July 1, 2022
Kleinfelder Project No.: 20193552.001A

Mr. Andrew Lathe, Project Manager
Bridge Program
Maine Department of Transportation
State House Station 16
Augusta, ME 04333

RE: **Final Design Geotechnical Report**
Superstructure Replacement of Mill Hill Bridge on the Deer Isle-Stonington Town Line (Bridge No.3063) WIN 022356.00
Deer Isle, Maine

Dear Mr. Lathe:

This report presents the results of our geotechnical evaluation for the proposed superstructure replacement for the Mill Hill Bridge in Deer Isle and Stonington, Maine. This final report includes our recommendation relating to the geotechnical aspects of the project design and supersedes our reported dated May 17, 2021. Our scope of work included a subsurface exploration, laboratory testing, engineering analysis, and preparation of this report. The site is suitable for the proposed development provided that the conclusions and recommendations presented in this report are incorporated into the design and construction of the project.

We appreciate the opportunity to provide geotechnical engineering services to you on this project. If you have any questions regarding this report or if we can be of further service, please to not hesitate to contact the undersigned at (617) 497-7800.

Respectfully submitted,

KLEINFELDER, INC.

Massimiliano Rolandi

Massimiliano (Max) Rolandi, P.E.
Principal Geotechnical Engineer

Attachment

KLEINFELDER
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Final Design Geotechnical Report

For:

Superstructure Replacement of Mill Hill Bridge on the Deer Isle-Stonington Town Line (Bridge No. 3063)
Deer Isle and Stonington, Maine

Prepared for:

Maine Department of Transportation
Bridge Program
State House Station 16
Augusta, ME 04333

Prepared by:

Malinda Chea

Malinda Chea, Staff Professional
Geotechnical Engineer

Reviewed by:

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Massimiliano Rolandi, PE
Principal Geotechnical Engineer

July 1, 2022

Table of Contents

Section Page

TABLES

APPENDICES

1 INTRODUCTION

1.1 General

Bridge No. 3063, also called the Mill Hill Bridge, carries Route 15 over the Mill Pond outlet at the Deer Isle-Stonington town line. The Mill Hill Bridge was constructed in 1939 and is a single-span T-Girder bridge supported on granite block abutments with concrete caps.

The Maine Department of Transportation (MaineDOT) is currently considering the rehabilitation of the bridge superstructure or a complete replacement of the bridge. We understand that the MaineDOT preference is to replace the bridge superstructure and reuse the existing abutments. Construction at the Mill Hill Bridge on the Deer Isle-Stonington town line is programmed for the spring/summer of 2023.

Kleinfelder conducted geotechnical explorations and analyses to evaluate the suitability of reusing the existing abutments to support the new superstructure, and to provide geotechnical recommendations to be used for design and construction. The purpose of this Final Design Geotechnical Report is to summarize our findings and geotechnical recommendations for the project.

Conclusions and recommendations presented in this report are based on our understanding of proposed site improvements as presented below, subsurface conditions encountered at locations of our explorations, and our engineering analysis based on field and laboratory data. Recommendations presented herein should not be extrapolated to other areas or used for other projects. An information sheet prepared by GBA (the Geoprofessionals Business Association, Inc.) is included in Appendix F. We recommend that individuals using this report read the limitations along with the referenced GBA document.

1.2 Site and Project Description

The Mill Hill Bridge on the Deer Isle-Stonington town line is a single-span bridge which runs in a general east-west direction and carries Route 15 over the tidal outlet to Mill Pond. The bridge site is subjected to strong tidal flows in both directions. The bridge was built in 1938 and improvements have been made over the years which include a bridge widening with replacement of the curb and rail with a 'Maine rail' concrete bridge rail system in 2004. A copy of the 1932/1938 Drawings is included in Appendix A.

The roadway on the bridge has two opposing 11-foot-wide traffic lanes with variable width gravel shoulders. The top of the bridge slopes from about El. 17 on the south to about El. 15 on the north. The superstructure consists of reinforced concrete T-beams, supported on two abutments which consist of stacked granite blocks. The approximate clear span between the abutments is 40 feet. The north abutment (referred to as 'No.1 abutment' on 1932 Drawings) has a total height extending from the top of the abutment to the base of the abutment foundation of 16.21 feet. The south abutment (referred to as 'No. 2 abutment') has a total height extending from the top of the abutment to base of the abutment foundation of 16.91 feet. The abutment No. 1 and 2 foundation width is shown to be about 10.4 feet and 10.9 feet, respectively.

Based on the 1938-1939 construction field book, No. 1 abutment wingwalls bear on bedrock, but the bearing conditions below the abutment are not indicated; and No.2 abutment bears on bedrock.

The following superstructure loads were provided by Kleinfelder's structural engineers for the existing and proposed bridge. We understand that the eccentricity of loading is about B/3 for the proposed superstructure replacement. The live load reactions are expected to remain unchanged following superstructure replacement. At the time of this report, no significant grade changes are planned as part of the proposed construction.

TABLE 1-1 **- SUMMARY OF STRUCTURAL LOADING INFORMATION**

Analysis	Load Combination	Existing	Proposed
Bearing	Strength I Bearing Pressure	369 kip	332 kip
	Service I Bearing Pressure	241	217 kip
Settlement	Increase in Service I Bearing Pressure	NA	-24 kip

The proposed project includes the replacement of the existing bridge superstructure and the partial reuse of the existing bridge abutments. The existing bridge abutment walls will be demolished to approximately El. 6 and will be replaced with cast-in-place concrete walls, founded on top of the existing granite blocks present below El.6.

New return retaining walls are proposed on the side of the abutment walls. The return retaining walls will be perpendicular the abutment walls and will tie into the proposed abutment wall replacement. The new retaining wall will have a base width of 8 feet, a stem width of approximately 1.65 feet and a footing thickness of approximately 1.75 feet.

1.3 Datum

The elevations referenced in the 1932 Drawings do not indicate a datum. Based upon the recently completed survey, the elevations provided on the 1932 drawings are about 83 feet higher than the North American Vertical Datum of 1988 (NAVD88). Elevations referenced in this report are based on NAVD88.

1.4 Project Authorization

Kleinfelder has performed geotechnical engineering services in accordance with our proposal dated August 20, 2021. Our work on this project was authorized by the agreement between MaineDOT and Kleinfelder dated September 10, 2021.

2 GEOLOGIC SETTING

The Maine Geologic Survey (MGS) Surficial Geology of the Deer Isle Quadrangle, Maine, open file no. 74-12 (1974) indicates the surficial soils in the vicinity of the project site consists of till and bedrock outcrops. Till within this area reportedly consists of a mixture of sand, silt, clay, and stones. The till can be either a basal till which is very compact and fine-grained; or an ablation till which is loose, sandy, and stony. The till generally overlies bedrock but may overlie or include sand and gravel.

The Maine Geologic Survey (MGS) Bedrock Geologic Map of Maine (1985) indicates that bedrock within the vicinity of the area consist of Devonian granite which is part of the Castine Formation.

3 FIELD EXPLORATION

A subsurface exploration program consisting of four (4) auger probes and two (2) borings was carried out between May 16 and 17th, 2019. The probes were advanced to assess the abutment geometry. The borings were advanced through the bridge abutments to assess the bearing conditions below the granite block foundations:

- Probes BB-SDIHP-101 and BB-SDIHP-102 were advanced behind Abutment No. 2.
- Boring BB-SDIHP-103 was advanced through Abutment No. 2.
- Probes BB-SDIHP-104 and BB-SDIHP-105 were advanced behind Abutment No. 1.
- Boring BB-SDIHP-106 was advanced through Abutment No. 1.

The approximate exploration locations are shown on Sheet B-1 in Appendix B. The borings and probes were laid out by a Kleinfelder geo-professional by taping distances from existing site features. The as-drilled locations shown on the plans should be considered accurate only to the degree implied by the method used. Ground surface elevations were estimated from topographic information provided by Maine DOT.

New England Boring Contractors of Hermon, Maine advanced the probes and borings with a mobile truck-mounted drill rig. The probes were advanced with a solid stem auger. The borings were advanced through the existing abutments using rock coring drilling methods as indicated on the boring logs. Boring BB-SDIHP-103 was advanced to a depth of about 30.4 feet, and BB-SDIHP-106 was advanced to a depth of about 34.2 feet below existing grade. Standard Penetration Tests (SPT) were not performed. A HQ-sized rock core barrel was used when coring through the abutment and rock. Upon completion, the probes were backfilled with the cuttings and the borings were backfilled with grout; the ground surface was restored with asphalt cold patch at each location.

A Kleinfelder geo-professional provided technical direction and observation during the explorations, maintained boring logs, and described the retrieved soil and bedrock samples in general accordance with the MaineDOT Geotechnical Section Key to Soil and Rock Descriptions and Terms dated January 2022. Descriptions of the rock encountered in the explorations are included in the boring logs provided in Appendix C. A copy of the MaineDOT Geotechnical Section Key to Soil and Rock Descriptions and Terms are also provided in Appendix C.

4 LABORATORY TESTING

Laboratory testing was performed on representative samples to substantiate field classifications and provide engineering parameters for geotechnical design. Laboratory testing consisted of one (1) Bulk Density and Compressive Strength of Rock Core Specimens by ASTM D7012 Method C. The selected sample was typical of the rock cores retrieved. The tests were performed by GeoTesting Express, Inc. of Acton, Massachusetts. The laboratory test report is included in Appendix D.

5 SUBSURFACE CONDITIONS

Subsurface conditions encountered during Kleinfelder's field exploration program are described below, in general order of occurrence. The subsurface descriptions in this report are based on a limited number of explorations. Variations may occur and should be expected between boring locations. The strata boundaries shown in our boring logs are based on our interpretations and the actual transition may be gradual. Refer to the boring logs in Appendix B for detailed descriptions of the core samples collected.

5.1 Abutment No. 1 (North)

Boring BB-SDIHP-106 was advanced through the north abutment location to a total depth of 34.2 feet. **Table 5-1** below summarizes the subsurface conditions encountered in this boring.

TABLE 5-1- SUMMARY OF THE SUBSURFACE CONDITIONS AT NO.1 ABUTMENT

Material	Top Depth (ft. bgs)	Thickness (ft.)	Description
Bituminous Pavement	0	0.8	
Abutment (Concrete)	0.8	6	Concrete
Abutment (Granite Block)	6.8	11.6	Light gray granite slabs with black and white mottling
Leveling Slab (Concrete)	18.4	3.4	Concrete
Bedrock	21.8	-	Pink Granite with black mottles, fine-grained, hard, slightly weathered, and slightly to highly fractured

As noted above, the 1938 construction field book indicates that bedrock was not exposed at the abutment subgrade. Based on the conditions encountered in boring BB-SDIHP-106, we assume the gravel was removed and replaced with concrete. Probes BB-SDIHP-104 and BB-SDIHP-105 were advanced behind the Abutment No. 1 and extended to a depth of about 2.5 feet and 7.3 feet below ground surface (bgs), respectively. Refusal was interpreted to be on the back of the abutment.

Based on the boring, probes, and survey, Kleinfelder estimated that the Abutment No. 1 bears on a concrete leveling slab at El. -3.2 and has a bottom width of 10.4 feet.

5.2 Abutment No. 2 (South)

Boring BB-SDIHP-103 was advanced at the south abutment location to a total depth of 30.4 feet. **Table 5-2** below summarizes the subsurface conditions encountered in this boring.

TABLE 5-2 - SUMMARY OF THE SUBSURFACE CONDITIONS AT NO.2 ABUTMENT

Material	Top Depth (ft. bgs)	Thickness (ft.)	Description
Bituminous Pavement	0	0.6	-
Abutment (Concrete)	0.6	7	Concrete
Abutment (Granite Block)	7.6	10.9	Light gray granite slabs with black and white mottling
Leveling Slab (Concrete)	18.5	3.2	Concrete
Bedrock	21.7	-	Pink Granite with black mottles, fine-grained, hard, slightly weathered, and slightly to highly fractured

Probes BB-SDIHP-101 and BB-SDIHP-102 were advanced behind the Abutment No. 2 and extended to a depth of about 5.4 feet and 7.0 feet below ground surface, respectively. Refusal was interpreted to be on the back of the abutment.

Based on the boring, probes, and survey, Kleinfelder estimated that the Abutment No. 2 bears on concrete leveling slab at El. -2.0 and has a bottom width of 10.96 feet.

5.3 Groundwater

The bridge is located over the tidal outlet to Mill Pond. As such, groundwater levels will be significantly impacted by the tide levels. The Preliminary Design Hydrology, Hydraulics, and Scour Report prepared by Northstar Hydro Inc of Winthrop, ME, dated September 6, 2019, includes the following tide information based on the NOAA Tide Station, Oceanville, #8414249, Harmonic Station.

TABLE 5-3 - TIDE ELEVATIONS

Tide Level	Elevation
Highest Predicted Tide, 2019	6.49
Mean High-High Water (MHHW)	5.02
Mean High Water (MHW)	4.6

Due to the introduction of water into the borings, the water level measurements in the borings (11 feet bgs in boring BB-SDIHP-103 and 10.7 feet bgs in boring BB-SDIHP-106) are not considered reflective of groundwater conditions. Groundwater levels are known to fluctuate due to local and regional factors including, but not limited to, tide cycles, precipitation and snow events, site topography, and seasonal changes.

6 GEOTECHNICAL ASSESSMENT AND RECOMMENDATIONS

Our geotechnical assessment of the existing bridge abutments is based on Load and Resistance Factor Design (LRFD) methodology. These recommendations were developed in general accordance with the AASHTO LRFD Bridge Design Specifications, 8th Edition, 2018 (herein also referred to as AASHTO LRFD), the AASHTO Guide Specifications for LRFD Seismic Bridge Design– 2nd Edition, 2011 (AASHTO LRFD Seismic) and the MaineDOT Bridge Design Guide (2003 with 2018 updates). Supporting calculations are provided in Appendix D. The calculations are based on the geometry of the abutments described herein.

6.1 Strength Limit State

6.1.1 Bearing Resistance - Existing Abutment Walls

The borings indicate that the existing stacked granite abutments bear on concrete leveling pads placed on bedrock. Therefore, the nominal bearing resistance was estimated by treating the rock mass as a continuum using the Hoek Brown Failure criterion. Duncan Wyllie, in his book "Foundations on Rock";1999, provides a detailed derivation of bearing capacity of a homogeneous rock mass based on the Hoek Brown Failure Criterion (Hoek, 1983) utilizing the estimated rock mass strength and geologic strength index (GSI). **Error! Reference source not found.** provides a summary of the strength and extreme limit state bearing resistance for the conditions analyzed. Per AASHTO 8th edition, the following resistance factors have been applied:

- 1.0 for the Service Limit per Section 10.5.5.1,
- 0.45 for the Strength Limit per Table 10.5.5.2.2-1, and
- 1.0 for the Extreme Limit per Section 10.5.5.3.3.

TABLE 6-1 - FACTORED BEARING RESISTANCE

Rock Type	Rock Strength (psi)	GSI	Factored Bearing Resistance (ksf)			Elastic Modulus (ksi)
			Service Limit	Strength Limit	Extreme Limit	
Granite	9,245	50	70	430	960	765

Notes for Tab 6-1:

1. Rock strength from geotechnical laboratory testing on rock core sample recovered during drilling
2. Service Limit State factored BR based on Table C10.6.2.6.1-1 of AASHTO LRFD.

6.1.2 Bearing Resistance - Return Retaining Walls

Kleinfelder performed an evaluation of the net factored bearing resistance of the proposed return retaining walls using the semi-empirical procedures as described in Section 10.6.3.1.3 of the AASHTO LRFD. The factored bearing resistance of the proposed retaining wall footings bearing on gravel borrow backfill, as a function of the base width and footing eccentricity are summarized in **Table 6-2**.

TABLE 6-2 – SUMMARY OF FACTORED BEARING RESISTANCE BASED ON FOOTING WIDTH (B=7FT TO B=9FT) AND ASSUMED ECCENTRICITY (E=B/3 TO E=B/12)

Eccentricity (B - ft)	Factored BR as a function of the footing base (B) width (ksf)		
	B=7 ft	B=8 ft	B=9 ft
B/3	1.4	1.6	1.7
B/6	2.8	3.2	3.5
B/9	3.3	3.7	4.1
B/12	3.6	4.0	4.4

Notes for Tab 6-2:

1. B/3 - the maximum allowed eccentricity per AASHTO LRFD
2. Resistance factor of 0.45 - Table 5-2 of the MaineDOT Bridge Design Guide

Once a value of the factored bearing resistance is selected, the structural engineer should perform the required overturning check and verify that the eccentricity is less than or equal to the corresponding value of the eccentricity assumed in the calculations (for example, if a value of 4.4 ksf is selected for B=9, the eccentricity must be up to a maximum of 9/12=0.75 feet).
The calculated factored bearing resistance values account for a resistance factor equal to 0.45 in accordance with Table 5-2 of the MaineDOT Bridge Design Guide.

6.1.3 Sliding and Overturning

The sliding and overturning stability analyses at the strength limit state of the existing abutment walls and of the proposed return retaining walls should be performed using the parameters provided in **Table 6-3**. These values are based on Table 3-3 of the MaineDOT Bridge Design Guide, assuming gravel borrow backfill of the existing and proposed walls.

TABLE 6-3 – LATERAL EARTH PRESSURES PARAMETERS FOR ANALYSIS OF EXISTING ABUTMENTS

Parameter	Value
Backfill Angle of Internal Friction, ϕ	36 degrees
Backfill Total Unit Weight, γ	135 pcf
Batter of existing abutment walls	67.4 degrees
Friction Factor Backfill/Wall	0.50
Friction Factor Bedrock/Lev. Pad	0.70
Friction Factor Lev. Pad/Granite Blocks	0.55
Design Groundwater Elevation	5 feet
Backfill Slope Proposed Retaining Walls	5 degrees
Active Earth Pressure Coefficient, K_A	0.26
At-Rest Earth Pressure Coefficient, K_0	0.41
Passive Earth Pressure Coefficient, K_P	5.60

Notes for Table 6.3:

1. The active earth pressure coefficient is based on Rankine theory as described in section 3.6.5.2 of the MaineDOT Bridge Design Guide, with a slope of backfill of 5 degrees.

2. The passive earth pressure coefficient is based on Coulomb theory as described in section 3.6.5.2 of the MaineDOT Bridge Design Guide, with a slope of backfill of 5 degrees and interface friction angle of 27 degrees.
3. The passive EP coefficient is provided for a base width of 9 feet

The granite abutments are constructed on a concrete leveling pad placed on the granite bedrock. A friction factor 0.70 should be assumed at the bedrock to concrete leveling slab, and 0.55 should be assumed at the concrete leveling slab-to-granite abutment interface based on AASHTO LRFD Table C3.11.5.3-1. A resistance factor of 0.8 should be used for cast in place concrete on soil and for cast in place concrete on rock (leveling pad on bedrock), in accordance with Table 5-3 of the MaineDOT Bridge Design Guide.

Per Section 5.3.9 of the MaineDOT Bridge Design Guide, the location of the resultant force shall be within the middle two-thirds of the base for foundations bearing on soil (proposed return retaining walls) and within the middle nine-tenths of the base for foundations bearing on rock (existing abutment walls).

6.2 Service Limit State

6.2.1 Settlement

The existing bridge was constructed in 1938. The existing abutment walls have undergone settlement due to the existing bridge loads. As indicated in Error! Reference source not found., the proposed service bearing pressure is less than the existing pressure and therefore additional settlement of the existing abutment walls is not anticipated.

The settlement of the proposed return retaining walls was calculated using the modified Hough method, assuming a footing base width of 7 to 9 feet for footing pressure up to the recommended bearing resistance as presented in Table 6-3. Settlement of the proposed retaining wall is summarized in **Table 6-4**.

TABLE 6-4 – SUMMARY OF SETTLEMENT BASED ON APPLIED FOOTING PRESSURE FOR B=7FT TO B=9FT

B=7 feet					
Footing Pressure (ksf)	1.5	2	2.5	3	3.6
Settlement (in)	0.4	0.5	0.55	0.6	0.7

B=8 feet					
Footing Pressure (ksf)	2	2.5	3	3.5	4
Settlement (in)	0.5	0.6	0.65	0.7	0.8

B=9 feet					
Footing Pressure (ksf)	2.5	3	3.5	4	4.4
Settlement (in)	0.6	0.65	0.7	0.8	0.85

Notes for Table 6-4:

1) Settlement evaluated for a maximum factored bearing resistance assuming an eccentricity of B/12.

6.2.2 Service Limit State Global Stability

The existing abutments bear on competent bedrock and there is a net decrease in the proposed bearing pressure of the new superstructure. Therefore, global stability analysis for the existing abutments was not performed.

A service limit state global stability analysis was performed for the proposed return retaining walls assuming gravel borrow backfill material and slope and wall geometry as shown in the attached figures. The factor of safety for global stability at the service limit state resulted to be above 1.5.

6.3 Extreme Event Limit State

6.3.1 Seismic Site Class and Seismic Design Parameters

The recommended seismic design parameters for the site have been evaluated in accordance with the AASHTO Guide Specifications for LRFD Seismic Bridge Design– 2nd Edition, 2011, based on the data obtained from the recent explorations at the project site.

The following seismic parameters are recommended for design:

TABLE 6-5 – SEISMIC DESIGN PARAMETERS

Parameter	Value
Site Class	B
Peak ground acceleration, PGA	0.055g
Short-period spectral acceleration, S_s	0.120g
Long-period spectral acceleration, S_1	0.038g
Spectral acceleration coefficient, A_S ($A_s = F_{pga}$*PGA)	0.055g
Short period acceleration coefficient, S_{DS} ($S_{DS} = F_a$*S_s)	0.120g
1-sec period acceleration coefficient, S_{D1} ($S_{D1} = F_v$*S_1)	0.038g
Seismic Design Category	A

Notes for Table 6-5

1. Coefficients Fpga, Fa and Fv all equal to 1 based on Table 3.4.2.3-1 of AASHTO LRFD Seismic.

A Mononobe-Okabe seismic active earth pressure coefficient of 0.30 and a seismic passive earth pressure coefficient of 3.5 can be used in extreme event limit state seismic stability analyses as appropriate.

6.3.2 Extreme Event Limit State Global Stability

An extreme event limit state global stability analysis was performed for the proposed return retaining walls assuming gravel borrow backfill material, a horizontal seismic coefficient (k_h=A_s) of 0.055, equal to the peak ground acceleration for site class B, and slope and wall geometry as shown in the attached figures. The factor of safety for global stability at the service limit state resulted to be well above 1.0.

7 CONSTRUCTION CONSIDERATIONS

We understand only minor changes in grade are planned as part of the proposed construction. We understand that construction may include removal and replacement of up to 10 feet of the existing abutments to facilitate widening the abutment to support the wider roadway and construction of approach slabs. The following sections discuss this aspect of construction. Construction site safety is the sole responsibility of the contractor, who shall also be solely responsible for the means, methods, and sequencing of construction operations.

7.1 General

The top of the bridge slopes from about El. 17 on the south to about El. 15. The profile in Figure 3 shows that the excavations to replace the upper portion of the abutments will extend to about El. 6.2. Based on the tide elevations in **Table 5-3**, the excavations will terminate above the Mean High-High Water (MHHW) but below the Highest predicted tide, 2019. Therefore, groundwater control may be required.

7.2 Site Clearing and Pavement Subgrade Preparation

Following removal of the fill and abutment to the proposed depths, the undercutting should be extended through loose layers, if necessary, to provide a suitable subgrade for placement and compaction of new fill.

The exposed subgrades should be proof rolled to a firm and unyielding condition with a minimum 10-ton smooth-drum vibratory roller except within 5 feet behind the abutment where hand operated equipment should be used. One of the objectives of proof-rolling is to identify zones of weakness in the subgrade. Areas of observed or suspected instability should be further explored under the observation of a representative of the Geotechnical Engineer by methods that may include test pits or laboratory testing, to identify causes and/or extents of instability. Soils which exhibit excessive rutting, pumping, or instability, as determined by a representative of the Geotechnical Engineer, should be stabilized. Stabilization techniques may include, but are not limited to, scarifying, moisture conditioning, and re-compacting loose/soft soils or removing loose/soft soils and replacing them with approved, compacted fill. The development areas should be firm and stable as observed by a representative of the Geotechnical Engineer prior to placement of backfill or construction of structural elements. The pavement areas should be stable, as determined by a representative of the Geotechnical Engineer, prior to pavement construction. The granular base thickness below pavements should be uniform and the pavement subgrade should be graded to provide positive drainage of the granular base section.

The performance of pavements will be dependent upon a number of factors, including subgrade conditions at the time of paving, drainage considerations including management of rainwater runoff, and traffic type and frequency. Stormwater should not be allowed to seep below pavements from adjacent areas. Proper drainage below the pavement section helps prevent softening of the subgrade and has a significant impact on pavement performance and pavement life.

7.3 Fill Recommendations

The aggregate subbase course should meet the property and gradation requirements for Aggregate Subbase Course Type D as provided in MaineDOT Standard Specifications section 703.06.

Backfill for walls and to replace material excavated from behind abutments should meet the property and gradation requirements for Granular Borrow - Material for Underwater Backfill as provided in MaineDOT Standard Specifications section 703.19.

Proposed import fill should be approved by the Geotechnical Engineer prior to delivery to the site. A sample should be obtained from each on-site or import source prior to use. If, at any time, the representative of the Geotechnical Engineer observes a change in material type of the fill soils, additional testing should be performed. A grain-size analysis (AASHTO T-088) should be performed on material desired for use as fill and backfill. If the gradation of the soil is acceptable, a modified proctor compaction test (AASHTO T-180) should be performed to determine the range of acceptable moisture contents and maximum dry density.

Fills should be placed on a stable, near level subgrade prepared in accordance with the recommendations above and approved by a representative of the Geotechnical Engineer. The contractor should not place subbase materials on subgrade surfaces that are muddy, frozen, or contain pooled water, frost, or ice. Frozen soils or soils that contain frost or ice are not suitable fill sources.

Fills should be placed in lifts not exceeding a loose thickness of 12 inches where ride-on compaction equipment can be used or 8 inches where hand-operated compaction equipment is used.

Quality control observation and testing of in-place fill densities should be performed throughout construction. In addition, fills should be stable without significant movement under construction equipment as judged by a representative of the Geotechnical Engineer. However, conformance with the drawings and specifications remains the contractor's responsibility. Fills should be compacted to 95 percent of maximum dry density as determined by AASHTO T-180. Moisture should be controlled to within 2 percent of optimum during compaction.

7.4 Temporary Excavations

As described above, excavations up to 11 feet below road grade are anticipated. If the roadway is closed during construction, open excavations may be feasible. However, if one lane of traffic must be maintained, temporary excavation support systems will be required.

Slopes of open cuts should be determined by the contractor. Where groundwater seepage is present, slope inclinations will likely need to be flatter, and/or excavations shored. Stockpiled (excavated) materials should be placed no closer than a distance equal to the depth of the excavation or 4 feet from the edge of an excavation, whichever is greater.

Recommended temporary excavation support systems include internally braced sheetpiles, soldier pile and lagging with soldier piles installed with rock sockets, or modular boxes. The design of a temporary lateral earth support system should consider appropriate construction and traffic surcharges including, but not limited to surcharges from vehicle traffic and construction equipment. Lateral pressures acting on the excavation support system should be calculated based on each stage (cantilevered, braced) of the excavation. Bracing or anchoring of the system should be used as needed to keep lateral movement of the excavation support system within tolerable limits. The designer should consider the lateral deflection of the support system at each stage of construction, including initial cantilever and during removal of bracing.

The contractor should prepare a workplan for the excavations that details the means, methods, and sequencing of work to be performed and presents a mitigation plan in the event tolerable movements or vibrations are exceeded. The contractor should also prepare and submit for review a temporary excavation support/shoring design signed and sealed by a Professional Engineer registered in Maine.

All excavations must comply with applicable local, state, and federal safety regulations. The responsibility for excavation safety and stability of temporary construction slopes lies solely with the contractor. We are providing this information below solely as a service to our client for preliminary planning and cost estimating. Under no circumstances should this information provided be interpreted to mean that Kleinfelder is assuming responsibility for final engineering of excavations or shoring, construction site safety, or the contractors' activities; such responsibility is not being implied and should not be inferred.

7.5 Water Control Considerations

As described above, excavations may extend below the Highest predicted tide. Therefore, groundwater control may be necessary during the course of the work.

Groundwater should be controlled so as to complete excavations, subgrade preparation, and foundation construction in dry conditions and to maintain the integrity of the existing soil deposits and bearing surfaces. It is essential to dewater prior to excavating below groundwater level. Construction dewatering may require a combination of wells, well points, and/or open pumping with collector sumps and trenches. The dewatering system should be designed by an experienced specialty contractor and should be designed and operated to prevent pumping of soil, disturbance of subgrades, and adverse effects to existing site features. The dewatering design should include an evaluation of settlement based on the proposed system to assess the influence on the site and surrounding features.

Foundations should be constructed in excavations immediately after foundation subgrades are approved by the on-site representative of the Geotechnical Engineer. Water entering foundation excavations should be removed and the subgrade scarified, moisture conditioned, and re-compacted in accordance with Section 7.3 of this report, prior to foundation placement.

After precipitation events, the contractor should re-proof-roll previously placed fill soils and/or subgrade soils prior to placing subsequent lifts. This re-proof-rolling operation should be observed by the on-site representative of the Geotechnical Engineer to evaluate whether or not the previously suitable soils have become loose/unsuitable as a result of precipitation events. If loose/unsuitable soils are observed during re-proof-rolling, those soils will either need to be removed or scarified, moisture conditioned, and re-compacted. Approved import soil should be considered for replacement of loose/unsuitable subgrade and/or fill materials if weather conditions make moisture conditioning the on-site soils impractical, in consideration of the project cost and/or schedule.

If site grading and construction is to be performed during or shortly after seasons of wet weather, the owner and contractors should be fully aware of the potential impact of wet weather. Rainstorms can cause delays to construction and damage to previously completed work, such as saturating a compacted subgrade. Runoff can also cause erosion. Earthwork during rainy months will require extra effort and caution by the contractors. The soils may be too wet to compact, which will require moisture conditioning to dry the soil. This may require scarifying multiple times and allowing the scarified soil to dry. The contractor is responsible to protect its work to avoid damage by rainstorms, including the use of a minimum 6-inch-thick layer of compacted gravel above the prepared subgrade, smooth rolling to seal off a pad or subgrade surface to facilitate drainage and to reduce rain damage, and the covering of stockpiles with plastic sheeting. Additional protection measures include, but are not limited to, diversion berms, ditches, erosion matting, rock drains, etc. Ponded water should be pumped out of subgrade areas immediately. We recommend that the grading contractor submit a wet weather construction plan outlining procedures they will employ to protect their work and to minimize damage to their work by rainstorms.

Both temporary and permanent grading should be performed so that pooling or ponding of water does not occur in the construction area. During construction, it is important the site is graded such that water is not allowed to pond on-site.

8 LIMITATIONS

This work was performed in a manner consistent with that level of care and skill ordinarily exercised by other members of Kleinfelder's profession practicing in the same locality, under similar conditions and at the date the services are provided. Our conclusions, opinions, and recommendations are based on a limited number of observations and data. It is possible that conditions could vary between or beyond the data evaluated. Kleinfelder makes no other representation, guarantee, or warranty, express or implied, regarding the services, communication (oral or written), report, opinion, or instrument of service provided.

This report may be used only by the Client and the registered design professional in responsible charge and only for the purposes stated for this specific engagement within a reasonable time from its issuance, but in no event later than four (4) years from the date of the report.

The work performed was based on project information provided by Client. If Client does not retain Kleinfelder to review any plans and specifications, including any revisions or modifications to the plans and specifications, Kleinfelder assumes no responsibility for the suitability of our recommendations. In addition, if there are any changes in the field to the plans and specifications, Client must obtain written approval from Kleinfelder's engineer that such changes do not affect our recommendations. Failure to do so will vitiate Kleinfelder's recommendations.

Recommendations contained in this report are based on our field observations and subsurface explorations, limited laboratory tests, and our present knowledge of the proposed construction. It is possible that soil, rock, or groundwater conditions could vary between or beyond the points explored. If soil, rock, or groundwater conditions are encountered during construction that differ from those described herein, the client is responsible for ensuring that Kleinfelder is notified immediately so that we may reevaluate the recommendations of this report.

As the geotechnical engineering firm that performed the geotechnical evaluation for this project, Kleinfelder should be retained to confirm that the recommendations of this report are properly incorporated in the design of this project, and properly implemented during construction. This may avoid misinterpretation of the information by other parties and will allow us to review and modify our recommendations if variations in the soil conditions are encountered. As a minimum Kleinfelder should be retained to provide the following continuing services for the project:

- Review the project plans and specifications, including any revisions or modifications;
- Observe and evaluate the site earthwork operations to confirm subgrade soils are suitable for placement of pavements.

9 REFERENCES

AASHTO LRFD Bridge Design Specifications, Eighth Edition, 2018

Dickson, Stephen M., 2003, “Coastal Landslide Hazards in the Castine Quadrangle, Maine”, Maine Geological Survey (Department of Conservation), Open-File Map 03-101, scale 1:24000

Maine Department of Transportation, “Bridge Design Guide” (2003 with Chapter 5 Substructures (Updated March 2018)

Steward, David B (1998). United States Geological Survey (USGS) Map I-2551 “Geology of Northern Penobscot Bay, Maine.”

Thompson, W., C. Halsted, R.G. Marvinney, and R. Tucker (2013). Maine Geological Survey (MGS) Open-File No. 13-7, “Castine Quadrangle, Maine.”

Appendix A – Existing Bridge Plans and Field Notes

Attachment A:
Record information of existing bridge

TRAFFIC – Temporary bridge better located upstream.

EXISTING SUPERSTRUCTURE – 3-panel pony steel pin-connected (Warren type) truss span built by Canton Bridge Co. in 1907, with 3" transverse and 2" longitudinal plank in fair condition.

EXISTING SUBSTRUCTURE – Abutments of roughly split granite fairly laid in mortar and in fair condition. Mortar in tide range eaten away. Retaining walls of causeway of boulders and roughly split granite fairly laid dry and in poor condition.

WATER COURSE – Salt tidal inlet. Ordinary high tide at Elev. 88.5. Monthly high tide at Elev. 90.2. Extreme high tide reported as 2" over low part of road on causeway at Elev. 94.1. During emptying and filling of pond water drops 2' going thru bridge at about 8 miles per hour.

FOUNDATION – Stream bed large rocks, probably over lying ledge. No penetrations.

PWA PROJECT NO. ME. 1096-12F

TOWN 05-35
BRIDGE 3063

MAINE HIGHWAY COMMISSION
BRIDGE DIVISION

MILL HILL BRIDGE
OVER
OUTLET TO HOLTS MILL POND
BETWEEN THE TOWNS OF
STONINGTON & DEER ISLE
HANCOCK CO.

SURVEY PLAN
Sheet 1 of 4 Augusta, Me. Dec. 12, 1932

31~145

T

STATE HIGHWAY COMMISSION, AUGUSTA, MAINE

Book #1 of

Town STONINGTON — DEER ISLE. MILL HILL Bridge

Contractor (s) J.R. Clanchette & Co.

Engineer (s) George W. Stangel

rk started 10/17/1938 Suspended 2/21/1939

sumed 4/13/1939 Completed May 6, 1939

neral index, Page 1st 2 of Book 1

eld changes, Page — of Book

Materials used in project, Page Thru out of Book 1

GENERAL NOTES

Notebooks should contain the following:

Layout of project, date & name of participants.

Computations for final quantities on all contract items. Extensive computations may be made on letter size paper, the sheets numbered and references made in notebook under the proper item that they are in the letter file.

Records of all samples and tests, noting source (Manufacturer, name of pit and city or town).

Record of materials actually used in project.

Note if sand or gravel was washed.

Account and description of all work done on a "Force Account Bases".

Location of right of way markers, if placed.

Recommended concrete mixes and mixes used.

Description and sketches of changes in plans.

If project is suspended, record work to be completed when work is resumed.

Disposal of salvaged material from old bridge.

NOTEBOOKS ARE TO BE PROMPTLY TURNED IN TO THE BRIDGE OFFICE WHEN PROJECT IS SUSPENDED OR COMPLETED.

Structural Earth Excav

✓

Boulders over 1 c.y.

Date	Size	Vol. c.f	
11-5-38	1.1'×7×3.5	27.5	Abut #2
	2.5'×4.0×3	30.0	"
11-7-38	3×2×6.5	39.0	"
11-8-38	2×6×2.5	30.	"
11-9-38	2×4×4.5	36	"
	4.5×3×2	27	"
	4.5×3×2.2	29.7	"
11-10-38	6×3×1.5	27	"
	6×3×1.5	27	"
	5.5×2.5×2.5	34.4	"
" -11-38	3×1.5×5.5	—	
	3.5×2.5×3	27	
11-15-38	9.5×1.5×2.5	35.6	

370.2 c.f

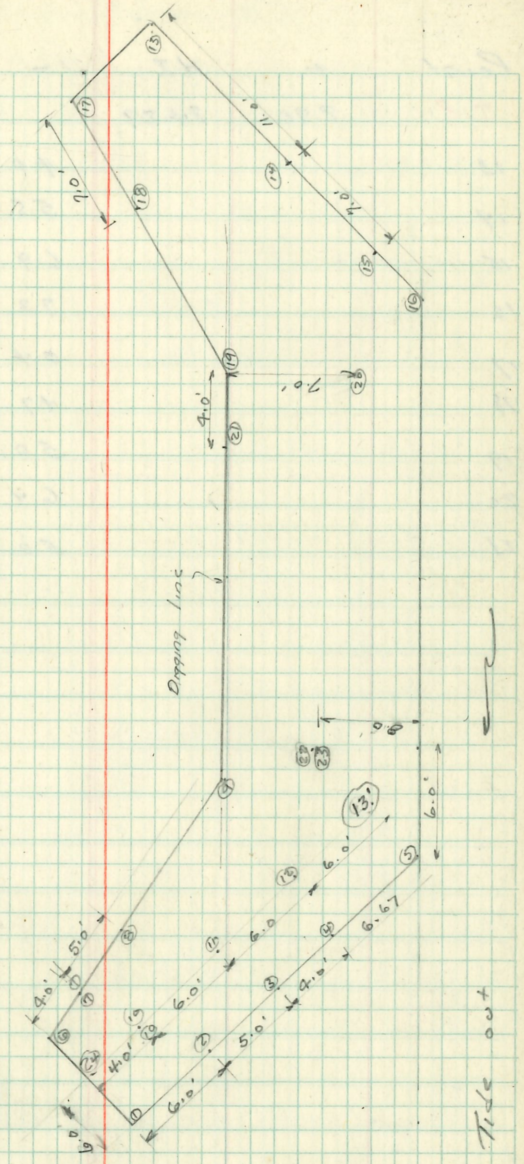
= 13.7 c.y to be added to exc.
as pay for boulders over 1.c.y is twice st. exc.

Point	+	H I	−	Elev	
T.B.M	0.58	91.48		90.90	
			11.05	80.43	Ledge Rock in channel.
T.P.			10.09	81.39	
	3.90	85.29			
1			1.2	84.1	
2			1.2	84.1	
3			2.8	82.5	
4			5.0	80.3	
5			6.6	78.7	
7			1.0	84.3	
8			3.0	82.3	
9			4.4	80.9	
10			0.0	85.3	
11			2.4	82.9	
12			5.2	80.1	
13			6.8	78.5	
22			5.3	80.0	
23			6.3	79.0	
T.B.M	0.56	91.46		90.90	
1			5.1	86.4	
6			2.4	89.1	
7			4.3	87.2	
10			5.9	85.6	
24			4.5	87.0	

Con't page 22

Top of ledge and bottom of Excav.
Abutment No. 2.

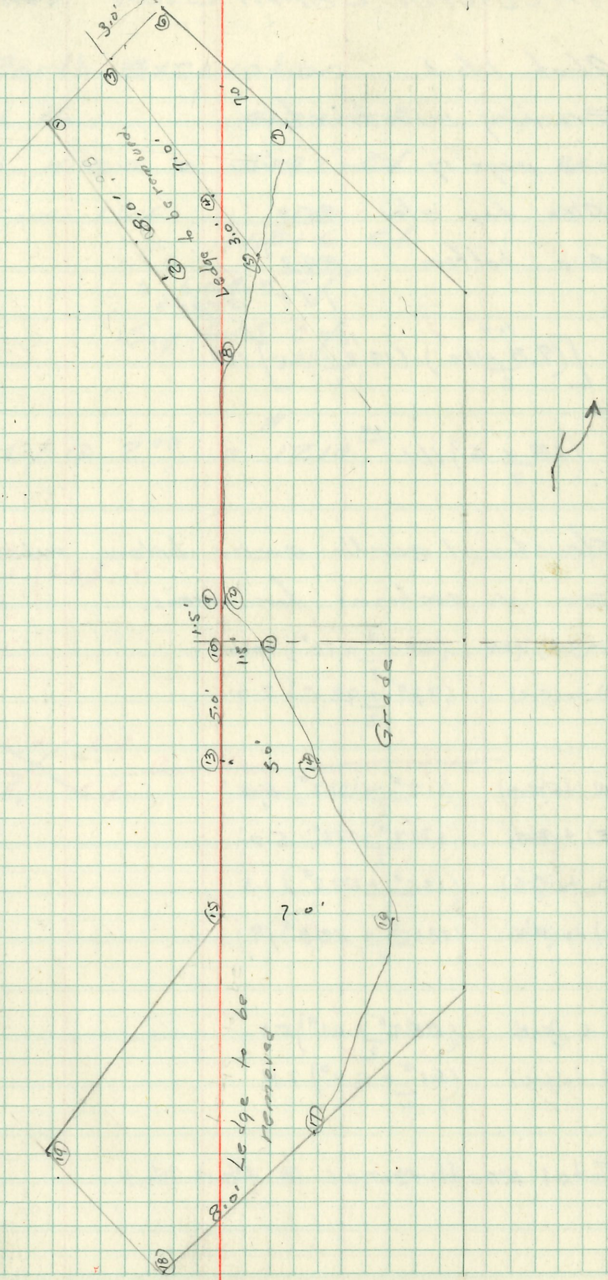
Ledge before removal
11/18/38

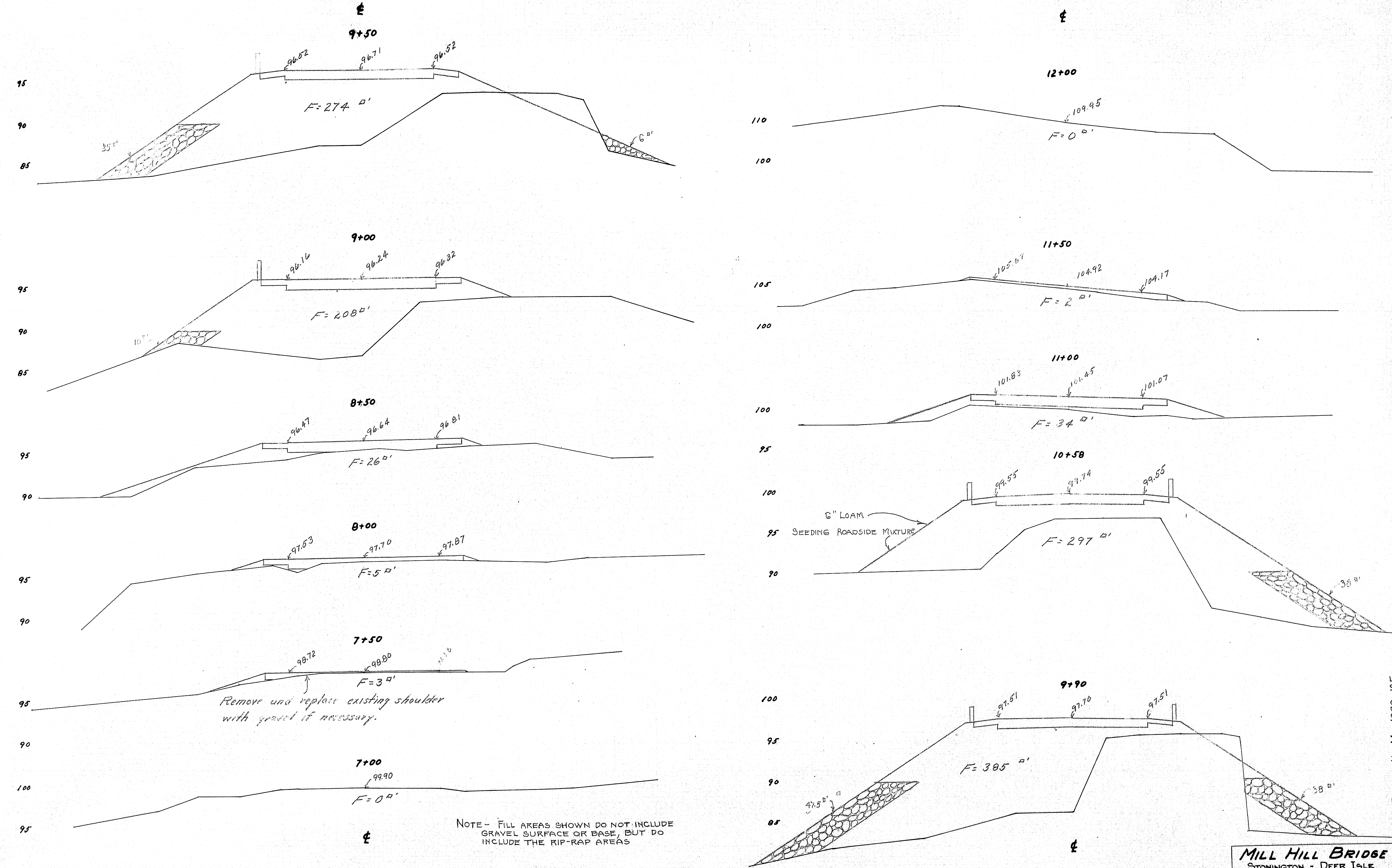


Point	+	H.I	−	Elev	
T.P	2.90	84.29		81.39	
13			4.0	80.3	✓
14			5.5	78.8	✓
15			6.9	77.4	✓
16			7.3	77.0	✓
17			3.4	80.9	✓
18			4.7	79.6	✓
19			5.0	79.3	✓
20			6.4	77.9	✓
21			5.0	79.3	✓

Point 12/20/38	+	H.I	−	Elev.
T.P	2.67	85.57		82.90
1			4.9	80.7
2			7.8	77.8
3			4.5	81.1
4			7.2	78.4
5			8.3	77.3
6			6.9	78.7
7			8.1	77.5
8			7.6	78.0
9			6.9	78.7
10			6.9	78.7
11			8.2	77.4
12			8.1	77.5
13			7.7	77.9
14			8.2	77.4
15			6.7	78.9
16			8.2	77.4
17			6.9	78.7
18			3.3	82.3
19			3.0	82.6

Top of ledge and bottom of excav.
Abutment No. 1





Remove and replace existing shoulder with gravel if necessary.

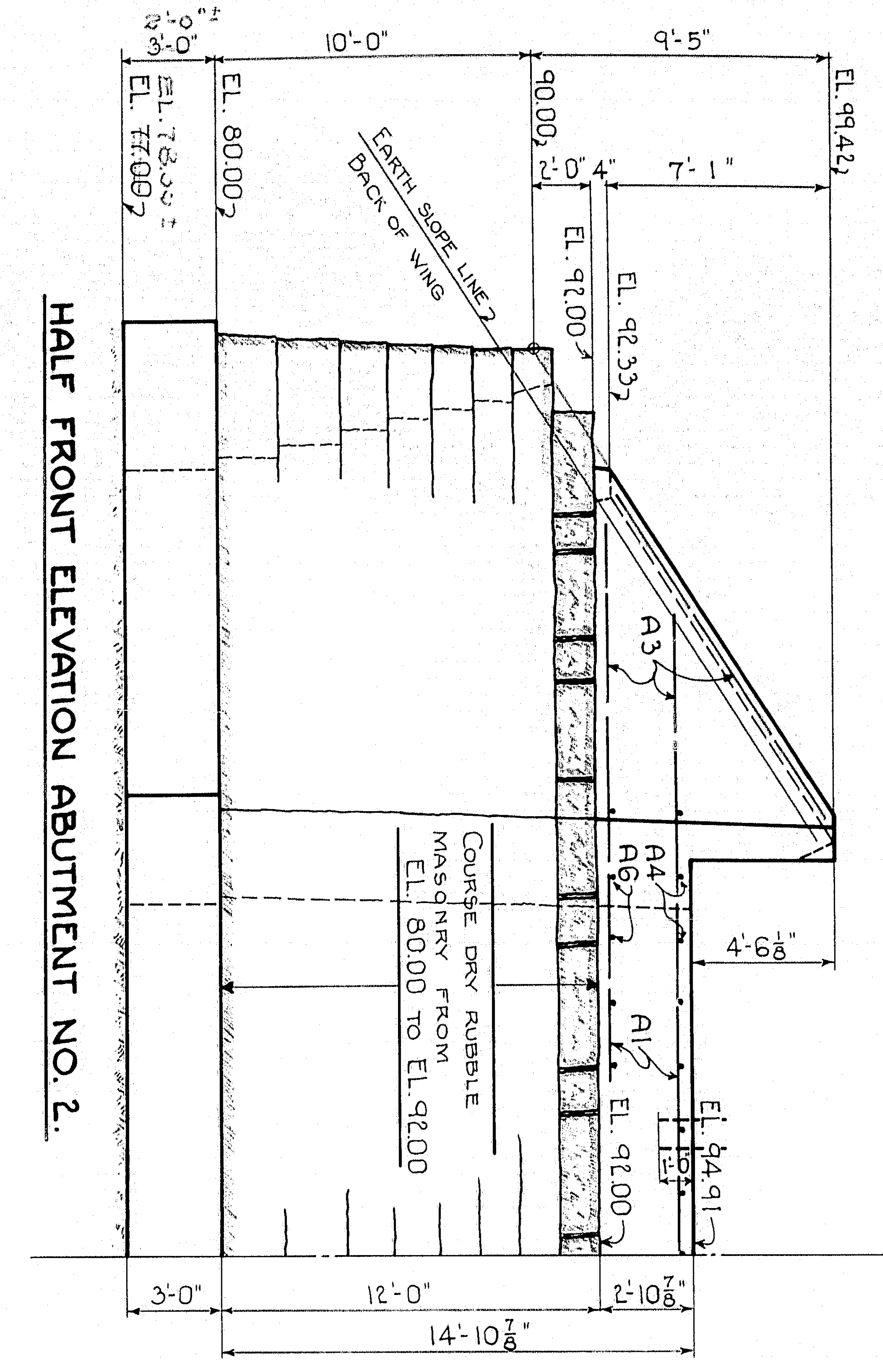
NOTE- FILL AREAS SHOWN DO NOT INCLUDE GRAVEL SURFACE OR BASE, BUT DO INCLUDE THE RIP-RAP AREAS

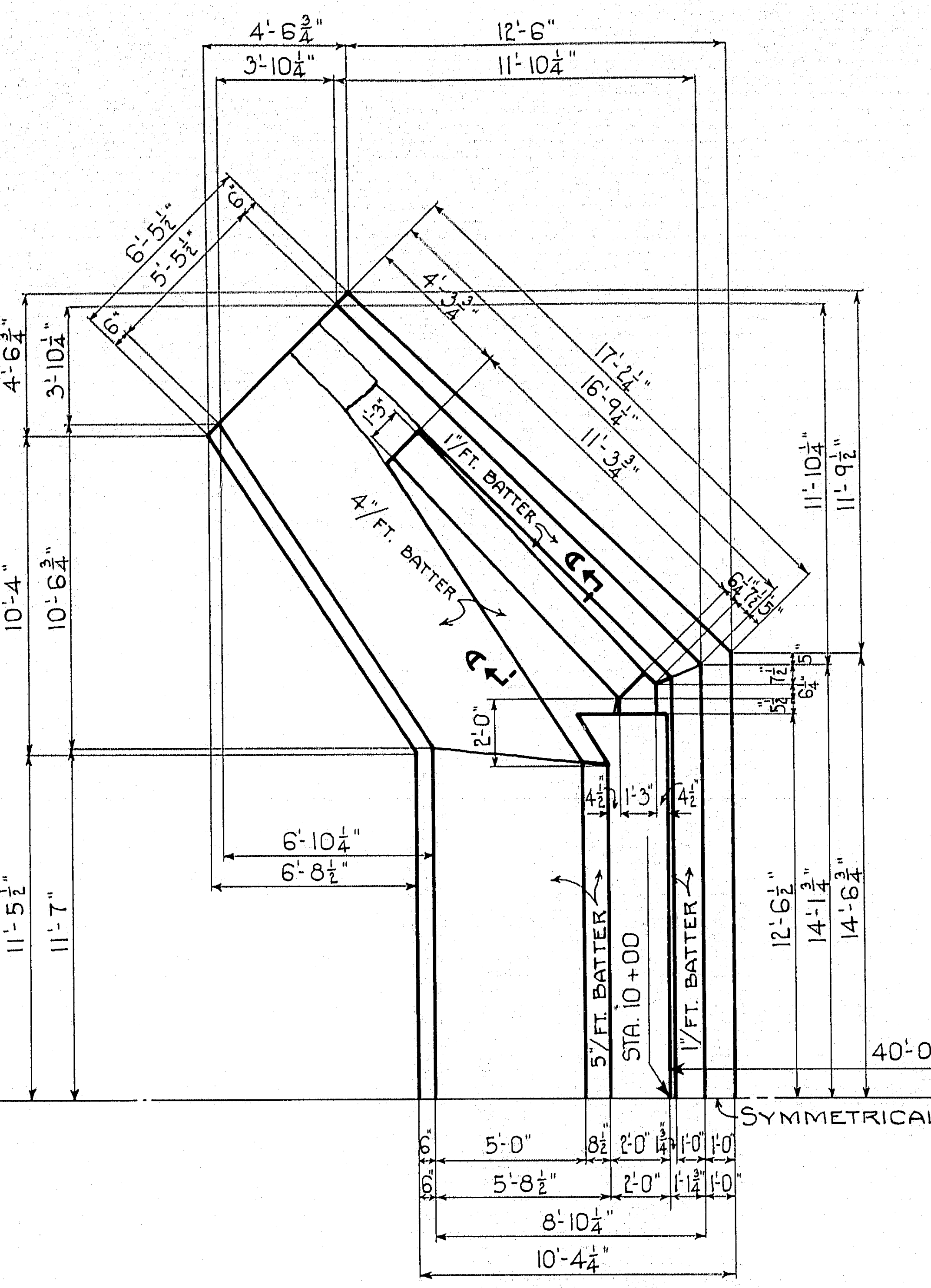
PWA PROJECT NO. ME. 1096-12F

MILL HILL BRIDGE
STONINGTON - DEER ISLE
Sheet 2 of 4

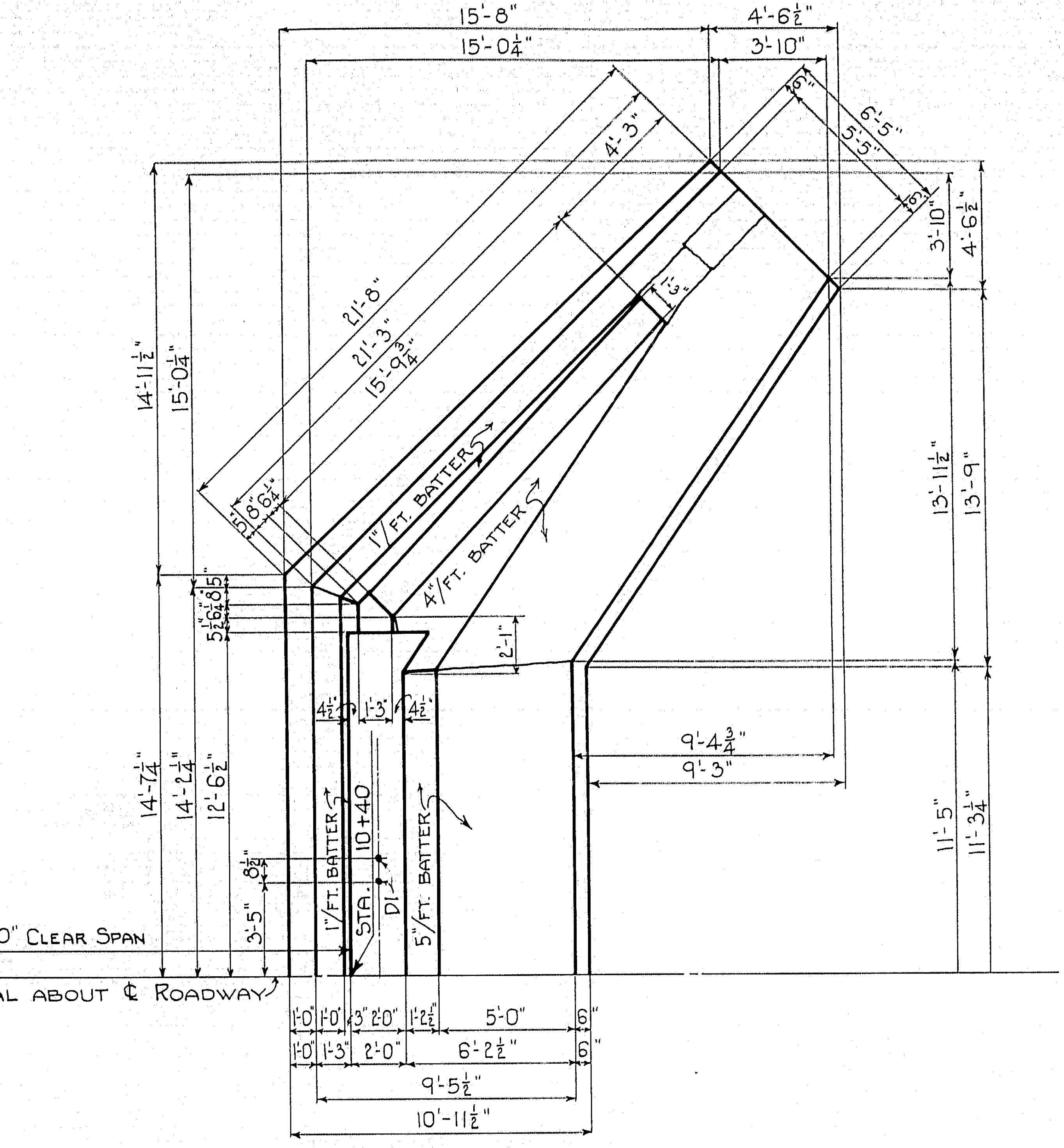
31-146

0 1 2 3 4 5 INCHES

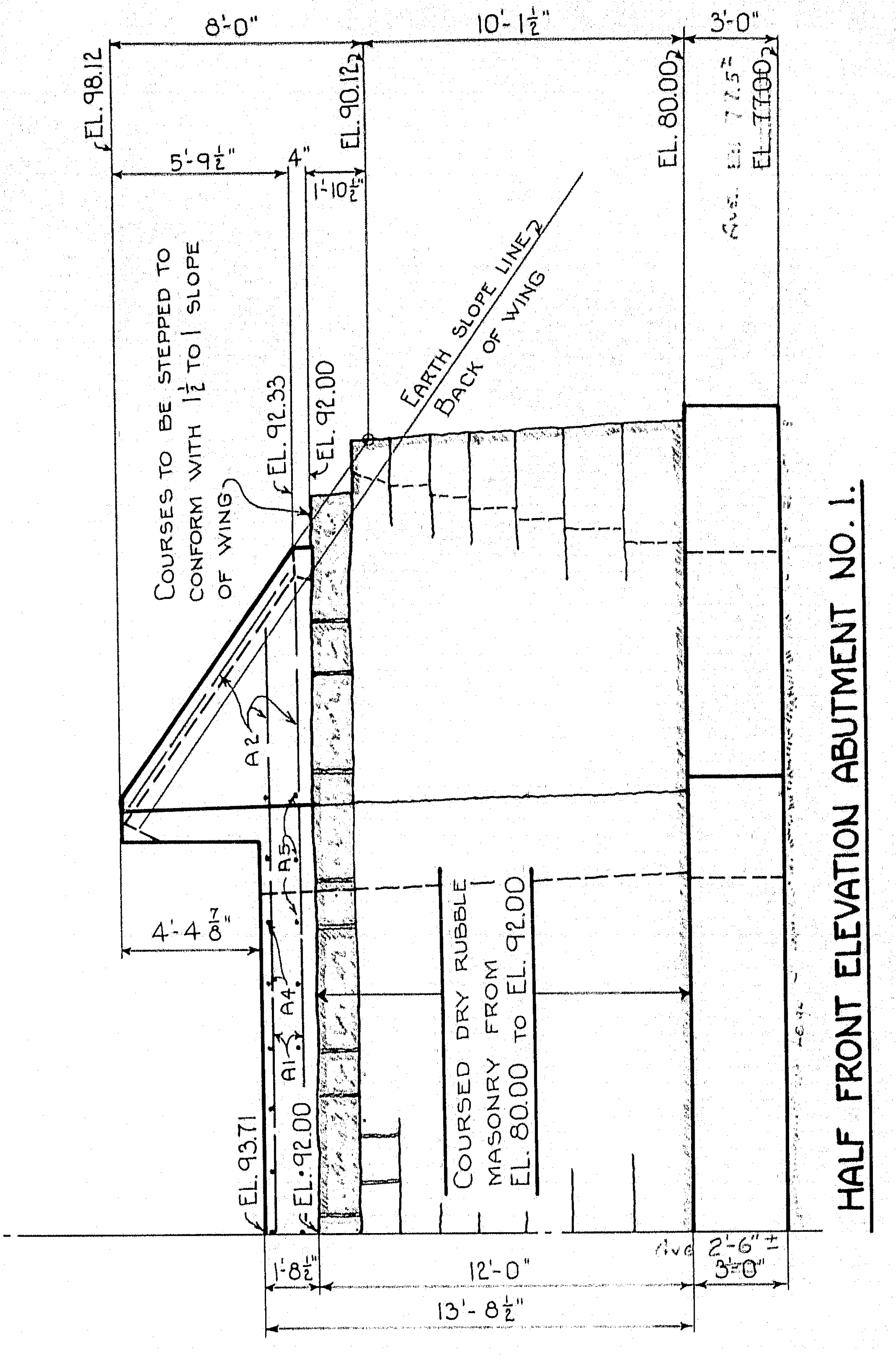


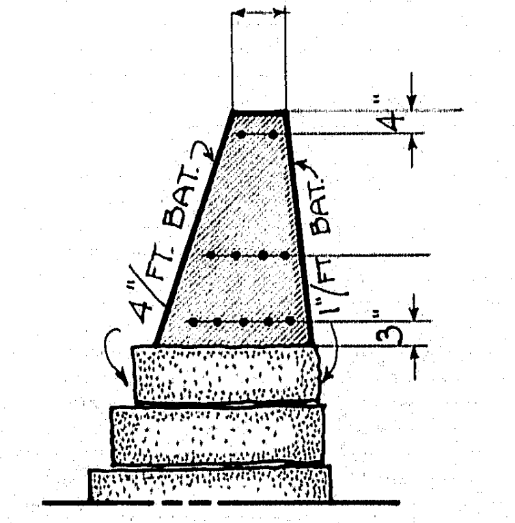


HALF PLAN ABUT. NO. 1.



HALF PLAN ABUT. NO. 2.

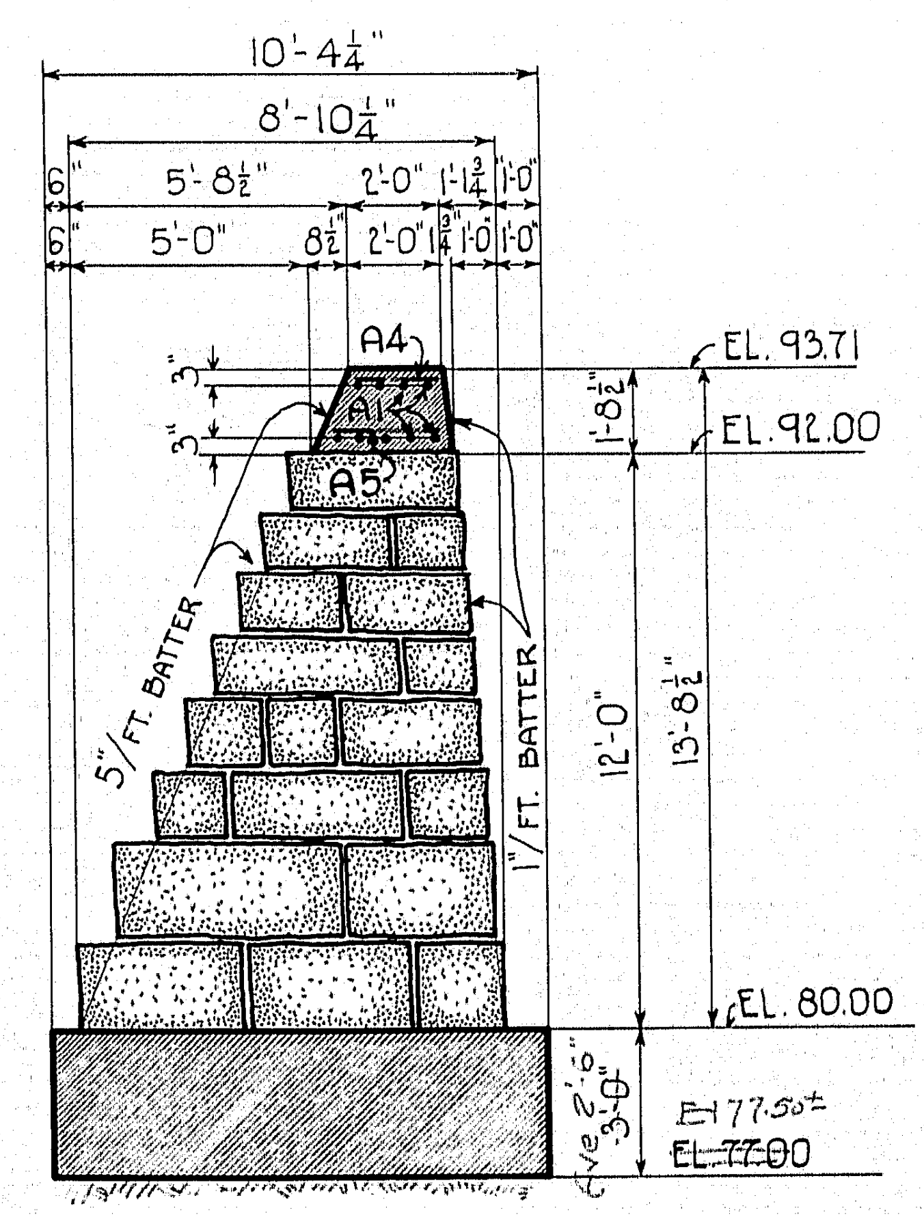




SECTION A-A

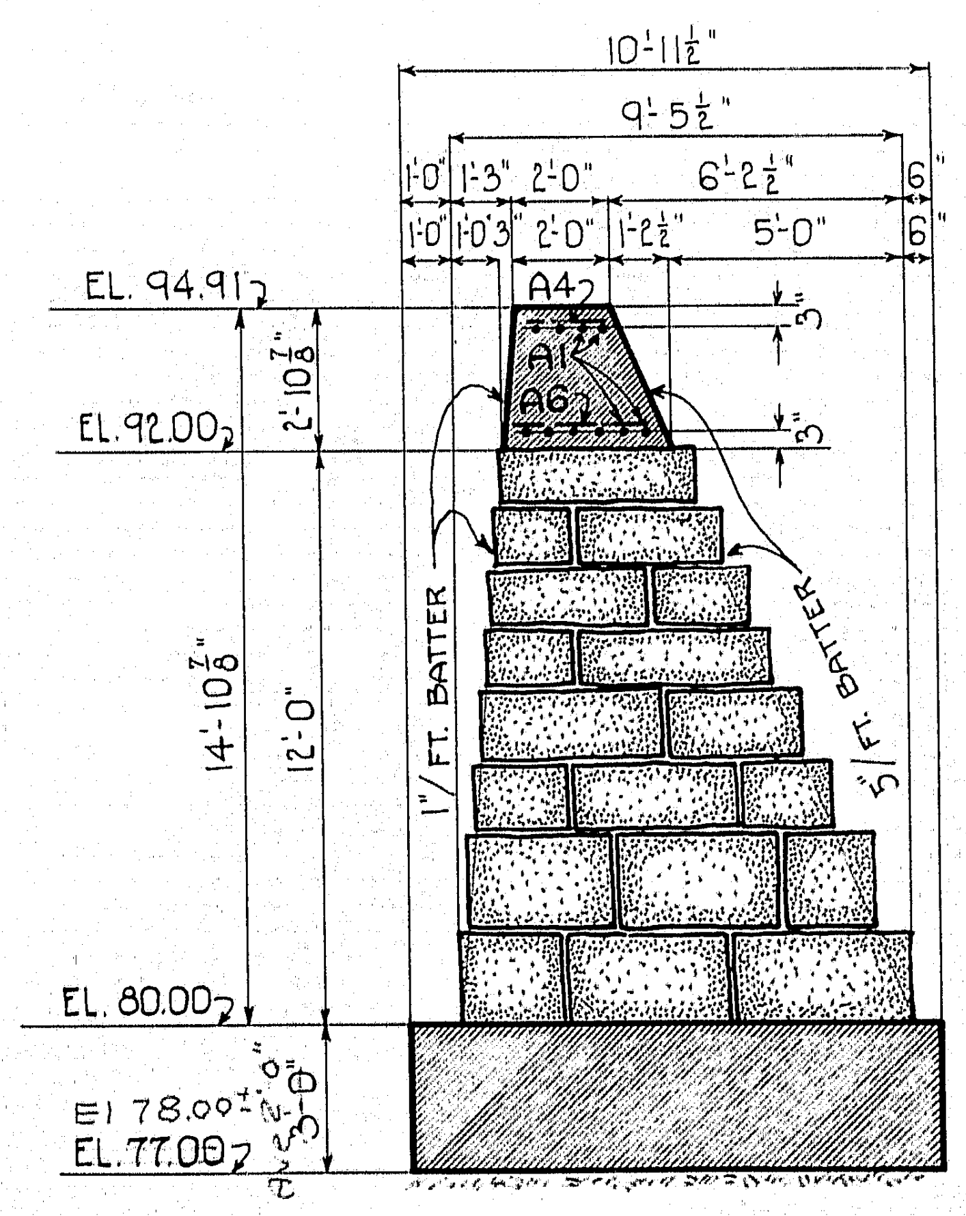
SECTION THRU WINGS ON No. 2. ABUTMENT IS SIMILAR.

NOTE- A2 AND A3 TO BE LAPPED TO A1 AND ARE TO BE BENT IN FIELD TO FIT.



SECTION NO. 1. ABUT.

NOTE- A1 @ 6" o.c.
A4-A5-A6 @ 2'-0" o.c.



SECTION NO. 2. ABUT.

PWA PROJECT NO. ME. 1096-12F

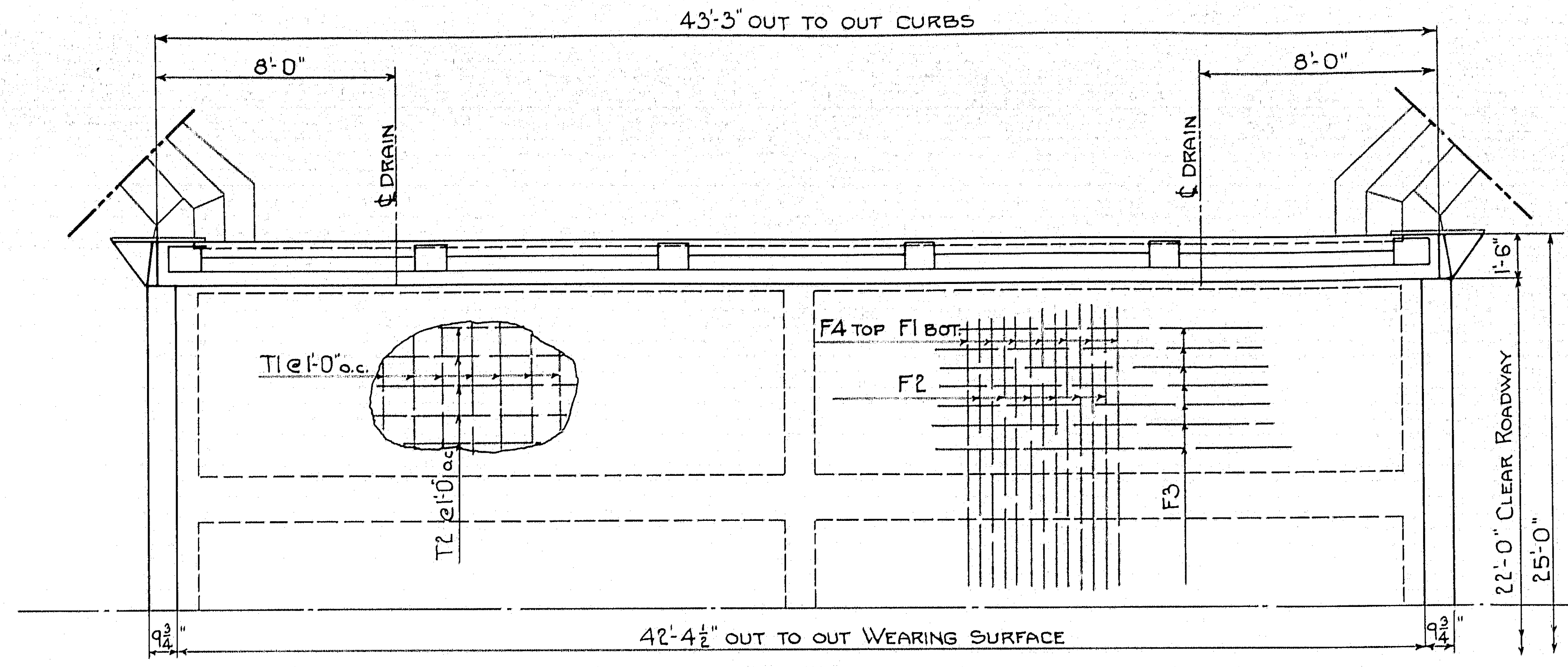
DESIGN & TRACED ~ Hamilton
CHECKED - G. Bragdon

TOWN	05-35
BRIDGE	3063

STATE HIGHWAY COMMISSION
BRIDGE DIVISION
MILL HILL BRIDGE
OVER
OUTLET TO HOLTS MILL POND
BETWEEN THE TOWNS OF
STONINGTON AND DEER ISLE
HANCOCK CO.
SUBSTRUCTURE
SHEET 3 OF 4 AUGUSTA, ME. JULY 1938.

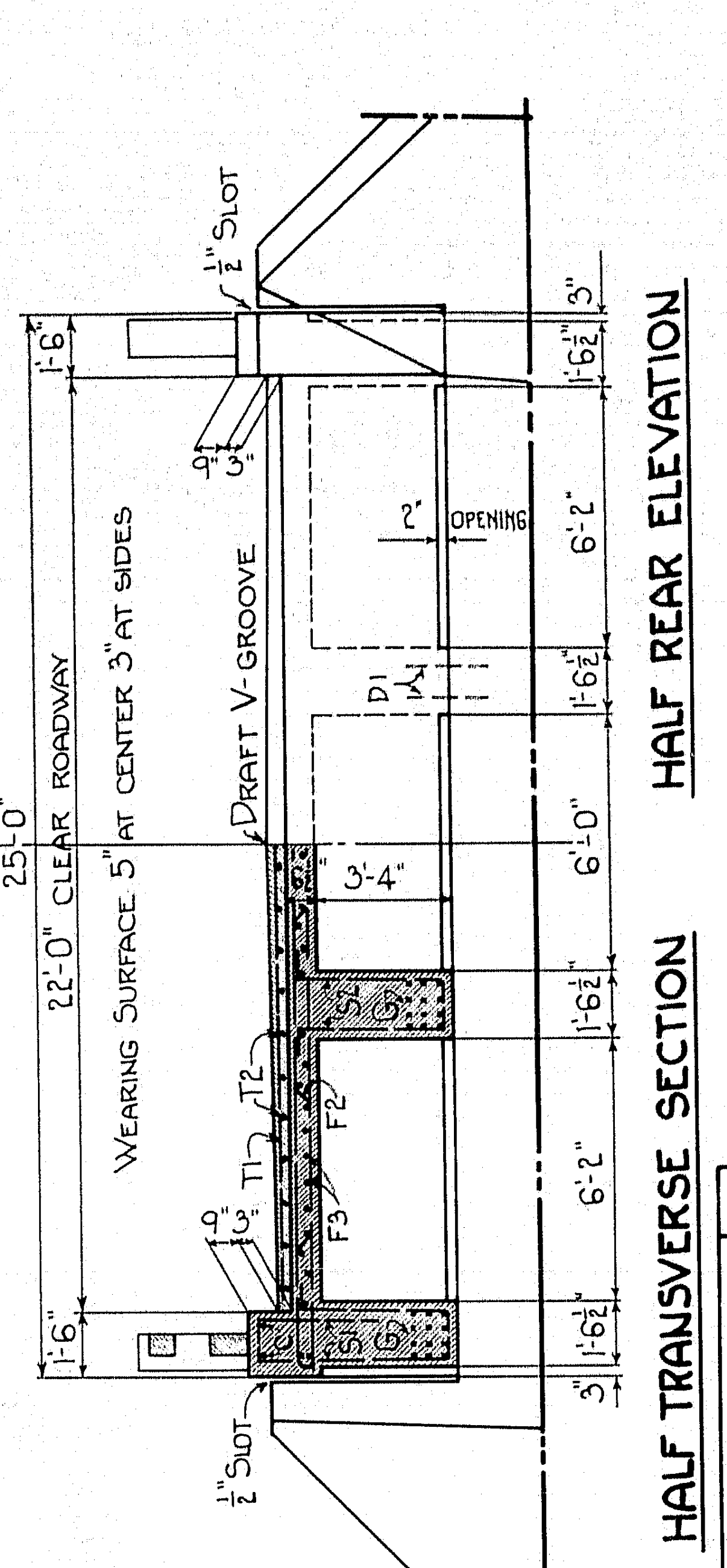
31~147

0 1 2 3 4 5 INCHES



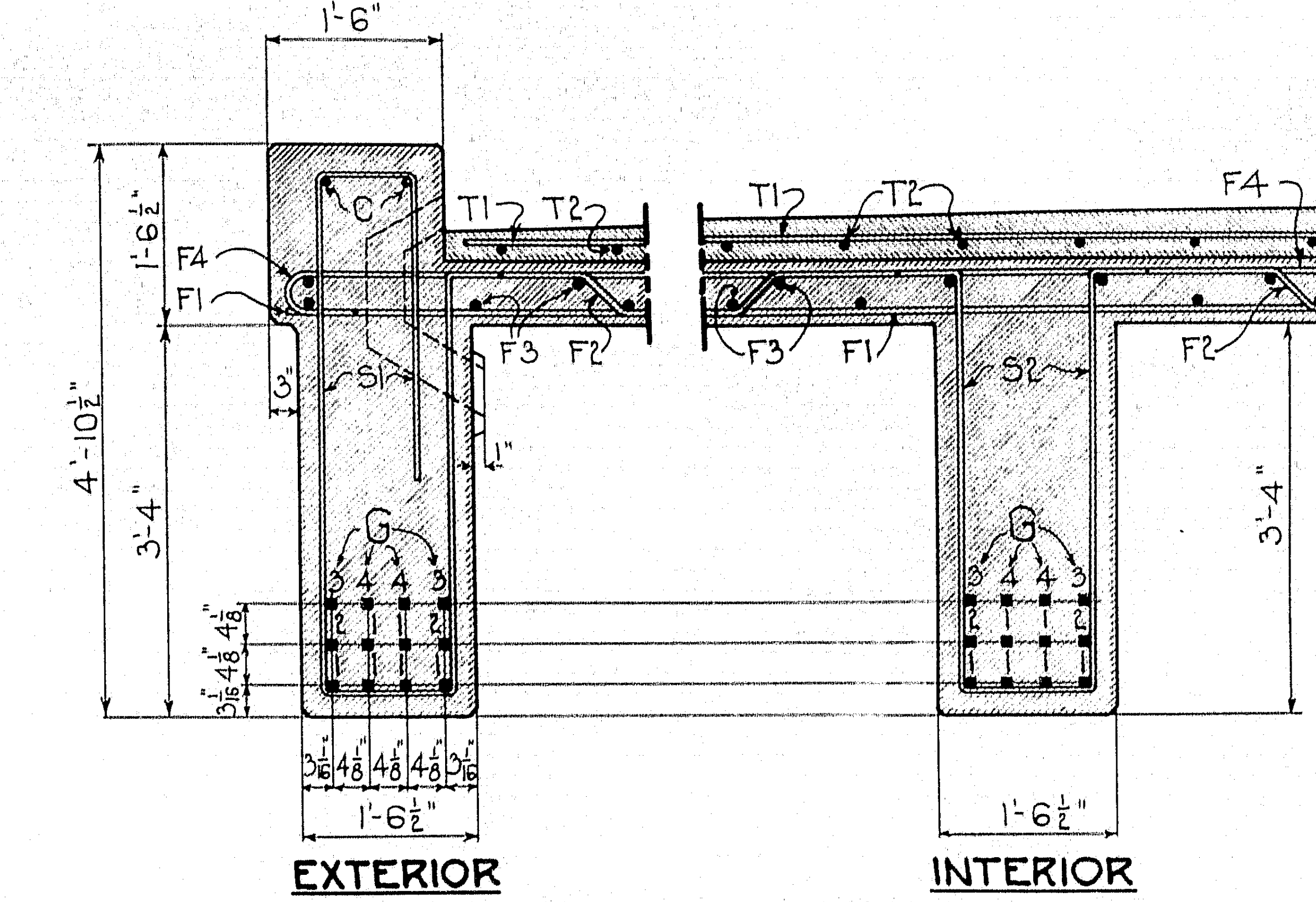
HALF PLAN

NOTE - COVER THE ½" SLOTS AND THE 2" OPENINGS BETWEEN THE SUPERSTRUCTURE AND THE SUB-STRUCTURE WITH 2 LAYERS OF HEAVY ROOFING 10" WIDE. COAT SURFACE OF CONCRETE AND BACK SIDE OF EACH LAYER AS APPLIED WITH HOT TAR OR ASPHALT. THE AREA TO BE COVERED BY THE ROOFING IS TO BE RECESSED ¼" BY NAILING THIN STRIPS TO THE FORMS BEFORE THE CONCRETE IS PLACED.



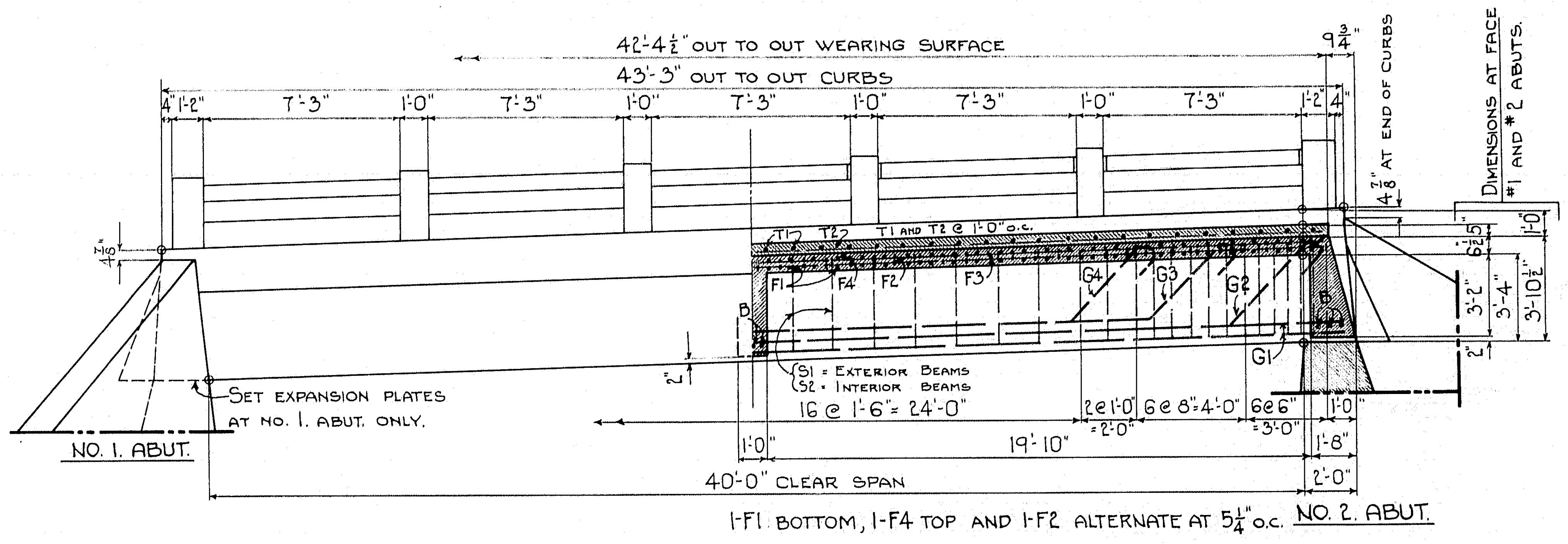
HALF REAR ELEVATION

HALF TRANSVERSE SECTION



EXTERIOR **INTERIOR**

GIRDER DETAILS

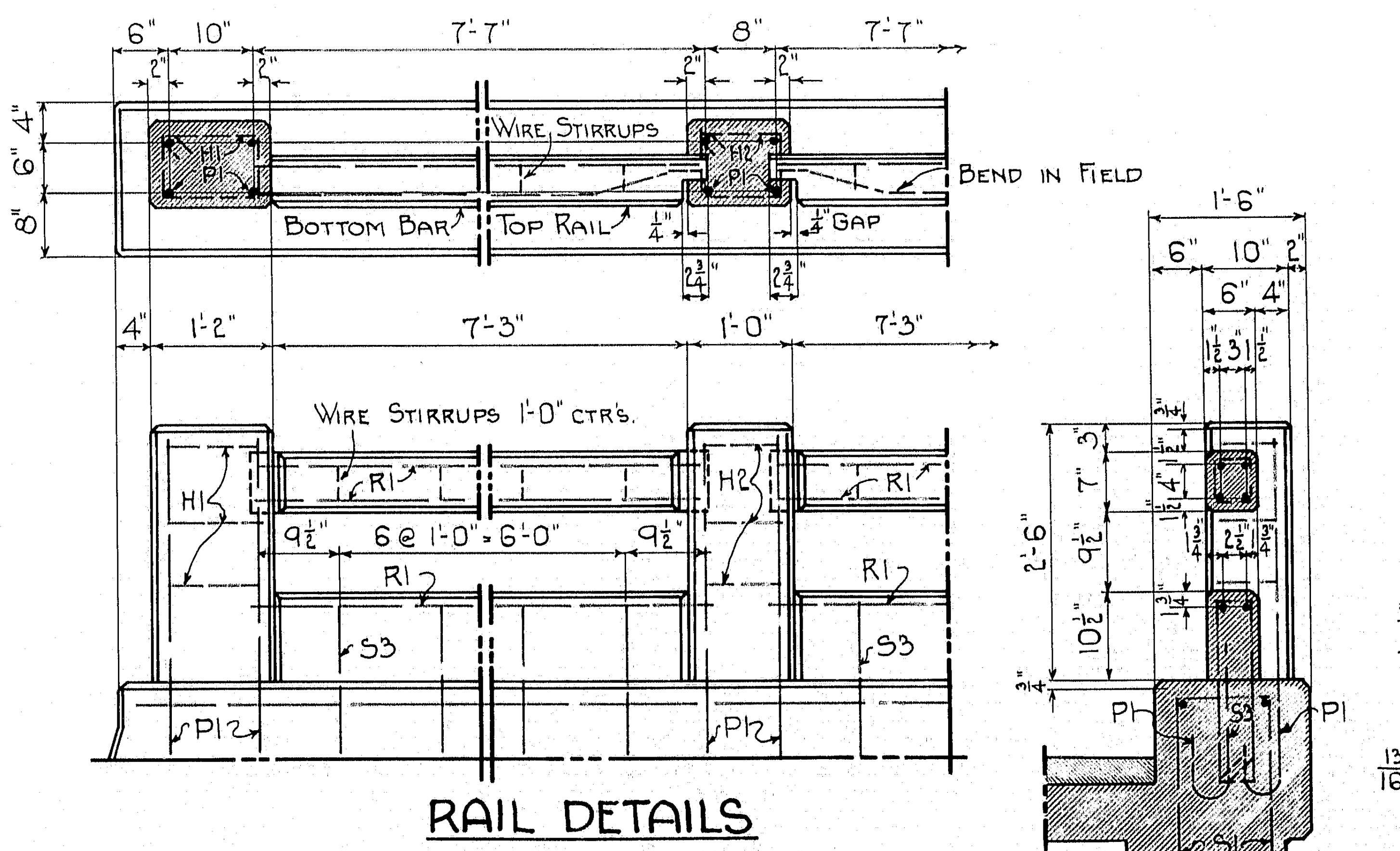


1-F1 BOTTOM, 1-F4 TOP AND 1-F2 ALTERNATE AT 5¼" o.c.

HALF SIDE ELEVATION

HALF LONGITUDINAL SECTION

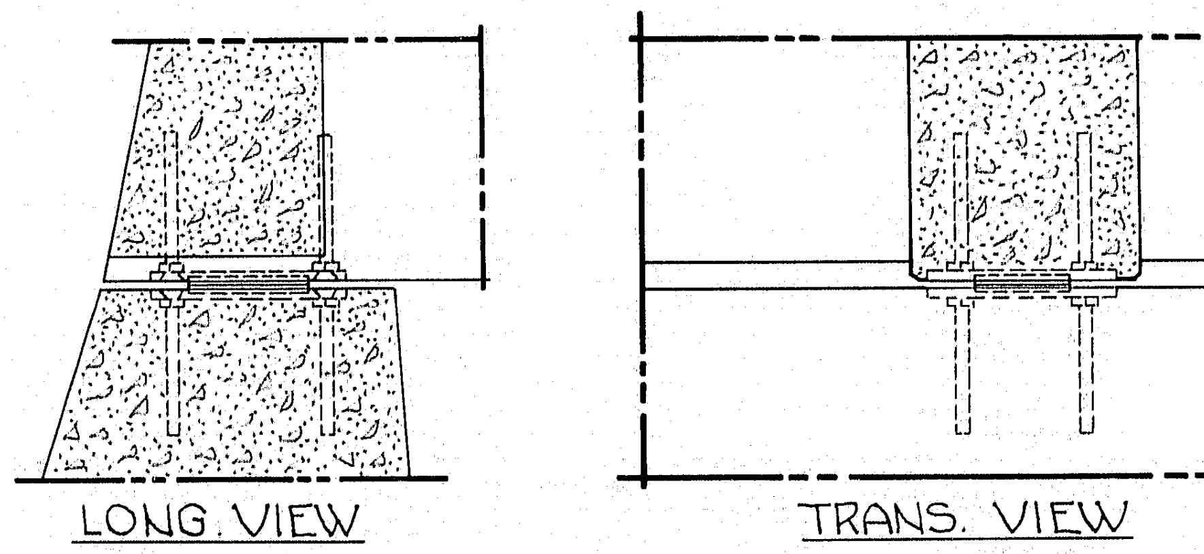
NOTE - BRIDGE ON +3% GRADE

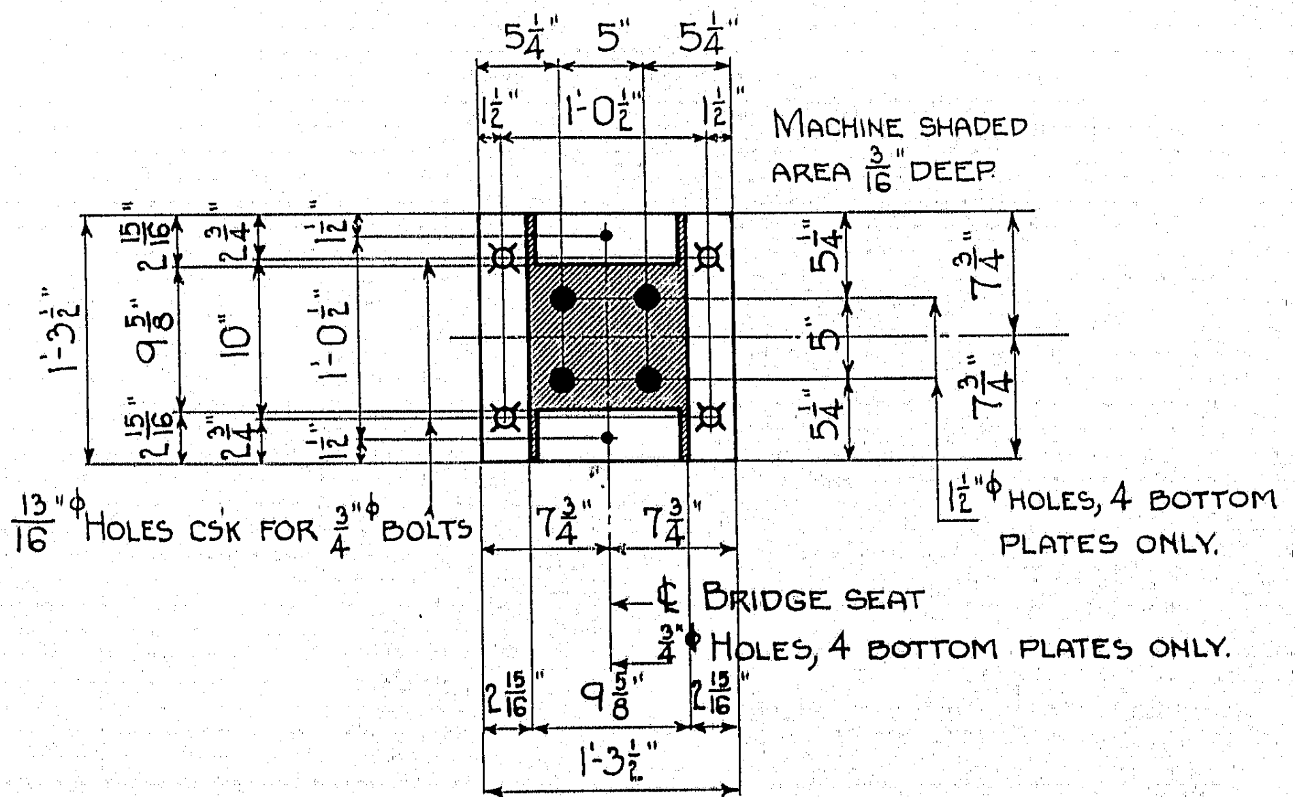


RAIL DETAILS

CURB TO BE CAST WITH SLAB. STEEL FOR POSTS TO BE SET; ALSO LOWER BAR, BEFORE CURB IS PLACED. THE LOWER BAR IS TO BE CAST IN PLACE. THE LONGITUDINAL STEEL IS TO PROJECT 2". THE TOP BAR IS TO BE PRECAST AND SET IN POSITION SO THAT THE ENDS PROJECT INTO POST FORMS 2½". WRAP THE TONGUE END WITH 2 LAYERS OF HEAVY ROOFING. BUILD POST FORMS AND CAST POSTS. ALL EXPOSED EDGES OF CONCRETE TO BE CHAMFERED ½" UNLESS OTHERWISE INDICATED. CUT AWAY ALL EXPOSED ROOFING.

WIRE STIRRUPS FOR RAIL BARS SHALL BE CONSTRUCTED IN THE FIELD FROM A SINGLE STRAND OF #9 ANNEALED WIRE. IN FORMING THE STIRRUPS, MAKE ONE COMPLETE TURN AROUND EACH REINFORCING BAR.





REQUIRED

8 - STEEL PLATES 15½" x ¾" x 1'-3½"

32 - BOLTS ¾"φ x 1'-0", NUTS, C'SK HEAD, THREADED FULL LENGTH

8 - "LUBRITE" OR APPROVED EQUIVILENT BRONZE PLATES 9½" x ⅜" x 0'-9½"

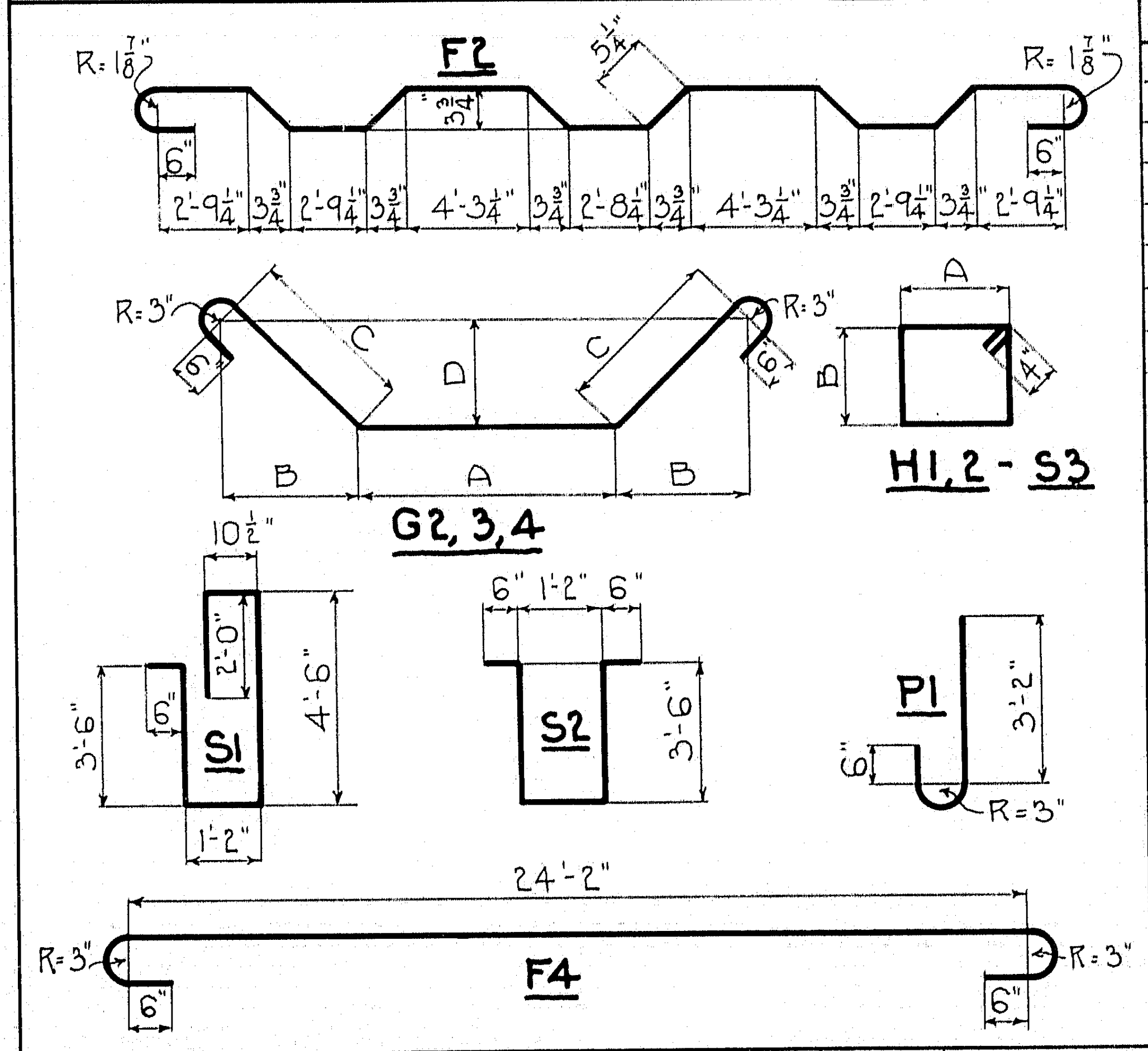
GROOVES IN STEEL PLATES MAY BE CUT 3/32" TO ⅛" DEEPER THAN BEARING AREA TO PERMIT MACHINING.

BRONZE PLATES TO BE FINISHED TRUE AND SMOOTH, EITHER BY ROLLING OR A FINAL FINISHING CUT.

EXPANSION DETAILS

STEEL SCHEDULE

BENT BARS

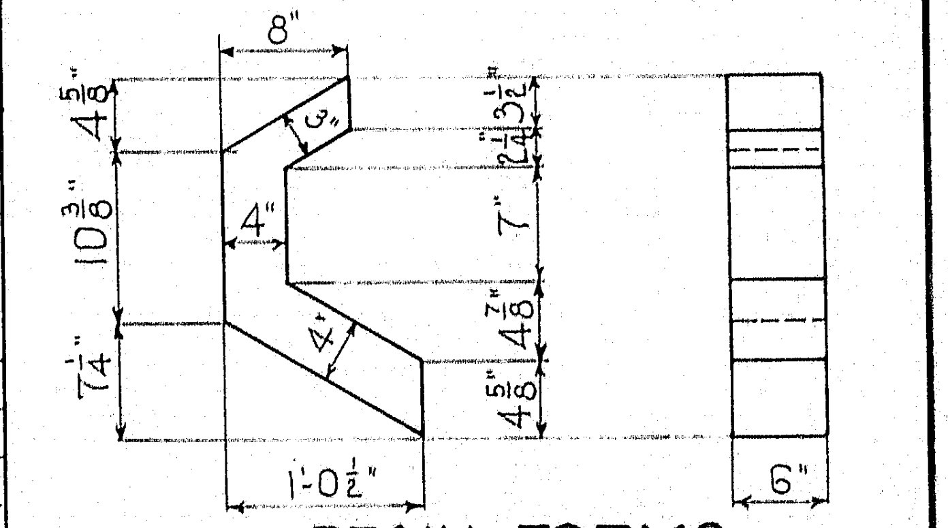


MARK	SIZE	NO. REQ'D	A	B	C	D	LENGTH	LOCATION
F2	⅝"φ	48	—	—	—	—	26'-11"	SLAB
G4	1⅛"□	8	23'-10"	2'-9¼"	3'-8"	2'-5"	33'-8¾"	ALL GIRDERS
G3	1⅛"□	8	29'-4"	2'-9¼"	3'-8"	2'-5"	39'-2¾"	" "
G2	1⅛"□	8	34'-10"	3'-1¼"	4'-1¾"	2'-9"	45'-8¼"	" "
H1	½"φ	12	11¼"	7¼"	—	—	3'-9"	END RAIL POSTS
H2	½"φ	24	9¼"	7¼"	—	—	3'-5"	INT. " "
S1	½"φ	90	—	—	—	—	12'-6½"	EXT. GIRDERS.
S2	½"φ	90	—	—	—	—	9'-2"	INT. "
S3	½"φ	70	0'-3½"	1'-9½"	—	—	4'-10"	LOWER RAIL BAR
P1	¾"φ	48	—	—	—	—	4'-5½"	RAIL POSTS
F4	⅝"φ	48	—	—	—	—	26'-8¾"	SLAB (TOP)

STRAIGHT BARS

MARK	SIZE	NO	LENGTH	LOCATION
G1	1⅛"□	24	43'-4"	ALL GIRDERS
B	¾"φ	9	24'-2"	CROSS BEAMS
F1	⅝"φ	48	24'-8"	SLAB (BOT.)
F3	⅝"φ	33	42'-0"	SLAB
C1	½"φ	4	43'-0"	CURBS
T1	⅜"φ	43	21'-10"	WEARING SURFACE
T2	⅜"φ	22	42'-2"	" "
R1	½"φ	40	7'-7"	TOP RAIL BAR
R2	½"φ	20	7'-9"	BOT. " "
A1	¾"φ	19	26'-0"	ABUT. CAPS
A2	"	22	15'-0"	WING #1 ABUT.
A3	"	24	21'-0"	" #2 "
A4	½"φ	30	1'-8"	ABUT. CAPS (TOP)
A5	"	15	2'-2"	#1 ABUT. CAP. (BOT.)
A6	"	15	2'-8"	#2 " " (")
D1	1"φ	4	2'-0"	ABUT. #2 BR. SEAT

ALL STEEL TO BE PLAIN BARS STRUCTURAL GRADE.
ALL DIMENSIONS TO ℄ BARS.



DRAIN FORMS

26" GAGE GAL. IRON. (4-REQ'D)

DESIGN + TRACED - Hamilton
CHECKED - G. Bragdon

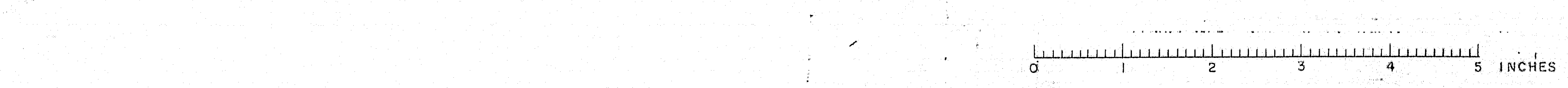
TOWN	05-35
BRIDGE	3063

~ DESIGN ~
LOADING - H-15
fs = 16000#/□"
fc = 900#/□"
n = 10

PWA PROJECT NO. ME. 1096-12F.

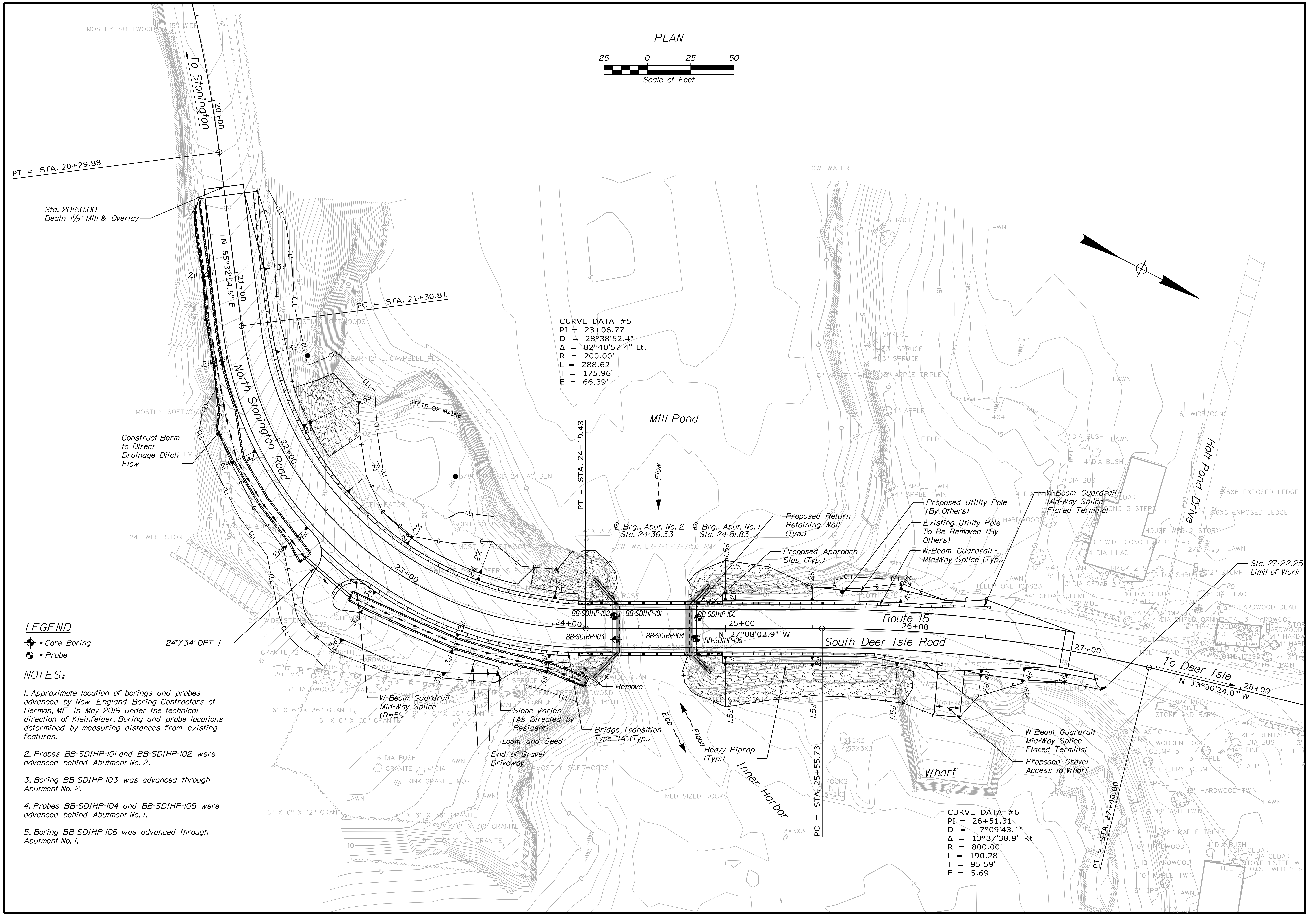
STATE HIGHWAY COMMISSION
BRIDGE DIVISION
MILL HILL BRIDGE
OVER
OUTLET TO HOLTS MILL POND
BETWEEN THE TOWNS OF
STONINGTON AND DEER ISLE
HANCOCK CO.
SUPERSTRUCTURE
SHEET 4 OF 4 AUGUSTA, ME. JULY, 1938.

31~148



Appendix B – Boring Location Plan and Interpretative Subsurface Profile

PLAN



LEGEND

⊕ = Core Boring

● = Probe

NOTES:

1. Approximate location of borings and probes advanced by New England Boring Contractors of Hermon, ME in May 2019 under the technical direction of Kleinfelder. Boring and probe locations determined by measuring distances from existing features.

2. Probes BB-SDIHP-101 and BB-SDIHP-102 were advanced behind Abutment No. 2.

3. Boring BB-SDIHP-103 was advanced through Abutment No. 2.

4. Probes BB-SDIHP-104 and BB-SDIHP-105 were advanced behind Abutment No. 1.

5. Boring BB-SDIHP-106 was advanced through Abutment No. 1.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

2235600

BRIDGE NO. 3063
WIN
022356.00
BRIDGE PLANS

PROJ. MANAGER	ANDREW LATHE	BY	DATE
DESIGN-DETAILED	T. TURCOTTE	T. WOLFEL	9/2019
CHECKED-REVIEWED	K. WOOD	T. MARSHALL	9/2019
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

MILL HILL BRIDGE
MILL POND OUTLET
DEER ISLE & STONINGTON HANCOCK COUNTY

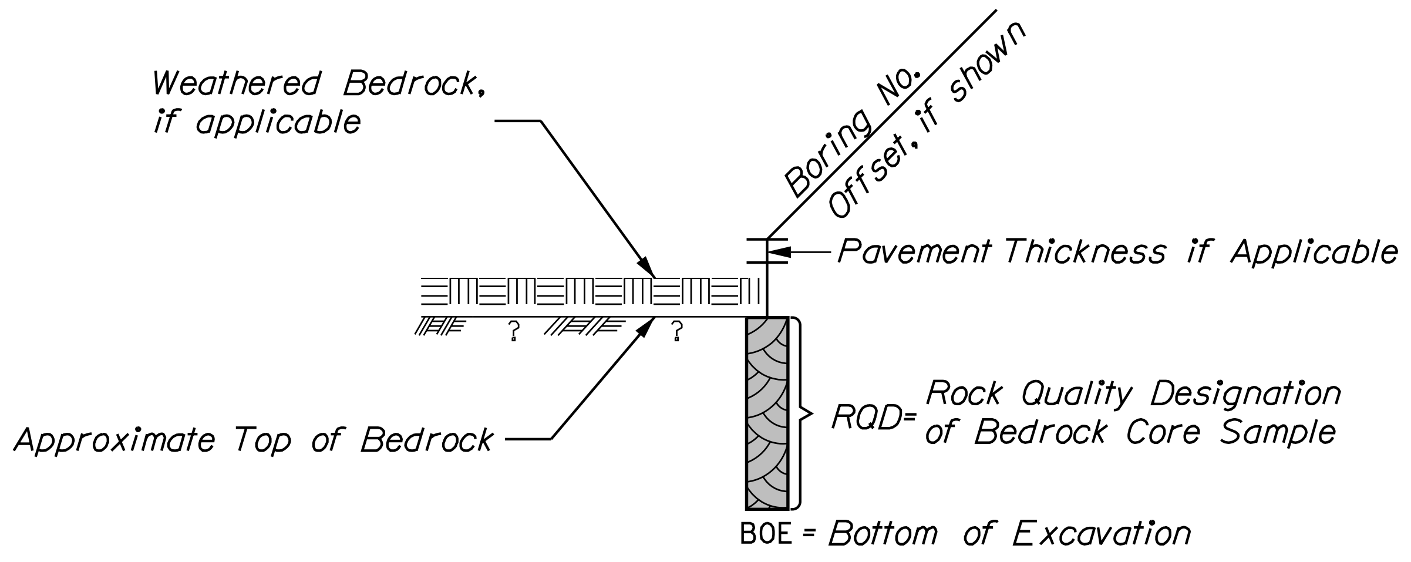
BORING LOCATION PLAN

SHEET NUMBER
6
OF 40

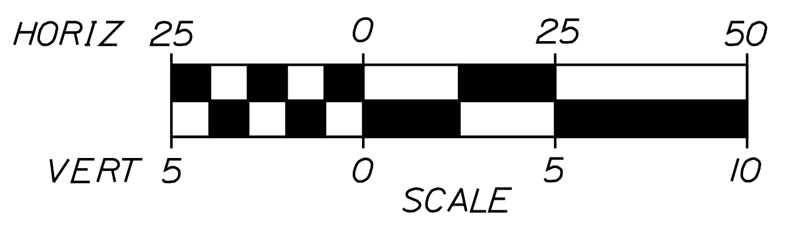
Filename: ...\006_Boring Location Plan.dgn Division: BRIDGE Username: Date:5/11/2022

Filename: ...\007_Interpretive Subsurface Profile.dgn Division: BRIDGE Username: Date:5/11/2022

LEGEND



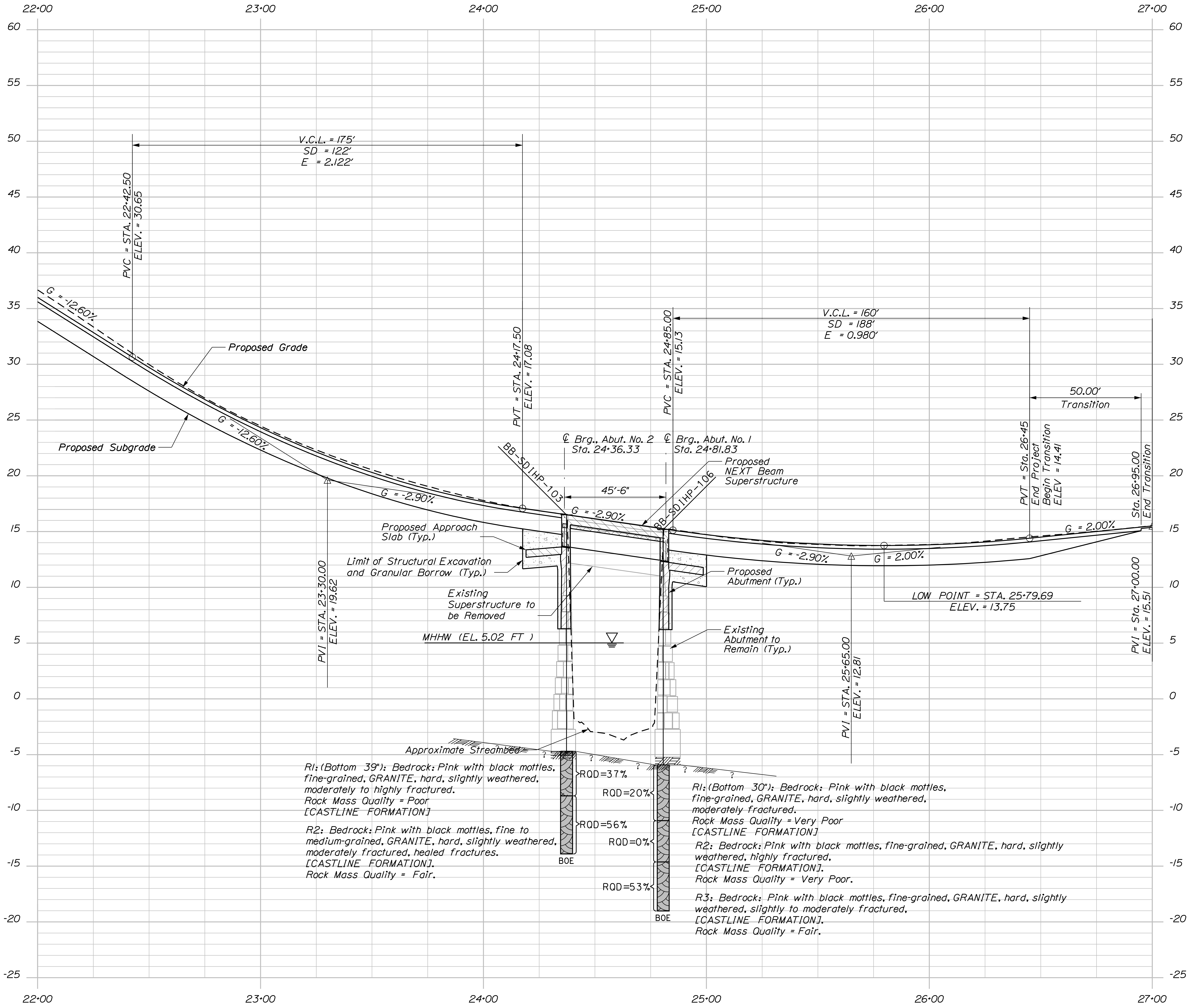
PROFILE



Notes:

This generalized interpretive soil profile is intended to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized, and have been developed by interpretations of widely spaced explorations and samples. Actual soil and bedrock transitions may vary and are probably more erratic. For more specific information refer to the exploration logs.

This interpretive subsurface profile was created through the borings nearest to the construction baseline. Probes BB-SDIHP-101, BB-SDIHP-102, BB-SDIHP-104 and BB-SDIHP-105 not shown for clarity. Refer to Boring Logs for information specific to these borings.



R1: (Bottom 39"): Bedrock: Pink with black mottles, fine-grained, GRANITE, hard, slightly weathered, moderately to highly fractured.
Rock Mass Quality = Poor
[CASTLINE FORMATION]

R2: Bedrock: Pink with black mottles, fine to medium-grained, GRANITE, hard, slightly weathered, moderately fractured, healed fractures.
[CASTLINE FORMATION].
Rock Mass Quality = Fair.

R1: (Bottom 30"): Bedrock: Pink with black mottles, fine-grained, GRANITE, hard, slightly weathered, moderately fractured.
Rock Mass Quality = Very Poor
[CASTLINE FORMATION]

R2: Bedrock: Pink with black mottles, fine-grained, GRANITE, hard, slightly weathered, highly fractured.
[CASTLINE FORMATION].
Rock Mass Quality = Very Poor.

R3: Bedrock: Pink with black mottles, fine-grained, GRANITE, hard, slightly weathered, slightly to moderately fractured.
[CASTLINE FORMATION].
Rock Mass Quality = Fair.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

2235600

BRIDGE NO. 3063
WIN
022356.00
BRIDGE PLANS

PROJ. MANAGER	ANDREW LATHE		
		BY	DATE
DESIGN-DETAILED	T. TURCOTTE	T. WOLFEL	9/2019
CHECKED-REVIEWED	K. WOOD	T. MARSHALL	9/2019
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

MILL HILL BRIDGE
MILL POND OUTLET
DEER ISLE & STONINGTON HANCOCK COUNTY

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
7
OF 40

Appendix C – Boring Logs by Kleinfelder

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: Mill Hill Bridge #3063 carries Route 15 over Mill Pond Outlet

Location: Deer Isle, ME

Boring No.: BB-SDIHP-101

WIN: 022356.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	16.28	**Auger ID/OD:**	2 1/2" Solid Stem
Operator:	Mike Porter	**Datum:**	NAVD88	**Sampler:**	NA
Logged By:	M. Chea	**Rig Type:**	Mobile Drill Truck-Mounted	**Hammer Wt./Fall:**	NA
Date Start/Finish:	5/16/19/ 5/16/19	**Drilling Method:**	Solid Stem Auger	**Core Barrel:**	NA
Boring Location:	Sta. 24+34.78, 7.08 ft Lt.	**Casing ID/OD:**	NA	**Water Level*:**	Not encountered
Hammer Efficiency Factor: NA		**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.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA ↓		▨	The probe encountered refusal at 5.4 feet below ground surface. Cuttings was observed to consist of brown granular fill.	
5								10.9		5.4	
										Bottom of Exploration at 5.4 feet below ground surface.	
10											
15											
20											
25											

Remarks:

Backfilled borehole with cuttings. Restored ground with asphalt cold patch.

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: Mill Hill Bridge #3063 carries Route 15 over Mill Pond Outlet
Location: Deer Isle, ME

Boring No.: BB-SDIHP-102

WIN: 022356.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	16.31	**Auger ID/OD:**	2 1/2" Solid Stem
Operator:	Mike Porter	**Datum:**	NAVD88	**Sampler:**	NA
Logged By:	M. Chea	**Rig Type:**	Mobile Drill Truck-Mounted	**Hammer Wt./Fall:**	NA
Date Start/Finish:	5/16/19/ 5/16/19	**Drilling Method:**	Solid Stem Auger	**Core Barrel:**	NA
Boring Location:	Sta. 24+35.75, 7.08 ft Lt.	**Casing ID/OD:**	NA	**Water Level*:**	Not Encountered
Hammer Efficiency Factor:	NA	**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		▨	The probe encountered refusal at 7.0 feet below ground surface. Cuttings was observed to consist of brown granular fill.	
5									▨		
								9.3		7.0	
										Bottom of Exploration at 7.0 feet below ground surface.	
10											
15											
20											
25											

Remarks:

Backfilled borehole with cuttings. Restored ground with asphalt cold patch.

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Page 1 of 1

Boring No.: BB-SDIHP-102

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Mill Hill Bridge #3063 carries Route 15 over Mill Pond Outlet
Location: Deer Isle, ME

Boring No.: BB-SDIHP-103

WIN: 022356.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	16.50	**Auger ID/OD:**	3.5" Hollow Stem
Operator:	Mike Porter	**Datum:**	NAVD88	**Sampler:**	NA
Logged By:	M. Chea	**Rig Type:**	Mobile Drill Truck-Mounted	**Hammer Wt./Fall:**	NA
Date Start/Finish:	5/16/19/ 5/16/19	**Drilling Method:**	Coring	**Core Barrel:**	HQ - 2.5"
Boring Location:	Sta. 24+37.27, 6.39 ft Rt.	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	11 ft
Hammer Efficiency Factor: NA		**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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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							HSA	15.9		6.8" BITUMINOUS PAVEMENT 0.6	
	C1	60/57	1.20 - 6.20				HQ			Encountered the top of concrete at about 0.9 feet. Auger through concrete to about 1.2 feet and began to core. C1: min/ft: 4.3, 3.0, 2.9, 5.4, 4.0 Light gray, CONCRETE, hard, Rec = 95% (CONCRETE)	
5											
	C2	60/56	6.20 - 11.20					8.9		C2 - min/ft: 1.5, 2.6, 2.0, 2.2, 2.4 Top 17" of C2: Similar to C1 (CONCRETE) Bottom of concrete is at about 7.6 feet. 7.6 Bottom 39" of C2: Light gray with black and white mottles, fine-grained, GRANITE SLABs, hard, seperation between adjacent blocks, Rec = 93% (ABUTMENT)	
10											
	C3	60/54	11.20 - 16.20							C3 - min/ft: 2.4, 2.4, 2.7, 3.1, 3.9 Similar to Bottom 39" of C2 except Rec = 90% (ABUTMENT)	
15											
	C4	60/46	16.20 - 21.20					-2.0		C4 - min/ft: 2.9, 3.6, 2.6, 2.8, 2.7 Top 28" of C4: Similar to Bottom 39" of C2 (ABUTMENT) Bottom of abutment is at about 18.5 feet. 18.5 Bottom 18 of C4": Light gray, CONCRETE, hard,(LEVELING CONCRETE SLAB)	
20											
	R1	48/45	21.20 - 25.20	RQD = 38%				-5.2		R1 - min/ft: 2.2, 2.2, 2.2, 3.0 Top 6" of R1: Similar to Bottom 18" of C4 (LEVELING CONCRETE SLAB) Bottom of level concrete slab is at about 21.7 feet. 21.7 Top of Bedrock at Elev. -5.2 ft Bottom 39" of R1: Bedrock: Pink with black mottles, fine-grained, GRANITE, hard, slightly weathered, moderately to highly fractured.	qp = 9245 psi
25											

Remarks:

C = Concrete or Granite Block Core
Backfilled borehole with grout. Restored ground surface with asphalt cold patch.

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: Mill Hill Bridge #3063 carries Route 15 over Mill Pond Outlet
Location: Deer Isle, ME

Boring No.: BB-SDIHP-103
WIN: 022356.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	16.50	**Auger ID/OD:**	3.5" Hollow Stem
Operator:	Mike Porter	**Datum:**	NAVD88	**Sampler:**	NA
Logged By:	M. Chea	**Rig Type:**	Mobile Drill Truck-Mounted	**Hammer Wt./Fall:**	NA
Date Start/Finish:	5/16/19/ 5/16/19	**Drilling Method:**	Coring	**Core Barrel:**	HQ - 2.5"
Boring Location:	Sta. 24+37.27, 6.39 ft Rt.	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	11 ft
Hammer Efficiency Factor:	NA	**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
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140 lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_V = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

	Sample Information										
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	R2	62/62	25.20 - 30.37	RQD = 56%			HQ ↓		▨	[CASTLINE FORMATION]. Rock Mass Quality = Poor. 93% Recovery R2: Bedrock: Pink with black mottles, fine to medium-grained, GRANITE, hard, slightly weathered, moderately fractured, healed fractures. [CASTLINE FORMATION]. Rock Mass Quality = Fair. R2: Core Times (min) 25.2-26.2 ft (2.2) 26.2-27.2 ft (1.9) 27.2-28.2 ft (2.1) 28.2-29.2 (2.0) 29.2-30.37 (2.7) 100% Recovery.	
30								-13.9		30.4 **Bottom of Exploration at 30.4 feet below ground surface.**	
35											
40											
45											
50											

Remarks:

C = Concrete or Granite Block Core
Backfilled borehole with grout. Restored ground surface with asphalt cold patch.

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: Mill Hill Bridge #3063 carries Route 15 over Mill Pond Outlet
Location: Deer Isle, ME

Boring No.: BB-SDIHP-104

WIN: 022356.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	15.0	**Auger ID/OD:**	2 1/2" Solid Stem
Operator:	Mike Porter	**Datum:**	NAVD88	**Sampler:**	NA
Logged By:	M. Chea	**Rig Type:**	Mobile Drill Truck-Mounted	**Hammer Wt./Fall:**	NA
Date Start/Finish:	5/16/19/ 5/16/19	**Drilling Method:**	Solid Stem Auger	**Core Barrel:**	NA
Boring Location:	Sta. 24+82.20, 5.76 ft Rt.	**Casing ID/OD:**	NA	**Water Level*:**	Not Encountered
Hammer Efficiency Factor:	NA	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA ↓		▨	The probe encountered refusal at 2.5 feet below ground surface. Cuttings was observed to consist of brown granular fill.	
								12.5		2.5 **Bottom of Exploration at 2.5 feet below ground surface.**	
5											
10											
15											
20											
25											

Remarks:

Backfilled borehole with cuttings. Restored ground with asphalt cold patch.

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: Mill Hill Bridge #3063 carries Route 15 over Mill Pond Outlet
Location: Deer Isle, ME

Boring No.: BB-SDIHP-105

WIN: 022356.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	14.96	**Auger ID/OD:**	2 1/2" Solid Stem
Operator:	Mike Porter	**Datum:**	NAVD88	**Sampler:**	NA
Logged By:	M. Chea	**Rig Type:**	Mobile Drill Truck-Mounted	**Hammer Wt./Fall:**	NA
Date Start/Finish:	5/16/19/ 5/16/19	**Drilling Method:**	Solid Stem Auger	**Core Barrel:**	NA
Boring Location:	Sta. 24+83.42, 5.91 ft Rt.	**Casing ID/OD:**	NA	**Water Level*:**	Not Encountered
Hammer Efficiency Factor:	NA	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☐		

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MV = Unsuccessful Field Vane Shear Test Attempt

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

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

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

Depth (ft.)	Sample Information: 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		▨	The probe encountered refusal at 7.3 feet below ground surface. Cuttings was observed to consist of brown granular fill.	
5							↓		▨		
7.3								7.7		7.3 **Bottom of Exploration at 7.3 feet below ground surface.**	
10											
15											
20											
25											

Remarks:

Backfilled borehole with cuttings. Restored ground with asphalt cold patch.

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: Mill Hill Bridge #3063 carries Route 15 over Mill Pond Outlet
Location: Deer Isle, ME

Boring No.: BB-SDIHP-106

WIN: 022356.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	15.18	**Auger ID/OD:**	3.5" Hollow Stem
Operator:	Mike Porter	**Datum:**	NAVD88	**Sampler:**	NA
Logged By:	M. Chea	**Rig Type:**	Mobile Drill Truck-Mounted	**Hammer Wt./Fall:**	NA
Date Start/Finish:	5/17/19/ 5/17/19	**Drilling Method:**	Coring	**Core Barrel:**	HQ - 2.5"
Boring Location:	Sta. 24+80.63, 5.27 ft Lt.	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	10.7 ft
Hammer Efficiency Factor:	NA	**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
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 Information							Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows				
0							HSA	14.4		10" BITUMINOUS PAVEMENT 0.8	
	C1	60/56	1.10 - 6.10				HQ			Encountered top of concrete at about 0.8 feet. Auger through concrete to about 1.1 feet and began to core. C1 - min/ft: 2.9, 3.3, 2.4, 3.8, 3.9 Light gray, CONCRETE, hard, Rec = 93% (CONCRETE) Encountered a 3 inch void at about 3.2 feet.	
5											
	C2	60/55	6.10 - 11.10					8.4		C2 - min/ft: 3.1, 2.3, 1.9, 2.5, 2.4 Top 8" of C2: Similar to C1 (CONCRETE) Bottom of concrete is at about 6.8 feet. 6.8	
										Bottom 47 of C2": Light gray with black and white mottles, fine-grained, GRANITE SLABS, hard, Rec = 92% (ABUTMENT)	
10											
	C3	60/50	11.10 - 16.10							C3 - min/ft: 3.0, 3.8, 4.6, 4.8, 2.4 Similar to Bottom 47" of C2 except Rec = 83% (ABUTMENT)	
15											
	C4	60/60	16.10 - 21.10							C4 - min/ft: 3.1, 3.9, 2.9, 3.1, 2.9 Top 28" of C4: Similar to Bottom 47" of C2 (ABUTMENT) Bottom of abutment is at about 18.4 feet.	
								-3.2		18.4 Bottom 32" of C4: Light gray, CONCRETE, hard, Rec = 100% (LEVELING CONCRETE SLAB)	
20											
	R1	60/38	21.10 - 26.10	RQD = 20%				-6.6		R1 - min/ft: 2.2, 2.0, 1.8, 3.0, 2.7 Top 8" of R1: Similar to Bottom 32" of C4 (LEVELING CONCRETE SLAB) Bottom of leveling concrete slab is at about 21.8 feet. 21.8	
										Top of Bedrock at Elev. -6.6 ft. Bottom 30" of R1: Bedrock: Pink with black mottles, fine-grained, GRANITE, hard, slightly weathered, moderately fractured.	
25							↓				

Remarks:

C = Concrete or Granite Block Core
Backfilled borehole with grout. Restored ground surface with asphalt cold patch.

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:** Mill Hill Bridge #3063 carries Route 15 over Mill Pond Outlet **Location:** Deer Isle, ME	**Boring No.:** BB-SDIHP-106 **WIN:** 022356.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	15.18	**Auger ID/OD:**	3.5" Hollow Stem
Operator:	Mike Porter	**Datum:**	NAVD88	**Sampler:**	NA
Logged By:	M. Chea	**Rig Type:**	Mobile Drill Truck-Mounted	**Hammer Wt./Fall:**	NA
Date Start/Finish:	5/17/19/ 5/17/19	**Drilling Method:**	Coring	**Core Barrel:**	HQ - 2.5"
Boring Location:	Sta. 24+80.63, 5.27 ft Lt.	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	10.7 ft
Hammer Efficiency Factor:	NA	**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 Information							Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows				
25										[CASTLINE FORMATION. Rock Mass Quality = Very Poor. 63% Recovery.	
	R2	44/18	26.10 - 29.77	RQD = 0%			HQ			R2: Bedrock: Pink with black mottles, fine grained, GRANITE, hard, slightly weathered, highly fractured. [CASTLINE FORMATION]. Rock Mass Quality = Very Poor. R2 Core Times (min) 26.1-27.1 ft (1.3) 27.1-28.1 ft (1.6) 28.1-29.1 ft (1.5) 29.1-29.77 ft (1.4) 41% Recovery.	
	R3	53/53	29.80 - 34.22	RQD = 53%						R3: Bedrock: Pink with black mottles, fine grained, GRANITE, hard, slightly weathered, slightly to moderately fractured. [CASTLINE FORMATION]. Rock Mass Quality = Fair. R3 Core Times (min) 29.8-30.8 ft (1.7) 30.8-31.8 ft (1.8) 31.8-32.8 ft (1.9)	
30											
								-19.0		32.8-33.8 ft (2.5) 33.8-34.22 ft (1.6) 100% Recovery.	
										34.2	
35										**Bottom of Exploration at 34.2 feet below ground surface.**	
40											
45											
50											

Remarks:

C = Concrete or Granite Block Core
Backfilled borehole with grout. Restored ground surface with asphalt cold patch.

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.

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)

Rock Quality Based on RQD

Rock 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 quality (very poor, poor, etc.)
ref: ASTM D6032 and FHWA NHI-16-072 GEC 5 - Geotechnical Site Characterization, Table 4-12
Recovery (inch/inch and percentage)
Rock Core Rate (X.X ft - Y.Y ft (min:sec))

Sample Container Labeling Requirements:

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

January 2020

Appendix D – Geotechnical Laboratory Test Results

Client:	Kleinfelder, Inc.				
Project:	Superstructure Replace Mill Hill Bridge				
Location:	Stonington/Deer Isle, ME			Project No:	GTX-310166
Boring ID:	BB-SDIHP-103	Sample Type:	cylinder	Tested By:	tlm
Sample ID:	C5	Test Date:	06/24/19	Checked By:	smd
Depth :	21.7-22.5	Test Id:	510079		

Test Comment: ---
Visual Description: See photograph(s)
Sample Comment: ---

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

Boring ID	**Sample Number**	**Depth**	**Bulk Density, pcf**	**Compressive strength, psi**	**Failure Type**	**Meets ASTM D4543**	**Note(s)**
BB-SDIHP-103	C5	22.01 - 22.46 ft	163	9245	1	Yes	---

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

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

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

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

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Client:	Kleinfelder, Inc.	Test Date:	6/21/2019
Project Name:	Superstructure Replace Mill Hill Bridge	Tested By:	cmh
Project Location:	Stonington/Deer Isle, ME	Checked By:	jsc
GTX #:	310166		
Boring ID:	BB-SDIHP-103		
Sample ID:	C5		
Depth:	22.01-22.46 ft		
Visual Description:	See photographs		

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

BULK DENSITY

	1	2	Average		
Specimen Length, in:	5.49	5.48	5.49		
Specimen Diameter, in:	2.48	2.48	2.48		
Specimen Mass, g:	1137.49				
Bulk Density, lb/ft^3	163			**Minimum Diameter Tolerence Met?**	YES
Length to Diameter Ratio:	2.2			**Length to Diameter Ratio Tolerance Met?**	YES

DEVIATION FROM STRAIGHTNESS (Procedure S1)

Maximum gap between side of core and reference surface plate:

Is the maximum gap ≤ 0.02 in.? YES

Maximum difference must be < 0.020 in.

Straightness Tolerance Met? YES

END FLATNESS AND PARALLELISM (Procedure FP1)

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

Difference between max and min readings, in:

0° = 0.00060 90° = 0.00050

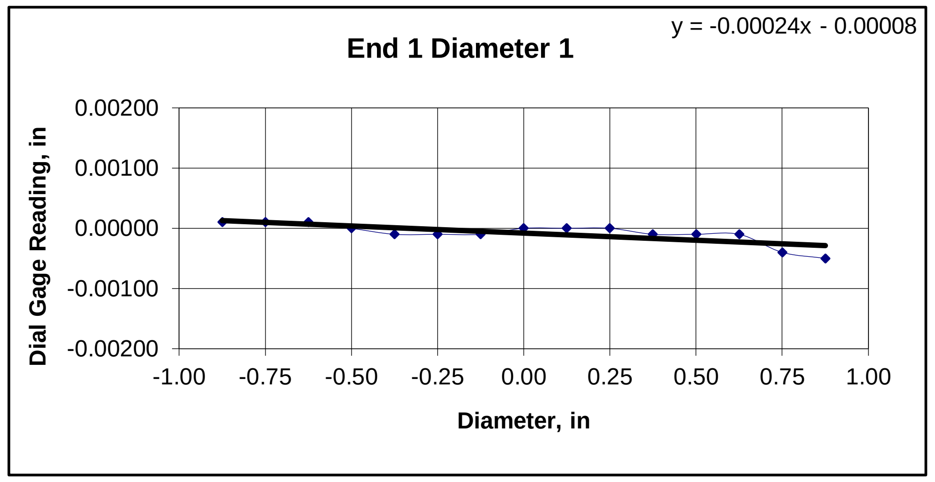
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00030	0.00020	0.00020	0.00010	0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010	-0.00010	-0.00020	-0.00030
Diameter 2, in (rotated 90°)	0.00020	0.00000	0.00000	0.00000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

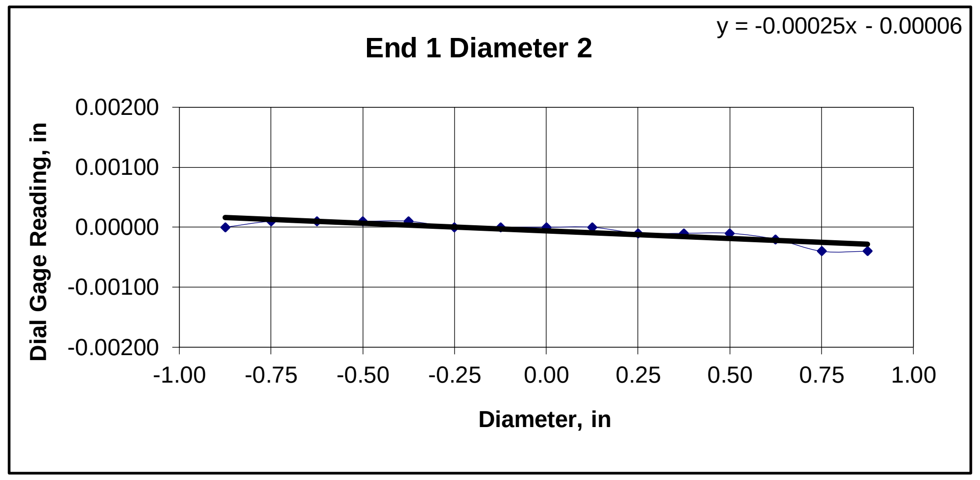
Difference between max and min readings, in:

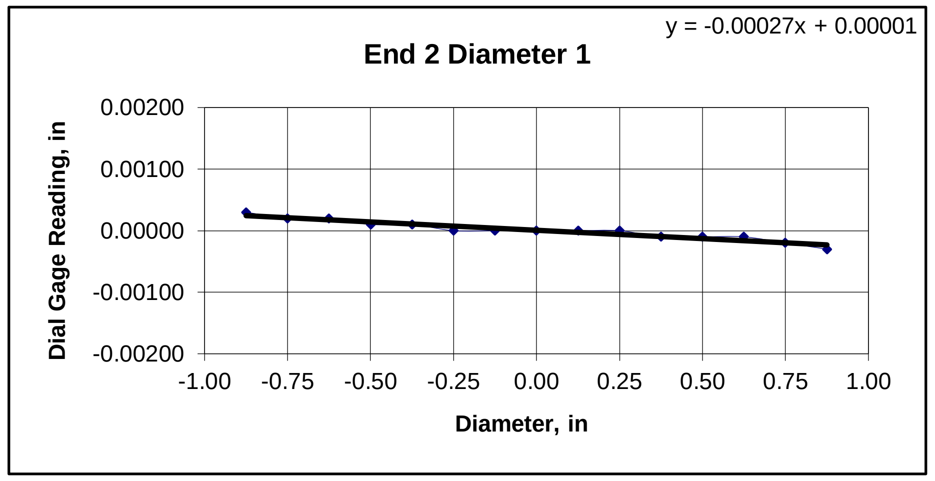
0° = 0.0006 90° = 0.0005

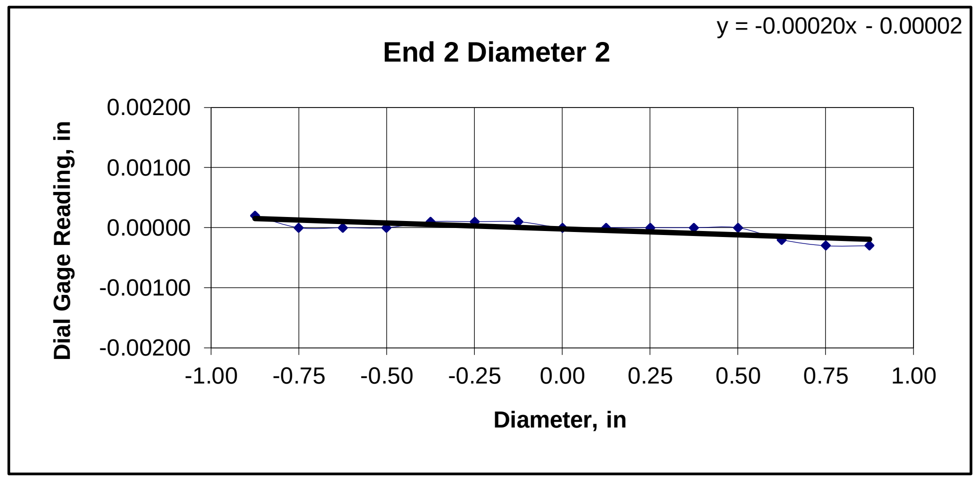
Maximum difference must be < 0.0020 in. Difference = ± 0.00030

Flatness Tolerance Met? YES


End 1 Diameter 1


End 1 Diameter 2


End 2 Diameter 1


End 2 Diameter 2

DIAMETER 1

End 1:

Slope of Best Fit Line	0.00024
Angle of Best Fit Line:	0.01359

End 2:

Slope of Best Fit Line	0.00027
Angle of Best Fit Line:	0.01555
Maximum Angular Difference:	0.00196

Parallelism Tolerance Met? YES

Spherically Seated

DIAMETER 2

End 1:

Slope of Best Fit Line	0.00025
Angle of Best Fit Line:	0.01457

End 2:

Slope of Best Fit Line	0.00020
Angle of Best Fit Line:	0.01130
Maximum Angular Difference:	0.00327

Parallelism Tolerance Met? YES

Spherically Seated

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

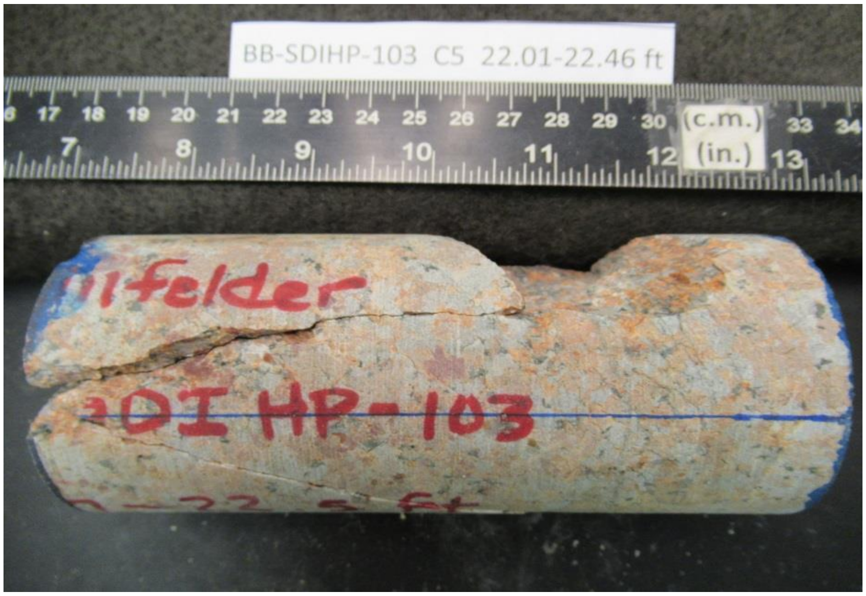
END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00060	2.480	0.00024	0.014	YES
Diameter 2, in (rotated 90°)	0.00050	2.480	0.00020	0.012	YES
END 2					
Diameter 1, in	0.00060	2.480	0.00024	0.014	YES
Diameter 2, in (rotated 90°)	0.00050	2.480	0.00020	0.012	YES

Maximum angle of departure must be ≤ 0.25°

Perpendicularity Tolerance Met? YES

Client:	Kleinfelder, Inc.
Project Name:	Superstructure Replace Mill Hill Bridge
Project Location:	Stonington/Deer Isle, ME
GTX #:	310166
Test Date:	6/24/2019
Tested By:	cmh
Checked By:	jsc
Boring ID:	BB-SDIHP-103
Sample ID:	C5
Depth, ft:	22.01-22.46

No photo available



After break

Appendix E – Geotechnical Calculations

Strength Limit State I

SHEET 1 OF 4

PROJECT NO. 20193552.001A

PROJECT Mill Hill Bridge Project

REVIEWED BY J. MacGregor DATE 6-14-19

SUBJECT Stonington-Deer Isle, Maine

BY M.Chea DATE 6/4/19

Rock Mass Rating System (Bieniawski, 1989)

RMR is based on the following factors:

① Uniaxial compressive strength of rock material
② RQD
③ Spacing of discontinuities
④ Condition of discontinuities
⑤ Groundwater conditions
⑥ Orientation of discontinuities –

- BB-SDIHP-103 – 21.7 to 25.2'

	Value (See Pg 4)
1. Rock Strength = 105 MPa	12
2. RQD = 37.5%	8
3. Spacing of discontinuities	5
4. Condition of discontinuities	25
5. Wet	7
6. Favorable	-2
Σ =	55

- BB-SDIHP-103 – 25.2' to 30.4'

	Value
1. Rock strength = 105 MPa	12
2. RQD = 56%	13
3. Spacing of discontinuities	5
4. Condition of discontinuities	25
5. Wet	7
6. Favorable	-2
Σ	60

SHEET 2 OF 4

PROJECT NO. 20193552.001A

PROJECT Mill Hill Bridge Project

REVIEWED BY J. MacGregor DATE 6-14-19

SUBJECT Stonington - Deer Isle, Maine

BY M. Chea DATE 6/4/19

◦ BB-SDIHP-106 - 21.1' to 26.1'

	Value
1. Rock Strength = 105 MPA	12
2. RQD = 20%	3
3. Spacing of discontinuties	5
4. Conditions of discontinuities	10
5. Wet	7
6. Favorable	-2
	Σ = 35

◦ BB-SDIHP-106 - 26.1' to 29.8'

	Value
1. Rock strength = 105 MPa	12
2. RQD = 0%	3
3. Spacing of discontinuties	5
4. Conditions of discontinuities	10
5. Wet	7
6. Favorable	-2
	Σ = 35

◦ BB-SDIHP-106 - 29.8' to 24.2'

	Value
1. Rock Strength ≈ 105 MPA	12
2. RQD = 53%	13
3. Spacing of discontinuities	5
4. Conditions of discontinuities	25
5. Wet	7
6. Favorable	-2
	Σ = 60

SHEET 3 OF 4

KLEINFELDER
Bright People. Right Solutions.

PROJECT NO. 20193552.001A

PROJECT Mill Hill Bridge Project

REVIEWED BY J. MacGregor DATE 6-14-19

SUBJECT Stonington-Deer Isle, ME

BY M. Chea DATE 6/4/19

- Calculate Geological Strength Index (GSI) from Hoek (1997)
  - Based on Hoek & Brown: Practical Estimates of Rock Mass Strength (1997)

- $GSI = RMR_{89} - 5$,

where RMR_{89} has groundwater rating set to 15 and adjustment for Joint orientation set to 0.

Boring	Depth (ft)	RMR	GSI
BB-SDIHP-103	21.7 - 25.2	55	50
	25.2 - 30.4	60	55
BB-SDIHP-106	21.1 - 26.1	35	30
	26.1 - 29.8	35	30
	24.8 - 34.2	60	55

- Average GSI = 44

Sheet 4 of 4

Table 4: Rock Mass Rating System (After Bieniawski 1989).

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS

	Parameter		Range of values						
1	Strength of intact rock material	Point-load strength index	>10 MPa	4 - 10 MPa	2 - 4 MPa	1 - 2 MPa	For this low range - uniaxial compressive test is preferred		
		Uniaxial comp. strength	>250 MPa	100 - 250 MPa	50 - 100 MPa	25 - 50 MPa	5 - 25 MPa	1 - 5 MPa	< 1 MPa
	Rating		15	12	7	4	2	1	0
2	Drill core Quality *RQD*		90% - 100%	75% - 90%	50% - 75%	25% - 50%	< 25%		
	Rating		20	17	13	8	3		
3	Spacing of		> 2 m	0.6 - 2 . m	200 - 600 mm	60 - 200 mm	< 60 mm		
	Rating		20	15	10	8	5		
4	Condition of discontinuities (See E)		Very rough surfaces Not continuous No separation Unweathered wall rock	Slightly rough surfaces Separation < 1 mm Slightly weathered walls	Slightly rough surfaces Separation < 1 mm Highly weathered walls	Slickensided surfaces or Gouge < 5 mm thick or Separation 1-5 mm Continuous	Soft gouge >5 mm thick or Separation > 5 mm Continuous		
	Rating		30	25	20	10	0		
5	Groundwater	Inflow per 10 m tunnel length (l/m)	None	< 10	10 - 25	25 - 125	> 125		
		(Joint water press)/ (Major principal σ)	0	< 0.1	0.1, - 0.2	0.2 - 0.5	> 0.5		
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing		
	Rating		15	10	7	4	0		

B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS (See F)

Strike and dip orientations		Very favourable	Favourable	Fair	Unfavourable	Very Unfavourable
Ratings	Tunnels & mines	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	

C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS

Rating	100 ← 81	80 ← 61	60 ← 41	40 ← 21	< 21
Class number	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

D. MEANING OF ROCK CLASSES

Class number	I	II	III	IV	V
Average stand-up time	20 yrs for 15 m span	1 year for 10 m span	1 week for 5 m span	10 hrs for 2.5 m span	30 min for 1 m span
Cohesion of rock mass (kPa)	> 400	300 - 400	200 - 300	100 - 200	< 100
Friction angle of rock mass (deg)	> 45	35 - 45	25 - 35	15 - 25	< 15

E. GUIDELINES FOR CLASSIFICATION OF DISCONTINUITY conditions

Discontinuity length (persistence) Rating	< 1 m 6	1 - 3 m 4	3 - 10 m 2	10 - 20 m 1	> 20 m 0
Separation (aperture) Rating	None 6	< 0.1 mm 5	0.1 - 1.0 mm 4	1 - 5 mm 1	> 5 mm 0
Roughness Rating	Very rough 6	Rough 5	Slightly rough 3	Smooth 1	Slickensided 0
Infilling (gouge) Rating	None 6	Hard filling < 5 mm 4	Hard filling > 5 mm 2	Soft filling < 5 mm 2	Soft filling > 5 mm 0
Weathering Ratings	Unweathered 6	Slightly weathered 5	Moderately weathered 3	Highly weathered 1	Decomposed 0

F. EFFECT OF DISCONTINUITY STRIKE AND DIP ORIENTATION IN TUNNELLING**

Strike perpendicular to tunnel axis		Strike parallel to tunnel axis	
Drive with dip - Dip 45 - 90°	Drive with dip - Dip 20 - 45°	Dip 45 - 90°	Dip 20 - 45°
Very favourable	Favourable	Very unfavourable	Fair
Drive against dip - Dip 45-90°	Drive against dip - Dip 20-45°	Dip 0-20 - Irrespective of strike°	
Fair	Unfavourable	Fair	

* Some conditions are mutually exclusive . For example, if infilling is present, the roughness of the surface will be overshadowed by the influence of the gouge. In such cases use A.4 directly.

** Modified after Wickham et al (1972).

Objective: Calculate the allowable bearing Resistance below the south (No. 2) abument

References:
Das, Braja M. (2002), "Principles of Geotechnical Engineering." Pacific Grove, CA. 5th Edition.
AASHTO Standard Specifications for Highway Bridges, 8th Edition, 2017

Design Basis: The bearing resistance was determined per AASHTO LRFD Section 4.4.7.1.2 with extreme event and service limit state resistance factor of 1.0 and strength limit state resistance factor of 0.45 for the semi-empirical method of all soil.

Given: The subsurface conditions are based on borings BB-SDIHP-103 advanced through the south abutment. The borings were advanced by New England Boring Contactor and observed by Kleinfelder. The boring logs did not provide stabilized or long term groundwater data, therefore assume groundwater at the pond water level.

Based on the conditions observed in boring BB-SDIHP-103, the south abutment bears on concrete which directly bears on Granite.

Assumptions: The rock strength parameters for the Granite were developed in in Calc-001.

Abutment geometry is assumed based on the section of No. 2 abutment shown in sheet 3 of the 1938 Drawings (Attachment 1).

Calculation:

FOUNDATION GEOMETRY- South Abutment

$B := 10.9\ ft$	Abutment Width
$L := 22.9\ ft$	Abutment Length - 11' 5" * 2
$D_{footing} := 18.5\ ft$	Based on boring BB-SDIHP-103
$D_{scour} := 0\ ft$	scour depth - Bearing on rock so assume 0 ft
$D_f := 0\ ft$	Conservatively, assumed abutment Embedment Depth is 0 ft

Calc By: M. Chea 6/13/2019
Check By: JM 9/12/19 & MR 6/8/22

SOIL and ROCK PROPERTIES

OVERBURDEN SOIL PROPERTIES

$\gamma_w := 62.4 \cdot pcf$ (Unit Weight of Water)

$\gamma_0 := 120 \cdot pcf$ (Unit Weight of overburnden above foundation base)

$D_w := 0\ ft$ (Depth to Groundwater from ground surface on canal side)

$\gamma_{0b} := \gamma_0 - \gamma_w = 57.6\ pcf$

ROCK PROPERTIES

$\gamma_1 := 160\ pcf$ Dry unit weight

$\gamma' := \gamma_1 - \gamma_w = 97.6\ pcf$ Effective Unit Weight of Foundation Soils

$q_u := 9.245\ ksi$ Uniaxial compressive strength of intact rock

Hoek-Brown Parameters (refer to Calc-001)

$s := 0.0039$

$a := 0.506$

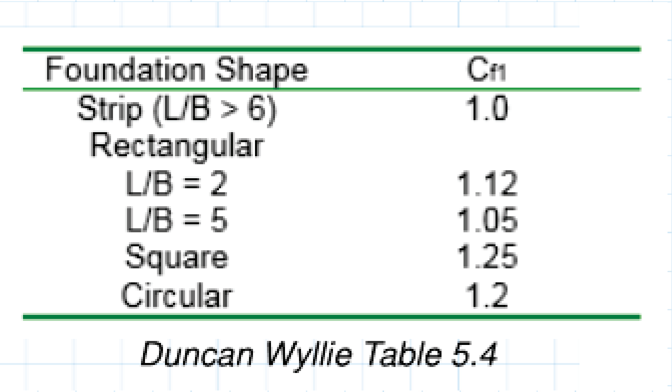
$m_b := 5.366$

BEARING RESISTANCE (Based on Wyllie, 1999)

Enter Correction Factor from Duncan Wyllie Table 5.4

$\frac{L}{B} = 2.1$

$C_{f1} := 1.12$



Foundation Shape	C_{f1}
Strip (L/B > 6)	1.0
Rectangular	
L/B = 2	1.12
L/B = 5	1.05
Square	1.25
Circular	1.2

Duncan Wyllie Table 5.4

Bearing Resistance (ignoring effects of footing embedment):

Nominal Bearing Resistance

$$q_{n\_unconfined} := C_{f1} \cdot s^{0.5} \cdot q_u \cdot \left(1 + \left(m_b \cdot s^{-0.5} + 1\right)^{0.5}\right) = 961.26\ ksf$$

Factored Bearing Resistance for extreme limit state (resistance factor = 1.0)

$$q_{R\_service\_unconfined} := q_{n\_unconfined} \cdot 1 = 961.3\ ksf$$

Factored Bearing Resistance for strength limit state (resistance factor = 0.45)

$$q_{n\_strength\_unconfined} := q_{n\_unconfined} \cdot 0.45 = 432.57\ ksf$$

Objective: Calculate the nominal and factored bearing resistance below the north (No. 1) abument

References:
Das, Braja M. (2002), "Principles of Geotechnical Engineering." Pacific Grove, CA. 5th Edition.
AASHTO Standard Specifications for Highway Bridges, 8th Edition, 2017

Design Basis: The bearing resistance was determined per AASHTO LRFD Section 4.4.7.1.2 with service and extreme event limit state resistance factor of 1.0 and strength limit state resistance factor of 0.45 for the semi-empirical method of all soil.

Given: The subsurface conditions are based on borings BB-SDIHP-106 advanced through the south abutment. The borings were advanced by New England Boring Contactor and observed by Kleinfelder. The boring logs did not provide stabilized or long term groundwater data, therefore assume groundwater at the pond water level.

Based on the conditions observed in boring BB-SDIHP-106, the north abutment bears on concrete which directly bears on Granite.

Assumptions: The rock strength parameters for the Granite were developed in in Calc-001.

Abutment geometry is assumed based on the section of No. 1 abutment shown in sheet 3 of the 1938 Drawings.

Calculation:

FOUNDATION GEOMETRY- South Abutment

$B := 10.4\ ft$	Based on 1938 drawings
$L := 23.0\ ft$	Based on 1938 drawings
$D_{footing} := 18.4\ ft$	Based on boring BB-SDIHP-106
$D_{scour} := 0\ ft$	(Assumed scour depth at pier)
$D_f := 0\ ft$	Conservatively, assumed 0 ft Embedment Depth

Calc By: M. Chea 6/13/2019
Check By: JM 9/12/19 & MR 6/8/22

SOIL PROPERTIES

OVERBURDEN SOIL PROPERTIES

$\gamma_w := 62.4 \cdot pcf$ (Unit Weight of Water)

$\gamma_0 := 120 \cdot pcf$ (Unit Weight of overburnden above foundation base)

$D_w := 0\ ft$ (Depth to Groundwater from ground surface on canal side)

$\gamma_{0b} := \gamma_0 - \gamma_w = 57.6\ pcf$

ROCK PROPERTIES

$\gamma_1 := 160\ pcf$ Dry unit weight

$\gamma' := \gamma_1 - \gamma_w = 97.6\ pcf$ Effective Unit Weight of Foundation Soils

$q_u := 9.245\ ksi$ Uniaxial compressive strength of intact rock

Hoek-Brown Parameters (refer to Calc-001)

$s := 0.0039$

$a := 0.506$

$m_b := 5.366$

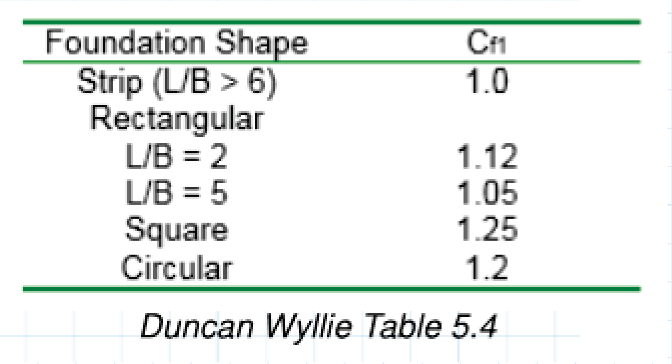
Calc By: M. Chea 6/13/2019
Check By: JM 9/12/19 & MR 6/8/22

BEARING RESISTANCE (Based on Wyllie, 1999)

Enter Correction Factor from Duncan Wyllie Table 5.4

$$\frac{L}{B}=2.2$$

$$C_{f1}:=1.12$$



Foundation Shape	C_{f1}
Strip (L/B > 6)	1.0
Rectangular	
L/B = 2	1.12
L/B = 5	1.05
Square	1.25
Circular	1.2

Duncan Wyllie Table 5.4

Bearing Resistance (ignoring effects of footing embedment):

Nominal Bearing Resistance

$$q_{n\_unconfined}:=C_{f1}\cdot s^{0.5}\cdot q_u\cdot\left(1+\left(m_b\cdot s^{-0.5}+1\right)^{0.5}\right)=961.26\ \mathbf{ksf}$$

Factored Bearing Resistance for extreme limit state (resistance factor = 1.0)

$$q_{R\_service\_unconfined}:=q_{n\_unconfined}\cdot 1=961.26\ \mathbf{ksf}$$

Factored Bearing Resistance for strength limit state (resistance factor = 0.45)

$$q_{R\_Strength\_unconfined}:=q_{n\_unconfined}\cdot 0.45=432.57\ \mathbf{ksf}$$

Calc-002c Service Limit State Bearing Capacity

Abutments bear on concrete leveling pad on granite
Granite Rock quality Poor to Fair
recommend 70 ksf

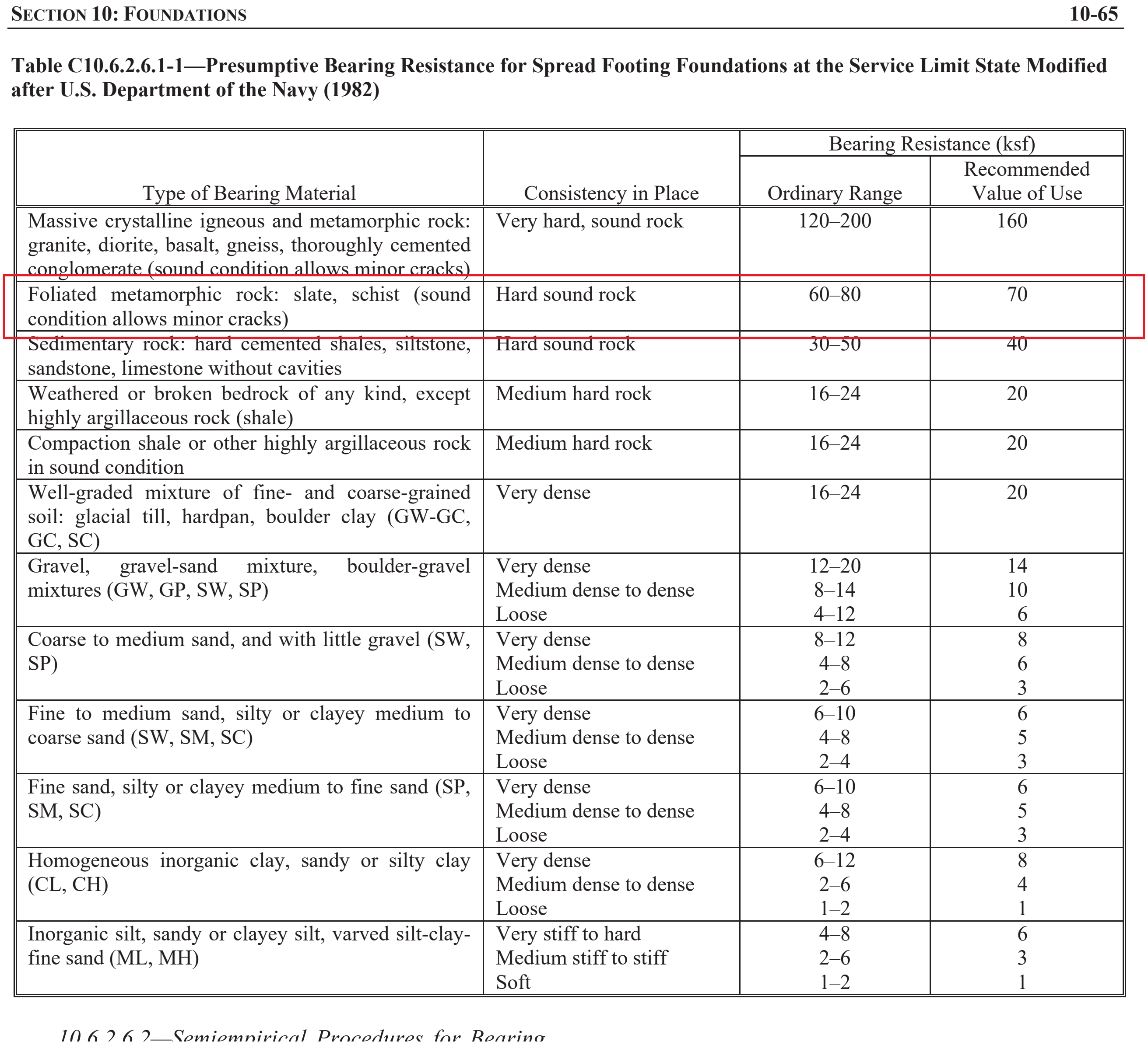


Table C10.6.2.6.1-1—Presumptive Bearing Resistance for Spread Footing Foundations at the Service Limit State Modified after U.S. Department of the Navy (1982)

Type of Bearing Material	Consistency in Place	Bearing Resistance (ksf)	
		Ordinary Range	Recommended Value of Use
Massive crystalline igneous and metamorphic rock: granite, diorite, basalt, gneiss, thoroughly cemented conglomerate (sound condition allows minor cracks)	Very hard, sound rock	120–200	160
Foliated metamorphic rock: slate, schist (sound condition allows minor cracks)	Hard sound rock	60–80	70
Sedimentary rock: hard cemented shales, siltstone, sandstone, limestone without cavities	Hard sound rock	30–50	40
Weathered or broken bedrock of any kind, except highly argillaceous rock (shale)	Medium hard rock	16–24	20
Compaction shale or other highly argillaceous rock in sound condition	Medium hard rock	16–24	20
Well-graded mixture of fine- and coarse-grained soil: glacial till, hardpan, boulder clay (GW-GC, GC, SC)	Very dense	16–24	20
Gravel, gravel-sand mixture, boulder-gravel mixtures (GW, GP, SW, SP)	Very dense Medium dense to dense Loose	12–20 8–14 4–12	14 10 6
Coarse to medium sand, and with little gravel (SW, SP)	Very dense Medium dense to dense Loose	8–12 4–8 2–6	8 6 3
Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)	Very dense Medium dense to dense Loose	6–10 4–8 2–4	6 5 3
Fine sand, silty or clayey medium to fine sand (SP, SM, SC)	Very dense Medium dense to dense Loose	6–10 4–8 2–4	6 5 3
Homogeneous inorganic clay, sandy or silty clay (CL, CH)	Very dense Medium dense to dense Loose	6–12 2–6 1–2	8 4 1
Inorganic silt, sandy or clayey silt, varved silt-clay-fine sand (ML, MH)	Very stiff to hard Medium stiff to stiff Soft	4–8 2–6 1–2	6 3 1

10.6.2.6.2—Semiempirical Procedures for Bearing

Factored Bearing Resistance of Strip Footing

Project: ME DOT Mill Hill Bridge
Location: Deer Isle, Maine
Calculated By: MR **Date:** 4/27/2022
Checked By: MK **Date:** 6/23/2022

Objective: Evaluate Nominal and Factored bearing resistance for the proposed Mill Hill Bridge retaining walls in Deer Isle Maine. Calculate the bearing resistance for wall foundation base width between 6 and 9 feet, for eccentricity varying from B/3 to B/12, with B= foundation base width.

References: 1) Soil Mechanics in Engineering Practice - Terzaghi, Peck and Mesri
2) MED DOT Bridge Manual
3) AASHTO LRFD Bridge Specifications
4) Boring Logs by Kleinfelder

Assumptions: 1) Assume rigid foundation.
2) Assume maximum eccentricity (1/3 footing width) for footings bearing on soil.
3) Assume strip footing of Length of 20 feet based on existing 60% drawings
4) Wall geometry as defined in the attached cross sections
5) Assume granular material (gravel borrow) above bedrock

Analysis: 1. Evaluate the soil stratigraphy, soil properties and groundwater elevation:
Soil stratigraphy and soil properties assumed based on granular backfill/sand to top of bedrock
groundwater elevation was assumed to be MHHW - El. 5+/-
2. Evaluate problem geometry, bottom of footing elevation and depth of embedment based on plans from kleinfelder
Bottom of footing elevation was provided by Kleinfelder's structural engineer
See attached cross section for the assumed problem geometry and the evaluated cross section
3. Evaluate the nominal bearing resistance using the theoretical method (Munfakh et al., 2001)
The case of footing on sloping ground for cohesionless soil was considered
Meyerhof factor $N\gamma q$ was evaluated for sloping ground from fig. 10.6.3.1.2c-2
4. Evaluate the factored bearing resistance using resistance factor in AASHTO LRFD table 10.5.5.2.2-1

Bearing Resistance Results: Results of the bearing resistance analysis, for strip footings at SLS1, are summarized in the table below

Eccentricity (B - ft)	Factored BR as a function of the footing width (ksf)				**Footing Embedm. (ft)**	**GW El (ft)**	**Resistance Factor**
	6 ft	7 ft	8 ft	9 ft			
B/3	1.2	1.4	1.6	1.7	10.5	5	0.45
B/6	2.5	2.8	3.2	3.5			
B/9	2.9	3.3	3.7	4.1			
B/12	3.1	3.6	4.0	4.4			

Eccentricity assumed to be B/3 (max allowed by AASHTO LRFD) to B/12
Footing embedment from BOF El provided by structural eng., and problem geometry
Factored BR based on theoretical method (Munfakh et al., 2001)
Resistance factor from AASHTO LRFD table 10.5.5.2.2-1

Once a value of the factored BR is selected, the structural engineer should perform the required overturning check and verify that the eccentricity is less then or equal to the corresponding value of the eccentricity assumed in the BR calculations. For example, if a value of 4.7 ksf is selected for B=9, the eccentricity must be up to 9/12=0.75 feet

Conclusions: The factored bearing resistance of the proposed wall, with the dimensions by Kleinfelder's structural engineer, as shown in the attached cross sections, is as indicated in the above summary table, depending on the assumed footing width and eccentricity values

Attachments: 1) Bearing resistance calculations
2) Plans and cross sections

Factored Bearing Resistance of Strip Footing

Project: ME DOT Mill Hill Bridge
Location: Deer Isle, maine
Calculated By: MR **Date:** 4/27/2022
Checked By: MK **Date:** 6/23/2022

ASSUMPTIONS AND INPUT

			Basis for Assumption (if applicable)
Sand Unit Weight, γ =	135	pcf	Assumed - Gravel Borrow Backfill - See ME DOT Table 3-3
Friction Angle, ϕ =	36	degrees	
Cohesion Factor, N_{cq} =	N/A		Cohesion factor not selected (c=0); Embedment factor =0 per AASHTO LRFD 10.6.3.1.2c-1; Weight factor evaluated from figure 10.6.3.1.2c-2
Embedment Factor, N_q =	0		
Unit Weight Factor, $N_{\gamma q}$ =	46.6		
Cohesion, c =	0	psf	Assumed granular material (sand/backfill) above top of BR
Depth to Groundwater, D_w =	10.5	ft	Assumed MHHW El.5 +/-
FTG Embedment Depth, D_f =	10.5	ft	Evaluated from attached cross section and BOF El. 5+/-
Resistance Factor RF =	0.45		

CALCULATIONS

- Account for Groundwater using factors C_{wq} and $C_{w\gamma}$ using Table 10.6.3.1.2a-2
- Account for sloping ground by replacing N_q and N_γ with factors N_{cq} and $N_{c\gamma}$ in accordance with Section 10.6.3.1.2c (Nq=0)
- Shape Correction Factors s_c, s_γ, and s_q determined using equation in AASHTO Table 10.6.3.1.2a-3
- Load inclination factors i_c, i_γ, and i_q assumed to be 1.0 => Vertical load
- Depth Correction Factor d_q estimated using AASHTO Table 10.6.3.1.2a-4
- Nominal Bearing Resistance, q_n, determined using equation 10.6.3.1.2a-1

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + 0.5\gamma B' N_{\gamma m} C_{w\gamma}$$

where: $N_{qm} = N_q s_q d_q i_q = 0$ (being Nq=0)

$N_{\gamma m} = N_{\gamma q} s_\gamma i_\gamma$

$N_{cm} = N_c s_c i_c$ = N/A (being C=0)

- Allowable Bearing Capacity: $q_f = q_n \times RF$

DETERMINE BEARING RESISTANCE, q_r FOR STRIP FOOTING ASSUMING FOOTING DEPTH D = 6FT

B ft	L ft	eB ft	eL ft	B' ft	L' ft	B'/L'	D_f/B'	C_{wq}	$C_{w\gamma}$
6	20	2.0	6.7	2.0	6.7	0.30	5.25	1.0	0.50
6	20	1.0	3.3	4.0	13.3	0.30	2.63	1.0	0.50
6	20	0.7	2.2	4.7	15.6	0.30	2.25	1.0	0.50
6	20	0.5	1.7	5.0	16.7	0.30	2.10	1.0	0.50

s_c (if applicable)	s_γ	s_q	d_q
N/A	0.88	1.00	N/A
	0.88	1.00	
	0.88	1.00	
	0.88	1.00	

B [ft]	N_q	$N_{\gamma q}$	N_c	q_n [psf]	**q_r** **[ksf]**
6	0.0	46.6	N/A	2,768	**1.2**
6	0.0	46.6	N/A	5,536	**2.5**
6	0.0	46.6	N/A	6,459	**2.9**
6	0.0	46.6	N/A	6,920	**3.1**

Factored Bearing Resistance of Strip Footing

Project: ME DOT Mill Hill Bridge
Location: Deer Isle, maine
Calculated By: MR **Date:** 4/27/2022
Checked By: MK **Date:** 6/23/2022

ASSUMPTIONS AND INPUT

			Basis for Assumption (if applicable)
Sand Unit Weight, γ =	135	pcf	Assumed - Gravel Borrow Backfill - See ME DOT Table 3-3
Friction Angle, ϕ =	36	degrees	
Cohesion Factor, N_{cq} =	N/A		Cohesion factor not selected (c=0); Embedment factor =0 per AASHTO LRFD 10.6.3.1.2c-1; Weight factor evaluated from figure 10.6.3.1.2c-2
Embedment Factor, N_q =	0		
Unit Weight Factor, $N_{\gamma q}$ =	46.6		
Cohesion, c =	0	psf	Assumed granular material (sand/backfill) above top of BR
Depth to Groundwater, D_w =	10.5	ft	Assumed MHHW El.5 +/-
FTG Embedment Depth, D_f =	10.5	ft	Evaluated from attached cross section and BOF El. 5+/-
Resistance Factor RF =	0.45		

CALCULATIONS

- Account for Groundwater using factors C_{wq} and $C_{w\gamma}$ using Table 10.6.3.1.2a-2
- Account for sloping ground by replacing N_q and N_γ with factors N_{cq} and $N_{c\gamma}$ in accordance with Section 10.6.3.1.2c (Nq=0)
- Shape Correction Factors s_c, s_γ, and s_q determined using equation in AASHTO Table 10.6.3.1.2a-3
- Load inclination factors i_c, i_γ, and i_q assumed to be 1.0 => Vertical load
- Depth Correction Factor d_q estimated using AASHTO Table 10.6.3.1.2a-4
- Nominal Bearing Resistance, q_n, determined using equation 10.6.3.1.2a-1

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + 0.5\gamma B' N_{\gamma m} C_{w\gamma}$$

where:

$$N_{qm} = N_q s_q d_q i_q = 0 \text{ (being Nq=0)}$$

$$N_{\gamma m} = N_{\gamma q} s_\gamma i_\gamma$$

$$N_{cm} = N_c s_c i_c = \text{N/A (being C=0)}$$

- Allowable Bearing Capacity: $q_f = q_n \times RF$

DETERMINE BEARING RESISTANCE, q_r FOR STRIP FOOTING ASSUMING FOOTING DEPTH D = 7FT

B ft	L ft	eB ft	eL ft	B' ft	L' ft	B'/L'	D_f/B'	C_{wq}	$C_{w\gamma}$
7	20	2.3	6.7	2.3	6.7	0.35	4.50	1.0	0.50
7	20	1.2	3.3	4.7	13.3	0.35	2.25	1.0	0.50
7	20	0.8	2.2	5.4	15.6	0.35	1.93	1.0	0.50
7	20	0.6	1.7	5.8	16.7	0.35	1.80	1.0	0.50

s_c (if applicable)	s_γ	s_q	d_q
N/A	0.86	1.00	N/A
	0.86	1.00	
	0.86	1.00	
	0.86	1.00	

B [ft]	N_q	$N_{\gamma q}$	N_c	q_n [psf]	**q_r** **[ksf]**
7	0.0	46.6	N/A	3,156	**1.4**
7	0.0	46.6	N/A	6,312	**2.8**
7	0.0	46.6	N/A	7,364	**3.3**
7	0.0	46.6	N/A	7,890	**3.6**

Factored Bearing Resistance of Strip Footing

Project: ME DOT Mill Hill Bridge
Location: Deer Isle, maine
Calculated By: MR **Date:** 4/27/2022
Checked By: MK **Date:** 6/23/2022

ASSUMPTIONS AND INPUT

			Basis for Assumption (if applicable)
Sand Unit Weight, γ =	135	pcf	Assumed - Gravel Borrow Backfill - See ME DOT Table 3-3
Friction Angle, ϕ =	36	degrees	
Cohesion Factor, N_{cq} =	N/A		Cohesion factor not selected (c=0); Embedment factor =0 per AASHTO LRFD 10.6.3.1.2c-1; Weight factor evaluated from figure 10.6.3.1.2c-2
Embedment Factor, N_q =	0		
Unit Weight Factor, $N_{\gamma q}$ =	46.6		
Cohesion, c =	0	psf	Assumed granular material (sand/backfill) above top of BR
Depth to Groundwater, D_w =	10.5	ft	Assumed MHHW El.5 +/-
FTG Embedment Depth, D_f =	10.5	ft	Evaluated from attached cross section and BOF El. 5+/-
Resistance Factor RF =	0.45		

CALCULATIONS

- Account for Groundwater using factors C_{wq} and $C_{w\gamma}$ using Table 10.6.3.1.2a-2
- Account for sloping ground by replacing N_q and N_γ with factors N_{cq} and $N_{c\gamma}$ in accordance with Section 10.6.3.1.2c (Nq=0)
- Shape Correction Factors s_c, s_γ, and s_q determined using equation in AASHTO Table 10.6.3.1.2a-3
- Load inclination factors i_c, i_γ, and i_q assumed to be 1.0 => Vertical load
- Depth Correction Factor d_q estimated using AASHTO Table 10.6.3.1.2a-4
- Nominal Bearing Resistance, q_n, determined using equation 10.6.3.1.2a-1

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + 0.5\gamma B' N_{\gamma m} C_{w\gamma}$$

where: $N_{qm} = N_q s_q d_q i_q = 0$ (being Nq=0)

$N_{\gamma m} = N_{\gamma q} s_\gamma i_\gamma$

$N_{cm} = N_c s_c i_c$ = N/A (being C=0)

- Allowable Bearing Capacity: $q_f = q_n \times RF$

DETERMINE BEARING RESISTANCE, q_r FOR STRIP FOOTING ASSUMING FOOTING DEPTH D = 8FT

B ft	L ft	eB ft	eL ft	B' ft	L' ft	B'/L'	D_f/B'	C_{wq}	$C_{w\gamma}$
8	20	2.7	6.7	2.7	6.7	0.40	3.94	1.0	0.50
8	20	1.3	3.3	5.3	13.3	0.40	1.97	1.0	0.50
8	20	0.9	2.2	6.2	15.6	0.40	1.69	1.0	0.50
8	20	0.7	1.7	6.7	16.7	0.40	1.58	1.0	0.50

s_c (if applicable)	s_γ	s_q	d_q
N/A	0.84	1.00	N/A
	0.84	1.00	
	0.84	1.00	
	0.84	1.00	

B [ft]	N_q	$N_{\gamma q}$	N_c	q_n [psf]	**q_r [ksf]**
8	0.0	46.6	N/A	3,523	**1.6**
8	0.0	46.6	N/A	7,046	**3.2**
8	0.0	46.6	N/A	8,220	**3.7**
8	0.0	46.6	N/A	8,807	**4.0**

Factored Bearing Resistance of Strip Footing

Project: ME DOT Mill Hill Bridge
Location: Deer Isle, maine
Calculated By: MR **Date:** 4/27/2022
Checked By: MK **Date:** 6/23/2022

ASSUMPTIONS AND INPUT

			Basis for Assumption (if applicable)
Sand Unit Weight, γ =	135	pcf	Assumed - Gravel Borrow Backfill - See ME DOT Table 3-3
Friction Angle, ϕ =	36	degrees	
Cohesion Factor, N_{cq} =	N/A		Cohesion factor not selected (c=0); Embedment factor =0 per AASHTO LRFD 10.6.3.1.2c-1; Weight factor evaluated from figure 10.6.3.1.2c-2
Embedment Factor, N_q =	0		
Unit Weight Factor, $N_{\gamma q}$ =	46.6		
Cohesion, c =	0	psf	Assumed granular material (sand/backfill) above top of BR
Depth to Groundwater, D_w =	10.5	ft	Assumed MHHW El.5 +/-
FTG Embedment Depth, D_f =	10.5	ft	Evaluated from attached cross section and BOF El. 5+/-
Resistance Factor RF =	0.45		

CALCULATIONS

- Account for Groundwater using factors C_{wq} and $C_{w\gamma}$ using Table 10.6.3.1.2a-2
- Account for sloping ground by replacing N_q and N_γ with factors N_{cq} and $N_{c\gamma}$ in accordance with Section 10.6.3.1.2c (Nq=0)
- Shape Correction Factors s_c, s_γ, and s_q determined using equation in AASHTO Table 10.6.3.1.2a-3
- Load inclination factors i_c, i_γ, and i_q assumed to be 1.0 => Vertical load
- Depth Correction Factor d_q estimated using AASHTO Table 10.6.3.1.2a-4
- Nominal Bearing Resistance, q_n, determined using equation 10.6.3.1.2a-1

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + 0.5\gamma B' N_{\gamma m} C_{w\gamma}$$

where:
$N_{qm} = N_q s_q d_q i_q = 0$ (being Nq=0)
$N_{\gamma m} = N_{\gamma q} s_\gamma i_\gamma$
$N_{cm} = N_c s_c i_c$ = N/A (being C=0)

- Allowable Bearing Capacity: $q_f = q_n \times RF$

DETERMINE BEARING RESISTANCE, q_r FOR STRIP FOOTING ASSUMING FOOTING DEPTH D = 9FT

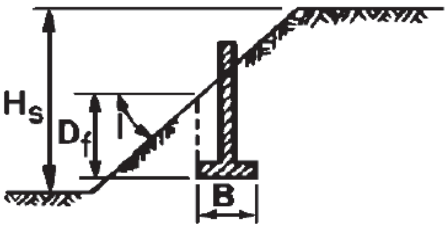
B ft	L ft	eB ft	eL ft	B' ft	L' ft	B'/L'	D_f/B'	C_{wq}	$C_{w\gamma}$
9	20	3.0	6.7	3.0	6.7	0.45	3.50	1.0	0.50
9	20	1.5	3.3	6.0	13.3	0.45	1.75	1.0	0.50
9	20	1.0	2.2	7.0	15.6	0.45	1.50	1.0	0.50
9	20	0.8	1.7	7.5	16.7	0.45	1.40	1.0	0.50

s_c (if applicable)	s_γ	s_q	d_q
N/A	0.82	1.00	N/A
	0.82	1.00	
	0.82	1.00	
	0.82	1.00	

B [ft]	N_q	$N_{\gamma q}$	N_c	q_n [psf]	**q_r [ksf]**
9	0.0	46.6	N/A	3,869	**1.7**
9	0.0	46.6	N/A	7,738	**3.5**
9	0.0	46.6	N/A	9,028	**4.1**
9	0.0	46.6	N/A	9,672	**4.4**

Factored Bearing Resistance of Strip Footing

Project: ME DOT Mill Hill Bridge
Location: Deer Isle, maine
Calculated By: MR **Date:** 4/27/2022
Checked By: MK **Date:** 6/23/2022



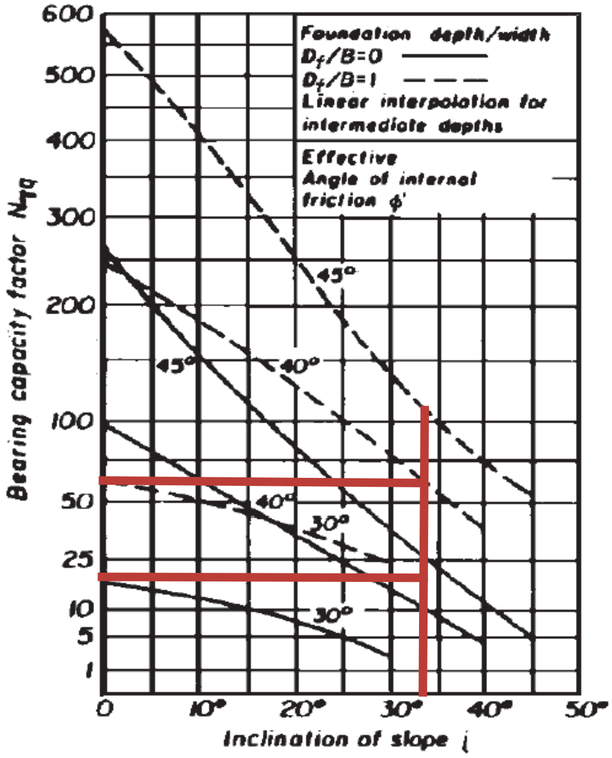
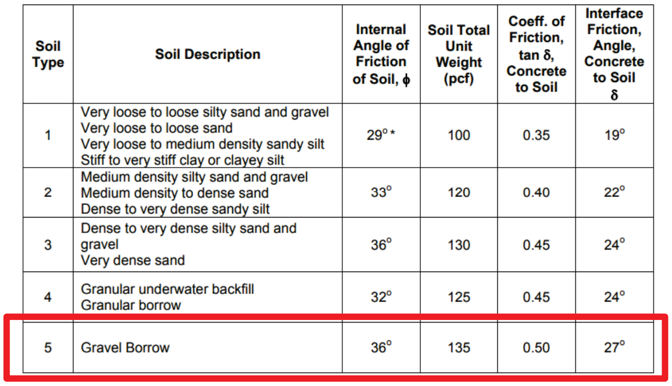


Table 3-3 Material Classification



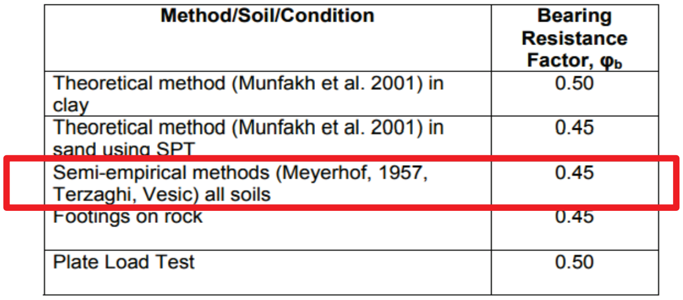
Soil Type	Soil Description	Internal Angle of Friction of Soil, φ	Soil Total Unit Weight (pcf)	Coeff. of Friction, tan δ, Concrete to Soil	Interface Friction, Angle, Concrete to Soil δ
1	Very loose to loose silty sand and gravel Very loose to loose sand Very loose to medium density sandy silt Stiff to very stiff clay or clayey silt	29°*	100	0.35	19°
2	Medium density silty sand and gravel Medium density to dense sand Dense to very dense sandy silt	33°	120	0.40	22°
3	Dense to very dense silty sand and gravel Very dense sand	36°	130	0.45	24°
4	Granular underwater backfill Granular borrow	32°	125	0.45	24°
5	Gravel Borrow	36°	135	0.50	27°

* The value given for the internal angle of friction (φ) for stiff to very stiff silty clay or clayey silt should be used with caution due to the large possible variation with different moisture contents.

Df=10.5 B= 6ft, 7ft, 8ft, 9ft
=> Df/B greater than or equal to 1
=> Use dotted lines for Df/B=1
=> Interpolate f'=40 deg and f'=30 deg

φ'(deg) = 36
Nγq (30)= 22
Nγq (40)= 63
Nγq (36)= 46.6

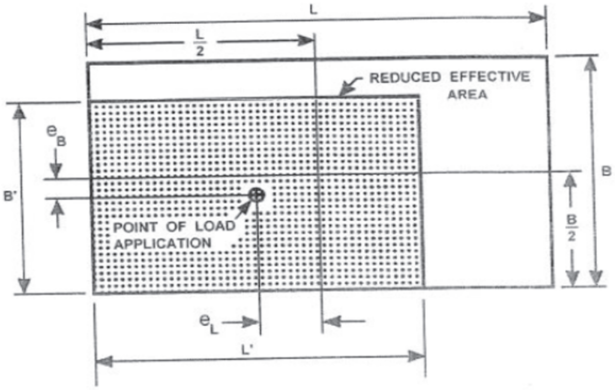
Table 5-2 Bearing Resistance Factors



Method/Soil/Condition	Bearing Resistance Factor, φ_b
Theoretical method (Munfakh et al. 2001) in clay	0.50
Theoretical method (Munfakh et al. 2001) in sand using SPT	0.45
Semi-empirical methods (Meyerhof, 1957, Terzaghi, Vesic) all soils	0.45
Footings on rock	0.45
Plate Load Test	0.50

Factored Bearing Resistance of Strip Footing

Project: ME DOT Mill Hill Bridge
Location: Deer Isle, maine
Calculated By: MR **Date:** 4/27/2022
Checked By: MK **Date:** 6/23/2022



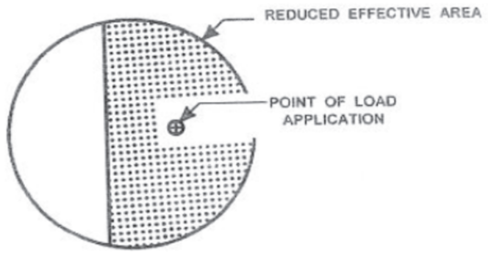


Figure C10.6.1.3-1—Reduced Footing Dimensions

$$B' = B - 2e_B \qquad (10.6.1.3\text{-}1)$$

$$L' = L - 2e_L$$

where:

e_B = eccentricity parallel to dimension B (ft)

e_L = eccentricity parallel to dimension L (ft)

10.6.3.3—Eccentric Load Limitations

The eccentricity of loading at the strength limit state, evaluated based on factored loads shall not exceed:

- One-third of the corresponding footing dimension, B or L, for footings on soils, or 0.45 of the corresponding footing dimensions B or L, for footings on rock.

C10.6.3.3

A comprehensive parametric study was conducted for cantilevered retaining walls of various heights and soil conditions. The base widths obtained using the LRFD load factors and eccentricity of $B/3$ were comparable to those of ASD with an eccentricity of $B/6$. For foundations on rock, to obtain equivalence with ASD specifications, a maximum eccentricity of $B/2$ would be needed for LRFD. However, a slightly smaller maximum eccentricity has been specified to account for the potential unknown future loading that could push the resultant outside the footing dimensions.

Table 10.6.3.1.2a-2—Coefficients C_{wq} and $C_{w\gamma}$ for Various Groundwater Depths

D_w	C_{wq}	$C_{w\gamma}$
0.0	0.5	0.5
D_f	1.0	0.5
$>1.5B + D_f$	1.0	1.0

Table 10.6.3.1.2a-3—Shape Correction Factors s_c, s_γ, s_q

Factor	Friction Angle	Cohesion Term (s_c)	Unit Weight Term (s_γ)	Surcharge Term (s_q)
Shape Factors s_c, s_γ, s_q	$\phi_f = 0$	$1 + \left(\frac{B}{5L}\right)$	1.0	1.0
	$\phi_f > 0$	$1 + \left(\frac{B}{L}\right)\left(\frac{N_q}{N_c}\right)$	$1 - 0.4\left(\frac{B}{L}\right)$	$1 + \left(\frac{B}{L}\tan\phi_f\right)$

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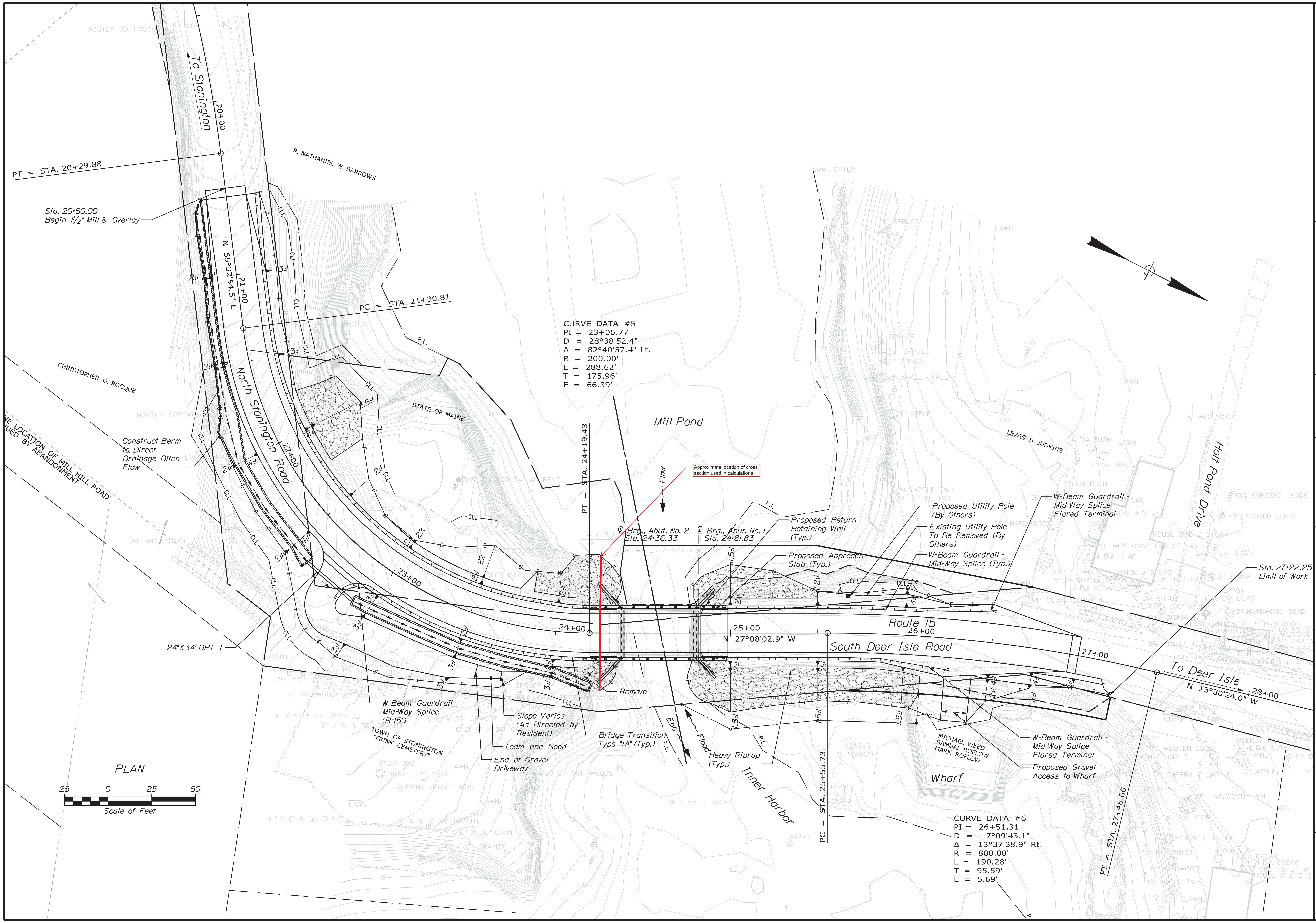

CURVE DATA #5
PI = 23+06.77
D = 28°38'52.4"
Δ = 82°40'57.4" Lt.
R = 200.00'
L = 288.62'
T = 175.96'
E = 66.39'

CURVE DATA #6
PI = 26+51.31
D = 7°09'43.1"
Δ = 13°37'38.9" Rt.
R = 800.00'
L = 190.28'
T = 95.59'
E = 5.69'

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

2235601

WIN
022356.01

BRIDGE NO. 3063

BRIDGE PLANS

PROJ. MANAGER	ANDREW LATHE	BY	DATE
DESIGN-DETAILED	T. WOLFEL	T. WOLFEL	1/2022
CHECKED-REVIEWED	T. TURCOTTE	K. WOOD	1/2022
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE
P.E. NUMBER
DATE

MILL HILL BRIDGE
MILL POND OUTLET
DEER ISLE & STONINGTON HANCOCK COUNTY

PLAN

SHEET NUMBER
3
OF 20

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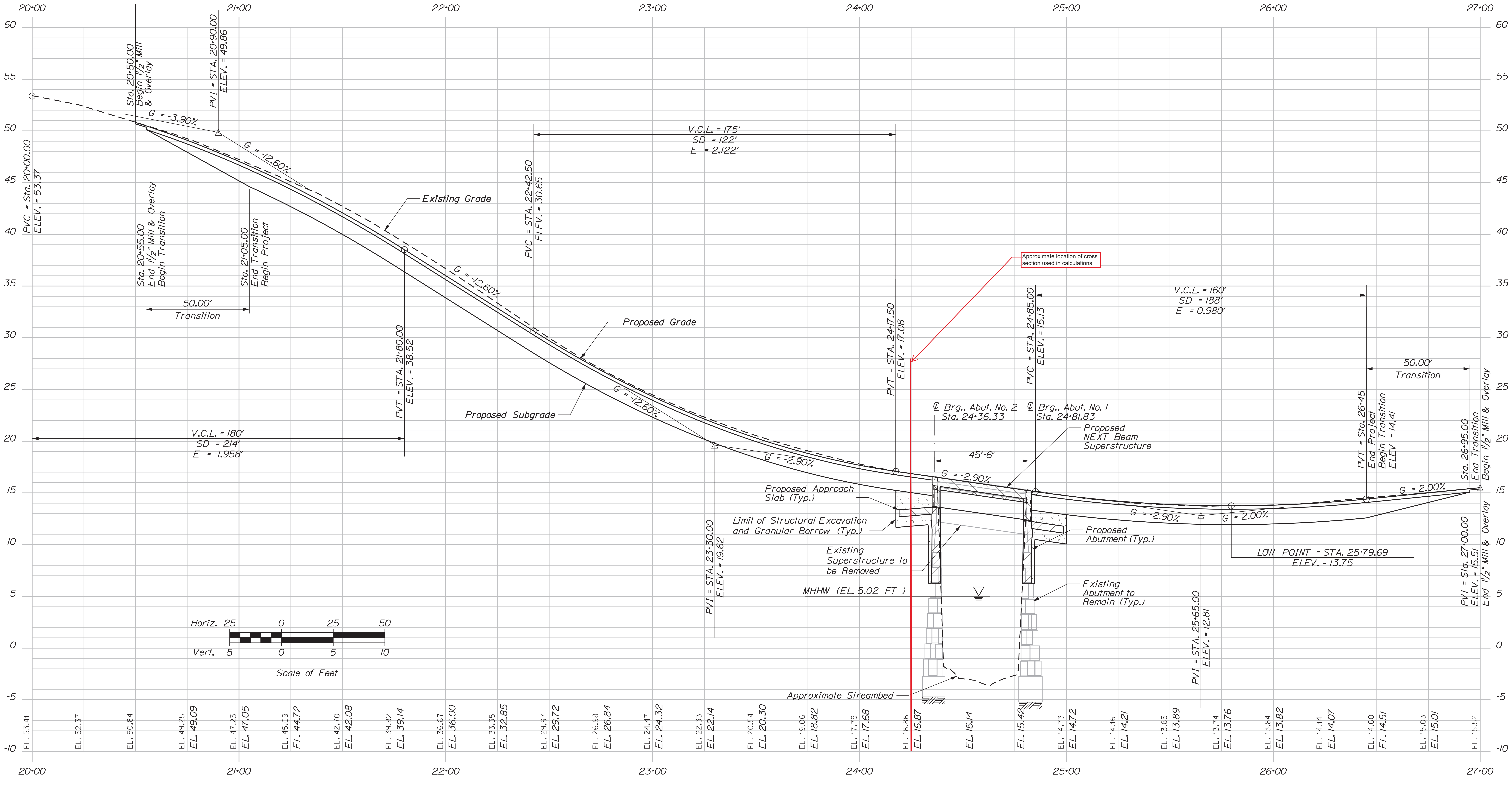
Division: BRIDGE
Username:
Date:1/24/2022



PROFILE

Note:

Abutments are numbered to match the original 1938 plan numbering

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

2235601

WIN
022356.01

BRIDGE PLANS

BRIDGE NO. 3063

PROJ. MANAGER	ANDREW LATHE	BY	DATE
DESIGN-DETAILED	T. WOLFEL	T. WOLFEL	1/2022
CHECKED-REVIEWED	T. TURCOTTE	K. WOOD	1/2022
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

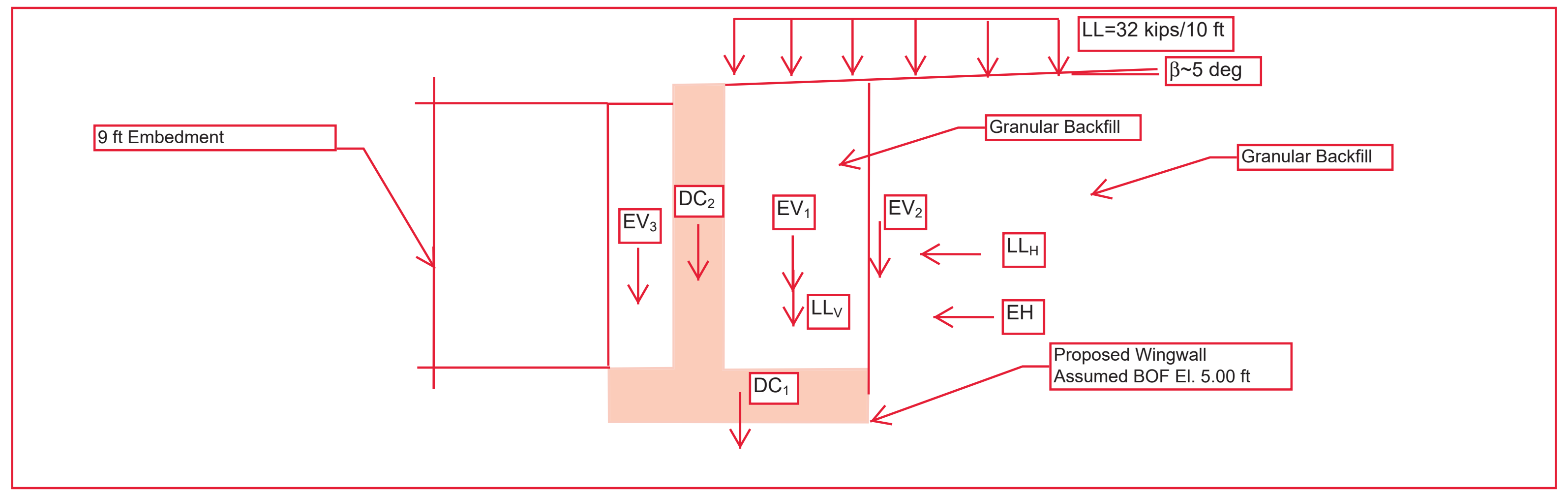
P.E. NUMBER

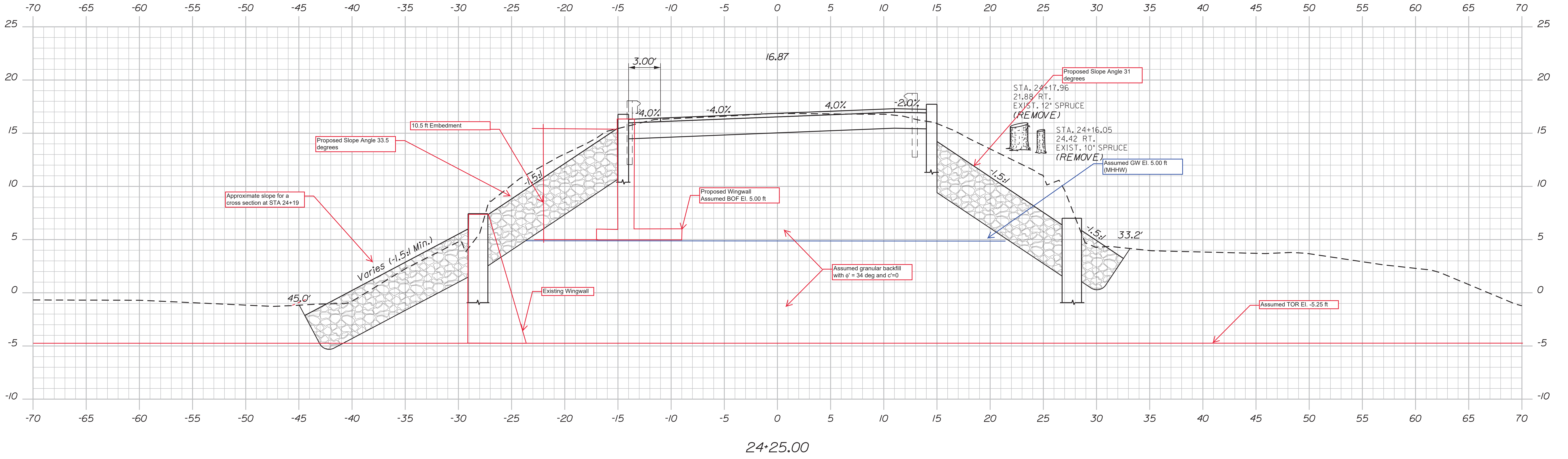
DATE

MILL HILL BRIDGE
MILL POND OUTLET
DEER ISLE & STONINGTON HANCOCK COUNTY

PROFILE

SHEET NUMBER
4
OF 20





24+25.00

24+00.00

Filename: ...\Drawings\BRIDGE\MSTA\xsect.DGN
Division: BRIDGE
Username:
Date:1/24/2022

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

2235601

BRIDGE NO. 3063
WIN
022356.01
BRIDGE PLANS

SIGNATURE

P.E. NUMBER

DATE

		BY	DATE
PROJ. MANAGER	ANDREW LATHE		
DESIGN-DETAILED	T. WOLFEL	T. WOLFEL	1/2022
CHECKED-REVIEWED	T. TURCOTTE	K. WOOD	1/2022
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

MILL HILL BRIDGE
MILL POND OUTLET
DEER ISLE & STONINGTON HANCOCK COUNTY

CROSS SECTIONS

SHEET NUMBER
15
OF 20

Sta. 24+00.00 to Sta. 24+25.00

3.4 Construction Loads

The construction live load to be used for constructibility checks is 50 psf applied over the entire deck area. Consideration should be given to slab placement sequence for calculation of maximum force effects.

3.5 Railroad Loads

Railroad bridges should be designed according to the latest American Railroad Engineering and Maintenance-of-Way Association specifications (AREMA, 2002), with the Cooper live loading as determined by the railroad company.

3.6 Earth Loads

3.6.1 General

Earth pressures considered for wall and substructure design must use the appropriate soil weight shown in Table 3-3.

Table 3-3 Material Classification

Soil Type	Soil Description	Internal Angle of Friction of Soil, ϕ	Soil Total Unit Weight (pcf)	Coeff. of Friction, tan δ, Concrete to Soil	Interface Friction, Angle, Concrete to Soil δ
1	Very loose to loose silty sand and gravel Very loose to loose sand Very loose to medium density sandy silt Stiff to very stiff clay or clayey silt	29° *	100	0.35	19°
2	Medium density silty sand and gravel Medium density to dense sand Dense to very dense sandy silt	33°	120	0.40	22°
3	Dense to very dense silty sand and gravel Very dense sand	36°	130	0.45	24°
4	Granular underwater backfill Granular borrow	32°	125	0.45	24°
5	Gravel Borrow	36°	135	0.50	27°

* The value given for the internal angle of friction (ϕ) for stiff to very stiff silty clay or clayey silt should be used with caution due to the large possible variation with different moisture contents.

Earth Pressure Parameters

Project: ME DOT Mill Hill Bridge
Location: Deer Isle, Maine
Calculated By: MR **Date:** 4/27/2022
Checked By: MK **Date:** 6/23/2022

Objective: Evaluate active, at-rest and passive EP coefficients. The active earth pressure coefficient is based on Rankine theory as described in section 3.6.5.2 of the MaineDOT Bridge Design Guide, with a slope of backfill of 5 degrees.The passive earth pressure coefficient is based on Coulomb theory as described in section 3.6.5.2 of the MaineDOT Bridge Design Guide, with a slope of backfill of 5 degrees and interface friction angle of 27 degrees.

References:
1) Soil Mechanics in Engineering Practice - Terzaghi, Peck and Mesri
2) MED DOT Bridge Manual
3) AASHTO LRFD Bridge Specifications

For a sloped backfill surface where β > 0°, the coefficient of active earth pressure (Rankine), K_a, may be taken as:

$$K_a = \cos\ \beta \cdot \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}$$

$$K_p = \frac{\sin(\alpha - \phi)^2}{\sin\alpha^2 \cdot \sin(\alpha + \delta) \cdot \left(1 - \sqrt{\frac{\sin(\phi + \delta)\cdot\sin(\phi + \beta)}{\sin(\alpha + \delta)\cdot\sin(\alpha + \beta)}}\right)^2}$$

Wall Stem (ft)= 1.70
Wall Base Thick. (ft)= 1.75
H wall (ft)= 10.5
L wall (ft)= 20
B wall (ft)= 9

Backfill Slope β= 5
α= 59
Internal Fricrion Angle ϕ'= 36
Interface Friction Angle δ= 27

Active EP coefficient ka= 0.26
At rest EP coefficient ko= 0.41
Passive EP coefficient kp= 5.60

Service Limit State I

ME DOT Mill Hill Bridge
Stonington, ME
Summary of Soil Unit Weight

Layer No.	Layer ID	Density	γ_{dry} (kN/m^3)	γ_{dry} (pcf)	Gs (-)	γ_{sat} (pcf)	γ_m (pcf)
1	Sand	**Dense**	**19.5**	121.7	**2.65**	138	**135**

Table 21. Relationship among relative density, penetration resistance, dry unit weight, and angle of internal friction of cohesionless soils (after Duncan and Buchignani, 1976)

Descriptive Relative Density	Relative Density **	Standard Penetration Resistance N_1 (see Note) *	Static Cone Resistance q_c	Angle of Internal Friction ϕ	Dry Unit Weight
	%	blows/foot	tsf or kgf/cm^2	degrees	KN/m^3
Very Loose	< 15	< 4	< 50	< 30	< 14
Loose	15 - 35	4 - 10	50 - 100	30 - 32	14 - 16
Medium Dense	35 - 65	10 - 30	100 - 150	32 - 35	16 - 18
Dense	65 - 85	30 - 50	150 - 200	35 - 38	18 - 20
Very Dense	85 - 100	> 50	> 200	> 38	> 20

* N_1 = N -value corrected to an effective vertical overburden pressure of 1.0tsf or 100 kPa

** Freshly deposited, normally consolidated sand

Note: As originally proposed, this correlation used the uncorrected SPT blowcount, N_1. However, hammers delivering 60% of the theoretical energy have been the most commonly used hammers for SPT tests, and it seems likely that the data on which the correlation was based was obtained primarily from tests with such hammers. It therefore seems logical to use $N_{1,60}$ with this correlation, and it is the recommendation of this report that this be done.
1kN/cubic meter=6.24 pcf

γ_m= [(G_s-1)/G_s) x γ_{dry}] + γ_w
γ_m = Moist unit weight (pcf)
γ_{dry} = Dry unit weight (pcf)
γ_w = Unit weight of water (pcf)
G_s = Soil specific gravity

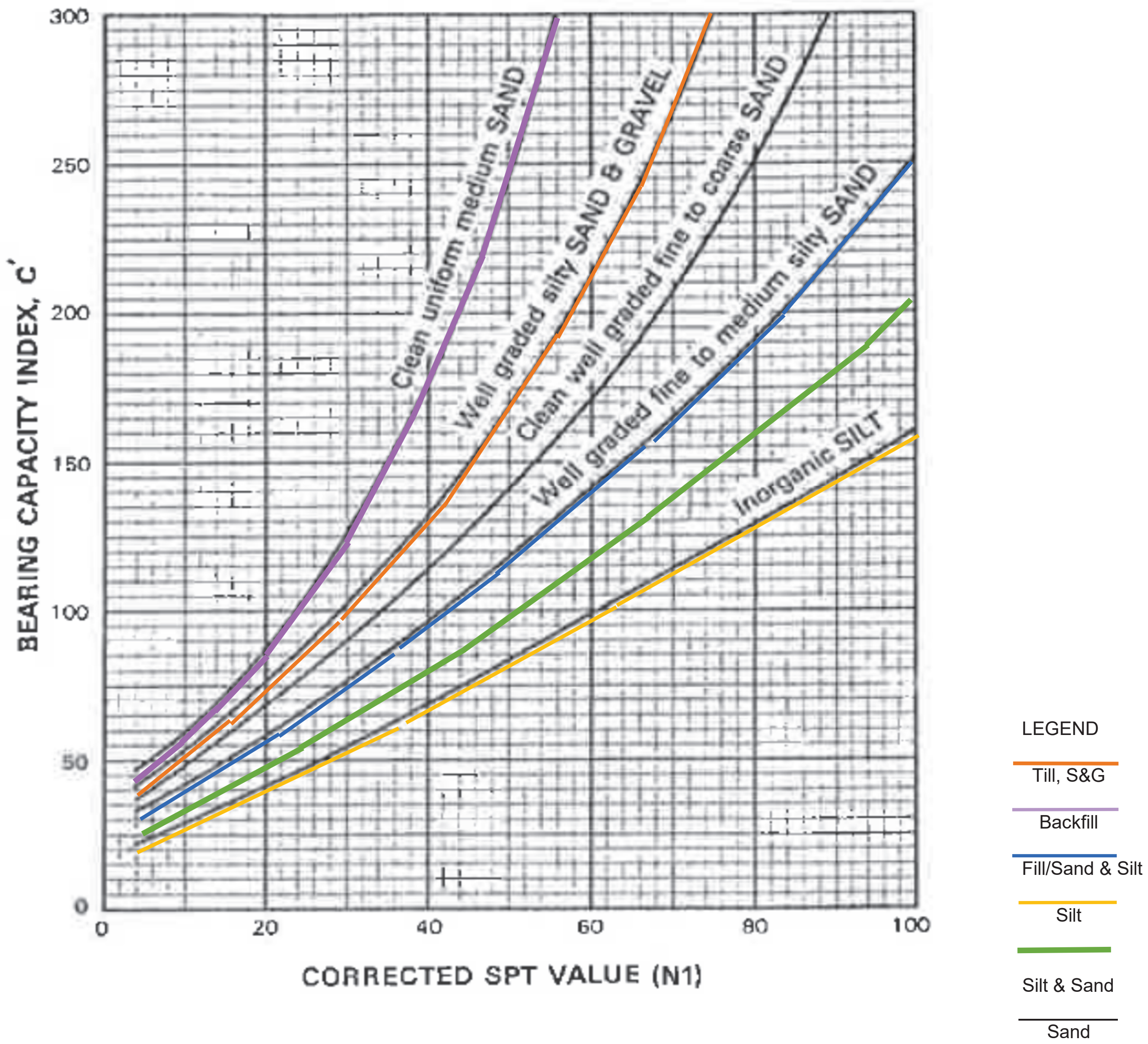
Neglect N=100 bpf in Fill and N=2 bpf in S&G for the evaluation of the average N-value

Prepared by:MR Date: 4/27/2022
Checked by:MK Date: 6/23/2022

ME DOT Mill Hill Bridge
Stonington, ME
Bearing Capacity Index Chart

Reference:

Figure 10.6.2.4.2-1, Bearing Capacity Index (C') versus Corrected SPT Value (N1,60) graph



ME DOT Mill Hill Bridge
Stonington, ME
Elastic Settlement - Spread Footings (Strip Footings)

7-foot wide footing; footing at approx. El.5 feet

Data:

Footing Width W (ft)	**7.0**		
Distance from ftg center x (ft)	**0.0**	Net Load From Footing (psf)	1500
Boring Ground Surface El. (ft)	**16.0**	Overexc. Below BOF (ft)	0.0
Lowest Ex.Ground Surface El. (ft)	**16.0**	Backfill Below BOF (ft)	0.0
Finished Grade Elevation (ft)	**16.0**	Avg Unit Weight of Unsuitable (pcf)	
		Unit Weight of Backfill (pcf)	
FTG Depth Below Finished grade (ft)	**11.0**	Raise in Grade above Existing (ft)	0.0
Bottom of Footing Elevation (ft)	5.0	Total Fill (ft)	0.0
Depth of Unsuitable in Boring (ft)	**0.0**	Unload from Cut (psf)	0
Bottom of Unsuitable El. (ft)	16.0	Load from Fill (psf)	0.0
GW Depth Below Existing Grade (ft)	**11.0**		
GW Elevation (ft)	5.0		
FTG Depth Below Boring (ft)	11.0		
FTG El. Below Boring (ft)	5.0		

Soil Properties (Includes all Soils Strata Encountered in all Borings):

Layer No.	Layer ID	Gamma Sat (pcf)	CR	RR	OC/NC	Evaluation of C' a	b	c	Drained/ Undrained
1	Sand	135	-	-	-	0.016	1.48	29.94	D
-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-

Notes:

1-Stratigraphy based on existing borings (top of rock) and cross section by KLF
2-Energy ratio ER= 70 Automatic Hammer
3 - From Figure 10.6.2.4.2-1 (Attached-AASHTO LRFD) and interpolation $C'=aN_{1,60}^2+bN_{1,60}+c$
4 - Sublayer settlement evaluated as ε times the sublayer thickness
5 - If applicable, overexcavate unsuitable and replace with Backfill (assume N=30 for Backfill)
4.5 Indicates bottom of footing

Calculations:

Geostatic Pressure										Vertical Stress Change (Bussinesq)							Consolidation Settlement			Elastic Settlement									Settlement		
Layer	**Drained/ Undrained**	**Layer Top (ft)**	**Layer Bottom (ft)**	**Layer Top Elevation (ft)**	**Layer Bottom Elevation (ft)**	**Layer Center Depth (ft)**	**Unit Weight (pcf)**	**Total Vertical Pressure (psf)**	**Effective Vertical Pressure (psf)**	**Layer Center Elevation (ft)**	**Depth Below Footing Base z (ft)**	**alpha (deg)**	**beta (deg)**	**Influence Factor (-)**	**Delta Sigma FTGS (psf)**	**Final Effective Vertical Pressure (psf)**	**NC/OC**	**CR (-)**	**RR (-)**	**N (bl/ft)(1,5,6)**	**C_E** energy ratio **(2)**	**N60 (bl/ft)**	**C_N (-)**	**Cs Sampler liner**	**C_B** boring diameter	**C_R** rod length	**N1,60 (bl/ft)(2,3)**	**C' (3)**	**Sublayer Thickness (ft)**	**Sublayer Strain ε (-)**	**Sublayer Settlement (in) (4)**
Sand	D			16.0	16.0	0	135	0	0	16	-11.0	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-
Sand	D	0	**11**	16.0	5.0	5.5	135	742	742	10.5	-5.5	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-
Sand	D	11	**13**	5.0	3.0	12.0	135	1620	1558	4	1.0	-74.0546	148.10921	0.990992648	1486	3044	-	-	-	**Assumed Medium Dense Backfill**							20	66	2	0.0044	0.11
Sand	D	13	**15**	3.0	1.0	14.0	135	1890	1703	2	3.0	-49.39871	98.797411	0.863439568	1295	2998	-	-	-								20	66	2	0.0037	0.09
Sand	D	15	**17**	1.0	-1.0	16.0	135	2160	1848	0	5.0	-34.99202	69.98404	0.687883339	1032	2880	-	-	-								20	66	2	0.0029	0.07
Sand	D	17	**19**	-1.0	-3.0	18.0	135	2430	1993	-2	7.0	-26.56505	53.130102	0.549815144	825	2818	-	-	-								20	66	2	0.0023	0.05
Sand	D	19	**21**	-3.0	-5.0	20.0	135	2700	2138	-4	9.0	-21.25051	42.501011	0.451167911	677	2815	-	-	-								20	66	2	0.0018	0.04

Settlement 0.4

ME DOT Mill Hill Bridge
Stonington, ME
Elastic Settlement - Spread Footings (Strip Footings)

7-foot wide footing; footing at approx. El.5 feet

Data:

Footing Width W (ft) 7.0
Distance from ftg center x (ft) 0.0

Boring Ground Surface El. (ft) 16.0
Lowest Ex.Ground Surface El. (ft) 16.0
Finished Grade Elevation (ft) 16.0

FTG Depth Below Finished grade (ft) 11.0
Bottom of Footing Elevation (ft) 5.0
Depth of Unsuitable in Boring (ft) 0.0
Bottom of Unsuitable El. (ft) 16.0
GW Depth Below Existing Grade (ft) 11.0
GW Elevation (ft) 5.0
FTG Depth Below Boring (ft) 11.0
FTG El. Below Boring (ft) 5.0

Net Load From Footing (psf) 2000

Overexc. Below BOF (ft) 0.0
Backfill Below BOF (ft) 0.0
Avg Unit Weight of Unsuitable (pcf)
Unit Weight of Backfill (pcf)
Raise in Grade above Existing (ft) 0.0
Total Fill (ft) 0.0
Unload from Cut (psf) 0
Load from Fill (psf) 0.0

Soil Properties (Includes all Soils Strata Encountered in all Borings):

Layer No.	Layer ID	Gamma Sat (pcf)	CR	RR	OC/NC
1	Sand	135	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-

Evaluation of C'			Drained/
a	b	c	Undrained
0.016	1.48	29.94	D
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-

Notes:

1-Stratigraphy based on existing borings (top of rock) and cross section by KLF
2-Energy ratio ER= 70 Automatic Hammer
3 - From Figure 10.6.2.4.2-1 (Attached-AASHTO LRFD) and interpolation $C'=aN_{1,60}^2+bN_{1,60}+c$
4 - Sublayer settlement evaluated as ε times the sublayer thickness
5 - If applicable, overexcavate unsuitable and replace with Backfill (assume N=30 for Backfill)
4.5 Indicates bottom of footing

Calculations:

Geostatic Pressure										Vertical Stress Change (Bussinesq)							Consolidation Settlement			Elastic Settlement									Settlement		
Layer	Drained/ Undrained	Layer Top (ft)	Layer Bottom (ft)	Layer Top Elevation (ft)	Layer Bottom Elevation (ft)	Layer Center Depth (ft)	Unit Weight (pcf)	Total Vertical Pressure (psf)	Effective Vertical Pressure (psf)	Layer Center Elevation (ft)	Depth Below Footing Base z (ft)	alpha (deg)	beta (deg)	Influence Factor (-)	Delta Sigma FTGS (psf)	Final Effective Vertical Pressure (psf)	NC/OC	CR (-)	RR (-)	N (bl/ft)$^{(1,5,6)}$	C_E energy ratio $^{(2)}$	N60 (bl/ft)	C_N (-)	Cs Sampler liner	C_B boring diameter	C_R rod length	N1,60 (bl/ft)$^{(2,3)}$	C' $^{(3)}$	Sublayer Thickness (ft)	Sublayer Strain ε (-)	Sublayer Settlement (in) $^{(4)}$
Sand	D			16.0	16.0	0	135	0	0	16	-11.0	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-
Sand	D	0	11	16.0	5.0	5.5	135	742	742	10.5	-5.5	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-
Sand	D	11	13	5.0	3.0	12.0	135	1620	1558	4	1.0	-74.0546	148.10921	0.990992648	1982	3540	-	-	-	Assumed Medium Dense Backfill							20	66	2	0.0054	0.13
Sand	D	13	15	3.0	1.0	14.0	135	1890	1703	2	3.0	-49.39871	98.797411	0.863439568	1727	3430	-	-	-								20	66	2	0.0046	0.11
Sand	D	15	17	1.0	-1.0	16.0	135	2160	1848	0	5.0	-34.99202	69.98404	0.687883339	1376	3224	-	-	-								20	66	2	0.0037	0.09
Sand	D	17	19	-1.0	-3.0	18.0	135	2430	1993	-2	7.0	-26.56505	53.130102	0.549815144	1100	3093	-	-	-								20	66	2	0.0029	0.07
Sand	D	19	21	-3.0	-5.0	20.0	135	2700	2138	-4	9.0	-21.25051	42.501011	0.451167911	902	3041	-	-	-								20	66	2	0.0023	0.06

Settlement 0.5

ME DOT Mill Hill Bridge
Stonington, ME
Elastic Settlement - Spread Footings (Strip Footings)

7-foot wide footing; footing at approx. El.5 feet

Data:

Footing Width W (ft)	7.0		
Distance from ftg center x (ft)	0.0	Net Load From Footing (psf)	2500
Boring Ground Surface El. (ft)	16.0	Overexc. Below BOF (ft)	0.0
Lowest Ex.Ground Surface El. (ft)	16.0	Backfill Below BOF (ft)	0.0
Finished Grade Elevation (ft)	16.0	Avg Unit Weight of Unsuitable (pcf)	
		Unit Weight of Backfill (pcf)	
FTG Depth Below Finished grade (ft)	11.0	Raise in Grade above Existing (ft)	0.0
Bottom of Footing Elevation (ft)	5.0	Total Fill (ft)	0.0
Depth of Unsuitable in Boring (ft)	0.0	Unload from Cut (psf)	0
Bottom of Unsuitable El. (ft)	16.0	Load from Fill (psf)	0.0
GW Depth Below Existing Grade (ft)	11.0		
GW Elevation (ft)	5.0		
FTG Depth Below Boring (ft)	11.0		
FTG El. Below Boring (ft)	5.0		

Soil Properties (Includes all Soils Strata Encountered in all Borings):

Layer No.	Layer ID	Gamma Sat (pcf)	CR	RR	OC/NC	Evaluation of C' a	b	c	Drained/ Undrained
1	Sand	135	-	-	-	0.016	1.48	29.94	D
-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-

Notes:

1-Stratigraphy based on existing borings (top of rock) and cross section by KLF
2-Energy ratio ER= 70 Automatic Hammer
3 - From Figure 10.6.2.4.2-1 (Attached-AASHTO LRFD) and interpolation $C'=aN_{1,60}^2+bN_{1,60}+c$
4 - Sublayer settlement evaluated as ε times the sublayer thickness
5 - If applicable, overexcavate unsuitable and replace with Backfill (assume N=30 for Backfill)
4.5 Indicates bottom of footing

Calculations:

Geostatic Pressure										Vertical Stress Change (Bussinesq)							Consolidation Settlement			Elastic Settlement									Settlement		
Layer	**Drained/ Undrained**	**Layer Top (ft)**	**Layer Bottom (ft)**	**Layer Top Elevation (ft)**	**Layer Bottom Elevation (ft)**	**Layer Center Depth (ft)**	**Unit Weight (pcf)**	**Total Vertical Pressure (psf)**	**Effective Vertical Pressure (psf)**	**Layer Center Elevation (ft)**	**Depth Below Footing Base z (ft)**	**alpha (deg)**	**beta (deg)**	**Influence Factor (-)**	**Delta Sigma FTGS (psf)**	**Final Effective Vertical Pressure (psf)**	**NC/OC**	**CR (-)**	**RR (-)**	**N (bl/ft)$^{(1,5,6)}$**	**C_E** energy ratio **$^{(2)}$**	**N60 (bl/ft)**	**C_N (-)**	**Cs Sampler liner**	**C_B** boring diameter	**C_R** rod length	**N1,60 (bl/ft)$^{(2,3)}$**	**C' $^{(3)}$**	**Sublayer Thickness (ft)**	**Sublayer Strain ε (-)**	**Sublayer Settlement (in) $^{(4)}$**
Sand	D			16.0	16.0	0	135	0	0	16	-11.0	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-
Sand	D	0	11	16.0	5.0	5.5	135	742	742	10.5	-5.5	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-
Sand	D	11	13	5.0	3.0	12.0	135	1620	1558	4	1.0	-74.0546	148.10921	0.990992648	2477	4035	-	-	-	Assumed Medium Dense Backfill							20	66	2	0.0063	0.15
Sand	D	13	15	3.0	1.0	14.0	135	1890	1703	2	3.0	-49.39871	98.797411	0.863439568	2159	3861	-	-	-								20	66	2	0.0054	0.13
Sand	D	15	17	1.0	-1.0	16.0	135	2160	1848	0	5.0	-34.99202	69.98404	0.687883339	1720	3568	-	-	-								20	66	2	0.0043	0.10
Sand	D	17	19	-1.0	-3.0	18.0	135	2430	1993	-2	7.0	-26.56505	53.130102	0.549815144	1375	3368	-	-	-								20	66	2	0.0035	0.08
Sand	D	19	21	-3.0	-5.0	20.0	135	2700	2138	-4	9.0	-21.25051	42.501011	0.451167911	1128	3266	-	-	-								20	66	2	0.0028	0.07

Settlement **0.53**

ME DOT Mill Hill Bridge
Stonington, ME
Elastic Settlement - Spread Footings (Strip Footings)

7-foot wide footing; footing at approx. El.5 feet

Data:

Footing Width W (ft) 7.0
Distance from ftg center x (ft) 0.0
Net Load From Footing (psf) 3000

Boring Ground Surface El. (ft) 16.0
Lowest Ex.Ground Surface El. (ft) 16.0
Finished Grade Elevation (ft) 16.0

FTG Depth Below Finished grade (ft) 11.0
Bottom of Footing Elevation (ft) 5.0
Depth of Unsuitable in Boring (ft) 0.0
Bottom of Unsuitable El. (ft) 16.0
GW Depth Below Existing Grade (ft) 11.0
GW Elevation (ft) 5.0
FTG Depth Below Boring (ft) 11.0
FTG El. Below Boring (ft) 5.0

Overexc. Below BOF (ft) 0.0
Backfill Below BOF (ft) 0.0
Avg Unit Weight of Unsuitable (pcf)
Unit Weight of Backfill (pcf)
Raise in Grade above Existing (ft) 0.0
Total Fill (ft) 0.0
Unload from Cut (psf) 0
Load from Fill (psf) 0.0

Soil Properties (Includes all Soils Strata Encountered in all Borings):

Layer No.	Layer ID	Gamma Sat (pcf)	CR	RR	OC/NC
1	Sand	135	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-

Evaluation of C'			Drained/
a	b	c	Undrained
0.016	1.48	29.94	D
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-

Notes:

1-Stratigraphy based on existing borings (top of rock) and cross section by KLF
2-Energy ratio ER= 70 Automatic Hammer
3 - From Figure 10.6.2.4.2-1 (Attached-AASHTO LRFD) and interpolation $C'=aN_{1,60}^2+bN_{1,60}+c$
4 - Sublayer settlement evaluated as ε times the sublayer thickness
5 - If applicable, overexcavate unsuitable and replace with Backfill (assume N=30 for Backfill)
4.5 Indicates bottom of footing

Calculations:

Geostatic Pressure										Vertical Stress Change (Bussinesq)							Consolidation Settlement			Elastic Settlement									Settlement		
Layer	**Drained/ Undrained**	**Layer Top (ft)**	**Layer Bottom (ft)**	**Layer Top Elevation (ft)**	**Layer Bottom Elevation (ft)**	**Layer Center Depth (ft)**	**Unit Weight (pcf)**	**Total Vertical Pressure (psf)**	**Effective Vertical Pressure (psf)**	**Layer Center Elevation (ft)**	**Depth Below Footing Base z (ft)**	**alpha (deg)**	**beta (deg)**	**Influence Factor (-)**	**Delta Sigma FTGS (psf)**	**Final Effective Vertical Pressure (psf)**	**NC/OC**	**CR (-)**	**RR (-)**	**N (bl/ft)$^{(1,5,6)}$**	**C_E** energy ratio **$^{(2)}$**	**N60 (bl/ft)**	**C_N (-)**	**Cs Sampler liner**	**C_B** boring diameter	**C_R** rod length	**N1,60 (bl/ft)$^{(2,3)}$**	**C' $^{(3)}$**	**Sublayer Thickness (ft)**	**Sublayer Strain ε (-)**	**Sublayer Settlement (in) $^{(4)}$**
Sand	D			16.0	16.0	0	135	0	0	16	-11.0	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-
Sand	D	0	11	16.0	5.0	5.5	135	742	742	10.5	-5.5	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-
Sand	D	11	13	5.0	3.0	12.0	135	1620	1558	4	1.0	-74.0546	148.10921	0.990992648	2973	4531	-	-	-	Assumed Medium Dense Backfill							20	66	2	0.0070	0.17
Sand	D	13	15	3.0	1.0	14.0	135	1890	1703	2	3.0	-49.39871	98.797411	0.863439568	2590	4293	-	-	-								20	66	2	0.0061	0.15
Sand	D	15	17	1.0	-1.0	16.0	135	2160	1848	0	5.0	-34.99202	69.98404	0.687883339	2064	3912	-	-	-								20	66	2	0.0049	0.12
Sand	D	17	19	-1.0	-3.0	18.0	135	2430	1993	-2	7.0	-26.56505	53.130102	0.549815144	1649	3643	-	-	-								20	66	2	0.0040	0.10
Sand	D	19	21	-3.0	-5.0	20.0	135	2700	2138	-4	9.0	-21.25051	42.501011	0.451167911	1354	3492	-	-	-								20	66	2	0.0032	0.08

Settlement 0.6

ME DOT Mill Hill Bridge
Stonington, ME
Elastic Settlement - Spread Footings (Strip Footings)

7-foot wide footing; footing at approx. El.5 feet

Data:

Footing Width W (ft) 7.0
Distance from ftg center x (ft) 0.0
Net Load From Footing (psf) 3600

Boring Ground Surface El. (ft) 16.0
Lowest Ex.Ground Surface El. (ft) 16.0
Finished Grade Elevation (ft) 16.0

FTG Depth Below Finished grade (ft) 11.0
Bottom of Footing Elevation (ft) 5.0
Depth of Unsuitable in Boring (ft) 0.0
Bottom of Unsuitable El. (ft) 16.0
GW Depth Below Existing Grade (ft) 11.0
GW Elevation (ft) 5.0
FTG Depth Below Boring (ft) 11.0
FTG El. Below Boring (ft) 5.0

Overexc. Below BOF (ft) 0.0
Backfill Below BOF (ft) 0.0
Avg Unit Weight of Unsuitable (pcf)
Unit Weight of Backfill (pcf)
Raise in Grade above Existing (ft) 0.0
Total Fill (ft) 0.0
Unload from Cut (psf) 0
Load from Fill (psf) 0.0

Soil Properties (Includes all Soils Strata Encountered in all Borings):

Layer No.	Layer ID	Gamma Sat (pcf)	CR	RR	OC/NC
1	Sand	135	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-

Evaluation of C'			Drained/
a	b	c	Undrained
0.016	1.48	29.94	D
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-

Notes:

1-Stratigraphy based on existing borings (top of rock) and cross section by KLF
2-Energy ratio ER= 70 Automatic Hammer
3 - From Figure 10.6.2.4.2-1 (Attached-AASHTO LRFD) and interpolation $C'=aN_{1,60}^2+bN_{1,60}+c$
4 - Sublayer settlement evaluated as ε times the sublayer thickness
5 - If applicable, overexcavate unsuitable and replace with Backfill (assume N=30 for Backfill)
4.5 Indicates bottom of footing

Calculations:

Geostatic Pressure										Vertical Stress Change (Bussinesq)							Consolidation Settlement			Elastic Settlement									Settlement		
Layer	Drained/ Undrained	Layer Top (ft)	Layer Bottom (ft)	Layer Top Elevation (ft)	Layer Bottom Elevation (ft)	Layer Center Depth (ft)	Unit Weight (pcf)	Total Vertical Pressure (psf)	Effective Vertical Pressure (psf)	Layer Center Elevation (ft)	Depth Below Footing Base z (ft)	alpha (deg)	beta (deg)	Influence Factor (-)	Delta Sigma FTGS (psf)	Final Effective Vertical Pressure (psf)	NC/OC	CR (-)	RR (-)	N (bl/ft)(1,5,6)	C_E energy ratio (2)	N60 (bl/ft)	C_N (-)	Cs Sampler liner	C_B boring diameter	C_R rod length	N1,60 (bl/ft)(2,3)	C' (3)	Sublayer Thickness (ft)	Sublayer Strain ε (-)	Sublayer Settlement (in) (4)
Sand	D			16.0	16.0	0	135	0	0	16	-11.0	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-
Sand	D	0	11	16.0	5.0	5.5	135	742	742	10.5	-5.5	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-
Sand	D	11	13	5.0	3.0	12.0	135	1620	1558	4	1.0	-74.0546	148.10921	0.990992648	3568	5125	-	-	-	Assumed Medium Dense Backfill							20	66	2	0.0078	0.19
Sand	D	13	15	3.0	1.0	14.0	135	1890	1703	2	3.0	-49.39871	98.797411	0.863439568	3108	4811	-	-	-								20	66	2	0.0068	0.16
Sand	D	15	17	1.0	-1.0	16.0	135	2160	1848	0	5.0	-34.99202	69.98404	0.687883339	2476	4324	-	-	-								20	66	2	0.0056	0.13
Sand	D	17	19	-1.0	-3.0	18.0	135	2430	1993	-2	7.0	-26.56505	53.130102	0.549815144	1979	3973	-	-	-								20	66	2	0.0045	0.11
Sand	D	19	21	-3.0	-5.0	20.0	135	2700	2138	-4	9.0	-21.25051	42.501011	0.451167911	1624	3763	-	-	-								20	66	2	0.0037	0.09

Settlement 0.7

ME DOT Mill Hill Bridge
Stonington, ME
Elastic Settlement - Spread Footings (Strip Footings)

8-foot wide footing; footing at approx. El.5 feet

Data:

Footing Width W (ft) 8.0
Distance from ftg center x (ft) 0.0
Net Load From Footing (psf) 2000

Boring Ground Surface El. (ft) 16.0
Lowest Ex.Ground Surface El. (ft) 16.0
Finished Grade Elevation (ft) 16.0
FTG Depth Below Finished grade (ft) 11.0
Bottom of Footing Elevation (ft) 5.0
Depth of Unsuitable in Boring (ft) 0.0
Bottom of Unsuitable El. (ft) 16.0
GW Depth Below Existing Grade (ft) 11.0
GW Elevation (ft) 5.0
FTG Depth Below Boring (ft) 11.0
FTG El. Below Boring (ft) 5.0

Overexc. Below BOF (ft) 0.0
Backfill Below BOF (ft) 0.0
Avg Unit Weight of Unsuitable (pcf)
Unit Weight of Backfill (pcf)
Raise in Grade above Existing (ft) 0.0
Total Fill (ft) 0.0
Unload from Cut (psf) 0
Load from Fill (psf) 0.0

Soil Properties (Includes all Soils Strata Encountered in all Borings):

Layer No.	Layer ID	Gamma Sat (pcf)	CR	RR	OC/NC
1	Sand	135	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-

Evaluation of C'			Drained/
a	b	c	Undrained
0.016	1.48	29.94	D
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-

Notes:

1-Stratigraphy based on existing borings (top of rock) and cross section by KLF
2-Energy ratio ER= 70 Automatic Hammer
3 - From Figure 10.6.2.4.2-1 (Attached-AASHTO LRFD) and interpolation $C'=aN_{1,60}^2+bN_{1,60}+c$
4 - Sublayer settlement evaluated as ε times the sublayer thickness
5 - If applicable, overexcavate unsuitable and replace with Backfill (assume N=30 for Backfill)
4.5 Indicates bottom of footing

Calculations:

Geostatic Pressure										Vertical Stress Change (Bussinesq)							Consolidation Settlement			Elastic Settlement									Settlement		
Layer	**Drained/ Undrained**	**Layer Top (ft)**	**Layer Bottom (ft)**	**Layer Top Elevation (ft)**	**Layer Bottom Elevation (ft)**	**Layer Center Depth (ft)**	**Unit Weight (pcf)**	**Total Vertical Pressure (psf)**	**Effective Vertical Pressure (psf)**	**Layer Center Elevation (ft)**	**Depth Below Footing Base z (ft)**	**alpha (deg)**	**beta (deg)**	**Influence Factor (-)**	**Delta Sigma FTGS (psf)**	**Final Effective Vertical Pressure (psf)**	**NC/OC**	**CR (-)**	**RR (-)**	**N (bl/ft) (1,5,6)**	**C_E energy ratio (2)**	**N60 (bl/ft)**	**C_N (-)**	**Cs Sampler liner**	**C_B boring diameter**	**C_R rod length**	**N1,60 (bl/ft) (2,3)**	**C' (3)**	**Sublayer Thickness (ft)**	**Sublayer Strain ε (-)**	**Sublayer Settlement (in) (4)**
Sand	D			16.0	16.0	0	135	0	0	16	-11.0	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-
Sand	D	0	11	16.0	5.0	5.5	135	742	742	10.5	-5.5	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-
Sand	D	11	13	5.0	3.0	12.0	135	1620	1558	4	1.0	-75.96376	151.92751	0.993834627	1988	3545	-	-	-	Assumed Medium Dense Backfill							20	66	2	0.0054	0.13
Sand	D	13	15	3.0	1.0	14.0	135	1890	1703	2	3.0	-53.1301	106.2602	0.895911961	1792	3495	-	-	-								20	66	2	0.0047	0.11
Sand	D	15	17	1.0	-1.0	16.0	135	2160	1848	0	5.0	-38.65981	77.319617	0.740099655	1480	3328	-	-	-								20	66	2	0.0039	0.09
Sand	D	17	19	-1.0	-3.0	18.0	135	2430	1993	-2	7.0	-29.74488	59.489763	0.604734891	1209	3203	-	-	-								20	66	2	0.0031	0.07
Sand	D	19	21	-3.0	-5.0	20.0	135	2700	2138	-4	9.0	-23.96249	47.924978	0.502521133	1005	3143	-	-	-								20	66	2	0.0025	0.06

Settlement 0.5

ME DOT Mill Hill Bridge
Stonington, ME
Elastic Settlement - Spread Footings (Strip Footings)

8-foot wide footing; footing at approx. El.5 feet

Data:

Footing Width W (ft) 8.0
Distance from ftg center x (ft) 0.0
Net Load From Footing (psf) 2500

Boring Ground Surface El. (ft) 16.0
Lowest Ex.Ground Surface El. (ft) 16.0
Finished Grade Elevation (ft) 16.0

FTG Depth Below Finished grade (ft) 11.0
Bottom of Footing Elevation (ft) 5.0
Depth of Unsuitable in Boring (ft) 0.0
Bottom of Unsuitable El. (ft) 16.0
GW Depth Below Existing Grade (ft) 11.0
GW Elevation (ft) 5.0
FTG Depth Below Boring (ft) 11.0
FTG El. Below Boring (ft) 5.0

Overexc. Below BOF (ft) 0.0
Backfill Below BOF (ft) 0.0
Avg Unit Weight of Unsuitable (pcf)
Unit Weight of Backfill (pcf)
Raise in Grade above Existing (ft) 0.0
Total Fill (ft) 0.0
Unload from Cut (psf) 0
Load from Fill (psf) 0.0

Soil Properties (Includes all Soils Strata Encountered in all Borings):

Layer No.	Layer ID	Gamma Sat (pcf)	CR	RR	OC/NC	Evaluation of C' a	b	c	Drained/ Undrained
1	Sand	135	-	-	-	0.016	1.48	29.94	D
-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-

Notes:

1-Stratigraphy based on existing borings (top of rock) and cross section by KLF
2-Energy ratio ER= 70 Automatic Hammer
3 - From Figure 10.6.2.4.2-1 (Attached-AASHTO LRFD) and interpolation $C'=aN_{1,60}^2+bN_{1,60}+c$
4 - Sublayer settlement evaluated as ε times the sublayer thickness
5 - If applicable, overexcavate unsuitable and replace with Backfill (assume N=30 for Backfill)
4.5 Indicates bottom of footing

Calculations:

Geostatic Pressure										Vertical Stress Change (Bussinesq)							Consolidation Settlement			Elastic Settlement									Settlement		
Layer	**Drained/ Undrained**	**Layer Top (ft)**	**Layer Bottom (ft)**	**Layer Top Elevation (ft)**	**Layer Bottom Elevation (ft)**	**Layer Center Depth (ft)**	**Unit Weight (pcf)**	**Total Vertical Pressure (psf)**	**Effective Vertical Pressure (psf)**	**Layer Center Elevation (ft)**	**Depth Below Footing Base z (ft)**	**alpha (deg)**	**beta (deg)**	**Influence Factor (-)**	**Delta Sigma FTGS (psf)**	**Final Effective Vertical Pressure (psf)**	**NC/OC**	**CR (-)**	**RR (-)**	**N (bl/ft)$^{(1,5,6)}$**	**C_E** energy ratio **$^{(2)}$**	**N60 (bl/ft)**	**C_N (-)**	**Cs Sampler liner**	**C_B** boring diameter	**C_R** rod length	**N1,60 (bl/ft)$^{(2,3)}$**	**C' $^{(3)}$**	**Sublayer Thickness (ft)**	**Sublayer Strain ε (-)**	**Sublayer Settlement (in) $^{(4)}$**
Sand	D			16.0	16.0	0	135	0	0	16	-11.0	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-
Sand	D	0	11	16.0	5.0	5.5	135	742	742	10.5	-5.5	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-
Sand	D	11	13	5.0	3.0	12.0	135	1620	1558	4	1.0	-75.96376	151.92751	0.993834627	2485	4042	-	-	-	Assumed Medium Dense Backfill							20	66	2	0.0063	0.15
Sand	D	13	15	3.0	1.0	14.0	135	1890	1703	2	3.0	-53.1301	106.2602	0.895911961	2240	3943	-	-	-								20	66	2	0.0055	0.13
Sand	D	15	17	1.0	-1.0	16.0	135	2160	1848	0	5.0	-38.65981	77.319617	0.740099655	1850	3698	-	-	-								20	66	2	0.0046	0.11
Sand	D	17	19	-1.0	-3.0	18.0	135	2430	1993	-2	7.0	-29.74488	59.489763	0.604734891	1512	3505	-	-	-								20	66	2	0.0037	0.09
Sand	D	19	21	-3.0	-5.0	20.0	135	2700	2138	-4	9.0	-23.96249	47.924978	0.502521133	1256	3395	-	-	-								20	66	2	0.0030	0.07

Settlement 0.6

ME DOT Mill Hill Bridge
Stonington, ME
Elastic Settlement - Spread Footings (Strip Footings)

8-foot wide footing; footing at approx. El.5 feet

Data:

Footing Width W (ft) 8.0
Distance from ftg center x (ft) 0.0
Net Load From Footing (psf) 3000

Boring Ground Surface El. (ft) 16.0
Lowest Ex.Ground Surface El. (ft) 16.0
Finished Grade Elevation (ft) 16.0

FTG Depth Below Finished grade (ft) 11.0
Bottom of Footing Elevation (ft) 5.0
Depth of Unsuitable in Boring (ft) 0.0
Bottom of Unsuitable El. (ft) 16.0
GW Depth Below Existing Grade (ft) 11.0
GW Elevation (ft) 5.0
FTG Depth Below Boring (ft) 11.0
FTG El. Below Boring (ft) 5.0

Overexc. Below BOF (ft) 0.0
Backfill Below BOF (ft) 0.0
Avg Unit Weight of Unsuitable (pcf)
Unit Weight of Backfill (pcf)
Raise in Grade above Existing (ft) 0.0
Total Fill (ft) 0.0
Unload from Cut (psf) 0
Load from Fill (psf) 0.0

Soil Properties (Includes all Soils Strata Encountered in all Borings):

Layer No.	Layer ID	Gamma Sat (pcf)	CR	RR	OC/NC
1	Sand	135	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-

Evaluation of C'			Drained/
a	b	c	Undrained
0.016	1.48	29.94	D
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-

Notes:

1-Stratigraphy based on existing borings (top of rock) and cross section by KLF
2-Energy ratio ER= 70 Automatic Hammer
3 - From Figure 10.6.2.4.2-1 (Attached-AASHTO LRFD) and interpolation C'=a$N_{1,60}^2$+b$N_{1,60}$+c
4 - Sublayer settlement evaluated as ε times the sublayer thickness
5 - If applicable, overexcavate unsuitable and replace with Backfill (assume N=30 for Backfill)
4.5 Indicates bottom of footing

Calculations:

Geostatic Pressure										Vertical Stress Change (Bussinesq)						
Layer	Drained/ Undrained	Layer Top (ft)	Layer Bottom (ft)	Layer Top Elevation (ft)	Layer Bottom Elevation (ft)	Layer Center Depth (ft)	Unit Weight (pcf)	Total Vertical Pressure (psf)	Effective Vertical Pressure (psf)	Layer Center Elevation (ft)	Depth Below Footing Base z (ft)	alpha (deg)	beta (deg)	Influence Factor (-)	Delta Sigma FTGS (psf)	Final Effective Vertical Pressure (psf)
Sand	D			16.0	16.0	0	135	0	0	16	-11.0	-	-	-	-	-
Sand	D	0	11	16.0	5.0	5.5	135	742	742	10.5	-5.5	-	-	-	-	-
Sand	D	11	13	5.0	3.0	12.0	135	1620	1558	4	1.0	-75.96376	151.92751	0.993834627	2982	4539
Sand	D	13	15	3.0	1.0	14.0	135	1890	1703	2	3.0	-53.1301	106.2602	0.895911961	2688	4391
Sand	D	15	17	1.0	-1.0	16.0	135	2160	1848	0	5.0	-38.65981	77.319617	0.740099655	2220	4068
Sand	D	17	19	-1.0	-3.0	18.0	135	2430	1993	-2	7.0	-29.74488	59.489763	0.604734891	1814	3807
Sand	D	19	21	-3.0	-5.0	20.0	135	2700	2138	-4	9.0	-23.96249	47.924978	0.502521133	1508	3646

	Consolidation Settlement			Elastic Settlement									Settlement		
Layer	NC/OC	CR (-)	RR (-)	N (bl/ft)$^{(1,5,6)}$	C_E energy ratio $^{(2)}$	N60 (bl/ft)	C_N (-)	Cs Sampler liner	C_B boring diameter	C_R rod length	N1,60 (bl/ft)$^{(2,3)}$	C' $^{(3)}$	Sublayer Thickness (ft)	Sublayer Strain ε (-)	Sublayer Settlement (in) $^{(4)}$
Sand	-	-	-		-	-	-	-	-	-	-	-	-	-	-
Sand	-	-	-		-	-	-	-	-	-	-	-	-	-	-
Sand	-	-	-	Assumed Medium Dense Backfill							20	66	2	0.0070	0.17
Sand	-	-	-								20	66	2	0.0062	0.15
Sand	-	-	-								20	66	2	0.0052	0.12
Sand	-	-	-								20	66	2	0.0043	0.10
Sand	-	-	-								20	66	2	0.0035	0.08

Settlement 0.63

ME DOT Mill Hill Bridge
Stonington, ME
Elastic Settlement - Spread Footings (Strip Footings)

8-foot wide footing; footing at approx. El.5 feet

Data:

Footing Width W (ft) 8.0
Distance from ftg center x (ft) 0.0
Boring Ground Surface El. (ft) 16.0
Lowest Ex.Ground Surface El. (ft) 16.0
Finished Grade Elevation (ft) 16.0
FTG Depth Below Finished grade (ft) 11.0
Bottom of Footing Elevation (ft) 5.0
Depth of Unsuitable in Boring (ft) 0.0
Bottom of Unsuitable El. (ft) 16.0
GW Depth Below Existing Grade (ft) 11.0
GW Elevation (ft) 5.0
FTG Depth Below Boring (ft) 11.0
FTG El. Below Boring (ft) 5.0

Net Load From Footing (psf) 3500
Overexc. Below BOF (ft) 0.0
Backfill Below BOF (ft) 0.0
Avg Unit Weight of Unsuitable (pcf)
Unit Weight of Backfill (pcf)
Raise in Grade above Existing (ft) 0.0
Total Fill (ft) 0.0
Unload from Cut (psf) 0
Load from Fill (psf) 0.0

Soil Properties (Includes all Soils Strata Encountered in all Borings):

Layer No.	Layer ID	Gamma Sat (pcf)	CR	RR	OC/NC
1	Sand	135	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-

Evaluation of C'			Drained/
a	b	c	Undrained
0.016	1.48	29.94	D
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-

Notes:

1-Stratigraphy based on existing borings (top of rock) and cross section by KLF
2-Energy ratio ER= 70 Automatic Hammer
3 - From Figure 10.6.2.4.2-1 (Attached-AASHTO LRFD) and interpolation $C'=aN_{1,60}^2+bN_{1,60}+c$
4 - Sublayer settlement evaluated as ε times the sublayer thickness
5 - If applicable, overexcavate unsuitable and replace with Backfill (assume N=30 for Backfill)
4.5 Indicates bottom of footing

Calculations:

Geostatic Pressure										Vertical Stress Change (Bussinesq)							Consolidation Settlement			Elastic Settlement									Settlement		
Layer	Drained/ Undrained	Layer Top (ft)	Layer Bottom (ft)	Layer Top Elevation (ft)	Layer Bottom Elevation (ft)	Layer Center Depth (ft)	Unit Weight (pcf)	Total Vertical Pressure (psf)	Effective Vertical Pressure (psf)	Layer Center Elevation (ft)	Depth Below Footing Base z (ft)	alpha (deg)	beta (deg)	Influence Factor (-)	Delta Sigma FTGS (psf)	Final Effective Vertical Pressure (psf)	NC/OC	CR (-)	RR (-)	N (bl/ft)$^{(1,5,6)}$	C_E energy ratio $^{(2)}$	N60 (bl/ft)	C_N (-)	Cs Sampler liner	C_B boring diameter	C_R rod length	N1,60 (bl/ft)$^{(2,3)}$	C' $^{(3)}$	Sublayer Thickness (ft)	Sublayer Strain ε (-)	Sublayer Settlement (in) $^{(4)}$
Sand	D			16.0	16.0	0	135	0	0	16	-11.0	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-
Sand	D	0	11	16.0	5.0	5.5	135	742	742	10.5	-5.5	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-
Sand	D	11	13	5.0	3.0	12.0	135	1620	1558	4	1.0	-75.96376	151.92751	0.993834627	3478	5036	-	-	-	Assumed Medium Dense Backfill							20	66	2	0.0077	0.19
Sand	D	13	15	3.0	1.0	14.0	135	1890	1703	2	3.0	-53.1301	106.2602	0.895911961	3136	4838	-	-	-								20	66	2	0.0069	0.17
Sand	D	15	17	1.0	-1.0	16.0	135	2160	1848	0	5.0	-38.65981	77.319617	0.740099655	2590	4438	-	-	-								20	66	2	0.0058	0.14
Sand	D	17	19	-1.0	-3.0	18.0	135	2430	1993	-2	7.0	-29.74488	59.489763	0.604734891	2117	4110	-	-	-								20	66	2	0.0048	0.11
Sand	D	19	21	-3.0	-5.0	20.0	135	2700	2138	-4	9.0	-23.96249	47.924978	0.502521133	1759	3897	-	-	-								20	66	2	0.0040	0.09

Settlement 0.7

ME DOT Mill Hill Bridge
Stonington, ME
Elastic Settlement - Spread Footings (Strip Footings)

8-foot wide footing; footing at approx. El.5 feet

Data:

Footing Width W (ft) 8.0
Distance from ftg center x (ft) 0.0
Boring Ground Surface El. (ft) 16.0
Lowest Ex.Ground Surface El. (ft) 16.0
Finished Grade Elevation (ft) 16.0
FTG Depth Below Finished grade (ft) 11.0
Bottom of Footing Elevation (ft) 5.0
Depth of Unsuitable in Boring (ft) 0.0
Bottom of Unsuitable El. (ft) 16.0
GW Depth Below Existing Grade (ft) 11.0
GW Elevation (ft) 5.0
FTG Depth Below Boring (ft) 11.0
FTG El. Below Boring (ft) 5.0

Net Load From Footing (psf) 4000
Overexc. Below BOF (ft) 0.0
Backfill Below BOF (ft) 0.0
Avg Unit Weight of Unsuitable (pcf)
Unit Weight of Backfill (pcf)
Raise in Grade above Existing (ft) 0.0
Total Fill (ft) 0.0
Unload from Cut (psf) 0
Load from Fill (psf) 0.0

Soil Properties (Includes all Soils Strata Encountered in all Borings):

Layer No.	Layer ID	Gamma Sat (pcf)	CR	RR	OC/NC
1	Sand	135	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-

Evaluation of C'			Drained/
a	b	c	Undrained
0.016	1.48	29.94	D
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-

Notes:

1-Stratigraphy based on existing borings (top of rock) and cross section by KLF
2-Energy ratio ER= 70 Automatic Hammer
3 - From Figure 10.6.2.4.2-1 (Attached-AASHTO LRFD) and interpolation $C'=aN_{1,60}^2+bN_{1,60}+c$
4 - Sublayer settlement evaluated as ε times the sublayer thickness
5 - If applicable, overexcavate unsuitable and replace with Backfill (assume N=30 for Backfill)
4.5 Indicates bottom of footing

Calculations:

Geostatic Pressure										Vertical Stress Change (Bussinesq)							Consolidation Settlement			Elastic Settlement									Settlement		
Layer	Drained/ Undrained	Layer Top (ft)	Layer Bottom (ft)	Layer Top Elevation (ft)	Layer Bottom Elevation (ft)	Layer Center Depth (ft)	Unit Weight (pcf)	Total Vertical Pressure (psf)	Effective Vertical Pressure (psf)	Layer Center Elevation (ft)	Depth Below Footing Base z (ft)	alpha (deg)	beta (deg)	Influence Factor (-)	Delta Sigma FTGS (psf)	Final Effective Vertical Pressure (psf)	NC/OC	CR (-)	RR (-)	N (bl/ft)(1,5,6)	C_E energy ratio (2)	N60 (bl/ft)	C_N (-)	Cs Sampler liner	C_B boring diameter	C_R rod length	N1,60 (bl/ft)(2,3)	C' (3)	Sublayer Thickness (ft)	Sublayer Strain ε (-)	Sublayer Settlement (in) (4)
Sand	D			16.0	16.0	0	135	0	0	16	-11.0	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-
Sand	D	0	11	16.0	5.0	5.5	135	742	742	10.5	-5.5	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-
Sand	D	11	13	5.0	3.0	12.0	135	1620	1558	4	1.0	-75.96376	151.92751	0.993834627	3975	5533	-	-	-	Assumed Medium Dense Backfill							20	66	2	0.0083	0.20
Sand	D	13	15	3.0	1.0	14.0	135	1890	1703	2	3.0	-53.1301	106.2602	0.895911961	3584	5286	-	-	-								20	66	2	0.0075	0.18
Sand	D	15	17	1.0	-1.0	16.0	135	2160	1848	0	5.0	-38.65981	77.319617	0.740099655	2960	4808	-	-	-								20	66	2	0.0063	0.15
Sand	D	17	19	-1.0	-3.0	18.0	135	2430	1993	-2	7.0	-29.74488	59.489763	0.604734891	2419	4412	-	-	-								20	66	2	0.0052	0.13
Sand	D	19	21	-3.0	-5.0	20.0	135	2700	2138	-4	9.0	-23.96249	47.924978	0.502521133	2010	4148	-	-	-								20	66	2	0.0044	0.10

Settlement 0.8

ME DOT Mill Hill Bridge
Stonington, ME
Elastic Settlement - Spread Footings (Strip Footings)

9-foot wide footing; footing at approx. El.5 feet

Data:

Footing Width W (ft) 9.0
Distance from ftg center x (ft) 0.0
Net Load From Footing (psf) 2500

Boring Ground Surface El. (ft) 16.0
Lowest Ex.Ground Surface El. (ft) 16.0
Finished Grade Elevation (ft) 16.0

FTG Depth Below Finished grade (ft) 11.0
Bottom of Footing Elevation (ft) 5.0
Depth of Unsuitable in Boring (ft) 0.0
Bottom of Unsuitable El. (ft) 16.0
GW Depth Below Existing Grade (ft) 11.0
GW Elevation (ft) 5.0
FTG Depth Below Boring (ft) 11.0
FTG El. Below Boring (ft) 5.0

Overexc. Below BOF (ft) 0.0
Backfill Below BOF (ft) 0.0
Avg Unit Weight of Unsuitable (pcf)
Unit Weight of Backfill (pcf)
Raise in Grade above Existing (ft) 0.0
Total Fill (ft) 0.0
Unload from Cut (psf) 0
Load from Fill (psf) 0.0

Soil Properties (Includes all Soils Strata Encountered in all Borings):

Layer No.	Layer ID	Gamma Sat (pcf)	CR	RR	OC/NC
1	Sand	135	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-

Evaluation of C'			Drained/
a	b	c	Undrained
0.016	1.48	29.94	D
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-

Notes:

1-Stratigraphy based on existing borings (top of rock) and cross section by KLF
2-Energy ratio ER= 70 Automatic Hammer
3 - From Figure 10.6.2.4.2-1 (Attached-AASHTO LRFD) and interpolation $C'=aN_{1,60}^2+bN_{1,60}+c$
4 - Sublayer settlement evaluated as ε times the sublayer thickness
5 - If applicable, overexcavate unsuitable and replace with Backfill (assume N=30 for Backfill)
4.5 Indicates bottom of footing

Calculations:

Geostatic Pressure										Vertical Stress Change (Bussinesq)							Consolidation Settlement			Elastic Settlement									Settlement		
Layer	Drained/ Undrained	Layer Top (ft)	Layer Bottom (ft)	Layer Top Elevation (ft)	Layer Bottom Elevation (ft)	Layer Center Depth (ft)	Unit Weight (pcf)	Total Vertical Pressure (psf)	Effective Vertical Pressure (psf)	Layer Center Elevation (ft)	Depth Below Footing Base z (ft)	alpha (deg)	beta (deg)	Influence Factor (-)	Delta Sigma FTGS (psf)	Final Effective Vertical Pressure (psf)	NC/OC	CR (-)	RR (-)	N (bl/ft)(1,5,6)	C_E energy ratio (2)	N60 (bl/ft)	C_N (-)	Cs Sampler liner	C_B boring diameter	C_R rod length	N1,60 (bl/ft)(2,3)	C' (3)	Sublayer Thickness (ft)	Sublayer Strain ε (-)	Sublayer Settlement (in) (4)
Sand	D			16.0	16.0	0	135	0	0	16	-11.0	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-
Sand	D	0	11	16.0	5.0	5.5	135	742	742	10.5	-5.5	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-
Sand	D	11	13	5.0	3.0	12.0	135	1620	1558	4	1.0	-77.47119	154.94238	0.995604624	2489	4047	-	-	-	Assumed Medium Dense Backfill							20	66	2	0.0063	0.15
Sand	D	13	15	3.0	1.0	14.0	135	1890	1703	2	3.0	-56.30993	112.61986	0.919490427	2299	4002	-	-	-								20	66	2	0.0056	0.14
Sand	D	15	17	1.0	-1.0	16.0	135	2160	1848	0	5.0	-41.98721	83.974425	0.783075851	1958	3806	-	-	-								20	66	2	0.0048	0.11
Sand	D	17	19	-1.0	-3.0	18.0	135	2430	1993	-2	7.0	-32.73523	65.470453	0.653306293	1633	3626	-	-	-								20	66	2	0.0039	0.09
Sand	D	19	21	-3.0	-5.0	20.0	135	2700	2138	-4	9.0	-26.56505	53.130102	0.549815144	1375	3513	-	-	-								20	66	2	0.0033	0.08

Settlement 0.6

ME DOT Mill Hill Bridge
Stonington, ME
Elastic Settlement - Spread Footings (Strip Footings)

9-foot wide footing; footing at approx. El.5 feet

Data:

Footing Width W (ft) 9.0
Distance from ftg center x (ft) 0.0

Boring Ground Surface El. (ft) 16.0
Lowest Ex.Ground Surface El. (ft) 16.0
Finished Grade Elevation (ft) 16.0

FTG Depth Below Finished grade (ft) 11.0
Bottom of Footing Elevation (ft) 5.0
Depth of Unsuitable in Boring (ft) 0.0
Bottom of Unsuitable El. (ft) 16.0
GW Depth Below Existing Grade (ft) 11.0
GW Elevation (ft) 5.0
FTG Depth Below Boring (ft) 11.0
FTG El. Below Boring (ft) 5.0

Net Load From Footing (psf) 3000

Overexc. Below BOF (ft) 0.0
Backfill Below BOF (ft) 0.0
Avg Unit Weight of Unsuitable (pcf)
Unit Weight of Backfill (pcf)
Raise in Grade above Existing (ft) 0.0
Total Fill (ft) 0.0
Unload from Cut (psf) 0
Load from Fill (psf) 0.0

Soil Properties (Includes all Soils Strata Encountered in all Borings):

Layer No.	Layer ID	Gamma Sat (pcf)	CR	RR	OC/NC
1	Sand	135	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-

Evaluation of C'			Drained/
a	b	c	Undrained
0.016	1.48	29.94	D
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-

Notes:

1-Stratigraphy based on existing borings (top of rock) and cross section by KLF
2-Energy ratio ER= 70 Automatic Hammer
3 - From Figure 10.6.2.4.2-1 (Attached-AASHTO LRFD) and interpolation $C'=aN_{1,60}^2+bN_{1,60}+c$
4 - Sublayer settlement evaluated as ε times the sublayer thickness
5 - If applicable, overexcavate unsuitable and replace with Backfill (assume N=30 for Backfill)
4.5 Indicates bottom of footing

Calculations:

Geostatic Pressure										Vertical Stress Change (Bussinesq)							Consolidation Settlement			Elastic Settlement									Settlement		
Layer	**Drained/ Undrained**	**Layer Top (ft)**	**Layer Bottom (ft)**	**Layer Top Elevation (ft)**	**Layer Bottom Elevation (ft)**	**Layer Center Depth (ft)**	**Unit Weight (pcf)**	**Total Vertical Pressure (psf)**	**Effective Vertical Pressure (psf)**	**Layer Center Elevation (ft)**	**Depth Below Footing Base z (ft)**	**alpha (deg)**	**beta (deg)**	**Influence Factor (-)**	**Delta Sigma FTGS (psf)**	**Final Effective Vertical Pressure (psf)**	**NC/OC**	**CR (-)**	**RR (-)**	**N (bl/ft)(1,5,6)**	**C_E energy ratio (2)**	**N60 (bl/ft)**	**C_N (-)**	**Cs Sampler liner**	**C_B boring diameter**	**C_R rod length**	**N1,60 (bl/ft)(2,3)**	**C' (3)**	**Sublayer Thickness (ft)**	**Sublayer Strain ε (-)**	**Sublayer Settlement (in) (4)**
Sand	D			16.0	16.0	0	135	0	0	16	-11.0	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-
Sand	D	0	11	16.0	5.0	5.5	135	742	742	10.5	-5.5	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-
Sand	D	11	13	5.0	3.0	12.0	135	1620	1558	4	1.0	-77.47119	154.94238	0.995604624	2987	4544	-	-	-	Assumed Medium Dense Backfill							20	66	2	0.0071	0.17
Sand	D	13	15	3.0	1.0	14.0	135	1890	1703	2	3.0	-56.30993	112.61986	0.919490427	2758	4461	-	-	-								20	66	2	0.0063	0.15
Sand	D	15	17	1.0	-1.0	16.0	135	2160	1848	0	5.0	-41.98721	83.974425	0.783075851	2349	4197	-	-	-								20	66	2	0.0054	0.13
Sand	D	17	19	-1.0	-3.0	18.0	135	2430	1993	-2	7.0	-32.73523	65.470453	0.653306293	1960	3953	-	-	-								20	66	2	0.0045	0.11
Sand	D	19	21	-3.0	-5.0	20.0	135	2700	2138	-4	9.0	-26.56505	53.130102	0.549815144	1649	3788	-	-	-								20	66	2	0.0038	0.09

Settlement 0.6

ME DOT Mill Hill Bridge
Stonington, ME
Elastic Settlement - Spread Footings (Strip Footings)

9-foot wide footing; footing at approx. El.5 feet

Data:

Footing Width W (ft) 9.0
Distance from ftg center x (ft) 0.0

Boring Ground Surface El. (ft) 16.0
Lowest Ex.Ground Surface El. (ft) 16.0
Finished Grade Elevation (ft) 16.0

FTG Depth Below Finished grade (ft) 11.0
Bottom of Footing Elevation (ft) 5.0
Depth of Unsuitable in Boring (ft) 0.0
Bottom of Unsuitable El. (ft) 16.0
GW Depth Below Existing Grade (ft) 11.0
GW Elevation (ft) 5.0
FTG Depth Below Boring (ft) 11.0
FTG El. Below Boring (ft) 5.0

Net Load From Footing (psf) 3500

Overexc. Below BOF (ft) 0.0
Backfill Below BOF (ft) 0.0
Avg Unit Weight of Unsuitable (pcf)
Unit Weight of Backfill (pcf)
Raise in Grade above Existing (ft) 0.0
Total Fill (ft) 0.0
Unload from Cut (psf) 0
Load from Fill (psf) 0.0

Soil Properties (Includes all Soils Strata Encountered in all Borings):

Layer No.	Layer ID	Gamma Sat (pcf)	CR	RR	OC/NC
1	Sand	135	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-

Evaluation of C'			Drained/
a	b	c	Undrained
0.016	1.48	29.94	D
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-

Notes:

1-Stratigraphy based on existing borings (top of rock) and cross section by KLF
2-Energy ratio ER= 70 Automatic Hammer
3 - From Figure 10.6.2.4.2-1 (Attached-AASHTO LRFD) and interpolation $C'=aN_{1,60}^2+bN_{1,60}+c$
4 - Sublayer settlement evaluated as ε times the sublayer thickness
5 - If applicable, overexcavate unsuitable and replace with Backfill (assume N=30 for Backfill)
4.5 Indicates bottom of footing

Calculations:

Geostatic Pressure										Vertical Stress Change (Bussinesq)							Consolidation Settlement			Elastic Settlement									Settlement		
Layer	**Drained/ Undrained**	**Layer Top (ft)**	**Layer Bottom (ft)**	**Layer Top Elevation (ft)**	**Layer Bottom Elevation (ft)**	**Layer Center Depth (ft)**	**Unit Weight (pcf)**	**Total Vertical Pressure (psf)**	**Effective Vertical Pressure (psf)**	**Layer Center Elevation (ft)**	**Depth Below Footing Base z (ft)**	**alpha (deg)**	**beta (deg)**	**Influence Factor (-)**	**Delta Sigma FTGS (psf)**	**Final Effective Vertical Pressure (psf)**	**NC/OC**	**CR (-)**	**RR (-)**	**N (bl/ft)(1,5,6)**	C_E energy ratio **(2)**	**N60 (bl/ft)**	C_N (-)	**Cs Sampler liner**	C_B boring diameter	C_R rod length	**N1,60 (bl/ft)(2,3)**	**C' (3)**	**Sublayer Thickness (ft)**	**Sublayer Strain ε (-)**	**Sublayer Settlement (in) (4)**
Sand	D			16.0	16.0	0	135	0	0	16	-11.0	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-
Sand	D	0	11	16.0	5.0	5.5	135	742	742	10.5	-5.5	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-
Sand	D	11	13	5.0	3.0	12.0	135	1620	1558	4	1.0	-77.47119	154.94238	0.995604624	3485	5042	-	-	-	Assumed Medium Dense Backfill							20	66	2	0.0077	0.19
Sand	D	13	15	3.0	1.0	14.0	135	1890	1703	2	3.0	-56.30993	112.61986	0.919490427	3218	4921	-	-	-								20	66	2	0.0070	0.17
Sand	D	15	17	1.0	-1.0	16.0	135	2160	1848	0	5.0	-41.98721	83.974425	0.783075851	2741	4589	-	-	-								20	66	2	0.0060	0.14
Sand	D	17	19	-1.0	-3.0	18.0	135	2430	1993	-2	7.0	-32.73523	65.470453	0.653306293	2287	4280	-	-	-								20	66	2	0.0050	0.12
Sand	D	19	21	-3.0	-5.0	20.0	135	2700	2138	-4	9.0	-26.56505	53.130102	0.549815144	1924	4063	-	-	-								20	66	2	0.0042	0.10

Settlement **0.72**

ME DOT Mill Hill Bridge
Stonington, ME
Elastic Settlement - Spread Footings (Strip Footings)

9-foot wide footing; footing at approx. El.5 feet

Data:

Footing Width W (ft)	**9.0**		
Distance from ftg center x (ft)	**0.0**	Net Load From Footing (psf)	4000
Boring Ground Surface El. (ft)	**16.0**	Overexc. Below BOF (ft)	0.0
Lowest Ex.Ground Surface El. (ft)	**16.0**	Backfill Below BOF (ft)	0.0
Finished Grade Elevation (ft)	**16.0**	Avg Unit Weight of Unsuitable (pcf)	
		Unit Weight of Backfill (pcf)	
FTG Depth Below Finished grade (ft)	**11.0**	Raise in Grade above Existing (ft)	0.0
Bottom of Footing Elevation (ft)	5.0	Total Fill (ft)	0.0
Depth of Unsuitable in Boring (ft)	**0.0**	Unload from Cut (psf)	0
Bottom of Unsuitable El. (ft)	16.0	Load from Fill (psf)	0.0
GW Depth Below Existing Grade (ft)	**11.0**		
GW Elevation (ft)	5.0		
FTG Depth Below Boring (ft)	11.0		
FTG El. Below Boring (ft)	5.0		

Soil Properties (Includes all Soils Strata Encountered in all Borings):

Layer No.	Layer ID	Gamma Sat (pcf)	CR	RR	OC/NC
1	Sand	135	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-

Evaluation of C'			Drained/
a	b	c	Undrained
0.016	1.48	29.94	D
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-

Notes:

1-Stratigraphy based on existing borings (top of rock) and cross section by KLF

2-Energy ratio ER= 70 Automatic Hammer

3 - From Figure 10.6.2.4.2-1 (Attached-AASHTO LRFD) and interpolation $C'=aN_{1,60}^2+bN_{1,60}+c$

4 - Sublayer settlement evaluated as ε times the sublayer thickness

5 - If applicable, overexcavate unsuitable and replace with Backfill (assume N=30 for Backfill)

4.5 Indicates bottom of footing

Calculations:

Geostatic Pressure										Vertical Stress Change (Bussinesq)							Consolidation Settlement			Elastic Settlement									Settlement		
Layer	**Drained/ Undrained**	**Layer Top (ft)**	**Layer Bottom (ft)**	**Layer Top Elevation (ft)**	**Layer Bottom Elevation (ft)**	**Layer Center Depth (ft)**	**Unit Weight (pcf)**	**Total Vertical Pressure (psf)**	**Effective Vertical Pressure (psf)**	**Layer Center Elevation (ft)**	**Depth Below Footing Base z (ft)**	**alpha (deg)**	**beta (deg)**	**Influence Factor (-)**	**Delta Sigma FTGS (psf)**	**Final Effective Vertical Pressure (psf)**	**NC/OC**	**CR (-)**	**RR (-)**	**N (bl/ft)$^{(1,5,6)}$**	C_E energy ratio $^{(2)}$	**N60 (bl/ft)**	C_N (-)	**Cs Sampler liner**	C_B boring diameter	C_R rod length	**N1,60 (bl/ft)$^{(2,3)}$**	**C' $^{(3)}$**	**Sublayer Thickness (ft)**	**Sublayer Strain ε (-)**	**Sublayer Settlement (in) $^{(4)}$**
Sand	D			16.0	16.0	0	135	0	0	16	-11.0	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-
Sand	D	0	11	16.0	5.0	5.5	135	742	742	10.5	-5.5	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-
Sand	D	11	13	5.0	3.0	12.0	135	1620	1558	4	1.0	-77.47119	154.94238	0.995604624	3982	5540	-	-	-	Assumed Medium Dense Backfill							20	66	2	0.0084	0.20
Sand	D	13	15	3.0	1.0	14.0	135	1890	1703	2	3.0	-56.30993	112.61986	0.919490427	3678	5381	-	-	-								20	66	2	0.0076	0.18
Sand	D	15	17	1.0	-1.0	16.0	135	2160	1848	0	5.0	-41.98721	83.974425	0.783075851	3132	4980	-	-	-								20	66	2	0.0065	0.16
Sand	D	17	19	-1.0	-3.0	18.0	135	2430	1993	-2	7.0	-32.73523	65.470453	0.653306293	2613	4606	-	-	-								20	66	2	0.0055	0.13
Sand	D	19	21	-3.0	-5.0	20.0	135	2700	2138	-4	9.0	-26.56505	53.130102	0.549815144	2199	4338	-	-	-								20	66	2	0.0047	0.11

Settlement **0.8**

ME DOT Mill Hill Bridge
Stonington, ME
Elastic Settlement - Spread Footings (Strip Footings)

9-foot wide footing; footing at approx. El.5 feet

Data:

Footing Width W (ft) 9.0
Distance from ftg center x (ft) 0.0

Boring Ground Surface El. (ft) 16.0
Lowest Ex.Ground Surface El. (ft) 16.0
Finished Grade Elevation (ft) 16.0

FTG Depth Below Finished grade (ft) 11.0
Bottom of Footing Elevation (ft) 5.0
Depth of Unsuitable in Boring (ft) 0.0
Bottom of Unsuitable El. (ft) 16.0
GW Depth Below Existing Grade (ft) 11.0
GW Elevation (ft) 5.0
FTG Depth Below Boring (ft) 11.0
FTG El. Below Boring (ft) 5.0

Net Load From Footing (psf) 4400

Overexc. Below BOF (ft) 0.0
Backfill Below BOF (ft) 0.0
Avg Unit Weight of Unsuitable (pcf)
Unit Weight of Backfill (pcf)
Raise in Grade above Existing (ft) 0.0
Total Fill (ft) 0.0
Unload from Cut (psf) 0
Load from Fill (psf) 0.0

Soil Properties (Includes all Soils Strata Encountered in all Borings):

Layer No.	Layer ID	Gamma Sat (pcf)	CR	RR	OC/NC
1	Sand	135	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-
-	-	-	-	-	-

Evaluation of C'			Drained/
a	b	c	Undrained
0.016	1.48	29.94	D
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-

Notes:

1-Stratigraphy based on existing borings (top of rock) and cross section by KLF
2-Energy ratio ER= 70 Automatic Hammer
3 - From Figure 10.6.2.4.2-1 (Attached-AASHTO LRFD) and interpolation $C'=aN_{1,60}^2+bN_{1,60}+c$
4 - Sublayer settlement evaluated as ε times the sublayer thickness
5 - If applicable, overexcavate unsuitable and replace with Backfill (assume N=30 for Backfill)
4.5 Indicates bottom of footing

Calculations:

Geostatic Pressure										Vertical Stress Change (Bussinesq)							Consolidation Settlement			Elastic Settlement									Settlement		
Layer	**Drained/ Undrained**	**Layer Top (ft)**	**Layer Bottom (ft)**	**Layer Top Elevation (ft)**	**Layer Bottom Elevation (ft)**	**Layer Center Depth (ft)**	**Unit Weight (pcf)**	**Total Vertical Pressure (psf)**	**Effective Vertical Pressure (psf)**	**Layer Center Elevation (ft)**	**Depth Below Footing Base z (ft)**	**alpha (deg)**	**beta (deg)**	**Influence Factor (-)**	**Delta Sigma FTGS (psf)**	**Final Effective Vertical Pressure (psf)**	**NC/OC**	**CR (-)**	**RR (-)**	**N (bl/ft)$^{(1,5,6)}$**	**C_E** energy ratio **$^{(2)}$**	**N60 (bl/ft)**	**C_N (-)**	**Cs Sampler liner**	**C_B** boring diameter	**C_R** rod length	**N1,60 (bl/ft)$^{(2,3)}$**	**C' $^{(3)}$**	**Sublayer Thickness (ft)**	**Sublayer Strain ε (-)**	**Sublayer Settlement (in) $^{(4)}$**
Sand	D			16.0	16.0	0	135	0	0	16	-11.0	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-
Sand	D	0	11	16.0	5.0	5.5	135	742	742	10.5	-5.5	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-
Sand	D	11	13	5.0	3.0	12.0	135	1620	1558	4	1.0	-77.47119	154.94238	0.995604624	4381	5938	-	-	-	Assumed Medium Dense Backfill							20	66	2	0.0088	0.21
Sand	D	13	15	3.0	1.0	14.0	135	1890	1703	2	3.0	-56.30993	112.61986	0.919490427	4046	5749	-	-	-								20	66	2	0.0080	0.19
Sand	D	15	17	1.0	-1.0	16.0	135	2160	1848	0	5.0	-41.98721	83.974425	0.783075851	3446	5294	-	-	-								20	66	2	0.0069	0.17
Sand	D	17	19	-1.0	-3.0	18.0	135	2430	1993	-2	7.0	-32.73523	65.470453	0.653306293	2875	4868	-	-	-								20	66	2	0.0059	0.14
Sand	D	19	21	-3.0	-5.0	20.0	135	2700	2138	-4	9.0	-26.56505	53.130102	0.549815144	2419	4558	-	-	-								20	66	2	0.0050	0.12

Settlement **0.8**

Extreme Event Limit State I

Objective: Determine Seismic design parameters for the Mill Hill Bridge at the Stonington-Deer Isle townline.

References: 1. AASHTO LRFD Bridge Design Specifications, 8th Ed
2. AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Ed. with addenda through 2015

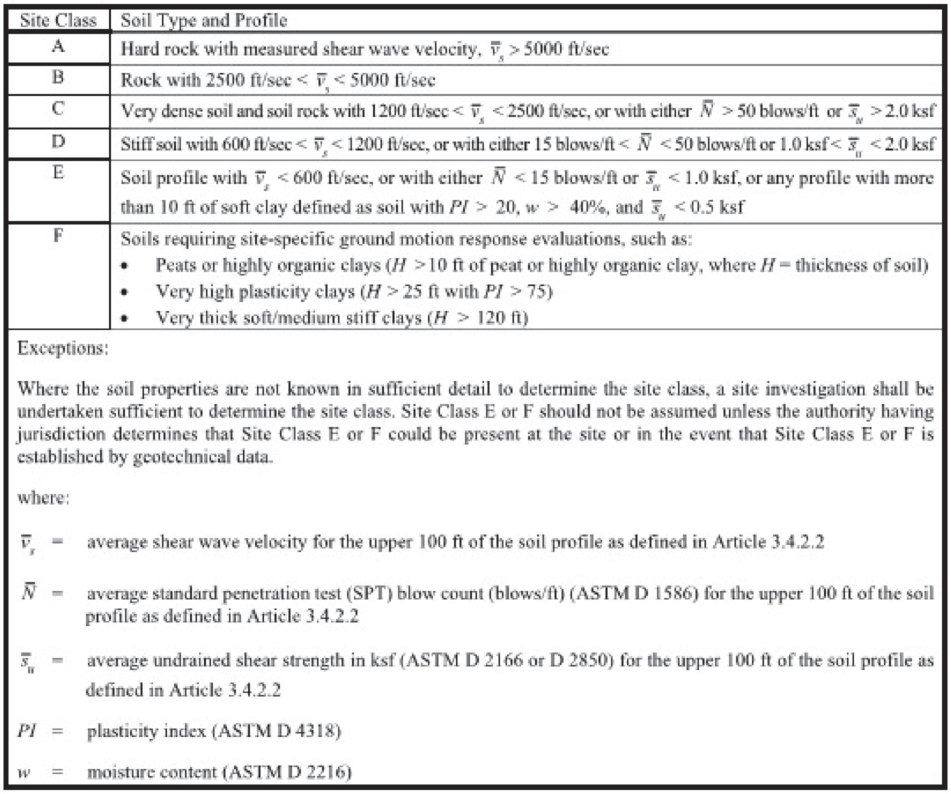
Soil Borings: Seismic parameters based on borings BB-SDIHP-103 and BB-SDIHP-106. The borings were advanced by New England Boring Contractors in May 2018 and overseen by Kleinfelder professionals.

Calculation:

Determine Site Class (Based on AASHTO Table 3.10.3.1-1):

Abutments bear on Rock
Shear wave velocity is unknown
Therefore, Site Class = B

Table 3.4.2.1-1—Site Class Definitions



Site Class	Soil Type and Profile
A	Hard rock with measured shear wave velocity, $\bar{v}_s > 5000$ ft/sec
B	Rock with 2500 ft/sec < $\bar{v}_s$ < 5000 ft/sec
C	Very dense soil and soil rock with 1200 ft/sec < $\bar{v}_s$ < 2500 ft/sec, or with either $\bar{N}$ > 50 blows/ft or $\bar{s}_u$ > 2.0 ksf
D	Stiff soil with 600 ft/sec < $\bar{v}_s$ < 1200 ft/sec, or with either 15 blows/ft < $\bar{N}$ < 50 blows/ft or 1.0 ksf < $\bar{s}_u$ < 2.0 ksf
E	Soil profile with $\bar{v}_s$ < 600 ft/sec, or with either $\bar{N}$ < 15 blows/ft or $\bar{s}_u$ < 1.0 ksf, or any profile with more than 10 ft of soft clay defined as soil with PI > 20, w > 40%, and $\bar{s}_u$ < 0.5 ksf
F	Soils requiring site-specific ground motion response evaluations, such as: • Peats or highly organic clays (H >10 ft of peat or highly organic clay, where H = thickness of soil) • Very high plasticity clays (H > 25 ft with PI > 75) • Very thick soft/medium stiff clays (H > 120 ft)

Exceptions:

Where the soil properties are not known in sufficient detail to determine the site class, a site investigation shall be undertaken sufficient to determine the site class. Site Class E or F should not be assumed unless the authority having jurisdiction determines that Site Class E or F could be present at the site or in the event that Site Class E or F is established by geotechnical data.

where:

$\bar{v}_s$ = average shear wave velocity for the upper 100 ft of the soil profile as defined in Article 3.4.2.2

$\bar{N}$ = average standard penetration test (SPT) blow count (blows/ft) (ASTM D 1586) for the upper 100 ft of the soil profile as defined in Article 3.4.2.2

$\bar{s}_u$ = average undrained shear strength in ksf (ASTM D 2166 or D 2850) for the upper 100 ft of the soil profile as defined in Article 3.4.2.2

PI = plasticity index (ASTM D 4318)

w = moisture content (ASTM D 2216)

Calc By: J. MacGregor, 3/11/2021
Check By: M. Chea, 3/11/2021

Enter Seismic Parameters from figures provided in AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Ed. with addenda through 2015 (Attachment A):

$PGA := 0.055\ g$

$S_S := 0.120\ g$

$S_1 := 0.038\ g$

Determine Site Coefficients from tables in AASHTO Guide Specifications for LRFD Seismic Bridge Design

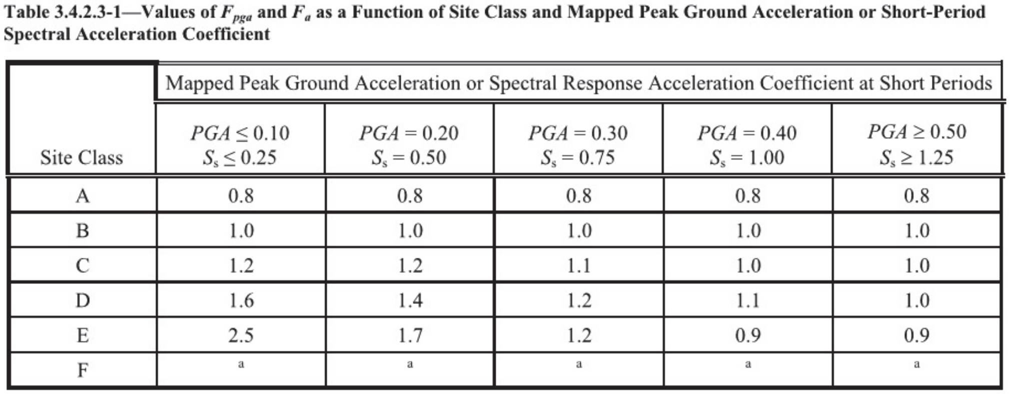


Table 3.4.2.3-1—Values of F_{pga} and F_a as a Function of Site Class and Mapped Peak Ground Acceleration or Short-Period Spectral Acceleration Coefficient

Site Class	Mapped Peak Ground Acceleration or Spectral Response Acceleration Coefficient at Short Periods $PGA \le 0.10$ $S_s \le 0.25$	 $PGA = 0.20$ $S_s = 0.50$	 $PGA = 0.30$ $S_s = 0.75$	 $PGA = 0.40$ $S_s = 1.00$	 $PGA \ge 0.50$ $S_s \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	a	a	a	a	a

$F_{pga} := 1.0$

$F_a := 1.0$

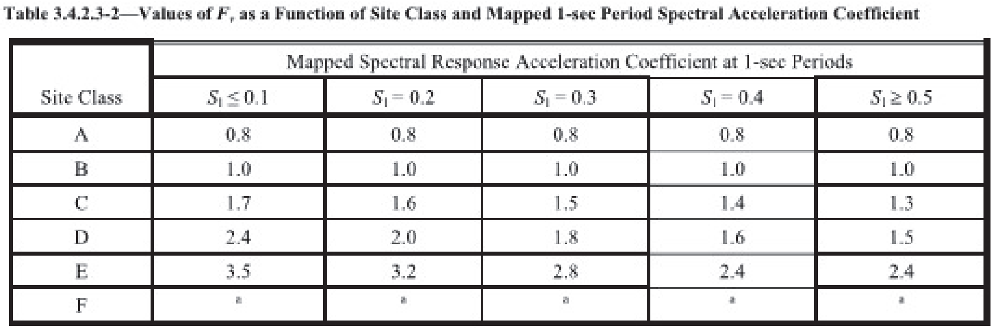


Table 3.4.2.3-2—Values of F_v as a Function of Site Class and Mapped 1-sec Period Spectral Acceleration Coefficient

Site Class	Mapped Spectral Response Acceleration Coefficient at 1-sec Periods $S_1 \le 0.1$	 $S_1 = 0.2$	 $S_1 = 0.3$	 $S_1 = 0.4$	 $S_1 \ge 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	a	a	a	a	a

$F_v := 1.0$

$A_S := PGA \cdot F_{pga} = 0.055\ g$ Eq. 3.4.1-1

$S_{DS} := S_S \cdot F_a = 0.12\ g$ Eq. 3.4.1-2

$S_{D1} := S_1 \cdot F_v = 0.038\ g$ Eq. 3.4.1-3

Determine Seismic Design Category from tables in AASHTO Guide Specifications for LRFD Seismic Bridge Design

$$SDC := \text{“A”}$$

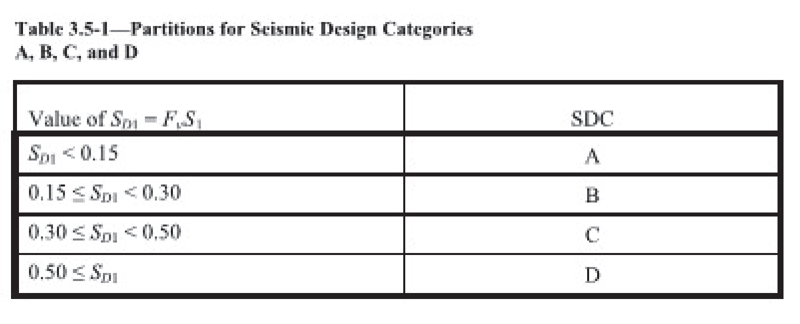
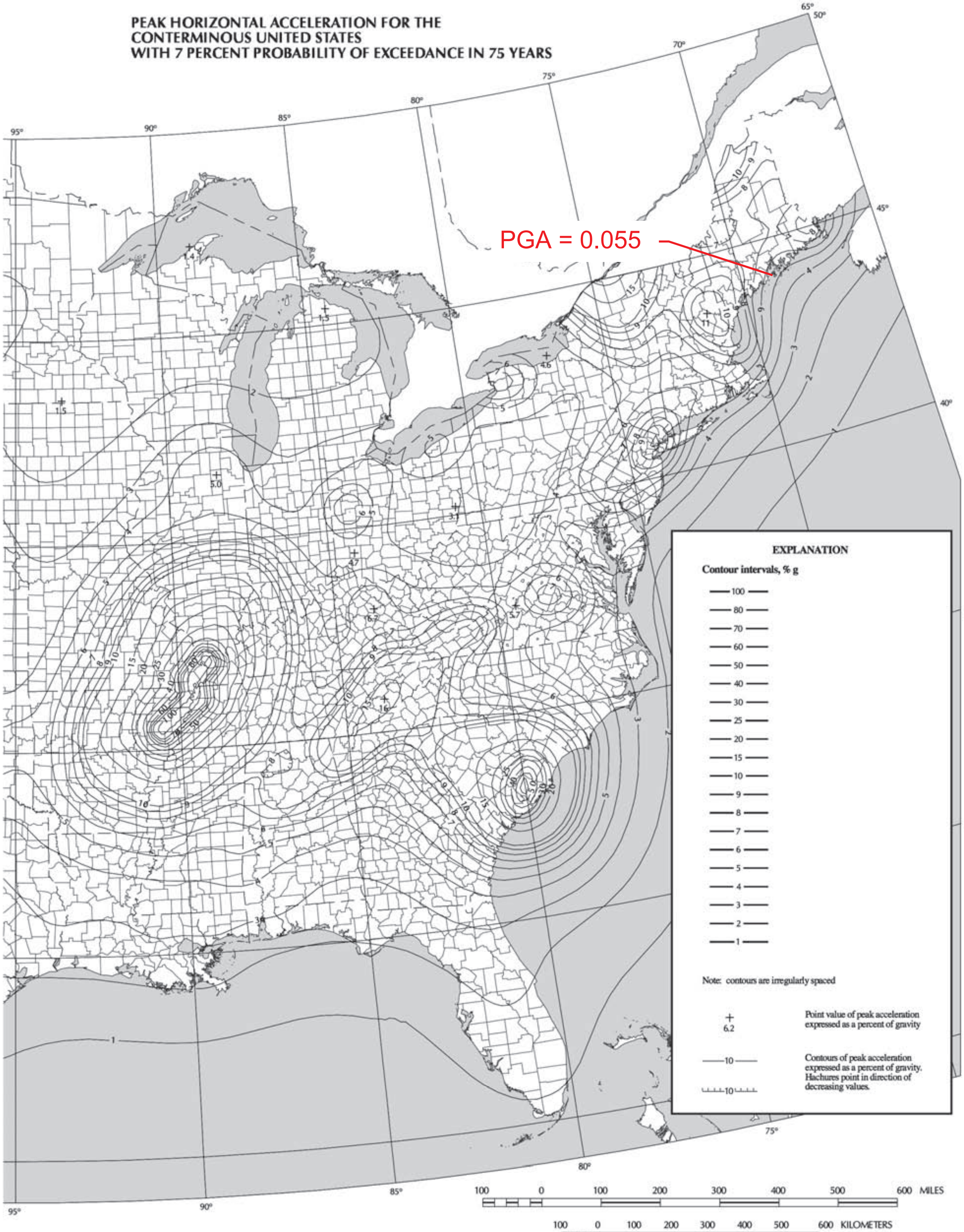


Table 3.5-1—Partitions for Seismic Design Categories A, B, C, and D

Value of $S_{D1} = F_v S_1$	SDC
$S_{D1} < 0.15$	A
$0.15 \leq S_{D1} < 0.30$	B
$0.30 \leq S_{D1} < 0.50$	C
$0.50 \leq S_{D1}$	D

Calc By: J. MacGregor, 3/11/2021
Check By: M. Chea, 3/11/2021



PGA = 0.055

Figure 3.4.1-2b—Horizontal Peak Ground Acceleration Coefficient for the Conterminous United States (*PGA*) with Seven Percent Probability of Exceedance in 75 yr (Approx. 1000-yr Return Period)

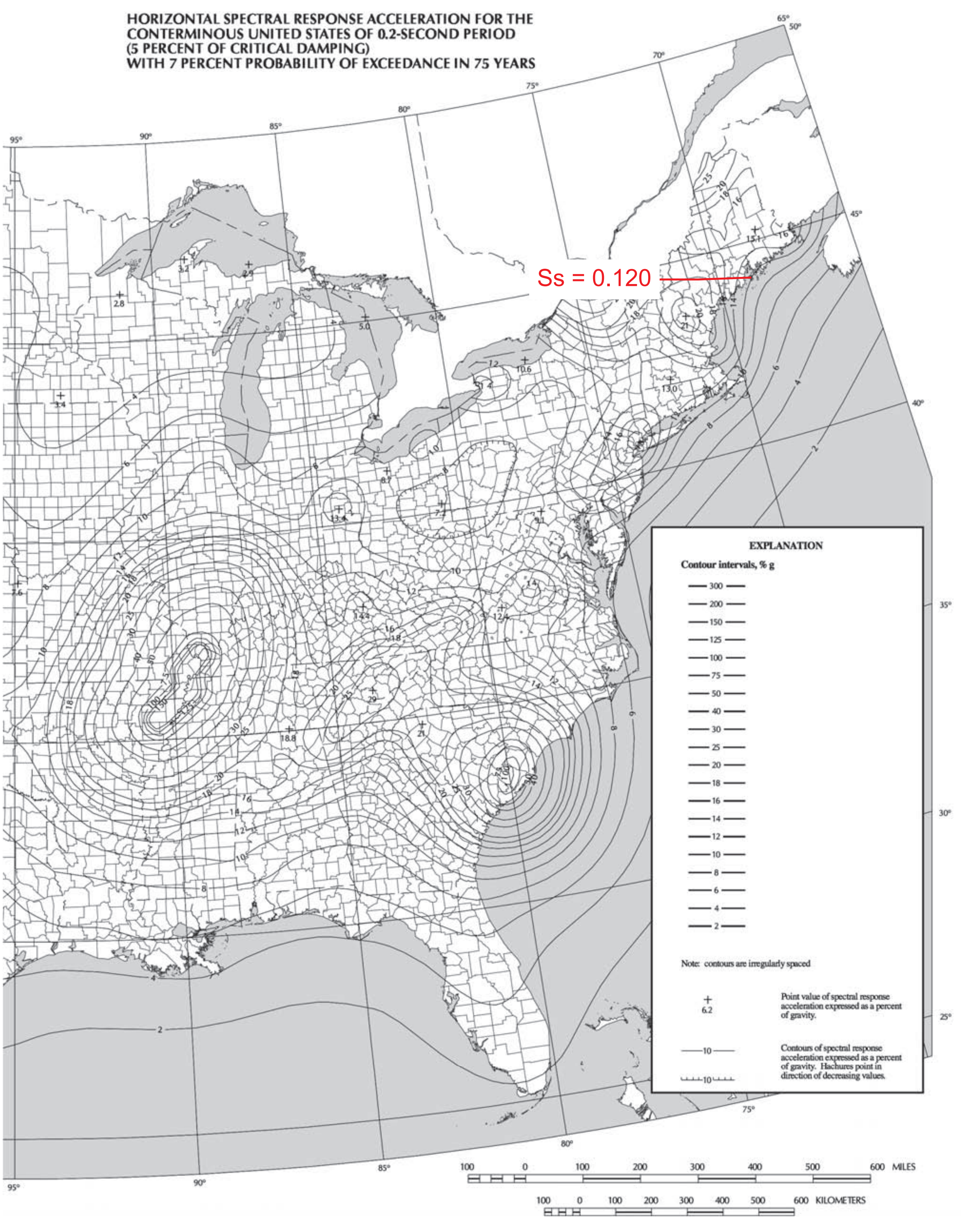


Figure 3.4.1-3b—Horizontal Response Spectral Acceleration Coefficient for the Conterminous United States at Period of 0.2-sec (S_s) with Seven Percent Probability of Exceedance in 75 yr (Approx. 1000-yr Return Period) and Five Percent Critical Damping

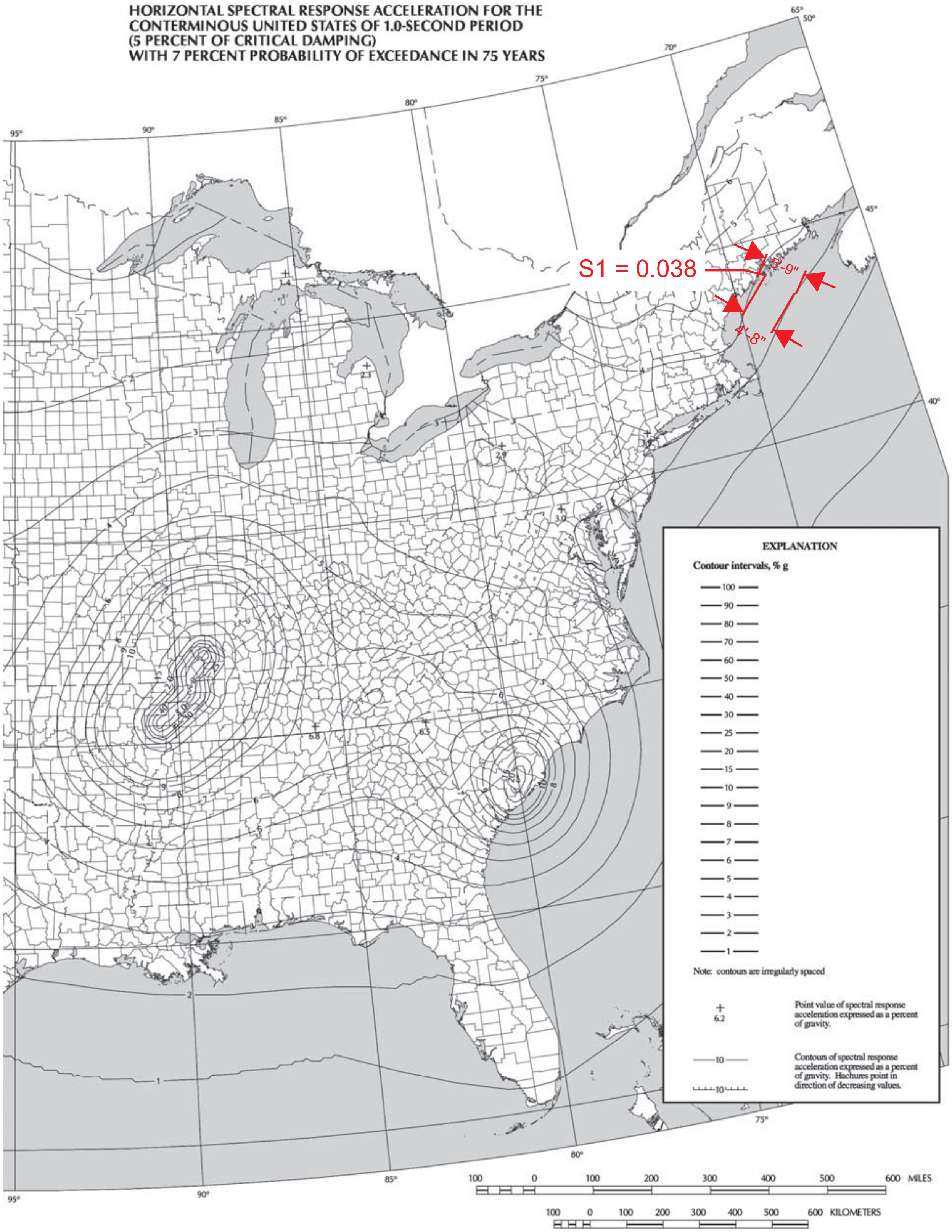


Figure 3.4.1-4b—Horizontal Response Spectral Acceleration Coefficient for the Conterminous United States at Period of 1.0-sec (S_1) with Seven Percent Probability of Exceedance in 75 yr (Approx. 1000-yr Return Period) and Five Percent Critical Damping

CONFIDENTIAL

Mononobe-Okabe Earth Pressure Coefficient

Project: ME DOT Mill Hill Bridge
Location: Deer Isle, Maine
Calculated By: MR **Date:** 4/27/2022
Checked By: MK **Date:** 6/23/2022

Objective: Evaluate the Mononobe-Okabe active earth pressure coefficient. Assume backfill parameters as described in Table 3-3 of the Maine DOT manual for gravel borrow backfill

References:
1) Soil Mechanics in Engineering Practice - Terzaghi, Peck and Mesri
2) ME DOT Bridge Manual
3) AASHTO LRFD Bridge Specifications

B wall (ft)= 9
Wall Stem (ft)= 1.70

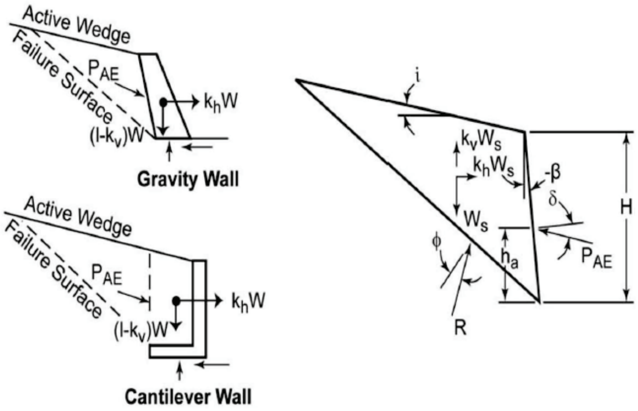


Figure A11.3.1-1—Mononobe-Okabe Method Force Diagrams

γ=	135		
H wall (ft)=	10.5		
h wall (ft)=	11.0		
Internal Friction Angle ϕ'=	36	0.63	Rad
δMO=	3.3	0.06	Rad
Interface Friction Angle δ=	27	0.47	Rad
Kh (-)=	0.055		
Kv (-)=	0.055		
Backfill Slope i=	5	0.09	Rad
Wall back slope to vertical β=	0	0.00	Rad

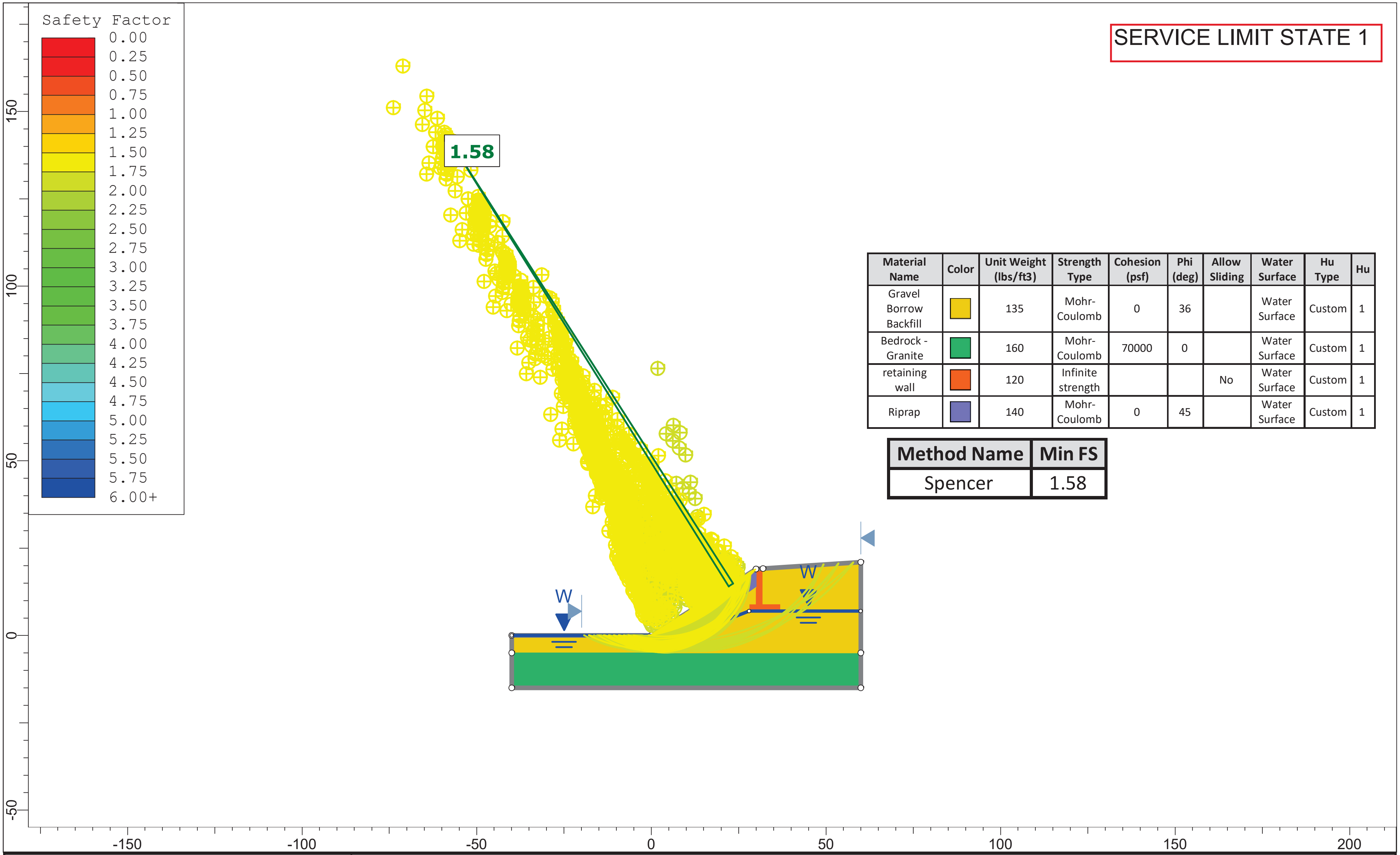
K_{AE} = 0.29

$$K_{AE} = \frac{\cos^2(\phi-\theta_{MO}-\beta)}{\cos\theta_{MO}\cos^2\beta\cos(\delta+\beta+\theta_{MO})} \times \left[1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\theta_{MO}-i)}{\cos(\delta+\beta+\theta_{MO})\cos(i-\beta)}}\right]^{-2} \qquad \text{(A11.3.1-1)}$$

where:

K_{AE} = seismic active earth pressure coefficient (dim)
γ = unit weight of soil (kcf)
H = height of wall (ft)
h = height of wall at back of wall heel considering height of sloping surcharge, if present (ft)
ϕ_f = friction angle of soil (degrees)
θ_{MO} = arc tan $[k_h/(1-k_v)]$ (degrees)
δ = wall backfill interface friction angle (degrees)
k_h = horizontal seismic acceleration coefficient (dim.)
k_v = vertical seismic acceleration coefficient (dim.)
i = backfill slope angle (degrees)
β = slope of wall to the vertical, negative as shown (degrees)

Limit Equilibrium Calculations



SERVICE LIMIT STATE 1

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Allow Sliding	Water Surface	Hu Type	Hu
Gravel Borrow Backfill	🟨	135	Mohr-Coulomb	0	36		Water Surface	Custom	1
Bedrock - Granite	🟩	160	Mohr-Coulomb	70000	0		Water Surface	Custom	1
retaining wall	🟧	120	Infinite strength			No	Water Surface	Custom	1
Riprap	🟪	140	Mohr-Coulomb	0	45		Water Surface	Custom	1

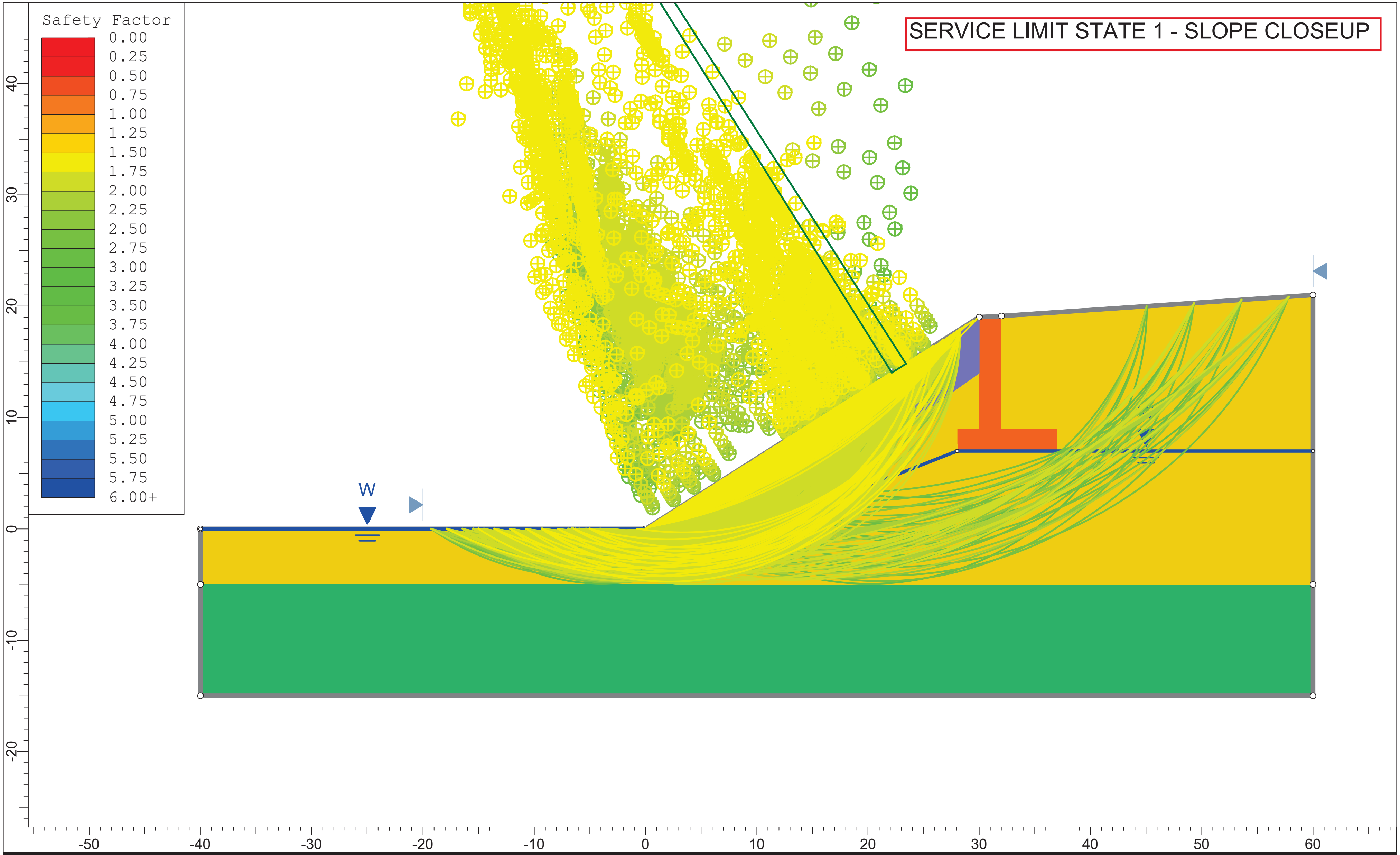
Method Name	Min FS
Spencer	1.58

rocscience

SLIDEINTERPRET 9.018

Project	Deer Island Bridge		
Group	Group 1	*Scenario*	Master Scenario
Drawn By	MR	*Company*	KLF
Date	6/6/2022, 8:42:02 PM	*File Name*	Slide1-Service.slmd

Checked by: MK Date: 6/23/2022



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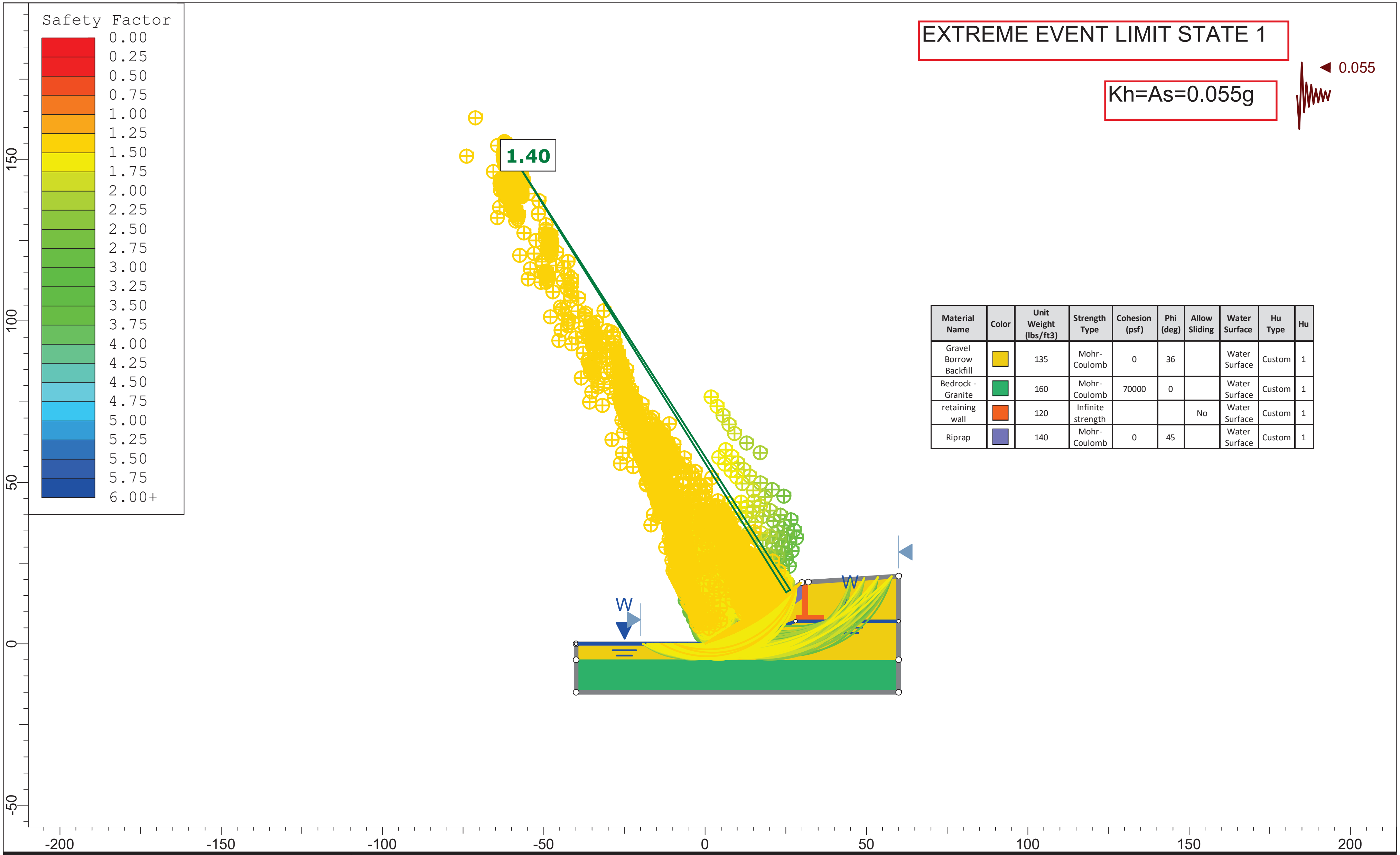
SLIDEINTERPRET 9.018

Project	Deer Island Bridge		
Group	Group 1	*Scenario*	Master Scenario
Drawn By	MR	*Company*	KLF
Date	6/6/2022, 8:42:02 PM	*File Name*	Slide1-Service.slmd

Checked by: MK Date: 6/23/2022



EXTREME EVENT LIMIT STATE 1

Kh=As=0.055g

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Allow Sliding	Water Surface	Hu Type	Hu
Gravel Borrow Backfill	🟨	135	Mohr-Coulomb	0	36		Water Surface	Custom	1
Bedrock - Granite	🟩	160	Mohr-Coulomb	70000	0		Water Surface	Custom	1
retaining wall	🟧	120	Infinite strength			No	Water Surface	Custom	1
Riprap	🟪	140	Mohr-Coulomb	0	45		Water Surface	Custom	1

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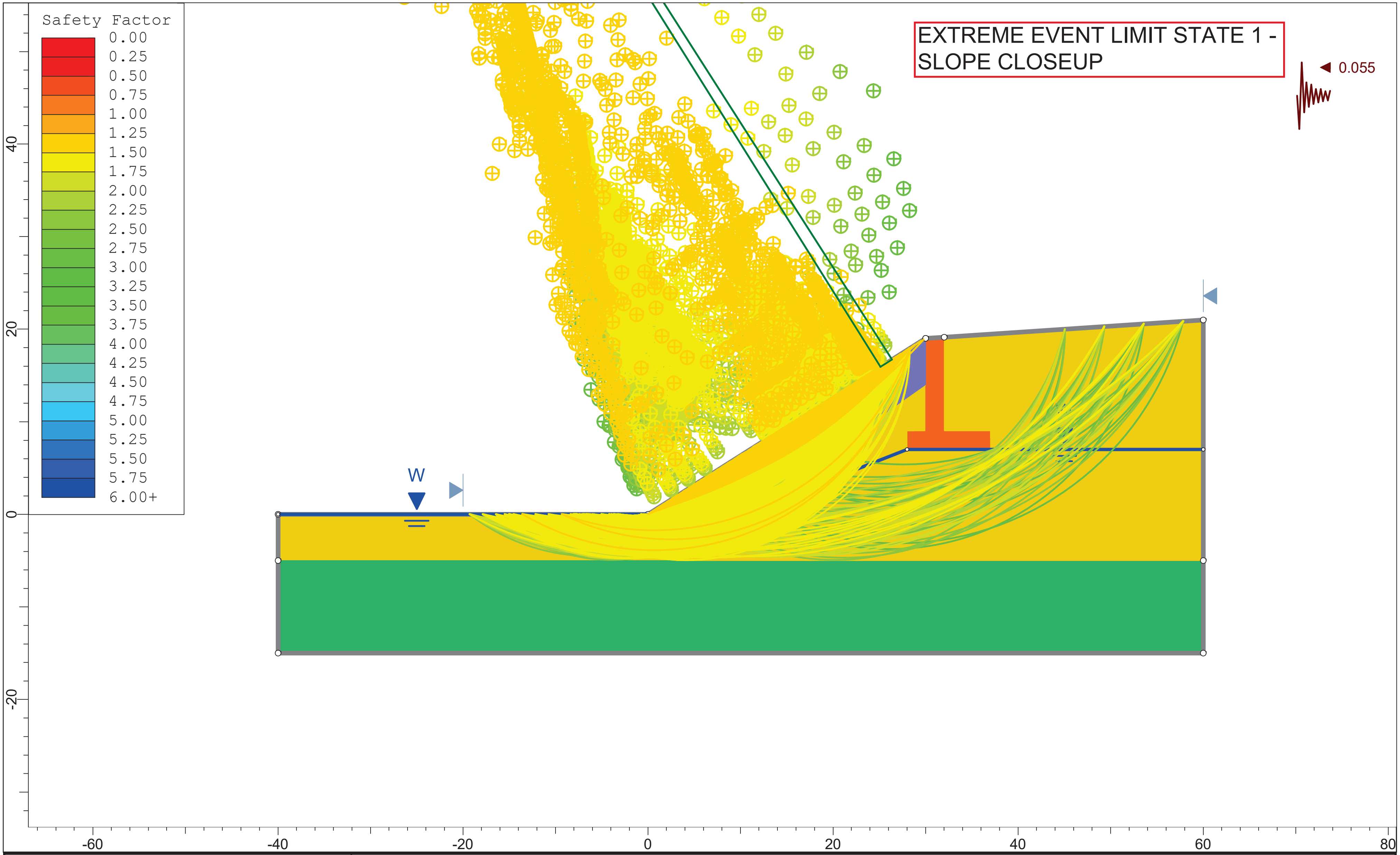
SLIDEINTERPRET 9.018

Project	Deer Island Bridge		
Group	Deer Island Bridge	*Scenario*	Seismic
Drawn By	MR	*Company*	KLF
Date	6/6/2022, 8:42:02 PM	*File Name*	Slide1-Service.slmd

Checked by: MK Date: 6/23/2022



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SLIDEINTERPRET 9.018

Project	Deer Island Bridge		
Group	Deer Island Bridge	*Scenario*	Seismic
Drawn By	MR	*Company*	KLF
Date	6/6/2022, 8:42:02 PM	*File Name*	Slide1-Service.slmd

Checked by: MK Date: 6/23/2022

Appendix F – GBA Information Sheet

Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, you can benefit from a lowered exposure to problems associated with subsurface conditions at project sites and development of them that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed herein, contact your GBA-member geotechnical engineer. Active engagement in GBA exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Understand the Geotechnical-Engineering Services Provided for this Report

Geotechnical-engineering services typically include the planning, collection, interpretation, and analysis of exploratory data from widely spaced borings and/or test pits. Field data are combined with results from laboratory tests of soil and rock samples obtained from field exploration (if applicable), observations made during site reconnaissance, and historical information to form one or more models of the expected subsurface conditions beneath the site. Local geology and alterations of the site surface and subsurface by previous and proposed construction are also important considerations. Geotechnical engineers apply their engineering training, experience, and judgment to adapt the requirements of the prospective project to the subsurface model(s). Estimates are made of the subsurface conditions that will likely be exposed during construction as well as the expected performance of foundations and other structures being planned and/or affected by construction activities.

The culmination of these geotechnical-engineering services is typically a geotechnical-engineering report providing the data obtained, a discussion of the subsurface model(s), the engineering and geologic engineering assessments and analyses made, and the recommendations developed to satisfy the given requirements of the project. These reports may be titled investigations, explorations, studies, assessments, or evaluations. Regardless of the title used, the geotechnical-engineering report is an engineering interpretation of the subsurface conditions within the context of the project and does not represent a close examination, systematic inquiry, or thorough investigation of all site and subsurface conditions.

Geotechnical-Engineering Services are Performed for Specific Purposes, Persons, and Projects, and At Specific Times

Geotechnical engineers structure their services to meet the specific needs, goals, and risk management preferences of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client.

Likewise, geotechnical-engineering services are performed for a specific project and purpose. For example, it is unlikely that a geotechnical-engineering study for a refrigerated warehouse will be the same as one prepared for a parking garage; and a few borings drilled during a preliminary study to evaluate site feasibility will not be adequate to develop geotechnical design recommendations for the project.

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project or purpose;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, the reliability of a geotechnical-engineering report can be affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying the recommendations in it. A minor amount of additional testing or analysis after the passage of time – if any is required at all – could prevent major problems.

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read the report in its entirety. Do not rely on an executive summary. Do not read selective elements only. *Read and refer to the report in full.*

You Need to Inform Your Geotechnical Engineer About Change

Your geotechnical engineer considered unique, project-specific factors when developing the scope of study behind this report and developing the confirmation-dependent recommendations the report conveys. Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the elevation, configuration, location, orientation, function or weight of the proposed structure and the desired performance criteria;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project or site changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept* *responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface using various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing is performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgement to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team through project completion to obtain informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, they are <u>not</u> final, because the geotechnical engineer who developed them relied heavily on judgement and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* exposed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a continuing member of the design team, to:

- confer with other design-team members;
- help develop specifications;
- review pertinent elements of other design professionals' plans and specifications; and
- be available whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction-phase observations.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for information purposes only.* To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. This happens in part because soil and rock on project sites are typically heterogeneous and not manufactured materials with well-defined engineering properties like steel and concrete. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are <u>Not</u> Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually provide environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures.* If you have not obtained your own environmental information about the project site, ask your geotechnical consultant for a recommendation on how to find environmental risk-management guidance.

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, the engineer's services were not designed, conducted, or intended to prevent migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will <u>not</u> of itself be sufficient to prevent moisture infiltration. Confront the risk of moisture infiltration*** by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are <u>not</u> building-envelope or mold specialists.***

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