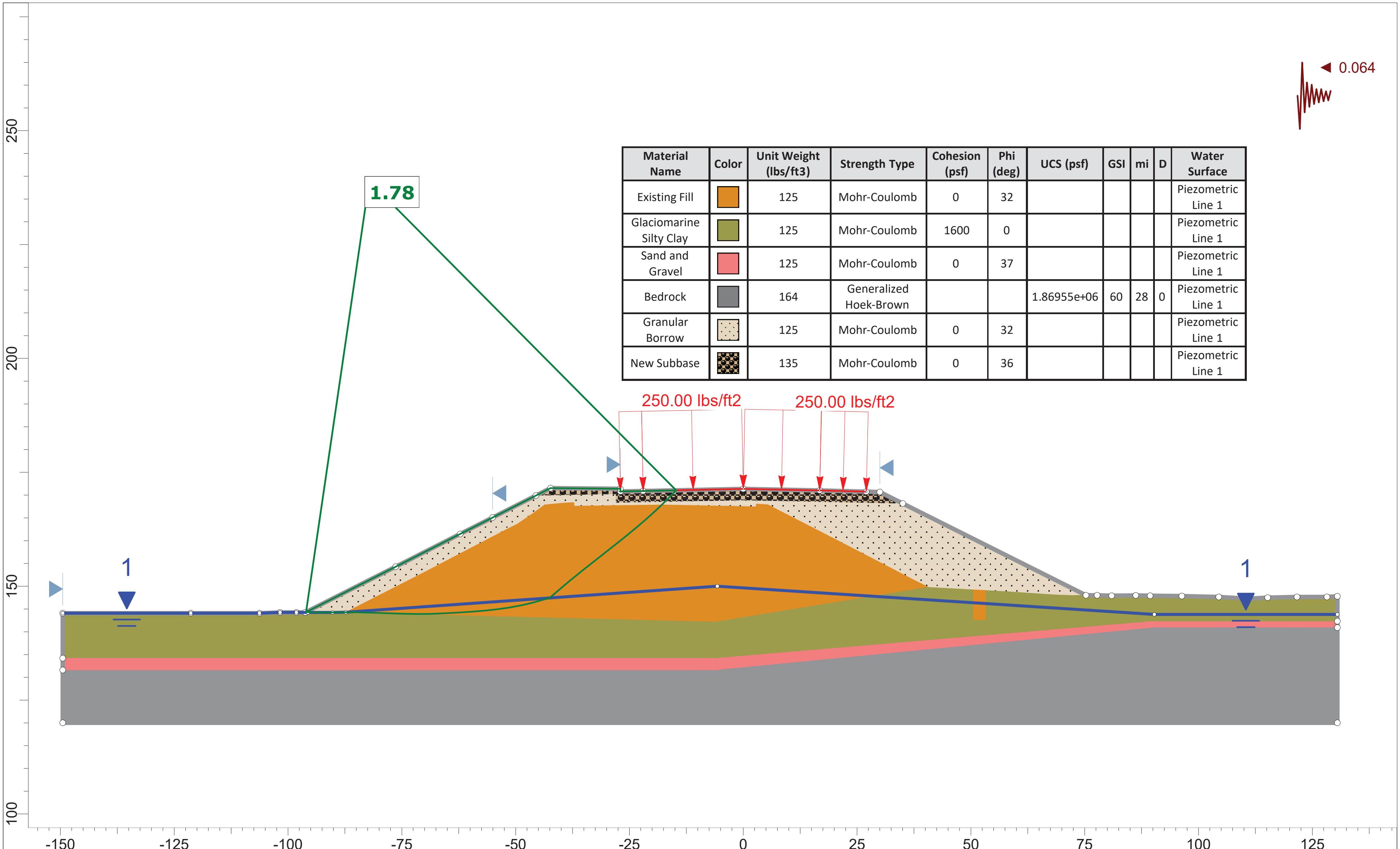
SLIDEINTERPRET 9.010

Project	19126013 MaineDOT I-295 Exit 20 Merrill Road Bridge No. 5720		
Group	South Slope Static	*Scenario*	**Figure A.4**
Drawn By	KAR/AH/MEL	*Company*	Golder Associates, Inc.
Date	7/2/2021	*File Name*	Desert Rd 60+10 Phase 2 - Seismic.slmd



GOLDER MEMBER OF WSP	*Project* 19126013 MaineDOT I-295 Exit 20 Merrill Road Bridge No. 5720		
	Group North Slope Seismic	*Scenario*	**Figure B.1**
	Drawn By KAR/AH/MEL	*Company*	Golder Associates, Inc.
SLIDEINTERPRET 9.010	*Date* 7/2/2021	*File Name*	Desert Rd 60+10 Phase 2 - Seismic.slmd



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	UCS (psf)	GSI	mi	D	Water Surface
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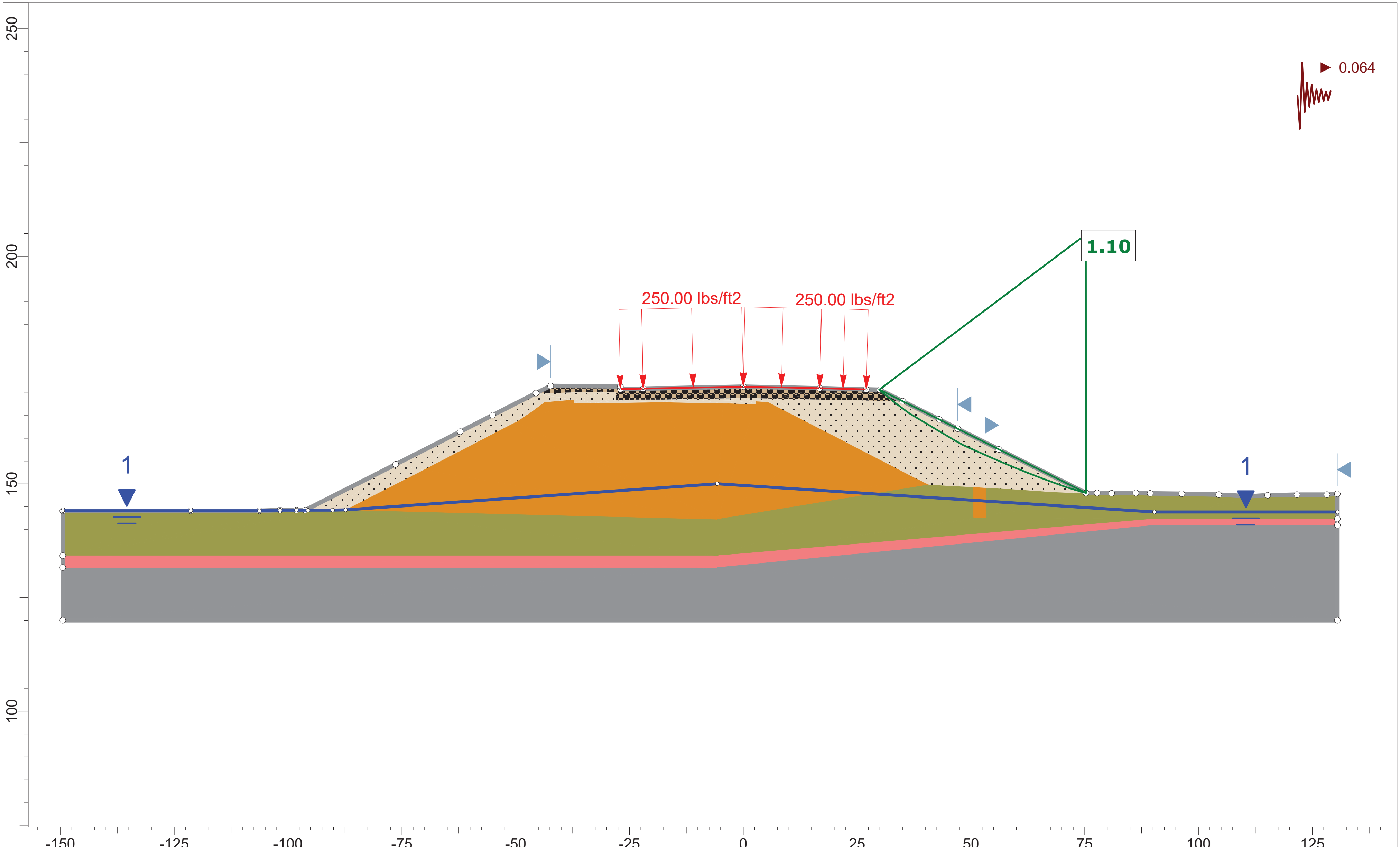
Granular Borrow		125	Mohr-Coulomb	0	32					Piezometric Line 1
New Subbase		135	Mohr-Coulomb	0	36					Piezometric Line 1

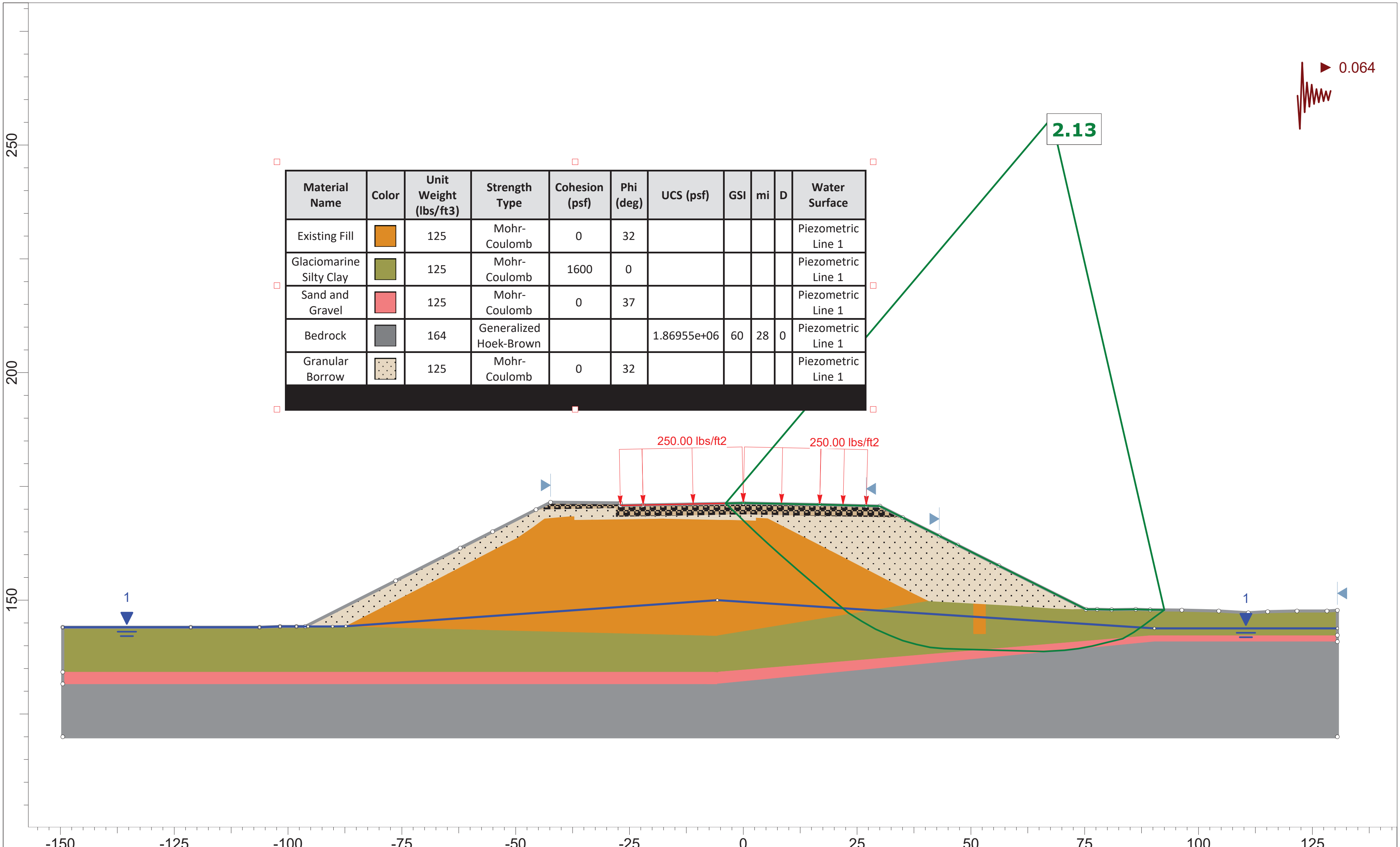
GOLDER
MEMBER OF WSP

SLIDEINTERPRET 9.010

Project	19126013 MaineDOT I-295 Exit 20 Merrill Road Bridge No. 5720		
Group	North Slope Seismic	*Scenario*	**Figure B.2**
Drawn By	KAR/AH/MEL	*Company*	Golder Associates, Inc.
Date	7/2/2021	*File Name*	Desert Rd 60+10 Phase 2 - Seismic.slmd



GOLDER MEMBER OF WSP	Project	19126013 MaineDOT I-295 Exit 20 Merrill Road Bridge No. 5720	
	Group: South Slope Seismic	Scenario: **Figure B.3**	
	Drawn By: KAR/AH/MEL	Company: Golder Associates, Inc.	
SLIDEINTERPRET 9.010	Date: 7/2/2021	File Name: Desert Rd 60+10 Phase 2 - Seismic.slmd	



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	UCS (psf)	GSI	mi	D	Water Surface
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Granular Borrow		125	Mohr-Coulomb	0	32					Piezometric Line 1

[redacted]

GOLDER
MEMBER OF WSP

SLIDEINTERPRET 9.010

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

Group	South Slope Seismic	*Scenario*	**Figure B.4**
Drawn By	KAR/AH/MEL	*Company*	Golder Associates, Inc.
Date	7/2/2021	*File Name*	Desert Rd 60+10 Phase 2 - Seismic.slmd

CALCULATIONS

Date:	7/2/2021	**Made by:**	AH
Project No.:	21450508	**Checked by:**	KAR/MEL
Subject:	Global Stability Analysis Abutment No. 2 Embankment	**Reviewed by:**	JEL
Project Short Title:	MaineDOT I-295 Exit 20 Desert Road Bridge Replacement No. 5720		

OBJECTIVE
Calculate global factor of safety for the Abutment No. 2 proposed bridge approach embankment.

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 9th Ed, 2020.
2. Email communication between Golder and HNTB, subject "RE: Freeport I-295 Exit 20 and Exit 22 - Request for Sections and Design Info", dated June 22, 2020.
3. Golder geotechnical test boring logs for 100-series borings (Appendix A, Preliminary Geotechnical Design Report, dated December 21, 2020).
4. HNTB for State of Maine Department of Transportation. Merrill Road Bridge over Interstate 295 and Signalized Intersections, Exit 20 Interchange: 60% Plans, dated May 21, 2021.
5. Das, Braja M. 2011. Principles of Foundation Engineering, 7th Edition. Cengage Learning.
6. FHWA. 2017. Geotechnical Engineering Circular No. 5: Geotechnical Site Characterization. Publication No. FHWA NHI-16-072.
7. GeoTesting Express laboratory testing results, dated February 5, 2020 (Appendix C, Preliminary Geotechnical Design Report, dated December 21, 2020).
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9. Golder calculation titled "Seismic Site Class" (Appendix E, Preliminary Geotechnical Design Report, dated December 21, 2020).
10. Rocscience Slide Software Package Version 2020 9.007 64-bit, build date May 29, 2020.
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ATTACHMENTS
1. Slide output figures
2. HNTB 60% Plans

ASSUMPTIONS
1. The load applied by the road and traffic for final design conditions is modeled as a 2 ft equivalent load of soil (Reference 1, Table 3.11.6.4-1) based on a 11 ft abutment height (Reference 2). 2 ft x 125 pcf (fill) = 250 psf.

2. A static FS ≥ 1.3 is recommended for embankment final design conditions per Section 5.9.2 in Reference 8. A pseudo-static FS > 1.1 is recommended in Reference 11.

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. The stratigraphy was not updated based 200-series borings and rock probes near this abutment.

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

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

Date:	7/2/2021	**Made by:**	AH
Project No.:	21450508	**Checked by:**	KAR/MEL
Subject:	Global Stability Analysis Abutment No. 2 Embankment	**Reviewed by:**	JEL
Project Short Title:	MaineDOT I-295 Exit 20 Desert Road Bridge Replacement No. 5720		

CALCULATION

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

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

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

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

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

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

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

Material Name	Unit Weight (pcf)	Strength Type	Cohesion (psf)	Friction Angle (°)	UCS (psf)	GSI	m_i	D
Existing Fill	125	Mohr-Coulomb	0	32	-	-	-	-
Glaciomarine Silty Clay	125	Mohr-Coulomb	1600	0	-	-	-	-
Sand and Gravel	125	Mohr-Coulomb	0	37	-	-	-	-
Bedrock	164	Generalized Hoek-Brown	-	-	1,869,552	60	28	0
Granular Borrow	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. This analysis evaluates the fills at Abutment No. 2. Both the northern and the southern fill slopes were analyzed at Abutment No. 2. The results of the Slide stability analyses are summarized in the following table.

Date:	7/2/2021	**Made by:**	AH
Project No.:	21450508	**Checked by:**	KAR/MEL
Subject:	Global Stability Analysis Abutment No. 2 Embankment	**Reviewed by:**	JEL
Project Short Title:	MaineDOT I-295 Exit 20 Desert Road Bridge Replacement No. 5720		

Baseline	Station	Feature	Slope	Lowest Factor of Safety (Spencer Method)	
				NonCircular Failure Surface - Proposed Fill	NonCircular Failure Surface - Below Road
Desert Road	62+75	Southeast Embankment	North	1.29 (Fig. A.1)	2.27 (Fig. A.2)
			South	1.27 (Fig. A.3)	2.17 (Fig. A.4)

Circular Surfaces:
Circular failure surfaces using Spencer and Bishop simplified methods were also evaluated for each scenario. The circular surfaces that were produced were similar in location and size to the noncircular surfaces and had factors of safety similar in magnitude to the noncircular factors of safety. The circular factors of safety ranged from 1.27 to 1.29 for surfaces through the proposed fill and from 2.33 to 2.37 for surfaces extending below the road and through the in situ soil soils, including through the glaciomarine deposit.

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

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

Baseline	Station	Feature	Slope	Lowest Factor of Safety (Spencer Method)	
				NonCircular Failure Surface - Proposed Fill	NonCircular Failure Surface - Below Road
Desert Road	62+75	Southeast Embankment	North	1.11 (Fig. B.1)	1.92 (Fig. B.2)
			South	1.09 (Fig. B.3)	1.87 (Fig. B.4)

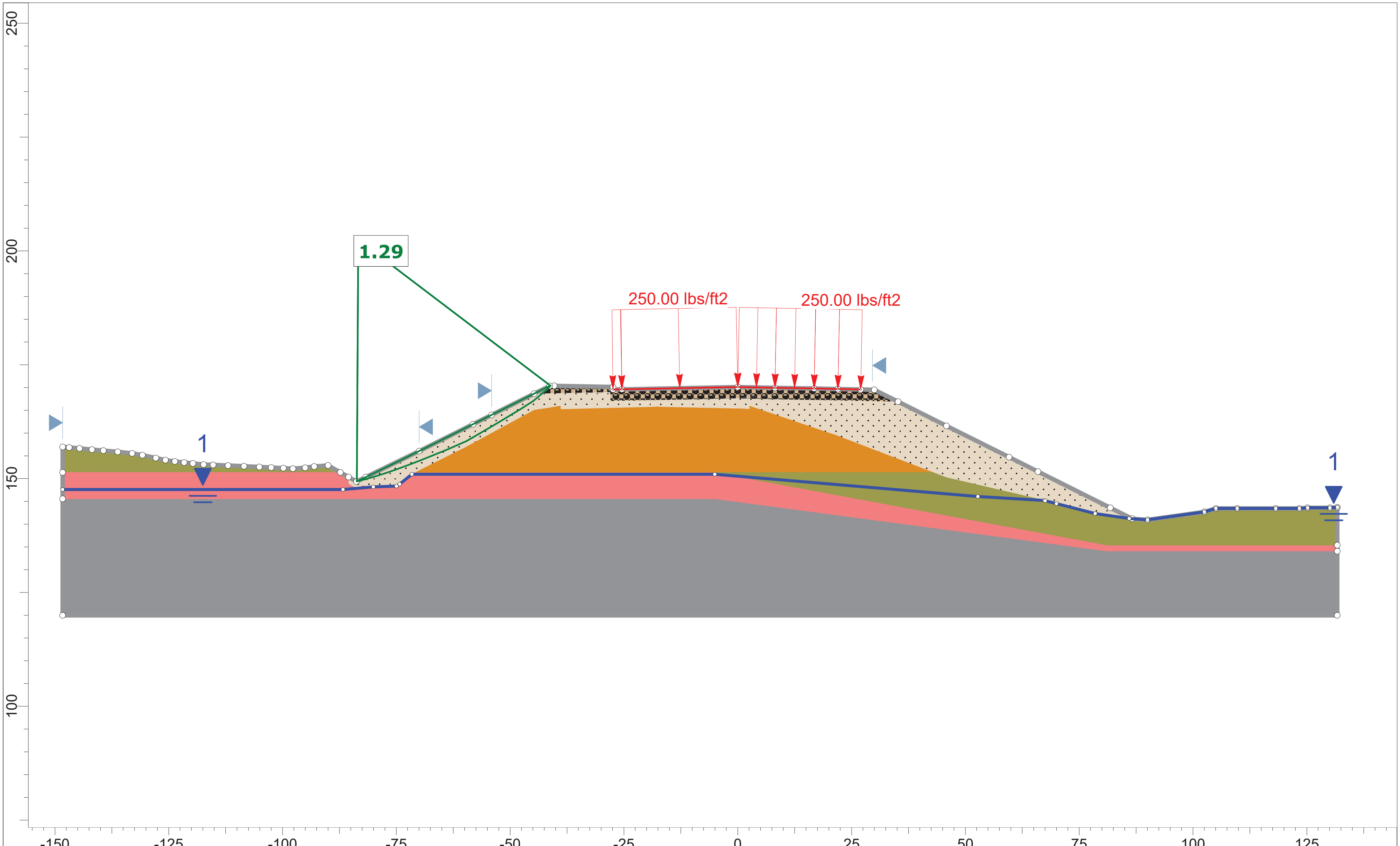
Circular Surfaces:
Circular failure surfaces using Spencer and Bishop simplified methods were also evaluated for each scenario. The circular surfaces that were produced were similar in location and size to the noncircular surfaces and had factors of safety similar in magnitude to the noncircular factors of safety. The circular factors of safety ranged from 1.09 to 1.11 for surfaces through the proposed fill and from 1.98 to 2.02 for surfaces extending below the road and through the in situ soil soils, including through the glaciomarine deposit.

CONCLUSIONS

The proposed embankment and slope grading system produces a global stability factor of safety less than the recommended factor of safety of 1.3 for potential surficial slope failures in the embankment fill when using embankment fill engineering parameters recommended in the MaineDOT Bridge Design Guide. Failure surfaces wtih FS < 1.3 are surficial in nature through the embankment slope and not through the in situ soils.

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 > 1.1 based on Ref. 11) 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 with the exception of the existing north slope potential failure surface.



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

Group	North Slope Static	*Scenario*	**Figure A.1**
Drawn By	KAR/AH/MEL	*Company*	Golder Associates, Inc.
Date	7/2/2021	*File Name*	Desert Rd 62+75 Phase 2 - Seismic.slmd

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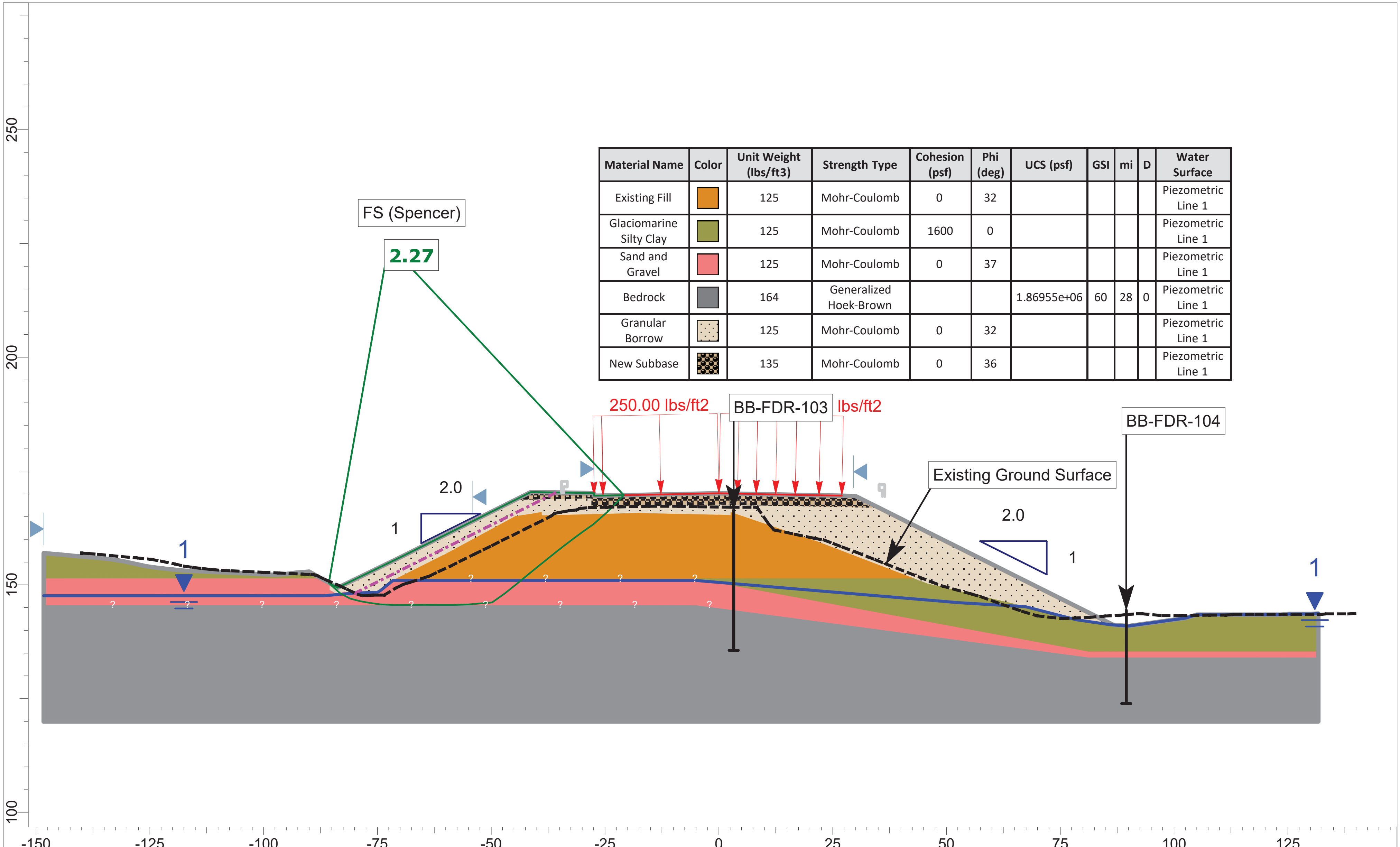

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	UCS (psf)	GSI	mi	D	Water Surface
Existing Fill		125	Mohr-Coulomb	0	32					Piezometric Line 1
Glaciomarine Silty Clay		125	Mohr-Coulomb	1600	0					Piezometric Line 1
Sand and Gravel		125	Mohr-Coulomb	0	37					Piezometric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

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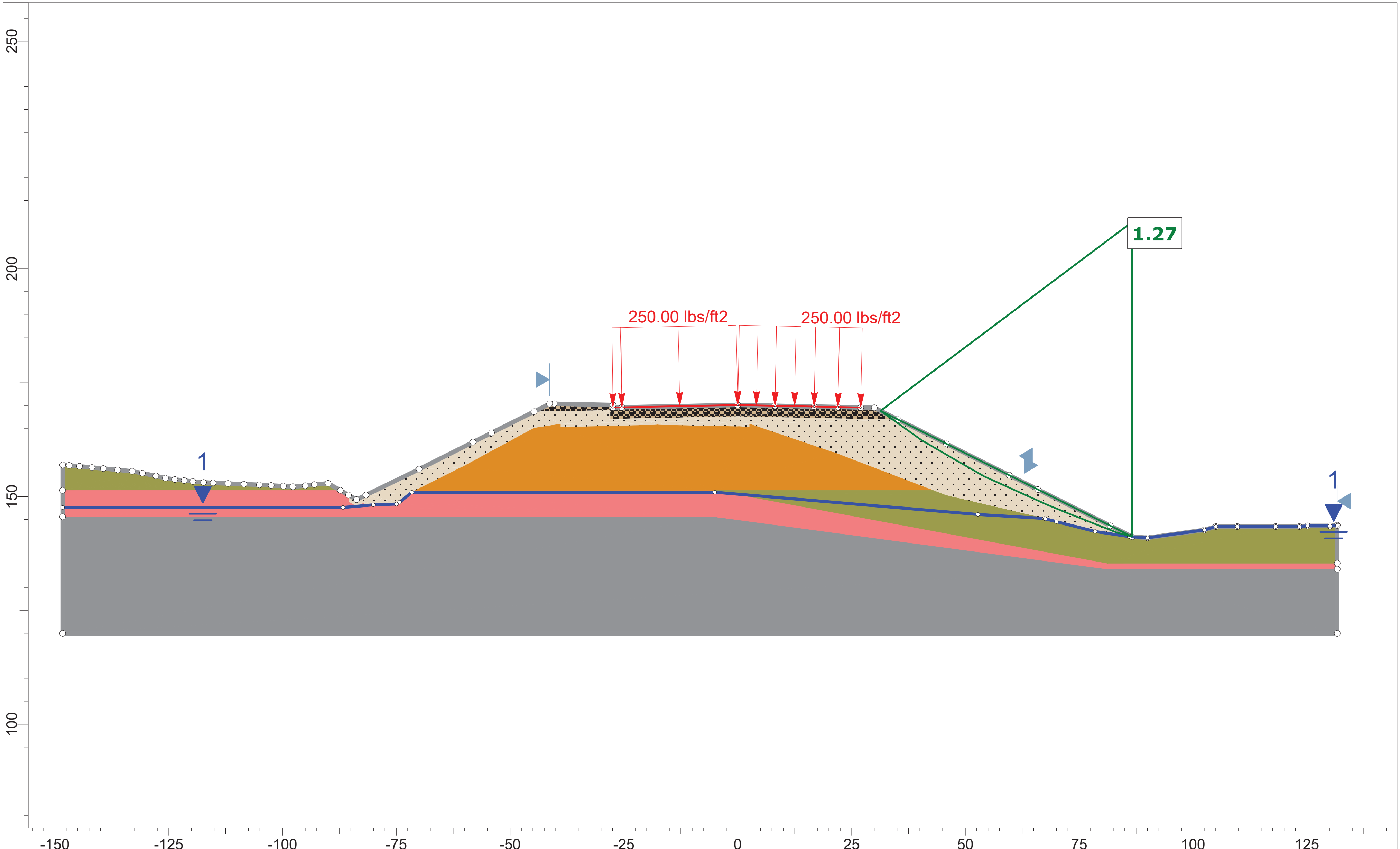
SLIDEINTERPRET 9.010

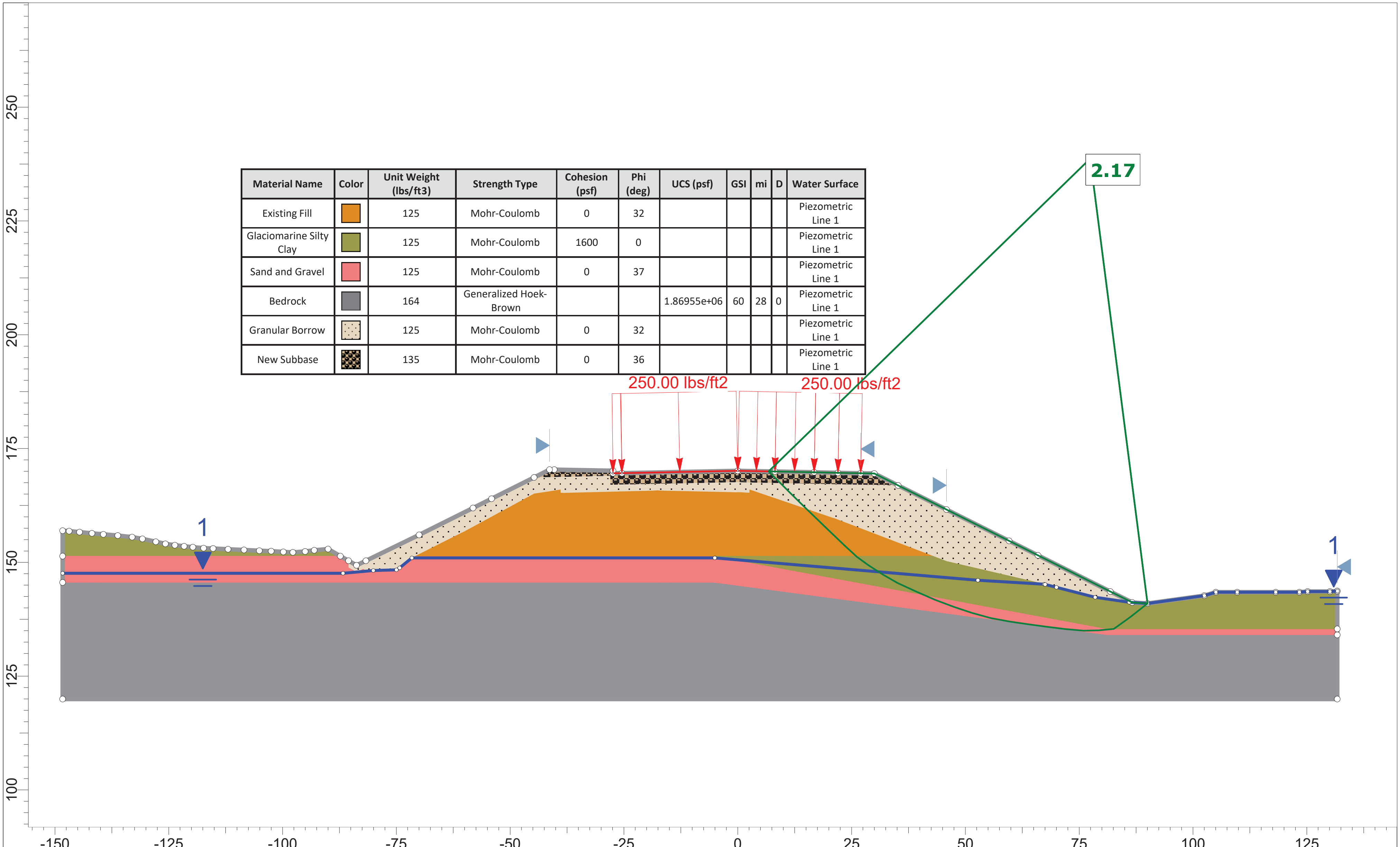
Project	19126013 MaineDOT I-295 Exit 20 Merrill Road Bridge No. 5720		
Group	North Slope Static	*Scenario*	**Figure A.2**
Drawn By	KAR/AH/MEL	*Company*	Golder Associates, Inc.
Date	7/2/2021	*File Name*	Desert Rd 62+75 Phase 2 - Seismic.slmd



GOLDER
MEMBER OF WSP

SLIDEINTERPRET 9.010

Project	19126013 MaineDOT I-295 Exit 20 Merrill Road Bridge No. 5720		
Group	South Slope Static	Scenario	**Figure A.3**
Drawn By	KAR/AH/MEL	Company	Golder Associates, Inc.
Date	7/2/2021	File Name	Desert Rd 62+75 Phase 2 - Seismic.slmd



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

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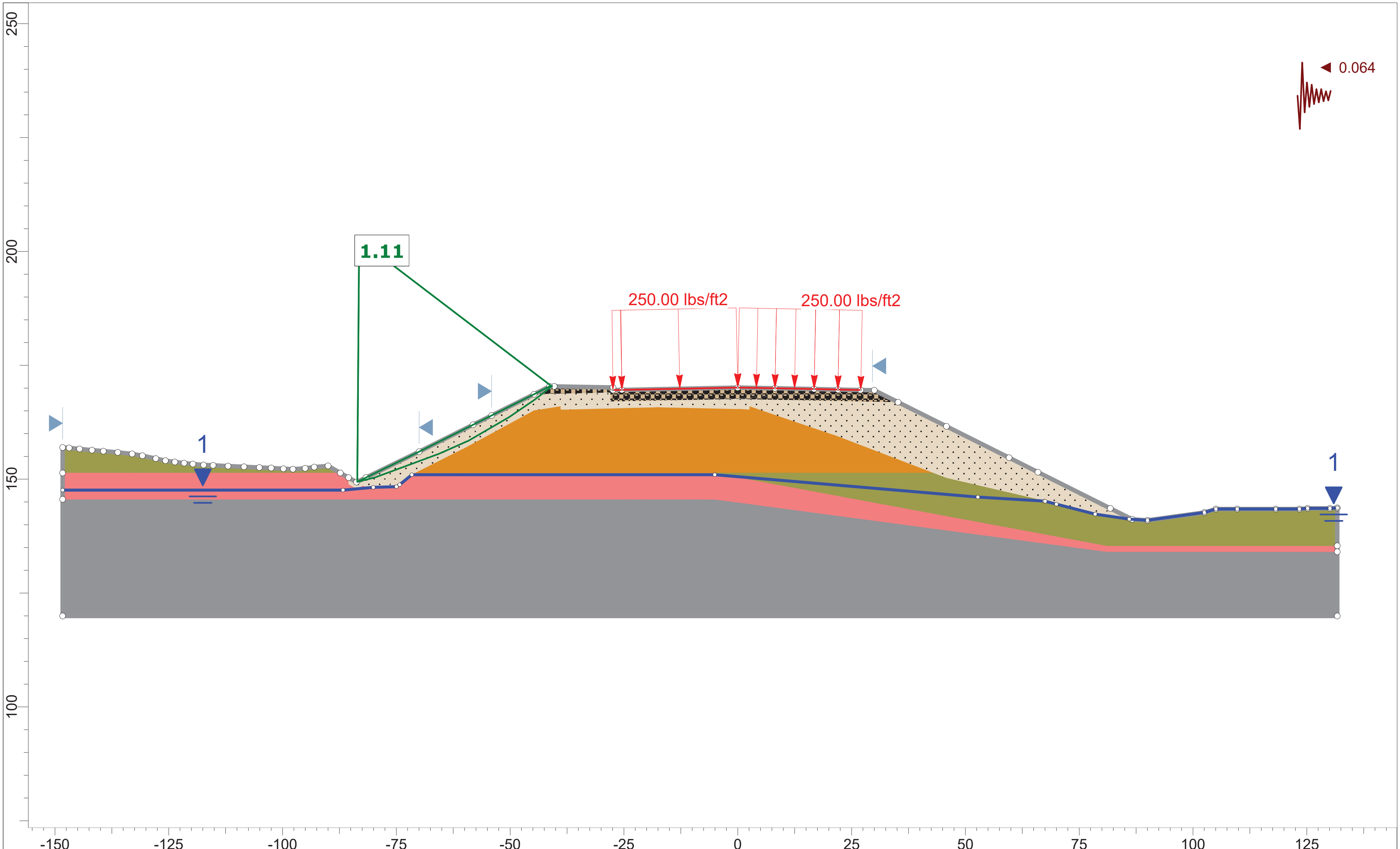
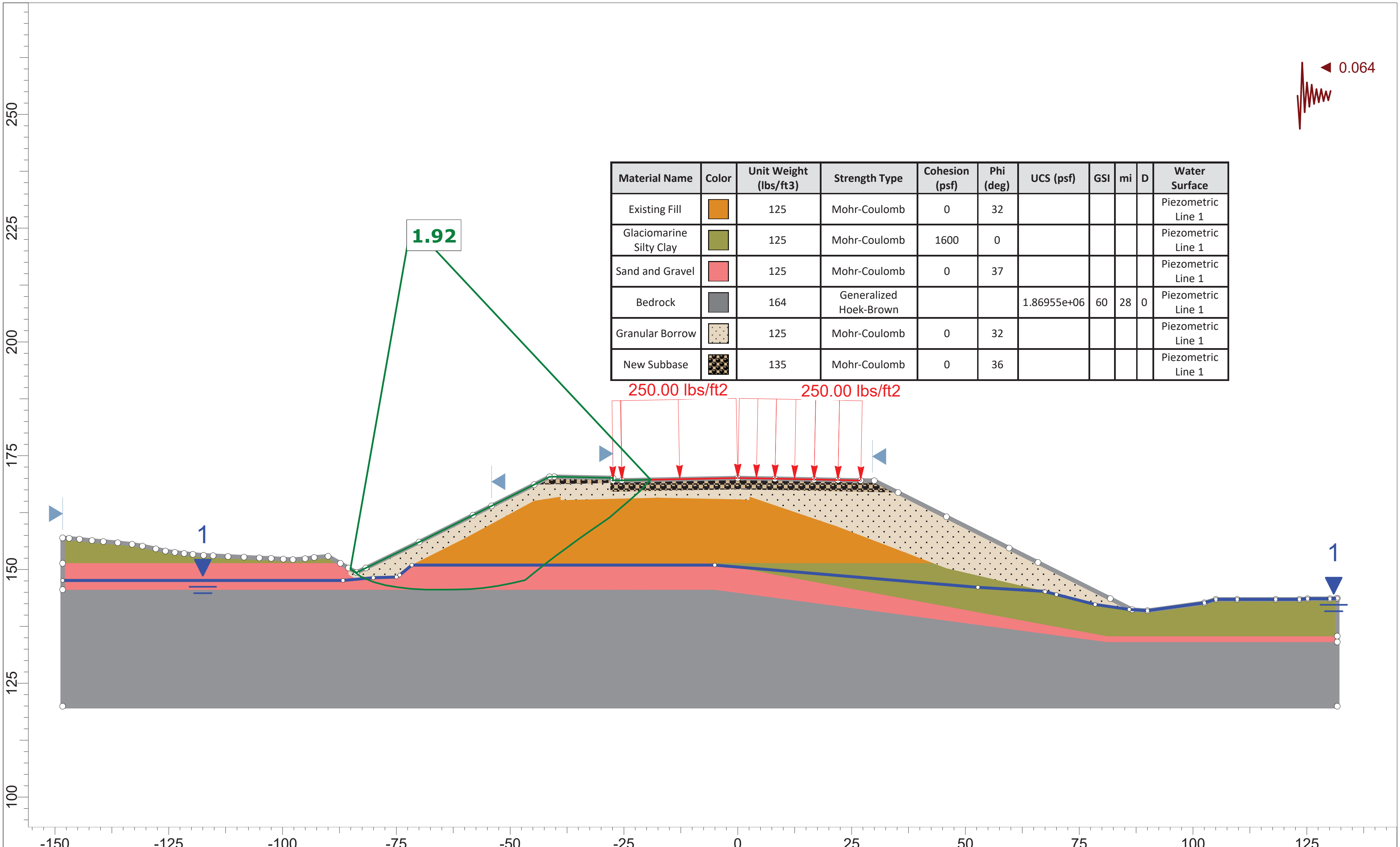
SLIDEINTERPRET 9.010

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

Group	South Slope Static	*Scenario*	**Figure A.4**
Drawn By	KAR/AH/MEL	*Company*	Golder Associates, Inc.
Date	7/2/2021	*File Name*	Desert Rd 62+75 Phase 2 - Seismic.slmd



GOLDER MEMBER OF WSP	*Project* 19126013 MaineDOT I-295 Exit 20 Merrill Road Bridge No. 5720		
	Group North Slope Seismic	*Scenario*	**Figure B.1**
	Drawn By KAR/AH/MEL	*Company*	Golder Associates, Inc.
SLIDEINTERPRET 9.010	*Date* 7/2/2021	*File Name*	Desert Rd 62+75 Phase 2 - Seismic.slmd



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

GOLDER
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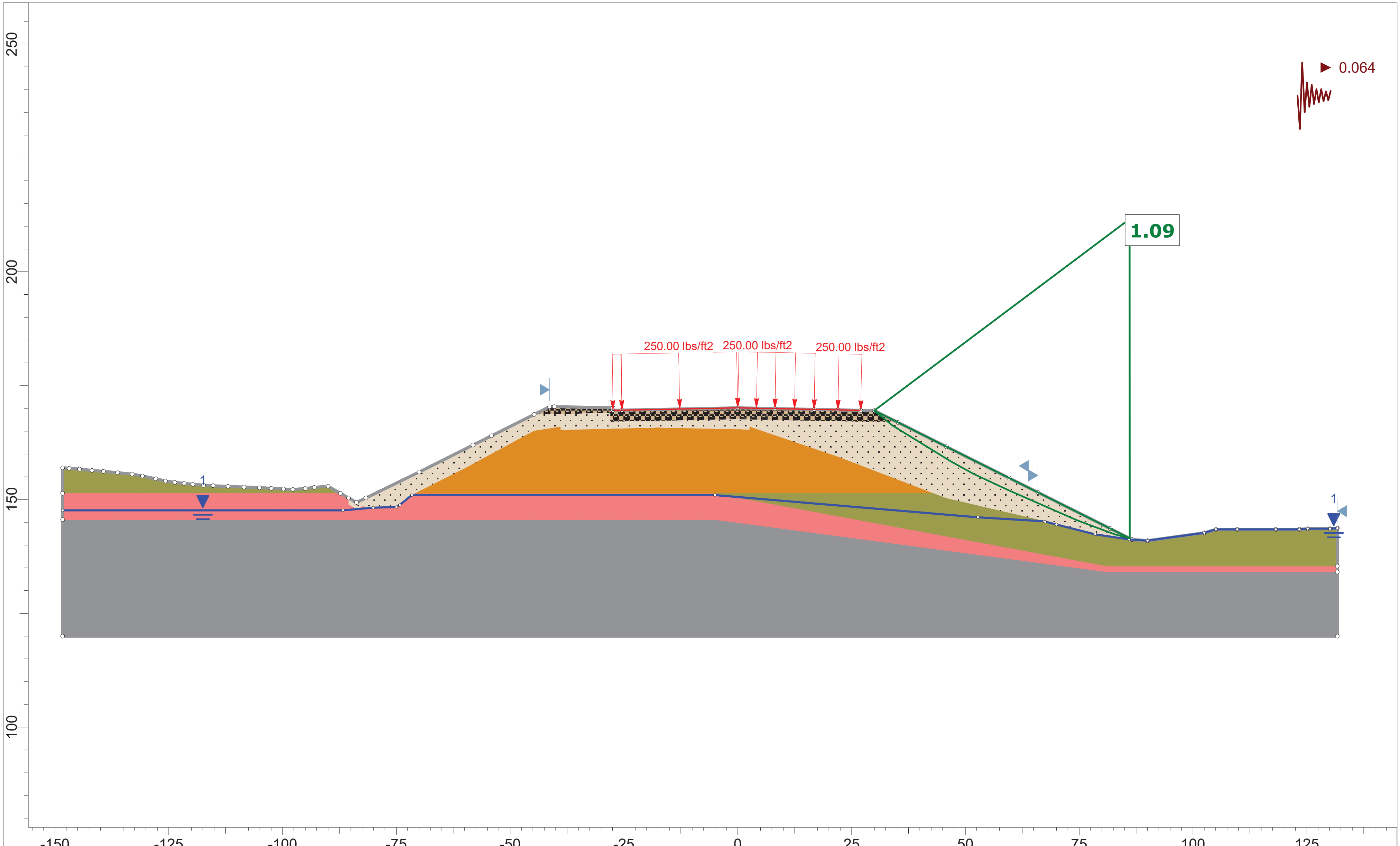
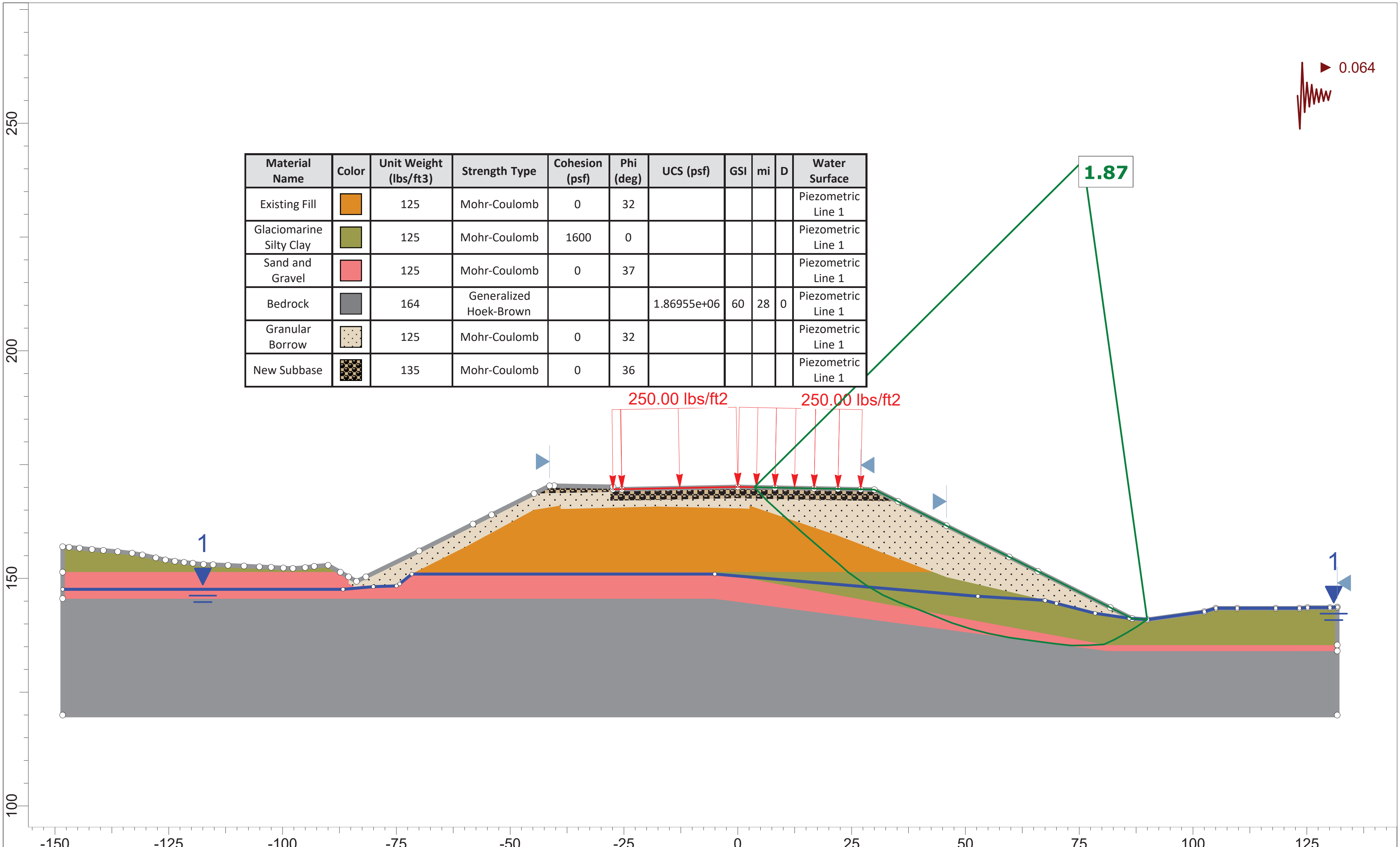
Project	19126013 MaineDOT I-295 Exit 20 Merrill Road Bridge No. 5720		
Group	North Slope Seismic	*Scenario*	**Figure B.2**
Drawn By	KAR/AH/MEL	*Company*	Golder Associates, Inc.
Date	7/2/2021	*File Name*	Desert Rd 62+75 Phase 2 - Seismic.slmd

SLIDEINTERPRET 9.010



GOLDER MEMBER OF WSP	*Project*	19126013 MaineDOT I-295 Exit 20 Merrill Road Bridge No. 5720	
	Group South Slope Seismic	*Scenario*	**Figure B.3**
	Drawn By KAR/AH/MEL	*Company*	Golder Associates, Inc.
SLIDEINTERPRET 9.010	*Date* 7/2/2021	*File Name*	Desert Rd 62+75 Phase 2 - Seismic.slmd

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



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Project	19126013 MaineDOT I-295 Exit 20 Merrill Road Bridge No. 5720		
Group	South Slope Seismic	*Scenario*	**Figure B.4**
Drawn By	KAR/AH/MEL	*Company*	Golder Associates, Inc.
Date	7/2/2021	*File Name*	Desert Rd 62+75 Phase 2 - Seismic.slmd

APPENDIX C

Settlement

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Settle3 Model Development & Results

Date:	6/29/2021	**Made by:**	MEL
Project No.:	21450908	**Checked by:**	DAF
Subject:	Basis for Model Development: 3D Settlement for Site Improvements	**Reviewed by:**	JEL
Project Short Title:	MaineDOT WIN 023627.00 - Freeport Exit 20 Desert Road		

OBJECTIVE

Estimate the settlement expected to take place at the proposed embankments and abutment and pile locations using a three-dimensional model of the proposed roadway and embankment fills.

REFERENCES

1. Geotechnical test boring logs: Appendix A in: Golder Associates, Inc., Preliminary Geotechnical Design Report: I-295 Desert Road Bridge Replacement #5720 (Exit 20), Freeport, Maine, MaineDOT WIN 023627.00, December 21, 2020.
2. Subsurface investigation laboratory testing results: Appendix D in: Golder Associates, Inc., Preliminary Geotechnical Design Report: I-295 Desert Road Bridge Replacement #5720 (Exit 20), Freeport, Maine, MaineDOT WIN 023627.00, December 21, 2020.
3. Summary of Laboratory Soil Index and Classification Test Results: Table 5 in: Golder Associates, Inc., Preliminary Geotechnical Design Report: I-295 Desert Road Bridge Replacement #5720 (Exit 20), Freeport, Maine, MaineDOT WIN 023627.00, December 21, 2020.
4. Interpreted Subsurface Profile, Sheet 3 from the report and updated from Sheet 3 in: Golder Associates, Inc., Preliminary Geotechnical Design Report: I-295 Desert Road Bridge Replacement #5720 (Exit 20), Freeport, Maine, MaineDOT WIN 023627.00, December 21, 2020.
5. HNTB, May 21, 2021, Freeport Merrill Road Bridge, Interstate 295: 60% Plans, Filename: Freeport_023627_Exit_20_60%25_Plans.pdf.
6. Rocscience Settle3 Ver. 5.006, Build date: June 30, 2021
7. Das, B. M. (1997). Principles of geotechnical engineering. Boston: PWS.

MODEL PARAMETERS

1. The existing stratigraphy (Ref. 4) and water table surface are estimated from samples and groundwater measurements encountered during drilling (Ref. 1) and interpreted across the site (Ref. 4).

2. Glaciomarine unit weight and consolidation parameters are based on laboratory measured parameters in Golder's historical project database for this unit (Presumpscot Formation) from regional project locations. We assume the clay is sufficiently overconsolidated and will experience recompression settlement only after loading. CR and CC are the strain-based recompression and compression ratios, respectively. Cr and Cc are the void ratio based recompression and compression indices, respectively. Void ratio, e, is estimated based on the average water content from Ref. 3 and an assumed specific gravity of G_s = 2.7.

γ (pcf) =	125
CC =	0.25
C_c =	0.42
CR =	0.02
C_r =	0.03
c_v (ft^2/year)=	120
c_v (ft^2/day)=	0.33
e_0 =	0.68
OCR	> 3.0

3. Material properties for the Fill and Sand and Gravel (Ref. 4) were correlated using average of the N-values encountered in all borings for each layer (Ref. 1), and are as follows:

N_{Fill} =	23
$N_{Sand\&Gravel}$ =	13

4. The existing topographic elevations for all site features and the proposed elevations for the embankments, roadway, and MSE walls are provided in Ref. 5.

5. The Settle3 model (Ref. 6) is from Station 57+00 To Station 64+00 and includes proposed design features provided in Ref. 5 between these stations.

6. Cohesionless materials were modeled to have elastic settlment only. Cohesive soils were modeled to have consolidation settlement only - elastic settlment and secondary compression are not considered.

Settle3 Model Development & Results

Date:	6/29/2021	**Made by:**	MEL
Project No.:	21450908	**Checked by:**	DAF
Subject:	Basis for Model Development: 3D Settlement for Site Improvements	**Reviewed by:**	JEL
Project Short Title:	MaineDOT WIN 023627.00 - Freeport Exit 20 Desert Road		

A. Coarse-grained soil modulus E_s for elastic settlement (using Reference 7).

$$\frac{E_s}{p_a} = \alpha N_{60}$$

atmospheric pressure, p_a (ksf) =	2.116	
α =	5	sand with fines
	10	clean normally consolidated sand
	15	clean overconsolidated sand

Material	Material Info	N_{60}	α	E_s (ksf)	γ (pcf)
Fill	fine to coarse Sand, some silt, little gravel	23	5	244	125
Sand and Gravel	Gravel or fine to coarse Sand, some silt	13	5	138	125

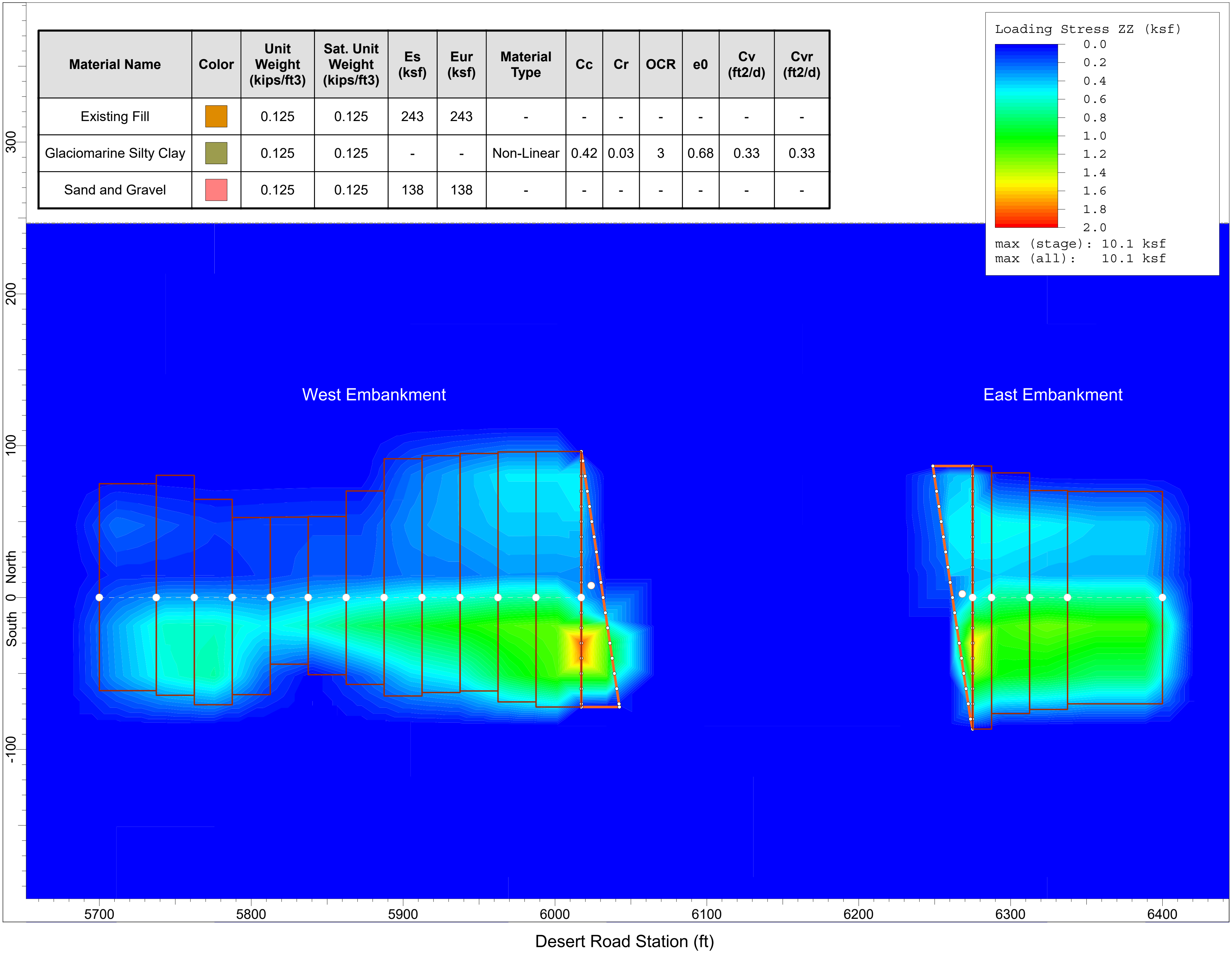
Unit weight assumed based on information in Ref. 1, Ref. 2, and Ref. 3

MODEL RESULTS

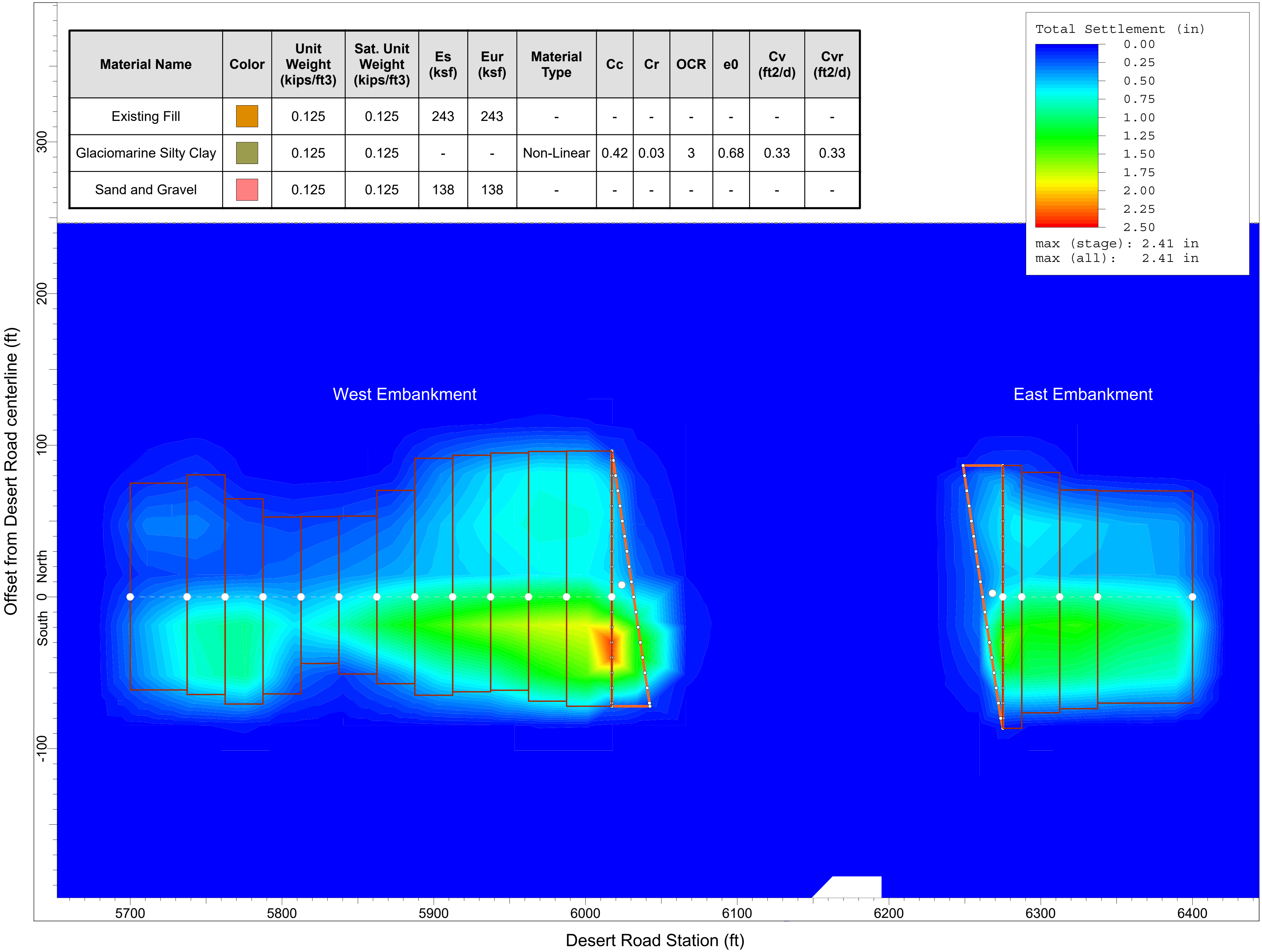
Attachment 1 - Loading from Proposed Site Improvements West and East of I-295 Median
Attachment 2 - Total Settlement from Proposed Site Improvements West and East of I-295 Median
Attachment 3 - Total Settlement from Proposed Site Improvements along Abutment 1
Attachment 4 - Total Settlement from Proposed Site Improvements along Abutment 2
Attachment 5 - Total Settlement from Proposed Site Improvements at Station 60+10
Attachment 6 - Total Settlement from Proposed Site Improvements at Station 62+75
Attachment 7 - Total Settlement from Proposed Site Improvements along Profile A-A'

Attachment 1 - Loading from Proposed Site Improvements West and East of I-295 Median

Material Name	Color	Unit Weight (kips/ft3)	Sat. Unit Weight (kips/ft3)	Es (ksf)	Eur (ksf)	Material Type	Cc	Cr	OCR	e0	Cv (ft2/d)	Cvr (ft2/d)
Existing Fill		0.125	0.125	243	243	-	-	-	-	-	-	-
Glaciomarine Silty Clay		0.125	0.125	-	-	Non-Linear	0.42	0.03	3	0.68	0.33	0.33
Sand and Gravel		0.125	0.125	138	138	-	-	-	-	-	-	-

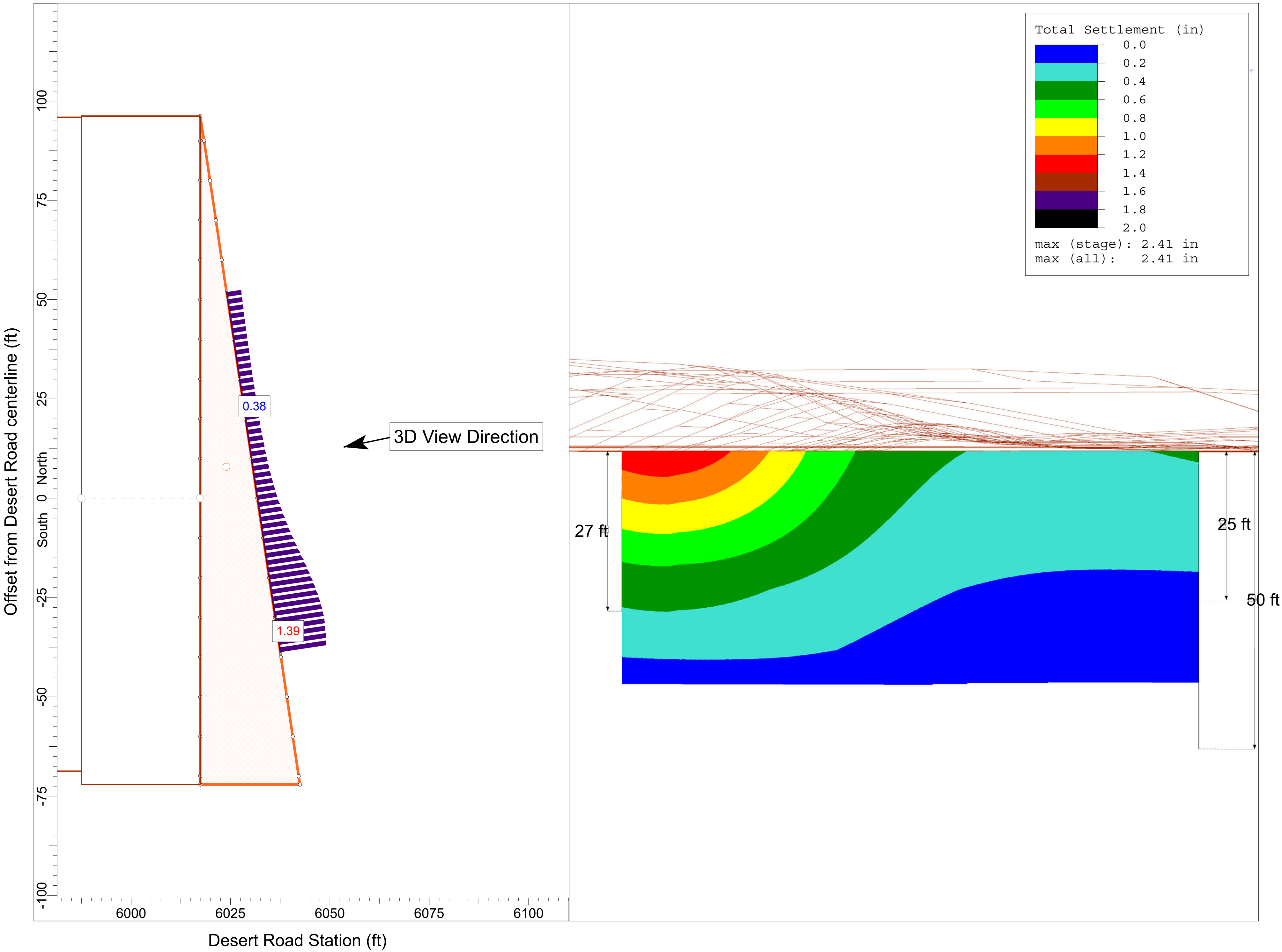


Attachment 2 - Total Settlement from Proposed Site Improvements West and East of I-295 Median

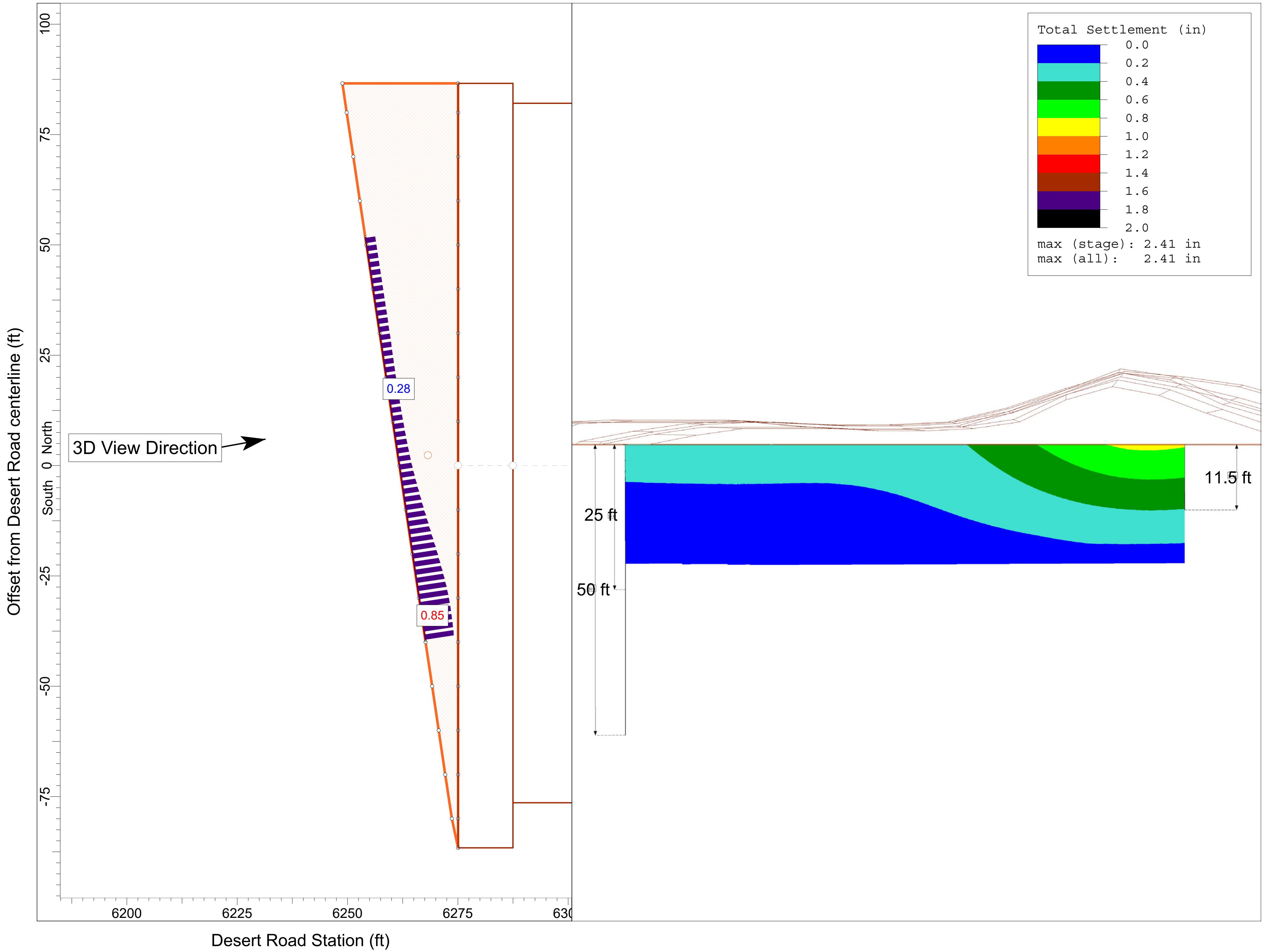


Material Name	Color	Unit Weight (kips/ft3)	Sat. Unit Weight (kips/ft3)	Es (ksf)	Eur (ksf)	Material Type	Cc	Cr	OCR	e0	Cv (ft2/d)	Cvr (ft2/d)
Existing Fill		0.125	0.125	243	243	-	-	-	-	-	-	-
Glaciomarine Silty Clay		0.125	0.125	-	-	Non-Linear	0.42	0.03	3	0.68	0.33	0.33
Sand and Gravel		0.125	0.125	138	138	-	-	-	-	-	-	-

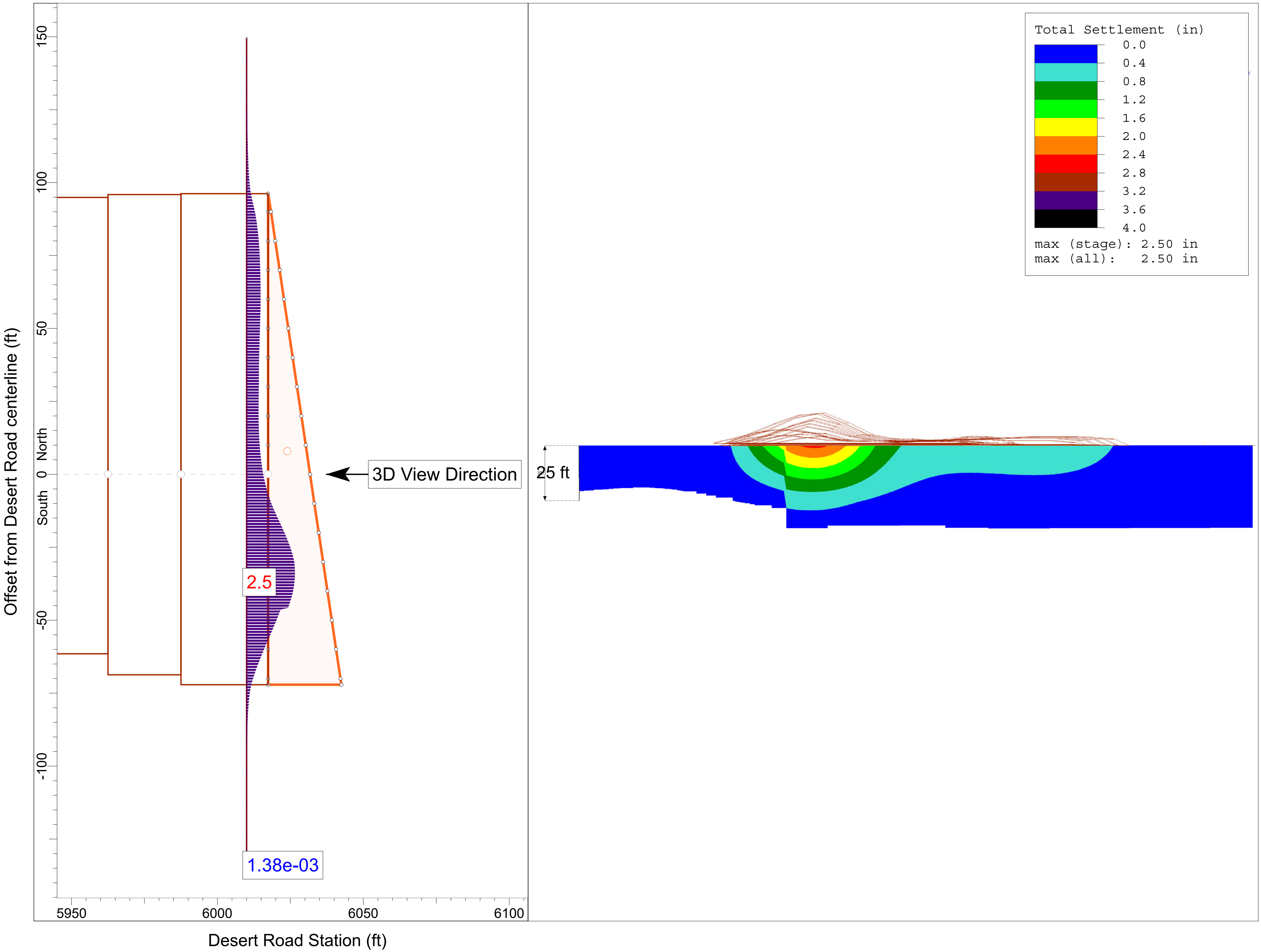
Attachment 3 - Total Settlement from Proposed Site Improvements along Abutment 1



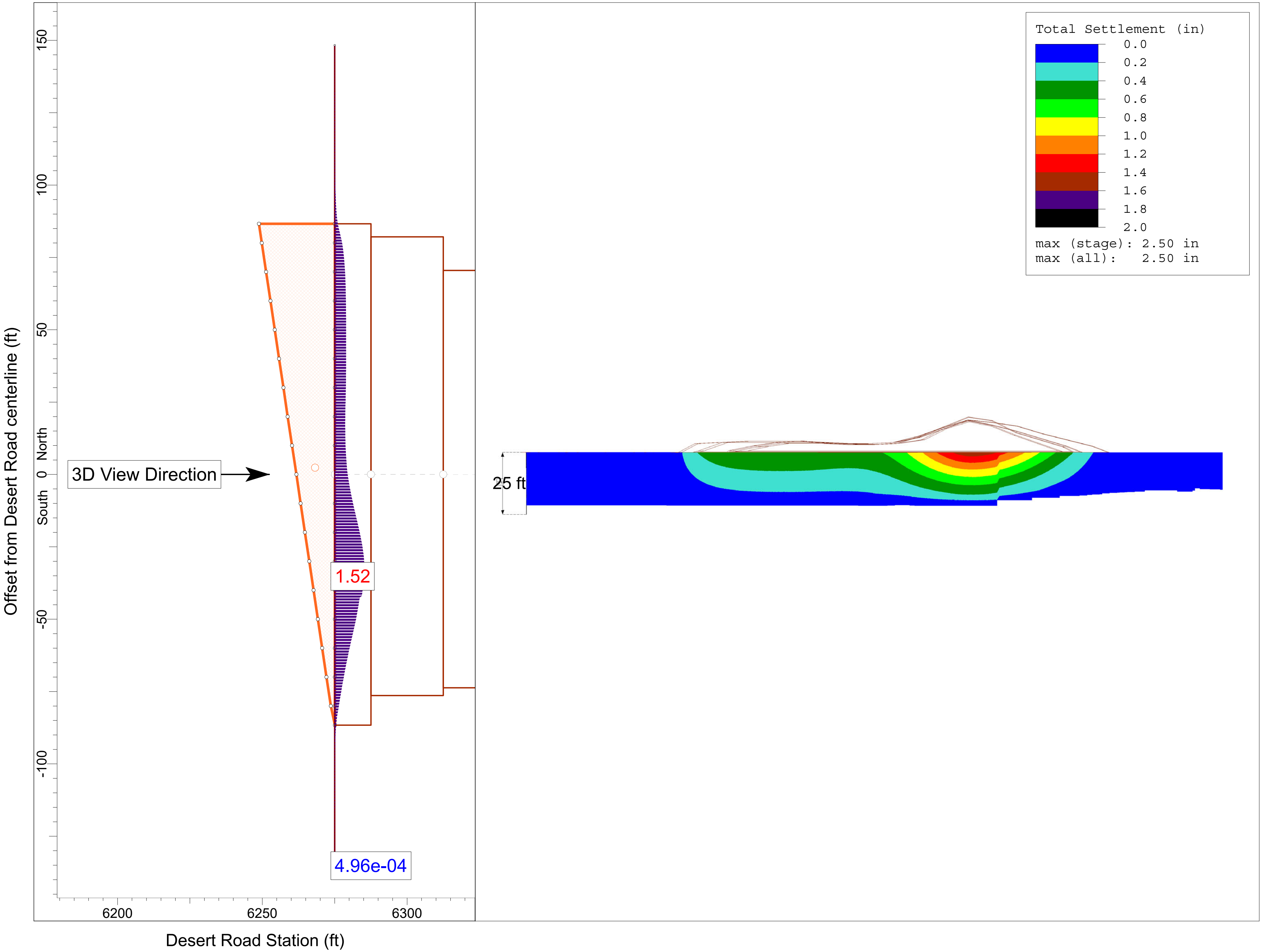
Attachment 4 - Total Settlement from Proposed Site Improvements along Abutment 2



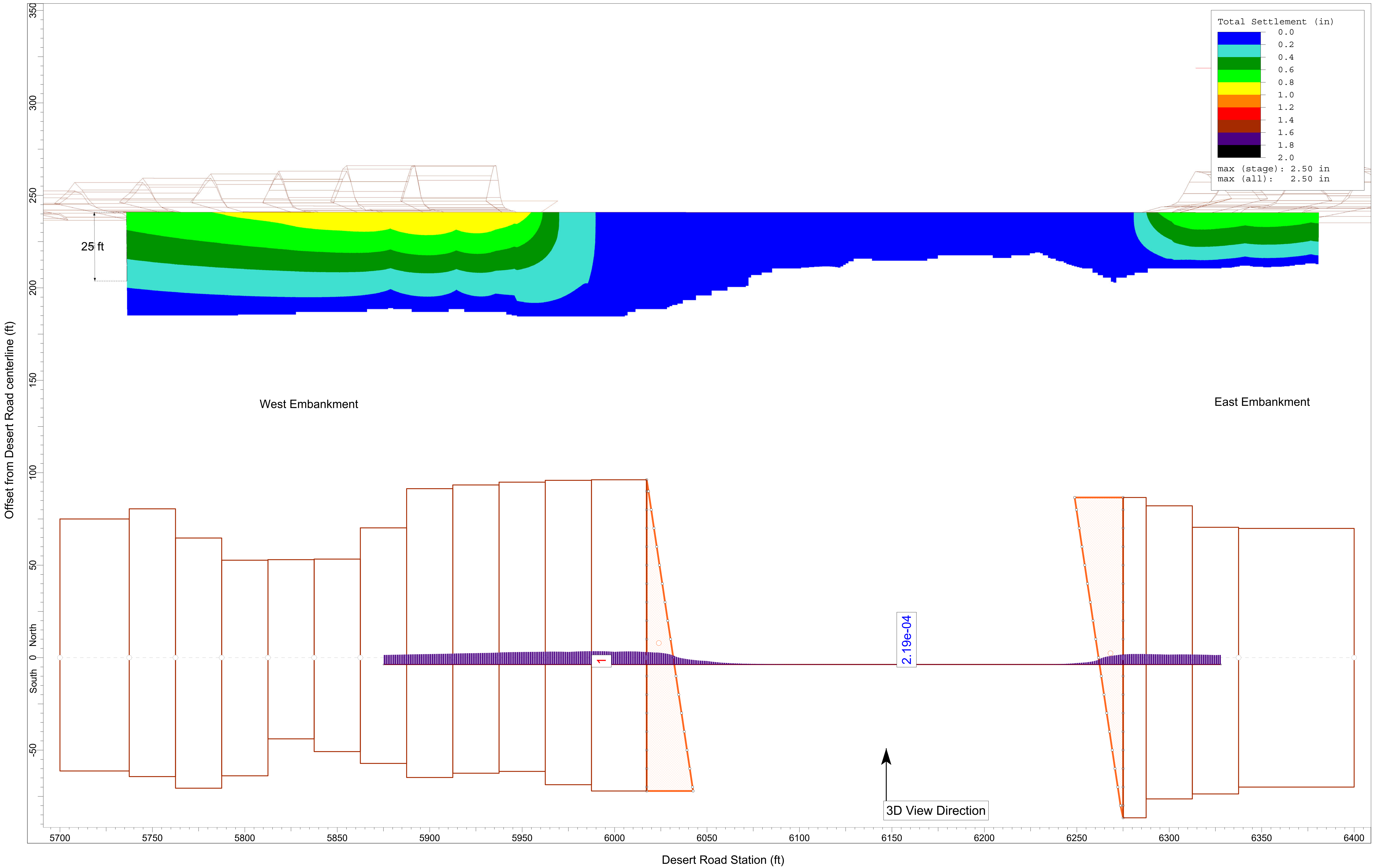
Attachment 5 - Total Settlement from Proposed Site Improvements at Station 60+10



Attachment 6 - Total Settlement from Proposed Site Improvements at Station 62+75



Attachment 7 - Total Settlement from Proposed Site Improvements along Profile A-A'



APPENDIX D

Abutment Lateral Earth Pressure

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CALCULATIONS

Date:	6/14/2020	**Made by:**	MEL
Project No.:	21450908	**Checked by:**	BK
Subject:	Lateral Earth Pressure	**Reviewed by:**	CCB
Project Short Title:	MaineDOT I-295 Exit 20 Merrill Road Bridge Replacement No. 5720		

OBJECTIVE

Determine lateral earth pressure acting on the proposed bridge abutments from the 60% plans (May 7, 2021).

REFERENCES

1. Golder interpreted subsurface section A-A' (Figure 3, Preliminary Geotechnical Design Report, dated September 2020).
2. AASHTO LRFD Bridge Design Specifications, 9th Ed, 2020 (LRFD).
3. HNTB, May 7, 2021. Merrill Road Bridge 60% plans. Abutment 1 Reinforcement Sections Sheet 94 of 113.
4. HNTB, May 21, 2021. Merrill Road Bridge 60% plans. Desert Road Profile 2 Sheet 8 of 113.
5. 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.
6. MassDOT LRFD Bridge Manual - Part 1, January 2020 Revision (https://www.mass.gov/doc/chapter-3-lrfd-bridge-design-guidelines/download)
7. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.

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 160 feet at Abutment No. 1 and 159.4 feet at Abutment No. 2 (Ref. 4).

CALCULATION

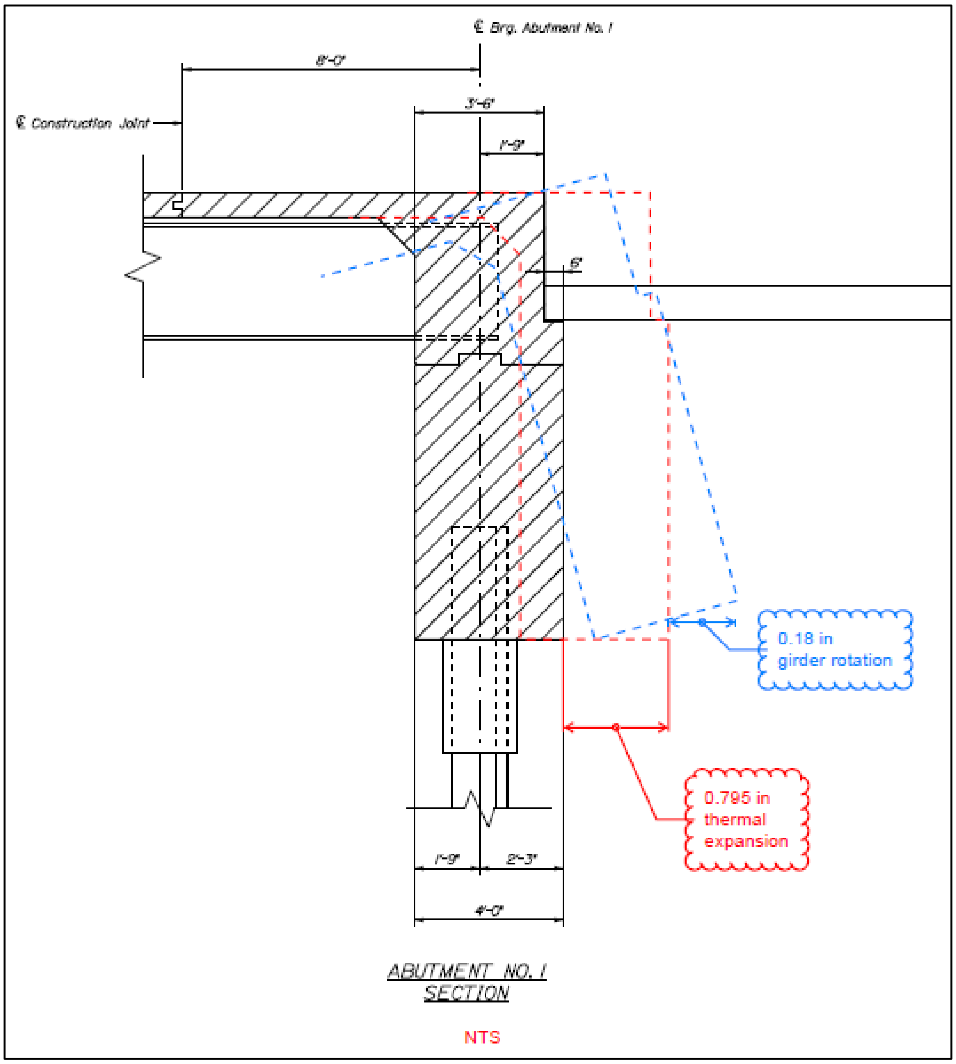
1. Calculate expected wall rotation for the integral abutments to select earth pressure case.

As per Ref. 2 Table C3.11.1-1, full passive earth pressure development requires that wall rotation (the ratio of lateral abutment movement to abutment height) exceeding 0.02 for medium dense sand similarly to that used for the wall backfill. 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. 6) would be more realistic.

Maximum lateral thermal movement =	0.795	inches	=	0.07	feet	(Ref. 3)
Maximum girder rotation =	0.18	inches	=	0.02	feet	(Ref. 3)
Abutment height (both)=	11	feet				(Ref. 4)
Maximum wall rotation =	0.007					

CALCULATIONS

Date:	6/14/2020	**Made by:**	MEL
Project No.:	21450908	**Checked by:**	BK
Subject:	Lateral Earth Pressure	**Reviewed by:**	CCB
Project Short Title:	MaineDOT I-295 Exit 20 Merrill Road Bridge Replacement No. 5720		



← Ref. 3 Schematic

Maximum wall rotation is estimated to be 0.007 and thus full passive earth pressure are not expected to develop. Partial passive earth pressure is determined using Ref. 6. Active earth pressure is determined using the Rankine method.

2. Calculate Rankine active earth pressure coefficient.

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

for horizontal backfill surface, where:

ϕ = internal friction angle of fill — ϕ = 32 degrees ("Granular borrow", Ref. 7, Table 3-3)

K_a = 0.31

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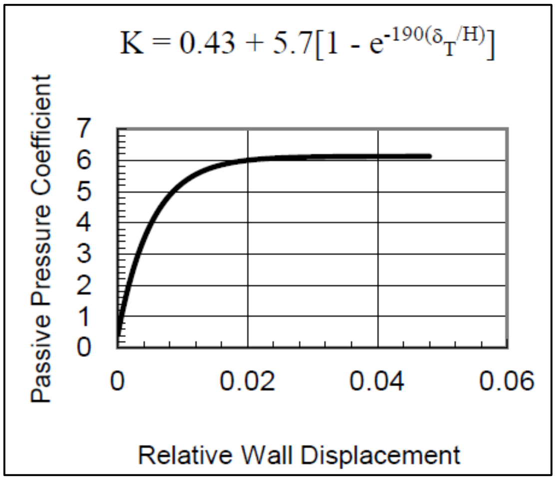
CALCULATIONS

Date:	6/14/2020	**Made by:**	MEL
Project No.:	21450908	**Checked by:**	BK
Subject:	Lateral Earth Pressure	**Reviewed by:**	CCB
Project Short Title:	MaineDOT I-295 Exit 20 Merrill Road Bridge Replacement No. 5720		

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 = 0.007 (relative wall displacement)

K_p = 4.63

4. Determine the UNFACTORED passive pressure P_p acting on the abutments.

$$P_p = \frac{1}{2} \cdot \gamma_{soil} \cdot H_{abut}{}^2 \cdot k_p$$ (Ref. 7, page 5-51)

where:

γ_{soil} = unit weight of fill — γ_{soil} = 125 pcf ("Granular borrow", Ref. 7, Table 3-3)

H_{abut} = height of the abutment backwall — H_{abut} = 11 feet (Ref. 4)

P_p = 35,014 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 Abutment No. 1, the load acts at:	163.7	feet elevation
For Abutment No. 2, the load acts at:	163.1	feet elevation

CONCLUSIONS

For the designer-given wall geometry, thermal lateral movement, and girder rotation, the calculated rotation is less than that required to develop full passive pressure. Thus the MassDOT method was used to determine the partial passive earth pressure. For the recommended soil parameters and given wall geometry, the partial passive earth pressure coefficient is K_p = 4.63, which corresponds to an unfactored passive earth force of P_p = 35,014 pounds per linear foot acting at elevations 163.7 feet and 163.1 feet on Abutment No. 1 and No. 2, respectively. The Rankine method was used to determine the active earth pressure coefficient of K_a = 0.31.

APPENDIX E

Abutment 1 Pile Design

GOLDER
MEMBER OF WSP

CALCULATIONS

Date:	7/14/2021	**Made by:**	DAF
Project No.:	21450908	**Checked by:**	KAR
Subject:	Pile Design at Abutment 1	**Reviewed by:**	JEL
Project Title:	MaineDOT Desert Road Bridge 5720 Freeport (Exit 20) Phase 2		

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, girder rotation, and final design loads.

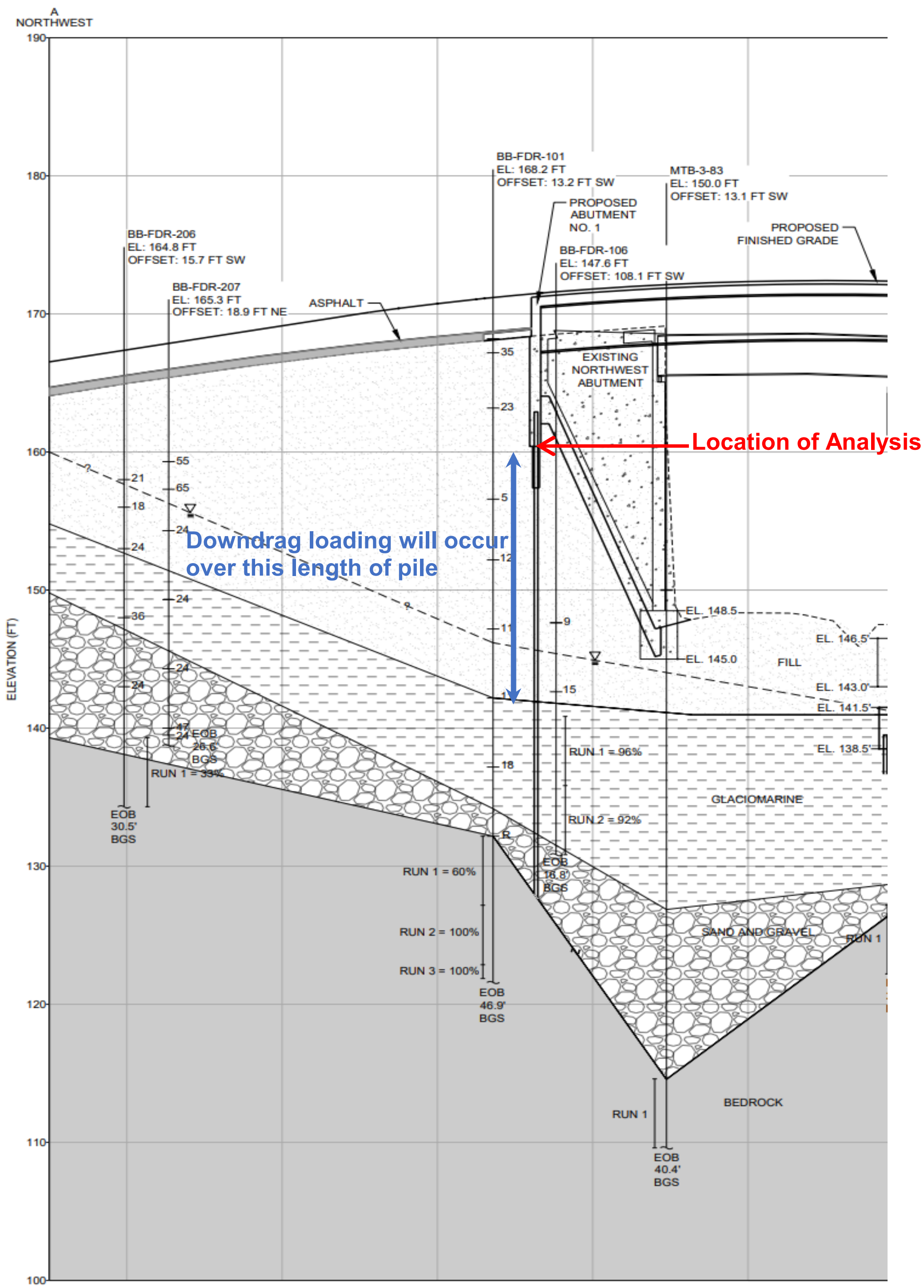
METHOD
Use the procedure outlined in AASHTO LRFD (Ref. 1) and the design method provided in the MaineDOT Bridge Design Guide (Ref. 2).

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 9th Ed. 2020.
2. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.
3. Email communication between Golder and HNTB, subject "RE: Freeport I-295 Exit 20 and Exit 22 - Request for Sections and Design Info", dated June 22, 2020.
4. Isenhower, W.M. et al. LPile v2019 Technical Manual: A Program for the Analysis of Deep Foundations Under Lateral Loading. Ensoft, Inc. Dated March 2020.
5. Golder interpreted subsurface profile A-A' (Figure 3, Preliminary Geotechnical Design Report, dated December 21, 2020).
6. Golder geotechnical test boring logs (Appendix A, Preliminary Geotechnical Design Report, dated December 21, 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. HNTB calculation titled "Freeport Bridges_Loads_Bottom of Footing_flat.pdf", dated May 26, 2021.
11. Oregon Department of Transportation, Geo-Environmental Section. Geotechnical Design Manual: Chapter 8 - Foundations, Version 2.1. Dated May 6, 2019.
12. HNTB for State of Maine Department of Transportation. Merrill Road Bridge over Interstate 295 and Signalized Intersections, Exit 20 Interchange: 60% Plans, dated May 21, 2021.
13. Golder calculation titled "Pile Design at Proposed Abutment 1 - Downdrag Analysis", dated May 12, 2021
14. HNTB for State of Maine Department of Transportation. Merrill Road Bridge over Interstate 295, Freeport, Cumberland: Abutment 1 Reinforcement Sections, Sheet 94, dated May 7, 2021.

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.

Date:	7/14/2021	**Made by:**	DAF
Project No.:	21450908	**Checked by:**	KAR
Subject:	Pile Design at Abutment 1	**Reviewed by:**	JEL
Project Title:	MaineDOT Desert Road Bridge 5720 Freeport (Exit 20) Phase 2		



CALCULATIONS

Date:	7/14/2021	**Made by:**	DAF
Project No.:	21450908	**Checked by:**	KAR
Subject:	Pile Design at Abutment 1	**Reviewed by:**	JEL
Project Title:	MaineDOT Desert Road Bridge 5720 Freeport (Exit 20) Phase 2		

ATTACHMENTS

1. LPile analysis output for Strength I
2. LPile analysis output for Strength I with Plastic Hinge
3. Downdrag Analysis
4. Driveability Analysis

CALCULATION

1. Select the preliminary pile size.

Determine the factored applied superstructure vertical dead and live load (P_u) distributed to each pile.

Design P_u = 465 kips (Total factored axial load including downdrag and pile weight, Ref. 13)

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 under combined axial and flexural loading (Ref. 2, page 5-42)
ϕ_f =	1.00	for flexural resistance in the upper zone of the pile under combined axial and flexural loading (Ref. 2, page 5-42)

Determine the maximum required nominal axial pile resistance (Ref. 1, Article 6.9.2.1).

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

$R_{n.upper}$ = 664 kips

$$R_{n.lower} = \frac{P_u}{\phi_{cl}}$$

$R_{n.lower}$ = 930 kips

$$R_n = \max\left(R_{n.upper}, R_{n.lower}\right)$$

R_n = 930 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)

$A_{s.req} =$ 23.3 in^2

Select a pile size with an area of $A_{s.req}$ or greater.

Preferred selection is HP 14x89 based on June 16, 2020 meeting with MaineDOT and HNTB.
Check that preferred selection satisfies pile area requirement:

HP 14x89 $A_s =$ 26.1 in^2 (Ref. 4, Table 5.6.3)
A_s > $A_{s.req}$ OK

2. 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:	32.6	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 due to abutment thermal expansion or contraction:	0.795	in	(Ref. 10)
Lateral deflection due to girder rotation:	0.18	in	(Ref. 14)
Total lateral deflection at pile head:	0.975	in	
Axial load:	465,000	lbs	(Ref. 10)

Soil Layers

Layer	Depth below base of abutment[1]	Lateral Model	Effective Unit Weight (pcf)	Undrained Shear Strength (psf)[2]	Friction Angle (°)[2]	Subgrade Modulus (pci)[3]	Major Principal Strain at 50%[3]	UCS (psi)[2]
Existing Fill (above water table)	0 - 14.8 ft	Sand (Reese)	125	-	32	124.8	-	-

Existing Fill (below water table)	14.8 - 18.1 ft	Sand (Reese)	62.6	-	32	75.5	-	-
Glaciomarine Silty Clay	18.1 - 27.7 ft	Stiff Clay w/o Free Water (Reese)	62.6	1600	-	-	0.005	-
Sand and Gravel	27.7 - 32.6 ft	Sand (Reese)	62.6	-	37	40.5	-	-
Bedrock	>32.6 ft	Strong Rock (Vuggy Limestone)	101.6	-	-	-	-	12983

1) Ref. 5
2) Ref. 6
3) Ref. 7. Interpolation based on average N_{60} value for each layer.

The full LPile output is provided in Attachment 1.

Obtain the maximum moment at the top of the pile.

$M_{u.Top}$ = 2908 in-kips (LPile)

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

$l_{b.top}$ = 4.6 ft (LPile)
$l_{b.top}$ = 54.7 in

$l_{b.2nd}$ = 12.5 ft (LPile)
$l_{b.2nd}$ = 149.6 in

3. 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):

$$F\lambda_{top} = \frac{K_{top} l_{b.top}}{r_y} \leq 120$$ (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} = 18.57 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} = 42.32 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)

$P_{e.top}$ = 21656 kips
$P_{e.2nd}$ = 4172 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

$P_o/P_{e.2nd}$ = 0.31 ≤ 2.25

thus use Ref. 1 Eqn 6.9.4.1.1-1:

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

$P_{n.top}$ = 1272 kips

$P_{n.2nd}$ = 1145 kips

$$P_{n.bottom} = (0.658^{(0)}) \times F_y A_s$$ (0 for a fully braced pile - Ref. 8, Appendix B, Eqn 6-9)

$P_{n.bottom}$ = 1305 kips

Calculate the factored structural pile resistance, P_r, for both segments of the upper zone of the pile as well as the lower zone of the pile.

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

$P_{r.top}$ = 891 kips

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

$P_{r.2nd}$ = 801 kips

$$P_{r.bottom} = \phi_{cl} P_{n.bottom}$$

$P_{r.bottom}$ = 653 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.

$$\frac{P_u}{P_{r.top}} =$$ 0.52 OK

$$\frac{P_u}{P_{r.2nd}} =$$ 0.58 OK

Since the lower zone of the pile will have virtually no moment, the entire section can carry the required vertical loads. Make sure the applied load will not exceed the resistance of the lower zone.

$$\text{Check} \left(\frac{P_u}{P_{r.bottom}} < 1\right)$$

$$\frac{P_u}{P_{r.bottom}} =$$ 0.71 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)

λr_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)

$M_p' =$ 1656 in-kips = 1656234.4 inch-lb

If the applied moment exceeds the moment that would cause a plastic hinge, it can be assumed that the pile head has entered plastic deformation and therefore the moment that can be applied to the pile head cannot exceed M_p'.

$M_{u.Top} =$ 2908 in-kips (From Step 2)

$M_{u.Top}$ > M_p' Plastic Hinge Forms

4. 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 3 with the new unbraced lengths.

$l_{b.top} =$ 3.3 ft (LPile)

$l_{b.top} =$ 39.2 in

$l_{b.2nd} =$ 13.1 ft (LPile)

$l_{b.2nd} =$ 157.7 in

$M_{u.2nd} =$ 1201 in-kips (LPile)

Since a plastic hinge developed at the pile head, the value of K for the top segment becomes 2.1 (Ref. 2, page 5-43).

$K_{top} =$ 2.1 (Ref. 1, Table C4.6.2.5-1)

$K_{2nd} =$ 1.0 (Ref. 1, Table C4.6.2.5-1)

$\lambda_{top} =$ 23.29 < 120 OK

$\lambda_{2nd} =$ 44.63 < 120 OK

$P_{e.top} =$ 13775 kips

$P_{e.2nd} =$ 3751 kips

$P_o/P_{e.top}$ = 0.09 ≤ 2.25 (to select P_n equation)
$P_o/P_{e.2nd}$ = 0.35 ≤ 2.25 (to select P_n equation)

$P_{n.top}$ = 1254 kips
$P_{n.2nd}$ = 1128 kips

$P_{r.top}$ = 878 kips
$P_{r.2nd}$ = 790 kips

$\frac{P_u}{P_{r.top}} =$ 0.53 > 0.20 OK

$\frac{P_u}{P_{r.2nd}} =$ 0.59 > 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: 0.94 < 1 OK

5. 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 = 41.0 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

6. 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)

$R_{ndr} =$ 715 kips

The nominal pile driving resistance (R_{ndr}) should not exceed the nominal structural pile resistance (P_n) or the maximum driving force (P_0) calculated above.

$P_{n.top}$ =	1254	kips	(From Step 4)
$P_{n.2nd}$ =	1128	kips	(From Step 4)

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

CONCLUSIONS

HP 14x89 piles were evaluated for Abutment 1. The results of the analysis indicate that a maximum moment of 2908 in-kips (242 ft-kips) occurs at the top of the pile, with a maximum bridge expansion or contraction of 0.8 inches, a maximum lateral deflection due to girder rotation of 0.2 inches, and maximum factored axial load of 465 kips including downdrag and pile weight at the strength 1 load case. The results indicate that the depth to bedrock is sufficient for driven piles to achieve fixity. The design is controlled by pile driving and a nominal pile driving resistance of 715 kips is recommended based on the factored design load and the assumption that pile driving will be established by dynamic testing.

Attachment 1

===

LPile for Windows, Version 2019-11.005

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

===

This copy of LPile is being used by:

KAR
Golder Associates, Inc.

Serial Number of Security Device: 239146533

This copy of LPile is licensed for exclusive use by:

GOLDER, Manchester, NH

Files Used for Analysis

Path to file locations:
\\golderassociates.sharepoint.com@SSL\DavWWWRoot\sites\139980\Project Files\5 Technical Work\06 Analysis\Phase II Pile Design\LPILE\Abutment 1 Strength I\

Name of input data file:
Freeport Exit 20 Abutment 1 - KAR check - girder rotation.lp11d

Name of output report file:
Freeport Exit 20 Abutment 1 - KAR check - girder rotation.lp11o

Name of plot output file:
Freeport Exit 20 Abutment 1 - KAR check - girder rotation.lp11p

Name of runtime message file:
Freeport Exit 20 Abutment 1 - KAR check - girder rotation.lp11r

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Date and Time of Analysis

Date: July 14, 2021 Time: 15:54:14

Problem Title

Project Name: MaineDOT I-295 Exit 20 Merrill Road Bridge No. 5720
Job Number: 19126013
Client: MaineDOT
Engineer: KAR
Description: Northwest Abutment Pile Design - Strength I

Program Options and Settings

Computational Options:
- Conventional Analysis

Engineering Units Used for Data Input and Computations:
- US Customary System Units (pounds, feet, inches)

Analysis Control Options:
- Maximum number of iterations allowed = 500
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 100.0000 in
- Number of pile increments = 100

Loading Type and Number of Cycles of Loading:
- Static loading specified

- Use of p-y modification factors for p-y curves not selected
- Analysis uses layering correction (Method of Georgiadis)
- No distributed lateral loads are entered
- Loading by lateral soil movements acting on pile not selected
- Input of shear resistance at the pile tip not selected
- Input of moment resistance at the pile tip not selected
- Input of side resistance moment along pile not selected
- Computation of pile-head foundation stiffness matrix not selected
- Push-over analysis of pile not selected
- Buckling analysis of pile not selected

Output Options:
- Output files use decimal points to denote decimal symbols.
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (nodal spacing of output points) = 1
- No p-y curves to be computed and reported for user-specified depths
- Print using wide report formats

Pile Structural Properties and Geometry

Number of pile sections defined = 1
Total length of pile = 32.600 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	32.600	14.6950

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is a H weak axis steel pile
Length of section = 32.600000 ft
Pile width = 13.830000 in
Shear capacity of section = 0.0000 lbs

Ground Slope and Pile Batter Angles

Ground Slope Angle = 0.000 degrees
= 0.000 radians

Pile Batter Angle = 0.000 degrees
= 0.000 radians

Soil and Rock Layering Information

The soil profile is modelled using 5 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer = 0.0000 ft
Distance from top of pile to bottom of layer = 14.800000 ft

Effective unit weight at top of layer = 125.000000 pcf
Effective unit weight at bottom of layer = 125.000000 pcf
Friction angle at top of layer = 32.000000 deg.
Friction angle at bottom of layer = 32.000000 deg.
Subgrade k at top of layer = 124.800000 pci
Subgrade k at bottom of layer = 124.800000 pci

Layer 2 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer = 14.800000 ft
Distance from top of pile to bottom of layer = 18.100000 ft
Effective unit weight at top of layer = 62.600000 pcf
Effective unit weight at bottom of layer = 62.600000 pcf
Friction angle at top of layer = 32.000000 deg.
Friction angle at bottom of layer = 32.000000 deg.
Subgrade k at top of layer = 75.500000 pci
Subgrade k at bottom of layer = 75.500000 pci

Layer 3 is stiff clay without free water

Distance from top of pile to top of layer = 18.100000 ft
Distance from top of pile to bottom of layer = 27.700000 ft
Effective unit weight at top of layer = 62.600000 pcf
Effective unit weight at bottom of layer = 62.600000 pcf
Undrained cohesion at top of layer = 1600. psf
Undrained cohesion at bottom of layer = 1600. psf
Epsilon-50 at top of layer = 0.005000
Epsilon-50 at bottom of layer = 0.005000

Layer 4 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer = 27.700000 ft
Distance from top of pile to bottom of layer = 32.600000 ft
Effective unit weight at top of layer = 62.600000 pcf
Effective unit weight at bottom of layer = 62.600000 pcf
Friction angle at top of layer = 37.000000 deg.
Friction angle at bottom of layer = 37.000000 deg.
Subgrade k at top of layer = 40.500000 pci
Subgrade k at bottom of layer = 40.500000 pci

Layer 5 is strong rock (vuggy limestone)

Distance from top of pile to top of layer = 32.600000 ft
Distance from top of pile to bottom of layer = 50.000000 ft
Effective unit weight at top of layer = 101.600000 pcf
Effective unit weight at bottom of layer = 101.600000 pcf
Uniaxial compressive strength at top of layer = 12983. psi
Uniaxial compressive strength at bottom of layer = 12983. psi

(Depth of the lowest soil layer extends 17.400 ft below the pile tip)

Summary of Input Soil Properties

Layer Num.	Soil Type Name (p-y Curve Type)	Layer Depth ft	Effective Unit Wt. pcf	Undrained Cohesion psf	Angle of Friction deg.	Uniaxial qu psi	E50 or krm	kpy pci
1	Sand	0.00	125.0000	--	32.0000	--	--	124.8000
	(Reese, et al.)	14.8000	125.0000	--	32.0000	--	--	124.8000
2	Sand	14.8000	62.6000	--	32.0000	--	--	75.5000
	(Reese, et al.)	18.1000	62.6000	--	32.0000	--	--	75.5000
3	Stiff Clay	18.1000	62.6000	1600.	--	--	0.00500	--
	w/o Free Water	27.7000	62.6000	1600.	--	--	0.00500	--
4	Sand	27.7000	62.6000	--	37.0000	--	--	40.5000
	(Reese, et al.)	32.6000	62.6000	--	37.0000	--	--	40.5000
5	Strong Rock	32.6000	101.6000	--	--	12983.	--	--
	(Vuggy Limestone)	50.0000	101.6000	--	--	12983.	--	--

Static Loading Type

Static loading criteria were used when computing p-y curves for all analyses.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load No.	Load Type	Condition 1	Condition 2	Axial Thrust Force, lbs	Compute Top y vs. Pile Length	Run Analysis
1	5	y = -0.975000 in	S = 0.0000 in/in	465000.	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	=	32.600000 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	465.000

Definition of Run Messages:

Y = part of pipe section has yielded.

Axial Thrust Force = 465.000 kips

Bending Curvature rad/in.	Bending Moment in-kip	Bending Stiffness kip-in2	Depth to N Axis in	Max Total Stress ksi	Run Msg

0.00000418	39.4155497	9438779.	156.0376435	18.8875060	
0.00000835	78.8310995	9438779.	81.6925717	19.7684019	
0.00001253	118.2466492	9438779.	56.9108812	20.6492980	
0.00001670	157.6621990	9438779.	44.5200359	21.5301933	
0.00002088	197.0777487	9438779.	37.0855287	22.4110895	
0.00002506	236.4932984	9438779.	32.1291906	23.2919854	
0.00002923	275.9088482	9438779.	28.5889491	24.1728812	
0.00003341	315.3243979	9438779.	25.9337679	25.0537772	
0.00003758	354.7399477	9438779.	23.8686271	25.9346731	
0.00004176	394.1554974	9438779.	22.2165143	26.8155689	
0.00004594	433.5710471	9438779.	20.8647858	27.6964648	
0.00005011	472.9865969	9438779.	19.7383453	28.5773606	
0.00005429	512.4021466	9438779.	18.7852033	29.4582566	
0.00005846	551.8176964	9438779.	17.9682245	30.3391524	
0.00006264	591.2332461	9438779.	17.2601762	31.2200483	
0.00006681	630.6487958	9438779.	16.6406340	32.1009442	
0.00007099	670.0643456	9438779.	16.0939790	32.9818400	
0.00007517	709.4798953	9438779.	15.6080635	33.8627359	
0.00007934	748.8954450	9438779.	15.1732970	34.7436318	
0.00008352	788.3109948	9438779.	14.7820072	35.6245276	
0.00008769	827.7265445	9438779.	14.4279830	36.5054235	
0.00009187	867.1420943	9438779.	14.1061429	37.3863195	
0.00009605	906.5576440	9438779.	13.8122888	38.2672153	
0.0001002	945.9731937	9438779.	13.5429226	39.1481112	
0.0001044	985.3887435	9438779.	13.2951057	40.0290070	
0.0001086	1025.	9438779.	13.0663517	40.9099029	
0.0001127	1064.	9438779.	12.8545424	41.7907988	
0.0001169	1104.	9438779.	12.6578623	42.6716947	
0.0001211	1143.	9438779.	12.4747463	43.5525906	
0.0001253	1182.	9438779.	12.3038381	44.4334864	
0.0001295	1222.	9438779.	12.1439562	45.3143823	
0.0001336	1261.	9438779.	11.9940670	46.1952782	
0.0001378	1301.	9438779.	11.8532619	47.0761741	
0.0001420	1340.	9438779.	11.7207395	47.9570699	
0.0001462	1380.	9438779.	11.5957898	48.8379658	
0.0001503	1419.	9438779.	11.4777818	49.7188617	
0.0001545	1458.	9433639.	11.3670960	50.0000000	Y
0.0001587	1495.	9418541.	11.2641477	50.0000000	Y
0.0001629	1530.	9395673.	11.1681210	50.0000000	Y
0.0001712	1598.	9331800.	10.9943887	50.0000000	Y
0.0001796	1661.	9249108.	10.8419386	50.0000000	Y
0.0001879	1720.	9152913.	10.7075643	50.0000000	Y
0.0001963	1776.	9047501.	10.5885833	50.0000000	Y
0.0002046	1829.	8936330.	10.4827291	50.0000000	Y
0.0002130	1879.	8822190.	10.3880673	50.0000000	Y
0.0002213	1927.	8705599.	10.3033989	50.0000000	Y
0.0002297	1973.	8589387.	10.2270557	50.0000000	Y
0.0002380	2017.	8472768.	10.1584484	50.0000000	Y
0.0002464	2059.	8357810.	10.0962941	50.0000000	Y
0.0002547	2100.	8244798.	10.0398817	50.0000000	Y
0.0002631	2140.	8133966.	9.9885881	50.0000000	Y
0.0002714	2178.	8025508.	9.9418642	50.0000000	Y
0.0002798	2216.	7919580.	9.8992241	50.0000000	Y
0.0002881	2252.	7816310.	9.8602356	50.0000000	Y

0.0002965	2288.	7715796.	9.8245125	50.0000000	Y
0.0003048	2322.	7618116.	9.7917085	50.0000000	Y
0.0003132	2356.	7523326.	9.7615115	50.0000000	Y
0.0003215	2390.	7431463.	9.7336391	50.0000000	Y
0.0003299	2422.	7342540.	9.7078385	50.0000000	Y
0.0003382	2454.	7255678.	9.6842426	50.0000000	Y
0.0003466	2486.	7171718.	9.6623145	50.0000000	Y
0.0003550	2516.	7086908.	9.6407330	50.0000000	Y
0.0003633	2543.	7000533.	9.6197794	50.0000000	Y
0.0003717	2570.	6913993.	9.5990744	50.0000000	Y
0.0003800	2594.	6827404.	9.5787658	50.0000000	Y
0.0003884	2618.	6741020.	9.5589435	50.0000000	Y
0.0003967	2640.	6655508.	9.5394404	50.0000000	Y
0.0004051	2661.	6569715.	9.5201513	50.0000000	Y
0.0004134	2681.	6485158.	9.5011879	50.0000000	Y
0.0004218	2700.	6401882.	9.4827107	50.0000000	Y
0.0004301	2718.	6319308.	9.4645010	50.0000000	Y
0.0004385	2735.	6238091.	9.4463322	50.0000000	Y
0.0004468	2752.	6158124.	9.4290013	50.0000000	Y
0.0004552	2767.	6079154.	9.4114989	50.0000000	Y
0.0004635	2782.	6001843.	9.3944082	50.0000000	Y
0.0004719	2796.	5925666.	9.3777834	50.0000000	Y
0.0004802	2810.	5850857.	9.3611587	50.0000000	Y
0.0004886	2823.	5777832.	9.3449499	50.0000000	Y
0.0004969	2835.	5705432.	9.3289874	50.0000000	Y
0.0005303	2880.	5431084.	9.2675962	50.0000000	Y
0.0005637	2919.	5178009.	9.2100273	50.0000000	Y
0.0005972	2952.	4943933.	9.1553729	50.0000000	Y
0.0006306	2982.	4728429.	9.1042653	50.0000000	Y
0.0006640	3007.	4528860.	9.0558319	50.0000000	Y
0.0006974	3030.	4344464.	9.0096933	50.0000000	Y
0.0007308	3050.	4173353.	8.9664559	50.0000000	Y
0.0007642	3068.	4014678.	8.9250764	50.0000000	Y
0.0007976	3084.	3866959.	8.8859993	50.0000000	Y
0.0008310	3099.	3729328.	8.8488422	50.0000000	Y
0.0008644	3113.	3600877.	8.8132568	50.0000000	Y
0.0008978	3125.	3480314.	8.7792545	50.0000000	Y
0.0009312	3136.	3367427.	8.7469879	50.0000000	Y

Summary of Results for Nominal Moment Capacity for Section 1

Load No.	Axial Thrust kips	Nominal Moment Capacity in-kips
1	465.0000000000	3136.

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	319356.
2	14.8000	14.8000	Yes	No	319356.	233723.
3	18.1000	35.5370	No	No	553079.	169286.
4	27.7000	18.6731	No	No	722365.	676707.
5	32.6000	32.6000	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 = -0.975000 inches
Rotation of pile head = 0.000E+00 radians
Axial load on pile head = 465000.0 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	-0.9750	2907557.	-55600.	0.00	83638.	5.25E+09	0.00	0.00	0.00
0.3260	-0.9708	2688527.	-55409.	0.00209	78694.	5.25E+09	39.0784	157.4792	0.00
0.6520	-0.9587	2466448.	-55166.	0.00376	73681.	7.22E+09	85.0148	346.9123	0.00
0.9780	-0.9414	2243240.	-54737.	0.00498	68642.	7.84E+09	134.4094	558.5557	0.00

1.3040	-0.9197	2020054.	-54113.	0.00601	63604.	8.46E+09	184.6473	785.4173	0.00
1.6300	-0.8944	1797995.	-53295.	0.00687	58592.	9.00E+09	233.6949	1022.	0.00
1.9560	-0.8660	1578091.	-52289.	0.00759	53628.	9.35E+09	280.4888	1267.	0.00
2.2820	-0.8350	1361278.	-51102.	0.00820	48734.	9.44E+09	326.1498	1528.	0.00
2.6080	-0.8018	1148430.	-49746.	0.00872	43930.	9.44E+09	367.4969	1793.	0.00
2.9340	-0.7668	940340.	-48237.	0.00915	39233.	9.44E+09	403.6882	2060.	0.00
3.2600	-0.7302	737720.	-46586.	0.00950	34659.	9.44E+09	440.7214	2361.	0.00
3.5860	-0.6924	541287.	-44789.	0.00977	30225.	9.44E+09	477.5703	2698.	0.00
3.9120	-0.6538	351756.	-42827.	0.00995	25947.	9.44E+09	525.5064	3145.	0.00
4.2380	-0.6146	170001.	-40669.	0.01006	21844.	9.44E+09	578.1125	3680.	0.00
4.5640	-0.5751	-3034.	-38307.	0.01009	18075.	9.44E+09	629.0625	4279.	0.00
4.8900	-0.5356	-166441.	-35779.	0.01006	21764.	9.44E+09	663.6354	4847.	0.00
5.2160	-0.4964	-319565.	-33116.	0.00996	25220.	9.44E+09	697.9111	5500.	0.00
5.5420	-0.4577	-461768.	-30328.	0.00980	28430.	9.44E+09	727.1449	6215.	0.00
5.8680	-0.4197	-592495.	-27439.	0.00958	31381.	9.44E+09	750.1074	6991.	0.00
6.1940	-0.3827	-711296.	-24469.	0.00931	34062.	9.44E+09	768.0895	7851.	0.00
6.5200	-0.3469	-817806.	-21428.	0.00899	36467.	9.44E+09	786.5739	8870.	0.00
6.8460	-0.3124	-911661.	-18324.	0.00863	38585.	9.44E+09	800.4027	10023.	0.00
7.1720	-0.2794	-992580.	-15175.	0.00824	40412.	9.44E+09	809.7805	11340.	0.00
7.4980	-0.2479	-1060359.	-11997.	0.00781	41942.	9.44E+09	815.0148	12860.	0.00
7.8240	-0.2182	-1114864.	-8808.	0.00736	43172.	9.44E+09	815.2029	14613.	0.00
8.1500	-0.1903	-1156054.	-5632.	0.00689	44102.	9.44E+09	808.4988	16617.	0.00
8.4760	-0.1643	-1183999.	-2496.	0.00641	44733.	9.44E+09	794.7054	18920.	0.00
8.8020	-0.1402	-1198890.	571.7004	0.00591	45069.	9.44E+09	773.6913	21586.	0.00
9.1280	-0.1181	-1201036.	3543.	0.00541	45117.	9.44E+09	745.3668	24699.	0.00
9.4540	-0.09785	-1190870.	6389.	0.00492	44888.	9.44E+09	709.6677	28373.	0.00
9.7800	-0.07957	-1168945.	9081.	0.00443	44393.	9.44E+09	666.5295	32770.	0.00
10.1060	-0.06319	-1135939.	11589.	0.00395	43648.	9.44E+09	615.8422	38129.	0.00
10.4320	-0.04864	-1092652.	13884.	0.00349	42671.	9.44E+09	557.3706	44824.	0.00
10.7580	-0.03587	-1040010.	15934.	0.00305	41482.	9.44E+09	490.6019	53499.	0.00
11.0840	-0.02479	-979077.	17698.	0.00263	40107.	9.44E+09	411.5038	64937.	0.00
11.4100	-0.01529	-911108.	19014.	0.00224	38573.	9.44E+09	261.3384	66847.	0.00
11.7360	-0.00728	-838453.	19776.	0.00188	36933.	9.44E+09	127.8643	68757.	0.00
12.0620	-6.15E-04	-763209.	20048.	0.00154	35234.	9.44E+09	11.1172	70667.	0.00
12.3880	0.00481	-687219.	19895.	0.00124	33519.	9.44E+09	-89.1751	72576.	0.00
12.7140	0.00911	-612076.	19381.	9.74E-04	31823.	9.44E+09	-173.5462	74486.	0.00
13.0400	0.01243	-539127.	18567.	7.36E-04	30176.	9.44E+09	-242.7434	76396.	0.00
13.3660	0.01487	-469487.	17510.	5.27E-04	28604.	9.44E+09	-297.6807	78306.	0.00
13.6920	0.01655	-404048.	16264.	3.46E-04	27127.	9.44E+09	-339.3932	80216.	0.00
14.0180	0.01758	-343499.	14878.	1.91E-04	25760.	9.44E+09	-368.9931	82126.	0.00
14.3440	0.01804	-288337.	13398.	5.99E-05	24515.	9.44E+09	-387.6301	84036.	0.00
14.6700	0.01805	-238891.	11864.	-4.93E-05	23399.	9.44E+09	-396.4533	85946.	0.00
14.9960	0.01766	-195331.	10620.	-1.39E-04	22416.	9.44E+09	-239.9172	53150.	0.00
15.3220	0.01696	-155296.	9690.	-2.12E-04	21512.	9.44E+09	-235.3680	54305.	0.00
15.6480	0.01600	-118746.	8786.	-2.69E-04	20687.	9.44E+09	-226.8337	55461.	0.00
15.9740	0.01485	-85578.	7922.	-3.11E-04	19938.	9.44E+09	-214.9486	56616.	0.00
16.3000	0.01357	-55635.	7109.	-3.40E-04	19262.	9.44E+09	-200.3365	57772.	0.00
16.6260	0.01219	-28715.	6358.	-3.58E-04	18655.	9.44E+09	-183.6057	58927.	0.00
16.9520	0.01077	-4584.	5676.	-3.65E-04	18110.	9.44E+09	-165.3466	60082.	0.00
17.2780	0.00934	17020.	5067.	-3.62E-04	18391.	9.44E+09	-146.1303	61238.	0.00
17.6040	0.00793	36375.	4533.	-3.51E-04	18828.	9.44E+09	-126.5091	62393.	0.00
17.9300	0.00659	53766.	4077.	-3.32E-04	19220.	9.44E+09	-107.0171	63549.	0.00
18.2560	0.00533	69479.	3274.	-3.07E-04	19575.	9.44E+09	-303.2720	222551.	0.00
18.5820	0.00419	80499.	2122.	-2.76E-04	19824.	9.44E+09	-285.4956	266769.	0.00

18.9080	0.00317	87089.	1043.	-2.41E-04	19972.	9.44E+09	-266.3777	328435.	0.00
19.2340	0.00230	89536.	41.1326	-2.04E-04	20028.	9.44E+09	-245.8007	418029.	0.00
19.5600	0.00157	88154.	-876.8572	-1.68E-04	19996.	9.44E+09	-223.5192	555941.	0.00
19.8860	9.88E-04	83286.	-1703.	-1.32E-04	19887.	9.44E+09	-199.0122	787701.	0.00
20.2120	5.39E-04	75308.	-2427.	-9.93E-05	19707.	9.44E+09	-171.0192	1241412.	0.00
20.5380	2.12E-04	64657.	-2969.	-7.03E-05	19466.	9.44E+09	-105.7929	1956000.	0.00
20.8640	-1.09E-05	52338.	-3165.	-4.60E-05	19188.	9.44E+09	5.4598	1956000.	0.00
21.1900	-1.49E-04	40063.	-3009.	-2.69E-05	18911.	9.44E+09	74.2828	1956000.	0.00
21.5160	-2.21E-04	28895.	-2647.	-1.26E-05	18659.	9.44E+09	110.6274	1956000.	0.00
21.8420	-2.47E-04	19398.	-2189.	-2.59E-06	18444.	9.44E+09	123.5474	1956000.	0.00
22.1680	-2.41E-04	11777.	-1711.	3.87E-06	18272.	9.44E+09	120.7420	1956000.	0.00
22.4940	-2.17E-04	5995.	-1263.	7.56E-06	18142.	9.44E+09	108.3895	1956000.	0.00
22.8200	-1.82E-04	1867.	-872.7356	9.19E-06	18049.	9.44E+09	91.1773	1956000.	0.00
23.1460	-1.45E-04	-867.1378	-552.6771	9.39E-06	18026.	9.44E+09	72.4518	1956000.	0.00
23.4720	-1.09E-04	-2492.	-304.4979	8.70E-06	18063.	9.44E+09	54.4292	1956000.	0.00
23.7980	-7.69E-05	-3281.	-122.8720	7.50E-06	18081.	9.44E+09	38.4265	1956000.	0.00
24.1240	-5.02E-05	-3480.	1.3543	6.10E-06	18085.	9.44E+09	25.0839	1956000.	0.00
24.4500	-2.91E-05	-3293.	78.9026	4.70E-06	18081.	9.44E+09	14.5625	1956000.	0.00
24.7760	-1.34E-05	-2880.	120.5129	3.42E-06	18072.	9.44E+09	6.7106	1956000.	0.00
25.1020	-2.39E-06	-2362.	135.9732	2.33E-06	18060.	9.44E+09	1.1934	1956000.	0.00
25.4280	4.82E-06	-1825.	133.5963	1.46E-06	18048.	9.44E+09	-2.4086	1956000.	0.00
25.7540	9.06E-06	-1322.	120.0213	8.11E-07	18036.	9.44E+09	-4.5315	1956000.	0.00
26.0800	1.12E-05	-888.5063	100.2384	3.53E-07	18027.	9.44E+09	-5.5824	1956000.	0.00
26.4060	1.18E-05	-539.3970	77.7534	5.72E-08	18019.	9.44E+09	-5.9130	1956000.	0.00
26.7320	1.16E-05	-280.3720	54.8305	-1.13E-07	18013.	9.44E+09	-5.8063	1956000.	0.00
27.0580	1.09E-05	-109.9934	32.7696	-1.94E-07	18009.	9.44E+09	-5.4723	1956000.	0.00
27.3840	1.01E-05	-23.2783	12.1898	-2.21E-07	18007.	9.44E+09	-5.0491	1956000.	0.00
27.7100	9.21E-06	-13.8161	2.0710	-2.29E-07	18007.	9.44E+09	-0.1241	52683.	0.00
28.0360	8.31E-06	-6.2424	1.6068	-2.33E-07	18007.	9.44E+09	-0.1132	53303.	0.00
28.3620	7.39E-06	-0.3964	1.1862	-2.34E-07	18007.	9.44E+09	-0.1019	53923.	0.00
28.6880	6.47E-06	3.8908	0.8103	-2.34E-07	18007.	9.44E+09	-0.09026	54543.	0.00
29.0140	5.56E-06	6.7938	0.4804	-2.31E-07	18007.	9.44E+09	-0.07844	55162.	0.00
29.3400	4.66E-06	8.4912	0.1969	-2.28E-07	18007.	9.44E+09	-0.06649	55782.	0.00
29.6660	3.78E-06	9.1646	-0.03969	-2.25E-07	18007.	9.44E+09	-0.05445	56402.	0.00
29.9920	2.91E-06	8.9978	-0.2290	-2.21E-07	18007.	9.44E+09	-0.04235	57022.	0.00
30.3180	2.05E-06	8.1761	-0.3709	-2.17E-07	18007.	9.44E+09	-0.03019	57642.	0.00
30.6440	1.21E-06	6.8862	-0.4651	-2.14E-07	18007.	9.44E+09	-0.01795	58261.	0.00
30.9700	3.73E-07	5.3165	-0.5112	-2.12E-07	18007.	9.44E+09	-0.00562	58881.	0.00
31.2960	-4.51E-07	3.6567	-0.5088	-2.10E-07	18007.	9.44E+09	0.00685	59501.	0.00
31.6220	-1.27E-06	2.0991	-0.4572	-2.09E-07	18007.	9.44E+09	0.01949	60121.	0.00
31.9480	-2.08E-06	0.8382	-0.3559	-2.08E-07	18007.	9.44E+09	0.03234	60741.	0.00
32.2740	-2.90E-06	0.07161	-0.2038	-2.08E-07	18007.	9.44E+09	0.04542	61360.	0.00
32.6000	-3.71E-06	0.00	0.00	-2.08E-07	18007.	9.44E+09	0.05876	30990.	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 = -0.97500000 inches
Computed slope at pile head = 0.000000 radians
Maximum bending moment = 2907557. inch-lbs
Maximum shear force = -55600. 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 = 16
Number of zero deflection points = 4

Summary of Pile-head Responses for Conventional Analyses

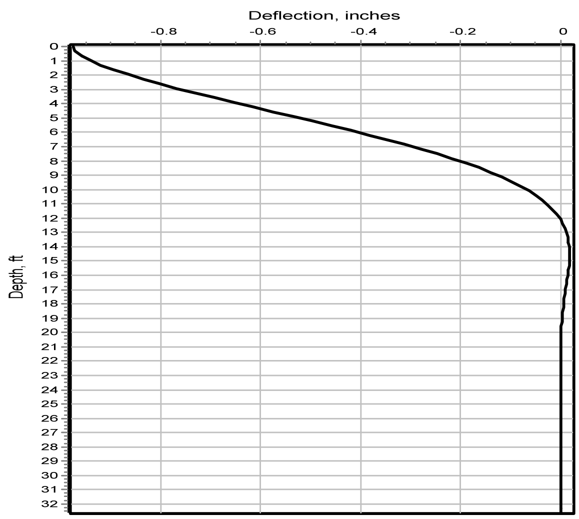
Definitions of Pile-head Loading Conditions:

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

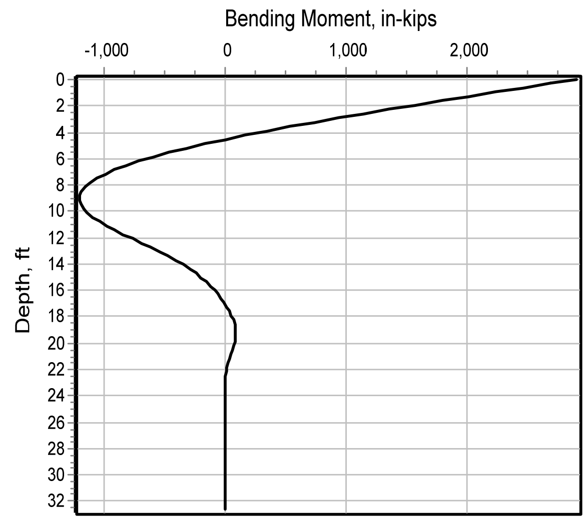
Load Case No.	Load Type 1	Pile-head Load 1	Load Type 2	Pile-head Load 2	Axial Loading lbs	Pile-head Deflection inches	Pile-head Rotation radians	Max Shear in Pile lbs	Max Moment in Pile in-lbs
1	y, in	-0.9750	S, rad	0.00	465000.	-0.9750	0.00	-55600.	2907557.

Maximum pile-head deflection = -0.9750000000 inches
Maximum pile-head rotation = 0.0000000000 radians = 0.000000 deg.

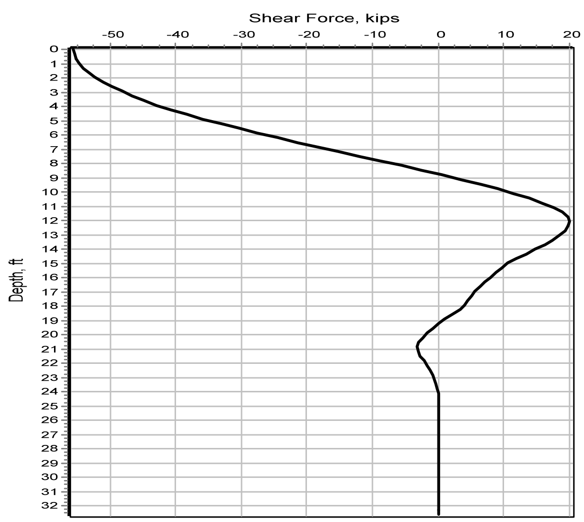
The analysis ended normally.



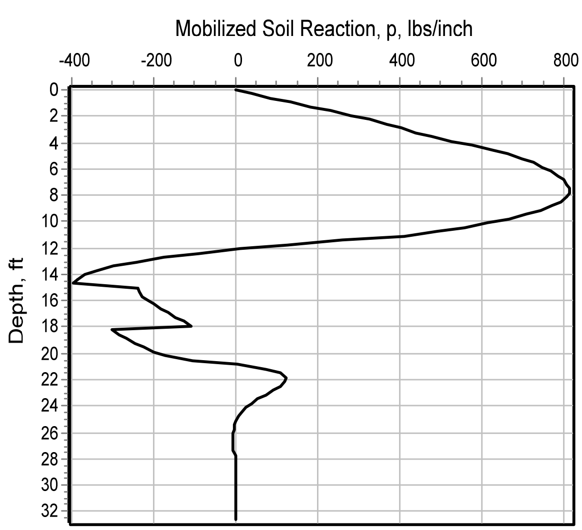
Deflection, inches
Depth, ft



Bending Moment, in-kips
Depth, ft



Shear Force, kips
Depth, ft



Mobilized Soil Reaction, p, lbs/inch
Depth, ft

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:
\\golderassociates.sharepoint.com@SSL\DavWWWRoot\sites\139980\Project Files\5 Technical Work\06 Analysis\Phase II Pile Design\LPILE\Abutment 1 Strength I\

Name of input data file:
Freeport Exit 20 Abutment 1 - KAR check - girder rotation Plastic Hinge.lp11d

Name of output report file:
Freeport Exit 20 Abutment 1 - KAR check - girder rotation Plastic Hinge.lp11o

Name of plot output file:
Freeport Exit 20 Abutment 1 - KAR check - girder rotation Plastic Hinge.lp11p

Name of runtime message file:
Freeport Exit 20 Abutment 1 - KAR check - girder rotation Plastic Hinge.lp11r

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Date and Time of Analysis

Date: July 14, 2021 Time: 15:57:30

Problem Title

Project Name: MaineDOT I-295 Exit 20 Merrill Road Bridge No. 5720
Job Number: 19126013
Client: MaineDOT
Engineer: KAR
Description: Northwest Abutment Pile Design - Strength I

Program Options and Settings

Computational Options:
- Conventional Analysis

Engineering Units Used for Data Input and Computations:
- US Customary System Units (pounds, feet, inches)

Analysis Control Options:
- Maximum number of iterations allowed = 500
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 100.0000 in
- Number of pile increments = 100

Loading Type and Number of Cycles of Loading:
- Static loading specified

- Use of p-y modification factors for p-y curves not selected
- Analysis uses layering correction (Method of Georgiadis)
- No distributed lateral loads are entered
- Loading by lateral soil movements acting on pile not selected
- Input of shear resistance at the pile tip not selected
- Input of moment resistance at the pile tip not selected
- Input of side resistance moment along pile not selected
- Computation of pile-head foundation stiffness matrix not selected
- Push-over analysis of pile not selected
- Buckling analysis of pile not selected

Output Options:
- Output files use decimal points to denote decimal symbols.
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (nodal spacing of output points) = 1
- No p-y curves to be computed and reported for user-specified depths
- Print using wide report formats

Pile Structural Properties and Geometry

Number of pile sections defined = 1
Total length of pile = 32.600 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	32.600	14.6950

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is a H weak axis steel pile
Length of section = 32.600000 ft
Pile width = 13.830000 in
Shear capacity of section = 0.0000 lbs

Ground Slope and Pile Batter Angles

Ground Slope Angle = 0.000 degrees
= 0.000 radians

Pile Batter Angle = 0.000 degrees
= 0.000 radians

Soil and Rock Layering Information

The soil profile is modelled using 5 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer = 0.0000 ft
Distance from top of pile to bottom of layer = 14.800000 ft

Effective unit weight at top of layer = 125.000000 pcf
Effective unit weight at bottom of layer = 125.000000 pcf
Friction angle at top of layer = 32.000000 deg.
Friction angle at bottom of layer = 32.000000 deg.
Subgrade k at top of layer = 124.800000 pci
Subgrade k at bottom of layer = 124.800000 pci

Layer 2 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer = 14.800000 ft
Distance from top of pile to bottom of layer = 18.100000 ft
Effective unit weight at top of layer = 62.600000 pcf
Effective unit weight at bottom of layer = 62.600000 pcf
Friction angle at top of layer = 32.000000 deg.
Friction angle at bottom of layer = 32.000000 deg.
Subgrade k at top of layer = 75.500000 pci
Subgrade k at bottom of layer = 75.500000 pci

Layer 3 is stiff clay without free water

Distance from top of pile to top of layer = 18.100000 ft
Distance from top of pile to bottom of layer = 27.700000 ft
Effective unit weight at top of layer = 62.600000 pcf
Effective unit weight at bottom of layer = 62.600000 pcf
Undrained cohesion at top of layer = 1600. psf
Undrained cohesion at bottom of layer = 1600. psf
Epsilon-50 at top of layer = 0.005000
Epsilon-50 at bottom of layer = 0.005000

Layer 4 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer = 27.700000 ft
Distance from top of pile to bottom of layer = 32.600000 ft
Effective unit weight at top of layer = 62.600000 pcf
Effective unit weight at bottom of layer = 62.600000 pcf
Friction angle at top of layer = 37.000000 deg.
Friction angle at bottom of layer = 37.000000 deg.
Subgrade k at top of layer = 40.500000 pci
Subgrade k at bottom of layer = 40.500000 pci

Layer 5 is strong rock (vuggy limestone)

Distance from top of pile to top of layer = 32.600000 ft
Distance from top of pile to bottom of layer = 50.000000 ft
Effective unit weight at top of layer = 101.600000 pcf
Effective unit weight at bottom of layer = 101.600000 pcf
Uniaxial compressive strength at top of layer = 12983. psi
Uniaxial compressive strength at bottom of layer = 12983. psi

(Depth of the lowest soil layer extends 17.400 ft below the pile tip)

Summary of Input Soil Properties

Layer Num.	Soil Type Name (p-y Curve Type)	Layer Depth ft	Effective Unit Wt. pcf	Undrained Cohesion psf	Angle of Friction deg.	Uniaxial qu psi	E50 or krm	kpy pci
1	Sand	0.00	125.0000	--	32.0000	--	--	124.8000
	(Reese, et al.)	14.8000	125.0000	--	32.0000	--	--	124.8000
2	Sand	14.8000	62.6000	--	32.0000	--	--	75.5000
	(Reese, et al.)	18.1000	62.6000	--	32.0000	--	--	75.5000
3	Stiff Clay	18.1000	62.6000	1600.	--	--	0.00500	--
	w/o Free Water	27.7000	62.6000	1600.	--	--	0.00500	--
4	Sand	27.7000	62.6000	--	37.0000	--	--	40.5000
	(Reese, et al.)	32.6000	62.6000	--	37.0000	--	--	40.5000
5	Strong Rock	32.6000	101.6000	--	--	12983.	--	--
	(Vuggy Limestone)	50.0000	101.6000	--	--	12983.	--	--

Static Loading Type

Static loading criteria were used when computing p-y curves for all analyses.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 2

Load No.	Load Type	Condition 1	Condition 2	Axial Thrust Force, lbs	Compute Top y vs. Pile Length	Run Analysis
1	5	y = -0.975000 in	S = 0.0000 in/in	465000.	N.A.	Yes
2	4	y = -0.975000 in	M = 1656234. in-lbs	465000.	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	=	32.600000 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	465.000

Definition of Run Messages:

Y = part of pipe section has yielded.

Axial Thrust Force = 465.000 kips

Bending Curvature rad/in.	Bending Moment in-kip	Bending Stiffness kip-in2	Depth to N Axis in	Max Total Stress ksi	Run Msg

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0.00000418	39.4155497	9438779.	156.0376435	18.8875060	
0.00000835	78.8310995	9438779.	81.6925717	19.7684019	
0.00001253	118.2466492	9438779.	56.9108812	20.6492980	
0.00001670	157.6621990	9438779.	44.5200359	21.5301933	
0.00002088	197.0777487	9438779.	37.0855287	22.4110895	
0.00002506	236.4932984	9438779.	32.1291906	23.2919854	
0.00002923	275.9088482	9438779.	28.5889491	24.1728812	
0.00003341	315.3243979	9438779.	25.9337679	25.0537772	
0.00003758	354.7399477	9438779.	23.8686271	25.9346731	
0.00004176	394.1554974	9438779.	22.2165143	26.8155689	
0.00004594	433.5710471	9438779.	20.8647858	27.6964648	
0.00005011	472.9865969	9438779.	19.7383453	28.5773606	
0.00005429	512.4021466	9438779.	18.7852033	29.4582566	
0.00005846	551.8176964	9438779.	17.9682245	30.3391524	
0.00006264	591.2332461	9438779.	17.2601762	31.2200483	
0.00006681	630.6487958	9438779.	16.6406340	32.1009442	
0.00007099	670.0643456	9438779.	16.0939790	32.9818400	
0.00007517	709.4798953	9438779.	15.6080635	33.8627359	
0.00007934	748.8954450	9438779.	15.1732970	34.7436318	
0.00008352	788.3109948	9438779.	14.7820072	35.6245276	
0.00008769	827.7265445	9438779.	14.4279830	36.5054235	
0.00009187	867.1420943	9438779.	14.1061429	37.3863195	
0.00009605	906.5576440	9438779.	13.8122888	38.2672153	
0.0001002	945.9731937	9438779.	13.5429226	39.1481112	
0.0001044	985.3887435	9438779.	13.2951057	40.0290070	
0.0001086	1025.	9438779.	13.0663517	40.9099029	
0.0001127	1064.	9438779.	12.8545424	41.7907988	
0.0001169	1104.	9438779.	12.6578623	42.6716947	
0.0001211	1143.	9438779.	12.4747463	43.5525906	
0.0001253	1182.	9438779.	12.3038381	44.4334864	
0.0001295	1222.	9438779.	12.1439562	45.3143823	
0.0001336	1261.	9438779.	11.9940670	46.1952782	
0.0001378	1301.	9438779.	11.8532619	47.0761741	
0.0001420	1340.	9438779.	11.7207395	47.9570699	
0.0001462	1380.	9438779.	11.5957898	48.8379658	
0.0001503	1419.	9438779.	11.4777818	49.7188617	
0.0001545	1458.	9433639.	11.3670960	50.0000000	Y
0.0001587	1495.	9418541.	11.2641477	50.0000000	Y
0.0001629	1530.	9395673.	11.1681210	50.0000000	Y
0.0001712	1598.	9331800.	10.9943887	50.0000000	Y
0.0001796	1661.	9249108.	10.8419386	50.0000000	Y
0.0001879	1720.	9152913.	10.7075643	50.0000000	Y
0.0001963	1776.	9047501.	10.5885833	50.0000000	Y
0.0002046	1829.	8936330.	10.4827291	50.0000000	Y
0.0002130	1879.	8822190.	10.3880673	50.0000000	Y
0.0002213	1927.	8705599.	10.3033989	50.0000000	Y
0.0002297	1973.	8589387.	10.2270557	50.0000000	Y
0.0002380	2017.	8472768.	10.1584484	50.0000000	Y
0.0002464	2059.	8357810.	10.0962941	50.0000000	Y
0.0002547	2100.	8244798.	10.0398817	50.0000000	Y
0.0002631	2140.	8133966.	9.9885881	50.0000000	Y
0.0002714	2178.	8025508.	9.9418642	50.0000000	Y
0.0002798	2216.	7919580.	9.8992241	50.0000000	Y

0.0002881	2252.	7816310.	9.8602356	50.0000000	Y
0.0002965	2288.	7715796.	9.8245125	50.0000000	Y
0.0003048	2322.	7618116.	9.7917085	50.0000000	Y
0.0003132	2356.	7523326.	9.7615115	50.0000000	Y
0.0003215	2390.	7431463.	9.7336391	50.0000000	Y
0.0003299	2422.	7342540.	9.7078385	50.0000000	Y
0.0003382	2454.	7255678.	9.6842426	50.0000000	Y
0.0003466	2486.	7171718.	9.6623145	50.0000000	Y
0.0003550	2516.	7086908.	9.6407330	50.0000000	Y
0.0003633	2543.	7000533.	9.6197794	50.0000000	Y
0.0003717	2570.	6913993.	9.5990744	50.0000000	Y
0.0003800	2594.	6827404.	9.5787658	50.0000000	Y
0.0003884	2618.	6741020.	9.5589435	50.0000000	Y
0.0003967	2640.	6655508.	9.5394404	50.0000000	Y
0.0004051	2661.	6569715.	9.5201513	50.0000000	Y
0.0004134	2681.	6485158.	9.5011879	50.0000000	Y
0.0004218	2700.	6401882.	9.4827107	50.0000000	Y
0.0004301	2718.	6319308.	9.4645010	50.0000000	Y
0.0004385	2735.	6238091.	9.4463322	50.0000000	Y
0.0004468	2752.	6158124.	9.4290013	50.0000000	Y
0.0004552	2767.	6079154.	9.4114989	50.0000000	Y
0.0004635	2782.	6001843.	9.3944082	50.0000000	Y
0.0004719	2796.	5925666.	9.3777834	50.0000000	Y
0.0004802	2810.	5850857.	9.3611587	50.0000000	Y
0.0004886	2823.	5777832.	9.3449499	50.0000000	Y
0.0004969	2835.	5705432.	9.3289874	50.0000000	Y
0.0005303	2880.	5431084.	9.2675962	50.0000000	Y
0.0005637	2919.	5178009.	9.2100273	50.0000000	Y
0.0005972	2952.	4943933.	9.1553729	50.0000000	Y
0.0006306	2982.	4728429.	9.1042653	50.0000000	Y
0.0006640	3007.	4528860.	9.0558319	50.0000000	Y
0.0006974	3030.	4344464.	9.0096933	50.0000000	Y
0.0007308	3050.	4173353.	8.9664559	50.0000000	Y
0.0007642	3068.	4014678.	8.9250764	50.0000000	Y
0.0007976	3084.	3866959.	8.8859993	50.0000000	Y
0.0008310	3099.	3729328.	8.8488422	50.0000000	Y
0.0008644	3113.	3600877.	8.8132568	50.0000000	Y
0.0008978	3125.	3480314.	8.7792545	50.0000000	Y
0.0009312	3136.	3367427.	8.7469879	50.0000000	Y

Summary of Results for Nominal Moment Capacity for Section 1

Load No.	Axial Thrust kips	Nominal Moment Capacity in-kips
1	465.0000000000	3136.

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	319356.
2	14.8000	14.8000	Yes	No	319356.	233723.
3	18.1000	35.5370	No	No	553079.	169286.
4	27.7000	18.6731	No	No	722365.	676707.
5	32.6000	32.6000	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 = -0.975000 inches
Rotation of pile head = 0.000E+00 radians
Axial load on pile head = 465000.0 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	-0.9750	2907557.	-55600.	0.00	83638.	5.25E+09	0.00	0.00	0.00
0.3260	-0.9708	2688527.	-55409.	0.00209	78694.	5.25E+09	39.0784	157.4792	0.00
0.6520	-0.9587	2466448.	-55166.	0.00376	73681.	7.22E+09	85.0148	346.9123	0.00

0.9780	-0.9414	2243240.	-54737.	0.00498	68642.	7.84E+09	134.4094	558.5557	0.00
1.3040	-0.9197	2020054.	-54113.	0.00601	63604.	8.46E+09	184.6473	785.4173	0.00
1.6300	-0.8944	1797995.	-53295.	0.00687	58592.	9.00E+09	233.6949	1022.	0.00
1.9560	-0.8660	1578091.	-52289.	0.00759	53628.	9.35E+09	280.4888	1267.	0.00
2.2820	-0.8350	1361278.	-51102.	0.00820	48734.	9.44E+09	326.1498	1528.	0.00
2.6080	-0.8018	1148430.	-49746.	0.00872	43930.	9.44E+09	367.4969	1793.	0.00
2.9340	-0.7668	940340.	-48237.	0.00915	39233.	9.44E+09	403.6882	2060.	0.00
3.2600	-0.7302	737720.	-46586.	0.00950	34659.	9.44E+09	440.7214	2361.	0.00
3.5860	-0.6924	541287.	-44789.	0.00977	30225.	9.44E+09	477.5703	2698.	0.00
3.9120	-0.6538	351756.	-42827.	0.00995	25947.	9.44E+09	525.5064	3145.	0.00
4.2380	-0.6146	170001.	-40669.	0.01006	21844.	9.44E+09	578.1125	3680.	0.00
4.5640	-0.5751	-3034.	-38307.	0.01009	18075.	9.44E+09	629.0625	4279.	0.00
4.8900	-0.5356	-166441.	-35779.	0.01006	21764.	9.44E+09	663.6354	4847.	0.00
5.2160	-0.4964	-319565.	-33116.	0.00996	25220.	9.44E+09	697.9111	5500.	0.00
5.5420	-0.4577	-461768.	-30328.	0.00980	28430.	9.44E+09	727.1449	6215.	0.00
5.8680	-0.4197	-592495.	-27439.	0.00958	31381.	9.44E+09	750.1074	6991.	0.00
6.1940	-0.3827	-711296.	-24469.	0.00931	34062.	9.44E+09	768.0895	7851.	0.00
6.5200	-0.3469	-817806.	-21428.	0.00899	36467.	9.44E+09	786.5739	8870.	0.00
6.8460	-0.3124	-911661.	-18324.	0.00863	38585.	9.44E+09	800.4027	10023.	0.00
7.1720	-0.2794	-992580.	-15175.	0.00824	40412.	9.44E+09	809.7805	11340.	0.00
7.4980	-0.2479	-1060359.	-11997.	0.00781	41942.	9.44E+09	815.0148	12860.	0.00
7.8240	-0.2182	-1114864.	-8808.	0.00736	43172.	9.44E+09	815.2029	14613.	0.00
8.1500	-0.1903	-1156054.	-5632.	0.00689	44102.	9.44E+09	808.4988	16617.	0.00
8.4760	-0.1643	-1183999.	-2496.	0.00641	44733.	9.44E+09	794.7054	18920.	0.00
8.8020	-0.1402	-1198890.	571.7004	0.00591	45069.	9.44E+09	773.6913	21586.	0.00
9.1280	-0.1181	-1201036.	3543.	0.00541	45117.	9.44E+09	745.3668	24699.	0.00
9.4540	-0.09785	-1190870.	6389.	0.00492	44888.	9.44E+09	709.6677	28373.	0.00
9.7800	-0.07957	-1168945.	9081.	0.00443	44393.	9.44E+09	666.5295	32770.	0.00
10.1060	-0.06319	-1135939.	11589.	0.00395	43648.	9.44E+09	615.8422	38129.	0.00
10.4320	-0.04864	-1092652.	13884.	0.00349	42671.	9.44E+09	557.3706	44824.	0.00
10.7580	-0.03587	-1040010.	15934.	0.00305	41482.	9.44E+09	490.6019	53499.	0.00
11.0840	-0.02479	-979077.	17698.	0.00263	40107.	9.44E+09	411.5038	64937.	0.00
11.4100	-0.01529	-911108.	19014.	0.00224	38573.	9.44E+09	261.3384	66847.	0.00
11.7360	-0.00728	-838453.	19776.	0.00188	36933.	9.44E+09	127.8643	68757.	0.00
12.0620	-6.15E-04	-763209.	20048.	0.00154	35234.	9.44E+09	11.1172	70667.	0.00
12.3880	0.00481	-687219.	19895.	0.00124	33519.	9.44E+09	-89.1751	72576.	0.00
12.7140	0.00911	-612076.	19381.	9.74E-04	31823.	9.44E+09	-173.5462	74486.	0.00
13.0400	0.01243	-539127.	18567.	7.36E-04	30176.	9.44E+09	-242.7434	76396.	0.00
13.3660	0.01487	-469487.	17510.	5.27E-04	28604.	9.44E+09	-297.6807	78306.	0.00
13.6920	0.01655	-404048.	16264.	3.46E-04	27127.	9.44E+09	-339.3932	80216.	0.00
14.0180	0.01758	-343499.	14878.	1.91E-04	25760.	9.44E+09	-368.9931	82126.	0.00
14.3440	0.01804	-288337.	13398.	5.99E-05	24515.	9.44E+09	-387.6301	84036.	0.00
14.6700	0.01805	-238891.	11864.	-4.93E-05	23399.	9.44E+09	-396.4533	85946.	0.00
14.9960	0.01766	-195331.	10620.	-1.39E-04	22416.	9.44E+09	-239.9172	53150.	0.00
15.3220	0.01696	-155296.	9690.	-2.12E-04	21512.	9.44E+09	-235.3680	54305.	0.00
15.6480	0.01600	-118746.	8786.	-2.69E-04	20687.	9.44E+09	-226.8337	55461.	0.00
15.9740	0.01485	-85578.	7922.	-3.11E-04	19938.	9.44E+09	-214.9486	56616.	0.00
16.3000	0.01357	-55635.	7109.	-3.40E-04	19262.	9.44E+09	-200.3365	57772.	0.00
16.6260	0.01219	-28715.	6358.	-3.58E-04	18655.	9.44E+09	-183.6057	58927.	0.00
16.9520	0.01077	-4584.	5676.	-3.65E-04	18110.	9.44E+09	-165.3466	60082.	0.00
17.2780	0.00934	17020.	5067.	-3.62E-04	18391.	9.44E+09	-146.1303	61238.	0.00
17.6040	0.00793	36375.	4533.	-3.51E-04	18828.	9.44E+09	-126.5091	62393.	0.00
17.9300	0.00659	53766.	4077.	-3.32E-04	19220.	9.44E+09	-107.0171	63549.	0.00
18.2560	0.00533	69479.	3274.	-3.07E-04	19575.	9.44E+09	-303.2720	222551.	0.00

18.5820	0.00419	80499.	2122.	-2.76E-04	19824.	9.44E+09	-285.4956	266769.	0.00
18.9080	0.00317	87089.	1043.	-2.41E-04	19972.	9.44E+09	-266.3777	328435.	0.00
19.2340	0.00230	89536.	41.1326	-2.04E-04	20028.	9.44E+09	-245.8007	418029.	0.00
19.5600	0.00157	88154.	-876.8572	-1.68E-04	19996.	9.44E+09	-223.5192	555941.	0.00
19.8860	9.88E-04	83286.	-1703.	-1.32E-04	19887.	9.44E+09	-199.0122	787701.	0.00
20.2120	5.39E-04	75308.	-2427.	-9.93E-05	19707.	9.44E+09	-171.0192	1241412.	0.00
20.5380	2.12E-04	64657.	-2969.	-7.03E-05	19466.	9.44E+09	-105.7929	1956000.	0.00
20.8640	-1.09E-05	52338.	-3165.	-4.60E-05	19188.	9.44E+09	5.4598	1956000.	0.00
21.1900	-1.49E-04	40063.	-3009.	-2.69E-05	18911.	9.44E+09	74.2828	1956000.	0.00
21.5160	-2.21E-04	28895.	-2647.	-1.26E-05	18659.	9.44E+09	110.6274	1956000.	0.00
21.8420	-2.47E-04	19398.	-2189.	-2.59E-06	18444.	9.44E+09	123.5474	1956000.	0.00
22.1680	-2.41E-04	11777.	-1711.	3.87E-06	18272.	9.44E+09	120.7420	1956000.	0.00
22.4940	-2.17E-04	5995.	-1263.	7.56E-06	18142.	9.44E+09	108.3895	1956000.	0.00
22.8200	-1.82E-04	1867.	-872.7356	9.19E-06	18049.	9.44E+09	91.1773	1956000.	0.00
23.1460	-1.45E-04	-867.1378	-552.6771	9.39E-06	18026.	9.44E+09	72.4518	1956000.	0.00
23.4720	-1.09E-04	-2492.	-304.4979	8.70E-06	18063.	9.44E+09	54.4292	1956000.	0.00
23.7980	-7.69E-05	-3281.	-122.8720	7.50E-06	18081.	9.44E+09	38.4265	1956000.	0.00
24.1240	-5.02E-05	-3480.	1.3543	6.10E-06	18085.	9.44E+09	25.0839	1956000.	0.00
24.4500	-2.91E-05	-3293.	78.9026	4.70E-06	18081.	9.44E+09	14.5625	1956000.	0.00
24.7760	-1.34E-05	-2880.	120.5129	3.42E-06	18072.	9.44E+09	6.7106	1956000.	0.00
25.1020	-2.39E-06	-2362.	135.9732	2.33E-06	18060.	9.44E+09	1.1934	1956000.	0.00
25.4280	4.82E-06	-1825.	133.5963	1.46E-06	18048.	9.44E+09	-2.4086	1956000.	0.00
25.7540	9.06E-06	-1322.	120.0213	8.11E-07	18036.	9.44E+09	-4.5315	1956000.	0.00
26.0800	1.12E-05	-888.5063	100.2384	3.53E-07	18027.	9.44E+09	-5.5824	1956000.	0.00
26.4060	1.18E-05	-539.3970	77.7534	5.72E-08	18019.	9.44E+09	-5.9130	1956000.	0.00
26.7320	1.16E-05	-280.3720	54.8305	-1.13E-07	18013.	9.44E+09	-5.8063	1956000.	0.00
27.0580	1.09E-05	-109.9934	32.7696	-1.94E-07	18009.	9.44E+09	-5.4723	1956000.	0.00
27.3840	1.01E-05	-23.2783	12.1898	-2.21E-07	18007.	9.44E+09	-5.0491	1956000.	0.00
27.7100	9.21E-06	-13.8161	2.0710	-2.29E-07	18007.	9.44E+09	-0.1241	52683.	0.00
28.0360	8.31E-06	-6.2424	1.6068	-2.33E-07	18007.	9.44E+09	-0.1132	53303.	0.00
28.3620	7.39E-06	-0.3964	1.1862	-2.34E-07	18007.	9.44E+09	-0.1019	53923.	0.00
28.6880	6.47E-06	3.8908	0.8103	-2.34E-07	18007.	9.44E+09	-0.09026	54543.	0.00
29.0140	5.56E-06	6.7938	0.4804	-2.31E-07	18007.	9.44E+09	-0.07844	55162.	0.00
29.3400	4.66E-06	8.4912	0.1969	-2.28E-07	18007.	9.44E+09	-0.06649	55782.	0.00
29.6660	3.78E-06	9.1646	-0.03969	-2.25E-07	18007.	9.44E+09	-0.05445	56402.	0.00
29.9920	2.91E-06	8.9978	-0.2290	-2.21E-07	18007.	9.44E+09	-0.04235	57022.	0.00
30.3180	2.05E-06	8.1761	-0.3709	-2.17E-07	18007.	9.44E+09	-0.03019	57642.	0.00
30.6440	1.21E-06	6.8862	-0.4651	-2.14E-07	18007.	9.44E+09	-0.01795	58261.	0.00
30.9700	3.73E-07	5.3165	-0.5112	-2.12E-07	18007.	9.44E+09	-0.00562	58881.	0.00
31.2960	-4.51E-07	3.6567	-0.5088	-2.10E-07	18007.	9.44E+09	0.00685	59501.	0.00
31.6220	-1.27E-06	2.0991	-0.4572	-2.09E-07	18007.	9.44E+09	0.01949	60121.	0.00
31.9480	-2.08E-06	0.8382	-0.3559	-2.08E-07	18007.	9.44E+09	0.03234	60741.	0.00
32.2740	-2.90E-06	0.07161	-0.2038	-2.08E-07	18007.	9.44E+09	0.04542	61360.	0.00
32.6000	-3.71E-06	0.00	0.00	-2.08E-07	18007.	9.44E+09	0.05876	30990.	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 = -0.97500000 inches
Computed slope at pile head = 0.000000 radians
Maximum bending moment = 2907557. inch-lbs
Maximum shear force = -55600. 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 = 16
Number of zero deflection points = 4

Computed Values of Pile Loading and Deflection
for Lateral Loading for Load Case Number 2

Pile-head conditions are Displacement and Moment (Loading Type 4)
Displacement of pile head = -0.975000 inches
Moment at pile head = 1656234.0 in-lbs
Axial load at pile head = 465000.0 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	-0.9750	1656234.	-40988.	0.00679	55392.	9.25E+09	0.00	0.00	0.00
0.3260	-0.9471	1482907.	-40911.	0.00745	51480.	9.25E+09	39.0782	161.4160	0.00
0.6520	-0.9167	1309038.	-40669.	0.00803	47555.	9.44E+09	85.0143	362.7934	0.00
0.9780	-0.8842	1135483.	-40239.	0.00854	43637.	9.44E+09	134.4085	594.6585	0.00
1.3040	-0.8499	963129.	-39615.	0.00898	39747.	9.44E+09	184.6458	849.9251	0.00
1.6300	-0.8140	792874.	-38797.	0.00934	35904.	9.44E+09	233.6927	1123.	0.00
1.9560	-0.7768	625598.	-37791.	0.00963	32128.	9.44E+09	280.4860	1413.	0.00
2.2820	-0.7386	462143.	-36605.	0.00986	28438.	9.44E+09	326.1462	1727.	0.00
2.6080	-0.6997	303330.	-35248.	0.01002	24854.	9.44E+09	367.4925	2055.	0.00
2.9340	-0.6602	149913.	-33740.	0.01011	21391.	9.44E+09	403.6830	2392.	0.00
3.2600	-0.6205	2561.	-32088.	0.01014	18064.	9.44E+09	440.7153	2778.	0.00
3.5860	-0.5809	-138049.	-30292.	0.01012	21123.	9.44E+09	477.5632	3216.	0.00
3.9120	-0.5414	-271246.	-28341.	0.01003	24129.	9.44E+09	519.7913	3756.	0.00
4.2380	-0.5024	-396283.	-26259.	0.00989	26952.	9.44E+09	544.6180	4241.	0.00
4.5640	-0.4640	-512688.	-24099.	0.00970	29579.	9.44E+09	559.9438	4721.	0.00
4.8900	-0.4264	-620136.	-21899.	0.00947	32005.	9.44E+09	564.4717	5178.	0.00
5.2160	-0.3899	-718479.	-19644.	0.00919	34225.	9.44E+09	588.7304	5907.	0.00
5.5420	-0.3545	-807269.	-17302.	0.00888	36229.	9.44E+09	608.5275	6715.	0.00
5.8680	-0.3205	-886139.	-14893.	0.00853	38009.	9.44E+09	622.8101	7603.	0.00
6.1940	-0.2878	-954809.	-12436.	0.00814	39559.	9.44E+09	633.2575	8607.	0.00
6.5200	-0.2567	-1013068.	-9934.	0.00774	40874.	9.44E+09	646.0987	9845.	0.00
6.8460	-0.2273	-1060676.	-7389.	0.00731	41949.	9.44E+09	655.1569	11276.	0.00
7.1720	-0.1996	-1097457.	-4819.	0.00686	42779.	9.44E+09	658.7007	12911.	0.00
7.4980	-0.1736	-1123331.	-2247.	0.00640	43363.	9.44E+09	656.3053	14786.	0.00
7.8240	-0.1495	-1138314.	304.1734	0.00593	43701.	9.44E+09	647.7650	16948.	0.00
8.1500	-0.1272	-1142525.	2809.	0.00546	43796.	9.44E+09	632.9185	19458.	0.00
8.4760	-0.1068	-1136189.	5244.	0.00498	43653.	9.44E+09	611.6403	22399.	0.00
8.8020	-0.08824	-1119635.	7582.	0.00452	43280.	9.44E+09	583.8266	25882.	0.00

9.1280	-0.07148	-1093303.	9798.	0.00406	42685.	9.44E+09	549.3713	30067.	0.00
9.4540	-0.05649	-1057740.	11867.	0.00361	41883.	9.44E+09	508.1240	35191.	0.00
9.7800	-0.04321	-1013602.	13760.	0.00318	40886.	9.44E+09	459.8144	41631.	0.00
10.1060	-0.03157	-961664.	15450.	0.00277	39714.	9.44E+09	403.9053	50043.	0.00
10.4320	-0.02150	-902819.	16897.	0.00239	38386.	9.44E+09	335.8939	61117.	0.00
10.7580	-0.01289	-838153.	17960.	0.00203	36926.	9.44E+09	207.6616	63027.	0.00
11.0840	-0.00564	-769677.	18549.	0.00169	35380.	9.44E+09	93.5794	64937.	0.00
11.4100	3.66E-04	-699189.	18720.	0.00139	33789.	9.44E+09	-6.2596	66847.	0.00
11.7360	0.00524	-628269.	18528.	0.00111	32188.	9.44E+09	-92.0362	68757.	0.00
12.0620	0.00909	-558285.	18026.	8.69E-04	30609.	9.44E+09	-164.1671	70667.	0.00
12.3880	0.01203	-490391.	17269.	6.52E-04	29076.	9.44E+09	-223.2656	72576.	0.00
12.7140	0.01419	-425545.	16304.	4.62E-04	27612.	9.44E+09	-270.1019	74486.	0.00
13.0400	0.01565	-364512.	15178.	2.98E-04	26235.	9.44E+09	-305.5645	76396.	0.00
13.3660	0.01652	-307879.	13933.	1.59E-04	24956.	9.44E+09	-330.6239	78306.	0.00
13.6920	0.01689	-256075.	12609.	4.18E-05	23787.	9.44E+09	-346.2971	80216.	0.00
14.0180	0.01684	-209377.	11240.	-5.47E-05	22733.	9.44E+09	-353.6164	82126.	0.00
14.3440	0.01646	-167933.	9857.	-1.33E-04	21797.	9.44E+09	-353.6001	84036.	0.00
14.6700	0.01580	-131774.	8486.	-1.95E-04	20981.	9.44E+09	-347.2273	85946.	0.00
14.9960	0.01494	-100830.	7410.	-2.43E-04	20283.	9.44E+09	-202.9158	53150.	0.00
15.3220	0.01390	-72914.	6636.	-2.79E-04	19652.	9.44E+09	-192.9872	54305.	0.00
15.6480	0.01275	-47897.	5904.	-3.04E-04	19088.	9.44E+09	-180.7724	55461.	0.00
15.9740	0.01152	-25611.	5225.	-3.19E-04	18585.	9.44E+09	-166.7536	56616.	0.00
16.3000	0.01025	-5857.	4602.	-3.26E-04	18139.	9.44E+09	-151.3956	57772.	0.00
16.6260	0.00897	11584.	4042.	-3.25E-04	18268.	9.44E+09	-135.1442	58927.	0.00
16.9520	0.00771	26949.	3546.	-3.17E-04	18615.	9.44E+09	-118.4252	60082.	0.00
17.2780	0.00649	40480.	3115.	-3.03E-04	18920.	9.44E+09	-101.6452	61238.	0.00
17.6040	0.00534	52426.	2750.	-2.84E-04	19190.	9.44E+09	-85.1929	62393.	0.00
17.9300	0.00427	63028.	2448.	-2.60E-04	19429.	9.44E+09	-69.4410	63549.	0.00
18.2560	0.00331	72520.	1785.	-2.32E-04	19644.	9.44E+09	-269.1811	318127.	0.00
18.5820	0.00246	77838.	769.7122	-2.00E-04	19764.	9.44E+09	-250.0024	397064.	0.00
18.9080	0.00174	79272.	-167.7362	-1.68E-04	19796.	9.44E+09	-229.2658	514774.	0.00
19.2340	0.00115	77137.	-1020.	-1.35E-04	19748.	9.44E+09	-206.6390	702924.	0.00
19.5600	6.83E-04	71781.	-1779.	-1.05E-04	19627.	9.44E+09	-181.3727	1039149.	0.00
19.8860	3.32E-04	63596.	-2430.	-7.65E-05	19442.	9.44E+09	-151.4217	1784392.	0.00
20.2120	8.42E-05	53045.	-2809.	-5.23E-05	19204.	9.44E+09	-42.1249	1956000.	0.00
20.5380	-7.75E-05	41810.	-2815.	-3.27E-05	18950.	9.44E+09	38.7316	1956000.	0.00
20.8640	-1.71E-04	31136.	-2572.	-1.76E-05	18709.	9.44E+09	85.6937	1956000.	0.00
21.1900	-2.15E-04	21749.	-2194.	-6.60E-06	18498.	9.44E+09	107.4146	1956000.	0.00
21.5160	-2.23E-04	13991.	-1766.	8.09E-07	18322.	9.44E+09	111.5036	1956000.	0.00
21.8420	-2.09E-04	7928.	-1344.	5.35E-06	18186.	9.44E+09	104.2506	1956000.	0.00
22.1680	-1.81E-04	3455.	-963.0921	7.71E-06	18085.	9.44E+09	90.5706	1956000.	0.00
22.4940	-1.48E-04	364.6191	-641.0161	8.50E-06	18015.	9.44E+09	74.0900	1956000.	0.00
22.8200	-1.15E-04	-1592.	-383.9903	8.25E-06	18043.	9.44E+09	57.3138	1956000.	0.00
23.1460	-8.37E-05	-2670.	-190.0689	7.36E-06	18067.	9.44E+09	41.8280	1956000.	0.00
23.4720	-5.70E-05	-3106.	-52.4946	6.17E-06	18077.	9.44E+09	28.5065	1956000.	0.00
23.7980	-3.54E-05	-3103.	37.8906	4.88E-06	18077.	9.44E+09	17.7027	1956000.	0.00
24.1240	-1.88E-05	-2827.	90.9313	3.65E-06	18070.	9.44E+09	9.4143	1956000.	0.00
24.4500	-6.84E-06	-2405.	116.0306	2.57E-06	18061.	9.44E+09	3.4176	1956000.	0.00
24.7760	1.26E-06	-1928.	121.4841	1.67E-06	18050.	9.44E+09	-0.6295	1956000.	0.00
25.1020	6.23E-06	-1460.	114.1629	9.67E-07	18040.	9.44E+09	-3.1134	1956000.	0.00
25.4280	8.83E-06	-1039.	99.4405	4.49E-07	18030.	9.44E+09	-4.4134	1956000.	0.00
25.7540	9.74E-06	-683.9117	81.2796	9.24E-08	18022.	9.44E+09	-4.8713	1956000.	0.00
26.0800	9.55E-06	-403.1310	62.4117	-1.33E-07	18016.	9.44E+09	-4.7748	1956000.	0.00
26.4060	8.70E-06	-195.1189	44.5607	-2.57E-07	18011.	9.44E+09	-4.3515	1956000.	0.00

26.7320	7.54E-06	-53.5539	28.6750	-3.08E-07	18008.	9.44E+09	-3.7700	1956000.	0.00
27.0580	6.29E-06	30.3564	15.1491	-3.13E-07	18007.	9.44E+09	-3.1451	1956000.	0.00
27.3840	5.09E-06	66.1124	4.0198	-2.93E-07	18008.	9.44E+09	-2.5448	1956000.	0.00
27.7100	4.00E-06	62.8741	-1.0630	-2.66E-07	18008.	9.44E+09	-0.05382	52683.	0.00
28.0360	3.00E-06	58.7648	-1.2484	-2.41E-07	18008.	9.44E+09	-0.04094	53303.	0.00
28.3620	2.11E-06	53.9847	-1.3853	-2.18E-07	18008.	9.44E+09	-0.02906	53923.	0.00
28.6880	1.30E-06	48.7191	-1.4776	-1.97E-07	18008.	9.44E+09	-0.01812	54543.	0.00
29.0140	5.70E-07	43.1394	-1.5288	-1.78E-07	18008.	9.44E+09	-0.00804	55162.	0.00
29.3400	-8.97E-08	37.4042	-1.5420	-1.61E-07	18007.	9.44E+09	0.00128	55782.	0.00
29.6660	-6.89E-07	31.6603	-1.5201	-1.47E-07	18007.	9.44E+09	0.00993	56402.	0.00
29.9920	-1.24E-06	26.0446	-1.4654	-1.35E-07	18007.	9.44E+09	0.01802	57022.	0.00
30.3180	-1.74E-06	20.6850	-1.3799	-1.25E-07	18007.	9.44E+09	0.02567	57642.	0.00
30.6440	-2.21E-06	15.7027	-1.2652	-1.17E-07	18007.	9.44E+09	0.03297	58261.	0.00
30.9700	-2.66E-06	11.2131	-1.1224	-1.12E-07	18007.	9.44E+09	0.04005	58881.	0.00
31.2960	-3.09E-06	7.3279	-0.9522	-1.08E-07	18007.	9.44E+09	0.04698	59501.	0.00
31.6220	-3.51E-06	4.1562	-0.7549	-1.06E-07	18007.	9.44E+09	0.05387	60121.	0.00
31.9480	-3.92E-06	1.8058	-0.5306	-1.04E-07	18007.	9.44E+09	0.06079	60741.	0.00
32.2740	-4.32E-06	0.3843	-0.2791	-1.04E-07	18007.	9.44E+09	0.06779	61360.	0.00
32.6000	-4.73E-06	0.00	0.00	-1.04E-07	18007.	9.44E+09	0.07491	30990.	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 = -0.97500000 inches
Computed slope at pile head = 0.00678664 radians
Maximum bending moment = 1656234. inch-lbs
Maximum shear force = -40988. 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 = 4

Summary of Pile-head Responses for Conventional Analyses

Definitions of Pile-head Loading Conditions:

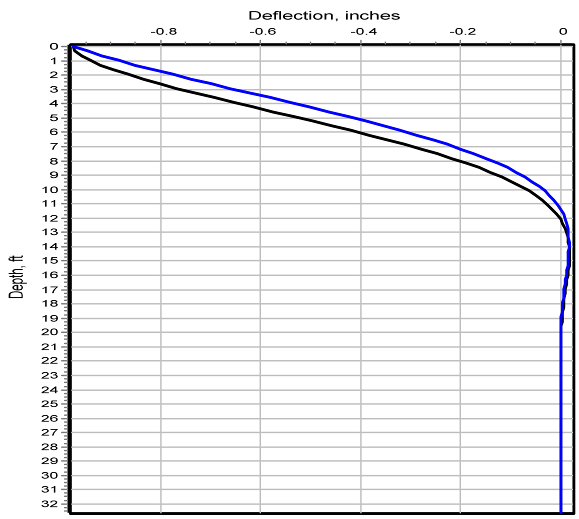
Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians
Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
Load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

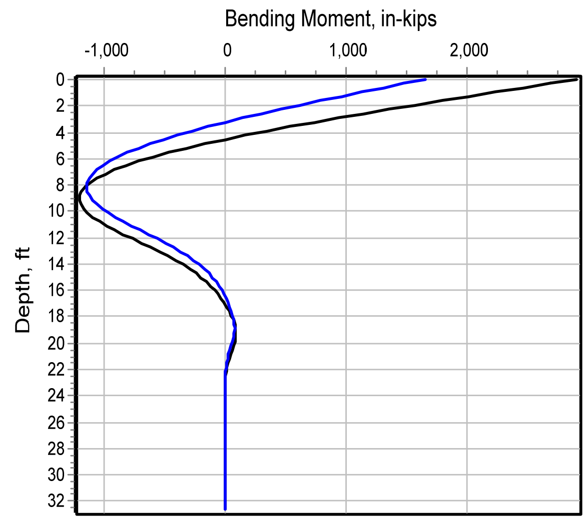
Load Load Load Axial Pile-head Pile-head Max Shear Max Moment

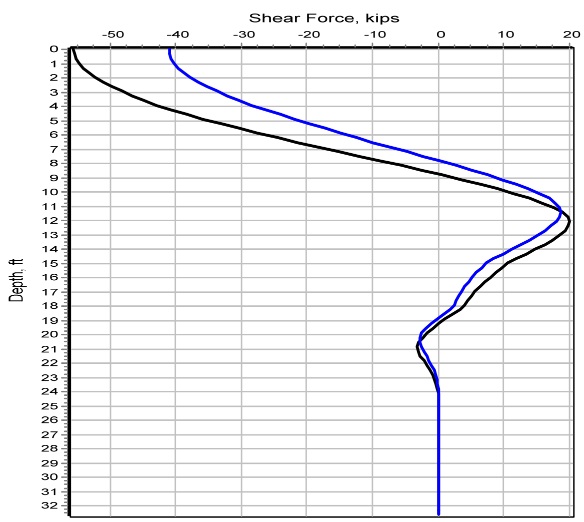
Case No.	Type 1	Pile-head Load 1	Type 2	Pile-head Load 2	Loading lbs	Deflection inches	Rotation radians	in Pile lbs	in Pile in-lbs
1	y, in	-0.9750	S, rad	0.00	465000.	-0.9750	0.00	-55600.	2907557.
2	y, in	-0.9750	M, in-lb	1656234.	465000.	-0.9750	0.00679	-40988.	1656234.

Maximum pile-head deflection = -0.9750000000 inches
Maximum pile-head rotation = 0.0067866398 radians = 0.388846 deg.

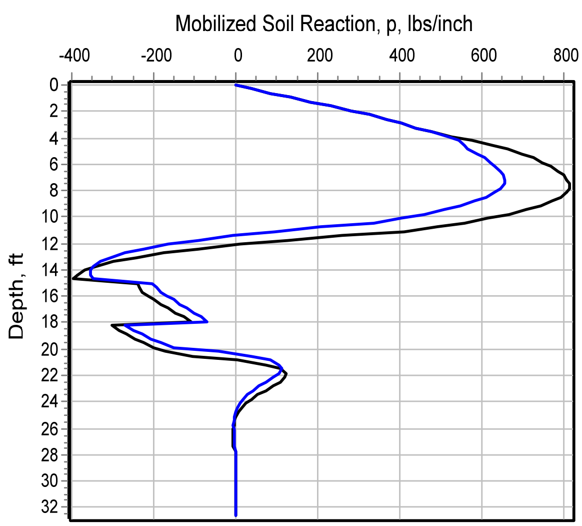
The analysis ended normally.







Legend:
----- First Iteration Load Case
(with axial load and lateral deflection applied to pile head)
----- Second Iteration Load Case
(with axial load, lateral deflection, and plastic hinge moment applied to pile head)



Attachment 3

GOLDER MEMBER OF WSP	SUBJECT: MaineDOT Desert Road Bridge 5720 (Exit 20)		
	Job No.: 21450908	Prepared: MSG/MEL	5/26/2021
	Location: Freeport, Maine	Checked: KAR	6/4/2021
	Date: 5/12/2021	Reviewed: CCB	6/7/2021

Pile Design at Proposed Abutment 1 - Downdrag Analysis

Description:

Evaluate the downdrag load for Abutment 1 driven piles using APILE

References:

1 Golder Associates Inc.; "Preliminary Geotechnical Design Report - I-295 Desert Road Bridge Replacement"; December 21, 2020
2 AASHTO; "AASHTO LRFD Bridge Design Specifications - 9th Edition", 2020
3 FHWA; Design and Construction of Driven Pile Foundations - Volume 1; FHWA GEC 012; FHWA-NHI-16-009; July 2016
4 FHWA; Design and Construction of Driven Pile Foundations - Volume 2; FHWA GEC 012; FHWA-NHI-16-009; July 2016
5 FHWA; Design and Construction of Driven Pile Foundations - Comprehensive Design Examples; FHWA GEC 012; FHWA-NHI-16-064; September 2016
6 Wyllie, DA; Foundations on Rock, 2nd Edition; E&FN Spon; 1999
7 Siegel, TC et al; "Alternative Design Approach for Drag Load and Downdrag of Deep Foundations within the LRFD Framework"; Proceedings 38th Deep Foundations; 2013
8 Geotechnical Design Manual, Chapter 8 - Foundations, Oregon Department of Transportation, Geo-Environmental Section, Version 2.1, May 6,
9 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.
10 Golder settlement model created using Rocscience Settle3 software package, Version 5.010 64-bit, build date Mar 5, 2021
11 HNTB. May 21, 2021. Merrill Road Bridge, Interstate 295: 60% Plans.
12 HNTB. May 26, 2021. Freeport Bridges_Loads_Bottom of Footing_flat.pdf.
13 Golder's Phase II Updated Interpreted Subsurface Profile using HNTB design references.

Assumptions:

1. Settlement greater than or equal to 0.4" is needed for downdrag to fully develop (Ref 2).
2. The soil profile analyzed (Ref 1) is the interpreted profile where maximum settlement occurs along the abutment.
3. Any downdrag load that may develop along the back of the abutment due to settlement is not included in the pile downdrag analysis.
4. The FHWA automated computation method provided in APILE is used for the software computations of unit load transfers and axial pile capacity.

Calculations:

				Top Elev (ft)	Bot. Elev (ft)	Thickness (ft)
Granular Backfill Unit Weight:	γ	125	pcf	171.0	160.0	11.0
Granular Backfill Friction Angle:	φ	32	deg			
Passive Earth Pressure Coefficient	K_p	3.93				
Active Earth Pressure Coefficient	K_a	0.31				

Strength I Loads

Strength I Factored Vertical Load per pile (kips) = 357 No. piles = 9 (Ref. 12)

Starting elevation of the pile	-	160.0	ft	Ref. 11
Ending elevation of the pile	-	127.4	ft	Ref. 13
Box perimeter of pile	P:	57.05	in	for HP14x89
Segment Length:	L:	12	in	
Cross-sectional Area:	A_s:	26.1	in^2	for HP14x89
Elastic Modulus of Pile	E:	29000	ksi	
Nominal Weight of Pile		0.089	kip/ft	
Factored Pile Strength = ϕFyAs	P_r:	652.5	kips	Ref 3 Eq 8-35
	Fy :	50	ksi	Ref 3 Table 8-2
	ϕ :	0.5	-	Ref 2 Article 6.5.4.2 for axial resistance of H-piles in compression and subject to severe driving conditions

Non-Cohesive Soil Layers -Nordlund/Thurman Method

	Parameters					
Soil Layer	**ϕ_f (deg)**	**γ (pcf)**	**γ_{pDD}**	Top Elev (ft)	Bot. Elev (ft)	Thickness (ft)
Soil 1	32	125	1.1	160.0	141.9	18.1	Fill, Ref. 1 and Ref. 13
Soil 3	37	125	1.1	132.3	127.4	4.9	Sand/Gravel, Ref. 1 and Ref. 13

ϕ_f: Based on empirical correlation to avg of N_{60} values encountered in all borings for layer

γ: Unit Weight

γ_{pDD}: STR 1 Load Factor for Downdrag (Ref 1 and Ref 8)

Cohesive Soil Layers - Alpha Method

Pile Depth	D_b:	9.6 ft	Ref 2, C10.7.3.8.6b
Pile Width	D:	1.15 ft	
Ratio D_b/D	D_b:	8 D	

GOLDER MEMBER OF WSP	SUBJECT: MaineDOT Desert Road Bridge 5720 (Exit 20)			
	Job No.: 21450908	**Prepared:**	MSG/MEL	5/26/2021
	Location: Freeport, Maine	**Checked:**	KAR	6/4/2021
	Date: 5/12/2021	**Reviewed:**	CCB	6/7/2021

	Parameters								
Soil Layer	S_u (ksf)	γ (pcf)	α	q_s (ksf)	γ_{pDD}	Top Elev (ft)	Bot. Elev (ft)	Thickness (ft)	
Soil 2	1.60	125	1.0	1.60	1.4	141.9	132.3	9.6	Glaciomarine clay, Ref. 1 and Ref. 13

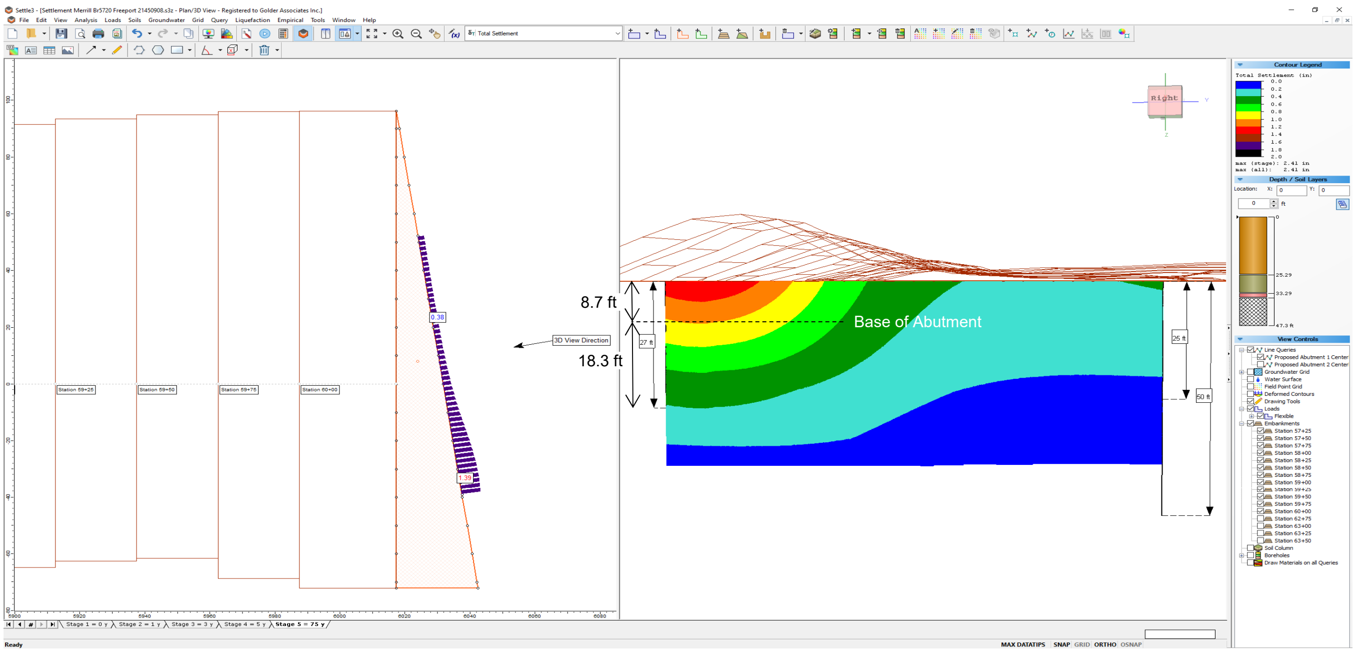
1. Identify deepest depth below the ground surface along the abutment centerline where settlement less than or equal to 0.4 inches.

Ref. 2 Article 3.11.8 indicates that full downdrag loading occurs where settlement is equal to or greater than 0.4 inches.

From the Settle3 model (Ref 10) image below, maximum depth to the 0.4" or less settlement contour below base of pile cap: **18.3**

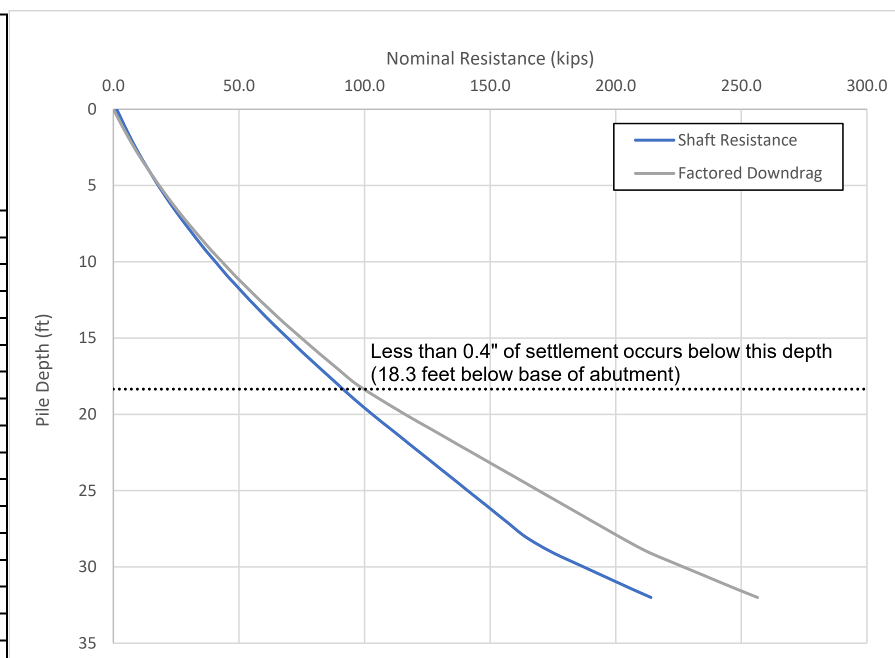
Thus, downdrag loading below this elevation is not considered fully developed and is not included in this analysis.

Settle3 Model from Abutment 1 loading.



2. Determine downdrag loading using APILE (Ensoft) Shaft Resistance

Pile Depth Below Pile Cap	Type of Soil	Shaft Resistance	Factored Downdrag Load	Total Factored Axial Load w/ Down Drag & Pile Weight
(feet)		(kips)	(kips)	(kips)
0	1	1.3	0	357.3
1	1	4.2	3	360.6
2	1	7.2	6	364.0
3	1	10.5	10	367.7
4	1	14.1	14	371.8
5	1	17.9	18	376.0
6	1	21.9	23	380.5
7	1	26.2	27	385.3
8	1	30.8	32	390.5
9	1	35.5	38	395.8
10	1	40.6	43	401.5
11	1	45.8	49	407.3
12	1	51.4	55	413.5
13	1	57.1	61	419.9
14	1	63.1	68	426.6
15	1	69.4	75	433.6
16	1	75.8	82	440.7



17	1	82.4	89	448.1
18	1	89.1	97	455.5
19	**2**	**95.9**	**106**	**465.1**
20	2	103.1	116	475.3
21	2	110.7	127	486.0
22	2	118.3	137	496.8
23	2	125.9	148	507.5
24	2	133.5	159	518.2
25	2	141.1	169	528.9
26	2	148.7	180	539.7
27	2	156.3	191	550.4
28	2	163.9	201	561.1
29	3	174.2	213	572.5
30	3	187.3	227	587.0
31	3	200.5	242	601.7
32	3	214.0	256	616.6

Type of Soil	g_{pDD}	Downdrag Load Factor
1	1.1	Strength I Load Factor for Down Drag (Ref 1, Ref 8 Table 8.2)
2	1.4	
3	1.1	
1 - 3	1.0	Service and Extreme Load Factor for Down Drag (Ref 2, Ref 8

652.5	Factored Strength of Pile
465.1	Maximum Factored Axial Load w/ Downdrag & Pile Weight
OK	OK?

Total Factored Downdrag Load,	**106 kips**	(per pile)
Strength I Limit State	**955 kips**	(per abutment - 9 piles/abutment)
Total Factored Downdrag Load,	**96 kips**	(per pile)
Extreme & Service Limit States	**863 kips**	(per abutment - 9 piles/abutment)

Conclusions:

Based on the Settle3 model, downdrag is estimated to develop along the upper 18.3 feet of the pile. A total factored downdrag load of 106 kips per pile was calculated for the Strength I load case and a total factored downdrag load of 96 kips per pile was calculated for the Extreme and Service limit load cases and will be conservatively applied to the top of the pile in the lateral response analysis.

==

APILE for Windows, Version 2019.9.6

Serial Number : 161219145

A Program for Analyzing the Axial Capacity
and Short-term Settlement of Driven Piles
under Axial Loading.

==

This program is licensed to :

Golder Associates, Ltd.
Gladstone, MI

Path to file locations : C:\Users\dfalish\Golder Associates\21450908 MaineDOT Desert Rd Bridge 5720 p2 Freeport - Project Files\5 Technical Work\06 Analysis\MSG_Downdrag\HP14x89\
Name of input data file : HP14x89_Abutment 1.ap9d
Name of output file : HP14x89_Abutment 1.ap9o
Name of plot output file : HP14x89_Abutment 1.ap9p

Time and Date of Analysis

Date: June 04, 2021 Time: 15:39:30

1

****
* INPUT INFORMATION *
****

New Pile

DESIGNER :

JOB NUMBER :

METHOD FOR UNIT LOAD TRANSFERS :

- FHWA (Federal Highway Administration)
Unfactored Unit Side Friction and Unit Side Resistance are used.

COMPUTATION METHOD(S) FOR PILE CAPACITY :

- FHWA (Federal Highway Administration)

TYPE OF LOADING :
- COMPRESSION

PILE TYPE :

H-Pile/Steel Pile

DATA FOR AXIAL STIFFNESS :

- MODULUS OF ELASTICITY = 0.290E+08 PSI
- CROSS SECTION AREA = 26.10 IN2

NONCIRCULAR PILE PROPERTIES :

- TOTAL PILE LENGTH, TL = 43.60 FT.
- BATTER ANGLE = 0.00 DEG
- PILE STICKUP LENGTH, PSL = 0.00 FT.
- ZERO FRICTION LENGTH, ZFL = 11.00 FT.
- PERIMETER OF PILE = 57.05 IN.
- TIP AREA OF PILE = 26.10 IN2
- INCREMENT OF PILE LENGTH
  USED IN COMPUTATION = 1.00 FT.

SOIL INFORMATIONS :

DEPTH FT.	SOIL TYPE	LATERAL EARTH PRESSURE	EFFECTIVE UNIT WEIGHT LB/FT^3	FRICTION ANGLE DEGREES	BEARING CAPACITY FACTOR
0.00	SAND	0.80*	125.00	32.00	28.00**
11.00	SAND	0.80*	125.00	32.00	28.00**
11.00	SAND	0.80*	125.00	32.00	28.00**
25.80	SAND	0.80*	125.00	32.00	28.00**
25.80	SAND	0.80*	62.60	32.00	28.00**
29.10	SAND	0.80*	62.60	32.00	28.00**
29.10	CLAY	0.80*	62.60	0.00	8.00**
38.70	CLAY	0.80*	62.60	0.00	8.00**
38.70	SAND	0.80*	62.60	37.00	44.00**
43.60	SAND	0.80*	62.60	37.00	44.00**
43.60	SAND	0.80*	82.60	50.00	50.00**

50.00	SAND	0.80*	82.60	50.00	50.00**

* VALUE ASSUMED BY THE PROGRAM
** VALUE ESTIMATED BY THE PROGRAM BASED ON FRICTION ANGLE

MAXIMUM UNIT FRICTION KSF	MAXIMUM UNIT BEARING KSF	UNDISTURB SHEAR STRENGTH KSF	REMOLDED SHEAR STRENGTH KSF	BLOW COUNT	UNIT SKIN FRICTION KSF	UNIT END BEARING KSF
0.10E-05	0.10E-05	0.00	0.00	0.00	0.00	0.00
0.10E-05	0.10E-05	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	1.60	1.60	0.00	0.00	0.00
0.10E+08*	0.10E+08*	1.60	1.60	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00

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

DEPTH FT.	LRFD FACTOR ON UNIT FRICTION	LRFD FACTOR ON UNIT BEARING
0.00	1.000	1.000
11.00	1.000	1.000
11.00	1.000	1.000
25.80	1.000	1.000
25.80	1.000	1.000
29.10	1.000	1.000
29.10	1.000	1.000
38.70	1.000	1.000
38.70	1.000	1.000
43.60	1.000	1.000
43.60	1.000	1.000
50.00	1.000	1.000

1

****
* COMPUTATION RESULT *
****

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

PILE PENETRATION FT.	SKIN FRICTION KIP	END BEARING KIP	ULTIMATE CAPACITY KIP
0.00	0.0	0.0	0.0
1.00	0.0	0.0	0.0
2.00	0.0	0.0	0.0
3.00	0.0	0.0	0.0
4.00	0.0	0.0	0.0
5.00	0.0	0.0	0.0
6.00	0.0	0.0	0.0
7.00	0.0	0.0	0.0
8.00	0.0	0.0	0.0
9.00	0.0	0.4	0.4
10.00	0.0	1.7	1.7
11.00	1.3	3.0	4.3
12.00	4.2	4.3	8.5
13.00	7.2	5.6	12.9
14.00	10.5	6.0	16.5
15.00	14.1	6.0	20.1
16.00	17.9	6.0	23.9
17.00	21.9	6.0	27.9
18.00	26.2	6.0	32.2
19.00	30.8	6.0	36.7
20.00	35.5	6.0	41.5
21.00	40.6	6.0	46.6
22.00	45.8	6.0	51.8
23.00	51.4	6.0	57.3
24.00	57.1	6.0	63.1
25.00	63.1	6.0	69.1
26.00	69.4	6.0	75.4
27.00	75.8	6.0	81.8
28.00	82.4	5.8	88.2
29.00	89.1	5.0	94.1
30.00	95.9	4.3	100.2
31.00	103.1	3.6	106.6
32.00	110.7	2.8	113.5
33.00	118.3	2.6	120.9
34.00	125.9	2.6	128.5
35.00	133.5	2.6	136.1
36.00	141.1	2.6	143.7
37.00	148.7	4.7	153.4
38.00	156.3	12.3	168.7
39.00	163.9	20.0	183.9
40.00	174.2	27.6	201.8
41.00	187.3	35.3	222.6
42.00	200.5	43.1	243.6
43.00	214.0	64.3	278.3

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

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

T-Z CURVE NO.	NO. OF POINTS	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
1	10	0.0000E+00		
			0.0000E+00	0.0000E+00
			0.0000E+00	0.2906E-01
			0.0000E+00	0.5629E-01
			0.0000E+00	0.1035E+00
			0.0000E+00	0.1453E+00
			0.0000E+00	0.1816E+00
			0.0000E+00	0.3632E+00
			0.0000E+00	0.5448E+00
			0.0000E+00	0.9080E+00
			0.0000E+00	0.3632E+01
2	10	0.5525E+01		
			0.0000E+00	0.0000E+00
			0.0000E+00	0.2906E-01
			0.0000E+00	0.5629E-01
			0.0000E+00	0.1035E+00
			0.0000E+00	0.1453E+00
			0.0000E+00	0.1816E+00
			0.0000E+00	0.3632E+00
			0.0000E+00	0.5448E+00
			0.0000E+00	0.9080E+00
			0.0000E+00	0.3632E+01
3	10	0.1096E+02		
			0.0000E+00	0.0000E+00
			0.1132E+01	0.2906E-01
			0.1887E+01	0.5629E-01
			0.2831E+01	0.1035E+00
			0.3397E+01	0.1453E+00
			0.3775E+01	0.1816E+00
			0.3775E+01	0.3632E+00
			0.3775E+01	0.5448E+00
			0.3775E+01	0.9080E+00
			0.3775E+01	0.3632E+01
4	10	0.1100E+02		
			0.0000E+00	0.0000E+00
			0.1182E+01	0.2906E-01
			0.1970E+01	0.5629E-01

			0.2954E+01	0.1035E+00
			0.3545E+01	0.1453E+00
			0.3939E+01	0.1816E+00
			0.3939E+01	0.3632E+00
			0.3939E+01	0.5448E+00
			0.3939E+01	0.9080E+00
			0.3939E+01	0.3632E+01
5	10	0.1843E+02		
			0.0000E+00	0.0000E+00
			0.1979E+01	0.2906E-01
			0.3299E+01	0.5629E-01
			0.4949E+01	0.1035E+00
			0.5938E+01	0.1453E+00
			0.6598E+01	0.1816E+00
			0.6598E+01	0.3632E+00
			0.6598E+01	0.5448E+00
			0.6598E+01	0.9080E+00
			0.6598E+01	0.3632E+01
6	10	0.2576E+02		
			0.0000E+00	0.0000E+00
			0.2767E+01	0.2906E-01
			0.4612E+01	0.5629E-01
			0.6918E+01	0.1035E+00
			0.8302E+01	0.1453E+00
			0.9224E+01	0.1816E+00
			0.9224E+01	0.3632E+00
			0.9224E+01	0.5448E+00
			0.9224E+01	0.9080E+00
			0.9224E+01	0.3632E+01
7	10	0.2580E+02		
			0.0000E+00	0.0000E+00
			0.2772E+01	0.2906E-01
			0.4619E+01	0.5629E-01
			0.6929E+01	0.1035E+00
			0.8315E+01	0.1453E+00
			0.9239E+01	0.1816E+00
			0.9239E+01	0.3632E+00
			0.9239E+01	0.5448E+00
			0.9239E+01	0.9080E+00
			0.9239E+01	0.3632E+01
8	10	0.2748E+02		
			0.0000E+00	0.0000E+00
			0.2873E+01	0.2906E-01
			0.4788E+01	0.5629E-01
			0.7181E+01	0.1035E+00
			0.8618E+01	0.1453E+00
			0.9575E+01	0.1816E+00
			0.9575E+01	0.3632E+00
			0.9575E+01	0.5448E+00
			0.9575E+01	0.9080E+00
			0.9575E+01	0.3632E+01
9	10	0.2906E+02		
			0.0000E+00	0.0000E+00
			0.2958E+01	0.2906E-01

			0.4930E+01	0.5629E-01
			0.7394E+01	0.1035E+00
			0.8873E+01	0.1453E+00
			0.9859E+01	0.1816E+00
			0.9859E+01	0.3632E+00
			0.9859E+01	0.5448E+00
			0.9859E+01	0.9080E+00
			0.9859E+01	0.3632E+01
10	10	0.2910E+02		
			0.0000E+00	0.0000E+00
			0.2960E+01	0.2906E-01
			0.4933E+01	0.5629E-01
			0.7400E+01	0.1035E+00
			0.8880E+01	0.1453E+00
			0.9867E+01	0.1816E+00
			0.8880E+01	0.3632E+00
			0.8880E+01	0.5448E+00
			0.8880E+01	0.9080E+00
			0.8880E+01	0.3632E+01
11	10	0.3393E+02		
			0.0000E+00	0.0000E+00
			0.3333E+01	0.2906E-01
			0.5556E+01	0.5629E-01
			0.8333E+01	0.1035E+00
			0.1000E+02	0.1453E+00
			0.1111E+02	0.1816E+00
			0.1000E+02	0.3632E+00
			0.1000E+02	0.5448E+00
			0.1000E+02	0.9080E+00
			0.1000E+02	0.3632E+01
12	10	0.3866E+02		
			0.0000E+00	0.0000E+00
			0.3330E+01	0.2906E-01
			0.5550E+01	0.5629E-01
			0.8326E+01	0.1035E+00
			0.9991E+01	0.1453E+00
			0.1110E+02	0.1816E+00
			0.9991E+01	0.3632E+00
			0.9991E+01	0.5448E+00
			0.9991E+01	0.9080E+00
			0.9991E+01	0.3632E+01
13	10	0.3870E+02		
			0.0000E+00	0.0000E+00
			0.3331E+01	0.2906E-01
			0.5551E+01	0.5629E-01
			0.8327E+01	0.1035E+00
			0.9992E+01	0.1453E+00
			0.1110E+02	0.1816E+00
			0.1110E+02	0.3632E+00
			0.1110E+02	0.5448E+00
			0.1110E+02	0.9080E+00
			0.1110E+02	0.3632E+01
14	10	0.4118E+02		
			0.0000E+00	0.0000E+00

			0.5783E+01	0.2906E-01
			0.9638E+01	0.5629E-01
			0.1446E+02	0.1035E+00
			0.1735E+02	0.1453E+00
			0.1928E+02	0.1816E+00
			0.1928E+02	0.3632E+00
			0.1928E+02	0.5448E+00
			0.1928E+02	0.9080E+00
			0.1928E+02	0.3632E+01
15	10	0.4356E+02		
			0.0000E+00	0.0000E+00
			0.5940E+01	0.2906E-01
			0.9900E+01	0.5629E-01
			0.1485E+02	0.1035E+00
			0.1782E+02	0.1453E+00
			0.1980E+02	0.1816E+00
			0.1980E+02	0.3632E+00
			0.1980E+02	0.5448E+00
			0.1980E+02	0.9080E+00
			0.1980E+02	0.3632E+01
16	10	0.4360E+02		
			0.0000E+00	0.0000E+00
			0.5940E+01	0.2906E-01
			0.9900E+01	0.5629E-01
			0.1485E+02	0.1035E+00
			0.1782E+02	0.1453E+00
			0.1980E+02	0.1816E+00
			0.1980E+02	0.3632E+00
			0.1980E+02	0.5448E+00
			0.1980E+02	0.9080E+00
			0.1980E+02	0.3632E+01
17	10	0.4683E+02		
			0.0000E+00	0.0000E+00
			0.5940E+01	0.2906E-01
			0.9900E+01	0.5629E-01
			0.1485E+02	0.1035E+00
			0.1782E+02	0.1453E+00
			0.1980E+02	0.1816E+00
			0.1980E+02	0.3632E+00
			0.1980E+02	0.5448E+00
			0.1980E+02	0.9080E+00
			0.1980E+02	0.3632E+01
18	10	0.4996E+02		
			0.0000E+00	0.0000E+00
			0.5940E+01	0.2906E-01
			0.9900E+01	0.5629E-01
			0.1485E+02	0.1035E+00
			0.1782E+02	0.1453E+00
			0.1980E+02	0.1816E+00
			0.1980E+02	0.3632E+00
			0.1980E+02	0.5448E+00
			0.1980E+02	0.9080E+00
			0.1980E+02	0.3632E+01

TIP LOAD KIP	TIP MOVEMENT IN.
0.0000E+00	0.0000E+00
0.4018E+01	0.9080E-02
0.8037E+01	0.1816E-01
0.1607E+02	0.3632E-01
0.3215E+02	0.2361E+00
0.4822E+02	0.7627E+00
0.5787E+02	0.1326E+01
0.6430E+02	0.1816E+01
0.6430E+02	0.2724E+01
0.6430E+02	0.3632E+01

LOAD VERSUS SETTLEMENT CURVE
****

TOP LOAD KIP	TOP MOVEMENT IN.	TIP LOAD KIP	TIP MOVEMENT IN.
0.3364E+00	0.2648E-03	0.4426E-01	0.1000E-03
0.3364E+01	0.2648E-02	0.4426E+00	0.1000E-02
0.1687E+02	0.1326E-01	0.2213E+01	0.5000E-02
0.3391E+02	0.2658E-01	0.4426E+01	0.1000E-01
0.6706E+02	0.5301E-01	0.8851E+01	0.2000E-01
0.1338E+03	0.1169E+00	0.1717E+02	0.5000E-01
0.1749E+03	0.1674E+00	0.1959E+02	0.8000E-01
0.1980E+03	0.1993E+00	0.2120E+02	0.1000E+00
0.2489E+03	0.3278E+00	0.2924E+02	0.2000E+00
0.2536E+03	0.6320E+00	0.4020E+02	0.5000E+00
0.2623E+03	0.9379E+00	0.4886E+02	0.8000E+00
0.2657E+03	0.1140E+01	0.5229E+02	0.1000E+01
0.2777E+03	0.2149E+01	0.6430E+02	0.2000E+01

GOLDER
MEMBER OF WSP

CALCULATIONS

Date:	6/23/2021	**Made by:**	DAF
Project No.:	21450908	**Checked by:**	KAR
Subject:	Pile Driveability at Abutment 1	**Reviewed by:**	JEL
Project Title:	MaineDOT Desert Road Bridge 5720 Freeport (Exit 20) Phase 2		

OBJECTIVE

Perform a driveability analysis using GRLWEAP to determine if the proposed piles can be driven to the nominal pile driving resistance at Abutment 1 while maintaining blow counts and pile stresses within the specified limits.

REFERENCES

1. GRLWEAP Software Package Version 2010-8, Built November 28, 2018.
2. AASHTO LRFD Bridge Design Specifications, 9th Edition, 2020
3. MaineDOT 2020 Standard Specifications Section 501.042
4. Golder interpreted subsurface section A-A' (Figure 3, Preliminary Geotechnical Design Report, dated December 2020).
5. HNTB calculation titled "Freeport Bridges_Loads_Bottom of Footing_flat.pdf", dated May 26, 2021.

ASSUMPTIONS

1. The pile will be bearing on bedrock with 90% tip resistance, 10% shaft resistance.
2. The number of hammer blows at the required resistance indicated by the wave equation analysis will be between 3 and 15 blows per inch (Ref. 3)
3. The factored axial load is 465 kips including downdrag for Abutment 1 with HP 14x89 piles.
4. The hammer cushion will consist of 50% aluminum and 50% conbest, 2" thick.
5. The HP 14x89 piles will be driven a total length of approximately 33 feet.
6. A resistance factor of 0.65 will be used for a driving criteria established by dynamic testing (Ref. 2).
7. A Delmag D 30 single acting diesel pile driving hammer is assumed for the analysis as it is a common pile driving hammer used on MaineDOT projects.

ATTACHMENTS

1. GRLWEAP Table of recommended quake and damping values for impact driven piles.
2. GRLWEAP output for a variable resistance analysis using the DELMAG D30 hammer.

CALCULATION

1. Determine the input parameters:

Nominal pile driving resistance:

$P_u =$ 465 Applied axial load including downdrag

$\phi_{mon} =$ 0.65 Resistance factor associated with pile monitoring method

$$R_{ndr} = \frac{P_u}{\phi_{mon}}$$

Abutment 1: $R_{ndr} =$ 715 kips

GOLDER
MEMBER OF WSP

CALCULATIONS

Date:	6/23/2021	**Made by:**	DAF
Project No.:	21450908	**Checked by:**	KAR
Subject:	Pile Driveability at Abutment 1	**Reviewed by:**	JEL
Project Title:	MaineDOT Desert Road Bridge 5720 Freeport (Exit 20) Phase 2		

Soil layering:

Abutment 1

Layer (Ref. 4)	Depth below base of abutment
Existing Fill (above water table)	0.0 - 14.8 ft
Existing Fill (below water table)	14.8 - 18.1 ft
Glaciomarine Silty Clay	18.1 - 27.7 ft
Sand and Gravel	27.7 - 32.6 ft
Bedrock	>32.6 ft

Shaft quake =	0.1	in	Shaft damping =	0.1	s/ft	(Attachment 1)
Toe quake =	0.04	in	Toe damping =	0.15	s/ft	

2. Analyze an open ended diesel (OED) hammer to determine the blow count and stress at R_{ndr} for the abutment.

Hammer	Energy / Power (lb-ft)	Fuel Setting	Location	R_{ndr} (kips)	Blows/ft	Blows/in	Stress (ksi)
DELMAG D 30	59,730	Fuel Setting 3 (81%)	Abutment 1	715	117.6	9.8	43.4

CONCLUSIONS

The analysis indicates the Delmag D30 hammer operated at fuel setting 3 (81% of the combustion pressure) should be able to drive the piles at Abutment 1 to the nominal structural pile resistance while staying at a blow count between 3 and 15 blows per inch and limiting the driving stresses to below 45 kips per square inch (ksi), in accordance with Section 501.042 of the MaineDOT 2020 Standard Specifications.

CALCULATIONS

Date:	6/23/2021	**Made by:**	DAF
Project No.:	21450908	**Checked by:**	KAR
Subject:	Pile Driveability at Abutment 1	**Reviewed by:**	JEL
Project Title:	MaineDOT Desert Road Bridge 5720 Freeport (Exit 20) Phase 2		

Attachment 1

Recommended Quake Values for Impact Driven Piles*

	Soil Type	Pile Type or Size	Quake (in) Quake (mm)
Shaft Quake	All soil types	All Types	0.10 2.5
Toe Quake	All soil types, soft Rock	Non-displacement piles** i.e. driving unplugged	0.10 2.5
	Very dense or hard soils	Displacement Piles*** of diameter or width D	D/120 D/120
	Soils which are not very dense or hard	Displacement Piles*** of diameter or width D	D/60 D/60
	Hard Rock	All Types	0.04 1.0

*For vibratory driven piles in cohesive soils, quakes should be doubled.
** Non-displacement piles are sheet pile, H-Piles, or open-ended pipe piles which are not plugging during driving. Normally it can be assumed that pipe piles with diameters of 30 inches (900 mm) or more will not plug during driving while H-Piles and pipe piles of diameter 20 inches (500 mm) or less will plug during driving into a bearing layer. Between 20 and 30 inches (500 and 750 mm), pipe piles may or may not plug.
*** Displacement piles are closed-ended pipe piles, pipe piles, or H-Piles that are plugged during driving and solid concrete piles. Normally, we would analyze H-Piles and pipe piles with diameters 20 inches (500 mm) or less as displacement piles.

Recommended quake values, both shaft and toe, for vibratory driven piles are somewhat speculative recommendations as not much experience exists.

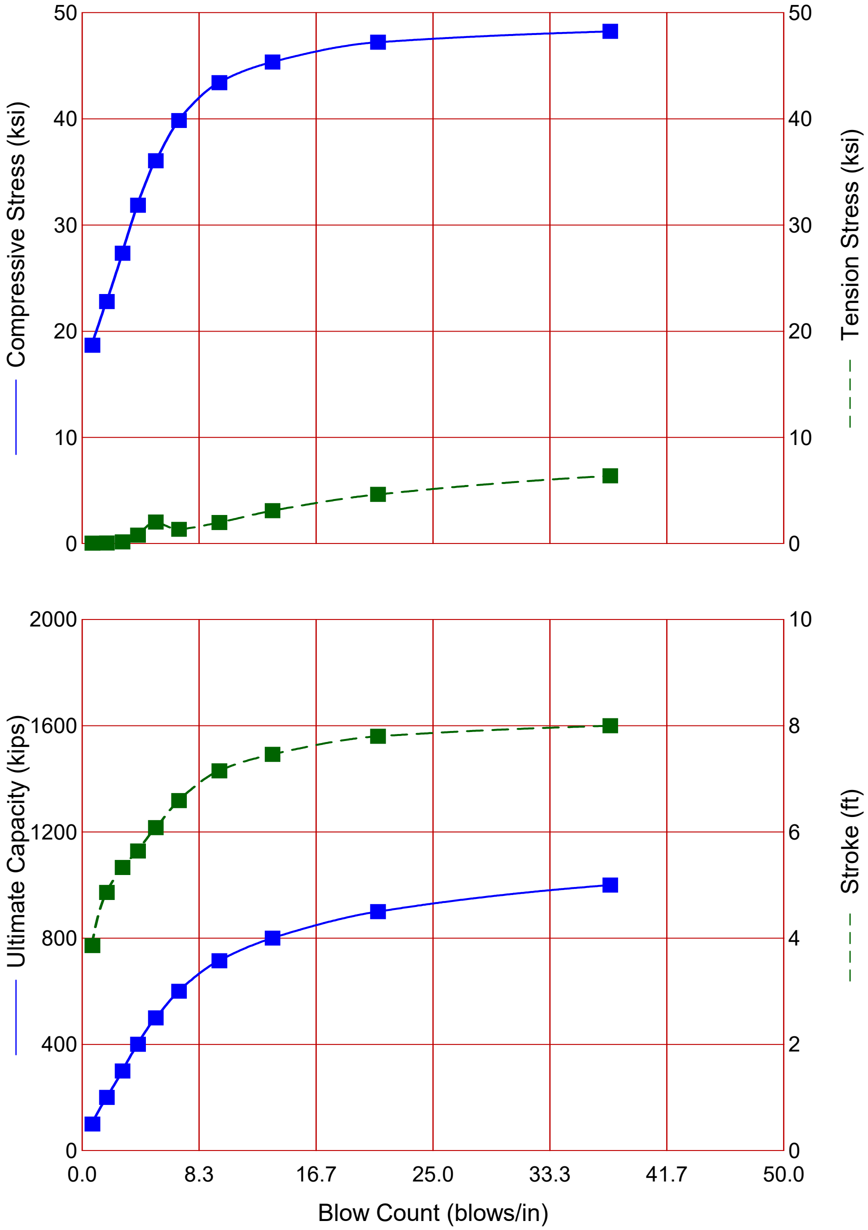
Recommended Damping Values for Impact Driven Piles*

	Soil Type	Damping Factor s/ft	Damping Factor s/m
Shaft Damping	Non-cohesive soils**	0.05	0.16
	Cohesive soils**	0.20	0.65
Toe damping	In all soil types	0.15	0.50

* For vibratory driven piles, use double values (Smith-viscous).
** For mixed soils, intermediate values may be appropriate; for example, a sandy silt or clayey sand may be modeled with 0.10 s/ft (0.33 s/m), a cohesive silt or a sandy clay with 0.15 s/ft (0.50 s/m).

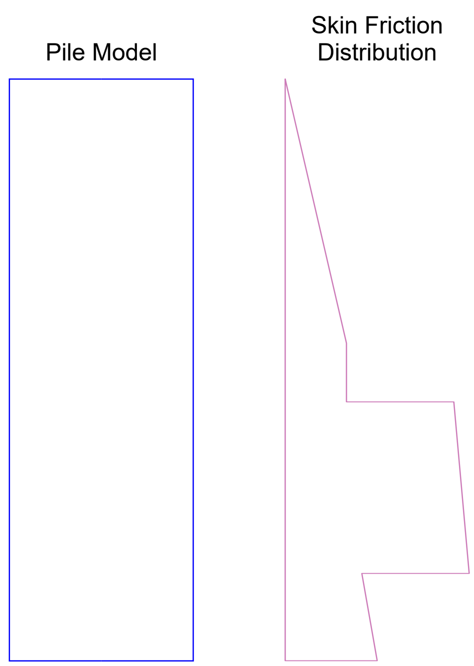
Golder Associates Inc.
21450908 Freeport Exit 20 Abutment 1

25-Jun-2021
GRLWEAP Version 2010



DELMAG D 30

Ram Weight	6.60	kips
Efficiency	0.800	
Pressure	1200 (81%)	psi
Helmet Weight	2.70	kips
Hammer Cushion	109975	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.100	s/ft
Toe Damping	0.150	s/ft
Pile Length	32.60	ft
Pile Penetration	32.60	ft
Pile Top Area	26.10	in2



Res. Shaft = 10 %
(Proportional)

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
100.0	18.68	0.05	0.7	3.86	21.38
200.0	22.79	0.07	1.8	4.86	19.22
300.0	27.34	0.17	2.9	5.33	18.55
400.0	31.85	0.79	4.0	5.64	18.90
500.0	36.04	2.03	5.3	6.08	20.05
600.0	39.83	1.35	6.9	6.59	21.57
715.0	43.40	1.99	9.8	7.15	23.18
800.0	45.34	3.10	13.6	7.46	24.05
900.0	47.20	4.64	21.1	7.80	25.41
1000.0	48.23	6.38	37.6	8.00	26.19

APPENDIX F

Abutment 2 Pile Design

GOLDER
MEMBER OF WSP

CALCULATIONS

Date:	7/14/2021	**Made by:**	KAR
Project No.:	21450908	**Checked by:**	AH
Subject:	Pile Design at Abutment 2	**Reviewed by:**	JEL
Project Title:	MaineDOT Desert Road Bridge 5720 Freeport (Exit 20) Phase 2		

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, girder rotation, and final design loads.

METHOD

Use the procedure outlined in AASHTO LRFD (Ref. 1) and the design method provided in the MaineDOT Bridge Design Guide (Ref. 2).

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 9th Ed. 2020.
2. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.
3. Email communication between Golder and HNTB, subject "RE: Freeport I-295 Exit 20 and Exit 22 - Request for Sections and Design Info", dated June 22, 2020.
4. Isenhower, W.M. et al. LPile v2019 Technical Manual: A Program for the Analysis of Deep Foundations Under Lateral Loading. Ensoft, Inc. Dated March 2020.
5. Golder updated Phase II interpreted subsurface profiles A-A' and B-B', dated June 2021.
6. Golder geotechnical test boring logs (Appendix A, Preliminary Geotechnical Design Report, dated December 2020).
7. Bridge Software Institute. FB-MultiPier Soil Parameter Table (US Customary Units). Accessed June 2021. 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. HNTB calculation titled "Freeport Bridges_Loads_Bottom of Footing_flat.pdf", dated May 26, 2021.
11. Oregon Department of Transportation, Geo-Environmental Section. Geotechnical Design Manual: Chapter 8 - Foundations, Version 2.1. Dated May 6, 2019.
12. HNTB for State of Maine Department of Transportation. Merrill Road Bridge over Interstate 295 and Signalized Intersections, Exit 20 Interchange: 60% Plans, dated May 21, 2021.
13. Golder calculation titled "Pile Design at Proposed Abutment 2 - Downdrag Analysis", dated June 28, 2021.
14. HNTB for State of Maine Department of Transportation. Merrill Road Bridge over Interstate 295, Freeport, Cumberland: Abutment 1 Reinforcement Sections, Sheet 94, dated May 7, 2021.

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. The soil profile is analyzed at the location along the abutment where maximum settlement (and thus maximum downdrag load) is anticipated to occur.

GOLDER
MEMBER OF WSP

CALCULATIONS

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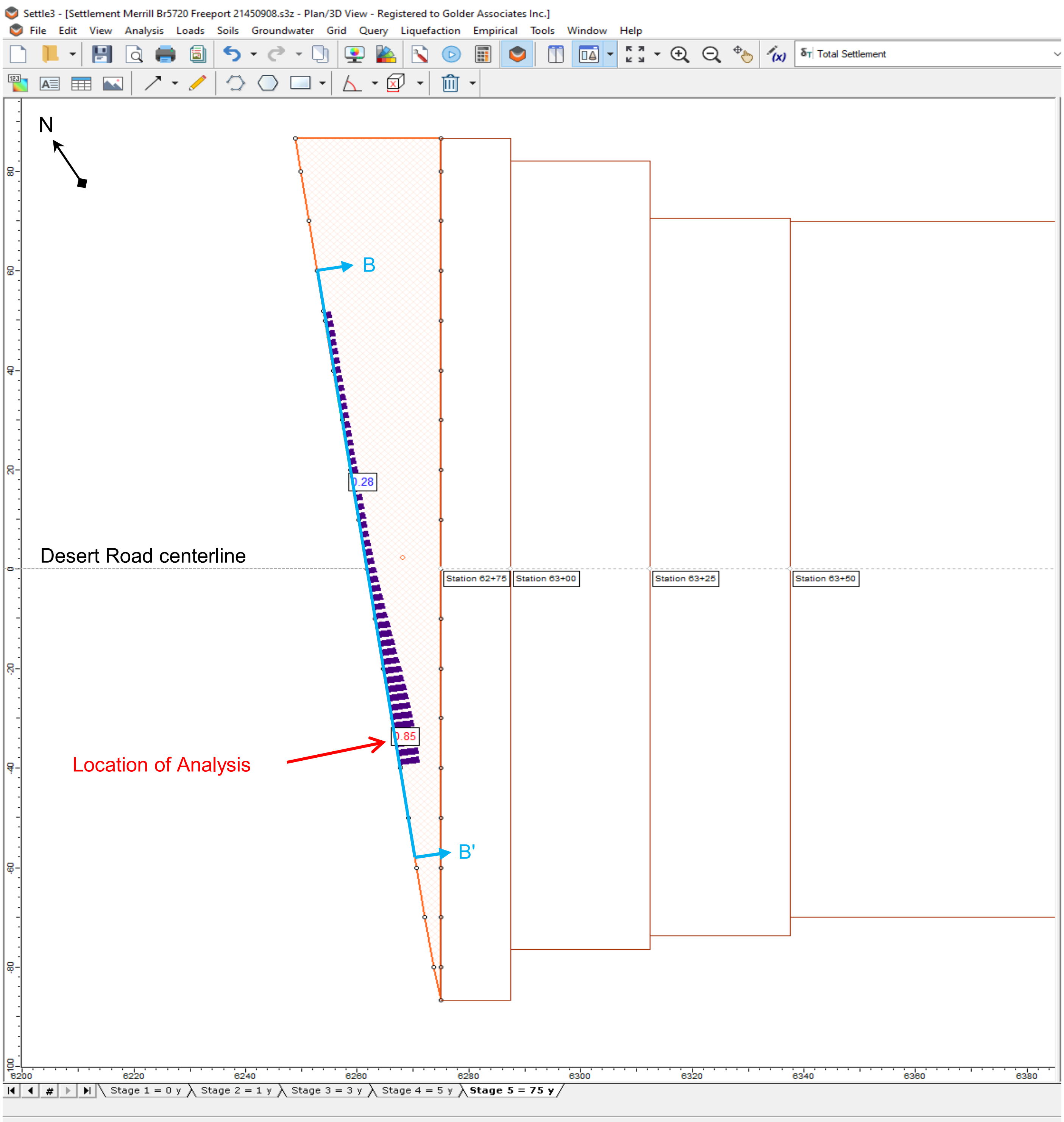

https://golderassociates.sharepoint.com/sites/139980/Project Files/5 Technical Work/06 Analysis/Phase II Pile Design/21450908 Freeport Exit 20 Abutment 2 Pile Design girder rotation.xlsx

Date:	7/14/2021	**Made by:**	KAR
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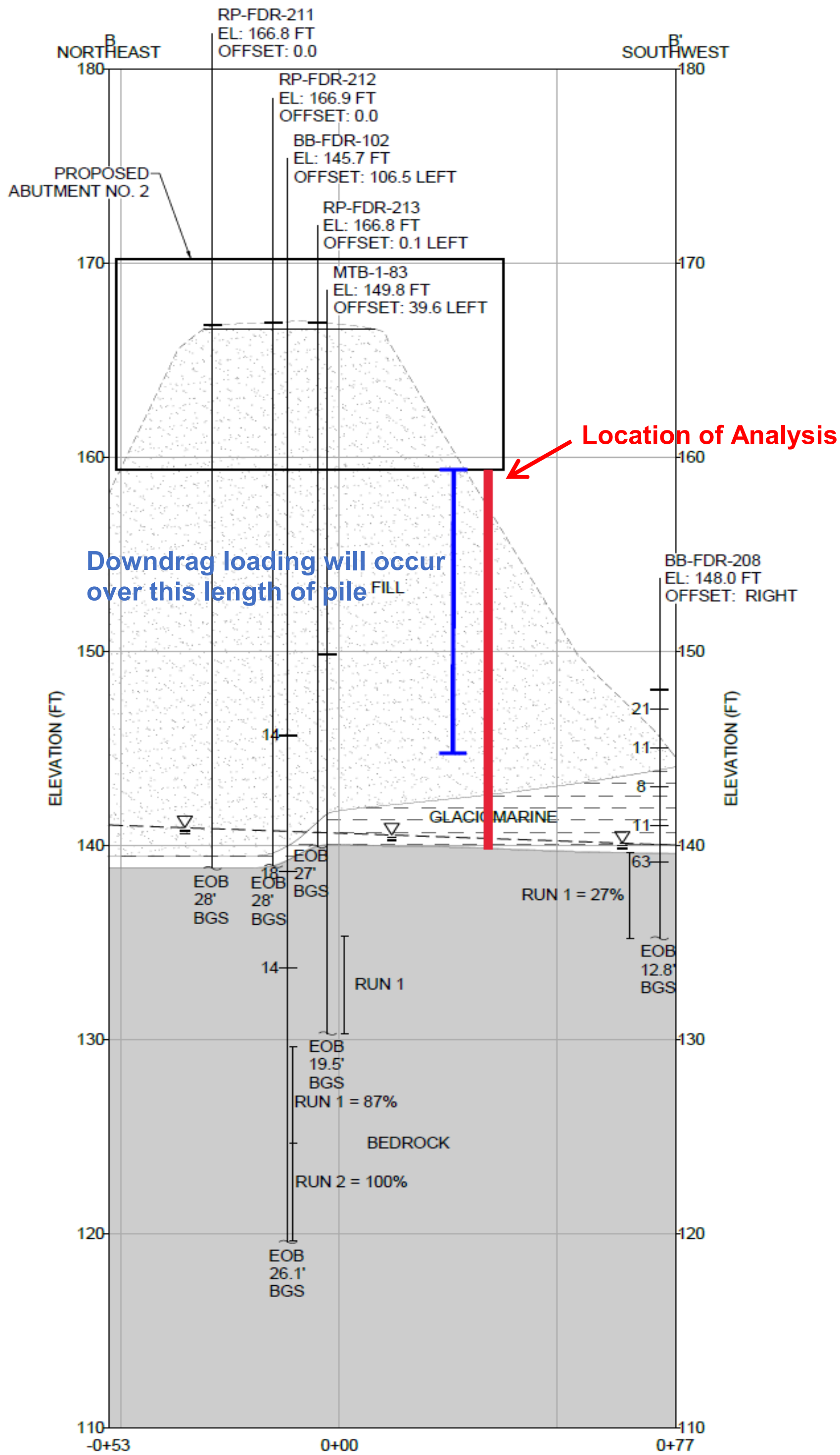
Project Title:	MaineDOT Desert Road Bridge 5720 Freeport (Exit 20) Phase 2		



GOLDER
MEMBER OF WSP

CALCULATIONS

Date:	7/14/2021	**Made by:**	KAR
Project No.:	21450908	**Checked by:**	AH
Subject:	Pile Design at Abutment 2	**Reviewed by:**	JEL
Project Title:	MaineDOT Desert Road Bridge 5720 Freeport (Exit 20) Phase 2		

ATTACHMENTS

1. LPile analysis output for Strength I
2. LPile analysis output for Strength I with Plastic Hinge
3. Downdrag Analysis
4. Driveability Analysis

CALCULATION

1. Select the preliminary pile size.

Determine the factored applied superstructure vertical dead and live load (P_u) distributed to each pile.

Strength I factored vertical load per abutment =	3216	kips	(Ref. 10, page 2)
divided by	9	piles at Abutment 2 (Ref. 12, Sheet 87) =	
Strength I factored vertical load per pile =	357	kips	
Strength I factored downdrag load per pile =	76	kips	(Ref. 13)

Pile weight = 0.089 kip/ft x 19.6 ft = 2 kips

P_u = 435 kips (Total factored axial load including downdrag and pile weight)

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 under combined axial and flexural loading (Ref. 2, page 5-42)
ϕ_f =	1.00	for flexural resistance in the upper zone of the pile under combined axial and flexural loading (Ref. 2, page 5-42)

Determine the maximum required nominal axial pile resistance (Ref. 1, Article 6.9.2.1).

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

$R_{n.upper}$ = 621 kips

$$R_{n.lower} = \frac{P_u}{\phi_{cl}}$$

$R_{n.lower}$ = 870 kips

$$R_n = \max\left(R_{n.upper}, R_{n.lower}\right)$$

R_n = 870 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 14x89 based on June 16, 2020 meeting with MaineDOT and HNTB.
Check that preferred selection satisfies pile area requirement:

HP 14x89 A_s = 26.1 in^2 (Ref. 4, Table 5.6.3)

A_s > $A_{s.req}$ OK

2. 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:	19.6	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 due to abutment thermal expansion or contraction:	0.795	in	(Ref. 10, page 2)
Lateral deflection due to girder rotation:	0.18	in	(Ref. 14)
Total lateral deflection at pile head:	0.975	in	
Axial load:	435,000	lbs	(from Step 1)

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	0.0 - 15.3 ft	Sand (Reese)	125	-	32	124.8	-	-
Glaciomarine Silty Clay (above water table)	15.3 - 19.0 ft	Stiff Clay with Free Water (Reese)	125	1600	-	500	0.005	-
Glaciomarine Silty Clay (below water table)	19.0 - 19.6 ft	Stiff Clay w/o Free Water (Reese)	62.6	1600	-	-	0.005	-
Bedrock	> 19.6 ft	Strong Rock (Vuggy Limestone)	101.6	-	-	-	-	12983

1) Ref. 5
2) Ref. 6
3) Ref. 7 - interpolation based on average N_{60} value for each layer.

The full LPile output is provided in Attachment 1.

Obtain the maximum moment at the top of the pile.

$M_{u.Top}$ = 2962 in-kips (LPile)

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

$l_{b.top}$ = 4.6 ft (LPile)
$l_{b.top}$ = 55.2 in

$l_{b.2nd}$ = 12.2 ft (LPile)
$l_{b.2nd}$ = 147.0 in

3. 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} = 18.74 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} = 41.59 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)

$P_{e.top}$ =	21273	kips
$P_{e.2nd}$ =	4319	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
$P_o/P_{e.2nd}$ = 0.30 ≤ 2.25

thus use Ref. 1 Eqn 6.9.4.1.1-1:

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

$P_{n.top}$ = 1272 kips
$P_{n.2nd}$ = 1150 kips

$$P_{n.bottom} = (0.658^{(0)}) \times F_y A_s$$ (0 for a fully braced pile - Ref. 8, Appendix B, Eqn 6-9)

$P_{n.bottom}$ = 1305 kips

Calculate the factored structural pile resistance, P_r, for both segments of the upper zone of the pile as well as the lower zone of the pile.

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

$P_{r.top}$ = 890 kips

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

$P_{r.2nd}$ = 805 kips

$$P_{r.bottom} = \phi_{cl} P_{n.bottom}$$

$P_{r.bottom}$ = 653 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.

$$\frac{P_u}{P_{r.top}} = 0.49$$ OK

$$\frac{P_u}{P_{r.2nd}} = 0.54$$ OK

Since the lower zone of the pile will have virtually no moment, the entire section can carry the required vertical loads. Make sure the applied load will not exceed the resistance of the lower zone.

$$\text{Check}\left(\frac{P_u}{P_{r.bottom}} < 1\right)$$

$$\frac{P_u}{P_{r.bottom}} =$$ 0.67 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:

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

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

$S_y =$ 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 $\lambda_{pf} < \lambda_f \leq \lambda_{rf}$,

M_n = 3080 in-kips

Factored flexural resistance:

ϕ_f = 1.00 (Ref. 2, page 5-42)

$$M_r = \phi_f M_n$$

M_r = 3080 in-kips

Calculate the moment that will cause a plastic hinge at the top of the pile, M_p'(Ref. 2, Article 6.9.2.2).

$$M'_p = \frac{9}{8}\left(1 - \frac{P_u}{P_{r.top}}\right) M_r$$ (Ref. 8, Appendix B, Eqn 6-24)

M_p' = 1772 in-kips = 1772170.5 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'.

$M_{u.Top}$ =	2962	in-kips	(From Step 3)
$M_{u.Top}$	>	M_p'	Plastic Hinge Forms

4. 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.

$l_{b.top}$ =	3.4	ft	(LPile)
$l_{b.top}$ =	41.0	in	
$l_{b.2nd}$ =	12.6	ft	(LPile)
$l_{b.2nd}$ =	151.2	in	
$M_{u.2nd}$ =	1207	in-kips	(LPile)

Since a plastic hinge developed at the pile head, the value of K for the top segment becomes 2.1 (Ref. 2, page 5-43).

K_{top} =	2.1	(Ref. 1, Table C4.6.2.5-1)
K_{2nd} =	1.0	(Ref. 1, Table C4.6.2.5-1)

λ_{top} =	24.34	<	120	OK
λ_{2nd} =	42.78	<	120	OK

$P_{e.top}$ = 12609 kips
$P_{e.2nd}$ = 4082 kips

$P_o/P_{e.top}$ = 0.10 ≤ 2.25 (to select P_n equation)
$P_o/P_{e.2nd}$ = 0.32 ≤ 2.25 (to select P_n equation)

$P_{n.top}$ = 1250 kips
$P_{n.2nd}$ = 1142 kips

$P_{r.top}$ = 875 kips
$P_{r.2nd}$ = 799 kips

$\frac{P_u}{P_{r.top}} =$ 0.50 > 0.20 OK

$\frac{P_u}{P_{r.2nd}} =$ 0.54 > 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: 0.89 < 1 OK

5. 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 = 42.6 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

$$V_r = \Phi_v V_n$$

$\phi_v =$ 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

6. 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)

$R_{ndr} =$ 669 kips

The nominal pile driving resistance (R_{ndr}) should not exceed the nominal structural pile resistance (P_n) or the maximum driving force (P_0) calculated above.

$P_{n.top}$ = 1250 kips (From Step 4)
$P_{n.2nd}$ = 1142 kips (From Step 4)

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

7. 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 = 3.33 kips (LPile)
ϕ_v = 1.00 (Ref. 1, Article 6.5.4.2)
$V_{factored}$ at pile tip = 3.33 kips

	Unfactored Vertical Dead Load per Abut. (Ref. 10, pg. 2)	Load Factor (Ref. 1, Table 3.4.1-1)
	kips	-
DC_{super}	633	1.25
DC_{sub}	636	1.25
DW	86	1.50

Strength I factored dead load per abutment = 1715 kips
divided by 9 piles at Abutment 2 (Ref. 12, Sheet 87) =
P = 191 kips

V / P = 0.017

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
0.017	<	0.2

The chosen pile section can be considered pinned.

Date:	7/14/2021	**Made by:**	KAR
Project No.:	21450908	**Checked by:**	AH
Subject:	Pile Design at Abutment 2	**Reviewed by:**	JEL
Project Title:	MaineDOT Desert Road Bridge 5720 Freeport (Exit 20) Phase 2		

CONCLUSIONS

The results of the analysis indicate that a maximum moment of 2962 in-kips (247 ft-kips) occurs at the top of the pile under the Strength I load case with a maximum bridge expansion or contraction of 0.8 inches and a maximum lateral deflection due to girder rotation of 0.2 inches. The results indicate that the depth to bedrock is sufficient for driven piles to limit translation of the pile tip to a negligible amount, i.e., less than approximately 1/8 inch, 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, girder rotation, and final design loads.

Attachment 1

==

LPile for Windows, Version 2019-11.005

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

==

This copy of LPile is being used by:

KAR
Golder Associates, Inc.

Serial Number of Security Device: 239146533

This copy of LPile is licensed for exclusive use by:

GOLDER, Manchester, NH

Files Used for Analysis

Path to file locations:
\\golderassociates.sharepoint.com@SSL\DavWWWRoot\sites\139980\Project Files\5 Technical Work\06 Analysis\Phase II Pile Design\LPILE\Abutment 2 Strength I\

Name of input data file:
Freeport Exit 20 Abutment 2 girder rotation.lp11d

Name of output report file:
Freeport Exit 20 Abutment 2 girder rotation.lp11o

Name of plot output file:
Freeport Exit 20 Abutment 2 girder rotation.lp11p

Name of runtime message file:
Freeport Exit 20 Abutment 2 girder rotation.lp11r

Date and Time of Analysis

Date: July 14, 2021 Time: 15:59:54

Problem Title

Project Name: MaineDOT I-295 Exit 20 Desert Road Bridge No. 5720
Job Number: 21450908
Client: MaineDOT
Engineer: KAR
Description: Abutment 2 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 = 19.600 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	19.600	14.6950

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is a H weak axis steel pile
Length of section = 19.600000 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 = 15.300000 ft

Effective unit weight at top of layer = 125.000000 pcf
Effective unit weight at bottom of layer = 125.000000 pcf
Friction angle at top of layer = 32.000000 deg.
Friction angle at bottom of layer = 32.000000 deg.
Subgrade k at top of layer = 124.800000 pci
Subgrade k at bottom of layer = 124.800000 pci

Layer 2 is stiff clay with water-induced erosion

Distance from top of pile to top of layer = 15.300000 ft
Distance from top of pile to bottom of layer = 19.000000 ft
Effective unit weight at top of layer = 125.000000 pcf
Effective unit weight at bottom of layer = 125.000000 pcf
Undrained cohesion at top of layer = 1600. psf
Undrained cohesion at bottom of layer = 1600. psf
Epsilon-50 at top of layer = 0.005000
Epsilon-50 at bottom of layer = 0.005000
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 = 19.000000 ft
Distance from top of pile to bottom of layer = 19.600000 ft
Effective unit weight at top of layer = 62.600000 pcf
Effective unit weight at bottom of layer = 62.600000 pcf
Undrained cohesion at top of layer = 1600. psf
Undrained cohesion at bottom of layer = 1600. psf
Epsilon-50 at top of layer = 0.005000
Epsilon-50 at bottom of layer = 0.005000

Layer 4 is strong rock (vuggy limestone)

Distance from top of pile to top of layer = 19.600000 ft
Distance from top of pile to bottom of layer = 50.000000 ft
Effective unit weight at top of layer = 101.600000 pcf
Effective unit weight at bottom of layer = 101.600000 pcf
Uniaxial compressive strength at top of layer = 12983. psi
Uniaxial compressive strength at bottom of layer = 12983. psi

(Depth of the lowest soil layer extends 30.400 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	--	32.0000	--	--	124.8000
	(Reese, et al.)	15.3000	125.0000	--	32.0000	--	--	124.8000
2	Stiff Clay	15.3000	125.0000	1600.	--	--	0.00500	500.0000
	with Free Water	19.0000	125.0000	1600.	--	--	0.00500	500.0000
3	Stiff Clay	19.0000	62.6000	1600.	--	--	0.00500	--
	w/o Free Water	19.6000	62.6000	1600.	--	--	0.00500	--
4	Strong Rock	19.6000	101.6000	--	--	12983.	--	--
	(Vuggy Limestone)	50.0000	101.6000	--	--	12983.	--	--

Static Loading Type

Static loading criteria were used when computing p-y curves for all analyses.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load No.	Load Type	Condition 1	Condition 2	Axial Thrust Force, lbs	Compute Top y vs. Pile Length	Run Analysis
1	5	y = -0.975000 in	S = 0.0000 in/in	435000.	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 = 19.600000 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	435.000

Definition of Run Messages:

Y = part of pipe section has yielded.

Axial Thrust Force = 435.000 kips

Bending Curvature rad/in.	Bending Moment in-kip	Bending Stiffness kip-in2	Depth to N Axis in	Max Total Stress ksi	Run Msg
0.00000425	40.0990456	9438779.	144.0737922	17.7410647	
0.00000850	80.1980912	9438779.	75.7106461	18.6372360	
0.00001274	120.2971368	9438779.	52.9229307	19.5334074	
0.00001699	160.3961825	9438779.	41.5290730	20.4295784	
0.00002124	200.4952281	9438779.	34.6927584	21.3257498	
0.00002549	240.5942737	9438779.	30.1352154	22.2219212	
0.00002974	280.6933193	9438779.	26.8798275	23.1180924	
0.00003399	320.7923649	9438779.	24.4382865	24.0142635	
0.00003823	360.8914105	9438779.	22.5393102	24.9104349	
0.00004248	400.9904562	9438779.	21.0201292	25.8066061	
0.00004673	441.0895018	9438779.	19.7771629	26.7027775	
0.00005098	481.1885474	9438779.	18.7413577	27.5989488	

0.00005523	521.2875930	9438779.	17.8649071	28.4951200	
0.00005948	561.3866386	9438779.	17.1136637	29.3912914	
0.00006372	601.4856842	9438779.	16.4625861	30.2874627	
0.00006797	641.5847299	9438779.	15.8928933	31.1836339	
0.00007222	681.6837755	9438779.	15.3902231	32.0798052	
0.00007647	721.7828211	9438779.	14.9434051	32.9759765	
0.00008072	761.8818667	9438779.	14.5436206	33.8721478	
0.00008497	801.9809123	9438779.	14.1838146	34.7683191	
0.00008921	842.0799579	9438779.	13.8582758	35.6644903	
0.00009346	882.1790036	9438779.	13.5623315	36.5606617	
0.00009771	922.2780492	9438779.	13.2921214	37.4568330	
0.0001020	962.3770948	9438779.	13.0444288	38.3530042	
0.0001062	1002.	9438779.	12.8165517	39.2491755	
0.0001105	1043.	9438779.	12.6062035	40.1453468	
0.0001147	1083.	9438779.	12.4114367	41.0415181	
0.0001190	1123.	9438779.	12.2305819	41.9376894	
0.0001232	1163.	9438779.	12.0621997	42.8338607	
0.0001274	1203.	9438779.	11.9050431	43.7300320	
0.0001317	1243.	9438779.	11.7580256	44.6262033	
0.0001359	1283.	9438779.	11.6201966	45.5223745	
0.0001402	1323.	9438779.	11.4907210	46.4185458	
0.0001444	1363.	9438779.	11.3688615	47.3147171	
0.0001487	1403.	9438779.	11.2539655	48.2108884	
0.0001529	1444.	9438779.	11.1454526	49.1070597	
0.0001572	1484.	9438751.	11.0428102	50.0000000	Y
0.0001614	1522.	9429640.	10.9472446	50.0000000	Y
0.0001657	1560.	9412595.	10.8581657	50.0000000	Y
0.0001742	1630.	9358708.	10.6973886	50.0000000	Y
0.0001827	1696.	9283509.	10.5569141	50.0000000	Y
0.0001912	1758.	9193715.	10.4334619	50.0000000	Y
0.0001997	1816.	9094026.	10.3244254	50.0000000	Y
0.0002082	1871.	8986798.	10.2279443	50.0000000	Y
0.0002167	1923.	8875634.	10.1420359	50.0000000	Y
0.0002252	1973.	8762427.	10.0652455	50.0000000	Y
0.0002337	2021.	8647690.	9.9966068	50.0000000	Y
0.0002422	2066.	8533251.	9.9349004	50.0000000	Y
0.0002507	2110.	8420020.	9.8792269	50.0000000	Y
0.0002591	2153.	8307885.	9.8290540	50.0000000	Y
0.0002676	2194.	8197534.	9.7836823	50.0000000	Y
0.0002761	2234.	8089498.	9.7425033	50.0000000	Y
0.0002846	2273.	7983937.	9.7050645	50.0000000	Y
0.0002931	2310.	7880983.	9.6709633	50.0000000	Y
0.0003016	2347.	7780739.	9.6398398	50.0000000	Y
0.0003101	2383.	7683284.	9.6113710	50.0000000	Y
0.0003186	2418.	7588351.	9.5853874	50.0000000	Y
0.0003271	2452.	7495950.	9.5616498	50.0000000	Y
0.0003356	2486.	7406379.	9.5398233	50.0000000	Y
0.0003441	2518.	7316824.	9.5188626	50.0000000	Y
0.0003526	2548.	7225653.	9.4983357	50.0000000	Y
0.0003611	2576.	7133382.	9.4783012	50.0000000	Y
0.0003696	2602.	7041200.	9.4584658	50.0000000	Y
0.0003781	2627.	6948782.	9.4391951	50.0000000	Y
0.0003866	2651.	6857065.	9.4202497	50.0000000	Y
0.0003951	2673.	6765488.	9.4014792	50.0000000	Y

0.0004036	2694.	6674708.	9.3830767	50.0000000	Y
0.0004121	2714.	6585278.	9.3650609	50.0000000	Y
0.0004206	2733.	6497326.	9.3474897	50.0000000	Y
0.0004291	2750.	6409739.	9.3299713	50.0000000	Y
0.0004376	2767.	6324218.	9.3127442	50.0000000	Y
0.0004461	2783.	6239945.	9.2962005	50.0000000	Y
0.0004546	2799.	6156855.	9.2794441	50.0000000	Y
0.0004631	2813.	6075700.	9.2633234	50.0000000	Y
0.0004716	2827.	5995548.	9.2473354	50.0000000	Y
0.0004801	2841.	5917409.	9.2315354	50.0000000	Y
0.0004886	2853.	5840533.	9.2161292	50.0000000	Y
0.0004971	2866.	5765099.	9.2009795	50.0000000	Y
0.0005056	2877.	5691443.	9.1859736	50.0000000	Y
0.0005395	2919.	5410963.	9.1283921	50.0000000	Y
0.0005735	2955.	5153025.	9.0744546	50.0000000	Y
0.0006075	2986.	4915807.	9.0237309	50.0000000	Y
0.0006415	3013.	4697588.	8.9758365	50.0000000	Y
0.0006755	3037.	4496247.	8.9303610	50.0000000	Y
0.0007095	3058.	4310271.	8.8876853	50.0000000	Y
0.0007435	3077.	4138399.	8.8469591	50.0000000	Y
0.0007774	3093.	3978856.	8.8086420	50.0000000	Y
0.0008114	3108.	3830725.	8.7720830	50.0000000	Y
0.0008454	3122.	3692914.	8.7372846	50.0000000	Y
0.0008794	3134.	3564174.	8.7044216	50.0000000	Y
0.0009134	3145.	3443658.	8.6726649	50.0000000	Y
0.0009474	3156.	3331015.	8.6423887	50.0000000	Y

Summary of Results for Nominal Moment Capacity for Section 1

Load No.	Axial Thrust kips	Nominal Moment Capacity in-kips
1	435.0000000000	3156.

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	350765.
2	15.3000	256.0114	No	No	350765.	6992.
3	19.0000	24.3995	No	No	357757.	10580.
4	19.6000	19.6000	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 = -0.975000 inches
Rotation of pile head = 0.000E+00 radians
Axial load on pile head = 435000.0 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	-0.9750	2961638.	-56398.	0.00	83697.	5.10E+09	0.00	0.00	0.00
0.1960	-0.9734	2828385.	-56332.	0.00133	80689.	5.10E+09	22.5096	54.3897	0.00
0.3920	-0.9687	2693922.	-56249.	0.00246	77654.	6.67E+09	47.9215	116.3507	0.00
0.5880	-0.9618	2558752.	-56104.	0.00336	74603.	7.19E+09	75.6220	184.9245	0.00
0.7840	-0.9529	2423145.	-55892.	0.00415	71541.	7.57E+09	104.7502	258.5398	0.00
0.9800	-0.9423	2287347.	-55610.	0.00486	68476.	7.94E+09	134.7173	336.2600	0.00
1.1760	-0.9301	2151601.	-55258.	0.00551	65412.	8.31E+09	164.8988	417.0104	0.00
1.3720	-0.9164	2016145.	-54834.	0.00609	62354.	8.66E+09	195.1097	500.7709	0.00
1.5680	-0.9014	1881207.	-54341.	0.00661	59309.	8.96E+09	224.6265	586.0963	0.00
1.7640	-0.8853	1747007.	-53779.	0.00708	56279.	9.21E+09	252.7131	671.3861	0.00
1.9600	-0.8681	1613749.	-53151.	0.00750	53271.	9.37E+09	281.0783	761.5127	0.00
2.1560	-0.8500	1481631.	-52457.	0.00789	50289.	9.44E+09	309.1288	855.3638	0.00
2.3520	-0.8310	1350846.	-51700.	0.00824	47337.	9.44E+09	335.2051	948.7125	0.00
2.5480	-0.8112	1221570.	-50882.	0.00856	44419.	9.44E+09	360.0272	1044.	0.00
2.7440	-0.7907	1093975.	-50008.	0.00885	41539.	9.44E+09	383.5463	1141.	0.00
2.9400	-0.7696	968222.	-49081.	0.00911	38700.	9.44E+09	404.2771	1236.	0.00
3.1360	-0.7479	844459.	-48106.	0.00933	35907.	9.44E+09	425.2057	1337.	0.00

3.3320	-0.7257	722833.	-47077.	0.00953	33161.	9.44E+09	449.3747	1456.	0.00
3.5280	-0.7031	603509.	-45994.	0.00969	30468.	9.44E+09	471.4720	1577.	0.00
3.7240	-0.6801	486638.	-44858.	0.00983	27830.	9.44E+09	494.6681	1711.	0.00
3.9200	-0.6568	372381.	-43657.	0.00994	25250.	9.44E+09	526.8119	1886.	0.00
4.1160	-0.6333	260942.	-42380.	0.01002	22735.	9.44E+09	558.5847	2074.	0.00
4.3120	-0.6097	152527.	-41030.	0.01007	20288.	9.44E+09	589.8461	2275.	0.00
4.5080	-0.5860	47336.	-39607.	0.01009	17913.	9.44E+09	620.4560	2490.	0.00
4.7040	-0.5622	-54434.	-38112.	0.01009	18074.	9.44E+09	650.2740	2720.	0.00
4.9000	-0.5385	-152594.	-36562.	0.01007	20289.	9.44E+09	667.8513	2917.	0.00
5.0960	-0.5149	-247020.	-34966.	0.01002	22421.	9.44E+09	689.7277	3151.	0.00
5.2920	-0.4914	-337567.	-33320.	0.00994	24465.	9.44E+09	709.5958	3396.	0.00
5.4880	-0.4681	-424103.	-31630.	0.00985	26418.	9.44E+09	727.3304	3654.	0.00
5.6840	-0.4451	-506508.	-29901.	0.00973	28278.	9.44E+09	742.8198	3925.	0.00
5.8800	-0.4223	-584674.	-28139.	0.00960	30042.	9.44E+09	755.9663	4210.	0.00
6.0760	-0.3999	-658509.	-26348.	0.00944	31709.	9.44E+09	766.6864	4509.	0.00
6.2720	-0.3779	-727935.	-24531.	0.00927	33276.	9.44E+09	778.5680	4846.	0.00
6.4680	-0.3563	-792869.	-22686.	0.00908	34742.	9.44E+09	789.7799	5213.	0.00
6.6640	-0.3352	-853231.	-20818.	0.00887	36105.	9.44E+09	799.2865	5608.	0.00
6.8600	-0.3146	-908954.	-18929.	0.00865	37362.	9.44E+09	807.1234	6035.	0.00
7.0560	-0.2945	-959981.	-17023.	0.00842	38514.	9.44E+09	813.3399	6496.	0.00
7.2520	-0.2750	-1006263.	-15104.	0.00818	39559.	9.44E+09	817.9993	6997.	0.00
7.4480	-0.2560	-1047764.	-13177.	0.00792	40496.	9.44E+09	821.1776	7544.	0.00
7.6440	-0.2377	-1084455.	-11243.	0.00766	41324.	9.44E+09	822.8751	8142.	0.00
7.8400	-0.2200	-1116318.	-9309.	0.00738	42043.	9.44E+09	822.3197	8791.	0.00
8.0360	-0.2030	-1143347.	-7378.	0.00710	42653.	9.44E+09	819.2721	9493.	0.00
8.2320	-0.1866	-1165552.	-5458.	0.00681	43154.	9.44E+09	813.6931	10255.	0.00
8.4280	-0.1709	-1182959.	-3553.	0.00652	43547.	9.44E+09	805.5504	11084.	0.00
8.6240	-0.1559	-1195608.	-1671.	0.00622	43833.	9.44E+09	794.8194	11987.	0.00
8.8200	-0.1417	-1203556.	182.3132	0.00592	44012.	9.44E+09	781.4818	12975.	0.00
9.0160	-0.1281	-1206873.	2002.	0.00562	44087.	9.44E+09	765.5251	14058.	0.00
9.2120	-0.1152	-1205648.	3780.	0.00532	44059.	9.44E+09	746.9414	15249.	0.00
9.4080	-0.1030	-1199984.	5512.	0.00502	43932.	9.44E+09	725.7262	16566.	0.00
9.6040	-0.09157	-1189999.	7191.	0.00473	43706.	9.44E+09	701.8758	18027.	0.00
9.8000	-0.08081	-1175828.	8811.	0.00443	43386.	9.44E+09	675.3851	19658.	0.00
9.9960	-0.07073	-1157621.	10365.	0.00414	42975.	9.44E+09	646.2431	21489.	0.00
10.1920	-0.06133	-1135544.	11847.	0.00385	42477.	9.44E+09	614.4280	23563.	0.00
10.3880	-0.05260	-1109778.	13252.	0.00357	41895.	9.44E+09	579.8989	25931.	0.00
10.5840	-0.04452	-1080522.	14572.	0.00330	41235.	9.44E+09	542.5843	28668.	0.00
10.7800	-0.03707	-1047988.	15801.	0.00304	40501.	9.44E+09	502.3638	31878.	0.00
10.9760	-0.03023	-1012409.	16931.	0.00278	39698.	9.44E+09	459.0383	35715.	0.00
11.1720	-0.02399	-974031.	17943.	0.00253	38831.	9.44E+09	401.3458	39352.	0.00
11.3680	-0.01832	-933186.	18782.	0.00229	37909.	9.44E+09	311.8346	40042.	0.00
11.5640	-0.01319	-890377.	19417.	0.00207	36943.	9.44E+09	228.4659	40733.	0.00
11.7600	-0.00859	-846078.	19864.	0.00185	35943.	9.44E+09	151.2792	41423.	0.00
11.9560	-0.00448	-800726.	20136.	0.00165	34919.	9.44E+09	80.2693	42113.	0.00
12.1520	-8.46E-04	-754726.	20249.	0.00145	33881.	9.44E+09	15.3891	42804.	0.00
12.3480	0.00235	-708448.	20216.	0.00127	32836.	9.44E+09	-43.4467	43494.	0.00
12.5440	0.00513	-662230.	20051.	0.00110	31793.	9.44E+09	-96.3581	44184.	0.00
12.7400	0.00752	-616376.	19769.	9.40E-04	30758.	9.44E+09	-143.4963	44875.	0.00
12.9360	0.00955	-571159.	19383.	7.92E-04	29737.	9.44E+09	-185.0401	45565.	0.00
13.1320	0.01125	-526820.	18905.	6.55E-04	28737.	9.44E+09	-221.1927	46256.	0.00
13.3280	0.01263	-483570.	18348.	5.29E-04	27760.	9.44E+09	-252.1780	46946.	0.00
13.5240	0.01374	-441592.	17725.	4.14E-04	26813.	9.44E+09	-278.2373	47636.	0.00
13.7200	0.01458	-401040.	17045.	3.09E-04	25897.	9.44E+09	-299.6267	48327.	0.00

13.9160	0.01519	-362044.	16320.	2.14E-04	25017.	9.44E+09	-316.6137	49017.	0.00
14.1120	0.01559	-324707.	15561.	1.29E-04	24174.	9.44E+09	-329.4741	49707.	0.00
14.3080	0.01580	-289110.	14775.	5.21E-05	23371.	9.44E+09	-338.4901	50398.	0.00
14.5040	0.01583	-255312.	13973.	-1.58E-05	22608.	9.44E+09	-343.9473	51088.	0.00
14.7000	0.01572	-223351.	13161.	-7.54E-05	21886.	9.44E+09	-346.1325	51779.	0.00
14.8960	0.01548	-193248.	12348.	-1.27E-04	21207.	9.44E+09	-345.3318	52469.	0.00
15.0920	0.01512	-165006.	11540.	-1.72E-04	20569.	9.44E+09	-341.8287	53159.	0.00
15.2880	0.01467	-138613.	10743.	-2.10E-04	19974.	9.44E+09	-335.9025	53850.	0.00
15.4840	0.01414	-114043.	9884.	-2.41E-04	19419.	9.44E+09	-393.9181	65536.	0.00
15.6800	0.01354	-91622.	8968.	-2.67E-04	18913.	9.44E+09	-385.4571	66974.	0.00
15.8760	0.01288	-71311.	8072.	-2.87E-04	18455.	9.44E+09	-376.0232	68655.	0.00
16.0720	0.01219	-53062.	7200.	-3.03E-04	18043.	9.44E+09	-365.7197	70589.	0.00
16.2680	0.01146	-36822.	6353.	-3.14E-04	17676.	9.44E+09	-354.6365	72795.	0.00
16.4640	0.01071	-22535.	5533.	-3.21E-04	17354.	9.44E+09	-342.8502	75298.	0.00
16.6600	0.00995	-10139.	4741.	-3.25E-04	17074.	9.44E+09	-330.4247	78130.	0.00
16.8560	0.00918	432.2337	3979.	-3.27E-04	16855.	9.44E+09	-317.4106	81333.	0.00
17.0520	0.00841	9247.	3249.	-3.25E-04	17054.	9.44E+09	-303.8441	84965.	0.00
17.2480	0.00765	16379.	2550.	-3.22E-04	17215.	9.44E+09	-289.7454	89099.	0.00
17.4440	0.00690	21904.	1886.	-3.17E-04	17339.	9.44E+09	-275.1162	93837.	0.00
17.6400	0.00616	25901.	1257.	-3.11E-04	17430.	9.44E+09	-259.9353	99318.	0.00
17.8360	0.00543	28454.	664.1935	-3.05E-04	17487.	9.44E+09	-244.1524	105738.	0.00
18.0320	0.00472	29649.	109.3205	-2.97E-04	17514.	9.44E+09	-227.6784	113389.	0.00
18.2280	0.00403	29577.	-405.8234	-2.90E-04	17513.	9.44E+09	-210.3692	122719.	0.00
18.4240	0.00336	28334.	-879.0072	-2.83E-04	17484.	9.44E+09	-191.9979	134462.	0.00
18.6200	0.00270	26021.	-1307.	-2.76E-04	17432.	9.44E+09	-172.2018	149920.	0.00
18.8160	0.00206	22749.	-1687.	-2.70E-04	17358.	9.44E+09	-150.3699	171687.	0.00
19.0120	0.00143	18639.	-2120.	-2.65E-04	17266.	9.44E+09	-218.3234	358661.	0.00
19.2080	8.14E-04	13317.	-2600.	-2.61E-04	17145.	9.44E+09	-189.6033	547605.	0.00
19.4040	2.05E-04	6943.	-2943.	-2.58E-04	17002.	9.44E+09	-102.4101	1176000.	0.00
19.6000	-4.01E-04	0.00	0.00	-2.57E-04	16845.	9.44E+09	2605.	7647071.	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 = -0.97500000 inches
Computed slope at pile head = 0.000000 radians
Maximum bending moment = 2961638. inch-lbs
Maximum shear force = -56398. 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 = 2

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	-0.9750	S, rad	0.00	435000.	-0.9750	0.00	-56398.	2961638.

Maximum pile-head deflection = -0.9750000000 inches
Maximum pile-head rotation = 0.0000000000 radians = 0.000000 deg.

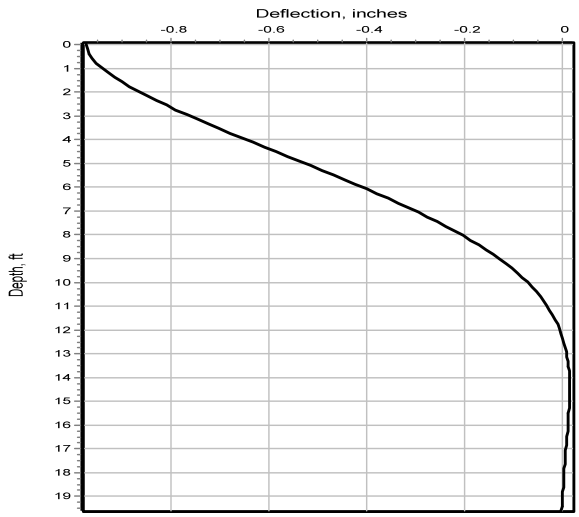
Summary of Warning Messages

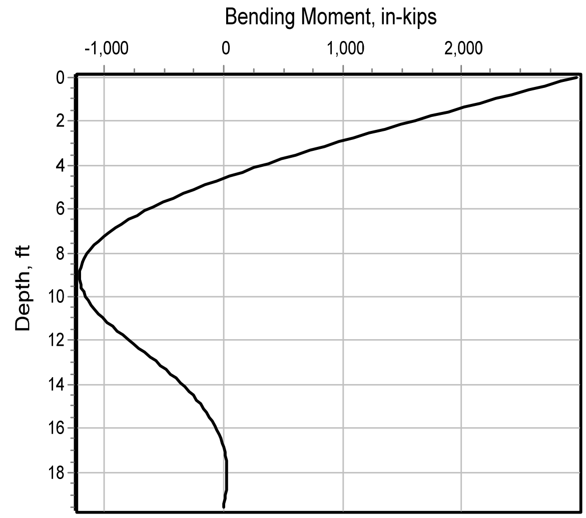
The following warning was reported 115 times

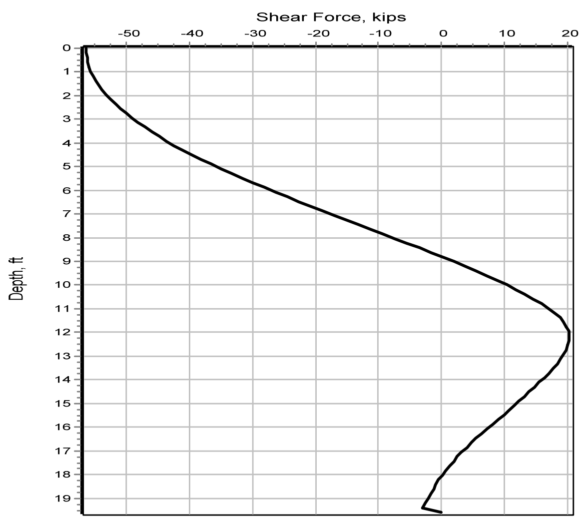
**** Warning ****

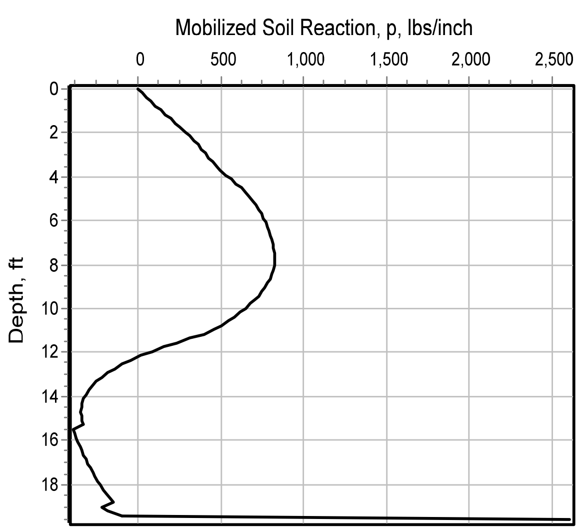
This warning is for an input value for uniaxial compressive strength that has been specified for a soil defined using the vuggy limestone criteria. The input value is outside of the range of 1,000 to 2,500 psi (6,895 to 17,237 kPa) which were used in actual field tests on which this theory is based. Higher or lower values may be applicable but the user is warned about the theoretical and testing limitations.

The analysis ended normally.


Deflection, inches
-0.8
-0.6
-0.4
-0.2
0
Depth, ft
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19


Bending Moment, in-kips
-1,000
0
1,000
2,000
Depth, ft
0
2
4
6
8
10
12
14
16
18


Shear Force, kips
-50
-40
-30
-20
-10
0
10
20
Depth, ft
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19


Mobilized Soil Reaction, p, lbs/inch
0
500
1,000
1,500
2,000
2,500
Depth, ft
0
2
4
6
8
10
12
14
16
18

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:
\\golderassociates.sharepoint.com@SSL\DavWWWRoot\sites\139980\Project Files\5 Technical Work\06 Analysis\Phase II Pile Design\LPILE\Abutment 2 Strength I\

Name of input data file:
Freeport Exit 20 Abutment 2 girder rotation Plastic Hinge.lp11d

Name of output report file:
Freeport Exit 20 Abutment 2 girder rotation Plastic Hinge.lp11o

Name of plot output file:
Freeport Exit 20 Abutment 2 girder rotation Plastic Hinge.lp11p

Name of runtime message file:
Freeport Exit 20 Abutment 2 girder rotation Plastic Hinge.lp11r

----
Date and Time of Analysis

Date: July 14, 2021 Time: 16:03:25

Problem Title

Project Name: MaineDOT I-295 Exit 20 Desert Road Bridge No. 5720
Job Number: 21450908
Client: MaineDOT
Engineer: KAR
Description: Abutment 2 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 = 19.600 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	19.600	14.6950

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is a H weak axis steel pile
Length of section = 19.600000 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 = 15.300000 ft

Effective unit weight at top of layer = 125.000000 pcf
Effective unit weight at bottom of layer = 125.000000 pcf
Friction angle at top of layer = 32.000000 deg.
Friction angle at bottom of layer = 32.000000 deg.
Subgrade k at top of layer = 124.800000 pci
Subgrade k at bottom of layer = 124.800000 pci

Layer 2 is stiff clay with water-induced erosion

Distance from top of pile to top of layer = 15.300000 ft
Distance from top of pile to bottom of layer = 19.000000 ft
Effective unit weight at top of layer = 125.000000 pcf
Effective unit weight at bottom of layer = 125.000000 pcf
Undrained cohesion at top of layer = 1600. psf
Undrained cohesion at bottom of layer = 1600. psf
Epsilon-50 at top of layer = 0.005000
Epsilon-50 at bottom of layer = 0.005000
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 = 19.000000 ft
Distance from top of pile to bottom of layer = 19.600000 ft
Effective unit weight at top of layer = 62.600000 pcf
Effective unit weight at bottom of layer = 62.600000 pcf
Undrained cohesion at top of layer = 1600. psf
Undrained cohesion at bottom of layer = 1600. psf
Epsilon-50 at top of layer = 0.005000
Epsilon-50 at bottom of layer = 0.005000

Layer 4 is strong rock (vuggy limestone)

Distance from top of pile to top of layer = 19.600000 ft
Distance from top of pile to bottom of layer = 50.000000 ft
Effective unit weight at top of layer = 101.600000 pcf
Effective unit weight at bottom of layer = 101.600000 pcf
Uniaxial compressive strength at top of layer = 12983. psi
Uniaxial compressive strength at bottom of layer = 12983. psi

(Depth of the lowest soil layer extends 30.400 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	--	32.0000	--	--	124.8000
	(Reese, et al.)	15.3000	125.0000	--	32.0000	--	--	124.8000
2	Stiff Clay	15.3000	125.0000	1600.	--	--	0.00500	500.0000
	with Free Water	19.0000	125.0000	1600.	--	--	0.00500	500.0000
3	Stiff Clay	19.0000	62.6000	1600.	--	--	0.00500	--
	w/o Free Water	19.6000	62.6000	1600.	--	--	0.00500	--
4	Strong Rock	19.6000	101.6000	--	--	12983.	--	--
	(Vuggy Limestone)	50.0000	101.6000	--	--	12983.	--	--

Static Loading Type

Static loading criteria were used when computing p-y curves for all analyses.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 2

Load No.	Load Type	Condition 1	Condition 2	Axial Thrust Force, lbs	Compute Top y vs. Pile Length	Run Analysis
1	5	y = -0.975000 in	S = 0.0000 in/in	435000.	N.A.	Yes
2	4	y = -0.975000 in	M = 1772170. in-lbs	435000.	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	=	19.600000 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	435.000

Definition of Run Messages:

Y = part of pipe section has yielded.

Axial Thrust Force = 435.000 kips

Bending Curvature rad/in.	Bending Moment in-kip	Bending Stiffness kip-in2	Depth to N Axis in	Max Total Stress ksi	Run Msg
0.00000425	40.0990456	9438779.	144.0737922	17.7410647	
0.00000850	80.1980912	9438779.	75.7106461	18.6372360	
0.00001274	120.2971368	9438779.	52.9229307	19.5334074	
0.00001699	160.3961825	9438779.	41.5290730	20.4295784	
0.00002124	200.4952281	9438779.	34.6927584	21.3257498	
0.00002549	240.5942737	9438779.	30.1352154	22.2219212	
0.00002974	280.6933193	9438779.	26.8798275	23.1180924	
0.00003399	320.7923649	9438779.	24.4382865	24.0142635	
0.00003823	360.8914105	9438779.	22.5393102	24.9104349	
0.00004248	400.9904562	9438779.	21.0201292	25.8066061	
0.00004673	441.0895018	9438779.	19.7771629	26.7027775	

0.00005098	481.1885474	9438779.	18.7413577	27.5989488	
0.00005523	521.2875930	9438779.	17.8649071	28.4951200	
0.00005948	561.3866386	9438779.	17.1136637	29.3912914	
0.00006372	601.4856842	9438779.	16.4625861	30.2874627	
0.00006797	641.5847299	9438779.	15.8928933	31.1836339	
0.00007222	681.6837755	9438779.	15.3902231	32.0798052	
0.00007647	721.7828211	9438779.	14.9434051	32.9759765	
0.00008072	761.8818667	9438779.	14.5436206	33.8721478	
0.00008497	801.9809123	9438779.	14.1838146	34.7683191	
0.00008921	842.0799579	9438779.	13.8582758	35.6644903	
0.00009346	882.1790036	9438779.	13.5623315	36.5606617	
0.00009771	922.2780492	9438779.	13.2921214	37.4568330	
0.0001020	962.3770948	9438779.	13.0444288	38.3530042	
0.0001062	1002.	9438779.	12.8165517	39.2491755	
0.0001105	1043.	9438779.	12.6062035	40.1453468	
0.0001147	1083.	9438779.	12.4114367	41.0415181	
0.0001190	1123.	9438779.	12.2305819	41.9376894	
0.0001232	1163.	9438779.	12.0621997	42.8338607	
0.0001274	1203.	9438779.	11.9050431	43.7300320	
0.0001317	1243.	9438779.	11.7580256	44.6262033	
0.0001359	1283.	9438779.	11.6201966	45.5223745	
0.0001402	1323.	9438779.	11.4907210	46.4185458	
0.0001444	1363.	9438779.	11.3688615	47.3147171	
0.0001487	1403.	9438779.	11.2539655	48.2108884	
0.0001529	1444.	9438779.	11.1454526	49.1070597	
0.0001572	1484.	9438751.	11.0428102	50.0000000	Y
0.0001614	1522.	9429640.	10.9472446	50.0000000	Y
0.0001657	1560.	9412595.	10.8581657	50.0000000	Y
0.0001742	1630.	9358708.	10.6973886	50.0000000	Y
0.0001827	1696.	9283509.	10.5569141	50.0000000	Y
0.0001912	1758.	9193715.	10.4334619	50.0000000	Y
0.0001997	1816.	9094026.	10.3244254	50.0000000	Y
0.0002082	1871.	8986798.	10.2279443	50.0000000	Y
0.0002167	1923.	8875634.	10.1420359	50.0000000	Y
0.0002252	1973.	8762427.	10.0652455	50.0000000	Y
0.0002337	2021.	8647690.	9.9966068	50.0000000	Y
0.0002422	2066.	8533251.	9.9349004	50.0000000	Y
0.0002507	2110.	8420020.	9.8792269	50.0000000	Y
0.0002591	2153.	8307885.	9.8290540	50.0000000	Y
0.0002676	2194.	8197534.	9.7836823	50.0000000	Y
0.0002761	2234.	8089498.	9.7425033	50.0000000	Y
0.0002846	2273.	7983937.	9.7050645	50.0000000	Y
0.0002931	2310.	7880983.	9.6709633	50.0000000	Y
0.0003016	2347.	7780739.	9.6398398	50.0000000	Y
0.0003101	2383.	7683284.	9.6113710	50.0000000	Y
0.0003186	2418.	7588351.	9.5853874	50.0000000	Y
0.0003271	2452.	7495950.	9.5616498	50.0000000	Y
0.0003356	2486.	7406379.	9.5398233	50.0000000	Y
0.0003441	2518.	7316824.	9.5188626	50.0000000	Y
0.0003526	2548.	7225653.	9.4983357	50.0000000	Y
0.0003611	2576.	7133382.	9.4783012	50.0000000	Y
0.0003696	2602.	7041200.	9.4584658	50.0000000	Y
0.0003781	2627.	6948782.	9.4391951	50.0000000	Y
0.0003866	2651.	6857065.	9.4202497	50.0000000	Y

0.0003951	2673.	6765488.	9.4014792	50.0000000	Y
0.0004036	2694.	6674708.	9.3830767	50.0000000	Y
0.0004121	2714.	6585278.	9.3650609	50.0000000	Y
0.0004206	2733.	6497326.	9.3474897	50.0000000	Y
0.0004291	2750.	6409739.	9.3299713	50.0000000	Y
0.0004376	2767.	6324218.	9.3127442	50.0000000	Y
0.0004461	2783.	6239945.	9.2962005	50.0000000	Y
0.0004546	2799.	6156855.	9.2794441	50.0000000	Y
0.0004631	2813.	6075700.	9.2633234	50.0000000	Y
0.0004716	2827.	5995548.	9.2473354	50.0000000	Y
0.0004801	2841.	5917409.	9.2315354	50.0000000	Y
0.0004886	2853.	5840533.	9.2161292	50.0000000	Y
0.0004971	2866.	5765099.	9.2009795	50.0000000	Y
0.0005056	2877.	5691443.	9.1859736	50.0000000	Y
0.0005395	2919.	5410963.	9.1283921	50.0000000	Y
0.0005735	2955.	5153025.	9.0744546	50.0000000	Y
0.0006075	2986.	4915807.	9.0237309	50.0000000	Y
0.0006415	3013.	4697588.	8.9758365	50.0000000	Y
0.0006755	3037.	4496247.	8.9303610	50.0000000	Y
0.0007095	3058.	4310271.	8.8876853	50.0000000	Y
0.0007435	3077.	4138399.	8.8469591	50.0000000	Y
0.0007774	3093.	3978856.	8.8086420	50.0000000	Y
0.0008114	3108.	3830725.	8.7720830	50.0000000	Y
0.0008454	3122.	3692914.	8.7372846	50.0000000	Y
0.0008794	3134.	3564174.	8.7044216	50.0000000	Y
0.0009134	3145.	3443658.	8.6726649	50.0000000	Y
0.0009474	3156.	3331015.	8.6423887	50.0000000	Y

Summary of Results for Nominal Moment Capacity for Section 1

Load No.	Axial Thrust kips	Nominal Moment Capacity in-kips
1	435.0000000000	3156.

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	350765.
2	15.3000	256.0114	No	No	350765.	6992.
3	19.0000	24.3995	No	No	357757.	10580.
4	19.6000	19.6000	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 = -0.975000 inches
Rotation of pile head = 0.000E+00 radians
Axial load on pile head = 435000.0 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	-0.9750	2961638.	-56398.	0.00	83697.	5.10E+09	0.00	0.00	0.00
0.1960	-0.9734	2828385.	-56332.	0.00133	80689.	5.10E+09	22.5096	54.3897	0.00
0.3920	-0.9687	2693922.	-56249.	0.00246	77654.	6.67E+09	47.9215	116.3507	0.00
0.5880	-0.9618	2558752.	-56104.	0.00336	74603.	7.19E+09	75.6220	184.9245	0.00
0.7840	-0.9529	2423145.	-55892.	0.00415	71541.	7.57E+09	104.7502	258.5398	0.00
0.9800	-0.9423	2287347.	-55610.	0.00486	68476.	7.94E+09	134.7173	336.2600	0.00
1.1760	-0.9301	2151601.	-55258.	0.00551	65412.	8.31E+09	164.8988	417.0104	0.00
1.3720	-0.9164	2016145.	-54834.	0.00609	62354.	8.66E+09	195.1097	500.7709	0.00
1.5680	-0.9014	1881207.	-54341.	0.00661	59309.	8.96E+09	224.6265	586.0963	0.00
1.7640	-0.8853	1747007.	-53779.	0.00708	56279.	9.21E+09	252.7131	671.3861	0.00
1.9600	-0.8681	1613749.	-53151.	0.00750	53271.	9.37E+09	281.0783	761.5127	0.00
2.1560	-0.8500	1481631.	-52457.	0.00789	50289.	9.44E+09	309.1288	855.3638	0.00
2.3520	-0.8310	1350846.	-51700.	0.00824	47337.	9.44E+09	335.2051	948.7125	0.00
2.5480	-0.8112	1221570.	-50882.	0.00856	44419.	9.44E+09	360.0272	1044.	0.00
2.7440	-0.7907	1093975.	-50008.	0.00885	41539.	9.44E+09	383.5463	1141.	0.00
2.9400	-0.7696	968222.	-49081.	0.00911	38700.	9.44E+09	404.2771	1236.	0.00

3.1360	-0.7479	844459.	-48106.	0.00933	35907.	9.44E+09	425.2057	1337.	0.00
3.3320	-0.7257	722833.	-47077.	0.00953	33161.	9.44E+09	449.3747	1456.	0.00
3.5280	-0.7031	603509.	-45994.	0.00969	30468.	9.44E+09	471.4720	1577.	0.00
3.7240	-0.6801	486638.	-44858.	0.00983	27830.	9.44E+09	494.6681	1711.	0.00
3.9200	-0.6568	372381.	-43657.	0.00994	25250.	9.44E+09	526.8119	1886.	0.00
4.1160	-0.6333	260942.	-42380.	0.01002	22735.	9.44E+09	558.5847	2074.	0.00
4.3120	-0.6097	152527.	-41030.	0.01007	20288.	9.44E+09	589.8461	2275.	0.00
4.5080	-0.5860	47336.	-39607.	0.01009	17913.	9.44E+09	620.4560	2490.	0.00
4.7040	-0.5622	-54434.	-38112.	0.01009	18074.	9.44E+09	650.2740	2720.	0.00
4.9000	-0.5385	-152594.	-36562.	0.01007	20289.	9.44E+09	667.8513	2917.	0.00
5.0960	-0.5149	-247020.	-34966.	0.01002	22421.	9.44E+09	689.7277	3151.	0.00
5.2920	-0.4914	-337567.	-33320.	0.00994	24465.	9.44E+09	709.5958	3396.	0.00
5.4880	-0.4681	-424103.	-31630.	0.00985	26418.	9.44E+09	727.3304	3654.	0.00
5.6840	-0.4451	-506508.	-29901.	0.00973	28278.	9.44E+09	742.8198	3925.	0.00
5.8800	-0.4223	-584674.	-28139.	0.00960	30042.	9.44E+09	755.9663	4210.	0.00
6.0760	-0.3999	-658509.	-26348.	0.00944	31709.	9.44E+09	766.6864	4509.	0.00
6.2720	-0.3779	-727935.	-24531.	0.00927	33276.	9.44E+09	778.5680	4846.	0.00
6.4680	-0.3563	-792869.	-22686.	0.00908	34742.	9.44E+09	789.7799	5213.	0.00
6.6640	-0.3352	-853231.	-20818.	0.00887	36105.	9.44E+09	799.2865	5608.	0.00
6.8600	-0.3146	-908954.	-18929.	0.00865	37362.	9.44E+09	807.1234	6035.	0.00
7.0560	-0.2945	-959981.	-17023.	0.00842	38514.	9.44E+09	813.3399	6496.	0.00
7.2520	-0.2750	-1006263.	-15104.	0.00818	39559.	9.44E+09	817.9993	6997.	0.00
7.4480	-0.2560	-1047764.	-13177.	0.00792	40496.	9.44E+09	821.1776	7544.	0.00
7.6440	-0.2377	-1084455.	-11243.	0.00766	41324.	9.44E+09	822.8751	8142.	0.00
7.8400	-0.2200	-1116318.	-9309.	0.00738	42043.	9.44E+09	822.3197	8791.	0.00
8.0360	-0.2030	-1143347.	-7378.	0.00710	42653.	9.44E+09	819.2721	9493.	0.00
8.2320	-0.1866	-1165552.	-5458.	0.00681	43154.	9.44E+09	813.6931	10255.	0.00
8.4280	-0.1709	-1182959.	-3553.	0.00652	43547.	9.44E+09	805.5504	11084.	0.00
8.6240	-0.1559	-1195608.	-1671.	0.00622	43833.	9.44E+09	794.8194	11987.	0.00
8.8200	-0.1417	-1203556.	182.3132	0.00592	44012.	9.44E+09	781.4818	12975.	0.00
9.0160	-0.1281	-1206873.	2002.	0.00562	44087.	9.44E+09	765.5251	14058.	0.00
9.2120	-0.1152	-1205648.	3780.	0.00532	44059.	9.44E+09	746.9414	15249.	0.00
9.4080	-0.1030	-1199984.	5512.	0.00502	43932.	9.44E+09	725.7262	16566.	0.00
9.6040	-0.09157	-1189999.	7191.	0.00473	43706.	9.44E+09	701.8758	18027.	0.00
9.8000	-0.08081	-1175828.	8811.	0.00443	43386.	9.44E+09	675.3851	19658.	0.00
9.9960	-0.07073	-1157621.	10365.	0.00414	42975.	9.44E+09	646.2431	21489.	0.00
10.1920	-0.06133	-1135544.	11847.	0.00385	42477.	9.44E+09	614.4280	23563.	0.00
10.3880	-0.05260	-1109778.	13252.	0.00357	41895.	9.44E+09	579.8989	25931.	0.00
10.5840	-0.04452	-1080522.	14572.	0.00330	41235.	9.44E+09	542.5843	28668.	0.00
10.7800	-0.03707	-1047988.	15801.	0.00304	40501.	9.44E+09	502.3638	31878.	0.00
10.9760	-0.03023	-1012409.	16931.	0.00278	39698.	9.44E+09	459.0383	35715.	0.00
11.1720	-0.02399	-974031.	17943.	0.00253	38831.	9.44E+09	401.3458	39352.	0.00
11.3680	-0.01832	-933186.	18782.	0.00229	37909.	9.44E+09	311.8346	40042.	0.00
11.5640	-0.01319	-890377.	19417.	0.00207	36943.	9.44E+09	228.4659	40733.	0.00
11.7600	-0.00859	-846078.	19864.	0.00185	35943.	9.44E+09	151.2792	41423.	0.00
11.9560	-0.00448	-800726.	20136.	0.00165	34919.	9.44E+09	80.2693	42113.	0.00
12.1520	-8.46E-04	-754726.	20249.	0.00145	33881.	9.44E+09	15.3891	42804.	0.00
12.3480	0.00235	-708448.	20216.	0.00127	32836.	9.44E+09	-43.4467	43494.	0.00
12.5440	0.00513	-662230.	20051.	0.00110	31793.	9.44E+09	-96.3581	44184.	0.00
12.7400	0.00752	-616376.	19769.	9.40E-04	30758.	9.44E+09	-143.4963	44875.	0.00
12.9360	0.00955	-571159.	19383.	7.92E-04	29737.	9.44E+09	-185.0401	45565.	0.00
13.1320	0.01125	-526820.	18905.	6.55E-04	28737.	9.44E+09	-221.1927	46256.	0.00
13.3280	0.01263	-483570.	18348.	5.29E-04	27760.	9.44E+09	-252.1780	46946.	0.00
13.5240	0.01374	-441592.	17725.	4.14E-04	26813.	9.44E+09	-278.2373	47636.	0.00

13.7200	0.01458	-401040.	17045.	3.09E-04	25897.	9.44E+09	-299.6267	48327.	0.00
13.9160	0.01519	-362044.	16320.	2.14E-04	25017.	9.44E+09	-316.6137	49017.	0.00
14.1120	0.01559	-324707.	15561.	1.29E-04	24174.	9.44E+09	-329.4741	49707.	0.00
14.3080	0.01580	-289110.	14775.	5.21E-05	23371.	9.44E+09	-338.4901	50398.	0.00
14.5040	0.01583	-255312.	13973.	-1.58E-05	22608.	9.44E+09	-343.9473	51088.	0.00
14.7000	0.01572	-223351.	13161.	-7.54E-05	21886.	9.44E+09	-346.1325	51779.	0.00
14.8960	0.01548	-193248.	12348.	-1.27E-04	21207.	9.44E+09	-345.3318	52469.	0.00
15.0920	0.01512	-165006.	11540.	-1.72E-04	20569.	9.44E+09	-341.8287	53159.	0.00
15.2880	0.01467	-138613.	10743.	-2.10E-04	19974.	9.44E+09	-335.9025	53850.	0.00
15.4840	0.01414	-114043.	9884.	-2.41E-04	19419.	9.44E+09	-393.9181	65536.	0.00
15.6800	0.01354	-91622.	8968.	-2.67E-04	18913.	9.44E+09	-385.4571	66974.	0.00
15.8760	0.01288	-71311.	8072.	-2.87E-04	18455.	9.44E+09	-376.0232	68655.	0.00
16.0720	0.01219	-53062.	7200.	-3.03E-04	18043.	9.44E+09	-365.7197	70589.	0.00
16.2680	0.01146	-36822.	6353.	-3.14E-04	17676.	9.44E+09	-354.6365	72795.	0.00
16.4640	0.01071	-22535.	5533.	-3.21E-04	17354.	9.44E+09	-342.8502	75298.	0.00
16.6600	0.00995	-10139.	4741.	-3.25E-04	17074.	9.44E+09	-330.4247	78130.	0.00
16.8560	0.00918	432.2337	3979.	-3.27E-04	16855.	9.44E+09	-317.4106	81333.	0.00
17.0520	0.00841	9247.	3249.	-3.25E-04	17054.	9.44E+09	-303.8441	84965.	0.00
17.2480	0.00765	16379.	2550.	-3.22E-04	17215.	9.44E+09	-289.7454	89099.	0.00
17.4440	0.00690	21904.	1886.	-3.17E-04	17339.	9.44E+09	-275.1162	93837.	0.00
17.6400	0.00616	25901.	1257.	-3.11E-04	17430.	9.44E+09	-259.9353	99318.	0.00
17.8360	0.00543	28454.	664.1935	-3.05E-04	17487.	9.44E+09	-244.1524	105738.	0.00
18.0320	0.00472	29649.	109.3205	-2.97E-04	17514.	9.44E+09	-227.6784	113389.	0.00
18.2280	0.00403	29577.	-405.8234	-2.90E-04	17513.	9.44E+09	-210.3692	122719.	0.00
18.4240	0.00336	28334.	-879.0072	-2.83E-04	17484.	9.44E+09	-191.9979	134462.	0.00
18.6200	0.00270	26021.	-1307.	-2.76E-04	17432.	9.44E+09	-172.2018	149920.	0.00
18.8160	0.00206	22749.	-1687.	-2.70E-04	17358.	9.44E+09	-150.3699	171687.	0.00
19.0120	0.00143	18639.	-2120.	-2.65E-04	17266.	9.44E+09	-218.3234	358661.	0.00
19.2080	8.14E-04	13317.	-2600.	-2.61E-04	17145.	9.44E+09	-189.6033	547605.	0.00
19.4040	2.05E-04	6943.	-2943.	-2.58E-04	17002.	9.44E+09	-102.4101	1176000.	0.00
19.6000	-4.01E-04	0.00	0.00	-2.57E-04	16845.	9.44E+09	2605.	7647071.	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 = -0.97500000 inches
Computed slope at pile head = 0.000000 radians
Maximum bending moment = 2961638. inch-lbs
Maximum shear force = -56398. 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 = 2

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 = -0.975000 inches
Moment at pile head = 1772170.0 in-lbs
Axial load at pile head = 435000.0 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	-0.9750	1772170.	-42640.	0.00634	56847.	9.16E+09	0.00	0.00	0.00
0.1960	-0.9595	1665159.	-42613.	0.00678	54432.	9.16E+09	22.5096	55.1745	0.00
0.3920	-0.9431	1557834.	-42531.	0.00719	52009.	9.41E+09	47.9213	119.5128	0.00
0.5880	-0.9257	1450377.	-42385.	0.00757	49584.	9.44E+09	75.6216	192.1355	0.00
0.7840	-0.9075	1342968.	-42173.	0.00792	47159.	9.44E+09	104.7495	271.4871	0.00
0.9800	-0.8885	1235796.	-41892.	0.00824	44740.	9.44E+09	134.7164	356.6260	0.00
1.1760	-0.8687	1129054.	-41539.	0.00853	42331.	9.44E+09	164.8975	446.4398	0.00
1.3720	-0.8483	1022936.	-41116.	0.00880	39935.	9.44E+09	195.1080	540.9323	0.00
1.5680	-0.8273	917637.	-40622.	0.00904	37558.	9.44E+09	224.6244	638.5714	0.00
1.7640	-0.8058	813347.	-40061.	0.00926	35204.	9.44E+09	252.7106	737.6163	0.00
1.9600	-0.7838	710248.	-39433.	0.00945	32877.	9.44E+09	281.0754	843.4484	0.00
2.1560	-0.7614	608522.	-38739.	0.00961	30581.	9.44E+09	309.1255	954.9469	0.00
2.3520	-0.7386	508351.	-37981.	0.00975	28320.	9.44E+09	335.2013	1067.	0.00
2.5480	-0.7155	409905.	-37164.	0.00987	26097.	9.44E+09	360.0229	1183.	0.00
2.7440	-0.6922	313346.	-36289.	0.00996	23918.	9.44E+09	383.5414	1303.	0.00
2.9400	-0.6687	218828.	-35363.	0.01002	21784.	9.44E+09	404.2718	1422.	0.00
3.1360	-0.6450	126492.	-34387.	0.01006	19700.	9.44E+09	425.1999	1550.	0.00
3.3320	-0.6213	36475.	-33359.	0.01009	17668.	9.44E+09	449.3683	1701.	0.00
3.5280	-0.5976	-51065.	-32276.	0.01008	17998.	9.44E+09	471.4651	1856.	0.00
3.7240	-0.5739	-135984.	-31140.	0.01006	19914.	9.44E+09	494.6605	2027.	0.00
3.9200	-0.5503	-218132.	-29939.	0.01002	21769.	9.44E+09	526.3349	2250.	0.00
4.1160	-0.5268	-297313.	-28682.	0.00995	23556.	9.44E+09	542.7829	2423.	0.00
4.3120	-0.5035	-373415.	-27390.	0.00987	25274.	9.44E+09	555.9881	2597.	0.00
4.5080	-0.4804	-446347.	-26071.	0.00977	26920.	9.44E+09	565.6565	2770.	0.00
4.7040	-0.4575	-516035.	-24733.	0.00965	28493.	9.44E+09	571.5027	2938.	0.00
4.9000	-0.4350	-582431.	-23387.	0.00951	29992.	9.44E+09	573.3978	3100.	0.00
5.0960	-0.4128	-645506.	-22019.	0.00936	31416.	9.44E+09	589.4978	3359.	0.00
5.2920	-0.3910	-705155.	-20616.	0.00919	32762.	9.44E+09	603.7540	3632.	0.00
5.4880	-0.3696	-761285.	-19182.	0.00901	34029.	9.44E+09	616.0828	3921.	0.00
5.6840	-0.3486	-813812.	-17720.	0.00881	35215.	9.44E+09	626.4124	4226.	0.00
5.8800	-0.3281	-862667.	-16237.	0.00860	36318.	9.44E+09	634.6838	4549.	0.00
6.0760	-0.3082	-907791.	-14737.	0.00838	37336.	9.44E+09	640.8507	4891.	0.00
6.2720	-0.2887	-949138.	-13221.	0.00815	38269.	9.44E+09	648.9508	5287.	0.00
6.4680	-0.2698	-986653.	-11685.	0.00791	39116.	9.44E+09	656.9252	5726.	0.00
6.6640	-0.2515	-1020283.	-10132.	0.00766	39875.	9.44E+09	663.6358	6206.	0.00
6.8600	-0.2338	-1049982.	-8565.	0.00740	40546.	9.44E+09	668.9672	6729.	0.00
7.0560	-0.2167	-1075712.	-6987.	0.00713	41126.	9.44E+09	672.2939	7296.	0.00
7.2520	-0.2002	-1097448.	-5405.	0.00686	41617.	9.44E+09	673.5023	7910.	0.00
7.4480	-0.1844	-1115180.	-3822.	0.00659	42017.	9.44E+09	672.5419	8577.	0.00
7.6440	-0.1693	-1128906.	-2244.	0.00631	42327.	9.44E+09	669.3686	9301.	0.00
7.8400	-0.1548	-1138642.	-675.7561	0.00603	42547.	9.44E+09	663.9451	10091.	0.00

8.0360	-0.1409	-1144415.	876.7818	0.00574	42677.	9.44E+09	656.2402	10953.	0.00
8.2320	-0.1278	-1146265.	2408.	0.00546	42719.	9.44E+09	646.2281	11898.	0.00
8.4280	-0.1153	-1144249.	3914.	0.00517	42674.	9.44E+09	633.8882	12936.	0.00
8.6240	-0.1034	-1138434.	5388.	0.00489	42542.	9.44E+09	619.2034	14081.	0.00
8.8200	-0.09227	-1128903.	6824.	0.00460	42327.	9.44E+09	602.1593	15349.	0.00
9.0160	-0.08177	-1115754.	8217.	0.00432	42030.	9.44E+09	582.7415	16761.	0.00
9.2120	-0.07193	-1099097.	9562.	0.00405	41654.	9.44E+09	560.9336	18341.	0.00
9.4080	-0.06273	-1079056.	10853.	0.00378	41202.	9.44E+09	536.7128	20122.	0.00
9.6040	-0.05417	-1055771.	12084.	0.00351	40676.	9.44E+09	510.0444	22146.	0.00
9.8000	-0.04622	-1029395.	13249.	0.00325	40081.	9.44E+09	480.8742	24470.	0.00
9.9960	-0.03888	-1000097.	14343.	0.00300	39420.	9.44E+09	449.1159	27171.	0.00
10.1920	-0.03212	-968060.	15359.	0.00275	38696.	9.44E+09	414.6319	30362.	0.00
10.3880	-0.02593	-933482.	16290.	0.00252	37916.	9.44E+09	377.2014	34215.	0.00
10.5840	-0.02029	-896579.	17112.	0.00229	37083.	9.44E+09	321.5459	37281.	0.00
10.7800	-0.01517	-857669.	17778.	0.00207	36205.	9.44E+09	244.8794	37971.	0.00
10.9760	-0.01055	-817186.	18270.	0.00186	35291.	9.44E+09	173.4712	38661.	0.00
11.1720	-0.00642	-775534.	18600.	0.00166	34351.	9.44E+09	107.3669	39352.	0.00
11.3680	-0.00274	-733091.	18781.	0.00147	33393.	9.44E+09	46.5726	40042.	0.00
11.5640	5.16E-04	-690204.	18825.	0.00130	32425.	9.44E+09	-8.9422	40733.	0.00
11.7600	0.00336	-647190.	18745.	0.00113	31454.	9.44E+09	-59.2418	41423.	0.00
11.9560	0.00583	-604339.	18553.	9.74E-04	30486.	9.44E+09	-104.4214	42113.	0.00
12.1520	0.00795	-561911.	18260.	8.29E-04	29529.	9.44E+09	-144.6041	42804.	0.00
12.3480	0.00973	-520140.	17878.	6.94E-04	28586.	9.44E+09	-179.9377	43494.	0.00
12.5440	0.01121	-479232.	17419.	5.69E-04	27662.	9.44E+09	-210.5922	44184.	0.00
12.7400	0.01241	-439366.	16893.	4.55E-04	26763.	9.44E+09	-236.7565	44875.	0.00
12.9360	0.01335	-400698.	16310.	3.50E-04	25890.	9.44E+09	-258.6361	45565.	0.00
13.1320	0.01406	-363359.	15681.	2.55E-04	25047.	9.44E+09	-276.4497	46256.	0.00
13.3280	0.01455	-327457.	15014.	1.69E-04	24236.	9.44E+09	-290.4275	46946.	0.00
13.5240	0.01485	-293077.	14319.	9.17E-05	23460.	9.44E+09	-300.8080	47636.	0.00
13.7200	0.01498	-260287.	13603.	2.28E-05	22720.	9.44E+09	-307.8364	48327.	0.00
13.9160	0.01496	-229134.	12875.	-3.82E-05	22017.	9.44E+09	-311.7617	49017.	0.00
14.1120	0.01480	-199647.	12140.	-9.16E-05	21351.	9.44E+09	-312.8356	49707.	0.00
14.3080	0.01453	-171839.	11406.	-1.38E-04	20724.	9.44E+09	-311.3101	50398.	0.00
14.5040	0.01415	-145710.	10679.	-1.77E-04	20134.	9.44E+09	-307.4362	51088.	0.00
14.7000	0.01369	-121244.	9962.	-2.11E-04	19582.	9.44E+09	-301.4623	51779.	0.00
14.8960	0.01316	-98415.	9263.	-2.38E-04	19066.	9.44E+09	-293.6332	52469.	0.00
15.0920	0.01257	-77185.	8583.	-2.60E-04	18587.	9.44E+09	-284.1885	53159.	0.00
15.2880	0.01194	-57508.	7927.	-2.77E-04	18143.	9.44E+09	-273.3625	53850.	0.00
15.4840	0.01127	-39329.	7192.	-2.89E-04	17733.	9.44E+09	-351.7339	73393.	0.00
15.6800	0.01058	-23085.	6378.	-2.97E-04	17366.	9.44E+09	-340.7850	75751.	0.00
15.8760	0.00988	-8720.	5590.	-3.01E-04	17042.	9.44E+09	-329.2471	78405.	0.00
16.0720	0.00917	3826.	4830.	-3.01E-04	16931.	9.44E+09	-317.2015	81382.	0.00
16.2680	0.00846	14615.	4098.	-2.99E-04	17175.	9.44E+09	-304.7205	84715.	0.00
16.4640	0.00776	23716.	3397.	-2.94E-04	17380.	9.44E+09	-291.8671	88446.	0.00
16.6600	0.00708	31196.	2726.	-2.87E-04	17549.	9.44E+09	-278.6949	92626.	0.00
16.8560	0.00641	37126.	2086.	-2.79E-04	17683.	9.44E+09	-265.2470	97322.	0.00
17.0520	0.00577	41579.	1478.	-2.69E-04	17783.	9.44E+09	-251.5550	102618.	0.00
17.2480	0.00515	44630.	903.0689	-2.58E-04	17852.	9.44E+09	-237.6370	108628.	0.00
17.4440	0.00455	46355.	360.7778	-2.47E-04	17891.	9.44E+09	-223.4948	115501.	0.00
17.6400	0.00398	46832.	-147.9647	-2.35E-04	17902.	9.44E+09	-209.1094	123447.	0.00
17.8360	0.00344	46141.	-622.5324	-2.24E-04	17886.	9.44E+09	-194.4346	132763.	0.00
18.0320	0.00293	44362.	-1062.	-2.12E-04	17846.	9.44E+09	-179.3870	143899.	0.00
18.2280	0.00245	41579.	-1466.	-2.02E-04	17783.	9.44E+09	-163.8292	157563.	0.00
18.4240	0.00198	37879.	-1832.	-1.92E-04	17700.	9.44E+09	-147.5391	174958.	0.00

18.6200	0.00154	33354.	-2159.	-1.83E-04	17598.	9.44E+09	-130.1508	198331.	0.00
18.8160	0.00112	28100.	-2442.	-1.75E-04	17479.	9.44E+09	-111.0192	232505.	0.00
19.0120	7.19E-04	22225.	-2789.	-1.69E-04	17347.	9.44E+09	-183.7715	601042.	0.00
19.2080	3.28E-04	15327.	-3183.	-1.64E-04	17191.	9.44E+09	-151.0403	1082292.	0.00
19.4040	-5.37E-05	7590.	-3329.	-1.61E-04	17016.	9.44E+09	26.8404	1176000.	0.00
19.6000	-4.31E-04	0.00	0.00	-1.60E-04	16845.	9.44E+09	2804.	7647071.	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 = -0.97500000 inches
Computed slope at pile head = 0.00634291 radians
Maximum bending moment = 1772170. inch-lbs
Maximum shear force = -42640. 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 = 16
Number of zero deflection points = 2

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	-0.9750	S, rad	0.00	435000.	-0.9750	0.00	-56398.	2961638.
2	y, in	-0.9750	M, in-lb	1772170.	435000.	-0.9750	0.00634	-42640.	1772170.

Maximum pile-head deflection = -0.9750000000 inches
Maximum pile-head rotation = 0.0063429137 radians = 0.363422 deg.

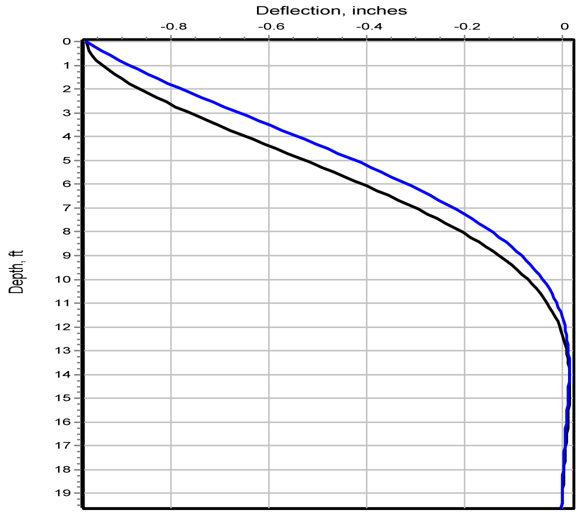
Summary of Warning Messages

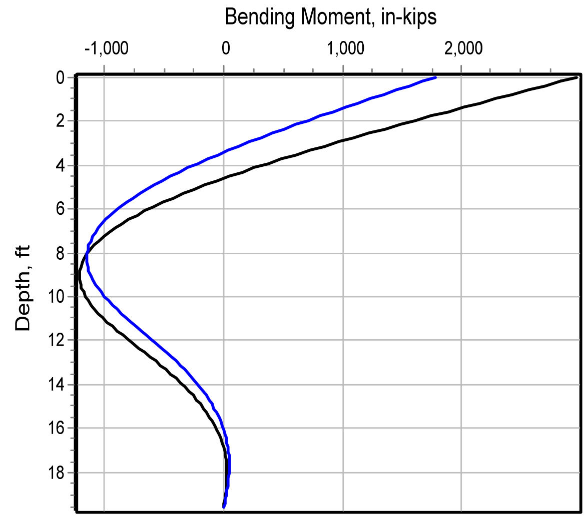
The following warning was reported 117 times

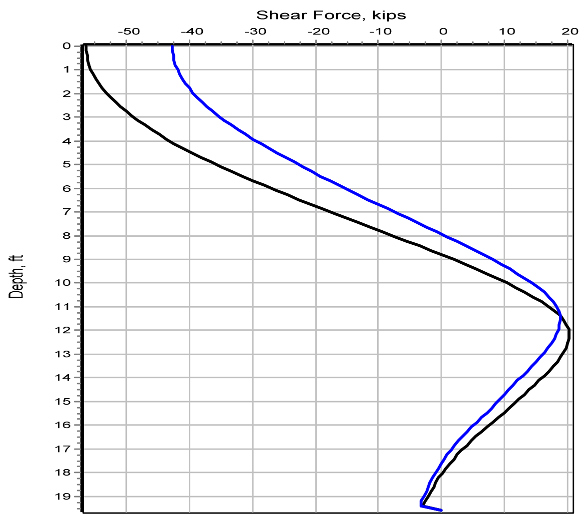
**** Warning ****

This warning is for an input value for uniaxial compressive strength that has been specified for a soil defined using the vuggy limestone criteria. The input value is outside of the range of 1,000 to 2,500 psi (6,895 to 17,237 kPa) which were used in actual field tests on which this theory is based. Higher or lower values may be applicable but the user is warned about the theoretical and testing limitations.

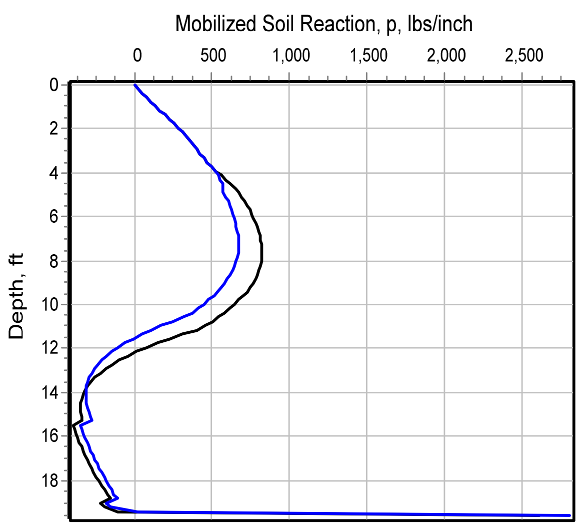
The analysis ended normally.







Legend:
----- First Iteration Load Case
(with axial load and lateral deflection applied to pile head)
----- Second Iteration Load Case
(with axial load, lateral deflection, and plastic hinge moment applied to pile head)



Attachment 3

GOLDER MEMBER OF WSP	**SUBJECT:** MaineDOT Desert Road Bridge 5720 (Exit 20)		
	Job No.: 21450908	**Prepared:**	KAR
	Location: Freeport, Maine	**Checked:**	AH
	Date: 6/28/2021	**Reviewed:**	JEL

Pile Design at Proposed Abutment 2 - Downdrag Analysis

Description:

Evaluate the downdrag load for Abutment 2 driven piles using APILE

References:

1 Golder Associates Inc.; "Preliminary Geotechnical Design Report - I-295 Desert Road Bridge Replacement"; December 21, 2020

2 AASHTO; "AASHTO LRFD Bridge Design Specifications - 9th Edition", 2020

3 FHWA; Design and Construction of Driven Pile Foundations - Volume 1; FHWA GEC 012; FHWA-NHI-16-009; July 2016

4 FHWA; Design and Construction of Driven Pile Foundations - Volume 2; FHWA GEC 012; FHWA-NHI-16-009; July 2016

5 FHWA; Design and Construction of Driven Pile Foundations - Comprehensive Design Examples; FHWA GEC 012; FHWA-NHI-16-064; September 2016

6 Wyllie, DA; Foundations on Rock, 2nd Edition; E&FN Spon; 1999

7 Siegel, TC et al; "Alternative Design Approach for Drag Load and Downdrag of Deep Foundations within the LRFD Framework"; Proceedings 38th Deep Foundations; 2013

8 Geotechnical Design Manual, Chapter 8 - Foundations, Oregon Department of Transportation, Geo-Environmental Section, Version 2.1, May 6, 2019.

9 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.

10 Golder settlement model created using Rocscience Settle3 software package, Version 5.010 64-bit, build date Mar 5, 2021

11 HNTB. May 21, 2021. Merrill Road Bridge, Interstate 295: 60% Plans.

12 HNTB. May 26, 2021. Freeport Bridges_Loads_Bottom of Footing_flat.pdf.

13 Golder's Phase II Updated Interpreted Subsurface Profile using HNTB design references.

Assumptions:

1. Settlement greater than or equal to 0.4" is needed for downdrag to fully develop (Ref 2).
2. The soil profile analyzed (Ref 1) is the interpreted profile where maximum settlement occurs along the abutment.
3. Any downdrag load that may develop along the back of the abutment due to settlement is not included in the pile downdrag analysis.
4. The FHWA automated computation method provided in APILE is used for the software computations of unit load transfers and axial pile

Calculations:

				Top Elev (ft)	Bot. Elev (ft)	Thickness (ft)
Granular Backfill Unit Weight:	γ	125	pcf	171.0	159.3	11.7
Granular Backfill Friction Angle:	φ	32	deg			
Passive Earth Pressure Coefficient	K_p	3.93				
Active Earth Pressure Coefficient	K_a	0.31				

Strength I Loads

Strength I Factored Vertical Load per pile (kips) = 357 No. piles = 9 (Ref. 12)

Starting elevation of the pile	-	159.3	ft	Ref. 11
Ending elevation of the pile	-	139.7	ft	Ref. 13
Box perimeter of pile	P:	57.05	in	for HP14x89
Segment Length:	L:	6	in	
Cross-sectional Area:	A_s:	26.1	in^2	for HP14x89
Elastic Modulus of Pile	E:	29000	ksi	
Nominal Weight of Pile		0.089	kip/ft	
Factored Pile Strength = ϕFyAs	P_r:	652.5	kips	Ref 3 Eq 8-35
	Fy :	50	ksi	Ref 3 Table 8-2
	ϕ :	0.5	-	Ref 2 Article 6.5.4.2 for axial resistance of H-piles in compression and subject to severe driving conditions

Non-Cohesive Soil Layers -Nordlund/Thurman Method

	Parameters					
Soil Layer	**ϕ_f (deg)**	**γ (pcf)**	**γ_{pDD}**	Top Elev (ft)	Bot. Elev (ft)	Thickness (ft)
Soil 1	32	125	1.1	159.3	144.0	15.3

Fill, Ref. 1 and Ref. 13

ϕ_f: Based on empirical correlation to avg of N_{60} values encountered in all borings for layer

γ: Unit Weight

γ_{pDD}: STR 1 Load Factor for Downdrag (Ref 1 and Ref 8)

Cohesive Soil Layers - Alpha Method

GOLDER MEMBER OF WSP	SUBJECT: MaineDOT Desert Road Bridge 5720 (Exit 20)		
	Job No.: 21450908	**Prepared:**	KAR
	Location: Freeport, Maine	**Checked:**	AH
	Date: 6/28/2021	**Reviewed:**	JEL

Pile Depth	D_b:	4.3 ft	Ref 2, C10.7.3.8.6b
Pile Width	D:	1.15 ft	
Ratio D_b/D	D_b:	4 D	

	Parameters								
Soil Layer	S_u (ksf)	γ (pcf)	α	q_s (ksf)	γ_{pDD}	Top Elev (ft)	Bot. Elev (ft)	Thickness (ft)	
Soil 2	1.60	125	1.0	1.60	1.4	144.0	139.7	4.3	Glaciomarine clay, Ref. 1 and Ref. 13

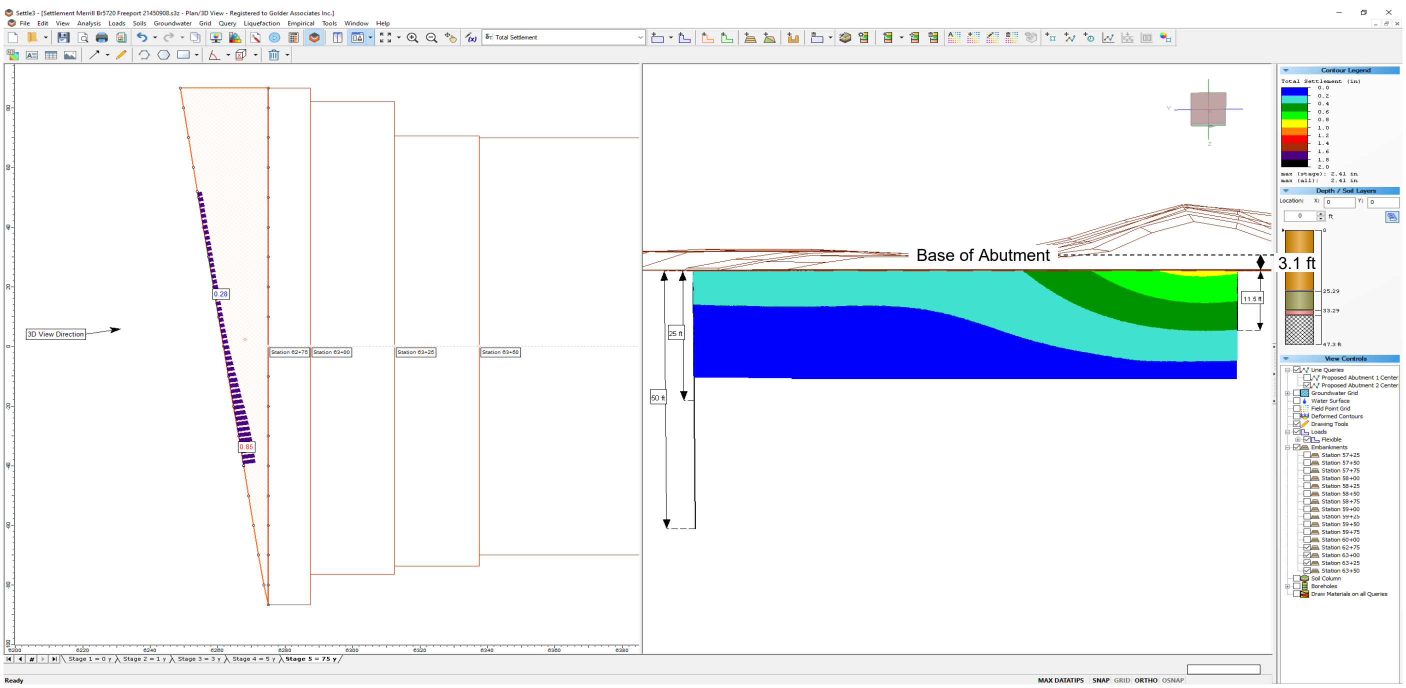
1. Identify deepest depth below the ground surface along the abutment centerline where settlement less than or equal to 0.4 inches.

Ref. 2 Article 3.11.8 indicates that full downdrag loading occurs where settlement is equal to or greater than 0.4 inches.

From the Settle3 model (Ref 10) image below, maximum depth to the 0.4" or less settlement contour below base of abutment (ft): **14.6**

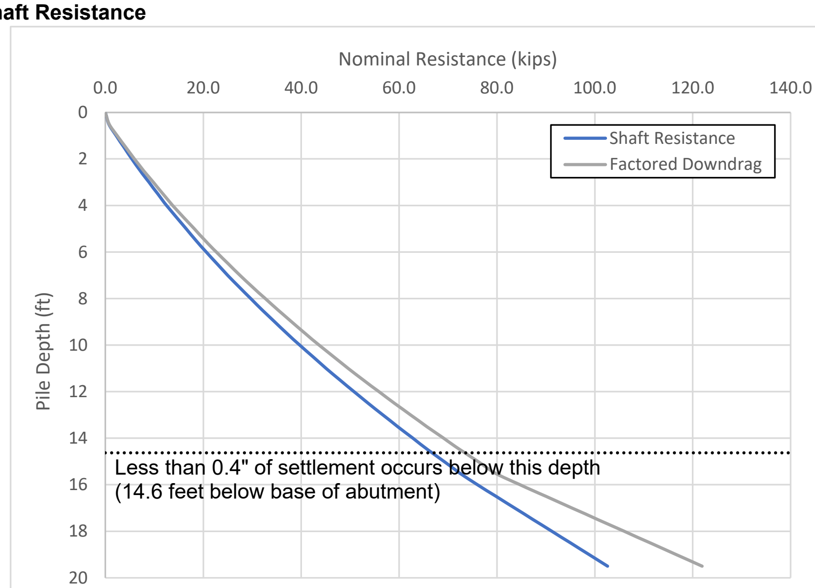
Thus, downdrag loading below this elevation is not considered fully developed and is not included in this analysis.

Settle3 Model from Abutment 2 loading.



2. Determine downdrag loading using APILE (Ensoft) Shaft Resistance

Pile Depth Below Pile Cap	Type of Soil	Shaft Resistance	Factored Downdrag Load	Total Factored Axial Load w/ Down Drag & Pile Weight
(feet)		(kips)	(kips)	(kips)
0	1	0.0	0	357.3
0.5	1	0.7	1	358.1
1	1	2.2	2	359.8
1.5	1	3.8	4	361.6
2	1	5.4	6	363.5
2.5	1	7.1	8	365.4
3	1	8.9	10	367.4
3.5	1	10.7	12	369.4
4	1	12.5	14	371.4
4.5	1	14.5	16	373.7
5	1	16.5	18	375.9
5.5	1	18.5	20	378.2



6	1	20.6	23	380.5
6.5	1	22.8	25	383.0
7	1	25.0	28	385.5
7.5	1	27.3	30	388.0
8	1	29.7	33	390.7
8.5	1	32.1	35	393.4
9	1	34.6	38	396.2
9.5	1	37.1	41	399.0
10	1	39.7	44	401.9
10.5	1	42.4	47	404.9
11	1	45.1	50	407.9
11.5	1	47.9	53	411.0
12	1	50.8	56	414.3
12.5	1	53.7	59	417.5
13	1	56.7	62	420.9
13.5	1	59.7	66	424.2
14	1	62.8	69	427.7
14.5	1	65.9	72	431.1
15	**1**	**69.2**	**76**	**434.8**
15.5	1	72.4	80	438.4
16	2	76.0	85	443.4
16.5	2	79.8	90	448.8
17	2	83.6	95	454.2
17.5	2	87.4	101	459.5
18	2	91.2	106	464.9
18.5	2	95.0	111	470.3
19	2	98.8	117	475.6
19.5	2	102.6	122	481.0

Type of Soil	g_{pDD}	Downdrag Load Factor
1	1.1	Strength I Load Factor for Down Drag (Ref 1, Ref 8 Table 8.2)
2	1.4	
1 - 3	1.0	Service and Extreme Load Factor for Down Drag (Ref 2, Ref 8)

Total Factored Downdrag Load, Strength I Limit State	**76 kips**	(per pile)
	685 kips	(per abutment - 9 piles/abutment)
Total Factored Downdrag Load, Extreme & Service Limit States	**69 kips**	(per pile)
	623 kips	(per abutment - 9 piles/abutment)

Conclusions:

Based on the Settle3 model, downdrag is estimated to develop along the upper 14.6 feet of the pile. A total factored downdrag load of 76 kips per pile was calculated for the Strength I load case and a total factored downdrag load of 69 kips per pile was calculated for the Extreme and Service limit load cases. The total factored downdrag load will be conservatively applied to the top of the pile in the lateral response analysis.

==

APILE for Windows, Version 2019.9.3

Serial Number : 156241926

A Program for Analyzing the Axial Capacity
and Short-term Settlement of Driven Piles
under Axial Loading.

==

This program is licensed to :

Golder Associates
Houston, Texas

Path to file locations : C:\Users\MGore\Golder Associates\21450908 MaineDOT Desert Rd Bridge 5720 p2 Freeport - 5 Technical Work\06 Analysis\Phase II Pile Design\Downdrag\HP14x89\
Name of input data file : HP14x89_Abutment 2_MSG.ap9d
Name of output file : HP14x89_Abutment 2_MSG.ap9o
Name of plot output file : HP14x89_Abutment 2_MSG.ap9p

Time and Date of Analysis

Date: June 28, 2021 Time: 14:09:57

1

****
* INPUT INFORMATION *
****

New Pile

DESIGNER :

JOB NUMBER :

METHOD FOR UNIT LOAD TRANSFERS :

- FHWA (Federal Highway Administration)
  Unfactored Unit Side Friction and Unit Side Resistance are used.

COMPUTATION METHOD(S) FOR PILE CAPACITY :

- FHWA (Federal Highway Administration)

TYPE OF LOADING :
- COMPRESSION

PILE TYPE :

H-Pile/Steel Pile

DATA FOR AXIAL STIFFNESS :

- MODULUS OF ELASTICITY = 0.290E+08 PSI
- CROSS SECTION AREA = 26.10 IN2

NONCIRCULAR PILE PROPERTIES :

- TOTAL PILE LENGTH, TL = 31.30 FT.
- BATTER ANGLE = 0.00 DEG
- PILE STICKUP LENGTH, PSL = 0.00 FT.
- ZERO FRICTION LENGTH, ZFL = 11.70 FT.
- PERIMETER OF PILE = 57.05 IN.
- TIP AREA OF PILE = 26.10 IN2
- INCREMENT OF PILE LENGTH
  USED IN COMPUTATION = 0.50 FT.

SOIL INFORMATIONS :

DEPTH FT.	SOIL TYPE	LATERAL EARTH PRESSURE	EFFECTIVE UNIT WEIGHT LB/FT^3	FRICTION ANGLE DEGREES	BEARING CAPACITY FACTOR
0.00	SAND	0.80*	125.00	32.00	28.00**
11.70	SAND	0.80*	125.00	32.00	28.00**
11.70	SAND	0.80*	125.00	32.00	28.00**
27.00	SAND	0.80*	125.00	32.00	28.00**
27.00	CLAY	0.80*	125.00	0.00	8.00**
30.70	CLAY	0.80*	125.00	0.00	8.00**
30.70	CLAY	0.80*	62.60	0.00	8.00**
31.30	CLAY	0.80*	62.60	0.00	8.00**
31.30	SAND	0.80*	101.60	50.00	50.00**
40.00	SAND	0.80*	101.60	50.00	50.00**

* VALUE ASSUMED BY THE PROGRAM
** VALUE ESTIMATED BY THE PROGRAM BASED ON FRICTION ANGLE

MAXIMUM UNIT FRICTION KSF	MAXIMUM UNIT BEARING KSF	UNDISTURB SHEAR STRENGTH KSF	REMOLDED SHEAR STRENGTH KSF	BLOW COUNT	UNIT SKIN FRICTION KSF	UNIT END BEARING KSF
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	1.60	1.60	0.00	0.00	0.00
0.10E+08*	0.10E+08*	1.60	1.60	0.00	0.00	0.00
0.10E+08*	0.10E+08*	1.60	1.60	0.00	0.00	0.00
0.10E+08*	0.10E+08*	1.60	1.60	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00

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

DEPTH FT.	LRFD FACTOR ON UNIT FRICTION	LRFD FACTOR ON UNIT BEARING
0.00	1.000	1.000
11.70	1.000	1.000
11.70	1.000	1.000
27.00	1.000	1.000
27.00	1.000	1.000
30.70	1.000	1.000
30.70	1.000	1.000
31.30	1.000	1.000
31.30	1.000	1.000
40.00	1.000	1.000

DEPTH FT.	PLASTIC INDEX PI %	YIELD STRESS RATIO	Qc FROM CPT KSF
0.00	0.00	0.00	0.000E+00
11.70	0.00	0.00	0.000E+00
11.70	0.00	0.00	0.000E+00
27.00	0.00	0.00	0.000E+00
27.00	0.00	0.00	0.000E+00
30.70	0.00	0.00	0.000E+00

30.70	0.00	0.00	0.000E+00
31.30	0.00	0.00	0.000E+00
31.30	0.00	0.00	0.000E+00
40.00	0.00	0.00	0.000E+00

1

****
* COMPUTATION RESULT *
****

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

PILE PENETRATION FT.	TOTAL SKIN FRICTION KIP	END BEARING KIP	ULTIMATE CAPACITY KIP
0.00	0.0	0.3	0.3
0.50	0.0	0.5	0.5
1.00	0.0	0.7	0.7
1.50	0.0	0.9	0.9
2.00	0.0	1.1	1.1
2.50	0.0	1.4	1.4
3.00	0.0	1.7	1.7
3.50	0.0	2.0	2.0
4.00	0.0	2.3	2.3
4.50	0.0	2.6	2.6
5.00	0.0	2.9	2.9
5.50	0.0	3.1	3.1
6.00	0.0	3.4	3.4
6.50	0.0	3.7	3.7
7.00	0.0	4.0	4.0
7.50	0.0	4.3	4.3
8.00	0.0	4.6	4.6
8.50	0.0	4.9	4.9
9.00	0.0	5.1	5.1
9.50	0.0	5.3	5.3
10.00	0.0	5.5	5.5
10.50	0.0	5.7	5.7
11.00	0.0	5.8	5.8
11.50	0.0	5.9	5.9
12.00	0.7	5.9	6.7
12.50	2.2	6.0	8.2
13.00	3.8	6.0	9.8
13.50	5.4	6.0	11.4
14.00	7.1	6.0	13.1
14.50	8.9	6.0	14.8
15.00	10.7	6.0	16.6
15.50	12.5	6.0	18.5
16.00	14.5	6.0	20.4

16.50	16.5	6.0	22.4
17.00	18.5	6.0	24.5
17.50	20.6	6.0	26.6
18.00	22.8	6.0	28.8
18.50	25.0	6.0	31.0
19.00	27.3	6.0	33.3
19.50	29.7	6.0	35.7
20.00	32.1	6.0	38.1
20.50	34.6	6.0	40.6
21.00	37.1	6.0	43.1
21.50	39.7	6.0	45.7
22.00	42.4	6.0	48.4
22.50	45.1	6.0	51.1
23.00	47.9	6.0	53.9
23.50	50.8	6.0	56.8
24.00	53.7	6.0	59.7
24.50	56.7	6.0	62.6
25.00	59.7	5.8	65.5
25.50	62.8	5.4	68.2
26.00	65.9	5.0	71.0
26.50	69.2	4.7	73.8
27.00	72.4	4.3	76.7
27.50	76.0	3.9	79.9
28.00	79.8	3.6	83.4
28.50	83.6	3.2	86.8
29.00	87.4	2.8	90.2
29.50	91.2	10.4	101.6
30.00	95.0	24.8	119.9
30.50	98.8	39.3	138.1
31.00	102.6	53.7	156.3

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

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

T-Z CURVE NO.	NO. OF POINTS	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
1	10	0.0000E+00		
			0.0000E+00	0.0000E+00
			0.0000E+00	0.2906E-01
			0.0000E+00	0.5629E-01
			0.0000E+00	0.1035E+00
			0.0000E+00	0.1453E+00

			0.0000E+00	0.1816E+00
			0.0000E+00	0.3632E+00
			0.0000E+00	0.5448E+00
			0.0000E+00	0.9080E+00
			0.0000E+00	0.3632E+01
2	10	0.5875E+01		
			0.0000E+00	0.0000E+00
			0.0000E+00	0.2906E-01
			0.0000E+00	0.5629E-01
			0.0000E+00	0.1035E+00
			0.0000E+00	0.1453E+00
			0.0000E+00	0.1816E+00
			0.0000E+00	0.3632E+00
			0.0000E+00	0.5448E+00
			0.0000E+00	0.9080E+00
			0.0000E+00	0.3632E+01
3	10	0.1166E+02		
			0.0000E+00	0.0000E+00
			0.4082E+00	0.2906E-01
			0.6804E+00	0.5629E-01
			0.1021E+01	0.1035E+00
			0.1225E+01	0.1453E+00
			0.1361E+01	0.1816E+00
			0.1361E+01	0.3632E+00
			0.1361E+01	0.5448E+00
			0.1361E+01	0.9080E+00
			0.1361E+01	0.3632E+01
4	10	0.1170E+02		
			0.0000E+00	0.0000E+00
			0.5157E+00	0.2906E-01
			0.8594E+00	0.5629E-01
			0.1289E+01	0.1035E+00
			0.1547E+01	0.1453E+00
			0.1719E+01	0.1816E+00
			0.1719E+01	0.3632E+00
			0.1719E+01	0.5448E+00
			0.1719E+01	0.9080E+00
			0.1719E+01	0.3632E+01
5	10	0.1938E+02		
			0.0000E+00	0.0000E+00
			0.2081E+01	0.2906E-01
			0.3469E+01	0.5629E-01
			0.5204E+01	0.1035E+00
			0.6244E+01	0.1453E+00
			0.6938E+01	0.1816E+00
			0.6938E+01	0.3632E+00
			0.6938E+01	0.5448E+00
			0.6938E+01	0.9080E+00
			0.6938E+01	0.3632E+01
6	10	0.2696E+02		
			0.0000E+00	0.0000E+00
			0.2896E+01	0.2906E-01
			0.4827E+01	0.5629E-01
			0.7240E+01	0.1035E+00

			0.8688E+01	0.1453E+00
			0.9654E+01	0.1816E+00
			0.9654E+01	0.3632E+00
			0.9654E+01	0.5448E+00
			0.9654E+01	0.9080E+00
			0.9654E+01	0.3632E+01
7	10	0.2700E+02		
			0.0000E+00	0.0000E+00
			0.2901E+01	0.2906E-01
			0.4834E+01	0.5629E-01
			0.7252E+01	0.1035E+00
			0.8702E+01	0.1453E+00
			0.9669E+01	0.1816E+00
			0.8702E+01	0.3632E+00
			0.8702E+01	0.5448E+00
			0.8702E+01	0.9080E+00
			0.8702E+01	0.3632E+01
8	10	0.2888E+02		
			0.0000E+00	0.0000E+00
			0.3333E+01	0.2906E-01
			0.5556E+01	0.5629E-01
			0.8333E+01	0.1035E+00
			0.1000E+02	0.1453E+00
			0.1111E+02	0.1816E+00
			0.1000E+02	0.3632E+00
			0.1000E+02	0.5448E+00
			0.1000E+02	0.9080E+00
			0.1000E+02	0.3632E+01
9	10	0.3066E+02		
			0.0000E+00	0.0000E+00
			0.3333E+01	0.2906E-01
			0.5556E+01	0.5629E-01
			0.8333E+01	0.1035E+00
			0.1000E+02	0.1453E+00
			0.1111E+02	0.1816E+00
			0.1000E+02	0.3632E+00
			0.1000E+02	0.5448E+00
			0.1000E+02	0.9080E+00
			0.1000E+02	0.3632E+01
10	10	0.3070E+02		
			0.0000E+00	0.0000E+00
			0.3333E+01	0.2906E-01
			0.5556E+01	0.5629E-01
			0.8333E+01	0.1035E+00
			0.1000E+02	0.1453E+00
			0.1111E+02	0.1816E+00
			0.1000E+02	0.3632E+00
			0.1000E+02	0.5448E+00
			0.1000E+02	0.9080E+00
			0.1000E+02	0.3632E+01
11	10	0.3103E+02		
			0.0000E+00	0.0000E+00
			0.3333E+01	0.2906E-01
			0.5556E+01	0.5629E-01

			0.8333E+01	0.1035E+00
			0.1000E+02	0.1453E+00
			0.1111E+02	0.1816E+00
			0.1000E+02	0.3632E+00
			0.1000E+02	0.5448E+00
			0.1000E+02	0.9080E+00
			0.1000E+02	0.3632E+01
12	10	0.3126E+02		
			0.0000E+00	0.0000E+00
			0.3333E+01	0.2906E-01
			0.5556E+01	0.5629E-01
			0.8333E+01	0.1035E+00
			0.1000E+02	0.1453E+00
			0.1111E+02	0.1816E+00
			0.1000E+02	0.3632E+00
			0.1000E+02	0.5448E+00
			0.1000E+02	0.9080E+00
			0.1000E+02	0.3632E+01
13	10	0.3130E+02		
			0.0000E+00	0.0000E+00
			0.3333E+01	0.2906E-01
			0.5556E+01	0.5629E-01
			0.8333E+01	0.1035E+00
			0.1000E+02	0.1453E+00
			0.1111E+02	0.1816E+00
			0.1111E+02	0.3632E+00
			0.1111E+02	0.5448E+00
			0.1111E+02	0.9080E+00
			0.1111E+02	0.3632E+01
14	10	0.3568E+02		
			0.0000E+00	0.0000E+00
			0.3333E+01	0.2906E-01
			0.5556E+01	0.5629E-01
			0.8333E+01	0.1035E+00
			0.1000E+02	0.1453E+00
			0.1111E+02	0.1816E+00
			0.1111E+02	0.3632E+00
			0.1111E+02	0.5448E+00
			0.1111E+02	0.9080E+00
			0.1111E+02	0.3632E+01
15	10	0.3996E+02		
			0.0000E+00	0.0000E+00
			0.3333E+01	0.2906E-01
			0.5556E+01	0.5629E-01
			0.8333E+01	0.1035E+00
			0.1000E+02	0.1453E+00
			0.1111E+02	0.1816E+00
			0.1111E+02	0.3632E+00
			0.1111E+02	0.5448E+00
			0.1111E+02	0.9080E+00
			0.1111E+02	0.3632E+01

TIP LOAD KIP	TIP MOVEMENT IN.
0.0000E+00	0.0000E+00
0.3357E+01	0.9080E-02
0.6714E+01	0.1816E-01
0.1343E+02	0.3632E-01
0.2686E+02	0.2361E+00
0.4029E+02	0.7627E+00
0.4834E+02	0.1326E+01
0.5371E+02	0.1816E+01
0.5371E+02	0.2724E+01
0.5371E+02	0.3632E+01

LOAD VERSUS SETTLEMENT CURVE
****

TOP LOAD KIP	TOP MOVEMENT IN.	TIP LOAD KIP	TIP MOVEMENT IN.
0.1554E+00	0.1610E-03	0.3697E-01	0.1000E-03
0.1554E+01	0.1610E-02	0.3697E+00	0.1000E-02
0.7771E+01	0.8052E-02	0.1849E+01	0.5000E-02
0.1554E+02	0.1610E-01	0.3697E+01	0.1000E-01
0.3111E+02	0.3221E-01	0.7395E+01	0.2000E-01
0.6477E+02	0.7541E-01	0.1435E+02	0.5000E-01
0.8462E+02	0.1130E+00	0.1636E+02	0.8000E-01
0.9680E+02	0.1376E+00	0.1771E+02	0.1000E+00
0.1285E+03	0.2503E+00	0.2443E+02	0.2000E+00
0.1347E+03	0.5535E+00	0.3359E+02	0.5000E+00
0.1420E+03	0.8571E+00	0.4082E+02	0.8000E+00
0.1448E+03	0.1058E+01	0.4368E+02	0.1000E+01
0.1549E+03	0.2063E+01	0.5371E+02	0.2000E+01

Attachment 4

GOLDER
MEMBER OF WSP

CALCULATIONS

Date:	6/29/2021	**Made by:**	KAR
Project No.:	21450908	**Checked by:**	DAF
Subject:	Pile Driveability at Abutment 2	**Reviewed by:**	JEL
Project Title:	MaineDOT Desert Road Bridge 5720 Freeport (Exit 20) Phase 2		

OBJECTIVE

Perform a driveability analysis using GRLWEAP to determine if the proposed piles can be driven to the nominal pile driving resistance at Abutment 2 while maintaining blow counts and pile stresses within the specified limits.

REFERENCES

1. GRLWEAP Software Package Version 2010-8, Built November 28, 2018.
2. AASHTO LRFD Bridge Design Specifications, 9th Edition, 2020
3. MaineDOT 2020 Standard Specifications Section 501.042
4. Golder updated Phase II interpreted subsurface profiles A-A' and B-B', dated June 2021.
5. HNTB calculation titled "Freeport Bridges_Loads_Bottom of Footing_flat.pdf", dated May 26, 2021.

ASSUMPTIONS

1. The pile will be bearing on bedrock with 90% tip resistance, 10% shaft resistance.
2. The number of hammer blows at the required resistance indicated by the wave equation analysis will be between 3 and 15 blows per inch (Ref. 3)
3. The factored axial load is 435 kips including downdrag for Abutment 2 with HP 14x89 piles.
4. The hammer cushion will consist of 50% aluminum and 50% conbest, 2" thick.
5. The HP 14x89 piles will be driven a total length of approximately 20 feet.
6. A resistance factor of 0.65 will be used for a driving criteria established by dynamic testing (Ref. 2).
7. A Delmag D30 single acting diesel pile driving hammer is assumed for the analysis as it is a common pile driving hammer used on MaineDOT projects.

ATTACHMENTS

1. GRLWEAP table of recommended quake and damping values for impact driven piles.
2. GRLWEAP output for a variable resistance analysis using the DELMAG D30 hammer.

CALCULATION

1. Determine the input parameters:

Nominal pile driving resistance:

$P_u =$ 435 Applied axial load including downdrag

$\phi_{mon} =$ 0.65 Resistance factor associated with pile monitoring method

$$R_{ndr} = \frac{P_u}{\phi_{mon}}$$

Abutment 2: $R_{ndr} =$ 669 kips

Date:	6/29/2021	**Made by:**	KAR
Project No.:	21450908	**Checked by:**	DAF
Subject:	Pile Driveability at Abutment 2	**Reviewed by:**	JEL
Project Title:	MaineDOT Desert Road Bridge 5720 Freeport (Exit 20) Phase 2		

Soil layering:

Abutment 2

Layer (Ref. 4)	Depth below base of abutment
Existing Fill	0.0 - 15.3 ft
Glaciomarine Silty Clay (above water table)	15.3 - 19.0 ft
Glaciomarine Silty Clay (below water table)	19.0 - 19.6 ft
Bedrock	> 19.6 ft

Shaft quake = 0.1 in   Shaft damping = 0.1 s/ft
Toe quake = 0.04 in   Toe damping = 0.15 s/ft   (Attachment 1)

2. Analyze an open ended diesel (OED) hammer to determine the blow count and stress at R_{ndr} for the abutment.

Hammer	Energy / Power (lb-ft)	Fuel Setting	Location	R_{ndr} (kips)	Blows/ft	Blows/in	Stress (ksi)
DELMAG D 30	59,730	Fuel Setting 3 (81%)	Abutment 2	669	91.2	7.6	43.2

CONCLUSIONS

The analysis indicates the Delmag D30 hammer operated at fuel setting 3 (81% of the combustion pressure) should be able to drive the piles at Abutment 2 to the nominal structural pile resistance while staying at a blow count between 3 and 15 blows per inch and limiting the driving stresses to below 45 kips per square inch (ksi), in accordance with Section 501.042 of the MaineDOT 2020 Standard Specifications.

GOLDER
MEMBER OF WSP

CALCULATIONS

Date:	6/29/2021	**Made by:**	KAR
Project No.:	21450908	**Checked by:**	DAF
Subject:	Pile Driveability at Abutment 2	**Reviewed by:**	JEL
Project Title:	MaineDOT Desert Road Bridge 5720 Freeport (Exit 20) Phase 2		

Attachment 1

Recommended Quake Values for Impact Driven Piles*

	Soil Type	Pile Type or Size	Quake (in) Quake (mm)
Shaft Quake	All soil types	All Types	0.10 2.5
Toe Quake	All soil types, soft Rock	Non-displacement piles** i.e. driving unplugged	0.10 2.5
	Very dense or hard soils	Displacement Piles*** of diameter or width D	D/120 D/120
	Soils which are not very dense or hard	Displacement Piles*** of diameter or width D	D/60 D/60
	Hard Rock	All Types	0.04 1.0

*For vibratory driven piles in cohesive soils, quakes should be doubled.
** Non-displacement piles are sheet pile, H-Piles, or open-ended pipe piles which are not plugging during driving. Normally it can be assumed that pipe piles with diameters of 30 inches (900 mm) or more will not plug during driving while H-Piles and pipe piles of diameter 20 inches (500 mm) or less will plug during driving into a bearing layer. Between 20 and 30 inches (500 and 750 mm), pipe piles may or may not plug.
*** Displacement piles are closed-ended pipe piles, pipe piles, or H-Piles that are plugged during driving and solid concrete piles. Normally, we would analyze H-Piles and pipe piles with diameters 20 inches (500 mm) or less as displacement piles.

Recommended quake values, both shaft and toe, for vibratory driven piles are somewhat speculative recommendations as not much experience exists.

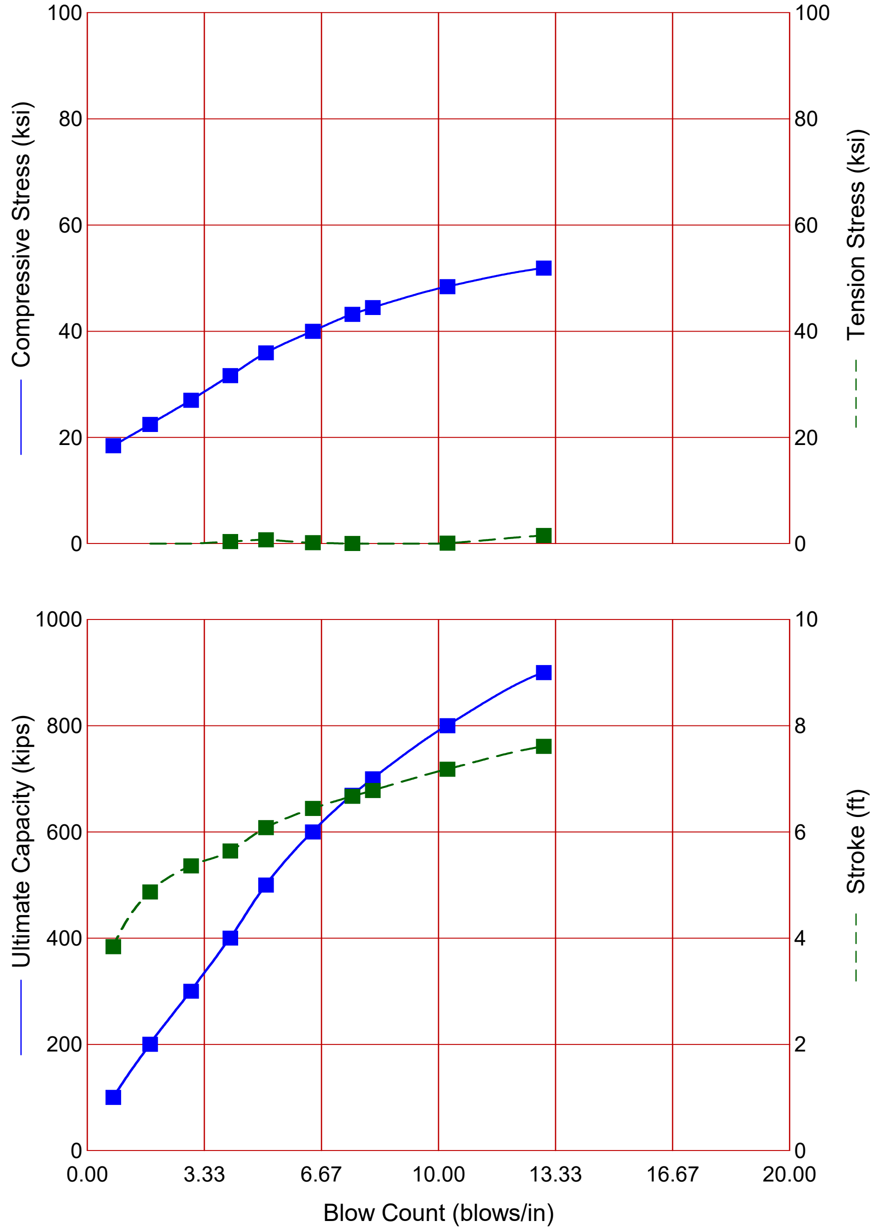
Recommended Damping Values for Impact Driven Piles*

	Soil Type	Damping Factor s/ft	Damping Factor s/m
Shaft Damping	Non-cohesive soils**	0.05	0.16
	Cohesive soils**	0.20	0.65
Toe damping	In all soil types	0.15	0.50

* For vibratory driven piles, use double values (Smith-viscous).
** For mixed soils, intermediate values may be appropriate; for example, a sandy silt or clayey sand may be modeled with 0.10 s/ft (0.33 s/m), a cohesive silt or a sandy clay with 0.15 s/ft (0.50 s/m).

Golder Associates Inc.
21450908 Freeport Exit 20 Abutment 2

29-Jun-2021
GRLWEAP Version 2010

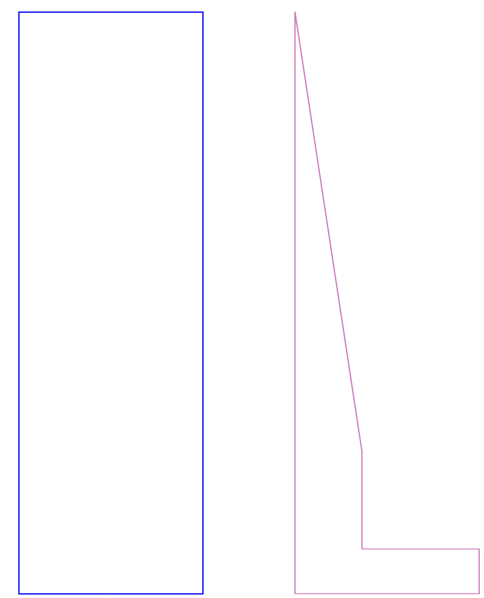


DELMAG D 30

Ram Weight	6.60	kips
Efficiency	0.800	
Pressure	1200 (81%)	psi
Helmet Weight	2.70	kips
Hammer Cushion	109975	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.100	s/ft
Toe Damping	0.150	s/ft
Pile Length	19.60	ft
Pile Penetration	19.60	ft
Pile Top Area	26.10	in2

Pile Model

Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
100.0	18.44	0.00	0.7	3.84	21.65
200.0	22.45	0.00	1.8	4.87	19.01
300.0	27.02	0.00	2.9	5.37	18.23
400.0	31.47	0.41	4.1	5.64	18.15
500.0	35.77	0.74	5.1	6.08	19.35
600.0	40.10	0.19	6.4	6.44	20.26
669.0	43.23	0.03	7.6	6.67	20.85
700.0	44.55	0.00	8.1	6.78	21.15
800.0	48.45	0.24	10.3	7.18	22.30
900.0	51.94	1.61	13.0	7.61	23.64

APPENDIX G

Pier Pile Design

CALCULATIONS

Date:	8/4/2021	**Made by:**	MLM
Project No.:	21450908	**Checked by:**	KAR
Subject:	Pile Design at Center Pier	**Reviewed by:**	JEL
Project Title:	MaineDOT I-295 Exit 20 Desert Road Bridge Replacement No. 5720		

OBJECTIVE
Determine pile design requirements at the proposed bridge pier. Perform a driveability analysis using GRLWEAP to determine if the proposed piles can be driven to the nominal pile driving resistance while maintaining blow counts and pile stresses within the specified limits.

METHOD
Use the procedure outlined in AASHTO LRFD (Ref. 1) and the design method provided in the MaineDOT Bridge Design Guide (Ref. 2).

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 9th Ed. 2020.
2. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.
3. Email communication between Golder and HNTB, subject "RE: Freeport I-295 Exit 20 and Exit 22 - Request for Sections and Design Info", dated June 22, 2020.
4. Isenhower, W.M. et al. LPile v2019 Technical Manual: A Program for the Analysis of Deep Foundations Under Lateral Loading. Ensoft, Inc. Dated March 2020.
5. Golder interpreted subsurface section A-A' (Figure 3, Preliminary Geotechnical Design Report, dated December 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. GRLWEAP Software Package Version 2010-8, Built November 28, 2018.
11. MaineDOT 2020 Standard Specifications Section 501.042
12. HNTB calculation titled "Freeport Bridges_Loads_Bottom of Footing_flat.pdf", dated May 26, 2021.
13. HNTB 98% PS&E Plans, dated July 30, 2021.

ASSUMPTIONS
1. The selected pile orientation is strong axis bending, due to expected greater resistance needed at the pier.
2. The vertical load is assumed to be evenly distributed. All thermal movement will be taken up by the abutments.
3. The spacing between the two rows of piles at the pier is assumed to be >5B and thus reduction due to pile group interaction is not necessary (Ref. 1, Article 10.7.2.4).
4. The pile will be bearing on bedrock with 90% tip resistance, 10% shaft resistance.
5. The number of hammer blows at the required resistance indicated by the wave equation analysis will be between 3 and 15 blows per inch (Ref. 11)
6. The hammer cushion will consist of 50% aluminum and 50% conbest, 2" thick.
7. The HP 14x89 piles will be driven a total length of approximately 12 feet.
9. A resistance factor of 0.65 will be used for a driving criteria established by dynamic testing (Ref. 1).
10. A Delmag D 30 single acting diesel pile driving hammer is assumed for the analysis as it is a common pile driving hammer used on MaineDOT projects.

CALCULATIONS

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CALCULATION

1. Select the preliminary pile size.

Determine the factored applied superstructure vertical dead and live load (P_u) distributed to each pile.

P_u = 414 kips (Ref. 12 & 13)

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).

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

$R_{n.upper}$ = 591 kips

$$R_{n.lower} = \frac{P_u}{\phi_{cl}}$$

$R_{n.lower}$ = 827.4 kips

$$R_n = \max\left(R_{n.upper}, R_{n.lower}\right)$$

R_n = 827.4 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)

$A_{s.req}$ = 20.7 in^2

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Select a pile size with an area of $A_{s.req}$ or greater.

Preferred selection is HP 14x89 based on June 16, 2020 meeting with MaineDOT and HNTB.
Check that preferred selection satisfies pile area requirement:

HP 14x89 A_s = 26.1 in^2 (Ref. 4, Table 5.6.3)
A_s > $A_{s.req}$ OK

2. Determine the nominal pile driving resistance.

While driving the pile, the maximum stress that is permitted in the pile is:

$$\sigma_{dr} = 0.9\Phi_{da}F_y$$ (Ref. 8, Appendix B, Eqn 7-22)

ϕ_{da} = 1.00 (Ref. 1, Article 6.5.4.2)

σ_{dr} = 45 ksi

Calculate the nominal pile driving resistance (R_{ndr}) from the applied load divided by the resistance factor associated with the pile monitoring method. The driving criteria will be established by dynamic testing.

ϕ_{mon} = 0.65 (Ref. 1, Table 10.5.5.2.3-1)

$$R_{ndr} = \frac{P_u}{\phi_{mon}}$$ (Ref. 8, Appendix B, Eqn 7-25)

R_{ndr} = 636 kips

3. Determine the input parameters:
Soil layering:

Layer (Refs. 5 & 6)	Depth below base of Pier
Glaciomarine Silty Clay	0 - 10 ft
Sand and Gravel	10 - 12 ft
Bedrock	>12 ft

Shaft quake = 0.1 in Shaft damping = 0.2 s/ft
Toe quake = 0.04 in Toe damping = 0.15 s/ft (Attachment 1)

CALCULATIONS

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2. Analyze an open ended diesel (OED) hammer to determine the blow count and stress at R_{ndr} for the abutment.

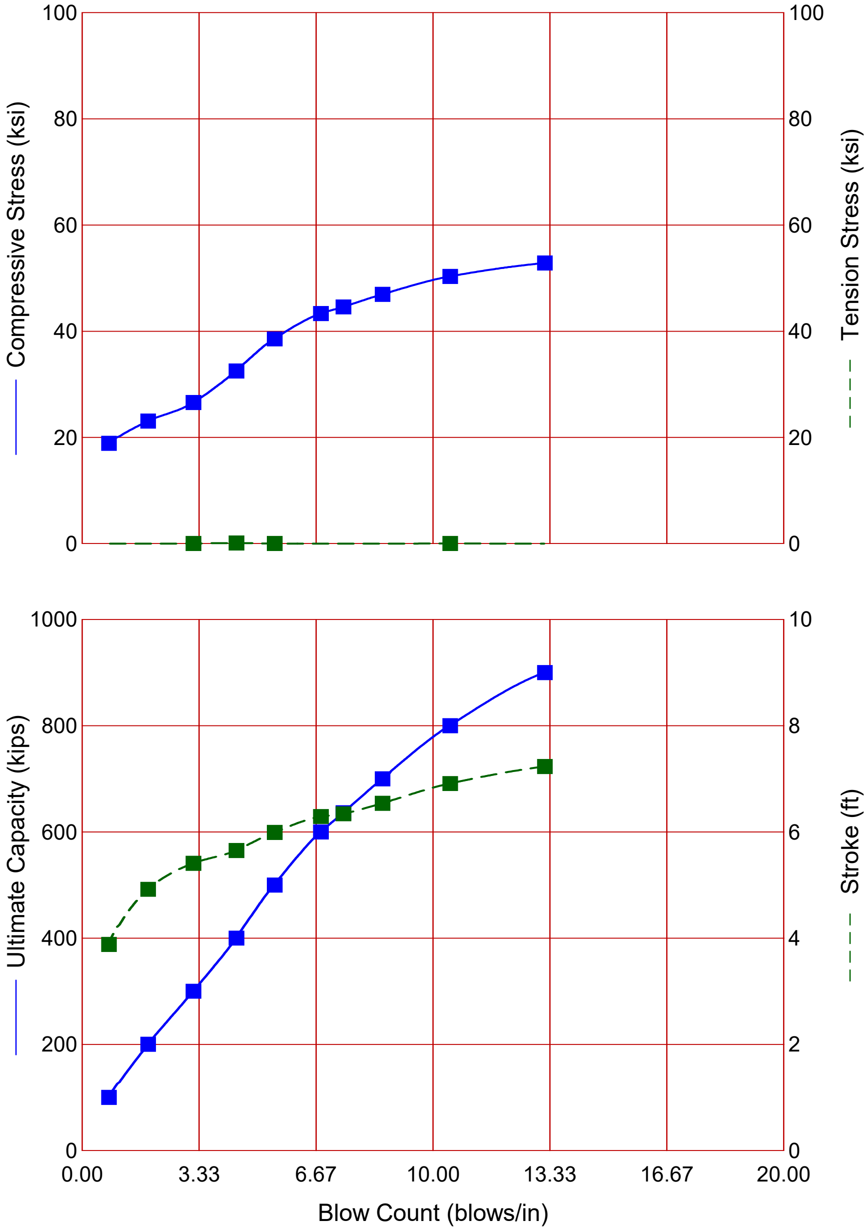
Hammer	Energy / Power (lb-ft)	Fuel Setting	Location	R_{ndr} (kips)	Blows/ft	Blows/in	Stress (ksi)
DELMAG D 30	59,730	Fuel Setting 3 (81%)	Pier	636	89.4	7.5	44.6

CONCLUSIONS

The analysis indicates the Delmag D30 hammer operated at fuel setting 3 (81% of the combustion pressure) should be able to drive the piles at the pier to the nominal structural pile resistance while staying at a blow count between 3 and 15 blows per inch and limiting the driving stresses to below 45 kips per square inch (ksi), in accordance with Section 501.042 of the MaineDOT 2020 Standard Specifications.

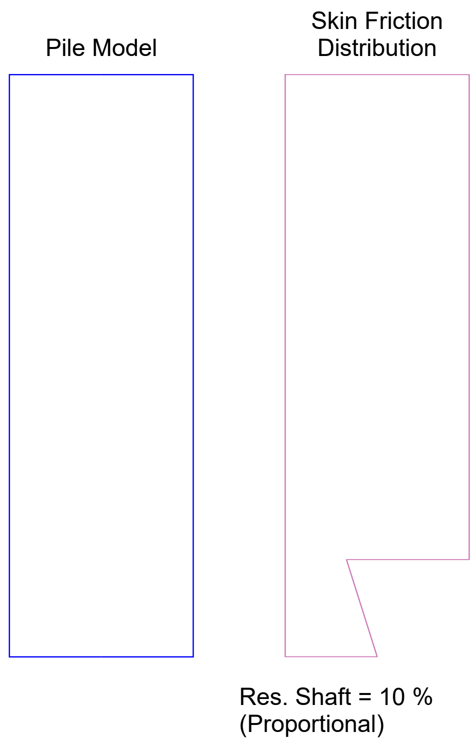
Golder Associates Inc.
21450908 Freeport Exit 20 Pier

16-Aug-2021
GRLWEAP Version 2010



DELMAG D 30

Ram Weight	6.60	kips
Efficiency	0.800	
Pressure	1200 (81%)	psi
Helmet Weight	2.70	kips
Hammer Cushion	109975	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.200	s/ft
Toe Damping	0.150	s/ft
Pile Length	12.00	ft
Pile Penetration	12.00	ft
Pile Top Area	26.10	in2



Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
100.0	18.88	0.00	0.8	3.88	21.44
200.0	23.09	0.00	1.9	4.92	18.70
300.0	26.56	0.05	3.2	5.41	17.61
400.0	32.52	0.12	4.4	5.65	17.28
500.0	38.55	0.04	5.5	5.99	17.90
600.0	43.33	0.00	6.8	6.29	18.40
636.0	44.58	0.00	7.5	6.34	18.38
700.0	46.94	0.00	8.6	6.54	18.81
800.0	50.31	0.05	10.5	6.91	19.81
900.0	52.84	0.00	13.2	7.23	20.61

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