REPORT

Supplemental Geotechnical Design Report Part II

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

FREEPORT, MAINE

MAINEDOT WIN 023627.00

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1.0 INTRODUCTION

This Supplemental Geotechnical Design Report (SGDR) Part II 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 site location). This is the second of two supplemental reports associated with the geotechnical design at the site, and specifically pertains to the geotechnical design of the drilled shaft and spread footing foundations for the traffic mast arms and light standards in the project development area.

Our design calculations and references are made in conjunction with the HNTB 98% Design Plans[1]. Golder had previously submitted the Preliminary Geotechnical Design Report (PGDR)[2] 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[7] serves as the basis for this report. Our work presented herein was completed in accordance with Golder's proposed scope of work[3] 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, pier foundations, and the approach embankments. This is the subject of a separate report[4].
- Part II pertains to geotechnical foundation designs and recommendations for traffic mast arms and light standards and luminaires within the project development area. It includes a supplemental subsurface investigation to identify the soil at the traffic mast arm and light standard locations. This is the subject of this report.

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.

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

1 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.

2 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.

3 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.

4 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.

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 SGDR Part I[Error! Bookmark not defined.].

Additional site development includes installation of seven (7) mast arm structures and 14 light standard structures to support traffic safety improvements at the site. These structures require geotechnical foundation design to support the axial, shear, moment, and torsional loading imposed on them. Foundation design and recommendations for these structures are discussed in this report, SGDR Part II.

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 II:

- Planned, coordinated, and monitored a geotechnical field program to establish the soil properties at or near the proposed locations of the traffic mast arm and light standard structures.
- Performed geotechnical drilled shaft and/or spread footing foundation design for seven (7) sign mast arm foundations and 14 light standard foundations using the loading and minimum drilled shaft diameter provided by HNTB[5].

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), 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-

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5 Hodgdon, S. (HNTB). "Freeport - Lighting and Signal Foundation Locations and Design Loads " Message to Melissa Landon (Golder Associates). June 29, 2021. E-mail

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.

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). Two borings were completed near each proposed abutment location, one boring at the proposed pier location, and one boring in the I-295 median approximately 100 feet south of the proposed pier location. For each abutment location, one boring was performed in the southbound lane of Desert Road 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 bedrock. Each boring had 10 feet of rock core drilled as well. Refer to the PGDR[2] for the methods used, boring logs, and interpreted 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

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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.

Subsurface Profile shown in Sheet 5 and the Interpretive Subsurface Cross Section shown in Sheet 6 of this report.

3.2 Supplemental Geotechnical Investigation

Golder completed 17 test borings (BB-FDR-201 through BB-FDR-217) between May 10 and May 26, 2021 along Desert Road between the I-295 southbound and northbound entrance/exit ramp intersections so support design of the proposed traffic mast arm and light standard structures. The field program included Standard Penetration Test (SPT) sampling of coarse-grained and fine-grained materials. A Golder geotechnical engineer or geologist monitored drilling activities, selected sampling intervals, logged subsurface conditions encountered, and obtained soil samples and rock core for use in visual description and classification and subsequent laboratory testing. The as-drilled boring locations were surveyed by MaineDOT following completion of the drilling program. Boring location coordinates and ground surface elevations are summarized in Table 1. Boring locations with respect to existing site features are illustrated in Sheet 2, Sheet 3, and Sheet 4.

Borings were completed by S.W. Cole Explorations, LLC (S.W.COLEx) of Bangor, Maine using a Diedrich D-50 truck-mounted rig. S.W.COLEx drilled the borings using solid-stem augers from the ground surface, followed by the cased and washed methods in which the boring was advanced by driving 5.5 inch diameter casing in 5-foot lengths, the soil in the casing was washed out with a roller bit and water to the depth where samples were subsequently collected. Several borings (BB-FDR-202, -205, -206, -207, -217) were vacuum excavated for the purpose of utility clearance, and the first 5 feet (6 feet for BB-FDR-205) of subsurface materials were not logged. All borings were advanced to at least a depth of 20 feet before termination, or, if bedrock was encountered shallower than 20 feet, to refusal at the bedrock surface after which 5 feet of rock core was collected.

Standard Penetration Testing (SPT) was performed using a calibrated automatic hammer system and a standard 2-inch split spoon sampler in accordance with American Society for Testing and Materials (ASTM) D1586 for all borings. Sampling was conducted continuously between the ground surface and 12 feet depth, and then at 5-foot intervals from 15 feet depth to boring termination for all borings. Split spoons were driven 24 inches by a 140-pound hammer dropped 30 inches, and the number of hammer blows required to advance the split spoon sampler through each 6-inch increment was recorded. Soil samples were collected and stored in jars for subsequent characterization and laboratory testing. Measured, uncorrected N-values, calculated as the sum of the hammer blows to advance the sampler during the 6-inch to 18-inch interval, are provided in the boring logs in Appendix A. A hammer efficiency factor of 0.974 provided by S.W.COLEx was used to convert the measured N-values to N_{60} values, which are also provided in the boring logs in Appendix A.

For each boring, 5 feet of rock core was collected using NQ-size (1-⅞ inch inside diameter) diamond-tipped core barrels in all borings following refusal of either the casing or rollercone bit to advance. Rock core samples were placed in wooden boxes and transported to the Golder office. Total Core Recovery (TCR), calculated Rock Quality Designation (RQD), and coring rates were recorded for each core run and are provided in the boring logs in Appendix A. A detailed summary of rock quality parameters for the recovered rock core is presented in Table 2, and photographs of the rock core are presented in Appendix B.

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

3.3 Supplemental Rock Probes

Golder completed three (3) rock probes on June 22, 2021 on Desert Road at the location of proposed Abutment No. 2 (RP-FDR-201, RP-FDR-202, and RP-FDR-203). Rock probes were completed by Maine Drilling and Blasting (MD&B) of Gardiner, Maine using an Atlas Copco D 755 track-mounted rig. MD&B drilled the borings using hollow piping advanced by air hammering, with 12-foot sections in length of pipe added as needed from a rotary pipe holder attached to the rig. Golder geotechnical engineers or geologists monitored drilling activities and logged the depth at which bedrock was encountered by the drilling. 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, and rock probe locations with respect to existing site features are illustrated in Sheet 3. Rock probes were used to develop Interpretive Subsurface Cross Section B-B' at Abutment No. 2 shown in Sheet 6. A detailed description of the rock probe field activities is provided in Golder's Supplemental Geotechnical Design Report Part I[4].

4.0 LABORATORY TESTING PROGRAM

Laboratory testing of soil samples was performed by GeoTesting Express (GTX) of Acton, Massachusetts in accordance with applicable American Society for Testing Materials (ASTM) and American Association of State Highway Transportation Officials (AASHTO) testing procedures. Geotechnical laboratory tests were performed on SPT split spoon soil samples representative of each soil type collected from the borings to assist in soil classification. The types and numbers of each of the laboratory tests conducted on soil samples are summarized in Table 4-1. Measured index and classification test results from soil samples are summarized in Table 3. Soil testing results are also included on the boring logs in Appendix A. Complete laboratory testing results are provided in Appendix C.

Table 4-1: Laboratory Testing of Soils

Soil Laboratory Test	Test Standard	No. Tests Completed
Grainsize (sieve)	ASTM D6913 AASHTO T 88	27
Water Content	ASTM D2216 AASHTO T 265	31
Atterberg Limits with Natural Water Content	ASTM D4318 AASHTO T 89/90	2

5.0 SUBSURFACE CONDITIONS

Soils encountered at the borings were found to generally include fill materials placed during construction of the bridge and roadway, naturally occurring silt and clay associated with the Presumpscot Formation, and sand and gravel interpreted as glacial till over bedrock. Detailed descriptions of the soil and bedrock conditions encountered at the borings are provided in the boring logs in Appendix A. The following descriptions summarize the major stratigraphic units from the existing ground surface to depth. Elevations presented are based on the NAD83 (2011) Maine 2000 West datum.

Asphalt Pavement: Asphalt pavement thicknesses observed in borings BB-FDR-201, BB-FDR-202, BB-FDR-205, BB-FDR-206, BB-FDR-207, and BB-FDR-217 ranged from between 3-inches and 12-inches thick.

Topsoil: Topsoil thickness observed in borings BB-FDR-203, BB-FDR-210, and BB-FDR-212 ranged from 0.3 feet to 0.4 feet thick.

Fill: Fill was encountered in all borings. The layer was observed to be between 2.2 feet and 14.7 feet thick and start between elevation 148.0 feet and 164.3 feet. Generally, the thickness of the fill is greater behind the bridge abutments. The depth of fill ranges from 0 feet to 1.0 feet. The fill consisted of fine to coarse SAND with gravelly to trace gravel fractions and silty to trace silt fractions, and sandy fine GRAVEL with trace silt. Laboratory classifications generally described the layer as SM, SW-SM, SW, SP, SM-SP, or GP (USCS classification) and A-3, A-2-4, A-1-a, or A-1-b (AASHTO classification). N_{60}-values for the fill, corrected for hammer efficiency, ranged from 3 to 91. In borings BB-FDR-202, -204, -205, -208, and -211, the fill layer transitions to the Presumpscot Formation silty or sandy clay and silt between 2.3 feet and 15.3 feet below ground surface (bgs). In borings BB-FDR-206, -207, and -213, the fill layer transitions to the sand and gravel layer between 4.0 and 10.0 feet bgs. In borings BB-FDR-201, -203, -209, -210, -212, -214, -215, -216, and -217, the fill layer transitions to bedrock between 2.8 and 15.0 feet bgs.

Buried Asphalt Pavement: A buried asphalt pavement layer with a thickness of between 1 inch and 3 inches was observed within the fill in borings BB-FDR-202, -209, -211 and -215.

Presumpscot Formation: Presumpscot silty or sandy clay and silt was encountered in BB-FDR-202, -204, -205, -207, -208 and -211. The layer was observed to be between 0.7 feet and 9.3 feet thick and start between elevation 133.9 feet and 150.3 feet. The Presumpscot Formation consists of CLAY with gravel fractions ranging up to "trace", sand fractions ranging from "some" to "trace", and silt fractions ranging up to "silty", and SILT with gravel fractions ranging up to "little" and sand fractions up to "sandy". Laboratory classifications generally described the layer as CL or ML-CL (USCS classification) and A-6 or A-4 (AASHTO classification). SPT N_{60}-values ranged from 3 to 34 (soft to hard). The clay layer transitions to a layer of sand and gravel at 9.7 feet, 8.0 feet, and 8.0 feet bgs for BB-FDR-204, BB-FDR-208, and BB-FDR-211, respectively, and to bedrock at 19.5 feet and 18.4 feet bgs for BB-FDR-202 and BB-FDR-205, respectively. BB-FDR-207 terminated in the clay layer at 26.6 feet bgs.

Sand and Gravel: A layer of sand and gravel (interpreted as glacial till) was encountered in borings BB-FDR-204, -206, -207, -208, -211 and -213. The layer was observed to be between 0.4 feet and 15.5 feet thick and start between elevation 139.3 feet and 155.3 feet. The glacial till consisted of fine to coarse SAND with gravel ranging up to “gravelly” and silt fractions ranging from "silty" to "trace". Laboratory classifications generally described the layer as SW-SM, SM-SC, or SM (USCS classification) and A-2-4, A-3, or A-1-b (AASHTO classification). SPT N_{60}-values ranged from 16 to refusal. The glacial till layer transitions to bedrock at 14.5 feet, 25.5 feet, 8.4 feet, 9.3 feet, and 12.0 feet bgs for BB-FDR-204, BB-FDR-206, BB-FDR-208, BB-FDR-211, and BB-FDR-213, respectively, and to clay at 24.5 feet bgs for BB-FDR-207.

Bedrock: The bedrock surface was encountered in all 200-series borings except BB-FDR-207 and varied in elevation by approximately 21.6 feet across the site. The deepest bedrock surface (elevation 114.6) was encountered in historical boring MTB-3-83, which is located on the southern corner of the existing Abutment No. 1. The shallowest bedrock was encountered in the borings BB-FDR-210, -212, and -214 located in the southeastern corner of the exploration area. In general, bedrock appears to slope down towards the low point at MTB-3-83 from each side of the site. Therefore, on the northwestern approach side, the bedrock is sloping down from southwest to northeast towards MTB-3-83; and on the southeastern approach side, the bedrock is sloping down from northeast to southwest towards MTB-3-83.

Between 4.4 feet and 5.5 feet of rock core was collected in each boring. The predominant bedrock lithology encountered was gray, coarse-grained, strongly foliated, fresh to slightly weathered gneiss, interpreted to be part of the Vassalboro Formation. The RQD (rock quality designation) ranged from very poor (19%) to fair (73%), and the estimated RMR (rock mass rating) ranged from 46 to 67. Table 2 provides detailed information about the recovery, rock quality designation (RQD), rock mass rating (RMR), and descriptions of lithology, rock mass, and discontinuities.

Groundwater: Groundwater levels were measured in the borings BB-FDR-204, -208 and -213, and elevations were between 140.0 feet and 145.5 feet. Groundwater levels shown on the Interpretive Subsurface Profile A-A' (Sheet 5) were interpreted based on water level measurements from the 100-series borings (See PGDR[2]).

6.0 GEOTECHNICAL ANALYSES AND RECOMMENDATIONS

Golder used the geotechnical data collected during the supplemental and preliminary geotechnical investigations to develop design parameters for the traffic mast arm and high mast lighting structure drilled shaft foundations. These parameters were based on correlations with SPT N_{60} values and engineering judgement and were used for subsequent geotechnical design.

Table 6-1: Mast Arm Parameters for Foundation Design

Structure	Loading Description[1]	Minimum Foundation Diameter[1] (ft / in)	Station[2]	Offset[2]	Proposed Ground Elevation[2] (ft)	Existing Ground Elevation[2] (ft)
MA1 (40' mast arm)	50 foot mast arm	3 / 36	55+75.00	45.5 feet LT	153.0	153.0
MA2 (20' mast arm)	25 foot mast arm	3 / 36	56+79.94	22.6 feet RT	157.0	156.5
MA3 (25' mast arm)	25 foot mast arm	3 / 36	56+07.00	31.0 feet RT	152.5	152.5
MA4 (45' mast arm)	50 foot mast arm	3 / 36	64+59.82	45.5 feet LT	162.1	160.1
MA5 (40' mast arm)	50 foot mast arm	3 / 36	65+45.27	45.5 feet LT	158.8	158.2
MA6 (40' mast arm)	50 foot mast arm	3 / 36	65+20.09	45.5 feet RT	158.5	157.5
MA7 (45' mast arm)	50 foot mast arm	3 / 36	67+05.00	30.5 feet LT	156.0	156.0

1. Based on mast arm and high mast lighting load scenario information[5] provided to Golder by HNTB.
2. From HNTB 98% Design Plans[1].

HNTB[5] provided Golder with stationing and offset locations, design loads, and minimum foundation diameters for seven (7) mast arm structures and 14 light standard structures. Table 6-1 and Table 6-2 provide information related to the design of the mast arm structures and light standards, respectively. Sheet 2 through Sheet 4 show the locations of each of these structures with respect to site features and BB-FDR-200 series borings.

Table 6-2: Light Standard Parameters for Foundation Design

Structure	Loading Description[1]	Minimum Foundation Diameter[1] (ft / in)	Station[2]	Offset[2]	Proposed Ground Elevation[2] (ft)	Existing Ground Elevation[2] (ft)
Pole 1	Light Standard	2.5 / 30	54+00	34.0 feet LT	145.0	145.0
Pole 2	Light Standard	2.5 / 30	55+35	28.4 feet LT	152.0	151.0
Pole 3	Light Standard	2.5 / 30	300+75	18.0 feet LT	152.3	152.9
Pole 4	Light Standard	2.5 / 30	56+80	30.0 feet LT	158.0	157.0
Pole 5	Light Standard	2.5 / 30	58+15	30.0 feet LT	164.0	162.5
Pole 6	Light Standard	2.5 / 30	59+50	30.0 feet LT	169.0	166.5
Pole 7	Light Standard	2.5 / 30	63+25	30.0 feet LT	168.0	164.5
Pole 8	Light Standard	2.5 / 30	64+25	30.0 feet LT	163.5	161.0
Pole 9	Light Standard	2.5 / 30	201+00	14.0 feet RT	160.3	160.3
Pole 10	Light Standard	2.5 / 30	65+75	30.0 feet LT	158.0	157.6
Pole 11	Light Standard	2.5 / 30	66+60	48.0 feet RT	155.5	155.0
Pole 12	Light Standard	2.5 / 30	67+75	78.0 feet RT	154.0	154.0
Pole 13	Light Standard	2.5 / 30	105+75	15.0 feet RT	156.0	154.5
Pole 14	Light Standard	2.5 / 30	104+40	15.0 feet RT	150.5	150.0

1. Based on mast arm and high mast lighting load scenario information[5] provided to Golder by HNTB.
2. From HNTB 98% Design Plans[1].

Golder evaluated the subsurface profiles at or nearby each of the proposed mast arm and light standard structures. Rather than perform separate foundation analyses for each structure, we aggregated this information into design profiles based on soil layering (material type and thickness) and soil and rock engineering properties relevant to the foundation design. Appendix D summarizes the subsurface characteristics and interpreted engineering parameters for each of the 21 locations as well as the six (6) design subsurface profiles for the mast arm or standard lighting foundations. Golder chose these design subsurface profiles based on layering and material properties profiles in order to provide a uniform drilled shaft or spread footing foundation design for the 14 standard lighting structures and seven (7) mast arm structures. The design subsurface profiles considered both the presence of cohesionless soils only (proposed and in situ fills and sand/gravel above bedrock), the presence of the Presumpscot glaciomarine clay, and the location of the bedrock surface.

Golder evaluated drilled shaft lengths assuming the top of shaft is at the proposed ground surface, ignoring any reveal height above the ground surface. We assumed the upper 2 feet of soil does not contribute to soil-shaft frictional interaction in the analyses that are subsequently discussed for each for following: axial resistance (Section 6.1.2), uplift evaluation (Section 6.1.3), and torsion evaluation (Section 6.1.5). The upper 2 feet of soil is incorporated into the lateral resistance and pushover evaluations (Section 6.1.4). We additionally investigated the potential for settlement-induced downdrag loading to develop for use in evaluation of axial resistance (Section 6.1.1). Results are summarized in Section 6.1.6.

Golder evaluated spread footing dimensions assuming a 1 foot thick footing with a minimum depth of 5 feet on soil or shallower if bedrock is encountered at less than 5 feet below the proposed finished grade. Dimensions were determined based on the following analyses: eccentricity and overturning resistance (Section 6.2.1), bearing resistance (Section 6.2.2), sliding resistance (Section 6.2.3), torsional resistance (Section 6.2.4), and settlement (Section 6.2.5). Results are summarized in Section 6.2.6.

6.1 Drilled Shaft Foundations

6.1.1 Settlement and Downdrag Loading

AASHTO LRFD[14] Article 3.11.8 indicates that downdrag on drilled shafts can be assumed to fully develop in soil layers where settlement is equal to or greater than 0.4 inches. We evaluated the settlement (immediate for cohesionless soils, consolidation for fine-grained glaciomarine soil) for the foundation in response to the embankment fills, where appropriate, along with foundation self-weight and applied axial load.

For the scenario where a drilled shaft is founded in the stratigraphy of cohesionless soils above glaciomarine clay, we estimated consolidation settlement of the drilled shaft to estimate downdrag loading. The loads imposed by the proposed embankment fill, shaft self-weight, and the structural axial load component were distributed through the subsurface soils using Boussinesq stress distribution theory. The subsurface soils were then discretized into layers less than 10 feet thick, and the effective stress, imposed stresses, and other soil parameters were used to evaluate consolidation settlement within each layer using one-dimensional consolidation theory for cohesive soil. Golder 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 as undisturbed samples were not collected for subsequent laboratory testing because of high soil stiffness.

For the analyses performed, the estimated consolidation settlement for the design stratigraphy with the glaciomarine layer was less than 0.4 inches. Thus, downdrag loading was assumed to be negligible and was not included in the geotechnical analysis. Where the design stratigraphy included only cohesionless soils over a similar depth, we assumed immediate settlement of the foundation was smaller than consolidation settlement. Thus, downdrag loading was assumed to be negligible and was not included in the geotechnical analysis. Refer to the full methodology of the analysis, material properties, and calculations in Appendix E.

6.1.2 Axial Resistance

In accordance with the MaineDOT Bridge Design Guide[15] Section 5.8, the design methods in AASHTO LRFD[14] and FHWA GEC 10[16] were used to evaluate geotechnical resistance for axial compression loads through a

[14] AASHTO LRFD Bridge Design Specifications, 9th Ed. 2020.

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

[16] FHWA. Drilled Shafts: Construction Procedures and Design Methods. Publication No. FHWA-NHI 18-024 and FHWA GEC 010. September 2018.

combination of side friction and end bearing tip resistance. The nominal axial geotechnical resistance was calculated for a single vertical drilled shaft at each design profile location. Contributions from side resistance of the soils in the top two feet of each shaft were neglected in the analysis to account for the effects of seasonal moisture changes, disturbance during construction, cyclic lateral loading, and low lateral stresses from freshly placed concrete. Below this depth, side and tip resistance were evaluated in accordance with AASHTO LRFD Article 10.8.3.5 for both cohesionless and cohesive soils. Resistance factors of 0.55 and 0.45 were applied to side resistance in cohesionless and cohesive soils, respectively. Resistance factors of 0.50 and 0.40 were applied to tip resistance in cohesionless and cohesive soils, respectively. Table 6-3 presents our recommended values of factored geotechnical axial compression resistance for each design profile analyzed. The full analysis methodology and calculations are provided in Appendix E.

6.1.3 Uplift Resistance

Although it is our understanding uplift loads will not be applied to the drilled shafts, uplift resistance will be provided by side friction acting along the perimeter of the shaft. Based on side friction resistance determined per AASHTO LRFD[14] guidance, and resistance factors of 0.45 and 0.35 for uplift in cohesionless and cohesive soils, respectively (as per AASHTO LRFD Table 10.5.5.2.4-1), recommended factored geotechnical uplift resistances for each design profile are provided in Table 6-3.

6.1.4 Lateral Resistance

The computer program LPile[17] was used to evaluate the lateral geotechnical resistance of the drilled shaft for each design profile. A summary of the input soil and rock parameters and p-y lateral models used in the LPile analyses is provided in Appendix D.

A pushover analysis was performed with LPile to evaluate lateral geotechnical resistance at the Strength I and Extreme I limit states for each design profile. The shaft was modeled as a simple linear elastic beam with an elastic modulus equal to that of concrete and a moment of inertia equal to that of the uncracked circular cross section. Shaft head deflection was computed for a constant factored axial design load and various multiples of the factored shear and moment design loads, up to $1/\varphi$ times the factored shear and moment design loads. A resistance factor (φ) of 0.67 was used for Strength I loads and a resistance factor of 0.8 was used for Extreme I loads. In accordance with FHWA GEC 10[16] Section 9.3.3.3.1, the drilled shaft design was considered stable if the analyses each converged to a solution with a computed head deflection no larger than 10% of the shaft diameter. Furthermore, Golder designed the mast arm foundations so that total drilled shaft rotation at the end of the pushover analysis would result in an estimated vertical movement at the tip of the mast arm of less than 6 inches.

Lateral geotechnical resistance at the Service I limit state was evaluated for each design profile by modeling the shaft as a nonlinear reinforced concrete shaft in flexure. In accordance with FHWA GEC 10[16], a limiting requirement of 0.5 inches was used for the shaft head deflection under Service I loads. Table 6-3 presents the computed head deflections under Service I loads for each design profile analyzed. In support of HNTB's structural design, Golder has provided the maximum bending moment in the drilled shaft for each of the design profiles at the Strength I, Extreme I, and Service I limits states in Table 6-4. The full analysis methodology and calculations are provided in Appendix E.

[17] Ensoft, Inc. LPile software package, version 2019.11.05, release date 03/05/20.

6.1.5 Torsional Resistance

The design method in Florida Department of Transportation (FDOT) Modifications to LRFD[18] was used to evaluate torsional resistance of the drilled shaft for the mast arm and standard lighting design profiles. In accordance with the FDOT method, the drilled shaft was assumed to be installed entirely within cohesionless soil for the torsional calculation. We assumed that the drilled shaft circumference is in contact with soil and determined torsional resistance assuming no casing is in place. The nominal torsional resistance was calculated for a single vertical drilled shaft at each design profile location. Contributions from skin friction resistance of the soils in the top two feet of each shaft were neglected in the analysis to account for the effects of seasonal moisture changes, disturbance during construction, cyclic lateral loading, and low lateral stresses from freshly placed concrete. A resistance factor of 1.0 was applied to torsional resistance for mast arm structures, and a resistance factor of 0.9 was applied to torsional resistance for standard lighting structures. For the mast arms analyzed, shaft length was controlled by the torsional resistance, and torsion demand capacity ratios ranging from 0.82 to 0.89 were calculated under the Extreme I load case. Table 6-3 presents our recommended values of factored torsional resistance for each design profile analyzed. The full analysis methodology and calculations are provided in Appendix E.

6.1.6 Summary

Table 6-3 presents the factored geotechnical resistance values and lateral deflection for the drilled shaft designs we analyzed for each for the design soil profiles chosen, as previously discussed. As discussed, the design soil profiles were chosen based on engineering judgment to represent subsurface layering and material properties that include fill and in situ cohesionless soils, glaciomarine clay when present, and proposed loading. Table 6-4 presents the maximum bending moment derived from the lateral analyses for the design profiles and shafts. Table 6-5 presents Golder's recommended foundation dimensions for each of the traffic mast arms and the standard light poles, which were determined based on our engineering judgment, evaluation of design loading scenarios, and comparison of the actual subsurface soil conditions with the design soil profile conditions.

[18] Florida Department of Transportation. FDOT Modifications to LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals (LRFDLTS-1). Structures Manual, Volume 3. January 2021.

Table 6-3: Factored Geotechnical Resistance and Lateral Deflection for Drilled Shaft Design Scenarios

Design Soil Profile	Shaft Diameter[1] (in)	Shaft Length[1,2] (ft)	Factored Axial Compression Resistance (kips)	Factored Uplift Resistance (kips)	Lateral Shaft Head Deflection[3] (in)	Factored Torsional Resistance (kip-ft)
25-foot mast arm with glaciomarine (MA2)	36	13	95	22	0.13	51
25-foot mast arm without glaciomarine (MA3)	36	7.5	238	21	0.13	55
50-foot mast arm with glaciomarine (MA6)	36	12	96	38	0.11	159
Standard Lighting without glaciomarine (LS9)	30	5.5	57	6	0.13	11
Standard Lighting with glaciomarine (LS14)	30	5.5	38	2	0.07	2.2

1. The drilled shaft diameters and lengths are adequate for the Strength I, Extreme I, Service I, and Service II limit load cases, considering axial, shear, moment, and torsion loading provide by HNTB. See Appendix E – Drilled Shaft Design Calculations, for details.
2. Shaft length assumes the top of the shaft is at the proposed final grade. This length ignores the maximum 3 inches of reveal above ground level allowable in certain conditions noted in MaineDOT Standard Specification Section 626.034 and in MaineDOT Standard Details 626(01) and 626(02).If the location of the shaft allows for HNTB to specify a reveal above the ground surface, we recommend the shaft length be increased by 0.5 feet.
3. At the governing Service Limit State.

Table 6-4: Maximum Bending Moments for Drilled Shaft Design Scenarios

Design Soil Profile	Shaft Diameter[1] (in)	Shaft Length[1,2] (ft)	Load Cases		
			Strength I M_{max} (kip-ft)	Extreme I M_{max} (kip-ft)	Service I M_{max} (kip-ft)
25-foot mast arm with glaciomarine (MA2)	36	13	29	76	40
25-foot mast arm without glaciomarine (MA3)	36	7.5	29	75	40
50-foot mast arm with glaciomarine (MA6)	36	12	148	184	134
Standard Lighting without glaciomarine (LS9)	30	5.5	1	18	10
Standard Lighting with glaciomarine (LS14)	30	5.5	1	18	10

1. The drilled shaft diameters and lengths are adequate for the Strength I, Extreme I, Service I, and Service II limit load cases, considering axial, shear, moment, and torsion loading provide by HNTB. See Appendix E – Drilled Shaft Design Calculations, for details.
2. Shaft length assumes the top of the shaft is at the proposed final grade. This length ignores the maximum 3 inches of reveal above ground level allowable in certain conditions noted in MaineDOT Standard Specification Section 626.034 and in MaineDOT Standard Details 626(01) and 626(02).If the location of the shaft allows for HNTB to specify a reveal above the ground surface, we recommend the shaft length be increased by 0.5 feet.
3. The drilled shaft was modeled as a simple linear elastic beam with an elastic modulus equal to that of concrete and a moment of inertia equal to that of the uncracked circular cross section.

Table 6-5: Mast Arm and Light Standard Recommended Drill Shaft Foundations

Structure	Loading[1]	Shaft Diameter[1,2] (in)	Shaft Length[2,3] (ft)	Station[4]	Offset[4]	Proposed Ground Elevation[4] (ft)	Expected Soil[5]
MA1	50 ft mast arm	36	12	55+75.00	45.5 feet LT	153.0	F/G/BR
MA2	25 ft mast arm	36	13	56+79.94	22.6 feet RT	157.0	F/G/S&G/BR
MA3	25 ft mast arm	36	7.5	56+07.00	31.0 feet RT	152.5	F/BR
MA6	50 ft mast arm	36	12	65+20.09	45.5 feet RT	158.5	F/G/S&G/BR
Pole 2	Light Standard	30	5.5	55+35	28.4 feet LT	152.0	F/BR
Pole 3	Light Standard	30	5.5	300+75	18.0 feet LT	152.3	F/BR
Pole 4	Light Standard	30	5.5	56+80	30.0 feet LT	158.0	F/BR
Pole 5	Light Standard	30	5.5	58+15	30.0 feet LT	164.0	F/G/BR
Pole 6	Light Standard	30	5.5	59+50	30.0 feet LT	169.0	F/S&G/BR
Pole 7	Light Standard	30	5.5	63+25	30.0 feet LT	168.0	F/BR
Pole 8	Light Standard	30	5.5	64+25	30.0 feet LT	163.5	F/BR
Pole 9	Light Standard	30	5.5	201+00	14.0 feet RT	160.3	F/BR
Pole 11	Light Standard	30	5.5	66+60	48.0 feet RT	155.5	F/BR
Pole 12	Light Standard	30	5.5	67+75	78.0 feet RT	154.0	F/BR
Pole 13	Light Standard	30	5.5	105+75	15.0 feet RT	156.0	F/G/S&G/BR
Pole 14	Light Standard	30	5.5	104+40	15.0 feet RT	150.5	G/S&G/BR

1. Based on mast arm and high mast lighting load scenario information[5] provided to Golder by HNTB.
2. The final concrete dimensions and reinforcing steel design for the drilled shafts are the responsibility of the project structural engineer.
3. Shaft length assumes the top of the shaft is at the proposed final grade. This length ignores the maximum 3 inches of reveal above ground level allowable in certain conditions noted in MaineDOT Standard Specification Section 626.034 and in MaineDOT Standard Details 626(01) and 626(02).If the location of the shaft allows for HNTB to specify a reveal above the ground surface, we recommend the shaft length be increased by 0.5 feet.
4. From HNTB 98% Design Plans[1].
5. Soil types: F = fill, G = glaciomarine, S&G = sand and gravel, BR = bedrock. Symbols are listed left to right from shallowest to deepest.

6.2 Spread Footing Foundations

6.2.1 Eccentricity (Overturning) Evaluation

Eccentricity was evaluated using AASHTO LRFD[14] and FHWA GEC 6[19] using the factored loads provided by HNTB[5] applied at the top of the mast arm or light standard pedestal, and loading from the foundation self-weight and weight of soil materials overlying the foundation. Per AASHTO LRFD Article 10.6.1.3, effective spread footing

[19] FHWA. Shallow Foundations. Publication No. FHWA-SA-02-054 and FHWA GEC 6. September 2002.

dimensions (reduced from the proposed foundation width and length) were used in the analyses to account for the reduced effective bearing area caused by eccentric loads. Maximum and minimum load factors were applied to the permanent foundation and soil loads as outlined in AASHTO LRFD Table 3.4.1-1. The analysis limited the eccentricity of the loading at the extreme limit state to 25% of the footing dimension in any direction for footings on soils and rock per AASHTO[20]. Table 6-6 presents our recommended footing dimensions for each design profile analyzed to maintain values of eccentricity within these limits. The full analysis methodology and calculations are provided in Appendix F.

6.2.2 Bearing Resistance

Golder evaluated ultimate bearing resistance using the methods prescribed in FHWA GEC 6[19] and AASHTO LRFD[14]. Bearing resistance was evaluated for using effective footing dimensions, an assumed thickness of 1 foot, and accounted for axial loading and loading from the foundation self-weight and weight of soil materials overlying the foundation. Maximum and minimum load factors were applied to the permanent foundation and soil loads as outlined in AASHTO LRFD Table 3.4.1-1. A resistance factor of 0.45 was applied to the Strength I load case, while a resistance factor of 1.0 was used in the other limit states evaluated per AASTO LRFD 10.5.5.2.2-1. In the case of bearing, a satisfactory capacity to demand ratio (CDR > 1.0) occurs when total resistance is greater than the applied loading. Table 6-6 presents our recommended values of factored bearing resistance for each design profile analyzed. Table 6-7 presents the factored applied bearing pressures for each loading scenario, accounting for HNTB's[5] loading and self-weight of footing and soil. The full analysis methodology and calculations are provided in Appendix F.

6.2.3 Sliding Resistance

Sliding resistance for spread footings considers a combination of the frictional resistance at the interface of the underlying soil or rock materials with the bottom of footing, and the passive resistance pressure on the side of the footing opposite the applied forces. Sliding resistance incorporated the applied loading from HNTB[5] and loading from the foundation self-weight and weight of soil materials overlying the foundation. A sliding coefficient of 0.70 was used for the case of concrete on clean bedrock. A sliding coefficient of 0.45 was used for the case of concrete on cohesionless soil. Minimum load factors were applied to the permanent foundation and soil loads as outlined in AASHTO LRFD Table 3.4.1-1. A resistance factor of 0.8 was used for shear resistance between the soil or rock and foundation at all limit states evaluated, and a resistance factor of 0.5 was used for passive resistance at all limit states evaluated per AASHTO LRFD Table 10.5.5.2.2-1. A satisfactory capacity to demand ratio (CDR > 1.0) occurs when total resistance is greater than the applied shear loads. Table 6-6 presents our recommended values of factored sliding resistance for each design profile analyzed. The full analysis methodology and calculations are provided in Appendix F.

6.2.4 Torsional Resistance

Torsional resistance for spread footings considers a combination of the torsional resistance of the pedestal, the passive resistance pressure along each side of the footing, and the frictional resistance at the interface of the underlying soil or rock materials with the bottom of footing. The design method in Florida Department of Transportation (FDOT) Modifications to LRFD[18] was used to evaluate torsional resistance of the pedestal portion of the foundation over a length less the upper 2 feet of the pedestal length, as described in Section 6.1.5 for drilled shafts. We determined the resistance along the sides of the footing assuming passive resistance developed along half of the footing dimension in a triangular distribution, ranging from negligible resistance at the

[20] AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals, 1st Ed. 2015 with 2017, 2018, and 2020 Interim Revisions.

midpoint of each of the sides to full passive resistance at the edge of the footing. We determined the interface frictional resistance on the effective footing area based on the applied axial loading and loading from the foundation self-weight and weight of soil materials overlying the foundation. A resistance factor of 1.0 was applied to torsional resistance for mast arm structures. For the mast arms analyzed, the torsion demand capacity ratios were 0.64 to 0.83 were calculated under the Extreme I load case. Our analysis of spread footings for mast arms demonstrated spread footings with the proposed dimensions are sufficient to resist the Extreme 1 shear load and torsional load. Based on this analysis and by inspection of loads (where torsional loading is less than shear loading), spread footings sized for Extreme I shear load are sufficiently sized to the resist the Extreme I torsional load. Table 6-6 presents our recommended values of factored torsional resistance for each design profile analyzed. The full analysis methodology and calculations are provided in Appendix F.

6.2.5 Settlement

Settlement of the spread footings was determined for the applied loads and loading from the foundation self-weight and weight of soil materials overlying the foundation, distributed through the subsurface soils using Boussinesq stress distribution theory for the given foundation geometry. The subsurface soils were discretized into layers, and the effective stress, imposed stresses, and other soil parameters were used to evaluate settlement within each layer using the Hough method and correlations to N_{60} for cohesionless soils. Table 6-6 presents the analyzed settlements for the design profiles analyzed. Refer to the full methodology of the analysis, material properties, and calculations in Appendix F.

6.2.6 Summary

Table 6-6 presents the factored geotechnical resistance values and eccentricity we analyzed for the design soil profiles chosen, as previously discussed. As discussed, the design soil profiles were chosen based on engineering judgment to represent subsurface layering and material properties that include fill and in situ cohesionless soils, glaciomarine clay when present, and proposed loading. Table 6-7 presents the factored bearing pressures for each loading scenario we analyzed. Table 6-8 presents Golder's recommended foundation dimensions for each of the traffic mast arms and the standard light poles, which were determined based on our engineering judgment, evaluation of design loading scenarios, and comparison of the actual subsurface soil conditions with the design soil profile conditions. If bedrock is encountered shallower than 3.8 feet depth, the minimum depth evaluated, the footing dimensions may need to be increased or bedrock removed to provide adequate resistance to overturning for mast arms and light standards. If rock is not encountered at the depths anticipated, the minimum embedment for spread footings founded on soil should be 5 feet to provide protection against frost heaving.

Table 6-6: Factored Geotechnical Resistance Values and Settlement for Spread Footing Designs

Design Soil Profile	Footing Dimension[1,2] (ft x ft)	Bottom of Footing Depth[2] (ft)	E[3,4] (ft)	Pedestal Diameter (in)	Factored Bearing Resist.[5] (ksf)	Factored Sliding Resist.[5] (kips)	Factored Torsional Resist.[4] (kip-ft)	Settlement[6] (in)
50-foot mast arm on rock (MA5)	10.5 x 10.5	4.8	2.5	36	14.1	44.1	223	< 0.1
50-foot mast arm on soil (MA7)	10.5 x 10.5	5.0	2.4	36	13.7	29.1	171	0.2
Lighting Standard on rock (LS1)	5.5 x 5.5	3.8	1.3	30	14.1	8.7[6]	-[7]	< 0.1
Lighting Standard on soil (LS2)	5.0 x 5.0	5.0	1.2	30	16.6	6.4	-[7]	0.4

1. Assumes a footing thickness of 1 foot.
2. The spread footing dimensions are adequate for the Strength I, Extreme I, Service I, and Service II limit load cases, considering axial, shear, moment, and torsion loading provide by HNTB. See Appendix F – Spread Footing Design Calculations, for details.
3. E = Two-way eccentricity
4. At the governing Extreme I Limit State.
5. At the Strength I Limit State
6. At the Service I Limit State
7. Our analysis of spread footings for mast arms demonstrated spread footings with the proposed dimensions are sufficient to resist the Extreme 1 shear load and torsional load. Based on this analysis and by inspection of loads (where torsional loading is less than shear loading), spread footings sized for Extreme I shear load are sufficiently sized to the resist the Extreme I torsional load.

Table 6-7: Summary of Bearing Pressures for Spread Footing Designs

Design Soil Profile	Footing Dimension[1,2] (ft x ft)	Load Cases			
		$q_{applied}$ (ksf) Strength I	$q_{applied}$ (ksf) Extreme I	$q_{applied}$ (ksf) Service I	$q_{applied}$ (ksf) Service II
50-foot mast arm on rock (MA5)	10.5 x 10.5	1.94	1.76	1.46	1.33
50-foot mast arm on soil (MA7)	10.5 x 10.5	1.48	1.40	1.11	1.02
Light Standard on rock (LS1)	5.5 x 5.5	0.74	1.33	0.97	0.58
Light Standard on soil (LS2)	5.0 x 5.0	0.95	1.44	0.99	0.74

1. Assumes a footing thickness of 1 foot.
2. The spread footing dimensions are adequate for the Strength I, Extreme I, Service I, and Service II limit load cases, considering axial, shear, moment, and torsion loading provide by HNTB. See Appendix F – Spread Footing Design Calculations, for details.

Table 6-8: Mast Arm and Light Standard Recommended Spread Footing Foundations

Structure	Loading[1]	Minimum Footing Dimension[2] (ft x ft)	Bottom of Footing Depth[3,4] (ft)	Pedestal Diameter (in)	Station[5]	Offset[5]	Proposed Ground Elevation[5] (ft)	Expected Soil[6]
MA4	50 foot mast arm	10.5 x 10.5	5.0	36	64+59.82	45.5 feet LT	162.0	F/BR
MA5	50 foot mast arm	10.5 x 10.5	5.0	36	65+45.27	45.5 feet LT	158.5	F/BR
MA7	50 foot mast arm	10.5 x 10.5	5.0	36	67+05.00	30.5 feet LT	156.0	F/BR
Pole 1	Light Standard	5.5 x 5.5	4	30	54+00	34.0 feet LT	145.0	F/BR
Pole 10	Light Standard	5.5 x 5.5	4.0	30	65+75	30.0 feet LT	158.0	F/BR

1. Based on mast arm and high mast lighting load scenario information[5] provided to Golder by HNTB.
2. Assumes footing thickness of 1 foot. The final concrete dimensions and reinforcing steel design for the spread footings are the responsibility of the project structural engineer.
3. If, during excavation for construction, bedrock is encountered at less than 5 feet below the proposed ground surface, we recommend the spread footing dimensions for the 50 foot mast arm or standard lighting on rock be used. If bedrock is not encountered within 5 feet of the proposed ground surface, we recommend spread footing dimensions for the 50 foot mast arm or standard lighting on soil be used.
4. Minimum recommended bottom of footing depths for spread footings founded on rock are provided. If rock is not encountered at a depth of 4 feet, subsurface soils should be excavated either to the bedrock surface for bearing on rock or to a depth of 5 feet for bearing on soil, whichever is shallower.
5. From HNTB 98% Design Plans[1]
6. Soil types: F = fill, G = glaciomarine, S&G = sand and gravel, BR = bedrock.

7.0 CONSTRUCTION CONSIDERATIONS

We recommend the drilled shaft and spread footing foundation locations be cleared, grubbed, and stripped of existing vegetation, pavement, and topsoil, and any 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 prior to start of foundation construction. We recommend unsuitable soils be replaced with Granular Borrow materials and placement methods in accordance with MaineDOT Standard Specifications.

Drilled shaft foundations should be constructed in accordance with MaineDOT Standard Specification Section 626.034. Foundations should be cast-in-place using temporary casing, if necessary, with no more than 2 feet of permanent casing (e.g., tubular form) below the ground surface. We recommend concrete for drilled shafts be placed (via tremie methods) as soon after excavation as practicable to prevent debris from collecting in the excavated area. The Contractor should provide temporary dewatering of excavations for foundations such that concrete is placed in the dry. The concrete for drilled shafts should be placed in accordance with Section 502.10 as temporary casing is withdrawn to prevent debris from contaminating the foundation and to ensure concrete is cast against the surrounding soil. The level of the concrete inside the temporary casing should be above the bottom of the casing throughout concrete placement.

Spread footing foundations should be constructed in accordance with MaineDOT Standard Specification Section 626.034. Foundations should be constructed of cast-in-place reinforced concrete. The Contractor should provide temporary dewatering of excavations for foundations such that concrete is placed in the dry. Where bedrock is encountered at depths of 5 feet below final grade or shallower, spread footings should be installed directly on cleaned bedrock prepared in accordance with MaineDOT Standard Specification 203.042. Where bedrock is deeper than 5 feet, spread footings should be founded on soil where the base of the spread footing is no shallower than 5 feet. For spread footings on soil, we recommend the bearing soils are over-excavated by 1 foot and backfilled with compacted Gravel Borrow to facilitate drainage and sliding resistance. Spread footings should be backfilled using Gravel Borrow meeting the requirements of MaineDOT Standard Specification Section 703.20 and placed in accordance with MaineDOT Standard Specification Sections 626.034 and Section 206.

8.0 REPORT AND EXPLORATION LIMITATIONS

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

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

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

Signature Page

STATE OF MAINE
CHRISTOPHER C. BENDA No. 15164
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Tables

Table 1: Subsurface Exploration Locations
Supplemental Geotechnical Design Report - Part II
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)	Comments[4,5]
	Northing (ft)	Easting (ft)	Stationing	Offset			
BB-FDR-201	368572.689	1050746.277	54+95.4	34.0 L	148.7	13.5	Bedrock at 141.2 ft elevation (7.5 ft bgs)
BB-FDR-202	368530.791	1050827.612	55+87.0	41.2 L	153.4	24.5	Bedrock at 133.9 ft elevation (19.5 ft bgs)
BB-FDR-203	368449.474	1050811.641	56+15.6	36.6 R	153.8	15.0	Bedrock at 143.6 ft elevation (10.2 ft bgs)
BB-FDR-204	368421.892	1050865.815	56+76.0	32.4 R	154.0	20.0	Bedrock at 139.5 ft elevation (14.5 ft bgs)
BB-FDR-205	368414.126	1050929.394	57+34.6	6.7 R	159.3	24.0	Bedrock at 140.3 ft elevation (19.0 ft bgs)
BB-FDR-206	368334.079	1051070.308	58+96.5	6.2 R	164.8	30.5	Bedrock at 139.3 ft elevation (25.5 ft bgs)
BB-FDR-207	368357.075	1051099.843	59+10.9	28.4 L	165.3	26.6	Bedrock not encountered
BB-FDR-208	368077.192	1051402.965	63+12.5	66.3 R	148.0	12.8	Bedrock at 139.6 ft elevation (8.4 ft bgs)
BB-FDR-209	368039.242	1051524.474	64+31.2	45.1 R	154.4	12.5	Bedrock at 146.9 ft elevation (7.5 ft bgs)
BB-FDR-210	368112.279	1051576.552	64+54.1	41.7 L	159.8	15.0	Bedrock at 155.5 ft elevation (4.3 ft bgs)
BB-FDR-211	367969.612	1051616.775	65+29.6	84.4 R	152.6	14.5	Bedrock at 143.1 ft elevation (9.5 ft bgs)
BB-FDR-212	368088.769	1051665.502	65+53.0	42.1 L	159.0	12.5	Bedrock at 154.0 ft elevation (5.0 ft bgs)
BB-FDR-213	367983.42	1051667.985	65+72.5	61.4 R	152.9	17.0	Bedrock at 140.9 ft elevation (12.0 ft bgs)
BB-FDR-214	368073.057	1051724.46	66+17.5	34.0 L	157.8	7.8	Bedrock at 155.0 ft elevation (2.8 ft bgs)
BB-FDR-215	367980.564	1051763.733	66+58.2	57.4 R	154.5	13.7	Bedrock at 145.8 ft elevation (8.7 ft bgs)
BB-FDR-216	368073.115	1051792.158	66+91.1	33.4 L	157.0	14.9	Bedrock at 148.2 ft elevation (8.8 ft bgs)
BB-FDR-217	368485.335	1050886.362	56+61.2	32.6 L	156.4	20.0	Bedrock at 141.4 ft elevation (15.0 ft bgs)
RP-FDR-201	368177.503	1051399.442	60+22.7	52.6 L	166.8	28.0	Bedrock at 138.8 ft elevation (28 bgs)
RP-FDR-202	368172.198	1051397.756	60+26.8	25.6 L	166.9	28.0	Bedrock at 138.9 ft elevation (28 bgs)
RP-FDR-203	368163.876	1051397.103	60+28.3	15.1 L	166.8	27.0	Bedrock at 139.8 ft elevation (27 bgs)

Notes:

1. Borings BB-FDR-201 through BB-FDR-217 were performed by S.W. Cole from May 10 to 26, 2021. Rock Probes RP-FDR-201, 202, and 203 were performed by Maine Drilling and Blasting on June 22, 2021.
2. Test boring locations are shown in Sheets 2 through 4 entitled "Boring Location Plan".
3. As-drilled locations and elevations are derived from survey files within the email titled "FW Exit 20 and 22 Borings and Probes Freeport" received by Golder on June 11, 2021 from MaineDOT and survey files within the email "WIN 23627 Exit 20 - Borings and Probes Freeport" received by Golder on June 23, 2021 from MaintDOT.
4. Boring logs presented in Appendix A.
5. ft = feet
6. bgs = below ground surface

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

GOLDER MEMBER OF WSP

Table 2: Summary of Rock Core Quality
Supplemental Geotechnical Design Report - Part II
I-295 Dessert Road Bridge Replacement #5720 (Exit 20)
Freeport, Maine
MaineDOT WIN 023627.00

Test Boring Designation	Core Size (in)	Existing Ground Surface Elevation[1] (ft)	Run No.	Run Midpoint Depth Below Bedrock Surface (ft)	Run Depth Below Ground Surface (ft) Start	Run Depth Below Ground Surface (ft) End	Run Depth Below Ground Surface (ft) Midpoint	Run Length (ft)	TCR[2] Length (ft)	TCR[2] %	RQD[3] Length (ft)	RQD[3] %	RQD[3] Designation	Physical Rock Parameters: Weathering[4]	Physical Rock Parameters: Estimated Field Strength[4]	Physical Rock Parameters: Rock Mass Rating [RMR][5]	Lithologic, Rock Mass and Discontinuity Description[6]
BB-FDR-201	NX (2.875)	148.7	R1	3.5	8.5	13.5	11.0	5.0	4.4	88%	1.2	24%	Very Poor	Moderately weathered (W3)	Strong (R4)	51	8.5-13.5 ft: Grey, medium to coarse grained, moderately foliated, GNEISS; moderately weathered (W3), strong (R4), discontinuities low angle to moderate (10°-40°), very closely to closely spaced (0.05-0.55 ft) [VASSALBORO FORMATION].
BB-FDR-202	NX (2.875)	153.4	R1	2.5	19.5	24.5	22.0	5.0	4.6	92%	3.1	61%	Fair	Slightly weathered (W2)	Extremely strong (R6)	67	19.5-24.5 ft: Grey, medium to coarse grained, moderately foliated, GNEISS; slightly weathered (W2), extremely strong (R6), discontinuities low angle to steep (10°-60°), very closely to closely spaced (0.1-0.8 ft) [VASSALBORO FORMATION].
BB-FDR-203	NX (2.875)	153.8	R1	2.4	10.2	15.0	12.6	4.8	3.5	73%	3.5	73%	Fair	Slightly weathered (W2)	Very strong (R5)	66	10.2-15.0 ft: Grey, medium to coarse grained, slightly foliated, GNEISS; slightly weathered (W2), very strong (R5), discontinuities low angle to moderate (10°-40°), very closely to closely spaced (0.05-0.6 ft) [VASSALBORO FORMATION].
BB-FDR-204	NX (2.875)	154.0	R1	2.5	15.0	20.0	17.5	5.0	4.6	92%	2.3	46%	Poor	Slightly weathered (W2)	Very strong (R5)	62	15.0-20.0 ft: Grey and white, medium to coarse grained, moderately foliated, GNEISS; slightly weathered (W2), very strong (R5), discontinuities low angle to moderate (5°-40°), very closely to closely spaced (0.05-0.7 ft) [VASSALBORO FORMATION].
BB-FDR-205	NX (2.875)	159.3	R1	2.5	19.0	24.0	21.5	5.0	4.9	98%	1.1	22%	Very Poor	Slightly weathered (W2)	Medium strong (R3)	50	19.0-19.85 ft and 21.2-23.9 ft: Grey, medium to coarse grained, strongly foliated, GNEISS; slightly weathered (W2), medium strong (R3), discontinuities low angle to steep (5°-62°), very closely to closely spaced (0.05-0.5 ft) [VASSALBORO FORMATION].
														Moderately weathered (W3)	Medium strong (R3)	47	19.85-21.2 ft: White, coarse grained, PEGMATITE; moderately weathered (W3), medium strong (R3), discontinuities low angle (5°-10°), very closely to closely spaced (0.05-0.3 ft) [VASSALBORO FORMATION].
BB-FDR-206	NX (2.875)	164.8	R1	2.5	25.5	30.5	28.0	5.0	4.8	96%	1.7	33%	Poor	Slightly weathered (W2)	Very strong (R5)	59	10.2-15.0 ft: Grey, medium to coarse grained, moderately foliated, GNEISS; slightly weathered (W2), very strong (R5), discontinuities low angle to moderate (10°-40°), very closely to closely spaced (0.1-0.55 ft), consists of coarse grained quartz bands [VASSALBORO FORMATION].

Table 2: Summary of Rock Core Quality
Supplemental Geotechnical Design Report - Part II
I-295 Dessert Road Bridge Replacement #5720 (Exit 20)
Freeport, Maine
MaineDOT WIN 023627.00

Test Boring Designation	Core Size (in)	Existing Ground Surface Elevation[1] (ft)	Run No.	Run Midpoint Depth Below Bedrock Surface (ft)	Run Depth Below Ground Surface (ft) Start	Run Depth Below Ground Surface (ft) End	Run Depth Below Ground Surface (ft) Midpoint	Run Length (ft)	TCR[2] Length (ft)	TCR[2] %	RQD[3] Length (ft)	RQD[3] %	RQD[3] Designation	Physical Rock Parameters Weathering[4]	Physical Rock Parameters Estimated Field Strength[4]	Physical Rock Parameters Rock Mass Rating [RMR][5]	Lithologic, Rock Mass and Discontinuity Description[6]
BB-FDR-208	NX (2.875)	148.0	R1	2.2	8.4	12.8	10.6	4.4	3.8	86%	1.2	27%	Poor	Slightly weathered (W2)	Strong (R4)	53	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°), very closely spaced (0.15 ft) [VASSALBORO FORMATION].
														Fresh (W1)	Extremely strong (R6)	62	8.9-12.2 ft: Dark grey, fine grained, dolomitic LIMESTONE; fresh (W1), extremely strong (R6), discontinuities low angle to steep (5°-70°), closely spaced (0.2-0.45 ft) [VASSALBORO FORMATION].
BB-FDR-209	NX (2.875)	154.4	R1	2.5	7.5	12.5	10.0	5.0	4.6	92%	1.1	22%	Very Poor	Slightly weathered (W2)	Very strong (R5)	58	7.5-12.5 ft: White/grey, medium to coarse grained, moderately foliated, GNEISS; slightly weathered (W2), very strong (R5), discontinuities low angle to moderate (10°-45°), very closely to closely spaced (0.1-0.5 ft). Contains quartz band from 8.4 ft to 8.9 ft. [VASSALBORO FORMATION].
BB-FDR-210	NX (2.875)	159.8	R1	2.5	5.0	10.0	7.5	5.0	3.5	70%	1.7	34%	Poor	Slightly weathered (W2)	Strong (R4)	54	5.0-10.0 ft: Grey, medium to coarse grained, moderately foliated, GNEISS; slightly weathered (W2), strong (R4), discontinuities low angle to moderate (5°-47°), very closely to closely spaced (0.05-0.5 ft) [VASSALBORO FORMATION].
			R2	7.5	10.0	15.0	12.5	5.0	5.0	100%	3.1	61%	Fair	Fresh (W1)	Strong (R4)	61	10.0-15.0 ft: Grey, medium to coarse grained, moderately foliated, GNEISS; fresh (W2), strong (R4), discontinuities low angle (5°-32°), closely spaced (0.17-0.7 ft) [VASSALBORO FORMATION].
BB-FDR-211	NX (2.875)	152.6	R1	2.5	9.5	14.5	12.0	5.0	2.5	50%	0.9	19%	Very Poor	Moderately weathered (W3)	Strong (R4)	51	9.5-14.5 ft: Grey, medium to coarse grained, slightly foliated, GNEISS; moderately weathered (W3), strong (R4), discontinuities low angle to moderate (5°-42°), very closely to closely spaced (0.05-0.45 ft) [VASSALBORO FORMATION].
BB-FDR-212	NX (2.875)	159.0	R1	2.8	7.0	12.5	9.8	5.5	5.0	91%	3.1	55%	Fair	Fresh (W1)	Extremely strong (R6)	67	7.0-7.6 ft: White, coarse grained, PEGMATITE; fresh (W1), extremely strong (R6), discontinuities low angle (13°), very closely to closely spaced (0.15-0.5 ft) [VASSALBORO FORMATION].
														Slightly weathered (W2)	Strong (R4)	59	7.6-12.0 ft: Grey, medium to coarse grained, moderately foliated, GNEISS; slightly weathered (W2), strong (R4), discontinuities low angle to moderate (5°-45°), very closely to closely spaced (0.05-0.6 ft) [VASSALBORO FORMATION].

Table 2: Summary of Rock Core Quality
Supplemental Geotechnical Design Report - Part II
I-295 Dessert Road Bridge Replacement #5720 (Exit 20)
Freeport, Maine
MaineDOT WIN 023627.00

Test Boring Designation	Core Size (in)	Existing Ground Surface Elevation[1] (ft)	Run						TCR[2]		RQD[3]			Physical Rock Parameters			Lithologic, Rock Mass and Discontinuity Description[6]
			No.	Midpoint Depth Below Bedrock Surface (ft)	Depth Below Ground Surface (ft) Start	End	Midpoint	Length (ft)	Length (ft)	%	Length (ft)	%	Designation	Weathering[4]	Estimated Field Strength[4]	Rock Mass Rating [RMR][5]	
BB-FDR-213	NX (2.875)	152.9	R1	2.5	12.0	17.0	14.5	5.0	4.4	88%	0.9	19%	Very Poor	Moderately weathered (W3)	Medium strong (R3)	48	12.0-17.0 ft: Greenish white, fine grained, APLITE; moderately weathered (W3), medium strong (R3), discontinuities low angle to moderate (4°-52°), very closely to closely spaced (0.1-0.6 ft) [VASSALBORO FORMATION].
BB-FDR-214	NX (2.875)	157.8	R1	2.5	2.8	7.8	5.3	5.0	4.3	86%	1.7	33%	Poor	Slightly weathered (W2)	Medium strong (R3)	51	2.8-3.9 ft: Grey, medium to coarse grained, strongly foliated, GNEISS; slightly weathered (W2), medium strong (R3), discontinuities horizontal to low angle (3°-15°), very closely to closely spaced (0.05-0.2 ft) [VASSALBORO FORMATION].
														Fresh (W1)	Medium strong (R3)	53	3.9-7.1 ft: White/grey, medium to coarse grained, moderately foliated, GNEISS; fresh (W1), medium strong (R3), discontinuities low angle to moderate (5°-37°), very closely to closely spaced (0.1-0.7 ft) [VASSALBORO FORMATION].
BB-FDR-215	NX (2.875)	154.5	R1	2.5	8.8	13.8	11.3	5.0	4.9	98%	2.5	50%	Poor	Slightly weathered (W2)	Strong (R4)	56	8.75-10.05 ft: Grey, medium to coarse grained, moderately foliated, GNEISS; slightly weathered (W2), strong (R4), discontinuities low angle (5°-23°), very closely spaced (0.05-0.15 ft). Consists of some quartz veins [VASSALBORO FORMATION].
														Fresh (W1)	Extremely strong (R6)	67	10.05-13.65 ft: Grey, medium to coarse grained, strongly foliated, GNEISS; fresh (W1), extremely strong (R6), discontinuities low angle (5°-24°), very closely to closely spaced (0.05-0.7 ft) [VASSALBORO FORMATION].
BB-FDR-216	NX (2.875)	157.0	R1	2.8	9.4	14.9	12.2	5.5	4.8	87%	2.7	49%	Poor	Slightly weathered (W2)	Strong (R4)	58	9.4-10.7 ft and 12.0-14.2 ft: Grey, medium to coarse grained, moderately foliated, GNEISS; slightly weathered (W2), strong (R4), discontinuities low angle to moderate (10°-53°), closely spaced (0.2-0.6 ft) [VASSALBORO FORMATION].
														Highly weathered (W4)	Very weak (R1)	46	10.7-12.0 ft: Greenish white, fine grained, APLITE; highly weathered (W4), very weak (R1), discontinuities low angle to moderate (7°-35°), very closely to closely spaced (0.1-0.25 ft) [VASSALBORO FORMATION].
BB-FDR-217	NX (2.875)	156.4	R1	2.5	15.0	20.0	17.5	5.0	5.0	100%	3.3	66%	Fair	Slightly weathered (W2)	Strong (R4)	61	15.0-20.0 ft: White and grey, medium to coarse grained, moderately foliated, GNEISS; slightly weathered (W2), strong (R4), discontinuities low angle to moderate (12°-38°), very closely to closely spaced (0.1-0.8 ft). Consists of a quartz band from 15.7 to 16.3 ft [VASSALBORO FORMATION].

Notes:

1. 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.

Prepared by: BK

Table 2: Summary of Rock Core Quality
Supplemental Geotechnical Design Report - Part II
I-295 Dessert Road Bridge Replacement #5720 (Exit 20)
Freeport, Maine
MaineDOT WIN 023627.00

Test Boring Designation	Core Size (in)	Existing Ground Surface Elevation[1] (ft)	Run						TCR [2]		RQD [3]			Physical Rock Parameters			Lithologic, Rock Mass and Discontinuity Description[6]
			No.	Midpoint Depth Below Bedrock Surface (ft)	Depth Below Ground Surface (ft) Start	End	Midpoint	Length (ft)	Length (ft)	%	Length (ft)	%	Designation	Weathering[4]	Estimated Field Strength[4]	Rock Mass Rating [RMR][5]	

2. TCR = total core recovery. Total core recovery is the length of core recovered divided by the length of the run.

3. RQD = rock quality designation. RQD is the total length of intact, full diameter core pieces recovered with a length greater than or equal to twice the core diameter (i.e., length of 4 inches) measured along the core axis. The percent RQD is the total length of RQD measured versus the run length. Note that vertical discontinuities are not included in determination of RQD.

4. Weathering and Estimated Field Strength based on Tables II.4 and II.3 (respectively) in Willey, 2004 (based on ISRM, 1981).
5. Rock Mass Rating (RMR) System (Bieniawski, 1989) assigns numerical ratings to six parameters, including the strength of the intact rock, the RQD, the discontinuity spacing, groundwater conditions, and orientation of discontinuities. These ratings are summed to give the RMR value. The rating adjustment for joint orientation was assigned a value of 0; correlation of geologic field mapping data of exposed rock outcrops with the rock core samples and proposed foundation type may allow for a different rating adjustment for joint orientation, and thus a modification to the RMR value shown on this table.
6. Mapped bedrock formation taken from: Berry & Hussey, 1998; Hussey, 1985; and West & Hussey, 2017.

7. ft = feet, in = inches

Checked by: JRS
Reviewed by: JRS

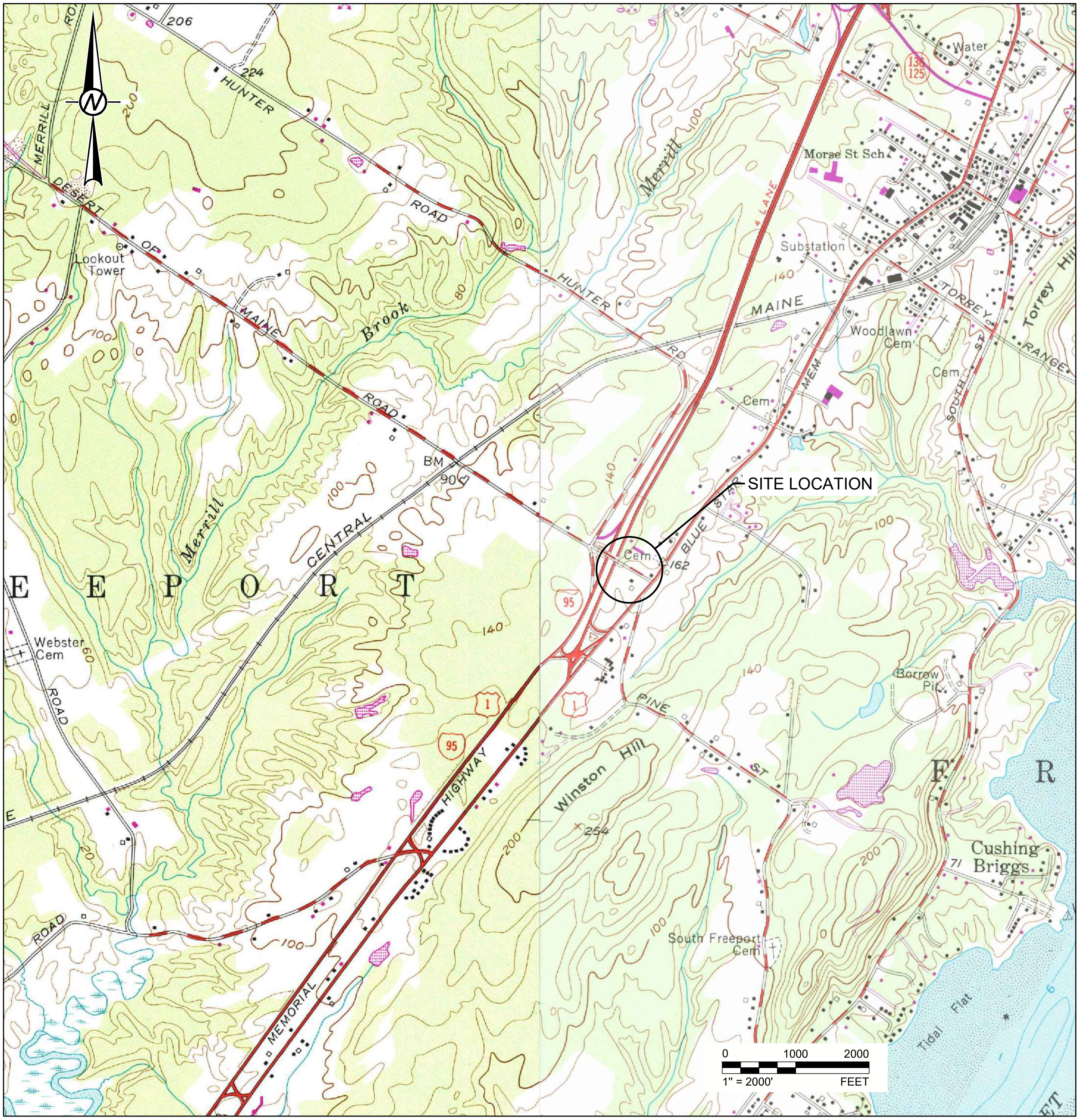
Table 3: Summary of Laboratory Soil Index and Classification Testing Results
Supplemental Geotechnical Design Report - Part II
I-295 Desert Road Bridge Replacement #5720 (Exit 22)
Freeport, Maine
MaineDOT WIN 023627.00

Test Boring Designation[1]	Ground Surface Elevation[2] (feet)	Sample Number[3]	Sample Depth Below Ground Surface (feet)	Approximate Sample Elevation (feet)	Laboratory Testing[4]						Soil Classification[5]	
					Sieve Minus No. 200 (%)	Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	AASHTO	USCS
BR-FDR-201	148.7	3D	4.3 - 6.3	160.6 - 158.6	28.0	11.1	-	-	-	-	A-2-4	SM
BR-FDR-201	148.7	4D	6.3 - 8.3	158.6 - 156.6	-	9.1	-	-	-	-	-	-
BR-FDR-202	153.4	2D	7.0 - 9.0	157.8 - 155.8	12.0	10.0	-	-	-	-	A-1-b	SW-SM
BR-FDR-202	153.4	4D	15.0 - 17.0	149.8 - 147.8	78.0	25.9	-	-	-	-	A-6	ML-CL
BR-FDR-203	153.8	5DB	8.0 - 10.0	156.8 - 154.8	13.0	11.0	-	-	-	-	A-1-b	SM
BR-FDR-204	154.0	4D	6.0 - 8.0	158.8 - 156.8	8.8	9.3	-	-	-	-	A-1-b	SW
BR-FDR-204	154.0	6DB	10.0 - 12.0	154.8 - 152.8	33.0	17.5	-	-	-	-	A-2-4	SM
BR-FDR-205	159.3	1D	6.0 - 8.0	158.8 - 156.8	8.9	8.8	-	-	-	-	A-3	SW
BR-FDR-205	159.3	2DA	8.0 - 10.0	156.8 - 154.8	6.7	8.2	-	-	-	-	A-1-b	SP
BR-FDR-205	159.3	4DB	15.0 - 17.0	149.8 - 147.8	83.0	29.7	-	-	-	-	A-4	ML-CL
BB-FDR-206	164.8	4D	15.0 - 17.0	149.8 - 147.8	9.2	9.8	-	-	-	-	A-3	SW-SM
BB-FDR-206	164.8	5DB	20.0 - 22.0	144.8 - 142.8	20.0	11.0	-	-	-	-	A-1-b	SM-SC
BB-FDR-207	165.3	1D	5.0 - 7.0	159.8 - 157.8	11.0	6.5	-	-	-	-	A-2-4	SM-SP
BB-FDR-207	165.3	5D	20.0 - 22.0	144.8 - 142.8	9.6	8.3	-	-	-	-	A-1-b	SM-SP
BB-FDR-207	165.3	6D	25.0 - 27.0	139.8 - 137.8	80.0	28.0	-	-	-	-	A-4	ML-CL
BB-FDR-208	148.0	3D	4.0 - 6.0	144.0 - 142.0	79.0	27.0	-	-	-	-	A-4	CL
BB-FDR-208	148.0	4D	6.0 - 8.0	142.0 - 140.0	78.0	26.7	29	20	9	0.7	A-4	CL
BB-FDR-211	152.6	2DB	2.0 - 4.0	146.0 - 144.0	-	21.2	-	-	-	-	-	-
BB-FDR-211	152.6	3DB	4.0 - 6.0	148.6 - 146.6	25.0	7.7	-	-	-	-	A-2-4	SC
BB-FDR-211	152.6	4D	6.0 - 8.0	146.6 - 144.6	68.0	22.0	23	17	6	0.8	A-4	ML-CL
BB-FDR-211	152.6	5D	8.0 - 10.0	144.6 - 142.6	41.0	15.6	-	-	-	-	A-4	SM
BB-FDR-212	159.0	3D	4.0 - 6.0	148.6 - 146.6	33.0	11.7	-	-	-	-	A-2-4	SM
BB-FDR-213	152.9	3D	4.0 - 6.0	148.9 - 146.9	-	9.8	-	-	-	-	-	-
BB-FDR-213	152.9	4D	8.0 - 10.0	144.9 - 142.9	20.0	9.5	-	-	-	-	A-2-4	SM
BB-FDR-213	152.9	6D	10 - 12	142.6 - 140.6	27.0	26.6	-	-	-	-	A-2-4	SC
BB-FDR-215	154.5	3DA	4.0 - 6.0	150.5 - 148.5	12.0	11.1	-	-	-	-	A-1-b	SM
BB-FDR-216	157.0	3D	4.0 - 6.0	150.5 - 148.5	33.0	12.6	-	-	-	-	A-2-4	SM
BB-FDR-216	157.0	4DB	6.0 - 8.0	148.5 - 146.5	4.6	2.8	-	-	-	-	A-1-a	GP
BB-FDR-217	156.4	2D	6.0 - 8.0	148.5 - 146.5	11.0	7.8	-	-	-	-	A-1-b	SW-SM
BB-FDR-217	156.4	3D	8.0 - 10.0	148.4 - 146.4	-	10.3	-	-	-	-	-	-
BB-FDR-217	156.4	4D	10.0 - 12.0	146.4 - 144.4	9.5	16.1	-	-	-	-	A-3	SW-SM

Notes:
1. All test boring (BB-FDR-XXX) locations are illustrated in Sheet 2 through Sheet 4 entitled "Boring Location Plan".
2. As-drilled elevations are derived from survey files within the email titled "FW Exit 20 and 22 Borings and Probes Freeport" and "WIN 23627 Exit 20 - Borings and Probes Freeport" received by Golder on June 11 and 23, 2021 from MaintDOT.
3. Laboratory testing was performed by GeoTesting Express, Inc.
4. Atterberg Limits ASTM D4318; Particle Size ASTM D6913; Moisture Content ASTM D2216
5. AASHTO and USCS symbols assigned based on interpreted laboratory test results provided to Golder by GeoTesting Express, Inc. on June 16, 2021.
6. Complete laboratory soil test results are provided in Appendix C.

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

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

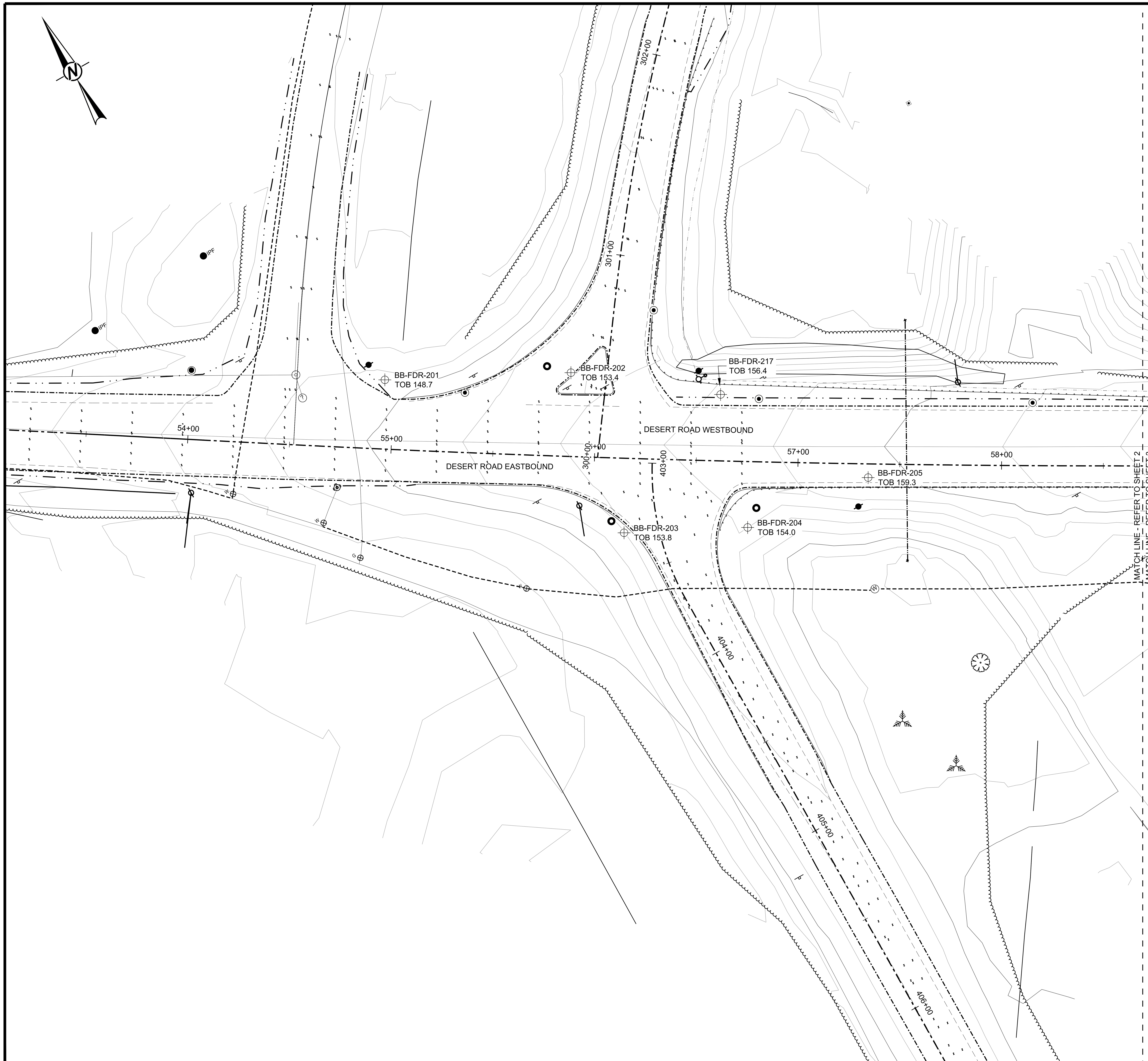
PROJECT
I-295 MERRILL ROAD, BRIDGE REPLACEMENT #5720 (EXIT 20)
FREEPORT, MAINE
MAINEDOT WIN 023627.00

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

◉	PROPOSED LIGHT STANDARD FOUNDATION
O	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.
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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

BRIDGE PLANS
WIN 023627.00
Bridge No. 5720

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

SIGNATURE
P.E. NUMBER
DATE

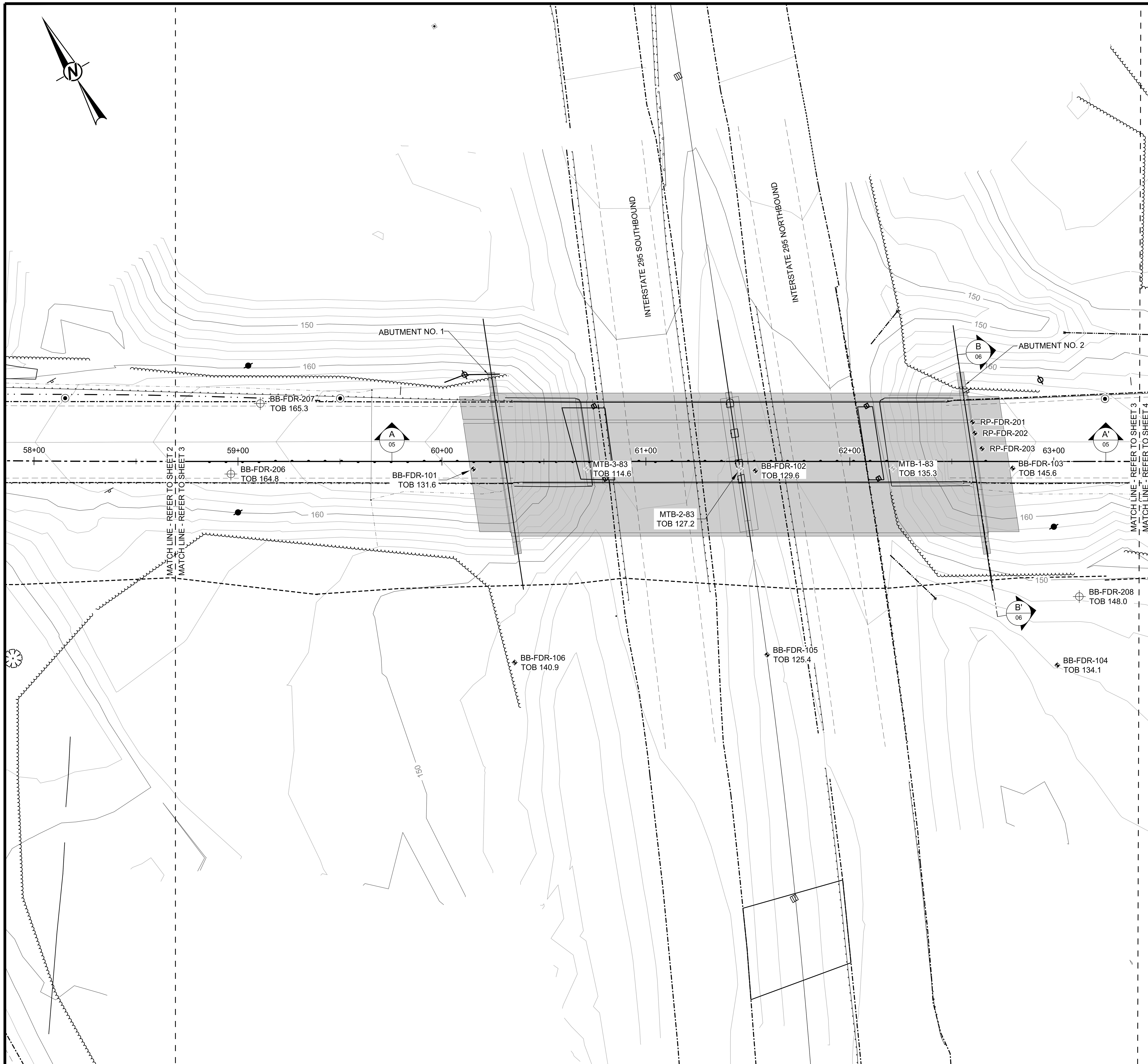
		BY	DATE
PROJ. MANAGER	MEL		
DESIGN-DETAILED	MEL	AAZ	2021/08/20
CHECKED-REVIEWED	CCB	AAZ	2021/08/20
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

MERRILL ROAD BRIDGE
INTERSTATE 295
FREEPORT CUMBERLAND

BORING LOCATION PLAN

SHEET NUMBER
2
OF 6

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



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

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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.

GOLDER
MEMBER OF WSP

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

023627.00

WIN
023627.00

BRIDGE PLANS

Bridge No. 5720

SIGNATURE

P.E. NUMBER

DATE

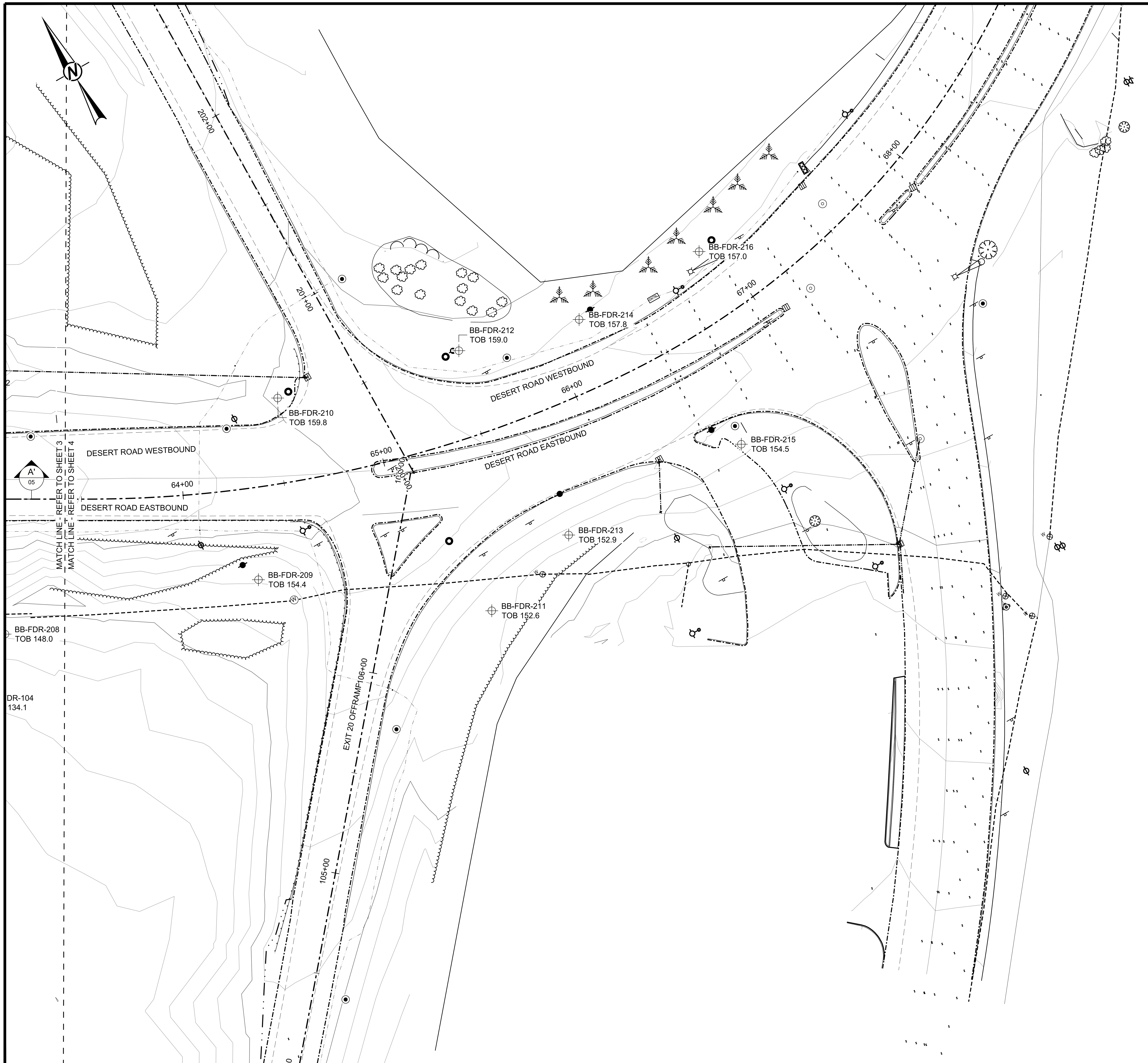
	BY	DATE
PROJ. MANAGER	MEL	
DESIGN-DETAILED	MEL	
CHECKED-REVIEWED	CCB	
DESIGN2-DETAILED2	AAZ	2021/08/20
DESIGN3-DETAILED3	AAZ	2021/08/20
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

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



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◉	PROPOSED LIGHT STANDARD FOUNDATION
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MTB-1-83 TOB 135.3	HISTORICAL BORINGS (SEE SHEET 3)
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GOLDER
MEMBER OF WSP

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

023627.00

WIN 023627.00

BRIDGE PLANS

Bridge No. 5720

	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			

SIGNATURE

P.E. NUMBER

DATE

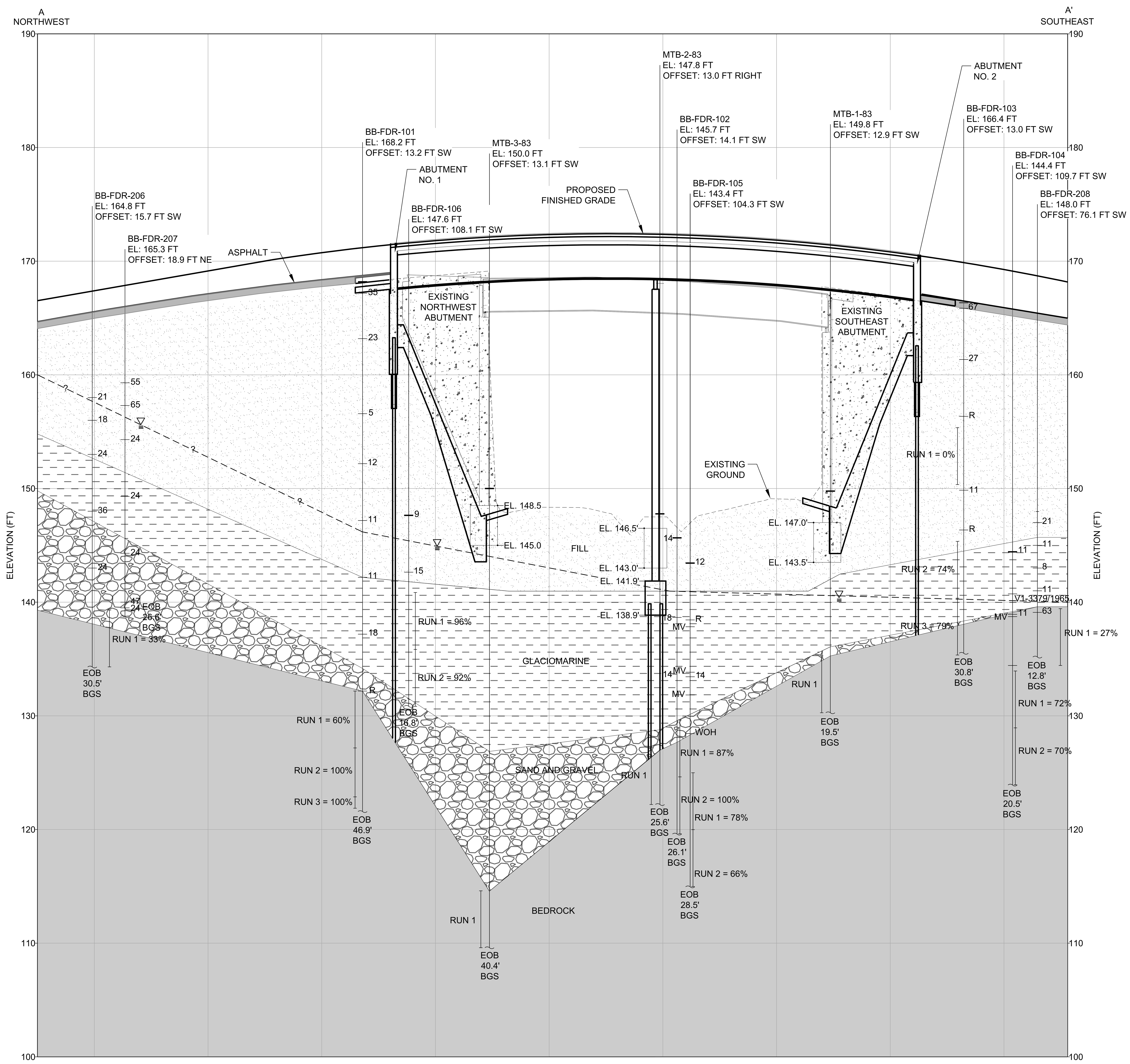
MERRILL ROAD BRIDGE
INTERSTATE 295
FREEPORT CUMBERLAND

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:

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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).
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

	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			

SIGNATURE

P.E. NUMBER

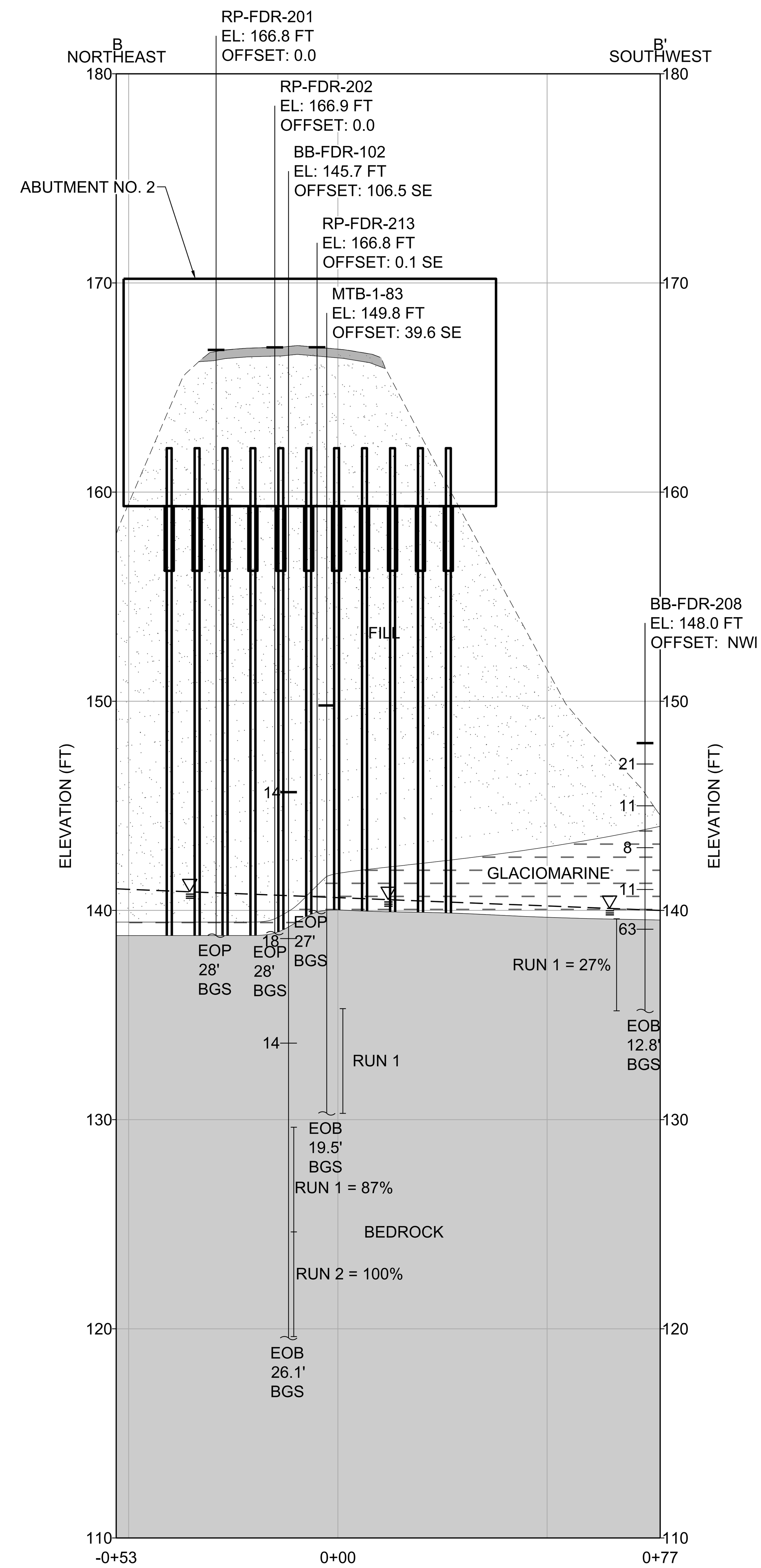
DATE

MERRILL ROAD BRIDGE
INTERSTATE 295
FREEPORT CUMBERLAND

INTERPRETIVE SUBSURFACE
PROFILE A-A'

SHEET NUMBER
5
OF 6

Filename: 21450908_0120_004
Division:
Username:
Date:2021-7-2



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

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BROWN TO GRAY, LOOSE TO MEDIUM DENSE, SILTY FINE TO COARSE SAND AND GRAVEL (FILL)

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EXISTING GROUND SURFACE

INTERPRETED GROUNDWATER SURFACE

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GOLDER
MEMBER OF WSP

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

023627.00

WIN 023627.00

BRIDGE PLANS

Bridge No. 5720

	BY	DATE
PROJ. MANAGER	MEL	
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

APPENDIX A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM

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

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

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

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

MODIFIED BURMISTER SYSTEM

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

TERMS DESCRIBING DENSITY/CONSISTENCY

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

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

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

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

Rock Quality Designation (RQD):

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

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

Correlation of RQD to Rock Mass Quality

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

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

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

Sample Container Labeling Requirements:

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

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

Boring No.: BB-FDR-201
WIN: 023627.00

Driller:	SW Cole	**Elevation (ft.)**	148.7 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:	05/10/21 (10:25);05/10/21 (11:55)	**Drilling Method:**	Pin Auger / Cased Wash	**Core Barrel:**	NQ
Boring Location:	N 368572.689 ft, E 1050746.277 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 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	24/13	0.25 - 2.25	3/2/5/4	7	11	SSA	148.5	Asphalt thickness of 3 in (ASPHALT) —0.3	
									Dark brown, moist, medium dense, fine to coarse SAND, trace coarse gravel, trace silt, well-graded (FILL)	
	2D	24/17	2.25 - 4.25	10/10/12/7	22	36			Dark brown, moist, dense, fine to coarse SAND, trace fine gravel, trace silt, well-graded (FILL)	
	3D	24/17	4.25 - 6.25	8/9/11/12	20	32			Brown, moist, dense, fine to coarse SAND, some fine gravel, some silt, well-graded (FILL)	3D: GTX #620960 WC = 11.1% Fines = 28.3% A-2-4(0), SM
5										
	4D	3.6/10	6.25 - 6.55	50(3")			100		Brown, moist, very dense, fine to coarse SAND, some fine gravel, some silt, well-graded (FILL)	4D: WC = 9.1%
								141.2	Auger refusal at 7.5 ft bgs. Offset 3.5 ft towards Desert Road. —7.5	
	R1	60/53	8.50 - 13.50	RQD = 24%			NQ		Top of bedrock at elev. 141.2 ft. Coarse sand, fine gravel and clay in wash at 8 ft bgs. Grey, medium to coarse grained, moderately foliated, GNEISS; moderately weathered (W3), strong (R4) discontinuities low angle to moderate (10-40 deg), very closely to closely spaced (0.05-0.55 ft) [VASSALBORO FORMATION]. Rock Mass Quality: Very Poor Rock Core Rate (min:sec): 8.5-9.5 ft (1:26) 9.5-10.5 ft (1:19) 10.5-11.5 ft (1:48) 11.5-12.5 ft (1:46) 12.5-13.5 ft (2:06) 88% Recovery	
10										
								135.2	—13.5	
15									**Bottom of Exploration at 13.5 feet below ground surface.** Boring backfilled with cuttings to ground surface.	
20										
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.

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-202

WIN: 023627.00

Driller:	SW Cole	**Elevation (ft.)**	153.4 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 (7:55);05/25/21 (9:50)	**Drilling Method:**	Pin Auger / Cased Wash	**Core Barrel:**	NQ
Boring Location:	N 368530.791 ft, E 1050827.612 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 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 DAYL	152.6		Asphalt thickness of 10 in (ASPHALT) — 0.8 Used vac truck to daylight down to 5 ft bgs, no sampling.	
5	1D	24/14	5.00 - 7.00	32/17/21/20	38	62	26			Brown, wet (from wash out), very dense, fine to coarse SAND, little fine gravel, little silt, well-graded (FILL)	
							24				
	2D	24/18	7.00 - 9.00 7.50 - 7.67	19/20/19/18	39	63	32	145.9 145.4		Brown, wet (from wash out), very dense, fine to coarse SAND, little fine gravel, little silt, well-graded (FILL) — 7.5 Asphalt thickness of 2 in (ASPHALT) — 8.0	GTX #620961 WC = 10.0% Fines = 11.6% A-1-b, SW-SM
							37				
							34			Dense sand in wash from 9-10 ft bgs.	
10	3D	24/6	10.00 - 12.00	6/3/3/3	6	10	OPEN	143.2		Brown, wet, loose, fine to coarse SAND, little silt, little fine gravel, well- graded (FILL). — 10.3 In 3D SS shoe - Grey, wet, fine Sandy SILT, non-plastic, poorly-graded. (GLACIOMARINE) Silt and clay in wash 12-15 ft bgs.	
15	4D	24/24	15.00 - 17.00	6/8/11/16	19	31				Brown and grey, wet, hard, Silty CLAY, some sand, low plasticity, 1-2 in layers of brown sandy SILT and <1 in layers of Silty CLAY throughout sample. T_V=358 psf q_p=4 ksf In SS Shoe - Grey, wet, Sandy SILT, non-plastic, some fine sand, poorly-graded. (GLACIOMARINE) Weathered rock and Silty CLAY in wash 18.4-19.5 ft bgs. Top of bedrock at elev. 133.9 ft bgs. — 19.5	GTX #620962 WC = 25.9% Fines = 78.0% A-6, ML-CL
20	R1	60/55	19.50 - 24.50	RQD = 61%			NQ	133.9		Grey, medium to coarse grained, moderately foliated, GNEISS; slightly weathered (W2), extremely strong (R6), discontinuities low angle to steep (10-60 deg), very closely to closely spaced (0.1-0.8 ft) [VASSALBORO FORMATION]. Rock Mass Quality: Fair Rock Core Rate (min:sec) 19.5-20.5 ft (2:43) 20.5-21.5 ft (3:05) 21.5-22.5 ft (3:31)	
25								128.9			

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.

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-202
WIN: 023627.00

Driller:	SW Cole	**Elevation (ft.)**	153.4 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 (7:55);05/25/21 (9:50)	**Drilling Method:**	Pin Auger / Cased Wash	**Core Barrel:**	NQ
Boring Location:	N 368530.791 ft, E 1050827.612 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										22.5-23.5 ft (4:14) 23.5-24.5 ft (4:26) 92% Recovery	
										24.5	
										Bottom of Exploration at 24.5 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.
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-203

WIN: 023627.00

Driller:	SW Cole	**Elevation (ft.)**	153.8 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:	5/10/21 (13:35) ; 5/10/21 (15:16)	**Drilling Method:**	Pin Auger / Cased Wash	**Core Barrel:**	NQ
Boring Location:	N 368449.474 ft, E 1050811.641 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
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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	1D	24/11	0.00 - 2.00	3/5/10/6	15	24		153.5		1DA, Top 4 in: Brown, damp, medium dense, fine to medium Sandy SILT, non-plastic, some roots and organics (TOPSOIL) 0.3	
										1DB, Bottom 8 in: Brown, damp, medium dense, fine to coarse SAND, little fine gravel, well-graded (FILL)	
	2D	24/17	2.00 - 4.00	5/4/5/6	9	15				2DA, Top 8 in: Brown, damp, medium dense, fine to coarse SAND, little fine gravel, well-graded (FILL)	
										2DB, Bottom 9 in: Brown, damp, medium dense , fine to coarse SAND, little fine gravel, little silt, well-graded (FILL)	
5	3D	24/18	4.00 - 6.00	10/9/15/16	24	39				Brown, moist, dense, fine to coarse SAND, little fine gravel,well-graded (FILL)	
	4D	21.96/21	6.00 - 7.83	17/19/36/50(4")	55	89	32			Brown, moist, very dense, fine to coarse SAND, trace silt, poorly-graded (FILL)	
							36				
	5D	24/12	8.00 - 10.00	26/18/16/9	34	55				5DA, Top 2 in: Medium sand, fine gravel and silt in wash (FILL) 5DB, 2 to 6 in: Dark brown, wet, very dense, fine to coarse SAND, some fine gravel, little silt, well-graded (FILL) 5DC, Bottom 4 in: Dark brown and black, wet, very stiff, Silty PEAT, some organic material (wood chips and roots) , little fine to medium sand, non-plastic (FILL) Wood in wash from 9 to 9.8 ft bgs. Attempted SPT sample 10 to 10.2 ft bgs but unable to penetrate. 10.2	5DB: GTX #620963 WC = 11.0% Fines = 13.0% A-1-b, SM
10	R1	58/42	10.20 - 15.03	RQD = 73%			NQ	143.6		Top of bedrock at elev. 143.6 ft Grey, medium to coarse grained, slightly foliated, GNEISS; slightly weathered (W2), very strong (R5), discontinuities low angle to moderate (10-40 deg), very closely to closely spaced (0.05-0.6 ft) [VASSALBORO FORMATION]. Rock Mass Quality: Fair	
15								138.8		Rock Core Rate (min:sec) 10.2-11.2 ft (1:26) 11.2-12.2 ft (1:54) 12.2-13.2 ft (2:19) 13.4-14.2 ft (3:21) 14.2-15.0 ft (2:52) 73% Recovery 15.0	
20										**Bottom of Exploration at 15.0 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.

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Boring No.: BB-FDR-203

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-204
WIN: 023627.00

Driller:	SW Cole	**Elevation (ft.)**	154.0 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:	5/11/21 (08:35) ; 5/11/21 (09:44)	**Drilling Method:**	Pin Auger / Cased Wash	**Core Barrel:**	NQ
Boring Location:	N 368421.892 ft, E 1050865.815 ft	**Casing ID/OD:**	5.5 in	**Water Level*:**	8.5 ft
Hammer Efficiency Factor:	0.974	**Hammer Type:**	Automatic ☐ Hydraulic ☒ Rope & Cathead ☐		

Definitions:
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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
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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	24/13	0.00 - 2.00	2/2/3/2	5	8	PUSH			1DA, Top 6 in: Brown, moist, loose, SILT, some fine to medium sand, non-plastic, some roots (FILL) 1DB, Bottom 7 in: Brown, moist, loose , fine to coarse SAND, trace fine gravel, trace silt, well-graded (FILL)	
	2D	24/14	2.00 - 4.00	1/1/1/1	2	3				2D: Brown, moist, very loose, fine to coarse SAND, little silt, trace fine gravel, well-graded (FILL)	
5	3D	24/15	4.00 - 6.00	2/2/2/3	4	6				Brown, moist, loose, fine to coarse SAND, little silt, trace fine gravel, well-graded (FILL)	
	4D	24/16	6.00 - 8.00	5/3/2/1	5	8				Brown, moist, loose, fine to coarse SAND, trace fine gravel, trace silt, well-graded (FILL)	GTX #620964 WC = 9.3% Fines = 8.8% A-1-b, SW
	5D	24/18	8.00 - 10.00	4/4/5/7	9	15	35	145.0		5DA, Top 9 in: Brown, moist, medium dense, fine to coarse SAND, little gravel, trace silt, well-graded (FILL) 9.0	
10								144.3		5DB, Bottom 9 in: Dark brown, moist, stiff, SILT, trace fine to medium sand, high organic content, black peat layers present (GLACIOMARINE) 9.7	
	6D	24/23	10.00 - 12.00	4/4/6/7	10	16	71			6DA, Top 13 in: Dark brown, wet, medium dense, fine to coarse SAND, some silt, little fine gravel, organic layer in top 2 inches (SAND AND GRAVEL) 6DB, Bottom 10 in: Brown, moist, medium dense, SAND, some silt, non-plastic (SAND AND GRAVEL) 14.5	6DB: GTX #620965 WC = 17.5% Fines = 33.1% A-2-4(0), SM
15	R1	60/55	15.00 - 20.00	RQD = 46%			NQ	139.5		Top of bedrock at elev. 139.5 ft. Grey and white, medium to coarse grained, moderately foliated, GNEISS; slightly weathered (W2), very strong (R5), discontinuities low angle to moderate (5-40 deg), very closely to closely spaced (0.05-0.7 ft) [VASSALBORO FORMATION]. Rock Mass Quality: Poor Rock Core Rate (min:sec) 15-16 ft (2:06) 16-17 ft (2:56) 17-18 ft (2:17) 18-19 ft (2:20) 19-20 ft (2:36) 92% Recovery	
20								134.0		20.0 **Bottom of Exploration at 20.0 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.

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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-205
WIN: 023627.00

Driller:	SW Cole	**Elevation (ft.)**	159.3 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:	05/26/21 (7:55);05/26/21 (10:05)	**Drilling Method:**	Pin Auger / Cased Wash	**Core Barrel:**	NQ
Boring Location:	N 368414.126 ft, E 1050929.394 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 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 DAYL	158.7		Asphalt thickness of 7 in (ASPHALT) 0.6	
										Used vac truck to daylight down to 6 ft bgs, no sampling.	
5							2				
	1D	24/11	6.00 - 8.00	10/11/9/12	20	32	34			Brown, moist, dense, fine to coarse SAND, trace silt, well-graded (FILL)	1D: GTX #620966 WC = 8.8% Fines = 8.9% A-3, SW
							70				
	2D	24/20	8.00 - 10.00	16/15/15/16	30	49	87			2DA, Top 13 in: Brown, wet, dense, medium to coarse SAND, some fine gravel, trace silt, well-graded (FILL) 2DB, Bottom 7 in: Brown, moist, dense, fine to coarse SAND, little fine gravel, well-graded (FILL)	2DA: GTX #620967 WC = 8.2% Fines = 6.7% A-1-b, SP
							101				
10	3D	24/7	10.00 - 12.00	10/8/6/7	14	23	PUSH			Brown, wet, medium dense, fine to coarse SAND, little fine gravel, well-graded (FILL)	
							10				
							12				
							21				
15	4D	24/12	15.00 - 17.00	3/1/1/4	2	3	15	144.0		4DA, Top 5 in: Brown, wet, very loose, fine to coarse Sandy fine GRAVEL, well-graded (FILL) 15.3	4DB: GTX #620968 WC = 29.7% Fines = 83.5% A-4, ML-CL
							26			4DB, Bottom 7 in: Grey to brown, moist, very soft, SILT, little fine sand, low plasticity, poorly-graded. T_V=1500 psf (GLACIOMARINE)	
							59				
							200	140.9		Casing refusal at 18.4 ft bgs. Weathered rock in wash from 18.4 to 19 ft bgs. 18.4	
	R1	60/59	19.00 - 24.00	RQD = 22%			NQ			Top of bedrock at elev. 140.9 ft. 19.0-19.85 ft and 21.2-23.9 ft: Grey, medium to coarse grained, strongly foliated, GNEISS; slightly weathered (W2), medium strong (R3), discontinuities low angle to steep (5- 62 deg), very closely to closely spaced (0.05-0.5 ft) [VASSALBORO FORMATION]. 19.85-21.2 ft: White, coarse grained, PEGMATITE; moderately weathered (W3), medium strong (R3), discontinuities low angle (5-10 deg), very closely to closely spaced (0.05-0.3 ft) [VASSALBORO FORMATION].	
20											
								135.3			
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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Boring No.: BB-FDR-205

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-205
WIN: 023627.00

Driller:	SW Cole	**Elevation (ft.)**	159.3 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:	05/26/21 (7:55);05/26/21 (10:05)	**Drilling Method:**	Pin Auger / Cased Wash	**Core Barrel:**	NQ
Boring Location:	N 368414.126 ft, E 1050929.394 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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MD = Unsuccessful Split Spoon Sample Attempt
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V = Field Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Field Vane Shear Test Attempt
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SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140 lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25										Rock Mass Quality: Very Poor Rock Core Rate (min:sec) 19-20 ft (4:57) 20-21 ft (4:12) 21-22 ft (4:37) 22-23 ft (4:42) 23-24 ft (4:03) 98% Recovery	
										24.0	
30										**Bottom of Exploration at 24.0 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-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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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				
							250 202			Weathered rock, coarse sand, and gravel in wash from 23.9 to 25.5 ft bgs. 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.

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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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		25.5 Top of bedrock at elev. 139.3 ft. 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.
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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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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)
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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
25								140.8		24.5	

Remarks:

Hammer calibration obtained from SW Cole calibration report.
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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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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
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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.

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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-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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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 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) — 8.0	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			
	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 — 12.8	
								135.2			
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.

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Boring No.: BB-FDR-208

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-209
WIN: 023627.00

Driller:	SW Cole	**Elevation (ft.)**	154.4 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:	5/13/21 (09:58) ; 5/14/21 (11:20)	**Drilling Method:**	Pin Auger / Cased Wash	**Core Barrel:**	NQ
Boring Location:	N 368039.242 ft, E 1051524.474 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 140lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)
0	1D	24/17	0.00 - 2.00	3/4/5/7	9	15	PUSH	
							58	
	2D	24/12	2.00 - 4.00	9/15/6/4	21	34	93	151.9
							127	151.8
	3D	24/2	4.00 - 6.00	2/4/5/13	9	15	189	
5							26 207	
							100 105	147.6
	R1	60/55	7.50 - 12.50	RQD = 22%			NQ	
10								
								141.9
15								
20								
25								

Visual Description and Remarks

Brown, moist, medium dense, fine to medium SAND, little fine gravel, well-graded (FILL)

2DA, Top 6 in: Brown, moist, dense, fine to coarse SAND, little fine gravel, trace silt, well-graded (FILL)
2.5

2DB, 6-7 in: Asphalt thickness of 1 in (ASPHALT)
2.6

2DC, Bottom 5 in: Grey and brown, moist, dense, fine to coarse Sandy fine GRAVEL, poorly-graded (FILL)
Grey and brown, moist, medium dense, fine to coarse Sandy fine GRAVEL, poorly-graded (FILL)
Casing refusal at 6.5 ft bgs. Offset 4 ft south due to water loss in formation.
6.8

Top of bedrock at elev. 147.6 ft.
White/grey, medium to coarse grained, moderately foliated, GNEISS; slightly weathered (W2), very strong (R5), discontinuities low angle to moderate (10-45 deg), very closely to closely spaced (0.1-0.5 ft). Contains quartz band from 8.4 ft to 8.9 ft. [VASSALBORO FORMATION].
Rock Mass Quality: Very Poor
Rock Core Rate (min:sec)
7.5 to 8.5 ft (2:26)
8.5 to 9.5 ft (3:06)
9.5 to 10.5 ft (4:37)
10.5 to 11.5 ft (4:52)
11.5 to 12.5 ft (4:41)
92% Recovery
12.5

Bottom of Exploration at 12.5 feet below ground surface.
Boring backfilled with cuttings to ground surface.

Laboratory Testing Results/ AASHTO and Unified Class.

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.

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-210
WIN: 023627.00

Driller:	SW Cole	**Elevation (ft.)**	159.8 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:	5/11/21 (11:15) ; 5/11/21 (12:55)	**Drilling Method:**	Pin Auger / Cased Wash	**Core Barrel:**	NQ
Boring Location:	N 368112.279 ft, E 1051576.552 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 140lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/10	0.00 - 2.00	2/10/10/18	20	32		159.4		1DA, Top 5 in: Brown, damp, very stiff, SILT, some fine to coarse sand, non-plastic, roots and organic material (TOPSOIL)	
							44			0.4	
	2D	21/0	2.00 - 3.75	7/10/6/50(3")	16	26	50			1DB, Bottom 5 in: Brown, damp, dense, medium to coarse Sandy fine GRAVEL, trace fine gravel, trace silt, well-graded (FILL) 2D: No recovery, some Sandy GRAVEL in shoe.	
							62				
								155.5		4.3	
5	R1	60/42	5.00 - 10.00	RQD = 34%			NQ			Top of bedrock at elev. 155.5 ft. Bedrock chips in wash at 4.5 ft bgs R1: Grey, medium to coarse grained, moderately foliated, GNEISS; slightly weathered (W2), strong (R4), discontinuities low angle to moderate (5-47 deg), very closely to closely spaced (0.05-0.5 ft) [VASSALBORO FORMATION]. Rock Mass Quality: Poor Rock Core Rate (min:sec) 5-6 ft (1:27) 6-7 ft (2:42) 7-8 ft (3:11) 8-9 ft (2:19) 9-10 ft (2:27) 70% Recovery	
10	R2	60/60	10.00 - 15.00	RQD = 61%			NQ			R2: Grey, medium to coarse grained, moderately foliated, GNEISS; fresh (W2), strong (R4), discontinuities low angle (5-32 deg), closely spaced (0.17-0.7 ft) [VASSALBORO FORMATION]. Rock Mass Quality: Fair Rock Core Rate (min:sec) 10-11 ft (2:22) 11-12 ft (2:49) 12-13 ft (3:37) 13-14 ft (3:42) 14-15 ft (3:57) 100% Recovery	
15								144.8		15.0	
										Bottom of Exploration at 15.0 feet below ground surface. Boring backfilled with cuttings to ground surface.	
20											
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.

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-211

WIN: 023627.00

Driller:	SW Cole	**Elevation (ft.)**	152.6 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:	5/13/21 (12:20) ; 5/13/21 (13:50)	**Drilling Method:**	Pin Auger / Cased Wash	**Core Barrel:**	NQ
Boring Location:	N 367969.612 ft, E 1051616.775 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 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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)
0	1D	24/17	0.00 - 2.00	5/10/10/7	20	32	PUSH	152.0
								151.8
	2D	24/20	2.00 - 4.00	4/4/8/7	12	19	16	150.3
							19	
	3D	24/23	4.00 - 6.00	4/8/9/10	17	28	48	
5							46	
	4D	24/19	6.00 - 8.00	6/11/10/19	21	34	110	
							200 225	144.6
	5D	15.6/17	8.00 - 9.30	20/24/50(4")				
	R1	60/30	9.50 - 14.50	RQD = 19%			NQ	143.3
10								
15								138.1
20								
25								

Visual Description and Remarks

1DA, Top 7 in: Brown, moist, dense, coarse to fine SAND, little silt, trace fine gravel, well-graded (FILL)
0.6
1DB: Asphalt thickness of 2 in (ASPHALT)
0.8
1DC, Bottom 8 in: Brown, moist, dense, fine to coarse SAND, some fine gravel, trace silt, well-graded (FILL)
2DA, Top 4 in: Brown, moist, medium dense, fine to coarse SAND, some fine gravel, trace silt, well-graded (FILL)
2.3
2DB, Bottom 16 in: Brown, stiff, SILT, trace fine sand, non-plastic, poorly-graded (GLACIOMARINE). q_p = 6.1 ksf (Pocket Penetrometer)
3DA, Top 4 in: Brown, moist, stiff, SILT, fine sand, non-plastic, poorly-graded (GLACIOMARINE).
3DB, 4-8 in: Brown, moist, medium dense, medium to coarse SAND, little fine gravel, little silt, well-graded (GLACIOMARINE)
3DC, Bottom 15 in: Brown, moist, stiff, SILT, some fine to coarse SAND, low plasticity, sand lenses. q_p = 7.2 ksf (Pocket Penetrometer) (GLACIOMARINE)
4D: Brown, moist, very stiff, SILT, little fine to coarse sand, trace fine gravel, low plasticity (GLACIOMARINE) q_p = 7.2 ksf (Pocket Penetrometer)
8.0
5D: Brown, wet, very dense, Silty fine to coarse SAND, trace gravel, well-graded (SAND AND GRAVEL)
9.3
Top of bedrock at elev. 143.3 ft.
Grey, medium to coarse grained, slightly foliated, GNEISS; moderately weathered (W3), strong (R4), discontinuities low angle to moderate (5-42 deg), very closely to closely spaced (0.05-0.45 ft) [VASSALBORO FORMATION].
Rock Mass Quality: Very Poor
Rock Core Rate (min:sec)
9.5-10.5 ft (1:56)
10.5-11.5 ft (1:22)
11.5-12.5 ft (0:56)
12.5-13.5 ft (1:31)
13.5-14.5 ft (3:01)
50% Recovery
14.5

Bottom of Exploration at 14.5 feet below ground surface.
Boring backfilled with cuttings to ground surface.

Laboratory Testing Results/ AASHTO and Unified Class.

2DB:
WC = 21.2%

3DB:
GTX #620981
WC = 7.7%
Fines = 25.1%
A-2-4, SC

4D:
GTX #620982, 620959
WC = 22.0%
Fines = 68.2%
A-4, ML-CL
LL = 23
PL = 17
PI = 6
LI = 0.8

5D:
GTX #620985
WC = 15.6%
Fines = 41.4%
A-4, SM

Remarks:

Hammer calibration obtained from SW Cole calibration report.
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Boring No.: BB-FDR-211

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-212
WIN: 023627.00

Driller:	SW Cole	**Elevation (ft.)**	159.0 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:	5/11/21 (13:50) ; 5/11/21 (14:55)	**Drilling Method:**	Pin Auger / Cased Wash	**Core Barrel:**	NQ
Boring Location:	N 368088.769 ft, E 1051665.502 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 140lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/21	0.00 - 2.00	7/9/10/4	19	31	76	158.6		1DA, Top 5 in: Brown, damp, very stiff, SILT, some fine to medium sand, trace fine gravel, non-plastic, organic material (TOPSOIL) 0.4	
							105			1DB, Bottom 16 in: Brown, dry, dense, fine to coarse SAND, some fine gravel, little silt, 2 distinct layers of asphalt (FILL)	
	2D	24/21	2.00 - 4.00	4/5/10/17	15	24	OPEN			2D: Brown, moist, stiff, medium to coarse Sandy SILT, little fine gravel, non-plastic. q_p = 1.4 ksf (Pocket Penetrometer)(FILL)	
	3D	17.04/16	4.00 - 5.42	21/24/50(5")						3D: Brown, moist, very dense, fine to coarse SAND, some silt, trace fine gravel, well-graded (FILL) 5.0	GTX #620983 WC = 11.7% Fines = 33.1% A-2-4, SM
5								154.0		Top of bedrock at elev. 154.0 ft. Bedrock chips in wash at 5 ft bgs. Weathered bedrock noted from 5 to 7 ft bgs.	
	R1	66/60	7.00 - 12.50	RQD = 55%			NQ			7.0-7.6 ft: White, coarse grained, PEGMATITE; fresh (W1), extremely strong (R6), discontinuities low angle (13 deg), very closely to closely spaced (0.15-0.5 ft) [VASSALBORO FORMATION]. 7.6-12.0 ft: Grey, medium to coarse grained, moderately foliated, GNEISS; slightly weathered (W2), strong (R4), discontinuities low angle to moderate (5-45 deg), very closely to closely spaced (0.05-0.6 ft) [VASSALBORO FORMATION]. Rock Mass Quality: Fair Rock Core Rate (min:sec) 7-8 ft (2:34) 8-9 ft (1:43) 9-10 ft (2:36) 10-11 ft (3:44) 11-12 ft (4:16) 12-12.5 ft (2:20) 91% Recovery 12.5	
10											
								146.5		**Bottom of Exploration at 12.5 feet below ground surface.** Boring backfilled with cuttings to ground surface.	
15											
20											
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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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-213

WIN: 023627.00

Driller:	SW Cole	**Elevation (ft.)**	152.9 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:	5/13/21 (14:35) ; 5/14/21 (08:45)	**Drilling Method:**	Pin Auger / Cased Wash	**Core Barrel:**	NQ
Boring Location:	N 367983.42 ft, E 1051667.985 ft	**Casing ID/OD:**	5.5 in	**Water Level*:**	10.8 ft
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 140lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person

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

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/16	0.00 - 2.00	2/4/9/12	13	21	PUSH			Brown, moist, medium dense, medium to coarse SAND, some gravel, trace silt, well-graded (FILL)	
	2D	24/22	2.00 - 4.00	10/8/7/12	15	24	37			2DA, Top 11 in: Brown, moist, medium dense, medium to coarse SAND, little gravel, trace silt, well-graded (FILL) 2DB, Bottom 11 in: Brown, moist, medium dense, Silty fine to medium SAND, trace fine gravel, poorly-graded (FILL)	
							39	148.9		4.0	
	3D	24/19	4.00 - 6.00	17/15/21/35	36	58	42			Brown, moist, very dense, Silty fine to medium SAND, little fine gravel, poorly-graded (SAND AND GRAVEL)	3D: WC = 9.8%
5							109				
	4D	24/23	6.00 - 8.00	23/24/21/35	45	73	163			Brown, moist, very dense, coarse to fine SAND, little fine gravel, little silt, well-graded (SAND AND GRAVEL)	4D: GTX #620984 WC = 9.5% Fines = 19.8% A-2-4(0), SM
							22 60				
	5D	24/21	8.00 - 10.00	14/19/20/31	39	63	91			5DA, Top 6 in: Brown, moist, very dense, coarse to fine SAND, little fine gravel, little silt, well-graded (SAND AND GRAVEL)	
							120				
10	6D	13.92/8	10.00 - 11.16	21/29/50(2")	79	128				5DB, Bottom 15 in: Orange, damp, medium dense Gravelly SAND, little clay. SCHIST present, completely weathered (W5), extremely weak (W0), discontinuities horizontal and vertical (SAND AND GRAVEL) 6D: Orange brown, wet, very dense, SAND, some clay, little fine gravel, weathered biotite schist (SAND AND GRAVEL)	6D: GTX #620974 WC = 26.6% Fines = 27.2% A-2-4(0), SM
								140.9		12.0	
	R1	60/53	12.00 - 17.00	RQD = 19%			NQ			Top of bedrock at elev. 140.9 ft. Greenish white, fine grained, APLITE; moderately weathered (W3), medium strong (R3), discontinuities low angle to moderate (4-52 deg), very closely to closely spaced (0.1-0.6 ft) [VASSALBORO FORMATION]. Rock Mass Quality: Very Poor Rock Core Rate (min:sec) 12-13 ft (4:18) 13-14 ft (5:51) 14-15 ft (3:49) 15-16 ft (4:01) 16-17 ft (3:27) 88% Recovery	
15											
								135.9		17.0	
										Bottom of Exploration at 17.0 feet below ground surface. Boring backfilled with cuttings to ground surface.	
20											
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.

Boring No.: BB-FDR-213

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-214
WIN: 023627.00

Driller:	SW Cole	**Elevation (ft.)**	157.8 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:	5/12/21 (08:30) ; 5/12/21 (09:16)	**Drilling Method:**	Pin Auger / Cased Wash	**Core Barrel:**	NQ
Boring Location:	N 368073.057 ft, E 1051724.46 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 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	2/5/5/11	10	16	6			Brown, dry, medium dense, fine to coarse SAND, little silt, trace fine gravel, well-graded (FILL)	
							20				
	2D	3/0	2.00 - 2.25	50(3")			NQ			No recovery. Some broken rock in the shoe.	
	R1	60/56	2.80 - 7.80	RQD = 33%				155.0		2.8 Top of bedrock at elev. 155.0 ft. 2.8-3.9 ft: Grey, medium to coarse grained, strongly foliated, GNEISS; slightly weathered (W2), medium strong (R3), discontinuities horizontal to low angle (3-15 deg), very closely to closely spaced (0.05-0.2 ft) [VASSALBORO FORMATION]. 3.9-7.1 ft: White/grey, medium to coarse grained, moderately foliated, GNEISS; fresh (W1), medium strong (R3), discontinuities low angle to moderate (5-37 deg), very closely to closely spaced (0.1-0.7 ft) [VASSALBORO FORMATION]. Rock Mass Quality: Poor Rock Core Rate (min:sec) 2.8-3.8 ft (2:38) 3.8-4.8 ft (2:48) 4.8-5.8 ft (2:21) 5.8-6.8 ft (2:33) 6.8-7.8 ft (3:01) 86% Recovery	
5											
								150.0			
										7.8 **Bottom of Exploration at 7.8 feet below ground surface.** Boring backfilled with cuttings to ground surface.	
10											
15											
20											
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.

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-215 **WIN:** 023627.00

Driller:	SW Cole	**Elevation (ft.)**	154.5 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:	5/17/21 (11:05) ; 5/17/21 (14:00)	**Drilling Method:**	Pin Auger / Cased Wash	**Core Barrel:**	NQ
Boring Location:	N 367980.564 ft, E 1051763.733 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 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	24/14	0.00 - 2.00	4/8/7/13	15	24	5			Brown, damp, medium dense, medium to coarse SAND, some fine gravel, well-graded(FILL)	
							21				
	2D	24/18	2.00 - 4.00	16/11/12/9	23	37	32			Brown, damp, dense, fine to coarse SAND, some fine gravel, well-graded (FILL)	
							23				
	3D	24/16	4.00 - 6.00	7/4/34/50(6")	38	62	19	149.7		3DA, Top 10 in: Dark brown, moist, very dense, fine to coarse SAND, little fine gravel, little silt, well-graded (FILL) 4.8	3DA: GTX #620975 WC = 11.1% Fines = 12.4% A-1-b, SM
5							82	149.4		3DB, 10-13 in: Asphalt thickness of 3 in (ASPHALT) 5.1	
	4D	24/14	6.00 - 8.00	14/29/27/16	56	91	207			3DC, Bottom 3 in: Brown, damp, very dense, fine to coarse SAND, some gravel, poorly-graded (FILL) 4D: Brown, moist, very dense, fine to coarse SAND, little fine gravel, well-graded (FILL) Casing refusal at 6.2 ft bgs.	
	5D	2.5/0	8.50 - 8.71	50 (2.5")			NQ			5D: No recovery.	
	R1	60/59	8.75 - 13.75	RQD = 50%				145.8		8.8 Top of bedrock at elev. 145.8 ft. 8.75-10.05 ft: Grey, medium to coarse grained, moderately foliated, GNEISS; slightly weathered (W2), strong (R4), discontinuities low angle (5-23 deg), very closely spaced (0.05-0.15 ft). Consists of some quartz veins [VASSALBORO FORMATION]. 10.05-13.65 ft: Grey, medium to coarse grained, strongly foliated, GNEISS; fresh (W1), extremely strong (R6), discontinuities low angle (5-24 deg), very closely to closely spaced (0.05-0.7 ft) [VASSALBORO FORMATION]. Rock Mass Quality: Poor Rock Core Rate (min:sec) 8.75-9.5 ft (4:02) 9.5-9.75 ft (0:28) 9.75-10.75 ft (3:04) 10.75-11.75 ft (2:49) 11.75-12.75 ft (3:27) 12.75-13.75 ft (4:03) 98% Recovery	
10											
								140.8			
15											
20										13.8 **Bottom of Exploration at 13.8 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.

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-216
WIN: 023627.00

Driller:	SW Cole	**Elevation (ft.)**	157.0 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 (09:40) ; 5/12/21 (11:32)	**Drilling Method:**	Pin Auger / Cased Wash	**Core Barrel:**	NQ
Boring Location:	N 368073.115 ft, E 1051792.158 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 140lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/12	0.00 - 2.00	2/2/3/5	5	8	PUSH			Brown, moist, loose, fine to coarse SAND, some silt, trace fine gravel, well-graded (FILL)	
	2D	24/12	2.00 - 4.00	5/4/5/5	9	15				Brown, moist, medium dense, medium to coarse SAND, little fine gravel, some silt, well-graded (FILL)	
5	3D	24/11	4.00 - 6.00	4/3/2/3	5	8				Brown, moist, loose, fine to coarse SAND, some silt, little gravel, broken rock in top 4 in mixed with sand, well-graded (FILL).	3D: GTX #620976 WC = 12.6% Fines = 32.9% A-2-4(0), SM
	4D	24/12	6.00 - 8.00	5/4/5/5	9	15	48			4DA, Top 4 in: Brown, moist, medium dense, fine to coarse SAND, some silt, little gravel, well-graded (FILL).	4DB: GTX #620977 WC = 2.8% Fines = 4.6% A-1-a, GP
							100			4DB, Bottom 8 in: Grey/brown, loose, GRAVEL, some sand, trace silt, well-graded (FILL)	
	5D	3.6/4	8.00 - 8.30	50(4")			26	148.2		5D: Brown, moist, very dense, medium to coarse, Gravelly SAND, little silt, well-graded (FILL) — 8.8	
10	R1	66/58	9.40 - 14.90	RQD = 49%			NQ			Top of bedrock at elev. 148.2 ft. 9.4-10.7 ft and 12.0-14.2 ft: Grey, medium to coarse grained, moderately foliated, GNEISS; slightly weathered (W2), strong (R4), discontinuities low angle to moderate (10-53 deg), closely spaced (0.2-0.6 ft) [VASSALBORO FORMATION]. 10.7-12.0 ft: Greenish white, fine grained, APLITE; highly weathered (W4) very weak (R1), discontinuities low angle to moderate (7-35 deg), very closely to closely spaced (0.1-0.25 ft) [VASSALBORO FORMATION].	
15								142.1		Rock Mass Quality: Poor Rock Core Rate (min:sec) 9.4-10.4 ft (2:48) 10.4-11.4 ft (1:42) 11.4-12.4 ft (3:27) 12.4-12.9 ft (3:50) 12.9-13.9 ft (2:54) 13.9-14.9 ft (4:42) 87% Recovery — 14.9	
20										**Bottom of Exploration at 14.9 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.

Boring No.: BB-FDR-216

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-217
WIN: 023627.00

Driller:	SW Cole	**Elevation (ft.)**	156.4 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:	05/18/21 (8:20);05/25/21 (12:40)	**Drilling Method:**	Pin Auger / Cased Wash	**Core Barrel:**	NQ
Boring Location:	N 368485.335 ft, E 1050886.362 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 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	24/10	0.60 - 2.60	8/9/10/11	19	31	~~SSA~~ DAYL	155.8	Asphalt thickness of 7 in ——0.6	
									Brown, dry, dense, fine to coarse SAND, some fine gravel, broken concrete present, well-graded (FILL)	
									Used vac truck to daylight down to 6 ft bgs, no sampling In shoe of SS was a 1-1.5 in diameter piece of black PVC pipe. Work stopped to daylight hole to confirm no utilities	
5										
	2D	24/6	6.00 - 8.00	6/25/48/62	73	119	PUSH		Brown, wet, very dense, fine to coarse SAND, some fine gravel, little silt, well-graded (FILL)	2D: GTX #620978 WC = 7.8% Fines = 11.2% A-1-b, SW-SM
							24			
	3D	24/9	8.00 - 10.00	3/7/12/13	19	31	46		Brown, moist, dense, fine to coarse SAND, little fine gravel, trace silt, well-graded (FILL)	3D: WC = 10.3%
							38			
10	4D	24/6	10.00 - 12.00	5/4/3/3	7	11	OPEN		Brown, moist, medium dense, fine to coarse SAND, little fine gravel, trace silt, well-graded (FILL)	4D: GTX #620979 WC = 16.1% Fines = 9.5% A-3, SW-SM
15								141.4	Weathered rock in wash from 14.6-15 ft bgs Top of bedrock at elev. 141.4 ft. ——15.0	
	R1	60/60	15.00 - 20.00	RQD = 66%			NQ		White and grey, medium to coarse grained, moderately foliated, GNEISS; slightly weathered (W2), strong (R4), discontinuities low angle to moderate (12-38 deg), very closely to closely spaced (0.1-0.8 ft). Consists of a quartz band from 15.7 to 16.3 ft [VASSALBORO FORMATION]. Rock Mass Quality: Fair Rock Core Rate (min:sec) 15-16 ft (2:45) 16-17 ft (2:26) 17-18 ft (2:09) 18-19 ft (2:41) 19-20 ft (3:02) 100% Recovery ——20.0	
20								136.4		
25									**Bottom of Exploration at 20.0 feet below ground surface.** Boring backfilled with cuttings to ground surface.	

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.

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

Rock Core Photographs

APPENDIX B
ROCK CORE PHOTOGRAPHS
I-295 DESERT ROAD BRIDGE REPLACEMENT #5720 (EXIT 20)
FREEPORT, MAINE
MAINEDOT WIN 023627.00

Boring	Date Cored	Run	Depth Below Surface feet	Recovery		RQD	
				Feet	%	Feet	%
BB-FDR-201	5/10/2021	R1	8.5 - 13.5	4.4 / 5.0	88	1.8 / 5.0	36
BB-FDR-203	5/10/2021	R1	10.2 - 15.0	3.5 / 4.8	73	1.0 / 4.8	21
BB-FDR-204	5/11/2021	R1	15.0 - 20.0	4.8 / 5.0	95	2.5 / 5.0	50
BB-FDR-210	5/11/2021	R1	5.0 - 10.0	3.5 / 5.0	70	1.9 / 5.0	38

From top to bottom of photo:
Row 1 = BB-FDR-201 Run 1: 8.5 - 13.5 ft bgs
Row 2 = BB-FDR-203 Run 1: 10.2 - 15.0 ft bgs
Row 3 = BB-FDR-204 Run 1: 15.0 - 20.0 ft bgs
Row 4 = BB-FDR-210 Run 1: 5.0 - 10.0 ft bgs

APPENDIX B
ROCK CORE PHOTOGRAPHS
I-295 DESERT ROAD BRIDGE REPLACEMENT #5720 (EXIT 20)
FREEPORT, MAINE
MAINEDOT WIN 023627.00

Boring	Date Cored	Run	Depth Below Surface feet	Recovery		RQD	
				Feet	%	Feet	%
BB-FDR-210	5/11/2021	R2	10.0 - 15.0	5.0 / 5.0	100	4.3 / 5.0	86
BB-FDR-212	5/11/2021	R1	7.0 - 12.5	5.0 / 5.5	91	3.3 / 5.5	60
BB-FDR-214	5/12/2021	R1	2.8 - 7.8	4.3 / 5.0	86	2.1 / 5.0	42
BB-FDR-216	5/12/2021	R1	9.4 - 14.9	4.8 / 5.5	87	2.8 / 5.5	51

From top to bottom of photo:
Row 1 = BB-FDR-210 Run 2: 10.0 - 15.0 ft bgs
Row 2 = BB-FDR-212 Run 1: 7.0 - 12.5 ft bgs
Row 3 = BB-FDR-214 Run 1: 2.8 - 7.8 ft bgs
Row 4 = BB-FDR-216 Run 1: 9.4 - 14.9 ft bgs

APPENDIX B
ROCK CORE PHOTOGRAPHS
I-295 DESERT ROAD BRIDGE REPLACEMENT #5720 (EXIT 20)
FREEPORT, MAINE
MAINEDOT WIN 023627.00

Boring	Date Cored	Run	Depth Below Surface feet	Recovery		RQD	
				Feet	%	Feet	%
BB-FDR-208	5/13/2021	R1	8.4 - 12.8	3.9 / 4.4	89	1.2 / 4.4	27
BB-FDR-211	5/13/2021	R1	9.5 - 14.5	2.5 / 5.0	50	1.0 / 5.0	20
BB-FDR-209	5/14/2021	R1	7.5 - 12.5	4.6 / 5.0	92	1.2 / 5.0	24
BB-FDR-213	5/14/2021	R1	12.0 - 17.0	4.4 / 5.0	88	1.6 / 5.0	32

From top to bottom of photo:
Row 1 = BB-FDR-208 Run 1: 8.4 - 12.8 ft bgs
Row 2 = BB-FDR-211 Run 1: 9.5 - 14.5 ft bgs
Row 3 = BB-FDR-209 Run 1: 7.5 - 12.5 ft bgs
Row 4 = BB-FDR-213 Run 1: 12.0 - 17.0 ft bgs

GOLDER
MEMBER OF WSP

APPENDIX B
ROCK CORE PHOTOGRAPHS
I-295 DESERT ROAD BRIDGE REPLACEMENT #5720 (EXIT 20)
FREEPORT, MAINE
MAINEDOT WIN 023627.00

Boring	Date Cored	Run	Depth Below Surface feet			Recovery				RQD			
						Feet			%	Feet			%
BB-FDR-215	5/17/2021	R1	8.8	-	13.8	4.9	/	5.0	98	2.4	/	5.0	48
BB-FDR-206	5/24/2021	R1	25.5	-	30.5	4.8	/	5.0	96	2.5	/	5.0	50
BB-FDR-202	5/25/2021	R1	19.5	-	24.5	4.5	/	5.0	90	3.9	/	5.0	78
BB-FDR-217	5/27/2021	R1	15.0	-	20.0	5.0	/	5.0	100	3.5	/	5.0	70

From top to bottom of photo:
Row 1 = BB-FDR-215 Run 1: 8.8 - 13.8 ft bgs
Row 2 = BB-FDR-206 Run 2: 25.5 - 30.5 ft bgs
Row 3 = BB-FDR-202 Run 1: 19.5 - 24.5 ft bgs
Row 4 = BB-FDR-217 Run 1: 15.0 - 20.0 ft bgs

APPENDIX B
ROCK CORE PHOTOGRAPHS
I-295 DESERT ROAD BRIDGE REPLACEMENT #5720 (EXIT 20)
FREEPORT, MAINE
MAINEDOT WIN 023627.00

Boring	Date Cored	Run	Depth Below Surface feet	Recovery		RQD	
				Feet	%	Feet	%
BB-FDR-205	5/26/2021	R1	19.0 - 24.0	4.9 / 5.0	98	1.1 / 5.0	22

From top to bottom of photo:
Row 1 = BB-FDR-205 Run 1: 19.0 - 24.0 ft bgs

APPENDIX C

Laboratory Test Results

Client:	Golder Associates			
Project:	Freeport Desert Rd Bridge Ex 20			
Location:	Freeport, ME		Project No:	GTX-313770
Boring ID:	---	Sample Type: ---	Tested By:	ckg
Sample ID:	---	Test Date: 06/10/21	Checked By:	bfs
Depth :	---	Test Id: 620996		

Moisture Content of Soil and Rock - ASTM D2216

Boring ID	Sample ID	Depth	Description	Moisture Content,%
BB-FDR-201	3D	4-6 ft	Moist, yellowish brown silty sand with gravel	11.1
BB-FDR-201	4D	6-8 ft	Moist, light brown silt	9.1
BB-FDR-202	2D	7-9 ft	Moist, yellowish brown sand with silt	10.0
BB-FDR-202	4D	15-17 ft	Moist, olive brown clay with sand	25.9
BB-FDR-203	5DB	8-10 ft	Moist, grayish brown silty sand with gravel	11.0
BB-FDR-204	4D	6-8 ft	Moist, yellowish brown sand with silt	9.3
BB-FDR-204	6DB	10-12 ft	Moist, yellowish brown silty sand	17.5
BB-FDR-205	1D	6-8 ft	Moist, yellowish brown sand with silt	8.8
BB-FDR-205	2DA	8-10 ft	Moist, brownish gray sand with silt and gravel	8.2
BB-FDR-205	4DB	15-17 ft	Moist, light olive silt with sand	29.7

Notes: Temperature of Drying : 110º Celsius

Client:	Golder Associates			
Project:	Freeport Desert Rd Bridge Ex 20			
Location:	Freeport, ME		Project No:	GTX-313770
Boring ID:	---	Sample Type: ---	Tested By:	ckg
Sample ID:	---	Test Date: 06/10/21	Checked By:	bfs
Depth :	---	Test Id: 621011		

Moisture Content of Soil and Rock - ASTM D2216

Boring ID	Sample ID	Depth	Description	Moisture Content,%
BB-FDR-206	4D	15-17 ft	Moist, yellowish brown sand with silt	9.8
BB-FDR-206	5DB	20-22 ft	Moist, gray silty sand	11.0
BB-FDR-207	1D	5-7 ft	Moist, yellowish brown sand with silt	6.5
BB-FDR-207	5D	20-22 ft	Moist, grayish brown sand with silt and gravel	8.3
BB-FDR-207	6D	25-27 ft	Moist, olive gray clay with sand	28.0
BB-FDR-208	3D	4-6 ft	Moist, grayish brown clay with sand	27.0
BB-FDR-208	4D	6-8 ft	Moist, olive brown clay with sand	26.7

Notes: Temperature of Drying : 110º Celsius

Client:	Golder Associates				
Project:	Freeport Desert Rd Bridge Ex 20				
Location:	Freeport, ME			Project No:	GTX-313770
Boring ID:	---	Sample Type:	---	Tested By:	ckg
Sample ID:	---	Test Date:	06/10/21	Checked By:	bfs
Depth :	---	Test Id:	621003		

Moisture Content of Soil and Rock - ASTM D2216

Boring ID	Sample ID	Depth	Description	Moisture Content,%
BB-FDR-211	2DB	2-4 ft	Moist, olive brown clay	21.2
BB-FDR-211	3DB	4-6 ft	Moist, yellowish brown silty sand with gravel	7.7
BB-FDR-211	4D	6-8 ft	Moist, olive brown sandy silty clay	22.0
BB-FDR-211	5D	8-10 ft	Moist, olive brown clayey sand	15.6
BB-FDR-212	3D	4-6 ft	Moist, olive yellow silty sand	11.7
BB-FDR-213	6D	10-12 ft	Moist, yellowish brown silty sand	26.6
BB-FDR-213	3D	4-6 ft	Moist, yellowish brown silt	9.8
BB-FDR-213	4D	8-10 ft	Moist, yellowish brown silty sand with gravel	9.5
BB-FDR-215	3DA	4-6 ft	Moist, dark yellowish brown silty sand with gravel	11.1

Notes: Temperature of Drying : 110º Celsius

Client:	Golder Associates				
Project:	Freeport Desert Rd Bridge Ex 20				
Location:	Freeport, ME			Project No:	GTX-313770
Boring ID:	---	Sample Type:	---	Tested By:	ckg
Sample ID:	---	Test Date:	06/10/21	Checked By:	bfs
Depth :	---	Test Id:	621008		

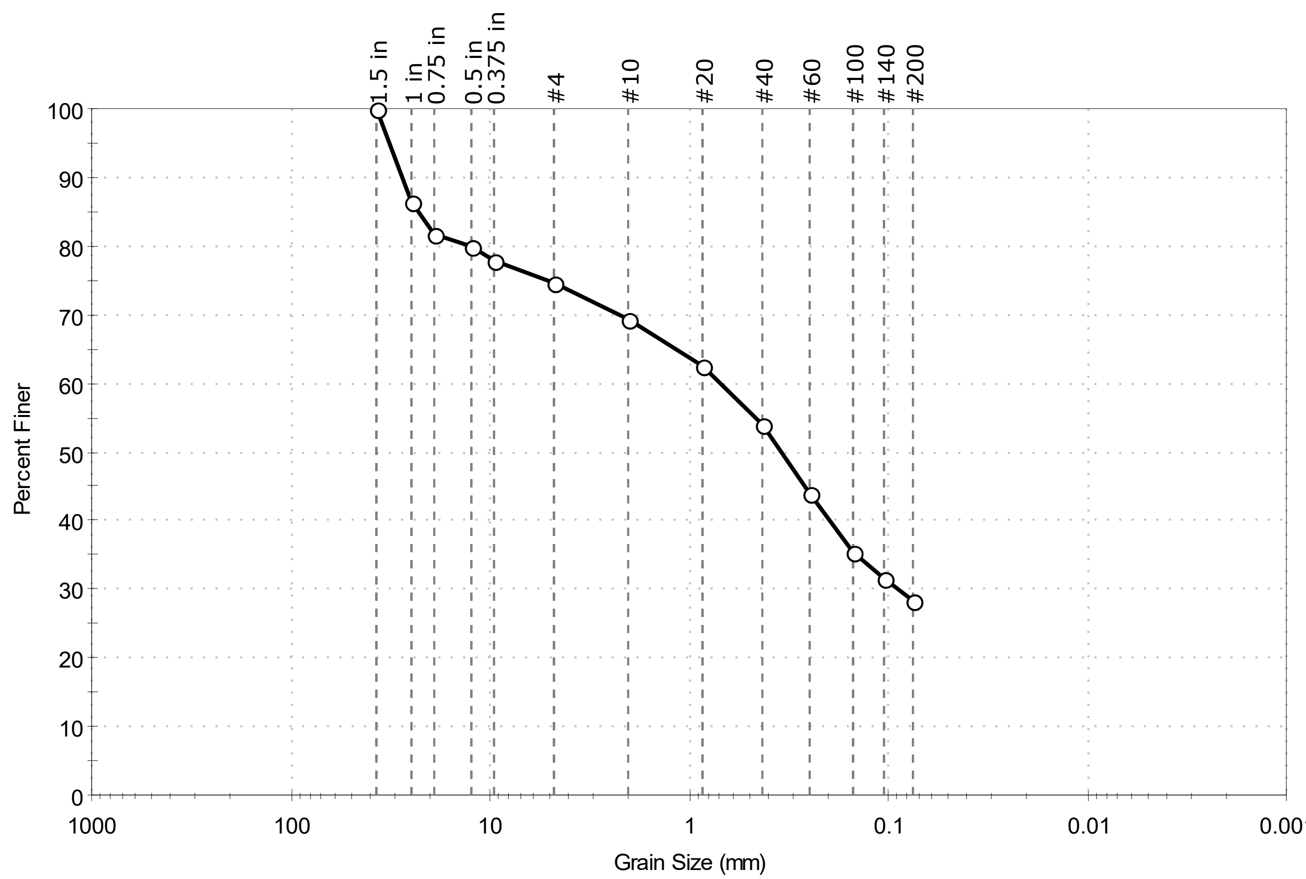
Moisture Content of Soil and Rock - ASTM D2216

Boring ID	Sample ID	Depth	Description	Moisture Content,%
BB-FDR-216	3D	4-6 ft	Moist, dark yellowish brown clayey sand with gravel	12.6
BB-FDR-216	4DB	6-8 ft	Moist, yellowish brown gravel with sand	2.8
BB-FDR-217	2D	6-8 ft	Moist, light brown sand with silt and gravel	7.8
BB-FDR-217	3D	8-10 ft	Moist, yellowish brown silty sand	10.3
BB-FDR-217	4D	10-12 ft	Moist, brownish yellow sand with silt	16.1

Notes: Temperature of Drying : 110º Celsius

Client: Golder Associates
Project: Freeport Desert Rd Bridge Ex 20
Location: Freeport, ME
Project No: GTX-313770
Boring ID: BB-FDR-201
Sample Type: jar
Tested By: ckg
Sample ID: 3D
Test Date: 06/11/21
Checked By: bfs
Depth : 4-6 ft
Test Id: 620960
Test Comment: ---
Visual Description: Moist, yellowish brown silty sand with gravel
Sample Comment: ---

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	25.4	46.3	28.3

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.5 in	37.50	100		
1 in	25.00	86		
0.75 in	19.00	82		
0.5 in	12.50	80		
0.375 in	9.50	78		
#4	4.75	75		
#10	2.00	69		
#20	0.85	63		
#40	0.42	54		
#60	0.25	44		
#100	0.15	35		
#140	0.11	32		
#200	0.075	28		

Coefficients

D_{85} =22.9963 mm
D_{30} =0.0897 mm
D_{60} =0.6915 mm
D_{15} =N/A
D_{50} =0.3437 mm
D_{10} =N/A
C_u =N/A
C_c =N/A

Classification

ASTM N/A

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

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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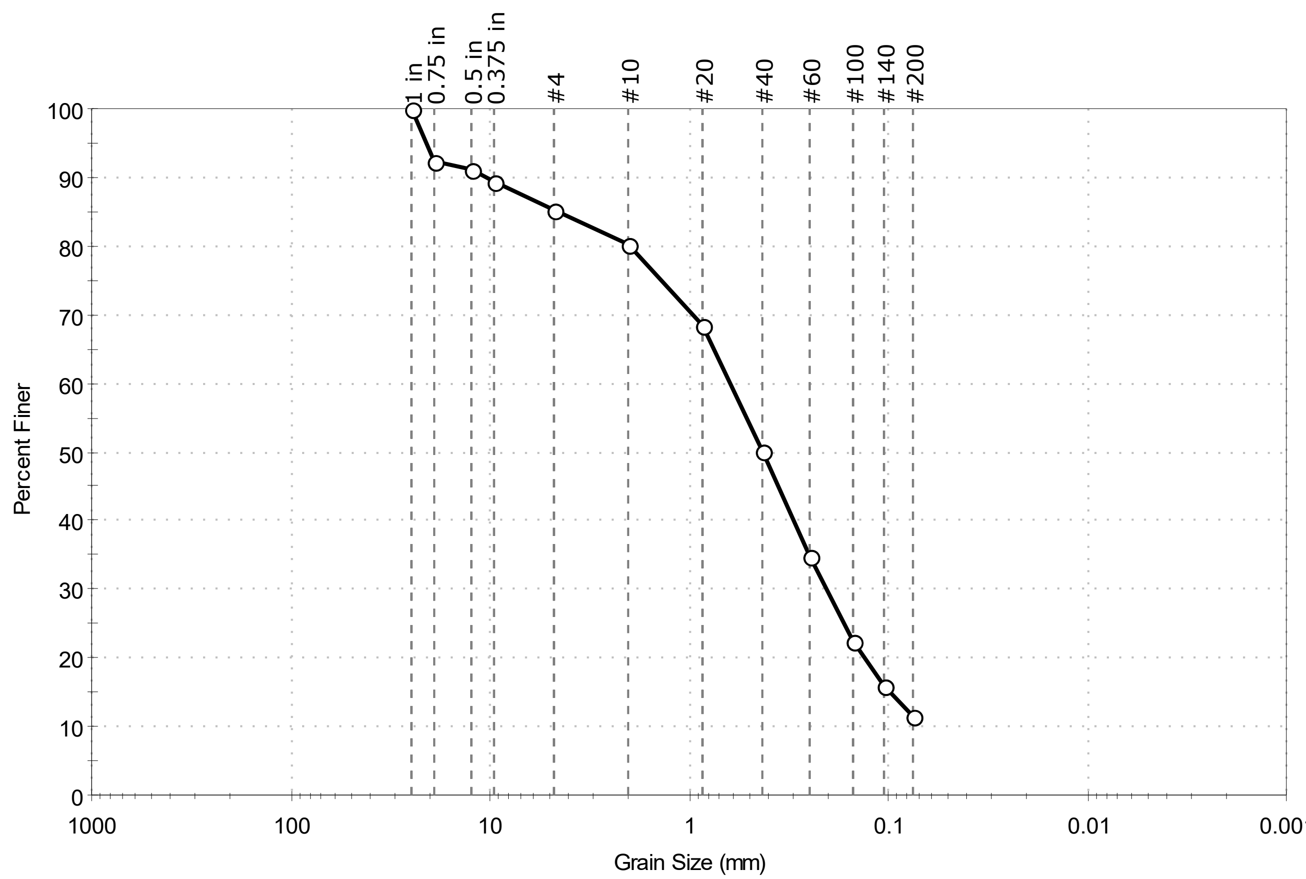
Client:	Golder Associates			
Project:	Freeport Desert Rd Bridge Ex 20			
Location:	Freeport, ME		Project No:	GTX-313770
Boring ID:	BB-FDR-202	Sample Type: jar	Tested By:	ckg
Sample ID:	2D	Test Date: 06/14/21	Checked By:	bfs
Depth :	7-9 ft	Test Id: 620961		

Test Comment: ---

Visual Description: Moist, yellowish brown sand with silt

Sample Comment: ---

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	14.6	73.8	11.6

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	92		
0.5 in	12.50	91		
0.375 in	9.50	89		
#4	4.75	85		
#10	2.00	80		
#20	0.85	68		
#40	0.42	50		
#60	0.25	35		
#100	0.15	22		
#140	0.11	16		
#200	0.075	12		

Coefficients

D_{85} =4.4701 mm
D_{30} =0.2058 mm
D_{60} =0.6167 mm
D_{15} =0.0977 mm
D_{50} =0.4212 mm
D_{10} =N/A
C_u =N/A
C_c =N/A

Classification

ASTM N/A

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

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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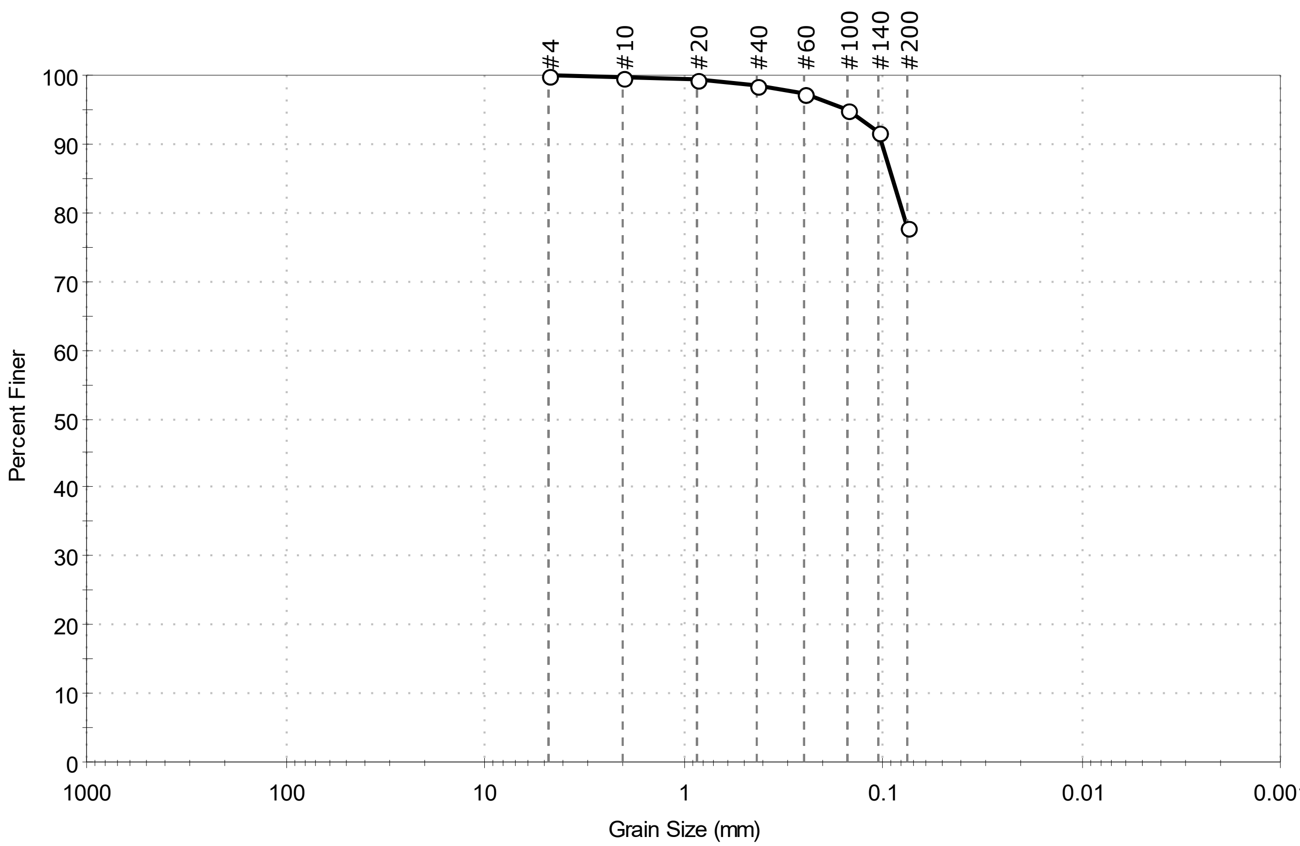
Client:	Golder Associates		
Project:	Freeport Desert Rd Bridge Ex 20		
Location:	Freeport, ME	Project No:	GTX-313770
Boring ID: BB-FDR-202	Sample Type: jar	Tested By:	ckg
Sample ID: 4D	Test Date: 06/11/21	Checked By:	bfs
Depth : 15-17 ft	Test Id: 620962		

Test Comment: ---
Visual Description: Moist, olive brown clay with sand
Sample Comment: ---

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	22.0	78.0

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	100		
#20	0.85	99		
#40	0.42	99		
#60	0.25	97		
#100	0.15	95		
#140	0.11	92		
#200	0.075	78		

Coefficients

D_{85} =0.0895 mm — D_{30} =N/A
D_{60} =N/A — D_{15} =N/A
D_{50} =N/A — D_{10} =N/A
C_u =N/A — C_c =N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

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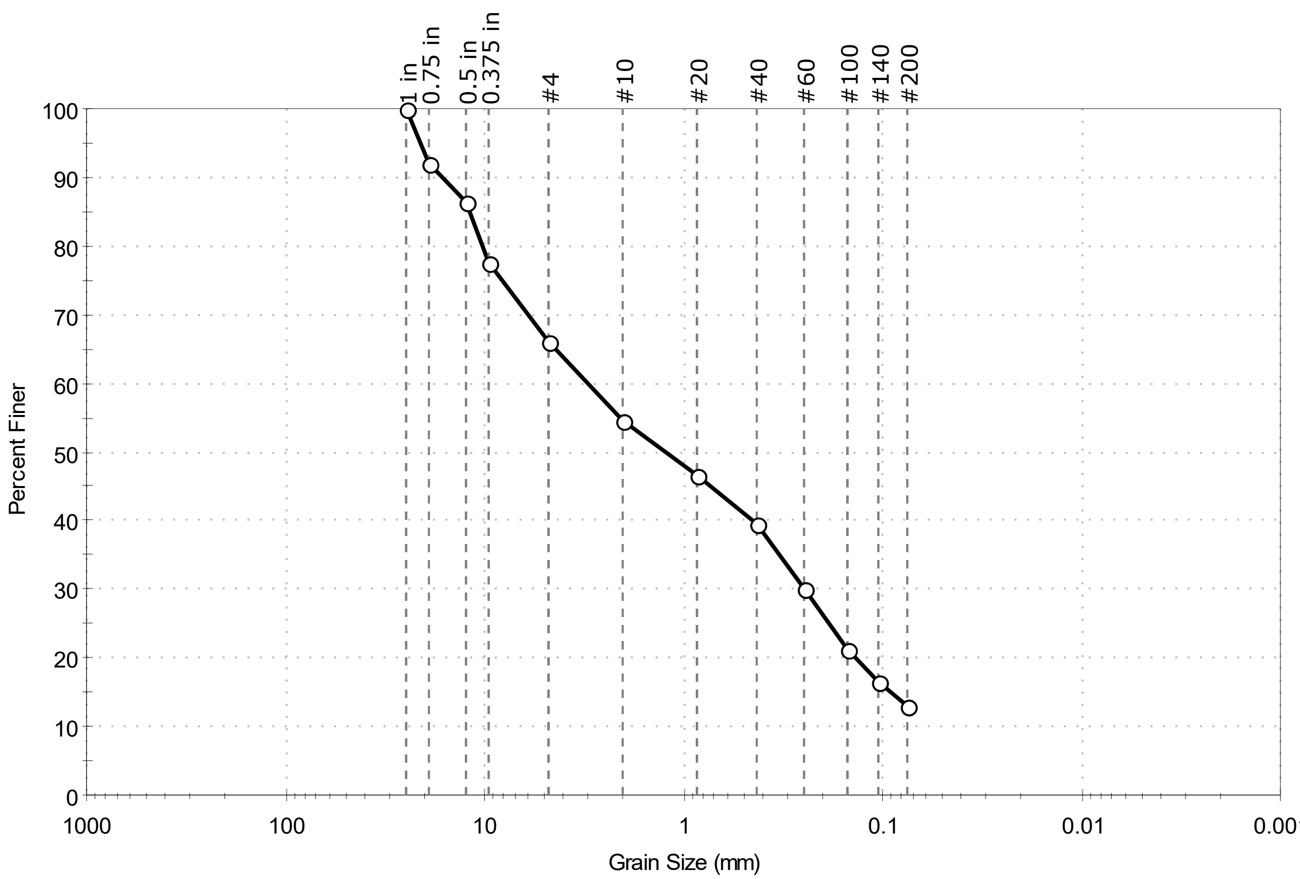
Client:	Golder Associates				
Project:	Freeport Desert Rd Bridge Ex 20				
Location:	Freeport, ME			Project No:	GTX-313770
Boring ID:	BB-FDR-203	Sample Type:	jar	Tested By:	ckg
Sample ID:	5DB	Test Date:	06/11/21	Checked By:	bfs
Depth :	8-10 ft	Test Id:	620963		
Test Comment:	---				
Visual Description:	Moist, grayish brown silty sand with gravel				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	33.8	53.2	13.0

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	92		
0.5 in	12.50	86		
0.375 in	9.50	78		
#4	4.75	66		
#10	2.00	54		
#20	0.85	47		
#40	0.42	40		
#60	0.25	30		
#100	0.15	21		
#140	0.11	17		
#200	0.075	13		

Coefficients

D_{85} =11.9990 mm | D_{30} =0.2490 mm
D_{60} =3.0106 mm | D_{15} =0.0915 mm
D_{50} =1.2307 mm | D_{10} =N/A
C_u =N/A | C_c =N/A

Classification

ASTM N/A

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

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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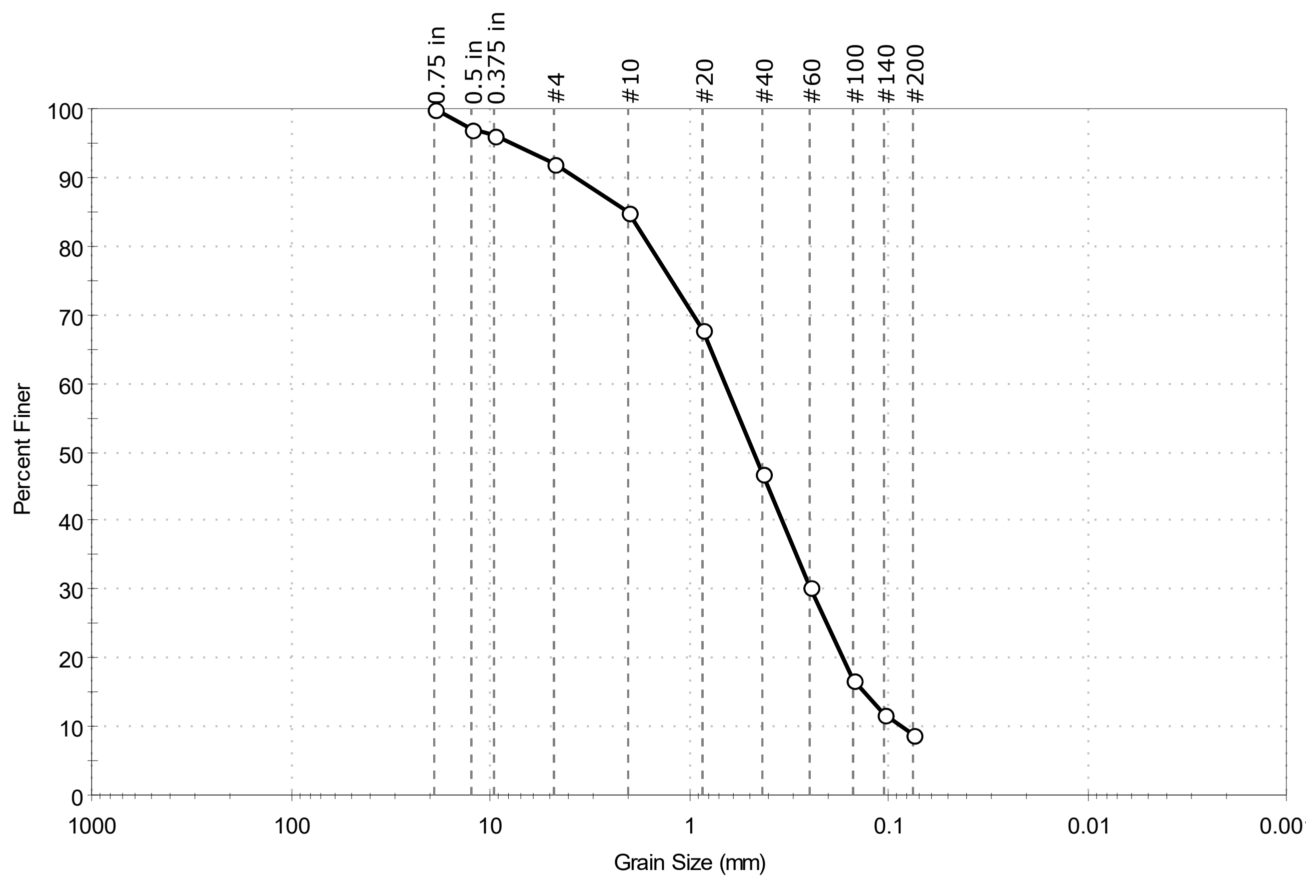
Client:	Golder Associates		
Project:	Freeport Desert Rd Bridge Ex 20		
Location:	Freeport, ME	Project No:	GTX-313770
Boring ID: BB-FDR-204	Sample Type: jar	Tested By:	ckg
Sample ID: 4D	Test Date: 06/11/21	Checked By:	bfs
Depth : 6-8 ft	Test Id: 620964		
Test Comment:	---		
Visual Description:	Moist, yellowish brown sand with silt		
Sample Comment:	---		

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	7.9	83.3	8.8

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	97		
0.375 in	9.50	96		
#4	4.75	92		
#10	2.00	85		
#20	0.85	68		
#40	0.42	47		
#60	0.25	30		
#100	0.15	17		
#140	0.11	12		
#200	0.075	8.8		

Coefficients

D_{85} =1.9981 mm D_{30} =0.2457 mm

D_{60} =0.6572 mm D_{15} =0.1324 mm

D_{50} =0.4722 mm D_{10} =0.0869 mm

C_u =7.563 C_c =1.057

Classification

ASTM   N/A

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

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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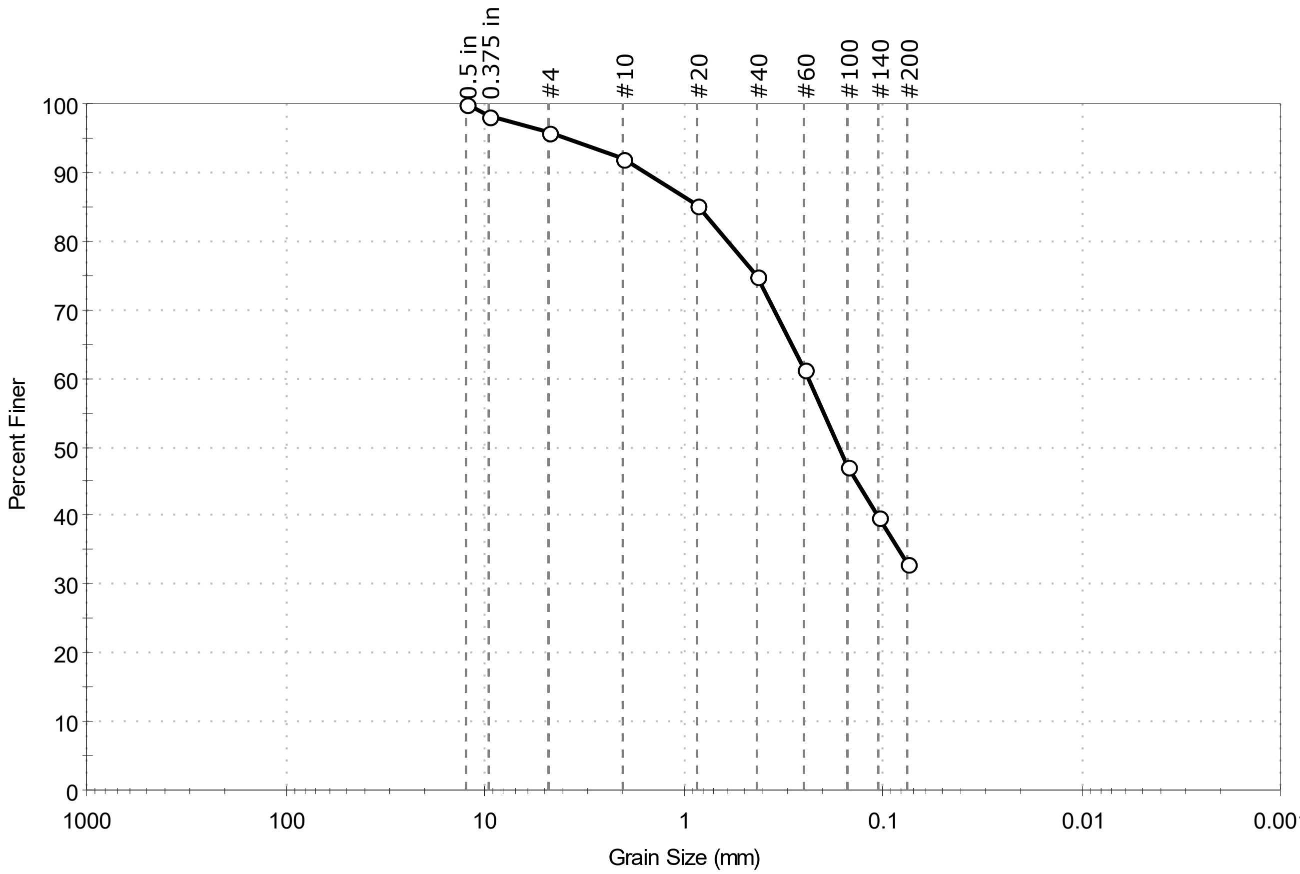
Client:	Golder Associates		
Project:	Freeport Desert Rd Bridge Ex 20		
Location:	Freeport, ME	Project No:	GTX-313770
Boring ID: BB-FDR-204	Sample Type: jar	Tested By:	ckg
Sample ID: 6DB	Test Date: 06/11/21	Checked By:	bfs
Depth : 10-12 ft	Test Id: 620965		

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

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	4.1	62.8	33.1

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.5 in	12.50	100		
0.375 in	9.50	98		
#4	4.75	96		
#10	2.00	92		
#20	0.85	85		
#40	0.42	75		
#60	0.25	61		
#100	0.15	47		
#140	0.11	40		
#200	0.075	33		

Coefficients

D_{85} =0.8300 mm D_{30} =N/A
D_{60} =0.2373 mm D_{15} =N/A
D_{50} =0.1654 mm D_{10} =N/A
C_u =N/A C_c =N/A

Classification

ASTM   N/A

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

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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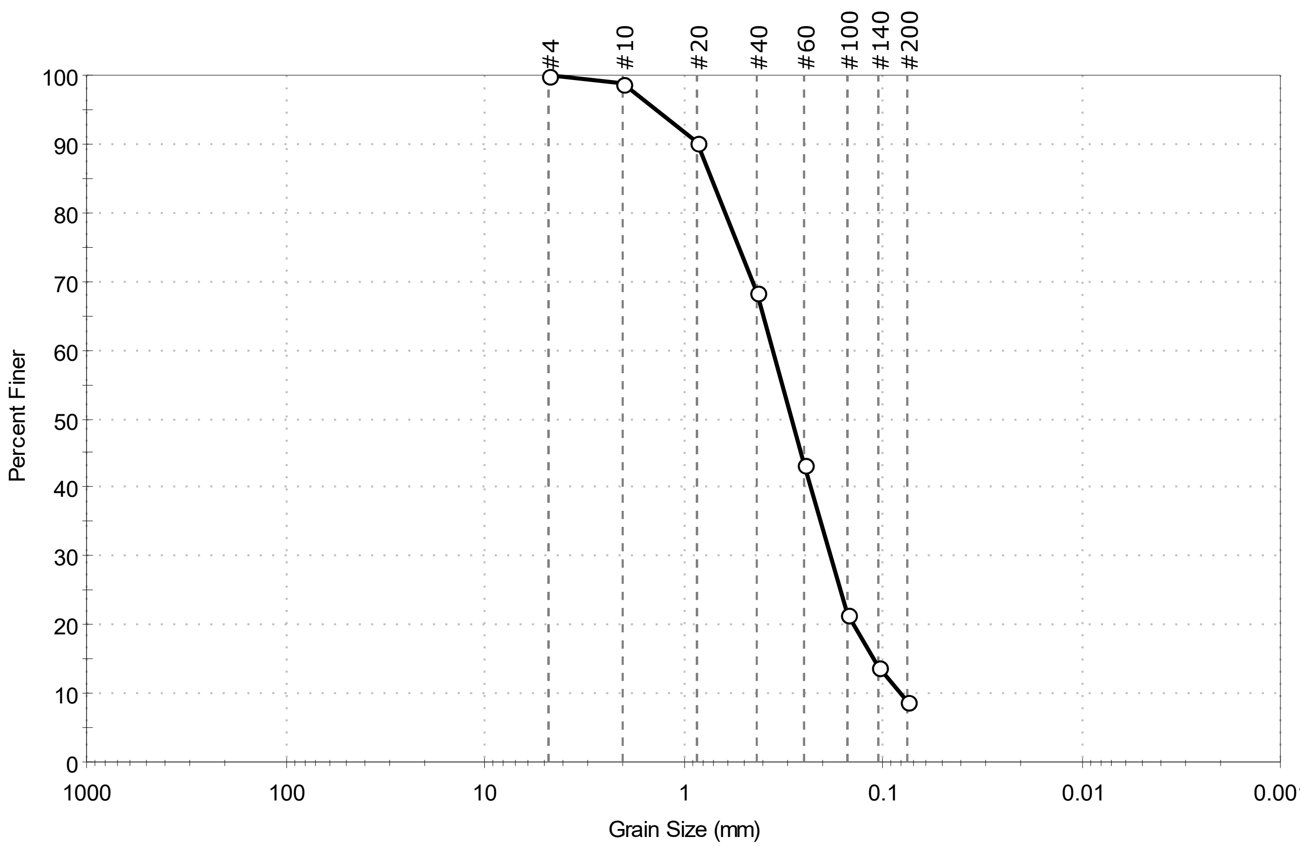
Client:	Golder Associates			
Project:	Freeport Desert Rd Bridge Ex 20			
Location:	Freeport, ME		Project No:	GTX-313770
Boring ID:	BB-FDR-205	Sample Type: jar	Tested By:	ckg
Sample ID:	1D	Test Date: 06/11/21	Checked By:	bfs
Depth :	6-8 ft	Test Id: 620966		

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

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	91.1	8.9

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	99		
#20	0.85	90		
#40	0.42	68		
#60	0.25	43		
#100	0.15	22		
#140	0.11	14		
#200	0.075	8.9		

Coefficients

D_{85} =0.7188 mm
D_{30} =0.1829 mm
D_{60} =0.3554 mm
D_{15} =0.1116 mm
D_{50} =0.2877 mm
D_{10} =0.0810 mm
C_u =4.388
C_c =1.162

Classification

ASTM N/A

AASHTO Fine Sand (A-3 (1))

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

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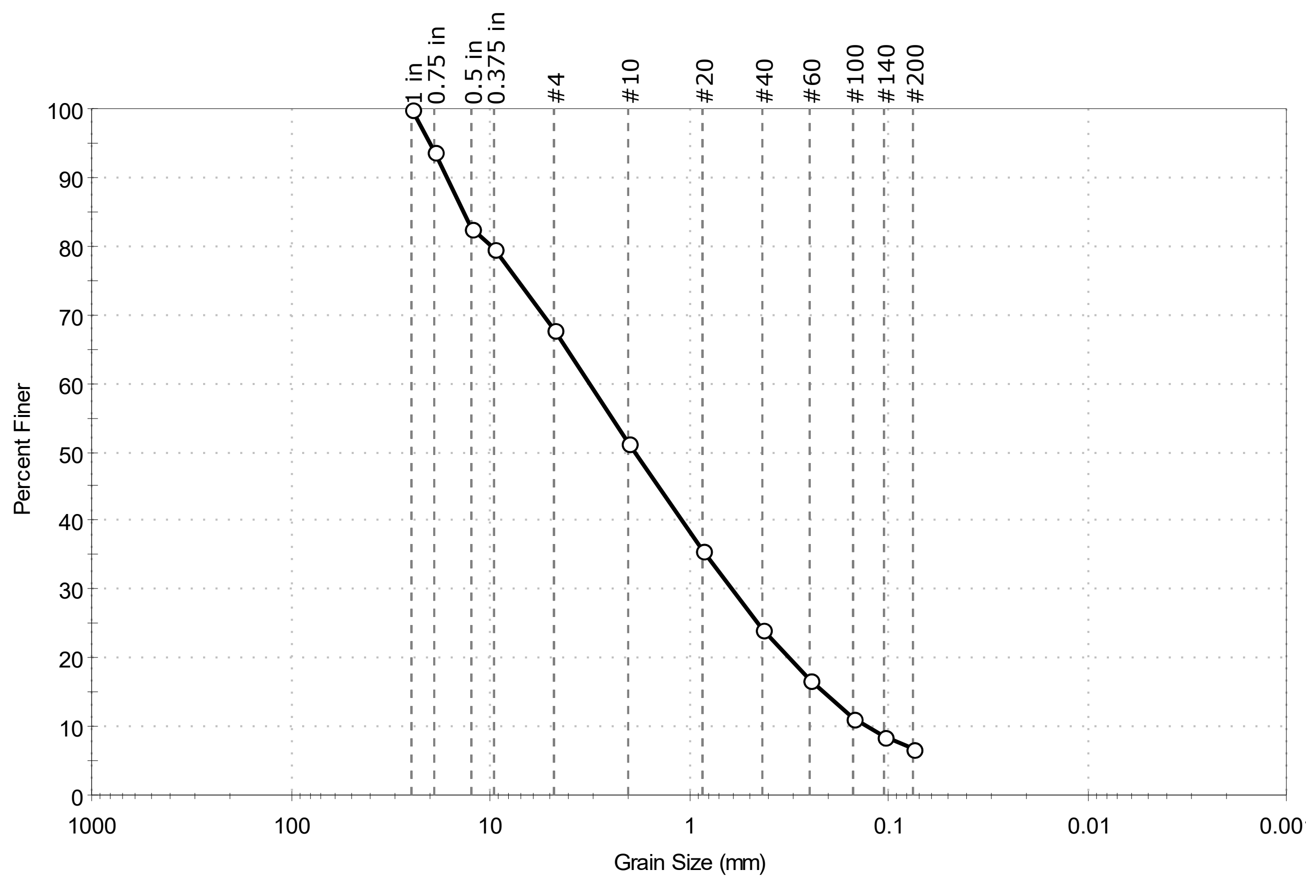
Client:	Golder Associates			
Project:	Freeport Desert Rd Bridge Ex 20			
Location:	Freeport, ME		Project No:	GTX-313770
Boring ID: BB-FDR-205		Sample Type: jar	Tested By:	ckg
Sample ID: 2DA		Test Date: 06/14/21	Checked By:	bfs
Depth : 8-10 ft		Test Id: 620967		
Test Comment:	---			
Visual Description:	Moist, brownish gray sand with silt and gravel			
Sample Comment:	--			

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	32.1	61.2	6.7

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	94		
0.5 in	12.50	82		
0.375 in	9.50	80		
#4	4.75	68		
#10	2.00	51		
#20	0.85	36		
#40	0.42	24		
#60	0.25	17		
#100	0.15	11		
#140	0.11	9		
#200	0.075	6.7		

Coefficients

D_{85} =13.7373 mm D_{30} =0.5987 mm
D_{60} =3.1459 mm D_{15} =0.2113 mm
D_{50} =1.8573 mm D_{10} =0.1275 mm
C_u =24.674 C_c =0.894

Classification

ASTM N/A

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

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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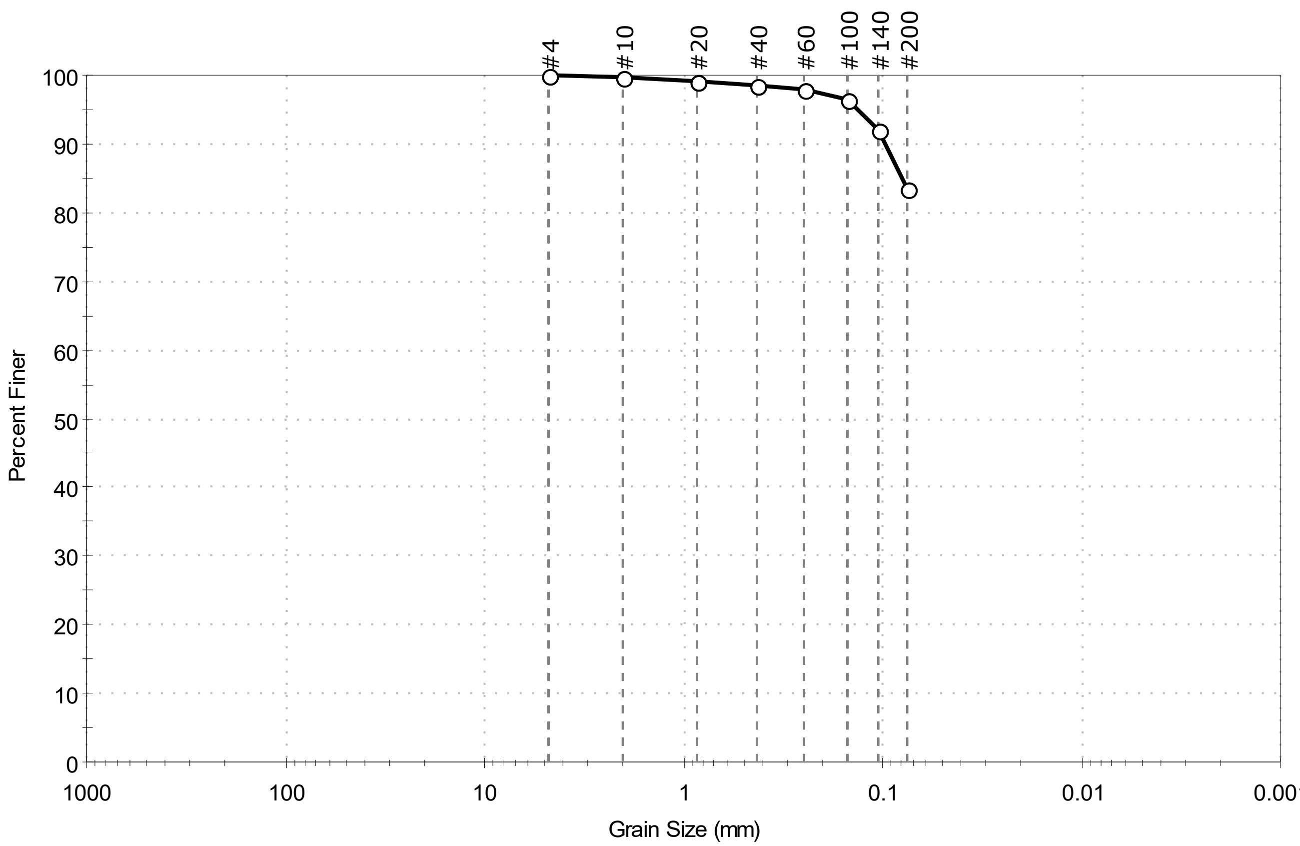
Client:	Golder Associates		
Project:	Freeport Desert Rd Bridge Ex 20		
Location:	Freeport, ME	Project No:	GTX-313770
Boring ID: BB-FDR-205	Sample Type: jar	Tested By:	ckg
Sample ID: 4DB	Test Date: 06/11/21	Checked By:	bfs
Depth : 15-17 ft	Test Id: 620968		
Test Comment:	---		
Visual Description:	Moist, light olive silt with sand		
Sample Comment:	---		

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	16.5	83.5

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	100		
#20	0.85	99		
#40	0.42	98		
#60	0.25	98		
#100	0.15	96		
#140	0.11	92		
#200	0.075	83		

Coefficients

D_{85} =0.0798 mm　　D_{30} =N/A

D_{60} =N/A　　D_{15} =N/A

D_{50} =N/A　　D_{10} =N/A

C_u =N/A　　C_c =N/A

Classification

ASTM　N/A

AASHTO　Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

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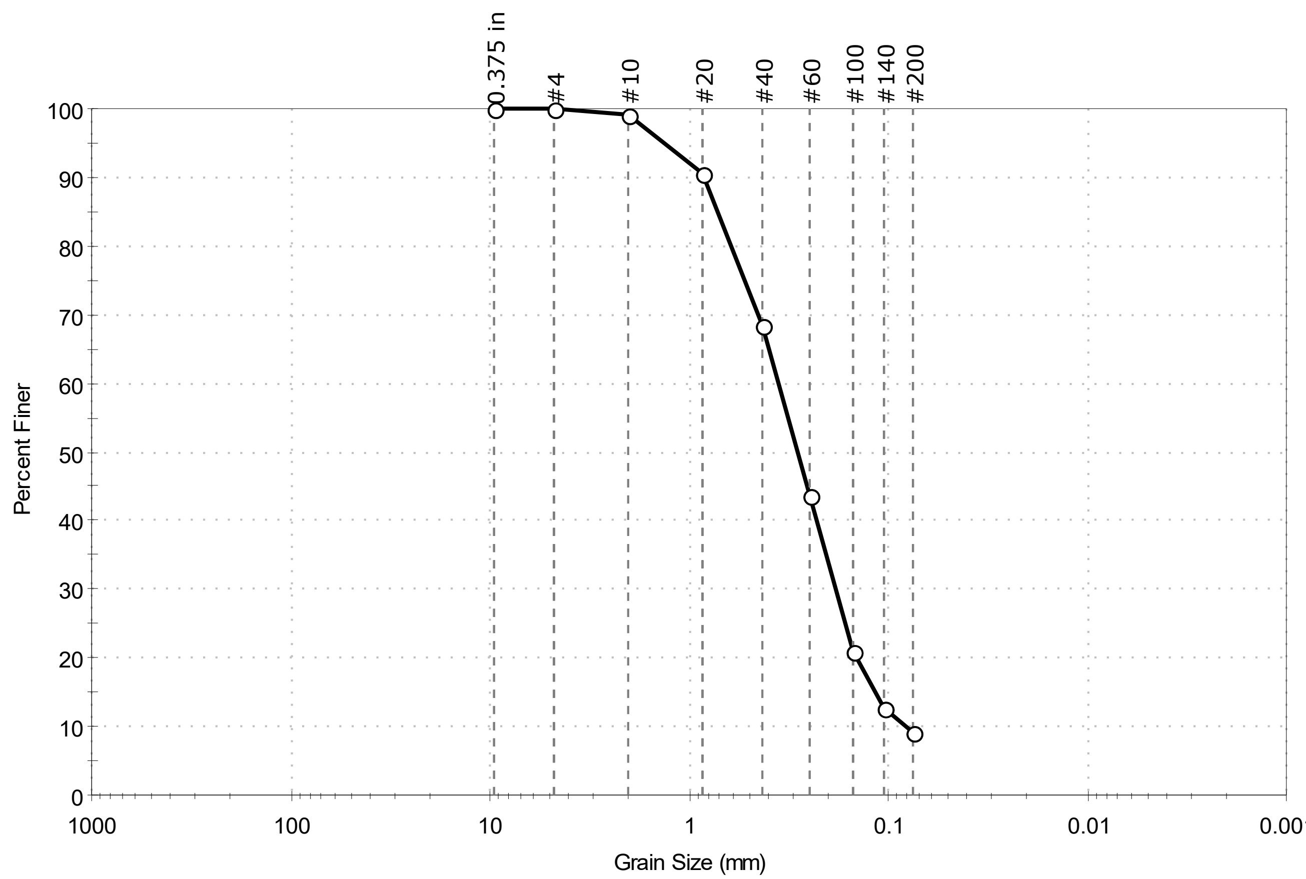
Client:	Golder Associates		
Project:	Freeport Desert Rd Bridge Ex 20		
Location:	Freeport, ME	Project No:	GTX-313770
Boring ID: BB-FDR-206	Sample Type: jar	Tested By:	ckg
Sample ID: 4D	Test Date: 06/14/21	Checked By:	bfs
Depth : 15-17 ft	Test Id: 620969		
Test Comment:	---		
Visual Description:	Moist, yellowish brown sand with silt		
Sample Comment:	---		

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.1	90.7	9.2

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.375 in	9.50	100		
#4	4.75	100		
#10	2.00	99		
#20	0.85	91		
#40	0.42	69		
#60	0.25	44		
#100	0.15	21		
#140	0.11	13		
#200	0.075	9.2		

Coefficients

D_{85} =0.7118 mm D_{30} =0.1842 mm

D_{60} =0.3544 mm D_{15} =0.1174 mm

D_{50} =0.2865 mm D_{10} =0.0814 mm

C_u =4.354 C_c =1.176

Classification

ASTM N/A

AASHTO Fine Sand (A-3 (1))

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

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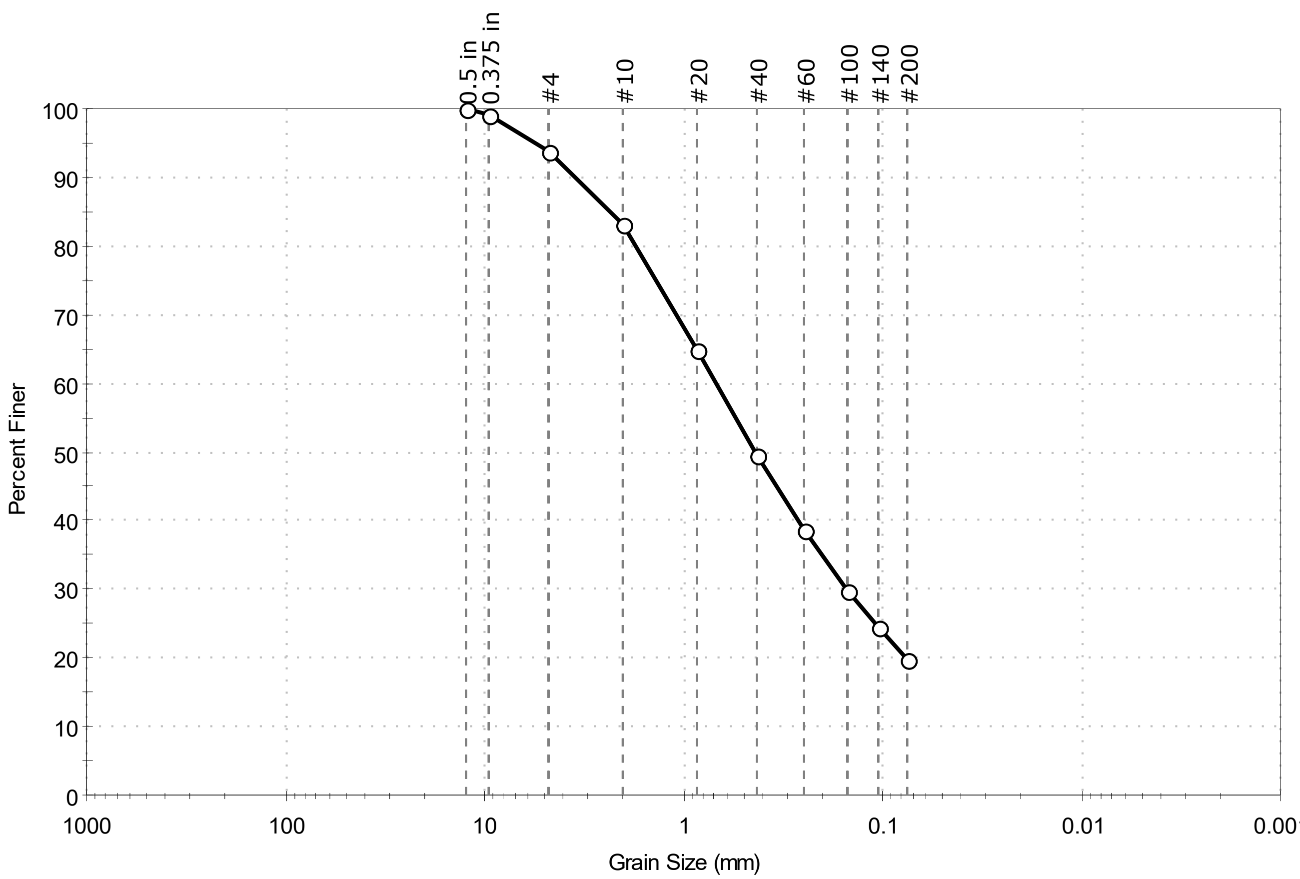
Client:	Golder Associates		
Project:	Freeport Desert Rd Bridge Ex 20		
Location:	Freeport, ME	Project No:	GTX-313770
Boring ID: BB-FDR-206	Sample Type: jar	Tested By:	ckg
Sample ID: 5DB	Test Date: 06/14/21	Checked By:	bfs
Depth : 20-22 ft	Test Id: 620970		
Test Comment:	---		
Visual Description:	Moist, gray silty sand		
Sample Comment:	---		

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	6.2	74.1	19.7

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.5 in	12.50	100		
0.375 in	9.50	99		
#4	4.75	94		
#10	2.00	83		
#20	0.85	65		
#40	0.42	50		
#60	0.25	39		
#100	0.15	30		
#140	0.11	24		
#200	0.075	20		

Coefficients

D_{85} =2.3258 mm
D_{30} =0.1526 mm
D_{60} =0.6795 mm
D_{15} =N/A
D_{50} =0.4329 mm
D_{10} =N/A
C_u =N/A
C_c =N/A

Classification

ASTM N/A

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

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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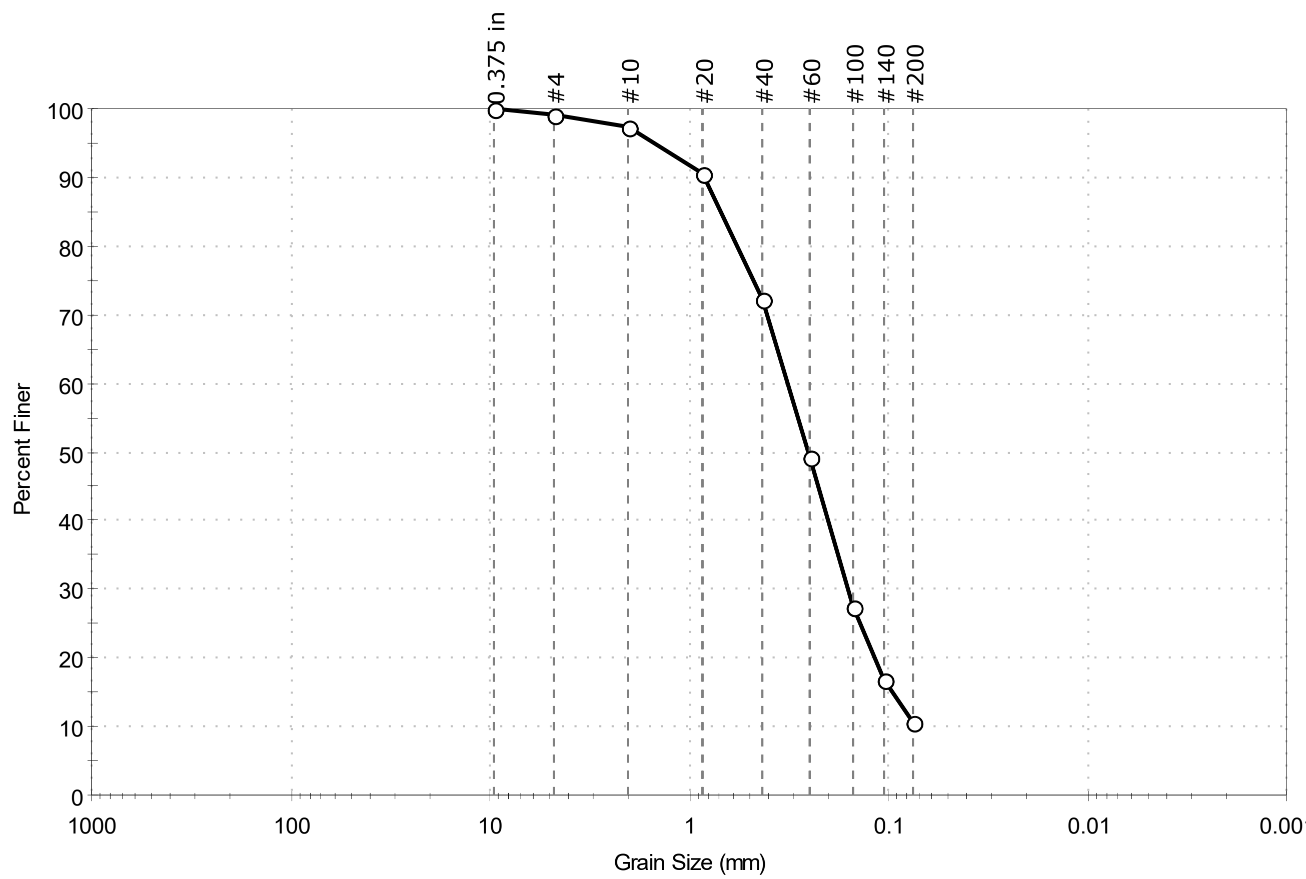
Client: Golder Associates			
Project: Freeport Desert Rd Bridge Ex 20			
Location: Freeport, ME		Project No:	GTX-313770
Boring ID: BB-FDR-207	Sample Type: jar	Tested By:	ckg
Sample ID: 1D	Test Date: 06/11/21	Checked By:	bfs
Depth : 5-7 ft	Test Id: 620971		

Test Comment: ---

Visual Description: Moist, yellowish brown sand with silt

Sample Comment: ---

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.9	88.4	10.7

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.375 in	9.50	100		
#4	4.75	99		
#10	2.00	97		
#20	0.85	91		
#40	0.42	72		
#60	0.25	49		
#100	0.15	28		
#140	0.11	17		
#200	0.075	11		

Coefficients

D_{85} =0.6872 mm D_{30} =0.1589 mm

D_{60} =0.3207 mm D_{15} =0.0960 mm

D_{50} =0.2550 mm D_{10} =N/A

C_u =N/A C_c =N/A

Classification

ASTM N/A

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

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

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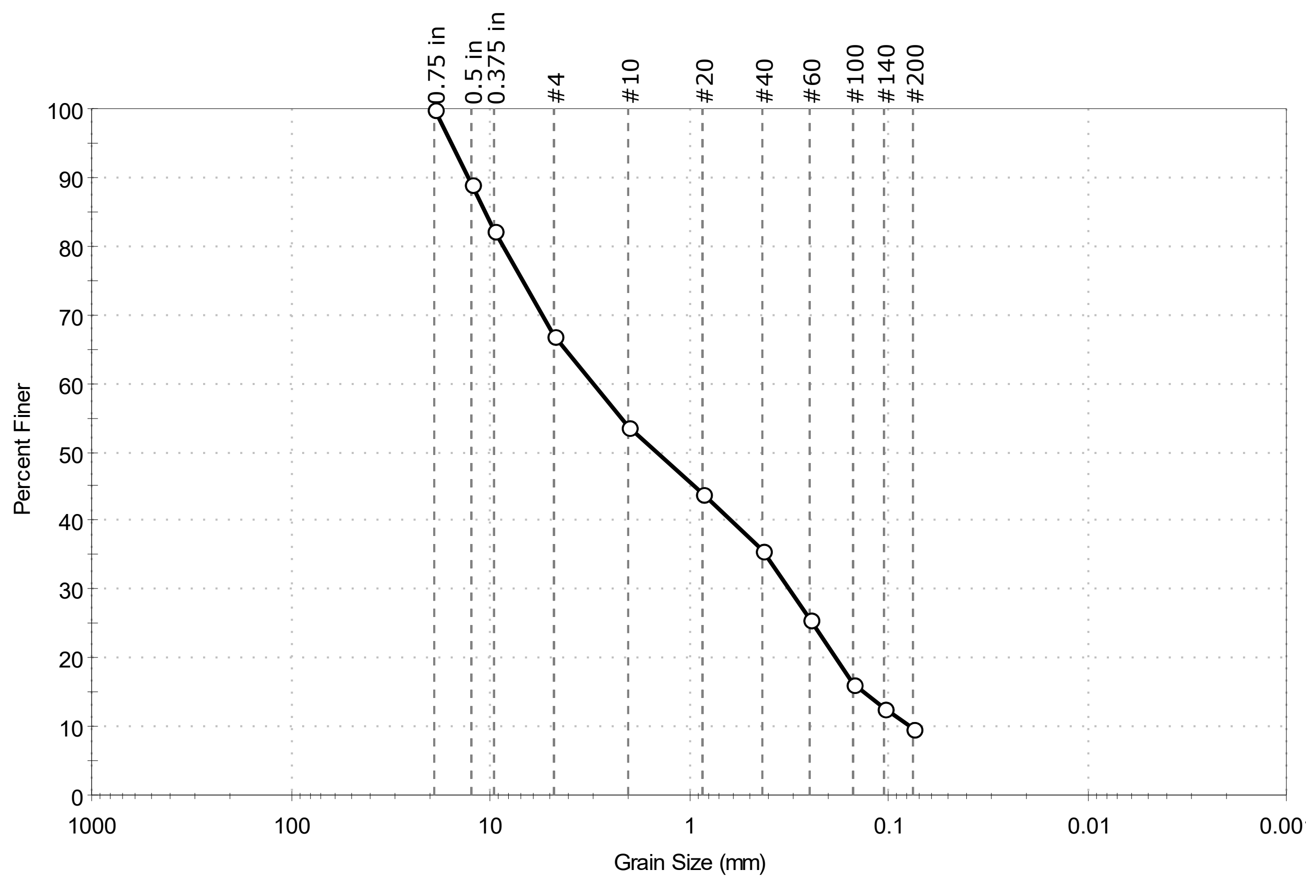
GeoTesting EXPRESS

Client:	Golder Associates		
Project:	Freeport Desert Rd Bridge Ex 20		
Location:	Freeport, ME	Project No:	GTX-313770
Boring ID: BB-FDR-207	Sample Type: jar	Tested By:	ckg
Sample ID: 5D	Test Date: 06/11/21	Checked By:	bfs
Depth : 20-22 ft	Test Id: 620972		

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

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	32.9	57.5	9.6

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	89		
0.375 in	9.50	82		
#4	4.75	67		
#10	2.00	54		
#20	0.85	44		
#40	0.42	36		
#60	0.25	26		
#100	0.15	16		
#140	0.11	13		
#200	0.075	9.6		

Coefficients

D_{85} =10.6107 mm	D_{30} =0.3154 mm
D_{60} =3.0121 mm	D_{15} =0.1330 mm
D_{50} =1.4526 mm	D_{10} =0.0782 mm
C_u =38.518	C_c =0.422

Classification

ASTM N/A

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

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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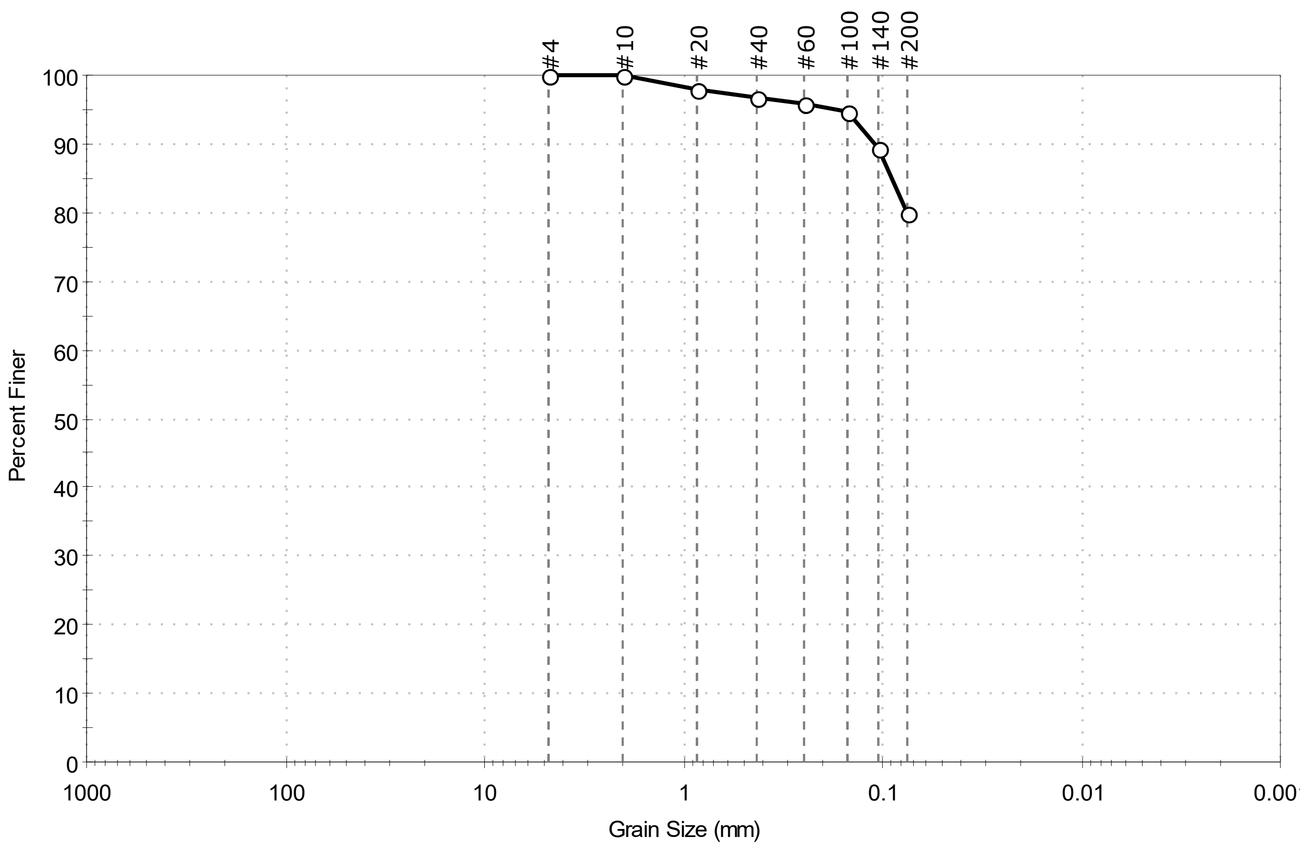
Client: Golder Associates			
Project: Freeport Desert Rd Bridge Ex 20			
Location: Freeport, ME		Project No:	GTX-313770
Boring ID: BB-FDR-207	Sample Type: jar	Tested By:	ckg
Sample ID: 6D	Test Date: 06/11/21	Checked By:	bfs
Depth : 25-27 ft	Test Id: 620973		

Test Comment: ---

Visual Description: Moist, olive gray clay with sand

Sample Comment: ---

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	20.0	80.0

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	100		
#20	0.85	98		
#40	0.42	97		
#60	0.25	96		
#100	0.15	95		
#140	0.11	89		
#200	0.075	80		

Coefficients

D_{85} = 0.0901 mm | D_{30} = N/A

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

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

C_u = N/A | C_c = N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

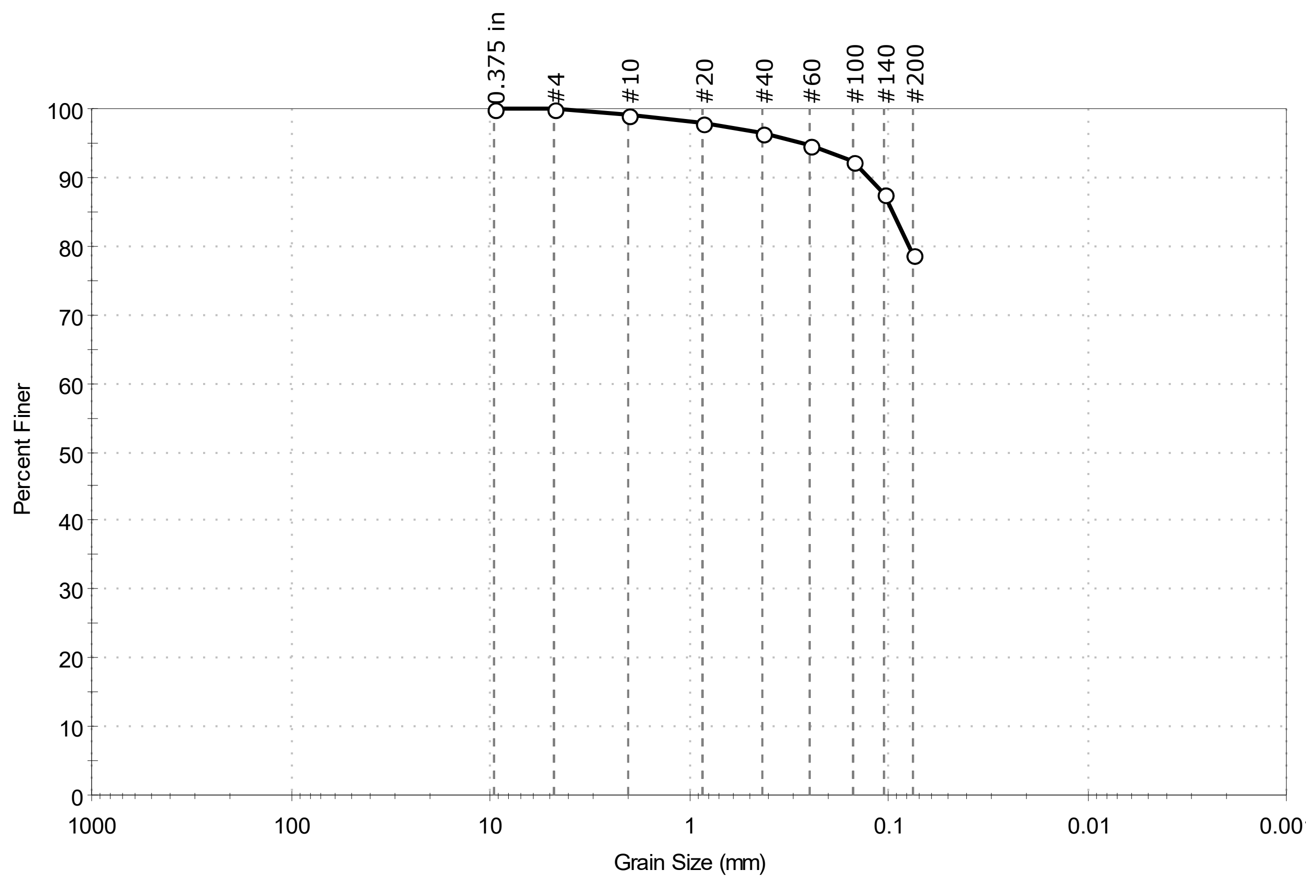
Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

Client:	Golder Associates			
Project:	Freeport Desert Rd Bridge Ex 20			
Location:	Freeport, ME		Project No:	GTX-313770
Boring ID:	BB-FDR-208	Sample Type: jar	Tested By:	ckg
Sample ID:	3D	Test Date: 06/11/21	Checked By:	bfs
Depth :	4-6 ft	Test Id: 620980		
Test Comment:	---			
Visual Description:	Moist, grayish brown clay with sand			
Sample Comment:	---			

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.1	21.0	78.9

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.375 in	9.50	100		
#4	4.75	100		
#10	2.00	99		
#20	0.85	98		
#40	0.42	96		
#60	0.25	95		
#100	0.15	92		
#140	0.11	88		
#200	0.075	79		

Coefficients

D_{85} =0.0953 mm D_{30} =N/A

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

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

C_u =N/A C_c =N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

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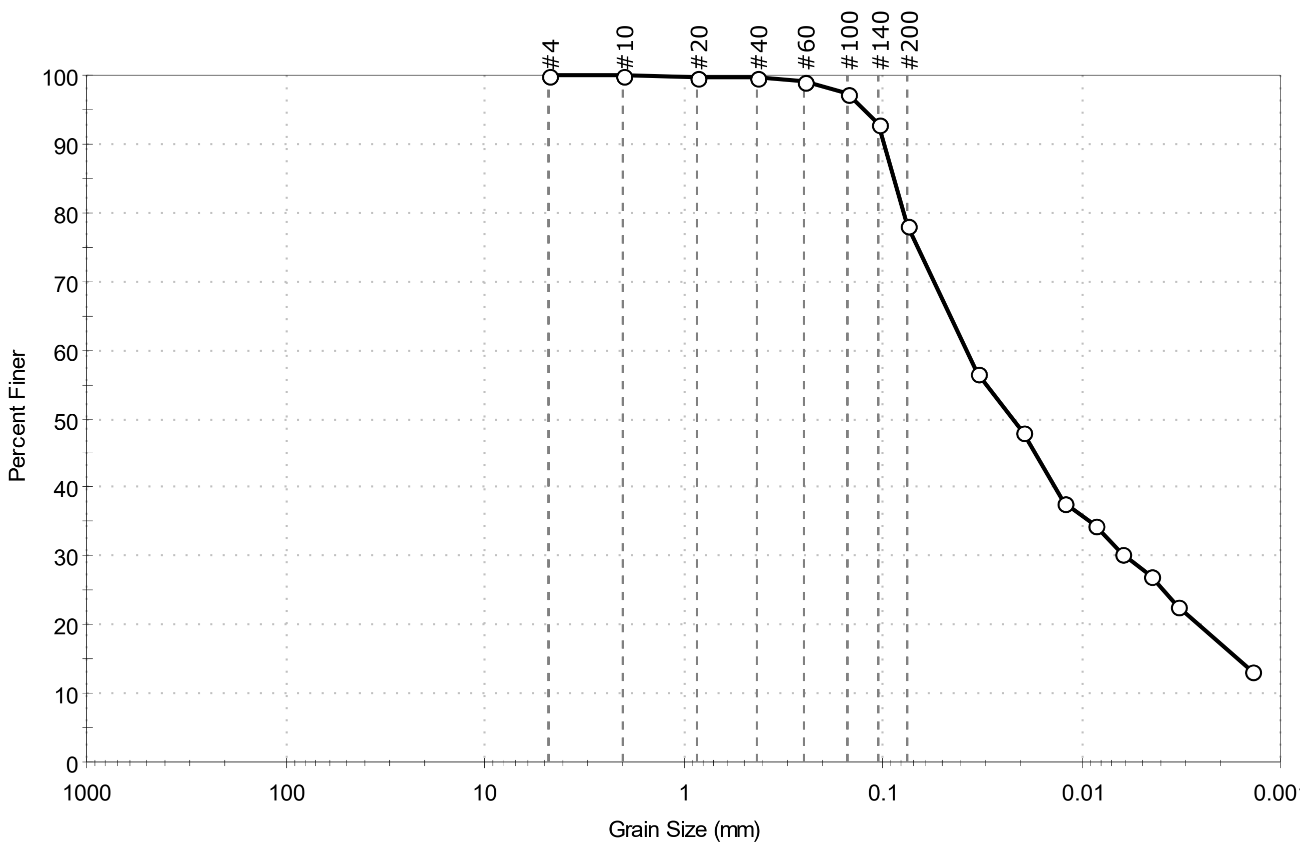
Client:	Golder Associates		
Project:	Freeport Desert Rd Bridge Ex 20		
Location:	Freeport, ME	Project No:	GTX-313770
Boring ID: BB-FDR-208	Sample Type: jar	Tested By:	ckg
Sample ID: 4D	Test Date: 06/15/21	Checked By:	bfs
Depth : 6-8 ft	Test Id: 620986		

Test Comment: ---
Visual Description: Moist, olive brown clay with sand
Sample Comment: ---

Particle Size Analysis - ASTM D6913/D7928



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	21.7	78.3

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	100		
#20	0.85	100		
#40	0.42	100		
#60	0.25	99		
#100	0.15	97		
#140	0.11	93		
#200	0.075	78		
Hydrometer	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0335	57		
---	0.0198	48		
---	0.0124	38		
---	0.0085	34		
---	0.0063	30		
---	0.0045	27		
---	0.0033	23		
---	0.0014	13		

Coefficients

D_{85} =0.0879 mm D_{30} =0.0061 mm
D_{60} =0.0379 mm D_{15} =0.0016 mm
D_{50} =0.0221 mm D_{10} =N/A
C_u =N/A C_c =N/A

Classification

ASTM   Lean CLAY with Sand (CL)

AASHTO   Silty Soils (A-4 (6))

Sample/Test Description

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

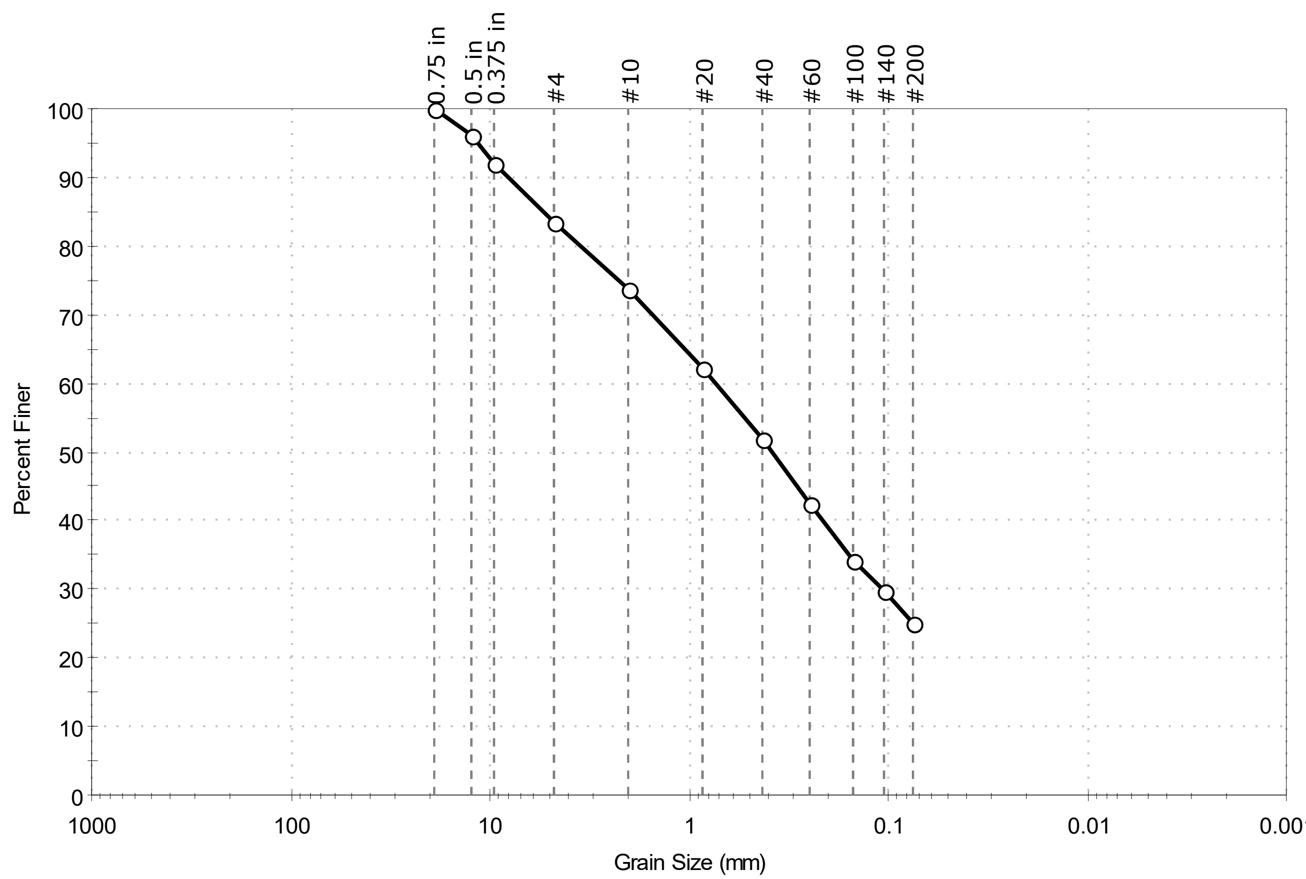
Client:	Golder Associates		
Project:	Freeport Desert Rd Bridge Ex 20		
Location:	Freeport, ME	Project No:	GTX-313770
Boring ID: BB-FDR-211	Sample Type: jar	Tested By:	ckg
Sample ID: 3DB	Test Date: 06/11/21	Checked By:	bfs
Depth : 4-6 ft	Test Id: 620981		

Test Comment: ---

Visual Description: Moist, yellowish brown silty sand with gravel

Sample Comment: ---

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	16.4	58.5	25.1

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	96		
0.375 in	9.50	92		
#4	4.75	84		
#10	2.00	74		
#20	0.85	62		
#40	0.42	52		
#60	0.25	42		
#100	0.15	34		
#140	0.11	30		
#200	0.075	25		

Coefficients

D_{85} =5.3380 mm | D_{30} =0.1071 mm

D_{60} =0.7346 mm | D_{15} =N/A

D_{50} =0.3846 mm | D_{10} =N/A

C_u =N/A | C_c =N/A

Classification

ASTM N/A

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

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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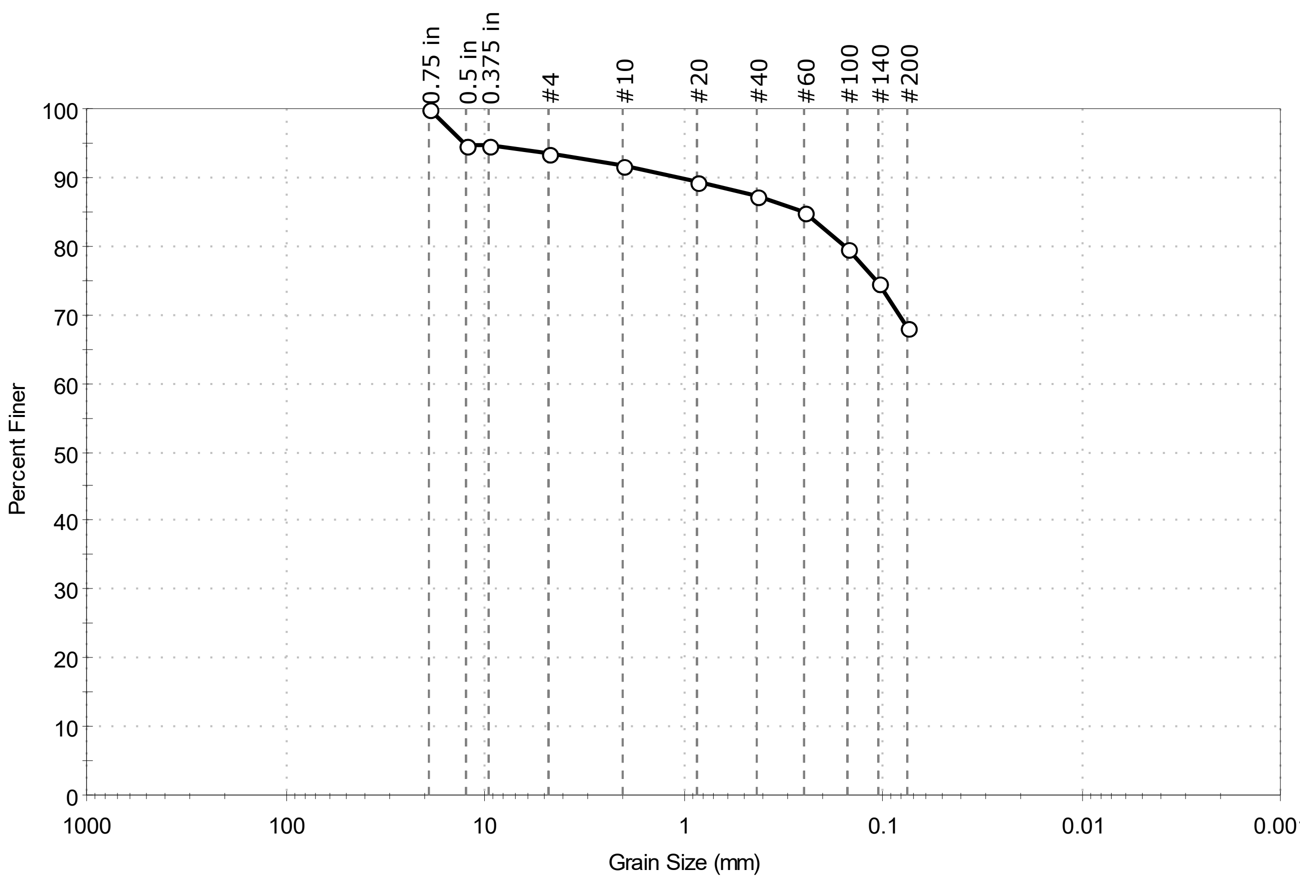
Client:	Golder Associates		
Project:	Freeport Desert Rd Bridge Ex 20		
Location:	Freeport, ME	Project No:	GTX-313770
Boring ID: BB-FDR-211	Sample Type: jar	Tested By:	ckg
Sample ID: 4D	Test Date: 06/11/21	Checked By:	bfs
Depth : 6-8 ft	Test Id: 620982		
Test Comment:	---		
Visual Description:	Moist, olive brown sandy silty clay		
Sample Comment:	---		

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	6.5	25.3	68.2

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	95		
0.375 in	9.50	95		
#4	4.75	94		
#10	2.00	92		
#20	0.85	89		
#40	0.42	87		
#60	0.25	85		
#100	0.15	80		
#140	0.11	75		
#200	0.075	68		

Coefficients

D_{85} =0.2536 mm D_{30} =N/A

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

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

C_u =N/A C_c =N/A

Classification

ASTM   Sandy Silty CLAY (CL-ML)

AASHTO   Silty Soils (A-4 (2))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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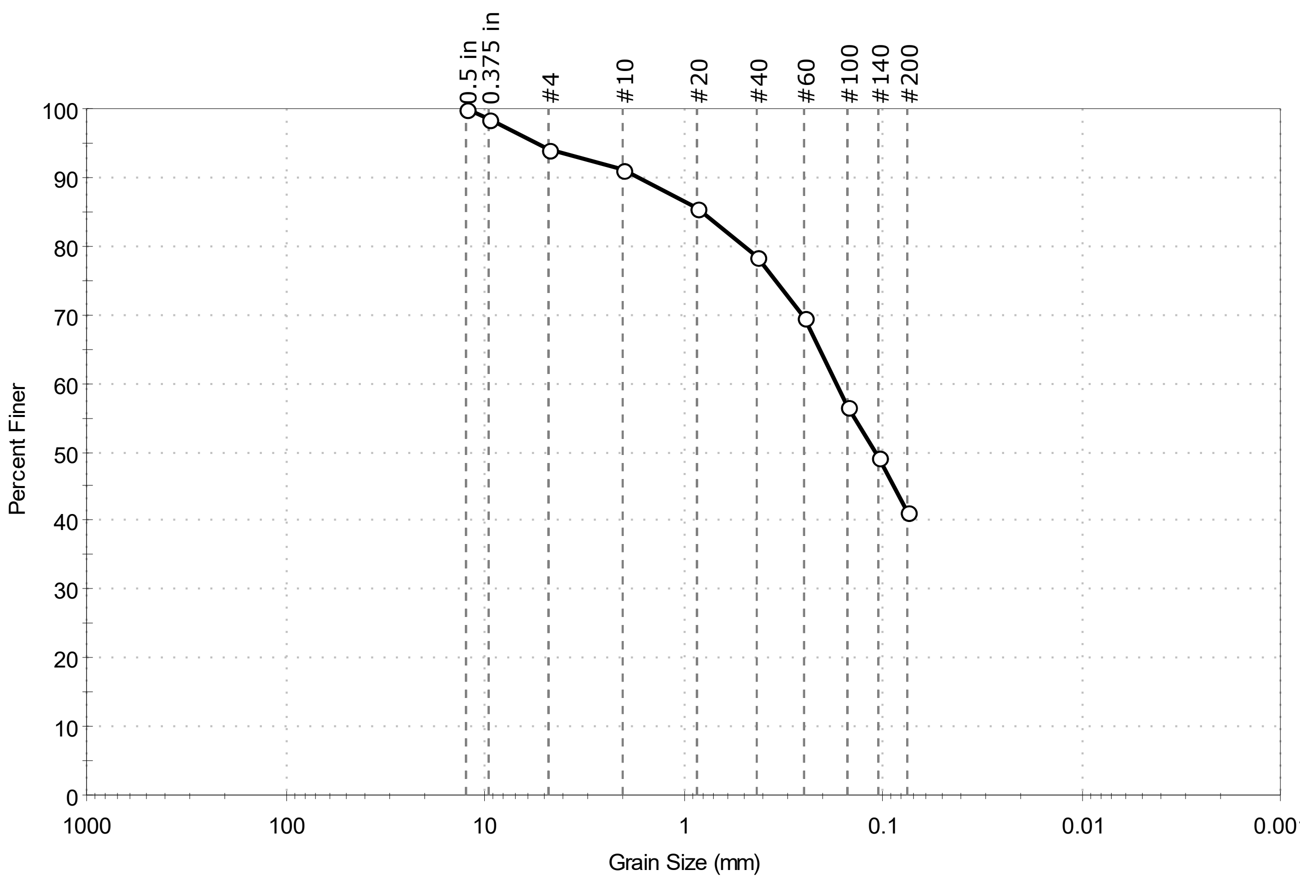
GeoTesting EXPRESS

Client:	Golder Associates		
Project:	Freeport Desert Rd Bridge Ex 20		
Location:	Freeport, ME	Project No:	GTX-313770
Boring ID: BB-FDR-211	Sample Type: jar	Tested By:	ckg
Sample ID: 5D	Test Date: 06/11/21	Checked By:	bfs
Depth : 8-10 ft	Test Id: 620985		
Test Comment:	---		
Visual Description:	Moist, olive brown clayey sand		
Sample Comment:	---		

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	5.8	52.8	41.4

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.5 in	12.50	100		
0.375 in	9.50	99		
#4	4.75	94		
#10	2.00	91		
#20	0.85	86		
#40	0.42	78		
#60	0.25	70		
#100	0.15	57		
#140	0.11	49		
#200	0.075	41		

Coefficients

D_{85} =0.7979 mm D_{30} =N/A
D_{60} =0.1707 mm D_{15} =N/A
D_{50} =0.1103 mm D_{10} =N/A
C_u =N/A C_c =N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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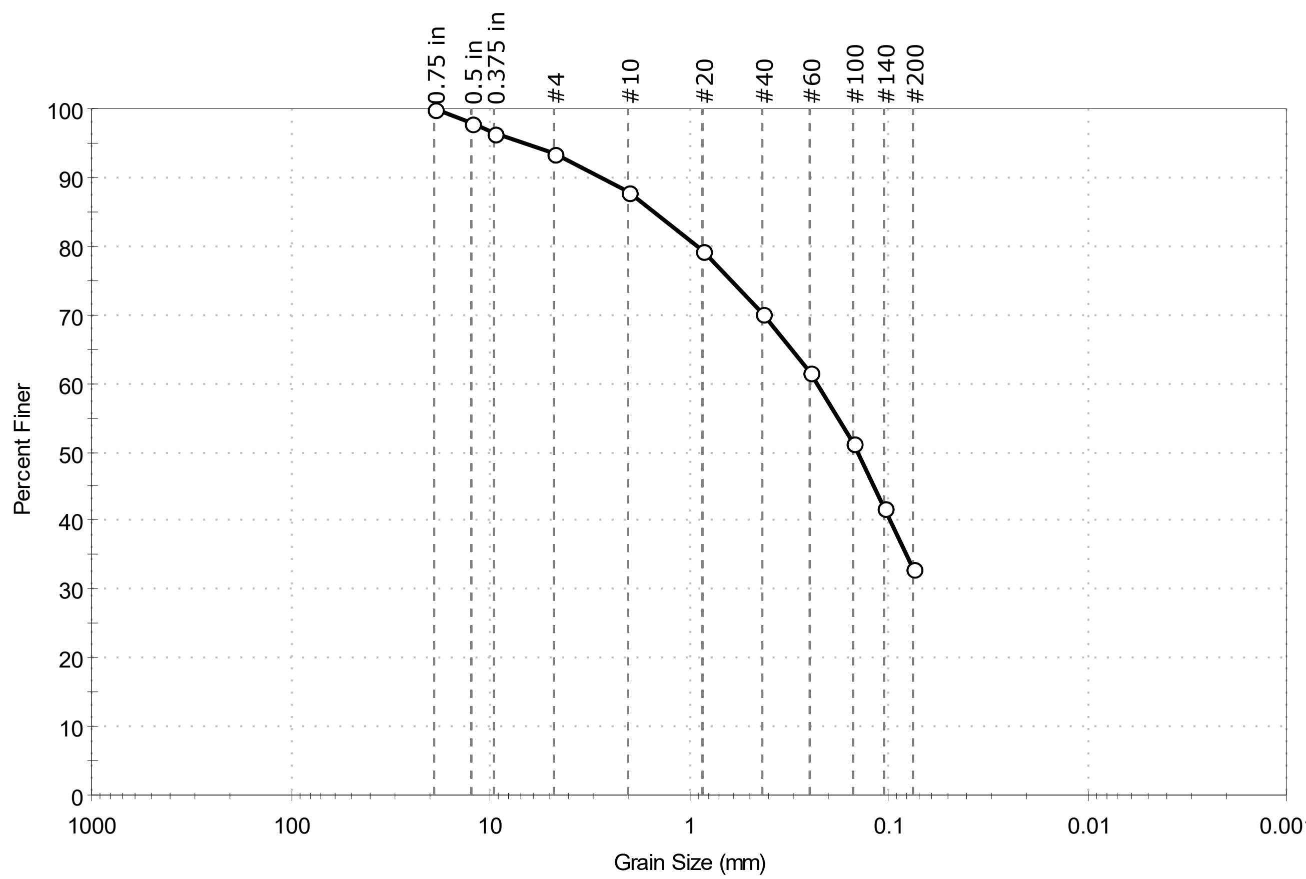
Client: Golder Associates			
Project: Freeport Desert Rd Bridge Ex 20			
Location: Freeport, ME		Project No:	GTX-313770
Boring ID: BB-FDR-212	Sample Type: jar	Tested By:	ckg
Sample ID: 3D	Test Date: 06/11/21	Checked By:	bfs
Depth : 4-6 ft	Test Id: 620983		

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

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	6.4	60.5	33.1

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	98		
0.375 in	9.50	97		
#4	4.75	94		
#10	2.00	88		
#20	0.85	79		
#40	0.42	70		
#60	0.25	62		
#100	0.15	51		
#140	0.11	42		
#200	0.075	33		

Coefficients

D_{85} =1.5123 mm D_{30} =N/A
D_{60} =0.2315 mm D_{15} =N/A
D_{50} =0.1430 mm D_{10} =N/A
C_u =N/A C_c =N/A

Classification

ASTM N/A

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

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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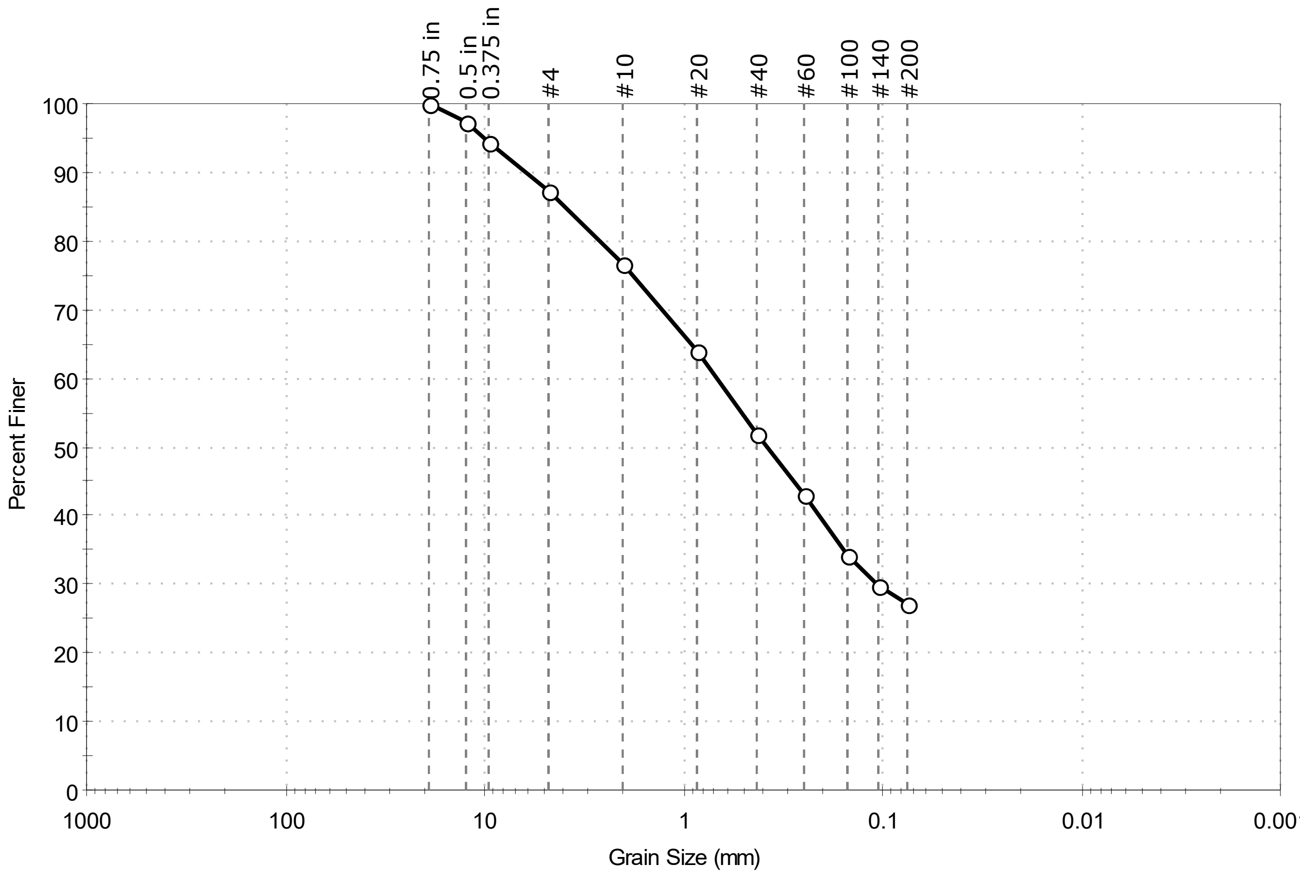
Client:	Golder Associates				
Project:	Freeport Desert Rd Bridge Ex 20				
Location:	Freeport, ME			Project No:	GTX-313770
Boring ID:	BB-FDR-213	Sample Type:	jar	Tested By:	ckg
Sample ID:	6D	Test Date:	06/11/21	Checked By:	bfs
Depth :	10-12 ft	Test Id:	620974		

Test Comment: ---

Visual Description: Moist, yellowish brown silty sand

Sample Comment: ---

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	12.8	60.0	27.2

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	97		
0.375 in	9.50	94		
#4	4.75	87		
#10	2.00	77		
#20	0.85	64		
#40	0.42	52		
#60	0.25	43		
#100	0.15	34		
#140	0.11	30		
#200	0.075	27		

Coefficients

D_{85} =3.9699 mm	D_{30} =0.1066 mm
D_{60} =0.6758 mm	D_{15} =N/A
D_{50} =0.3808 mm	D_{10} =N/A
C_u =N/A	C_c =N/A

Classification

ASTM N/A

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

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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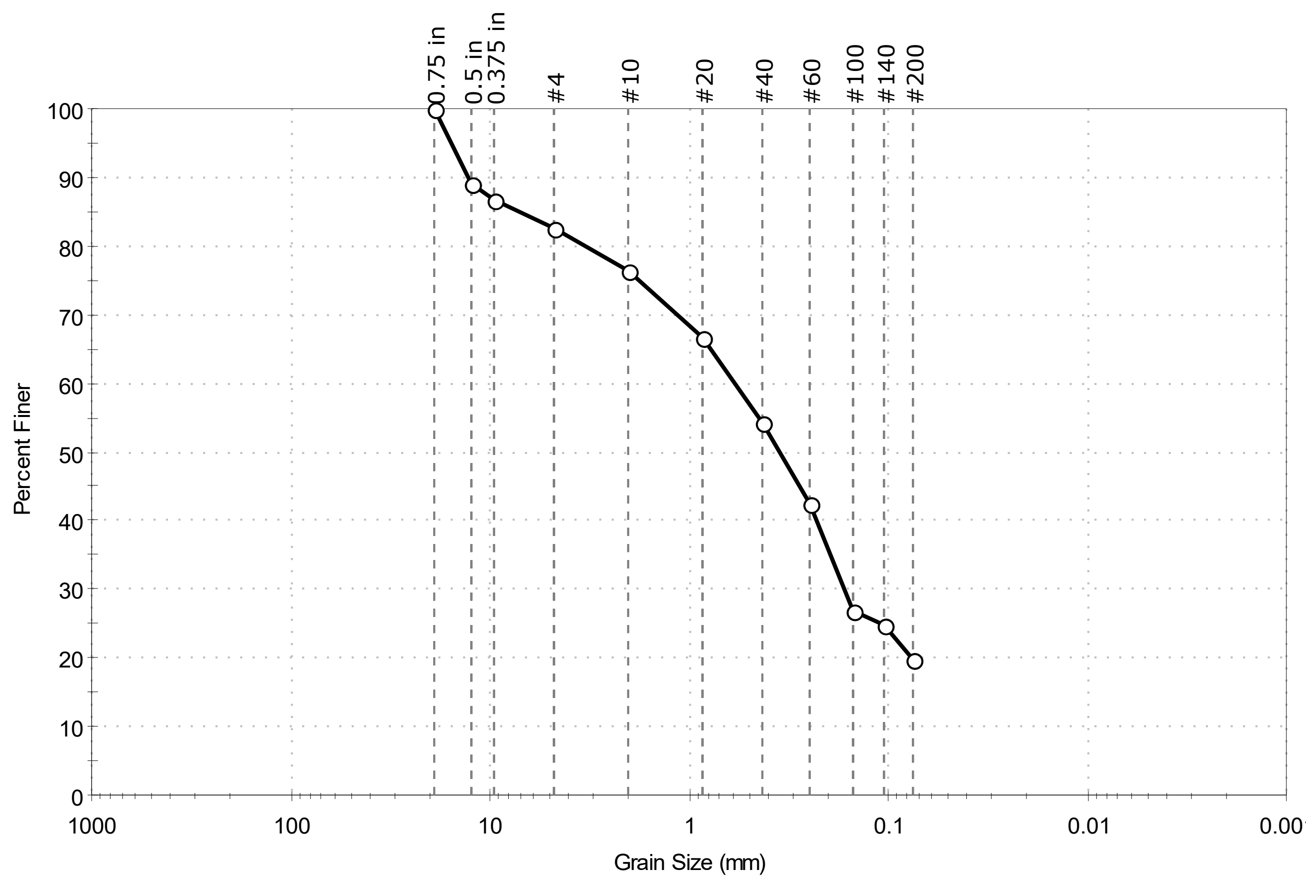
Client:	Golder Associates		
Project:	Freeport Desert Rd Bridge Ex 20		
Location:	Freeport, ME	Project No:	GTX-313770
Boring ID:	BB-FDR-213	Sample Type: jar	Tested By: ckg
Sample ID:	4D	Test Date: 06/11/21	Checked By: bfs
Depth :	8-10 ft	Test Id: 620984	
Test Comment:	---		
Visual Description:	Moist, yellowish brown silty sand with gravel		
Sample Comment:	--		

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	17.5	62.7	19.8

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	89		
0.375 in	9.50	87		
#4	4.75	83		
#10	2.00	76		
#20	0.85	67		
#40	0.42	54		
#60	0.25	42		
#100	0.15	27		
#140	0.11	25		
#200	0.075	20		

Coefficients

D_{85} =7.2523 mm D_{30} =0.1664 mm
D_{60} =0.5847 mm D_{15} =N/A
D_{50} =0.3499 mm D_{10} =N/A
C_u =N/A C_c =N/A

Classification

ASTM   N/A

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

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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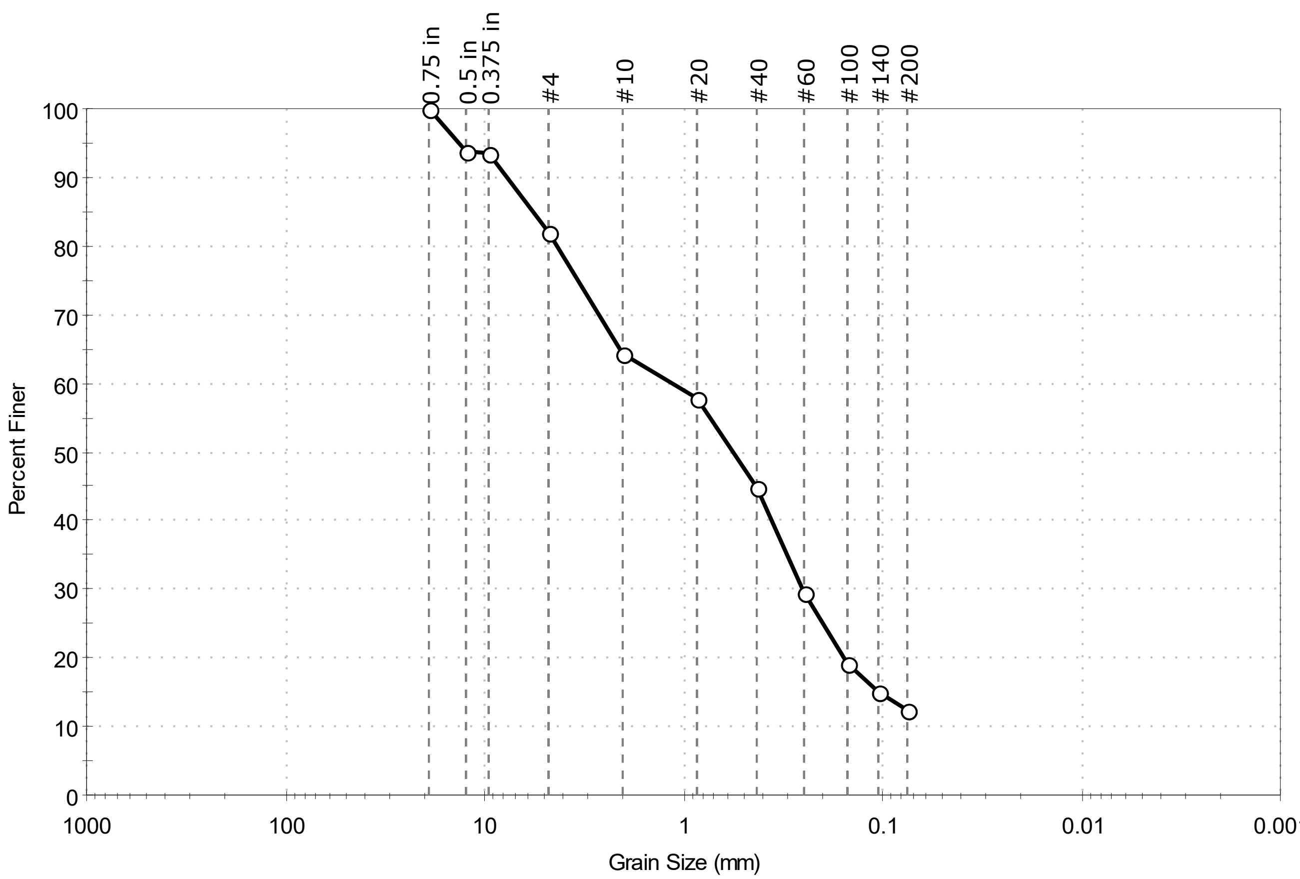
Client:	Golder Associates				
Project:	Freeport Desert Rd Bridge Ex 20				
Location:	Freeport, ME			Project No:	GTX-313770
Boring ID:	BB-FDR-215	Sample Type:	jar	Tested By:	ckg
Sample ID:	3DA	Test Date:	06/11/21	Checked By:	bfs
Depth :	4-6 ft	Test Id:	620975		
Test Comment:	---				
Visual Description:	Moist, dark yellowish brown silty sand with gravel				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	18.0	69.6	12.4

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	94		
0.375 in	9.50	93		
#4	4.75	82		
#10	2.00	64		
#20	0.85	58		
#40	0.42	45		
#60	0.25	29		
#100	0.15	19		
#140	0.11	15		
#200	0.075	12		

Coefficients

D_{85} =5.6762 mm D_{30} =0.2552 mm
D_{60} =1.1365 mm D_{15} =0.1062 mm
D_{50} =0.5619 mm D_{10} =N/A
C_u =N/A C_c =N/A

Classification

ASTM   N/A

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

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

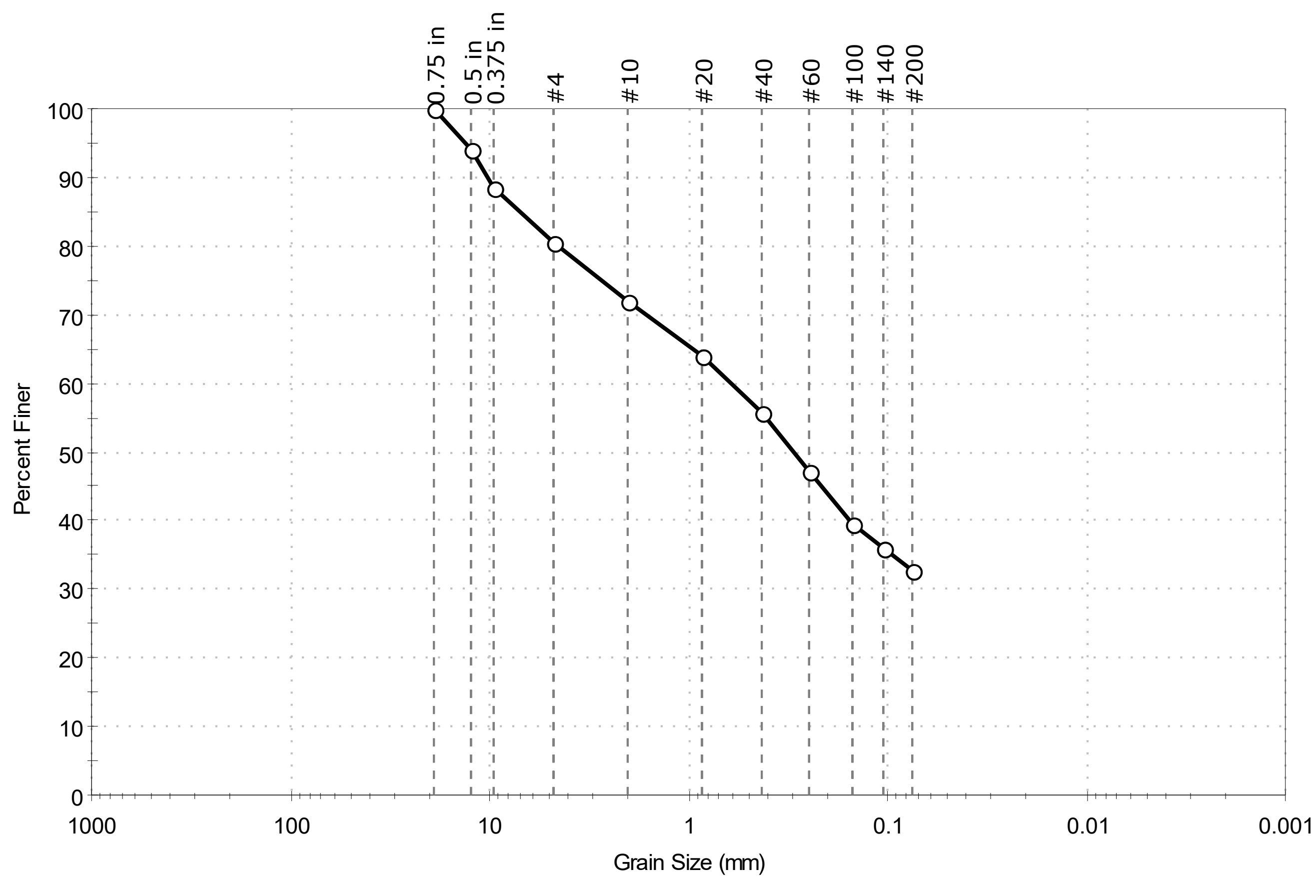
Sand/Gravel Hardness : HARD

printed 6/16/2021 12:55:57 PM

Client:	Golder Associates		
Project:	Freeport Desert Rd Bridge Ex 20		
Location:	Freeport, ME	Project No:	GTX-313770
Boring ID: BB-FDR-216	Sample Type: jar	Tested By:	ckg
Sample ID: 3D	Test Date: 06/11/21	Checked By:	bfs
Depth : 4-6 ft	Test Id: 620976		

Test Comment: ---
Visual Description: Moist, dark yellowish brown clayey sand with gravel
Sample Comment: ---

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	19.6	47.5	32.9

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	94		
0.375 in	9.50	88		
#4	4.75	80		
#10	2.00	72		
#20	0.85	64		
#40	0.42	56		
#60	0.25	47		
#100	0.15	40		
#140	0.11	36		
#200	0.075	33		

Coefficients

D_{85} =7.0427 mm D_{30} =N/A
D_{60} =0.6063 mm D_{15} =N/A
D_{50} =0.2981 mm D_{10} =N/A
C_u =N/A C_c =N/A

Classification

ASTM N/A

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

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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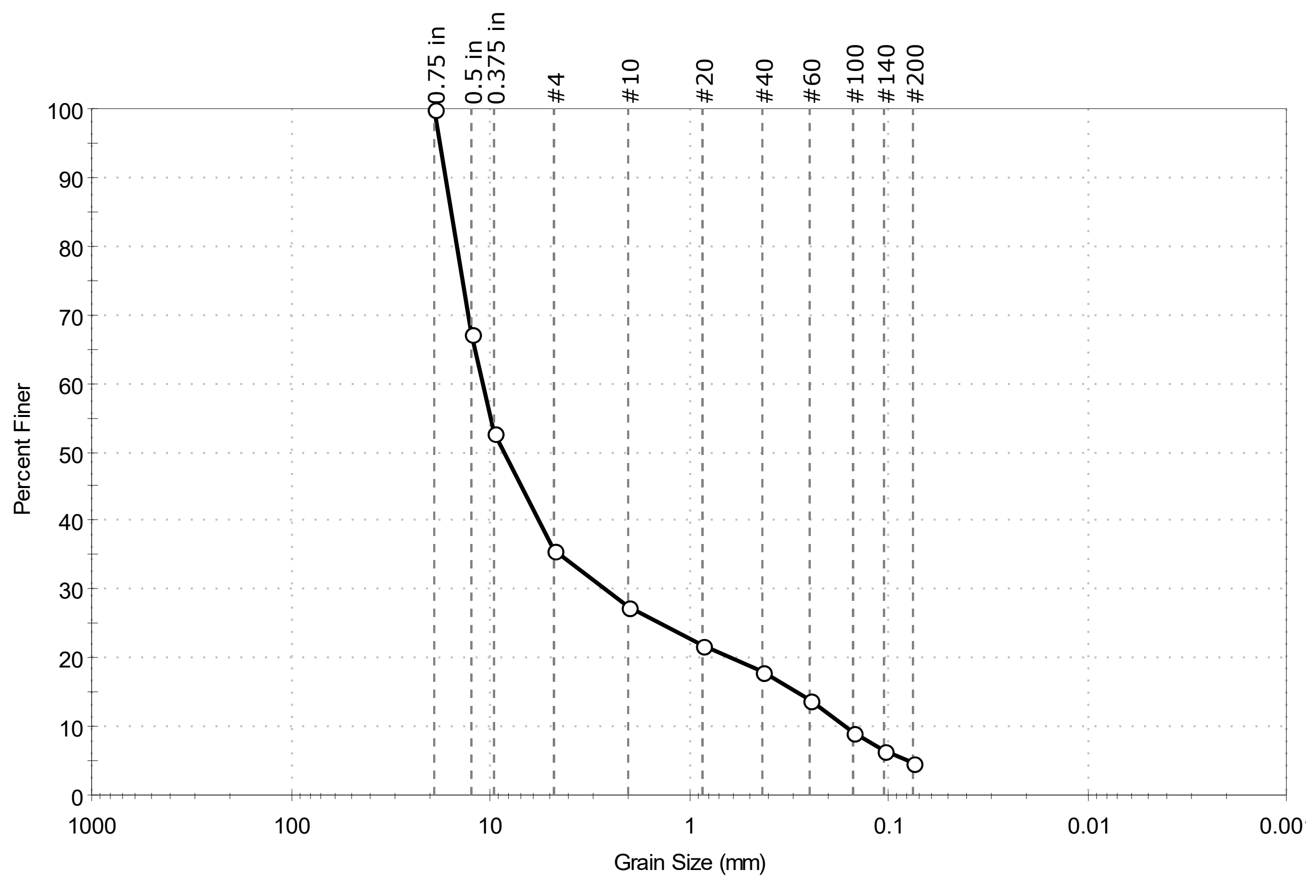
Client:	Golder Associates
Project:	Freeport Desert Rd Bridge Ex 20
Location:	Freeport, ME
Project No:	GTX-313770
Boring ID:	BB-FDR-216
Sample Type:	jar
Tested By:	ckg
Sample ID:	4DB
Test Date:	06/14/21
Checked By:	bfs
Depth :	6-8 ft
Test Id:	620977
Test Comment:	---
Visual Description:	Moist, yellowish brown gravel with sand
Sample Comment:	---

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	64.2	31.2	4.6

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	67		
0.375 in	9.50	53		
#4	4.75	36		
#10	2.00	27		
#20	0.85	22		
#40	0.42	18		
#60	0.25	14		
#100	0.15	9		
#140	0.11	6		
#200	0.075	4.6		

Coefficients

D_{85} =15.6735 mm D_{30} =2.6077 mm

D_{60} =10.8819 mm D_{15} =0.2934 mm

D_{50} =8.4834 mm D_{10} =0.1650 mm

C_u =65.951 C_c =3.787

Classification

ASTM   Poorly graded GRAVEL with Sand (GP)

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

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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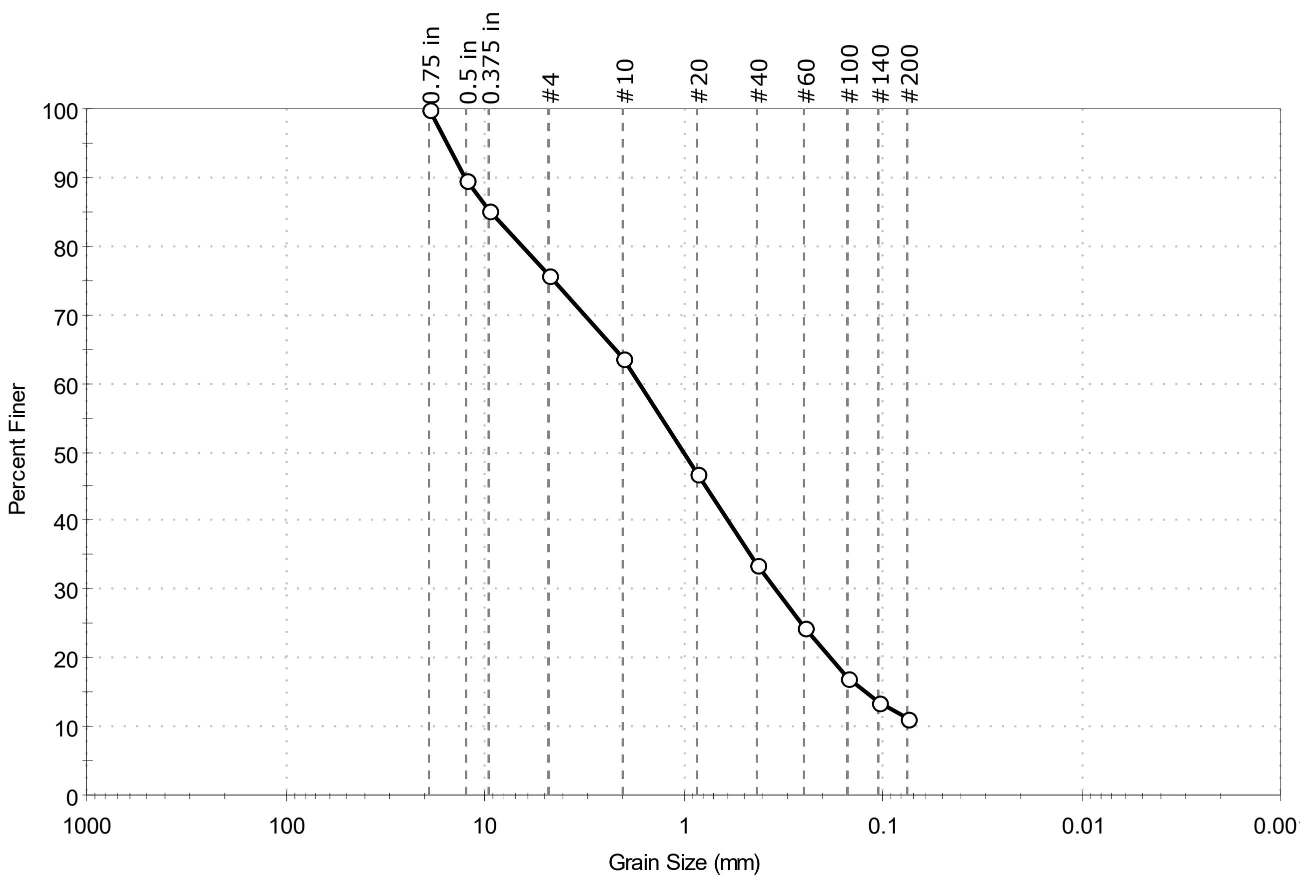
Client:	Golder Associates				
Project:	Freeport Desert Rd Bridge Ex 20				
Location:	Freeport, ME			Project No:	GTX-313770
Boring ID:	BB-FDR-217	Sample Type:	jar	Tested By:	ckg
Sample ID:	2D	Test Date:	06/14/21	Checked By:	bfs
Depth :	6-8 ft	Test Id:	620978		

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

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	24.1	64.7	11.2

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	90		
0.375 in	9.50	85		
#4	4.75	76		
#10	2.00	64		
#20	0.85	47		
#40	0.42	34		
#60	0.25	25		
#100	0.15	17		
#140	0.11	14		
#200	0.075	11		

Coefficients

D_{85} =9.3691 mm D_{30} =0.3427 mm
D_{60} =1.6582 mm D_{15} =0.1218 mm
D_{50} =0.9962 mm D_{10} =N/A
C_u =N/A C_c =N/A

Classification

ASTM N/A

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

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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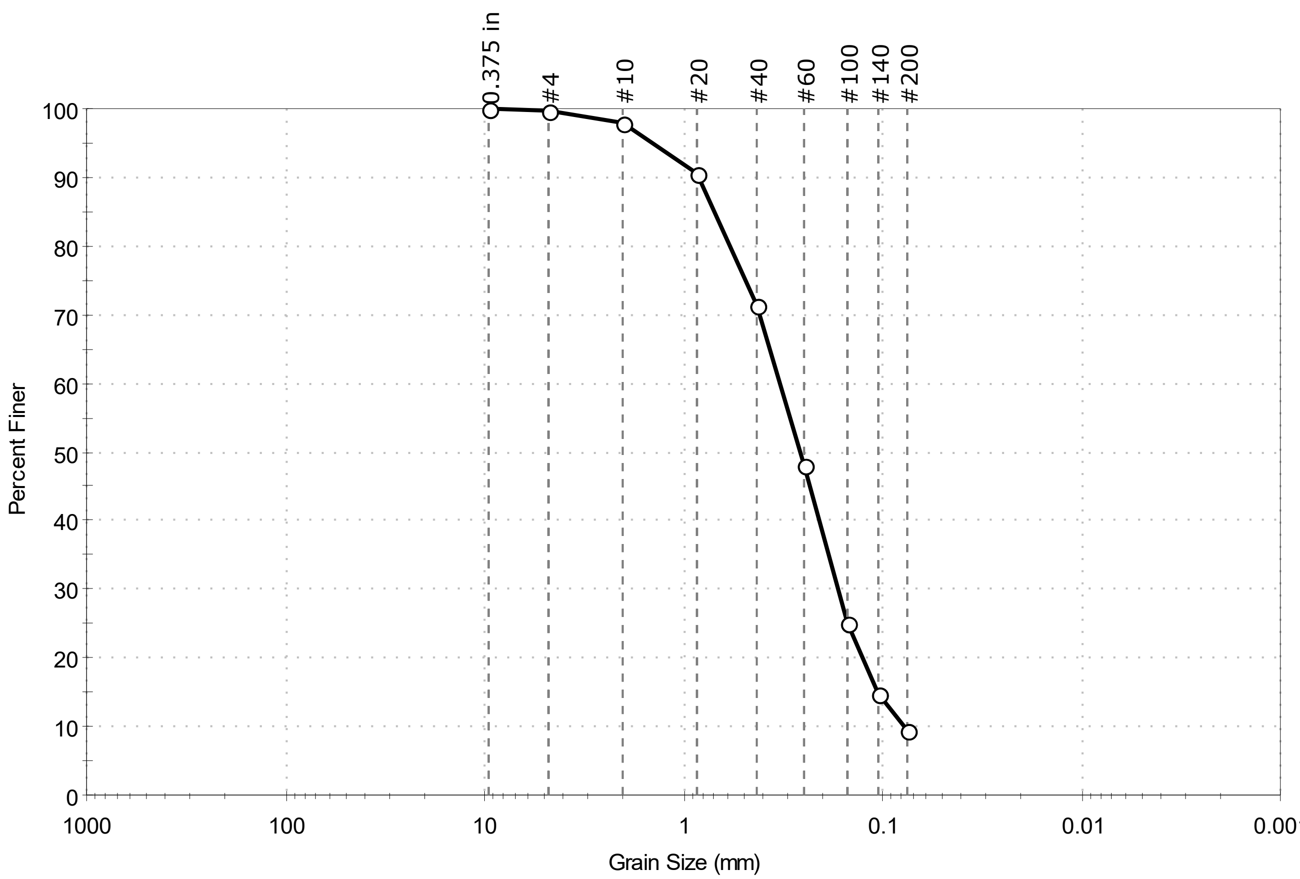
Client:	Golder Associates		
Project:	Freeport Desert Rd Bridge Ex 20		
Location:	Freeport, ME	Project No:	GTX-313770
Boring ID: BB-FDR-217	Sample Type: jar	Tested By:	ckg
Sample ID: 4D	Test Date: 06/11/21	Checked By:	bfs
Depth : 10-12 ft	Test Id: 620979		

Test Comment: ---
Visual Description: Moist, brownish yellow sand with silt
Sample Comment: ----

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.3	90.2	9.5

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.375 in	9.50	100		
#4	4.75	100		
#10	2.00	98		
#20	0.85	90		
#40	0.42	71		
#60	0.25	48		
#100	0.15	25		
#140	0.11	15		
#200	0.075	9.5		

Coefficients

D_{85} =0.6964 mm D_{30} =0.1671 mm
D_{60} =0.3281 mm D_{15} =0.1070 mm
D_{50} =0.2618 mm D_{10} =0.0774 mm
C_u =4.239 C_c =1.100

Classification

ASTM N/A

AASHTO Fine Sand (A-3 (1))

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

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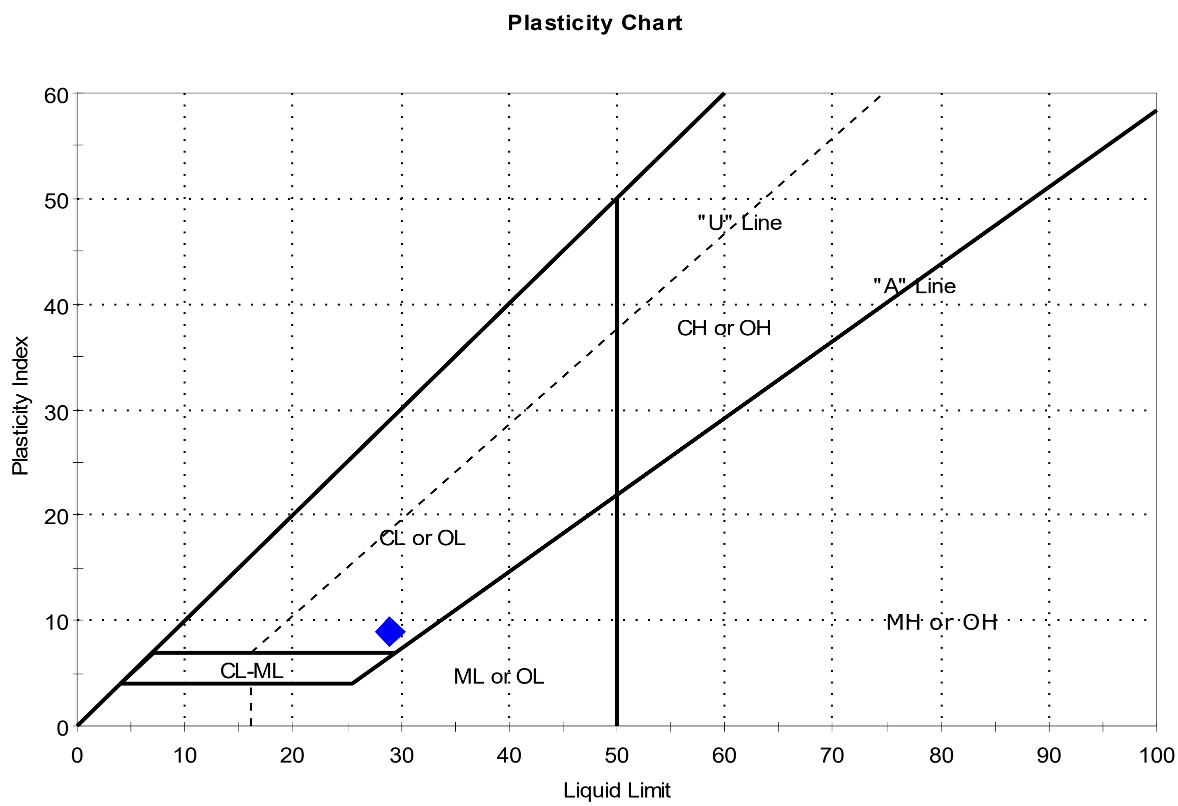
Client:	Golder Associates				
Project:	Freeport Desert Rd Bridge Ex 20				
Location:	Freeport, ME			Project No:	GTX-313770
Boring ID:	BB-FDR-208	Sample Type:	jar	Tested By:	cam
Sample ID:	4D	Test Date:	06/15/21	Checked By:	bfs
Depth :	6-8 ft	Test Id:	620958		
Test Comment:	---				
Visual Description:	Moist, olive brown clay with sand				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	4D	B-FDR-20	6-8 ft	27	29	20	9	0.7	Lean CLAY with Sand (CL)

Sample Prepared using the WET method

0% Retained on #40 Sieve

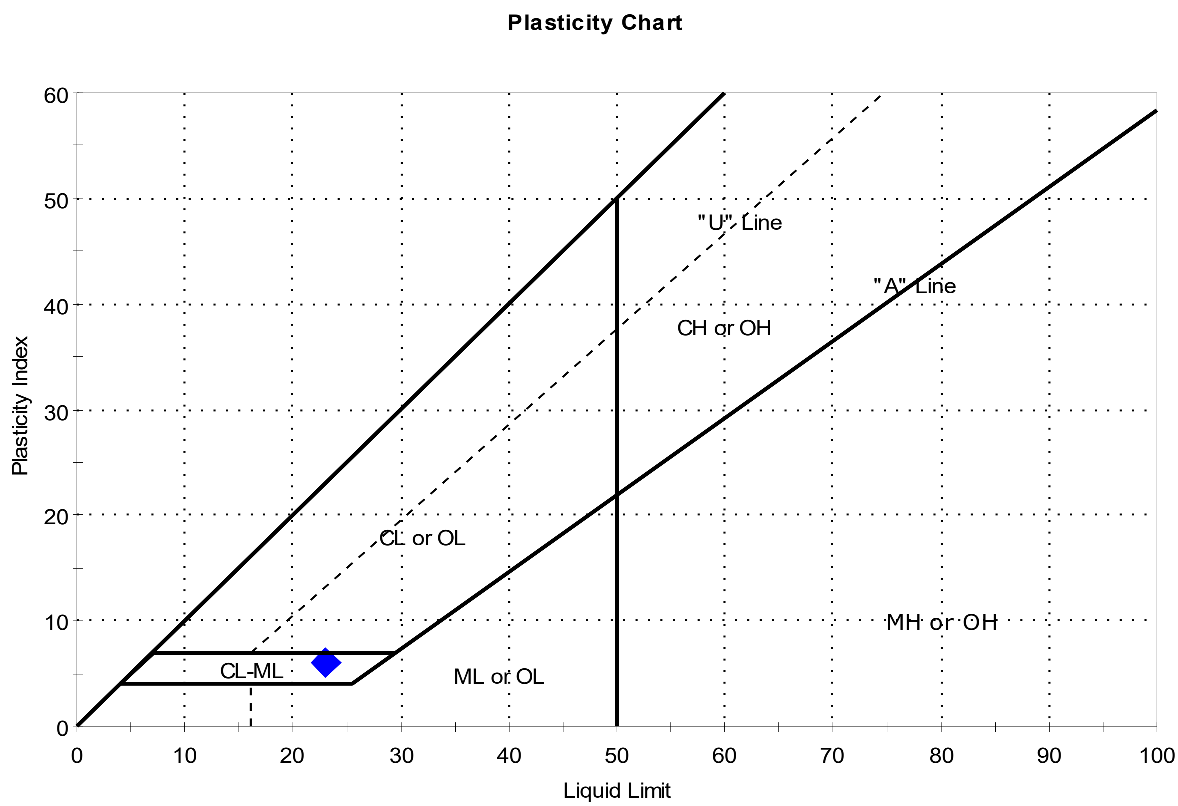
Dry Strength: MEDIUM

Dilatancy: SLOW

Toughness: LOW

Client:	Golder Associates		
Project:	Freeport Desert Rd Bridge Ex 20		
Location:	Freeport, ME	Project No:	GTX-313770
Boring ID:	BB-FDR-211	Sample Type: jar	Tested By: cam
Sample ID:	4D	Test Date: 06/16/21	Checked By: bfs
Depth :	6-8 ft	Test Id: 620959	
Test Comment:	---		
Visual Description:	Moist, olive brown sandy silty clay		
Sample Comment:	---		

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	4D	B-FDR-21	6-8 ft	22	23	17	6	0.8	Sandy Silty CLAY (CL-ML)

Sample Prepared using the WET method

13% Retained on #40 Sieve

Dry Strength: HIGH

Dilatancy: SLOW

Toughness: LOW

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APPENDIX D

Subsurface Layering and Engineering Properties for Drilled Shaft Design

GOLDER
MEMBER OF WSP

Date:	August 17, 2021		
Project No.:	21450908	**Made by:**	BK
Subject:	Subsurface Layering and Engineering Properties for Mast Arm Foundations	**Checked by:**	MEL
Project Title:	MaineDOT I-295 Exit 20 Desert Road Bridge Replacement No. 5720	**Reviewed by:**	CCB

Soil Type	Layering					Lateral Model[1]	Below GW table[2]?	Effective Unit Weight (pcf)[3]	Undrained Shear Strength[4] (psf)	Friction Angle[4] (°)	Subgrade Modulus[5] (pci)	Major Principal Strain at 50%[5]	UCS[3] (psi)	Design Note
	Depth (ft bgs)		Elevation (ft)		Thickness									
	Start	End	Start	End	(ft)									
MA1, 40' mast arm, 50-foot mast arm loading scenario, Station 55+75, Offset 45.5' LT, Layering[7]														
Granular Borrow Replacement[6]	0.0	1.0	153.0	152.0	1.0	Sand (Reese)	N	125	-	32	88	-	-	Drilled Shaft
Fill	1.0	9.9	152.0	143.2	8.9	Sand (Reese)	N	125	-	40	253	-	-	
Glaciomarine	9.9	12.1	143.2	141.0	2.2	Stiff Clay with Free Water (Reese)	N	119	2386	-	200	0.005	-	
Glaciomarine	12.1	19.2	141.0	133.9	7.1	Stiff Clay w/o Free Water (Reese)	Y	56.6	2386	-	200	0.005	-	
Bedrock	18.2	-	133.9	-	-	Strong Rock	Y	101.6	-	-	-	-	12983	
MA2, 20' mast arm, 25-foot mast arm loading scenario, Station 56+79.94, Offset 22.6 RT, Layering[8]														
Granular Borrow	0.0	0.5	157.0	156.5	0.5	Sand (Reese)	N	125	-	32	88	-	-	Drilled Shaft
Granular Borrow Replacement[6]	0.5	1.5	156.5	155.5	1.0	Sand (Reese)	N	125	-	32	88	-	-	
Fill	1.5	8.5	155.5	148.5	7.0	Sand (Reese)	N	125	-	29	20	-	-	
Fill	8.5	12.0	148.5	145.0	3.5	Sand (Reese)	Y	62.6	-	29	20	-	-	
Glaciomarine	12.0	12.7	145.0	144.3	0.7	Stiff Clay w/o Free Water (Reese)	Y	56.6	1746	-	200	0.005	-	
Sand/Gravel	12.7	17.5	144.3	139.5	4.8	Sand (Reese)	Y	62.6	-	32	53	-	-	
Bedrock	17.5	-	139.5	-	-	Strong Rock	Y	101.6	-	-	-	-	12983	
MA3, 25' mast arm, 25-foot mast arm loading scenario, Station 56+07.00, Offset 31.0' RT, Layering[9]														
Granular Borrow Replacement[6]	0.0	1.0	152.5	151.5	1.0	Sand (Reese)	N	125	-	32	88	-	-	Drilled Shaft
Fill	1.0	8.9	151.5	143.6	7.9	Sand (Reese)	N	125	-	37	187	-	-	
Bedrock	8.9	-	143.6	-	-	Strong Rock	N	164	-	-	-	-	12983	
MA4, 45' mast arm, 50-foot mast arm loading scenario, Station 64+59.82, Offset 45.5' LT, Layering[10]														
Granular Borrow	0.0	2.0	162.1	160.1	2.0	Sand (Reese)	N	125	-	32	88	-	-	Spread Footing on Soil
Granular Borrow Replacement[6]	2.0	3.0	160.1	159.1	1.0	Sand (Reese)	N	125	-	32	88	-	-	
Fill	3.0	4.1	159.1	158.0	1.1	Sand (Reese)	N	125	-	36	165	-	-	
Fill	4.1	6.6	158.0	155.5	2.5	Sand (Reese)	Y	62.6	-	36	100	-	-	
Bedrock	6.6	-	155.5	-	-	Strong Rock	Y	101.6	-	-	-	-	12983	
MA5, 40' mast arm, 50-foot mast arm loading scenario, Station 65+45.27, Offset 45.5' LT, Layering[11]														
Granular Borrow	0.0	0.6	158.8	158.2	0.6	Sand (Reese)	N	125	-	32	88	-	-	Spread Footing on Rock
Granular Borrow Replacement[6]	0.6	1.6	158.2	157.2	1.0	Sand (Reese)	N	125	-	32	88	-	-	
Fill	1.6	4.8	157.2	154.0	3.2	Sand (Reese)	N	125	-	36	165	-	-	
Bedrock	4.8	-	154.0	-	-	Strong Rock	N	164	-	-	-	-	12983	
MA6, 40' mast arm, 50-foot mast arm loading scenarios, Station 65+20.09, Offset 45.5' RT, Layering[12]														
Granular Borrow	0.0	1.0	158.5	157.5	1.0	Sand (Reese)	N	125	-	32	88	-	-	Drilled Shaft
Granular Borrow Replacement[6]	1.0	2.0	157.5	156.5	1.0	Sand (Reese)	N	125	-	32	88	-	-	
Fill	2.0	8.2	156.5	150.3	6.2	Sand (Reese)	N	125	-	36	165	-	-	
Glaciomarine	8.2	13.9	150.3	144.6	5.7	Stiff Clay with Free Water (Reese)	N	119	3608	-	200	0.005	-	
Sand/Gravel	13.9	15.2	144.6	143.3	1.3	Sand (Reese)	N	125	-	37	187	-	-	
Bedrock	15.4	-	143.3	-	-	Strong Rock	N	164	-	-	-	-	12983	

GOLDER
MEMBER OF WSP

Date:	August 17, 2021		
Project No.:	21450908	**Made by:**	BK
Subject:	Subsurface Layering and Engineering Properties for Mast Arm Foundations	**Checked by:**	MEL
Project Title:	MaineDOT I-295 Exit 20 Desert Road Bridge Replacement No. 5720	**Reviewed by:**	CCB

Soil Type	Layering: Depth (ft bgs) Start	Layering: Depth (ft bgs) End	Layering: Elevation (ft) Start	Layering: Elevation (ft) End	Layering: Thickness (ft)	Lateral Model[1]	Below GW table[2]?	Effective Unit Weight (pcf)[3]	Undrained Shear Strength[4] (psf)	Friction Angle[4] (°)	Subgrade Modulus[5] (pci)	Major Principal Strain at 50%[5]	UCS[3] (psi)	Design Note
MA7, 45' mast arm, 50-foot mast arm loading scenario, Station 67+05.00, Offset 30.5' LT, Layering[13]														
Granular Borrow Replacement[6]	0.0	1.0	156.0	155.0	1.0	Sand (Reese)	N	125	-	32	88	-	-	Spread Footing on Soil
Fill	1.0	7.8	155.0	148.2	6.8	Sand (Reese)	N	125	-	30	50	-	-	
Bedrock	7.8	-	148.2	-	-	Strong Rock	N	164	-	-	-	-	12983	
Design basis for a drilled shaft 50-foot mast arm foundation with shallow glaciomarine soil present (based on MA6)														
Granular Borrow	0.0	1.0	158.5	157.5	1.0	Sand (Reese)	N	125	-	32	88	-	-	Drilled Shaft, Friction angle and undrained shear strength reduced based on engineering judgement
Granular Borrow Replacement[6]	1.0	2.0	157.5	156.5	1.0	Sand (Reese)	N	125	-	32	88	-	-	
Fill	2.0	8.2	156.5	150.3	6.2	Sand (Reese)	N	125	-	36	165	-	-	
Glaciomarine	8.2	13.9	150.3	144.6	5.7	Stiff Clay with Free Water (Reese)	N	119	2300	-	200	0.005	-	
Sand/Gravel	13.9	15.2	144.6	143.3	1.3	Sand (Reese)	N	125	-	36	165	-	-	
Bedrock	15.4	-	143.3	-	-	Strong Rock	N	164	-	-	-	-	12983	
Design basis for a drilled shaft 25-foot mast arm foundation with deeper glaciomarine soil present (based on MA2)														
Granular Borrow	0.0	0.5	157.0	156.5	0.5	Sand (Reese)	N	125	-	32	88	-	-	Drilled Shaft
Granular Borrow Replacement[6]	0.5	1.5	156.5	155.5	1.0	Sand (Reese)	N	125	-	32	88	-	-	
Fill	1.5	8.5	155.5	148.5	7.0	Sand (Reese)	N	125	-	29	20	-	-	
Fill	8.5	12.0	148.5	145.0	3.5	Sand (Reese)	Y	62.6	-	29	20	-	-	
Glaciomarine	12.0	12.7	145.0	144.3	0.7	Stiff Clay w/o Free Water (Reese)	Y	56.6	1746	-	200	0.005	-	
Sand/Gravel	12.7	17.5	144.3	139.5	4.8	Sand (Reese)	Y	62.6	-	32	53	-	-	
Bedrock	17.5	-	139.5	-	-	Strong Rock	Y	101.6	-	-	-	-	12983	
Design basis for a drilled shaft 25-foot mast arm foundation without glaciomarine soil present (based on MA3)														
Granular Borrow Replacement[6]	0.0	1.0	152.5	151.5	1.0	Sand (Reese)	N	125	-	32	88	-	-	Drilled Shaft, Friction angle reduced based on engineering judgment
Fill	1.0	8.9	151.5	143.6	7.9	Sand (Reese)	N	125	-	36	165	-	-	
Bedrock	8.9	-	143.6	-	-	Strong Rock	N	164	-	-	-	-	12983	
Design basis for a spread footing 50-foot mast arm foundation on bedrock without glaciomarine soil present (based on MA5)														
Granular Borrow	0.0	0.6	158.8	158.2	0.6	Sand (Reese)	N	125	-	32	88	-	-	Spread Footing on Rock
Granular Borrow Replacement[6]	0.6	1.6	158.2	157.2	1.0	Sand (Reese)	N	125	-	32	88	-	-	
Fill	1.6	4.8	157.2	154.0	3.2	Sand (Reese)	N	125	-	36	165	-	-	
Bedrock	4.8	-	154.0	-	-	Strong Rock	N	164	-	-	-	-	12983	
Design basis for a spread footing 50-foot mast arm foundation on soil without glaciomarine soil present (based on MA7)														
Granular Borrow Replacement[6]	0.0	1.0	156.0	155.0	1.0	Sand (Reese)	N	125	-	32	88	-	-	Spread Footing on Soil
Fill	1.0	7.8	155.0	148.2	6.8	Sand (Reese)	N	125	-	30	50	-	-	
Bedrock	7.8	-	148.2	-	-	Strong Rock	N	164	-	-	-	-	12983	

GOLDER
MEMBER OF WSP

Date:	August 17, 2021		
Project No.:	21450908	**Made by:**	BK
Subject:	Subsurface Layering and Engineering Properties for Mast Arm Foundations	**Checked by:**	MEL
Project Title:	MaineDOT I-295 Exit 20 Desert Road Bridge Replacement No. 5720	**Reviewed by:**	CCB

Soil Type	Layering					Lateral Model[1]	Below GW table[2]?	Effective Unit Weight (pcf)[3]	Undrained Shear Strength[4] (psf)	Friction Angle[4] (°)	Subgrade Modulus[5] (pci)	Major Principal Strain at 50%[5]	UCS[3] (psi)	Design Note
	Depth (ft bgs)		Elevation (ft)		Thickness									
	Start	End	Start	End	(ft)									

Notes:

1. Lateral model: Isenhower, W.M., Wang, S.T., and Vasquez, L.G. LPILE v2019 Technical Manual: A Program for the Analysis of Deep Foundations Under Lateral Loading. Ensoft, Inc. Release: March 2020.
2. Water table determined based on boring log stratigraphy (Appendix A, SGDR Part II)
3. Soil properties determined based on the PGDR (Golder Associates, Inc., December 21, 2020, Preliminary Geotechnical Design Report, I-295 Mallet Drive Bridge Replacement #5721 (Exit 22), Freeport, Maine, MaineDOT WIN 021726.00)
4. Soil properties correlated from SPT N_{60} values
5. Bridge Software Institute. FB-MultiPier Soil Parameter Table (US Customary Units). Accessed 7/2021. https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf
6. We propose to remove and replace the upper 1 foot of existing soil with new Gravel Borrow to remove organics.
7. Layering determined from stratigraphy from BB-FDR-202 for in situ soils; Starting elevation for Granular Borrow Replacement is the proposed elevation at Sta. 55+75 from HNTB 98% plans; Starting elevation for Granular Borrow Replacement is the existing ground surface from the HNTB 98% plans for the actual location.
8. Layering determined from stratigraphy from BB-FDR-204 for in situ soils; Starting elevation for Granular Borrow is the proposed elevation at Sta. 56+75 from HNTB 98% plans; Starting elevation for Granular Borrow Replacement is the existing ground surface from the HNTB 98% plans for the actual location.
9. Layering determined from stratigraphy from BB-FDR-203 for in situ soils; Starting elevation for Granular Borrow Replacement is the proposed elevation at Sta. 56+00 from HNTB 98% plans; Starting elevation for Granular Borrow Replacement is the existing ground surface from the HNTB 98% plans for the actual location.
10. Layering determined from stratigraphy from BB-FDR-210 for in situ soils; Starting elevation for Granular Borrow is the proposed elevation at Sta. 64+50 from HNTB 98% plans; Starting elevation for Granular Borrow Replacement is the existing ground surface from the HNTB 98% plans for the actual location.
11. Layering determined from stratigraphy from BB-FDR-212 for in situ soils; Starting elevation for Granular Borrow is the proposed elevation at Sta. 65+50 from HNTB 98% plans; Starting elevation for Granular Borrow Replacement is the existing ground surface from the HNTB 98% plans for the actual location.
12. Layering determined from stratigraphy from BB-FDR-211 for in situ soils; Starting elevation for Granular Borrow is the proposed elevation at Sta. 65+25 from HNTB 98% plans; Starting elevation for Granular Borrow Replacement is the existing ground surface from the HNTB 98% plans for the actual location.
13. Layering determined from stratigraphy from BB-FDR-216 for in situ soils; Starting elevation for Granular Borrow Replacement is the proposed elevation at Sta. 67+00 from HNTB 98% plans; Starting elevation for Granular Borrow Replacement is the existing ground surface from the HNTB 98% plans for the actual location.

GOLDER
MEMBER OF WSP

Date: August 17, 2021
Project No.: 21450908
Subject: Subsurface Layering and Engineering Properties for Light Standard Foundations
Project Title: MaineDOT I-295 Exit 20 Desert Road Bridge Replacement No. 5720

Made by: BK
Checked by: MEL
Reviewed by: CCB

Soil Type	Layering: Depth (ft bgs) Start	Depth (ft bgs) End	Elevation (ft) Start	Elevation (ft) End	Thickness (ft)	Lateral Model[1]	Below GW table[2]?	Effective Unit Weight (pcf)[3]	Undrained Shear Strength[4] (psf)	Friction Angle[4] (°)	Subgrade Modulus[5] (pci)	Major Principal Strain at 50%[5]	UCS[3] (psi)	Design Note
LS Pole 1 Station 54+00, Offset 34.0' LT, Layering[7]														
Granular Borrow Replacement[6]	0.0	1.0	145.0	144.0	1.0	Sand (Reese)	N	125	-	32	88	-	-	Spread Footing on Rock
Fill	1.0	3.8	144.0	141.2	2.8	Sand (Reese)	N	125	-	32	88	-	-	
Bedrock	3.8	-	141.2	-	-	Strong Rock	Y	101.5	-	-	-	-	12983	
LS Pole 2 Station 55+35, Offset 28.4' LT, Layering[8]														
Granular Borrow	0.0	1.0	152.0	151.0	1.0	Sand (Reese)	N	125	-	32	88	-	-	Drilled Shaft
Granular Borrow Replacement[6]	1.0	2.0	151.0	150.0	1.0	Sand (Reese)	N	125	-	32	88	-	-	
Fill	2.0	10.8	150.0	141.2	8.8	Sand (Reese)	N	125	-	33	108	-	-	
Bedrock	10.8	-	141.2	-	-	Strong Rock	Y	101.6	-	-	-	-	12983	
LS Pole 4 Station 56+80, Offset 30.0' LT, Layering[9]														
Granular Borrow	0.0	1.0	158.0	157.0	1.0	Sand (Reese)	N	125	-	32	88	-	-	Drilled Shaft
Granular Borrow Replacement[6]	1.0	2.0	157.0	156.0	1.0	Sand (Reese)	N	125	-	32	88	-	-	
Fill	2.0	16.6	156.0	141.4	14.6	Sand (Reese)	N	125	-	39	231	-	-	
Bedrock	16.6	-	141.4	-	-	Strong Rock	Y	101.6	-	-	-	-	12983	
LS Pole 5 Station 58+15, Offset 30.0' LT, Layering[10]														
Granular Borrow	0.0	1.5	164.0	162.5	1.5	Sand (Reese)	N	125	-	32	88	-	-	Drilled Shaft
Granular Borrow Replacement[6]	1.5	2.5	162.5	161.5	1.0	Sand (Reese)	N	125	-	32	88	-	-	
Fill	2.5	6.0	161.5	158.0	3.5	Sand (Reese)	N	125	-	35	146	-	-	
Fill	6.0	20.0	158.0	144.0	14.0	Sand (Reese)	Y	62.6	-	35	88	-	-	
Glaciomarine	20.0	23.1	144.0	140.9	3.1	Stiff Clay w/o Free Water (Reese)	Y	56.6	349	-	N/A	0.02	-	
Bedrock	23.1	-	140.9	-	-	Strong Rock	Y	101.6	-	-	-	-	12983	
LS Pole 6 Station 59+50, Offset 30.0' LT, Layering[11]														
Granular Borrow	0.0	2.5	169.0	166.5	2.5	Sand (Reese)	N	125	-	32	88	-	-	Drilled Shaft
Granular Borrow Replacement[6]	2.5	3.5	166.5	165.5	1.0	Sand (Reese)	N	125	-	32	88	-	-	
Fill	3.5	11.0	165.5	158.0	7.5	Sand (Reese)	N	125	-	33	108	-	-	
Fill	11.0	14.2	158.0	154.8	3.2	Sand (Reese)	Y	62.6	-	33	65	-	-	
Sand/Gravel	14.2	29.7	154.8	139.3	15.5	Sand (Reese)	Y	62.6	-	35	88	-	-	
Bedrock	29.7	-	139.3	-	-	Strong Rock	Y	101.6	-	-	-	-	12983	
LS Pole 7 Station 63+25, Offset 30.0' LT, Layering[12]														
Granular Borrow	0.0	3.5	168.0	164.5	3.5	Sand (Reese)	N	125	-	32	88	-	-	Drilled Shaft
Granular Borrow Replacement[6]	3.5	4.5	164.5	163.5	1.0	Sand (Reese)	N	125	-	32	88	-	-	
Fill	4.5	12.5	163.5	155.5	8.0	Sand (Reese)	N	125	-	36	165	-	-	
Bedrock	12.5	-	155.5	-	-	Strong Rock	N	164	-	-	-	-	12983	

GOLDER
MEMBER OF WSP

Date: August 17, 2021
Project No.: 21450908
Subject: Subsurface Layering and Engineering Properties for Light Standard Foundations
Project Title: MaineDOT I-295 Exit 20 Desert Road Bridge Replacement No. 5720

Made by: BK
Checked by: MEL
Reviewed by: CCB

Soil Type	Layering: Depth (ft bgs) Start	Layering: Depth (ft bgs) End	Layering: Elevation (ft) Start	Layering: Elevation (ft) End	Layering: Thickness (ft)	Lateral Model[1]	Below GW table[2]?	Effective Unit Weight (pcf)[3]	Undrained Shear Strength[4] (psf)	Friction Angle[4] (°)	Subgrade Modulus[5] (pci)	Major Principal Strain at 50%[5]	UCS[3] (psi)	Design Note
LS Pole 8 Station 64+25, Offset 30.0' LT, Layering[13]														
Granular Borrow	0.0	2.5	163.5	161.0	2.5	Sand (Reese)	N	125	-	32	88	-	-	Drilled Shaft
Granular Borrow Replacement[6]	2.5	3.5	161.0	160.0	1.0	Sand (Reese)	N	125	-	32	88	-	-	
Fill	3.5	8.0	160.0	155.5	4.5	Sand (Reese)	N	125	-	36	165	-	-	
Bedrock	8.0	-	155.5	-	-	Strong Rock	N	164	-	-	-	-	12983	
LS Pole 10 Station 65+75, Offset 30.0' LT, Layering[14]														
Granular Borrow	0.0	0.4	158.0	157.6	0.4	Sand (Reese)	N	125	-	32	88	-	-	Spread Footing on Rock
Granular Borrow Replacement[6]	0.4	1.4	157.6	156.6	1.0	Sand (Reese)	N	125	-	32	88	-	-	
Fill	1.4	4.0	156.6	154.0	2.6	Sand (Reese)	N	125	-	36	165	-	-	
Bedrock	4.0	-	154.0	-	-	Strong Rock	N	164	-	-	-	-	12983	
LS9 Pole 11 Station 66+60, Offset 48.0' RT, Layering[15]														
Granular Borrow	0.0	0.5	155.5	155.0	0.5	Sand (Reese)	N	125	-	32	88	-	-	Drilled Shaft
Granular Borrow Replacement[6]	0.5	1.5	155.0	154.0	1.0	Sand (Reese)	N	125	-	32	88	-	-	
Fill	1.5	9.7	154.0	145.8	8.2	Sand (Reese)	N	125	-	41	275	-	-	
Bedrock	9.7	-	145.8	-	-	Strong Rock	N	164	-	-	-	-	12983	
LS Pole 12 Station 67+75, Offset 78.0' RT, Layering[16]														
Granular Borrow Replacement[6]	0.0	1.0	154.0	153.0	1.0	Sand (Reese)	N	125	-	32	88	-	-	Drilled Shaft
Fill	1.0	8.2	153.0	145.8	7.2	Sand (Reese)	N	125	-	41	275	-	-	
Bedrock	8.2	-	145.8	-	-	Strong Rock	N	164	-	-	-	-	12983	
LS Pole 14 Station 104+40, Offset 15.0' RT, Layering[17]														
Granular Borrow	0.0	0.5	150.5	150.0	0.5	Sand (Reese)	N	125	-	32	88	-	-	Drilled Shaft
Granular Borrow Replacement[6]	0.5	1.5	150.0	149.0	1.0	Sand (Reese)	N	125	-	32	88	-	-	
Glaciomarine	1.5	5.9	149.0	144.6	4.4	Stiff Clay with Free Water (Reese)	N	119	3608	-	200	0.005	-	
Sand/Gravel	5.9	7.2	144.6	143.3	1.3	Sand (Reese)	N	125	-	37	187	-	-	
Bedrock	7.2	-	143.3	-	-	Strong Rock	N	164	-	-	-	-	12983	
LS Pole 13 Station 105+75, Offset 15.0' RT, Layering[18]														
Granular Borrow	0.0	1.5	156.0	154.5	1.5	Sand (Reese)	N	125	-	32	88	-	-	Drilled Shaft
Granular Borrow Replacement[6]	1.5	2.5	154.5	153.5	1.0	Sand (Reese)	N	125	-	32	88	-	-	
Fill	2.5	5.7	153.5	150.3	3.2	Sand (Reese)	N	125	-	35	146	-	-	
Glaciomarine	5.7	11.4	150.3	144.6	5.7	Stiff Clay with Free Water (Reese)	N	119	3608	-	200	0.005	-	
Sand/Gravel	11.4	12.7	144.6	143.3	1.3	Sand (Reese)	N	125	-	37	187	-	-	
Bedrock	12.7	-	143.3	-	-	Strong Rock	N	164	-	-	-	-	12983	
LS Pole 9 Station 201+00, Offset 14.0 RT, Layering[19]														
Granular Borrow Replacement[6]	0.0	1.0	160.3	159.3	1.0	Sand (Reese)	N	125	-	32	88	-	-	Drilled Shaft
Fill	1.0	6.3	159.3	154.0	5.3	Sand (Reese)	N	125	-	36	165	-	-	
Bedrock	6.3	-	154.0	-	-	Strong Rock	N	164	-	-	-	-	12983	

GOLDER
MEMBER OF WSP

Date:	August 17, 2021		
Project No.:	21450908	**Made by:**	BK
Subject:	Subsurface Layering and Engineering Properties for Light Standard Foundations	**Checked by:**	MEL
Project Title:	MaineDOT I-295 Exit 20 Desert Road Bridge Replacement No. 5720	**Reviewed by:**	CCB

Soil Type	Layering					Lateral Model[1]	Below GW table[2]?	Effective Unit Weight (pcf)[3]	Undrained Shear Strength[4] (psf)	Friction Angle[4] (°)	Subgrade Modulus[5] (pci)	Major Principal Strain at 50%[5]	UCS[3] (psi)	Design Note
	Depth (ft bgs)		Elevation (ft)		Thickness (ft)									
	Start	End	Start	End										
LS Pole 3 Station 300+75, Offset 18.0' LT, Layering[20]														
Granular Borrow Replacement[6]	0.0	0.4	152.3	151.9	0.4	Sand (Reese)	N	125	-	32	88	-	-	Drilled Shaft
Fill	0.4	9.1	151.9	143.2	8.7	Sand (Reese)	N	125	-	40	253	-	-	
Glaciomarine	9.1	11.3	143.2	141.0	2.2	Stiff Clay with Free Water (Reese)	N	119	2386	-	200	0.005	-	
Glaciomarine	11.3	18.4	141.0	133.9	7.1	Stiff Clay w/o Free Water (Reese)	Y	56.6	2386	-	200	0.005	-	
Bedrock	18.4	-	133.9	-	-	Strong Rock	Y	101.6	-	-	-	-	12983	
Design basis for a drilled shaft light standard foundation without glaciomarine soil present (based on LS Pole 9)														
Granular Borrow Replacement[6]	0.0	1.0	160.3	159.3	1.0	Sand (Reese)	N	125	-	32	88	-	-	Design Drilled Shaft. Friction angle reduced based on engineering judgement
Fill	1.0	6.3	159.3	154.0	5.3	Sand (Reese)	N	125	-	32	88	-	-	
Bedrock	6.3	-	154.0	-	-	Strong Rock	N	164	-	-	-	-	12983	
Design basis for a drilled shaft light standard foundation with glaciomarine soil present (based on LS Pole 14)														
Granular Borrow	0.0	0.5	150.5	150.0	0.5	Sand (Reese)	N	125	-	32	88	-	-	Design Drilled Shaft. Friction angle and undrained shear strength reduced based on engineering judgement
Granular Borrow Replacement[6]	0.5	1.5	150.0	149.0	1.0	Sand (Reese)	N	125	-	32	88	-	-	
Glaciomarine	1.5	5.9	149.0	144.6	4.4	Stiff Clay with Free Water (Reese)	N	119	2300	-	200	0.005	-	
Sand/Gravel	5.9	7.2	144.6	143.3	1.3	Sand (Reese)	N	125	-	36	165	-	-	
Bedrock	7.2	-	143.3	-	-	Strong Rock	N	164	-	-	-	-	12983	
Design basis for a spread footing light standard foundation on rock (based on LS Pole 1)														
Granular Borrow Replacement[6]	0.0	1.0	145.0	144.0	1.0	Sand (Reese)	N	125	-	32	88	-	-	Design Spread Footing on Rock
Fill	1.0	3.8	144.0	141.2	2.8	Sand (Reese)	N	125	-	32	88	-	-	
Bedrock	3.8	-	141.2	-	-	Strong Rock	Y	101.5	-	-	-	-	12983	
Design basis for a spread footing light standard foundation on soil (based on LS Pole 2)														
Granular Borrow	0.0	1.0	152.0	151.0	1.0	Sand (Reese)	N	125	-	32	88	-	-	Design Spread Footing on Soil. Friction angle reduced based on engineering judgement
Granular Borrow Replacement[6]	1.0	2.0	151.0	150.0	1.0	Sand (Reese)	N	125	-	32	88	-	-	
Fill	2.0	10.8	150.0	141.2	8.8	Sand (Reese)	N	125	-	33	108	-	-	
Bedrock	10.8	-	141.2	-	-	Strong Rock	Y	101.6	-	-	-	-	12983	

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MEMBER OF WSP

Date:	August 17, 2021		
Project No.:	21450908	**Made by:**	BK
Subject:	Subsurface Layering and Engineering Properties for Light Standard Foundations	**Checked by:**	MEL
Project Title:	MaineDOT I-295 Exit 20 Desert Road Bridge Replacement No. 5720	**Reviewed by:**	CCB

Soil Type	Layering					Lateral Model[1]	Below GW table[2]?	Effective Unit Weight (pcf)[3]	Undrained Shear Strength[4] (psf)	Friction Angle[4] (°)	Subgrade Modulus[5] (pci)	Major Principal Strain at 50%[5]	UCS[3] (psi)	Design Note
	Depth (ft bgs)		Elevation (ft)		Thickness (ft)									
	Start	End	Start	End										

Notes:

1. Lateral model: Isenhower, W.M., Wang, S.T., and Vasquez, L.G. LPILE v2019 Technical Manual: A Program for the Analysis of Deep Foundations Under Lateral Loading. Ensoft, Inc. Release: March 2020.
2. Water table determined based on boring log stratigraphy (Appendix A, SGDR Part II)
3. Soil properties determined based on the PGDR (Golder Associates, Inc., December 21, 2020, Preliminary Geotechnical Design Report, I-295 Mallet Drive Bridge
4. Soil properties correlated from SPT N_{60} values
5. Bridge Software Institute. FB-MultiPier Soil Parameter Table (US Customary Units). Accessed 7/2021. https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf
6. We propose to remove and replace the upper 1 foot of existing soil with new Gravel Borrow to remove organics.
7. Layering determined from stratigraphy from BB-FDR-201 for in situ soils; Starting elevation for Granular Borrow Replacement is the proposed elevation at Sta. 54+00 from HNTB 98% plans; Starting elevation for Granular Borrow Replacement is the existing ground surface from the HNTB 98% plans for the actual location.
8. Layering determined from stratigraphy from BB-FDR-201 for in situ soils; Starting elevation for Granular Borrow is the proposed elevation at Sta. 55+25 from HNTB 98% plans; Starting elevation for Granular Borrow Replacement is the existing ground surface from the HNTB 98% plans for the actual location.
9. Layering determined from stratigraphy from BB-FDR-217 for in situ soils; Starting elevation for Granular Borrow is the proposed elevation at Sta. 56+25 from HNTB 98% plans; Starting elevation for Granular Borrow Replacement is the existing ground surface from the HNTB 98% plans for the actual location.
10. Layering determined from stratigraphy from BB-FDR-205 for in situ soils; Starting elevation for Granular Borrow is the proposed elevation at Sta. 58+25 from HNTB 98% plans; Starting elevation for Granular Borrow Replacement is the existing ground surface from the HNTB 98% plans for the actual location.
11. Layering determined from stratigraphy from BB-FDR-206 for in situ soils; Starting elevation for Granular Borrow is the proposed elevation at Sta. 50+50 from HNTB 98% plans; Starting elevation for Granular Borrow Replacement is the existing ground surface from the HNTB 98% plans for the actual location.
12. Layering determined from stratigraphy from BB-FDR-210 for in situ soils; Starting elevation for Granular Borrow is the proposed elevation at Sta. 63+25 from HNTB 98% plans; Starting elevation for Granular Borrow Replacement is the existing ground surface from the HNTB 98% plans for the actual location.
13. Layering determined from stratigraphy from BB-FDR-210 for in situ soils; Starting elevation for Granular Borrow is the proposed elevation at Sta. 64+25 from HNTB 98% plans; Starting elevation for Granular Borrow Replacement is the existing ground surface from the HNTB 98% plans for the actual location.
14. Layering determined from stratigraphy from BB-FDR-212 for in situ soils; Starting elevation for Granular Borrow is the proposed elevation at Sta. 65+75 from HNTB 98% plans; Starting elevation for Granular Borrow Replacement is the existing ground surface from the HNTB 98% plans for the actual location.
15. Layering determined from stratigraphy from BB-FDR-215 for in situ soils; Starting elevation for Granular Borrow Replacement is the proposed elevation at Sta. 66+50 from HNTB 98% plans; Starting elevation for Granular Borrow Replacement is the existing ground surface from the HNTB 98% plans for the actual location.
16. Layering determined from stratigraphy from BB-FDR-215 for in situ soils; Starting elevation for Granular Borrow Replacement is the proposed elevation at Sta. 67+75 from HNTB 98% plans; Starting elevation for Granular Borrow Replacement is the existing ground surface from the HNTB 98% plans for the actual location.
17. Layering determined from stratigraphy from BB-FDR-211 for in situ soils; Starting elevation for Granular Borrow is the proposed elevation at Sta. 104+50 from HNTB 98% plans; Starting elevation for Granular Borrow Replacement is the existing ground surface from the HNTB 98% plans for the actual location.
18. Layering determined from stratigraphy from BB-FDR-211 for in situ soils; Starting elevation for Granular Borrow is the proposed elevation at Sta. 105+75 from HNTB 98% plans; Starting elevation for Granular Borrow Replacement is the existing ground surface from the HNTB 98% plans for the actual location.
19. Layering determined from stratigraphy from BB-FDR-212 for in situ soils; Starting elevation for Granular Borrow Replacement is the proposed elevation at Sta. 201+00 from HNTB 98% plans; Starting elevation for Granular Borrow Replacement is the existing ground surface from the HNTB 98% plans for the actual location.
20. Layering determined from stratigraphy from BB-FDR-202 for in situ soils; Starting elevation for Granular Borrow Replacement is the proposed elevation at Sta. 300+75 from HNTB 98% plans; Starting elevation for Granular Borrow Replacement is the proposed ground surface from the HNTB 98% plans for the actual location.

APPENDIX E

Drilled Shaft Design Calculations

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CALCULATIONS

Date:	8/5/2021	**Made by:**	KAR
Project No.:	21450908	**Checked by:**	MSG
Subject:	Drilled Shaft Design for Mast Arm 2 at Station 56+79.94	**Reviewed by:**	JEL
Project Title:	MaineDOT Desert Road Bridge 5720 Freeport (Exit 20) Phase 2		

OBJECTIVE

Determine if the proposed drilled shaft foundation will provide adequate support for Mast Arm 2 (Station 56+79.94, Offset 22.6' RT) at Exit 20 based on the final design loads provided by HNTB.

METHOD

As per the MaineDOT Bridge Design Guide Section 5.8 (Ref. 2), use the procedure outlined in AASHTO LRFD Article 10.8 (Ref. 1) and FHWA GEC-10 (Ref. 4).

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. AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals, 1st Ed. 2015 with 2017, 2018, and 2020 Interim Revisions.
4. FHWA. Drilled Shafts: Construction Procedures and Design Methods. Publication No. FHWA-NHI 18-024 and FHWA GEC 010. September 2018.
5. Golder geotechnical test boring logs for 200-series borings.
6. Golder geotechnical test boring logs for 100-series borings (Appendix A, Preliminary Geotechnical Design Report, dated December 21, 2020).
7. Bridge Software Institute. FB-MultiPier Soil Parameter Table (US Customary Units). Accessed July 2021. https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf
8. Email communication between Golder and HNTB, subject "Freeport - Lighting and Signal Foundation Locations and Design Loads", dated June 29, 2021.
9. GeoTesting Express laboratory testing results, dated February 5, 2020 (Appendix C, Preliminary Geotechnical Design Report, dated December 21, 2020).
10. HNTB for State of Maine Department of Transportation. Merrill Road Bridge over Interstate 295 and Signalized Intersections, Exit 20 Interchange: 98% PS&E, dated July 30, 2021.
11. Holtz, R.D. and Kovacs, W.D. 1981. An Introduction to Geotechnical Engineering, 1st ed. Prentice Hall, Englewood Cliffs, NJ.
12. FHWA. Geotechnical Engineering Circular No. 6: Shallow Foundations. Report No. FHWA-SA-02-054. September 2002.
13. MaineDOT. Standard Details. March 2020.
14. VTrans. Materials & Research Engineering Instructions MREI 10-01. March 9, 2010.
15. Rodriguez, C.M., et al. Final Report: State of Practice and Literature Review on Foundations for Coastal Traffic Signal Mast Arm Structures. State of North Carolina Department of Transportation Research & Development, Report No. FHWA/NC/2018-17. May 14, 2020.
16. Florida Department of Transportation. FDOT Modifications to LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals (LRFDLTS-1). Structures Manual, Volume 3. January 2021.

ASSUMPTIONS

1. The drilled shafts are assumed to be installed either open hole or with temporary casing that is removed after construction. It is assumed that permanent casing will not be used, and the effect of permanent casing is not included in the analysis of nominal side resistance.
2. The topmost 1 foot of existing soil is removed and replaced by granular borrow at the drilled shaft location.

3. Typical steel reinforcement details are assumed for the purpose of the serviceability limit state lateral geotechnical analysis. Full steel reinforcement design and structural analysis will be performed by a non-Golder structural engineer.
4. Shaft head deflection is limited to 0.5 inches under Service loads and less than 10% of the shaft diameter under a "pushover" analysis (step 6). Furthermore, total shaft rotation at the end of the pushover analysis is limited so that the estimated vertical movement at the tip of the mast arm will not exceed 6 inches.
5. The glaciomarine silty clay is asssumed to be sufficiently overconsolidated such that it will experience only recompression settlement after loading (based on Golder's local engineering experience).
6. The glaciomarine silty clay is assumed to be undrained, and thus total stress analysis is used in calculating nominal shaft resistance via the alpha method as per Ref. 1 Article C10.8.3.5.1b and Ref. 4 page 10-18.

ATTACHMENTS

1. LPile analysis output for Strength I, Extreme I, and Service I (nonlinear model)

CALCULATION

Summary of calculation steps:
1. Define subsurface profile for analysis.
2. Select trial shaft diameter, shaft length, and shaft reinforcement for evaluation.
3. Calculate settlement and resulting downdrag loading at the drilled shaft location.
4. Define the factored design loads for the analysis.
5. Check geotechnical axial compression resistance of the shaft at the Strength I limit state.
6. Check lateral geotechnical resistance of the shaft at the Strength I and Extreme I limit states.
7. Check horizontal movement at the top of the shaft at the Service I limit state.
8. Check embedment length to resist torsion loading demand at the Extreme I limit state.
9. Check estimated vertical movement at the tip of the mast arm during the pushover analysis.

1. Define subsurface profile for analysis.

The soil stratigraphy encountered in nearby boring BB-FDR-204 is used for the analysis at Mast Arm 2.

Proposed top of drilled shaft elevation =	157.0	ft	(Ref. 10, Sheet 35, Station 56+75.00)
Existing ground surface elevation =	156.5	ft	(Ref. 10, Sheet 35, Station 56+75.00)
Elevation to which remove/replace soil =	155.5	ft	(Assumption 2)
Groundwater (GW) elevation extrapolated from BB-FDR-204 to drilled shaft location =	148.5	ft	(Ref. 5)

Layer	Depth below top of shaft[1]	Lateral Model	Effective Unit Weight (pcf)[2]	Average N_{60} Value[1]	Design Friction Angle (°)[2]	Undrained Shear Strength (psf)[2]	Subgrade Modulus (pci)[3]	Major Principal Strain at 50%[3]	UCS (psi)[4]
Granular Borrow	0 - 0.5 ft	Sand (Reese)	125	-	32	-	88	-	-

Granular Borrow Replacement	0.5 - 1.5 ft	Sand (Reese)	125	-	32	-	88	-	-
Existing Fill (above GW)	1.5 - 8.5 ft	Sand (Reese)	125	6	29	-	20	-	-
Existing Fill (below GW)	8.5 - 12 ft	Sand (Reese)	62.6	6	29	-	20	-	-
Glaciomarine Silty Clay	12 - 12.7 ft	Stiff Clay w/o Free Water (Reese)	56.6	15	-	1746	-	0.005	-
Sand / Gravel	12.7 - 17.5 ft	Sand (Reese)	62.6	16	32	-	53	-	-
Bedrock	> 17.5 ft	Strong Rock (Vuggy Limestone)	101.6	-	-	-	-	-	12983

1) Ref. 5
2) Proposed soils: Ref. 2, Table 3-3. Existing soils: correlation to average N_{60} value for each layer, Ref. 5.
3) Ref. 7. Interpolation based on friction angle for cohesionless layers and on undrained shear strength for cohesive layers.
4) Ref. 6 and 9

Clay consolidation parameters for the Glaciomarine Silty Clay layer are based on Golder's local engineering experience ($C_{c\varepsilon}$, $C_{r\varepsilon}$, c_v) and calculated (e_0) from soil moisture contents determined in laboratory testing (Reference 9).

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

2. Select trial shaft diameter, shaft length, and shaft reinforcement for evaluation.

The analysis will be evaluated with the minimum shaft diameter to accommodate poles and anchorages as provided by HNTB (Reference 8):

Shaft diameter for Traffic Signals =	36	inches	=	3.0	feet
which corresponds to shaft circumference =	113	inches	=	9.4	feet
and shaft base area =	1018	in^2	=	7.1	ft^2

The analysis will be evaluated with the minimum shaft length necessary to meet the torsional resistance requirement in Step 8:

Shaft length = 13 feet

Ref. 3 Article 13.6.2.1 requires a minimum concrete cover of 3 inches over steel reinforcement (to protect against corrosion). Ref. 4 Section 12.4 recommends a minimum concrete cover of 4 inches for shaft diameters greater than 3 feet and smaller than 5 feet.

Concrete cover to edge of bar =	3	in		(Ref. 3, Article 13.6.2.1)
Concrete compressive strength, f'_c =	5	ksi		(Ref. 1, Article C5.6.4.2)

Ref. 1 Article 5.6.4.2 and Ref. 4 Section 12.3.1 recommend a steel longitudinal reinforcement area of 1% to 1.5% of the gross area of the section and require a minimum of six bars in a circular arrangement plus a minimum #5 bar size. The MaineDOT Standard Details (Ref. 13, pg. 626(01) and 626(02)) specify six #6-size bars for an 18-inch diameter shaft and eight #6-size bars for a 24-inch diameter shaft, corresponding to 1.04% and 0.78% steel area, respectively.

This analysis will be evaluated with twelve Grade 60 #6-size bars, arranged in a symmetrical circular pattern with single-bar bundles, for the purpose of geotechnical design only.

Steel reinforcement area per bar =	0.44	in^2		(Ref. 4, Table 6-2, #14-size)
Number of bars =	12			
Steel reinforcement area per shaft =	5.28	in^2	=	0.52% of total shaft area
Steel yield strength, F_y =	60	ksi		(Ref. 4, Section 12.2.2)
Steel elastic modulus, E =	29,000	ksi		(Ref. 4, Section 12.2.2)

As per Assumption 3, steel transverse reinforcement for structural design purposes is not analyzed. Tie hoop reinforcement will be assumed for constructability purposes, as per Ref. 4 Section 6.1. Assume #3-size ties (based on Ref. 13 page 626(02)) spaced a maximum of 12 inches apart (based on Ref. 1 Article 5.7.2.6), resulting in 13 ties for a shaft length of 13 feet.

Steel reinforcement area per tie, A_v =	0.11	in^2	(Ref. 4, Table 6-2, #3-size)
Steel yield strength for ties =	60	ksi	(Ref. 4, Section 12.2.2)
Spacing of the ties, s =	12	in	(Ref. 1, Article 5.7.2.6)
Number of ties =	13		

3. Calculate settlement and resulting downdrag loading at the drilled shaft location.

Ref. 1 Article 3.11.8 indicates that downdrag on drilled shafts can be assumed to fully develop in soil layers where settlement is equal to or greater than 0.4 inches. Calculate the estimated settlement due to embankment construction at the drilled shaft location and due to axial load on the drilled shaft in order to determine the potential for downdrag loading.

3a. Settlement due to embankment construction, calculated for soil below the base of the Granular Borrow Replacement:

Begin by determining the change in effective stress state within the soil at the proposed drilled shaft location to identify if settlement or heave will occur. The change in effective stress state due to change in stratigraphy is determined at an elevation of 155.5 ft (elevation of the base of the remove/replace soil at the drilled shaft location). Calculate the vertical stress increase from the proposed embankment loading.

Existing Conditions (Ref. 10, Sheet 35, Station 56+75.00):

Layer	Unit weight (pcf)	Elevation (ft)		Thickness (ft)	Stress Contribution of Layer (psf)
		Top	Bottom		
Existing Fill	125	156.5	155.5	1.0	125

After Construction (Ref. 10, Sheet 35, Station 56+75.00):

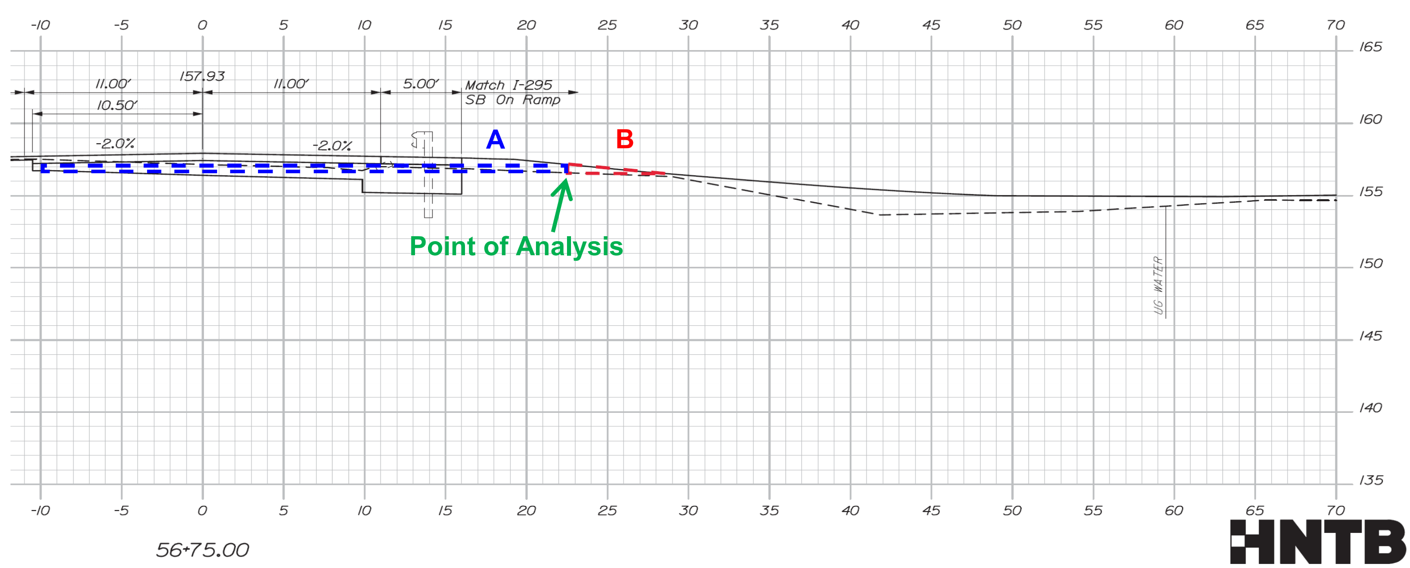
Layer	Unit weight (pcf)	Elevation (ft)		Thickness (ft)	Stress Contribution of Layer (psf)
		Top	Bottom		
Granular Borrow	125	157.0	156.5	0.5	63
Granular Borrow Replacement	125	156.5	155.5	1.0	125

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

	σ'_v at Elev. 155.5 ft (psf)	$\Delta\sigma'_v$ at Elev. 155.5 ft (psf)	Result
Existing Conditions	125	63	Settlement
After Construction	188		

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

Assumed Fill Loading (section view):



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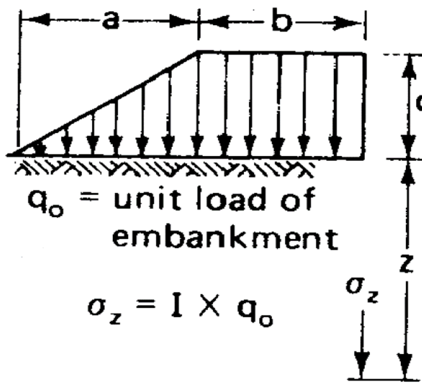
CALCULATIONS

Date:	8/5/2021	**Made by:**	KAR
Project No.:	21450908	**Checked by:**	MSG
Subject:	Drilled Shaft Design for Mast Arm 2 at Station 56+79.94	**Reviewed by:**	JEL
Project Title:	MaineDOT Desert Road Bridge 5720 Freeport (Exit 20) Phase 2		



(Ref. 11, Figure 8.23)

$$\sigma_z = q_0 \times I$$

(Ref. 11, Eq. 8-30)

where:
- σ_z = Vertical stress increase (psf)
- I = influence value from Ref. 11, Figure 8.23
- z = Depth to midpoint of layer (ft)

	Trapezoid A	Trapezoid B		
Stress applied by fill loading, q_0 =	63	63	psf	(From above)
Dimension of embankment slope, a =	0.0	6.0	ft	(Ref. 10, Sheet 35, Station 56+75.00)
Dimension of embankment top, b =	32.6	0.0	ft	(Ref. 10, Sheet 35, Station 56+75.00)

Trapezoid A:

Layer		Depth below footing (ft)	Layer Thickness (ft)	z (ft)	a/z	b/z	I	Stress Increase σ_z (psf)
Existing Fill	1	0-7	7.0	3.5	0.0	9.3	0.500	31.5
Existing Fill	2	7-10.5	3.5	8.8	0.0	3.7	0.500	31.5
Glaciomarine	3	10.5-11.2	0.7	10.9	0.0	3.0	0.492	31.0
Sand/Gravel	4	11.2-16	4.8	13.6	0.0	2.4	0.485	30.6

Trapezoid B:

Layer		Depth below footing (ft)	Layer Thickness (ft)	z (ft)	a/z	b/z	I	Stress Increase σ_z (psf)
Existing Fill	1	0-7	7.0	3.5	1.7	0.0	0.345	21.7
Existing Fill	2	7-10.5	3.5	8.8	0.7	0.0	0.190	12.0
Glaciomarine	3	10.5-11.2	0.7	10.9	0.6	0.0	0.167	10.5
Sand/Gravel	4	11.2-16	4.8	13.6	0.4	0.0	0.120	7.6

Total Footing (Trapezoid A + Trapezoid B):

Layer		Depth below footing (ft)	Layer Thickness (ft)	z (ft)	Stress Increase (psf)
Existing Fill	1	0-7	7.0	3.5	53.2
Existing Fill	2	7-10.5	3.5	8.8	43.5
Glaciomarine	3	10.5-11.2	0.7	10.9	41.5
Sand/Gravel	4	11.2-16	4.8	13.6	38.1

Use the Hough method to estimate settlement of the Existing Fill and Sand/Gravel layers (Layers 1, 2, and 4); use consolidation theory to estimate settlement of the Glaciomarine Silty Clay layer (Layer 3).

Hough method:

Determine σ'_{v0} and C' at layer midpoints for in situ existing soil. Use Ref. 12 Figures 5-18 and 5-19 to obtain corrected N' and C' for Layers 1, 2, and 4, assuming the Existing Fill matches "Clean well graded fine to coarse SAND" and the Sand/Gravel matches "Well graded silty SAND & GRAVEL".

Stress due to existing soil above footing	σ'_{v0} (psf)
	125

Layer		Layer Thickness (ft)	Effective Unit Weight of Layer (pcf)	σ'_{v0} at layer midpoint (psf)	σ'_{v0} at layer midpoint (kPa)	N_{60}	N'	C'
Existing Fill	1	7.0	125	563	26.9	6	11	50
Existing Fill	2	3.5	62.6	1110	53.1	6	8	45
Glaciomarine	3	0.7	56.6	1239	59.3	Not required for clay consol.		
Sand/Gravel	4	4.8	62.6	1409	67.4	16	19	74

Hough method general equation:

$$\Delta H_i = H_c \frac{1}{C'} \log\left(\frac{\sigma'_{v0} + \Delta\sigma_v}{\sigma'_{v0}}\right)$$ (Ref. 12. Eqn 5-24)

where:

- ΔH_i settlement in each layer, ft
- H_c initial height of layer i, ft
- C' bearing capacity index from Ref. 12, Figure 5-19
- $\Delta\sigma_v$ vertical stress increase, ksf

ΔH_i (Layer 1)	ft	0.005
	in	0.07

ΔH_i (Layer 2)	ft	0.001
	in	0.02

ΔH_i (Layer 4)	ft	0.001
	in	0.01

Consolidation theory general equation (Ref. 11, Eq. 8-11, 8-16, and 8-18b):

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

for normally consolidated clay

$$\Delta H_i = C_r \frac{H_0}{1+e_0} \log \frac{\sigma'_{v0} + \Delta\sigma_v}{\sigma'_{v0}}$$

for overconsolidated clay, $\sigma'_{v0} + \Delta\sigma_v \le \sigma'_p$

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

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

where:		Layer 3
H_0	initial height of layer i, ft	0.7
$\Delta\sigma_v$	surcharge load, psf	41.5
σ'_{v0}	in situ vertical effective stress, psf	1239
$\sigma'_{v0} + \Delta\sigma_v$	psf	1280
σ'_p	preconsolidated stress, psf	N/A
C_c	compression index, $C_c = C_{c\varepsilon}(1+e_0)$	0.42
C_r	recompression index, $C_r = C_{r\varepsilon}(1+e_0)$	0.03
e_0	initial void ratio	0.68
	Use equation (Assumption 5):	8-16
ΔH_i	ft	0.000
	in	0.00

Layer	Settlement due to embankment construction ΔH_i (in)
1	0.07
2	0.02
3	0.00
4	0.01

Total Settlement (in)	0.10

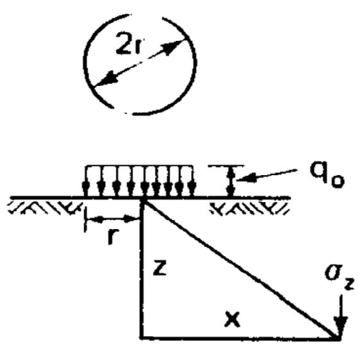
3b. Settlement due to axial load on the drilled shaft, calculated for soil below the base of the shaft:

Base of shaft elevation =	144.0	ft	(From Steps 1 and 2)
Axial load on drilled shaft =	3.5	kips	(Strength I, Ref. 8; see Step 4)
Shaft base area =	7.1	ft^2	(From Step 2)
Stress increase due to axial load =	495	psf	

	Concrete:	Steel:		
Shaft length =	13	13	ft	(From Step 2)
Portion of shaft base area =	7.03	0.04	ft^2	(From Step 2)
Portion of shaft volume =	91.4	0.5	ft^3	
Unit weight, γ =	140	490	pcf	(Assumed)
Portion of shaft weight =	12.8	0.2	kips	
Stress increase due to shaft weight =	1844		psf	

Total stress increase due to axial load and shaft weight =	2339	psf

Subdivide the subsurface soils into layers no larger than 10 feet thick and to a depth of either three times the footing width (shaft diameter) or to bedrock. Calculate the vertical stress increase at each layer, assuming the Boussinesq stress distribution method for stress under a uniformly loaded circular area (Ref. 11, Figure 8.22).



$$\sigma_z = \frac{I \times q_o}{100}$$ (Ref. 11, Figure 8.22)

where:

σ_z = Vertical stress increase (psf)

I = influence value from Ref. 11, Figure 8.22

z = Depth to midpoint of layer (ft)

q_0 = stress increase due to axial load =	2339	psf	(From above)
r = radius of uniformly loaded circular area =	1.5	ft	(Half of shaft diameter from Step 2)
x = offset distance to settlement location =	0	ft	(Calculate settlement below shaft center)

Layer		Depth below footing (ft)	Layer Thickness (ft)	z (ft)	x/r	z/r	I	Stress Increase σ_z (psf)
Sand/Gravel	1	0-4.5	4.5	2.3	0.0	1.5	43	1006

Use the Hough method to estimate settlement of the Sand/Gravel layer:
Determine σ'_{v0} and C' at layer midpoints for in situ existing soil. Use Ref. 12 Figures 5-18 and 5-19 to obtain corrected N' and C' for Layer 1, assuming the Sand/Gravel matches "Well graded silty SAND & GRAVEL".

Stress due to existing soil above footing	σ'_{v0} (psf)
	1276

Layer		Layer Thickness (ft)	Effective Unit Weight of Layer (pcf)	σ'_{v0} at layer midpoint (psf)	σ'_{v0} at layer midpoint (kPa)	N_{60}	N'	C'
Sand/Gravel	1	4.5	62.6	1417	67.8	16	19	74

Hough method general equation:

$$\Delta H_i = H_c \frac{1}{C'} \log\left(\frac{\sigma'_{v0} + \Delta\sigma_v}{\sigma'_{v0}}\right)$$ (Ref. 12. Eqn 5-24)

where:

ΔH_i	settlement in each layer, ft
H_c	initial height of layer i, ft
C'	bearing capacity index from Ref. 12, Figure 5-19
$\Delta\sigma_v$	vertical stress increase, ksf

Settlement due to axial load		
ΔH_i	ft	0.014
	in	0.17

Since the estimated settlement in each soil layer due to embankment construction and axial load on the drilled shaft is less than 0.4 inches, as per Ref. 1 Article 3.11.8 downdrag loading will be assumed to be negligible for this analysis.

4. Define the factored design loads for the analysis.

The factored design loads at the top of the shaft were provided by HNTB (Ref. 8). Since Ref. 8 indicates that Mast Arm 2 uses the "25' Mast Arm" foundation design, the corresponding loads were selected.

Factored Design Forces (25' Mast Arm)					
Load	Units	Strength I	Extreme I	Service I	Service II
Axial	kip	3.5	3.5	3	3
Moment	kip-ft	29	72	38	23
Shear	kip	0	3	2	0
Torsion	kip-ft	0	45	20	0

As per Step 3, no downdrag loading will be added.

5. Check geotechnical axial compression resistance of the shaft at the Strength I limit state.

Compute nominal side resistance for all layers through which the shaft extends and nominal tip resistance for the layer at the tip elevation. Select appropriate resistance factors and calculate factored resistances.

Note that as per Ref. 14 page 5, a minimum of the upper 2 feet of soil should be neglected for contributions to skin friction resistance. As per Ref. 1 Article C10.8.3.5.1b, for cohesive soils at least the top 5 feet of any shaft should not be taken to contribute to the development of resistance through skin friction, to account for the effects of seasonal moisture changes, disturbance during construction, cyclic lateral loading, and low lateral stresses from freshly placed concrete.

The factored resistance of the drilled shaft, R_R, shall be taken as:

$$R_R = \varphi R_n = \varphi_{qp} R_p + \varphi_{qs} R_s \qquad \text{(Ref. 1, Eq. 10.8.3.5-1)}$$

consisting of the nominal shaft tip resistance, R_p:

$$R_p = q_p A_p \qquad \text{(Ref. 1, Eq. 10.8.3.5-2)}$$

and the nominal shaft side resistance, R_s:

$$R_s = q_s A_s \qquad \text{(Ref. 1, Eq. 10.8.3.5-3)}$$

Calculate the unit side resistance q_s at the midpoints of the Existing Fill and Sand/Gravel layers. Use the beta method for cohesionless soils (Ref. 1, Article 10.8.3.5.2b).

$$q_s = \beta\sigma'_v$$ (Ref. 1, Eq. 10.8.3.5.2b-1)

$$\beta = (1 - \sin\phi'_f)\left(\frac{\sigma'_p}{\sigma'_v}\right)^{\sin\phi'_f}\tan\phi'_f$$ (Ref. 1, Eq. 10.8.3.5.2b-2)

For sands: $$\frac{\sigma'_p}{p_a} = 0.47(N_{60})^m$$ (Ref. 1, Eq. 10.8.3.5.2b-4)

For gravelly soils: $$\frac{\sigma'_p}{p_a} = 0.15(N_{60})$$ (Ref. 1, Eq. 10.8.3.5.2b-5)

where:

	Existing Fill	Sand / Gravel		
Depth below top of shaft to midpoint of layer =	6.8	12.9	ft	
Vertical effective stress at midpoint, σ'_v =	0.850	1.334	ksf	
Friction angle for layer, ϕ'_f =	29	32	deg	(From Step 1)
Friction angle for layer, ϕ'_f =	0.51	0.56	rad	
Average N_{60} value for layer =	6	16		(From Step 1)
Atmospheric pressure, p_a =	2.12	2.12	ksf	(Ref. 1, Article 10.8.3.5.2b)
Sand constant, m =	0.6	N/A		(Ref. 1, Article 10.8.3.5.2b)
Preconsolidation stress, σ'_p =	2.92	5.09	ksf	(Ref. 1, Eq. 10.8.3.5.2b-4)

	Existing Fill	Sand / Gravel	
β =	0.52	0.60	
q_s =	0.44	0.80	ksf

Calculate the unit side resistance q_s at the midpoint of the Glaciomarine Silty Clay layer. Use the alpha method for cohesive soils (Ref. 1, Article 10.8.3.5.1b).

$$q_s = \alpha S_u$$ (Ref. 1, Eq. 10.8.3.5.1b-1)

$$\alpha = 0.55 \text{ for } \frac{S_u}{p_a} \leq 1.5$$ (Ref. 1, Eq. 10.8.3.5.1b-2)

$$\alpha = 0.55 - 0.1\left(\frac{S_u}{p_a} - 1.5\right) \text{ for } 1.5 \leq \frac{S_u}{p_a} \leq 2.5$$ (Ref. 1, Eq. 10.8.3.5.1b-3)

where:

Undrained shear strength for layer, S_u =	1.746	ksf	(From Step 1)
Atmospheric pressure, p_a =	2.12	ksf	(Ref. 1, Article 10.8.3.5.1b)
S_u / p_a =	0.82		
Adhesion factor, α =	0.55		(Ref. 1, Eq. 10.8.3.5.1b-2)

Glaciomarine q_s = 0.96 ksf

Based on the shaft length selected in Step 2, the drilled shaft will terminate in the Sand/Gravel layer. Calculate the unit tip resistance q_p for the Sand/Gravel layer. Use the cohesionless soil method (Ref. 1, Article 10.8.3.5.2c). Note that as per Ref. 1 Article 10.8.3.5.2c, the value of q_p should be limited to 60 ksf unless greater values can be justified through load test data.

$$\text{If } N_{60} \leq 50, \text{then } q_p = 1.2N_{60} \leq 60 \text{ ksf}$$ (Ref. 1, Eq. 10.8.3.5.2c-1)

where:

Average N_{60} value for layer = 16 ksf (From Step 1)

Sand/Gravel q_p = 19.2 ksf

Compute nominal and factored side and tip resistances for all layers.

	q_s (ksf)	Shaft circumference (ft)	Length of shaft* (ft)	A_s (ft^2)	R_s (kips)	φ_{qs}**	$R_{R,s}$ (kips)
Existing Fill	0.44	9.4	10.0	94.2	41.6	0.55	22.9
Glaciomarine	0.96	9.4	0.7	6.6	6.3	0.45	2.9
Sand/Gravel	0.80	9.4	0.3	2.8	2.3	0.55	1.2

* neglecting top 2 feet of shaft for Existing Fill length as per Ref. 14, page 5
** Ref. 1, Table 10.5.5.2.4-1

	q_p (ksf)	A_p (ft^2)	R_p (kips)	φ_{qp}*	$R_{R,p}$ (kips)
Sand/Gravel	19.2	7.1	135.7	0.50	67.9

* Ref. 1, Table ´3

Factored geotechnical axial compression resistance of the drilled shaft:

$R_R = R_{R,s} + R_{R,p}$ = 95 kips

Check against Strength I factored axial design load:

Strength I factored axial design load:		Factored geotechnical axial compression resistance:	
3.5 kips	<	95 kips	OK

Demand capacity ratio = 0.04

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CALCULATIONS

Date:	8/5/2021	**Made by:**	KAR
Project No.:	21450908	**Checked by:**	MSG
Subject:	Drilled Shaft Design for Mast Arm 2 at Station 56+79.94	**Reviewed by:**	JEL
Project Title:	MaineDOT Desert Road Bridge 5720 Freeport (Exit 20) Phase 2		

6. Check lateral geotechnical resistance of the shaft at the Strength I and Extreme I limit states.

Perform a pushover analysis using LPile to compute shaft head deflection at various multiples of the factored shear and moment design loads, up to 1/φ times the factored shear and moment design loads. As per Ref. 4 Section 9.3.3.3.1, for a stable condition the analyses should each converge to a solution with a computed deflection no larger than 10% of the shaft diameter. For the pushover analysis the shaft should be modeled as a simple linear elastic beam rather than a nonlinear stress-strain model (Ref. 4, Section 9.3.3.3.1).

Elastic modulus for linear model = 4,000,000 psi (Ref. 4, page 9-21)

Moment of inertia for linear model, I = 82,448 in^4 (Ref. 4, page 9-21: $I = \pi D^4/64$)

Recommended resistance factor φ for lateral geotechnical resistance (Ref. 4, Table 9-1):

Limit State	φ	1/φ
Strength I	0.67	1.5
Extreme I	0.8	1.25

Computed shaft deflection at each load multiple analyzed with LPile:

Limit State	LPile Load Case	Multiple of Factored Design Loads	Loads Applied in LPile			Computed Lateral Head Deflection from LPile (in)	Deflection < 10% of Shaft Diameter?
			Axial (kips)	Moment (kip-ft)	Shear (kips)		
Strength I	1	0.25	3.5	7	0	0.01	Yes
	2	0.50	3.5	15	0	0.03	Yes
	3	0.75	3.5	22	0	0.04	Yes
	4	1.00	3.5	29	0	0.06	Yes
	5	1.25	3.5	36	0	0.08	Yes
	6	1.50	3.5	44	0	0.10	Yes
Extreme I	7	0.25	3.5	18	1	0.06	Yes
	8	0.50	3.5	36	2	0.13	Yes
	9	0.75	3.5	54	2	0.18	Yes
	10	1.00	3.5	72	3	0.28	Yes
	11	1.25	3.5	90	4	0.39	Yes
					maximum =	0.39	

The trial shaft length of 13 feet exhibits stable behavior up through 1/φ times the factored design loads, and the maximum computed deflection of 0.39 inches is less than 10% of the shaft diameter (10% of 36 inches = 3.6 inches). Thus, based on the pushover analysis, the trial shaft length of 13 feet satisfies the lateral geotechnical criterion at the Strength I and Extreme I limit states.

7. Check horizontal movement at the top of the shaft at the Service I limit state.

Use LPile to compute shaft head deflection at the Service I factored design loads. As per Ref. 4 page 9-27 and Ref. 14 page 3, deflection due to combined loading on the structure should be limited to 0.5 inches at the top of the shaft. For the serviceability analysis the shaft should be modeled as a nonlinear reinforced concrete shaft in flexure (Ref. 4, Section 9.3.3.3.3).

Computed shaft deflection analyzed with LPile:

Limit State	LPile Load Case	Multiple of Factored Design Loads	Loads Applied in LPile			Computed Lateral Head Deflection from LPile (in)	Deflection < 0.5 Inches?
			Axial (kips)	Moment (kip-ft)	Shear (kips)		
Service I	12	1.00	3	38	2	0.13	Yes

The computed deflection of 0.13 inches is less than the required limit of 0.5 inches. Thus the trial shaft length of 13 feet satisfies the lateral criterion at the Service I limit state. The LPile analysis output for factored design loads at the Service I limit state is included as Attachment 1.

8. Check embedment length to resist torsion loading demand at the Extreme I limit state.

Compute nominal and factored torsion resistance of the drilled shaft. Due to the limitations of the method in Ref. 15 Section 4.2.2 and Ref. 16 Section 13.6.1.1, assume the drilled shaft is installed entirely within cohesionless soil with an N-value equal to that of the existing fill.

$$T_u \leq \phi_{tor} \cdot T_n$$ (Ref. 16, Section 13.6.1.1)

$$T_n = \pi D L F_s \left(\frac{D}{2}\right)$$ (Ref. 16, Section 13.6.1.1)

$$F_s = \sigma'_v \omega_{fdot}$$ (Ref. 16, Section 13.6.1.1)

where:

Factored torsional loading demand, T_u =	45	kip-ft	(From Step 4 - Extreme I)
Resistance factor for torsion, ϕ_{tor} =	1.0	for mast arm structures (Ref. 16, Section 13.6.1.1)	
Shaft diameter, D =	3.0	ft	(From Step 2)
Shaft length =	13	ft	(From Step 2)
Midpoint of shaft =	6.5	ft	
Vertical effective stress at midpoint, σ'_v =	0.813	ksf	
Uncorrected N-value for Existing Fill =	4		(Ref. 5)
Load transfer ratio, ω_{fdot} = 1.5*(N-value/15) =	0.4		(Ref. 16, Section 13.6.1.1, for uncorrected N-values less than 15)
Unit skin friction, F_s =	0.33	ksf	
Shaft length contributing to resistance, L =	11	ft	(neglecting top 2 feet as per Ref. 14, page 5)

Nominal torsion resistance, T_n =	51	kip-ft
Factored torsion resistance, T_r =	51	kip-ft

Check against Extreme I factored torsion design load:

Extreme I factored torsion design load:		Factored torsion resistance:	
45 kip-ft	<	51 kip-ft	OK

Demand capacity ratio = 0.89

9. Check estimated vertical movement at the tip of the mast arm during the pushover analysis.

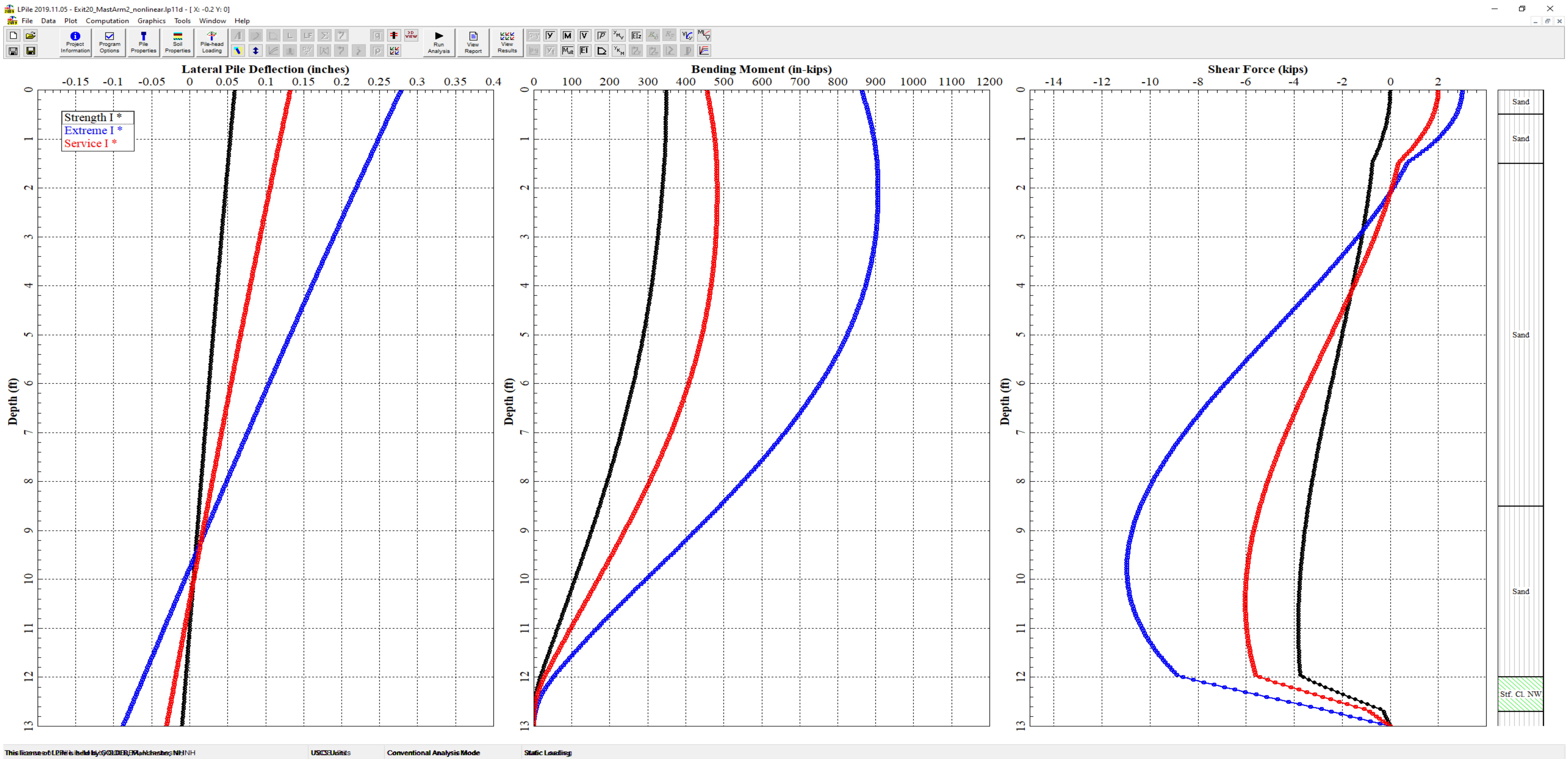
Computed maximum shaft head deflection =	0.39	in	(From LPile, Extreme I factored load × 1.25)			
Computed maximum shaft tip deflection =	0.13	in	(From LPile, Extreme I factored load × 1.25)			
Total shaft deflection =	0.53	in				
Shaft length =	13	ft	=	156	in	(From Step 2)
Total shaft rotation = deflection ÷ length =	0.003	rad				
Mast arm length for Mast Arm 2 =	20	ft	=	240	in	(Ref. 8)
Vertical movement at tip of mast arm = shaft rotation × mast arm length =	0.81	in				

The estimated vertical movement of 0.81 inches at the tip of the mast arm during the pushover analysis is less than the required limit of 6 inches (Assumption 4). Thus the total shaft rotation will be considered sufficiently small.

CONCLUSIONS

The results of the analysis indicate that the proposed drilled shaft foundation with a shaft diameter of 36 inches and a shaft length of 13 feet will provide adequate support for Mast Arm 2 at Exit 20 based on the final design loads provided by HNTB. A maximum lateral deflection of 0.13 inches occurs at the top of the shaft under the Service I load case, satisfying the limiting requirement of 0.5 inches. The shaft length is controlled by the torsional resistance, and a torsion demand capacity ratio of 0.89 was calculated under the Extreme I load case. Although reinforcement consisting of twelve Grade 60 #6 bars arranged in a circular pattern was used in Golder's modeling, it is understood that HNTB will perform the final structural check and generate the required reinforcement pattern.

Attachment 1



Date:	8/5/2021	**Made by:**	KAR
Project No.:	21450908	**Checked by:**	MSG
Subject:	Drilled Shaft Design for Mast Arm 3 at Station 56+07.00	**Reviewed by:**	JEL
Project Title:	MaineDOT Desert Road Bridge 5720 Freeport (Exit 20) Phase 2		

OBJECTIVE
Determine if the proposed drilled shaft foundation will provide adequate support for Mast Arm 3 (Station 56+07.00, Offset 31.0' RT) at Exit 20 based on the final design loads provided by HNTB.

METHOD
As per the MaineDOT Bridge Design Guide Section 5.8 (Ref. 2), use the procedure outlined in AASHTO LRFD Article 10.8 (Ref. 1) and FHWA GEC-10 (Ref. 4).

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. AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals, 1st Ed. 2015 with 2017, 2018, and 2020 Interim Revisions.
4. FHWA. Drilled Shafts: Construction Procedures and Design Methods. Publication No. FHWA-NHI 18-024 and FHWA GEC 010. September 2018.
5. Golder geotechnical test boring logs for 200-series borings.
6. Golder geotechnical test boring logs for 100-series borings (Appendix A, Preliminary Geotechnical Design Report, dated December 21, 2020).
7. Bridge Software Institute. FB-MultiPier Soil Parameter Table (US Customary Units). Accessed July 2021. https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf
8. Email communication between Golder and HNTB, subject "Freeport - Lighting and Signal Foundation Locations and Design Loads", dated June 29, 2021.
9. GeoTesting Express laboratory testing results, dated February 5, 2020 (Appendix C, Preliminary Geotechnical Design Report, dated December 21, 2020).
10. HNTB for State of Maine Department of Transportation. Merrill Road Bridge over Interstate 295 and Signalized Intersections, Exit 20 Interchange: 98% PS&E, dated July 30, 2021.
11. Holtz, R.D. and Kovacs, W.D. 1981. An Introduction to Geotechnical Engineering, 1st ed. Prentice Hall, Englewood Cliffs, NJ.
12. FHWA. Geotechnical Engineering Circular No. 6: Shallow Foundations. Report No. FHWA-SA-02-054. September 2002.
13. MaineDOT. Standard Details. March 2020.
14. VTrans. Materials & Research Engineering Instructions MREI 10-01. March 9, 2010.
15. Rodriguez, C.M., et al. Final Report: State of Practice and Literature Review on Foundations for Coastal Traffic Signal Mast Arm Structures. State of North Carolina Department of Transportation Research & Development, Report No. FHWA/NC/2018-17. May 14, 2020.
16. Florida Department of Transportation. FDOT Modifications to LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals (LRFDLTS-1). Structures Manual, Volume 3. January 2021.

ASSUMPTIONS
1. The drilled shafts are assumed to be installed either open hole or with temporary casing that is removed after construction. It is assumed that permanent casing will not be used, and the effect of permanent casing is not included in the analysis of nominal side resistance.
2. The topmost 1 foot of existing soil is removed and replaced by granular borrow at the drilled shaft location.

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Subject:	Drilled Shaft Design for Mast Arm 3 at Station 56+07.00	**Reviewed by:**	JEL
Project Title:	MaineDOT Desert Road Bridge 5720 Freeport (Exit 20) Phase 2		

3. Typical steel reinforcement details are assumed for the purpose of the serviceability limit state lateral geotechnical analysis. Full steel reinforcement design and structural analysis will be performed by a non-Golder structural engineer.
4. Shaft head deflection is limited to 0.5 inches under Service loads and less than 10% of the shaft diameter under a "pushover" analysis (step 5). Furthermore, total shaft rotation at the end of the pushover analysis is limited so that the estimated vertical movement at the tip of the mast arm will not exceed 6 inches.
5. The immediate settlement of the cohesionless soils is assumed to be sufficiently small that downdrag loading will be negligible.

ATTACHMENTS

1. LPile analysis output for Strength I, Extreme I, and Service I (nonlinear model)

CALCULATION

Summary of calculation steps:
1. Define subsurface profile for analysis.
2. Select trial shaft diameter, shaft length, and shaft reinforcement for evaluation.
3. Define the factored design loads for the analysis.
4. Check geotechnical axial compression resistance of the shaft at the Strength I limit state.
5. Check lateral geotechnical resistance of the shaft at the Strength I and Extreme I limit states.
6. Check horizontal movement at the top of the shaft at the Service I limit state.
7. Check embedment length to resist torsion loading demand at the Extreme I limit state.
8. Check estimated vertical movement at the tip of the mast arm during the pushover analysis.

1. Define subsurface profile for analysis.

The soil stratigraphy encountered in nearby boring BB-FDR-203 is used for the analysis at Mast Arm 3.

Proposed top of drilled shaft elevation =	152.5	ft	(Ref. 10, Sheet 33, Station 56+00.00)
Existing ground surface elevation =	152.5	ft	(Ref. 10, Sheet 33, Station 56+00.00)
Elevation to which remove/replace soil =	151.5	ft	(Assumption 2)
Groundwater elevation =	141.0	ft	(Assumed based on water level measurements obtained in borings near the proposed Abutment 1)

Layer	Depth below top of shaft[1]	Lateral Model	Effective Unit Weight (pcf)[2]	Average N_{60} Value[1]	Design Friction Angle (°)[2]	Undrained Shear Strength (psf)[2]	Subgrade Modulus (pci)[3]	Major Principal Strain at 50%[3]	UCS (psi)[4]
Granular Borrow Replacment	0 - 1 ft	Sand (Reese)	125	-	32	-	88	-	-
Existing Fill	1 - 8.9 ft	Sand (Reese)	125	50	36	-	165	-	-

Bedrock	> 8.9 ft	Strong Rock (Vuggy Limestone)	164	-	-	-	-	-	12983

1) Ref. 5
2) Proposed soils: Ref. 2, Table 3-3. Existing soils: correlation to average N_{60} value for each layer, Ref. 5.
3) Ref. 7. Interpolation based on friction angle for cohesionless layers and on undrained shear strength for cohesive layers.
4) Ref. 6 and 9

2. Select trial shaft diameter, shaft length, and shaft reinforcement for evaluation.

The analysis will be evaluated with the minimum shaft diameter to accommodate poles and anchorages as provided by HNTB (Reference 8):

Shaft diameter for Standard Lighting =	36	inches	=	3.0	feet
which corresponds to shaft circumference =	113	inches	=	9.4	feet
and shaft base area =	1018	in^2	=	7.1	ft^2

The analysis will be evaluated with the minimum shaft length necessary to meet the torsional resistance requirement in Step 8:

Shaft length = 7.5 feet

Ref. 3 Article 13.6.2.1 requires a minimum concrete cover of 3 inches over steel reinforcement (to protect against corrosion). Ref. 4 Section 12.4 recommends a minimum concrete cover of 4 inches for shaft diameters greater than 3 feet and smaller than 5 feet.

Concrete cover to edge of bar =	3	in	(Ref. 3, Article 13.6.2.1)
Concrete compressive strength, f'_c =	5	ksi	(Ref. 1, Article C5.6.4.2)

Ref. 1 Article 5.6.4.2 and Ref. 4 Section 12.3.1 recommend a steel longitudinal reinforcement area of 1% to 1.5% of the gross area of the section and require a minimum of six bars in a circular arrangement plus a minimum #5 bar size. The MaineDOT Standard Details (Ref. 13, pg. 626(01) and 626(02)) specify six #6-size bars for an 18-inch diameter shaft and eight #6-size bars for a 24-inch diameter shaft, corresponding to 1.04% and 0.78% steel area, respectively.

This analysis will be evaluated with twelve Grade 60 #6-size bars, arranged in a symmetrical circular pattern with single-bar bundles, for the purpose of geotechnical design only.

Steel reinforcement area per bar =	0.44	in^2		(Ref. 4, Table 6-2, #14-size)
Number of bars =	12			
Steel reinforcement area per shaft =	5.28	in^2	=	0.52% of total shaft area
Steel yield strength, F_y =	60	ksi		(Ref. 4, Section 12.2.2)
Steel elastic modulus, E =	29,000	ksi		(Ref. 4, Section 12.2.2)

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As per Assumption 3, steel transverse reinforcement for structural design purposes is not analyzed. Tie hoop reinforcement will be assumed for constructability purposes, as per Ref. 4 Section 6.1. Assume #3-size ties (based on Ref. 13 page 626(02)) spaced a maximum of 12 inches apart (based on Ref. 1 Article 5.7.2.6), resulting in 7 ties for a shaft length of 7.5 feet.

Steel reinforcement area per tie, A_v =	0.11	in^2	(Ref. 4, Table 6-2, #3-size)
Steel yield strength for ties =	60	ksi	(Ref. 4, Section 12.2.2)
Spacing of the ties, s =	12	in	(Ref. 1, Article 5.7.2.6)
Number of ties =	7		

3. Define the factored design loads for the analysis.

The factored design loads at the top of the shaft were provided by HNTB (Ref. 8). Since Ref. 8 indicates that Mast Arm 2 uses the "25' Mast Arm" foundation design, the corresponding loads were selected.

Factored Design Forces (25' Mast Arm)					
Load	Units	Strength I	Extreme I	Service I	Service II
Axial	kip	3.5	3.5	3	3
Moment	kip-ft	29	72	38	23
Shear	kip	0	3	2	0
Torsion	kip-ft	0	45	20	0

As per Assumption 5, no downdrag loading will be added.

4. Check geotechnical axial compression resistance of the shaft at the Strength I limit state.

Compute nominal side resistance for all layers through which the shaft extends and nominal tip resistance for the layer at the tip elevation. Select appropriate resistance factors and calculate factored resistances.

Note that as per Ref. 14 page 5, a minimum of the upper 2 feet of soil should be neglected for contributions to skin friction resistance. As per Ref. 1 Article C10.8.3.5.1b, for cohesive soils at least the top 5 feet of any shaft should not be taken to contribute to the development of resistance through skin friction, to account for the effects of seasonal moisture changes, disturbance during construction, cyclic lateral loading, and low lateral stresses from freshly placed concrete.

The factored resistance of the drilled shaft, R_R, shall be taken as:

$$R_R = \varphi R_n = \varphi_{qp} R_p + \varphi_{qs} R_s \qquad \text{(Ref. 1, Eq. 10.8.3.5-1)}$$

consisting of the nominal shaft tip resistance, R_p:

$$R_p = q_p A_p \qquad \text{(Ref. 1, Eq. 10.8.3.5-2)}$$

and the nominal shaft side resistance, R_s:

$$R_s = q_s A_s$$ (Ref. 1, Eq. 10.8.3.5-3)

Calculate the unit side resistance q_s at the midpoint of the Existing Fill layer. Use the beta method for cohesionless soils (Ref. 1, Article 10.8.3.5.2b).

$$q_s = \beta \sigma'_v$$ (Ref. 1, Eq. 10.8.3.5.2b-1)

$$\beta = (1 - \sin \phi'_f)\left(\frac{\sigma'_p}{\sigma'_v}\right)^{\sin \phi'_f} \tan \phi'_f$$ (Ref. 1, Eq. 10.8.3.5.2b-2)

For sands: $$\frac{\sigma'_p}{p_a} = 0.47(N_{60})^m$$ (Ref. 1, Eq. 10.8.3.5.2b-4)

For gravelly soils: $$\frac{\sigma'_p}{p_a} = 0.15(N_{60})$$ (Ref. 1, Eq. 10.8.3.5.2b-5)

where:

Depth below top of shaft to midpoint of layer =	4.3	ft				
Vertical effective stress at midpoint, σ'_v =	0.538	ksf				
Friction angle for layer, ϕ'_f =	36	deg	=	0.63	rad	(From Step 1)
Average N_{60} value for layer =	50		(From Step 1)			
Atmospheric pressure, p_a =	2.12	ksf	(Ref. 1, Article 10.8.3.5.2b)			
Sand constant, m =	0.6	for clean quartzitic sands (Ref. 1, Article 10.8.3.5.2b)				
Preconsolidation stress, σ'_p =	10.42	ksf	(Ref. 1, Eq. 10.8.3.5.2b-4)			
β =	1.71					
Existing Fill q_s =	0.92	ksf				

Based on the shaft length selected in Step 2, the drilled shaft will terminate in the Existing Fill layer. Calculate the unit tip resistance q_p for the Existing Fill layer. Use the cohesionless soil method (Ref. 1, Article 10.8.3.5.2c). Note that as per Ref. 1 Article 10.8.3.5.2c, the value of q_p should be limited to 60 ksf unless greater values can be justified through load test data.

$$\text{If } N_{60} \leq 50, \text{then } q_p = 1.2N_{60} \leq 60 \text{ ksf}$$ (Ref. 1, Eq. 10.8.3.5.2c-1)

where:

Average N_{60} value for layer =	50	ksf	(From Step 1)
Existing Fill q_p =	60.0	ksf	

Compute nominal and factored side and tip resistances for all layers.

	q_s (ksf)	Shaft circumference (ft)	Length of shaft* (ft)	A_s (ft^2)	R_s (kips)	φ_{qs}**	$R_{R,s}$ (kips)
Existing Fill	0.92	9.4	5.5	51.8	47.7	0.55	26.2

* neglecting top 2 feet of shaft for Existing Fill length as per Ref. 14, page 5
** Ref. 1, Table 10.5.5.2.4-1

	q_p (ksf)	A_p (ft^2)	R_p (kips)	φ_{qp}*	$R_{R,p}$ (kips)
Existing Fill	60.0	7.1	424.1	0.50	212.1

* Ref. 1, Table 10.5.5.2.4-1

Factored geotechnical axial compression resistance of the drilled shaft:

$R_R = R_{R,s} + R_{R,p} =$ 238 kips

Check against Strength I factored axial design load:

Strength I factored axial design load:		Factored geotechnical axial compression resistance:	
3.5 kips	<	238 kips	OK

Demand capacity ratio = 0.01

5. Check lateral geotechnical resistance of the shaft at the Strength I and Extreme I limit states.

Perform a pushover analysis using LPile to compute shaft head deflection at various multiples of the factored shear and moment design loads, up to 1/φ times the factored shear and moment design loads. As per Ref. 4 Section 9.3.3.3.1, for a stable condition the analyses should each converge to a solution with a computed deflection no larger than 10% of the shaft diameter. For the pushover analysis the shaft should be modeled as a simple linear elastic beam rather than a nonlinear stress-strain model (Ref. 4, Section 9.3.3.3.1).

Elastic modulus for linear model = 4,000,000 psi (Ref. 4, page 9-21)
Moment of inertia for linear model, I = 82,448 in^4 (Ref. 4, page 9-21: $I = \pi D^4/64$)

Recommended resistance factor φ for lateral geotechnical resistance (Ref. 4, Table 9-1):

Limit State	φ	1/φ
Strength I	0.67	1.5
Extreme I	0.8	1.25

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Project Title:	MaineDOT Desert Road Bridge 5720 Freeport (Exit 20) Phase 2		

Computed shaft deflection at each load multiple analyzed with LPile:

Limit State	LPile Load Case	Multiple of Factored Design Loads	Loads Applied in LPile			Computed Lateral Head Deflection from LPile (in)	Deflection < 10% of Shaft Diameter?
			Axial (kips)	Moment (kip-ft)	Shear (kips)		
Strength I	1	0.25	3.5	7.3	0.0	0.02	Yes
	2	0.50	3.5	14.5	0.0	0.04	Yes
	3	0.75	3.5	21.8	0.0	0.06	Yes
	4	1.00	3.5	29.0	0.0	0.07	Yes
	5	1.25	3.5	36.3	0.0	0.09	Yes
	6	1.50	3.5	43.5	0.0	0.11	Yes
Extreme I	7	0.25	3.5	18.0	0.8	0.06	Yes
	8	0.50	3.5	36.0	1.5	0.12	Yes
	9	0.75	3.5	54.0	2.3	0.19	Yes
	10	1.00	3.5	72.0	3.0	0.34	Yes
	11	1.25	3.5	90.0	3.8	0.66	Yes
					maximum =	0.66	

The trial shaft length of 7.5 feet exhibits stable behavior up through 1/φ times the factored design loads, and the maximum computed deflection of 0.66 inches is less than 10% of the shaft diameter (10% of 36 inches = 3.6 inches). Thus, based on the pushover analysis, the trial shaft length of 7.5 feet satisfies the lateral geotechnical criterion at the Strength I and Extreme I limit states.

6. Check horizontal movement at the top of the shaft at the Service I limit state.

Use LPile to compute shaft head deflection at the Service I factored design loads. As per Ref. 4 page 9-27 and Ref. 14 page 3, deflection due to combined loading on the structure should be limited to 0.5 inches at the top of the shaft. For the serviceability analysis the shaft should be modeled as a nonlinear reinforced concrete shaft in flexure (Ref. 4, Section 9.3.3.3.3).

Computed shaft deflection analyzed with LPile:

Limit State	LPile Load Case	Multiple of Factored Design Loads	Loads Applied in LPile			Computed Lateral Head Deflection from LPile (in)	Deflection < 0.5 Inches?
			Axial (kips)	Moment (kip-ft)	Shear (kips)		
Service I	12	1.00	3	38	2	0.13	Yes

The computed deflection of 0.13 inches is less than the required limit of 0.5 inches. Thus the trial shaft length of 7.5 feet satisfies the lateral criterion at the Service I limit state. The LPile analysis output for factored design loads at the Service I limit state is included as Attachment 1.

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Project Title:	MaineDOT Desert Road Bridge 5720 Freeport (Exit 20) Phase 2		

7. Check embedment length to resist torsion loading demand at the Extreme I limit state.

Compute nominal and factored torsion resistance of the drilled shaft. Use the method in Ref. 15 Section 4.2.2 and Ref. 16 Section 13.6.1.1.

$$T_u \leq \phi_{tor} \cdot T_n$$ (Ref. 16, Section 13.6.1.1)

$$T_n = \pi D L F_s \left(\frac{D}{2}\right)$$ (Ref. 16, Section 13.6.1.1)

$$F_s = \sigma'_v \omega_{fdot}$$ (Ref. 16, Section 13.6.1.1)

where:

Factored torsional loading demand, T_u =	45	kip-ft	(From Step 3 - Extreme I)
Resistance factor for torsion, ϕ_{tor} =	1.0	for mast arm structures (Ref. 16, Section 13.6.1.1)	
Shaft diameter, D =	3.0	ft	(From Step 2)
Shaft length =	7.5	ft	(From Step 2)
Midpoint of shaft =	3.75	ft	
Vertical effective stress at midpoint, σ'_v =	0.469	ksf	
Uncorrected N-value for Existing Fill =	31		(Ref. 5)
Load transfer ratio, ω_{fdot} =	1.5		(Ref. 16, Section 13.6.1.1, for uncorrected N-values of 15 or greater)
Unit skin friction, F_s =	0.70	ksf	
Shaft length contributing to resistance, L =	5.5	ft	(neglecting top 2 feet as per Ref. 14, page 5)
Nominal torsion resistance, T_n =	55	kip-ft	
Factored torsion resistance, T_r =	55	kip-ft	

Check against Extreme I factored torsion design load:

Extreme I factored torsion design load:		Factored torsion resistance:	
45 kip-ft	<	55 kip-ft	OK

Demand capacity ratio = 0.82

8. Check estimated vertical movement at the tip of the mast arm during the pushover analysis.

Computed maximum shaft head deflection =	0.66	in	(From LPile, Extreme I factored load × 1.25)
Computed maximum shaft tip deflection =	0.27	in	(From LPile, Extreme I factored load × 1.25)
Total shaft deflection =	0.93	in	
Shaft length =	7.5	ft	= 90 in (From Step 2)
Total shaft rotation = deflection ÷ length =	0.010	rad	

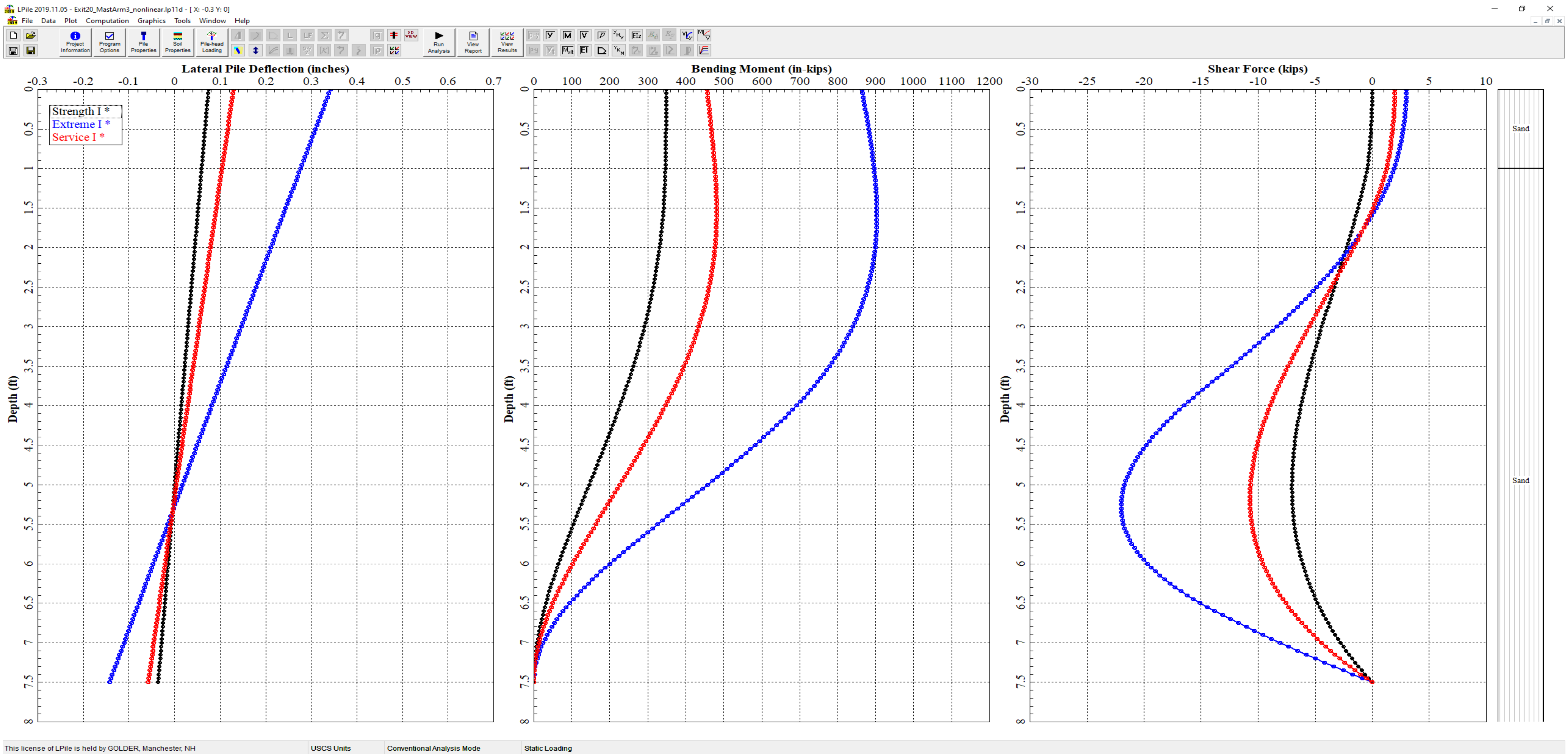
Mast arm length for Mast Arm 3 =	25	ft	=	300	in	(Ref. 8)
Vertical movement at tip of mast arm						
= shaft rotation × mast arm length =	3.08	in				

The estimated vertical movement of 3.08 inches at the tip of the mast arm during the pushover analysis is less than the required limit of 6 inches (Assumption 4). Thus the total shaft rotation will be considered sufficiently small.

CONCLUSIONS

The results of the analysis indicate that the proposed drilled shaft foundation with a shaft diameter of 36 inches and a shaft length of 7.5 feet will provide adequate support for Mast Arm 3 at Exit 20 based on the final design loads provided by HNTB. A maximum lateral deflection of 0.13 inches occurs at the top of the shaft under the Service I load case, satisfying the limiting requirement of 0.5 inches. The shaft length is controlled by the torsional resistance, and a torsion demand capacity ratio of 0.82 was calculated under the Extreme I load case. Although reinforcement consisting of twelve Grade 60 #6 bars arranged in a circular pattern was used in Golder's modeling, it is understood that HNTB will perform the final structural check and generate the required reinforcement pattern.

Attachment 1



GOLDER
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CALCULATIONS

Date:	8/5/2021	**Made by:**	KAR
Project No.:	21450908	**Checked by:**	MSG
Subject:	Drilled Shaft Design for Mast Arm 6 at Station 65+20.09	**Reviewed by:**	JEL
Project Title:	MaineDOT Desert Road Bridge 5720 Freeport (Exit 20) Phase 2		

OBJECTIVE
Determine if the proposed drilled shaft foundation will provide adequate support for Mast Arm 6 (Station 65+20.09, Offset 45.5' RT) at Exit 20 based on the final design loads provided by HNTB.

METHOD
As per the MaineDOT Bridge Design Guide Section 5.8 (Ref. 2), use the procedure outlined in AASHTO LRFD Article 10.8 (Ref. 1) and FHWA GEC-10 (Ref. 4).

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. AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals, 1st Ed. 2015 with 2017, 2018, and 2020 Interim Revisions.
4. FHWA. Drilled Shafts: Construction Procedures and Design Methods. Publication No. FHWA-NHI 18-024 and FHWA GEC 010. September 2018.
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7. Bridge Software Institute. FB-MultiPier Soil Parameter Table (US Customary Units). Accessed July 2021. https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf
8. Email communication between Golder and HNTB, subject "Freeport - Lighting and Signal Foundation Locations and Design Loads", dated June 29, 2021.
9. GeoTesting Express laboratory testing results, dated February 5, 2020 (Appendix C, Preliminary Geotechnical Design Report, dated December 21, 2020).
10. HNTB for State of Maine Department of Transportation. Merrill Road Bridge over Interstate 295 and Signalized Intersections, Exit 20 Interchange: 98% PS&E, dated July 30, 2021.
11. Holtz, R.D. and Kovacs, W.D. 1981. An Introduction to Geotechnical Engineering, 1st ed. Prentice Hall, Englewood Cliffs, NJ.
12. FHWA. Geotechnical Engineering Circular No. 6: Shallow Foundations. Report No. FHWA-SA-02-054. September 2002.
13. MaineDOT. Standard Details. March 2020.
14. VTrans. Materials & Research Engineering Instructions MREI 10-01. March 9, 2010.
15. Rodriguez, C.M., et al. Final Report: State of Practice and Literature Review on Foundations for Coastal Traffic Signal Mast Arm Structures. State of North Carolina Department of Transportation Research & Development, Report No. FHWA/NC/2018-17. May 14, 2020.
16. Florida Department of Transportation. FDOT Modifications to LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals (LRFDLTS-1). Structures Manual, Volume 3. January 2021.

ASSUMPTIONS
1. The drilled shafts are assumed to be installed either open hole or with temporary casing that is removed after construction. It is assumed that permanent casing will not be used, and the effect of permanent casing is not included in the analysis of nominal side resistance.
2. The topmost 1 foot of existing soil is removed and replaced by granular borrow at the drilled shaft location.

3. Typical steel reinforcement details are assumed for the purpose of the serviceability limit state lateral geotechnical analysis. Full steel reinforcement design and structural analysis will be performed by a non-Golder structural engineer.
4. Shaft head deflection is limited to 0.5 inches under Service loads and less than 10% of the shaft diameter under a "pushover" analysis (step 6). Furthermore, total shaft rotation at the end of the pushover analysis is limited so that the estimated vertical movement at the tip of the mast arm will not exceed 6 inches.
5. The glaciomarine silty clay is asssumed to be sufficiently overconsolidated such that it will experience only recompression settlement after loading (based on Golder's local engineering experience).
6. The glaciomarine silty clay is assumed to be undrained, and thus total stress analysis is used in calculating nominal shaft resistance via the alpha method as per Ref. 1 Article C10.8.3.5.1b and Ref. 4 page 10-18.

ATTACHMENTS
1. LPile analysis output for Strength I, Extreme I, and Service I (nonlinear model)

CALCULATION

Summary of calculation steps:
1. Define subsurface profile for analysis.
2. Select trial shaft diameter, shaft length, and shaft reinforcement for evaluation.
3. Calculate settlement and resulting downdrag loading at the drilled shaft location.
4. Define the factored design loads for the analysis.
5. Check geotechnical axial compression resistance of the shaft at the Strength I limit state.
6. Check lateral geotechnical resistance of the shaft at the Strength I and Extreme I limit states.
7. Check horizontal movement at the top of the shaft at the Service I limit state.
8. Check embedment length to resist torsion loading demand at the Extreme I limit state.
9. Check estimated vertical movement at the tip of the mast arm during the pushover analysis.

1. Define subsurface profile for analysis.

The soil stratigraphy encountered in nearby boring BB-FDR-211 is used for the analysis at Mast Arm 6.

Proposed top of drilled shaft elevation =	158.5	ft	(Ref. 10, Sheet 50, Station 65+25.00)
Existing ground surface elevation =	157.5	ft	(Ref. 10, Sheet 50, Station 65+25.00)
Elevation to which remove/replace soil =	156.5	ft	(Assumption 2)
Groundwater elevation =	140.0	ft	(Assumed based on water level measurements obtained in borings near the proposed Abutment 2)

Layer	Depth below top of shaft[1]	Lateral Model	Effective Unit Weight (pcf)[2]	Average N_{60} Value[1]	Design Friction Angle (°)[2]	Undrained Shear Strength (psf)[2]	Subgrade Modulus (pci)[3]	Major Principal Strain at 50%[3]	UCS (psi)[4]
Granular Borrow	0 - 1 ft	Sand (Reese)	125	-	32	-	88	-	-

Granular Borrow Replacement	1 - 2 ft	Sand (Reese)	125	-	32	-	88	-	-
Existing Fill	2 - 8.2 ft	Sand (Reese)	125	32	36	-	165	-	-
Glaciomarine Silty Clay	8.2 - 13.9 ft	Stiff Clay with Free Water (Reese)	119	27	-	2300	200	0.005	-
Sand / Gravel	13.9 - 15.2 ft	Sand (Reese)	125	120	36	-	165	-	-
Bedrock	> 15.2 ft	Strong Rock (Vuggy Limestone)	164	-	-	-	-	-	12983

1) Ref. 5
2) Proposed soils: Ref. 2, Table 3-3. Existing soils: correlation to average N_{60} value for each layer, Ref. 5.
3) Ref. 7. Interpolation based on friction angle for cohesionless layers and on undrained shear strength for cohesive layers.
4) Ref. 6 and 9

Clay consolidation parameters for the Glaciomarine Silty Clay layer are based on Golder's local engineering experience ($C_{c\varepsilon}$, $C_{r\varepsilon}$, c_v) and calculated (e_0) from soil moisture contents determined in laboratory testing (Reference 9).

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

2. Select trial shaft diameter, shaft length, and shaft reinforcement for evaluation.

The analysis will be evaluated with the minimum shaft diameter to accommodate poles and anchorages as provided by HNTB (Reference 8):

Shaft diameter for Traffic Signals =	36	inches	=	3.0	feet
which corresponds to shaft circumference =	113	inches	=	9.4	feet
and shaft base area =	1018	in^2	=	7.1	ft^2

The analysis will be evaluated with the minimum shaft length necessary to meet the torsional resistance requirement in Step 8:

Shaft length = 12 feet

Ref. 3 Article 13.6.2.1 requires a minimum concrete cover of 3 inches over steel reinforcement (to protect against corrosion). Ref. 4 Section 12.4 recommends a minimum concrete cover of 4 inches for shaft diameters greater than 3 feet and smaller than 5 feet.

Concrete cover to edge of bar =	3	in		(Ref. 3, Article 13.6.2.1)
Concrete compressive strength, f'_c =	5	ksi		(Ref. 1, Article C5.6.4.2)

Ref. 1 Article 5.6.4.2 and Ref. 4 Section 12.3.1 recommend a steel longitudinal reinforcement area of 1% to 1.5% of the gross area of the section and require a minimum of six bars in a circular arrangement plus a minimum #5 bar size. The MaineDOT Standard Details (Ref. 13, pg. 626(01) and 626(02)) specify six #6-size bars for an 18-inch diameter shaft and eight #6-size bars for a 24-inch diameter shaft, corresponding to 1.04% and 0.78% steel area, respectively.

This analysis will be evaluated with twelve Grade 60 #6-size bars, arranged in a symmetrical circular pattern with single-bar bundles, for the purpose of geotechnical design only.

Steel reinforcement area per bar =	0.44	in^2		(Ref. 4, Table 6-2, #14-size)
Number of bars =	12			
Steel reinforcement area per shaft =	5.28	in^2	=	0.52% of total shaft area
Steel yield strength, F_y =	60	ksi		(Ref. 4, Section 12.2.2)
Steel elastic modulus, E =	29,000	ksi		(Ref. 4, Section 12.2.2)

As per Assumption 3, steel transverse reinforcement for structural design purposes is not analyzed. Tie hoop reinforcement will be assumed for constructability purposes, as per Ref. 4 Section 6.1. Assume #3-size ties (based on Ref. 13 page 626(02)) spaced a maximum of 12 inches apart (based on Ref. 1 Article 5.7.2.6), resulting in 12 ties for a shaft length of 12 feet.

Steel reinforcement area per tie, A_v =	0.11	in^2	(Ref. 4, Table 6-2, #3-size)
Steel yield strength for ties =	60	ksi	(Ref. 4, Section 12.2.2)
Spacing of the ties, s =	12	in	(Ref. 1, Article 5.7.2.6)
Number of ties =	12		

3. Calculate settlement and resulting downdrag loading at the drilled shaft location.

Ref. 1 Article 3.11.8 indicates that downdrag on drilled shafts can be assumed to fully develop in soil layers where settlement is equal to or greater than 0.4 inches. Calculate the estimated settlement due to embankment construction at the drilled shaft location and due to axial load on the drilled shaft in order to determine the potential for downdrag loading.

3a. Settlement due to embankment construction, calculated for soil below the base of the Granular Borrow Replacement:

Begin by determining the change in effective stress state within the soil at the proposed drilled shaft location to identify if settlement or heave will occur. The change in effective stress state due to change in stratigraphy is determined at an elevation of 156.5 ft (elevation of the base of the remove/replace soil at the drilled shaft location). Calculate the vertical stress increase from the proposed embankment loading.

Existing Conditions (Ref. 10, Sheet 50, Station 65+25.00):

Layer	Unit weight (pcf)	Elevation (ft)		Thickness (ft)	Stress Contribution of Layer (psf)
		Top	Bottom		
Existing Fill	125	157.5	156.5	1.0	125

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After Construction (Ref. 10, Sheet 505, Station 65+25.00):

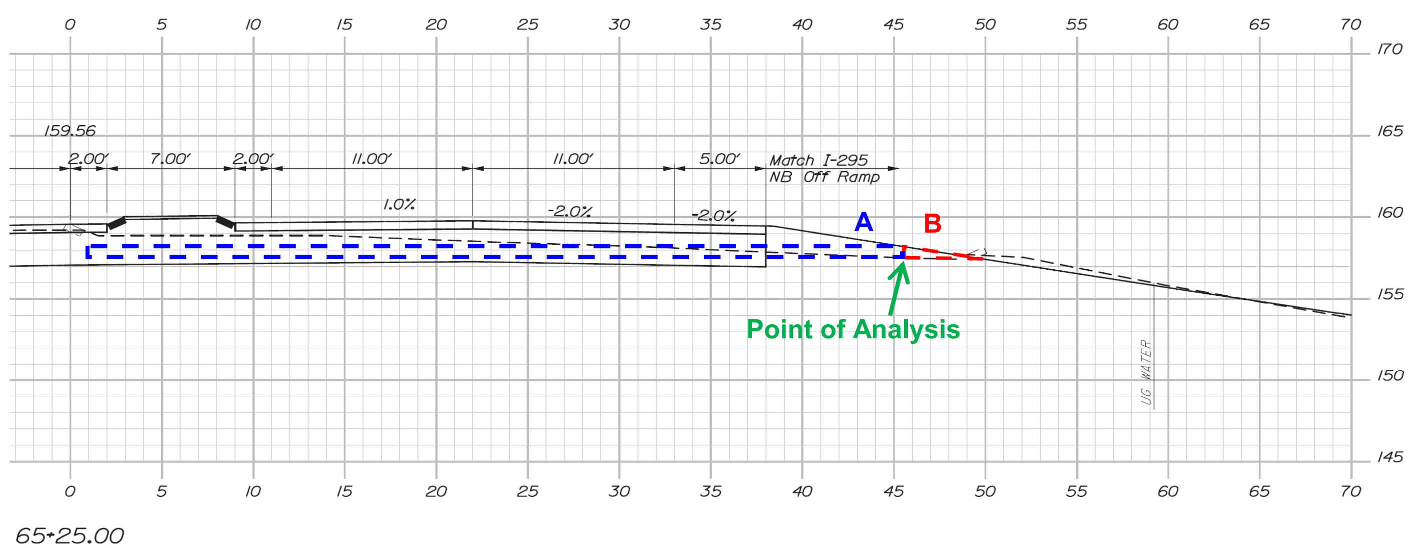
Layer	Unit weight (pcf)	Elevation (ft)		Thickness (ft)	Stress Contribution of Layer (psf)
		Top	Bottom		
Granular Borrow	125	158.5	157.5	1.0	125
Granular Borrow Replacement	125	157.5	156.5	1.0	125

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

	σ'_v at Elev. 155.5 ft (psf)	$\Delta\sigma'_v$ at Elev. 155.5 ft (psf)	Result
Existing Conditions	125	125	Settlement
After Construction	250		

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

Assumed Fill Loading (section view):



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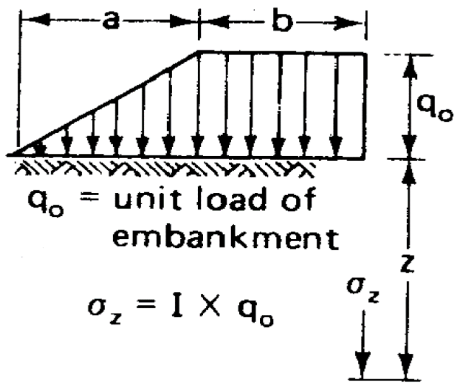

(Ref. 11, Figure 8.23)

$$\sigma_z = q_0 \times I$$ (Ref. 11, Eq. 8-30)

where:

σ_z = Vertical stress increase (psf)

I = influence value from Ref. 11, Figure 8.23

z = Depth to midpoint of layer (ft)

	Trapezoid A	Trapezoid B		
Stress applied by fill loading, q_0 =	125	125	psf	(From above)
Dimension of embankment slope, a =	0.0	4.5	ft	(Ref. 10, Sheet 50, Station 65+25.00)
Dimension of embankment top, b =	44.5	0.0	ft	(Ref. 10, Sheet 50, Station 65+25.00)

Trapezoid A:

Layer		Depth below footing (ft)	Layer Thickness (ft)	z (ft)	a/z	b/z	I	Stress Increase σ_z (psf)
Existing Fill	1	0-6.2	6.2	3.1	0.0	14.4	0.500	62.5
Glaciomarine	2	6.2-11.9	5.7	9.1	0.0	4.9	0.500	62.5
Sand/Gravel	3	11.9-13.2	1.3	12.6	0.0	3.5	0.500	62.5

Trapezoid B:

Layer		Depth below footing (ft)	Layer Thickness (ft)	z (ft)	a/z	b/z	I	Stress Increase σ_z (psf)
Existing Fill	1	0-6.2	6.2	3.1	1.5	0.0	0.322	40.3
Glaciomarine	2	6.2-11.9	5.7	9.1	0.5	0.0	0.145	18.1
Sand/Gravel	3	11.9-13.2	1.3	12.6	0.4	0.0	0.120	15.0

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Total Footing (Trapezoid A + Trapezoid B):

Layer		Depth below footing (ft)	Layer Thickness (ft)	z (ft)	Stress Increase (psf)
Existing Fill	1	0-6.2	6.2	3.1	102.8
Glaciomarine	2	6.2-11.9	5.7	9.1	80.6
Sand/Gravel	3	11.9-13.2	1.3	12.6	77.5

Use the Hough method to estimate settlement of the Existing Fill and Sand/Gravel layers (Layers 1 and 3); use consolidation theory to estimate settlement of the Glaciomarine Silty Clay layer (Layer 2).

Hough method:

Determine σ'_{v0} and C' at layer midpoints for in situ existing soil. Use Ref. 12 Figures 5-18 and 5-19 to obtain corrected N' and C' for Layers 1, 2, and 4, assuming the Existing Fill matches "Clean well graded fine to coarse SAND" and the Sand/Gravel matches "Well graded silty SAND & GRAVEL".

Stress due to existing soil above footing	σ'_{v0} (psf)
	125

Layer		Layer Thickness (ft)	Effective Unit Weight of Layer (pcf)	σ'_{v0} at layer midpoint (psf)	σ'_{v0} at layer midpoint (kPa)	N_{60}	N'	C'
Existing Fill	1	6.2	125	512	24.5	32	63	180
Glaciomarine	2	5.7	119	1239	59.3	Not required for clay consol.		
Sand/Gravel	3	1.3	125	1660	79.4	120	132	300

Hough method general equation:

$$\Delta H_i = H_c \frac{1}{C'} \log\left(\frac{\sigma'_{v0} + \Delta\sigma_v}{\sigma'_{v0}}\right)$$ (Ref. 12. Eqn 5-24)

where:

ΔH_i	settlement in each layer, ft
H_c	initial height of layer i, ft
C'	bearing capacity index from Ref. 12, Figure 5-19
$\Delta\sigma_v$	vertical stress increase, ksf

ΔH_i (Layer 1)	ft	0.003
	in	0.03

ΔH_i (Layer 3)	ft	0.000
	in	0.00

Consolidation theory general equation (Ref. 11, Eq. 8-11, 8-16, and 8-18b):

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

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

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

where:		Layer 2
H_0	initial height of layer i, ft	5.7
$\Delta\sigma_v$	surcharge load, psf	80.6
σ'_{v0}	in situ vertical effective stress, psf	1239
$\sigma'_{v0} + \Delta\sigma_v$	psf	1320
σ'_p	preconsolidated stress, psf	N/A
C_c	compression index, $C_c = C_{c\varepsilon}(1+e_0)$	0.42
C_r	recompression index, $C_r = C_{r\varepsilon}(1+e_0)$	0.03
e_0	initial void ratio	0.68
	Use equation (Assumption 5):	8-16
ΔH_i	ft	0.003
	in	0.04

Layer	Settlement due to embankment construction ΔH_i (in)
1	0.03
2	0.04
3	0.00
Total Settlement (in)	0.07

3b. Settlement due to axial load on the drilled shaft, calculated for soil below the base of the shaft:

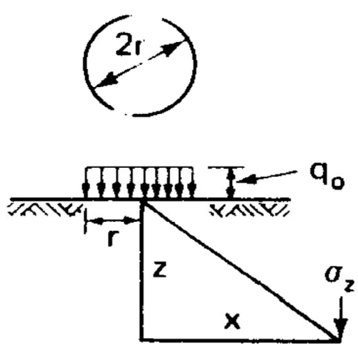
Base of shaft elevation =	146.5	ft	(From Steps 1 and 2)
Axial load on drilled shaft =	11	kips	(Strength I, Ref. 8; see Step 4)

Shaft base area =	7.1	ft^2	(From Step 2)
Stress increase due to axial load =	1556	psf	

	Concrete:	Steel:		
Shaft length =	12	12	ft	(From Step 2)
Portion of shaft base area =	7.03	0.04	ft^2	(From Step 2)
Portion of shaft volume =	84.4	0.4	ft^3	
Unit weight, γ =	140	490	pcf	(Assumed)
Portion of shaft weight =	11.8	0.2	kips	
Stress increase due to shaft weight =	1702		psf	

Total stress increase due to axial load and shaft weight =	3258	psf

Subdivide the subsurface soils into layers no larger than 10 feet thick and to a depth of either three times the footing width (shaft diameter) or to bedrock. Calculate the vertical stress increase at each layer, assuming the Boussinesq stress distribution method for stress under a uniformly loaded circular area (Ref. 11, Figure 8.22).



$$\sigma_z = \frac{I \times q_o}{100}$$ (Ref. 11, Figure 8.22)

where:

σ_z = Vertical stress increase (psf)
I = influence value from Ref. 11, Figure 8.22
z = Depth to midpoint of layer (ft)

q_0 = stress increase due to axial load =	3258	psf	(From above)
r = radius of uniformly loaded circular area =	1.5	ft	(Half of shaft diameter from Step 2)
x = offset distance to settlement location =	0	ft	(Calculate settlement below shaft center)

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Layer		Depth below footing (ft)	Layer Thickness (ft)	z (ft)	x/r	z/r	I	Stress Increase σ_z (psf)
Glaciomarine	1	0-1.9	1.9	1.0	0.0	0.6	70	2281
Sand/Gravel	2	1.9-3.2	1.3	2.6	0.0	1.7	30	977

Use the Hough method to estimate settlement of the Sand/Gravel layer (Layer 2); use consolidation theory to estimate settlement of the Glaciomarine Silty Clay layer (Layer 1).

Hough method:

Determine σ'_{v0} and C' at layer midpoints for in situ existing soil. Use Ref. 12 Figures 5-18 and 5-19 to obtain corrected N' and C' for Layer 1, assuming the Sand/Gravel matches "Well graded silty SAND & GRAVEL".

Stress due to existing soil above footing	σ'_{v0} (psf)
	1352

Layer		Layer Thickness (ft)	Effective Unit Weight of Layer (pcf)	σ'_{v0} at layer midpoint (psf)	σ'_{v0} at layer midpoint (kPa)	N_{60}	N'	C'
Glaciomarine	1	1.9	119	1465	70.1	Not required for clay consol.		
Sand/Gravel	2	1.3	125	1660	79.4	120	132	300

Hough method general equation:

$$\Delta H_i = H_c \frac{1}{C'} \log\left(\frac{\sigma'_{v0} + \Delta\sigma_v}{\sigma'_{v0}}\right) \quad \text{(Ref. 12. Eqn 5-24)}$$

where:

ΔH_i	settlement in each layer, ft
H_c	initial height of layer i, ft
C'	bearing capacity index from Ref. 12, Figure 5-19
$\Delta\sigma_v$	vertical stress increase, ksf

ΔH_i (Layer 2)		
	ft	0.001
	in	0.01

Consolidation theory general equation (Ref. 11, Eq. 8-11, 8-16, and 8-18b):

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

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

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

where:			Layer 1
H_0	initial height of layer i, ft		1.9
$\Delta\sigma_v$	surcharge load, psf		2281
σ'_{v0}	in situ vertical effective stress, psf		1465
$\sigma'_{v0} + \Delta\sigma_v$	psf		3746
σ'_p	preconsolidated stress, psf		N/A
C_c	compression index, $C_c = C_{c\varepsilon}(1+e_0)$		0.42
C_r	recompression index, $C_r = C_{r\varepsilon}(1+e_0)$		0.03
e_0	initial void ratio		0.68
	Use equation (Assumption 5):		8-16
	ΔH_i	ft	0.015
		in	0.19

Layer	Settlement due to axial load ΔH_i (in)
1	0.19
2	0.01
Total Settlement (in)	0.20

Since the estimated settlement in each soil layer due to embankment construction and axial load on the drilled shaft is less than 0.4 inches, as per Ref. 1 Article 3.11.8 downdrag loading will be assumed to be negligible for this analysis.

4. Define the factored design loads for the analysis.

The factored design loads at the top of the shaft were provided by HNTB (Ref. 8). Since Ref. 8 indicates that Mast Arm 2 uses the "50' Mast Arm" foundation design, the corresponding loads were selected.

Factored Design Forces (25' Mast Arm)					
Load	Units	Strength I	Extreme I	Service I	Service II
Axial	kip	11	10	9	9
Moment	kip-ft	148	176	129	119
Shear	kip	0	5	3	0
Torsion	kip-ft	0	142	62	0

As per Step 3, no downdrag loading will be added.

5. Check geotechnical axial compression resistance of the shaft at the Strength I limit state.

Compute nominal side resistance for all layers through which the shaft extends and nominal tip resistance for the layer at the tip elevation. Select appropriate resistance factors and calculate factored resistances.

Note that as per Ref. 14 page 5, a minimum of the upper 2 feet of soil should be neglected for contributions to skin friction resistance. As per Ref. 1 Article C10.8.3.5.1b, for cohesive soils at least the top 5 feet of any shaft should not be taken to contribute to the development of resistance through skin friction, to account for the effects of seasonal moisture changes, disturbance during construction, cyclic lateral loading, and low lateral stresses from freshly placed concrete.

The factored resistance of the drilled shaft, R_R, shall be taken as:

$$R_R = \varphi R_n = \varphi_{qp} R_p + \varphi_{qs} R_s$$ (Ref. 1, Eq. 10.8.3.5-1)

consisting of the nominal shaft tip resistance, R_p:

$$R_p = q_p A_p$$ (Ref. 1, Eq. 10.8.3.5-2)

and the nominal shaft side resistance, R_s:

$$R_s = q_s A_s$$ (Ref. 1, Eq. 10.8.3.5-3)

Calculate the unit side resistance q_s at the midpoint of the Existing Fill layer. Use the beta method for cohesionless soils (Ref. 1, Article 10.8.3.5.2b).

$$q_s = \beta \sigma'_v$$ (Ref. 1, Eq. 10.8.3.5.2b-1)

$$\beta = (1 - \sin \phi'_f)\left(\frac{\sigma'_p}{\sigma'_v}\right)^{\sin \phi'_f} \tan \phi'_f$$ (Ref. 1, Eq. 10.8.3.5.2b-2)

For sands: $$\frac{\sigma'_p}{p_a} = 0.47(N_{60})^m$$ (Ref. 1, Eq. 10.8.3.5.2b-4)

For gravelly soils: $\frac{\sigma'_p}{p_a} = 0.15(N_{60})$ (Ref. 1, Eq. 10.8.3.5.2b-5)

where:

Depth below top of shaft to midpoint of layer =	5.1	ft				
Vertical effective stress at midpoint, σ'_v =	0.638	ksf				
Friction angle for layer, ϕ'_f =	36	deg	=	0.63	rad	(From Step 1)
Average N_{60} value for layer =	32		(From Step 1)			
Atmospheric pressure, p_a =	2.12	ksf	(Ref. 1, Article 10.8.3.5.2b)			
Sand constant, m =	0.6	for clean quartzitic sands (Ref. 1, Article 10.8.3.5.2b)				
Preconsolidation stress, σ'_p =	7.97	ksf	(Ref. 1, Eq. 10.8.3.5.2b-4)			
β =	1.32					
Existing Fill q_s =	0.84	ksf				

Calculate the unit side resistance q_s at the midpoint of the Glaciomarine Silty Clay layer. Use the alpha method for cohesive soils (Ref. 1, Article 10.8.3.5.1b).

$$q_s = \alpha S_u$$ (Ref. 1, Eq. 10.8.3.5.1b-1)

$$\alpha = 0.55 \text{ for} \frac{S_u}{p_a} \leq 1.5$$ (Ref. 1, Eq. 10.8.3.5.1b-2)

$$\alpha = 0.55 - 0.1\left(\frac{S_u}{p_a} - 1.5\right) \text{ for } 1.5 \leq \frac{S_u}{p_a} \leq 2.5$$ (Ref. 1, Eq. 10.8.3.5.1b-3)

where:

Undrained shear strength for layer, S_u =	2.300	ksf	(From Step 1)
Atmospheric pressure, p_a =	2.12	ksf	(Ref. 1, Article 10.8.3.5.1b)
S_u / p_a =	1.08		
Adhesion factor, α =	0.55		(Ref. 1, Eq. 10.8.3.5.1b-2)
Glaciomarine q_s =	1.27	ksf	

Based on the shaft length selected in Step 2, the drilled shaft will terminate in the Glaciomarine Silty Clay layer. Calculate the unit tip resistance q_p for the Glaciomarine Silty Clay layer. Use the cohesive soil method (Ref. 1, Article 10.8.3.5.1c). Note that as per Ref. 1 Article 10.8.3.5.1c, if the soil within two diameters of the tip has S_u < 0.5 ksf, the value of N_c should be multiplied by 0.67.

$$q_p = N_c S_u \leq 80.0 \text{ ksf}$$ (Ref. 1, Eq. 10.8.3.5.1c-1)

$$N_c = 6\left[1 + 0.2\left(\frac{Z}{D}\right)\right] \leq 9$$ (Ref. 1, Eq. 10.8.3.5.1c-2)

where:

Undrained shear strength for layer, S_u = 2.300 ksf (From Step 1)
Diameter of drilled shaft, D = 3.0 ft (From Step 2)
Penetration of drilled shaft into layer, Z = 3.8 ft

N_c = 7.52
Glaciomarine q_p = 17.30 ksf

Compute nominal and factored side and tip resistances for all layers.

	q_s (ksf)	Shaft circumference (ft)	Length of shaft* (ft)	A_s (ft^2)	R_s (kips)	φ_{qs}**	$R_{R,s}$ (kips)
Existing Fill	0.84	9.4	6.2	58.4	49.2	0.55	27.1
Glaciomarine	1.27	9.4	3.8	35.8	45.3	0.45	20.4

* neglecting top 2 feet of shaft for Existing Fill length as per Ref. 14, page 5
** Ref. 1, Table 10.5.5.2.4-1

	q_p (ksf)	A_p (ft^2)	R_p (kips)	φ_{qp}*	$R_{R,p}$ (kips)
Glaciomarine	17.30	7.1	122.3	0.40	48.9

* Ref. 1, Table 10.5.5.2.4-1

Factored geotechnical axial compression resistance of the drilled shaft:

$R_R = R_{R,s} + R_{R,p}$ = 96 kips

Check against Strength I factored axial design load:

Strength I factored axial design load: 11 kips < Factored geotechnical axial compression resistance: 96 kips OK

Demand capacity ratio = 0.11

6. Check lateral geotechnical resistance of the shaft at the Strength I and Extreme I limit states.

Perform a pushover analysis using LPile to compute shaft head deflection at various multiples of the factored shear and moment design loads, up to 1/φ times the factored shear and moment design loads. As per Ref. 4 Section 9.3.3.3.1, for a stable condition the analyses should each converge to a solution with a computed deflection no larger than 10% of the shaft diameter. For the pushover analysis the shaft should be modeled as a simple linear elastic beam rather than a nonlinear stress-strain model (Ref. 4, Section 9.3.3.3.1).

Elastic modulus for linear model = 4,000,000 psi (Ref. 4, page 9-21)
Moment of inertia for linear model, I = 82,448 in^4 (Ref. 4, page 9-21: $I = \pi D^4/64$)

Recommended resistance factor φ for lateral geotechnical resistance (Ref. 4, Table 9-1):

Limit State	φ	1/φ
Strength I	0.67	1.5
Extreme I	0.8	1.25

Computed shaft deflection at each load multiple analyzed with LPile:

Limit State	LPile Load Case	Multiple of Factored Design Loads	Loads Applied in LPile			Computed Lateral Head Deflection from LPile (in)	Deflection < 10% of Shaft Diameter?
			Axial (kips)	Moment (kip-ft)	Shear (kips)		
Strength I	1	0.25	11	37	0	0.03	Yes
	2	0.50	11	74	0	0.05	Yes
	3	0.75	11	111	0	0.08	Yes
	4	1.00	11	148	0	0.10	Yes
	5	1.25	11	185	0	0.13	Yes
	6	1.50	11	222	0	0.16	Yes
Extreme I	7	0.25	10	44	1	0.04	Yes
	8	0.50	10	88	3	0.08	Yes
	9	0.75	10	132	4	0.12	Yes
	10	1.00	10	176	5	0.16	Yes
	11	1.25	10	220	6	0.21	Yes
					maximum =	0.21	

The trial shaft length of 12 feet exhibits stable behavior up through 1/φ times the factored design loads, and the maximum computed deflection of 0.21 inches is less than 10% of the shaft diameter (10% of 36 inches = 3.6 inches). Thus, based on the pushover analysis, the trial shaft length of 12 feet satisfies the lateral geotechnical criterion at the Strength I and Extreme I limit states.

7. Check horizontal movement at the top of the shaft at the Service I limit state.

Use LPile to compute shaft head deflection at the Service I factored design loads. As per Ref. 4 page 9-27 and Ref. 14 page 3, deflection due to combined loading on the structure should be limited to 0.5 inches at the top of the shaft. For the serviceability analysis the shaft should be modeled as a nonlinear reinforced concrete shaft in flexure (Ref. 4, Section 9.3.3.3.3).

Computed shaft deflection analyzed with LPile:

Limit State	LPile Load Case	Multiple of Factored Design Loads	Loads Applied in LPile			Computed Lateral Head Deflection from LPile (in)	Deflection < 0.5 Inches?
			Axial (kips)	Moment (kip-ft)	Shear (kips)		
Service I	12	1.00	9	129	3	0.11	Yes

The computed deflection of 0.11 inches is less than the required limit of 0.5 inches. Thus the trial shaft length of 12 feet satisfies the lateral criterion at the Service I limit state. The LPile analysis output for factored design loads at the Service I limit state is included as Attachment 1.

8. Check embedment length to resist torsion loading demand at the Extreme I limit state.

Compute nominal and factored torsion resistance of the drilled shaft. Due to the limitations of the method in Ref. 15 Section 4.2.2 and Ref. 16 Section 13.6.1.1, assume the drilled shaft is installed entirely within cohesionless soil with an N-value equal to that of the existing fill.

$$T_u \leq \phi_{tor} \cdot T_n$$ (Ref. 16, Section 13.6.1.1)

$$T_n = \pi D L F_s \left(\frac{D}{2}\right)$$ (Ref. 16, Section 13.6.1.1)

$$F_s = \sigma'_v \omega_{fdot}$$ (Ref. 16, Section 13.6.1.1)

where:

Factored torsional loading demand, T_u =	142	kip-ft	(From Step 4 - Extreme I)
Resistance factor for torsion, ϕ_{tor} =	1.0	for mast arm structures (Ref. 16, Section 13.6.1.1)	
Shaft diameter, D =	3.0	ft	(From Step 2)
Shaft length =	12	ft	(From Step 2)
Midpoint of shaft =	6	ft	
Vertical effective stress at midpoint, σ'_v =	0.750	ksf	
Uncorrected N-value for Existing Fill =	20		(Ref. 5)
Load transfer ratio, ω_{fdot} =	1.5		(Ref. 16, Section 13.6.1.1, for uncorrected N-values of 15 or greater)
Unit skin friction, F_s =	1.13	ksf	
Shaft length contributing to resistance, L =	10	ft	(neglecting top 2 feet as per Ref. 14, page 5)
Nominal torsion resistance, T_n =	159	kip-ft	
Factored torsion resistance, T_r =	159	kip-ft	

Check against Extreme I factored torsion design load:

Extreme I factored torsion design load:		Factored torsion resistance:	
142 kip-ft	<	159 kip-ft	OK

Demand capacity ratio = 0.89

9. Check estimated vertical movement at the tip of the mast arm during the pushover analysis.

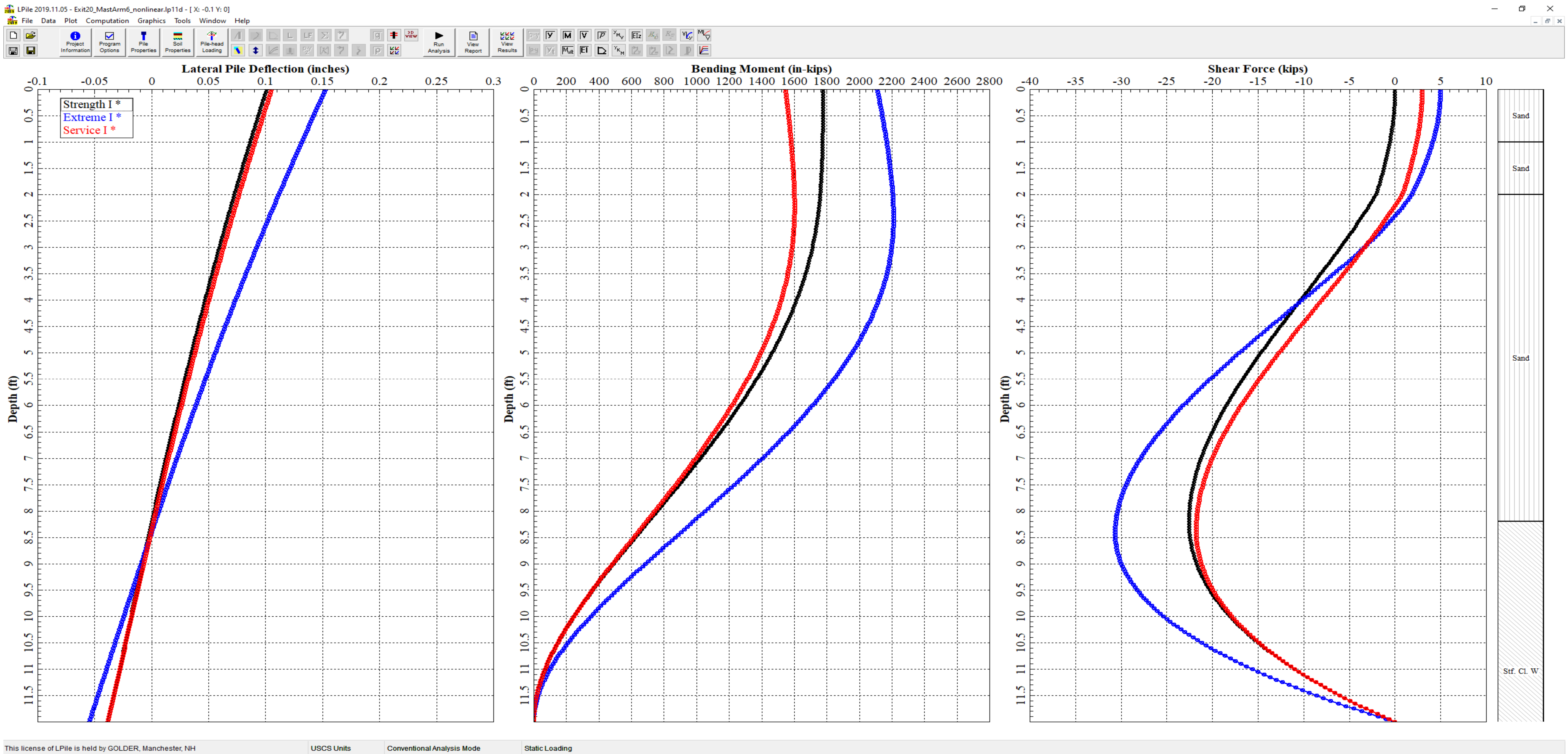
Computed maximum shaft head deflection =	0.21	in	(From LPile, Extreme I factored load × 1.25)			
Computed maximum shaft tip deflection =	0.07	in	(From LPile, Extreme I factored load × 1.25)			
Total shaft deflection =	0.28	in				
Shaft length =	12	ft	=	144	in	(From Step 2)
Total shaft rotation = deflection ÷ length =	0.002	rad				
Mast arm length for Mast Arm 6 =	40	ft	=	480	in	(Ref. 8)
Vertical movement at tip of mast arm = shaft rotation × mast arm length =	0.92	in				

The estimated vertical movement of 0.92 inches at the tip of the mast arm during the pushover analysis is less than the required limit of 6 inches (Assumption 4). Thus the total shaft rotation will be considered sufficiently small.

CONCLUSIONS

The results of the analysis indicate that the proposed drilled shaft foundation with a shaft diameter of 36 inches and a shaft length of 12 feet will provide adequate support for Mast Arm 6 at Exit 20 based on the final design loads provided by HNTB. A maximum lateral deflection of 0.11 inches occurs at the top of the shaft under the Service I load case, satisfying the limiting requirement of 0.5 inches. The shaft length is controlled by the torsional resistance, and a torsion demand capacity ratio of 0.89 was calculated under the Extreme I load case. Although reinforcement consisting of twelve Grade 60 #6 bars arranged in a circular pattern was used in Golder's modeling, it is understood that HNTB will perform the final structural check and generate the required reinforcement pattern.

Attachment 1



CALCULATIONS

Date:	8/10/2021	**Made by:**	KAR
Project No.:	21450908	**Checked by:**	MSG
Subject:	Drilled Shaft Design for Standard Lighting 9 at Station 201+00.00	**Reviewed by:**	JEL
Project Title:	MaineDOT Desert Road Bridge 5720 Freeport (Exit 20) Phase 2		

OBJECTIVE

Determine if the proposed drilled shaft foundation will provide adequate support for Standard Lighting 9 (Station 201+00.00, Offset 14.0' RT) at Exit 20 based on the final design loads provided by HNTB.

METHOD

As per the MaineDOT Bridge Design Guide Section 5.8 (Ref. 2), use the procedure outlined in AASHTO LRFD Article 10.8 (Ref. 1) and FHWA GEC-10 (Ref. 4).

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 9th Ed. 2020.
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3. AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals, 1st Ed. 2015 with 2017, 2018, and 2020 Interim Revisions.
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5. Golder geotechnical test boring logs for 200-series borings.
6. Golder geotechnical test boring logs for 100-series borings (Appendix A, Preliminary Geotechnical Design Report, dated December 21, 2020).
7. Bridge Software Institute. FB-MultiPier Soil Parameter Table (US Customary Units). Accessed July 2021. https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf
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10. HNTB for State of Maine Department of Transportation. Merrill Road Bridge over Interstate 295 and Signalized Intersections, Exit 20 Interchange: 98% PS&E, dated July 30, 2021.
11. Holtz, R.D. and Kovacs, W.D. 1981. An Introduction to Geotechnical Engineering, 1st ed. Prentice Hall, Englewood Cliffs, NJ.
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13. MaineDOT. Standard Details. March 2020.
14. VTrans. Materials & Research Engineering Instructions MREI 10-01. March 9, 2010.
15. Rodriguez, C.M., et al. Final Report: State of Practice and Literature Review on Foundations for Coastal Traffic Signal Mast Arm Structures. State of North Carolina Department of Transportation Research & Development, Report No. FHWA/NC/2018-17. May 14, 2020.
16. Florida Department of Transportation. FDOT Modifications to LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals (LRFDLTS-1). Structures Manual, Volume 3. January 2021.

ASSUMPTIONS

1. The drilled shafts are assumed to be installed either open hole or with temporary casing that is removed after construction. It is assumed that permanent casing will not be used, and the effect of permanent casing is not included in the analysis of nominal side resistance.
2. The topmost 1 foot of existing soil is removed and replaced by granular borrow at the drilled shaft location.

3. Typical steel reinforcement details are assumed for the purpose of the serviceability limit state lateral geotechnical analysis. Full steel reinforcement design and structural analysis will be performed by a non-Golder structural engineer.
4. Shaft head deflection is limited to 0.5 inches under Service loads and less than 10% of the shaft diameter under a "pushover" analysis (step 5). Furthermore, total shaft rotation at the end of the pushover analysis is limited so that the estimated vertical movement at the tip of the mast arm will not exceed 6 inches.
5. The immediate settlement of the cohesionless soils is assumed to be sufficiently small that downdrag loading will be negligible.

ATTACHMENTS

1. LPile analysis output for Strength I, Extreme I, and Service I (nonlinear model)

CALCULATION

Summary of calculation steps:
1. Define subsurface profile for analysis.
2. Select trial shaft diameter, shaft length, and shaft reinforcement for evaluation.
3. Define the factored design loads for the analysis.
4. Check geotechnical axial compression resistance of the shaft at the Strength I limit state.
5. Check lateral geotechnical resistance of the shaft at the Strength I and Extreme I limit states.
6. Check horizontal movement at the top of the shaft at the Service I limit state.
7. Check embedment length to resist torsion loading demand at the Extreme I limit state.

1. Define subsurface profile for analysis.

The soil stratigraphy encountered in nearby boring BB-FDR-212 is used for the analysis at Standard Lighting 9.

Proposed top of drilled shaft elevation =	160.3	ft	(Ref. 10, Sheet 65, Station 201+00.00)
Existing ground surface elevation =	160.3	ft	(Ref. 10, Sheet 65, Station 201+00.00)
Elevation to which remove/replace soil =	159.3	ft	(Assumption 2)
Groundwater elevation =	140.0	ft	(Assumed based on water level measurements obtained in borings near the proposed Abutment 2)

Layer	Depth below top of shaft[1]	Lateral Model	Effective Unit Weight (pcf)[2]	Average N_{60} Value[1]	Design Friction Angle (°)[2]	Undrained Shear Strength (psf)[2]	Subgrade Modulus (pci)[3]	Major Principal Strain at 50%[3]	UCS (psi)[4]
Granular Borrow Replacment	0 - 1 ft	Sand (Reese)	125	-	32	-	88	-	-
Existing Fill	1 - 6.3 ft	Sand (Reese)	125	17	32	-	88	-	-

Bedrock	> 6.3 ft	Strong Rock (Vuggy Limestone)	164	-	-	-	-	-	12983

1) Correlated from design friction angle using Peck, Hanson, and Thornburn (1974): $\phi = 27.1 + 0.3N_{60} - 0.00054[N_{60}]^2$
2) Proposed soils: Ref. 2, Table 3-3. Existing soils: correlation to average N_{60} value for each layer, Ref. 5.
3) Ref. 7. Interpolation based on friction angle for cohesionless layers and on undrained shear strength for cohesive layers.
4) Ref. 6 and 9

2. Select trial shaft diameter, shaft length, and shaft reinforcement for evaluation.

The analysis will be evaluated with the minimum shaft diameter to accommodate poles and anchorages as provided by HNTB (Reference 8):

Shaft diameter for Standard Lighting =	30	inches	=	2.5	feet
which corresponds to shaft circumference =	94	inches	=	7.9	feet
and shaft base area =	707	in^2	=	4.9	ft^2

The analysis will be evaluated with the minimum shaft length specified in the MaineDOT Standard Details (Ref. 13, page 626(01)):

Shaft length = 5.5 feet

Ref. 3 Article 13.6.2.1 requires a minimum concrete cover of 3 inches over steel reinforcement (to protect against corrosion). Ref. 4 Section 12.4 recommends a minimum concrete cover of 4 inches for shaft diameters greater than 3 feet and smaller than 5 feet.

Concrete cover to edge of bar =	3	in	(Ref. 3, Article 13.6.2.1)
Concrete compressive strength, f'_c =	5	ksi	(Ref. 1, Article C5.6.4.2)

Ref. 1 Article 5.6.4.2 and Ref. 4 Section 12.3.1 recommend a steel longitudinal reinforcement area of 1% to 1.5% of the gross area of the section and require a minimum of six bars in a circular arrangement plus a minimum #5 bar size. The MaineDOT Standard Details (Ref. 13, pg. 626(01) and 626(02)) specify six #6-size bars for an 18-inch diameter shaft and eight #6-size bars for a 24-inch diameter shaft, corresponding to 1.04% and 0.78% steel area, respectively.

This analysis will be evaluated with ten Grade 60 #6-size bars, arranged in a symmetrical circular pattern with single-bar bundles, for the purpose of geotechnical design only.

Steel reinforcement area per bar =	0.44	in^2		(Ref. 4, Table 6-2, #14-size)
Number of bars =	10			
Steel reinforcement area per shaft =	4.4	in^2	=	0.62% of total shaft area
Steel yield strength, F_y =	60	ksi		(Ref. 4, Section 12.2.2)
Steel elastic modulus, E =	29,000	ksi		(Ref. 4, Section 12.2.2)

As per Assumption 3, steel transverse reinforcement for structural design purposes is not analyzed. Tie hoop reinforcement will be assumed for constructability purposes, as per Ref. 4 Section 6.1. Assume #3-size ties (based on Ref. 13 page 626(02)) spaced a maximum of 12 inches apart (based on Ref. 1 Article 5.7.2.6), resulting in 5 ties for a shaft length of 5.5 feet.

Steel reinforcement area per tie, A_v =	0.11	in^2	(Ref. 4, Table 6-2, #3-size)
Steel yield strength for ties =	60	ksi	(Ref. 4, Section 12.2.2)
Spacing of the ties, s =	12	in	(Ref. 1, Article 5.7.2.6)
Number of ties =	5		

3. Define the factored design loads for the analysis.

The factored design loads at the top of the shaft were provided by HNTB (Ref. 8). The loads corresponding to "Standard Lighting" were selected.

Factored Design Forces (Standard Lighting)					
Load	Units	Strength I	Extreme I	Service I	Service II
Axial	kip	1.5	1.5	1.5	1.5
Moment	kip-ft	0.5	17.5	9.5	0.5
Shear	kip	0	1	0.5	0
Torsion	kip-ft	0	0.5	0.25	0

As per Assumption 5, no downdrag loading will be added.

4. Check geotechnical axial compression resistance of the shaft at the Strength I limit state.

Compute nominal side resistance for all layers through which the shaft extends and nominal tip resistance for the layer at the tip elevation. Select appropriate resistance factors and calculate factored resistances.

Note that as per Ref. 14 page 5, a minimum of the upper 2 feet of soil should be neglected for contributions to skin friction resistance. As per Ref. 1 Article C10.8.3.5.1b, for cohesive soils at least the top 5 feet of any shaft should not be taken to contribute to the development of resistance through skin friction, to account for the effects of seasonal moisture changes, disturbance during construction, cyclic lateral loading, and low lateral stresses from freshly placed concrete.

The factored resistance of the drilled shaft, R_R, shall be taken as:

$$R_R = \varphi R_n = \varphi_{qp} R_p + \varphi_{qs} R_s \qquad \text{(Ref. 1, Eq. 10.8.3.5-1)}$$

consisting of the nominal shaft tip resistance, R_p:

$$R_p = q_p A_p \qquad \text{(Ref. 1, Eq. 10.8.3.5-2)}$$

and the nominal shaft side resistance, R_s:

$$R_s = q_s A_s$$ (Ref. 1, Eq. 10.8.3.5-3)

Calculate the unit side resistance q_s at the midpoint of the Existing Fill layer. Use the beta method for cohesionless soils (Ref. 1, Article 10.8.3.5.2b).

$$q_s = \beta\sigma'_v$$ (Ref. 1, Eq. 10.8.3.5.2b-1)

$$\beta = (1 - \sin\phi'_f)\left(\frac{\sigma'_p}{\sigma'_v}\right)^{\sin\phi'_f} \tan\phi'_f$$ (Ref. 1, Eq. 10.8.3.5.2b-2)

For sands: $$\frac{\sigma'_p}{p_a} = 0.47(N_{60})^m$$ (Ref. 1, Eq. 10.8.3.5.2b-4)

For gravelly soils: $$\frac{\sigma'_p}{p_a} = 0.15(N_{60})$$ (Ref. 1, Eq. 10.8.3.5.2b-5)

where:

Depth below top of shaft to midpoint of layer =	3.3	ft				
Vertical effective stress at midpoint, σ'_v =	0.413	ksf				
Friction angle for layer, ϕ'_f =	32	deg	=	0.56	rad	(From Step 1)
Average N_{60} value for layer =	17		(From Step 1)			
Atmospheric pressure, p_a =	2.12	ksf	(Ref. 1, Article 10.8.3.5.2b)			
Sand constant, m =	0.6	for clean quartzitic sands (Ref. 1, Article 10.8.3.5.2b)				
Preconsolidation stress, σ'_p =	5.45	ksf	(Ref. 1, Eq. 10.8.3.5.2b-4)			
β =	1.15					
Existing Fill q_s =	0.48	ksf				

Based on the shaft length selected in Step 2, the drilled shaft will terminate in the Existing Fill layer. Calculate the unit tip resistance q_p for the Existing Fill layer. Use the cohesionless soil method (Ref. 1, Article 10.8.3.5.2c). Note that as per Ref. 1 Article 10.8.3.5.2c, the value of q_p should be limited to 60 ksf unless greater values can be justified through load test data.

$$\text{If } N_{60} \leq 50, \text{then } q_p = 1.2N_{60} \leq 60 \text{ ksf}$$ (Ref. 1, Eq. 10.8.3.5.2c-1)

where:

Average N_{60} value for layer =	17	ksf	(From Step 1)
Existing Fill q_p =	20.4	ksf	

Compute nominal and factored side and tip resistances for all layers.

Date:	8/10/2021	**Made by:**	KAR
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Project Title:	MaineDOT Desert Road Bridge 5720 Freeport (Exit 20) Phase 2		

	q_s (ksf)	Shaft circumference (ft)	Length of shaft* (ft)	A_s (ft^2)	R_s (kips)	φ_{qs}**	$R_{R,s}$ (kips)
Existing Fill	0.48	7.9	3.5	27.5	13.1	0.55	7.2

* neglecting top 2 feet of shaft for Existing Fill length as per Ref. 14, page 5
** Ref. 1, Table 10.5.5.2.4-1

	q_p (ksf)	A_p (ft^2)	R_p (kips)	φ_{qp}*	$R_{R,p}$ (kips)
Existing Fill	20.4	4.9	100.1	0.50	50.1

* Ref. 1, Table 10.5.5.2.4-1

Factored geotechnical axial compression resistance of the drilled shaft:

$R_R = R_{R,s} + R_{R,p} =$ 57 kips

Check against Strength I factored axial design load:

Strength I factored axial design load:		Factored geotechnical axial compression resistance:	
1.5 kips	<	57 kips	OK

Demand capacity ratio = 0.03

5. Check lateral geotechnical resistance of the shaft at the Strength I and Extreme I limit states.

Perform a pushover analysis using LPile to compute shaft head deflection at various multiples of the factored shear and moment design loads, up to 1/φ times the factored shear and moment design loads. As per Ref. 4 Section 9.3.3.3.1, for a stable condition the analyses should each converge to a solution with a computed deflection no larger than 10% of the shaft diameter. For the pushover analysis the shaft should be modeled as a simple linear elastic beam rather than a nonlinear stress-strain model (Ref. 4, Section 9.3.3.3.1).

Elastic modulus for linear model = 4,000,000 psi (Ref. 4, page 9-21)
Moment of inertia for linear model, I = 39,761 in^4 (Ref. 4, page 9-21: $I = \pi D^4/64$)

Recommended resistance factor φ for lateral geotechnical resistance (Ref. 4, Table 9-1):

Limit State	φ	1/φ
Strength I	0.67	1.5
Extreme I	0.8	1.25

Computed shaft deflection at each load multiple analyzed with LPile:

Limit State	LPile Load Case	Multiple of Factored Design Loads	Loads Applied in LPile			Computed Lateral Head Deflection from LPile (in)	Deflection < 10% of Shaft Diameter?
			Axial (kips)	Moment (kip-ft)	Shear (kips)		
Strength I	1	0.25	1.5	0.1	0.0	0.00	Yes
	2	0.50	1.5	0.3	0.0	0.00	Yes
	3	0.75	1.5	0.4	0.0	0.00	Yes
	4	1.00	1.5	0.5	0.0	0.01	Yes
	5	1.25	1.5	0.6	0.0	0.01	Yes
	6	1.50	1.5	0.8	0.0	0.01	Yes
Extreme I	7	0.25	1.5	4.4	0.3	0.06	Yes
	8	0.50	1.5	8.8	0.5	0.12	Yes
	9	0.75	1.5	13.1	0.8	0.20	Yes
	10	1.00	1.5	17.5	1.0	0.29	Yes
	11	1.25	1.5	21.9	1.3	0.47	Yes
					maximum =	0.47	

The trial shaft length of 5.5 feet exhibits stable behavior up through 1/φ times the factored design loads, and the maximum computed deflection of 0.47 inches is less than 10% of the shaft diameter (10% of 30 inches = 3 inches). Thus, based on the pushover analysis, the trial shaft length of 5.5 feet satisfies the lateral geotechnical criterion at the Strength I and Extreme I limit states.

6. Check horizontal movement at the top of the shaft at the Service I limit state.

Use LPile to compute shaft head deflection at the Service I factored design loads. As per Ref. 4 page 9-27 and Ref. 14 page 3, deflection due to combined loading on the structure should be limited to 0.5 inches at the top of the shaft. For the serviceability analysis the shaft should be modeled as a nonlinear reinforced concrete shaft in flexure (Ref. 4, Section 9.3.3.3.3).

Computed shaft deflection analyzed with LPile:

Limit State	LPile Load Case	Multiple of Factored Design Loads	Loads Applied in LPile			Computed Lateral Head Deflection from LPile (in)	Deflection < 0.5 Inches?
			Axial (kips)	Moment (kip-ft)	Shear (kips)		
Service I	12	1.00	1.5	9.5	0.5	0.13	Yes

The computed deflection of 0.13 inches is less than the required limit of 0.5 inches. Thus the trial shaft length of 5.5 feet satisfies the lateral criterion at the Service I limit state. The LPile analysis output for factored design loads at the Service I limit state is included as Attachment 1.

Date:	8/10/2021	**Made by:**	KAR
Project No.:	21450908	**Checked by:**	MSG
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Project Title:	MaineDOT Desert Road Bridge 5720 Freeport (Exit 20) Phase 2		

7. Check embedment length to resist torsion loading demand at the Extreme I limit state.

Compute nominal and factored torsion resistance of the drilled shaft. Use the method in Ref. 15 Section 4.2.2 and Ref. 16 Section 13.6.1.1.

$$T_u \leq \phi_{tor} \cdot T_n$$ (Ref. 16, Section 13.6.1.1)

$$T_n = \pi D L F_s \left(\frac{D}{2}\right)$$ (Ref. 16, Section 13.6.1.1)

$$F_s = \sigma'_v \omega_{fdot}$$ (Ref. 16, Section 13.6.1.1)

where:

Factored torsional loading demand, T_u =	0.5	kip-ft	(From Step 3 - Extreme I)
Resistance factor for torsion, ϕ_{tor} =	0.9		(Ref. 16, Section 13.6.1.1)

Note: using the minimum resistance factor specified in Ref. 16 Section 13.6.1.1 (the factor for overhead cantilever sign structures), since a resistance factor for lightpost structures is not available.

Shaft diameter, D =	2.5	ft	(From Step 2)
Shaft length =	5.5	ft	(From Step 2)
Midpoint of shaft =	2.75	ft	
Vertical effective stress at midpoint, σ'_v =	0.344	ksf	
Uncorrected N-value for Existing Fill =	10		(Correlated from design friction angle)
Load transfer ratio, ω_{fdot} = 1.5*(N-value/15) =	1.0		(Ref. 16, Section 13.6.1.1, for uncorrected N-values less than 15)
Unit skin friction, F_s =	0.34	ksf	
Shaft length contributing to resistance, L =	3.5	ft	(neglecting top 2 feet as per Ref. 14, page 5)
Nominal torsion resistance, T_n =	12	kip-ft	
Factored torsion resistance, T_r =	11	kip-ft	

Check against Extreme I factored torsion design load:

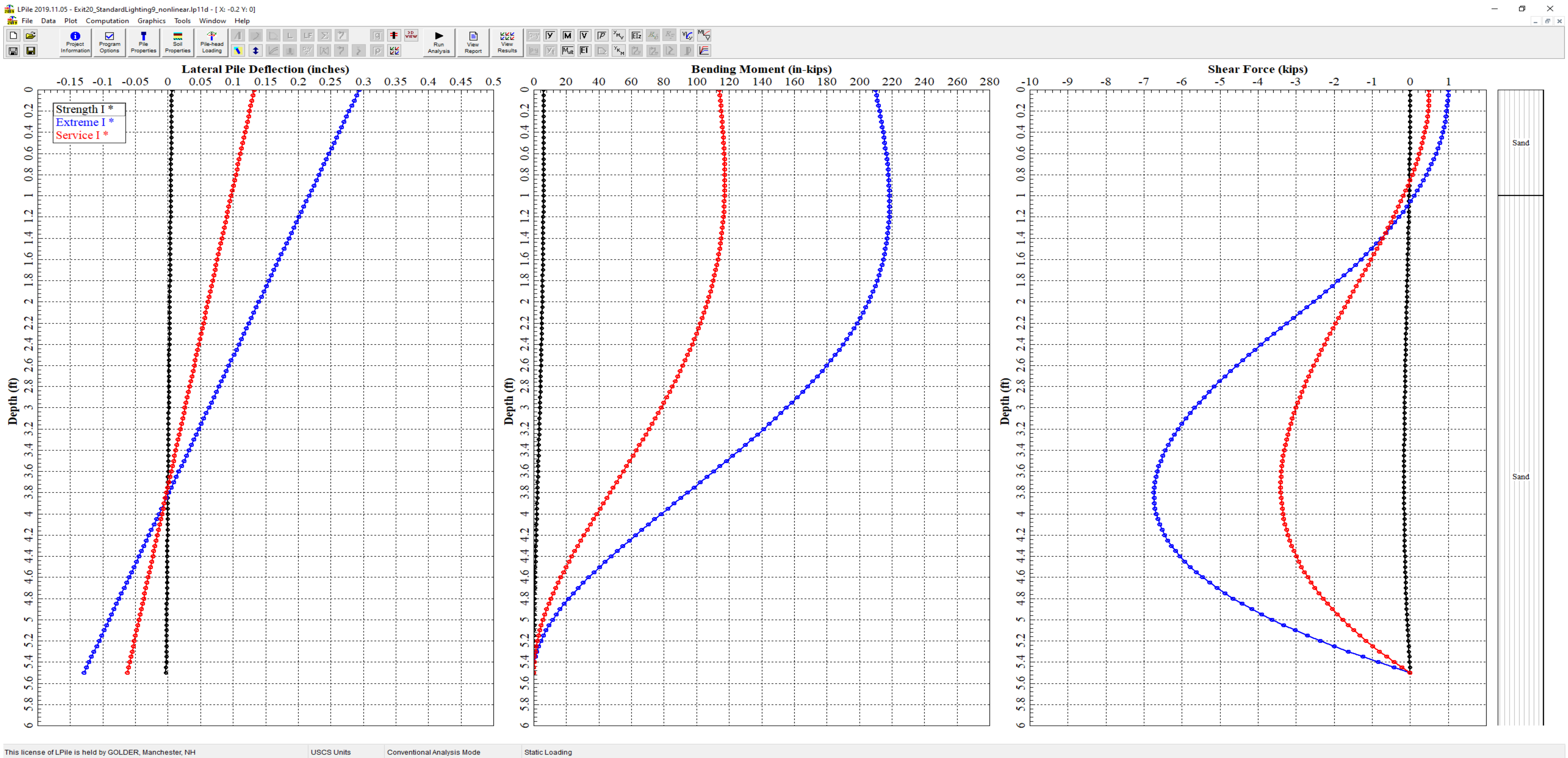
Extreme I factored torsion design load:		Factored torsion resistance:	
0.5 kip-ft	<	11 kip-ft	OK

Demand capacity ratio = 0.05

CONCLUSIONS

The results of the analysis indicate that the proposed drilled shaft foundation with a shaft diameter of 30 inches and a shaft length of 5.5 feet will provide adequate support for Standard Lighting 9 at Exit 20 based on the final design loads provided by HNTB. A maximum lateral deflection of 0.13 inches occurs at the top of the shaft under the Service I load case, satisfying the limiting requirement of 0.5 inches. Although reinforcement consisting of ten Grade 60 #6 bars arranged in a circular pattern was used in Golder's modeling, it is understood that HNTB will perform the final structural check and generate the required reinforcement pattern.

Attachment 1



GOLDER
MEMBER OF WSP

CALCULATIONS

Date:	8/3/2021	**Made by:**	KAR
Project No.:	21450908	**Checked by:**	MSG
Subject:	Drilled Shaft Design for Standard Lighting 14 at Station 104+40.00	**Reviewed by:**	JEL
Project Title:	MaineDOT Desert Road Bridge 5720 Freeport (Exit 20) Phase 2		

OBJECTIVE
Determine if the proposed drilled shaft foundation will provide adequate support for Standard Lighting 14 (Station 104+40.00, Offset 16.3' RT) at Exit 20 based on the final design loads provided by HNTB.

METHOD
As per the MaineDOT Bridge Design Guide Section 5.8 (Ref. 2), use the procedure outlined in AASHTO LRFD Article 10.8 (Ref. 1) and FHWA GEC-10 (Ref. 4).

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. AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals, 1st Ed. 2015 with 2017, 2018, and 2020 Interim Revisions.
4. FHWA. Drilled Shafts: Construction Procedures and Design Methods. Publication No. FHWA-NHI 18-024 and FHWA GEC 010. September 2018.
5. Golder geotechnical test boring logs for 200-series borings.
6. Golder geotechnical test boring logs for 100-series borings (Appendix A, Preliminary Geotechnical Design Report, dated December 21, 2020).
7. Bridge Software Institute. FB-MultiPier Soil Parameter Table (US Customary Units). Accessed July 2021. https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf
8. Email communication between Golder and HNTB, subject "Freeport - Lighting and Signal Foundation Locations and Design Loads", dated June 29, 2021.
9. GeoTesting Express laboratory testing results, dated February 5, 2020 (Appendix C, Preliminary Geotechnical Design Report, dated December 21, 2020).
10. HNTB for State of Maine Department of Transportation. Merrill Road Bridge over Interstate 295 and Signalized Intersections, Exit 20 Interchange: 98% PS&E, dated July 30, 2021.
11. Holtz, R.D. and Kovacs, W.D. 1981. An Introduction to Geotechnical Engineering, 1st ed. Prentice Hall, Englewood Cliffs, NJ.
12. FHWA. Geotechnical Engineering Circular No. 6: Shallow Foundations. Report No. FHWA-SA-02-054. September 2002.
13. MaineDOT. Standard Details. March 2020.
14. VTrans. Materials & Research Engineering Instructions MREI 10-01. March 9, 2010.
15. Rodriguez, C.M., et al. Final Report: State of Practice and Literature Review on Foundations for Coastal Traffic Signal Mast Arm Structures. State of North Carolina Department of Transportation Research & Development, Report No. FHWA/NC/2018-17. May 14, 2020.
16. Florida Department of Transportation. FDOT Modifications to LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals (LRFDLTS-1). Structures Manual, Volume 3. January 2021.

ASSUMPTIONS
1. The drilled shafts are assumed to be installed either open hole or with temporary casing that is removed after construction. It is assumed that permanent casing will not be used, and the effect of permanent casing is not included in the analysis of nominal side resistance.
2. The topmost 1 foot of existing soil is removed and replaced by granular borrow at the drilled shaft location.

3. Typical steel reinforcement details are assumed for the purpose of the serviceability limit state lateral geotechnical analysis. Full steel reinforcement design and structural analysis will be performed by a non-Golder structural engineer.
4. Shaft head deflection is limited to 0.5 inches under Service loads and less than 10% of the shaft diameter under a "pushover" analysis (step 6). Furthermore, total shaft rotation at the end of the pushover analysis is limited so that the estimated vertical movement at the tip of the mast arm will not exceed 6 inches.
5. The glaciomarine silty clay is asssumed to be sufficiently overconsolidated such that it will experience only recompression settlement after loading (based on Golder's local engineering experience).
6. The glaciomarine silty clay is assumed to be undrained, and thus total stress analysis is used in calculating nominal shaft resistance via the alpha method as per Ref. 1 Article C10.8.3.5.1b and Ref. 4 page 10-18.

ATTACHMENTS
1. LPile analysis output for Strength I, Extreme I, and Service I (nonlinear model)

CALCULATION

Summary of calculation steps:
1. Define subsurface profile for analysis.
2. Select trial shaft diameter, shaft length, and shaft reinforcement for evaluation.
3. Calculate settlement and resulting downdrag loading at the drilled shaft location.
4. Define the factored design loads for the analysis.
5. Check geotechnical axial compression resistance of the shaft at the Strength I limit state.
6. Check lateral geotechnical resistance of the shaft at the Strength I and Extreme I limit states.
7. Check horizontal movement at the top of the shaft at the Service I limit state.
8. Check embedment length to resist torsion loading demand at the Extreme I limit state.

1. Define subsurface profile for analysis.

The soil stratigraphy encountered in nearby boring BB-FDR-211 is used for the analysis at Standard Lighting 14.

Proposed top of drilled shaft elevation =	150.5	ft	(Ref. 10, Sheet 60, Station 104+50.00)
Existing ground surface elevation =	150.0	ft	(Ref. 10, Sheet 60, Station 104+50.00)
Elevation to which remove/replace soil =	149.0	ft	(Assumption 2)
Groundwater elevation =	140.0	ft	(Assumed based on water level measurements obtained in borings near the proposed Abutment 2)

Layer	Depth below top of shaft[1]	Lateral Model	Effective Unit Weight (pcf)[2]	Average N_{60} Value[1]	Design Friction Angle (°)[2]	Undrained Shear Strength (psf)[2]	Subgrade Modulus (pci)[3]	Major Principal Strain at 50%[3]	UCS (psi)[4]
Granular Borrow	0 - 0.5 ft	Sand (Reese)	125	-	32	-	88	-	-

Granular Borrow Replacement	0.5 - 1.5 ft	Sand (Reese)	125	-	32	-	88	-	-
Glaciomarine Silty Clay	1.5 - 5.9 ft	Stiff Clay with Free Water (Reese)	119	27	-	2300	200	0.005	-
Sand / Gravel	5.9 - 7.2 ft	Sand (Reese)	125	120	36	-	165	-	-
Bedrock	> 7.2 ft	Strong Rock (Vuggy Limestone)	164	-	-	-	-	-	12983

1) Ref. 5
2) Proposed soils: Ref. 2, Table 3-3. Existing soils: correlation to average N_{60} value for each layer, Ref. 5.
3) Ref. 7. Interpolation based on friction angle for cohesionless layers and on undrained shear strength for cohesive layers.
4) Ref. 6 and 9

Clay consolidation parameters for the Glaciomarine Silty Clay layer are based on Golder's local engineering experience ($C_{c\varepsilon}$, $C_{r\varepsilon}$, c_v) and calculated (e_0) from soil moisture contents determined in laboratory testing (Reference 9).

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

2. Select trial shaft diameter, shaft length, and shaft reinforcement for evaluation.

The analysis will be evaluated with the minimum shaft diameter to accommodate poles and anchorages as provided by HNTB (Reference 8):

Shaft diameter for Standard Lighting =	30	inches	=	2.5	feet
which corresponds to shaft circumference =	94	inches	=	7.9	feet
and shaft base area =	707	in^2	=	4.9	ft^2

The analysis will be evaluated with the minimum shaft length specified in the MaineDOT Standard Details (Ref. 13, page 626(01)):

Shaft length = 5.5 feet

Ref. 3 Article 13.6.2.1 requires a minimum concrete cover of 3 inches over steel reinforcement (to protect against corrosion). Ref. 4 Section 12.4 recommends a minimum concrete cover of 4 inches for shaft diameters greater than 3 feet and smaller than 5 feet.

Concrete cover to edge of bar =	3	in	(Ref. 3, Article 13.6.2.1)
Concrete compressive strength, f'_c =	5	ksi	(Ref. 1, Article C5.6.4.2)

Ref. 1 Article 5.6.4.2 and Ref. 4 Section 12.3.1 recommend a steel longitudinal reinforcement area of 1% to 1.5% of the gross area of the section and require a minimum of six bars in a circular arrangement plus a minimum #5 bar size. The MaineDOT Standard Details (Ref. 13, pg. 626(01) and 626(02)) specify six #6-size bars for an 18-inch diameter shaft and eight #6-size bars for a 24-inch diameter shaft, corresponding to 1.04% and 0.78% steel area, respectively.

This analysis will be evaluated with ten Grade 60 #6-size bars, arranged in a symmetrical circular pattern with single-bar bundles, for the purpose of geotechnical design only.

Steel reinforcement area per bar =	0.44	in^2	(Ref. 4, Table 6-2, #14-size)
Number of bars =	10		
Steel reinforcement area per shaft =	4.4	in^2	= 0.62% of total shaft area
Steel yield strength, F_y =	60	ksi	(Ref. 4, Section 12.2.2)
Steel elastic modulus, E =	29,000	ksi	(Ref. 4, Section 12.2.2)

As per Assumption 3, steel transverse reinforcement for structural design purposes is not analyzed. Tie hoop reinforcement will be assumed for constructability purposes, as per Ref. 4 Section 6.1. Assume #3-size ties (based on Ref. 13 page 626(02)) spaced a maximum of 12 inches apart (based on Ref. 1 Article 5.7.2.6), resulting in 5 ties for a shaft length of 5.5 feet.

Steel reinforcement area per tie, A_v =	0.11	in^2	(Ref. 4, Table 6-2, #3-size)
Steel yield strength for ties =	60	ksi	(Ref. 4, Section 12.2.2)
Spacing of the ties, s =	12	in	(Ref. 1, Article 5.7.2.6)
Number of ties =	5		

3. Calculate settlement and resulting downdrag loading at the drilled shaft location.

Ref. 1 Article 3.11.8 indicates that downdrag on drilled shafts can be assumed to fully develop in soil layers where settlement is equal to or greater than 0.4 inches. Calculate the estimated settlement due to embankment construction at the drilled shaft location and due to axial load on the drilled shaft in order to determine the potential for downdrag loading.

3a. Settlement due to embankment construction, calculated for soil below the base of the Granular Borrow Replacement:

Begin by determining the change in effective stress state within the soil at the proposed drilled shaft location to identify if settlement or heave will occur. The change in effective stress state due to change in stratigraphy is determined at an elevation of 149.0 ft (elevation of the base of the remove/replace soil at the drilled shaft location). Calculate the vertical stress increase from the proposed embankment loading.

Existing Conditions (Ref. 10, Sheet 60, Station 104+50.00):

Layer	Unit weight (pcf)	Elevation (ft)		Thickness (ft)	Stress Contribution of Layer (psf)
		Top	Bottom		
Glaciomarine Silty Clay	119	150.0	149.0	1.0	119

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After Construction (Ref. 10, Sheet 60, Station 104+50.00):

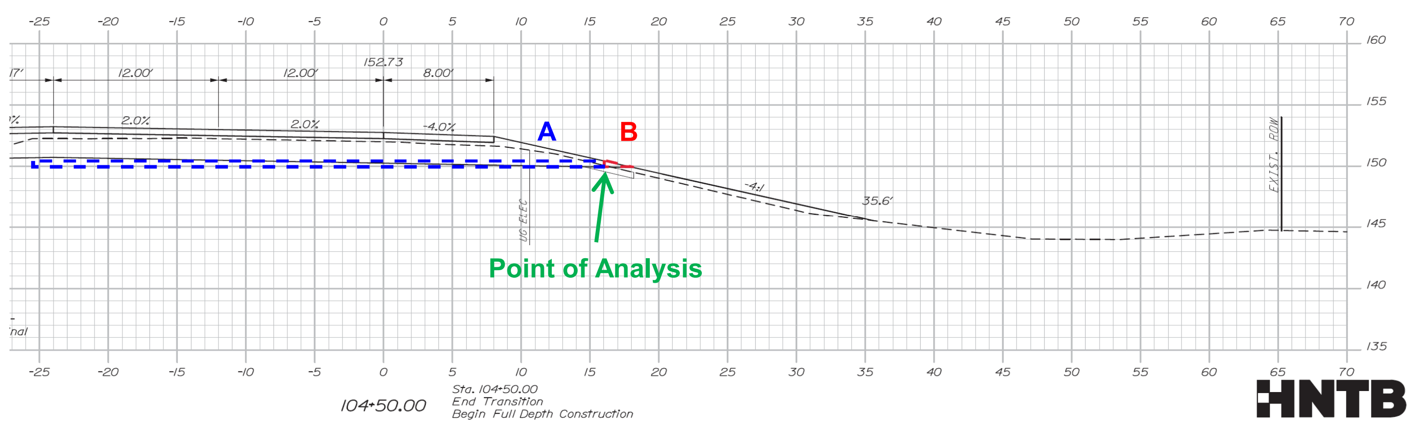
Layer	Unit weight (pcf)	Elevation (ft)		Thickness (ft)	Stress Contribution of Layer (psf)
		Top	Bottom		
Granular Borrow	125	150.5	150.0	0.5	63
Granular Borrow Replacement	125	150.0	149.0	1.0	125

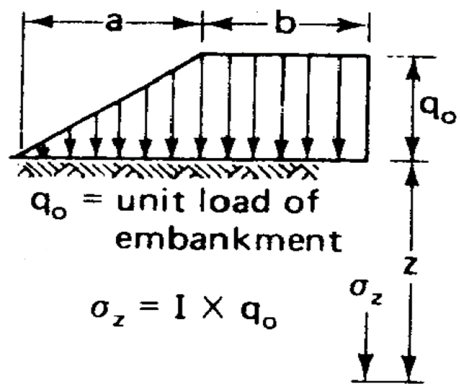
Calculate the increase or decrease in effective stress as a result of construction.

	σ'_v at Elev. 155.5 ft (psf)	$\Delta\sigma'_v$ at Elev. 155.5 ft (psf)	Result
Existing Conditions	119	69	Settlement
After Construction	188		

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

Assumed Fill Loading (section view):





(Ref. 11, Figure 8.23)

$$\sigma_z = q_0 \times I$$

(Ref. 11, Eq. 8-30)

where:

σ_z = Vertical stress increase (psf)

I = influence value from Ref. 11, Figure 8.23

z = Depth to midpoint of layer (ft)

	Trapezoid A	Trapezoid B		
Stress applied by fill loading, q_0 =	69	69	psf	(From above)
Dimension of embankment slope, a =	0.0	2.0	ft	(Ref. 10, Sheet 60, Station 104+50.00)
Dimension of embankment top, b =	41.5	0.0	ft	(Ref. 10, Sheet 60, Station 104+50.00)

Trapezoid A:

Layer		Depth below footing (ft)	Layer Thickness (ft)	z (ft)	a/z	b/z	I	Stress Increase σ_z (psf)
Glaciomarine	1	0-4.4	4.4	2.2	0.0	18.9	0.500	34.5
Sand/Gravel	2	4.4-5.7	1.3	5.1	0.0	8.2	0.500	34.5

Trapezoid B:

Layer		Depth below footing (ft)	Layer Thickness (ft)	z (ft)	a/z	b/z	I	Stress Increase σ_z (psf)
Glaciomarine	1	0-4.4	4.4	2.2	0.9	0.0	0.240	16.6
Sand/Gravel	2	4.4-5.7	1.3	5.1	0.4	0.0	0.120	8.3

Total Footing (Trapezoid A + Trapezoid B):

Layer		Depth below footing (ft)	Layer Thickness (ft)	z (ft)	Stress Increase (psf)
Glaciomarine	1	0-4.4	4.4	2.2	51.1
Sand/Gravel	2	4.4-5.7	1.3	5.1	42.8

Use the Hough method to estimate settlement of the Sand/Gravel layer (Layer 2); use consolidation theory to estimate settlement of the Glaciomarine Silty Clay layer (Layer 1).

Hough method:
Determine σ'_{v0} and C' at layer midpoints for in situ existing soil. Use Ref. 12 Figures 5-18 and 5-19 to obtain corrected N' and C' for Layer 2, assuming the Sand/Gravel matches "Well graded silty SAND & GRAVEL".

Stress due to existing soil above footing	σ'_{v0} (psf)
	119

Layer		Layer Thickness (ft)	Effective Unit Weight of Layer (pcf)	σ'_{v0} at layer midpoint (psf)	σ'_{v0} at layer midpoint (kPa)	N_{60}	N'	C'
Glaciomarine	1	4.4	119	381	18.2	Not required for clay consol.		
Sand/Gravel	2	1.3	125	724	34.7	120	199	300

Hough method general equation:

$$\Delta H_i = H_c \frac{1}{C'} \log\left(\frac{\sigma'_{v0} + \Delta\sigma_v}{\sigma'_{v0}}\right)$$ (Ref. 12. Eqn 5-24)

where:
- ΔH_i settlement in each layer, ft
- H_c initial height of layer i, ft
- C' bearing capacity index from Ref. 12, Figure 5-19
- $\Delta\sigma_v$ vertical stress increase, ksf

ΔH_i (Layer 2)		
	ft	0.000
	in	0.00

Consolidation theory general equation (Ref. 11, Eq. 8-11, 8-16, and 8-18b):

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

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

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

where:			Layer 1
H_0	initial height of layer i, ft		4.4
$\Delta\sigma_v$	surcharge load, psf		51.1
σ'_{v0}	in situ vertical effective stress, psf		381
$\sigma'_{v0} + \Delta\sigma_v$	psf		432
σ'_p	preconsolidated stress, psf		N/A
C_c	compression index, $C_c = C_{c\varepsilon}(1+e_0)$		0.42
C_r	recompression index, $C_r = C_{r\varepsilon}(1+e_0)$		0.03
e_0	initial void ratio		0.68
	Use equation (Assumption 5):		8-16
	ΔH_i	ft	0.005
		in	0.06

	Settlement due to embankment construction
Layer	ΔH_i (in)
1	0.06
2	0.00
Total Settlement (in)	0.06

3b. Settlement due to axial load on the drilled shaft, calculated for soil below the base of the shaft:

Base of shaft elevation =	145.0	ft	(From Steps 1 and 2)
Axial load on drilled shaft =	1.5	kips	(Strength I, Ref. 8; see Step 4)
Shaft base area =	4.9	ft^2	(From Step 2)
Stress increase due to axial load =	306	psf	

	Concrete:	Steel:		
Shaft length =	5.5	5.5	ft	(From Step 2)

CALCULATIONS

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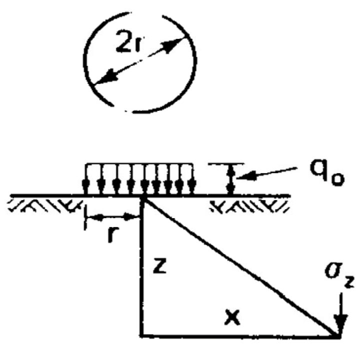
Portion of shaft base area =	4.88	0.03	ft^2	(From Step 2)
Portion of shaft volume =	26.8	0.2	ft^3	
Unit weight, γ =	140	490	pcf	(Assumed)
Portion of shaft weight =	3.8	0.1	kips	
Stress increase due to shaft weight =	782		psf	
Total stress increase due to axial load and shaft weight =	1088	psf		

Subdivide the subsurface soils into layers no larger than 10 feet thick and to a depth of either three times the footing width (shaft diameter) or to bedrock. Calculate the vertical stress increase at each layer, assuming the Boussinesq stress distribution method for stress under a uniformly loaded circular area (Ref. 11, Figure 8.22).



$$\sigma_z = \frac{I \times q_o}{100}$$ (Ref. 11, Figure 8.22)

where:

σ_z = Vertical stress increase (psf)

I = influence value from Ref. 11, Figure 8.22

z = Depth to midpoint of layer (ft)

q_0 = stress increase due to axial load =	1088	psf	(From above)
r = radius of uniformly loaded circular area =	1.25	ft	(Half of shaft diameter from Step 2)
x = offset distance to settlement location =	0	ft	(Calculate settlement below shaft center)

Layer		Depth below footing (ft)	Layer Thickness (ft)	z (ft)	x/r	z/r	I	Stress Increase σ_z (psf)
Glaciomarine	1	0-0.4	0.4	0.2	0.0	0.2	95	1033
Sand/Gravel	2	0.4-1.7	1.3	1.1	0.0	0.8	70	761

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Use the Hough method to estimate settlement of the Sand/Gravel layer (Layer 2); use consolidation theory to estimate settlement of the Glaciomarine Silty Clay layer (Layer 1).

Hough method:
Determine σ'_{v0} and C' at layer midpoints for in situ existing soil. Use Ref. 12 Figures 5-18 and 5-19 to obtain corrected N' and C' for Layer 1, assuming the Sand/Gravel matches "Well graded silty SAND & GRAVEL".

Stress due to existing soil above footing	σ'_{v0} (psf)
	595

Layer		Layer Thickness (ft)	Effective Unit Weight of Layer (pcf)	σ'_{v0} at layer midpoint (psf)	σ'_{v0} at layer midpoint (kPa)	N_{60}	N'	C'
Glaciomarine	1	0.4	119	619	29.6	Not required for clay consol.		
Sand/Gravel	2	1.3	125	724	34.7	120	199	300

Hough method general equation:

$$\Delta H_i = H_c \frac{1}{C'} \log\left(\frac{\sigma'_{v0} + \Delta\sigma_v}{\sigma'_{v0}}\right)$$ (Ref. 12. Eqn 5-24)

where:

ΔH_i	settlement in each layer, ft
H_c	initial height of layer i, ft
C'	bearing capacity index from Ref. 12, Figure 5-19
$\Delta\sigma_v$	vertical stress increase, ksf

ΔH_i (Layer 2)	ft	0.001
	in	0.02

Consolidation theory general equation (Ref. 11, Eq. 8-11, 8-16, and 8-18b):

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

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

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

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

where:		Layer 1
H_0	initial height of layer i, ft	0.4
$\Delta\sigma_v$	surcharge load, psf	1033
σ'_{v0}	in situ vertical effective stress, psf	619
$\sigma'_{v0} + \Delta\sigma_v$	psf	1652
σ'_p	preconsolidated stress, psf	N/A
C_c	compression index, $C_c = C_{c\varepsilon}(1+e_0)$	0.42
C_r	recompression index, $C_r = C_{r\varepsilon}(1+e_0)$	0.03
e_0	initial void ratio	0.68
	Use equation (Assumption 5):	8-16
ΔH_i	ft	0.003
	in	0.04

	Settlement due to axial load
Layer	ΔH_i (in)
1	0.04
2	0.02
Total Settlement (in)	0.06

Since the estimated settlement in each soil layer due to embankment construction and axial load on the drilled shaft is less than 0.4 inches, as per Ref. 1 Article 3.11.8 downdrag loading will be assumed to be negligible for this analysis.

4. Define the factored design loads for the analysis.

The factored design loads at the top of the shaft were provided by HNTB (Ref. 8). The loads corresponding to "Standard Lighting" were selected.

Factored Design Forces (Standard Lighting)					
Load	Units	Strength I	Extreme I	Service I	Service II
Axial	kip	1.5	1.5	1.5	1.5
Moment	kip-ft	0.5	17.5	9.5	0.5
Shear	kip	0	1	0.5	0
Torsion	kip-ft	0	0.5	0.25	0

As per Step 3, no downdrag loading will be added.

5. Check geotechnical axial compression resistance of the shaft at the Strength I limit state.

Compute nominal side resistance for all layers through which the shaft extends and nominal tip resistance for the layer at the tip elevation. Select appropriate resistance factors and calculate factored resistances.

Note that as per Ref. 14 page 5, a minimum of the upper 2 feet of soil should be neglected for contributions to skin friction resistance. As per Ref. 1 Article C10.8.3.5.1b, for cohesive soils at least the top 5 feet of any shaft should not be taken to contribute to the development of resistance through skin friction, to account for the effects of seasonal moisture changes, disturbance during construction, cyclic lateral loading, and low lateral stresses from freshly placed concrete.

The factored resistance of the drilled shaft, R_R, shall be taken as:

$$R_R = \varphi R_n = \varphi_{qp} R_p + \varphi_{qs} R_s$$ (Ref. 1, Eq. 10.8.3.5-1)

consisting of the nominal shaft tip resistance, R_p:

$$R_p = q_p A_p$$ (Ref. 1, Eq. 10.8.3.5-2)

and the nominal shaft side resistance, R_s:

$$R_s = q_s A_s$$ (Ref. 1, Eq. 10.8.3.5-3)

Calculate the unit side resistance q_s at the midpoint of the Glaciomarine Silty Clay layer. Use the alpha method for cohesive soils (Ref. 1, Article 10.8.3.5.1b).

$$q_s = \alpha S_u$$ (Ref. 1, Eq. 10.8.3.5.1b-1)

$$\alpha = 0.55 \text{ for } \frac{S_u}{p_a} \leq 1.5$$ (Ref. 1, Eq. 10.8.3.5.1b-2)

$$\alpha = 0.55 - 0.1\left(\frac{S_u}{p_a} - 1.5\right) \text{ for } 1.5 \leq \frac{S_u}{p_a} \leq 2.5$$ (Ref. 1, Eq. 10.8.3.5.1b-3)

where:

Undrained shear strength for layer, S_u =	2.300	ksf	(From Step 1)
Atmospheric pressure, p_a =	2.12	ksf	(Ref. 1, Article 10.8.3.5.1b)
S_u / p_a =	1.08		
Adhesion factor, α =	0.55		(Ref. 1, Eq. 10.8.3.5.1b-2)
Glaciomarine q_s =	1.27	ksf	

CALCULATIONS

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Based on the shaft length selected in Step 2, the drilled shaft will terminate in the Glaciomarine Silty Clay layer. Calculate the unit tip resistance q_p for the Glaciomarine Silty Clay layer. Use the cohesive soil method (Ref. 1, Article 10.8.3.5.1c). Note that as per Ref. 1 Article 10.8.3.5.1c, if the soil within two diameters of the tip has S_u < 0.5 ksf, the value of N_c should be multiplied by 0.67.

$$q_p = N_c S_u \leq 80.0 \text{ ksf}$$ (Ref. 1, Eq. 10.8.3.5.1c-1)

$$N_c = 6\left[1 + 0.2\left(\frac{Z}{D}\right)\right] \leq 9$$ (Ref. 1, Eq. 10.8.3.5.1c-2)

where:

Undrained shear strength for layer, S_u = 2.300 ksf (From Step 1)
Diameter of drilled shaft, D = 2.5 ft (From Step 2)
Penetration of drilled shaft into layer, Z = 4.0 ft

N_c = 7.92
Glaciomarine q_p = 18.22 ksf

Compute nominal and factored side and tip resistances for all layers.

	q_s (ksf)	Shaft circumference (ft)	Length of shaft* (ft)	A_s (ft^2)	R_s (kips)	φ_{qs}**	$R_{R,s}$ (kips)
Glaciomarine	1.27	7.9	0.5	3.9	5.0	0.45	2.2

* neglecting top 5 feet of shaft for Glaciomarine length as per Ref. 1, Article C10.8.3.5.1b
** Ref. 1, Table 10.5.5.2.4-1

	q_p (ksf)	A_p (ft^2)	R_p (kips)	φ_{qp}*	$R_{R,p}$ (kips)
Glaciomarine	18.22	4.9	89.4	0.40	35.8

* Ref. 1, Table 10.5.5.2.4-1

Factored geotechnical axial compression resistance of the drilled shaft:

$R_R = R_{R,s} + R_{R,p}$ = 38 kips

Check against Strength I factored axial design load:

Strength I factored axial design load:		Factored geotechnical axial compression resistance:	
1.5 kips	<	38 kips	OK

Demand capacity ratio = 0.04

GOLDER
MEMBER OF WSP

CALCULATIONS

Date:	8/3/2021	**Made by:**	KAR
Project No.:	21450908	**Checked by:**	MSG
Subject:	Drilled Shaft Design for Standard Lighting 14 at Station 104+40.00	**Reviewed by:**	JEL
Project Title:	MaineDOT Desert Road Bridge 5720 Freeport (Exit 20) Phase 2		

6. Check lateral geotechnical resistance of the shaft at the Strength I and Extreme I limit states.

Perform a pushover analysis using LPile to compute shaft head deflection at various multiples of the factored shear and moment design loads, up to 1/φ times the factored shear and moment design loads. As per Ref. 4 Section 9.3.3.3.1, for a stable condition the analyses should each converge to a solution with a computed deflection no larger than 10% of the shaft diameter. For the pushover analysis the shaft should be modeled as a simple linear elastic beam rather than a nonlinear stress-strain model (Ref. 4, Section 9.3.3.3.1).

Elastic modulus for linear model = 4,000,000 psi (Ref. 4, page 9-21)

Moment of inertia for linear model, I = 39,761 in^4 (Ref. 4, page 9-21: $I = \pi D^4/64$)

Recommended resistance factor φ for lateral geotechnical resistance (Ref. 4, Table 9-1):

Limit State	φ	1/φ
Strength I	0.67	1.5
Extreme I	0.8	1.25

Computed shaft deflection at each load multiple analyzed with LPile:

Limit State	LPile Load Case	Multiple of Factored Design Loads	Loads Applied in LPile: Axial (kips)	Loads Applied in LPile: Moment (kip-ft)	Loads Applied in LPile: Shear (kips)	Computed Lateral Head Deflection from LPile (in)	Deflection < 10% of Shaft Diameter?
Strength I	1	0.25	1.5	0.1	0.0	0.00	Yes
	2	0.50	1.5	0.3	0.0	0.00	Yes
	3	0.75	1.5	0.4	0.0	0.00	Yes
	4	1.00	1.5	0.5	0.0	0.00	Yes
	5	1.25	1.5	0.6	0.0	0.00	Yes
	6	1.50	1.5	0.8	0.0	0.01	Yes
Extreme I	7	0.25	1.5	4.4	0.3	0.04	Yes
	8	0.50	1.5	8.8	0.5	0.07	Yes
	9	0.75	1.5	13.1	0.8	0.10	Yes
	10	1.00	1.5	17.5	1.0	0.14	Yes
	11	1.25	1.5	21.9	1.3	0.18	Yes
					maximum =	0.18	

The trial shaft length of 5.5 feet exhibits stable behavior up through 1/φ times the factored design loads, and the maximum computed deflection of 0.18 inches is less than 10% of the shaft diameter (10% of 30 inches = 3 inches). Thus, based on the pushover analysis, the trial shaft length of 5.5 feet satisfies the lateral geotechnical criterion at the Strength I and Extreme I limit states.

Date:	8/3/2021	**Made by:**	KAR
Project No.:	21450908	**Checked by:**	MSG
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Project Title:	MaineDOT Desert Road Bridge 5720 Freeport (Exit 20) Phase 2		

7. Check horizontal movement at the top of the shaft at the Service I limit state.

Use LPile to compute shaft head deflection at the Service I factored design loads. As per Ref. 4 page 9-27 and Ref. 14 page 3, deflection due to combined loading on the structure should be limited to 0.5 inches at the top of the shaft. For the serviceability analysis the shaft should be modeled as a nonlinear reinforced concrete shaft in flexure (Ref. 4, Section 9.3.3.3.3).

Computed shaft deflection analyzed with LPile:

Limit State	LPile Load Case	Multiple of Factored Design Loads	Loads Applied in LPile			Computed Lateral Head Deflection from LPile (in)	Deflection < 0.5 Inches?
			Axial (kips)	Moment (kip-ft)	Shear (kips)		
Service I	12	1.00	1.5	9.5	0.5	0.07	Yes

The computed deflection of 0.07 inches is less than the required limit of 0.5 inches. Thus the trial shaft length of 5.5 feet satisfies the lateral criterion at the Service I limit state. The LPile analysis output for factored design loads at the Service I limit state is included as Attachment 1.

8. Check embedment length to resist torsion loading demand at the Extreme I limit state.

Compute nominal and factored torsion resistance of the drilled shaft. Due to the limitations of the method in Ref. 15 Section 4.2.2 and Ref. 16 Section 13.6.1.1, assume the drilled shaft is installed entirely within cohesionless soil with an N-value equal to that of the glaciomarine silty clay.

$$T_u \le \phi_{tor} \cdot T_n$$ (Ref. 16, Section 13.6.1.1)

$$T_n = \pi D L F_s \left(\frac{D}{2}\right)$$ (Ref. 16, Section 13.6.1.1)

$$F_s = \sigma'_v \omega_{fdot}$$ (Ref. 16, Section 13.6.1.1)

where:

Factored torsional loading demand, T_u =	0.5	kip-ft	(From Step 4 - Extreme I)
Resistance factor for torsion, ϕ_{tor} =	0.9		(Ref. 16, Section 13.6.1.1)

Note: using the minimum resistance factor specified in Ref. 16 Section 13.6.1.1 (the factor for overhead cantilever sign structures), since a resistance factor for lightpost structures is not available.

Shaft diameter, D =	2.5	ft	(From Step 2)
Shaft length =	5.5	ft	(From Step 2)
Midpoint of shaft =	2.75	ft	
Vertical effective stress at midpoint, σ'_v =	0.336	ksf	
Uncorrected N-value for Glaciomarine =	17		(Ref. 5)
Load transfer ratio, ω_{fdot} =	1.5		(Ref. 16, Section 13.6.1.1, for uncorrected N-values of 15 or greater)
Unit skin friction, F_s =	0.50	ksf	

CALCULATIONS

Date:	8/3/2021	**Made by:**	KAR
Project No.:	21450908	**Checked by:**	MSG
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Shaft length contributing to resistance, L = 0.5 ft (neglecting top 5 feet as per Ref. 1, Article C10.8.3.5.1b)

Nominal torsion resistance, T_n = 2.5 kip-ft

Factored torsion resistance, T_r = 2.2 kip-ft

Check against Extreme I factored torsion design load:

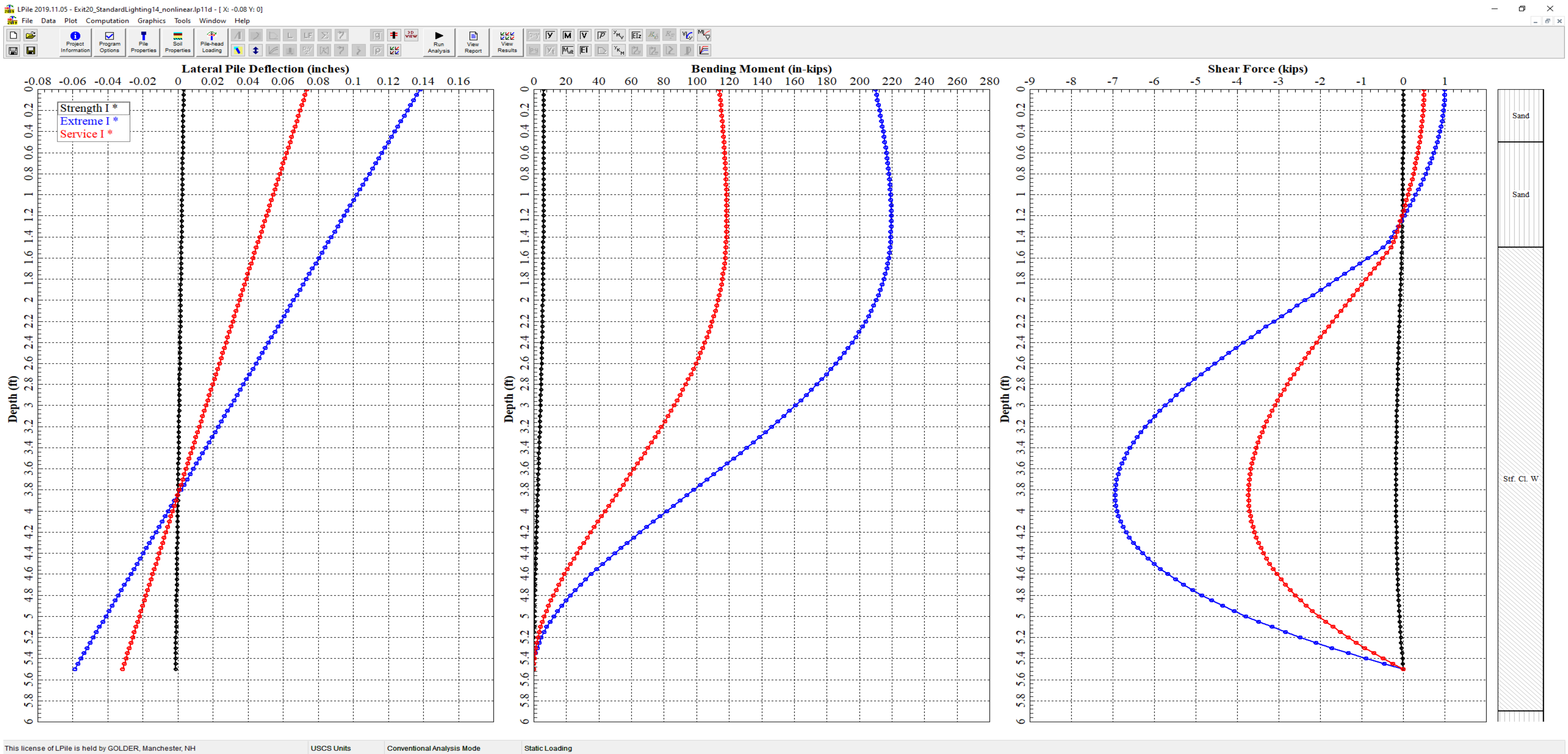
Extreme I factored torsion design load:		Factored torsion resistance:	
0.5 kip-ft	<	2.2 kip-ft	OK

Demand capacity ratio = 0.22

CONCLUSIONS

The results of the analysis indicate that the proposed drilled shaft foundation with a shaft diameter of 30 inches and a shaft length of 5.5 feet will provide adequate support for Standard Lighting 14 at Exit 20 based on the final design loads provided by HNTB. A maximum lateral deflection of 0.07 inches occurs at the top of the shaft under the Service I load case, satisfying the limiting requirement of 0.5 inches. Although reinforcement consisting of ten Grade 60 #6 bars arranged in a circular pattern was used in Golder's modeling, it is understood that HNTB will perform the final structural check and generate the required reinforcement pattern.

Attachment 1



APPENDIX F

Spread Footing Design Calculations

GOLDER MEMBER OF WSP	SUBJECT:	MaineDOT Freeport Foundation Design - Mast Arm and Light			
	Job No.:	21450908	Prepared:	MSG	8/5/2021
	Location:	WIN 023627.00 Freeport Desert Road	Checked:	AH/EDF	8/6/2021
	Date:	8/2/2021	Reviewed:	CCB	8/11/2021

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2	Calculation Equations Sheet
6	LRFD Shallow Foundation Geotechnical Calculation Mast Arm on Rock
10	LRFD Shallow Foundation Geotechnical Calculation Mast Arm on Soil
14	LRFD Shallow Foundation Geotechnical Calculation Light Standard on Rock
17	LRFD Shallow Foundation Geotechnical Calculation Light Standard on Soil

GOLDER MEMBER OF WSP	**SUBJECT:**	MaineDOT Freeport Foundation Design - Mast Arm and Light			
	Job No.:	21450908	**Prepared:**	MSG	8/5/2021
	Location:	WIN 023627.00 Freeport Desert Road	**Checked:**	AH/EDF	8/6/2021
	Date:	8/2/2021	**Reviewed:**	CCB	8/11/2021

Calculation Equations Sheet

Equations	**Variable**	**Equation Number**

Bearing Capacity Factors (REF 3)

N_q = bearing capacity factor for the surcharge term:

$$= e^{\pi \tan\phi} \tan^2 (45° + \frac{\phi}{2})$$

N_q — 21450908-CALC-001-EQ1

N_c = bearing capacity factor for the cohesion term:

$$= (N_q - 1)\cot\phi \quad \text{for } \phi > 0°$$

$$= 2 + \pi = 5.14 \quad \text{for } \phi = 0°$$

N_c — 21450908-CALC-001-EQ2

$$N_\gamma = 2\,(N_q + 1)\tan(\phi)$$

N_γ — 21450908-CALC-001-EQ3

Share Correction Factors for Ultimate Bearing Capacity (REF 3)

TABLE 5-2: SHAPE CORRECTION FACTORS (AASHTO 1996)

Factor	Friction Angle	Cohesion Term (s_c)	Unit Weight Term (s_γ)	Surcharge Term (s_q)
Shape Factors, s_c, s_γ, s_q	$\phi = 0$	$1+\left(\frac{B_f}{5L_f}\right)$	1.0	1.0
	$\phi > 0$	$1+\left(\frac{B_f}{L_f}\right)\left(\frac{N_q}{N_c}\right)$	$1-0.4\left(\frac{B_f}{L_f}\right)$	$1+\left(\frac{B_f}{L_f}\tan\phi\right)$

Note: Shape (eccentricity) factors, s, should not be applied simultaneously with inclined loading factors, i. See Section 5.2.3.7.

s_c — 21450908-CALC-001-EQ4

s_γ — 21450908-CALC-001-EQ5

s_q — 21450908-CALC-001-EQ6

Embedment Depth Correction (REF 3)

Friction Angle, ϕ (degrees)	D_f/B_f	d_q
32	1	1.20
	2	1.30
	4	1.35
	8	1.40
37	1	1.20
	2	1.25
	4	1.30
	8	1.35
42	1	1.15
	2	1.20
	4	1.25
	8	1.30

Note: The depth correction factor should be used only when the soils above the footing bearing elevation are as competent as the soils beneath the footing level; otherwise, the depth correction factor should be taken as 1.0.

d_q — 21450908-CALC-001-EQ7

Groundwater Depth Correction Factor (REF 3)

$$C_{wq} = 0.5 + 0.5\left(\frac{D_W}{\ \ }\right) \le 1.0$$

C_{wq} — 21450908-CALC-001-EQ8

$$C_{W_\gamma} = 0.5 + 0.5\left(\frac{D_w}{1.5B_f + D_f}\right) \le 1.0$$

$C_{w\gamma}$ 21450908-CALC-001-EQ9

$$C_{W_q} = 0.5 + 0.5\left(\frac{D_W}{D_f}\right) \le 1.0$$

Factor	Friction Angle	Cohesion Term (c)	Unit Weight Term (γ)	Surcharge Term (q)	
Base Inclination Factors, b_c, b_γ, b_q		b_c	b_γ	b_q	21450908-CALC-001-EQ10
	$\phi = 0$	$1-\left(\frac{\alpha}{147.3}\right)$	1.0	1.0	21450908-CALC-001-EQ11
	$\phi > 0$	$b_q - \left(\frac{1-b_q}{N_c \tan\phi}\right)$	$(1\text{-}0.017\alpha \tan\phi)^2$	$(1\text{-}0.017\alpha \tan\phi)^2$	21450908-CALC-001-EQ12

Definitions: ϕ = friction angle, degrees; α = footing inclination from horizontal, upward +, degrees

Ultimate Bearing Capacity (REF 3)

$$q_{ult} = cN_c s_c b_c + qN_q C_{W_q} s_q b_q d_q + 0.5\gamma B_f N_\gamma C_{W_\gamma} s_\gamma b_\gamma \qquad (5\text{-}14)$$

21450908-CALC-001-EQ13

where: s_c, s_γ and s_q are shape correction factors
b_c, b_γ and b_q are correction factors for the inclination of the base
d_q is a correction factor to account for the shearing resistance along the failure surface passing through cohesionless material above the bearing elevation (recall that the embedment is modeled as a surcharge pressure only, applied at the bearing elevation)
$C_{W\gamma}$ and C_{Wq} are correction factors considering the location of the ground water table
N_c, N_γ and N_q are obtained from Table 5-1 or Figure 5-6
N_c and N_γ are replaced with N_{cq} and $N_{\gamma q}$ for the sloping ground or footing near slope case

Additional Loads from Concrete and Soil (NOT provided by Structural Engineer)

Concrete Load Calculations

$$P_F = B_f \times L_f \times t_f \times \gamma_c$$

Foundation Load P_F 21450908-CALC-001-EQ14

$$P_p = A_p \times H_p \times \gamma_c$$

Pedestal Load P_P 21450908-CALC-001-EQ15

$$P_{Tc} = P_F + P_P$$

Total Concrete Load P_{Tc} 21450908-CALC-001-EQ16

Soil Load Calculations

$$V_s = (B_f * L_f * H_p) - (A_p * H_p)$$

Soil Volume V_s 21450908-CALC-001-EQ17

$$P_{Ts} = V_s \gamma_s$$

Total Soil Load P_{Ts} 21450908-CALC-001-EQ18

Check for Eccentricity (REF 10)

$$e = \frac{Factored\ Applied\ Moments}{Factored\ Applied\ Vertical\ Loads}$$

Eccentricity e 21450908-CALC-001-EQ19

$$B'_f = B_f - 2e_B$$

Effective Width and Length B'_f 21450908-CALC-001-EQ20

$L'_f = L_f - 2e_L$		L'$_f$	21450908-CALC-001-EQ21
$A' = B'_f L'_f$	Effective Area	A'	21450908-CALC-001-EQ22
$CDR_e = OK\ if\ e < \frac{B_f}{4}\ for\ soil\ and\ rock$	Eccentricity Check AASHTO Section 13.7.1 (REF 10)	CDR$_e$	21450908-CALC-001-EQ23
$\gamma_p P = \gamma_p(P_{Tc} + P_{Ts}) + P_1$	Factored Load	Υ$_p$P	21450908-CALC-001-EQ24
$M = M_1 + (D_f \times V_1)$	Factored Moment	M	21450908-CALC-001-EQ25

Check for Bearing Capacity (REF 3)

$Q_R = q_{u,centric} \times \varphi_{tb}$	Factored Bearing Capacity	Q$_R$	21450908-CALC-001-EQ26
$q_{app} = \frac{\gamma_{tb} P}{A'}$	Applied Bearing Pressure	q$_{app}$	21450908-CALC-001-EQ27
$q_{app} = \frac{\gamma_{tb} P}{B'L}$	soil (AASHTO Eq. 11.6.3.2-1)		
$q_{app} = \frac{\gamma_{tb} P}{BL}(1 + \frac{6e}{B})$	rock (AASHTO Eq. 11.6.3.2-2)		
$CDR_b = OK\ if\ Q_R > q_{app}$	Bearing Check	CDR$_b$	21450908-CALC-001-EQ28

Check for Sliding Capacity (REF 3)

$\gamma_T P = \gamma_T(P_{Tc} + P_{Ts}) + P_1$	Factored Load	Υ$_T$P	21450908-CALC-001-EQ29
$Q_T = \gamma_T P \tan \varphi_s$	Resistance at Interface	QT	21450908-CALC-001-EQ30
$Q_{ep} = 0.5 K_p \gamma_s t_f^2$	Resistance due to Passive Pressure	Qep	21450908-CALC-001-EQ31
$Q_{Rs} = \varphi_T Q_T + \varphi_{ep} Q_{ep}$	Total Factored Resistance	Q$_{RS}$	21450908-CALC-001-EQ32
$V_T = \gamma_T V_1$	Factored Shear Loading	V$_T$	21450908-CALC-001-EQ33
$CDR_s = OK\ if\ Q_{Rs} > V_T$	Sliding Check	CDR$_s$	21450908-CALC-001-EQ34

Calculation for Settlement (REF 4)

$\Delta\sigma_v = \frac{P_1}{A'}$		Δσ$_v$	21450908-CALC-001-EQ35
$\sigma'_{v0} = \gamma_{si}\frac{D_{fli} - D_{ili}}{2}$, top layer	$\sigma'_{v0} = (D_{fl1}\gamma_1) + \gamma_{s2}\frac{D_{fl2} - D_{il2}}{2}$,	σ'$_{v0}$	21450908-CALC-001-EQ35

layers below

ΔH_i 21450908-CALC-001-EQ36

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

GOLDER MEMBER OF WSP					
SUBJECT:	MaineDOT Freeport Foundation Design - Mast Arm and Light				
Job No.:	21450908	**Prepared:**	MSG	8/5/2021	
Location:	WIN 023627.00 Freeport Desert Road	**Checked:**	AH/EDF	8/6/2021	
Date:	8/2/2021	**Reviewed:**	CCB	8/11/2021	

AASHTO LRFD Shallow Foundation Design - Based on Mast Arm 5 (MA5) (50 ft loading) (Sta. 65+45.27)

Purpose and Scope

The purpose of this calculation package is design shallow foundations in accordance with American Association of State Highway (AASHTO) Load Resistance Factor Design (LRFD) Bridge Design Specifications and Federal Highway Administration (FHWA) guidance.

Summary of Results and Conclusions

Foundation eccentricity, overturning resistance, bearing capacity resistance, and sliding resistance are satisfactory for the applied loads using a 10.5 foot x 10.5 foot x 1 foot square foundation. Estimated settlement for the foundation is less than 1/4 inch.

References

(1) Fang, HY; Foundation Engineering Handbook - 2nd Edition; 1991
(2) Bowles, JE; Foundation Analysis and Design - 4th Edition; 1988
(3) Federal Highway Administration (FHWA); Geotechnical Engineering Circular (GEC) No. 6 - Shallow Foundations; September 2002.
(4) American Association of State Highway and Transportation Officials (AASHTO); LRFD Bridge Design Specifications - 9th Edition; 2020.
(5) HNTB - Foundation Location Tables and Design Loads Correspondance (Dated June 29, 2021)
(6) National Cooperative Highway Research Program (NCHRP) - LRFD Design and Construction of Shallow Foundations for Highway Bridge Structures - Report 651, 2010
(7) Golder Associates; Mast Arm and Light Design Basis, 2021.
(8) HNTB 98% Plans
(9) Golder SGDR 200-series boring logs
(10) AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals, 1st Ed. 2015 with 2017, 2018, and 2020 Interim Revisions.
(11) Florida Department of Transportation. FDOT Modifications to LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals (LRFDLTS-1). Structures Manual, Volume 3. January 2021.

Assumptions

(1) Geotechnical field and laboratory testing results provide appropriate design characterization of subsurface conditions
(2) Loads and pressures provided by HNTB represent critical loading conditions for the project area(s).
(3) Maximum acceptable settlement for the foundation assumed to be 1 inch

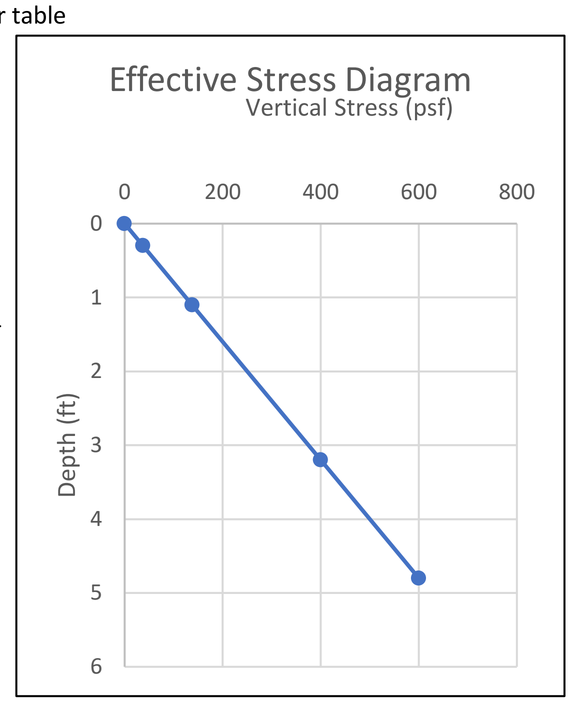
Soil Conditions (based on Boring BB-FDR-212, Ref. 9)

Layer (i)	Material Type	Depth Start (D_{ili}) (ft)	Depth End (D_{fli}) (ft)	Average N_{60} value	Friction Angle (ϕ') (deg)	Cohesion (c') (psf)	Effective Unit Weight (Υ') (pcf)	UCS (psi)	Rock Mass Modulus (ksi)	Poisson's Ratio	Depth at Midpoint (D_m) ft	Vertical Stress at Midpoint (σ'_{v0mi}) psf
											0	0
1	Soil	0	0.6	29	32	0	125				0.3	37.5
2	Soil	0.6	1.6	29	32	0	125				1.1	137.5
3	Soil	1.6	4.8	32	33	0	125				3.2	400
4	Rock	4.8					164	12983	1336	0	4.8	600

Depth to Water D_w = 4.2 ft <-Artificially high to represent no influence of water table

Foundation Characteristics

Embedment Depth: 5 feet or bedrock, whichever is encountered first D_f = 4.8 ft
Foundation Width: B_f = 10.5 ft L/B = 1
Foundation Length: L_f = 10.5 ft
Foundation Thickness: t_f = 1.0 ft EPRI Rigidity Factors (REF 4)
Footing Inclination from Horizontal (upward +) α = 0 deg β_z = 1.08 10.6.2.4.4-1
B/2 = 5.25 ft
Assumed Unit Weight of Concrete Υ_c = 150 pcf



Surcharge (σ_{v0}) at Base of the Footing q = 600 psf
Cohesion of Underlying Soil c = 0 psf
Unit Weight of Underlying Soil Υ = 125 pcf
Coefficient of Passive Pressure K_p = 3.39
Coefficient of Active Pressure K_a = 0.29

Calculations for Additional Loading from Foundation Concrete and Surrounding Soil

Assume 36" Column/Pedestal loading (based on minimum shaft diameter, Ref. 5)

Foundation Dimensions:	Foundation Width	B_f =	10.5	ft
	Foundation Length	L_f =	10.5	ft
	Foundation Thickness	t_f =	1.0	ft
Unit Weight of Concrete =		Υ_c =	150	pcf
Pedestal Diameter		D_P =	3	ft
Area of Pedestal below Ground		A_P =	7.1	ft^2
Height of Pedestal Below Ground		H_P =	3.8	ft

Load from Concrete	P_F =	16538 lb	EQ 14	Volume for Soil Above Footing	V_s =	392.09 ft^3	EQ 17
	P_P =	4029 lb	EQ 15	Unit Weight of Soil =	Υ_s =	125 pcf	
	P_{Tc} =	20567 lb	EQ 16	Load from Soil	P_{Ts} =	49011 lb	EQ 18
		21 kip				49 kip	

(A) Loading Conditions (Ref. 5)

Standard Lighting Factored Design Forces					
Load	Units	Strength I	Extreme I	Service 1	Service II
Axial, P_1	kip	11	10	9	9
Moment, M_2	kip-ft	148	176	129	119
Shear, V_1	kip	0	5	3	0
Torsion, M_1	kip-ft	0	142	62	0

Note: LRFD load factors for Service I equal to 1.0

(B) Check Eccentricity and Overturning Based on Moments about Toe of Footing

Table 5-6 (REF 3) Allowable Bearing Pressure: q_{all} = 0.5 MPa 10442.75 psf Value determined for weak schist/gneiss

Factor of Safety Applied for ASD Bearing Capacity FS = 3

Nominal Bearing Pressure q_n = 31328 psf

For Strength 1 case, load factor is applied:

Load Factor Minimum $\Upsilon_{P,\,Concrete}$ = 0.9 $\Upsilon_{P,\,Soil}$ = 1 Per AASHTO Section 3.4.2.1

Load Factor Maximum $\Upsilon_{P,\,Concrete}$ = 1.25 $\Upsilon_{P,\,Soil}$ = 1.35

For Other Cases Υ_P = 1

	EQ 24	EQ 25	EQ 19		EQ 23	
Load Group	$\Upsilon_P P$	M	e_B	e_L	Eccentricity	Eccentricity
	(kip)	(kip-ft)	(ft)	(ft)	CDR_e (B)	CDR_e (L)
Strength I	78.5	148	1.88	1.88	OK	OK
Extreme I	79.6	200	2.51	2.51	OK	OK
Service 1	78.6	143.4	1.82	1.82	OK	OK
Service 2	78.6	119	1.51	1.51	OK	OK

(1/4)B =	2.63
(1/4)L =	2.63

Per AASHTO 13.7.1 (Ref. 10), the eccentricity at the extreme limit state based on the factored loads shall not exceed 0.25 B or L.

(C) Determine CDR against Bearing Capacity Failure

Load Factor Maximum (soil) $\Upsilon_{P,\,soil}$ = 1.35 Table 3.4.1-2 AASHTO

Load Factor Maximum (concrete) $\Upsilon_{P,\,concrete}$ = 1.25 Table 3.4.1-2 AASHTO

Resistance Factor for Strength ϕ_{tb} = 0.45 Table 10.5.5.2.2-1 AASHTO

Resistance Factor, Extreme, Service I, Service II ϕ_{tb} = 1 Per AASHTO Section 10.5.5.1

	EQ 26		EQ 20	EQ 21	EQ 22	EQ 27	EQ 28
Load Group	Q_R	$\Upsilon_{tb}P$	B	L	e_B	q_{app}	Bearing
	(ksf)	(kip)	(ft)	(ft)	(ft)	(ksf)	CDR_b
Strength I	14.10	102.9	10.50	10.5	1.9	1.94	OK
Extreme I	31.33	79.6	10.50	10.5	2.5	1.76	OK
Service 1	31.33	78.6	10.50	10.5	1.8	1.46	OK
Service 2	31.33	78.6	10.50	10.5	1.5	1.33	OK

Note: Vertical load increased by maximum factor for conservative evaluation. Q_R = nominal bearing resistance multipled by resistance factors given in AASHTO.

(D) Determine CDR against Sliding

Load Factor for Sliding Minimum (concrete) Υ_T = 0.9 Table 3.4.1-2 AASHTO

Load Factor for Sliding Minimum (soil) Υ_T = 1 Table 3.4.1-2 AASHTO

Resistance Factor for Shear Between Footing Base and Soil $\phi_T = 0.8$ Table 10.5.5.2.2-1 AASHTO

Resistance Factor for Passive Soil $\phi_{ep} = 0.5$ Table 10.5.5.2.2-1 AASHTO

Interface Friction Angle for Concrete/Rock $\phi = 35$ deg Ref. 3, Table 5-15, mass concrete on clean sound rock

Load Group	EQ 29 $\gamma_T P$ (kip)	EQ 30 Q_T (kip)	$\phi_T Q_T$ (kip)	EQ 31 Q_{ep} (kip)	$\phi_{ep} Q_{ep}$ (kip)	EQ 32 Q_{Rs} (kip)	EQ 33 Shear, V_T (kip)	EQ 34 Sliding CDR_s
Strength I	78.5	54.98	43.98	0.21	0.11	44.09	0	OK
Extreme I	79.6	55.72	44.58	0.21	0.11	44.68	5	OK
Service 1	78.6	55.02	44.02	0.21	0.11	44.12	3	OK
Service 2	78.6	55.02	44.02	0.21	0.11	44.12	0	OK

Note: Shear load increased by maximum load factor for conservative evaluation. Resistance values used for footing interface with soil and passive given in AASHTO.

(E) Check embedment length to resist torsion loading demand at the Extreme I limit state. 0.7002075

Pedestal Diameter Resistance

$$T_u \leq \phi_{tor} \cdot T_n$$ (Ref. 11, Section 13.6.1.1)

$$T_n = \pi D L F_s \left(\frac{D}{2}\right)$$ (Ref. 11, Section 13.6.1.1)

$$F_s = \sigma'_v \omega_{fdot}$$ (Ref. 11, Section 13.6.1.1)

Pedestal diameter, D =	3.0	ft	
Pedestal length =	3.8	ft	
Midpoint of shaft =	1.9	ft	
Vertical effective stress at midpoint, σ'_v =	0.24	ksf	
Uncorrected N-value for Existing Fill =	18		(Correlated from design friction angle)
Load transfer ratio, ω_{fdot} = 1.5 =	1.5		(Ref. 11, Section 13.6.1.1, for uncorrected N-values greater than 15)
Unit skin friction, F_s =	0.36	ksf	
Shaft length contributing to resistance, L =	1.8	ft	(neglecting top 2 feet as per Ref. 14, page 5)
Nominal torsion resistance, T_n =	9	kip-ft	
Resistance factor for torsion, ϕ_{tor} =	1.0		(Ref. 11, Section 13.6.1.1, Minimum resistance factor for cantilever sign structures)
Factored torsion resistance, T_r =	9	kip-ft	

Spread Footing Side Wall Passive Resistance

$B_f/2$=	5.25	ft	
$L_f/2$=	5.25	ft	
Moment arm (1/6 B_f or L_f) =	3.5	ft	
Spread Footing Height =	1.0	ft	
Depth to midpoint of Spread Footing =	4.3	ft	
Vertical effective stress at midpoint, σ'_v =	0.54	ksf	
Passive earth pressure coefficient, k_p =	3.4		
Passive Earth Pressure (per side) =	4.7	kips	
Nominal torsion resistance, T_n (per side) =	16.6	kip-ft	
Nominal torsion resistance, T_n =	66.2	kip-ft	
Resistance factor for torsion, ϕ_{tor} =	1.0		(Ref. 11, Section 13.6.1.1, Minimum resistance factor for cantilever sign structures)
Factored torsion resistance, T_r =	66.2	kip-ft	

Assume that each side of the spread footing experiences passive earth pressure over a distance of half of the footing length or width in response to torsion, where the passive pressure increases from the midpoint of the side to the edge. Assume the other side of the footing width moves away from the soil and does not contribute to torsional resistance.

ϕ' (deg) = 33 $-\delta/\phi'$ = 0 (assumed)

k'_p = 8.3 R = 0.404 Figure 3.11.5.4-1 (Ref. 4)

Spread Footing Basal Resistance

B'_f =	7.9	ft	Effective footing dimensions are 0.75B by 0.75L per AASHTO 13.7.1 (Ref. 10) where the
L'_f =	7.9	ft	eccentricity at Strength I factored loads shall not exceed 0.25 B or L.
Moment arm (1/6 B'_f or L'_f) =	2.6	ft	
Total Axial Load =	80.6	kips	Sum: applied axial load for Strength I and weight of soil and concrete above footing base
Interface Friction Angle for Concrete/Rock, ϕ =	35	deg	Ref. 3, Table 5-15, mass concrete on clean sound rock
Frictional Load at Base of Footing =	56.4	kips	
Nominal torsion resistance, T_n =	148.1		
Resistance factor for torsion, ϕ_{tor} =	1.0		(Ref. 11, Section 13.6.1.1, Minimum resistance factor for cantilever sign structures)
Factored torsion resistance, T_r =	148.1	kip-ft	

Total factored torsion resistance, T_r = 223.4 kip-ft

Check against Extreme I factored torsion design load

Extreme I factored torsion design load: 142 kip-ft < Total factored torsion resistance: 223 kip-ft OK

Demand capacity ratio = 0.64

(F) Calculate Settlement at Foundation Location using Service I Loads

Determine Stress Increase at Rock Interface

$q_f = \Delta\sigma_v = q_{app}$ (Service I) = 1456 psf Per Calcuation Section B and D , applied vertical stress on eccentric footing area (B' x L) EQ 35
Conservatively assume that entire applied load is considered as change in vertical stress to system

Use the elastic settlement of foundations on broken or jointed rock provided in REF 4 10.6.2.4.4

EQ 36

Layer		Layer Thickness (ft)	σ'_{v0} at layer midpoint (Variable)	(psf)	(kPa)	σ'_{v0} at layer bottom (Variable)	(psf)	(kPa)	N_{60}	N'	C'
Soil	1	0.6	σ'_{v0m1}	37.5	1.8	σ'_{v0b1}	75	3.6	NA	NA	NA
	2	1	σ'_{v0m2}	137.5	6.6	σ'_{v0b2}	200	9.6	NA	NA	NA
	3	3.2	σ'_{v0m3}	400	19.1	σ'_{v0b3}	600	28.7	15	30	85
Rock	4	4.8	<- Top Elevation of Layer								

σ'_{v0bf} Initial Stress at Foundation Bottom 600 psf

$$p = q_0(1-v^2)\frac{rI_p}{144E_m}$$

EQ 37 (REF 4, 10.6.2.4.4-1)

Applied Vertical Stress at Base of Loaded Area	q_0 =	2.056	ksf
Poisson's Ratio	v =	0.27	
Influence Area	r or B/2	5.25	ft
Foundation Shape and Rigidity Factor	β_z =	1.08	
Rigidity Influence Coefficient	I_p =	1.64	
Intact Rock Modulus	E_i =	7820	ksi
Rock Mass Modulus	E_m =	192411350	psf
		1336	ksi

Influence area is radius or B/s (circular or square footing), (REF 4 10.6.4.4-1)

$$I_p = \frac{(\sqrt{\pi})}{\beta_z}$$

EQ 38 (REF 4, 10.6.2.4.4-2)

Based on the following parameters used in RocScience RSData using Generalized Hoek Brown Classification to Determine Rock Mass Modulus

GSI =	60	Good to Fair Blocky Quality	
mi =	12	For schists	
D =	0.8	Minimal disturbance	
UCS =	90	MPa	1869552 psf
E_i =	1.13E+09	psf	
v =	0.27		

p (bedrock elastic deformation)	ft	0.0001
	in	0.0010

(G) Conclusions

Foundation eccentricity, overturning resistance, bearing capacity resistance, and sliding resistance are satisfactory for the applied loads using a 10.5 foot x 10.5 foot x 1 foot square foundation. Estimated settlement for the foundation is less than 1/4 inch.

SUBJECT:	MaineDOT Freeport Foundation Design - Mast Arm and Light			
Job No.:	21450908	Prepared:	MSG	8/5/2021
Location:	WIN 023627.00 Freeport Desert Road	Checked:	AH/EDF	8/6/2021
Date:	8/2/2021	Reviewed:	CCB	8/11/2021

AASHTO LRFD Shallow Foundation Design - Based on Mast Arm 7 (MA7) (50 ft loading) (Sta. 67+05.00)

Purpose and Scope

The purpose of this calculation package is design shallow foundations in accordance with American Association of State Highway (AASHTO) Load Resistance Factor Design (LRFD) Bridge Design Specifications and Federal Highway Administration (FHWA) guidance.

Summary of Results and Conclusions

Foundation eccentricity, overturning resistance, bearing capacity resistance, and sliding resistance are satisfactory for the applied loads using a 10.5 foot x 10.5 foot x 1 foot square foundation. Estimated settlement for the foundation is less than 1/2 inch.

References

(1) Fang, HY; Foundation Engineering Handbook - 2nd Edition; 1991
(2) Bowles, JE; Foundation Analysis and Design - 4th Edition; 1988
(3) Federal Highway Administration (FHWA); Geotechnical Engineering Circular (GEC) No. 6 - Shallow Foundations; September 2002.
(4) American Association of State Highway and Transportation Officials (AASHTO); LRFD Bridge Design Specifications - 9th Edition; 2020.
(5) HNTB - Foundation Location Tables and Design Loads Correspondance (Dated June 29, 2021)
(6) National Cooperative Highway Research Program (NCHRP) - LRFD Design and Construction of Shallow Foundations for Highway Bridge Structures - Report 651, 2010
(7) Golder Associates; Mast Arm and Light Design Basis, 2021.
(8) HNTB 98% Plans
(9) Golder SGDR 200-series boring logs
(10) AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals, 1st Ed. 2015 with 2017, 2018, and 2020 Interim Revisions.

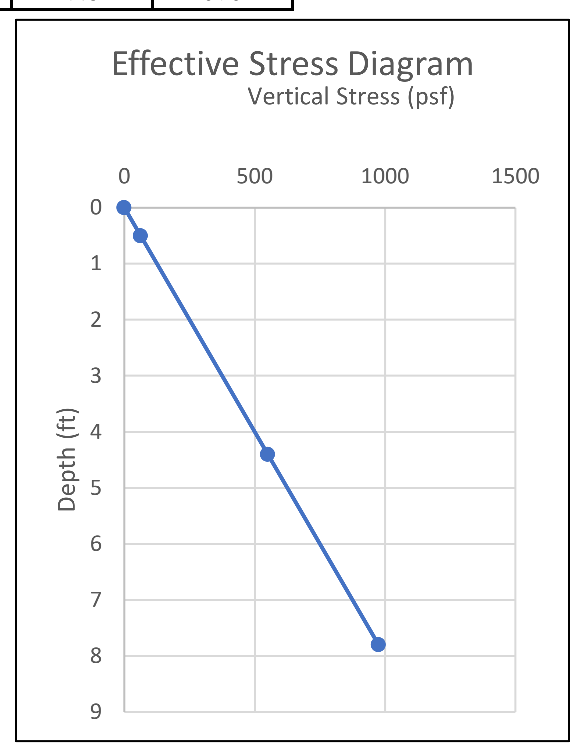
Assumptions

(1) Geotechnical field and laboratory testing results provide appropriate design characterization of subsurface conditions
(2) Loads and pressures provided by HNTB represent critical loading conditions for the project area(s).
(3) Maximum acceptable settlement for the foundation assumed to be 1 inch

Soil Conditions (based on Boring BB-FDR-216, Ref. 9)

Layer (i)	Material Type	Depth: Start (D_{ili}) (ft)	Depth: End (D_{fli}) (ft)	Average N_{60} value	Friction Angle (ϕ') (deg)	Cohesion (c') (psf)	Effective Unit Weight (Υ') (pcf)	UCS (psi)	Depth at Midpoint (D_m) ft	Vertical Stress at Midpoint (σ'_{v0mi}) psf
									0	0
1	Soil	0	1	11	32	0	125		0.5	62.5
2	Soil	1	7.8	32	32	0	125		4.4	550
3	Rock	7.8		36	36	0	164	12983	7.8	975

Depth to Water D_w = 8.2 ft



Foundation Characteristics

Embedment Depth: 5 feet or bedrock, whichever is encountered first D_f = 5 ft
Foundation Width: B_f = 10.5 ft
Foundation Length: L_f = 10.5 ft
Foundation Thickness: t_f = 1.0 ft
Footing Inclination from Horizontal (upward +) α = 0 deg
Assumed Unit Weight of Concrete Υ_c = 150 pcf

Surcharge (σ_{v0}) at Base of the Footing q = 625 psf
Cohesion of Underlying Soil c = 0 psf
Unit Weight of Underlying Soil Υ = 125 pcf
Coefficient of Passive Pressure K_p = 3.25
Coefficient of Active Pressure K_a = 0.31

Calculations for Additional Loading from Foundation Concrete and Surrounding Soil

Assume 36" Column/Pedestal loading (based on minimum shaft diameter, Ref. 5)

Foundation Dimensions:
Foundation Width B_f = 10.5 ft
Foundation Length L_f = 10.5 ft
Foundation Thickness t_f = 1.0 ft

Unit Weight of Concrete = γ_c = 150 pcf

Pedestal Diameter D_P = 3 ft
Area of Pedestal below Ground A_P = 7.1 ft^2
Height of Pedestal Below Ground H_P = 4 ft

Load from Concrete P_F = 16538 lb EQ 14
P_P = 4241 lb EQ 15
P_{Tc} = 20779 lb EQ 16
21 kip

Volume for Soil Above Footing V_s = 412.73 ft^3 EQ 17
Unit Weight of Soil = γ_s = 125 pcf
Load from Soil P_{Ts} = 51591 lb EQ 18
52 kip

(A) Calculate Ultimate Bearing Capacity

Bearing Capacity Factors EQ 1, 2, 3
N_c = 35.5
N_q = 23.2
N_γ = 30.2

Shape Factors EQ 4, 5, 6
s_c = 1.65
s_q = 1.62
s_γ = 0.60

Base Inclination Correction Factors EQ 10, 11, 12
b_c = 1.00
b_q = 1.00
b_γ = 1.00

Groundwater Location Corrections EQ 8, 9
C_{wq} = 0.6975904
$C_{w\gamma}$ = 1

Depth Correction Factors EQ 7
d_q = 1.12

$q_{u,centric}$ = 30344 psf EQ 13

(B) Loading Conditions (Ref. 5)

Standard Lighting Factored Design Forces					
Load	**Units**	**Strength I**	**Extreme I**	**Service 1**	**Service II**
Axial, P_1	kip	11	10	9	9
Moment, M_2	kip-ft	148	176	129	119
Shear, V_1	kip	0	5	3	0
Torsion, M_1	kip-ft	0	142	62	0

Note: LRFD load factors for Service I equal to 1.0

(C) Check Eccentricity and Overturning Based on Moments about Toe of Footing

For Strength 1 case, load factor is applied:
Load Factor Minimum $\gamma_{P, Concrete}$ = 0.9 $\gamma_{P, Soil}$ = 1 Per AASHTO Section 3.4.2.1 (Ref. 4)
Load Factor Maximum $\gamma_{P, Concrete}$ = 1.25 $\gamma_{P, Soil}$ = 1.35
For Other Cases γ_P = 1

	EQ 24	EQ 25	EQ 19		EQ 23	
Load Group	$\gamma_P P$ (kip)	M (kip-ft)	e_B (ft)	e_L (ft)	Eccentricity CDR_e (B)	Eccentricity CDR_e (L)
Strength I	81.3	148	1.82	1.82	OK	OK
Extreme I	82.4	201	2.44	2.44	OK	OK
Service 1	81.4	144	1.77	1.77	OK	OK
Service 2	81.4	119	1.46	1.46	OK	OK

(1/4)B =	2.63
(1/4)L =	2.63

Per AASHTO 13.7.1 (Ref. 10), the eccentricity at the extreme limit state based on the factored loads shall not exceed 0.25 B or L.

(D) Determine CDR against Bearing Capacity Failure

Load Factor Maximum (soil) $\gamma_{P, soil}$ = 1.35 Table 3.4.1-2 AASHTO (Ref. 4)
Load Factor Maximum (concrete) $\gamma_{P, concrete}$ = 1.25 Table 3.4.1-2 AASHTO (Ref. 4)
Resistance Factor for Strength ϕ_{tb} = 0.45 Table 10.5.5.2.2-1 AASHTO (Ref. 4)
Resistance Factor, Extreme, Service I, Service II ϕ_{tb} = 1 Per AASHTO Section 10.5.5.1 (Ref. 4)

	EQ 26		EQ 20	EQ 21	EQ 22	EQ 27	EQ 28
Load Group	Q_R (ksf)	$\gamma_{tb}P$ (kip)	B' (ft)	L (ft)	A' (ft^2)	q_{app} (ksf)	Bearing CDR_b
Strength I	13.65	106.6	6.86	10.5	72.0	1.48	OK
Extreme I	30.34	82.4	5.62	10.5	59.0	1.40	OK
Service 1	30.34	81.4	6.96	10.5	73.1	1.11	OK
Service 2	30.34	81.4	7.58	10.5	79.5	1.02	OK

Note: Vertical load increased by maximum load factor for conservative evaluation. Q_R = nominal bearing resistance multipled by resistance factors given in AASHTO.

(E) Determine CDR against Sliding

GOLDER MEMBER OF WSP				
SUBJECT:	MaineDOT Freeport Foundation Design - Mast Arm and Light			
Job No.:	21450908	**Prepared:**	MSG	8/5/2021
Location:	WIN 023627.00 Freeport Desert Road	**Checked:**	AH/EDF	8/6/2021
Date:	8/2/2021	**Reviewed:**	CCB	8/11/2021

Load Factor for Sliding Minimum (concrete)	γ_T = 0.9		Table 3.4.1-2 AASHTO (Ref. 4)
Load Factor for Sliding Minimum (soil)	γ_T = 1		Table 3.4.1-2 AASHTO (Ref. 4)
Resistance Factor for Shear Between Footing Base and Soil	ϕ_T = 0.8		Table 10.5.5.2.2-1 AASHTO (Ref. 4)
Resistance Factor for Passive Soil	ϕ_{ep} = 0.5		Table 10.5.5.2.2-1 AASHTO (Ref. 4)
Interface Friction Angle for Concrete/Soil	ϕ = 24	deg	Ref. 3, Table 5-15, mass concrete on silty med. to coarse sand

	EQ 29	EQ 30		EQ 31		EQ 32	EQ 33	EQ 34
Load Group	$\gamma_T P$	Q_T	$\phi_T Q_T$	Q_{ep}	$\phi_{ep} Q_{ep}$	Q_{Rs}	Shear, V_T	Sliding CDR_s
	(kip)	(kip)	(kip)	(kip)	(kip)	(kip)	(kip)	
Strength I	81.3	36.19	28.95	0.20	0.10	29.06	0	OK
Extreme I	82.4	36.67	29.34	0.20	0.10	29.44	5	OK
Service 1	81.4	36.23	28.98	0.20	0.10	29.08	3	OK
Service 2	81.4	36.23	28.98	0.20	0.10	29.08	0	OK

Note: Shear load increased by maximum factor for conservative evaluation. Resistance values used for footing interface with soil and passive given in AASHTO.

(F) Check embedment length to resist torsion loading demand at the Extreme I limit state.

Pedestal Diameter Resistance

$$T_u \leq \phi_{tor} \cdot T_n$$ (Ref. 11, Section 13.6.1.1)

$$T_n = \pi D L F_s \left(\frac{D}{2}\right)$$ (Ref. 11, Section 13.6.1.1)

$$F_s = \sigma'_v \omega_{fdot}$$ (Ref. 11, Section 13.6.1.1)

Pedestal diameter, D =	3.0	ft	
Pedestal length =	4.0	ft	
Midpoint of shaft =	2	ft	
Vertical effective stress at midpoint, σ'_v =	0.25	ksf	
Uncorrected N-value for Existing Fill =	20		(Correlated from design friction angle)
Load transfer ratio, ω_{fdot} = 1.5 =	1.5		(Ref. 11, Section 13.6.1.1, for uncorrected N-values greater than 15)
Unit skin friction, F_s =	0.38	ksf	
Shaft length contributing to resistance, L =	2.0	ft	(neglecting top 2 feet as per Ref. 14, page 5)
Nominal torsion resistance, T_n =	11	kip-ft	
Resistance factor for torsion, ϕ_{tor} =	1.0		(Ref. 11, Section 13.6.1.1, Minimum resistance factor for cantilever sign structures)
Factored torsion resistance, T_r =	4.0	kip-ft	

Spread Footing Side Wall Passive Resistance

Assume that each side of the spread footing experiences passive earth pressure over a distance of half of the footing length or width in response to torsion, where the passive pressure increases from the midpoint of the side to the edge. Assume the other side of the footing width moves away from the soil and does not contribute to torsional resistance.

$B_f/2$=	5.25	ft	
$L_f/2$=	5.25	ft	
Moment arm (1/6 B_f or L_f) =	3.5	ft	
Spread Footing Height =	1.0	ft	
Depth to midpoint of Spread Footing =	4.5	ft	
Vertical effective stress at midpoint, σ'_v =	0.56	ksf	ϕ' (deg) = 32; $-\delta/\phi'$ = 0 (assumed)
Passive earth pressure coefficient, k_p =	3.4		k'_p = 8.3; R = 0.404; Figure 3.11.5.4-1 (Ref. 4)
Passive Earth Pressure (per side) =	5.0	kips	
Nominal torsion resistance, T_n (per side) =	17.3	kip-ft	
Nominal torsion resistance, T_n =	69.3	kip-ft	
Resistance factor for torsion, ϕ_{tor} =	1.0		(Ref. 11, Section 13.6.1.1, Minimum resistance factor for cantilever sign structures)
Factored torsion resistance, T_r =	69.3	kip-ft	

Spread Footing Basal Resistance

B'_f =	7.9	ft	Effective footing dimensions are 0.75B by 0.75L per AASHTO 13.7.1 (Ref. 10) where the eccentricity at Strength I factored loads shall not exceed 0.25 B or L.
L'_f =	7.9	ft	
Moment arm (1/6 B'_f or L'_f) =	2.6	ft	
Total Axial Load =	83.4	kips	Sum: applied axial load for Strength I and weight of soil and concrete above footing base
Interface Friction Angle for Concrete/Soil =	24	deg	Ref. 3, Table 5-15, mass concrete on silty med. to coarse sand
Frictional Load at Base of Footing =	37.1	kips	
Nominal torsion resistance, T_n =	97.4		
Resistance factor for torsion, ϕ_{tor} =	1.0		(Ref. 11, Section 13.6.1.1, Minimum resistance factor for cantilever sign structures)
Factored torsion resistance, T_r =	97.4	kip-ft	

Total factored torsion resistance, T_r = 170.7 kip-ft

Check against Extreme I factored torsion design load

Extreme I factored torsion design load:		Total factored torsion resistance:	
142 kip-ft	<	171 kip-ft	OK

Demand capacity ratio = 0.83

(G) Calculate Settlement at Foundation Location using Service I Loads

Determine stress increase at Layer 3 midpoint

$q_f = \Delta\sigma_v = q_{app}$ (Service I) = 1113 psf — Per Calcuation Section B and D , applied vertical stress on eccentric footing area (B' x L) EQ 35
Conservatively assume that entire applied load is considered as change in vertical stress to system

Use the Hough method to estimate settlement of the Soil layer (Layer 2)

Hough Method:
Determine σ'_{v0} and C' at layer midpoints for in situ existing soil. Use Ref. 3 Figures 5-18 and 5-19 to obtain corrected N' and C' for Layer 3, assuming the Sand/Gravel matches "Well graded silty SAND & GRAVEL".

EQ 36

Layer		Layer Thickness	σ'_{v0} at layer midpoint			σ'_{v0} at layer bottom			N_{60}	N'	C'
		(ft)	(Variable)	(psf)	(kPa)	(Variable)	(psf)	(kPa)			
Soil	1	1	σ'_{v0m1}	62.5	3.0	σ'_{v0b1}	125	6.0	NA	NA	NA
	2	6.8	σ'_{v0m2}	550	26.3	σ'_{v0b2}	975	46.7	32	61	215

σ'_{v0bf} Initial Stress at Foundation Bottom 516 psf

Hough Method General Equation:

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

(REF 3. Eqn 5-24) EQ 37

where:

ΔH_i	settlement in each layer, ft		
H_c	initial height of layer i, ft	2.8	ft
C'	bearing capacity index from Ref. 12, Figure 5-19	0	
$\sigma'v_0$	initial stress, ksf	0.550	ksf
$\Delta\sigma_v$	vertical stress increase, ksf	1.113	ksf

ΔH_i (Layer 2)	ft	0.02
	in	0.18

(H) Conclusions

Foundation eccentricity, overturning resistance, bearing capacity resistance, and sliding resistance are satisfactory for the applied loads using a 10.5 foot x 10.5 foot x 1 foot square foundation. Estimated settlement for the foundation is less than 1/2 inch.

AASHTO LRFD Shallow Foundation Design - Based on Light Standard 1 (LS1, Sta. 54+00) Soil Profile

Purpose and Scope

The purpose of this calculation package is design shallow foundations in accordance with American Association of State Highway (AASHTO) Load Resistance Factor Design (LRFD) Bridge Design Specifications and Federal Highway Administration (FHWA) guidance.

Summary of Results and Conclusions

Foundation eccentricity, overturning resistance, bearing capacity resistance, and sliding resistance are satisfactory for the applied loads using a 5.5 foot x 5.5 foot x 1 foot square foundation. Estimated settlement for the foundation is less than 1/4 inch.

References

(1) Fang, HY; Foundation Engineering Handbook - 2nd Edition; 1991
(2) Bowles, JE; Foundation Analysis and Design - 4th Edition; 1988
(3) Federal Highway Administration (FHWA); Geotechnical Engineering Circular (GEC) No. 6 - Shallow Foundations; September 2002.
(4) American Association of State Highway and Transportation Officials (AASHTO); LRFD Bridge Design Specifications - 9th Edition; 2020.
(5) HNTB - Foundation Location Tables and Design Loads Correspondance (Dated June 29, 2021)
(6) National Cooperative Highway Research Program (NCHRP) - LRFD Design and Construction of Shallow Foundations for Highway Bridge Structures - Report 651, 2010
(7) Golder Associates; Mast Arm and Light Design Basis, 2021.
(8) HNTB 98% Plans
(9) Golder SGDR 200-series boring logs
(10) AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals, 1st Ed. 2015 with 2017, 2018, and 2020 Interim Revisions.

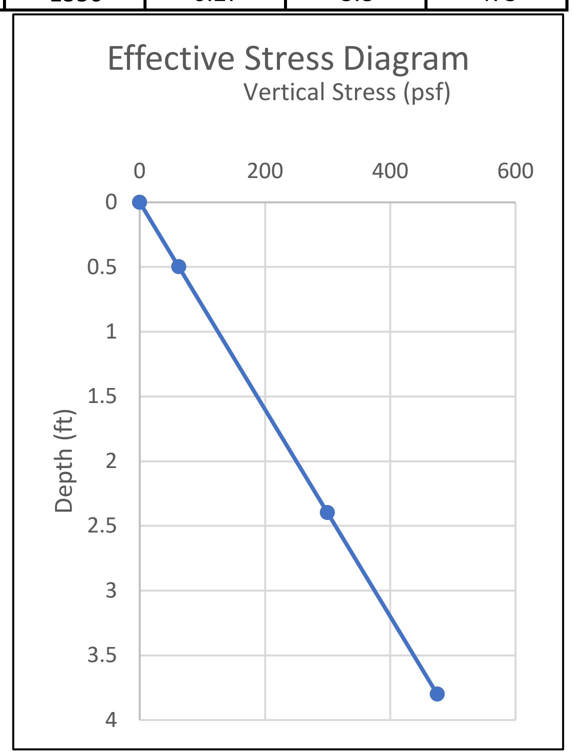
Assumptions

(1) Geotechnical field and laboratory testing results provide appropriate design characterization of subsurface conditions
(2) Loads and pressures provided by HNTB represent critical loading conditions for the project area(s).
(3) Maximum acceptable settlement for the foundation assumed to be 1 inch

Soil Conditions (based on Boring BB-FDR-201, Ref. 9)

Layer (i)	Material Type	Depth Start (D_{ili}) (ft)	Depth End (D_{fli}) (ft)	Average N_{60} value	Friction Angle (ϕ') (deg)	Cohesion (c') (psf)	Effective Unit Weight (ϒ') (pcf)	UCS (psi)	Rock Mass Modulus (ksi)	Poisson's Ratio	Depth at Midpoint (D_m) ft	Vertical Stress at Midpoint (σ'_{v0mi}) psf
											0	0
1	Soil	0	1	19	32	0	125				0.5	62.5
2	Soil	1	3.8	32	32	0	125				2.4	300
3	Rock	3.8					101.5	12983	1336	0.27	3.8	475

Depth to Water D_w = 3.8 ft



Foundation Characteristics

Embedment Depth:	5 feet or bedrock, whichever is encountered first		D_f =	3.8 ft	
Foundation Width:	B_f =	5.5 ft			
Foundation Length:	L_f =	5.5 ft	L/B =	1	
Foundation Thickness:	t_f =	1 ft	EPRI Rigidity Factors (REF 4)		
Footing Inclination from Horizontal (upward +)	α =	0 deg	β_z =	1.08	10.6.2.4.4-1
			B/2 =	2.75 ft	
Assumed Unit Weight of Concrete	Υ_c =	150 pcf			

Surcharge (σ_{v0}) at Base of the Footing	q =	475 psf
Cohesion of Underlying Soil	c =	0 psf
Unit Weight of Underlying Soil	ϒ =	125 pcf
Coefficient of Passive Pressure	K_p =	3.25
Coefficient of Active Pressure	K_a =	0.31

Calculations for Additional Loading from Foundation Concrete and Surrounding Soil

Assume 30" Column/Pedestal loading (based on minimum shaft diameter, Ref. 5)

Foundation Dimensions:	Foundation Width	B_f =	5.5 ft
	Foundation Length	L_f =	5.5 ft

Dimensions:

Foundation Thickness	t_f =	1 ft	
Unit Weight of Concrete =	Υ_c =	150 pcf	
Pedestal Diameter	D_P =	2.5 ft	*Ref. 5*
Area of Pedestal below Ground	A_P =	4.9 ft^2	
Height of Pedestal Below Ground	H_P =	2.8 ft	

Load from Concrete	P_F =	4538 lb	EQ 14	Volume for Soil Above Footing	V_s =	70.96 ft^3	EQ 17
	P_P =	2062 lb	EQ 15	Unit Weight of Soil =	Υ_s =	125 pcf	
	P_{Tc} =	6599 lb	EQ 16	Load from Soil	P_{Ts} =	8869 lb	EQ 18
		7 kip				9 kip	

(A) Loading Conditions (Ref. 5)

Standard Lighting Factored Design Forces					
Load	**Units**	**Strength I**	**Extreme I**	**Service 1**	**Service II**
Axial, P_1	kip	1.5	1.5	1.5	1.5
Moment, M_2	kip-ft	0.5	17.5	9.5	0.5
Shear, V_1	kip	0	1	0.5	0
Torsion, M_1	kip-ft	0	0.5	0.25	0

Note: LRFD load factors for Service I equal to 1.0

(B) Check Eccentricity and Overturning Based on Moments about Center of Footing

Table 5-6 (REF 3) Allowable Bearing Pressure: q_{all} = 0.5 MPa 10443 psf Value determined for weak schist/gneiss

Factor of Safety for ASD Bearing Capacity FS = 3

Nominal Bearing Pressure q_n= 31328 psf

For Strength 1 case, load factor is applied:

Load Factor Minimum $\Upsilon_{P,\ Concrete}$ = 0.9 $\Upsilon_{P,\ Soil}$ = 1 Per AASHTO Section 3.4.2.1

Load Factor Maximum $\Upsilon_{P,\ Concrete}$ = 1.25 $\Upsilon_{P,\ Soil}$ = 1.35

For Other Cases Υ_P = 1

	EQ 24	EQ 25	EQ 19		EQ 23	
Load Group	$\Upsilon_P P$ (kip)	M (kip-ft)	e_B (ft)	e_L (ft)	Eccentricity CDR_e (B)	Eccentricity CDR_e (L)
Strength I	16.3	0.5	0.03	0.03	OK	OK
Extreme I	17.0	21	1.26	1.26	OK	OK
Service 1	17.0	11.4	0.67	0.67	OK	OK
Service 2	17.0	0.5	0.03	0.03	OK	OK

(1/4)B =	1.38
(1/4)L =	1.38

Per AASHTO 13.7.1 (Ref. 10), the eccentricity at the extreme limit state based on the factored loads shall not exceed 0.25 B or L.

(C) Determine CDR against Bearing Capacity Failure

Load Factor Maximum (soil) $\Upsilon_{P,\ soil}$ = 1.35 Table 3.4.1-2 AASHTO

Load Factor Maximum (concrete) $\Upsilon_{P,\ concrete}$ = 1.25 Table 3.4.1-2 AASHTO

Resistance Factor for Strength ϕ_{tb} = 0.45 Table 10.5.5.2.2-1 AASHTO

Resistance Factor, Extreme, Service I, Service II ϕ_{tb} = 1 Per AASHTO Section 10.5.5.1

	EQ 26		EQ 20	EQ 21	EQ 22	EQ 27	EQ 28
Load Group	Q_R (ksf)	$\Upsilon_{tb} P$ (kip)	B (ft)	L (ft)	e_B (ft)	q_{app} (ksf)	Bearing CDR_b
Strength I	14.1	21.7	5.50	5.50	0.03	0.74	OK
Extreme I	31.3	17.0	5.50	5.50	1.26	1.33	OK
Service 1	31.3	17.0	5.50	5.50	0.67	0.97	OK
Service 2	31.3	17.0	5.50	5.50	0.03	0.58	OK

Note: Vertical load increased by maximum factor for conservative evaluation. Q_R = nominal bearing resistance multipled by resistance factors given in AASHTO.

(D) Determine CDR against Sliding

Load Factor for Sliding Minimum (concrete) Υ_T = 0.9 Table 3.4.1-2 AASHTO

Load Factor for Sliding Minimum (soil) Υ_T = 1 Table 3.4.1-2 AASHTO

Resistance Factor for Shear Between Footing Base and Soil ϕ_T = 0.8 Table 10.5.5.2.2-1 AASHTO

Resistance Factor for Passive Soil ϕ_{ep} = 0.5 Table 10.5.5.2.2-1 AASHTO

Interface Friction Angle for Concrete/Rock ϕ = 35 deg Ref. 3, Table 5-15, mass concrete on clean sound rock

	EQ 29	EQ 30		EQ 31		EQ 32	EQ 33	EQ 34
Load Group	$\gamma_T P$	Q_T	$\phi_T Q_T$	Q_{ep}	$\phi_{ep} Q_{ep}$	Q_{Rs}	Shear, V_T	Sliding CDR_s
	(kip)	(kip)	(kip)	(kip)	(kip)	(kip)	(kip)	
Strength I	16.3	11.42	9.14	0.20	0.10	9.24	0	OK
Extreme I	17.0	11.88	9.51	0.20	0.10	9.61	1	OK
Service 1	15.4	10.80	8.64	0.20	0.10	8.74	0.5	OK
Service 2	15.4	10.80	8.64	0.20	0.10	8.74	0	OK

Note: Shear load increased by maximum load factor for conservative evaluation. Resistance values used for footing interface with soil and passive given in AASHTO.

(E) Check embedment length to resist torsion loading demand at the Extreme I limit state.

5.0Our analysis of spread footings for mast arms demonstrated spread footings with the proposed dimensions are sufficient to resist the Extreme 1 shear load and torsional load. Based on this analysis and by inspection of loads (where torsional loading is less than shear loading), spread footings sized for Extreme I shear load are sufficiently sized to the resist the Extreme I torsional load.

(F) Calculate Settlement at Foundation Location using Service I Loads

Determine Stress Increase at Rock Interface

$q_f = \Delta\sigma_v = q_{app}$ (Service I) = 972 psf Per Calcuation Section B and D , applied vertical stress on eccentric footing area (B' x L) EQ 35

Conservatively assume that entire applied load is considered as change in vertical stress to system

Use the elastic settlement of foundations on broken or jointed rock provided in REF 4 10.6.2.4.4

EQ 36

Layer		Layer Thickness (ft)	σ'_{v0} at layer midpoint (Variable)	(psf)	(kPa)	σ'_{v0} at layer bottom (Variable)	(psf)	(kPa)	N_{60}	N'	C'
Soil	1	1	σ'_{v0m1}	62.5	3.0	σ'_{v0b1}	125	6.0	NA	NA	NA
Soil	2	2.8	σ'_{v0m2}	300	14.4	σ'_{v0b2}	475	22.7	19	38	125
Rock	3	3.8	<- Top Elevation of Layer								

σ'_{v0bf} Initial Stress at Foundation Bottom 475 psf

$$p = q_0(1 - v^2)\frac{rI_p}{144E_m}$$

EQ 37 (REF 4, 10.6.2.4.4-1)

Applied Vertical Stress at Base of Loaded Area	q_0 =	1.447 ksf
Poisson's Ratio	v =	0.27
Influence Area	r or B/2 =	2.75 ft
Foundation Shape and Rigidity Factor	β_z =	1.08
Rigidity Influence Coefficient	I_p =	1.64
Intact Rock Modulus	E_i =	7820 ksi
Rock Mass Modulus	E_m =	192411350 psf
		1336 ksi

Influence area is radius or B/s (circular or square footing), (REF 4 10.6.4.4-1)

$$I_p = \frac{(\sqrt{\pi})}{\beta_z}$$

EQ 38 (REF 4, 10.6.2.4.4-2)

Based on the following parameters used in RocScience RSData using Generalized Hoek Brown Classification to Determine Rock Mass Modulus

GSI =	60	Good to Fair Blocky Quality	
mi =	12	For schists	
D =	0.8	Minimal disturbance	
UCS =	90	MPa	1869552 psf
E_i =	1.13E+09	psf	
v =	0.27		

p (bedrock elastic deformation)	ft	0.0000
	in	0.0004

(G) Conclusions

Foundation eccentricity, overturning resistance, bearing capacity resistance, and sliding resistance are satisfactory for the applied loads using a 5.5 foot x 5.5 foot x 1 foot square foundation. Estimated settlement for the foundation is less than 1/4 inch.

GOLDER MEMBER OF WSP				
SUBJECT:	MaineDOT Freeport Foundation Design - Mast Arm and Light			
Job No.:	21450908	**Prepared:**	MSG	8/5/2021
Location:	WIN 023627.00 Freeport Desert Road	**Checked:**	AH/EDF	8/6/2021
Date:	8/2/2021	**Reviewed:**	CCB	8/11/2021

AASHTO LRFD Shallow Foundation Design - Based on Light Standard 2 (LS2) (Sta. 55+35)

Purpose and Scope

The purpose of this calculation package is design shallow foundations in accordance with American Association of State Highway (AASHTO) Load Resistance Factor Design (LRFD) Bridge Design Specifications and Federal Highway Administration (FHWA) guidance.

Summary of Results and Conclusions

Foundation eccentricity, overturning resistance, bearing capacity resistance, and sliding resistance are satisfactory for the applied loads using a 5 foot x 5 foot x 1 foot square foundation. Estimated settlement for the foundation is less than 1 inch.

References

(1) Fang, HY; Foundation Engineering Handbook - 2nd Edition; 1991
(2) Bowles, JE; Foundation Analysis and Design - 4th Edition; 1988
(3) Federal Highway Administration (FHWA); Geotechnical Engineering Circular (GEC) No. 6 - Shallow Foundations; September 2002.
(4) American Association of State Highway and Transportation Officials (AASHTO); LRFD Bridge Design Specifications - 9th Edition; 2020.
(5) HNTB - Foundation Location Tables and Design Loads Correspondance (Dated June 29, 2021)
(6) National Cooperative Highway Research Program (NCHRP) - LRFD Design and Construction of Shallow Foundations for Highway Bridge Structures - Report 651, 2010
(7) Golder Associates; Mast Arm and Light Design Basis, 2021.
(8) HNTB 98% Plans
(9) Golder SGDR 200-series boring logs
(10) AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals, 1st Ed. 2015 with 2017, 2018, and 2020 Interim Revisions.

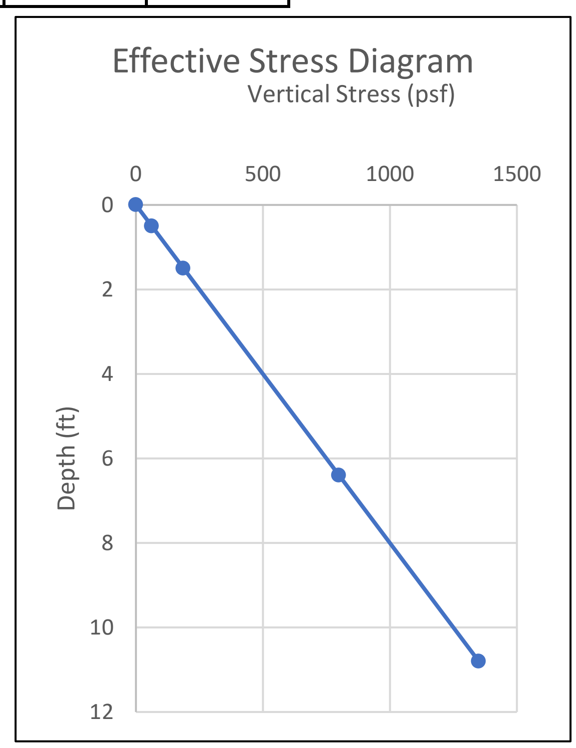
Assumptions

(1) Geotechnical field and laboratory testing results provide appropriate design characterization of subsurface conditions
(2) Loads and pressures provided by HNTB represent critical loading conditions for the project area(s).
(3) Maximum acceptable settlement for the foundation assumed to be 1 inch

Soil Conditions (based on Boring BB-FDR-201, Ref. 9)

Layer (i)	Material Type	Depth Start (D_{ili}) (ft)	Depth End (D_{fli}) (ft)	Average N_{60} value	Friction Angle (ϕ') (deg)	Cohesion (c') (psf)	Effective Unit Weight (Υ') (pcf)	UCS (psi)	Depth at Midpoint (D_m) ft	Vertical Stress at Midpoint (σ'_{v0mi}) psf
									0	0
1	Soil	0	1	11	32	0	125		0.5	62.5
2	Soil	1	2	29	32	0	125		1.5	187.5
3	Soil	2	10.8	32	33	0	125		6.4	800
4	Rock	10.8					101.6	12983	10.8	1350

Depth to Water D_w = 10.8 ft



Foundation Characteristics

Embedment Depth: 5 feet or bedrock, whichever is encountered first D_f = 5 ft
Foundation Width: B_f = 5.0 ft
Foundation Length: L_f = 5.0 ft
Foundation Thickness: t_f = 1 ft
Footing Inclination from Horizontal (upward +) α = 0 deg
Assumed Unit Weight of Concrete Υ_c = 150 pcf

Surcharge (σ_{v0}) at Base of the Footing q = 625 psf
Cohesion of Underlying Soil c = 0 psf
Unit Weight of Underlying Soil Υ = 125 pcf
Coefficient of Passive Pressure K_p = 3.39
Coefficient of Active Pressure K_a = 0.29

Calculations for Additional Loading from Foundation Concrete and Surrounding Soil

Assume 30" Column/Pedestal loading (based on minimum shaft diameter, Ref. 5)

Foundation Dimensions = Width B_f = 5 ft
Length L_f = 5 ft

Thickness t_f = 1 ft

Unit Weight of Concrete = Υ_c = 150 pcf

Pedestal Diameter D_P = 2.5 ft

Area of Pedestal below Ground A_P = 4.9 ft^2

Height of Pedestal Below Ground H_P = 4 ft

Load from Concrete	P_F = 3750	lb	EQ 14	Volume for Soil Above Footing	V_s = 80.37	ft^3	EQ 17
	P_P = 2945	lb	EQ 15	Unit Weight of Soil =	Υ_s = 125	pcf	
	P_{Tc} = 6695	lb	EQ 16	Load from Soil	P_{Ts} = 10046	lb	EQ 18
	7	kip			10	kip	

(A) Calculate Ultimate Bearing Capacity

Bearing Capacity Factors	*Shape Factors*	*Base Inclination Correction Factors*	*Groundwater Location Corrections*	Depth Correction Factors
EQ 1, 2, 3	EQ 4, 5, 6	EQ 10, 11, 12	EQ 8, 9	EQ 7
N_c = 38.6	s_c = 1.68	b_c = 1.00		
N_q = 26.1	s_q = 1.65	b_q = 1.00	C_{wq} = 0.932	d_q = 1.21
N_Υ = 35.2	s_Υ = 0.60	b_Υ = 1.00	$C_{w\Upsilon}$ = 1	

$q_{u,centric}$ = 36980 psf EQ 13

(B) Loading Conditions (Ref. 5)

Standard Lighting Factored Design Forces					
Load	**Units**	**Strength I**	**Extreme I**	**Service 1**	**Service II**
Axial, P_1	kip	1.5	1.5	1.5	1.5
Moment, M_2	kip-ft	0.5	17.5	9.5	0.5
Shear, V_1	kip	0	1	0.5	0
Torsion, M_1	kip-ft	0	0.5	0.25	0

Note: LRFD load factors for Service I equal to 1.0

(C) Check Eccentricity and Overturning Based on Moments about Center of Footing

For Strength 1 case, load factor is applied:

Load Factor Minimum $\Upsilon_{P, Concrete}$ = 0.9 $\Upsilon_{P, Soil}$ = 1 Per AASHTO Section 3.4.2.1

Load Factor Maximum $\Upsilon_{P, Concrete}$ = 1.25 $\Upsilon_{P, Soil}$ = 1.35

For Other Cases Υ_P = 1

	EQ 24	EQ 25	EQ 19		EQ 23	
Load Group	$\Upsilon_P P$ (kip)	M (kip-ft)	e_B (ft)	e_L (ft)	Eccentricity CDR_e (B)	Eccentricity CDR_e (L)
Strength I	17.6	0.5	0.03	0.03	OK	OK
Extreme I	18.2	23	1.23	1.23	OK	OK
Service 1	18.2	12	0.66	0.66	OK	OK
Service 2	18.2	0.5	0.03	0.03	OK	OK

(1/4)B =	1.25
(1/4)L =	1.25

Per AASHTO 13.7.1 (Ref. 10), the eccentricity at the extreme limit state based on the factored loads shall not exceed 0.25 B or L.

(D) Determine CDR against Bearing Capacity Failure

Load Factor Maximum (soil)	$\Upsilon_{P, soil}$ = 1.35	Table 3.4.1-2 AASHTO
Load Factor Maximum (concrete)	$\Upsilon_{P, concrete}$ = 1.25	Table 3.4.1-2 AASHTO
Resistance Factor for Strength	ϕ_{tb} = 0.45	Table 10.5.5.2.2-1 AASHTO
Resistance Factor, Extreme, Service I, Service II	ϕ_{tb} = 1	Per AASHTO Section 10.5.5.1

	EQ 26		EQ 20	EQ 21	EQ 22	EQ 27	EQ 28
Load Group	Q_R (ksf)	$\Upsilon_{tb}P$ (kip)	B' (ft)	L (ft)	A' (ft^2)	q_{app} (ksf)	Bearing CDR_b
Strength I	16.64	23.4	4.94	5.0	24.7	0.95	OK
Extreme I	36.98	18.2	2.53	5.0	12.7	1.44	OK
Service 1	36.98	18.2	3.68	5.0	18.4	0.99	OK
Service 2	36.98	18.2	4.95	5.0	24.7	0.74	OK

Note: Vertical load increased by maximum factor for conservative evaluation. Q_R = nominal bearing resistance multipled by resistance factors given in AASHTO.

(E) Check embedment length to resist torsion loading demand at the Extreme I limit state.

5.0ur analysis of spread footings for mast arms demonstrated spread footings with the proposed dimensions are sufficient to resist the Extreme 1 shear load and torsional load. Based on this analysis and by inspection of loads (where torsional loading is less than shear loading), spread footings sized for Extreme I shear load are sufficiently sized to the resist the Extreme I torsional load.

(F) Determine CDR against Sliding

Load Factor for Sliding Minimum (concrete)	Υ_T = 0.9		Table 3.4.1-2 AASHTO
Load Factor for Sliding Minimum (soil)	Υ_T = 1		Table 3.4.1-2 AASHTO
Resistance Factor for Shear Between Footing Base and Soil	ϕ_T = 0.8		Table 10.5.5.2.2-1 AASHTO
Resistance Factor for Passive Soil	ϕ_{ep} = 0.5		Table 10.5.5.2.2-1 AASHTO
Interface Friction Angle for Concrete/Soil	ϕ = 24	deg	Ref. 3, Table 5-15, mass concrete on silty medium to coarse sand

	EQ 29	EQ 30		EQ 31		EQ 32	EQ 33	EQ 34
Load Group	$\Upsilon_T P$	Q_T	$\phi_T Q_T$	Q_{ep}	$\phi_{ep} Q_{ep}$	Q_{Rs}	Shear, V_T	Sliding CDR_s
	(kip)	(kip)	(kip)	(kip)	(kip)	(kip)	(kip)	
Strength I	17.6	7.82	6.26	0.21	0.11	6.36	0	OK
Extreme I	18.2	8.12	6.50	0.21	0.11	6.60	1	OK
Service 1	18.2	8.12	6.50	0.21	0.11	6.60	0.5	OK
Service 2	18.2	8.12	6.50	0.21	0.11	6.60	0	OK

Note: Shear load increased by maximum factor for conservative evaluation. Resistance values used for footing interface with soil and passive given in AASHTO.

(G) Calculate Settlement at Foundation Location using Service I Loads

Determine stress increase at Layer 3 midpoint

$q_f = \Delta\sigma_v = q_{app}$ (Service I) = 990 psf Per Calcuation Section B and D , applied vertical stress on eccentric footing area (B' x L) EQ 35

Conservatively assume that entire applied load is considered as change in vertical stress to system

Use the Hough method to estimate settlement of the Soil layer (Layer 3)

Hough Method:

Determine σ'_{v0} and C' at layer midpoints for in situ existing soil. Use Ref. 3 Figures 5-18 and 5-19 to obtain corrected N' and C' for Layer 3, assuming the Sand/Gravel matches "Well graded silty SAND & GRAVEL".

EQ 36

Layer		Layer Thickness (ft)	σ'_{v0} at layer midpoint (Variable)	(psf)	(kPa)	σ'_{v0} at layer bottom (Variable)	(psf)	(kPa)	N_{60}	N'	C'
Soil	1	1	σ'_{v0m1}	62.5	3.0	σ'_{v0b1}	125	6.0	NA	NA	NA
	2	1	σ'_{v0m2}	187.5	9.0	σ'_{v0b2}	250	12.0	NA	NA	NA
	3	8.8	σ'_{v0m3}	800	38.3	σ'_{v0b3}	1350	64.6	15	24	85

σ'_{v0bf} Initial Stress at Foundation Bottom Stress 625 psf

Hough Method General Equation:

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

(REF 3. Eqn 5-24) EQ 37

where:

ΔH_i	settlement in each layer, ft	
H_c	initial height of layer i, ft	5.8 ft
C'	bearing capacity index from Ref. 12, Figure 5-19	85
σ'_{v0}	initial stress, ksf	0.800 ksf
$\Delta\sigma_v$	vertical stress increase, ksf	0.990 ksf

ΔH_i (Layer 3)	ft	0.04
	in	0.43

(H) Conclusions

Foundation eccentricity, overturning resistance, bearing capacity resistance, and sliding resistance are satisfactory for the applied loads using a 5 foot x 5 foot x 1 foot square foundation. Estimated settlement for the foundation is less than 1 inch.

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