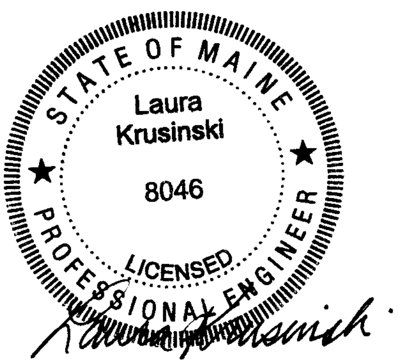
MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

BEARSLEY BROOK BRIDGE
OVER BEARSLEY BROOK
NEW SWEDEN, MAINE



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Aroostook County

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GEOTECHNICAL DESIGN SUMMARY

The purpose of this report is to make geotechnical recommendations for the replacement of Bearsley Brook Bridge over Bearsley Brook in New Sweden, Maine. Preliminary geotechnical recommendations were provided in an Interoffice Memorandum of January 24, 2008. The proposed replacement bridge will be a structural plate pipe arch. The replacement culvert will be built on the same alignment as the existing bridge. The following design recommendations for a plate pipe arch structure are discussed in detail in the attached report:

Limit State Design. Buried structures and their foundations shall be designed by the methods specified in the AASHTO LRFD Bridge Design Specifications, 4th Edition, 2007 with 2008 Interims (herein referred to as LRFD) Articles 12.6 and 12.7 so that they resist the factored loads given by the Service Load Combination I and Strength Load Combinations I and II.

Design Loads. Structural plate pipes shall be designed for forces resulting from horizontal and vertical earth pressure, pavement load, live load and vehicular dynamic load allowance. For vertical earth pressures, a load factor, γ_p, of 1.95 shall apply. External hydrostatic pressure, if present, can add significantly to the total thrust in a buried pipe arch and should be evaluated if site conditions warrant. Water buoyancy loads and resulting uplift shall be evaluated. Additional lateral earth pressure due to distribution of wheel loads due to construction load or vehicular live load through earth fill should be evaluated.

Backfill Envelope Soils. The Designer may assume Soil Type 4 (BDG Section 3.6.1) for backfill material soil properties. The backfill properties are as follows: $\phi = 32°$, $\gamma = 125$ pcf. Pipe envelope backfill soils 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.

Bearing Resistance and Stability. It is anticipated that the new structural plate pipe arch will be founded within the soft clay-silt soil deposit. The estimated factored bearing resistance of a strip footing on a saturated, silt clay soil is approximately 1.0 to 2.0 ksf. Partial removal of the soft foundation and replacement with a large, geotextile-wrapped granular mat foundation will increase the factored bearing resistance to approximately 3.0 ksf. Therefore, the bearing resistance for the plate invert on mat foundation as described below shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 3.0 ksf.

Foundation Support - Geotextile wrapped granular mat. A 2-foot thick granular mat will be required beneath the pipe arch in order to distribute the stresses evenly. Over excavation of the native soil and timber logs a minimum of 2 feet below the invert elevation will be necessary. After excavation, a reinforcement geotextile and a reinforcement geogrid shall be placed directly on the subgrade for the full length of the pipe arch and extend 3 feet beyond the span of the pipe arch.

A 2-foot thick layer of Granular Borrow, Material for Underwater for Underwater Backfill, MaineDOT 703.19 shall be placed on the geotextile and geogrid. The Granular Borrow shall be compacted in 12-inch lifts to 95% of the optimum density. The Granular Borrow mat shall be wrapped in the reinforcement geotextile. The total length and width of the reinforcement geotextile required must be sufficient to accommodate a wrap consisting of 2 feet of granular borrow thickness and a minimum overlap embedment of 3 feet below the pipe. The total area of the granular mat shall be sufficient to extend 3 feet beyond the footprint of the pipe arch to prevent downdrag loads on the pipe wall caused by a settling soil envelope and to provide support to the radius corners. Pre-shaping a shallow "V" into the upper surface of the granular mat prior to closing the wrap will to allow the pipe arch to maintain its shape.

Scour and Riprap. Stone riprap conforming to Item number 703.26 of the Standard Specification shall dress slopes to the top of the pipe arch. Properly riprapped slopes shall extend far enough from the structure to protect the structural portion of the soil envelope surrounding the pipe arch. Since the structure will be placed on erodible soils, a cut-off wall, scour curtain, or riprap-dressed invert aprons, extending below the maximum anticipated depth of scour, should be used.

Riprapped slopes shall be 3 feet thick and slope at a maximum 2H:1V. The toe of the riprap section shall be constructed 1 foot below the streambed and shall be underlain by a Class A Erosion Control Geotextile and a 1-foot thick layer of bedding material.

Settlement. To provide consist settlement between the pipe and the adjacent side fill, a uniform bedding consisting of a mat of compacted granular material should extend beyond the footprint of the pipe invert. The replacement buried structure will not be skewed to the roadway, therefore, unbalanced loading is not expected. Settlement is expected to be less than 1.0 inch.

Frost Protection. According to BDG Figure 5-1, Maine Design Freezing Index Map, the site has a design freezing index of approximately 2600 F-degree days. An assumed water content of 10% was used for moist, coarse grained soils above the water table. These components correlate to a frost depth of 9 feet. Therefore, any foundations placed on native soil should be founded a minimum of 9 feet below finished exterior grade for frost protection.

Seismic Design. In accordance with LRFD Article 3.10.1, seismic effects for box culverts and buried structures need not be considered, except where they cross active faults. There are no active faults mapped in the town on New Sweden, Maine.

Dewatering. The contractor should control groundwater and surface water to permit construction in the dry. The contractor may use temporary ditches, sumps, stone ditch protection, or riprap with geotextile underlayment to divert groundwater and surface water. Artesian groundwater was encountered within the lower glacial till unit and the bedrock formation at the site, and an upward seepage gradient may result in significant water in excavations; pumping from sumps will be needed to control water.

Construction Considerations. The 1936 bridge plans indicate the existing steel plate arch was constructed on a timber log mat consisting of 2 layers of 8-inch diameter logs. During drilling activities, the presence of a 6-inch timber log mat was noted. The wood and logs should be removed and replaced with crushed stone or compacted granular borrow for the replacement structure. In general, all wood, logs, soft clay, silt, peat or boulders encountered within a culvert bed excavation shall be removed to the full depth and replaced with compacted granular fill.

1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present geotechnical recommendations for replacement of Bearsley Brook Bridge, which carries State Route 161 over Bearsley Brook in New Sweden, Maine. This report presents the soils information obtained at the site during the subsurface investigation, foundation recommendations and geotechnical design recommendations for the bridge culvert replacement.

2.0 EXISTING AND PROPOSED CONDITIONS

Bearlsey Brook Bridge was built in the 1936 and is a 17-foot bolted steel structural plate arch constructed on timber grillage. The 1936 bridge plans indicate the timber grillage mat is 8-inches wider than the arch on each side and consists of 2 layers of 8-inch diameter logs. Each leg of the arch is bolted to the timber mat and butts against an 8-inch by 8-inch timber beam on top of a 8-inch by 12-inch timber beam, both which run the longitudinal length of the arch. The bridge structure pre-dating the 1936 plate arch consisted of a 15-foot timber stringer deck supported by stone-filled, log crib-type abutments.

Maine Department of Transportation (MaineDOT) Bridge Maintenance inspection reports indicate the arch is in poor condition with moderate distortion of the plates, heavy pitting and rusting, and scattered small holes below the flow line. One of the concrete headwalls has a full height crack and the other shows moderately severe cracking. 2006 MaineDOT Bridge Maintenance inspection reports indicate a Bridge Sufficiency Rating of 66.7. Considering the extensive deterioration of the pipe arch this will be a bridge replacement project, as it has been scoped.

The proposed replacement bridge will be a 17-foot 5-inch by 11-foot 6-inch structural plate pipe arch founded on a 2-foot thick geotextile-wrapped granular mat, to distribute the stresses evenly over the soft foundation soils. The bridge approach side slopes will be dressed with 2H:1V, 3-foot thick, plain riprap. The replacement bridge will be built on the same alignment as the existing bridge.

Preliminary foundation alternatives were provided by the Geotechnical Team Member in an internal Geotechnical Design Memorandum, dated January 24, 2008. Subsequent preliminary engineering assessments by the MaineDOT Bridge Program identified the most economical and practicable bridge replacement alternative for this site to be a structural plate pipe arch.

3.0 GEOLOGIC SETTING

Bearsley Brook Bridge on State Route 161 in New Sweden, Maine crosses Bearsley Brook as shown on Sheet 1 - Location Map, presented at the end of this report.

The Maine Geologic Survey "Surficial Geology of Caribou Quadrangle, Maine, Open-file No. 86-59" (1986) indicates that surficial soils in the vicinity of Bearsley Brook Bridge consist of swamp deposits. Swamp deposits are described as peat, silt, clay and sand, formed by the accumulation of sediments and organic material in depressions and other poorly drained areas. The dominant geologic material mapped in the area surrounding the bridge site is glacial till. Glacial till is a heterogeneous mixture of sand, silt, clay and stones. Till deposits typically conform to the bedrock surface, and were deposited directly by the glacial ice.

According to the Bedrock Geologic Map of Maine (1985), the bedrock at the Bearsley Brook Bridge site is the New Sweden Formation and consists of calcareous pelite, commonly known as mudstone.

4.0 SUBSURFACE INVESTIGATION

Subsurface conditions at the site were explored by drilling one test boring. The boring was terminated with a bedrock core. Test boring BB-NSBB-101 was drilled in the south approach of the existing plate arch. The boring location is shown on Sheet 2 - Boring Location Plan and Interpretive Subsurface Profile, found at the end of this report.

The boring was drilled on October 10 and 11, 2007 using the MaineDOT drill rig. Details and sampling methods used, field data obtained, and soil, bedrock and groundwater conditions encountered are presented in the boring log provided at the end of this memorandum.

The boring was drilled using solid stem auger and cased wash boring 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 standard penetration resistance, N-value, is the sum of the blows for the second and third intervals. The MaineDOT drill rig is newly equipped with a CME automatic hammer to drive the split spoon. The hammer was calibrated by MaineDOT in August of 2007 and was found to deliver approximately 30 percent more energy during driving than the standard rope and cathead system. All N-values discussed in this report are corrected values computed by applying an average energy transfer factor of 0.77 to the raw field N-values. This hammer efficiency factor (0.77) and both the raw field N-value and the corrected N-value are shown on the boring log. Attempts were made to conduct undrained vane shear tests.

Bedrock was cored in the boring using an NQ 2.0-inch I.D. core barrel and the Rock Quality Designation (RQD) of the core was calculated. The MaineDOT Geotechnical Team member selected the boring location and drilling methods, designated type and depth of sampling techniques, reviewed the field log for accuracy and identified field and laboratory testing requirements. A MaineDOT Certified Subsurface Investigator logged the subsurface conditions encountered. The boring was located in the field by use of a tape after completion of the drilling program.

Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring log provided in Appendix A – Boring Log and on Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile, found at the end of this report

5.0 LABORATORY TESTING

Laboratory testing for samples obtained in the boring consisted of one (1) standard grain size analysis and one (1) grain size analysis with hydrometer, with natural water contents. The results of soil laboratory tests are included as Appendix B - Laboratory Data, at the end of this report. Laboratory test information is also shown on the boring log provided in Appendix A – Boring Log and on Sheet 3 – Boring Log.

6.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered at test boring BB-NSBB-101 generally consisted of fill overlying a timber log mat, silt and clay silt, and glacial till, all underlain by metamorphosed siltstone. An interpretive subsurface profile depicting the detailed soil stratigraphy across the site is show on Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile found at the end of this report. A brief summary description of the strata encountered is as follows:

6.1 Fill and Timber Mat

A layer of fill was encountered in the boring. The encountered fill layer is approximately 7.5 feet thick and was underlain by approximately 6 inches of wood. The fill deposit generally consisted of grey-brown and brown, SAND and silty SAND, little to some angular to rounded fine gravel, little silt.

SPT N-values in fill layer ranged from 17 to 51 blows per foot (bpf) indicating that the fill unit is medium dense to very dense in consistency. One SPT N-value in the fill unit was greater than 50 bpf and was likely influenced by coarse aggregate.

One grain size analysis was conducted on a sample from the fill unit. The grain size analysis resulted in the soil being classified as A-1-b under the AASHTO Soil Classification System and SM under the Unified Soil Classification System. The natural water content of the tested sample was approximately 7 percent.

6.2 Silt and Clay Silt

An approximately 11-foot thick deposit of silt and clay silt was encountered below the granular fill and wood in boring BB-NSBB-101. The deposit encountered consisted of brown to brown-grey, SILT to CLAY SILT, little sand, trace of organics, wood, and angular coarse gravel. SPT N-values in the silt and clay silt layer ranged from 4 to 5 bpf indicating that the deposit is soft to medium stiff in consistency.

Both split-spoon samples attempted in this layer required two (2) attempts to achieve any recovery. The 10 to 12-foot sample did not result in recovery for either attempt. Two (2) attempts to conduct undrained vane shear tests in the clay silt deposit resulted in not being able to push the vanes into the soil unit, probably because of sand lenses.

6.3 Glacial Till

A layer of glacial till was encountered in the boring below the silt and clay silt deposit. The encountered glacial till deposit is approximately 15.2 feet thick. The till deposit generally consisted of grey, wet, SAND to sandy SILT, some silt, little clay, little to some fine to coarse gravel, and grey, angular to subrounded coarse to fine GRAVEL, some silt, little sand, strongly cemented, with boulders.

SPT N-values in glacial till ranged from 6 to 12 bpf in the upper till, indicating that the till unit is medium stiff to medium dense in consistency. The lower portion of the glacial till was strongly cemented and included a 1.1-foot diameter boulder. A soil sample in the lower glacial till deposit was cored with a core barrel.

Grain size analyses were conducted on one sample from the glacial till deposit. Grain size analyses resulted in the soil being classified as A-4 under the AASHTO Soil Classification System and SC-SM under the Unified Soil Classification System. An Atterberg Limits test on the sample determined the sample to be non-plastic. The natural water content of the tested sample was approximately 11 percent.

6.4 Groundwater

Artesian groundwater conditions were encountered when coring the cemented glacial till and bedrock. The artesian pressure was estimated to be approximately 4 feet of pressure head. Artesian conditions continued after demobilizing from the site, resulting in a continuously wet to icy pavement surface. On October 23, 2007, the MaineDOT Drill Crew returned and sealed the borehole with a 12-foot bentonite seal topped with 2 feet of Portland cement.

6.5 Bedrock

Bedrock at the site was encountered and cored at a depth of 34.2 feet below ground surface (bgs) and Elevation 563.90 feet in boring BB-NSBB-101.

The bedrock at the site is identified as dark grey, aphanitic, metamorphosed SILTSTONE, moderately hard, moderately weathered, very highly fractured, joint set along bedding, dipping at steep angles, closely spaced, tight to open, some with silt. The rock quality designation (RQD) of the bedrock was determined to range from 0 to 19 percent, correlating to a rock quality of very poor.

Refer to the boring log in Appendix A for more detailed documentation of the conditions encountered in the exploration.

7.0 FOUNDATION ALTERNATIVES

For development of the Preliminary Design Report for Bearsley Brook Bridge, foundation alternatives were provided in an internal Geotechnical Design Memorandum, dated January 24, 2008. The following foundation systems were considered for the replacement bridge and evaluated for practicality in the January 24, 2008 memorandum:

- Structural Plate Pipe, Pipe Arch or Concrete Box
- Cast-in-place (CIP) or precast concrete integral abutments supported on short, driven steel H-piles
- Cast-in-place (CIP) abutments founded on spread footings bearing on glacial till

All of these foundation types are viable, to varying degrees, as foundation alternatives for this site, however, a structural plate pipe arch was selected by the Designer as the most economical and practicable foundation type. This report addresses only that foundation type.

8.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

The following subsections will discuss geotechnical design recommendations for design and construction of a buried, structural plate pipe arch.

8.1 Limit State Design and Resistance Factors

Buried structures and their foundations shall be designed by the methods specified in LRFD Articles 12.6 and 12.7 so that they resist the factored loads given by the load combinations specified below:

- Service Load Combination I as specified in LRFD Table 3.4.1-1 – investigate the buried structure for deflection.

- Strength Load Combinations I and II, as specified in LRFD Table 3.4.1-1, and construction loads - investigate:

    - Wall area
    - Buckling
    - Seam failure
    - Flexibility limit for construction
    - Flexure

Due to the buried nature of the structure, the Extreme Event State does not control.

Resistance factors for the design of the pipe arch and the geotechnical evaluation of the foundation be taken as specified in the Table below:

Condition	Resistance Factor
Minimum wall area and buckling	1.00
Minimum longitudinal seam strength	0.67
Bearing Resistance to pipe arch foundation	0.45

Table. Resistance Factors for Structural Plate Pipes

8.2 Design Loads

Structural plate pipes shall be designed for forces resulting from horizontal and vertical earth pressure, pavement load, live load and vehicular dynamic load allowance. For vertical earth pressures, a load factor, γ_p, of 1.95 shall apply, per LRFD Table 3.4.1-2.

External hydrostatic pressure, if present, can add significantly to the total thrust in a buried pipe arch and should be evaluated if site conditions warrant.

Water buoyancy loads and resulting uplift shall be evaluated for buried structures with inverts below the water table to control flotation, as indicated in LRFD Article 3.7.2. The dead load on the crown of the structure should exceed the buoyancy of the pipe arch, using appropriate factored loads.

Additional lateral earth pressure due to distribution of wheel loads due to construction load or vehicular live load through earth fill should be determined in accordance with LRFD Article 3.6. Where the depth of fill is less than 2.0 feet, live loads shall be distributed to the upper portion of the pipe arch as specified in LFRD Article 4.6.2.10. Otherwise, live load/wheel load distributions through the earth fill can be determined as described in LRFD Article 3.6.1.2.6. For single span culverts, the effects of live load may be neglected where the depth of fill is more than 8 feet and exceeds the span length.

8.3 Backfill Envelope Soils

The Designer may assume Soil Type 4 (BDG Section 3.6.1) for backfill material soil properties. The backfill properties are as follows: $\phi = 32°$, $\gamma = 125$ pcf.

Pipe envelope backfill soils 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 around the structure.

Backfill material shall be placed in layers not exceeding 8-inch loose lifts compacted to a minimum of 90 percent of T99. Compaction equipment used to compact backfill within 3 feet from sides of the pipe shall be approved by the Resident prior to use. Backfill shall be placed, compacted and raised evenly on both sides of the pipe by working backfill operations from side to side. At least one foot of backfill shall be placed on top of the pipe prior to placing fill material specified to support pavement. The trench shall be kept to a minimum width required for placing the bedding, pipe and backfill. Ponding or jetting of backfill will not be permitted.

8.4 Bearing Resistance

Pipe structures and their foundations shall be designed to provide stability against bearing capacity failure.

It is anticipated that the new structural plate pipe arch will be founded within the soft clay-silt soil deposit. The estimated factored bearing resistance of conventional footing on a saturated, silt clay soil deposit is approximately 1.0 to 2.0 ksf. Partial removal of the soft soils and replacement with a large, geotextile-wrapped granular mat foundation will increase the factored bearing resistance to approximately 3.0 ksf. Therefore, the bearing resistance for the plate invert shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 3.0 ksf. This assumes a bearing resistance factor, $\varphi_{b,}$ for mat foundations on native soils to be 0.45. To control settlements, a limiting factored bearing resistance of 2.0 ksf is recommended at the service limit state. Supporting calculations are provided in Appendix C - Calculations.

The radial pressures at the corners and side of the pipe arch may be 2 to 5 times the loading on top of the pipe, depending on its shape. For this reason the geotextile-wrapped granular mat should extend a minimum of 3 feet beyond the radius corners. The Designer may consider less support directly under the invert to allow the pipe arch to maintain its shape, or to place the pipe on a shaped bed. Pre-shaping may consist of a shallow “V” graded into the upper surface of the granular mat prior to wrapping with geotextile.

The pipe structure and the foundation material of the pipe should be checked for bearing soil failure resulting from erosion by hydraulic gradients. The hydraulic design considerations in LRFD Article 2.6 and FHWA’s “Hydraulic Design of Highway Culverts” (1985) apply.

8.5 Foundation Support - Geotextile wrapped granular mat

A 2-foot thick granular mat will be required beneath the pipe arch in order to distribute the stresses evenly. Over excavation of the native soil and timber logs a minimum of 2 feet below the invert elevation will be necessary. After excavation, a reinforcement geotextile and a reinforcement geogrid shall be placed directly on the subgrade for the full length of the pipe arch. The geotextile will aid in preventing migration and mixing of the granular mat with the underlying poor soils. If the existing subgrade is inconsistent and difficult to work with, a working mat consisting of crushed stone may be installed.

Once the geotextile is placed, the reinforcement geogrid should be installed directly on top of the geotextile. The reinforcement geogrid will improve the bearing capacity of the soft foundation soils. The geogrid and the geotextile should extend a minimum of 3 feet beyond the limit of the structure in the direction parallel to the construction centerline. A 2 foot thick layer of Granular Borrow, Material for Underwater for Underwater Backfill, MaineDOT 703.19 shall be placed on the geotextile and geogrid. The Granular Borrow shall be placed in 1-foot lifts and compacted to 95% of the optimum density. The Granular Borrow mat shall be wrapped in the reinforcement geotextile. The total length and width of the reinforcement geotextile required must be sufficient to accommodate a wrap consisting of 2 feet of granular borrow thickness and a minimum overlap embedment of 3 feet below the pipe. The total area of the granular mat shall be sufficient to extend 3 feet beyond the footprint of the pipe arch to prevent downdrag loads on the box wall caused by a settling soil envelope and to provide support to the radius corners. Pre-shaping a shallow “V” into the upper surface of the granular mat prior to closing the wrap will to allow the pipe arch to maintain its shape.

8.6 Scour and Riprap

Buried structures shall be designed so that no movement of any part of the structure will occur as a result of scour. Wingwalls, headwalls or properly riprapped slopes shall extend far enough from the structure to protect the structural portion of the soil envelope surrounding the pipe arch. Since the structure will be placed on erodible soils, a cut-off wall, scour curtain, or riprap-dressed invert aprons, extending below the maximum anticipated depth of scour, should be used.

Stone riprap conforming to Item number 703.26 of the Standard Specification shall be placed at the toes of any wall footings or to dress slopes to the top of the pipe arch. Riprap shall be 3 feet thick. The riprap shall extend horizontally in front of any wall faces before sloping at maximum 2H:1V slope to the existing ground surface. The toe of the riprap section shall be constructed 1 foot below the streambed elevation. The riprap section shall be underlain by a Class A Erosion Control Geotextile and a 1-foot thick layer of bedding material conforming to Item number 703.19 of the Standard Specification.

8.7 Settlement

Consideration was given to potential settlements resulting from longitudinal differential settlement along the length of the pipe, differential settlement between the pipe and the backfill and settlement of the foundation due to unbalanced loading. Differential settlements are not expected, although uniform settlements up to 1.0 inch can be expected.

In our opinion, to provide consist settlement between the pipe invert foundation settlement and the adjacent side fill, a uniform bedding consisting of a mat of compacted granular material should extend beyond the footprint of the pipe invert. The granular mat should extend beyond both dimensions of the pipe invert footprint by at least 3 feet to provide uniform longitudinal and transverse settlement. The replacement buried structure will not be skewed to the roadway, therefore, unbalanced loading is not expected.

8.8 Frost Protection

In the case that project final engineering introduces foundations placed on native soils, the foundations should be designed with an appropriate embedment for frost protection. According to BDG Figure 5-1, Maine Design Freezing Index Map, the site has a design freezing index of approximately 2600 F-degree days. An assumed water content of 10% was used for moist, coarse grained soils above the water table. These components correlate to a frost depth of 9 feet. Therefore, any foundations placed on native soil should be founded a minimum of 9 feet below finished exterior grade for frost protection.

8.9 Seismic Design Considerations

In accordance with LRFD Article 3.10.1, seismic effects for box culverts and buried structures need not be considered, except where they cross active faults. According to the Maine Geological Survey Bedrock Geologic Map of Maine (1985) there are no active faults mapped in the town on New Sweden, Maine.

8.10 Dewatering

The native clay-silt soils within the project area are poorly draining. These soil units encountered above the watertable may be saturated and there may be localized sloughing and instability of excavation slopes. The contractor should control groundwater and surface water to permit construction in the dry. The contractor may use temporary ditches, sumps, stone ditch protection, or riprap with geotextile underlayment to divert groundwater and surface water. Artesian groundwater was encountered within the lower glacial till and bedrock formation at the site, and an upward seepage gradient may result in significant water in any excavation; we anticipate that pumping from sumps will be needed to control water. If the contractor excavates in the location of boring BB-NSBB-101 and disturbs the bentonite and Portland cement seal, artesian flow should be expected.

8.11 Construction Considerations

The existing bridge culvert plans indicate the steel plate arch was constructed on a timber log mat which is 8-inches wider than the arch on each side. The 1936 plans show the log mat consists of 2 layers of 8-inch diameter logs. During drilling activities, the presence of a 6-inch timber log mat was noted at the approximate elevation of the proposed pile invert. The wood and logs should be removed and replaced with crushed stone or compacted granular borrow for the replacement structure.

In general, all wood, logs, soft clay, silt, peat, boulders or other unsuitable material encountered within the pipe arch bed excavation shall be removed to the full depth and replaced with compacted granular fill or crushed stone.

Standard Specification Section 509 – Structural Plate Pipes, Pipe Arches, Arches and Metal Box Culverts does not currently include construction requirements controlling compaction of the soil envelope and bedding material. Therefore, compaction requirements must be specified by the Special Provision or by notes on the plans.

Artesian groundwater was encountered at the site. Artesian pressures may be released if the contractor excavates in the location of boring BB-NSBB-101 and disturbs the bentonite and Portland cement seal.

9.0 CLOSURE

This report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed replacement of Bearsley Brook Bridge in New Sweden, Maine in accordance with generally accepted geotechnical and foundation engineering practices. No other intended use 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.

<u>Appendix A</u>

Boring Log

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Bearsley Brook Bridge over Bearsley Brook, Route 161
Location: New Sweden, Maine

Boring No.: BB-NSBB-101
PIN: 15645.00

Driller:	MaineDOT	**Elevation (ft.)**	598.1	**Auger ID/OD:**	5" Solid Stem
Operator:	E. Giguere	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	G. Lidstone	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/10/07-10/11/07	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-1.88"
Boring Location:	1+82.4, 10.8 Rt.	**Casing ID/OD:**	NW	**Water Level*:**	Artesian - See Remarks
Hammer Efficiency Factor:	0.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	597.40		PAVEMENT. —0.70	
	1D	24/14	1.00 - 3.00	12/22/18/19	40	51				Grey-brown, dry to damp, very dense, fine to coarse SAND, some angular to rounded fine gravel, little silt, (Fill).	G#209968 A-1-b, SM WC=6.9%
								594.10		—4.00 Brown, damp, medium dense, silty fine to coarse SAND, little angular to rounded fine gravel, (Fill). (2D/A) 5.0-6.5' bgs.	
5	2D/AB	24/9	5.00 - 7.00	5/8/5/3	13	17	23				
							14	591.60		—6.50 (2D/B) 6.5-7.0' bgs. Brown, weathered, moist, medium dense, silty fine to coarse SAND, little fine gravel, (Fill).	
							256	590.60		—7.50 Wood from 7.5-8.0' bgs.	
							9	590.10		—8.00 Brown, medium stiff, SILT, little organics, fine sand seen in wash water.	
							12			2 sampling attempts for MD.	
10	MD	24/0	10.00 - 12.00	3/2/2/2	4	5	18				
							15				
							14	586.10		—12.00 Brown-grey, wet, soft, CLAY SILT, trace of organics, wood, trace fine sand, angular coarse gravel.	
							14				
							12				
15	3D/MV	24/6	15.00 - 17.00	6/1/2/2	3	4	17			2 sampling attempts for 3D. Failed 55x110 mm vane attempt, could not push. Roller Coned ahead from 15.0-18.0' bgs. Sand in wash water.	
							14				
							15				
	MV			Could not push			20			Failed 55x110 mm vane attempt.	
							32	579.10		—19.00 Grey, wet, medium dense, fine to medium SAND some silt, little clay, little fine gravel, trace coarse gravel.	
20	4D	24/13	20.00 - 22.00	6/6/3/4	9	12	38				G#209969 A-4, SC-SM Non-Plastic WC=11.2%
							43				
							47				
							54				
25							57				

Remarks:

Artesian flow noted after R1, R2 and R3 when extracting casing. Flow was neutralized with casing sticking up 4' above ground surface.
10/10/07; 13:00-17:00, 10/11/07; 08:00-12:30;
Artesian flow continued after demobilizing. Crew returned on 10/23/07 and augered to 16 ft with 5" SSA, put in a 12-ft of Bentonite seal, 2 ft of auger cuttings and 2 ft of Portland Type 2 Cement ontop. Used 1 50 lb bag of Holeplug 3/8-in coarse grade bentonitie chips.

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: Bearsley Brook Bridge over Bearsley Brook, Route 161
Location: New Sweden, Maine

Boring No.: BB-NSBB-101
PIN: 15645.00

Driller:	MaineDOT	**Elevation (ft.)**	598.1	**Auger ID/OD:**	5" Solid Stem
Operator:	E. Giguere	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	G. Lidstone	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/10/07-10/11/07	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-1.88"
Boring Location:	1+82.4, 10.8 Rt.	**Casing ID/OD:**	NW	**Water Level*:**	Artesian - See Remarks
Hammer Efficiency Factor:	0.77	**Hammer Type:**	Automatic ☒	Hydraulic ☐	Rope & Cathead ☐

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)
25	5D	24/9	25.00 - 27.00	3/2/3/6	5	6	52	573.10
							65	
							81	
							132	569.90
	R1	60/27	29.00 - 34.00				242	569.10
30							NQ	568.00
							93	
							90	
							75	
							92	
	R2	48/44	34.70 - 38.70	RQD = 0%			a55	563.90
35							NQ	563.40
	R3	51.6/47	38.70 - 43.00	RQD = 19%				559.40
40								
								555.10
45								
50								

Visual Description and Remarks

25.00
Similar to above. Grey, wet, medium stiff, fine sandy SILT, little clay, some angular coarse gravel, trace medium to coarse sand.

28.20
TILL, probably similar to till seen in R1.

29.00
R1: BOULDER from 29.0-30.1' bgs.
R1: 1.1-ft metasiltstone boulder.
Core Times (min:sec)
29.0-30.0' (3:10)
30.0-31.0' (3:45)
31.0-32.0' (3:46)
32.0-33.0' (5:12)
33.0-34.0' (3:05) 45% Recovery

30.10
Grey, angular to subrounded coarse to fine GRAVEL, some silt, little fine to medium sand, strongly cemented. (Glacial Till).
a55 blows for 0.2'.

34.20
Top of Bedrock at Elev. 563.9'.
Roller Coned ahead from 34.2-34.7' bgs.

34.70
R2:Bedrock: Dark grey, aphanitic, metamorphosed SILTSTONE, moderately weathered, moderately hard, very highly fractured, jointing along foliation at steep angles, closely spaced, tight to open, some with silt on surfaces.
R2:Core Times (min:sec)
34.7-35.7' (8:12)
35.7-36.7' (4:33)
36.7-37.7' (4:40)
37.7-38.7' (6:18) 98% Recovery

38.70
R3:Bedrock: Same as R2, and very highly fractured.
R3:Core Times (min:sec)
38.7-39.7' (12:15)
39.7-40.7' (7:26)
40.7-41.7' (7:53)
41.7-42.7' (8:40)
42.7-43.0' (6:07) 90% Recovery

43.00
Bottom of Exploration at 43.00 feet below ground surface.

Laboratory Testing Results/ AASHTO and Unified Class.

Remarks:

Artesian flow noted after R1, R2 and R3 when extracting casing. Flow was neutralized with casing sticking up 4' above ground surface.
10/10/07; 13:00-17:00, 10/11/07; 08:00-12:30;
Artesian flow continued after demobilizing. Crew returned on 10/23/07 and augered to 16 ft with 5" SSA, put in a 12-ft of Bentonite seal, 2 ft of auger cuttings and 2 ft of Portland Type 2 Cement ontop. Used 1 50 lb bag of Holeplug 3/8-in coarse grade bentonite chips.

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

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

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):

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

*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

Sample Container Labeling Requirements:

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

Appendix B

Laboratory Data

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): New Sweden Project Number: 15645.00

Boring & Sample Identification Number	Station (Feet)	Offset (Feet)	Depth (Feet)	Reference Number	G.S.D.C. Sheet	W.C.	L.L.	P.I.	Classification		
									Unified	AASHTO	Frost
BB-NSBB-101, 1D	1+82.4	10.8 Rt.	1.0-3.0	209968	1	6.9			SM	A-1-b	II
BB-NSBB-101, 4D	1+82.4	10.8 Rt.	20.0-22.0	209969	1	11.2	-N	P-	SC-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 MDOT and Corps of Engineers Classification Systems.

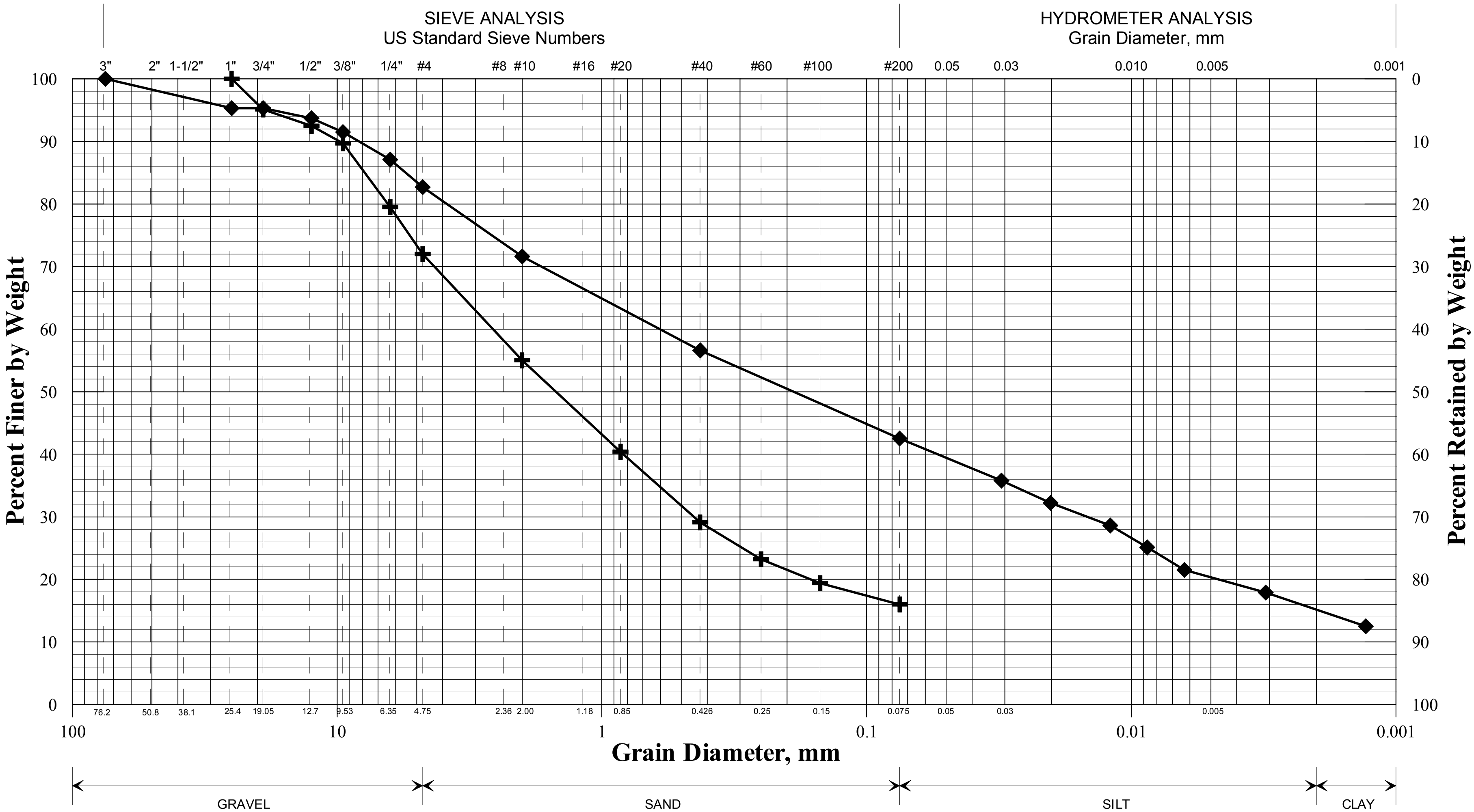
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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

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

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

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-NSBB-101/1D	1+82.4	10.8 RT	1.0-3.0	SAND, some gravel, little silt.	6.9			
◆	HB-NSBB-101/4D	1+82.4	10.8 RT	20.0-22.1	SAND, some silt, little gravel, little clay.	11.2	13	13	NP
■									
●									
▲									
✖									

PIN
015645.00
Town
New Sweden
Reported by/Date
WHITE, TERRY A 5/5/2008

SHEET 1

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
209968	BB-NSBB-101/1D	GEOTECHNICAL (DISTURBED)	10/10/2007	10/18/2007

Sample Type: **GEOTECHNICAL** Location: **OTHER** Station: **1+82.4** Offset, ft: **10.8 RT** Dbfg, ft: **1.0-3.0**

PIN: **015645.00** Town: **New Sweden** Sampler: **GIGUERE, ERVIN M**

TEST RESULTS

Sieve Analysis (T 27, T 11)

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	100.0
¾ in. [19.0 mm]	95.1
½ in. [12.5 mm]	92.5
⅜ in. [9.5 mm]	89.7
¼ in. [6.3 mm]	79.5
No. 4 [4.75 mm]	72.0
No. 10 [2.00 mm]	55.0
No. 20 [0.850 mm]	40.4
No. 40 [0.425 mm]	29.1
No. 60 [0.250 mm]	23.2
No. 100 [0.150 mm]	19.4
No. 200 [0.075 mm]	16.0

Wash Method
Procedure A

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

Liquid Limit @ 25 blows (T 89), %

Plastic Limit (T 90), %

Plasticity Index (T 90), %

Specific Gravity, Corrected to 20°C (T 100)

Loss on Ignition (T 267)

Loss, % H2O, %

Water Content (T 265), %

6.9

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: **11/6/2007**

Paper Copy: Lab File; Project File; Geotech File

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
209969	HB-NSBB-101/4D	GEOTECHNICAL (DISTURBED)	10/10/2007	10/12/2007

Sample Type: **GEOTECHNICAL** Location: **OTHER** Station: **1+82.4** Offset, ft: **10.8 RT** Dbfg, ft: **20.0-22.1**

PIN: **015645.00** Town: **New Sweden** Sampler: **GIGUERE, ERVIN M**

TEST RESULTS

Sieve Analysis (T-88)

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	100.0
1 in. [25.0 mm]	95.3
¾ in. [19.0 mm]	95.3
½ in. [12.5 mm]	93.7
⅜ in. [9.5 mm]	91.5
¼ in. [6.3 mm]	87.1
No. 4 [4.75 mm]	82.7
No. 10 [2.00 mm]	71.6
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	56.6
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	42.5
[0.0309 mm]	35.8
[0.0202 mm]	32.2
[0.0120 mm]	28.6
[0.0087 mm]	25.1
[0.0063 mm]	21.5
[0.0031 mm]	17.9
[0.0013 mm]	12.5

Wash Method

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

Liquid Limit @ 25 blows (T 89)
13

Plastic Limit (T 90)
13

Plasticity Index (T 90)
NP

Specific Gravity, Corrected to 20°C (T 100)
2.66

Loss on Ignition (T 267)
Loss, % H2O, %

Water Content (T 265), %
11.2

Vane Shear Test on Shelby Tubes (Maine DOT)

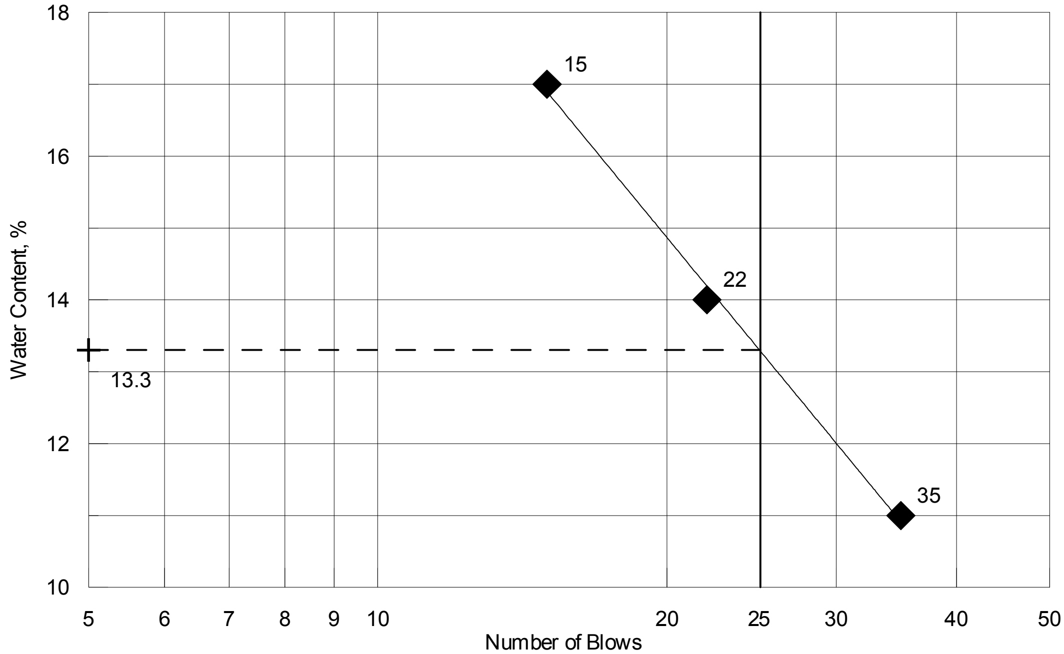
Depth taken in tube, ft	3 In. U. Shear tons/ft²	3 In. Remold tons/ft²	6 In. U. Shear tons/ft²	6 In. Remold tons/ft²	Water Content, %	Description of Material Sampled at the Various Tube Depths

Comments:

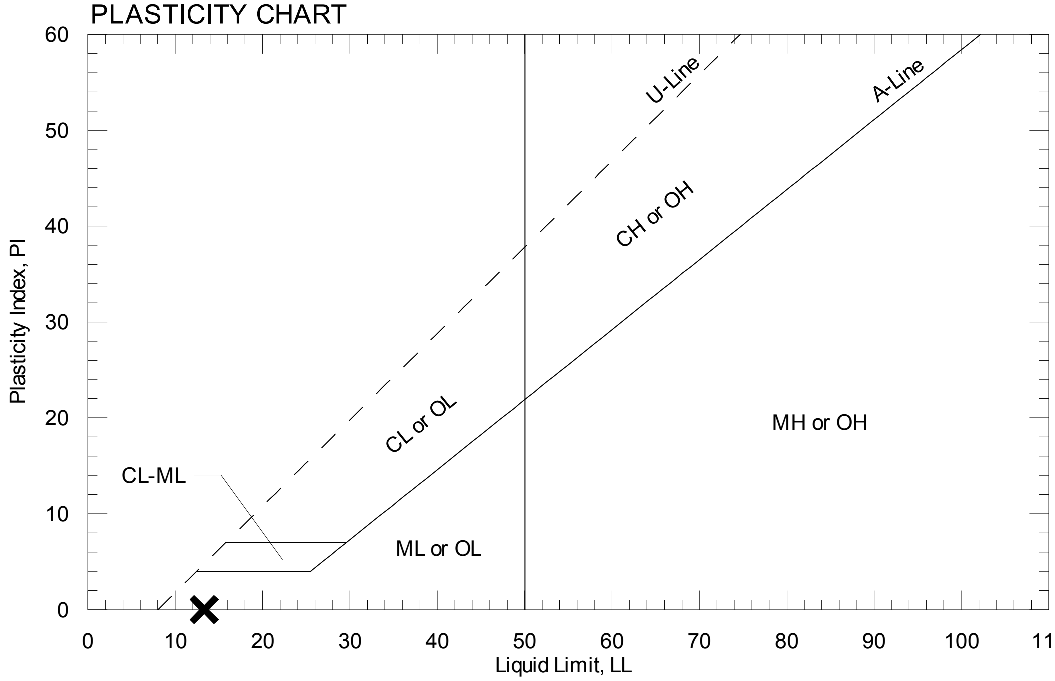
Maine DOT, Materials Testing & Exploration, 219 Hogan Road, Bangor, Maine 04401

TOWN	New Sweden	Reference No.	209969
PIN	015645.00	Water Content, %	11.2
Sampled	10/10/2007	Plastic Limit	13
Boring No./Sample No.	HB-NSBB-101/4D	Liquid Limit	13
Station	1+82.4	Plasticity Index	NP
Depth	20.0-22.1	Tested By	BBURR

FLOW CURVE



PLASTICITY CHART



AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **11/8/2007**

Paper Copy: Lab File; Project File; Geotech File

Appendix C

Calculations

Analysis: Nominal and factored bearing resistance
Structure: Strip footing directly founded on native soils

$psf := \frac{lbf}{ft^2}$ $pcf := \frac{lbf}{ft^3}$ $Mg := 1000 \cdot kg$ $kN := 1000 \cdot newton$ $kPa := \frac{kN}{m^2}$ $tonf := g \cdot ton$ $kip := 1000 \cdot lbf$

$ksf := \frac{kip}{ft^2}$ $ton := 2000 \cdot lbf$ $tsf := \frac{tonf}{ft^2}$ $psi := \frac{lbf}{in^2}$ $ksi := \frac{kip}{in^2}$

Assumptions:

1. Traditional strip foundation for precast or steel arch wall founded with 2 feet embedment below stream invert; water 2 feet above steambed.

2. Assumed pamaters for medium stiff silt, little organics to soft clay silt (N-values range from 5 to 4 bpf
saturated unit weight = 115 pcf
dry unit weight = 110 pcf
internal friction angle of 20 degree
undrained shear strength (c) 500 psf

3. Method used: Terzaghi, assume a strip foundation directly on soft soils.

4. Examine conditions: footing on ϕ–c soil (ref: Bowles Ex. 4-1 pg 231), effective stress analysis.

Foundation soil values

Available References:

ϕ : Lambe & Whitman Table 11.3 based on Hough, Basic Soils Engr, 1967
ϕ, SPT correlation, Lambe & Whitman, Fig 11.14, (from Peck, Hanson, Thornburn).
ϕ and γ correlations to soil description and N values, Bowles 5th Ed., Tables 3-4 and 2-6
γ sat : Holtz, Kovacs, Table 2-1 1981

Footing Width and Depth

$B := \begin{pmatrix} 2 \\ 5 \\ 10 \end{pmatrix} \cdot ft$ $D_f := 2.0 \cdot ft$ $D_w := -2 \cdot ft$ $\gamma_w := 62.4 \cdot pcf$

Foundation Soil (Undisturbed soft to medium stiff clay silt)

$\gamma 1_{sat} := 115 \cdot pcf$ $\gamma 1_d := 110 \cdot pcf$ $\gamma 1_t := \gamma 1_{sat}$ $\phi := 20deg$ $c_1 := 500 \cdot psf$

Nominal Bearing Resistance - based on Presumptive Bearing Capacity

For Service Limit States ONLY

Based on NavFac DM 7.2 pg 142-143 Table 1 - "Presumptive Values of Allowable Bearing Capacity Pressures for Spread Foundations".

Bearing Material:	Consistency in Place:	Allowable Bearing Pressure (tons per sq. foot):	Recommended Value:
Inorganic silt or clayey silt	Very stiff to hard	2 to 4	3 tsf
	Medium to stiff	1 to 3	1.5 tsf
	Soft	0.5 to 1	0.5 tsf

Recommend 1 tsf / 2 ksf

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

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

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

Terzaghi Bearing Capacity Factors - (Ref: Bowles Table 4-2, 5th Ed. pg 222) for ϕ=20

$N_c := 17.7$ $\qquad$ $N_q := 7.4$ $\qquad$ $N_\gamma := 5.0$

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

$N_{cm} := 14.83$ $\qquad$ $N_{qm} := 6.4$ $\qquad$ $N_{\gamma m} := 2.9$

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

Groundwater is 2 feet above the invert, therefore 4 ft above the bottom of the footing founded 2 ft down

$$q := \left(2\cdot ft\cdot\gamma_w + 2\cdot ft\cdot\gamma 1_{sat}\right) - 4\cdot ft\cdot\gamma_w \qquad q = 0.053\,tsf$$

Using Terzaghi Factors

$$q_n := c_1\cdot N_c\cdot s_c + q\cdot N_q + 0.5\cdot\left(\gamma 1_{sat} - \gamma_w\right)\cdot B\cdot N_\gamma\cdot s_\gamma \qquad q_n = \begin{pmatrix} 4.9 \\ 5.1 \\ 5.5 \end{pmatrix} tsf$$

Using Meyerhof Factors

$$q_{n\_m} := c_1 \cdot N_{cm} \cdot s_c + q \cdot N_{qm} + 0.5 \cdot (\gamma 1_{sat} - \gamma_w) \cdot B \cdot N_{\gamma m} \cdot s_\gamma \qquad q_{n\_m} = \begin{pmatrix} 4.1 \\ 4.2 \\ 4.4 \end{pmatrix} tsf$$

Factored Bearing Resistance for strength limit states

Terzaghi $\quad q_r := q_n \cdot 0.45 \quad \boxed{q_r = \begin{pmatrix} 2.2 \\ 2.3 \\ 2.5 \end{pmatrix} tsf}$

Strength limit state

Meyerhof $\quad q_{r\_m} := q_{n\_m} \cdot 0.45 \quad \boxed{q_{r\_m} = \begin{pmatrix} 1.9 \\ 1.9 \\ 2 \end{pmatrix} tsf}$

Recommend a limiting factored bearing resistance of **2 tsf** or 4 ksf, for strip footings 10 ft or smaller on soft silt clay unit for the strength limit state.

Factored Bearing Resistance for service limit states - approximate by allowable bearing pressures computed using ASD approach and a FS of 3 to control settlements

Terzaghi $\quad q_r := \frac{q_n}{3} \quad \boxed{q_r = \begin{pmatrix} 1.6 \\ 1.7 \\ 1.8 \end{pmatrix} tsf}$

Service limit state

Meyerhof $\quad q_{r\_m} := \frac{q_{n\_m}}{3} \quad \boxed{q_{r\_m} = \begin{pmatrix} 1.4 \\ 1.4 \\ 1.5 \end{pmatrix} tsf}$

Recommend a limiting factored bearing resistance of **1.5 tsf** or 3 ksf, for strip foundations founded on the clay silt soils, to contol settlements - i.e the Service Limit State.

Nominal Bearing Resistance for Strength Limit States: Terzaghi Method - $\phi = 0$ analysis

Terzaghi Bearing Capacity Factors - (Ref: Bowles Table 4-2, 5th Ed. pg 222) ϕ=0

$N_c := 5.7$ $\qquad N_q := 1.0$ $\qquad N_\gamma := 0$

Meyerhof Bearing Capacity Factors - (Ref: Bowles Table 4-4, 5th Ed. pg 222) ϕ=0

$N_{c\_m} := 5.14$ $\qquad N_{q\_m} := 1.0$ $\qquad N_{\gamma\_m} := 0$

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

Groundwater is 2 feet above the invert, therefore 4 ft above the bottom of the footing founded 2 ft down

$$q := \left(2\cdot ft\cdot\gamma_w + 2\cdot ft\cdot\gamma 1_{sat}\right) - 4\cdot ft\cdot\gamma_w \qquad q = 0.053\ tsf$$

Terzaghi

$$q_n := c_1\cdot N_c\cdot s_c + q\cdot N_q + 0.5\cdot\left(\gamma 1_{sat} - \gamma_w\right)\cdot B\cdot N_\gamma\cdot s_\gamma \qquad q_n = \begin{pmatrix} 1.5 \\ 1.5 \\ 1.5 \end{pmatrix} tsf$$

Meyerhof

$$q_{n\_m} := c_1\cdot N_{c\_m}\cdot s_c + q\cdot N_{q\_m} + 0.5\cdot\left(\gamma 1_{sat} - \gamma_w\right)\cdot B\cdot N_{\gamma\_m}\cdot s_\gamma \qquad q_{n\_m} = \begin{pmatrix} 1.3 \\ 1.3 \\ 1.3 \end{pmatrix} tsf$$

Factored Bearing Resistance for strength limit states

Terzaghi $\qquad q_r := q_n\cdot 0.45 \qquad \boxed{q_r = \begin{pmatrix} 0.7 \\ 0.7 \\ 0.7 \end{pmatrix} tsf}$ $\qquad$ Strength Limit State

Meyerhof $\qquad q_{r\_m} := q_{n\_m}\cdot 0.45 \qquad \boxed{q_{r\_m} = \begin{pmatrix} 0.6 \\ 0.6 \\ 0.6 \end{pmatrix} tsf}$

Factored bearing resistance drops to 0.5 to 1.0 tsf in a phi = 0 total stress analysis.

Factored Bearing Resistance for service limit states - approximate by allowable bearing pressures computed using ASD approach and a FS of 3 to control settlements

$$q_r := \frac{q_n}{3} \qquad q_r = \begin{pmatrix} 0.5 \\ 0.5 \\ 0.5 \end{pmatrix} tsf$$

Service Limit State

$$q_{r\_m} := \frac{q_{n\_m}}{3} \qquad q_{r\_m} = \begin{pmatrix} 0.4 \\ 0.4 \\ 0.4 \end{pmatrix} tsf$$

Analysis: Nominal and factored bearing resistance
Structure: Large mat footing directly founded on native soils

$psf := \frac{lbf}{ft^2}$ $pcf := \frac{lbf}{ft^3}$ $Mg := 1000 \cdot kg$ $kN := 1000 \cdot newton$ $kPa := \frac{kN}{m^2}$ $tonf := g \cdot ton$ $kip := 1000 \cdot lbf$

$ksf := \frac{kip}{ft^2}$ $ton := 2000 \cdot lbf$ $tsf := \frac{tonf}{ft^2}$ $psi := \frac{lbf}{in^2}$ $ksi := \frac{kip}{in^2}$

Assumptions:

1. Base of mat footing founded with 2 feet embedment below culvert invert.

2. Assumed pamaters for medium stiff silt, little organics to soft clay silt (N-values range from 5 to 4 bpf)
   saturated unit weight = 115 pcf
   dry unit weight = 110 pcf
   internal friction angle of 20 degree
   undrained shear strength (c) 500 psf

3. Method used: Terzaghi, assume a 20 ft by 40 ft mat foundation directly on soft soils.

4. Examine conditions: footing on ϕ–c soil (ref: Bowles Ex. 4-1 pg 231), effective stress analysis.

Foundation soil values

Available References:

ϕ : Lambe & Whitman Table 11.3 based on Hough, Basic Soils Engr, 1967
ϕ, SPT correlation, Lambe & Whitman, Fig 11.14, (from Peck, Hanson, Thornburn).
ϕ and γ correlations to soil description and N values, Bowles (5th Ed) Tables 3-4 and 2-6
γ sat : Holtz, Kovacs, Table 2-1 1981

Footing Width and Depth

$B := \begin{pmatrix} 10 \\ 15 \\ 20 \end{pmatrix} \cdot ft$ $D_f := 2.0 \cdot ft$ $D_w := -2 \cdot ft$ $\gamma_w := 62.4 \cdot pcf$

Foundation Soil

$\gamma 1_{sat} := 115 \cdot pcf$ $\gamma 1_d := 110 \cdot pcf$ $\gamma 1_t := \gamma 1_{sat}$ $\phi := 20deg$ $c_1 := 500 \cdot psf$

Nominal Bearing Resistance - based on Presumptive Bearing Capacity

For Service Limit States ONLY

Based on NavFac DM 7.2 pg 142-143 Table 1 - "Presumptive Values of Allowable Bearing Capacity Pressures for Spread Foundations".

Bearing Material:	Consistency in Place:	Allowable Bearing Pressure (tons per sq. foot):	Recommended Value:
Inorganic silt or clayey silt	Very stiff to hard	2 to 4	3 tsf
	Medium to stiff	1 to 3	1.5 tsf
	Soft	0.5 to 1	0.5 tsf

Recommend 1 tsf / 2 ksf

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

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

$s_\gamma := 0.8$ $\qquad$ $s_c := 1.3$

Terzaghi Bearing Capacity Factors - (Ref: Bowles Table 4-2, 5th Ed. pg 222) for ϕ=20 degrees

$N_c := 17.7$ $\qquad$ $N_q := 7.4$ $\qquad$ $N_\gamma := 5.0$

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

$N_{c\_m} := 14.83$ $\qquad$ $N_{q\_m} := 6.4$ $\qquad$ $N_{\gamma\_m} := 2.9$

Nominal Bearing Resistance per Terzaghi equation (Bowles, Table 4-1, 5th Ed., pg 220) for ϕ=20

Groundwater is 2 feet above the invert, therefore 4 ft above the bottom of the mat

$q := (2\cdot ft\cdot\gamma_w + \gamma1_{sat}\cdot 2\cdot ft) - 4\cdot ft\cdot\gamma_w$ $\qquad$ $q = 0.053\ tsf$

Using Terzaghi Factors

$q_n := c_1\cdot N_c\cdot s_c + q\cdot N_q + 0.5\cdot(\gamma1_{sat} - \gamma_w)\cdot B\cdot N_\gamma\cdot s_\gamma$ $\qquad$ $q_n = \begin{pmatrix} 6.7 \\ 6.9 \\ 7.2 \end{pmatrix} tsf$

Using Meyerhof Factors

$$q_{n\_m} := c_1 \cdot N_{c\_m} \cdot s_c + q \cdot N_{q\_m} + 0.5 \cdot (\gamma 1_{sat} - \gamma_w) \cdot B \cdot N_{\gamma\_m} \cdot s_\gamma \qquad q_{n\_m} = \begin{pmatrix} 5.5 \\ 5.6 \\ 5.8 \end{pmatrix} \text{tsf}$$

Factored Bearing Resistance for strength limit states

Terzaghi

$$q_r := q_n \cdot 0.45 \qquad \boxed{q_r = \begin{pmatrix} 3 \\ 3.1 \\ 3.2 \end{pmatrix} \text{tsf}}$$

Strength Limit State

Meyerhof

$$q_{r\_m} := q_{n\_m} \cdot 0.45 \qquad \boxed{q_{r\_m} = \begin{pmatrix} 2.5 \\ 2.5 \\ 2.6 \end{pmatrix} \text{tsf}}$$

Recommend a limiting factored bearing resistance of 2.5 tsf or 5 ksf for mat foundations founded on the clay silt soils for the STRENGTH LIMIT STATE.

Factored Bearing Resistance for service limit states

Terzaghi

$$q_r := \frac{q_n}{3} \qquad \boxed{q_r = \begin{pmatrix} 2.2 \\ 2.3 \\ 2.4 \end{pmatrix} \text{tsf}}$$

Service Limit State

Meyerhof

$$q_{r\_m} := \frac{q_{n\_m}}{3} \qquad \boxed{q_{r\_m} = \begin{pmatrix} 1.8 \\ 1.9 \\ 1.9 \end{pmatrix} \text{tsf}}$$

Recommend a limiting factored bearing resistance of 2 tsf or 4 ksf, for mat foundations founded on the clay silt soils, to contol settlements - i.e the service limit state.

Nominal Bearing Resistance for Strength Limit States: Total Stress Analysis - $\phi = 0$

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

$$s_\gamma := 0.8 \qquad s_c := 1.3$$

Terzaghi Bearing Capacity Factors - (Ref: Bowles Table 4-2, 5th Ed. pg 222) for ϕ=0 degrees

$$N_c := 5.7 \qquad N_q := 1.0 \qquad N_\gamma := 0.0$$

Meyerhof Bearing Capacity Factors - (Ref: Bowles Table 4-4, 5th Ed. pg 223) for ϕ=0 degrees

$N_{c\_m} := 5.14$ $\quad N_{q\_m} := 1.0$ $\quad N_{\gamma\_m} := 0.0$

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

Groundwater is 2 feet above the invert, therefore 4 ft above the bottom of the mat

$$q := \left(2\cdot ft\cdot\gamma_w + \gamma 1_{sat}\cdot 2\cdot ft\right) - 4\cdot ft\cdot\gamma_w \qquad q = 0.053\ tsf$$

Terzaghi

$$q_n := c_1\cdot N_c\cdot s_c + q\cdot N_q + 0.5\cdot\left(\gamma 1_{sat} - \gamma_w\right)\cdot B\cdot N_\gamma\cdot s_\gamma \qquad q_n = \begin{pmatrix} 1.9 \\ 1.9 \\ 1.9 \end{pmatrix} tsf$$

Meyerhof

$$q_{n\_m} := c_1\cdot N_{c\_m}\cdot s_c + q\cdot N_{q\_m} + 0.5\cdot\left(\gamma 1_{sat} - \gamma_w\right)\cdot B\cdot N_{\gamma\_m}\cdot s_\gamma \qquad q_{n\_m} = \begin{pmatrix} 1.7 \\ 1.7 \\ 1.7 \end{pmatrix} tsf$$

Factored Bearing Resistance for Strength Limit State

Terzaghi $\quad q_r := q_n\cdot 0.45 \quad \boxed{q_r = \begin{pmatrix} 0.9 \\ 0.9 \\ 0.9 \end{pmatrix} tsf}$ Strength Limit State

Meyerhof $\quad q_{r\_m} := q_{n\_m}\cdot 0.45 \quad \boxed{q_{r\_m} = \begin{pmatrix} 0.8 \\ 0.8 \\ 0.8 \end{pmatrix} tsf}$

Recommend a limiting factored bearing resistance of 1 tsf or 2 ksf, for mat foundations founded on **SATURATED** clay silt soils, for the strength liimit state.

Factored Bearing Resistance for Service Limit State

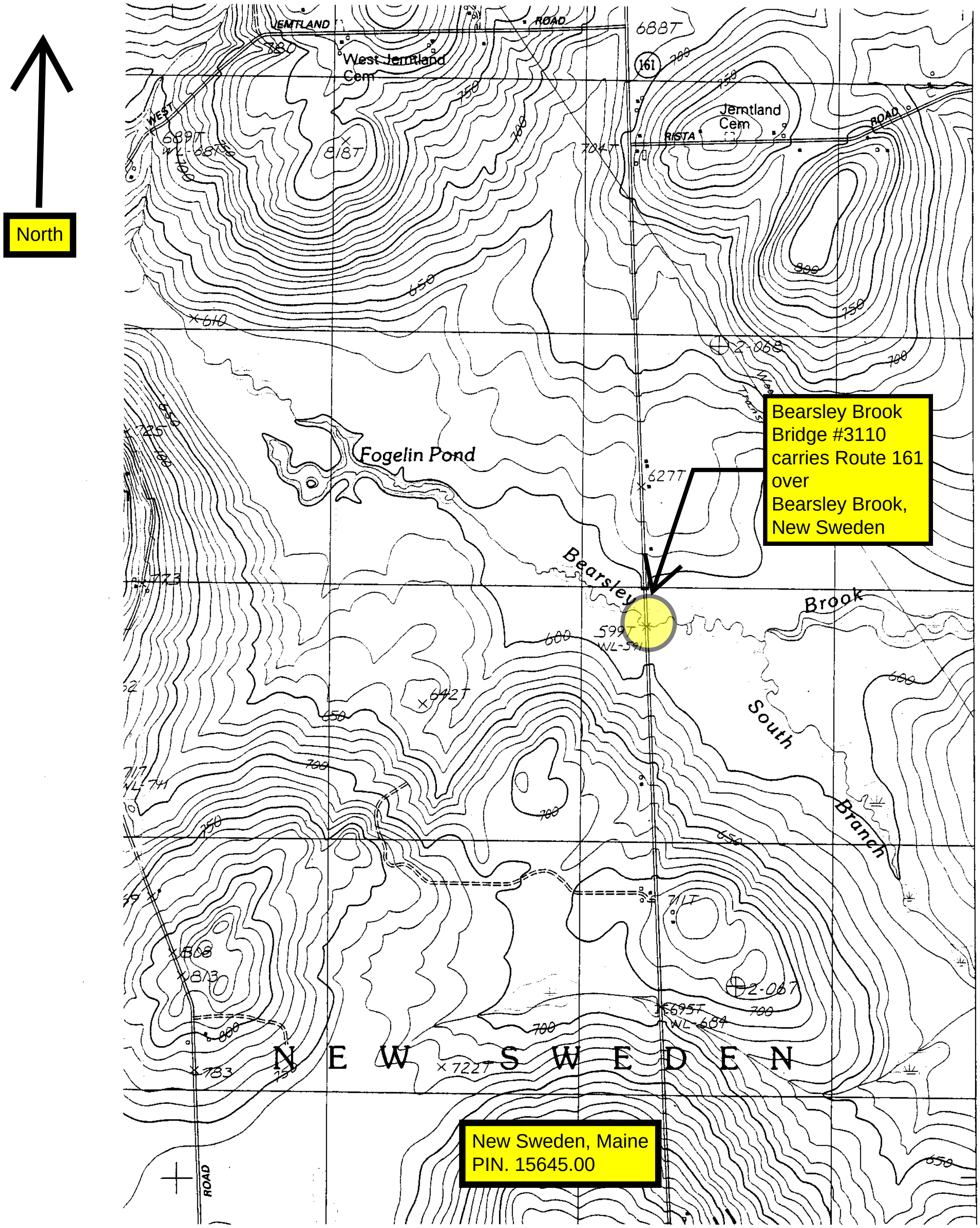
Terzaghi $\quad q_r := \dfrac{q_n}{3} \quad \boxed{q_r = \begin{pmatrix} 0.6 \\ 0.6 \\ 0.6 \end{pmatrix} tsf}$ Service Limit State

Meyerhof $\quad q_{r\_m} := \dfrac{q_{n\_m}}{3} \quad \boxed{q_{r\_m} = \begin{pmatrix} 0.6 \\ 0.6 \\ 0.6 \end{pmatrix} tsf}$

Recommend a limiting factored bearing resistance of 0.5 tsf or 1 ksf, for mat foundations founded on the clay silt soils, to contol settlements - i.e the service limit state.

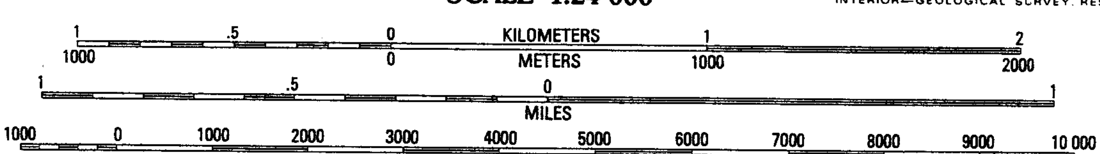
Sheets

MUD LAKE QUADRANGLE
MAINE – AROOSTOOK CO.
7.5 MINUTE SERIES (TOPOGRAPHIC)

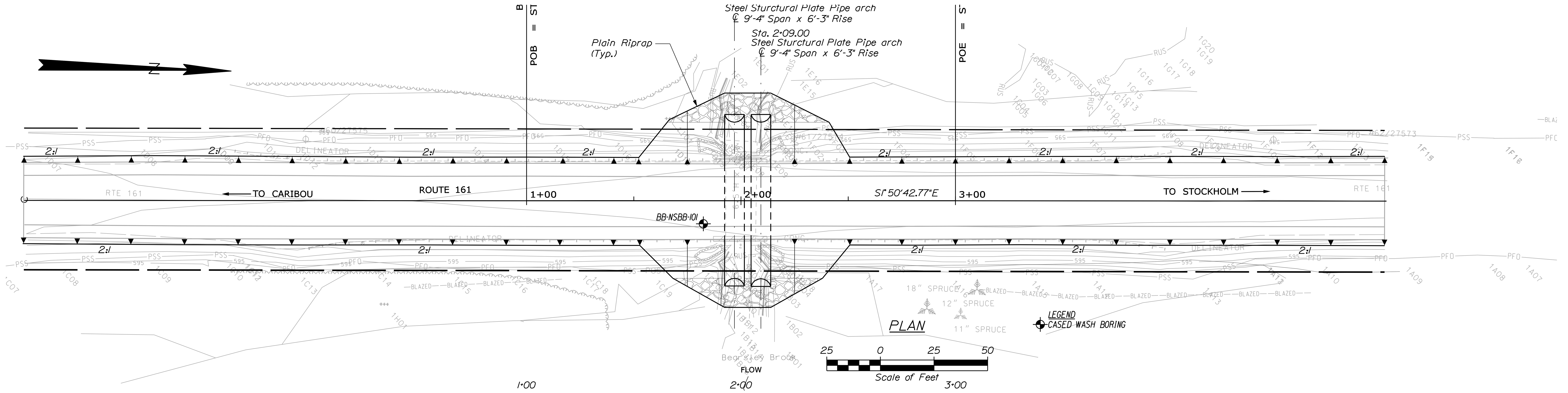


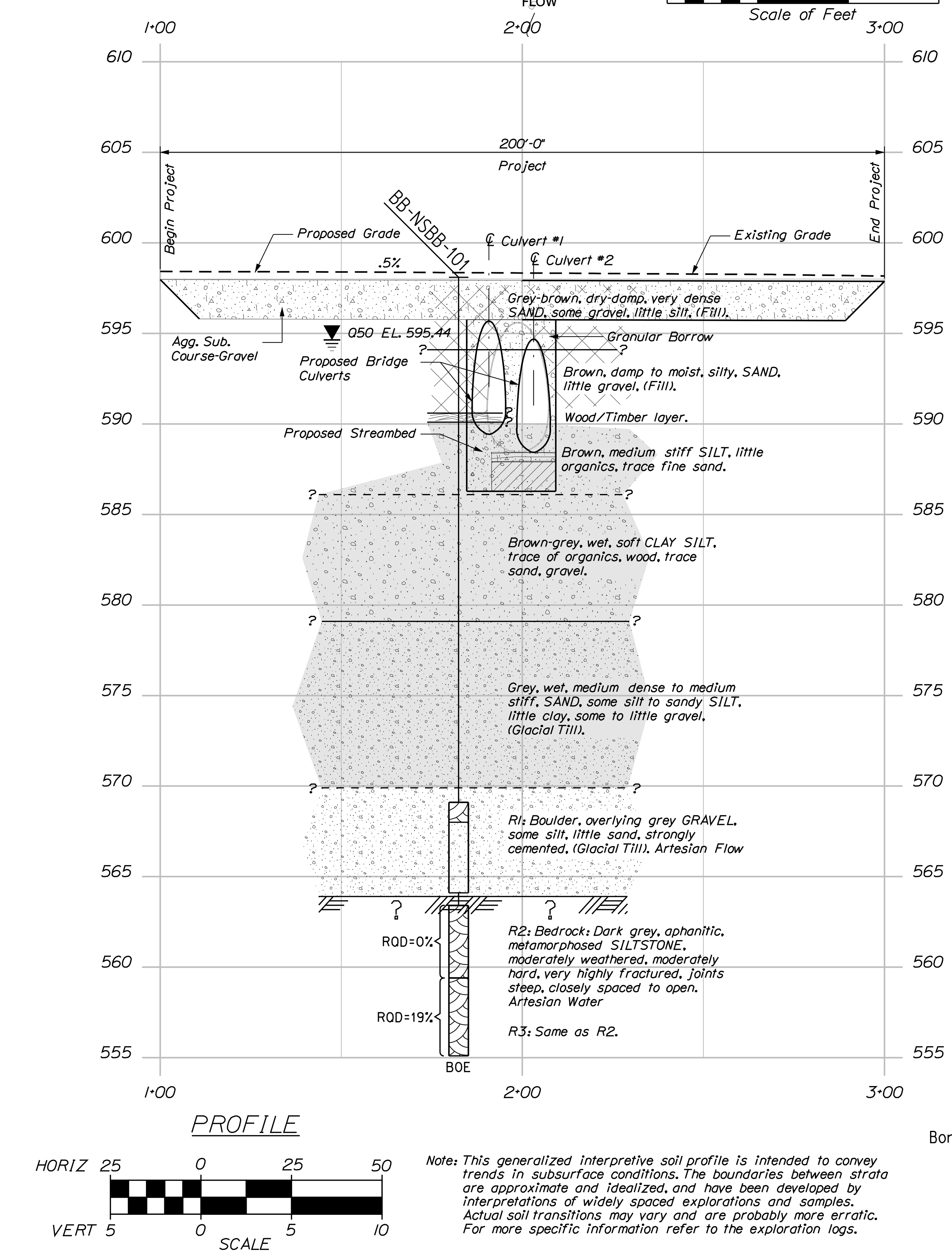
SCALE 1:24 000

INTERIOR—GEOLOGICAL SURVEY, RESTO



CONTOUR INTERVAL 10 FEET





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

Boring No. Offset, if shown
Strata Interface
Boring
RQD= Rock Quality Designation for Rock Core Sample
BOE= Bottom Of Exploration

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BRIDGE PLANS

PIN 15645.00

BRIDGE NO. 3110

	BY	DATE
PROJ. MANAGER	D. Anderson	
DESIGN-DETAILED	T. WHITE	MAY 2008
CHECKED-REVIEWED		
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

BEARSLEY BROOK BRIDGE
BEARSLEY BROOK
NEW SWEDEN AROOSTOOK COUNTY

BORING LOCATION PLAN & INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
2
OF _

Filename: ...\GEOTECH\MSTA\002_BLP&ISP1.dgn Division: GEOTECH Username: terry.white Date:6/10/2008

Filename: ...\msta\003_BORING LOGS1.dgn Division: GEOTECH Username: terry.white Date:6/10/2008

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: Bearsley Brook Bridge over Bearsley Brook, Route 161
Location: New Sweden, Maine

Boring No.: BB-NSBB-101

PIN: 15645.00

Driller:	MaineDOT	Elevation (ft.)	598.1	Auger ID/OD:	5" Solid Stem
Operator:	E. Giguere	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	G. Lidstone	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	10/10/07-10/11/07	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-1.88"
Boring Location:	1+82.4, 10.8 Rt.	Casing ID/OD:	NW	Water Level*:	Artesian - See Remarks
Hammer Efficiency Factor: 0.77		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows
0							SSA
	1D	24/14	1.00 - 3.00	12/22/18/19	40	51	
5	2D/AB	24/9	5.00 - 7.00	5/8/5/3	13	17	23
							14
							256
							9
							12
10	MD	24/0	10.00 - 12.00	3/2/2/2	4	5	18
							15
							14
							14
							12
15	3D/MV	24/6	15.00 - 17.00	6/1/2/2	3	4	17
							14
							15
	MV			Could not push			20
							32
20	4D	24/13	20.00 - 22.00	6/6/3/4	9	12	38
							43
							47
							54
							57
25	5D	24/9	25.00 - 27.00	3/2/3/6	5	6	52
							65
							81
							132
	R1	60/27	29.00 - 34.00				242 NQ
30							93
							90
							75
							92
	R2	48/44	34.70 - 38.70	RQD = 0%			a55 NQ
35							
	R3	51.6/47	38.70 - 43.00	RQD = 19%			
40							
45							
50							

Visual Description and Remarks

Elevation (ft.)	Depth (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	0	PAVEMENT.	
597.40	0.70	Grey-brown, dry to damp, very dense, fine to coarse SAND, some angular to rounded fine gravel, little silt, (Fill).	G#209968 A-1-b, SM WC=6.9%
594.10	4.00	Brown, damp, medium dense, silty fine to coarse SAND, little angular to rounded fine gravel, (Fill). (2D/A) 5.0-6.5' bgs.	
591.60	6.50	(2D/B) 6.5-7.0' bgs. Brown, weathered, moist, medium dense, silty fine to coarse SAND, little fine gravel, (Fill).	
590.60	7.50	Wood from 7.5-8.0' bgs.	
590.10	8.00	Brown, medium stiff, SILT, little organics, fine sand seen in wash water. 2 sampling attempts for MD.	
586.10	12.00	Brown-grey, wet, soft, CLAY SILT, trace of organics, wood, trace fine sand, angular coarse gravel. 2 sampling attempts for 3D. Failed 55x110 mm vane attempt, could not push. Roller Coned ahead from 15.0-18.0' bgs. Sand in wash water. Failed 55x110 mm vane attempt.	
579.10	19.00	Grey, wet, medium dense, fine to medium SAND some silt, little clay, little fine gravel, trace coarse gravel.	G#209969 A-4, SC-SM Non-Plastic WC=11.2%
573.10	25.00	Similar to above. Grey, wet, medium stiff, fine sandy SILT, little clay, some angular coarse gravel, trace medium to coarse sand.	
569.90	28.20	TILL, probably similar to till seen in R1.	
569.10	29.00	R1: BOULDER from 29.0-30.1' bgs. R1: 1.1-ft metasiltstone boulder. Core Times (min:sec) 29.0-30.0' (3:10) 30.0-31.0' (3:45) 31.0-32.0' (3:46) 32.0-33.0' (5:12) 33.0-34.0' (3:05) 45% Recovery	
568.00	30.10	Grey, angular to subrounded coarse to fine GRAVEL, some silt, little fine to medium sand, strongly cemented. (Glacial Till). a55 blows for 0.2'.	
563.90	34.20	Top of Bedrock at Elev. 563.9'. Roller Coned ahead from 34.2-34.7' bgs.	
563.40	34.70	R2:Bedrock: Dark grey, aphanitic, metamorphosed SILTSTONE, moderately weathered, moderately hard, very highly fractured, jointing along foliation at steep angles, closely spaced, tight to open, some with silt on surfaces. R2:Core Times (min:sec) 34.7-35.7' (8:12) 35.7-36.7' (4:33) 36.7-37.7' (4:40) 37.7-38.7' (6:18) 98% Recovery	
559.40	38.70	R3:Bedrock: Same as R2, and very highly fractured. R3:Core Times (min:sec) 38.7-39.7' (12:15) 39.7-40.7' (7:26) 40.7-41.7' (7:53) 41.7-42.7' (8:40) 42.7-43.0' (6:07) 90% Recovery	
555.10	43.00	Bottom of Exploration at 43.00 feet below ground surface.	

Remarks:

Artesian flow noted after R1, R2 and R3 when extracting casing. Flow was neutralized with casing sticking up 4' above ground surface.
10/10/07: 13:00-17:00, 10/11/07: 08:00-12:30;
Artesian flow continued after demobilizing. Crew returned on 10/23/07 and augered to 16 ft with 5" SSA, put in a 12-ft of Bentonite seal, 2 ft of auger cuttings and 2 ft of Portland Type 2 Cement ontop. Used 1 50 lb bag of Holeplug 3/8-in coarse grade bentontite chips.

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-NSBB-101

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

PIN 15645.00

BRIDGE PLANS

BRIDGE NO. 3110

	BY	DATE
PROJ. MANAGER	D. Anderson	
DESIGN-DETAILED	T. WHITE	MAY 2008
CHECKED-REVIEWED		
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

BEARSLEY BROOK BRIDGE
BEARSLEY BROOK
NEW SWEDEN AROOSTOOK COUNTY

BORING LOGS

SHEET NUMBER

3

OF _