MAINE DEPARTMENT OF TRANSPORTATION
HIGHWAY/BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

STATE ROUTE 32 STRUT
BREMEN, MAINE

STATE OF MAINE
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GEOTECHNICAL DESIGN SUMMARY

The purpose of this report is to present subsurface information and make geotechnical recommendations for the replacement of two undersized struts (pipe culverts) which carry Muscongus Brook under State Route 32 at the Bristol-Bremen town line. The proposed replacement structure will be a single-span structure with a span of approximately 14 feet. The superstructure will be supported on a Geosynthetic Reinforced Soil (GRS) Integrated Bridge System (IBS). The recommended design and construction process of GRS-IBS abutments are presented in Federal Highway Administration (FHWA) publications *Geosynthetic Reinforced Soil Integrated Bridge System Interim Implementation Guide* Publication No. FHWA-HRT-11-026 and *Geosynthetic Reinforced Soil Integrated Bridge System Synthesis Report* Publication No. FHWA-HRT-11-027 both dated January 2011. The following design and construction sections are discussed in detail in the attached report:

Geosynthetic Reinforced Soil (GRS) Integrated Bridge System (IBS) - The proposed abutments and wingwalls will be a U-shaped structure with an abutment width of approximately 41 feet and wingwall length of approximately 11.5 feet. The proposed structure will be placed directly on bedrock, with maximum wall height of less than 10 feet. A GRS-IBS structure consists of three main components: the reinforced soil foundation (RSF), the GRS abutment and the integrated approach. A GRS abutment is a type of gravity structure, and external stability should be evaluated for direct sliding, bearing capacity and global stability failure modes. Based on discussions with FHWA, the RSF will not be required for this structure as it will be founded on bedrock.

A cast-in-place concrete leveling pad directly on the bedrock surface shall be used to provide a stable, level foundation for the GRS-IBS structure. Precast (wet-cast) modular blocks (PCMB) have been proposed as facing elements for GRS-IBS abutments and wingwalls.

Abutment reinforced backfill below the 100-year flood elevation shall consist of open-graded aggregate meeting the requirements listed in MaineDOT Standard Specification Section 518.03, Concrete Repair Materials, Designation SP-2-89. In an effort to specify one type of aggregate for all GRS zones, it is recommended that the fill material above Q100 and in the integrated approach also meet the requirements of Section 518.03, Concrete Repair Materials, Designation SP-2-89.

All GRS-IBS structures currently in-service have used a biaxial, woven polypropylene (PP) geotextile in the abutments. Any geosynthetic meeting the requirements on the Plans can be used in the abutment.

Geotextile layers shall be placed in 6 inch layers to match the height of the proposed facing PCMBs. A minimum length of the base reinforcement of 5 feet shall be used extending the full width of the base. The base to height ratio (B/H) should begin at 0.3 not including the facing block, and follow the cut slope up to a B/H ratio of 0.7. These progressively longer reinforcements improve the quality of construction and stability of the structure and provide a transition from the substructure to the superstructure.

Construction of the GRS-IBS shall meet the requirements of the Project Plans and the Special Provision for this item, to be developed during final design.

Bearing Resistance - The factored bearing resistance at the strength limit state for the GRS-IBS abutments on bedrock shall not exceed 32 to 90 ksf for base widths ranging from 5 to 14 feet. A factored bearing resistance of 20 ksf shall be used to control settlement when analyzing the service limit state.

Sour and Riprap – For scour protection and protection of the GRS-IBS structure, the bridge approach slopes and slopes at abutments should be armored with 3 feet of plain riprap. The toe of the riprap section shall be constructed 1 foot below the streambed elevation. The riprap section shall be underlain by a 1 foot thick layer of bedding material conforming to item number 703.19 of the Standard Specification and Class "1" Erosion Control Geotextile.

Seismic Design Considerations – Seismic analysis is not required for single span bridges regardless of seismic zone. However, superstructure connections and minimum support lengths should be designed in accordance with LRFD requirements.

Construction Considerations - Local information indicates that stones from the original structure (prior to the struts) might still be buried in the highway embankment. It is anticipated that excavation activities may encounter these stones.

Regulatory agencies have requested that blasting methods not be used in the construction of the replacement structure. The bedrock at the site is hard and will be difficult to remove without the use of blasting techniques. The bedrock surface shall be cleared of all loose fractured bedrock, loose decomposed bedrock and soil.

The cast-in-place concrete leveling pads shall be placed to create a level surface for the facing elements and shall take into account all existing bedrock knobs. Steps in the leveling pad are allowed provided the height of the step is equivalent to the facing element unit height.

Careful attention should be given to the installation of the first row of blocks. Since all other courses of block are built off the first row, it is essential to ensure that the bottom row is level and even for construction.

1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present geotechnical recommendations for the proposed replacement of two struts (pipe culverts) which carry Muscongus Brook under State Route 32 at the Bristol-Bremen town line. State Route 32 is a Priority 4 Highway Corridor. The project also includes 300 feet of approach roadway construction. A subsurface investigation has been completed for this site. This report presents the soils and bedrock information obtained at the site and geotechnical design recommendations.

The existing struts have diameters of 4 feet (north) and 3 feet (south) and carry Muscongus Brook under State Route 32. These structures are undersized for the current site hydraulic needs. Existing roadway lanes are 11 feet wide without built shoulders and there is no guardrail at the Muscongus Brook crossing. The existing highway alignment will be maintained in the replacement. The culverts were installed by MaineDOT Maintenance staff to replace an earlier stone structure. An existing knob of bedrock in the middle of the strut location prevented installation of the struts at a grade that would allow fish passage. The new structure will be constructed to provide passage for alewives returning to Webber Pond via Muscongus Brook. The streambed appears to be soil upstream but has been scoured to bedrock downstream.

The proposed replacement structure will be a two-lane, single span Geosynthetic Reinforced Soil (GRS) Integrated Bridge System (IBS) supported superstructure. GRS-IBS structures use alternating layers of compacted fill and closely spaced geosynthetic reinforcement to provide support for the bridge superstructure which is placed directly on the GRS abutment without a joint and without cast-in-place concrete. The proposed roadway approaches will have 11 foot lanes and 5 foot shoulders. State Route 32 is a Priority 4 Highway Corridor with projected year 2023 Annual Average Daily Traffic (AADT) of 1080 cars and 9% heavy trucks.

2.0 GEOLOGIC SETTING

The existing struts carry Muscongus Brook under State Route 32 near the Bristol-Bremen town line. Muscongus Brook is the outlet of Webber Pond which drains into Muscongus Harbor as shown on Sheet 1 - Location Map found at the end of this report.

Mapping by the Natural Resources Conservation Service (NRCS) indicates that soils in the site vicinity are Buxton Silt-Loam with fines contents on the order of 36% and bedrock at depths greater than 7.5 feet. Surrounding soils are of the Lyman-Tunbridge complex with low fines content and shallow bedrock.

According to the Surficial Geology map of the Loud Island Quadrangle, Maine published by the Maine Geologic Survey (Open-File 76-36) the surficial soils in the vicinity of the site consist of glacial-marine deposits of the Presumpscot Formation. These soils are made up of mostly silt and clay with poor drainage and low permeability. They are typically composed of sediments that washed out of the Late Wisconsinian glacier and accumulated on the ocean floor.

According to the Bedrock Geologic Map of Maine (1985) published by the Maine Geologic Survey the bedrock in the vicinity of the site consist of calcareous sandstone and interbedded sandstone and impure limestone of the Bucksport Formation.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions at the site were explored by drilling three (3) test borings and three (3) auger probes and conducting a ground penetrating radar (GPR) survey to map the bedrock surface. Test borings HB-BREM-101 and HB-BREM-101A were conducted behind the location of the proposed Abutment No. 1 (south). Test boring HB-BREM-201 was conducted behind the location of the proposed Abutment No. 2 (north). The exploration locations are shown on Sheet 2 - Boring Location Plan and Interpretive Subsurface Profile found at the end of this report. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A – Boring Logs found end of this report.

Initial borings (HB-BREM-101 and HB-BREM-101A) were drilled on April 28, 2011 by the MaineDOT drill crew. An additional boring (HB-BREM-201) was drilled on July 18, 2011 also by the MaineDOT drill crew. The borings were drilled using solid-stem auger and cased wash boring techniques. Soil samples were obtained where possible using Standard Penetration Test (SPT) test methods. The standard penetration resistances (N-values) discussed in this report are corrected for average energy transfer. All N-values discussed in this report are corrected values computed by applying an average energy transfer factor of 0.84 to the raw field N-values. This hammer efficiency factor (0.84) and both the raw field N-value and the corrected N-value (N_{60}) are shown on the boring logs. The bedrock was cored in two of the borings using an NQ-2 inch core barrel and the Rock Quality Designation (RQD) of the core was calculated.

Auger probes to the assumed bedrock surface were conducted using solid stem augers. Visual soil identifications were made from soils observed on the auger flights The borings were located in the field using a tape during the exploration program. Boring logs and rock core photographs are provided in Appendix A – Boring Logs found end of this report.

Geophysical investigations were conducted by Hager-Richter Geoscience, Inc. of Salem, NH in September 2011 using GPR traverses. The purpose of the investigation was to determine the depth and configuration of the bedrock surface in the vicinity of the proposed construction. GPR data were acquired along traverses spaced approximately 2 to 5 feet apart and oriented parallel to the travel lanes. The Geophysical Survey report is presented in Appendix B found end of this report.

4.0 LABORATORY TESTING

Laboratory testing for samples obtained in the borings consisted of three (3) standard grain analyses with natural water content. The results of these laboratory tests are provided in Appendix C - Laboratory Data found at the end of this report. Moisture content information and other soil test results are included on the Boring Logs in Appendix A and on Sheet 3 – Boring Logs found at the end of this report.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered at the borings were generally fill soils underlain a thin layer of native soils underlain by bedrock. An interpretive subsurface profile depicting the site stratigraphy is shown on Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile found at the end of this report. The following paragraphs discuss the subsurface conditions encountered at the site in detail:

5.1 Fill

Fill material was encountered beneath the pavement in all of the borings. The fill material consisted of brown, moist, fine to coarse sandy silt, little gravel and brown, moist, silt, some fine to coarse sand, little gravel. The thickness of the fill was approximately 6.0 feet in boring HB-BREM-101. One corrected SPT N-value in the fill was 8 blows per foot (bpf) indicating that the fill is loose in consistency. Natural water contents obtained from fill samples ranged from approximately 17% to 28%. Grain size analyses conducted on samples of the fill indicate that the soil is classified as an A-4 by the AASHTO Classification System and an SM or ML by the Unified Soil Classification System.

5.2 Native Soils

A thin layer of native soils was encountered beneath the fill in boring HB-BREM-101. The native soils encountered consisted of brown, moist, gravel, some fine to coarse sand, little silt. The thickness of the native soils layer was approximately 1.0 foot. One corrected SPT N-value in the native soils was 6 bpf indicating that the native soil is loose in consistency. One natural water content obtained from native soils was approximately 6%. One grain size analysis conducted on a sample of the native soils indicated that the soil is classified as an A-1-b by the AASHTO Classification System and a GM by the Unified Soil Classification System.

5.3 Bedrock

Bedrock was encountered and cored in borings HB-BREM-101A and HB-BREM-201. Table 1 summarizes the depths to bedrock and corresponding elevations of the top of bedrock:

Boring Number	Approximate Depth to Bedrock	Approximate Bedrock Elevation	RQD
HB- BREM -101A	8.1 feet	30.7 feet	60%
HB- BREM -201	8.5 feet	29.7 feet	80%

Table 1 - Summary of Bedrock Depths, Elevations and RQD

Bedrock is visible in the streambed downstream from the existing struts. An assumed bedrock surface was encountered in boring HB-BREM-101 and in the auger probes at shallow depths. The bedrock surface was mapped using GPR through the roadway. Although GPR data is not precise, it corresponds reasonably well with boring and auger refusals. The bedrock contours developed from the GPR data are shown in the Geophysical Report in Appendix B found at the end of this report. A depiction of the approximate bedrock surface at the site is also shown on Sheet 4 – Cross Sections found at the end of this report. The GPR data support the physical data at the site by indicating the presence of a bedrock “knob” in the roadway at the existing steam crossing location.

The bedrock is identified as gray, hard, coarse-grained granite with two joint sets – one steeply dipping, tight set and another open, flat set with iron staining. The RQD of the bedrock was determined to range from 60 to 80 percent indicating a rock mass quality of fair to good.

5.4 Groundwater

Groundwater was not observed in the borings or auger probes. Groundwater levels are expected to fluctuate seasonally depending upon the local precipitation magnitudes.

5.5 Existing Pavement Structure

The existing pavement structure observed at the boring and auger probe locations was found to have 5 to 6 inches of Hot Mix Asphalt and approximately two (2) feet of poor quality gravel fill.

6.0 FOUNDATION ALTERNATIVES

The new structure will be constructed to provide passage for alewives returning to Webber Pond via Muscongus Brook. Regulatory agencies have requested that blasting methods not be used in the construction of the replacement structure. The following alternatives were considered for structure replacement:

- Geosynthetic Reinforced (GRS) Integrated Bridge System (IBS) on bedrock,
- An open bottom pipe arch on concrete spread footings on bedrock, and
- A four sided box culvert on bedrock.

After consideration of all the alternatives, a GRS-IBS structure was selected due to reduced construction time and reduced costs. GRS-IBS technology is a part of the FHWA "Every Day Counts" initiative. GRS-IBS technology will reduce construction time and costs when compared to traditional construction techniques. GRS-IBS is an ideal fit for this project where minimizing construction costs is of critical importance. GRS-IBS technology is untested in the State of Maine therefore, this site as a relatively low volume Priority 4 Highway Corridor would be an ideal location for first time application and future monitoring of the technology. GRS-IBS abutments use alternating layers of compacted fill and closely spaced geosynthetic reinforcement to provide support for the superstructure which is placed directly on the GRS abutments without a joint and without cast-in-place concrete other than the leveling pad. The GRS-IBS facing elements are not a structural component of this system although it is necessary to protect the other GRS-IBS elements.

7.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

The following section discusses geotechnical design recommendation for a GRS-IBS supported superstructure which has been identified as the optimal structure for this site. The recommended steps for design and construction of GRS-IBS abutments and wingwalls for sliding, bearing capacity and internal stability presented in this report are based in FHWA Publications *Geosynthetic Reinforced Soil Integrated Bridge System Interim Implementation Guide*, Publication No. FHWA-HRT-11-026 and *Geosynthetic Reinforced Soil Integrated Bridge System Synthesis Report* Publication No. FHWA-HRT-11-027 both dated January 2011 (referred to as FHWA 2011). Construction of the GRS-IBS structure will be in accordance with the Project Plans and Special Provision 636 – GRS-IBS Abutment to be developed during final design.

7.1 Geosynthetic Reinforced (GRS) Integrated Bridge System (IBS)

The GRS-IBS structure is a viable bridge system for use at this site. A GRS-IBS supported structure consists of three main components: the reinforced soil foundation (RSF), the GRS abutments and wingwalls, and the integrated approach.

A GRS abutment is a gravity structure for which the weight of the entire reinforced soil mass is considered in the design. External stability for this structure should be evaluated for direct sliding, bearing capacity and global stability failure modes. Because a GRS mass is relatively ductile and free of tensile strength, overturning about the toe, in a strict sense, is not a possible response to earth pressures at the back of the mass or loading on its top (FHWA 2011). Further, the integrated superstructure functions as a strut to resist overturning, and each GRS mass has a reinforced integration zone above its heel, also resisting the overturning mode of failure (FHWA 2011). Internal stability should be analyzed for vertical capacity, deformations and reinforcement strength.

Table 2 presents the resistance factors to be used in design:

Failure Mechanism	Symbol	Resistance Factor
Direct Sliding – Soil on Soil	ϕ_r	1.00
Direct Sliding - Soil on Rock	ϕ_r	0.90
Bearing Capacity	ϕ_{bc}	0.65
Global Stability	ϕ	0.65
Vertical Load Carrying Capacity (nominal)	ϕ_{cap}	0.45
Reinforcement Strength (ultimate)	ϕ_{reinf}	0.40

Table 2: GRS-IBS Design Resistance Factors

The abutments and wingwalls will be U-shaped structures founded directly on the bedrock with an abutment width of approximately 41 feet and a wingwall length of approximately 11.5 feet. The abutments and wingwalls will be constructed using geosynthetic layers of varying lengths to create a horizontal reinforced soil surface over the variable bedrock surface until a level surface is reached.

The final design layout of the GRS abutment should ensure that the face of the abutment is wide enough to allow for a guardrail lay down length of approximately 4.0 feet. AASHTO LRFD Bridge Design Specifications 5th Edition, 2010 (LRFD) Article 11.10.10.2 specifies that guardrail be placed a minimum distance of 3.0 feet from the face of mechanically stabilized earth walls.

7.2 GRS-IBS Foundation

The proposed GRS-IBS structure will be founded on the existing bedrock surface. Regulatory agencies have requested that blasting methods not be used in the construction of the replacement structure. Based on discussions with FHWA, the RSF will not be required for this structure as it will be founded directly on the existing bedrock surface.

A cast-in place concrete leveling pad with variable height to create a level surface for the facing elements will be placed on the bedrock surface for each GRS abutment. A minimum thickness of 2 inches is required for the leveling pad. Steps in the leveling pad are allowed provided the height of the step is equivalent to the facing element unit height. The top elevation of each section of the leveling pads for both GRS abutments shall result in equal final top of abutment elevations. Riprap protection may be needed to protect the leveling pads from long-term abrasion and ice damage.

7.3 Retained Soils

The native retained soils behind the proposed reinforced backfill consist of moist, loose sandy silt or silt, with gravel in the lowest stratum above bedrock. Table 3 presents the soil properties to be used in design:

Property	Symbol	
Retained Soil Total Unit Weight	γ_b	120 pcf
Retained Soil Effective Unit Weight (below Q1.1)	γ'_b	58 pcf
Retained Soil Undrained Shear Strength	c_b	0 psf
Retained Soil Effective Cohesion	c'_b	0 psf
Retained Soil Effective Friction Angle	ϕ'_b	28 degrees

Table 3 – Properties for Retained Soils

GRS-IBS abutments should be designed using the Rankine active earth pressure coefficient, K_{ab} of 0.36 assuming a level back slope. See Appendix D – Calculations for supporting documentation.

7.4 Facing Elements

A commonly used facing element for GRS walls and abutments is concrete masonry unit (CMU) with nominal dimensions of 8 inches by 8 inches by 16 inches. The blocks of this type that are available in the State of Maine do not meet the freeze-thaw requirements of MaineDOT Standard Specifications. The facing unit is not a structural element of GRS walls and abutments, so any facing element which meets the freeze-thaw specification may be used.

All facing blocks for the abutments and wingwalls shall be wet-cast concrete blocks. The "Redi-Scapes®" small-block system (6 inches high, 56 pounds) or approved equal is recommended for this application. Because the geosynthetic is placed between the facing blocks in the GRS-IBS geosynthetic thickness must be considered in calculating the elevation of the top of the leveling pad. The facing units shall meet the requirements of the Project Plans and Special Provision 636 – GRS-IBS Abutment to be developed for the Bid Documents.

The upper 2.0 feet of facing block elements are susceptible to movement due to the reduced weight above them. To prevent displacement, the hollow cores of these upper block courses shall be filled with concrete fill and pinned together with No. 4 epoxy coated rebar embedded with a 2-inch minimum cover. The geosynthetic will need to be removed from the hollow cores of these facing elements to allow for concrete placement. After the top block void is filled with concrete and rebar is inserted, a thin layer of concrete shall be placed on top of the block to form the coping cap. The concrete shall be ASTM Class A concrete with 4,000 psi compressive strength. If the facing blocks are solid, No. 4 epoxy coated rebar shall be drilled and grouted into the blocks to tie the upper 2.0 feet of the facing blocks together. Adhesive may be required on corner blocks as recommended by the manufacturer.

7.5 Reinforced Backfill

The reinforced backfill is a major structural component of a GRS-IBS structure. The reinforced backfill material shall consist of hard, durable soil particles or crushed stone materials. Fill soils shall be free of deleterious materials or other soft particles that have poor durability. Open-graded soils are preferred for ease of construction and drainage characteristics. Since the lower section of the abutments will be submerged during extreme events, an open-graded aggregate should be used because it is a free-draining material. The friction angle of the backfill shall be no less than 38 degrees.

Table 4 presents the reinforced backfill properties to be used in design:

Property	Symbol	
Reinforced Backfill Total Unit Weight	γ_r	125 pcf
Reinforced Backfill Effective Unit Weight (below Q1.1)	γ'_r	63 pcf
Friction Angle of Reinforced Backfill	ϕ_r	38 degrees
Cohesion of Reinforced Backfill	c_r	0 psf
Maximum Grain Size	d_{max}	½ inch

Table 4 – Properties for Reinforced Backfill

Free-draining, open-graded backfill up to the 100 year storm stage shall be aggregate meeting the requirements listed in MaineDOT Standard Specification Section 518.03 Concrete Repair Materials, Designation SP-2-89. Backfill above that elevation can be a well-graded gravel, however, in an effort to specify one type of aggregate for all GRS zones, it is recommended that the backfill material above Q100 also meet the requirements of Section 518.03, Concrete Repair Materials, Designation SP-2-89

The reinforced backfill should be compacted to a minimum 95 percent of maximum dry density according to AASHTO T-99. Compacted lifts shall be 6 inches thick to match the size of the facing blocks and shall be compacted using a vibratory roller for the main reinforced soil mass. Soil within 1.5 feet of the facing units shall be compacted with hand operated equipment because there is no rigid connection between the geotextile and the facing blocks. The top 5.0 feet of the abutment shall be compacted to 100 percent of maximum dry density according to AASHTO T-99.

The lateral stress distribution due to the weight of the reinforced fill is calculated using the Rankine active earth pressure coefficient, K_{ar} of 0.24. This coefficient is used to calculate the required reinforcement strength. For the internal stability analysis of the GRS mass, the ultimate load carrying capacity of the GRS mass is computed using the Rankine passive earth pressure coefficient, K_{pr} of 4.2. See Appendix C - Calculations for supporting documentation.

7.5.1 Integrated Approach Backfill

The area of approach directly behind the superstructure beams is called the Integration Zone and is necessary to provide a smooth transition from the approach way to the bridge deck. FHWA 2011 recommends that the fill material used for this zone be a well-graded gravel. However, in an effort to specify one type of aggregate for all GRS components (the reinforced fill zone and the integration zone) it is recommended that the fill material in the integration zone be open-graded gravel meeting the requirements of Section 518.03, Concrete Repair Materials, Designation SP-2-89 (see Table 4).

7.5.2 Geosynthetic Requirements

All existing GRS-IBS structures have used a biaxial, woven, polypropylene (PP) geotextile in the abutment. This geotextile has been used for several reasons including cost, ease of placement and compatibility with the connection requirements of the block facing. Any biaxial geosynthetic meeting the requirements of this section can be used in the abutment and the wingwalls.

A minimum ultimate strength of the reinforcement of at least 4800 pounds/foot shall be used in load-bearing applications. Limiting the required reinforcement strength to less than the reinforcement strength at 2 percent strain will ensure long-term performance and serviceability. Permittivity and apparent opening size shall be considered, particularly in sections of the abutments that may be submerged during extreme storm events or if rapid drawdown should occur after a storm event.

The geosynthetic reinforcement will be placed at each course of the facing elements, that is, at every 6 inch lift of the backfill. Layers adjacent to the superstructure beam ends should have the faces wrapped to prevent lateral spreading.

For structures with a span length of less than 25 feet, an initial base length of 5 feet shall be used with a minimum base to height ratio (B/H) of 0.3 not including the block facing. Above the base, the reinforcing should follow the cut slope with increasing lengths up to B/H ratio of 0.7. The reinforcement length can increase in zones above that point, although not every layer needs to extend the full length into the slope. For construction cut zones flatter than 1:1, reinforcement lengths greater than 1H may not be necessary.

A bearing reinforcement zone under the bridge seat is normally required to support the increased loads due to the bridge structure; however for a structure with a span of 14 feet and geosynthetic at every 6 inch lift this additional reinforcement may not be needed. If the required strength in the bearing reinforcement zone under the bridge seat does not exceed the allowable strength at 2 percent strain the intermediate layers may not be necessary.

7.5.3 Integration Zone

The integration zone creates a smooth transition between the highway and the bridge structure. It takes the place of an approach slab, and serves to limit the development of tension cracks at the cut slope-reinforced soil interface and to blend the approach with the roadway. The number of layers in this zone depends on the height of the superstructure, but each wrapped layer should be 12 inches or less in height. The top layer of the integration zone should extend beyond the cut slope to prevent moisture infiltration.

7.6 Bearing Resistance

The factored bearing resistance at the strength limit state for the GRS-IBS abutments on bedrock vs. foundation width is shown by the dashed line in the Figure 1 below. Once the dimensions of the RSF are determined a factored bearing resistance can be determined from the figure. This factored bearing resistance must be greater than the applied factored vertical bearing pressure determined by the structural designer.

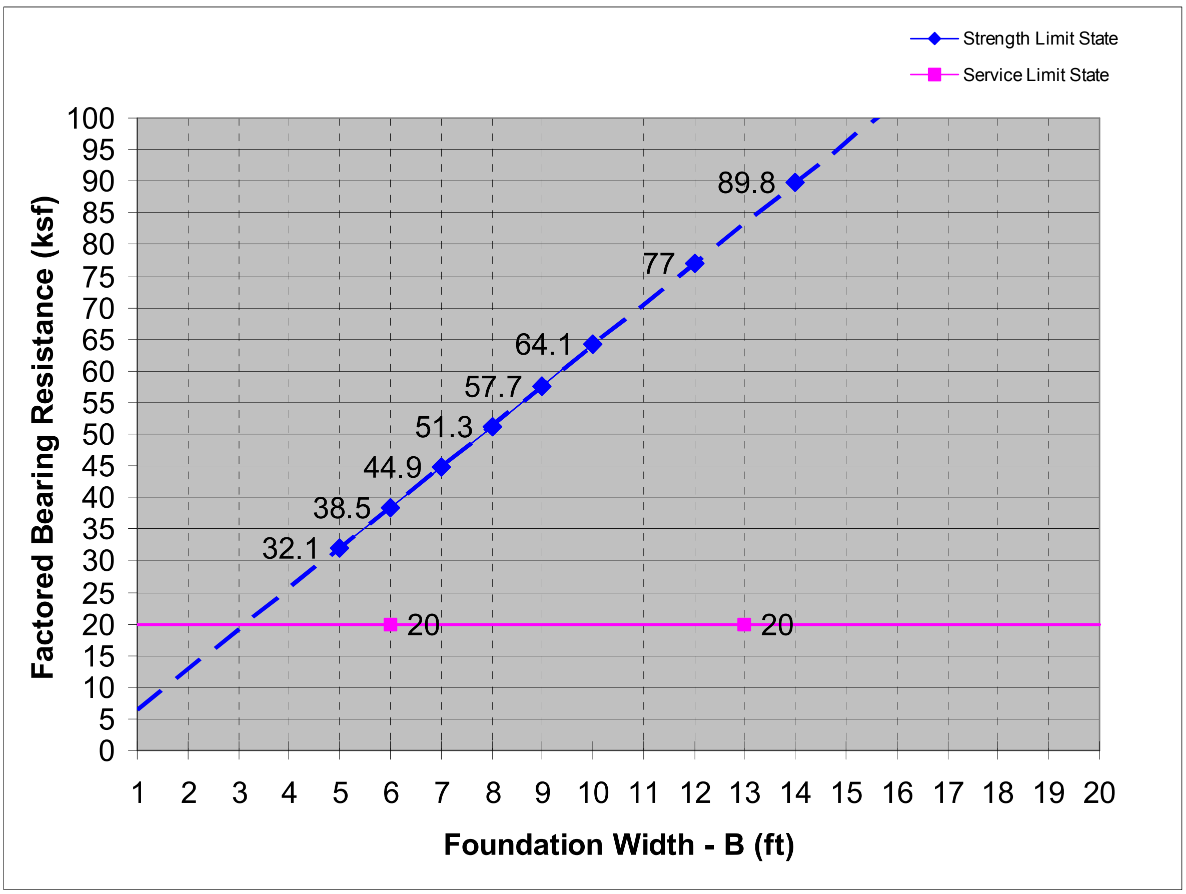


Figure 1 – Factored Bearing Resistance vs. Foundation Width

A factored bearing resistance of 20 ksf shall be used to control settlement when analyzing the service limit state as allowed in LRFD C10.6.2.6.1. See Appendix C - Calculations for supporting documentation.

7.7 Scour and Riprap

The GRS-IBS concrete leveling pad shall be formed directly on the bedrock surface. The bedrock surface shall be cleaned of all weathered, loose and potentially erodible or scourable materials. For scour protection and protection of the GRS-IBS structure, the bridge approach slopes and slopes at abutments should be armored with 3 feet of plain riprap. Refer to MaineDOT Bridge Design Guide (BDG) Section 2.3.11 for information regarding scour design.

Riprap conforming to Special Provisions 610 and 703 shall be placed at the toes of GRS-IBS abutments and wingwalls. Stone riprap shall conform to item number 703.26 of the MaineDOT Special Provision 703 and shall be placed at a maximum slope of 1.75H:1V. The toe of the riprap section shall be constructed 1 foot below the top of the leveling slab. Where possible, the riprap section shall be underlain by a 1 foot thick layer of bedding material conforming to item number 703.19 of the Standard Specification and Class "1" Erosion Control Geotextile per Standard Details 610(02) through 610(04).

7.8 Frost Protection

It is anticipated that the GRS-IBS abutments and wingwall footings will be founded directly on bedrock. For foundations on bedrock, heave due to frost is not a design issue and no requirements for minimum depth of embedment are necessary.

7.9 Seismic Design Considerations

In conformance with LRFD Article 4.7.4.2 seismic analysis is not required for single-span bridges regardless of seismic zone. According to Figure 2-2 of the MaineDOT BDG, the Route 32 Struts are not on the National Highway System (NHS). The bridge is not classified as a major structure since the construction costs will not exceed $10 million. These criteria eliminate the MaineDOT BDG requirement to design the foundations for seismic earth loads. However, superstructure connections and minimum support length requirements shall be satisfied per LRFD Articles 3.10.9 and 4.7.4.4, respectively.

7.10 Pavement Design

Projected Year 2023 traffic loading on State Route 32 is low with AADT of 1080 and 9% heavy trucks. Although Falling Weight Deflectometer (FWD) testing was not done for this project, a Resilient Modulus of 4500 psi should be used for pavement design. The minimum pavement thickness of 4 inches Hot Mix Asphalt (HMA) and 18 inches of ASC-Gravel would be adequate for these loadings, or the existing HMA and subbase thicknesses could be matched.

7.11 Construction Considerations

The existing culvert was installed with the outlet hanging several feet above the streambed, which is a possible indication of a knob of bedrock in the channel. Local information indicates that stones from the original structure (prior to the struts) might still be buried in the highway embankment. It is anticipated that excavation activities may encounter these stones.

Regulatory agencies have requested that blasting methods not be used in the construction of the replacement structure. The bedrock at the site is hard and will be difficult to remove without the use of blasting techniques. It is anticipated that the proposed structure will be constructed directly on the existing bedrock surface without any means of leveling.

The nature, slope and degree of fracturing in the bedrock bearing surfaces will not be evident until the foundation excavations are made. The bedrock surface shall be cleared of all loose fractured bedrock, loose decomposed bedrock and soil. The cleanliness and condition of the bedrock surface shall be confirmed by the Resident prior to placing concrete.

It is anticipated that there will be seepage of water from fractures and joints exposed in the bedrock surface. If areas of groundwater seepage are encountered during construction, it may become necessary to control groundwater or flatten the construction slopes used. The contractor should maintain the excavation so that all foundations are constructed in the dry.

The cast-in-place concrete leveling pads shall be placed to create a level surface for the facing elements and shall take into account all existing bedrock knobs. The top elevation of each section of the leveling pads for both GRS abutments shall result in equal final top of abutment elevations. Steps in the leveling pad are allowed provided the height of the step is equivalent to the facing element unit height. The thickness of the geosynthetic between the facing blocks will need to be considered in calculating the total height of the abutment.

Careful attention should be given to the installation of the first row of blocks. Since all other courses of block are built off the first row, it is essential to ensure that the bottom row is level and even for construction.

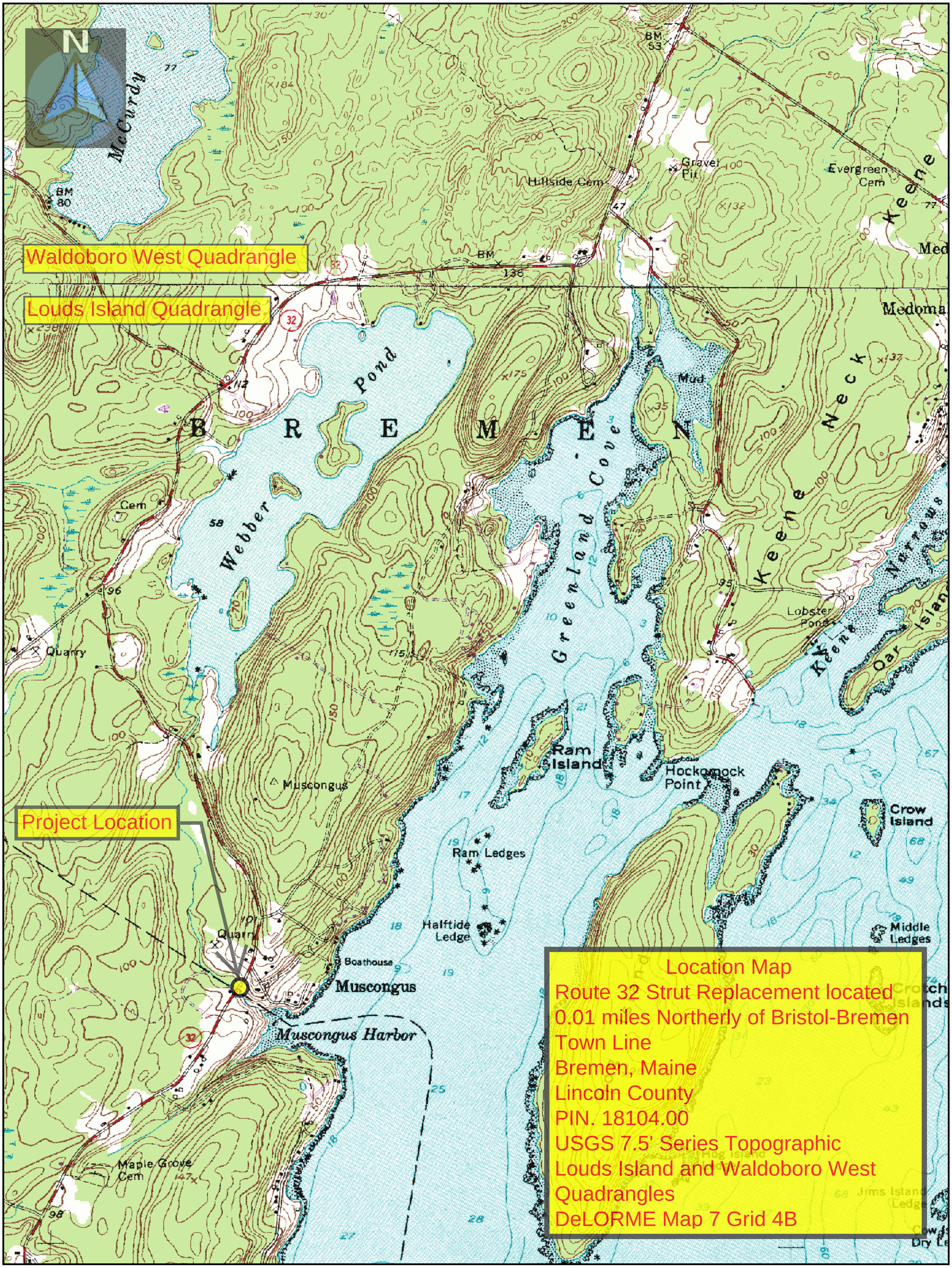
Excavated soils may be used as common borrow in accordance with MaineDOT Standard Specifications 203 and 703, but these soils may not be used as structural backfill for the GRS-IBS structure or as gravel in the new roadway construction.

8.0 CLOSURE

This report has been prepared for the use of the MaineDOT Highway Program for specific application to the proposed replacement of struts on State Route 32 at the Bristol – Bremen Town Line. No other intended use or warranty is implied. In the event that any changes in the nature, design or location of the proposed project are planned, this report should be reviewed by a geotechnical engineer to assess the appropriateness of the conclusions and recommendations and to modify the recommendation as appropriate to reflect the changes in design. Further, the analyses and recommendations are based in part upon limited soil explorations at discrete locations at the site. If variations from the conditions encountered during the investigation appear evident during construction, it may also become necessary to re-evaluate the recommendations made in this report.

We also recommend that we be provided the opportunity for a general review of the final design plans and specifications in order to verify that the earthwork and foundation recommendations have been properly interpreted and implemented in the design.

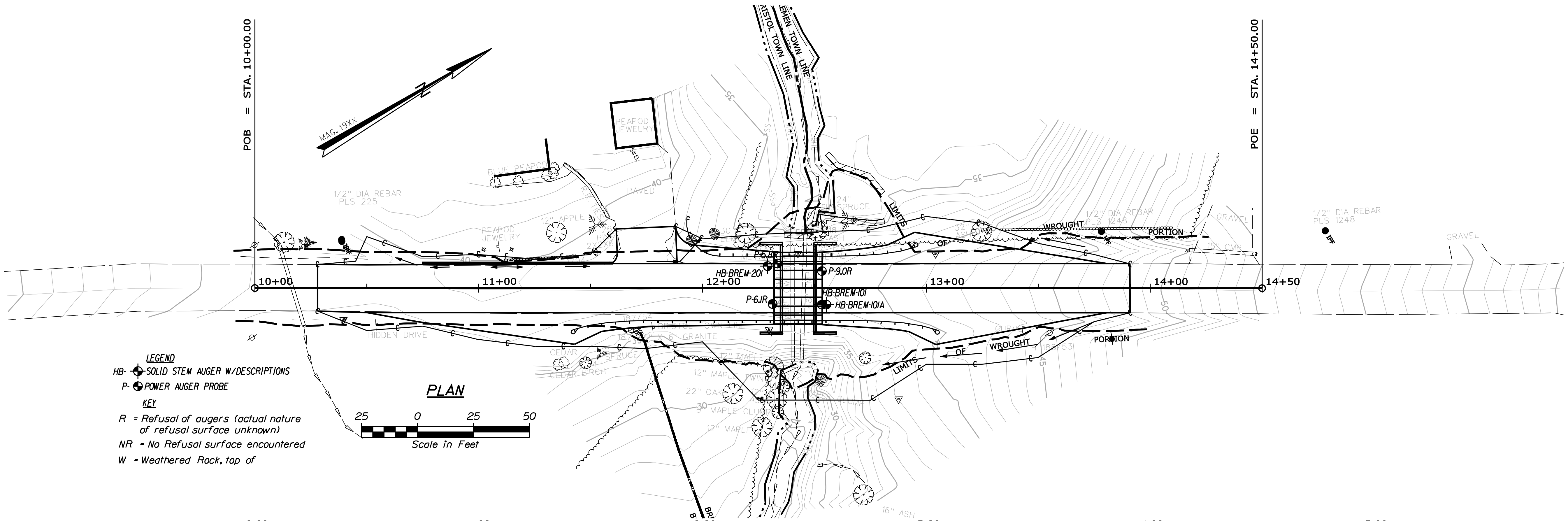
Sheets



Map Scale 1:24000

The Maine Department of Transportation provides this publication for information only. Reliance upon this information is at user risk. It is subject to revision and may be incomplete depending upon changing conditions. The Department assumes no liability if injuries or damages result from this information. This map is not intended to support emergency dispatch. Road names used on this map may not match official road names.

Filename: ...\MSTA\001_Geoplan & ISP1.dgn Division: GEOTECH Username: terry.white Date:4/4/2012



LEGEND

HB- SOLID STEM AUGER W/DESCRIPTIONS

P- POWER AUGER PROBE

KEY

R = Refusal of augers (actual nature of refusal surface unknown)

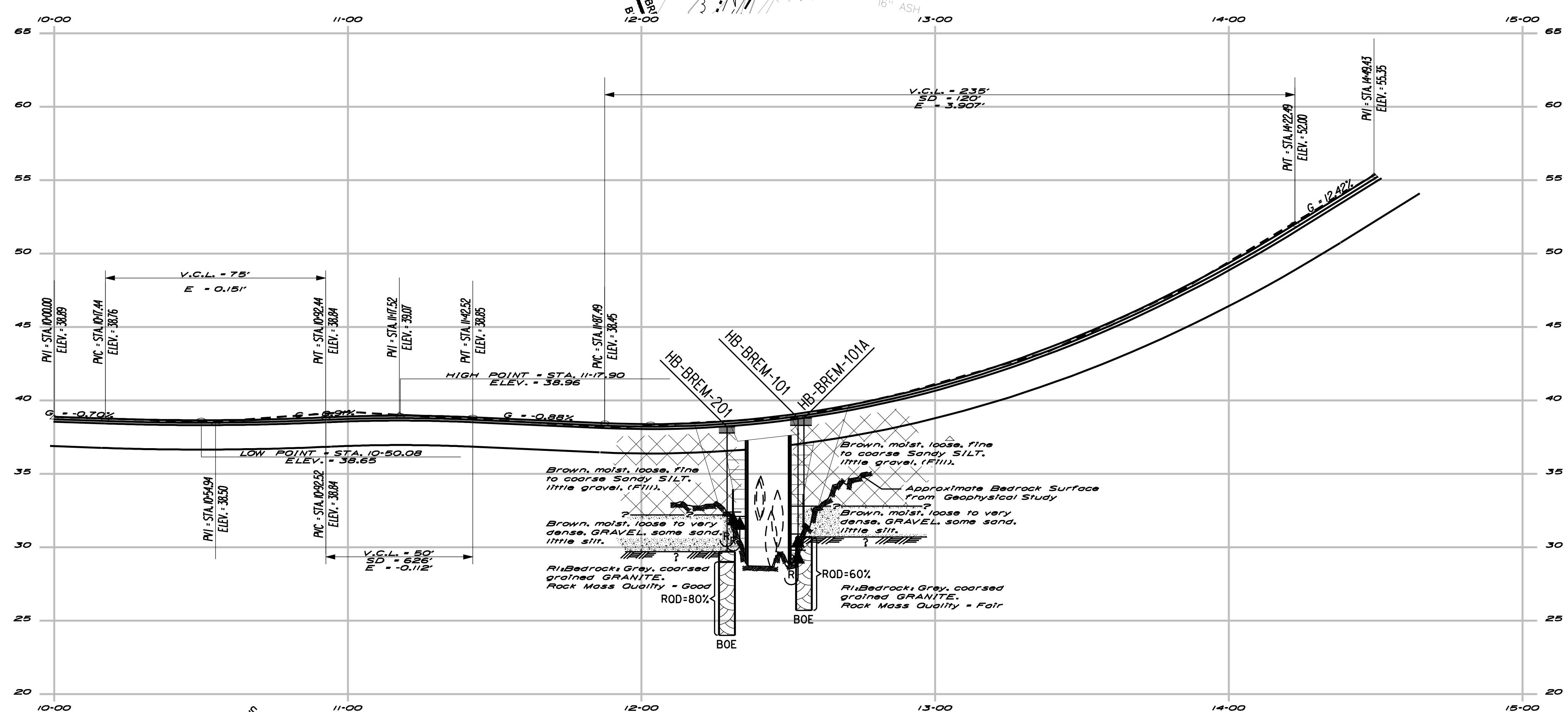
NR = No Refusal surface encountered

W = Weathered Rock, top of

PLAN

25 0 25 50

Scale in Feet



LEGEND

Boring No. Offset, if shown

Pavement Thickness if applicable

Strata Interface

Refusal

Boring

RQD= Rock Quality Designation for Rock Core Sample

BOE= Bottom Of Exploration

PROFILE

HORIZ 25 0 25 50

VERT 5 0 5 10

SCALE

Note: 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 transitions may vary and are probably more erratic. For more specific information refer to the exploration logs.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

FUNDING WITH NOAA

WIN 18104.00

HIGHWAY PLANS

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED	K.BRESKIN	T.WHITE JAN 2012
CHECKED-REVIEWED		
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

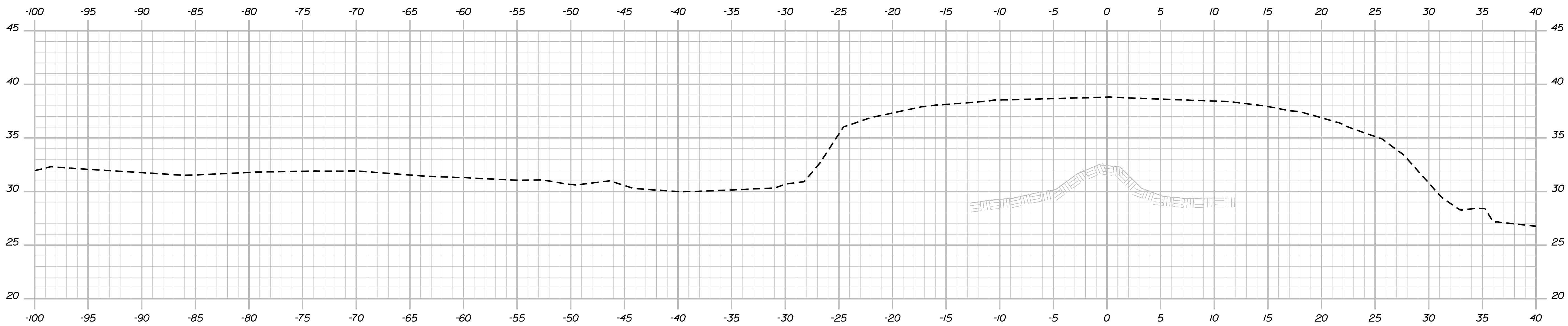
BREMEN
ROUTE 32 STRUT

GEOPLANS &
INTERPRETIVE SUBSURFACE PROFILE

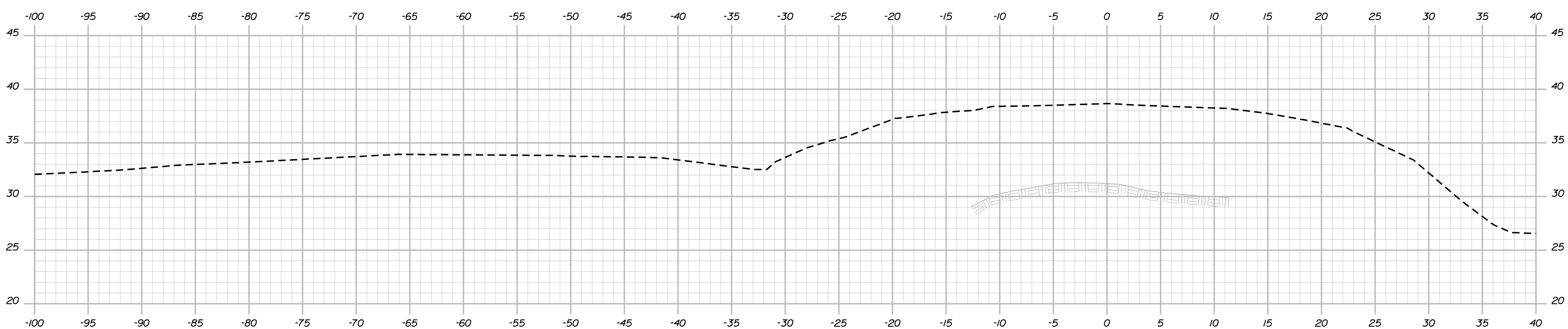
SHEET NUMBER

2

OF 4



12+48.00



12+36.00

LEGEND

Approximate Bedrock Surface from Geophysical Study by Hager-Richter Geoscience, Inc (2012)

CROSS SECTION

HORIZ 5 0 5 10

VERT 5 0 5 10

SCALE

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

FUNDING WITH NOAA

HIGHWAY PLANS

WIN
18104.00

	BY	DATE
PROJ. MANAGER	K.BRESKIN	
DESIGN-DETAILED	T.WHITE	JAN 2012
CHECKED-REVIEWED		
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

BREMEN
ROUTE 32 STRUT

CROSS SECTIONS

SHEET NUMBER

3

OF 4

Filename: ...\002_Xsect_12+38_12+50 1.dgn Division: GEOTECH Username: terry.white Date:4/4/2012

Filename: ...\msta\003_BORING LOGS1.dgn   Division: GEOTECH   Username: terry.white   Date:4/4/2012

Maine Department of Transportation
Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Strut Replacement Route 32
Location: Bremen, Maine

Boring No.: HB-BREM-201
WIN: 18104.00

Driller:	MaineDOT	Elevation (ft.)	38.2	Auger ID/OD:	5" Dia.
Operator:	Giguere/Giles	Datum:	NAVD88	Sampler:	N/A
Logged By:	K. Breskin	Rig Type:	CME 45C	Hammer Wt./Fall:	N/A
Date Start/Finish:	7/18/11-7/18/11	Drilling Method:	Solid Stem Auger	Core Barrel:	NQ-2"
Boring Location:	12+29.1, 9.9 ft Lt.	Casing ID/OD:	HW	Water Level*:	None Observed
Hammer Efficiency Factor: 0.84		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 = Insitu Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Insitu 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 = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane 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	37.80		5" Pavement 0.40	
										Similar to HB-BREM-101. (Fill)	
5											
								29.70		8.50 Top of Bedrock at Elev. 29.7 ft. Roller Coned ahead to 9.2 ft bgs.	
	R1	60/57	9.20 - 14.20	RQD = 80%			NQ-2			R1:Bedrock: Grey, coarse-grained, GRANITE, with two joint sets: steeply dipping - tight, and flat - open, with iron staining. Rock Mass Quality = Good. R1:Core Times (min:sec) 9.2-10.2 ft (5:50) 10.2-11.2 ft (3:30) 11.2-12.2 ft (3:10) 12.2-13.2 ft (4:50) 13.2-14.2 ft (5:00) 95% Recovery	
10											
								24.00		14.20 Bottom of Exploration at 14.20 feet below ground surface.	
15											
20											
25											

Remarks:

9.9 ft Left of CL heading North, 50.0 ft North of TL sign, and west of culvert.

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.

Page 1 of 1

Boring No.: HB-BREM-201

Maine Department of Transportation
Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Strut Replacement Route 32
Location: Bremen, Maine

Boring No.: HB-BREM-101
WIN: 18104.00

Driller:	MaineDOT	Elevation (ft.)	38.8	Auger ID/OD:	5" Dia.
Operator:	Giguere/Giles	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	4/28/11; 12:00-12:30	Drilling Method:	Solid Stem Auger	Core Barrel:	N/A
Boring Location:	12+53.3, 7.3 Rt.	Casing ID/OD:	N/A	Water Level*:	None Observed
Hammer Efficiency Factor: 0.84		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
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V = Insitu Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Insitu 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 = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane 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	38.30		6" Pavement 0.50	
	1D/A	24/18	1.00 - 3.00	6/3/3/4	6	8				1D (1.0-2.0 ft) Brown, moist, loose, fine to coarse Sandy SILT, little gravel, (Fill). 1D/A (2.0-3.0 ft) Brown, moist, loose, SILT, some fine to coarse sand, little gravel, (Fill).	G#245128 A-4, SM WC=16.9% G#245129 A-4, ML WC=27.7%
5	2D	24/14	5.00 - 7.00	4/2/2/14	4	6					G#245130 A-1-b, GM WC=6.4%
								32.30		Brown, moist, loose, GRAVEL, some fine to coarse sand, little silt. 6.50	
										Changed to very dense at 6.5 ft bgs.	
								30.90		7.90 Bottom of Exploration at 7.90 feet below ground surface. REFUSAL	
10											
15											
20											
25											

Remarks:

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.

Page 1 of 1

Boring No.: HB-BREM-101

Maine Department of Transportation
Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Strut Replacement Route 32
Location: Bremen, Maine

Boring No.: HB-BREM-101A
WIN: 18104.00

Driller:	MaineDOT	Elevation (ft.)	38.8	Auger ID/OD:	5" Dia.
Operator:	Giguere/Giles	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	4/28/11; 12:30-14:00	Drilling Method:	SSA/Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	12+55.3, 7.3 ft Rt.	Casing ID/OD:	HW	Water Level*:	None Observed
Hammer Efficiency Factor: 0.84		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 = Insitu Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Insitu 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 = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane 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	38.30		6" Pavement 0.50	
										See HB-BREM-101 for soil descriptions. (Fill)	
5							36				
							69				
							a100			a100 blows for 0.8 ft.	
								30.70		Roller Coned through Cobble from 7.8-8.1 ft bgs. 8.10	
	R1	60/60	8.10 - 13.10	RQD = 60%			NQ-2			Top of Bedrock at Elev. 30.7 ft. R1:Bedrock: Grey, coarse-grained, GRANITE, with two joint sets: steeply dipping, tight and flat, open, with iron staining. Rock Mass Quality = Fair. R1:Core Times (min:sec) 8.1-9.1 ft (2:30) 9.1-10.1 ft (3:45) 10.1-11.1 ft (4:00) 11.1-12.1 ft (3:50) 12.1-13.1 ft (4:00) 100% Recovery	
10											
								25.70		13.10 Bottom of Exploration at 13.10 feet below ground surface.	
15											
20											
25											

Remarks:

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.

Page 1 of 1

Boring No.: HB-BREM-101A

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
FUNDING WITH NOAA

WIN 18104.00
HIGHWAY PLANS

		BY	DATE
PROJ. MANAGER	------		
DESIGN-DETAILED	K.BRESKIN	T.WHITE	JAN 2012
CHECKED-REVIEWED	------	------	------
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1	------	------	------
REVISIONS 2	------	------	------
REVISIONS 3	------	------	------
REVISIONS 4	------	------	------
FIELD CHANGES	------	------	------

SIGNATURE

P.E. NUMBER

DATE

BREMEN
ROUTE 32 STRUT

BORING LOGS

SHEET NUMBER
4
OF 4

Appendix A

Boring Logs and
Rock Core Photographs

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Strut Replacement Route 32
Location: Bremen, Maine

Boring No.: HB-BREM-101
PIN: 18104.00

Driller:	MaineDOT	**Elevation (ft.)**	38.8	**Auger ID/OD:**	5" Dia.
Operator:	Giguere/Giles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/28/11; 12:00-12:30	**Drilling Method:**	Solid Stem Auger	**Core Barrel:**	N/A
Boring Location:	12+53.3, 7.3 Rt.	**Casing ID/OD:**	N/A	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.84	**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 = Insitu Vane Shear Test
MV = Unsuccessful Insitu 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 = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane 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	38.30		6" Pavement —0.50	
	1D/A	24/18	1.00 - 3.00	6/3/3/4	6	8				1D (1.0-2.0 ft) Brown, moist, loose, fine to coarse Sandy SILT, little gravel, (Fill). 1D/A (2.0-3.0 ft) Brown, moist, loose, SILT, some fine to coarse sand, little gravel, (Fill).	G#245128 A-4, SM WC=16.9% G#245129 A-4, ML WC=27.7%
5	2D	24/14	5.00 - 7.00	4/2/2/14	4	6					G#245130 A-1-b, GM WC=6.4%
								32.30		Brown, moist, loose, GRAVEL, some fine to coarse sand, little silt. —6.50 Changed to very dense at 6.5 ft bgs.	
								30.90		—7.90 **Bottom of Exploration at 7.90 feet below ground surface.** REFUSAL	
10											
15											
20											
25											

Remarks:

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: Strut Replacement Route 32
Location: Bremen, Maine

Boring No.: HB-BREM-101A
PIN: 18104.00

Driller:	MaineDOT	**Elevation (ft.)**	38.8	**Auger ID/OD:**	5" Dia.
Operator:	Giguere/Giles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/28/11; 12:30-14:00	**Drilling Method:**	SSA/Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	12+55.3, 7.3 ft Rt.	**Casing ID/OD:**	HW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
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SSA = Solid Stem Auger
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WOH = weight of 140lb. hammer
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S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
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$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
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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	38.30		6" Pavement 0.50	
										See HB-BREM-101 for soil descriptions. (Fill)	
5							36				
							69				
							a100			a100 blows for 0.8 ft.	
								30.70		Roller Coned through Cobble from 7.8-8.1 ft bgs. 8.10	
	R1	60/60	8.10 - 13.10	RQD = 60%			NQ-2			Top of Bedrock at Elev. 30.7 ft. R1:Bedrock: Grey, coarse-grained, GRANITE, with two joint sets: steeply dipping, tight and flat, open, with iron staining. Rock Mass Quality = Fair. R1:Core Times (min:sec) 8.1-9.1 ft (2:30) 9.1-10.1 ft (3:45) 10.1-11.1 ft (4:00) 11.1-12.1 ft (3:50) 12.1-13.1 ft (4:00) 100% Recovery 13.10	
10											
								25.70		**Bottom of Exploration at 13.10 feet below ground surface.**	
15											
20											
25											

Remarks:

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

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

Boring No.: HB-BREM-101A

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Strut Replacement Route 32	**Location:**	Bremen, Maine	**Boring No.:**	HB-BREM-201
				PIN:	18104.00
Driller:	MaineDOT	**Elevation (ft.)**	38.2	**Auger ID/OD:**	5" Dia.
Operator:	Giguere/Giles	**Datum:**	NAVD88	**Sampler:**	N/A
Logged By:	K. Breskin	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	N/A
Date Start/Finish:	7/18/11-7/18/11	**Drilling Method:**	Solid Stem Auger	**Core Barrel:**	NQ-2"
Boring Location:	12+29.1, 9.9 ft Lt.	**Casing ID/OD:**	HW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒	Hydraulic ☐	Rope & Cathead ☐

Definitions:
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MD = Unsuccessful Split Spoon Sample attempt
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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	37.80		5" Pavement — 0.40	
										Similar to HB-BREM-101. (Fill)	
5											
								29.70		— 8.50	
										Top of Bedrock at Elev. 29.7 ft. Roller Coned ahead to 9.2 ft bgs.	
	R1	60/57	9.20 - 14.20	RQD = 80%			NQ-2			R1:Bedrock: Grey, coarse-grained, GRANITE, with two joint sets: steeply dipping - tight, and flat - open, with iron staining. Rock Mass Quality = Good. R1:Core Times (min:sec) 9.2-10.2 ft (5:50) 10.2-11.2 ft (3:30) 11.2-12.2 ft (3;10) 12.2-13.2 ft (4;50) 13.2-14.2 ft (5:00) 95% Recovery	
10											
								24.00		— 14.20	
15										**Bottom of Exploration at 14.20 feet below ground surface.**	
20											
25											

Remarks:

9.9 ft Left of CL heading North, 50.0 ft North of TL sign, and west of culvert.

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.

State of Maine - Department of Transportation

Power Auger Probe Summary Sheet

Town(s): Bremen Project Number: 18104.00

Station (Feet)	Offset (Feet)	Weathered Rock (Feet)	Refusal (Feet)	No Refusal (Feet)	Water Depth (Ft.)	Comments / Date 4/28/2011
12+31.3	7.0 Rt.		5.8			5.0" Pavement.
12+33.5	11.0 Lt.		6.1			Shoulder EP
12+53.3	7.7 Lt.		9.0			5.0" Pavement

MaineDOT Drill Crew
Logged By: B. Wilder
Drill Rig: CME 45C

5" Solid Stem Auger


JUL 18 2011
3) REC= 58"
RQD= 33%
BB-202 R1
12-21-00
BB-201 R1
23.1'- 27.6'
4) REC=53"
RQD= 44%
12-22-00
TOP


JUL 18 2011


JUL 18 2011


JUL 18 2011

Appendix B

Geophysical Survey Report

HAGER-RICHTER
GEOSCIENCE, INC.

GEOPHYSICAL SURVEY
ROUTE 32
BRISTOL-BREMEN, MAINE

PIN 18104.00

MAINE DOT CONTRACT
No. 20110613000000006486

Prepared for:

Maine Department of Transportation
Highway Program
16 State House Station
Augusta, Maine 04333-0016

Prepared by:

Hager-Richter Geoscience, Inc.
8 Industrial Way - D10
Salem, New Hampshire 03079

File 11J66
September, 2011

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HAGER-RICHTER
GEOSCIENCE, INC.

CONSULTANTS IN GEOLOGY AND GEOPHYSICS
8 INDUSTRIAL WAY - D10
SALEM, NEW HAMPSHIRE 03079-5820
TELEPHONE (603) 893-9944
FAX (603) 893-8313

September 30, 2011
File 11J66

Kitty Breskin, P.E.
Geotechnical Design Engineer
Maine Department of Transportation
Highway Program
16 State House Station
Augusta, Maine 04333-0016

Phn: 207-592-7605
Email Kitty.Breskin@maine.gov

RE: **MaineDOT Contract**
No. 20110613000000006486
Ground Penetrating Radar Survey
Route 32
Bristol-Bremen, Maine
PIN 18104.00

Dear Ms. Breskin:

In this letter, we report the results of a ground penetrating radar (GPR) survey conducted by Hager-Richter Geoscience, Inc. (H-R) along a portion of State Route 32, Waldoboro Road, near the Bristol-Bremen town line in Maine for the Maine Department of Transportation (MaineDOT) in September, 2011. The geophysical survey was performed in support of a geotechnical investigation by MaineDOT for the replacement of two corrugated steel culverts.

INTRODUCTION

The Site is a portion of State Route 32 south of Coates Road near at the Bristol-Bremen town line. The general location of the Site is shown in Figure 1. According to information provided by MaineDOT, 36-inch and 45-inch diameter corrugated steel culverts carry an unnamed stream under Route 32 at this location, and the steel culverts are to be replaced by a single box culvert. MaineDOT required information on the depth of bedrock in the vicinity of the existing culverts. MaineDOT installed two borings (HB-101 and HB-101A) and three power auger probes (PA-1, PA-2, and PA-3) in the roadway near the culverts. Refusal on presumed bedrock was encountered at depths of approximately 5.8 feet to 9 feet below ground surface. Logs for the borings and probes are included in Appendix 1. Bedrock is described as unweathered granite and is visible in the stream bed on the east (downstream) side of the roadway.

OBJECTIVE

The objective of the geophysical survey was to determine the depth and configuration of the bedrock surface in the vicinity of the proposed construction.

THE SURVEY

The geophysical survey was conducted using the ground penetrating radar (GPR) method. Hager-Richter personnel were on-site on September 7, 2011. Eric Rickert and Michael Howley conducted the survey. The fieldwork was coordinated with Ms. Kitty Breskin, P.E., of the MaineDOT. A representative of MaineDOT was on site for a portion of the field work and coordinated traffic control services. Data analysis and interpretation were completed at the Hager-Richter offices.

GPR data were acquired along traverses spaced 2-5 feet apart and oriented parallel to the travel lanes. The area of interest extended approximately 30 feet north and south of the culverts along the paved roadway. MaineDOT provided site plans showing site features, surface topography, and boring and probe locations. Figure 2 is a Site Plan showing the locations of the GPR traverses and other site features.

EQUIPMENT

The GPR survey was conducted using a Sensors and Software Noggin SmartCart Plus digital GPR system equipped with a survey wheel to trigger recording of data at equal horizontal distances. The GPR system was used with a 250 MHz antenna and a 80 nsec[1] time window. The GPR data were processed using PulseEkko software licensed by Sensors and Software.

LIMITATIONS OF THE METHOD

HAGER-RICHTER GEOSCIENCE, INC. MAKES NO GUARANTEE THAT THE DEPTH OF BEDROCK WAS ACCURATELY DETERMINED IN THIS SURVEY. HAGER-RICHTER GEOSCIENCE, INC. IS NOT RESPONSIBLE FOR DETERMINING THE DEPTH OF BEDROCK WHERE THE INTERFACE CANNOT BE DETECTED BECAUSE OF SITE CONDITIONS. THE BEDROCK DEPTHS DETERMINED SHOULD NOT BE USED FOR CONTRACT BEDROCK REMOVAL QUANTITIES.

[1] ns, abbreviation for nanosecond, 1/1,000,000,000 second. Light and the GPR signal require about 1 ns to travel 1 ft in air. The GPR signal requires about 3.5 ns to travel 1 ft in unsaturated sandy soil.

There are limitations of the GPR technique: (1) surface conditions, (2) electrical conductivity and thickness of the subsurface layers, (3) electrical properties of the target(s), and (4) spacing of the traverses. Of these restrictions, only the last is controllable by us in most cases.

The condition of the survey surface can affect the quality of the GPR data and the depth of penetration of the GPR signal. For exterior sites, a surface covered with obstacles such as automobiles, dumpsters, thick leaf debris, materials piles, etc. limit the survey access. Similarly, for interior sites, a surface covered with obstacles such as desks, benches, laboratory equipment, etc. also limit access. Some floor coverings may limit the coupling of the GPR antenna with the subsurface.

The electrical conductivity of the subsurface determines the attenuation of the GPR signals, and thereby limits the maximum depth of exploration. The GPR signal does not penetrate clay-rich soils or soils contaminated with road salt. In some cases, the GPR signal may not penetrate below concrete pavement, and some asphalts are electrically conducting.

A strong contrast in the electrical conductivities of the ground and the target (for examples, UST, pipe, void, dry well, drum, contaminant plume) is required to obtain a reflection of the GPR signal. If the contrast is too small, then the reflection may be too weak to recognize, and the target can be missed.

Spacing of the traverses is limited by access at many sites, but where flexibility of traverse spacing is possible, the spacing is adjusted on the basis of the size of the target.

RESULTS

The geophysical survey consisted of a ground penetrating radar (GPR) survey along eight traverses oriented parallel to the travel lanes. The locations of the GPR traverses are shown in Figure 2.

Apparent GPR signal penetration was generally good across the area of interest, with two-way traveltime reflections received for 50 to 80ns. Based on site specific time-to-depth conversions for the GPR signal at the Site, the GPR signal penetration is estimated to have been approximately 8 to 12 feet.

GPR reflections consistent with those expected for the top of bedrock were recorded for significant portions of the GPR traverses. The depth of bedrock determined by the GPR method matches reasonably well with the depths of bedrock based on logs for the borings and probes provided by MaineDOT. Figure 3 is a contour plot of the approximate depth of the bedrock surface based on the GPR, boring, and probe data.

A measure of the accuracy of the depths of bedrock can be obtained by comparing the depths determined by the GPR method with depths reported in logs for borings and probes that intersect bedrock. Based on the comparison, and on the results from other similar surveys, we estimate the accuracy (standard deviation) of the depths of competent bedrock determined by the GPR survey in most locations to be about ±15% of the depth of bedrock, or ± 1 foot), whichever is greater.

Figure 4 shows examples of two GPR records for the Route 32 site. GPR reflections interpreted as the bedrock surface are shown as red dashed lines. GPR reflections from the two culverts are shown in purple, and the interpreted position of the culvert is also shown in purple. Where GPR reflections from the bedrock surface were not detected, the bedrock surface is either deeper than the effective penetration of the GPR signal, or the contrast between the bedrock surface and the overlying soils was not sufficient to generate a detectable reflection.

The bedrock surface gently undulates, is deeper at the location of the culverts, and is shallower on the north and south approaches. The depth of bedrock determined by the GPR survey varies between about 3 feet and 10 feet below ground surface.

CONCLUSIONS

Based on the results of the geophysical survey conducted by Hager-Richter Geoscience, Inc. along a portion of Route 32 at a culvert crossing located near the Bristol/Bremen town line in Maine in September, 2011, we conclude that the depth of bedrock below the surveyed portion of the roadway varies between about 3 and 10 feet and is deepest at the location of the culverts.

LIMITATIONS

This letter report was prepared for the exclusive use of Maine Department of Transportation (Client). No other party shall be entitled to rely on this Report or any information, documents, records, data, interpretations, advice or opinions given to Client by Hager-Richter Geoscience, Inc. (H-R) in the performance of its work. The Report relates solely to the specific project for which H-R has been retained and shall not be used or relied upon by Client or any third party for any variation or extension of this project, any other project or any other purpose without the express written permission of H-R. Any unpermitted use by Client or any third party shall be at Client's or such third party's own risk and without any liability to H-R.

H-R has used reasonable care, skill, competence and judgment in the performance of' its services for this project consistent with professional standards for those providing similar services at the same time, in the same locale, and under like circumstances. Unless otherwise stated, the work performed by H-R should be understood to be exploratory and interpretational in character and any results, findings or recommendations contained in this Report or resulting from the work proposed may include decisions which are judgmental in nature and not necessarily based solely on pure science or engineering. It should be noted that our conclusions might be modified if subsurface conditions were better delineated with additional subsurface exploration including, but not limited to, test pits, soil borings with collection of soil and water samples, and laboratory testing.

Except as expressly provided in this limitations section, H-R makes no other representation or warranty of any kind whatsoever, oral or written, expressed or implied; and all implied warranties of merchantability and fitness for a particular purpose, are hereby disclaimed.

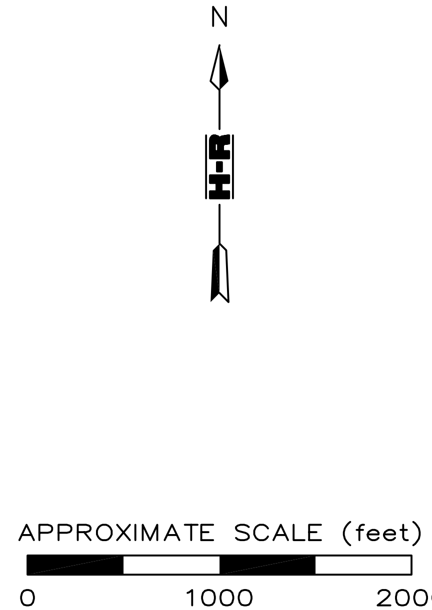
If you have any questions or comments on this letter report, please contact us at your convenience. It has been a pleasure to work with you on this project. We look forward to working with you again in the future.

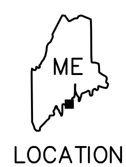
Sincerely yours,
HAGER-RICHTER GEOSCIENCE, INC.

Jeffrey Reid, P.G.
Senior Geophysicist

Dorothy Richter, P.G.
President

Attachments: Figures 1-4





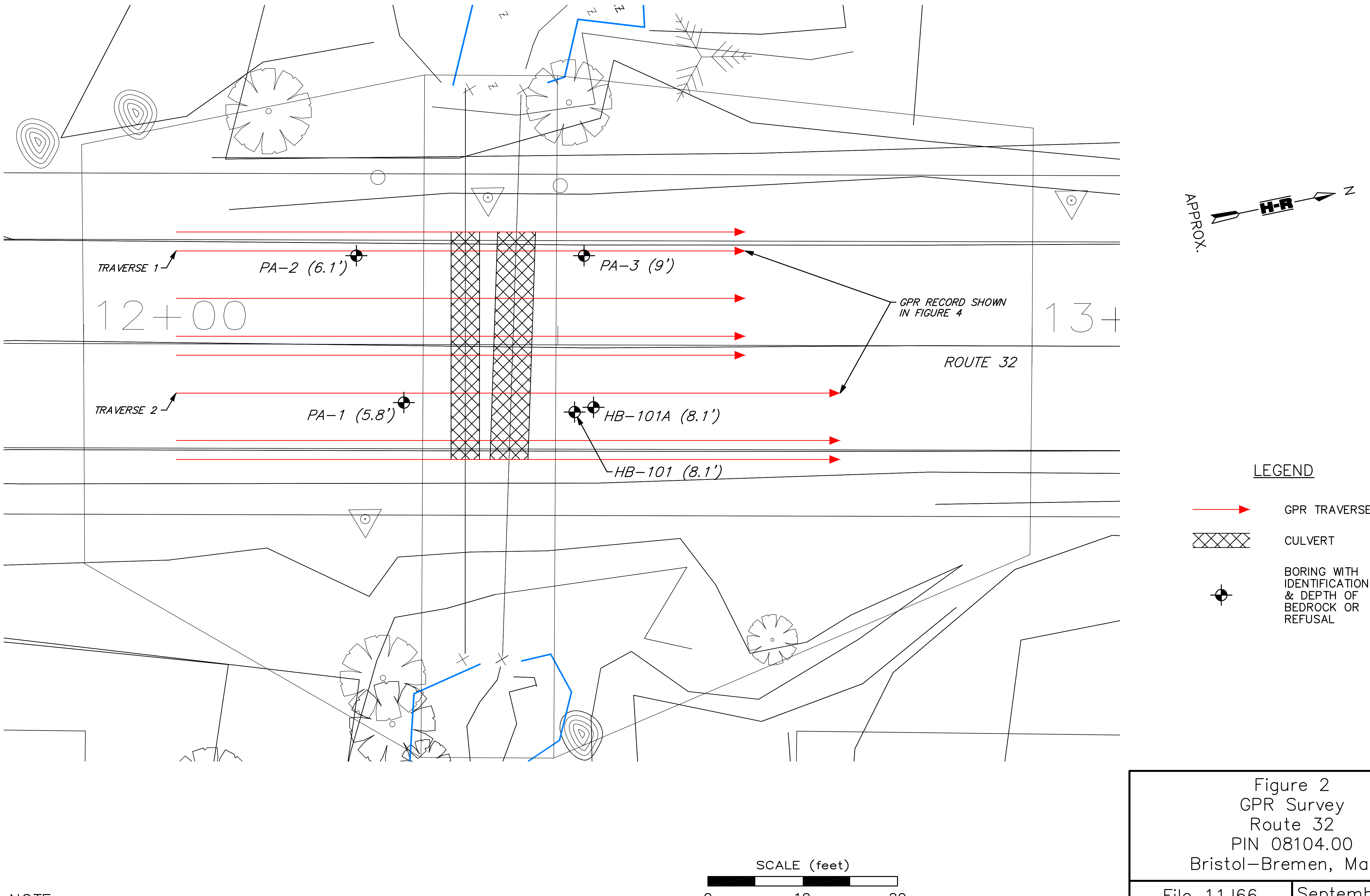
NOTE:

Modified from Google Earth aerial photograph.

Figure 1
General Site Location
Route 32
PIN 08104.00
Bristol–Bremen, Maine

File 11J66 | September, 2011

HAGER–RICHTER GEOSCIENCE, INC.
Salem, New Hampshire



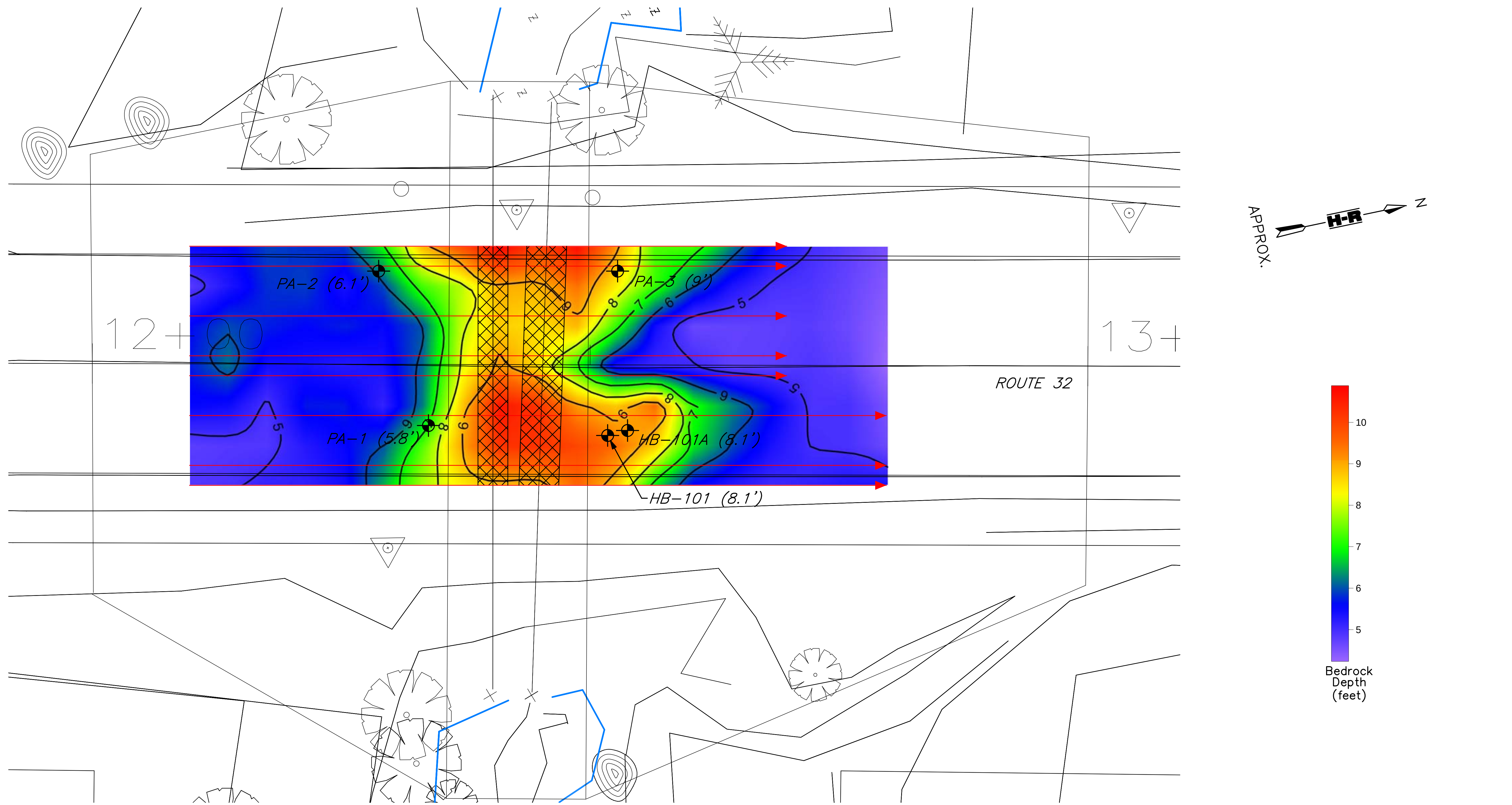
NOTE:

Modified from site plan provided by Maine Department of Transportation.

Figure 2
GPR Survey
Route 32
PIN 08104.00
Bristol–Bremen, Maine

File 11J66	September, 2011

HAGER–RICHTER GEOSCIENCE, INC.
Salem, New Hampshire



NOTES:

1. Modified from site plan provided by Maine Department of Transportation.
2. Contour interval = 0.5 feet.
3. The bedrock depths shown on this plot are a non-unique model. The contours represent interpolations based on the available data, including GPR data and boring information. The depths of the bedrock surface at any particular location may differ from that shown. Bedrock depths based on additional data may differ significantly.

SCALE (feet)

0 10 20

LEGEND

GPR TRAVERSE

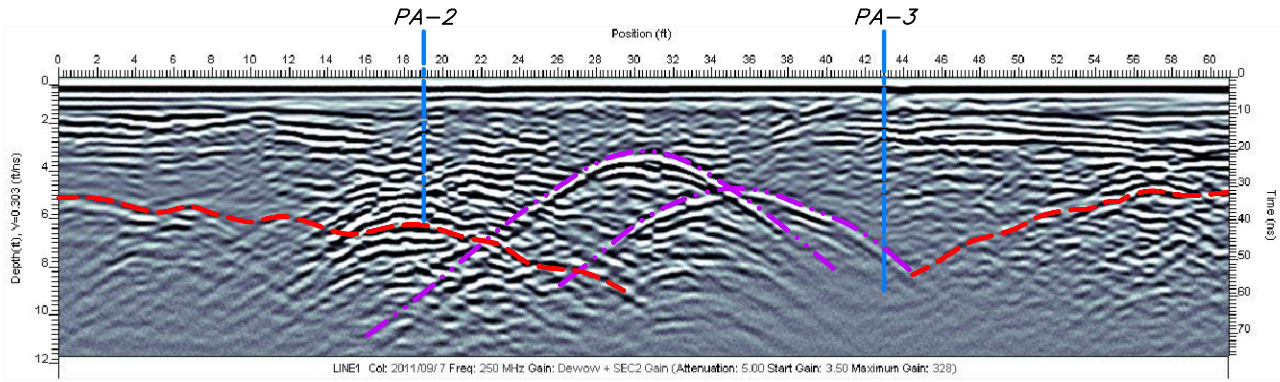
CULVERT

BORING WITH IDENTIFICATION & DEPTH OF BEDROCK OR REFUSAL

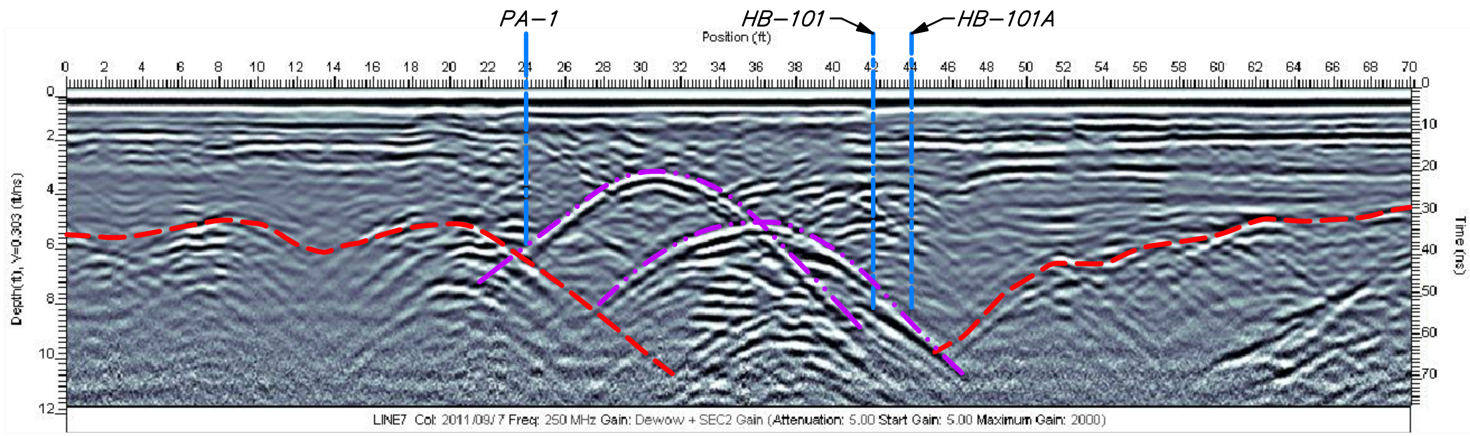
Figure 3
Bedrock Depth
Route 32
PIN 08104.00
Bristol–Bremen, Maine

File 11J66 | September, 2011

HAGER–RICHTER GEOSCIENCE, INC.
Salem, New Hampshire



TRAVERSE 1



TRAVERSE 2

SCALE (feet)

0 10 20

LEGEND

BEDROCK

BORING

GPR REFLECTION OF CULVERT

NOTE:

Location of GPR traverse shown on Figure 2.

Figure 4
Example GPR Traverses
Route 32
PIN 08104.00
Bristol–Bremen, Maine

File 11J66 | September, 2011

HAGER–RICHTER GEOSCIENCE, INC.
Salem, New Hampshire

Appendix C

Laboratory Data

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Bremen Project Number: 18104.00

Boring & Sample Identification Number	Station (Feet)	Offset (Feet)	Depth (Feet)	Reference Number	G.S.D.C. Sheet	W.C. %	L.L.	P.I.	Classification		
									Unified	AASHTO	Frost
HB-BREM-101, 1D	12+53.3	7.3 Rt.	1.0-2.0	245128	1	16.9			SM	A-4	III
HB-BREM-101, 1D/A	12+53.3	7.3 Rt.	2.0-3.0	245129	1	27.7			ML	A-4	IV
HB-BREM-101, 2D	12+53.3	7.3 Rt.	5.0-7.0	245130	1	6.4			GM	A-1-b	I

Classification of these soil samples is in accordance with AASHTO Classification System M-145-40. This classification is followed by the "Frost Susceptibility Rating" from zero (non-frost susceptible) to Class IV (highly frost susceptible). The "Frost Susceptibility Rating" is based upon the MaineDOT and Corps of Engineers Classification Systems.

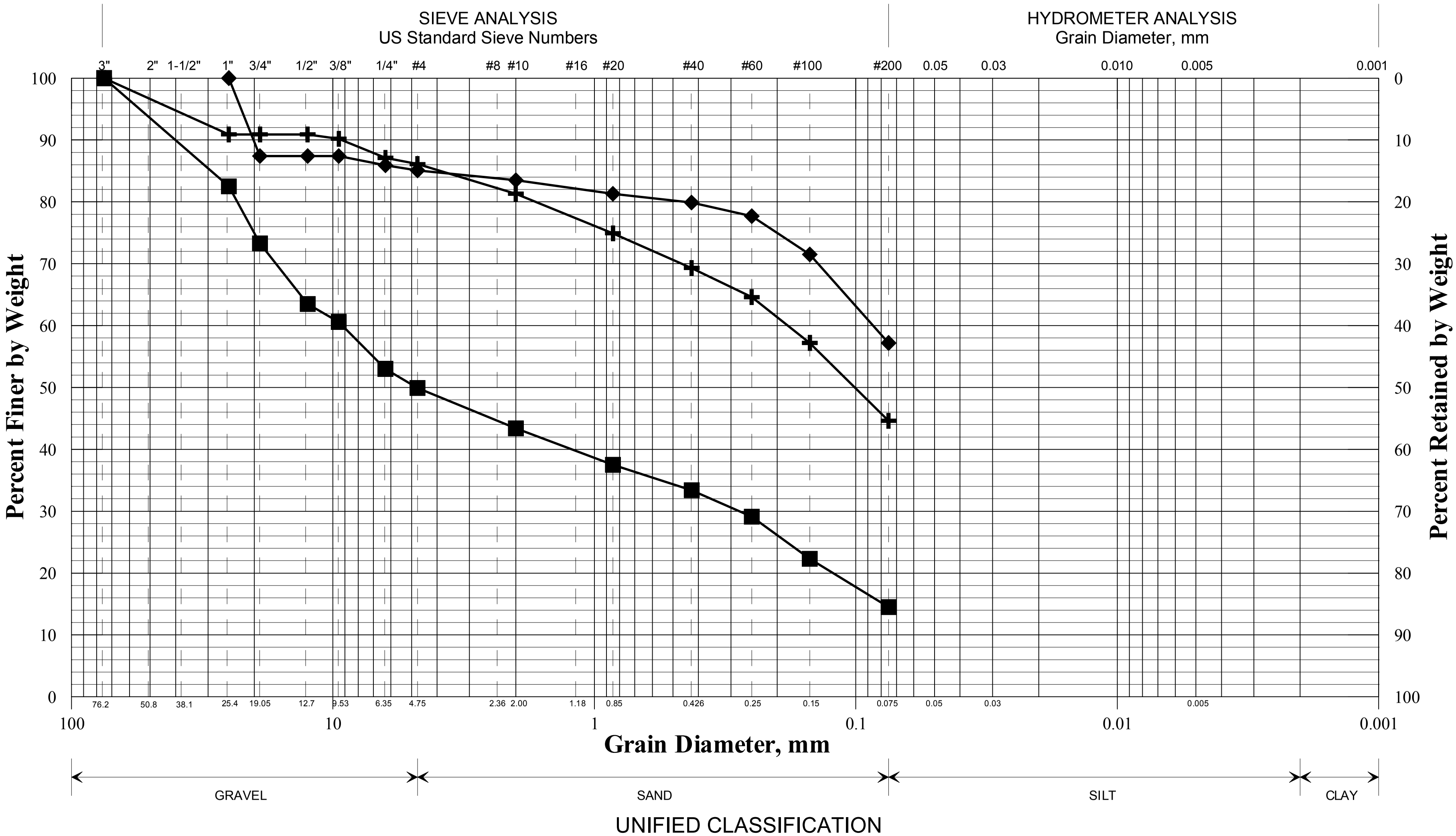
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

WC = water content as determined by AASHTO T 265-93 and/or ASTM D 2216-98

LL = Liquid limit as determined by AASHTO T 89-96 and/or ASTM D 4318-98

PI = Plasticity Index as determined by AASHTO 90-96 and/or ASTM D4318-98

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	HB-BREM-101/1D	12+53.3	7.3 RT	1.0-2.0	Sandy SILT, little gravel.	16.9			
◆	HB-BREM-101/1DA	12+53.3	7.3 RT	2.0-3.0	SILT, some sand, little gravel.	27.7			
■	HB-BREM-101/2D	12+53.3	7.3 RT	5.0-7.0	GRAVEL, some sand, little silt.	6.4			
●									
▲									
✖									

WIN
018104.00
Town
Bristol,Bremen
Reported by/Date
WHITE, TERRY A 1/4/2012

SHEET 1

<u>Appendix D</u>

Calculations

Earth Pressures:

Reference: Geosynthetic Reinforced Soil Integrated Bridge System Interim Implementation Guide Publication No. FHWA-HRT-11-026, dated January 2011 (FHWA 2011)

Effective active earth pressure coefficient for retained backfill:

$\phi_{ab} := 28 \cdot deg$ assumed effective friction angle for retained soil

Rankine Theory - Equation 1, pg 36, FHWA 201

$$K_{ab} := \tan\left(45 \cdot deg - \frac{\phi_{ab}}{2}\right)^2 \qquad K_{ab} = 0.36$$

Effective active earth pressure coefficient for reinforced backfill:

$\phi_r := 38 \cdot deg$ assumed effective friction angle for reinforced soil

Rankine Theory - Equation 1, pg 36, FHWA 201

$$K_{ar} := \tan\left(45 \cdot deg - \frac{\phi_r}{2}\right)^2 \qquad K_{ar} = 0.24$$

Effective passive earth pressure coefficient for reinforced soil:

$\phi_{pr} := 38 \cdot deg$ assumed effective friction angle for reinforced soil

Rankine Theory - Equation 26, pg 46, FHWA 2011

$$K_{pr} := \tan\left(45 \cdot deg + \frac{\phi_{pr}}{2}\right)^2 \qquad K_{pr} = 4.2$$

Service Limit State Bearing Resistance - Native Granular Soils:

Nominal and factored Bearing Resistance - GRS Abutment on bedrock

Presumptive Bearing Resistance for Service Limit State ONLY

Reference: AASHTO LRFD Bridge Design Specifications 5th Edition
Table C10.6.2.6.1-1 Presumptive Bearing Resistances for Spread Footings at the Service Limit State Modified after US Department of Navy (1982)

Type of Bearing Material: Bedrock of any kind

Consistency In Place: medium to hard rock

Bearing Resistance: Ordinary Range (ksf) 16 to 14

Recommended Value of Use: 20 ksf

Recommended Value: $20 \cdot ksf = 10 \cdot tsf$ $tsf := g \cdot \left(\frac{ton}{ft^2}\right)$

Therefore: $q_{nom} := 10 \cdot tsf$

Resistance factor at the **service limit state** = 1.0 (LRFD Article 10.5.5.1)

$q_{factored\_bc} := 10 \cdot tsf$ or $q_{factored\_bc} = 20 \cdot ksf$

Note: This bearing resistance is settlement limited (1 inch) and applies only a the service limit state.

StrengthLimit State Bearing Resistance - Bedrock:

Nominal and factored Bearing Resistance - GRS Abutment on bedrock

Reference: Geosynthetic Reinforced Soil Integrated Bridge System (GRS-IBS) Interim Implementation Guide Publication No. FHWA-HRT-11-026

1. The GRS Abutments will be founded on bedrock $D_{Abut} := 0 \cdot ft$

2. Assumed parameters for bedrock:

   Saturated unit weight: $\gamma_s := 135 \cdot pcf$

   Dry unit weight: $\gamma_d := 140 \cdot pcf$

   Internal friction angle: $\phi_{ns} := 45 \cdot deg$

   Undrained shear strength: $c_{ns} := 0 \cdot psf$

3. Bearing Capacity Factors - Table 4 GRS-IBS Implementation Guide

   For ϕ=45 deg $N_c := 133.9$ $N_q := 134.9$ $N_\gamma := 271.8$

4. Groundwater:

   Depth the water table: $D_w := 0 \cdot ft$ Unit Weight of water: $\gamma_w := 62.4 \cdot pcf$

5. Foundation width, B:

$$B := \begin{pmatrix} 5 \\ 6 \\ 7 \\ 8 \\ 9 \\ 10 \\ 12 \\ 14 \end{pmatrix} \cdot ft$$

6. Determine Nominal Bearing Resistance equation (GRS-IBS Implementation Guide Eq 21 (pg 43)

$$q_{nominal} := c_{ns} \cdot N_c + 0.5 \cdot B \cdot \left(\gamma_s - \gamma_w\right) \cdot N_\gamma + \left(\gamma_s - \gamma_w\right) \cdot D_{Abut} \cdot N_q$$

$$q_{nominal} = \begin{pmatrix} 49.3 \\ 59.2 \\ 69.1 \\ 78.9 \\ 88.8 \\ 98.7 \\ 118.4 \\ 138.1 \end{pmatrix} \cdot ksf$$

7. Determine factored resistance:

Resistance Factor: $\phi_{bc} := 0.65$

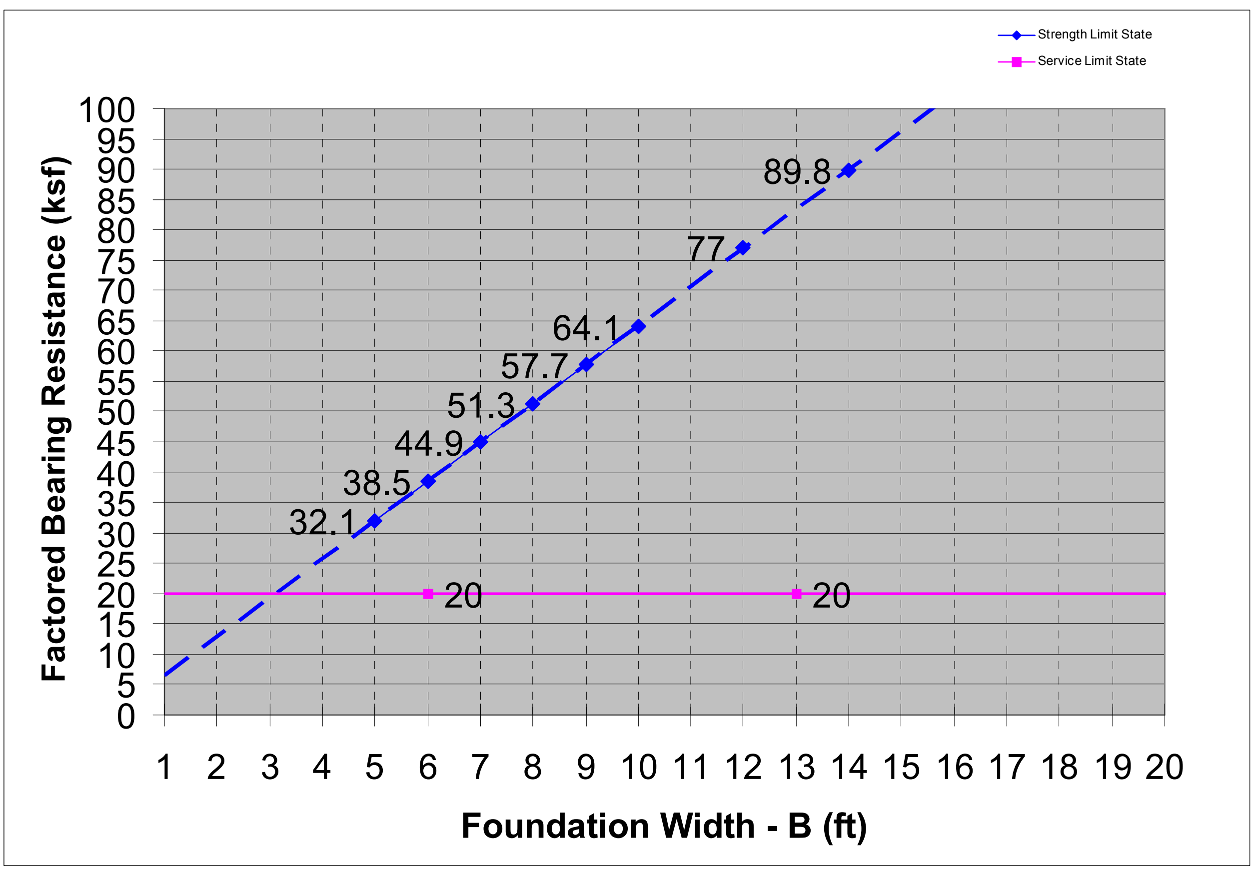
Geosynthetic Reinforced Soil Integrated Bridge System Interim Implementation Guide
Publication No. FHWA-HRT-11-026
Appendix C - pg 135

$$q_{factored} := q_{nominal} \cdot \phi_{bc}$$

$$q_{factored} = \begin{pmatrix} 16 \\ 19 \\ 22 \\ 26 \\ 29 \\ 32 \\ 38 \\ 45 \end{pmatrix} \cdot tsf \qquad B = \begin{pmatrix} 5 \\ 6 \\ 7 \\ 8 \\ 9 \\ 10 \\ 12 \\ 14 \end{pmatrix} \cdot ft \qquad q_{factored} = \begin{pmatrix} 32.1 \\ 38.5 \\ 44.9 \\ 51.3 \\ 57.7 \\ 64.1 \\ 77.0 \\ 89.8 \end{pmatrix} \cdot ksf$$

At Strength Limit State:

Recommend a limiting factored bearing resistance of 32 ksf for 5 feet of reinforcement



Conterminous 48 States

2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years

Sate- Maine
Zip Code- 04551
Zip Code Latitude= 44.019800
Zip Code Longitude= -069.443800

Site Class B

Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)		
0.0	0.065	PGA,	Site Class B
0.2	0.140	Ss,	Site Class B
1.0	0.042	S1,	Site Class B

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Spectral Response Accelerations SDs and SD1

State- Maine
Zip Code- 04551
Zip Code Latitude= 44.019800
Zip Code Longitude= -069.443800

As = FpgaPGA, SDs = FaSs, and SD1 = FvS1
Site Class D-Fpga = 1.60, Fa = 1.60, Fv = 2.40

Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)		
0.0	0.104	As,	Site Class D
0.2	0.223	SDs,	Site Class D
1.0	0.100	SD1,	Site Class D