MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For New Bridge Construction of:

GREYS BROOK BRIDGE
ROUTE 180 OVER GREYS BROOK
ELLSWORTH, MAINE

Prepared by:
Laura Krusinski, P.E.
Senior Geotechnical Engineer

Reviewed by:
Michael Moreau, P.E.
Geotechnical Engineer

Hancock County
PIN 10063.10

Soils Report No. 2011-11
Bridge No. 6436

Fed No. 010063.10
September 30, 2011

Table of Contents

Tables

Figures

Sheets

Appendices

GEOTECHNICAL DESIGN SUMMARY

The purpose of this report is to present subsurface information and make geotechnical recommendations for the new construction of Greys Brook Bridge which will be located on the new alignment of Route 180 in Ellsworth, Maine. The Maine Department of Transportation (MaineDOT) has selected the Greys Brook Bridge as a location to install a 34-foot span composite tubular arch bridge. Currently, it is proposed that the composite arch bridge structure will be founded on a pile cap supported on driven H-piles. The following design recommendations are discussed in detail in this report:

Arch Stem Wall and Pile Cap Design – Arch stem walls and pile caps shall be designed for all relevant strength, service and extreme limit states and load combinations specified in AASHTO LRFD Bridge Design Specifications 5th Edition, 2010 (LRFD). Arch pile caps shall be designed to resist all lateral earth loads, vehicular loads, arch dead and live loads, and lateral thrust forces transferred through the bridge arches.

A resistance factor (ϕ) of 1.0 shall be used to assess arch pile cap design at the service limit state, including: settlement, excessive horizontal movement, and movement resulting after scour due to the design flood. The overall stability of the foundation should be investigated at the Service I Load Combination and a resistance factor, ϕ, of 0.65.

Resistance factors for extreme limit state shall be taken as 1.0. Extreme limit state design shall check the nominal arch foundation resistance remaining after scour due to the check flood.

Calculation of passive earth pressures for resisting lateral thrust forces from the arch should assume a Rankine passive earth pressure coefficient, K_p, of 3.25, anticipating small arch pile cap movements, or a Coulomb K_p of 6.73 should the ratio of lateral pile cap movement to the pile cap stem wall height (y/H) exceed 0.005. Use a resistance factor for passive earth pressures (φ_{ep}) of 0.50 for earth pressure mobilized to resist lateral sliding forces. For designing the arch pile cap reinforcing steel to resist passive earth pressures, use a maximum load factor (γ_{EH}) of 1.50.

Arch stem walls shall include a drainage system behind the arch stem wall/pile cap to intercept any groundwater.

Driven H-Pile Design - H-piles should be end bearing and driven to the required resistance on bedrock or within bedrock. Piles should be 50 ksi, Grade A572 steel and be oriented for strong axis bending.

If structural frame analyses indicate that the H-pile design does not achieve fixity and requires a pinned boundary condition at the pile tip, the piles should be fitted with Rock Injector HP-80500 Pile Points, manufactured by Associated Pile and Fitting (APF), LLC, to improve penetration and friction at the pile tips.

The H-piles will be subjected to lateral loads and should be analyzed for combined axial compression and flexure resistance. The analysis shall assign a free, pinned or fixed condition at the pile tip that is consistent with the proposed pile tip condition.

The design of H-piles at the service limit state shall consider tolerable transverse and longitudinal movement of the piles, overall global stability of the pile group and pile group movements/stability considering changes in soil conditions after scour due to the design flood event.

Extreme limit state design checks for the H-piles shall assess load combinations related to ice loads, debris loads, the check flood for scour and certain hydraulic events. A resistance factor of 1.0 is used. Recommended streambed soil parameters for scour evaluations are provided in Section 7.7.

Preliminary estimates of the calculated factored axial compressive structural, geotechnical and drivability resistances of five H-pile sections for the strength, extreme and service limit states are provided in Tables 7-3 and 7-4 of this report. It is the structural designer's responsibility to recalculate the nominal and factored pile resistances based on actual unbraced lengths, effective lengths and critical buckling (see Section 7.2.1).

Lateral Pile Resistance - Lateral loads may be reacted by plumb or battered piles. We recommend the designer perform a series of lateral pile resistance analyses using L-Pile® software or FB-Pier software. Recommended geotechnical parameters for generation of soil-resistance (p-y) curves in lateral pile analyses are provided in Section 7.2.3 of this report.

Pile Quality Control - The contractor is required to perform a wave equation analysis. The first pile driven at each arch stem wall/pile cap should be dynamically tested to confirm capacity and verify the stopping criteria developed by the contractor in the wave equation analysis. With this level of quality control, the pile should be driven to a nominal resistance equal to the factored axial pile load divided by a resistance factor, φ_{dyn}, of 0.65.

Wingwalls – Wingwalls may consist of cantilever-type cast-in-place (CIP) walls. The CIP walls may be full height or only constructed up to elevation 111 feet (approximately one foot above Q1.1) and then completed with Prefabricated Concrete Modular Gravity (PCMG) or Mechanically Stabilized Earth (MSE) walls above the CIP walls. Three (3) subgrade options for the CIP wall spread footings are discussed in this report:

- Construct the footings on glacial till;
- Construct the footings on the clay-silt deposit at elevation 104.0 feet with over-excavation of 1 to 2 feet of the soft subgrade and replacing with ¾-inch crushed stone;
- Excavate the clayey silt deposit to approximate elevation 96.0 feet and replace with compacted granular fill up to elevation 104.0 feet for the subgrade of the CIP wall spread footing.

A sliding resistance factor, φ_τ, of 0.80 shall be applied to the nominal sliding resistance of cast-in-place footings constructed on either compacted granular fill or the native clayey silt. Sliding computations for resistance to lateral loads shall assume a maximum frictional coefficient of 0.62 = tan 32° for mass concrete on compacted fill and 0.36 = tan 20° for mass concrete on clayey silt.

The location of the resultant of the reaction forces at the strength limit state, based on factored loads should be within the middle one-half (1/2) of the footing width.

The bearing resistance for wall footings founded on glacial till or compacted structural fill shall be investigated at the strength limit state using factored loads and a factored bearing resistance provided in Figure 7-1 of this report. A factored bearing resistance of 6 ksf may be used to control settlements when analyzing the service limit state load combination. For footings bearing on 1-foot of ¾-inch crushed stone over native clayey silt at elevation 104 feet, use a factored bearing resistance of 8 ksf for all footing widths at the strength limit state, and a factored bearing resistance of 3 ksf to control settlements when analyzing the service limit state.

A resistance factor of 1.0 shall be used to assess spread footing design at the service limit state, including: settlement and excessive horizontal movement.

Independent wingwalls shall be designed as unrestrained meaning that they are free to rotate at the top in an active state of earth pressure. Earth loads shall be calculated using an active earth pressure coefficient, K_a, of 0.31, calculated using Rankine Theory for cantilever-type walls. The live load surcharge on wingwalls may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) of no less than 2 feet.

MSE Walls - MSE walls may be constructed above the CIP stem walls, above the ordinary high water (Q1.1) elevation of 109.6 feet. MSE walls will be designed by a Professional Engineer subcontracted by the Contractor as a design-build item.

MSE walls should be investigated at the strength limit state for bearing capacity failure, lateral sliding, excessive loss of base contact, pullout of soil reinforcements and structural failure. Sliding computations shall assume a maximum allowable frictional coefficient of 0.58 = tan 30° at the soil base to foundation soil interface. A sliding resistance factor, φ_τ, of 1.0 shall be applied to the nominal sliding resistance of the MSE mass founded on soil. For eccentricity design checks, the location of the resultant of the reaction forces shall be within the middle one-half (1/2) of the base width.

Bearing pressures should be computed using a uniform base distribution over the effective width. Calculated bearing resistance values for MSE reinforced soil volumes founded on compacted granular fill soils are provided in Figure 7-2 of this report. A factored bearing resistance of 6.0 ksf may be used when analyzing the service limit state to control settlement.

The reinforcing length shall be uniform throughout the entire height of the wall. An impervious geomembrane consisting of low-permeability, 2-sided, texture HDPE with a minimum thickness of 40 mils shall be installed near the top of the reinforced soil zone.

PCMG Retaining Walls - PCMG walls may be used to retain approach fills above the ordinary high water (Q1.1) elevation. The walls shall be designed by a Professional Engineer subcontracted by the Contractor as a design-build item. The bearing resistance for PCMG walls founded on granular fill shall be investigated at the strength limit state using the factored bearing resistances shown in Figure 7-3 of this report. Based on presumptive bearing resistance values, a factored bearing resistance of 6 ksf may be used to control settlement when analyzing the service limit state.

Global Stability - Stability analyses to determine factors of safety against global failure of the retaining walls and arch stem walls retaining the 27-foot high approach embankments at the arch stem walls were conducted. Stability analyses indicate that excavation and replacement of the upper 1 foot of the 3 feet of topsoil consisting of clayey silt, organics and roots, and the construction of the 30-foot high walls with a bottom of footing (BOF) elevation of 104 feet will achieve the minimum required factors of safety against global instability.

Settlement. The finished grade of the new alignment of Route 180 will require embankment fills approximately 27 feet high. Post-construction consolidation settlement of the bridge approach fills is estimated to be approximately 2 to 3 inches and will occur over a long period of time. Any settlement of bridge abutments will be due to axial compression of the foundation piles and is anticipated to be less than 0.5 inch.

Frost Protection - Foundations placed on granular fill or native subgrade soils should be founded a minimum of 6 feet below finished exterior grade for frost protection.

Seismic Design Considerations – Seismic analysis is not required for buried structures, except where they cross active faults. There are no known active faults in Maine.

Scour and Riprap - Streambed grain size parameters for scour analyses at the design and check flood events are provided in Section 7.7 of this report.

Plain riprap shall be placed at the toes of arch footings and wingwalls at a maximum slope of 1.75H:1V. 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 and Class 1 nonwoven erosion control geotextile. Riprap shall be 3 feet thick.

Construction Considerations – Construction of the arch footings and wingwalls will require soil excavation and pile driving. Cofferdams and temporary lateral earth support systems will be required to permit construction of arch footings and wingwalls.

Removal of the upper topsoil and clay silt, or additional removal of deeper units of the clayey silt deposit will result in the exposure of naturally deposited pockets of potentially sensitive clayey silts. These soils at the subgrade will be susceptible to disturbance and rutting as a result of exposure to water or construction traffic. If disturbance occurs, we recommend that the contractor remove and replace the disturbed materials with compacted MaineDOT Standard Specification 703.20, Gravel Borrow.

Furthermore, the silt clay soils may become saturated and water seepage may be encountered during construction. There may be localized sloughing and instability in some excavations and cut slopes. The contractor should control groundwater, surface water infiltration and soil erosion.

1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present geotechnical recommendations for the new construction of Greys Brook Bridge which will carry the realigned Route 180 at approximately Station 1072+22 over Greys Brook in Ellsworth, Maine. The proposed Greys Brook Bridge is new construction at a location where no bridge exists now. This report presents the subsurface information obtained at the site during the subsurface investigation and foundation recommendations and geotechnical design parameters for substructure design.

The MaineDOT Bridge Program has selected the proposed Greys Brook Bridge as a location to install a rigidified, concrete-filled composite tubular arch bridge structure developed by the University of Maine's Advanced Engineering Wood Composites (AEWC) Center in Orono, Maine. The carbon fiber tubes are inflated and infused with resin. After hardening, the tubes are transported to the bridge site and lowered into place and filled with concrete. The proposed arch structure will have a span length of approximately 34 feet and will be founded on reinforced concrete stem walls supported on two rows of driven H-piles. The vertical grade of the proposed new alignment of Route 180 at Station 1072+00 will require approximately 27 feet of new fill at the proposed bridge.

2.0 GEOLOGIC SETTING

Proposed Greys Brook Bridge on the new Route 180 alignment in Ellsworth, Maine will cross Greys Brook as shown on Sheet 1 - Location Map, presented at the end of this report.

The Maine Geologic Survey (MGS) Surficial Geology of Ellsworth Quadrangle, Maine, Open-file No. 82-3 (1982) indicates the surficial soils in the vicinity of the project consists primarily of glaciomarine deposits with a nearby glacial till soil unit contact. The predominant native soil units at the site based on our subsurface explorations are glaciomarine which consist of silt and clay overlying glacial till.

Glacial till generally consists of a heterogeneous mixture of sand, sit, clay and stones. Basal till is fine grained and very compact, with low permeability and poor drainage. The unit generally overlies bedrock, but may overlie or include sand and gravel. Glacial till was originally deposited directly by glacial ice, and commonly conforms to the topography of the bedrock surface.

The Bedrock Geologic Map of Maine, MGS, (1985), cites the bedrock at the proposed bridge site as the Ellsworth Formation consisting of metamorphic, interbedded pelite and sandstone. Bedrock cores obtained from test borings at the site are identified as phyllite.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions at the site were explored by drilling four test borings. The four borings were terminated with bedrock cores. Test borings BB-GB-104 and BB-GB-102 were drilled along the centerline of Route 180 at the proposed locations of Arch Footing 1 and Arch Footing 2, respectively. Test borings BB-GR-101 and BB-GR-103 were drilled approximately 61 feet Lt. and 48.0 feet Rt., respectively, to facilitate slope stability analyses.

The boring locations are shown on Sheet 2 - Boring Location Plan found at the end of this report. The borings were drilled between August 23 and 25, 2010 by Northern Test Boring (NTB) of Gorham, Maine. 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 and on Sheet 5 - Boring Logs found at the end of this report.

The borings were drilled using cased wash boring and solid stem auger techniques. Soil samples were typically obtained at 5-foot intervals using Standard Penetration Test (SPT) methods. During SPT sampling, the sampler is driven 24 inches and the hammer blows for each 6 inch interval of penetration are recorded. The sum of the blows for the second and third intervals is the N-value, or standard penetration resistance. The NTB dill rig is equipped with a Dietrich D-50 automatic hammer. The hammer was calibrated by NTB in March of 2010. The N-values presented for borings drilled with the Dietrich D-50 hammer are corrected values computed by applying an average energy transfer factor of 0.713 to the raw field N-values. The hammer efficiency factor of 0.713 and both the raw field N-value and the corrected N-value are shown on the boring logs.

The bedrock was cored in the four borings using an NQ-2" core barrel and the Rock Quality Designation (RQD) of the core was calculated. The MaineDOT Geotechnical Team member selected the boring locations and drilling methods, designated type and depth of sampling techniques, reviewed field logs for accuracy and identified field and laboratory testing requirements. A MaineDOT New England Transportation Technician Certification Program (NETTCP) Certified Subsurface Inspector and a consultant geologist logged the subsurface conditions encountered. The borings were staked in the field by the MaineDOT geotechnical team member and surveyed by the MaineDOT Survey Crew at the completion of the drilling program.

4.0 LABORATORY TESTING

A laboratory testing program was conducted on selected samples recovered from test borings to assist in soil classification, evaluation of engineering properties of the soils, and geologic assessment of the project site.

Laboratory testing consisted of four standard grain size analyses, four grain size analyses with hydrometer, eight natural water content tests, and four Atterberg Limits test. The tests were performed in the MaineDOT Materials and Testing Laboratory in Bangor, Maine. The results of soil laboratory tests are included as Appendix B – Laboratory Test Results.

Laboratory test information is also shown on the boring logs provided in Appendix A – Boring Logs and on Sheet 5 - Boring Logs.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered in the test borings generally consisted of topsoil, glaciomarine clayey silt, and glacial till underlain by metamorphic bedrock. The boring logs are provided in Appendix A – Boring Logs and on Sheet 5 – Boring Logs. The following paragraphs discuss the subsurface conditions encountered in detail:

5.1 Topsoil

A 0.3 to 3-foot layer of topsoil was encountered in the borings. The upper topsoil unit consists of roots and sod. A lower topsoil unit was encountered. The encountered thickness is approximately 2 to 2.7 feet thick. The lower topsoil unit consists of brown to dark brown to grey-brown, wet clayey silt, some fine sand, little organics, trace roots or dry sandy silt, trace roots.

SPT N-values in topsoil subunits were 2 to 6 blows per foot (bpf), indicating the topsoil units are soft to medium stiff in consistency.

5.2 Glaciomarine Clayey Silt

A glaciomarine deposit was encountered in all of the borings. The encountered thickness is approximately 8 to 10 feet. The glaciomarine deposit encountered consisted of grey, brownish-grey moist to wet, clayey silt, and trace fine sand layers, or silt with some clay, trace fine sand.

Four grain size analyses of the glaciomarine clay silt resulted in the soil being classified as A-4 and A-6 under the AASHTO Soil Classification System and CL under the Unified Soil Classification System (USCS).

Atterberg Limits tests on samples from the deposit determined moisture contents ranged from approximately 20 to 30 percent and plastic limits ranged from 19 to 20. For three of the four samples tested, the natural water contents were less than the liquid limits and greater than the plastic limits, and the calculated liquidity indexes (LI) were less than 1.0. Therefore, the clay-silt deposit is generally lightly consolidated. The exception is a clay-silt subunit encountered in BB-GB-103 from which a tested sample had a water content greater than the plastic limit and liquid limits and LI greater than 1.0. This glaciomarine subunit is normally consolidated.

Table 1 summarizes the results of Atterberg Limits test made from samples of the clay-silt unit:

Sample No.	Soil Description	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index
BB-GB-101, 2D	Clayey Silt, trace sand	22.6	31	19	12	0.30
BB-GB-102, 2D	Clayey Silt, trace sand	21.8	30	20	10	0.18
BB-GB-103, 3D	Clayey Silt, trace sand	30.1	29	20	9	1.12
BB-GB-104, 2D	Silt, some clay, trace sand	20.1	29	20	9	0.01

Table 5-1 Atterberg Limits Test Results

Vane shear testing conducted within the clay-silt layer showed measured undrained shear strengths of the layer to range from about 2986 psf to greater than 5650 psf, indicating that the clay-silt unit is very stiff to hard in consistency. Where SPT tests were conducted in the clay-silt, N-values ranged between 8 and 18 indicting that the clay-silt is medium stiff to very stiff in consistency. The remolded strength at one test interval was 2358 psf. Based on the ratio of peak to remolded shear strength at that one test interval, the clay-silt has a sensitivity of 1.27 and is classified a low sensitivity.

5.3 Glacial Till

A deposit of glacial till was encountered in all four borings. The encountered thickness is approximately 4.3 to 11.5 feet. The glacial till deposit encountered consisted of

- brown, wet, fine to coarse sand, some gravel, little silt
- brown, wet, fine to medium sand, trace silt
- brown, wet silty fine to coarse sand, trace gravel
- light brownish-grey, damp, fine to medium sand, some gravel, trace to little silt

SPT N-values in glacial till were 25 to 48 bpf in glacial till, indicating the till is medium dense to dense in consistency.

Four grain size analyses of the glacial till resulted in the soil being classified as A-4 and A-1-b under the AASHTO Soil Classification System and SM and SW-SM under the USCS.

5.4 Bedrock

Bedrock at the site was encountered and cored at depths ranging from approximately 15.3 to 24.5 feet below ground surface (bgs) and approximate Elevation 94.60 feet in boring BB-GB-104 to approximate Elevation 85.40 feet in boring BB-GB-103.

The bedrock at the site is identified as pinkish grey, fine grained, hard, slightly weathered to fresh, calcareous phyllite, with limited quartz inclusions and veins, thinly bedded at steeply dipping angles to chaotic orientation. Rock cores recovered ranged from very highly fractured to massive. The RQD of the bedrock was determined to range from 0 to 83 percent, correlating to a rock mass quality of very poor to good.

5.5 Groundwater

Groundwater was observed at depths ranging from approximately 3.2 feet to 4.0 feet bgs in the borings at the time of drilling. The water levels measured upon completion of drilling are indicated on the boring logs found in Appendix A. Note that water was introduced into the boreholes during the drilling operations. Therefore, the water levels indicated on the boring logs may not represent stabilized groundwater conditions. Groundwater levels will fluctuate with seasonal changes, precipitation, runoff, and construction activities.

6.0 FOUNDATION ALTERNATIVES

The MaineDOT Bridge Program has selected Greys Brook on the new alignment of Route 180 as a location to install a rigidified, inflatable, composite tubular arch bridge structure developed by the University of Maine's AEWC Advanced Structures & Composites Center in Orono, Maine. AEWC's tubular arches are made of Fiber Reinforced Polymer (FRP) composite materials. The carbon fiber tubes are inflated off-site and infused with resin. After hardening, the tubes are transported to the bridge site, lowered into place and filled with concrete. The tubular arches are covered with a corrugated, FRP composite deck material and backfill is placed over the tubular structure.

The following foundation alternatives can considered for proposed arch bridge:

- reinforced concrete arch stem walls/pile caps supported on H-piles or pipe piles driven to bedrock
- spread footings founded on seals cast on native glacial till unit
- spread footings founded on the clay-silt deposit

Due to the challenges to engineer the arch footings on clay-silt soils or glacial till to resist lateral thrust reactions, it is our understanding that the composite tubular arch bridge will be founded on driven piles. For the purposes of this geotechnical report it is assumed that driven H-piles will be used to support the arch bridge structure. Design recommendations for this foundation alternative are discussed in detail in Section 7.0 - Geotechnical Design Recommendations. If during final design, it is determined that the use of another pile section is necessary or spread footings on till are deemed feasible, additional geotechnical design recommendations and parameters will be developed and provided to the designer.

Design recommendations are also provided for independent cantilever-type retaining walls, PCMG walls and MSE walls, which may be used as wingwalls or to support bridge approaches.

The design of the FRP tubular arches and associated headwalls is the responsibility of the AEWC and will be supplied to the bridge designer and Contractor prior to construction of the structure.

7.0 GEOTECHNICAL DESIGN CONSIDERATIONS AND RECOMMENDATIONS

This section provides geotechnical design recommendations for H-pile supported arch stem walls/pile caps.

7.1 Arch Footing/Pile Cap Design

Arch stem walls and pile caps shall be designed for all relevant strength, service and extreme limit states and load combinations specified in AASHTO LRFD Bridge Design Specifications 5th Edition, 2010 (LRFD) Articles 3.4.1, 11.5.5., and 12.5. Arch pile caps shall be designed to resist all lateral earth loads, vehicular loads, arch dead and live loads, and lateral thrust forces transferred through the bridge arches. The design of arch pile caps at the strength limit state shall consider pile reinforced-concrete structural design.

A resistance factor (ϕ) of 1.0 shall be used to assess arch pile cap design at the service limit state, including: settlement, excessive horizontal movement, and movement resulting after scour due to the design flood. The overall stability of the foundation should be investigated at the Service I Load Combination and a resistance factor, ϕ, of 0.65.

Extreme limit state design of the pile cap supported on H-piles shall include pile structural resistance, pile geotechnical resistance and pile resistance in combined axial and flexure, and overall stability. Resistance factors for extreme limit state shall be taken as 1.0. Extreme limit state design shall also check that the nominal arch foundation resistance remaining after scour due to the check flood can support the extreme limit state loads with a resistance factor of 1.0. Recommended streambed soil parameters for scour evaluations are provided in Section 7.7.

The designer may assume Soil Type 4 (MaineDOT Bridge Design Guide (BDG) Section 3.6.1) for arch wall and pile cap backfill material soil properties. The backfill properties are as follows: $\phi = 32°$, $\gamma = 125$ pcf.

Calculation of passive earth pressures for <u>resisting</u> lateral thrust forces from the arch should assume a Rankine passive earth pressure coefficient, K_p, of 3.25, anticipating the arch pile caps experience small movements. Should the ratio of lateral pile cap movement to the pile cap stem wall height (y/H) exceed 0.005, then the calculation of passive earth pressure may assume a Coulomb passive earth pressure coefficient, K_p, of 6.73. Use a resistance factor for passive earth pressures (φ_{ep}) of 0.50 for earth pressure mobilized to resist lateral sliding forces, per LRFD Table 10.5.5.2.2-1. For designing the arch pile cap reinforcing steel to resist passive earth pressures, use a maximum load factor (γ_{EH}) of 1.50.

Additional lateral earth pressure due to live load surcharge is required per Section 3.6.8 of the MaineDOT BDG. The live load surcharge on arch stem walls/pile caps may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) taken from Table 7-1 below:

Arch Stem Wall Height (feet)	h_{eq} (feet)
5	4.0
10	3.0
>=20	2.0

Table 7-1 Equivalent Height of Soil for Estimating Live Load Surcharge on Arch Stem Walls and Pile Caps

Arch stem walls shall include a drainage system behind the arch stem wall/pile cap to intercept any groundwater. We recommend weep holes be constructed approximately 6 inches above Q1.1 (normal high water). Drainage behind the structure shall be in accordance with Section 5.4.1.4 Drainage, of the MaineDOT BDG.

Backfill within 10 feet of the arches, arch footing and side slope fill shall conform to Granular Borrow for Underwater Backfill - MaineDOT Specification 709.19. This gradation specifies 10 percent or less of the material passing the No. 200 sieve. This material is specified in order to reduce the amount of fines and to minimize frost action behind the structure.

7.2 Driven H-Pile Design

H-piles for support of the arches should be end bearing and driven to the required resistance on bedrock or within bedrock. Piles may be HP 12x53, 12x74, 14x73, 14x89, or 14x117 depending on the factored design axial and lateral loads. Piles should be 50 ksi, Grade A572 steel. The piles should be oriented for strong axis bending. Piles should be fitted with driving pile points to protect the tips and improve penetration. Piles may be plumb, battered or a combination of both.

If structural frame analyses indicate the H-pile design does not achieve fixity and requires a pinned boundary condition at the pile tip, the piles should be fitted with Rock Injector HP-80500 Pile Points, manufactured by Associated Pile and Fitting (APF), LLC, to improve penetration and friction at the pile tips.

Pile lengths at the proposed arch stem wall/pile caps, considering a nominal 2-foot pile embedment in the pile cap, will range from approximately 12 to 17 feet. This data is summarized in Table 7-2 below:

Proposed Structure	Approximate Bedrock Elevation (feet)	Estimated Arch Stem Wall/Pile Cap Bottom Elevation (feet)	Estimated Pile Embedment in Abutment (feet)	Estimated Pile Lengths after cut-off (feet)
Abutment 1 Pile Cap	94.60	104.0	2.0	11.4
Abutment 2 Pile Cap	89.70	104.0	2.0	16.3

Table 7-2 Estimated Pile Lengths for Plumb Piles

The pile lengths do not take into account consideration to accommodate locations where bedrock may be deeper than that encountered in the four borings, or the additional five feet of pile required for dynamic testing instrumentation or pile length needed to accommodate leads and driving equipment.

The center-to-center pile spacing should not be less than 30 inches or 2.5 to 3 times the pile diameter. The distance from the side of any pile to the nearest edge of the pile cap shall not be less than 9 inches. The tops of the piles should project at least 18 inches into the pile cap.

7.2.1 Piles - Strength Limit State Design

The design of pile foundations bearing on or within the bedrock at the ***strength limit state*** shall consider:

- compressive axial geotechnical resistance of individual piles bearing on bedrock
- structural resistance of individual piles in axial compression
- structural resistance of individual piles in combined axial loading and flexure
- geotechnical uplift resistance of piles in tension
- structural failure in tension of pile head to pile cap connection
- failure by lateral loading of piles

Strength limit state load combinations are specified in AASHTO LRFD Bridge Design Specifications 5th Edition, 2010 (LRFD) Articles 3.4.1, 11.5.5., and 12.5. The pile groups should be designed to resist all lateral earth loads, vehicular loads, arch dead and live loads, and lateral thrust forces transferred through the pile caps. Resistance factors at the strength limit state are provided in this section. The pile group resistance after scour due to the design flood shall provide adequate foundation resistance using the resistance factors given in this section.

A modified Strength Limit State analysis should be performed that includes the ice pressures specified in BDG Section 3.9 – Ice Loads.

Since the arch H-piles will be subjected to lateral loading, the piles should be analyzed for combined axial compression and flexure resistance as prescribed in LRFD Articles 6.9.2.2 and 6.15.2. The analysis shall assign a free, pinned or fixed condition at the pile tip that is consistent with the proposed pile tip condition. As the proposed piles will be short and may not achieve fixity, the resistance for the piles should be determined for compliance with the interaction equation and checked for buckling.

In accordance with LRFD 6.15.1, the structural analysis of pile groups subjected to lateral loads shall include explicit consideration of soil-structure interaction effects as specified in LRFD 10.7.3.9. Assumptions regarding a fixed, pinned or free condition at the pile tip should be also confirmed with soil-structure interaction analyses.

The nominal and factored axial geotechnical resistance in the strength limit state was calculated using the Canadian Geotechnical Society method and a resistance factor, φ_{stat}, of 0.45 and are provided in Table 7-3, below.

The nominal compressive structural resistance (P_n) for piles loaded in compression shall be as specified in LRFD 6.9.4.1. It is the responsibility of the structural designer to recalculate the nominal and factored pile structural compressive resistance (P_n) based on the "actual unbraced pile length (ℓ) and effective length factor (K)" or "on the actual elastic critical buckling resistance, P_e." Preliminary estimates of the factored structural axial compressive resistance of five H-pile sections were calculated using a resistance factor, ϕ_c, of 0.60 (for good driving conditions) an unbraced length (ℓ) of 0 feet, and an effective length factor (K) of 2.0.

Drivability analyses were performed to determine the resistance that might be achieved considering available diesel hammers. The maximum driving stresses in the pile, assuming the use of 50 ksi steel, shall be less than 45 ksi. The resistance factor for a single pile in axial compression when a dynamic test is performed given in LRFD Table 10.5.5.2.3-1 is φ_{dyn} = 0.65.

A summary of the calculated factored axial compressive structural, geotechnical and drivability resistances of five H-piles sections for the strength limit state is provided in Table 7-3 below. Supporting calculations are provided in Appendix C – Calculations.

	Strength Limit State Factored Axial Pile Resistance			
	Structural Resistance ϕ_c=0.60[1] (kips)	Geotechnical Resistance, φ_{stat} = 0.45 (kips)	Drivability Resistance φ_{dyn} = 0.65 (kips)	Governing Pile Axial Resistance (kips)
HP 12 x 53	464	95	259	259
HP 12 x 74	653	133	332	332
HP 14 x 73	641	128	377	377
HP 14 x 89	782	156	384	384
HP 14 x 117	1031	206	440	440

Table 7-3 Factored Axial Pile Resistances for H-Piles for Strength Limit State Design

LRFD Article 10.7.3.2.2 states that the factored axial compressive resistance of piles driven to hard rock is typically controlled by the structural resistance. However, for these site conditions the factored axial geotechnical resistance and the estimated factored resistance from the drivability analyses are less than the factored axial structural resistance. Therefore, the recommended governing resistance for pile design is the factored drivability resistance in Table 7-3, above. The maximum applied factored axial pile load should not exceed the governing factored pile resistance shown in Table 7-3 above.

The piles shall also be checked for resistance against combined axial load and flexure, per LRFD Article 6.15. This design axial load may govern the design. Per LRFD 6.5.4.2, at the strength limit state, the axial resistance factor ϕ_c = 0.70 and the flexural resistance factor ϕ_f = 1.0 shall be applied to the combined axial and flexural resistance of the pile in the interaction equation (LRFD Eq. 6.9.2.2-1 or -2). The combined axial compression and flexure should be evaluated in accordance with the applicable sections of LRFD Articles 6.9.2.2 and 6.15.2.

7.2.2 Service and Extreme Limit State Design

The design of H-piles at the service limit state shall consider tolerable transverse and longitudinal movement of the piles, overall global stability of the pile group and pile group movements/stability considering changes in soil conditions after scour due to the design flood event.

Extreme limit state design checks for the H-piles shall include pile axial bearing resistance, failure of the pile group by overturning (eccentricity), pile failure by uplift in tension and structural failure. The extreme event load combinations are those related to ice loads, debris loads, the check flood for scour and certain hydraulic events.

[1] Calculated using a resistance factor, ϕ_c, for good driving conditions, an unbraced length (ℓ) of 0 feet and a K of 2.0. The piling may not achieve fixity, therefore the factored structural resistance may be controlled by combined the axial and flexural resistance of the pile.

Extreme limit state design shall also check that the nominal pile foundation resistance remaining after scour due to the check flood can support the extreme limit state loads with a resistance factor of 1.0. Recommended stream bed soil parameters for scour evaluations are provided in Section 7.7.

The ice pressures for Extreme Event II shall be applied at the Q1.1 and Q50 elevations as defined in BDG Section 3.9 with the design ice thickness increased by 1 foot and a load factor of 1.0.

For the service and extreme limit states, resistance factors, ϕ, of 1.0 should be used for the calculation of structural, geotechnical and drivability axial pile resistances in accordance with LRFD Article 10.5.5.1 and 10.5.5.3. The exception is the service limit state resistance factor for the uplift resistance of piles which shall be 0.80.

The nominal and factored axial geotechnical piles resistance in the service and extreme limit state was calculated using the Canadian Geotechnical Society method and a resistance factor, φ, of 1.0. The calculated factored axial structural, geotechnical and drivability resistances of five H-pile sections for the service and extreme limit states and are provided below in Table 7-4. Supporting documentation is provided in Appendix C – Calculations.

	Service and Extreme Limit State Factored Axial Pile Resistance			
	Structural Resistance [2] ϕ=1.0 (kips)	Geotechnical Resistance, ϕ= 1.0 (kips)	Drivability Resistance ϕ = 1.0 (kips)	Governing Axial Pile Resistance (kips)
HP 12 x 53	774	210	398	398
HP 12 x 74	1089	295	510	510
HP 14 x 73	1069	285	580	580
HP 14 x 89	1304	348	590	590
HP 14 x 117	1719	458	676	676

Table 7-4 Factored Axial Pile Resistance for H-Piles for Service and Extreme Limit State Design

LRFD Article 10.7.3.2.2 states that the factored axial compressive resistance of piles driven to hard rock is typically controlled by the structural resistance. However, at this site the factored geotechnical pile resistance and the factored pile resistance from the drivability analyses are less than the factored axial pile structural resistance. Therefore, it is

[2] Calculated using a resistance factor of ϕ_c=1.0, an unbraced length (ℓ) of 0 feet and a K of 2.0. Short pile may not achieve fixity, therefore the factored structural resistance will be controlled by combined the axial and flexural resistance of the pile.

recommended that the governing resistance used in service/extreme limit state design be the calculated factored drivabilty resistances in the Table 7-4.

7.2.3 Lateral Pile Resistance

Lateral loads may be reacted by plumb or battered piles. We recommend the designer perform a series of lateral pile resistance analyses to evaluate pile top deflections and bending stresses under strength limit state design lateral loads using L-Pile® software or FB-Pier software. Similar software for analyzing pile response under lateral loads where the nonlinear soil behavior is modeled using soil resistance (p-y) curves may be used. These analyses should take into consideration pile batter, if any. There is not a performance criteria at this time for allowable lateral displacements at the pile head, therefore, the designer should consider performing lateral pile analyses to determine maximum factored lateral loads permissible based on the allowable displacement criteria. Furthermore, the designer should evaluate the associated pile stresses under factored lateral loads.

Recommended geotechnical parameters for generation of soil-resistance (p-y) curves in lateral pile analyses are provided in Table 7-5 below. In general, the model developed should emulate the soil at the site by using the soil layers (referenced in the tables below by elevations) and appropriate structural parameters and pile-head boundary conditions for the pile section being analyzed. It is recommended that the analyses be conducted assuming a fixed pile-head boundary condition.

Soil Layer	Approx. Elevation of Soil Layer (feet)	Water Table Condition	Effective Unit Weight lbs/in^3 (lbs/ft^3)	k_s (lb/in^3)	Cohesion (lb/in^2)	E_{50} for clays	Friction Angle
Clay-Silt (Glaciomarine)	110 – 99	Below	0.0304 (53)	-	2000	0.005	-
Sand, gravel, silt (Glacial Till)	58 – 87	Below	0.0336 (58)	120	-	-	36°

Table 7-5 Soil Parameters for Generation of Soil-Resistance (p-y) Curves

7.2.4 Driven Pile Resistance and Pile Quality Control

The contractor is required to perform a wave equation analysis of the proposed pile-hammer system and a dynamic pile test with signal matching at each arch pile cap. The first pile driven at each arch stem wall/pile cap should be dynamically tested to confirm nominal pile resistance and verify the stopping criteria developed by the contractor in the wave equation analysis. Restrikes will be not be required as part of the pile field quality control program unless pile behavior indicates the pile is not seated firmly on bedrock or if piles “walk” out of position.

With this level of quality control, the ultimate resistance that must be achieved in the wave equation analysis and dynamic testing will be the factored axial pile load divided by a resistance factor, ϕ_{dyn}, of 0.65. The maximum factored axial pile load should be shown on the plans.

Piles should be driven to an acceptable penetration resistance as determined by the contractor based on the results of a wave equation analysis and as approved by the Resident. Driving stresses in the pile determined in the drivability analysis shall be less than 45 ksi, in accordance with LRFD Article 10.7.8. A hammer should be selected which provides the required pile resistance when the penetration resistance for the final 3 to 6 inches is 5 to 10 blows per inch (bpi), which is the optimal range for diesel hammers. If an abrupt increase in driving resistance is encountered, the driving could be terminated when the penetration is less than 0.5-inch in 10 consecutive blows.

7.3 Return Wingwalls

MSE walls or PCMG walls may be constructed on all four corners above cantilever-type cast-in-place (CIP) walls constructed up to elevation 111.0 feet which is 1 foot above the Q1.1 elevation. Q1.1 is approximately elevation 109.6 feet. The MSE or PCMG walls will be designed by a Professional Engineer subcontracted by the Contractor as a design-build item. The walls shall be designed in accordance with LRFD and Special Provisions 635 or 636, which are included in Appendix D at the end of this report.

Spread footings for the CIP cantilever walls may be:

1.) Founded on glacial till, at approximate elevations ranging from 96.9 to 99.0 feet;

2.) Founded at frost depth on the clay-silt deposit at elevation 104.0 feet, with over-excavation and replacement of 1 to 2 feet of the clay-silt subgrade with ¾ inch stone;

3.) Founded at frost depth at elevation 104.0 feet with over-excavation of the clay-silt deposit in its entirety to approximate elev. 96.0 feet, replaced with compacted granular fill.

7.3.1 Cantilever-type Wingwalls

Cantilever-type wingwalls on spread footings shall be designed for all relevant strength, service and extreme limit states and load combinations specified in AASHTO LRFD Bridge Design Specifications 5th Edition, 2010, (LRFD) Articles 3.4.1, 11.5.5, and 12.5. Retaining wall spread footings shall be designed to resist all lateral earth loads, vehicular loads, and any forces transferred through the arches. The design of wingwall spread footings at the strength limit state shall consider bearing resistance, eccentricity (overturning), lateral sliding and reinforced-concrete structural design.

Extreme limit state design shall also check that the nominal foundation resistance remaining after scour due to the check flood can support the extreme limit state loads with a resistance factor of 1.0. Recommended stream bed soil parameters for scour evaluations are provided in Section 7.7. In general, spread footings at stream crossings should be founded a minimum of 2 feet below the calculated design scour depth.

Failure by sliding shall be investigated. A sliding resistance factor, φ_τ, of 0.80 shall be applied to the nominal sliding resistance of cast-in-place footings constructed on glacial till, compacted granular fill or ¾ inch crushed stone that replaces over-excavated clay silt soils. Sliding computations for resistance to lateral loads shall assume a maximum frictional coefficient of 0.62, for mass concrete on glacial till or compacted fill based on an internal friction angle of 32° in accordance with LRFD Article 10.6.3.4. For footings bearing on the clay-silt soils, or upon ¾ inch crushed stone that replaces over-excavated clay silt soils, use a maximum frictional coefficient of 0.36, based on an internal friction angle of 20° for undrained silt.

For spread footings on soil, the location of the resultant of the reaction forces at the strength limit state, based on factored loads should be within the middle one-half (1/2) of the footing width.

Wingwall spread footings shall be proportioned to provide stability against bearing capacity failure. Application of permanent and transient loads are specified in LRFD Article 11.5.5. The vertical stress may be calculated assuming a uniform stress distribution over the effective base as shown in LRFD Figure 11.6.3.2-1. The bearing resistance for spread footings founded on the <u>native clay-silt</u>, regardless if the clay-silt is over-excavated and replaced with ¾-inch stone, shall be evaluated at the strength limit state using a factored bearing resistance of 8 ksf, regardless of footing width. This assumes a bearing resistance factor, φ_b, for spread footings on soil of 0.45. For footings bearing on clay-silt soils, a factored bearing resistance of 3 ksf should be used for preliminary footing sizing and to control settlements when analyzing the service limit state load combination. The service limit state may control the footing design.

The bearing resistance for wall footings founded on <u>compacted structural fill or glacial till</u> shall be investigated using the factored bearing resistances provided in Figure 7.1 for footing widths ranging from 6 to 16 feet. This assumes a bearing resistance factor, φ_b, for spread footings on compacted fill of 0.45. A factored bearing resistance of 6 ksf may be used for preliminary footing sizing and to control settlements when analyzing the service limit state load combination. See Appendix C – Calculations for supporting documentation.

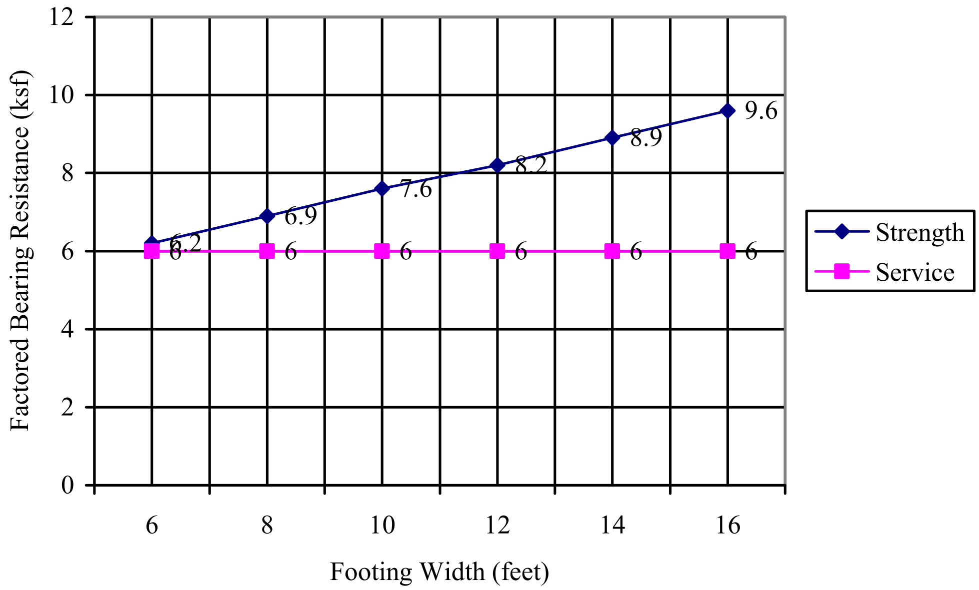


Figure 7-1 Factored Bearing Resistance for Spread Footings on Glacial Till

In no instance shall the factored bearing stress exceed the factored compressive resistance of the footing concrete or concrete fill, if permitted, which may be taken as 0.3 *f'c*. No footing shall be less than 2 feet wide regardless of the applied bearing pressure or bearing material.

A resistance factor of 1.0 shall be used to assess spread footing design at the service limit state, including: settlement and excessive horizontal movement. The overall stability of the foundation should be investigated at the Service I Load Combination and a resistance factor, ϕ, of 0.65.

The designer may assume Soil Type 4 (MaineDOT Bridge Design Guide (BDG) Section 3.6.1) for arch footing and wall backfill material soil properties. The backfill properties are as follows: ϕ = 32 degrees, γ = 125 pcf.

Independent wingwalls shall be designed as unrestrained meaning that they are free to rotate at the top in an active state of earth pressure. Earth loads shall be calculated using an active earth pressure coefficient, K_a, of 0.31, calculated using Rankine Theory for cantilever-type walls. The designer may assume BDG Soil Type 4 for backfill material soil properties. The backfill properties are as follows: ϕ = 32 degrees, γ = 125 pcf.

Additional lateral earth pressure due to live load surcharge may be required. The live load surcharge on wingwalls may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) taken from Table 7-6 below:

Retaining Wall Height (feet)	h_{eq} (feet)	
	Distance from wall pressure surface to edge of traffic: 0 feet	Distance from wall pressure surface to edge of traffic: >=1 foot
5	5.0	2.0
10	3.5	2.0
>=20	2.0	2.0

Table 7-6 Equivalent Height of Soil for Estimating Live Load Surcharge on Walls

Wingwall designs shall include a drainage system behind the arch or wall to intercept any groundwater. Drainage behind the structure shall be in accordance with Section 5.4.1.4 Drainage, of the MaineDOT BDG.

Backfill within 10 feet of the wingwalls, and side slope fill shall conform to Granular Borrow for Underwater Backfill - MaineDOT Specification 709.19. This gradation specifies 10 percent or less of the material passing the No. 200 sieve. This material is specified in order to reduce the amount of fines and to minimize frost action behind the structure.

7.3.2 MSE Walls

MSE walls may be constructed on top of the lower CIP walls above Q1.1. The Q1.1 elevation has been calculated to be 109.6 feet. MSE walls will be designed by a Professional Engineer subcontracted by the Contractor as a design-build item. The walls shall be designed in accordance with LRFD Article 11.10 and Special Provision 635, which is included in Appendix D at the end of this report. No utilities other than highway drainage are to be constructed within the reinforced zone unless access is provided to utilities without disrupting reinforcements and breakage or rupture of utility lines will to have a detrimental effect on the stability of the structure.

The MSE walls will be designed by the vendor for external and internal stability of the reinforced mass behind the facing. It is the responsibility of the MaineDOT to assure the MSE wall and approach embankment adequately meeting requirements for global stability. Special Provisions 635 also includes requirements for facing elements, reinforcing strips, backfill material and compaction, impervious membrane and drainage.

MSE walls shall be designed for all permanent and transient loads as specified in LRFD Articles 3.4.1 and 11.10.5.2. MSE walls should be investigated at the strength limit state for bearing capacity failure, lateral sliding, excessive loss of base contact, pullout of soil reinforcements and structural failure. Sliding computations shall assume a maximum allowable frictional coefficient of 0.58 = tan 30° at the soil base to foundation soil interface. A sliding resistance factor, φ_τ, of 1.0 shall be applied to the nominal sliding resistance of the MSE mass founded on soil.

For eccentricity design checks, the location of the resultant of the reaction forces shall be within the middle one-half (1/2) of the base width.

Bearing pressures should be computed using a uniform base distribution over the effective width. Preliminary bearing resistance calculations for the reinforced soil volume founded on compacted granular fill soils at the strength limit state resulted in a factored bearing resistance of 6.8 ksf for reinforced volumes with a reinforcement length of 14 feet. For bearing resistance recommendations for other reinforcement lengths, refer to Figure 7-2, below. This assumes a bearing resistance factor, $\varphi_{b,}$ for MSE walls of 0.65 per LRFD Table 11.5.6-1. Strict adherence to LRFD 10.6.3 will require this preliminary bearing resistance estimate to be reevaluated using the effective footing width by taking eccentricity into account. A factored bearing resistance of 6.0 ksf may be used when analyzing the service limit state and for preliminary MSE wall based sizing, assuming a resistance factor φ_b, of 1.0. See Appendix C – Calculations for supporting documentation.

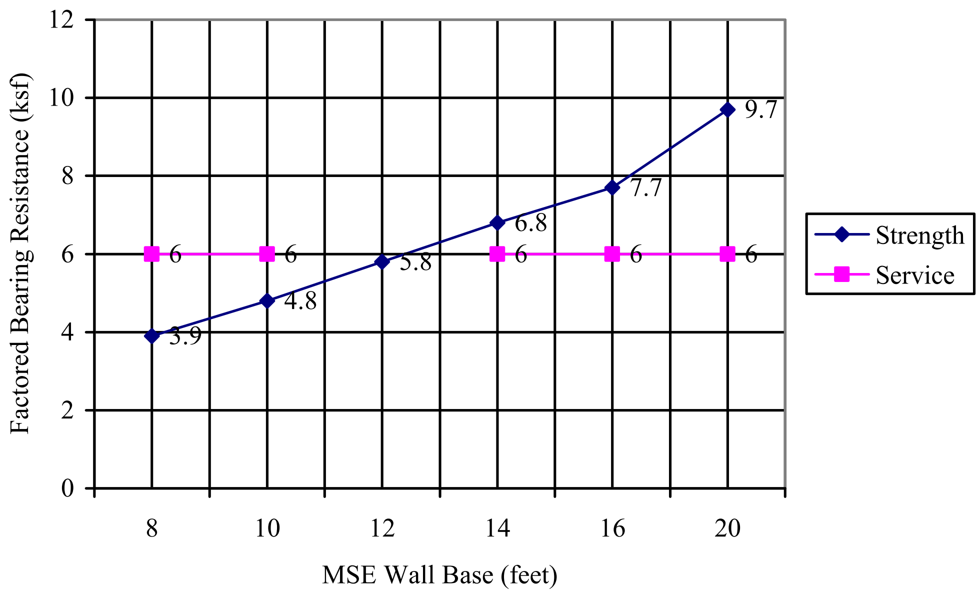


Figure 7-2 Factored Bearing Resistance for MSE Walls

Resistance factors for tensile resistance of steel reinforcements and connectors, and pullout resistance are provided in LRFD Table 11.5.6-1.

Earth pressures for external stability shall be calculated using an active earth pressure coefficient of K_a=0.31, calculated using Rankine Theory.

The live load surcharge on wingwalls may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) taken from Table 7-6.

A concrete leveling pad with a width of no less than 2.0 feet should be provided above the CIP wall to support the MSE wall panels. The leveling pad shall be located to provide an approximate 1.5 foot set-back of the MSE Wall panels from the face of the underlying CIP wall.

A resistance factor, ϕ, of 1.0 shall be used to assess the MSE volume design at the service limit state including: settlement, horizontal movement, overall global stability and wall movements/stability considering changes in soil conditions after scour due to the design flood event. The overall stability of the wall system should be investigated at the Service I load Combination with a resistance factor ϕ of 0.65. A resistance factor of 1.0 shall also be used to assess the MSE volume design at the extreme limit state. Extreme limit state design checks for MSE walls include bearing resistance, overturning (eccentricity), and internal stability. The extreme event load combinations are those related to ice loads, debris loads, the check flood for scour and certain hydraulic events.

The reinforcing length shall be uniform throughout the entire height of the wall. Backfill within the reinforced mass shall consist of Gravel Borrow meeting the requirements of MaineDOT 703.20 except that maximum particle size shall be 4 inches. Additional electrochemical requirements for the backfill within the reinforced mass are specified in Special Provision 636.

An impervious geomembrane consisting of low-permeability, 2-sided, texture HDPE with a minimum thickness of 40 mils shall be installed near the top of the reinforced soil zone to reduce the chance of water infiltration into the reinforce backfill. The membrane shall be bonded to the back of wall. The surface of the membrane shall be sloped to shed water infiltrating from the road surface above.

7.3.3 PCMG Walls

PCMG walls founded on fill soils may be used to retain approach fills above the ordinary high water (Q1.1) elevation. Should PCMG walls be used below Q1.1, the design flow velocity should be low and the potential for severe ice or wave action should be minimal. In general, PCMG wingwalls should be used only at stream crossings where the flow velocities are low, and the potential for severe ice or wave action is low. The walls shall be designed by a Professional Engineer subcontracted by the Contractor as a design-build item.

The bearing resistance for PCMG walls founded on a 6 by 12 inch leveling slab and the structural backfill behind the CIP walls shall be investigated at the strength limit state using factored loads and a factored bearing resistance provided in Figure 7-3, below. (The PCMG wall face should be set back approximately 1.5 feet from the face of the underlying CIP wall face.) Based on presumptive bearing resistance values, a factored bearing resistance of 6 ksf may be used to control settlement when analyzing the service limit state. The vertical stress may be calculated assuming a uniform distribution over the effective footing base as shown in LRFD Figure 11.6.3.2-1.

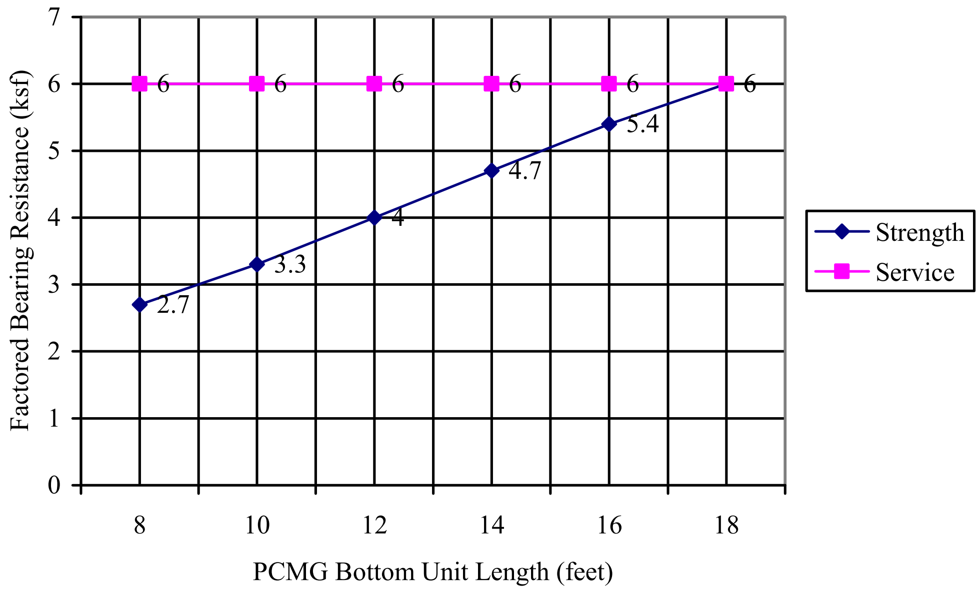


Figure 7-3 Factored Bearing Resistance for PCMG Walls

The live load surcharge may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) taken from Table 7-6.

For the lowest PCMG unit on soil the location of the resultant of the reaction forces at the strength limit state should be within the middle one-half (1/2) of the footing width.

The overall stability of the foundation should be investigated at the Service I Load Combination and a resistance factor, φ, of 0.65

Failure by sliding shall be investigated by the wall designer-supplier. A sliding resistance factor, φ_τ , of 0.90 shall be applied to the nominal sliding resistance of precast concrete wall segments founded on structural backfill. A sliding resistance factor of 0.90 shall be applied to the nominal sliding resistance of soil within the precast concrete units on granular bedding soils. Sliding computations for resistance to lateral loads shall assume a maximum frictional coefficient of 0.46 = 0.80 x tan 30° at the foundation soil to concrete unit interfaces and a maximum frictional coefficient of 0.58 = tan 30° at foundation soil to soil-infill interfaces. Recommended values of sliding frictional coefficients are based on LRFD Articles 11.11.4.2 and 10.6.3.4 and Table 10.5.5.2.2-1.

7.4 Global Stability of Embankments

It is recommended that all stumps, roots, organics, vegetation or other objectionable material be removed from the approach embankment plan area within 100 feet of the abutment locations.

Stability analyses to determine factors of safety against global failure of the walls and arch stem walls retaining the approach embankments at the arch stem walls were conducted. New approach fills with maximum heights on the order of 27 feet are proposed at the approaches to Abutments No. 1 and No. 2, respectively. The software used to conduct the stability analyses was GeoStudio Slope/W 6.20 which applied the Bishop method in the analyses. A minimum factor of safety of 1.5 is required in accordance with FHWA Soils and Foundations Manual, 2006.

Results of the slope stability analyses indicate that excavation and replacement of approximately 1 foot of the 3-foot layer of topsoil consisting of wet, soft to medium stiff clayey silt, some fine sand, little organics and roots, and the construction of approximately 30 foot high walls with a bottom of footing (BOF) at elevation 104 feet will achieve adequate factors of safety against global instability. Supporting calculations are provided in Appendix C – Calculations.

7.5 Settlement

The finished grade of the new alignment of Route 180 will require embankment fills approximately 27 feet high at the Greys Brook Bridge. Post-construction settlement of the bridge approach fills due to compression of the foundation soils is estimated to be approximately 2 to 3 inches and will occur over a long period of time. Any settlement of bridge abutments will be due to axial compression of the foundation piles and is anticipated to be less than 0.5 inch.

7.6 Frost Protection

Foundations placed on granular fill or native subgrade soils should be designed with an appropriate embedment for frost protection. According to BDG Figure 5-1, Maine Design Freezing Index Map, Ellsworth has a design freezing index of approximately 1400 F-degree days. An assumed water content of 10% was used for coarse grained soils above the water table. These components correlate to a frost depth of 6.6 feet. A similar analysis was performed using Modberg software by the US Army Cold Regions Research and Engineering Laboratory (CRREL). For the Modberg analysis, Ellsworth was assigned a design freezing index of approximately 1256 F-degree days. An assumed water content of 10% was used for coarse grained soils above the water table. These components correlate to a frost depth of 5.2 feet. We recommend foundations be designed with an embedment of 6.0 feet for frost protection. See Appendix C – Calculations for supporting documentation.

Riprap is not to be considered as contributing to the overall thickness of soils required for frost protection.

7.7 Scour and Riprap

Grain size analyses were performed on four soil samples taken from the glaciomarine deposit encountered in all four borings, for the purpose of generating grain size curves to determine parameters for scour analyses. The samples were assumed to be similar in nature to the soils likely to be exposed to scour conditions. The following streambed grain size parameters can be used in scour analyses:

- Average diameter of particle at 50% passing, D_{50} = 0.004 mm
- Average diameter of particle at 95% passing, D_{95} = 0.048 mm
- Soil Classification: AASHTO Soil Type: A-4

The consequences of changes in foundation conditions resulting from the design and check floods for scour shall be considered at the strength and extreme limit states, respectively. Design at the strength limit state should consider loss of lateral and vertical support due to scour with respect to factored strength limit state loads. Design at the extreme limit state should check that the nominal foundation resistance due to scour at the check flood event is no less than the extreme limit state loads. At the service limit state, the design shall limit movements and overall stability considering scour at the design flood.

Plain riprap conforming to Special Provisions 610 and 703 shall be placed at the toes of arch footings and wingwalls. Stone riprap shall conform to item number 703.26 of 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 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 nonwoven erosion control geotextile per Standard Details 610(02) through 610(04). The riprap layer shall be 3 feet thick.

7.8 Seismic Design Considerations

In conformance with LRFD Article 3.10.1, seismic analysis is not required for buried structures, expect where they cross active faults. There are no known active faults in Maine, therefore seismic analysis is not required.

7.9 Construction Considerations

Construction of the arch footings and wingwalls will require soil excavation and pile driving. Cofferdams and temporary lateral earth support systems will be required to permit construction of arch footings and wingwalls.

Water should be controlled by pumping from sumps. The contractor should maintain the excavation so that all foundations are constructed in the dry.

Removal (grubbing) of the clayey silt topsoil and over-excavation and replacement of 1 to 2 feet of the clayey silt deposit for construction of retaining wall footings at elevation 104 feet will expose potentially sensitive clayey silts. The clayey silt deposits at the site will be susceptible to disturbance and rutting as a result of exposure to water or construction traffic. The contractor shall protect the subgrade from exposure to water and any unnecessary construction traffic. If disturbance occurs, we recommend that the contractor remove and replace the disturbed materials with ¾ inch stone or compacted MaineDOT Standard

Specification 703.20, Gravel Borrow. Furthermore, the clayey silt soils may become saturated and water seepage may be encountered during construction. There may be localized sloughing and instability in some excavations and cut slopes. The contractor should control groundwater, surface water infiltration and soil erosion.

8.0 CLOSURE

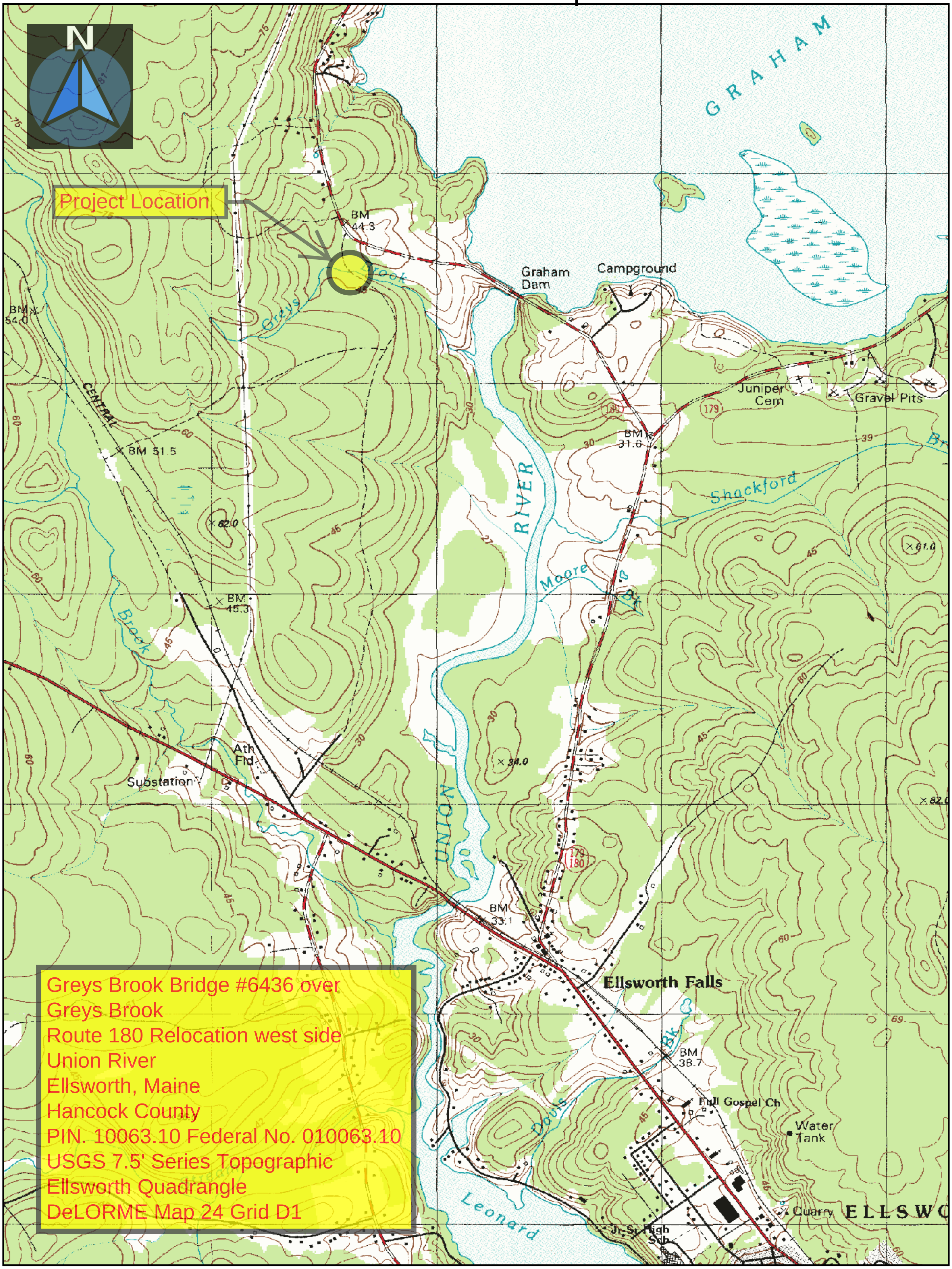
This report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed replacement of Greys Brook Bridge on the Relocated Route 180 in Ellsworth, Maine in accordance with generally accepted geotechnical and foundation engineering practices. No other intended 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 recommendations as appropriate to reflect the changes in design. Further, the analyses and recommendations are based in part upon limited soil explorations at discrete locations completed 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 and specifications in order that the earthwork and foundation recommendations may be properly interpreted and implemented in the design.

Sheets

Location Map

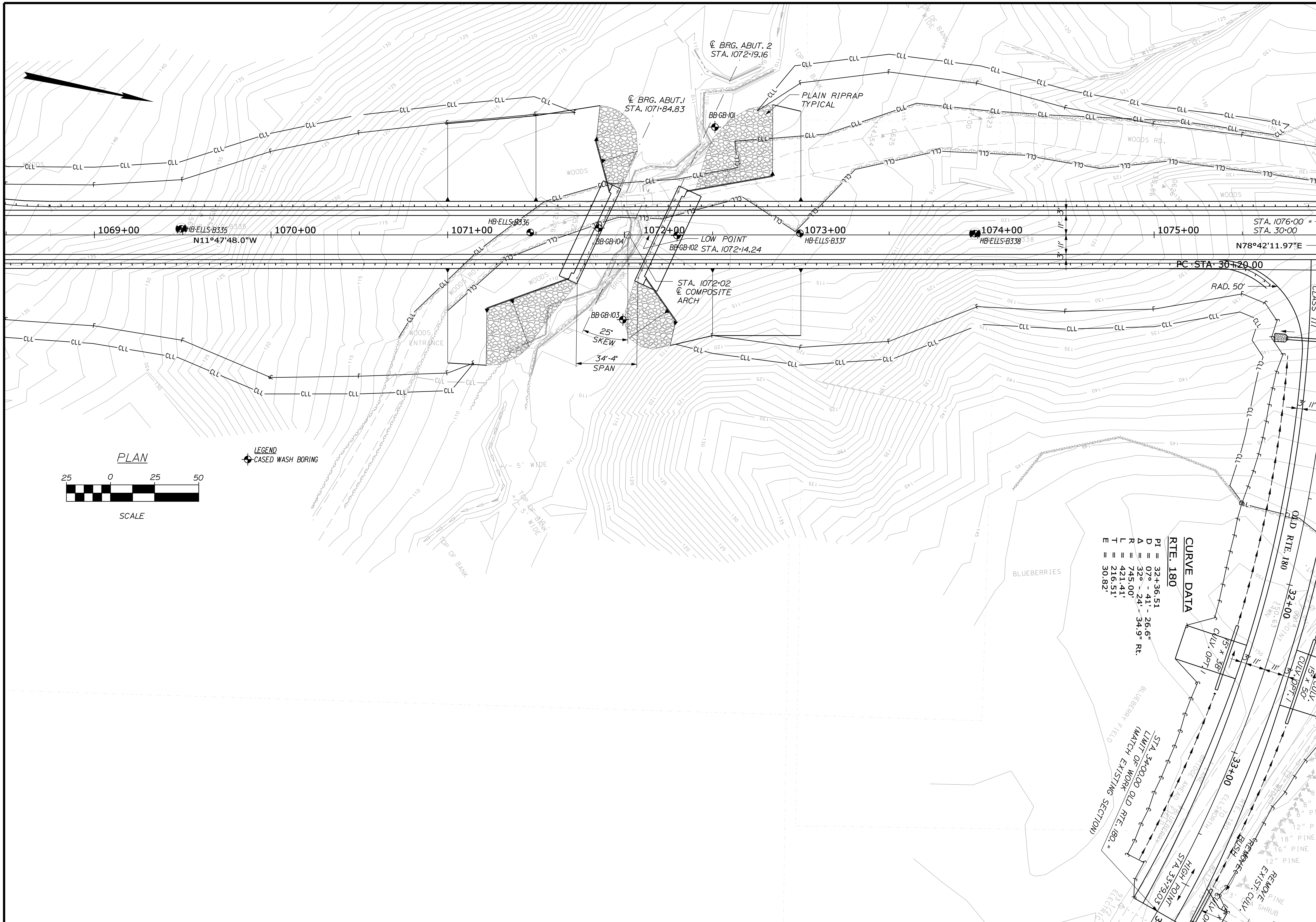
Sheet 1



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: ...\GEOTECH\MSTA\002_BLP GB1.dgn   Division: GEOTECH   Username: terry.white   Date:10/6/2011



STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

010063.10

BRIDGE PLANS

PIN
10063.10

BRIDGE NO. 6436

	BY	DATE
PROJ. MANAGER $PROJMANAGER$		
DESIGN-DETAILED	L.KRUSINSKI	T.WHITE	OCT 2011
CHECKED-REVIEWED		
DESIGN2-DETAILED2	$designer2$	$detailer2$	$projdate2$
DESIGN3-DETAILED3	$designer3$	$detailer3$	$projdate3$
REVISIONS 1		$revdate1$
REVISIONS 2		$revdate2$
REVISIONS 3	$revision3$	$revdate3$
REVISIONS 4	$revision4$	$revdate4$
FIELD CHANGES	$fieldchange$	$fielddate$

SIGNATURE

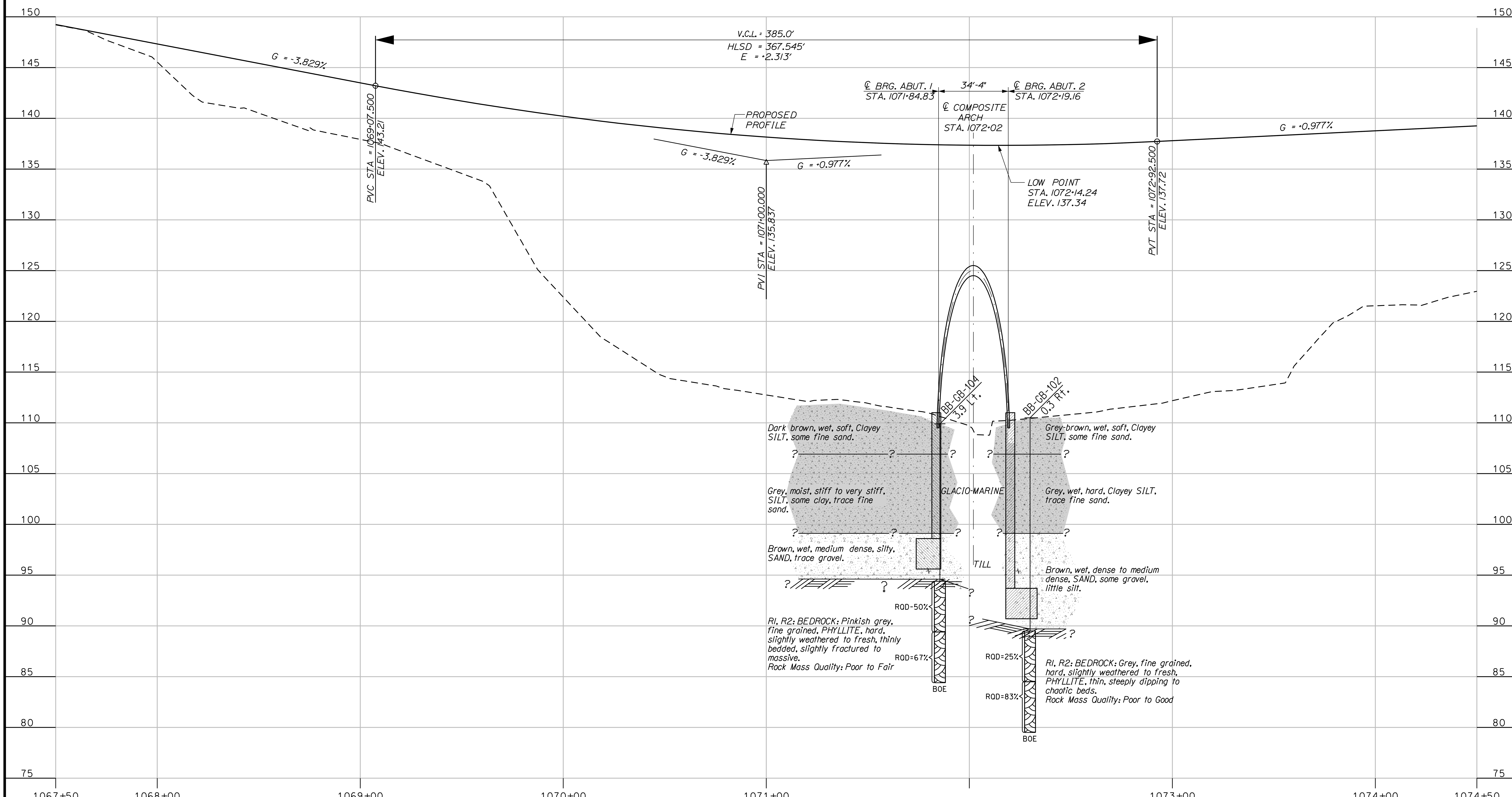
P.E. NUMBER

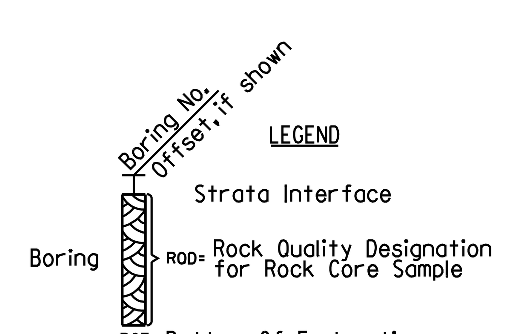
DATE

GREYS BROOK BRIDGE
GREYS BROOK
ELLSWORTH HANCOCK COUNTY

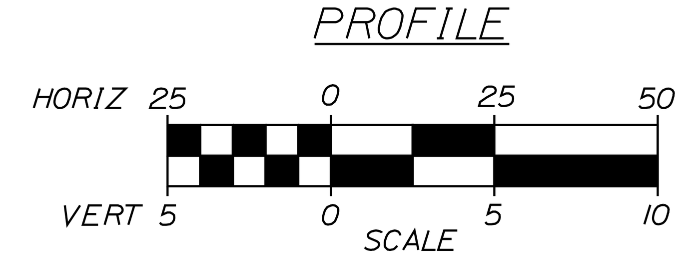
BORING LOCATION PLAN

SHEET NUMBER
2
OF 5





PROFILE



Notes:
1. 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.
2. Soil statagraphy for only borings BB-GB-104 and BB-GB-102 shown for clarity. Refer to Sheet No. 4 for generalized soil strata for borings BB-GB-101 and BB-GB-103.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

010063.10

PIN 10063.10

BRIDGE NO. 6436

BRIDGE PLANS

	BY	DATE
PROJ. MANAGER $PROJMANAGER$		
DESIGN-DETAILED	L.KRUSINSKI	T.WHITE	OCT 2011
CHECKED-REVIEWED		
DESIGN2-DETAILED2	$designer2$	$detailer2$	$projdate2$
DESIGN3-DETAILED3	$designer3$	$detailer3$	$projdate3$
REVISIONS 1		$revdate1$
REVISIONS 2	$revision2$	$revdate2$
REVISIONS 3	$revision3$	$revdate3$
REVISIONS 4	$revision4$	$revdate4$
FIELD CHANGES	$fieldchange$	$fielddate$

SIGNATURE
P.E. NUMBER
DATE

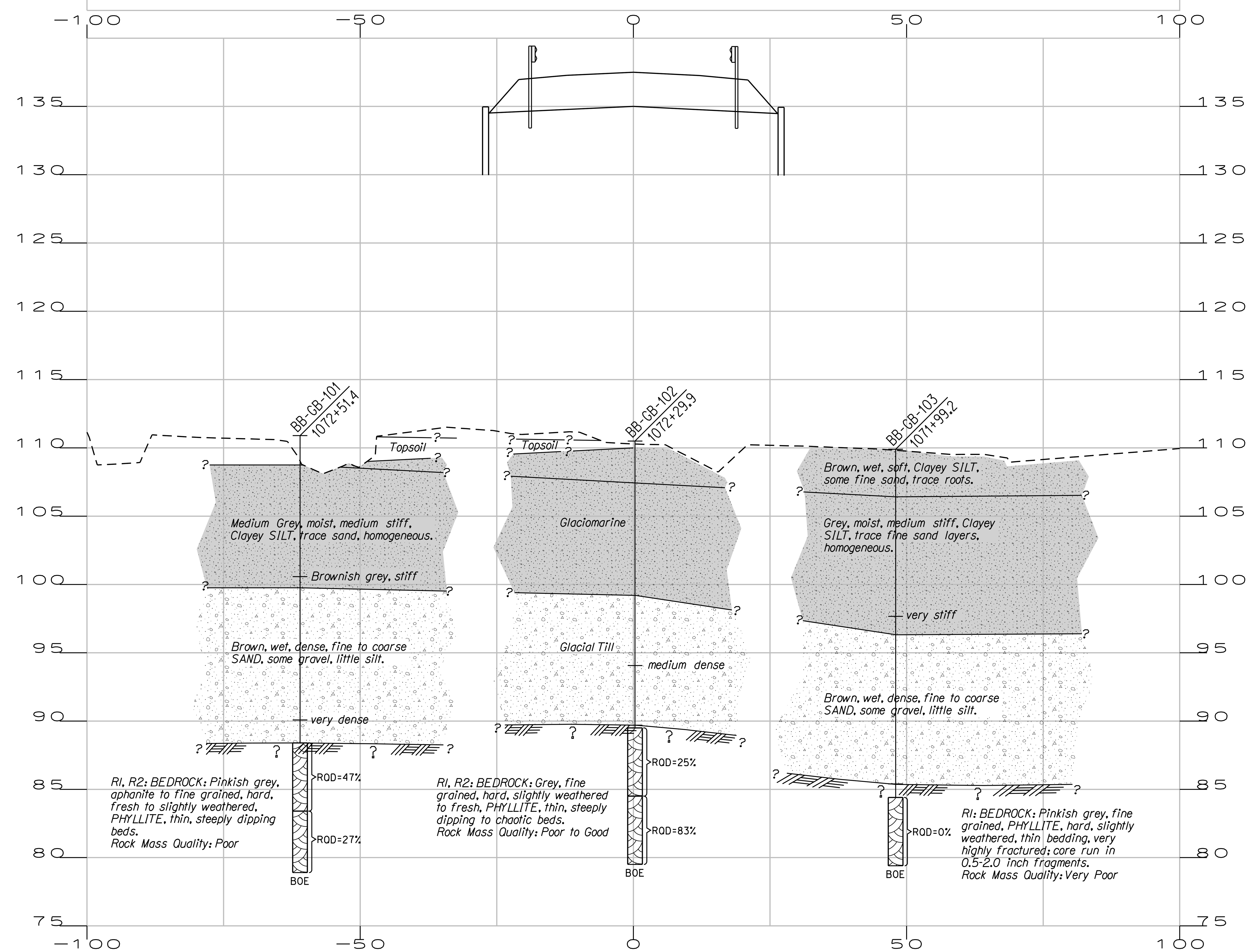
GREYS BROOK BRIDGE
GREYS BROOK
ELLSWORTH HANCOCK COUNTY

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
3
OF 5

Filename: ...\geotech\msta\004_ ISTS GB1.dgn Division: GEOTECH Username: terry.white Date:10/6/2011

SKEWED TRANSVERSE SECTION - STA. 1072+22.15 25° SKEW



LEGEND

Boring No. / Station

Pavement Thickness if applicable

Strata Interface

Boring — RQD= Rock Quality Designation for Rock Core Sample

BOE= Bottom Of Exploration

HORIZ 25 0 25 50

VERT 10 0 10 20

SCALE

Notes: 1. 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.

2. Soil statagraphy for only borings BB-GB-101, BB-GB-102 and BB-GB-103 shown for clarity. Refer to Sheet No. 3 for generalized soil strata for boring BB-GB-104.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

PROJECT NO. 10063.10

PIN 10063.10

BRIDGE NO. 6436

BRIDGE PLANS

	BY	DATE
PROJ. MANAGER	ERNIE MARTIN	
DESIGN-DETAILED	L.KRUSINSKI	T.WHITE	OCT 2011
CHECKED-REVIEWED		
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

GREYS BROOK BRIDGE
GREYS BROOK
ELLSWORTH HANCOCK COUNTY

INTERPRETIVE SOIL
TRANSVERSE SECTION

SHEET NUMBER

4

OF 5

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

Project: Greys Brook Bridge #6436 carries Route 180 over Greys Brook		Location: Ellsworth, Maine		Boring No.: BB-GB-104	WIN: 10063.10
Driller:	Northern Test Boring	Elevation (ft.)	109.9	Auger ID/OD:	5" Solid Stem
Operator:	Nick/Ryan	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	Diedrich D-50	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	8/25/10; 07:00-11:00	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	1071+85.5, 3.9 Lt.	Casing ID/OD:	HW	Water Level*:	4.0 ft bgs.
Hammer Efficiency Factor: 0.713		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.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0	1D	24/18	0.00 - 2.00	1/1/1/1	2	2	SSA	109.60	TOPSOIL, (Sod and Roots). 0.30 Dark brown, wet, soft, clayey SILT, some fine sand.	
3								106.90	3.00	
5	MV/2D	24/20	5.00 - 7.00	4/6/9/13	15	18			Failed 16x32 mm vane attempt, would not push. Grey, moist, very stiff to stiff, SILT, some clay, trace fine sand, no structure, some mottling otherwise homogeneous.	G#237516 A-4, CL WC=20.1% LL=29 PL=20 PI=9
10	MV/3D	24/18	10.00 - 12.00	5/6/6/12	12	14	6		Failed 16x32 mm vane attempt, would not push.	G#237517 A-4, SM WC=22.0%
11							27	98.90	11.00 Brown, wet, medium dense, silty, fine to coarse SAND, trace gravel. (Till). Roller Coned ahead to 15.0 ft bgs.	
12							52			
13							78			
14							73			
15	MD R1	3.6/0 60/52	15.00 - 15.30 15.50 - 20.50	50(3.6") RQD = 50%	---		40 NQ-2	94.60 94.40	Failed sample attempt. Roller Coned ahead to 15.5 ft bgs. 15.30 Top of Bedrock at Elev. 94.6 ft. 15.50 R1:Bedrock: Pinkish grey, fine grained PHYLITTE, hard, fresh to slightly discolored. Thinly bedded, frequent quartz veins, drill breaks along bedding at steep angles and cross bedding (low angles joints). Slightly fractured to massive. Rock Mass Quality: Poor. Ellsworth Formation. R1:Core Times (min:sec) 15.5-16.5 ft (4:35) 16.5-17.5 ft (4:50) 17.5-18.5 ft (3:55) 18.5-19.5 ft (4:40) 19.5-20.5 ft (4:00) 87% Recovery	
20	R2	60/60	20.50 - 25.50	RQD = 67%					R2:Bedrock: Pinkish grey, fine grained PHYLITTE, slightly weathered to fresh. Thin bedding, slightly fractured to massive. Rock Mass Quality: Fair. Ellsworth Formation. R2:Core Times (min:sec) 20.5-21.5 ft (-:--) 21.5-22.5 ft (-:--) 22.5-23.5 ft (-:--) 23.5-24.5 ft (-:--) 24.5-25.5 ft (-:--) 100% Recovery	
25								84.40	25.50 Bottom of Exploration at 25.50 feet below ground surface.	

Remarks:
Auto Hammer #185

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.: BB-GB-104

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

Project: Greys Brook Bridge #6436 carries Route 180 over Greys Brook		Location: Ellsworth, Maine		Boring No.: BB-GB-103	WIN: 10063.10
Driller:	Northern Test Boring	Elevation (ft.)	109.9	Auger ID/OD:	5" Solid Stem
Operator:	Nick/Ryan	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	Diedrich D-50	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	8/24/10; 07:00-11:30	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	1071+99.2, 48.0 Rt.	Casing ID/OD:	HW	Water Level*:	4.0 ft bgs.
Hammer Efficiency Factor: 0.713		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.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0	1D	24/5	0.00 - 2.00	1/1/1/1	2	2	SSA	109.60	TOPSOIL, (Sod and Roots). 0.30 Brown, wet, soft, clayey SILT, some fine sand, little organics. (Topsoil subunit).	
3								106.90	3.00	
5	MV/2D	24/20	5.00 - 7.00	3/3/4/3	7	8			Failed 16x32 mm vane attempt, would not push. Grey, moist, medium stiff, clayey SILT, trace fine sand layers, homogeneous, no structure. (Glaciomarine).	
10	V1 3D	24/18	10.00 - 10.11 10.00 - 12.00	Su=2986/2358 psf WOH/1/2/2	3	4	ªHP		16x32 mm vane raw torque readings: V1: 19.0/15.0 in-lbs ªHP-Hydraulic Push Grey, wet, very stiff, clayey SILT, trace fine sand, homogeneous. (Glaciomarine).	G#237514 A-4, CL WC=30.1% LL=29 PL=20 PI=9
11							ªHP			
12							ªHP			
13							51	96.90	13.00	
14							68			
15	4D	24/18	15.00 - 17.00	9/16/16/14	32	38	12		Brown, wet, dense, fine to coarse SAND, some gravel, little silt. (Till).	G#237515 A-1-b, SW-SM WC=10.7%
16							76			
17							64			
18							79			
19							95			
20	5D	24/10	20.00 - 22.00	13/20/20/14	40	48	5		Similar to above. Roller Coned ahead to 25.5 ft bgs.	
21							6			
22							59			
23							97			
24							125	85.40	24.50 Top of Bedrock at Elev. 85.4 ft.	
25	R1	60/24	25.50 - 30.50	RQD = 0%			NQ-2	84.40	25.50 R1:Bedrock: Pinkish grey, fine grained PHYLLITE, hard, slightly weathered, thin bedding, no orientation to jointing evident, very highly fractured, core run is in 1/2 to 2-inch fragments. Rock Mass Quality: very poor. Ellsworth Formation. R1:Core Times (min:sec) 25.5-26.5 ft (4:30) 26.5-27.5 ft (3:00) 27.5-28.5 ft (2:25) 28.5-29.5 ft (3:20) 29.5-30.5 ft (4:15) 40% Recovery	
30								79.40	30.50 Bottom of Exploration at 30.50 feet below ground surface.	

Remarks:
Auto Hammer #185

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.: BB-GB-103

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

Project: Greys Brook Bridge #6436 carries Route 180 over Greys Brook		Location: Ellsworth, Maine		Boring No.: BB-GB-102	WIN: 10063.10
Driller:	Northern Test Boring	Elevation (ft.)	110.5	Auger ID/OD:	5" Solid Stem
Operator:	Nick/Ryan	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	Diedrich D-50	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	8/24/10; 12:30-17:00	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	1072+29.9, 0.3 Rt.	Casing ID/OD:	HW	Water Level*:	3.6 ft bgs.
Hammer Efficiency Factor: 0.713		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.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0	1D	24/10	0.00 - 2.00	1/1/1/1	2	2	SSA	110.20	TOPSOIL (Sod and Roots). 0.30 Grey-brown, wet, soft, clayey SILT, some fine sand, trace roots.	
3								107.50	3.00	
5	V1 2D	24/20	5.00 - 5.11 5.00 - 7.00	Su=>5658 psf 3/4/6/7	10	12			16x32 mm vane raw torque readings: V1:>36.0 in-lbs Grey, wet, hard, clayey SILT, trace fine sand.	G#237512 A-4, CL WC=21.8% LL=30 PL=20 PI=10
10	3D V2	24/16	10.00 - 12.00 10.00 - 10.11	9/13/14/18 Su=>5658 psf	27	32	49		16x32 mm vane raw torque readings: V2:>36.0 in-lbs	
11							58	99.50	11.00 Brown, wet, dense, fine to coarse SAND, some gravel, little silt. (Till).	
12							74			
13							65			
14							65			
15	4D	24/15	15.00 - 17.00	11/10/11/13	21	25	49		Brown, wet, medium dense, fine to coarse SAND, some gravel, little silt. (Till).	G#237513 A-1-b, SM WC=11.4%
16							71			
17							80			
18							69			
19							68			
20	5D	9.6/6	20.00 - 20.80	19/50(3.6")	---		50	89.70 89.50	Similar to above. 20.80 Top of Bedrock at Elev. 89.7 ft. Roller Coned ahead to 21.0 ft bgs. 21.00	
21	R1	60/47	21.00 - 26.00	RQD = 25%			NQ-2		R1:Bedrock: Grey, fine grained, hard, slightly weathered, calcareous PHYLLITE, steepley dipping to chaotic bedding, limited quartz inclusions/veins, highly fractured. Rock Mass Quality: Poor. Ellsworth Formation. R1:Core Times (min:sec) 21.0-22.0 ft (4:00) 22.0-23.0 ft (4:20) 23.0-24.0 ft (5:15) 24.0-25.0 ft (5:45) 25.0-26.0 ft (6:10) 78% Recovery	
26	R2	60/52	26.00 - 31.00	RQD = 83%					R2:Bedrock: Grey, fine grained, hard, fresh, PHYLITTE, typically thinly bedded in steeply dipping, close, beds, massive. Rock Mass Quality: Good. Ellsworth Formation. R2:Core Times (min:sec) 26.0-27.0 ft (5:00) 27.0-28.0 ft (5:15) 28.0-29.0 ft (5:00) 29.0-30.0 ft (5:00) 30.0-31.0 ft (5:30) 87% Recovery	
31								79.50	31.00 Bottom of Exploration at 31.00 feet below ground surface.	

Remarks:
Auto Hammer #185

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.: BB-GB-102

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

Project: Greys Brook Bridge #6436 carries Route 180 over Greys Brook		Location: Ellsworth, Maine		Boring No.: BB-GB-101	WIN: 10063.10
Driller:	Northern Test Boring	Elevation (ft.)	110.9	Auger ID/OD:	5" Solid Stem
Operator:	Voltolina/Strattard	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	Be Schonewald	Rig Type:	Diedrich D-50	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	8/23/10; 12:00-16:40	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	1072+51.4, 61.0 Lt.	Casing ID/OD:	HW	Water Level*:	3.2 ft bgs.
Hammer Efficiency Factor: 0.713		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.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0	1D	24/6	0.00 - 2.00	2/3/2/2	5	6	SSA		Brown, dry, medium stiff, sandy SILT, trace roots (Topsoil Subsoil)	
2								108.90	2.00 Top of silt-clay at 2.0 ft bgs based on auger flight.	
5	2D	24/24	5.00 - 7.00	3/3/4/6	7	8			Medium grey, moist, medium stiff, clayey SILT, trace sand, no structure, homogeneous except one 2" thick mottled seam at 6.0 ft. (Glaciomarine Silt-Clay).	G#237510 A-6, CL WC=22.6% LL=31 PL=19 PI=12
10	3D/AB	24/24	10.00 - 12.00	2/3/6/7	9	11	---		3D/A (10.0-11.0 ft) Brownish grey, stiff, moist, SILT and CLAY.	
11							---	99.90	11.00 3D/B (11.0-11.7 ft) Brown, wet, fine to medium SAND, trace silt, (Till).	
12							---			
13							89			
14							82			
15	4D	24/9	15.00 - 17.00	12/11/19/27	30	36	ªWA		Brown, wet, dense, fine to coarse SAND, some gravel, little silt, (Till). ªWashed Ahead	G#237511 A-1-b, SM WC=9.3%
20	5D	16.8/5	20.00 - 21.40	10/21/50(4.8")	---				Light brownish grey, very dense, fine to medium SAND, some gravel, trace to little silt, trace coarse sand, (Till).	
22.5	R1	60/54	22.50 - 27.50	RQD = 47%			NQ-2	88.40	22.50 Top of Bedrock at Elev. 88.4 ft. R1:Bedrock: Pinkish grey, fine grained, hard, fresh to slightly weathered, calcareous feldspar and biotite-rich PHYLLITE. Typially thinly bedded with steeply dipping beds. Limited quartz inclusions veins. Close, low and high angle, stepped and planar, rough, fresh to slightly discolored, open breaks along beding (high angle) and cross bedding (low angle, possible joints). Rock Mass Quality: Poor. (Ellsworth Formation). R1:Core Times (min:sec) 22.5-23.5 ft (3:30) 23.5-24.5 ft (3:40) 24.5-25.5 ft (4:40) 25.5-26.5 ft (4:05) 26.5-27.5 ft (4:15) 90% Recovery	
27.5	R2	54/48	27.50 - 32.00	RQD = 27%					R2:Bedrock: Upper 1 ft same as R1 with significant quartz inclusions. Lower 3.5 ft predominantly quartz vein with alkali (? green) feldspar. Bottom of core highly fractured, partially decomposed. (Ellsworth Formation). Rock Mass Quality: Poor. R2:Core Times (min:sec) 27.5-28.5 ft (5:00) 28.5-29.5 ft (7:00) 29.5-30.5 ft (7:55) 30.5-31.5 ft (8:45) 31.5-32.0 ft (-:--) 89% Recovery	
32								78.90	32.00 Bottom of Exploration at 32.00 feet below ground surface.	

Remarks:
Auto Hammer #185

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.: BB-GB-101

Filename: ...\msta\005_ BORING LOGS GB1.dgn   Division: GEOTECH   Username: terry.white   Date:10/6/2011

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
PROJECT NO. 10063.10
BRIDGE NO. 6436   PIN 10063.10   BRIDGE PLANS

		BY	DATE
PROJ. MANAGER	ERNIE MARTIN		
DESIGN-DETAILED	L.KRUSINSKI	T.WHITE	OCT 2011
CHECKED-REVIEWED			
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE
P.E. NUMBER
DATE

GREYS BROOK BRIDGE
GREYS BROOK
ELLSWORTH   HANCOCK COUNTY

BORING LOGS

SHEET NUMBER
5
OF 5

Appendix A

Boring Logs

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Greys Brook Bridge #6436 carries Route 180 over Greys Brook
Location: Ellsworth, Maine

Boring No.: BB-GB-101
WIN: 10063.10

Driller:	Northern Test Boring	**Elevation (ft.)**	110.9	**Auger ID/OD:**	5" Solid Stem
Operator:	Voltolina/Strattard	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Be Schonewald	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/23/10; 12:00-16:40	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	1072+51.4, 61.0 Lt.	**Casing ID/OD:**	HW	**Water Level*:**	3.2 ft bgs.
Hammer Efficiency Factor:	0.713	**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	1D	24/6	0.00 - 2.00	2/3/2/2	5	6	SSA			Brown, dry, medium stiff, sandy SILT, trace roots (Topsoil Subsoil)	
								108.90		2.00 — Top of silt-clay at 2.0 ft bgs based on auger flight.	
5	2D	24/24	5.00 - 7.00	3/3/4/6	7	8				Medium grey, moist, medium stiff, clayey SILT, trace sand, no structure, homogeneous except one 2" thick mottled seam at 6.0 ft, (Glaciomarine Silt-Clay).	G#237510 A-6, CL WC=22.6% LL=31 PL=19 PI=12
10	3D/AB	24/24	10.00 - 12.00	2/3/6/7	9	11	---			3D/A (10.0-11.0 ft) Brownish grey, stiff, moist, SILT and CLAY.	
							---	99.90		11.00 — 3D/B (11.0-11.7 ft) Brown, wet, fine to medium SAND, trace silt, (Till).	

							89				
							82				
15	4D	24/9	15.00 - 17.00	12/11/19/27	30	36	aWA			Brown, wet, dense, fine to coarse SAND, some gravel, little silt, (Till). aWashed Ahead	G#237511 A-1-b, SM WC=9.3%
20	5D	16.8/5	20.00 - 21.40	10/21/50(4.8")	---					Light brownish grey, very dense, fine to medium SAND, some gravel, trace to little silt, trace coarse sand, (Till).	
	R1	60/54	22.50 - 27.50	RQD = 47%			NQ-2	88.40		22.50 — Top of Bedrock at Elev. 88.4 ft. R1:Bedrock: Pinkish grey, fine grained, hard, fresh to slightly weathered, calcareous feldspar and biotite-rich PHYLLITE. Typially thinly bedded with steeply dipping beds. Limited quartz inclusions veins. Close, low and high angle, stepped and planar, rough, fresh to slightly discolored,	
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: Greys Brook Bridge #6436 carries Route 180 over Greys Brook Location: Ellsworth, Maine	Boring No.: BB-GB-101 WIN: 10063.10

Driller:	Northern Test Boring	**Elevation (ft.)**	110.9	**Auger ID/OD:**	5" Solid Stem
Operator:	Voltolina/Strattard	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Be Schonewald	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/23/10; 12:00-16:40	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	1072+51.4, 61.0 Lt.	**Casing ID/OD:**	HW	**Water Level*:**	3.2 ft bgs.
Hammer Efficiency Factor:	0.713	**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.
25										open breaks along beding (high angle) and cross bedding (low angle, possible joints). Rock Mass Quality: Poor. (Ellsworth Formation). R1:Core Times (min:sec) 22.5-23.5 ft (3:30) 23.5-24.5 ft (3:40) 24.5-25.5 ft (4:40) 25.5-26.5 ft (4:05) 26.5-27.5 ft (4:15) 90% Recovery	
	R2	54/48	27.50 - 32.00	RQD = 27%						R2:Bedrock: Upper 1 ft same as R1 with significant quartz inclusions. Lower 3.5 ft predominantly quartz vein with alkali (? green) feldspar. Bottom of core highly fractured, partially decomposed. (Ellsworth Formation). Rock Mass Quality: Poor. R2:Core Times (min:sec) 27.5-28.5 ft (5:00) 28.5-29.5 ft (7:00) 29.5-30.5 ft (7:55) 30.5-31.5 ft (8:45) 31.5-32.0 ft (-:--) 89% Recovery	
30											
								78.90		32.00 **Bottom of Exploration at 32.00 feet below ground surface.**	
35											
40											
45											
50											

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:** Greys Brook Bridge #6436 carries Route 180 over Greys Brook **Location:** Ellsworth, Maine	**Boring No.:** BB-GB-102 **WIN:** 10063.10

Driller:	Northern Test Boring	**Elevation (ft.)**	110.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Nick/Ryan	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/24/10; 12:30-17:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	1072+29.9, 0.3 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	3.6 ft bgs.
Hammer Efficiency Factor:	0.713	**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	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/10	0.00 - 2.00	1/1/1/1	2	2	SSA	110.20		TOPSOIL (Sod and Roots). 0.30	
										Grey-brown, wet, soft, clayey SILT, some fine sand, trace roots.	
								107.50		3.00	
5	V1	24/20	5.00 - 5.11	Su=>5658 psf	10	12				16x32 mm vane raw torque readings: V1:>36.0 in-lbs	G#237512 A-4, CL WC=21.8% LL=30 PL=20 PI=10
	2D		5.00 - 7.00	3/4/6/7						Grey, wet, hard, clayey SILT, trace fine sand.	
10	3D	24/16	10.00 - 12.00	9/13/14/18	27	32	49			16x32 mm vane raw torque readings: V2:>36.0 in-lbs	
	V2		10.00 - 10.11	Su=>5658 psf				99.50		11.00	
							58			Brown, wet, dense, fine to coarse SAND, some gravel, little silt. (Till).	
							74				
							65				
							65				
15	4D	24/15	15.00 - 17.00	11/10/11/13	21	25	49			Brown, wet, medium dense, fine to coarse SAND, some gravel, little silt. (Till).	G#237513 A-1-b, SM WC=11.4%
							71				
							80				
							69				
							68				
20	5D	9.6/6	20.00 - 20.80	19/50(3.6")	---		50			Similar to above.	
								89.70		20.80	
	R1	60/47	21.00 - 26.00	RQD = 25%			NQ-2	89.50		Top of Bedrock at Elev. 89.7 ft. Roller Coned ahead to 21.0 ft bgs. 21.00	
										R1:Bedrock: Grey, fine grained, hard, slightly weathered, calcareous PHYLLITE, steepley dipping to chaotic bedding, limited quartz inclusions/veins, highly fractured. Rock Mass Quality: Poor. Ellsworth Formation.	
25										R1:Core Times (min:sec)	

Remarks:

Auto Hammer #185

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: Greys Brook Bridge #6436 carries Route 180 over Greys Brook
Location: Ellsworth, Maine

Boring No.: BB-GB-102
WIN: 10063.10

Driller:	Northern Test Boring	**Elevation (ft.)**	110.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Nick/Ryan	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/24/10; 12:30-17:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	1072+29.9, 0.3 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	3.6 ft bgs.
Hammer Efficiency Factor:	0.713	**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.
25										21.0-22.0 ft (4:00) 22.0-23.0 ft (4:20) 23.0-24.0 ft (5:15) 24.0-25.0 ft (5:45) 25.0-26.0 ft (6:10) 78% Recovery	
	R2	60/52	26.00 - 31.00	RQD = 83%						R2:Bedrock: Grey, fine grained, hard, fresh, PHYLITTE, typically thinly bedded in steeply dipping, close, beds, massive. Rock Mass Quality: Good. Ellsworth Formation. R2:Core Times (min:sec) 26.0-27.0 ft (5:00) 27.0-28.0 ft (5:15) 28.0-29.0 ft (5:00)	
30								79.50		29.0-30.0 ft (5:00) 30.0-31.0 ft (5:30) 87% Recovery 31.00 **Bottom of Exploration at 31.00 feet below ground surface.**	
35											
40											
45											
50											

Remarks:

Auto Hammer #185

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: Greys Brook Bridge #6436 carries Route 180 over Greys Brook
Location: Ellsworth, Maine

Boring No.: BB-GB-103
WIN: 10063.10

Driller:	Northern Test Boring	**Elevation (ft.)**	109.9	**Auger ID/OD:**	5" Solid Stem
Operator:	Nick/Ryan	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/24/10; 07:00-11:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	1071+99.2, 48.0 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	4.0 ft bgs.
Hammer Efficiency Factor:	0.713	**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	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/5	0.00 - 2.00	1/1/1/1	2	2	SSA	109.60		TOPSOIL, (Sod and Roots). —0.30— Brown, wet, soft, clayey SILT, some fine sand, little organics. (Topsoil subunit).	
								106.90		—3.00—	
5	MV/2D	24/20	5.00 - 7.00	3/3/4/3	7	8				Failed 16x32 mm vane attempt, would not push. Grey, moist, medium stiff, clayey SILT, trace fine sand layers, homogeneous, no structure. (Glaciomarine).	
10	V1 3D	24/18	10.00 - 10.11 10.00 - 12.00	Su=2986/2358 psf WOH/1/2/2	3	4	aHP			16x32 mm vane raw torque readings: V1: 19.0/15.0 in-lbs aHP-Hydraulic Push Grey, wet, very stiff, clayey SILT, trace fine sand, homogeneous. (Glaciomarine).	G#237514 A-4, CL WC=30.1% LL=29 PL=20 PI=9
							aHP				
							aHP				
							51	96.90		—13.00—	
							68				
15	4D	24/18	15.00 - 17.00	9/16/16/14	32	38	12			Brown, wet, dense, fine to coarse SAND, some gravel, little silt. (Till).	G#237515 A-1-b, SW-SM WC=10.7%
							76				
							64				
							79				
							95				
20	5D	24/10	20.00 - 22.00	13/20/20/14	40	48	5			Similar to above. Roller Coned ahead to 25.5 ft bgs.	
							6				
							59				
							97				
25							125	85.40		—24.50—	

Remarks:

Auto Hammer #185

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: Greys Brook Bridge #6436 carries Route 180 over Greys Brook
Location: Ellsworth, Maine

Boring No.: BB-GB-103
WIN: 10063.10

Driller:	Northern Test Boring	**Elevation (ft.)**	109.9	**Auger ID/OD:**	5" Solid Stem
Operator:	Nick/Ryan	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/24/10; 07:00-11:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	1071+99.2, 48.0 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	4.0 ft bgs.
Hammer Efficiency Factor:	0.713	**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.
25	R1	60/24	25.50 - 30.50	RQD = 0%			NQ-2	84.40		Top of Bedrock at Elev. 85.4 ft. 25.50 R1:Bedrock: Pinkish grey, fine grained PHYLLITE, hard, slightly weathered, thin bedding, no orientation to jointing evident, very highly fractured, core run is in 1/2 to 2-inch fragments. Rock Mass Quality: very poor. Ellsworth Formation. R1:Core Times (min:sec) 25.5-26.5 ft (4:30) 26.5-27.5 ft (3:00) 27.5-28.5 ft (2:25) 28.5-29.5 ft (3:20) 29.5-30.5 ft (4:15) 40% Recovery	
30								79.40		30.50 **Bottom of Exploration at 30.50 feet below ground surface.**	
35											
40											
45											
50											

Remarks:

Auto Hammer #185

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: Greys Brook Bridge #6436 carries Route 180 over Greys Brook
Location: Ellsworth, Maine

Boring No.: BB-GB-104
WIN: 10063.10

Driller:	Northern Test Boring	**Elevation (ft.)**	109.9	**Auger ID/OD:**	5" Solid Stem
Operator:	Nick/Ryan	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/25/10; 07:00-11:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	1071+85.5, 3.9 Lt.	**Casing ID/OD:**	HW	**Water Level*:**	4.0 ft bgs.
Hammer Efficiency Factor:	0.713	**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	1D	24/18	0.00 - 2.00	1/1/1/1	2	2	SSA	109.60		TOPSOIL, (Sod and Roots).	
										0.30 Dark brown, wet, soft, clayey SILT, some fine sand.	
								106.90		3.00	
5	MV/2D	24/20	5.00 - 7.00	4/6/9/13	15	18				Failed 16x32 mm vane attempt, would not push. Grey, moist, very stiff to stiff, SILT, some clay, trace fine sand, no structure, some mottling otherwise homogeneous.	G#237516 A-4, CL WC=20.1% LL=29 PL=20 PI=9
10	MV/3D	24/18	10.00 - 12.00	5/6/6/12	12	14	6			Failed 16x32 mm vane attempt, would not push.	G#237517 A-4, SM WC=22.0%
							27	98.90		11.00 Brown, wet, medium dense, silty, fine to coarse SAND, trace gravel. (Till). Roller Coned ahead to 15.0 ft bgs.	
							52				
							78				
							73				
15	MD	3.6/0	15.00 - 15.30	50(3.6")	---		40	94.60		Failed sample attempt. Roller Coned ahead to 15.5 ft bgs.	
	R1	60/52	15.50 - 20.50	RQD = 50%			NQ-2	94.40		15.30 Top of Bedrock at Elev. 94.6 ft. 15.50 R1:Bedrock: Pinkish grey, fine grained PHYLITTE, hard, fresh to slightly discolored. Thinly bedded, frequent quartz veins, drill breaks along bedding at steep angles and cross bedding (low angles joints). Slightly fractured to massive. Rock Mass Quality: Poor. Ellsworth Formation. R1:Core Times (min:sec) 15.5-16.5 ft (4:35) 16.5-17.5 ft (4:50) 17.5-18.5 ft (3:55) 18.5-19.5 ft (4:40) 19.5-20.5 ft (4:00) 87% Recovery	
20	R2	60/60	20.50 - 25.50	RQD = 67%						R2:Bedrock: Pinkish grey, fine grained PHYLITTE, slightly weathered to fresh. Thin bedding, slightly fractured to massive. Rock Mass Quality: Fair. Ellsworth Formation. R2:Core Times (min:sec) 20.5-21.5 ft (-:--) 21.5-22.5 ft (-:--)	
25											

Remarks:

Auto Hammer #185

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: Greys Brook Bridge #6436 carries Route 180 over Greys Brook
Location: Ellsworth, Maine

Boring No.: BB-GB-104
WIN: 10063.10

Driller:	Northern Test Boring	**Elevation (ft.)**	109.9	**Auger ID/OD:**	5" Solid Stem
Operator:	Nick/Ryan	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/25/10; 07:00-11:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	1071+85.5, 3.9 Lt.	**Casing ID/OD:**	HW	**Water Level*:**	4.0 ft bgs.
Hammer Efficiency Factor:	0.713	**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.
25								84.40		22.5-23.5 ft (-:--) 23.5-24.5 ft (-:--) 24.5-25.5 ft (-:--) 100% Recovery	
										25.50 **Bottom of Exploration at 25.50 feet below ground surface.**	
30											
35											
40											
45											
50											

Remarks:

Auto Hammer #185

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)

Color (Munsell color chart)
Moisture (dry, damp, moist, wet, saturated)
Density/Consistency (from above right hand side)
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., if applicable)
Cementation (weak, moderate, or strong, if applicable, ASTM D 2488)
Geologic Origin (till, marine clay, alluvium, etc.)
Unified Soil Classification Designation
Groundwater level

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. Consistency is rated according to standard penetration resistance.

Modified Burmister System

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

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 shear strength as indicated.

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

Rock Quality Designation (RQD):

$$RQD = \frac{\text{sum of the lengths of intact pieces of core* > 100 mm}}{\text{length of core advance}}$$

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

Correlation of RQD to Rock Mass Quality

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

Desired Rock Observations: (in this order)

Color (Munsell color chart)
Texture (aphanitic, fine-grained, etc.)
Lithology (igneous, sedimentary, metamorphic, 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, low angle - 5-35, mod. dipping - 35-55, steep - 55-85, vertical - 85-90)
- -spacing (very close - <5 cm, close - 5-30 cm, mod. close 30-100 cm, wide - 1-3 m, very wide >3 m)
- -tightness (tight, open or healed)
- -infilling (grain size, color, etc.)

Formation (Waterville, Ellsworth, Cape Elizabeth, etc.)
RQD and correlation to rock mass quality (very poor, poor, etc.)
ref: AASHTO Standard Specification for Highway Bridges 17th Ed. Table 4.4.8.1.2A
Recovery

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

Sample Container Labeling Requirements:

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

Appendix B

Laboratory Test Results

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Ellsworth Project Number: 10063.10

Boring & Sample	Station	Offset	Depth	Reference	G.S.D.C.	W.C.	L.L.	P.I.	Classification		
Identification Number	(Feet)	(Feet)	(Feet)	Number	Sheet	%			Unified	AASHTO	Frost
BB-GB-101, 2D	1072+51.4	61.0 Lt.	5.0-7.0	237510	1	22.6	31	12	CL	A-6	III
BB-GB-101, 4D	1072+51.4	61.0 Lt.	15.0-17.0	237511	1	9.3			SM	A-1-b	II
BB-GB-102, 2D	1072+29.9	0.3 Rt.	5.0-7.0	237512	1	21.8	30	10	CL	A-4	IV
BB-GB-102, 4D	1072+29.9	0.3 Rt.	15.0-17.0	237513	1	11.4			SM	A-1-b	II
BB-GB-103, 3D	1071+99.2	48.0 Rt.	10.0-12.0	237514	2	30.1	29	9	CL	A-4	IV
BB-GB-103, 4D	1071+99.2	48.0 Rt.	15.0-17.0	237515	2	10.7			SW-SM	A-1-b	0
BB-GB-104, 2D	1071+85.5	3.9 Lt.	5.0-7.0	237516	2	20.1	29	9	CL	A-4	IV
BB-GB-104, 3D	1071+85.5	3.9 Lt.	10.0-12.0	237517	2	22.0			SM	A-4	III

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.

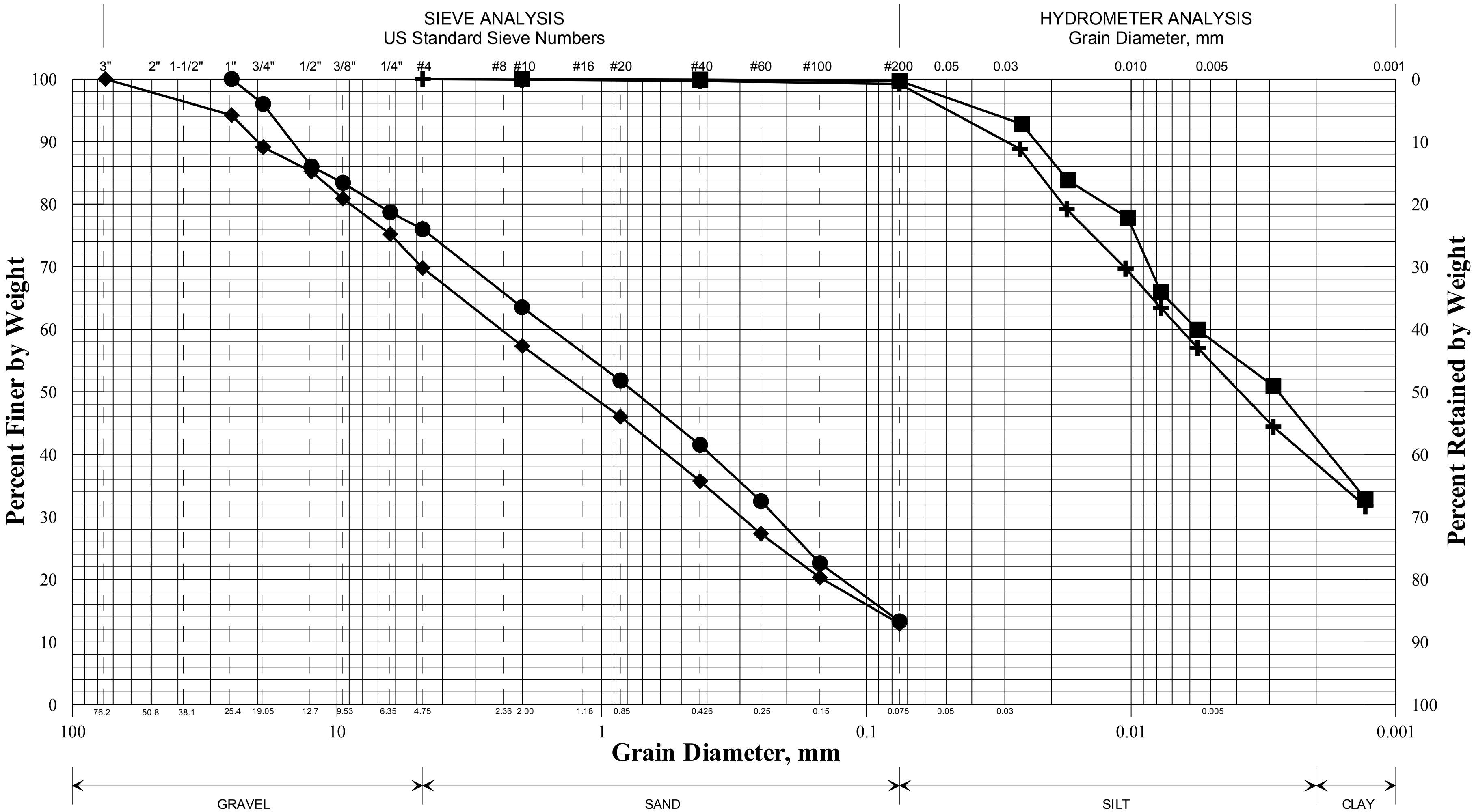
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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



UNIFIED CLASSIFICATION

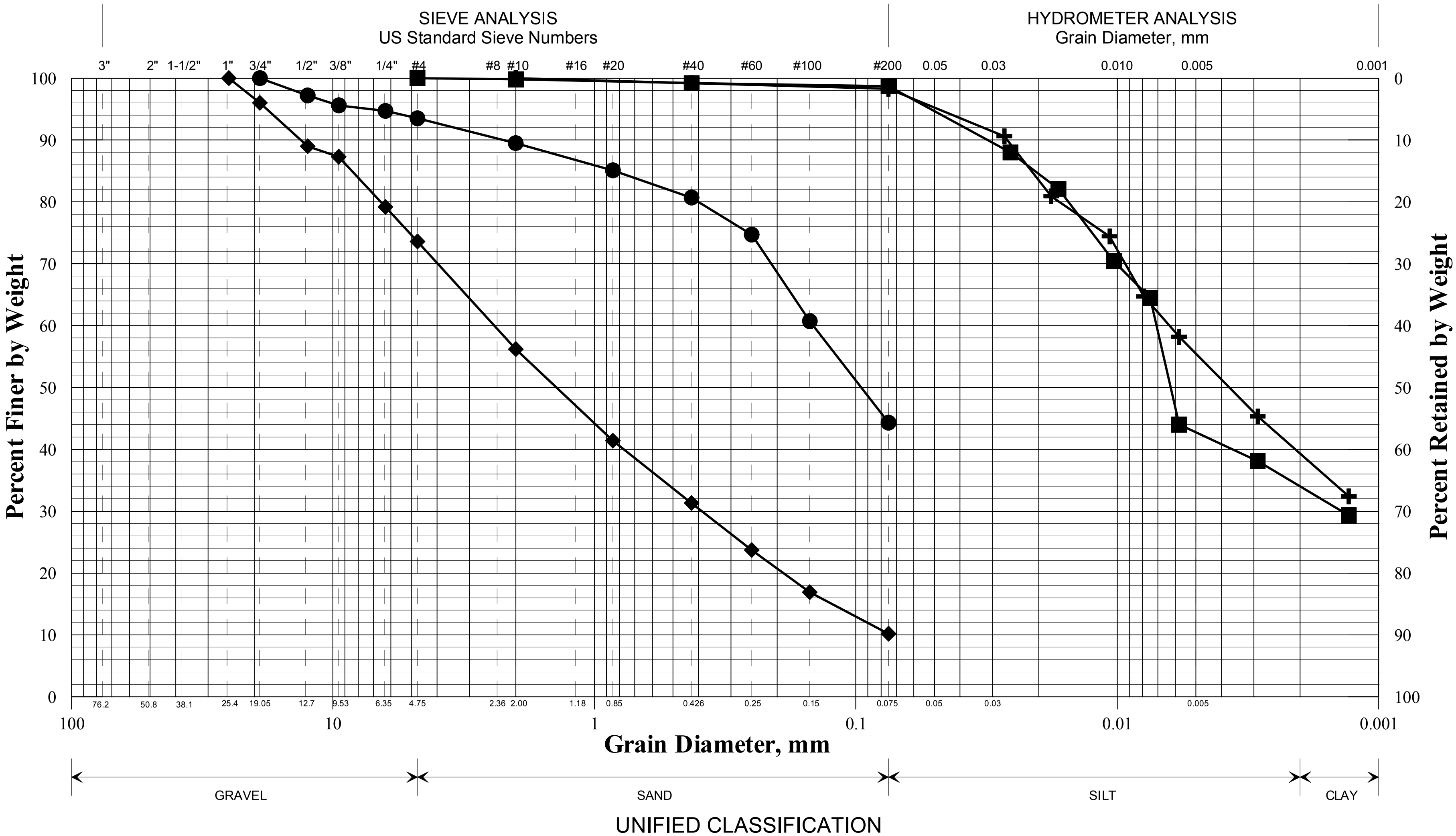
	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-GB-101/2D	1072+51.4	61.0 LT	5.0-7.0	Clayey SILT, trace sand.	22.6	31	19	12
◆	BB-GB-101/4D	1072+51.4	61.0 LT	15.0-17.0	SAND, some gravel, little silt.	9.3			
■	BB-GB-102/2D	1072+29.9	0.3 RT	5.0-7.0	Clayey SILT, trace sand.	21.8	30	20	10
●	BB-GB-102/4D	1072+29.9	0.3 RT	15.0-17.0	SAND, some gravel, little silt.	11.4			
▲									
✖									

PIN	
010063.10	
Town	
Ellsworth	
Reported by/Date	
WHITE, TERRY A	11/10/2010

SHEET 1

State of Maine Department of Transportation

GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-GB-103/3D	1071+99.2	48.0 RT	10.0-12.0	Clayey SILT, trace sand.	30.1	29	20	9
◆	BB-GB-103/4D	1071+99.2	48.0 RT	15.0-17.0	SAND, some gravel, little silt.	10.7			
■	BB-GB-104/2D	1071+85.5	3.9 LT	5.0-7.0	SILT, some clay, trace sand.	20.1	29	20	9
●	BB-GB-104/3D	1071+85.5	3.9 LT	10.0-12.0	Silty SAND, trace gravel.	22.0			
▲									
✖									

PIN
010063.10
Town
Ellsworth
Reported by/Date
WHITE, TERRY A 11/30/2010

SHEET 2

Maine DOT, Materials Testing & Exploration, 219 Hogan Road, Bangor, Maine 04401

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
237510	BB-GB-101/2D	GEOTECHNICAL (DISTURBED)	8/23/2010	9/21/2010

Sample Type: **GEOTECHNICAL** Location: **ROADWAY** Station: **1072+51.4** Offset, ft: **61.0 LT** Dbfg, ft: **5.0-7.0**

PIN: **010063.10** Town: **Ellsworth** Sampler: **SCHONEWALD, BE**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	
⅜ in. [9.5 mm]	
¼ in. [6.3 mm]	
No. 4 [4.75 mm]	100.0
No. 10 [2.00 mm]	99.9
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	99.7
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	99.2
[0.0263 mm]	88.8
[0.0175 mm]	79.2
[0.0105 mm]	69.7
[0.0077 mm]	63.4
[0.0056 mm]	57.0
[0.0029 mm]	44.4
[0.0013 mm]	31.7

Direct Shear (T 236)

Shear Angle, °			
Initial Water Content, %			
Normal Stress, psi			
Wet Density, lbs/ft³			
Dry Density, lbs/ft³			
Specimen Thickness, in			

Consolidation (T 216)

Trimmings, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	31
Plastic Limit (T 90), %	19
Plasticity Index (T 90), %	12
Specific Gravity, Corrected to 20°C (T 100)	2.65
Loss on Ignition (T 267) — Loss, % / H2O, %	
Water Content (T 265), %	22.6

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In. U. Shear tons/ft²	3 In. Remold tons/ft²	6 In. U. Shear tons/ft²	6 In. Remold tons/ft²	Water Content, %	Description of Material Sampled at the Various Tube Depths

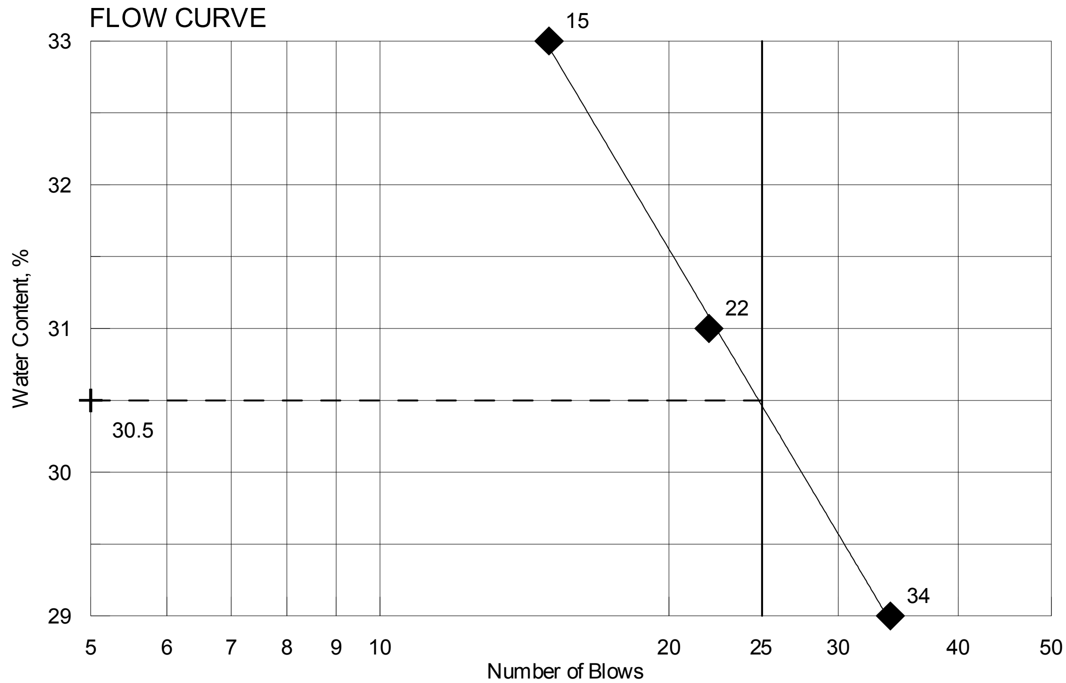
Comments:

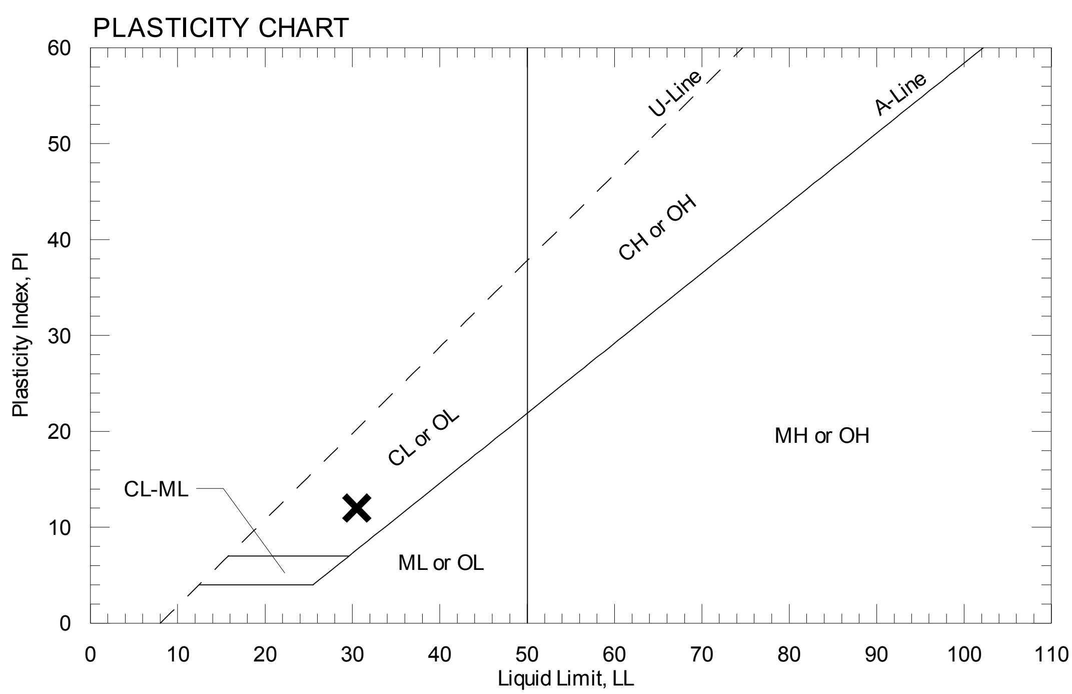
AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **11/3/2010**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Ellsworth	Reference No.	237510
PIN	010063.10	Water Content, %	22.6
Sampled	8/23/2010	Plastic Limit	19
Boring No./Sample No.	BB-GB-101/2D	Liquid Limit	31
Station	1072+51.4	Plasticity Index	12
Depth	5.0-7.0	Tested By	BBURR





Maine DOT, Materials Testing & Exploration, 219 Hogan Road, Bangor, Maine 04401

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
237512	BB-GB-102/2D	GEOTECHNICAL (DISTURBED)	8/24/2010	9/21/2010

Sample Type: **GEOTECHNICAL** Location: **ROADWAY** Station: **1072+29.9** Offset, ft: **0.3 RT** Dbfg, ft: **5.0-7.0**

PIN: **010063.10** Town: **Ellsworth** Sampler: **WILDER, BRUCE H**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	
⅜ in. [9.5 mm]	
¼ in. [6.3 mm]	
No. 4 [4.75 mm]	
No. 10 [2.00 mm]	100.0
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	99.9
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	99.7
[0.0259 mm]	92.8
[0.0173 mm]	83.8
[0.0103 mm]	77.8
[0.0077 mm]	65.9
[0.0056 mm]	59.9
[0.0029 mm]	50.9
[0.0013 mm]	32.9

Direct Shear (T 236)

Shear Angle, °			
Initial Water Content, %			
Normal Stress, psi			
Wet Density, lbs/ft³			
Dry Density, lbs/ft³			
Specimen Thickness, in			

Consolidation (T 216)

Trimmings, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	30
Plastic Limit (T 90), %	20
Plasticity Index (T 90), %	10
Specific Gravity, Corrected to 20°C (T 100)	2.66
Loss on Ignition (T 267) — Loss, %	
Loss on Ignition (T 267) — H2O, %	
Water Content (T 265), %	21.8

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In. U. Shear tons/ft²	3 In. Remold tons/ft²	6 In. U. Shear tons/ft²	6 In. Remold tons/ft²	Water Content, %	Description of Material Sampled at the Various Tube Depths

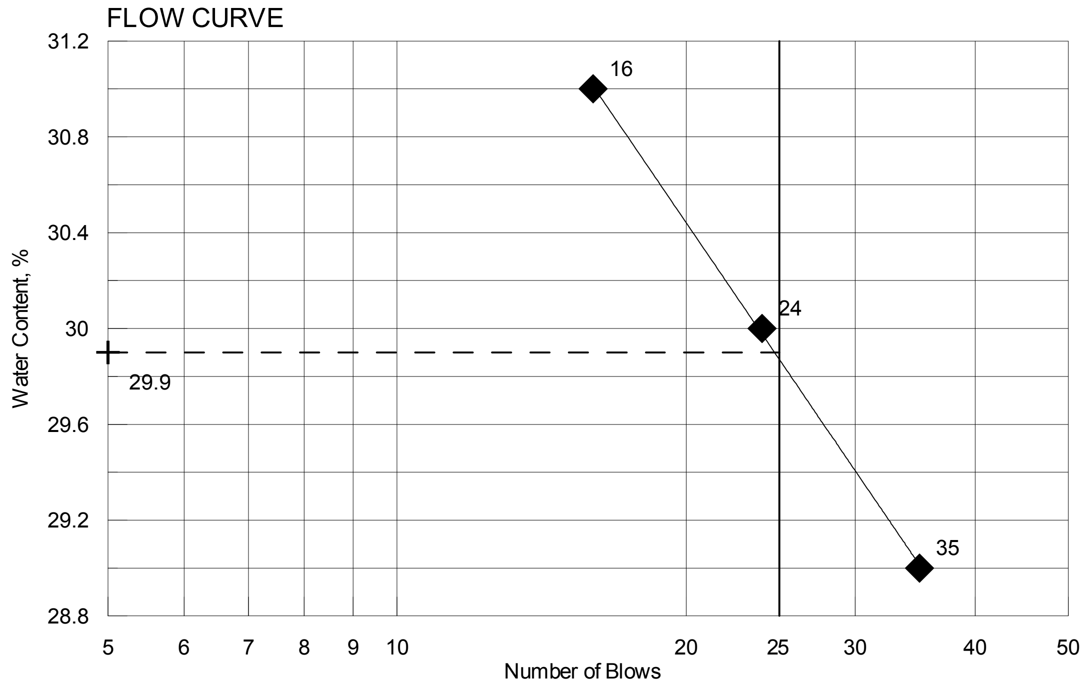
Comments:

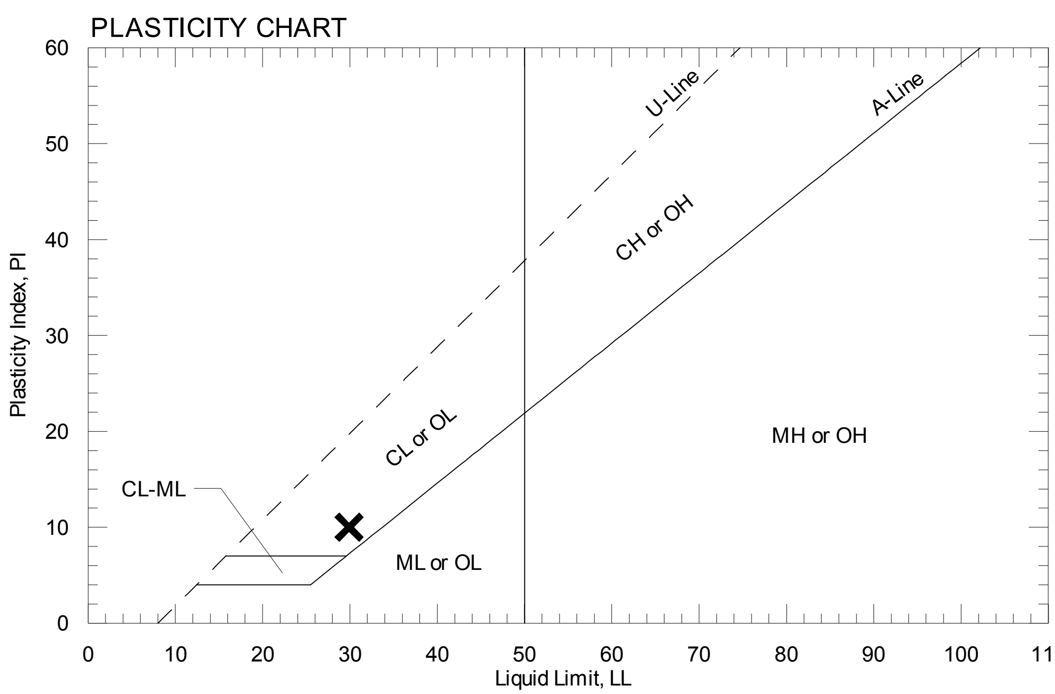
AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **10/26/2010**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Ellsworth	Reference No.	237512
PIN	010063.10	Water Content, %	21.8
Sampled	8/24/2010	Plastic Limit	20
Boring No./Sample No.	BB-GB-102/2D	Liquid Limit	30
Station	1072+29.9	Plasticity Index	10
Depth	5.0-7.0	Tested By	BBURR





Maine DOT, Materials Testing & Exploration, 219 Hogan Road, Bangor, Maine 04401

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
237514	BB-GB-103/3D	GEOTECHNICAL (DISTURBED)	8/24/2010	9/21/2010

Sample Type: **GEOTECHNICAL** Location: **ROADWAY** Station: **1071+99.2** Offset, ft: **48.0 RT** Dbfg, ft: **10.0-12.0**

PIN: **010063.10** Town: **Ellsworth** Sampler: **WILDER, BRUCE H**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	
⅜ in. [9.5 mm]	
¼ in. [6.3 mm]	
No. 4 [4.75 mm]	
No. 10 [2.00 mm]	100.0
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	99.2
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	98.3
[0.0270 mm]	90.6
[0.0179 mm]	80.9
[0.0107 mm]	74.4
[0.0079 mm]	64.7
[0.0058 mm]	58.2
[0.0029 mm]	45.3
[0.0013 mm]	32.4

Direct Shear (T 236)

Shear Angle, °			
Initial Water Content, %			
Normal Stress, psi			
Wet Density, lbs/ft³			
Dry Density, lbs/ft³			
Specimen Thickness, in			

Consolidation (T 216)

Trimmings, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	29
Plastic Limit (T 90), %	20
Plasticity Index (T 90), %	9
Specific Gravity, Corrected to 20°C (T 100)	2.64
Loss on Ignition (T 267): Loss, %	
Loss on Ignition (T 267): H2O, %	
Water Content (T 265), %	30.1

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In. U. Shear tons/ft²	3 In. Remold tons/ft²	6 In. U. Shear tons/ft²	6 In. Remold tons/ft²	Water Content, %	Description of Material Sampled at the Various Tube Depths

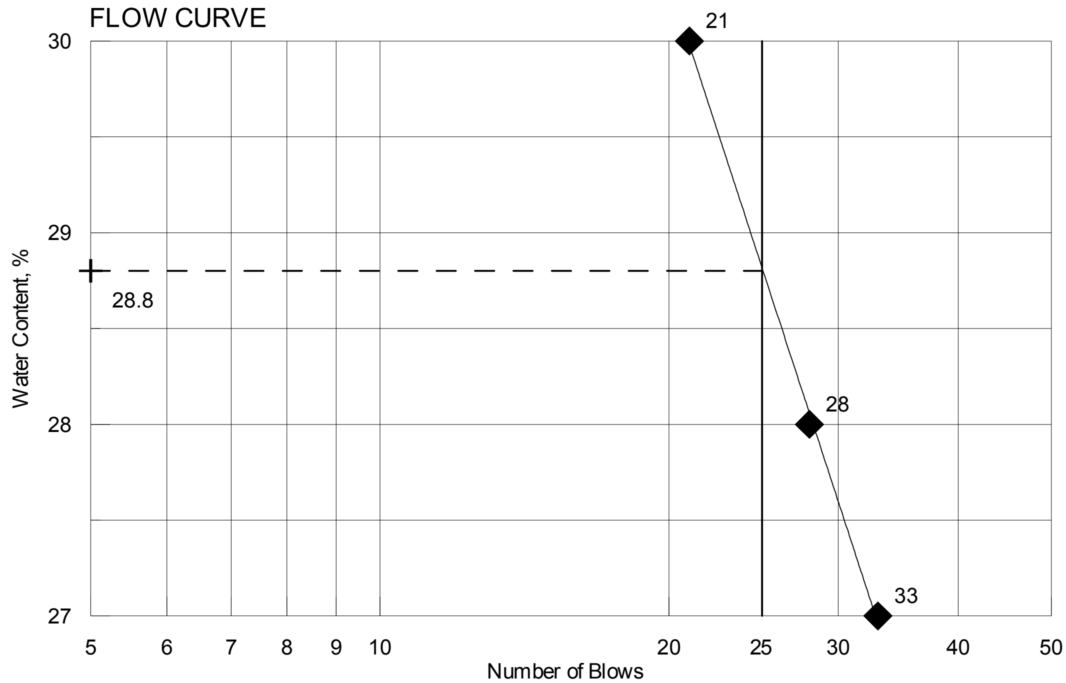
Comments:

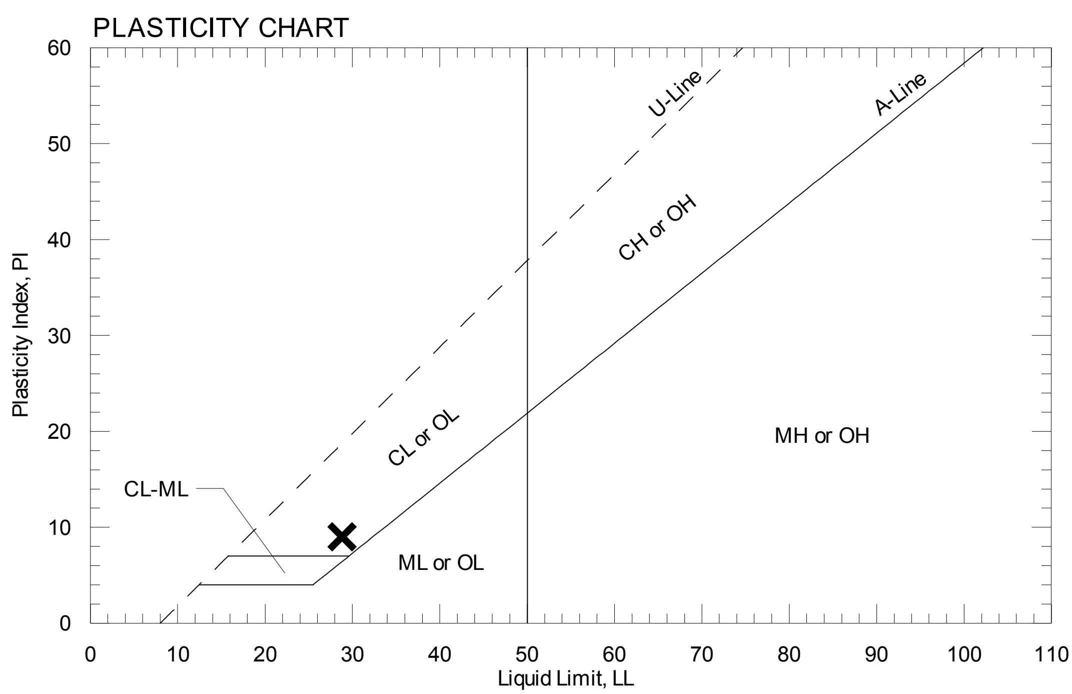
AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **11/23/2010**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Ellsworth	Reference No.	237514
PIN	010063.10	Water Content, %	30.1
Sampled	8/24/2010	Plastic Limit	20
Boring No./Sample No.	BB-GB-103/3D	Liquid Limit	29
Station	1071+99.2	Plasticity Index	9
Depth	10.0-12.0	Tested By	BBURR





Maine DOT, Materials Testing & Exploration, 219 Hogan Road, Bangor, Maine 04401

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
237516	**BB-GB-104/2D**	**GEOTECHNICAL (DISTURBED)**	**8/25/2010**	**9/21/2010**

Sample Type: **GEOTECHNICAL** Location: **ROADWAY** Station: **1071+85.5** Offset, ft: **3.9 LT** Dbfg, ft: **5.0-7.0**

PIN: **010063.10** Town: **Ellsworth** Sampler: **WILDER, BRUCE H**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	
⅜ in. [9.5 mm]	
¼ in. [6.3 mm]	
No. 4 [4.75 mm]	**100.0**
No. 10 [2.00 mm]	**99.8**
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	**99.2**
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	**98.7**
[0.0256 mm]	**88.0**
[0.0168 mm]	**82.1**
[0.0103 mm]	**70.4**
[0.0075 mm]	**64.5**
[0.0058 mm]	**44.0**
[0.0029 mm]	**38.1**
[0.0013 mm]	**29.3**

Direct Shear (T 236)

Shear Angle, °			
Initial Water Content, %			
Normal Stress, psi			
Wet Density, lbs/ft³			
Dry Density, lbs/ft³			
Specimen Thickness, in			

Consolidation (T 216)

Trimmings, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	**29**
Plastic Limit (T 90), %	**20**
Plasticity Index (T 90), %	**9**
Specific Gravity, Corrected to 20°C (T 100)	**2.73**
Loss on Ignition (T 267) — Loss, % / H2O, %	
Water Content (T 265), %	**20.1**

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In. U. Shear tons/ft²	3 In. Remold tons/ft²	6 In. U. Shear tons/ft²	6 In. Remold tons/ft²	Water Content, %	Description of Material Sampled at the Various Tube Depths

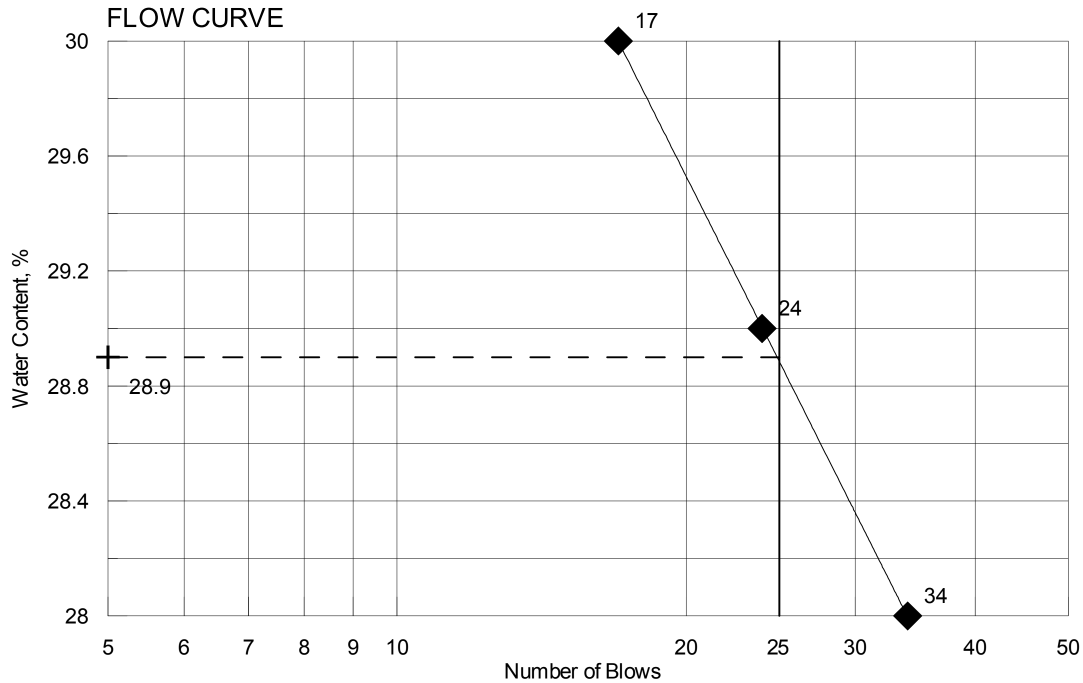
Comments:

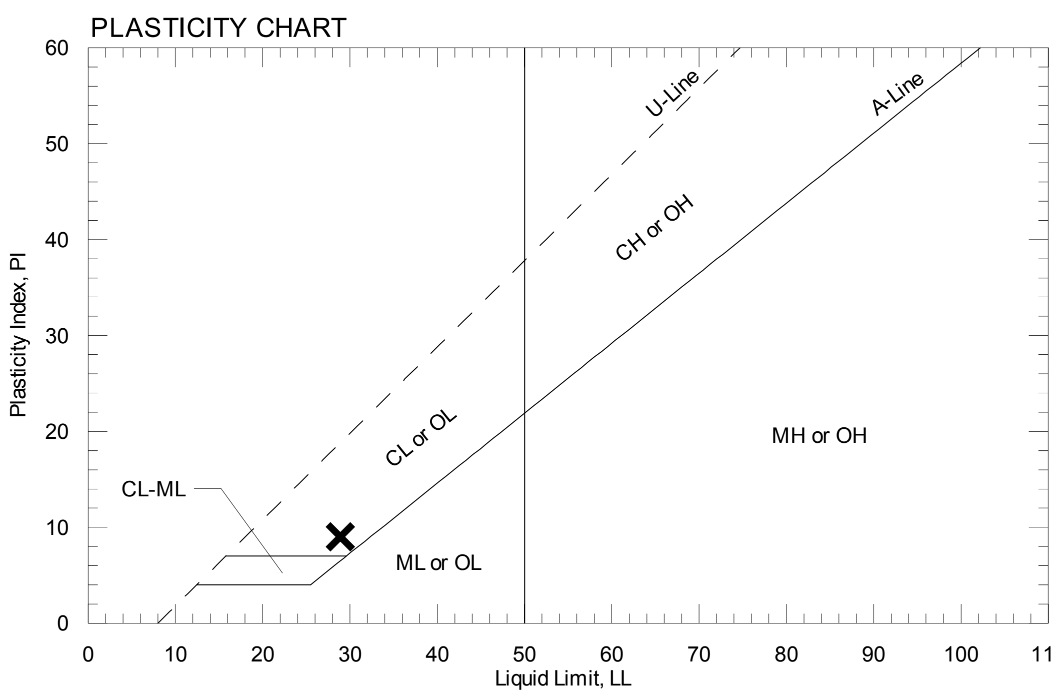
AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **10/26/2010**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Ellsworth	Reference No.	237516
PIN	010063.10	Water Content, %	20.1
Sampled	8/25/2010	Plastic Limit	20
Boring No./Sample No.	BB-GB-104/2D	Liquid Limit	29
Station	1071+85.5	Plasticity Index	9
Depth	5.0-7.0	Tested By	BBURR





Appendix C

Calculations

Analysis : Bearing Resistance of CIP Wingwalls at El. 104.0 on clayey silt, silt

Assumptions

1. Base of footing founded with 6 feet embedment for frost
   Footings bearing on Clayey Silt at Elevation 104.0 feet

2. Assumed parameters for undrained loose silt
   Saturated unit weight = 120 pcf (Bowles Table 3-4; Holtz, Kovacs, Table 2-1 1981)
   Dry unit weight = 117 pcf
   ϕ = 20 degrees, undrained (ref: Bowles, 5th Edition, Table 2-6).
   Su = undrained shear strength, c = 500 psf

3. Method used: Terzaghi, use strip equations since L>B

CIP Wall Base Widths and Depth

$$B := \begin{pmatrix} 8 \\ 10 \\ 12 \\ 14 \\ 16 \\ 18 \end{pmatrix} \cdot ft$$

Embed footings 6 feet for frost protection

$D_f := 6.0 \cdot ft$

$D_w := 0 \cdot ft$ $\qquad \gamma_w := 62.4 \cdot pcf$

Foundation Soil - Undrained Analysis - Clayey Silt medium stiff, based on vanes, very stiff to hard, Su = 2986 to >5658 psf ; based on SPT, medium stiff to stiff to very stiff..

$\gamma1_{sat} := 120 \cdot pcf$

$\gamma1_d := 117 \cdot pcf$

$\phi := 20 \cdot deg$

Based on range of undrained shear strengths, use lowest SPT correlation

$c_1 := 1000 \cdot psf$

Nominal Bearing Resistance - based on Presumptive Bearing Capacity

For Service Limit States ONLY

Method: LRFD Table C10.6.2.6.1-1, Presumptive Bearing Resistance for Spread Footings at the Service Limit State, based on *NavFac DM 7.2, May 1983, Foundations and Earth Structures*, Table 1, 7.2-142, "Presumptive Values of Allowable Bearing Pressures for Spread Foundations".

Bearing Material:	Consistency in Place:	Bearing Pressure Resistance Range (ksf)	Recommended Value (ksf)
Homogeneous inorganic clay, silty clay - CL	**med. dense-dense**	**2-6**	**4 ksf**
Inorganic silt, clayey silt ML ,	**med. stiff to stiff**	**2-6**	**3 ksf**

Recommend 3 ksf, to limit settlement to 1.0 inch for Service Limit State analyses and for preliminary footing sizing. (Therefore, need to lengthen footing of preliminary design stage - where the applied pressure was 5.51 ksf for Service).

Nominal Bearing Resistance for Strength Limit States: Terzaghi Method - ϕ and c soil.

Shape Factors for strip footing (Bowles 5th Ed., pg 220)

$s_\gamma := 1.0$ $s_c := 1.0$

Meyerhof Bearing Capacity Factors - (Ref: Bowles Table 4-4, 5th Ed. pg 223) for undrained silt ϕ = 20 degrees

$N_c := 14.83$ $N_q := 6.4$ $N_\gamma := 2.9$

Nominal Bearing Resistance per Terzaghi equation (Bowles, Table 4-1, 5th Ed., pg 220)

$q := D_f \cdot (\gamma 1_{sat} - \gamma_w)$ $q = 0.346 \cdot \text{ksf}$

$q_n := c_1 \cdot N_c \cdot s_c + q \cdot N_q + 0.5 \cdot (\gamma 1_{sat} - \gamma_w) \cdot B \cdot N_\gamma \cdot s_\gamma$

$$q_n = \begin{pmatrix} 17.7 \\ 17.9 \\ 18 \\ 18.2 \\ 18.4 \\ 18.5 \end{pmatrix} \cdot ksf$$

For cohesion of 1000 psf

Factored Bearing Resistance for strength limit states

Use a resistance factor per AASHTO LRFD Table 10.5.5.2.2-1

$\varphi_b := 0.45$

$q_r := q_n \cdot \varphi_b$

$$q_r = \begin{pmatrix} 8 \\ 8 \\ 8.1 \\ 8.2 \\ 8.3 \\ 8.3 \end{pmatrix} \cdot ksf \quad \textbf{for} \quad B = \begin{pmatrix} 8 \\ 10 \\ 12 \\ 14 \\ 16 \\ 18 \end{pmatrix} \cdot ft$$

For cohesion of 1000 psf

8 ksf for strength limit state design regardless of footing width.

Calderwood Engineering preliminary design estimated factored STRI at 7.85 ksf for a 11'9" wide footing.

Factored Bearing Resistance for extreme limit states

Use a resistance factor per AASHTO LRFD Table 10.5.5.2.2-1

$\varphi_b := 1.0$

$q_r := q_n \cdot \varphi_b$

$$q_r = \begin{pmatrix} 17.7 \\ 17.9 \\ 18 \\ 18.2 \\ 18.4 \\ 18.5 \end{pmatrix} \cdot ksf \quad \textbf{for} \quad B = \begin{pmatrix} 8 \\ 10 \\ 12 \\ 14 \\ 16 \\ 18 \end{pmatrix} \cdot ft$$

Analysis : Bearing Resistance of CIP Wingwalls at El. 104.0 on compacted structural fill

Assumptions

1. Base of footing founded with 6 feet embedment for frost
   Excavate Clayey Silt completely (down to Elev. 96 to 99) and replace with compacted structural fill.

2. Assumed parameters for compacted granular backfill
   Saturated unit weight = 130 pcf (Bowles Table 3-4; Holtz, Kovacs, Table 2-1 1981)
   Dry unit weight = 125 pcf
   ϕ : Lambe & Whitman Table 11.3 based on Hough, Basic Soils Engr, 1967
   ϕ and SPT correlation, Lambe & Whitman, Fig 11.14, (from Peck, Hanson, Thornburn).
   ϕ = 32 degrees (Bowles Tables 3-4 and 2-6).
   Su= undrained shear strength (c) 0 psf

3. Method used: Terzaghi, use strip equations since L>B

CIP Wall Base Widths and Depth

$$B := \begin{pmatrix} 6 \\ 8 \\ 10 \\ 12 \\ 14 \\ 16 \end{pmatrix} \cdot ft$$

Embed footings 6.0 feet for frost protection

$$D_f := 6.0 \cdot ft$$

$$D_w := 0 \cdot ft \qquad \gamma_w := 62.4 \cdot pcf$$

Foundation Soil - Excavate the Clayey Silt (medium stiff, Su = 500-1000) in its entirety and replace with compacted granular borrow

$$\gamma 1_{sat} := 130 \cdot pcf$$

$$\gamma 1_d := 125 \cdot pcf$$

$$\phi := 32 \cdot deg$$

$$c_1 := 0 \cdot psf$$

Nominal Bearing Resistance - based on Presumptive Bearing Capacity

For Service Limit States ONLY

Method**:** LRFD Table C10.6.2.6.1-1, Presumptive Bearing Resistance for Spread Footings at the Service Limit State, based on *NavFac DM 7.2, May 1983, Foundations and Earth Structures*, Table 1, 7.2-142, "Presumptive Values of Allowable Bearing Pressures for Spread Foundations".

Bearing Material:	Consistency in Place:	Bearing Pressure Resistance Range (ksf)	Recommended Value (ksf)
Coarse to medium sand and with little gravel	Med. dense to dense	4-8	6 ksf

Recommend 6 ksf, to limit settlement to 1.0 inch for Service Limit State analyses and for preliminary footing sizing. (OK - Calderwood Engr. preliminary design is 5.51 ksf for Service).

Nominal Bearing Resistance for Strength Limit States: Terzaghi Method - ϕ and c soil.

Shape Factors for strip footing (Bowles 5th Ed., pg 220)

$s_\gamma := 1.0$ $\qquad$ $s_c := 1.0$

Meyerhof Bearing Capacity Factors - (Ref: Bowles Table 4-4, 5th Ed. pg 223)

$N_c := 35.47$ $\qquad$ $N_q := 23.2$ $\qquad$ $N_\gamma := 22$

Nominal Bearing Resistance per Terzaghi equation (Bowles, Table 4-1, 5th Ed., pg 220)

$q := D_f \cdot (\gamma 1_{sat} - \gamma_w)$ $\qquad$ $q = 0.406 \cdot ksf$

$q_n := c_1 \cdot N_c \cdot s_c + q \cdot N_q + 0.5 \cdot (\gamma 1_{sat} - \gamma_w) \cdot B \cdot N_\gamma \cdot s_\gamma$

$$q_n = \begin{pmatrix} 13.9 \\ 15.4 \\ 16.8 \\ 18.3 \\ 19.8 \\ 21.3 \end{pmatrix} \cdot ksf$$

Factored Bearing Resistance for strength limit state **s**

Use a resistance factor per AASHTO LRFD Table 10.5.5.2.2-1

$\varphi_b := 0.45$

$q_r := q_n \cdot \varphi_b$

$q_r = \begin{pmatrix} 6.2 \\ 6.9 \\ 7.6 \\ 8.2 \\ 8.9 \\ 9.6 \end{pmatrix} \cdot ksf$ **for** $B = \begin{pmatrix} 6 \\ 8 \\ 10 \\ 12 \\ 14 \\ 16 \end{pmatrix} \cdot ft$

7.6 ksf for strength limit state design of footings 10 feet wide or more; to limit settlement to 1.0 inch design footing size such that the Service Limit State pressure is 6 ksf or less.

OK. Calderwood Engineering preliminary design estimated factored STRI at 7.85 ksf for a 11'9" wide footing.

Factored Bearing Resistance for extreme limit state **s**

Use a resistance factor per AASHTO LRFD Table 10.5.5.2.2-1

$\varphi_b := 1.0$

$q_r := q_n \cdot \varphi_b$

$q_r = \begin{pmatrix} 13.9 \\ 15.4 \\ 16.8 \\ 18.3 \\ 19.8 \\ 21.3 \end{pmatrix} \cdot ksf$ **for** $B = \begin{pmatrix} 6 \\ 8 \\ 10 \\ 12 \\ 14 \\ 16 \end{pmatrix} \cdot ft$

Analysis : Bearing Resistance of MSE Walls on top of CIP traditional cantilever wall - above Q1.1..
Construciton of CIP cantilever type wall will require over-excavation of 1-2 feet of clay-silt and replacement with 3/4-inch crushed stone and place CIP wall footing at 104.0 (6 foot embedment for frost.)
Elev. 114.0 is approximately Q1.1.

Assumptions

1. Base of footing founded with 0 feet embedment for frost. MSE wall facing elements constructed on CIP stemwall base.

2. Assumed parameters for compacted granular backfill
   Saturated unit weight = 130 pcf (Bowles Table 3-4; Holtz, Kovacs, Table 2-1 1981)
   Dry unit weight = 125 pcf
   ϕ : Lambe & Whitman Table 11.3 based on Hough, Basic Soils Engr, 1967
   ϕ and SPT correlation, Lambe & Whitman, Fig 11.14, (from Peck, Hanson, Thornburn).
   ϕ = 32 degrees (Bowles Tables 3-4 and 2-6).
   Su= undrained shear strength (c) 0 psf

3. Method used: Terzaghi, use strip equations since L>B

MSE Wall Base Widths and Depth

$D_f := .0\cdot ft$

$D_w := 0\cdot ft$

$\gamma_w := 62.4\cdot pcf$

$$B := \begin{pmatrix} 8 \\ 10 \\ 12 \\ 14 \\ 16 \\ 20 \end{pmatrix}\cdot ft$$

Foundation Soil - MSE Wall bearing on backfill soil behind underlying CIP wall

$\gamma1_{sat} := 130\cdot pcf$

$\gamma1_d := 125\cdot pcf$

$\phi := 32\cdot deg$

$c_1 := 0\cdot psf$

Nominal Bearing Resistance - based on Presumptive Bearing Capacity

For Service Limit States ONLY

Method**:** LRFD Table C10.6.2.6.1-1, Presumptive Bearing Resistance for Spread Footings at the Service Limit State, based on *NavFac DM 7.2, May 1983, Foundations and Earth Structures*, Table 1, 7.2-142, "Presumptive Values of Allowable Bearing Pressures for Spread Foundations".

Bearing Material:	Consistency in Place:	Bearing Pressure Resistance Range (ksf)	Recommended Value (ksf)
Coarse to medium sand and with little gravel	med. dense - dense	4-8	6 ksf

Recommend 6 ksf, to limit settlement to 1.0 inch for Service Limit State analyses and for preliminary footing sizing.

Nominal Bearing Resistance for Strength Limit States: Terzaghi Method - ϕ and c soil.

Shape Factors for square MSE Wall Base (Bowles 5th Ed., pg 220)

$s_\gamma := 1$ $\qquad$ $s_c := 1.$

Meyerhof Bearing Capacity Factors - (Ref: Bowles Table 4-4, 5th Ed. pg 223)

$N_c := 35.47$ $\qquad$ $N_q := 23.2$ $\qquad$ $N_\gamma := 22$

Nominal Bearing Resistance per Terzaghi equation (Bowles, Table 4-1, 5th Ed., pg 220)

$q := D_f \cdot (\gamma1_{sat} - \gamma_w)$ $\qquad$ $q = 0 \cdot ksf$

$$q_n := c_1 \cdot N_c \cdot s_c + q \cdot N_q + 0.5 \cdot (\gamma1_{sat} - \gamma_w) \cdot B \cdot N_\gamma \cdot s_\gamma$$

$$q_n = \begin{pmatrix} 5.9 \\ 7.4 \\ 8.9 \\ 10.4 \\ 11.9 \\ 14.9 \end{pmatrix} \cdot ksf$$

Factored Bearing Resistance for strength limit state **s**

Use a bearing resistance factor for MSE walls per AASHTO LRFD Table 11.5.6-1

$$\varphi_b := 0.65$$

$$q_r := q_n \cdot \varphi_b$$

$$q_r = \begin{pmatrix} 3.9 \\ 4.8 \\ 5.8 \\ 6.8 \\ 7.7 \\ 9.7 \end{pmatrix} \cdot ksf$$ **for** $$B = \begin{pmatrix} 8 \\ 10 \\ 12 \\ 14 \\ 16 \\ 20 \end{pmatrix} \cdot ft$$

Factored Bearing Resistance for extreme limit state **s**

Use a resistance factor per AASHTO LRFD Table 10.5.5.2.2-1

$$\varphi_b := 1.0$$

$$q_r := q_n \cdot \varphi_b$$

$$q_r = \begin{pmatrix} 5.9 \\ 7.4 \\ 8.9 \\ 10.4 \\ 11.9 \\ 14.9 \end{pmatrix} \cdot ksf$$ **for** $$B = \begin{pmatrix} 8 \\ 10 \\ 12 \\ 14 \\ 16 \\ 20 \end{pmatrix} \cdot ft$$

Analysis : Bearing resistance of PCMG Walls constructed above CIP cantilever walls. Assumes over-excavation of 1-2 feet clay-silt, replacement with 3/4 crushed stone and construction of CIP wall footing at Elevation 104. feet. Q1.1 is approximated at 114.0 feet. PCMG walls should be above Q1.1.

Assumptions

1. Base of footing founded with 0 feet embedment for frost. PCMG modular units are constructed on CIP stemwall base and the granular borrow backfilling the CIP cantilever wall.

2. Assumed parameters for compacted granular backfill
   Saturated unit weight = 130 pcf (Bowles Table 3-4; Holtz, Kovacs, Table 2-1 1981)
   Dry unit weight = 125 pcf
   ϕ : Lambe & Whitman Table 11.3 based on Hough, Basic Soils Engr, 1967
   ϕ and SPT correlation, Lambe & Whitman, Fig 11.14, (from Peck, Hanson, Thornburn).
   ϕ = 32 degrees (Bowles Tables 3-4 and 2-6).
   Su= undrained shear strength (c) 0 psf

3. Method used: Terzaghi, use strip equations since L>B

PCMG Base Widths and Depth

$D_f := .0 \cdot ft$

$D_w := 0 \cdot ft$

$\gamma_w := 62.4 \cdot pcf$

$$B := \begin{pmatrix} 8 \\ 10 \\ 12 \\ 14 \\ 16 \\ 18 \end{pmatrix} \cdot ft$$

Foundation Soil - PCMG Walls constructed above CIP walls on the granular borrow structural backfill

$\gamma 1_{sat} := 130 \cdot pcf$

$\gamma 1_d := 125 \cdot pcf$

$\phi := 32 \cdot deg$

$c_1 := 0 \cdot psf$

Nominal Bearing Resistance - based on Presumptive Bearing Capacity

For Service Limit States ONLY

Method: LRFD Table C10.6.2.6.1-1, Presumptive Bearing Resistance for Spread Footings at the Service Limit State, based on *NavFac DM 7.2, May 1983, Foundations and Earth Structures*, Table 1, 7.2-142, "Presumptive Values of Allowable Bearing Pressures for Spread Foundations".

Bearing Material:	Consistency in Place:	Bearing Pressure Resistance Range (ksf)	Recommended Value (ksf)
Coarse to medium sand and with little gravel	med. dense - dense	4-8	6 ksf

Recommend 6 ksf, to limit settlement to 1.0 inch for Service Limit State analyses and for preliminary footing sizing.

Nominal Bearing Resistance for Strength Limit States: Terzaghi Method - ϕ and c soil.

Shape Factors for square MSE Wall Base (Bowles 5th Ed., pg 220)

$s_{\gamma} := 1$ $\quad$ $s_c := 1.$

Meyerhof Bearing Capacity Factors - (Ref: Bowles Table 4-4, 5th Ed. pg 223)

$N_c := 35.47$ $\quad$ $N_q := 23.2$ $\quad$ $N_{\gamma} := 22$

Nominal Bearing Resistance per Terzaghi equation (Bowles, Table 4-1, 5th Ed., pg 220)

$$q := D_f \cdot (\gamma 1_{sat} - \gamma_w) \qquad q = 0 \cdot ksf$$

$$q_n := c_1 \cdot N_c \cdot s_c + q \cdot N_q + 0.5 \cdot (\gamma 1_{sat} - \gamma_w) \cdot B \cdot N_{\gamma} \cdot s_{\gamma}$$

$$q_n = \begin{pmatrix} 5.9 \\ 7.4 \\ 8.9 \\ 10.4 \\ 11.9 \\ 13.4 \end{pmatrix} \cdot ksf$$

*Factored Bearing Resistance for strength limit state***s**

Use a bearing resistance factor for PCMG walls per AASHTO LRFD Table 11.5.6-1

$$\varphi_b := 0.45$$

$$q_r := q_n \cdot \varphi_b$$

$$\boxed{q_r = \begin{pmatrix} 2.7 \\ 3.3 \\ 4 \\ 4.7 \\ 5.4 \\ 6 \end{pmatrix} \cdot ksf} \quad \textbf{for} \quad \boxed{B = \begin{pmatrix} 8 \\ 10 \\ 12 \\ 14 \\ 16 \\ 18 \end{pmatrix} \cdot ft}$$

*Factored Bearing Resistance for extreme limit state***s**

Use a resistance factor per AASHTO LRFD Table 10.5.5.2.2-1

$$\varphi_b := 1.0$$

$$q_r := q_n \cdot \varphi_b$$

$$\boxed{q_r = \begin{pmatrix} 5.9 \\ 7.4 \\ 8.9 \\ 10.4 \\ 11.9 \\ 13.4 \end{pmatrix} \cdot ksf} \quad \textbf{for} \quad \boxed{B = \begin{pmatrix} 8 \\ 10 \\ 12 \\ 14 \\ 16 \\ 18 \end{pmatrix} \cdot ft}$$

Bedrock Properties at the Site

RQD of bedrock cores
Abutment #1 Pile Group at CL: BB-GB-104, R1=50%, R2=67%

Abutment #2 Pile Group at CL: BB-GB-102, R1=25%, R2=83%
Left 61 feet: BB-GB-101 R1=47%, R2= 27%
Right 48 feet BB-GB-103, R1=0%

Rock Type: Sedimentary PHYLLITE

ϕ = 20-27 (AASHTO LRFD Table C.10.4.6.4-1);

uniaxial compressive strength = Co= 3500 to 35,000 psi - use **20,000 psi** for design AASHTO TABLE 4.4.8.1.2.B (17th Edition, 2002)

Average of upper bedrock cores at CL: 37%
Average of upper bedrock cores from all borings: 30%
Average of 10 of rock core at CL: 56%
Average of all bedrock cores: 43%

Use 37% for design purposes

Pile Properties

Use the following piles: 12x53, 12x74, 14x73, 14x89, 14x117

$$A_s := \begin{pmatrix} 15.5 \\ 21.8 \\ 21.4 \\ 26.1 \\ 34.4 \end{pmatrix} \cdot in^2 \qquad d := \begin{pmatrix} 11.78 \\ 12.13 \\ 13.6 \\ 13.83 \\ 14.21 \end{pmatrix} \cdot in \qquad b := \begin{pmatrix} 12.045 \\ 12.215 \\ 14.585 \\ 14.695 \\ 14.885 \end{pmatrix} \cdot in$$

$$A_{box} := \overrightarrow{(d \cdot b)} \qquad A_{box} = \begin{pmatrix} 141.89 \\ 148.168 \\ 198.356 \\ 203.232 \\ 211.516 \end{pmatrix} \cdot in^2$$

Nominal and Factored Structural Compressive Resistance of HP piles

Axial pile resistance may be controlled by structural resistance if driven to sound bedrock
Use LRFD Equation 6.9.2.1-1

$$F_y := 50 \cdot ksi$$

Nominal Axial Structural Resistance

Determine equivalent yield resistance Po=QFyAs (LRFD 6.9.4.1.1)

$Q := 1.0$ LRFD Article 6.9.4.2

$P_o := Q \cdot F_y \cdot A_s$

$$P_o = \begin{pmatrix} 775 \\ 1090 \\ 1070 \\ 1305 \\ 1720 \end{pmatrix} \cdot kip$$

Determine elastic critical buckling resistance Pe, LRFD eq. 6.9.4.1.2-1

E = Elastic Modulus $E := 29000 \cdot ksi$

K = effective length factor $K_{eff} := 2.0$ LRFD Table C4.6.2.5-1 (assume rotation free at pile tip

l = unbraced length $l_{unbraced} := .5 \cdot ft$

r s = radius of gyration

$$r_s := \begin{pmatrix} 2.86 \\ 2.92 \\ 3.49 \\ 3.53 \\ 3.59 \end{pmatrix} \cdot in$$

LRFD eq. 6.9.4.1.2-1

$$P_e := \overrightarrow{\left[\frac{\pi^2 \cdot E}{\left(\frac{K_{eff} \cdot l_{unbraced}}{r_s}\right)^2} \cdot A_s\right]} \qquad P_e = \begin{pmatrix} 251999 \\ 369452 \\ 518084 \\ 646435 \\ 881216 \end{pmatrix} \cdot kip$$

LRFD Article 6.9.4.1.1

$$\frac{P_e}{P_o} = \begin{pmatrix} 325.16 \\ 338.946 \\ 484.19 \\ 495.353 \\ 512.335 \end{pmatrix}$$

If Pe/Po > or = 0.44, then:

$$P_n := \overrightarrow{\left(0.658^{\frac{P_o}{P_e}} \cdot P_o\right)}$$

LRFD Eq. 6.9.4.1.1-1

then

$$P_n = \begin{pmatrix} 774 \\ 1089 \\ 1069 \\ 1304 \\ 1719 \end{pmatrix} \cdot kip$$

this applies to all the pile sizes use Po

If Pe/Po < 0.44, then: $P_{n1} := \overrightarrow{\left(0.877 \cdot P_e\right)}$

not : $P_{n1} = \begin{pmatrix} 221003 \\ 324009 \\ 454359 \\ 566924 \\ 772827 \end{pmatrix} \cdot kip$

Factored Axial Structural Resistance of single H pile

Resistance factor or H-pile in compression, good driving condtions LRFD 6.5.4.2

$$\phi_c := 0.6$$

The Factored Structural Resistance (Pr) per LRFD 6.9.2.1-1 is

$$P_r := \phi_c \cdot P_n$$

Factored structural compressive resistance, Pr $\boxed{P_r = \begin{pmatrix} 464 \\ 653 \\ 641 \\ 782 \\ 1031 \end{pmatrix} \cdot kip}$

Nominal and Factored Axial Geotechnical Resistance of HP piles

Geotechncial axial pile resistance for pile end bearing on rock is determined by CGS method (LRFD Table 10.5.5.2.3-1) and outlined in Canadian Foundation Engineering Manual, 4th Edition 2006, and FHWA LRFD Pile Foundation Design Example in FHWA-NHI-05-094.

Nominal unit bearing resistance of pile point, qp

Design value of compressive strength of rock core

Phyllite $q_{u\_1} := 20000 \cdot psi$

Spacing of discontinuities $s_d := 4 \cdot in$

Width of discontinuities. Joints are open to tight per boring logs $t_d := \frac{1}{64} \cdot in$

Pile width is b - matrix $D := b$

Embedment depth of pile in socket - pile is end bearing on rock $H_s := 0 \cdot ft$

Diameter of socket: $D_s := 12 \cdot in$

Depth factor $dd := 1 + 0.4 \cdot \frac{H_s}{D_s}$ and dd < 3

$dd = 1$ OK

Ksp

$$K_{sp} := \frac{3 + \frac{s_d}{D}}{10 \cdot \left(1 + 300 \cdot \frac{t_d}{s_d}\right)^{0.5}}$$

$$K_{sp} = \begin{pmatrix} 0.226 \\ 0.226 \\ 0.222 \\ 0.222 \\ 0.222 \end{pmatrix}$$

Ksp has a factor of safety of 3.0 in the CGS method. Remove in calculation of pile tip resistance, below.

Geotechnical tip resistance.

$$q_{p\_1} := 3 \cdot q_{u\_1} \cdot K_{sp} \cdot dd$$

$$q_{p\_1} = \begin{pmatrix} 1953 \\ 1951 \\ 1920 \\ 1918 \\ 1916 \end{pmatrix} \cdot ksf$$

Nominal geotechnical tip resistance, Rp - *Extreme Limit States and Service Limit States*

Case I

$$R_{p\_1} := \overrightarrow{\left(q_{p\_1} \cdot A_s\right)}$$

$$R_{p\_1} = \begin{pmatrix} 210 \\ 295 \\ 285 \\ 348 \\ 458 \end{pmatrix} \cdot kip$$

Factored Axial Geotechnical Compressive Resistance - *Strength Limit States*

Resistance factor, end bearing on rock Candadian Geotechnical Society method

$$\phi_{stat} := 0.45$$

LRFD Table 10.5.5.2.3-1

Factored Geotechnical Tip Resistance (Rr)

$$R_{r\_p1} := \phi_{stat} \cdot R_{p\_1}$$

$$R_{r\_p1} = \begin{pmatrix} 95 \\ 133 \\ 128 \\ 156 \\ 206 \end{pmatrix} \cdot kip$$

Drivability Analysis

Ref: LRFD Article 10.7.8

For steel piles in compression or tension, driving stresses are limited to 90% of fy

$\phi_{da} := 1.0$ resistance factor from LRFD Table 10.5.5.2.3-1, Drivablity Analysis, steel piles

$\sigma_{dr} := 0.90 \cdot 50 \cdot (ksi) \cdot \phi_{da}$

$\sigma_{dr} = 45 \cdot ksi$ driving stress cannot exceed 45 ksi

Limit driving stress to 45 ksi or limit blow count to 6-10 bpi which is optimal for diesel hammers

Compute the resistance that can be achieved in a drivablity analysis:

The resistance that must be achieved in a drivablity analysis will be the maximum factored pile load divided by the appropriate resistance factor for wave equation analysis and dynamic test which will be required for construction.

$\phi_{dyn} := 0.65$

Pile Size is 12 x 53

The 12x53 pile can be driven to the resistances below with a D 19-42 at a reasonable blow count and level of driving stress. See GRLWEAP results below:

State of Maine Dept. Of Transportation
12 x 53 Delmag D19-42

06-Apr-2011
GRLWEAP (TM) Version 2003

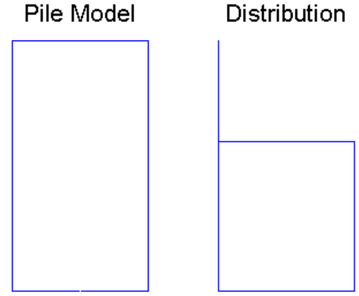
Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
100.0	21.50	0.03	1.1	5.40	14.97
200.0	29.10	0.34	2.4	6.52	13.96
300.0	36.25	1.78	3.9	7.18	14.48
350.0	41.10	2.19	4.7	7.57	15.21
370.0	42.89	2.36	5.0	7.72	15.45
390.0	44.43	2.31	5.5	7.83	15.63
400.0	45.37	2.30	5.7	7.94	15.81
450.0	49.20	3.31	6.8	8.36	16.59

DELMAG D 19-42

Efficiency	0.800	
Helmet	2.70	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	25.00	ft
Pile Penetration	15.00	ft
Pile Top Area	15.50	in2

Pile Model

Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Limiting driving stress to 45 ksi:

$$R_{ndr} := \left(\frac{45 - 42.89}{45.37 - 42.89}\right) \cdot (400 \cdot kip - 390 \cdot kip) + 390 \cdot kip$$

$$R_{ndr} = 398.5 \cdot kip$$

$$R_{fdr} := R_{ndr} \cdot \phi_{dyn}$$

$$\boxed{R_{fdr} = 259 \cdot kip}$$

Pile Size is 12 x 74

The 12x 74 pile can be driven to the resistances below with a D 19-42 at a reasonable blow count and level of driving stress. See GRLWEAP results below:

State of Maine Dept. Of Transportation — 05-Apr-2011
12 x 74 Delmag D19-42 — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
200.0	22.78	0.35	2.6	6.55	13.97
300.0	29.01	0.76	4.3	7.08	13.99
400.0	33.74	1.26	6.3	7.61	14.64
500.0	37.44	2.39	9.5	8.16	15.39
510.0	37.87	2.53	9.9	8.23	15.47
520.0	38.28	2.66	10.3	8.32	15.63
530.0	38.63	2.79	10.7	8.39	15.78
540.0	39.02	2.92	11.1	8.46	15.91

Limit blow count to 10 bpi

$R_{ndr} := 510 \cdot kip$

$R_{fdr} := R_{ndr} \cdot \phi_{dyn}$

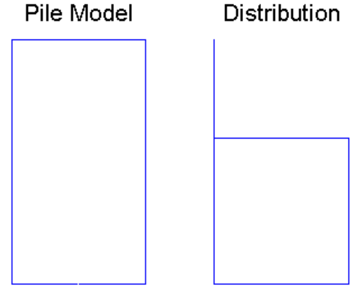
For a resistance factor for dynamic test of 0.65:

$\boxed{R_{fdr} = 332 \cdot kip}$ use this

DELMAG D 19-42

Efficiency	0.800	
Helmet	2.70	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.100	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	25.00	ft
Pile Penetration	15.00	ft
Pile Top Area	21.80	in2

Pile Model — Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Pile Size is 14 x 73

14 x 73 with Delmag 19-42 and 2.7 kip helmet

State of Maine Dept. Of Transportation — 05-Apr-2011
14 x 74 Delmag D19-42 — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
200.0	24.19	0.10	2.5	6.55	13.74
400.0	34.21	2.61	5.6	7.50	14.11
550.0	41.12	3.12	9.0	8.58	15.79
550.0	41.12	3.12	9.0	8.58	15.79
570.0	41.85	3.18	9.6	8.70	15.98
580.0	42.24	3.23	9.9	8.76	16.10
590.0	42.65	3.26	10.3	8.84	16.21
600.0	43.05	3.30	10.6	8.91	16.36
620.0	43.88	3.39	11.3	9.07	16.68

DELMAG D 19-42

Efficiency	0.800	
Helmet	2.70	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	25.00	ft
Pile Penetration	15.00	ft
Pile Top Area	21.40	in2

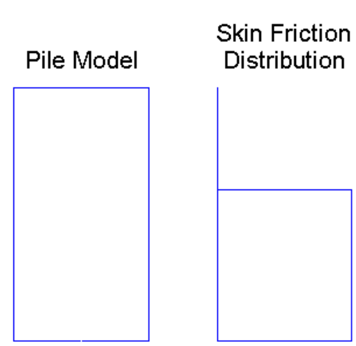
Limiting driving resistance to 10 bpi:

$R_{ndr} := 580 \cdot kip$

For a resistance factor for dynamic test of 0.65:

$R_{fdr} := R_{ndr} \cdot \phi_{dyn}$

$\boxed{R_{fdr} = 377 \cdot kip}$



Res. Shaft = 10 %
(Proportional)

Pile Size is 14 x 89

The 14 x 89 pile can be driven to the resistances below with a D 19-42 at a reasonable blow count and level of driving stress. See GRLWEAP results below:

State of Maine Dept. Of Transportation | 05-Apr-2011
14 x 89 Delmag D19-42 | GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
300.0	26.21	0.48	4.0	7.03	13.43
400.0	30.69	1.52	5.6	7.47	13.77
570.0	37.69	3.57	9.2	8.48	15.20
580.0	37.83	3.78	9.7	8.46	15.09
590.0	38.22	3.94	9.9	8.52	15.21
600.0	38.56	4.02	10.3	8.58	15.29
610.0	38.91	4.09	10.6	8.64	15.37

Limiting driving resistance to 10 bpi

$R_{ndr} := 590 \cdot kip$

For a resistance factor for dynamic test of 0.65:

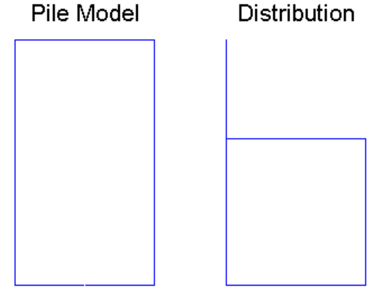
$R_{fdr} := R_{ndr} \cdot \phi_{dyn}$

$\boxed{R_{fdr} = 384 \cdot kip}$

DELMAG D 19-42

Efficiency	0.800
Helmet	2.70 kips
Hammer Cushion	109975 kips/in
Skin Quake	0.100 in
Toe Quake	0.040 in
Skin Damping	0.050 sec/ft
Toe Damping	0.150 sec/ft
Pile Length	25.00 ft
Pile Penetration	15.00 ft
Pile Top Area	26.10 in2

Pile Model | Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Pile Size is 14 x 117

The 14 x 117 pile can be driven to the resistances below with a D 36-32 at Fuel Setting 3 and a 2.7 kip helmet, at a reasonable blow count and level of driving stress. See GRLWEAP results below:

State of Maine Dept. Of Transportation — 05-Apr-2011
14 x 89 Delmag D36-32 — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
300.0	28.65	0.05	1.7	6.59	31.41
400.0	32.77	0.00	2.4	7.13	30.22
500.0	37.39	0.38	3.1	7.46	29.84
600.0	41.50	1.04	3.8	7.86	30.81
680.0	44.77	1.64	4.3	8.20	31.66
690.0	45.14	1.72	4.4	8.24	31.89
700.0	45.41	1.77	4.5	8.29	31.97

Limiting driving stress to 45 ksi:

$$R_{ndr} := \left(\frac{45 - 44.77}{44.14 - 44.77}\right)\cdot(690\cdot kip - 680\cdot kip) + 680\cdot kip$$

$$R_{ndr} = 676.3\cdot kip$$

For a resistance factor for dynamic test of 0.65:

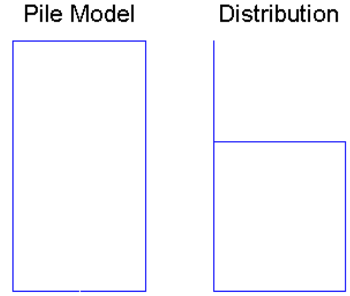
$$R_{fdr} := R_{ndr}\cdot\phi_{dyn}$$

$$\boxed{R_{fdr} = 440\cdot kip}$$

DELMAG D 36-32

Efficiency	0.800	
Helmet	2.70	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	25.00	ft
Pile Penetration	15.00	ft
Pile Top Area	34.40	in2

Pile Model — Skin Friction Distribution



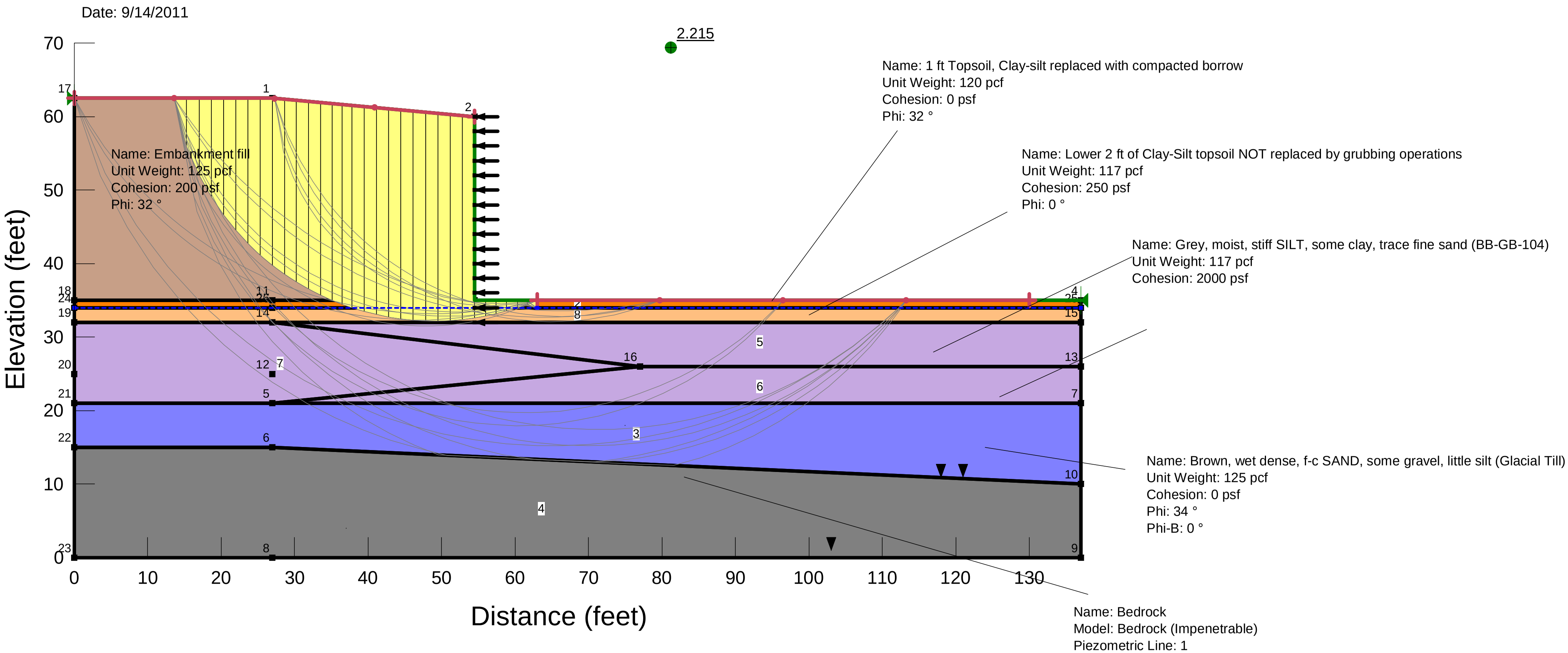
Res. Shaft = 10 %
(Proportional)

Slope Stability Analyses

Slope Stability Analysis Location	Factor of Safety	Sheet # (follows)
Abutment 1, South to North failure along CL of Rte. 180 into streambed. • 27.5 feet of new fill. • Top 1 foot of 3 feet of topsoil (silt clay) is grubbed and replaced with compacted granular borrow. • Soil Profile modeled on BB-GB-104 • Push tangents of failure surface to below El. 103 ft = Bottom of Arch Pile Cap or Elev. 28 in GeoSlope y-axis.	2.2	1
Abutment 1, Left to Right failure at Sta. 1071+85, 25° skew at southeast wingwall. • 27.5 feet of new fill. • Top 1 foot of 3 feet of topsoil (silt clay) is replaced with compacted granular borrow. • Soil Profile modeled on BB-GB-104 • Push tangents of failure surface to below El. 104 = BOF of CIP wingwall or Elev. 29' on GeoSlope Y-axis.	2.6	2
Abutment 2, Left to Right failure at Sta. 1072+02, 25° skew. • Height of new fill retained by Northeast Wingwall • 27.5 feet of new fill. • Top 1 foot of 3 feet of topsoil (silt clay) replaced with compacted granular borrow. • Soil Profile modeled on BB-GB-102 at CL and BB-GB-103 outboard of NE wingwall. • Push tangents of failure surface to below El. 104' = BOF of CIP wingwall or Elev. 29' on GeoSlope Y-axis.	2.0	3
Abutment 2, Left to Right failure at STA 1072+02, 25° skew. • Same as analysis above, but did not restrict tangents of failure surface to below the bottom of the proposed NE wingwall footing. Just modeled confining stem wall of the NE wingwall with point loads.	1.7	4

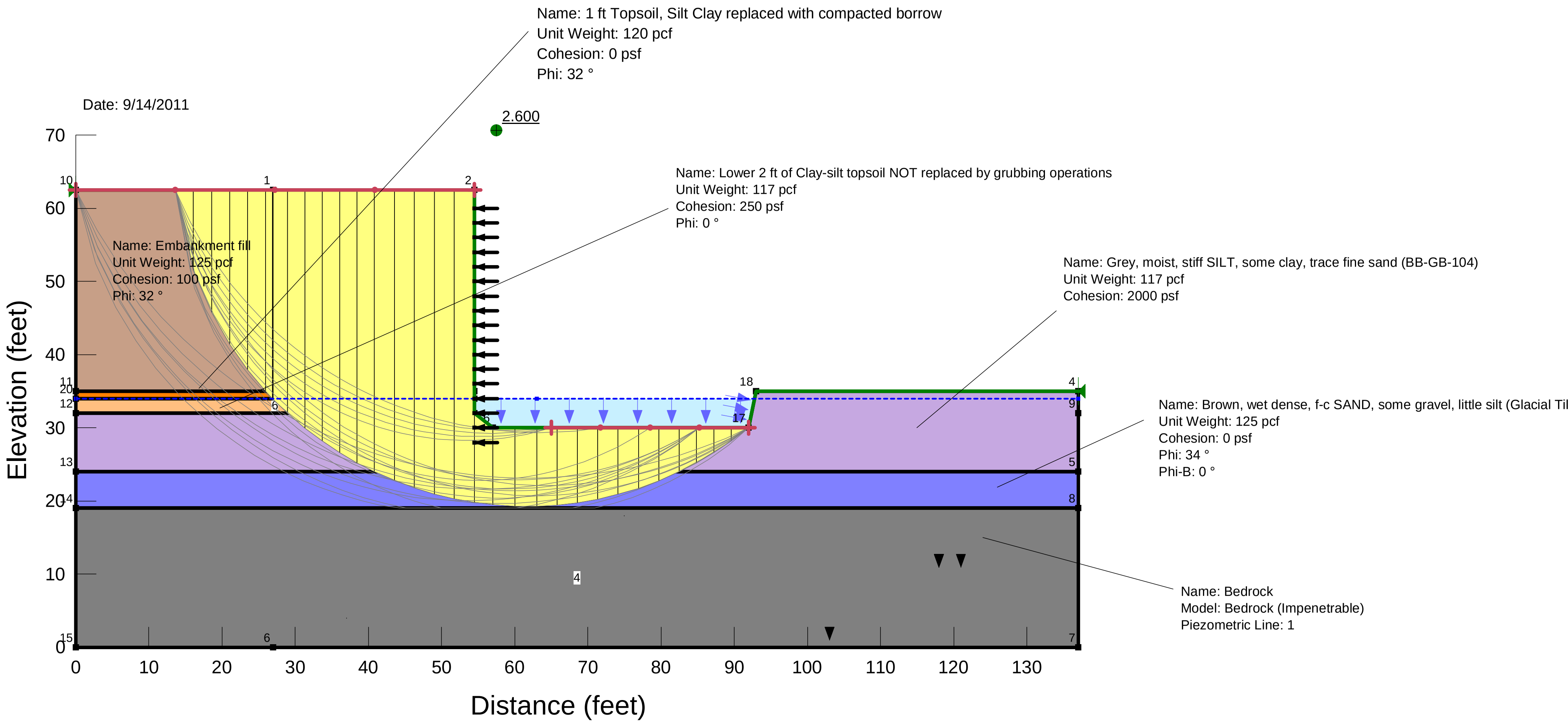
Title: L to Right Failure at Sta 1071+85, 25 deg skew
Comments: Conservative, wall stem and footing will be lower at Elev. 104 or y=29 ft here. Add cohesion to embankment soils to prevent internal failure surfaces "through the walls".
Date: 9/14/2011
File Name: Ellsworth Southeast Wingwall (11).gsz

Description: Push exit of failure surfaces to x=63 and beyond, to push slip surfaces to y=32 or to Elev. 107 in the lower zone of 2 ft of soft clay-silt. Add point loads for CIP wall stem and base down to Elev. 107



Title: South to North Failure along CL of Roadway into streambed- Model Soil Profile with BB-GB-104
Comments: Topsoil + surficial soft clayey silt excavated and replaced with compacted granular borrow.
Date: 9/14/2011
File Name: Ellsworth South Arch Abut (1) rev3.gsz

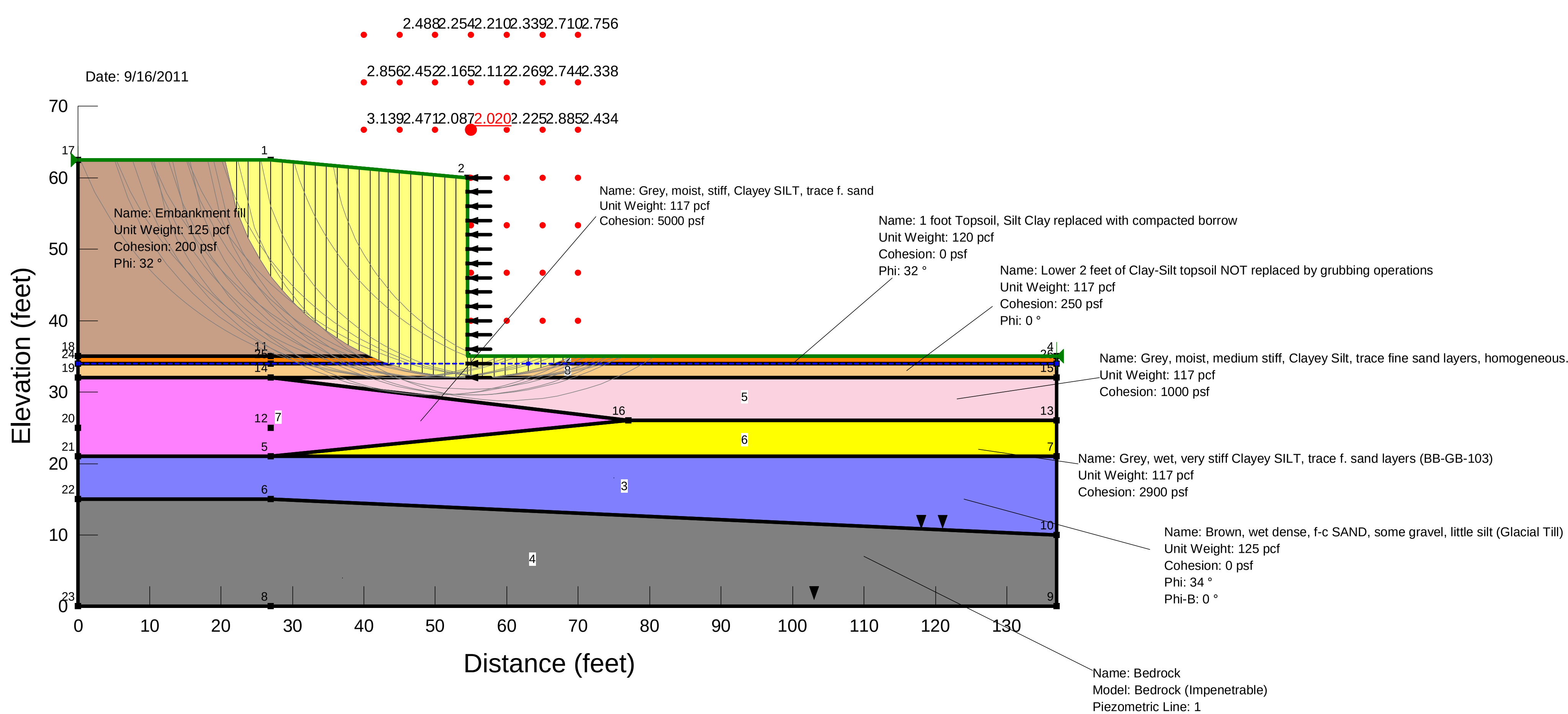
Description: push tangents of failures to El 103 = Bottom of Arch Pile Cap, or 28 Elev. ft here. Add point loads for arch wall stem and pile cap down to Elev. 103. Add cohesion to Embankment fill to prevent sloughs at top of wall.



Title: L to Right Failure at Sta 1072_02, 25 deg skew
Comments: 1 foot of topsoil (surficial soft clayey silt) excavated and replaced with compacted granular borrow.
Date: 9/16/2011
File Name: Ellsworth North East Wingwall (6 rev1).gsz

Description: push tangents of failures to El 104 , BOF, or 29 ft here. Add point loads for CIP wall stem and base down to Elev. 104. Add cohesion to Embankment fill to prevent sloughs at top of wall.

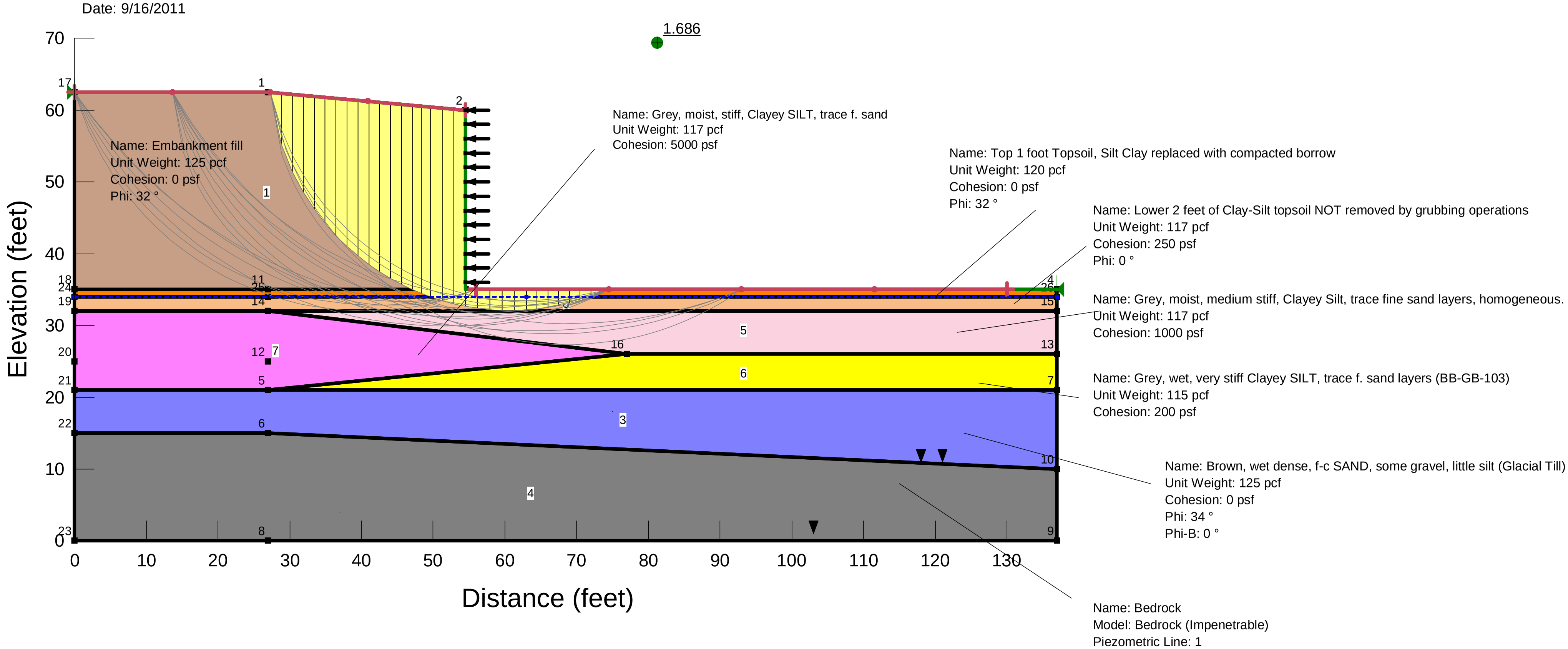
Shallow slip surfaces allowed to Y = 32 ft or Elev. 107 ft Footing will actually be placed at Elev. 104 ft. Conservative result shown here.



Title: L to Right Failure at Sta 1072_02, 25 deg skew
Comments: Topsoil + surficial soft clayey silt excavated and replaced with compacted granular borrow.
Date: 9/16/2011
File Name: Ellsworth North East Wingwall (4 rev1).gsz

Name: Ellsworth North East Wingwall (4 rev1).gsz

Date: 9/16/2011



By: Laura Krusinski
Date: 5/2011
Check By: MJM 7/2011

Consolidation Settlement Analysis

Estimation of Compression and Recompression Indexes – 4 sheets

FoSSA – Foundation Stress & Settlement Analysis - 3 sheets

Estimation of Compression Index & Recompression Index for Clayey Silt Units, OCR and input parameters for Consolidation Settlement Analyses

Thickness of silt clay subunit	Boring	N (bfp)	Vane test	Correlation of N to Su
8 ft	BB-GB-101	8		1000 psf
2 ft	BB-GB-101	11		1000 psf
6 ft	BB-GB-102	12	> 5658 psf	(1500 psf)
6 ft	BB-GB-102	32	> 5658 psf	(>4000 psf)
5 ft	BB-GB-103	8		1000 psf
5 ft	BB-GB-103	4	3000 psf	(500 psf)
4 ft	BB-GB-104	18		2000 psf
4 ft	BB-GB-104	14		2000 psf

Dismiss Correlations of Su to N in paretheses.

Average Su for upper 5 feet of Marine Clay

$$S_{u\_avg\_upper} := \frac{1000 + 5685 + 1000 + 2000}{4} \cdot psf \qquad S_{u\_avg\_upper} = 2421 \cdot psf$$

Average Su for lower 5 feet of Marine Clay (dismiss one test >5658 psf)

$$S_{u\_avg\_lower\_5\_ft} := \frac{1000 + 3000 + 2000}{3} \cdot psf \qquad S_{u\_avg\_lower\_5\_ft} = 2000 \cdot psf$$

Estimate Compression Index (Cc) and Recompression Index (Cr)

1. Consolidation Test Data from OC clay at Norton Bridge, Carmel

$C_c := 0.157$ $\qquad$ $C_r := 0.02$

2. Correlations

From A Summary of Geotechnical Engineering Information on the Presumpscot Formation Silty Clay, Andrews, (1996)

Cc = 0.18 - 0.34 - Bangor Area Clayey Silt
Cr = 8-10% of Cc

Use lower Cc since OC clay; use Cr higher to estimate highest consolidation settlement

Cc $C_c := 0.18$

Cr $C_r := 0.10 \cdot C_c$ $C_r = 0.018$

<u>Shansep Method to Backcalculate OCR</u>

Shansep Method - Reference Ladd (1991) for S and m variables

$S := 0.22$ for Maine silt clays

$m := 0.88 \cdot \left(1 - \frac{C_r}{C_c}\right)$ $m = 0.792$

Upper 5 foot Silt Clay subunit - for calculation of existing effective overburden pressure at the center of the upper 5 foot subunit, use soil profile (layer thicknesses) at BB-GB-102 and BB-GB-104

$$\sigma'_{vo} := (3 \cdot ft) \cdot 120 \cdot pcf + (1 \cdot ft) \cdot 115 \cdot pcf + [1.5 \cdot ft \cdot (115 \cdot pcf - 62.4 \cdot pcf)]$$

$$\sigma'_{vo} = 553.9 \cdot psf$$

$$OCR_{shan2} := \left(\frac{S_{u\_avg\_upper}}{0.22 \cdot \sigma'_{vo}}\right)^{1.299} \qquad OCR_{shan2} = 48.567$$

OCR >10 - The upper clay silt is overconsolidated.

Lower 5 foot Silt Clay subunit - for calculation of existing effective overburden pressure at the center of the lower 5 foot subunit, use soil profile (layer thicknesses) at BB-GB-102 and BB-GB-104

$$\sigma'_{vo} := (3 \cdot ft) \cdot 120 \cdot pcf + (1 \cdot ft) \cdot 115 \cdot pcf + [4 \cdot ft \cdot (115 \cdot pcf - 62.4 \cdot pcf)] + [2.5 \cdot ft \cdot (115 \cdot pcf - 62.4 \cdot pcf)]$$

$$\sigma'_{vo} = 816.9 \cdot psf$$

$$OCR_{shan2} := \left(\frac{S_{u\_avg\_lower\_5\_ft}}{0.22 \cdot \sigma'_{vo}} \right)^{1.299} \qquad OCR_{shan2} = 22.873$$

OCR > 10 - The lower clay silt is overconsolidated.

For the Fossa Consolidation Settlement Analysis, assume an initial void ratio based on the Carmel, Norton Bridge Site

Initial void ratio $e_o := 0.70$

Also check estimates of Cc and Cr for the 2 clayey silt units based on LL correlations

Correlations

Cc = -.5077 + 0.937e - 0.086 x LL Bangor Area Samples, Young (1966) per Andrews (1986)

Cc = 0.009(LL-10%) Terzaghi and Peck

Cr = 8-10% of Cc Andrews (1986)

Approx. 10 foot thick Clayey Silt:

$$LL := \begin{pmatrix} 31 \\ 30 \\ 29 \\ 29 \end{pmatrix}$$

Terzaghi Correlation

$$Cc := .009 \cdot (LL - 10) \qquad Cc = \begin{pmatrix} 0.189 \\ 0.18 \\ 0.171 \\ 0.171 \end{pmatrix}$$

$$Cr := Cc \cdot 0.10 \qquad Cr = \begin{pmatrix} 0.019 \\ 0.018 \\ 0.017 \\ 0.017 \end{pmatrix}$$

Young Correlation

$$Cc := -0.5506 + 0.937 \cdot 2 - 0.086 \cdot LL$$

$$Cc = \begin{pmatrix} -1.343 \\ -1.257 \\ -1.171 \\ -1.171 \end{pmatrix}$$

$$Cr := Cc \cdot 0.10$$

$$Cr = \begin{pmatrix} -0.134 \\ -0.126 \\ -0.117 \\ -0.117 \end{pmatrix}$$

The check with the Terzaghi Correlation indicates selected values ok.

By: L. Krusinski
March 20, 2011
Check by: MJM 6/2011

Greys Brook Bridge
Ellsworth, Maine
PIN 10063.10
Consolidation Settlement Analysis - Fossa

Grey's Brook Bridge, Ellsworth

PROJECT IDENTIFICATION

Title:	Grey's Brook Bridge, Ellsworth
Project Number:	10063.10 -
Client:	Calderwood Engr.
Designer:	Laura Krusinski
Station Number:	

Description:

Company's information:

Name:	MaineDOT
Street:	
	,
Telephone #:	
Fax #:	
E-Mail:	

Original file path and name: D:\O2KFile ..... ddocs\10063-10 Ellsworth\fossa run Ellsworth 1.F2S
Original date and time of creating this file: Mon Apr 11 14:01:44 2011

GEOMETRY: Analysis of a 2D geometry

By: L. Krusinski
March 20, 2011
Check by: MJM 6/2011

Greys Brook Bridge
Ellsworth, Maine
PIN 10063.10
Consolidation Settlement Analysis - Fossa

TABULTAED GEOMETRY INPUT OF FOUNDATION SOILS

Found. Soil #	Point #	Coordinates (X, Z) : (X) [ft.]	(Z) [ft.]	DESCRIPTION
1	1	328.08	330.00	Top soil
2	1	328.08	328.00	Upper OC Clay Silt
3	1	328.08	323.00	Lower OC Clay Silt
4	1	328.08	318.00	Glacial Till

TABULTAED GEOMETRY INPUT OF EMBANKMENT SOILS

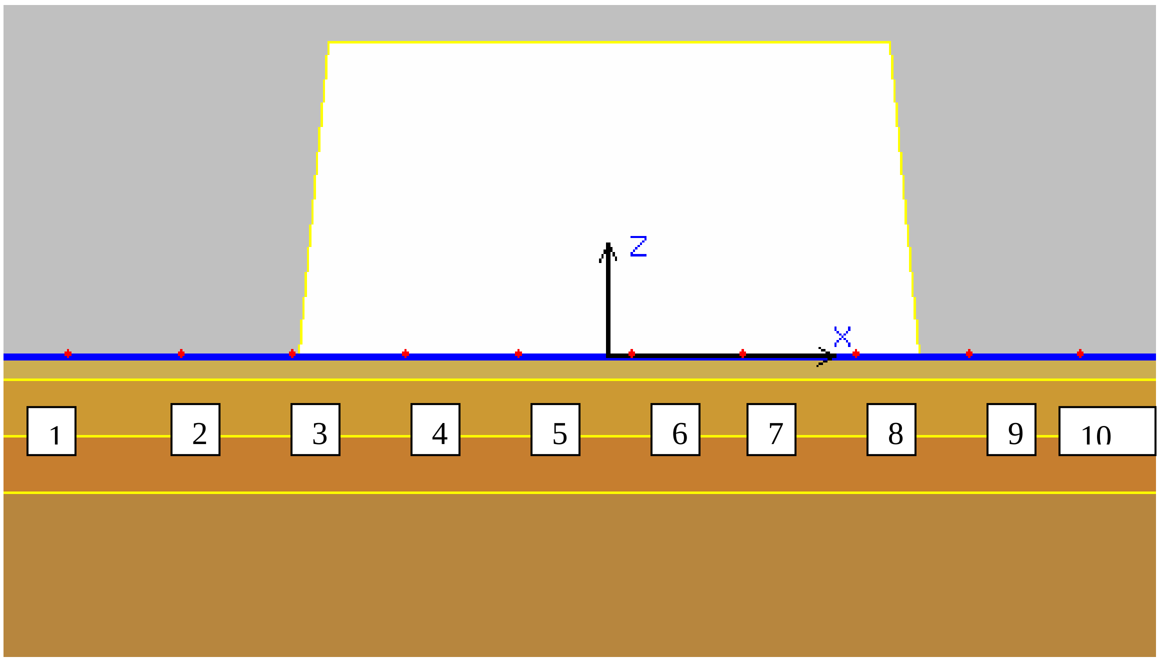
Embank. Soil #	Point #	Coordinates (X, Z) : (X) [ft.]	(Z) [ft.]	DESCRIPTION
1	1	353.13	358.00	Embankment Fill
	2	3701.46	358.00	

INPUT DATA FOR CONSOLIDATION -- $\alpha = 1/6$

Layer #	Underging Consolidation [Yes/No]	OCR = Pc / Po	Cc	Cr	e0	Cv [ft²/day]	Drains at :
1	No	N/A	N/A	N/A	N/A	N/A	N/A
2	Yes	10.00	0.18	0.02	0.70	0.5000	Top & Bot.
3	Yes	10.00	0.18	0.02	0.70	0.3000	Top & Bot.
4	No	N/A	N/A	N/A	N/A	N/A	N/A

By: L. Krusinski
March 20, 2011
Check by: MJM 6/2011

Greys Brook Bridge
Ellsworth, Maine
PIN 10063.10
Consolidation Settlement Analysis - Fossa



Settlement at Nodes 1 though 10 provided in the Table below

FoSSA -- Foundation Stress & Settlement Analysis
Present Date/Time: Mon Oct 03 13:34:29 2011

Grey's Brook Bridge, Ellsworth
D:\CKFiles\worddocs\10063-10 Ellsworth\fossa run Ellsworth1.FT8

ULTIMATE SETTLEMENT, Sc

Node #	X [ft.]	Y [ft.]	Original Z [ft.]	Settlement Sc [ft.]	Final Z * [ft.]	Total Settlement
1	280.00	0.00	330.00	0.00	330.00	0.1 in
2	290.00	0.00	330.00	0.01	329.99	0.12 in
3	300.00	0.00	330.00	0.06	329.94	0.72 in
4	310.00	0.00	330.00	0.18	329.82	2.16 in
5	320.00	0.00	330.00	0.19	329.81	2.28 in
6	330.00	0.00	330.00	0.19	329.81	2.28 in
7	340.00	0.00	330.00	0.19	329.81	
8	350.00	0.00	330.00	0.17	329.83	2.04 in
9	360.00	0.00	330.00	0.03	329.97	0.36 in
10	370.00	0.00	330.00	0.01	329.99	0.12 in

*Note: Final Z is calculated assuming only 'Ultimate Settlement' exists.

Method 1 - MaineDOT Design Freezing Index (DFI) Map and Depth of Frost Penetration Table, BDG Section 5.2.1.

From Design Freezing Index Map:
Ellsworth, Maine

DFI = 1400 degree-days

Case I - Medium to coarse grained fill soils -WC=10%.

Depth of Frost Penetration = 79.2 inch

$d := 79.2 \cdot in \qquad d = 79.2 \cdot in \qquad d = 6.6 \cdot ft$

Method 2 - ModBerg Software

Examine coarse grained soils without 4 inches of asphalt

--- ModBerg Results ---

Project Location: Ellsworth, Maine

Air Design Freezing Index = 1256 F-days
N-Factor = 0.80
Surface Design Freezing Index = 1005 F-days
Mean Annual Temperature = 44.6 deg F
Design Length of Freezing Season = 126 days

Layer #:Type	t	w%	d	Cf	Cu	Kf	Ku	L
1-Coarse	62.3	10.0	120.0	26	32	1.7	1.5	1,728

t = Layer thickness, in inches.
w% = Moisture content, in percentage of dry density.
d = Dry density, in lbs/cubic ft.
Cf = Heat Capacity of frozen phase, in BTU/(cubic ft degree F).
Cu = Heat Capacity of thawed phase, in BTU/(cubic ft degree F).
Kf = Thermal conductivity in frozen phase, in BTU/(ft hr degree).
Ku = Thermal conductivity in thawed phase, in BTU/(ft hr degree).
L = Latent heat of fusion, in BTU / cubic ft.

****
Total Depth of Frost Penetration = 5.20 ft = 62.3 in.
****

Recommendation:6.0 feet for design of spread footings constructed on soil

Appendix D

Special Provisions

SPECIAL PROVISION
SECTION 635
PREFABRICATED CONCRETE MODULAR GRAVITY WALL

The following replaces Section 635 in the Standard Specifications in its entirety:

635.01 Description. This work shall consist of the construction of a prefabricated modular reinforced concrete gravity wall in accordance with these specifications and in reasonably close conformance with the lines and grades shown on the plans, or established by the Resident.

Included in the scope of the Prefabricated Concrete Modular Gravity Wall construction are**:** all grading necessary for wall construction, excavation, compaction of the wall foundation, backfill, construction of leveling pads, placement of geotextile, segmental unit erection, and all incidentals necessity to complete the work.

The Prefabricated Concrete Modular Gravity Wall design shall follow the general dimensions of the wall envelope shown in the contract plans. The top of the leveling pad shall be located at or below the theoretical leveling pad elevation. The minimum wall embedment shall be at or below the elevation shown on the plans. The top of the face panels shall be at or above the top of the panel elevation shown on the plans.

The Contractor shall require the design-supplier to supply an on-site, qualified experienced technical representative to advise the Contractor concerning proper installation procedures. The technical representative shall be on-site during initial stages of installation and thereafter shall remain available for consultation as necessary for the Contractor or as required by the Resident. The work done by this representative is incidental.

635.02 Materials. Materials shall meet the requirements of the following subsections of Division 700 - Materials:

Gravel Borrow	703.20
Preformed Expansion Joint Material	705.01
Reinforcing Steel	709.01
Structural Pre-cast Concrete Units	712.061
Drainage Geotextile	722.02

The Contractor is cautioned that all of the materials listed are not required for every Prefabricated Concrete Modular Gravity Wall. The Contractor shall furnish the Resident a Certificate of Compliance certifying that the applicable materials comply with this section of the specifications. Materials shall meet the following additional requirements:

Concrete Units:

Tolerances. In addition to meeting the requirements of 712.061, all prefabricated units shall be manufactured with the following tolerances. All units not meeting the listed tolerances will be rejected.

1. All dimensions shall be within (edge to edge of concrete) ±3/16 inch.
2. Squareness. The length differences between the two diagonals shall not exceed 5/16 inch.
3. Surface Tolerances. For steel formed surfaces, and other formed surface, any surface defects in excess of 0.08 inch in 4 feet will be rejected. For textured surfaces, any surface defects in excess of 5/16 inch in 5 feet shall be rejected.

Joint Filler. (where applicable) Joints shall be filled with material approved by the Resident and supplied by the approved Prefabricated Concrete Modular Gravity Wall supplier. 4 inches wide, by 0.5 inch preformed expansion joint filler shall be placed in all horizontal joints between facing units. In all vertical joints, a space of 0.25 inch shall be provided. All Preformed Expansion Joint Material shall meet the requirements of subsection 502.03.

Woven Drainage Geotextile. Woven drainage geotextile 12 inches wide shall be bonded with an approved adhesive compound to the back face, covering all joints between units, including joints abutting concrete structures. Geotextile seam laps shall be 6 inches, minimum. The fabric shall be secured to the concrete with an adhesive satisfactory to the Resident. Dimensions may be modified per the wall supplier's recommendations, with written approval of the Resident.

Concrete Shear Keys. (where applicable) Shear keys shall have a thickness at least equal to the pre-cast concrete stem.

Concrete Leveling Pad. Cast-in-place concrete shall be Fill Concrete conforming to the requirements of Section 502 Structural Concrete. The horizontal tolerance on the surface of the pad shall be 0.25 inch in 10 feet. Dimensions may be modified per the wall supplier's recommendations, with written approval of the Resident.

Backfill and Bedding Material. Bedding and backfill material placed behind and within the reinforced concrete modules shall be gravel borrow conforming to the requirements of Subsection 703.20. The backfill materials shall conform to the following additional requirements: backfill and bedding material shall only contain particles that will pass the 3-inch square mesh sieve and the plasticity index (PI) as determined by AASHTO T90 shall not exceed 6. Compliance with the gradation and plasticity requirements shall be the responsibility of the Contractor, who shall furnish a copy of the backfill test results prior to construction.

The backfilling of the interior of the wall units and behind the wall shall progress simultaneously. The material shall be placed in layers not over 8 inches in depth, loose measure, and thoroughly compacted by mechanical or vibratory compactors. Puddling for compaction will not be allowed.

Materials Certificate Letter. The Contractor, or the supplier as his agent, shall furnish the Resident a Materials Certificate Letter for the above materials, including the backfill material, in accordance with Section 700 of the Standard Specifications. A copy of all test results performed by the Contractor or his supplier necessary to assure contract compliance shall also be furnished to the Resident. Acceptance will be based upon the materials Certificate Letter, accompanying test reports, and visual inspection by the Resident.

635.03 Design Requirements. The Prefabricated Concrete Modular Gravity Wall shall be designed and sealed by a licensed Professional Engineer registered in accordance with the laws of the State of Maine. The design to be performed by the wall system supplier shall be in accordance with AASHTO LRFD Bridge Design Specifications, current edition, except as required herein. Design shall consider Strength, Service and Extreme Limit States. Thirty days prior to beginning construction of the wall, the design computations shall be submitted to the Resident for review by the Department. Design calculations that consist of computer generated output shall be supplemented with at least one hand calculation and graphic demonstrating the design methodology used. Design calculations shall provide thorough documentation of the sources of equations used and material properties. The design by the wall system supplier shall consider the stability of the wall as outlined below:

A. Stability Analysis:

1. Overturning: Location of the resultant of the reaction forces shall be within the middle one-half of the base width.
2. Sliding: $R_R \geq \gamma_{p(max)} \cdot (EH+ES)$

   Where: R_R = Factored Sliding Resistance
   $\gamma_{p(max)}$ = Maximum Load Factor
   EH = Horizontal Earth Pressure
   ES = Earth Surcharge (as applicable)
3. Bearing Pressure: $q_R \geq$ Factored Bearing Pressure

   Where: q_R = Factored Bearing Resistance, as shown on the plans
   Factored Bearing Pressure = Determined considering the applicable loads and load factors which result in the maximum calculated bearing pressure.
4. Pullout Resistance: Pullout resistance shall be determined using nominal resistances and forces. The ratio of the sum of the nominal resistances to the sum of the nominal forces shall be greater than or equal to 1.5.

Live load surcharge on Prefabricated Concrete Modular Gravity walls shall be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) taken from LRFD Table 3.11.6.4-2 with consideration for the distance from the wall pressure surface to the edge of traffic. Traffic impact loads transmitted to the wall through guardrail posts shall be calculated and applied in compliance with LRFD Section 11, where Article 11.10.10.2 is modified such that the upper 3.5 feet of concrete modular units shall be designed for an additional horizontal load of γP_{H1}, where γP_{H1}=300 lbs per linear foot of wall.

B. Backfill and Wall Unit Soil Parameters. For overturning and sliding stability calculations, earth pressure shall be assumed acting on a vertical plane rising from the back of the lowest wall stem. For overturning, the unit weight of the backfill within the wall units shall be limited to 96 pcf. For sliding analyses, the unit weight of the backfill within the wall units can be assumed to be 120 pcf. Both analyses may assume a friction angle of 34 degrees for backfill within the wall units.

These unit weights and friction angles are based on a wall unit backfill meeting the requirements for select backfill in this specification. Backfill behind the wall units shall be assumed to have a unit weight of 120 pcf and a friction angle of 30 degrees. The friction angle of the foundation soils shall be assumed to be 30 degrees unless otherwise noted on the plans.

C. Internal Stability. Internal stability of the wall shall be demonstrated using accepted methods, such as Elias' Method, 1991. Shear keys shall not contribute to pullout resistance. Soil-to-soil frictional component along stem shall not contribute to pullout resistance. The failure plane used to determine pullout resistance shall be found by the Rankine theory only for vertical walls with level backfills. When walls are battered or with backslopes > 0 degrees are considered, the angle of the failure plane shall be per Jumikus Method. For computation of pullout force, the width of the backface of each unit shall be no greater than 4.5 feet. A unit weight of the soil inside the units shall be assumed no greater than 120 pcf when computing pullout. Coulomb theory may be used.

D. Safety Against Structural Failure. Prefabricated units shall be designed for all strength and reinforcement requirements in accordance with LRFD Section 5 and LRFD Article 11.11.5.

E. External loads which affect the internal stability such as those applied through piling, bridge footings, traffic, slope surcharge, hydrostatic and seismic loads shall be accounted for in the design.

F. The maximum calculated factored bearing pressure under the Prefabricated Concrete Modular Gravity block wall shall be clearly indicated on the design drawings.

G. Stability During Construction. Stability during construction shall be considered during design, and shall meet the requirements of the AASHTO LRFD Bridge Design Specifications, Extreme Limit State.

H. Hydrostatic forces. Unless specified otherwise, when a design high water surface is shown on the plans at the face of the wall, the design stresses calculated from that elevation to the bottom of wall must include a 3 feet minimum differential head of saturated backfill. In addition, the buoyant weight of saturated soil shall be used in the calculation of pullout resistance.

I. Design Life. Design life shall be in accordance with AASHTO requirements or 75 years; the more stringent requirements apply.

J. Not more than two vertically consecutive units shall have the same stem length, or the same unit depth. Walls with units with extended height curbs shall be designed for the added earth pressure. A separate computation for pullout of each unit with extended height curbs, or extended height coping, shall be prepared and submitted in the design package described above.

635.04 Submittals. The Contractor shall supply wall design computations, wall details, dimensions, quantities, and cross sections necessary to construct the wall. Thirty (30) days prior to beginning construction of the wall, the design computations and wall details shall be submitted to the Resident for review. The fully detailed plans shall be prepared in conformance with Subsection 105.7 of the Standard Specifications and shall include, but not be limited to the following items:

A. A plan and elevation sheet or sheets for each wall, containing the following: elevations at the top of leveling pads, the distance along the face of the wall to all steps in the leveling pads, the designation as to the type of prefabricated module, the distance along the face of the wall to where changes in length of the units occur, the location of the original and final ground line.

B. All details, including reinforcing bar bending details, shall be provided. Bar bending details shall be in accordance with Department standards.

C. All details for foundations and leveling pads, including details for steps in the leveling pads, as well as allowable and actual maximum bearing pressures shall be provided.

D. All prefabricated modules shall be detailed. The details shall show all dimensions necessary to construct the element, and all reinforcing steel in the element.

E. The wall plans shall be prepared and stamped by a Professional Engineer. Four sets of design drawings and detail design computations shall be submitted to the Resident.

F. Four weeks prior to the beginning of construction, the contractor shall supply the Resident with two copies of the design-supplier's Installation Manual. In addition, the Contractor shall have two copies of the Installation Manual on the project site.

635.05 Construction Requirements

Excavation. The excavation and use as fill or disposal of all excavated material shall meet the requirements of Section 203 -- Excavation and Embankment, except as modified herein.

Foundation. The area upon which the modular gravity wall structure is to rest, and within the limits shown on the submitted plans, shall be graded for a width equal to, or exceeding, the length of the module. Prior to wall and leveling pad construction, this foundation material shall be compacted to at least 95 percent of maximum laboratory dry density, determined using AASHTO T180, Method C or D. Frozen soils and soils unsuitable or incapable of sustaining the required compaction, shall be removed and replaced.

A concrete leveling pad shall be constructed as indicated on the plans. The leveling pad shall be cast to the design elevations as shown on the plans, or as required by the wall supplier upon written approval of the Resident. Allowable elevation tolerances are +0.01 feet and -0.02 feet from the design elevations. Leveling pads which do not meet this requirement shall be repaired or replaced as directed by the Resident at no additional cost to the Department. Placement of wall units may begin after 24 hours curing time of the concrete leveling pad.

Method and Equipment. Prior to erection of the Prefabricated Concrete Modular Gravity Wall, the Contractor shall furnish the Resident with detailed information concerning the proposed construction method and equipment to be used. The erection procedure shall be in accordance with the manufacturer’s instructions. Any pre-cast units that are damaged due to handling will be replaced at the Contractor’s expense.

Installation of Wall Units. A field representative from the wall system being used shall be available, as needed, during the erection of the wall. The services of the representative shall be at no additional cost to the Department. Vertical and horizontal joint fillers shall be installed as shown on the plans.

The maximum offset in any unit joint shall be 3/4 inch. The overall vertical tolerance of the wall, plumb from top to bottom, shall not exceed 1/2 inch per 10 feet of wall height. The prefabricated wall units shall be installed to a tolerance of plus or minus 3/4 inch in 10 feet in vertical alignment and horizontal alignment.

Select Backfill Placement. Backfill placement shall closely follow the erection of each row of prefabricated wall units. The Contractor shall decrease the lift thickness if necessary to obtain the specified density. The maximum lift thickness shall be 8 inches (loose). Gravel borrow backfill shall be compacted in accordance with Subsection 203.12 except that the minimum required compaction shall be 92 percent of maximum density as determined by AASHTO T180 Method C or D. Backfill compaction shall be accomplished without disturbance or displacement of the wall units. Sheepsfoot rollers will not be allowed. Whenever a compaction test fails, no additional backfill shall be placed over the area until the lift is recompacted and a passing test achieved.

The moisture content of the backfill material prior to and during compaction shall be uniform throughout each layer. Backfill material shall have a placement moisture content less than or equal to the optimum moisture content. Backfill material with a placement moisture content in excess of the optimum moisture content shall be removed and reworked until the moisture content is uniform and acceptable throughout the entire lift. The optimum moisture content shall be determined in accordance with AASHTO T180, Method C or D. At the end of the day’s operations, the Contractor shall shape the last level of backfill so as to direct runoff of rain water away from the wall face.

635.06 Method of Measurement. Prefabricated Concrete Modular Gravity Wall will be measured by the square foot of front surface not to exceed the dimensions shown on the contract plans or authorized by the Resident. Vertical and horizontal dimensions will be from the edges of the facing units. No field measurements for computations will be made unless the Resident specifies, in writing, a change in the limits indicated on the plans.

635.07 Basis of Payment. The accepted quantity of Prefabricated Concrete Modular Gravity Retaining Wall will be paid for at the contract unit price per square foot complete in place. Payment shall be full compensation for furnishing all labor, equipment and materials including excavation, foundation material, backfill material, pre-cast concrete units hardware, joint fillers, woven drainage geotextile, cast-in-place coping or traffic barrier and technical field representative. Cost of cast-in-place concrete for leveling pad will not be paid for separately, but will be considered incidental to the Prefabricated Concrete Modular Gravity Wall.

There will be no allowance for excavating and backfilling for the Prefabricated Concrete Modular Gravity Wall beyond the limits shown on the approved submitted plans, except for excavation required to remove unsuitable subsoil in preparation for the foundation, as approved by the Resident. Payment for excavating unsuitable material shall be full compensation for all costs of pumping, drainage, sheeting, bracing and incidentals for proper execution of the work.

Payment will be made under:

Pay Item	Pay Unit
635.14 Prefabricated Concrete Modular Gravity Wall	Square Foot

SPECIAL PROVISION
SECTION 636
MECHANICALLY STABILIZED EARTH RETAINING WALL

The following replaces Standard Specification Section 636 in its entirety:

636.01 Description The work under this item shall consist of design, fabrication, furnishing, transportation, and erection of Mechanically Stabilized Earth (MSE) retaining wall system of the required type, including miscellaneous items necessary for a complete installation.

The MSE retaining walls shall consist of reinforcing strips or reinforcing mesh earth wall systems utilizing architectural precast concrete facing panels supported on cast-in-place concrete leveling pads. All reinforcing strips or mesh material shall consist of galvanized steel. The wall structures shall be dimensioned to achieve the design criteria shown on the plans and specified herein.

The MSE retaining walls shall be constructed in accordance with these specifications and in conformity with the lines, grades, design criteria, and dimensions shown on the plans or established by the Geotechnical Engineer.

636.02 Quality Assurance. The MSE retaining wall system shall be one of the approved wall systems noted in the Contract Documents.

All necessary materials, except backfill and cast in-place concrete shall be obtained from the approved system designer.

Mechanically Stabilized Earth (MSE) retaining walls shall be designed and constructed as specified herein. The design shall be subject to review and acceptance by the Geotechnical Engineer. The acceptability of a MSE retaining wall design shall be at the sole discretion of the Geotechnical Engineer. Any additional design, construction or other costs arising as a result of rejection of a retaining wall design by the Geotechnical Engineer shall be borne by the Contractor.

Precast facing panels shall be manufactured in a concrete products plant with approved facilities. Before proceeding with production, precast sample units shall be provided for the Resident’s acceptance. These samples shall be kept at the plant to be used for comparison purposes during production.

All calculations and Shop Drawings shall be signed and sealed by a licensed Professional Engineer registered in accordance with the laws of the State of Maine and specializing in geotechnical construction.

The Contractor installing the MSE retaining walls shall have demonstrated experience constructing MSE walls and shall use personnel having demonstrated experience in the installation procedures recommended by the manufacturer and as specified herein.

All MSE walls shall be built in accordance with the plans and accepted shop drawings for the proposed wall systems.

A qualified representative from the wall design-supplier shall be present during construction of the MSE walls. The services of the qualified representative shall be at no additional cost to the project. The qualified experienced technical representative will advise the Contractor and the Resident concerning proper installation procedures.

The vendor's representative shall specify the required back-batter so that the final position of the wall is vertical. Furthermore, footing berms shall be placed in front of the first three (3) levels of panels erected, to maintain verticality.

636.03 Design Requirements The MSE retaining walls shall be designed to provide the grade separation shown on the plans with a service life of not less than 100 years.

The MSE wall system shall be designed in accordance with:

1. The manufacturer's requirements
2. The Contract Plans
3. The requirements specified herein
4. AASHTO LRFD Bridge Design Specifications, current edition
5. AASHTO LRFD Bridge Construction Specifications, current edition
6. FHWA-NHI-10-024, Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes – Volume I, November 2009,
7. FHWA-NHI-10-025, Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes – Volume II, November 2009,
8. FHWA-NHI-09-087, Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, November 2009.

Where conflicting requirements occur, the more stringent requirements shall govern.

The MSE wall design shall follow the general dimensions of the wall envelope shown on the plans. Base of footing elevation shall be as shown on the plans, or may be lower. All wall elements shall be within the right-of-way limits shown on the plans. The panels shall be placed so as not to interfere with drainage or other utilities, or other potential obstructions.

All appurtenances behind in front of, under, mounted upon, or passing through the wall such as drainage structures, utilities, fences, concrete parapet wall or other appurtenances shown on the plans shall be accounted for in the stability design of the wall.

Facing panels shall have tongue and groove, ship lap or similar approved connections along all joints, both vertical and horizontal. Where foundation conditions indicate large differential settlements, vertical full-height slip joints shall be provided. The shape of the panels shall be such that adjacent panels will have continuous, vertical joints, or as noted on the plans.

MSE facing panels shall be installed on cast-in-place concrete leveling pads. The top of the leveling pad shall be located at or below the theoretical leveling pad elevation. The minimum wall embedment shall be 4.0 ft as measured to the top of the leveling pad, or as shown on the plans, whichever is greater. The top of the face panels shall be at or above the top of the panel elevation shown on the plans. Where coping or barrier is used, the wall face shall extend up into the coping or barrier a minimum of 2 in.

The MSE walls shall be dimensioned so that the factored bearing pressure resistance of the foundation soils, as noted on the plans, is not exceeded. Requirements for over excavation of native foundation soils and replacement with compacted structural fill are detailed on the plans.

The design by the wall system supplier shall consider the stability of the wall as outlined below and in the Contract Documents:

(a) Failure Plane The theoretical failure plane within the reinforced soil mass shall be determined per LRFD Section 11 and be analyzed so that the soil stabilizing components extend sufficiently beyond the failure plane within the reinforced soil mass to stabilize the material. External loads which affect the internal stability such as those applied through piling, bridge footings, traffic, slope surcharge, hydrostatic, and seismic loads shall be accounted for in the design.

(b) External Stability - Load and Resistance Factors Loads and load combinations selected for design shall be consistent with AASHTO LRFD. Application of load factors shall be taken as specified in AASHTO LRFD. Sliding resistance factors and bearing resistance factors shall be consistent with LRFD Section 10. Overturning provisions of LRFD Section 11 shall apply.

MSE walls shall be designed to resist failure by instability of temporary construction slope. Passive pressure in front of the wall mass shall be assumed to be zero for design purposes. The factored applied bearing pressures under the MSE mass for each reinforced length shall be clearly indicated on the design drawing.

(c) Internal Stability - Load and Resistance Factors Evaluation of reinforcement pullout, reinforcement rupture and panel connection pullout or rupture shall be consistent with LRFD Section 11. Loads, load combinations and load factors shall be as specified in LRFD Article 11. Resistance factors for internal design shall be consistent with LRFD Article 11. Maximum reinforcement loads shall be calculated using the Simplified Method approach. Calculations for factored stresses and resistances shall be based upon assumed conditions at the end of the design life. The design life of steel soil reinforcements shall comply with LRFD Section 11.

(d) Backfill and Foundation Soils Parameters. The friction angle of the select backfill used in the reinforced fill zone for the internal stability design of the wall shall be assumed to be 34° unless noted otherwise. The friction angle of the foundation soils and random backfill shall be assumed to be 30° unless otherwise shown on the plans.

(e) Reinforcement Length. The soil reinforcement shall be the same length from the bottom to the top of each wall section. The reinforcement length defining the width of the entire reinforced soil mass may vary with wall height. The minimum length of the soil reinforcement shall be 8 ft, but shall not be less than 70 percent of the wall height, H, for walls with level surcharges, or 70 percent of H1 for walls with a sloped surcharge or walls supporting an abutment. The mechanical wall height, H or H1, shall be the vertical difference between the top of the leveling footing and the elevation at which the failure surface, as described above, intercepts the ground surface supported by the wall.

(f) Steel Reinforcement For steel reinforcements, all structural connections, tie strips and loop inserts, the following galvanization and carbon steel loss rates shall be assumed:

	Mil./year/side
Zinc galvanizing (first 2 years)	0.58
Zinc galvanizing (subsequent years to depletion):	0.16
Carbon Steel (after zinc depletion to 100 yrs):	0.47

Calculations for factored stresses and resistances in steel reinforcements and connections, including tie-strips and loop inserts, shall be based upon assumed conditions at the end of the design life. (or: The nominal long-term design strength in steel reinforcements and connections, including tie-strips and loop inserts shall be determined at the end of the service life.) The applied factored reinforcement loads shall be calculated in accordance with LRFD Section, and shall be checked against the nominal tensile strength multiplied by a resistance factor per LRFD Table 11.5.6-1. Transverse and longitudinal grid members shall be sized in accordance with ASTM A 185.

When the expected differential settlement normal to the wall exceeds 3 in, the lower level reinforcement facing connections shall be designed to accommodate the increased tensile forces due to settlement.

(g) Facing Panel Requirements

1. Facing panels shall be designed to resist compaction stresses that occur during wall erection.

2. The minimum thickness for concrete panels in the zone of embedded connections shall be 5.5 in and 3.5 in elsewhere. The minimum concrete cover shall be 1.5 in. Facing panels shall meet the design requirements of LRFD 11.10.2.3

3. The wall facing shall be designed to accommodate differential settlements of 1/100 ft.

4. The minimum spacing between adjacent panels shall be ¾ inches in order to accommodate differential settlements without impairing the appearance of the facing or compromising the structural integrity of the individual panels. Joints between panels shall be no more than 0.75 in. Joint between panels shall have a ship lap configuration or tongue and groove connection. There shall be no openings through the wall facing, except for utilities to pass through the wall. Slip joints to accommodate differential settlement shall be included where shown on the plans.

5. Where wall or wall sections intersect with an angle of 130º or less, a special vertical corner element panel shall be used. The corner element panel shall cover the joint of the panels that abut the corner and allow for independent movement of the abutting panels. Corner elements shall not be formed by connecting standard facing panels that abut the acute corner.

636.04 Materials The Contractor shall be responsible for the purchase or manufacture of the precast concrete facing panels, reinforcing mesh or strips, panel/reinforcement connections, bearing pads, joint filler, and all other necessary components. The Contractor shall furnish to the Resident the appropriate Certificates of Compliance certifying that the applicable wall materials meet the requirements of the project specifications. All materials used in the construction of the MSE retaining walls shall meet the requirements specified in the following subsections of the Maine Standard Specifications and as specified herein.

Materials not conforming to this section of the specifications, or from sources not listed in the contract documents, shall not be used without written consent from the Resident.

636.041 Reinforced Concrete Facing Panels Reinforced concrete facing panels shall meet the requirements specified in the following subsections:

Structural Precast Concrete Units	712.061
Drainage Geotextile	722.02

636.042 Precast Panel Tolerances and Surface Finish Concrete surface for the front face shall have a smooth steel formed finish, or as noted on the plans. The rear face shall have an unformed surface finish. The rear face of the panel shall be roughly screeded to eliminate open pockets of aggregate and surface distortions in excess of ¼ in. All uncoated steel projecting from the panel unit shall be galvanized in accordance with ASTM A 123/A 123M (AASHTO M 111) with a minimum coating thickness of 2 oz/ft^2.

Precast panel tolerances shall comply with the following; units that do not meet the listed tolerances will be rejected.

1. Panel dimensions (edge to edge of concrete) within ±3/16 in.
2. Panel thickness: ± ¼ in.
3. Squareness. The length difference between the two diagonals shall not exceed ½ in.
4. Distance between the centerline of dowel and dowel sleeve, and to centerline of reinforcing steel shall be ± 1/8 in.
5. Face of panel to centerline of dowel and dowel sleeve, and to centerline of reinforcing steel shall be ± 1/8 in.
6. Position of panel connection devices (Tie Strip) shall be ± 1 in.
7. Location of Coil and loop Imbeds shall be ± 1/8 in.
8. Warping of the exposed panel face shall not exceed 1/4 in. in 5 ft.
9. Surface defects on smooth-formed surfaces measured over a length of 5 ft shall not exceed 1/8 in. Surface defects on textured-finished surfaces measured over a length of 5 ft shall not exceed 5/16 in.

636.043 Reinforcing All reinforcing, tie strips, and attachment devices shall be carefully inspected to insure they are true to size and free from defects that may impair their strength and durability.

A. Reinforcing Mesh shall be shop fabricated from cold drawn steel wire conforming to the requirements of AASHTO M 32 (ASTM A 82-97) yield strength minimum of 65 ksi and shall be welded into the finished mesh fabric in accordance with AASHTO M 55 (ASTM A 185). Galvanizing shall be in accordance with AASHTO M 111 (ASTM A 123/A123M) after fabrication. The minimum coating thickness shall be 2 oz/ft^2. Any damage done to the mesh galvanization prior to the installation shall be repaired in an acceptable manner and provide a minimum galvanized coating of 2 oz/ft^2.

B. Reinforcing Strips shall be fabricated from hot rolled bars to the required shape and dimensions. Their physical and mechanical properties shall conform to AASHTO M 223 (ASTM A 572/A572M) Grade 65, or approved equal. Reinforcing strips shall be hot dipped galvanized in accordance with AASHTO M 111 (ASTM A 123/A123M) after fabrication. The minimum galvanization coating thickness shall be 2 oz/ft^2. Any damage done to the mesh galvanization prior to the installation shall be repaired 2 oz/ft^2.

C. Tie strips shall be fabricated of hot rolled steel conforming to ASTM A 1011/A1011M, Grade 50 or equivalent. Tie strips shall be hot dipped galvanized in accordance with AASHTO M 111 (ASTM A 123/A123M) after fabrication. The minimum coating thickness shall be 2 oz/ft^2.

D. The tie strips and reinforcing strips shall be cut to lengths and tolerances shown on the submitted plans. Holes for bolts shall be punched in the locations shown.

636.044 Attachment Devices

A. Steel clevis loop embeds shall be fabricated of cold drawn steel wire conforming to ASTM A 510, UNS G 10350 or AASHTO M 32 (ASTM A 82). Loop embeds shall be welded in accordance with AASHTO M 55 (ASTM A 185). Both shall have electrodeposited coatings of zinc applied in accordance with ASTM B 633.

B. Fasteners shall consist of hexagonal cap screw bolts and nuts, which are galvanized and conform to the requirements of AASHTO M 164 (ASTM A 325) or equivalent.

C. Connector pins and mat bars shall be fabricated from AASHTO M 183 (ASTM A 36/A36M) steel and welded to the soil reinforcement mats as shown on the plans. Galvanization shall conform to AASHTO M111 (ASTM A 123/A123M) with a minimum coating thickness of 2 oz/ft^2. Connector bars shall be fabricated of cold drawn steel wire conforming to the requirements of ASTM A 82 (AASHTO M 32) and galvanized in accordance with ASTM A 123/A123M.

D. Structural plate connectors and fasteners used for yokes to connect reinforcements to wall panels around pile or utility conflicts shall conform to the material requirements for reinforcing strips and fasteners in 677.042 (c).

636.045 Joint Materials Joint material shall be installed to the dimensions and thicknesses specified below, or in accordance with the plans or approved shop drawings.

A. Provide flexible foam strips for filler for vertical joints between panels, and in horizontal joints where pads are used.

B. Provide in horizontal joints between panels either preformed EPDM rubber pads conforming to ASTM D 2000 for 4AA, or 812 rubbers or neoprene elastomeric pads having a Durometer Hardness of 55±5, or high density polyethylene pads with a minimum density of 0.946 g/cm3 in accordance with ASTM D 1505.

636.046 Nonwoven Drainage Geotextile Cover all joints between panels on the back side of the wall with a geotextile fabric meeting the minimum requirements of 722.02 Class 2. Slit film and multifilament woven and resin bonded woven geotextile fabrics are not allowed for this application. The minimum width of the fabric shall be 12 in. Lap fabric at least 12 in. where splices are required. Nonwoven Drainage Geotextile shall be bonded with an approved adhesive compound to the back face covering all joints between panels. Adhesives used to hold the geotextile filter fabric material to the rear of the facing panels prior to backfill placement shall be supplied by the wall supplier and approved by the Resident.

636.047 Concrete Leveling Pad The cast-in-place leveling pad shall be constructed of Class A concrete conforming to the requirements of Section 502 - Structural Concrete. Leveling pad shall have minimum dimensions of 6 in thickness and 24 in width and be placed at the design elevation shown on the shop drawings within a 1/8 in tolerance.

636.048 Backfill Materials All backfill materials used in the MSE Walls volume shall conform to Gravel Borrow conforming to the requirements of Section 703.20, with the following additional requirements:

A. The maximum aggregate size is limited to 4 in (U.S Sieve Size - 102 mm)

B. Soundness The material shall be substantially free of shale or other soft, poor durability particles. The materials shall have a magnesium sulfate soundness loss, as determined by AASHTO T104 (ASTM C 88), of less than 30 percent after four cycles.

C. Electrochemical Requirements The backfill materials shall meet the following criteria:

Requirements		Test Methods
Resistivity	>3,000 ohm-centimeters	AASHTO T 288
pH between	Between 5 and 10, inclusive	AASHTO T 289
Chlorides	<100 parts per million	AASHTO T 291
Sulfates	<200 parts per million	AASHTO T 290
Organic Content	<1%	AASHTO T 267-86

D. The plasticity index (PI) as determined by AASHTO T90 shall not exceed 6.

E. The select backfill material shall exhibit a peak angle of internal friction of not less than 34 degrees, as determined by the standard Direct Shear Test, AASHTO T236 (ASTM D3080-72), on the portion finer than the 2 mm [#10 sieve], compacted to 95 percent of AASHTO T99, Methods C or D (with oversized correction as outlined in Note 7) at optimum moisture content. No testing is required for backfills where 80 percent of sizes are greater than 3/4 in. Before construction begins, the borrow material selected shall be subject to show conformance with this frictional requirement. Compliance with the test requirements shall be the responsibility of the Contractor, who shall furnish a copy of the backfill test results prior to construction.

636.049 Crushed Stone for Abutment Foundation and Beneath Leveling Pad Crushed stone used in the foundation layer below the abutment and beneath the concrete leveling pad shall meet the requirements of 703.22, Underdrain Backfill Material Type C.

636.050 Impervious Membrane An impervious geomembrane shall be installed near the top of the reinforced backfill to reduce the chance of water infiltrating into the reinforced backfill. The geomembrane shall be bonded to the inside face of the wall panels and extend perpendicularly from the wall face into the fill, while being parallel to the top of the wall. The membrane should be sloped to drain away from the facing and outlet beyond the reinforcing zone. The impervious geomembrane shall extend into the fill a distance of 1 ft beyond the MSE reinforcement. The geomembrane shall have a minimum thickness of 30 mil (0.03 in, 1/32 in)

The geomembrane shall have both sides textured with a rough finish to improve resistance against sliding. The texture shall be approved by the Resident before installation. The geomembrane shall be shown on the design drawings of the MSE submittal of the Contractor.

636.051 Acceptance of Material The Contractor shall furnish to the Resident a Certificate of Compliance certifying that the above materials comply with the applicable contract specifications including the backfill material, in accordance with Section 700. A copy of all test results performed by the Contractor necessary to assure contract compliance shall also be furnished to the Resident. Acceptance will be based on the Certificate of Compliance, accompanying test reports, and visual inspection by the Resident.

636.06 Submittals

A. Design computations demonstrating compliance with the criteria specified herein and shown on the plans, shall be prepared, signed and stamped by a licensed Professional Engineer licensed in the State of Maine and specializing in geotechnical engineering. Design calculations that consist of computer generated output shall be supplemented with at least one hand calculation and graphic demonstrating the design methodology used. Design calculations shall provide thorough documentation of the sources of equations used and material properties.

The design calculations shall include:

1. Statement of all assumptions made and copies of all references used in the calculations.

2. Analyses demonstrating compliance with all applicable earth, water, surcharges, seismic, or other loads, as specified herein and required by AASHTO LRFD.

3. Analyses or studies demonstrating durability and corrosion resistance of retaining wall systems for the proposed location and environment. The designer shall provide all corrosion protection devices necessary for the retaining wall to have a minimum service life of 100 years in the proposed location and environment.

B. A detailed resume of the wall designer listing similar projects with references, and demonstrating necessary experience to perform the MSE retaining wall design, including a brief description of each project that is similar in scope.

C. A detailed listing of MSE walls that the Contractor has constructed including a brief description of each project and a listing of personnel who will construct the walls demonstrating their experience in construction of MSE retaining walls. A reference shall be included for each project listed. As a minimum, the reference shall include an individual’s name, address and current phone number.

D. Manufacturer’s product data for the MSE wall system, including material, manufacture and erection specifications, all specified erection equipment necessary, details of buried MSE wall elements, special details required of reinforcing layout around drainage structures and sign foundations, structures design properties, type of backfill and details for connections between facing panels.

E. Details of precast yard and concrete mix design.

F. Shop drawing showing the configuration and all details, dimensions, quantities and cross sections necessary to construct the MSE wall, including but not limited to the following:

1. A plan view of the wall, which shall include Contract limits, stations and offsets, and the face of wall line shown on the plans.

2. An elevation view of the wall which shall include the elevation at the top of the wall at all horizontal and vertical break points and at least every 50 ft along the face of the wall, all steps in the leveling pads, the designation as to the type of retaining wall system(s), and an indication of the final ground line and calculated factored bearing pressures. The face of wall shown on the plans shall be indicated.

3. A typical cross section or cross sections showing the elevation relationship between existing ground conditions and proposed grades, and the proposed wall configuration, including details for the proposed methods for connecting to existing conditions. The sections shall also indicate the location of the face of wall shown on the plans.

4. General notes pertaining to design criteria and wall construction.

5. A listing of material quantities for each wall.

6. Details of sleeves and pipes and other embedded items to be installed through the walls.

7. Clearly indicated details for construction of walls or reinforcing elements around drainage, foundations, utilities or any other potential obstructions.

8. Details of the architectural treatment of facing panels.

9. Drainage design detail and design scheme.

10. Location of utilities.

11. Sequence and schedule of construction, including overall construction schedule.

12. Methods of excavation and backfill.

13. Method of maintaining stability of excavated trenches.

14. Method of monitoring plumbness and deviation of wall.

15. Excavation support system, if any.

16. Any acceptance testing and frequency.

17. Details and location of all necessary construction and expansion joints along the wall.

18. Connection details at the interface of the wall and any adjacent proposed cast in place retaining wall or abutment structure.

19. Details of impermeable membrane connection to abutment in roadway runoff collection system.

636.07 Delivery, Storage and Handling

A. Contractor shall check the material upon delivery to assure that the proper material has been received. A product certification should be provided with each shipment.

B. Material shall be stored above -20º F

C. Contractor shall prevent excessive mud, wet cement, epoxy and like substances which may affix themselves to the material from coming in contact with the material.

D. Material may be laid flat and stored outside for 30 days. For extended storage, material shall be stored in or beneath a trailer or covered with a colored tarpaulin to prevent long-term exposure.

636.08 Wall Excavation The excavation and use as fill disposal of all excavated material shall meet the requirements of Section 203 - Excavation and Embankment, except as modified herein. Temporary excavation support as required shall be the responsibility of the contractor.

636.09 Foundation Preparation. The foundation for the structure shall be graded level for a width equal to the length of reinforcement elements plus 5 ft, or as shown on the plans. Prior to wall construction the foundation shall be compacted with at least 10 passes of a smooth wheel vibratory roller weighing at least 10,000 lbs. Any foundation soils found to be unsuitable or incapable of sustaining the required compaction shall be removed and replaced with 703.20, Gravel Borrow. The foundation for the structure shall be approved by the Resident before erection is started.

A concrete leveling pad shall be constructed as indicated on the submitted plans. The leveling pad shall be cast to the design elevations as shown on the plans. Allowable elevation tolerances are +0.01 ft and -0.02 ft from the design elevations. Placement of wall panels may begin after 24 hours curing time of the concrete leveling pad.

636.10 Wall Erection A field representative from the proprietary wall system being used shall be available, as needed, during the erection of the wall. The services of the representative shall be at no additional cost to the project.

Precast concrete panels shall be placed so that their final position is vertical or battered as shown on the plans. The vendor representative shall specify the required back-batter so that the final position of the wall is vertical. Earth berms at the footing shall be placed to maintain the desired position of panels. For erection, panels are handled by means of lifting devices connected to the upper edge of the panel. Panels should be placed in successive horizontal lifts in the sequence shown on the approved shop drawings as backfill placement proceeds. As backfill material is placed behind the panels, the panels shall be maintained in position by means of temporary wedges or bracing according to the wall supplier’s recommendations.

Concrete facing vertical tolerances and horizontal alignment tolerances shall not exceed ¾ inch when measured with a 10 ft straightedge (¼ in/yd). During construction, the maximum allowable offset in any panel joint shall be ¾ in. The overall vertical tolerance of the wall (from top to bottom) shall not exceed ½ inch per 10 ft of wall height.

636.11 Backfill Placement Backfill shall not be placed between November 1st and April 1st. Backfill placement shall closely follow erection of each course of panels. Backfill shall be placed and compacted in such a manner as to avoid any damage or disturbance of the wall materials or misalignment of the facing panels or reinforcing elements. Any wall materials which become damaged during backfill placement shall be removed and replaced at the Contractor’s expense. Any misalignment or distortion of the wall facing panels due to placement of backfill outside the limits of this specification shall be corrected by the Contractor at his expense. Prior to the placement of the soil reinforcement, the backfill elevation after compaction shall be at the required elevation of the reinforcements. At each reinforcement level, the backfill shall be placed to the level of the connection. Backfill placement methods near the panels shall assure that no voids exist directly beneath the reinforcing element.

Gravel borrow backfill shall be compacted in accordance with Subsection 203.12 except that the minimum required compaction shall be 92 percent of maximum density as determined by AASHTO T180, Method C or D (with oversize correction, as outlined in Note 7 of that test). If 30 percent or more of the backfill material is greater than 19 mm [3/4 in] in size, AASHTO T180 is not applicable, and the acceptance criterion for control of compaction shall be either a minimum of 70 percent of the relative density of the material as determined by ASTM D4253 and D4254, or a method of compaction consisting of at least 4 (four) passes by a heavy roller.

Where spread footings support bridge or other structural loads, the top 5 ft below the bottom of footing elevation shall be compacted to 98 percent of the maximum density as determined by AASHTO T180, Method C or D (with oversize correction, as outlined in Note 7 of that test).

The moisture content (determined in accordance with AASHTO T180, Method C or D) of the backfill material prior to and during compaction shall be uniformly distributed throughout each layer. Backfill materials shall be placed at a moisture content not more than 2 percentage points less than or equal to the optimum moisture content. Backfill material with a placement moisture content in excess of the optimum moisture content shall be removed and reworked until the moisture content is uniformly acceptable throughout the entire lift.

At each reinforcing level, backfill shall be leveled before placing and bolting the reinforcing. The maximum lift thickness after compaction shall not exceed 12 in. The Contractor shall decrease this lift thickness, if necessary, to obtain the specified density.

Heavy compaction equipment shall not be used to compact backfill within 3 ft of the wall face. Compaction within 3 ft of the back face of the wall shall be achieved by at least three (3) passes of lightweight mechanical tamper, lightweight roller, or vibratory system. The specified lift thickness shall be adjusted as warranted by the type of compaction equipment actually used. No vehicular equipment shall be operated within 3 ft of the panels.

The frequency of sampling of the backfill material necessary to assure gradation control throughout construction shall be as directed by the Resident.

At the end of each day's operation, the Contractor shall slope the last level of the backfill away from the wall facing to rapidly direct runoff away from the wall face. In addition, the Contractor shall not allow surface runoff from adjacent areas to enter the wall construction site.

<u>636.12 Reinforcement Placement</u> Prior to placing the first layer of reinforcements (strips, mats or grids), backfill shall be placed and compacted in accordance with Subsection 677.11, Backfill Placement.

Bending of reinforcements in the horizontal plane resulting in a permanent deformation in their alignment shall not be allowed. Gradual bending in the vertical direction that does not result in permanent deformations is allowable.

Cutting of longitudinal or transverse reinforcement bars to avoid conflicts with utility obstructions or piles will not be allowed. A structural connection (yokes) from the wall panel to the reinforcement shall be used whenever it is necessary to avoid cutting or excessive skewing of reinforcement due to pile or utility conflicts.

Soil reinforcements shall be placed normal to the face of the wall, unless otherwise shown on the plans or directed by the Resident. If skewing of the soil reinforcements is required due to obstructions in the reinforced fill, rotatable bolted connections shall be used and the maximum skew angle shall not exceed 15º from the normal position except in the case of acute corner where redundant reinforcements are used. The tensile capacity of splayed reinforcement shall be reduced by the cosine of the splay angle.

<u>636.13 Method of Measurement</u> Mechanically Stabilized Earth Retaining Wall will be measured by the square foot of face area computed using the plan dimensions. No adjustment in the pay quantity will be made if the computed quantity, based on the working drawings, varies from the plan quantity.

Vertical dimension limits will be from the top of leveling pad to the top of the wall facing units, as shown on the plans. The horizontal dimension limits will be from the edges of the facing units at each end of a wall, as shown on the plans. No field measurements will be made unless the Resident specifies, in writing, a change to the limits indicated on the plans.

The wall surface area, as shown on the plans, includes the surface area of nominal panel joint openings and wall penetrations such as pipes and other utilities.

<u>636.14 Basis of Payment</u> The accepted quantity of Mechanically Stabilized Earth Retaining Wall will be paid for at the contract unit price per square foot. Payment shall be full compensation for design, fabrication and erection of MSE retaining walls, furnishing all labor, equipment and materials including concrete face panels, fasteners, reinforcing mesh, reinforcing strips, tie strips, hardware, joint fillers, coping, woven drainage geotextile, impervious membrane, select granular backfill and technical field representative. Cost of cast-in-place concrete for leveling pad will not be paid for separately but will be considered incidental to the Mechanically Stabilized Earth Retaining Wall.

Excavation, including extra excavation due to unsuitable foundation material, will be measured and paid for under Item 203.20 - Common Excavation. Foundation material and select backfill material in the reinforced zone will be considered incidental to the Mechanically Stabilized Earth Retaining Walls.

The unit price for Mechanically Stabilized Earth Wall shall include costs for:

1. All design work, preparation of written submittals and plans, revision of submittals, sample submittals and any other necessary preliminary work prior to and after acceptance of the retaining wall by the Resident.

2. All materials, including transportation, for the MSE walls, including facing panels, MSE reinforcing elements, attachment devices, fasteners, bearing blocks and shims, joint materials, copings, vertical corner elements, concrete masonry, reinforcing steel, crushed stone, select backfill and incidentals.

3. All labor and equipment required to excavate and prepare the wall foundation, form and cast the leveling pad, erect the MSE wall to the lines and grades shown on the plans, place and compact backfill, place and compact the drainage layer, and construct any other items necessary to complete the MSE wall.

4. All temporary sheeting, temporary excavation, and temporary dewatering necessary to perform the other work in this section.

There will be no allowance for excavating and backfilling for the Mechanically Stabilized Earth Retaining Wall beyond the limits shown on the approved submitted plans, except for excavation required to remove unsuitable subsoil in preparation for the foundation.

Payment will be made under:

Pay Item		Pay Unit
636.40	Mechanically Stabilized Earth Retaining Wall	Square foot