MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

DAVIS #2 BRIDGE
OVER DAVIS STREAM
JEFERSON, MAINE

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GEOTECHNICAL DESIGN SUMMARY

This report provides geotechnical recommendations for replacement of the Davis #2 Bridge over Davis Stream in Jefferson, Maine. MaineDOT is employing the "Detail-Build" project delivery method for this project. As such the exact replacement structure has yet to be determined. The project detail-build special provision will require that 1) existing abutments shall be removed in entirety, 2) abutments and wingwalls shall be founded on cast-in-place footings on bedrock and 3) the clear-span shall be 30 feet minimum. The replacement bridge design must conform to the requirements of the Bridge Design Guide (BDG) and the AASHTO LRFD Bridge Design Specifications, 5th Edition, 2010, (herein referred to as LRFD). The design and construction recommendations below are discussed in greater detail in Section 7.0 Evaluation and Recommendations.

Cantilever Abutments and Wingwalls – The abutments and wingwalls will be designed to resist all lateral earth loads, vehicular loads, superstructure loads, and any loads transferred through the superstructure. Abutments and wingwalls will be designed for all relevant strength, service and extreme limit states in accordance with LRFD.

The design of project abutments founded on spread footings at the strength limit state shall consider nominal bearing resistance, eccentricity (overturning), lateral sliding and structural failure. A sliding resistance factor, φ_τ, of 0.90 shall be applied to the nominal sliding resistance of abutments and wingwalls founded on spread footings on bedrock. A maximum frictional coefficient of 0.70 at the bedrock-concrete interface should be assumed. For footings on bedrock, the eccentricity of loading at the strength limit state, based on factored loads, shall not exceed three-eighths (3/8^{ths}) of the footing dimensions, in either direction.

The bedrock at the site is highly fractured. Excavation of several feet of friable, weathered bedrock may be required and should be planned and accounted for on the estimated quantities sheet. The full extent of the rock excavation needed will not be known until the foundation excavation is made.

The overall global stability of a foundation is typically investigated at the Service I Load Combination and a resistance factor, φ, of 0.65. We do not anticipate shear failure along adversely oriented joint surfaces in the rock mass below the foundations, and therefore a global stability evaluation may be waived.

Earth loads shall be calculated using an active earth pressure coefficient, K_a, of 0.31 calculated using Rankine Theory for cantilever wingwalls. The designer may assume Soil Type 4 [Bridge Design Guide (BDG) Section 3.6.1] for backfill soil properties. The backfill properties are as follows: ϕ = 32 degrees, γ = 125 pounds per cubic foot (pcf). Additional lateral earth pressure due to construction or live load surcharge is required for the abutments and wingwalls if an approach slab is not specified. If a structural approach slab is specified, some reduction of surcharge loads is permitted.

Factored Bedrock Bearing Resistance – The factored bearing resistance at the strength limit state for spread footings on bedrock should not exceed 15 kips per square foot (ksf).

Based on presumptive bearing resistance values, a factored bearing resistance of 16 ksf may be used when analyzing the service limit state and for preliminary footing sizing, as allowed in LRFD C10.6.2.6.1. In no instance shall the service limit state bearing stress exceed the nominal resistance of the footing concrete, which may be taken as $0.3f'c$. The minimum footing size is 2 feet wide regardless of the applied bearing pressure or bearing material.

Settlement – Settlement of the bridge abutments due to elastic compression of the bedrock and any silt seams in the bedrock will be negligible and will occur during construction. Settlement of wall footings constructed on bedrock will be negligible. The new bridge vertical alignment will not change significantly so settlement beneath the approaches will be negligible.

Frost Protection – Foundations placed on bedrock are not subject to heave by frost. Thus, there are no frost embedment requirements for project footings cast directly on sound bedrock. If needed, any foundation placed on granular soils should be founded a minimum of 5.0 feet below finish exterior grade for frost protection. Riprap is not considered as contributing to the overall thickness of soils required for frost protection.

Scour and Riprap – For scour protection of the abutment and wingwall footings, place the bottom of seals or footings directly on bedrock surfaces cleaned of all weathered, loose and potentially erodible/scourable rock.

Bridge approach slopes and slopes at wingwalls should be armored with 3 feet of riprap in accordance with the MaineDOT Bridge Design Guide (BDG) Section 2.3.11. The riprap section shall be underlain by Class A erosion control geotextile and a 1 foot thick layer of bedding material conforming to Standard Specification 703.19, Granular Borrow for Underwater Backfill, as shown in Standard Detail 610 (03) except where riprap is placed directly over exposed bedrock. Riprap shall meet the requirements of Section 703.26, Plain and Hand Laid Riprap.

Riprap shall extend 1.5 feet horizontally in front of walls before sloping down at a maximum 1.75H:1V slope to the existing ground surface. The toe of riprap sections shall be constructed 1 foot below the streambed elevation where feasible. If bedrock occurs in the stream bed, the riprap should be placed at the design slope down to the stream bedrock surface.

Seismic Design Considerations – In accordance with LRFD 4.7.4.2, seismic analysis is not required for single-span bridges regardless of seismic zone. However, superstructure connections and bridge seat dimensions must satisfy LRFD Articles 3.10.9 and 4.7.4.4, respectively. Section 7.7 presents seismic parameters for this site.

Construction Considerations –

<u>Excavation</u>

- Construction of new abutment and retaining wall structures will require soil and loose/weathered bedrock excavation. Earth support systems may be required.
- Remove the old abutments in their entirety.

- Prepare bedrock subgrade for abutment footings by creating level benches or a completely level surface. Bedrock excavation may use conventional equipment, but may also require drilling and blasting methods. All loose bedrock fragments and soil debris should be removed from bearing surfaces and the surfaces washed with high pressure water and air before concrete or seal concrete is placed for the abutment and wingwall foundations.

Blasting

- Where blasting is required, conduct pre and post-blast condition surveys, as well as, blast vibration monitoring at nearby residences and bridge structures in accordance with MaineDOT Standard Specification 105.2.6, Use of Explosives and industry standards at the time of blast.

Dewatering

- Control groundwater and surface water infiltration to permit construction in-the-dry.
- Cofferdams, temporary ditches, pumping from sumps, granular drainage blankets, stone ditch protection, or hand-laid riprap with geotextile underlayment may be needed to divert surface water or groundwater if significant seepage is encountered during excavation.

Reuse of Excavated Soil and Bedrock

- Do not use excavated existing subbase aggregate for pavement structure construction or to re-base shoulders or for abutment and wall backfill soil. Excavated subbase sand and gravel may be used as fill below subgrade elevation in fill embankment areas.
- Do not use excavated existing fill or native soils for fill anywhere beneath the pavement structure, dressing slopes, abutments or walls. Use these soils to dress slopes only below the bottom elevation of the shoulder subbase gravel.
- Silty native soils or existing fill soils may be used as common borrow in accordance with MaineDOT Standard Specification Sections 203 and 703. It may be necessary to spread out and dry portions of these soils that are excessively moist.

Embankment Fill Areas

- Bench existing fill slope soils in accordance with MaineDOT Standard Specification 203.09, Preparation of Embankment Area, where new fill slope extensions are constructed over existing slopes.

Erosion Control

- Use MaineDOT Best Management Practices February 2008 to minimize erosion of fine-grained soils found on the project site.

1.0 INTRODUCTION

The Maine Department of Transportation (MaineDOT) plans to replace Davis #2 Bridge carrying Goose Hill Road over Davis Stream in the Town of Jefferson, Lincoln County, Maine. We show the project location on Sheet 1, Site Location Map, appended to this report. We conducted subsurface investigations at the bridge site to develop geotechnical recommendations for the structure replacement. This report summarizes our findings, discusses our evaluation of the subsurface conditions and presents our geotechnical recommendations for design and construction of the bridge foundations.

No record was found for the construction date of the existing stacked stone abutments supported by spread footings founded on bedrock. A 2009 bridge inspection assigned the substructures a rating of 3 – Serious. The Davis #2 Bridge had a timber superstructure replacement and Abutment 1 concrete jacketing in 1948. The current bridge seats, backwalls, painted steel I beams and concrete deck were constructed in 1965, with 1-inch open joints at each end.

The bridge beam ends have heavy scaling to minor crushing. The structure is currently in poor to serious condition and in need of complete replacement. It also has insufficient bridge width for a two lane structure and substandard bridge and approach railings. Current plans call for the complete removal and replacement of the existing superstructure and substructure. As of the year 2009, the bridge sufficiency rating was 29.5.

The final bridge configuration will comprise of a single-lane, 14-foot wide superstructure. The horizontal and vertical alignment will remain virtually the same except for the reduced bridge width. The MaineDOT Bridge Program plans to use the "detail-build" project delivery method for this project. Consequently, there are no specific bridge type plans for the final substructures and superstructure. However, the project detail-build special provision will require that 1) existing abutments shall be removed in entirety, 2) abutments and wingwalls shall consist of cast-in-place reinforced concrete cantilever-type walls founded on spread footings on bedrock and 3) the clear-span shall be 30 feet minimum.

2.0 GEOLOGIC SETTING

The Davis #2 Bridge on Goose Hill Road in Jefferson crosses the Davis Stream approximately 1.0 mile south of the town line as shown on Sheet 1, Site Location Map, presented at the end of this report. Davis Stream flows in a southwesterly direction to Damariscotta Lake.

The Maine Geologic Survey (MGS) "Surficial Geology of Jefferson Quadrangle, Maine, Open-File No. 75-24" (1975) indicates that surficial soils in the vicinity of Davis #2 Bridge consists primarily of glacial marine deposits with numerous nearby moraine soil unit contacts. The predominant native soil units at the site based on our subsurface explorations are glaciomarine or moraine which consist of sands and silts.

According to the "Bedrock Geologic Map of Maine" MGS (1985), the bedrock at the Davis #2 Bridge site consists of Devonian-Ordovician calcareous sandstone, interbedded sandstone and impure limestone of the Bucksport Formation.

3.0 SUBSURFACE INVESTIGATION

MaineDOT investigated subsurface conditions at the site by drilling four test borings BB-JDS-101, BB-JDS-102, BB-JDS-103, and BB-JDS-104 on May 11 and 12, 2010. The approximate boring locations are shown on Sheet 2, Boring Location Plan and Interpretive Subsurface Profile, found at the end of this report. All of the soil borings were terminated with bedrock cores. We present the details and sampling methods used, field data obtained, and soil and groundwater conditions encountered in the boring logs in Appendix A and on Sheet 3, Boring Logs, provided at the end of this report.

The MaineDOT geotechnical team member selected the boring locations and drilling methods, designated the type and depth of sampling techniques, and identified field and laboratory testing requirements. A MaineDOT Inspector certified under the Northeast Transportation Technician Certification Program logged the subsurface conditions encountered on the field logs. The field crew tied down the boring locations by taping distances to adjacent site features. The boring locations were later picked up by MaineDOT survey.

The drill crew used solid stem auger and cased wash boring techniques to conduct the borings. Soil samples were obtained, where possible, at 5-foot intervals using Standard Penetration Test (SPT) methods. The standard penetration resistances, or N-values, discussed in this report are corrected for average hammer energy transfer. We compute the corrected or, N_{60}-values, by applying an average hammer energy transfer factor of 0.84 to the raw field N-values obtained with the MaineDOT drill rig. Bedrock was cored using an NQ-2 core barrel producing a 2.0-inch diameter rock core.

4.0 LABORATORY TESTING

We conducted a laboratory soil testing program on selected samples recovered from the test borings to evaluate soil classification, material reuse, and subgrade soil properties. Laboratory testing consisted of four standard grain size analyses with natural water contents tests. We present results of laboratory testing in Appendix B, Laboratory Test Data. The AASHTO and Unified Soil Classification System (USCS) soil classifications and water content data are also presented on the boring logs in Appendix A.

5.0 SUBSURFACE CONDITIONS

The surficial geology map shows that the bridge site is located in a region of glaciomarine sediments and numerous end moraine ridges. However, the bridge site is situated at the end of short fill extensions built into the Davis Stream cut channel. Consequently, the soil behind the abutments is predominantly granular fill overlying a thin veneer of glaciomarine or moraine soils. We found that the glaciomarine or moraine soil overlies bedrock. All of the boring locations are underlain by metasandstone bedrock. We provide an interpretive subsurface profile depicting the site stratigraphy on Sheet 2, Boring Location Plan and Interpretive Subsurface Profile, found at the end of this report. A summary description of the subsurface conditions follows:

5.1 Granular Fill

We encountered granular fill to a depth ranging between approximately 4.0 and 5.0 feet below ground surface (bgs). The granular fill generally consists of fine to coarse sand, with little to some gravel and little silt. The SPT N_{60}-values in the granular fill ranged from 7 to 35 blows per foot (bpf) indicating that the unit is loose to dense in consistency.

The granular fill samples subjected to laboratory testing had water contents of approximately 5 percent. Grain size analyses conducted on selected samples of the fill soils indicate that the soils are classified as A-1-b by the AASHTO Classification System and SM under the Unified Soil Classification System.

5.2 Glaciomarine and Moraine Soils

We generally encountered a layer of glaciomarine sediments or moraine soils beneath the granular fill. These soils generally comprised of silty fine to coarse sand with little gravel and occasional cobbles, or fine to coarse sand with some gravel and silt, or silt with some fine to coarse sand and gravel. The thickness of this soil unit ranged between approximately 4.7 and 8.6 feet. SPT N_{60}-values ranged from 8 to 63 bpf, indicating these deposits are loose to very dense in consistency.

The samples selected for testing had water contents ranging between approximately 12 and 14 percent. Grain size analyses of the tested samples indicate that the soils are classified as A-2-4 or A-4 by the AASHTO Classification System and SM or ML under the Unified Soil Classification System.

5.3 Bedrock

We encountered bedrock at approximate depths ranging from 9.7 to 12.6 feet bgs. Locally, the bedrock is mapped as Devonian-Ordovician calcareous sandstone, interbedded sandstone and impure limestone of the Bucksport Formation. Visual identification of rock cores indicates that the bedrock at all the cored boring locations is a grey, fine-grained, meta-sandstone that is hard, fresh to slightly weathered with very close to close joints. The bedrock contains fractures that are oriented 30 to 45 degrees from horizontal, generally follow bedding planes, and have minor silt in-filling and iron-staining. We determined that the rock quality designation (RQD) of the bedrock ranged from 15 to 60 percent which correlates to a very poor to fair rock mass quality. The table below summarizes the top of bedrock elevations at the boring locations:

Substructure	Boring	Station	Depth to Bedrock (feet bgs)	Elevation of Bedrock Surface (feet)
Abutment No. 1	BB-JDS-101	11+55.3, 8.0 RT	11.4	143.1
	BP-JDS-102	11+68.2, 6.2 LT	9.7	144.6
Abutment No. 2	BB-JDS-103	12+20.3, 10.3 LT	12.5	140.6
	BP-JDS-104	12+35.8, 1.4 RT	12.6	141.7

Bedrock Depth and Elevation at the Boring Locations

5.4 Groundwater

We observed the groundwater level at approximately 1.6 to 7.6 feet bgs in the borings. However, the groundwater level will fluctuate with seasonal changes, runoff, and adjacent construction activities.

For a more detailed description of the subsurface conditions, please refer to Appendix A, Boring Logs attached to this report.

6.0 FOUNDATION ALTERNATIVES

The final configuration of the bridge superstructure and substructure will be determined by the contractor. The presence of shallow bedrock indicates that abutments and wingwalls on spread footings is the most practical and durable substructure alternative. Consequently, the "detail-build" special provision will require cast-in-place cantilever-type abutments and wingwalls on spread footings on bedrock. Section 7.0, Evaluation and Recommendations, of this report provides geotechnical design recommendations for spread footings founded on bedrock.

7.0 EVALUATION AND RECOMMENDATIONS

The project "detail-build" special provision will require spread footings on bedrock to replace the bridge at the Jefferson site. The design methodology used in the following evaluation is referenced from the AASHTO LRFD Bridge Design Specifications, 5th Edition, 2010.

7.1 Spread Footings on Bedrock

The borings encountered bedrock approximately 10 to 13 feet below the existing bridge approaches at the boring locations. It is therefore considered feasible that cofferdams and spread footings could be practically and economically constructed to bear on bedrock. The boring logs indicate that the bedrock at the site is moderately to highly fractured. Thus, it will be necessary to excavate all dislodged, loose fractured or highly weathered bedrock before placing seal or spread footing concrete. The full extent of the weathered bedrock excavation needed will not be known until the foundation excavation is made.

7.2 Abutment and Wingwall Design

Abutments and wingwalls shall be proportioned for all applicable load combinations in LRFD Articles 3.4.1 and 11.5.5 and shall be designed for all relevant strength, service and extreme limit states. The design of project abutments and wingwalls founded on spread footings at the strength limit state shall consider nominal bearing resistance, eccentricity (overturning), lateral sliding and structural failure.

A sliding resistance factor, φ_τ, of 0.90 shall be applied to the nominal sliding resistance of cast-in-place abutments and wingwalls founded on spread footings on bedrock. Sliding computations for resistance to lateral loads shall assume a maximum frictional coefficient of 0.70 at the bedrock-concrete interface.

For footings on bedrock, the eccentricity of loading at the strength limit state, based on factored loads, shall not exceed three-eighths (3/8^{ths}) of the footing dimensions, in either direction.

For scour protection of the abutment and wingwall footings, place the bottom of seals or footings directly on bedrock surfaces cleaned of all weathered, loose and potentially erodible/scourable rock. As such, strength and extreme event limit state designs do not need to consider foundation resistance after the design or check floods for scour.

A resistance factor of 1.0 shall be used to assess spread footing design at the service limit state, including: settlement, excessive horizontal movement and overall stability. The overall global stability of a foundation is typically investigated at the Service I Load Combination and a resistance factor, φ, of 0.65. We do not anticipate shear failure along adversely oriented joint surfaces in the rock mass below the foundations, and therefore a global stability evaluation may be waived.

Cantilever-type abutments and 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 = 0.31, calculated using Rankine Theory for cantilever-type abutments and wingwalls. See Appendix C – Calculations, for supporting documentation. The designer may assume Soil Type 4 (BDG Section 3.6.1) for backfill material soil properties. The backfill properties are as follows: ϕ = 32 degrees, γ = 125 pcf.

Additional lateral earth pressure due to construction surcharge or live load surcharge is required per Section 3.6.8 of the BDG for the abutments and wingwalls if an approach slab is not specified. In the case where a structural approach slab is specified, reduction of the surcharge loads is permitted per LRFD Article 3.11.6.5. The live load surcharge on walls may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) of 2.0 feet, per LRFD Table 3.11.6.4-1. The live load surcharge on abutments may be estimated as a uniform earth pressure due to an equivalent height of soil (h_{eq}) taken from the table below:

Abutment Height (feet)	h_{eq} (feet)
5.0	4.0
10.0	3.0
$\geq$ 20.0	2.0

All abutment and wingwall designs shall include a drainage system behind them to intercept any groundwater. Drainage behind the structure shall be in accordance with Section 5.4.1.4, Drainage, of the BDG.

Backfill within 10 feet of the abutments and wingwalls and side slope fill shall conform to MaineDOT Specification 709.19, Granular Borrow for Underwater Backfill. This gradation specifies 10 percent or less of 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 and below the approach slab.

Slopes in front of and sloping down to the wingwalls should be constructed with riprap and not exceed 1.75H:1V.

7.3 Factored Bedrock Bearing Resistance

Substructure 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 stress distribution may be assumed to be a triangular or trapezoidal distribution over the effective base as shown in LRFD Figure 11.6.3.2-2. The factored bearing resistance for any structure founded on bedrock shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 15 ksf. This assumes a bearing resistance factor, φ_b, for spread footings on bedrock of 0.45, based on bearing resistance evaluation using semi-empirical methods. A factored bearing resistance of 16 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.

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

7.4 Settlement

No significant vertical or horizontal alignment changes are currently planned for the bridge. We anticipate that all foundations will be constructed on bedrock. Thus, we expect that any settlement of the bridge abutments will be due to the elastic compression of the bedrock and will be negligible.

No compressible soils or peat occur beneath the existing approach embankments and no profile changes are planned. Consequently, settlement beneath approach embankments will be negligible.

7.5 Frost Protection

We recommend that any abutment and return wing spread footings at this site be founded on bedrock. Therefore, heave due to frost is not a design issue, and no requirements for minimum embedment depth are necessary.

We have, however, evaluated the potential frost depth at the site for footings placed on soil. Based on State of Maine frost depth maps, BDG Figure 5-1, the site has a design-freezing index of approximately 1400 F-degree days. Considering an assumed water content of 10 percent, this correlates to a frost depth of 5.5 feet at this site. We also considered frost depth projections computed by Modberg software developed by the US Army Cold Regions Research and Engineering Laboratory. The results of the Modberg frost depth model indicate a potential frost depth of 5.0 feet. Consequently, we recommend that any foundations or leveling pads constructed on soil at this site be founded a minimum of 5.0 feet below finished exterior grade. This minimum embedment applies only to foundations constructed on soil and not those founded on bedrock.

7.6 Scour and Riprap

We expect that abutment and return wing spread footings will be founded on bedrock. The bedrock at the site is not considered to be erodible. Therefore, no specific scour protection recommendations are needed. We recommend any abutment or wingwall footing should be armored with riprap.

The riprap layer shall be at least 3 feet thick. Stone riprap shall conform to MaineDOT Standard Specification 703.26, Plain and Hand Laid Riprap. For wingwalls and retaining walls, the riprap shall extend 1.5 feet horizontally in front of the walls before sloping at maximum 1.75H:1V slope to the existing ground surface. The toe of riprap sections shall be constructed 1 foot below the streambed elevation, unless the streambed consists of bedrock. The riprap section shall be underlain by Class A erosion control geotextile and a 1 foot thick layer of bedding material conforming to Standard Specification 703.19, Granular Borrow for Underwater Backfill, as shown in Standard Detail 610 (03).

7.7 Seismic Design Considerations

In conformance with LRFD Article 4.7.4.2, seismic analysis is not required for single-span bridges, regardless of seismic zone, however, superstructure connections and bridge seat dimensions shall be satisfied per LRFD 3.10.9 and 4.7.4.4, respectively. Furthermore, the bridge is not classified as a major structure since construction costs will be less than $10 million dollars, nor is it classified as functionally important. Consequently, seismic earth loads do not need to be considered in bridge substructure design.

The following parameters were determined for the site from the USGS Seismic Parameters CD provided with the LRFD Manual and LRFD Articles 3.10.3.1 and 3.10.6:

- Peak Ground Acceleration coefficient (PGA) = 0.069g
- Design spectral acceleration coefficient at 0.2-second period, S_{DS} = 0.235g
- Design spectral acceleration coefficient at 1.0-second period, S_{D1} = 0.103g
- Site Class D (stiff soil with an average shear wave velocity = 600ft/sec < v_s < 1,200ft/sec)
- Seismic Zone 1, based on an S_{D1} < 0.15g

7.8 Construction Considerations

7.8.1 Excavation

Construction of the new abutment structures and any retaining walls will require soil and loose weathered rock excavation. Earth support systems may be required.

We anticipate that the existing abutments will be removed in their entirety. Cofferdams will be needed.

The abutment foundation subgrade should consist of sound bedrock. The bearing surface should be cleaned of all overburden soils, and loose, dislodged bedrock fragments should be removed by mechanical means. Mechanical means include expansive agents, use of hydraulic hoe ram, hydraulic splitters, or wedging and prying. We recommend final bedrock surface preparation by washing with a high pressure water jet.

The nature, slope, and degree of fracturing in the bedrock will not be evident until the foundation excavation is made. The bedrock surface shall be cleared of all loose fractured bedrock and loose decomposed bedrock and soil. Excavation of highly sloped and loose bedrock material may be done using conventional excavation methods, but may require drilling and blasting techniques. We recommend anchoring, doweling, benching or other means of improving sliding resistance if the prepared bedrock surface is steeper than 4:1 (H:V) in any direction. The final bearing surface shall then be washed with high pressure water and air prior to concrete being placed for the footing. The final bedrock surface shall be approved by the Resident prior to placing seal or footing concrete.

Surface water should be diverted from the foundation excavation throughout the period of construction. We recommend removing any groundwater encountered at the base of the foundation excavation by using a sump pump located in a corner of the excavation outside of the foundation footprint.

The silty native soils are susceptible to disturbance and rutting as a result of exposure to water or construction traffic. We recommend that the contractor protect the subgrade from exposure to water and any unnecessary construction traffic. If disturbance and rutting occur, we recommend that the contractor remove and replace the disturbed materials and replace with compacted gravel borrow.

7.8.2 Blasting

Bedrock excavation may be needed to achieve abutment and wingwall subgrade elevation. The contractor should conduct all blasting work for the project in accordance with MaineDOT Standard Specification 105.2.6, Use of Explosives. We also recommend that the contractor conduct pre and post-blast surveys, as well as, blast vibration monitoring at nearby residences and bridge structures in accordance with industry standards at the time of blast.

7.8.3 Dewatering

The contractor should control groundwater and surface water infiltration to permit construction in-the-dry. We recommend that the contractor use temporary ditches, sumps, granular drainage blankets, stone ditch protection, or hand-laid riprap with geotextile underlayment to divert surface water and groundwater if significant seepage is encountered during construction. We also recommend using French drains daylighted to nearby ditches if significant seepage is encountered in the subgrade along the construction areas.

7.8.4 Reuse of Excavated Soil and Bedrock

The project plans call for excavation of the existing approach areas to achieve planned grades. In the process, the contractor will excavate both the existing subbase gravel, and subgrade fill soils. We do not recommend using the excavated subbase aggregate to re-base the bridge approaches. Excavated subbase and any granular fill excavation may be used as fill below subgrade elevation in fill embankment areas provided all other requirements of MaineDOT Standard Specification Sections 203 and 703 are met.

We do not recommend using excavated native soils as fill directly beneath the pavement structure. The silty native soils are typically susceptible to strength loss when wet or disturbed. The excavated soils may be allowed as fill in accordance with the Standard Specification 203 as shown on Standard Detail 203 (01). This soil may also be used for dressing slopes, but only below the bottom elevation of the shoulder subbase gravel.

The native silty soils or existing fill soils may be used as common borrow in accordance with MaineDOT Standard Specification Sections 203 and 703. Contractors should expect that prior to placement and compaction it may be necessary to spread out and dry portions of these soils that are excessively moist.

7.8.5 Embankment Areas Outside of Abutment/Wingwall Backfill Envelope

Embankment approach slopes that are created or extended as part of the bridge construction effort should be designed as earth fill slopes no steeper than 2:1 (H:V). Slopes steeper than 2:1 (H:V) typically require reinforcement or rock fill surfacing.

We recommend that all new embankment fill be thoroughly and systematically compacted to the full limit of the slope. Where new fill slope extensions are constructed over existing slopes, we recommend benching the existing slope soils in accordance with MaineDOT Standard Specification 203.09, Preparation of Embankment Area, to prevent creation of a preferential slip plane under the new embankment fill.

7.8.6 Erosion Control Recommendations

The fine-grained soils along the project are susceptible to erosion. We recommend using appropriate erosion control measures during construction as described in the MaineDOT Best Management Practices February 2008 guidelines to minimize erosion of the fine-grained soils at the site.

8.0 CLOSURE

This report has been prepared for use by the MaineDOT Bridge Program for specific application to the replacement of the Davis #2 Bridge over the Davis Stream in Jefferson, Maine. We have prepared the report in accordance with generally accepted soil and foundation engineering practices. No other intended use or warranty is expressed or 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 completed at discrete locations on the project 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 recommend that we be provided the opportunity for a general review of the final design drawings and specifications in order that we may verify that the earthwork and foundation recommendations have been properly interpreted and implemented in the design.

REFERENCES

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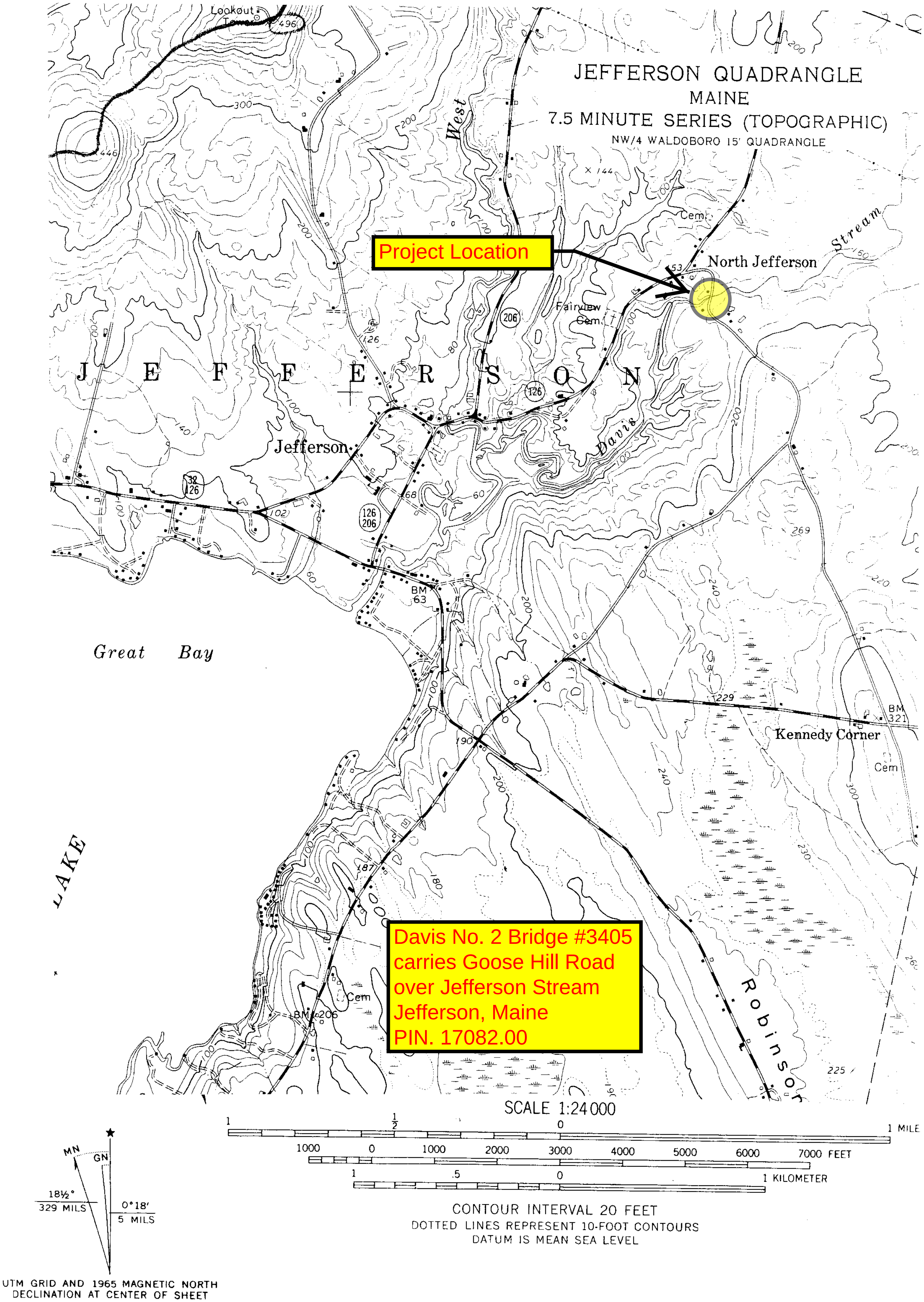
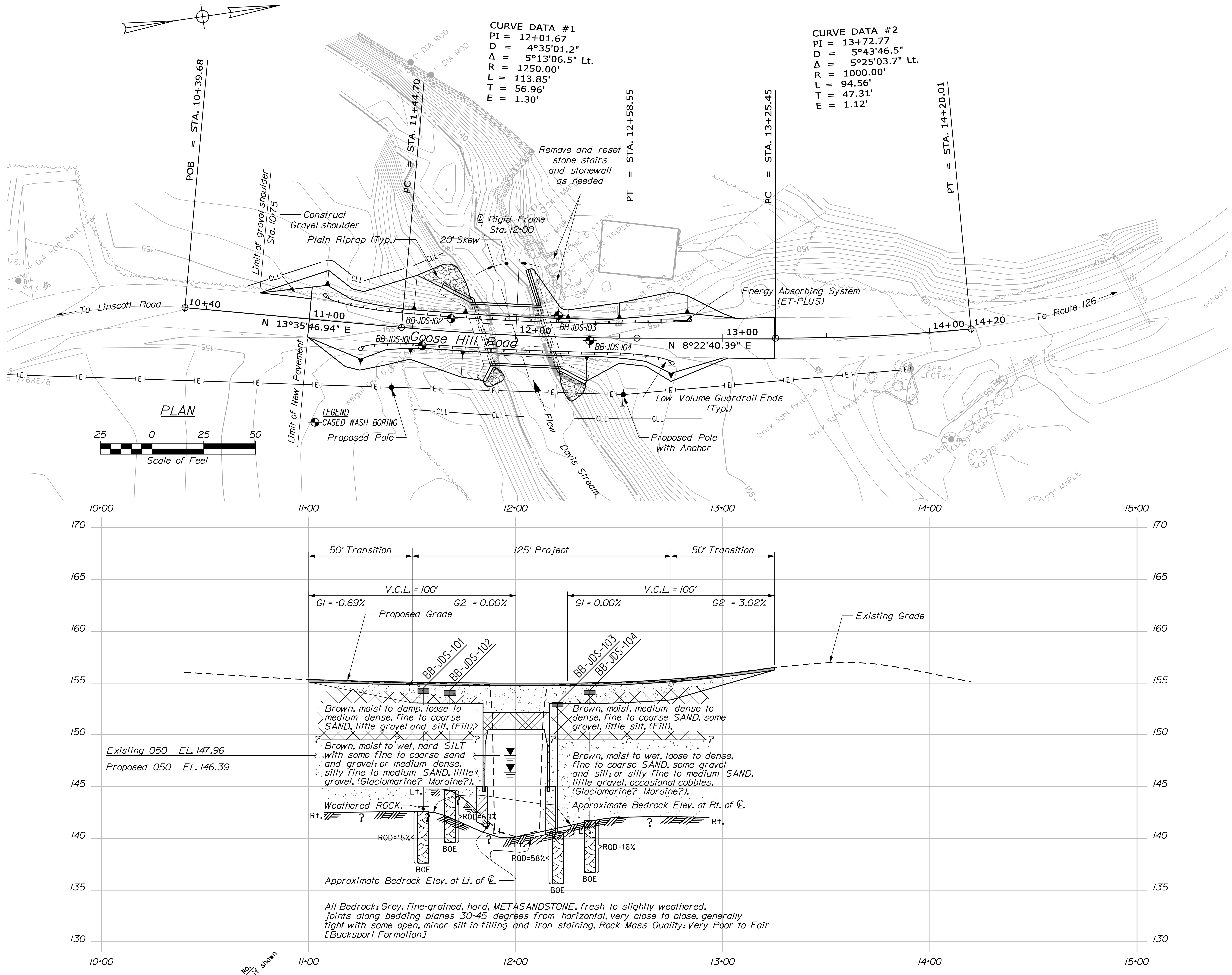
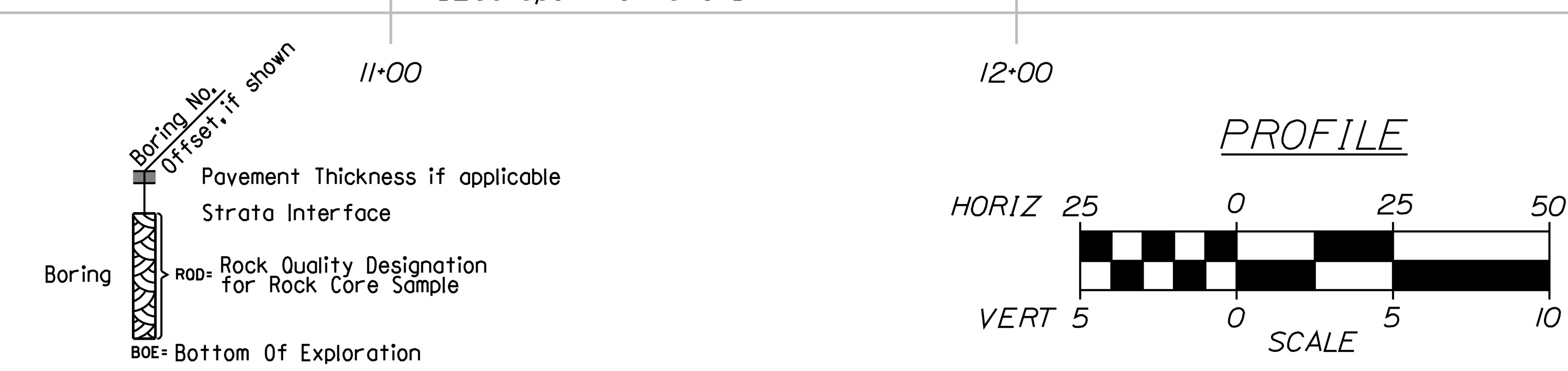
Sheets


JEFFERSON QUADRANGLE
MAINE
7.5 MINUTE SERIES (TOPOGRAPHIC)
NW/4 WALDOBORO 15' QUADRANGLE
Project Location
North Jefferson
Stream
Fairview Cem.
Cem
West
JEFFERSON
Jefferson
Davis
Lookout Tower
Great Bay
LAKE
Kennedy Corner
BM 321
BM 63
Robinson
Davis No. 2 Bridge #3405
carries Goose Hill Road
over Jefferson Stream
Jefferson, Maine
PIN. 17082.00
SCALE 1:24000
1 MILE
7000 FEET
1 KILOMETER
CONTOUR INTERVAL 20 FEET
DOTTED LINES REPRESENT 10-FOOT CONTOURS
DATUM IS MEAN SEA LEVEL
MN
GN
18½°
329 MILS
0°18′
5 MILS
UTM GRID AND 1965 MAGNETIC NORTH
DECLINATION AT CENTER OF SHEET





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

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BR-1708(200)X

PIN 17082.00

BRIDGE NO. 3405

BRIDGE PLANS

		BY	DATE
PROJ. MANAGER	S. BODGE		
DESIGN-DETAILED	M.MOREAU	T.WHITE	AUG 2010
CHECKED-REVIEWED			
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

DAVIS NO. 2
DAVIS STREAM
JEFFERSON LINCOLN COUNTY

BORING LOCATION PLAN & INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER

2

OF 3

Filename: ...\geotech\msta\005_BLP&ISP1.dgn Division: GEOTECH Username: terry.white Date:1/11/2011

Filename: ...\msta\006_BORING LOGS1.dgn   Division: GEOTECH   Username: terry.white   Date:1/11/2011

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

Project: Davis No.2 Bridge #3405 carries Goose Hill Road over Davis Stream
Location: Jefferson, Maine

Boring No.: BB-JDS-101   PIN: 17082.00

Driller:	MaineDOT	Elevation (ft.)	154.5	Auger ID/OD:	5" Solid Stem
Operator:	Giguere/Giles	Datum:	NAD 1983	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	5/11/10; 07:00-10:00	Drilling Method:	SSA and Cased Wash Boring	Core Barrel:	NQ-2
Boring Location:	11+55.3, 8.0 Rt.	Casing ID/OD:	NW	Water Level*:	1.6' bgs.
Hammer Efficiency Factor: 0.84		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA	154.00	PAVEMENT. 0.50	
	1D	24/18	1.00 - 3.00	14/10/8/5	18	25			Brown, damp, medium dense, fine to coarse SAND, little gravel and silt, (Fill).	G#237541 A-1-b, SM WC=5.0%
5	2D	4.8/4.8	5.00 - 5.40	50(4.8")	---			149.50	5.00 Olive-brown, moist, hard, SILT, some fine to coarse sand and gravel, (Glaciomarine? Moraine?).	
10	3D	16.8/16.8	10.00 - 11.40	19/25/50(4.8")	---		63		Similar to above, except wet. a51 blows for 0.4'.	G#237542 A-4, ML WC=13.8%
	R1	60/53	11.90 - 16.90	RQD = 15%			a51 NQ-2	143.10 142.60	11.40 Roller Coned ahead to 11.9' bgs. Weathered ROCK. 11.90 Bedrock: Grey, fine-grained, hard, METASANDSTONE, fresh to slightly weathered, joints along bedding planes typically 30-45 degrees from horizontal, very close to close, generally tightwith some open, minor silt in-filling and iron staining. Rock Mass Quality is Very Poor. [Bucksport Formation] R1:Core Times (min:sec) 11.9-12.9' (3:10) 12.9-13.9' (4:28) 13.9-14.9' (3:15) 14.9-15.9' (2:40) 15.9-16.9' (2:45) 88% Recovery	
15								137.60	16.90 Bottom of Exploration at 16.90 feet below ground surface.	
20										
25										

Remarks:
300-500# of down pressure on Core Barrel.

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-JDS-101

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

Project: Davis No.2 Bridge #3405 carries Goose Hill Road over Davis Stream
Location: Jefferson, Maine

Boring No.: BB-JDS-102   PIN: 17082.00

Driller:	MaineDOT	Elevation (ft.)	154.3	Auger ID/OD:	5" Solid Stem
Operator:	Giguere/Giles	Datum:	NAD 1983	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	5/11/10; 10:30-14:30	Drilling Method:	SSA and Cased Wash Boring	Core Barrel:	NQ-2
Boring Location:	11+68.2, 6.2 Lt.	Casing ID/OD:	NW	Water Level*:	None Observed
Hammer Efficiency Factor: 0.84		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA	153.80	PAVEMENT. 0.50	
	1D	24/15	1.00 - 3.00	5/2/3/5	5	7			Brown, moist, loose, fine to coarse SAND, little gravel, little silt, (Fill) .	
5	2D	24/12	5.00 - 7.00	2/5/11/17	16	22		149.30	5.00 Brown, wet, medium dense, silty, fine to medium SAND, little gravel, (Glaciomarine? Moraine?).	
							104		Cobble from 7.0-7.8' bgs.	
							90		Roller Coned ahead to 9.7' bgs.	
10	R1	60/53	9.70 - 14.70	RQD = 60%			NQ-2	144.60	9.70 Bedrock: Grey, fine-grained, hard, METASANDSTONE, fresh to slightly weathered, joints along bedding planes typically 30-45 degrees from horizontal, very close to close, generally tightwith some open, minor silt in-filling and iron staining. Rock Mass Quality is Fair. [Bucksport Formation] R1:Core Times (min:sec) 9.7-10.7' (2:05) 10.7-11.7' (2:30) 11.7-12.7' (2:40) 12.7-13.7' (2:30) 13.7-14.7' (3:00) 88% Recovery No water return.	
15								139.60	14.70 Bottom of Exploration at 14.70 feet below ground surface.	
20										
25										

Remarks:
400-600# of down pressure on Core Barrel.

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-JDS-102

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

Project: Davis No.2 Bridge #3405 carries Goose Hill Road over Davis Stream
Location: Jefferson, Maine

Boring No.: BB-JDS-103   PIN: 17082.00

Driller:	MaineDOT	Elevation (ft.)	153.1	Auger ID/OD:	5" Solid Stem
Operator:	Giguere/Giles	Datum:	NAD 1983	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	5/11/10; 13:30-15:00	Drilling Method:	SSA and Cased Wash Boring	Core Barrel:	NQ-2
Boring Location:	12+20.3, 10.3 Lt.	Casing ID/OD:	NW	Water Level*:	7.6' bgs.
Hammer Efficiency Factor: 0.84		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA	152.70	PAVEMENT. 0.40	
	1D	24/17	1.00 - 3.00	11/15/10/11	25	35			Brown, moist, dense, fine to coarse SAND, some gravel, little silt, (Fill).	G#237543 A-1-b, SM WC=5.4%
5	2D	24/16	5.00 - 7.00	2/2/4/8	6	8		148.60	4.50 Brown, wet, loose, fine to coarse SAND, some gravel and silt, (Glaciomarine? Moraine?).	G#237544 A-2-4, SM WC=12.4%
10	3D	24/14	10.00 - 12.00	11/21/24/22	45	63			Similar to above, except very dense.	
							24		a50 blows for 0.5'.	
	R1	60/55	12.50 - 17.50	RQD = 58%			a50 NQ-2	140.60	12.50 Bedrock: Grey, fine-grained, hard, METASANDSTONE, fresh to slightly weathered, joints along bedding planes typically 30-45 degrees from horizontal, very close to close, generally tight with some open, minor silt in-filling and iron staining. Rock Mass Quality is Fair. [Bucksport Formation] R1:Core Times (min:sec) 12.5-13.5' (3:15) 13.5-14.5' (3:10) 14.5-15.5' (3:00) 15.5-16.5' (3:10) 16.5-17.5' (2:45) 92% Recovery	
15								135.60	17.50 Bottom of Exploration at 17.50 feet below ground surface.	
20										
25										

Remarks:
400-600# of down pressure on Core Barrel.

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-JDS-103

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

Project: Davis No.2 Bridge #3405 carries Goose Hill Road over Davis Stream
Location: Jefferson, Maine

Boring No.: BB-JDS-104   PIN: 17082.00

Driller:	MaineDOT	Elevation (ft.)	154.3	Auger ID/OD:	5" Solid Stem
Operator:	Giguere/Giles	Datum:	NAD 1983	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	5/12/10; 07:30-10:00	Drilling Method:	SSA and Cased Wash Boring	Core Barrel:	NQ-2
Boring Location:	12+35.8, 1.4 Rt.	Casing ID/OD:	NW	Water Level*:	6.8' bgs.
Hammer Efficiency Factor: 0.84		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA	153.90	PAVEMENT. 0.40	
	1D	24/17	1.00 - 3.00	9/10/7/7	17	24			Brown, moist, medium dense, fine to coarse SAND, some gravel, little silt, (Fill).	
5	2D	24/20	5.00 - 7.00	6/9/25/21	34	48		150.30	4.00 Brown, moist, dense, silty fine to medium SAND, little gravel, occasional cobbles, (Moraine).	
									Cobble from 8.5-9.0' bgs.	
10	3D	24/18	10.00 - 12.00	7/10/17/42	27	38	23		Similar to above, except wet.	
							65		a100 blows for 0.6'.	
	R1	60/60	12.60 - 17.60	RQD = 16%			a100 NQ-2	141.70	12.60 Bedrock: Grey, fine-grained, hard, METASANDSTONE, fresh to slightly weathered, joints along bedding planes typically 30-45 degrees from horizontal, very close to close, generally tightwith some open, minor silt in-filling and iron staining. Rock Mass Quality is Very Poor. [Bucksport Formation] R1:Core Times (min:sec) 12.6-13.6' (2:55) 13.6-14.6' (2:40) 14.6-15.6' (2:40) 15.6-16.6' (3:10) 16.6-17.6' (3:12) 100% Recovery	
15								136.70	17.60 Bottom of Exploration at 17.60 feet below ground surface.	
20										
25										

Remarks:
400-600# of down pressure on Core Barrel.

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-JDS-104

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BR-1708(200)X

BRIDGE NO. 3405   PIN 17082.00   BRIDGE PLANS

		BY	DATE
PROJ. MANAGER	S. BODGE		
DESIGN-DETAILED	M.MOREAU	T.WHITE	AUG 2010
CHECKED-REVIEWED			
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

DAVIS NO. 2
DAVIS STREAM
JEFFERSON   LINCOLN COUNTY

BORING LOGS

SHEET NUMBER
3
OF 3

Appendix A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM

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

Desired Soil Observations: (in this order)

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):

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

*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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Davis No.2 Bridge #3405 carries Goose Hill Road over Davis Stream
Location: Jefferson, Maine

Boring No.: BB-JDS-101
PIN: 17082.00

Driller:	MaineDOT	**Elevation (ft.)**	154.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles	**Datum:**	NAD 1983	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/11/10; 07:00-10:00	**Drilling Method:**	SSA and Cased Wash Boring	**Core Barrel:**	NQ-2
Boring Location:	11+55.3, 8.0 Rt.	**Casing ID/OD:**	NW	**Water Level*:**	1.6' bgs.
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 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							SSA	154.00		PAVEMENT. 0.50	
	1D	24/18	1.00 - 3.00	14/10/8/5	18	25				Brown, damp, medium dense, fine to coarse SAND, little gravel and silt, (Fill).	G#237541 A-1-b, SM WC=5.0%
5								149.50		5.00	
	2D	4.8/4.8	5.00 - 5.40	50(4.8")	---					Olive-brown, moist, hard, SILT, some fine to coarse sand and gravel, (Glaciomarine? Moraine?).	
10	3D	16.8/16.8	10.00 - 11.40	19/25/50(4.8")	---		63			Similar to above, except wet.	G#237542 A-4, ML WC=13.8%
	R1	60/53	11.90 - 16.90	RQD = 15%			a51	143.10		a51 blows for 0.4'. 11.40	
							NQ-2	142.60		Roller Coned ahead to 11.9' bgs. Weathered ROCK. 11.90	
										Bedrock: Grey, fine-grained, hard, METASANDSTONE, fresh to slightly weathered, joints along bedding planes typically 30-45 degrees from horizontal, very close to close, generally tightwith some open, minor silt in-filling and iron staining. Rock Mass Quality is Very Poor. [Bucksport Formation]	
15										R1:Core Times (min:sec) 11.9-12.9' (3:10) 12.9-13.9' (4:28) 13.9-14.9' (3:15) 14.9-15.9' (2:40) 15.9-16.9' (2:45) 88% Recovery	
								137.60		16.90	
										Bottom of Exploration at 16.90 feet below ground surface.	
20											
25											

Remarks:

300-500# of down pressure on Core Barrel.

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: Davis No.2 Bridge #3405 carries Goose Hill Road over Davis Stream
Location: Jefferson, Maine

Boring No.: BB-JDS-102
PIN: 17082.00

Driller:	MaineDOT	**Elevation (ft.)**	154.3	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles	**Datum:**	NAD 1983	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/11/10; 10:30-14:30	**Drilling Method:**	SSA and Cased Wash Boring	**Core Barrel:**	NQ-2
Boring Location:	11+68.2, 6.2 Lt.	**Casing ID/OD:**	NW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	153.80	PAVEMENT. —0.50— Brown, moist, loose, fine to coarse SAND, little gravel, little silt, (Fill) .	
	1D	24/15	1.00 - 3.00	5/2/3/5	5	7				
5	2D	24/12	5.00 - 7.00	2/5/11/17	16	22		149.30	—5.00— Brown, wet, medium dense, silty, fine to medium SAND, little gravel, (Glaciomarine? Moraine?).	
							104		Cobble from 7.0-7.8' bgs.	
							90		Roller Coned ahead to 9.7' bgs.	
10	R1	60/53	9.70 - 14.70	RQD = 60%			NQ-2	144.60	—9.70— Bedrock: Grey, fine-grained, hard, METASANDSTONE, fresh to slightly weathered, joints along bedding planes typically 30-45 degrees from horizontal, very close to close, generally tightwith some open, minor silt in-filling and iron staining. Rock Mass Quality is Fair. [Bucksport Formation] R1:Core Times (min:sec) 9.7-10.7' (2:05) 10.7-11.7' (2:30) 11.7-12.7' (2:40) 12.7-13.7' (2:30) 13.7-14.7' (3:00) 88% Recovery No water return.	
15								139.60	—14.70— **Bottom of Exploration at 14.70 feet below ground surface.**	
20										
25										

Remarks:

400-600# of down pressure on Core Barrel.

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: Davis No.2 Bridge #3405 carries Goose Hill Road over Davis Stream
Location: Jefferson, Maine

Boring No.: BB-JDS-103
PIN: 17082.00

Driller:	MaineDOT	**Elevation (ft.)**	153.1	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles	**Datum:**	NAD 1983	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/11/10; 13:30-15:00	**Drilling Method:**	SSA and Cased Wash Boring	**Core Barrel:**	NQ-2
Boring Location:	12+20.3, 10.3 Lt.	**Casing ID/OD:**	NW	**Water Level*:**	7.6' bgs.
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	152.70		PAVEMENT. 0.40	
	1D	24/17	1.00 - 3.00	11/15/10/11	25	35				Brown, moist, dense, fine to coarse SAND, some gravel, little silt, (Fill).	G#237543 A-1-b, SM WC=5.4%
								148.60		4.50	
5	2D	24/16	5.00 - 7.00	2/2/4/8	6	8				Brown, wet, loose, fine to coarse SAND, some gravel and silt, (Glaciomarine? Moraine?).	G#237544 A-2-4, SM WC=12.4%
10	3D	24/14	10.00 - 12.00	11/21/24/22	45	63				Similar to above, except very dense.	
							24				
	R1	60/55	12.50 - 17.50	RQD = 58%			a50 NQ-2	140.60		a50 blows for 0.5'. 12.50 Bedrock: Grey, fine-grained, hard, METASANDSTONE, fresh to slightly weathered, joints along bedding planes typically 30-45 degrees from horizontal, very close to close, generally tight with some open, minor silt in-filling and iron staining. Rock Mass Quality is Fair. [Bucksport Formation] R1:Core Times (min:sec) 12.5-13.5' (3:15) 13.5-14.5' (3:10) 14.5-15.5' (3:00) 15.5-16.5' (3:10) 16.5-17.5' (2:45) 92% Recovery	
15											
								135.60		17.50 **Bottom of Exploration at 17.50 feet below ground surface.**	
20											
25											

Remarks:

400-600# of down pressure on Core Barrel.

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: Davis No.2 Bridge #3405 carries Goose Hill Road over Davis Stream
Location: Jefferson, Maine

Boring No.: BB-JDS-104
PIN: 17082.00

Driller:	MaineDOT	**Elevation (ft.)**	154.3	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles	**Datum:**	NAD 1983	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/12/10; 07:30-10:00	**Drilling Method:**	SSA and Cased Wash Boring	**Core Barrel:**	NQ-2
Boring Location:	12+35.8, 1.4 Rt.	**Casing ID/OD:**	NW	**Water Level*:**	6.8' bgs.
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	153.90		PAVEMENT. 0.40	
	1D	24/17	1.00 - 3.00	9/10/7/7	17	24				Brown, moist, medium dense, fine to coarse SAND, some gravel, little silt, (Fill).	
								150.30		4.00	
5	2D	24/20	5.00 - 7.00	6/9/25/21	34	48				Brown, moist, dense, silty fine to medium SAND, little gravel, occasional cobbles, (Moraine).	
										Cobble from 8.5-9.0' bgs.	
10	3D	24/18	10.00 - 12.00	7/10/17/42	27	38	23			Similar to above, except wet.	
							65				
	R1	60/60	12.60 - 17.60	RQD = 16%			a100 NQ-2	141.70		a100 blows for 0.6'. 12.60 Bedrock: Grey, fine-grained, hard, METASANDSTONE, fresh to slightly weathered, joints along bedding planes typically 30-45 degrees from horizontal, very close to close, generally tightwith some open, minor silt in-filling and iron staining. Rock Mass Quality is Very Poor. [Bucksport Formation]	
15										R1:Core Times (min:sec) 12.6-13.6' (2:55) 13.6-14.6' (2:40) 14.6-15.6' (2:40)	
								136.70		15.6-16.6' (3:10) 16.6-17.6' (3:12) 100% Recovery 17.60	
										Bottom of Exploration at 17.60 feet below ground surface.	
20											
25											

Remarks:

400-600# of down pressure on Core Barrel.

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

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

Appendix B

Laboratory Test Data

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Jefferson   Project Number: 17082.00

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-JDS-101, 1D	11+55.3	8.0 Rt.	1.0-3.0	237541	1	5.0			SM	A-1-b	II
BB-JDS-101, 3D	11+55.3	8.0 Rt.	10.0-11.4	237542	1	13.8			ML	A-4	IV
BB-JDS-103, 1D	12+20.3	10.3 Lt.	1.0-3.0	237543	1	5.4			SM	A-1-b	II
BB-JDS-103, 2D	12+20.3	10.3 Lt.	5.0-7.0	237544	1	12.4			SM	A-2-4	II

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

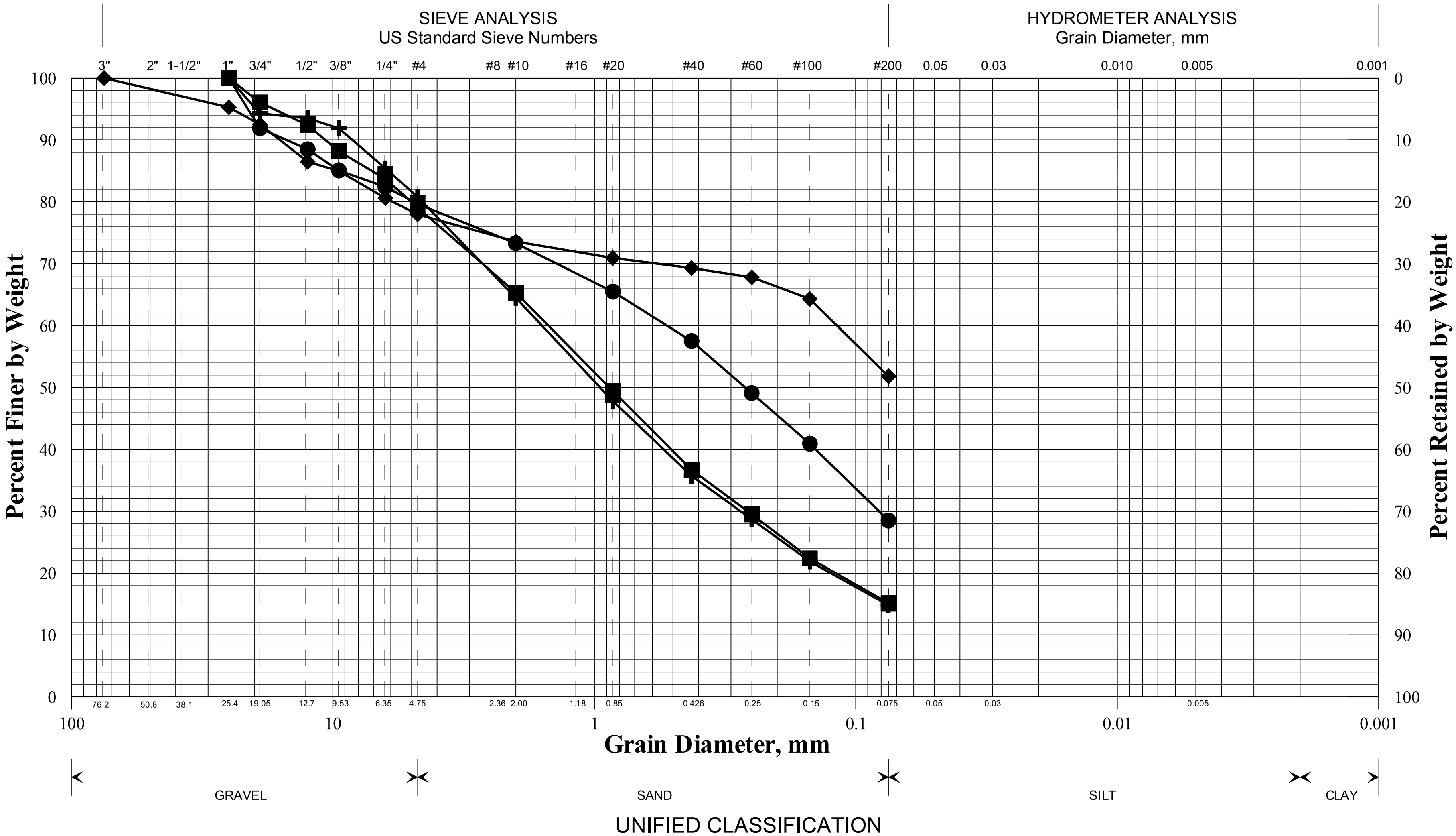
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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

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

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

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-JDS-101/1D	11+55.3	8.0 RT	1.0-3.0	SAND, little gravel, little silt.	5.0			
◆	BB-JDS-101/3D	11+55.3	8.0 RT	10.0-11.4	SILT, some sand, some gravel.	13.8			
■	BB-JDS-103/1D	12+20.3	10.3 LT	1.0-3.0	SAND, some gravel, little silt.	5.4			
●	BB-JDS-103/2D	12+20.3	10.3 LT	5.0-7.0	SAND, some silt, some gravel.	12.4			
▲									
✖									

PIN	
017082.00	
Town	
Jefferson	
Reported by/Date	
WHITE, TERRY A	9/29/2010

SHEET 1

Appendix C

Calculations

ABUTMENT AND WINGWALL PASSIVE AND ACTIVE EARTH PRESSURES:

Rankine Theory - Active Earth Pressure from MaineDOT Bridge Design Guide Section 3.6.5.2, pg. 3-7

Either Rankine or Coulomb may be used for long-heeled cantilever walls where the failure surface is uninterupted by the top of the wall stem. In general, use Rankine though.

Soil angle of internal friction: $\phi := 32\text{deg}$

Slope angle of backfill soil from horizontal: $\beta := 0\text{deg}$

$$K_a := \tan\left[45\text{deg} - \left(\frac{\phi}{2}\right)\right]^2$$

$K_a = 0.31$

Rankine Theory - Passive Earth Pressure from Bowles 5th Edition Section 11-5, pg 602

Soil angle of internal friction: $\phi := 32\text{deg}$

Slope angle of backfill soil from horizontal: $\beta := 0\text{deg}$

$$K_{p\_rank} := \frac{\cos(\beta) + \sqrt{\cos(\beta)^2 - \cos(\phi)^2}}{\cos(\beta) - \sqrt{\cos(\beta)^2 - \cos(\phi)^2}}$$

$K_{p\_rank} = 3.25$

Coulomb Theory - Active Earth Pressure from MaineDOT Bridge Design Guide
Section 3.6.5.2, pg. 3-7

For gravity walls , semi-gravity walls, prefabricated modular walls, and cantilever walls and abutments with short heels where wall and backfill interface friction is considered, use Coulomb Theory

Angle of back face of wall: $\alpha := 90deg$

Soil angle of internal friction: $\phi := 32deg$

Slope angle of backfill soil from horizontal: $\beta := 0deg$

$\delta =$
β $\delta := \beta$

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

$K_a = 0.31$

Coulomb Theory - Passive Earth Pressure from MaineDOT Bridge Design Guide
Section 3.6.6, pg. 3-8

Angle of back face of wall: $\alpha := 90deg$

Soil angle of internal friction: $\phi := 32deg$

Friction angle between fill and wall:
From LRFD Table 3.11.5.3-1, pg. 3-74, δ ranges from 17 to 22 $\delta := 20deg$

Angle of backfill from horizontal: $\beta := 0deg$

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

$K_p = 6.89$

Frost Protection:
Method 1

From the Maine Design Freezing Index Map:

DFI = 1400 degree-days

Site has Coarse Grained Soils With Wn > 10%

From the 2003 Bridge Design Guide Table 5-1:

$Frost\_depth := 66in$

$Frost\_depth = 66\cdot in$

$Frost\_depth = 5.5\cdot ft$

Method 2

```
------------------------
--- ModBerg Results ---
-----------------------

Project Location: Gardiner, Maine

Air Design Freezing Index         =  1489 F-days
N-Factor                          =  0.70
Surface Design Freezing Index     =  1042 F-days
Mean Annual Temperature           =  44.1 deg F
Design Length of Freezing Season  =  128 days

---------------------------------------------------------
Layer
#:Type            t     w%     d     Cf   Cu    Kf    Ku      L
---------------------------------------------------------
1-Asphalt        4.0    .1  140.0   28   28     .9    .9       0
2-Coarse        55.4  13.0  120.0   28   36    2.2   1.6   2,246
---------------------------------------------------------

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 = 4.95 ft = 59.4 in.
*********************************************************
```

Ok Use 5.0 feet

BEARING RESISTANCE - FOOTINGS ON BEDROCK:

SERVICE LIMIT STATE:

Method 1

Method: Based on LRFD Table C10.6.2.6.1-1 (Based on NAVFAC DM 7.2, May 1982) - "Presumptive Bearing Resistances for Spread Footing Foundations at the Service Limit State."

Description of Bedrock Materials:

Moderate to Highly Fractured Metasandstone, RQD as low as 15%

Bearing Material:	Weathered bedrock, RQD less than 25%
Consistency in Place:	Medium hard rock
Bearing Resistance:	Range 16 - 24 ksf
Recommeded Value	16 ksf

STRENGTH LIMIT STATE:

Method 2

Method: AASHTO Standard Specifications - 17th Edition, 2002

Section 4.4.8.1.2 - Footings on Broken or Jointed Rock, Pg. 62
Table 4.4.8.1.2A - for footings supported on Broken or Jointed Rock, Pg. 63

a. estimated Rock Mass Rating	Very Poor (Numerous joints < 2 inches apart)
b. Rock Category per 4.4.8.1.2B	B, Metasandstone
c. Unconfined compressive strength, Co	1000 psi
d. Nms, per Table 4.4.8.1.2A	Use q_{ult} of equivalent soil mass
e. $Q_{ult} = Q_{nom}$	q_{ult} of equivalent soil mass

Nominal Bearing Resistance for Spread Footings on Fractured Bedrock Using Equivalent Soil Mass:

Use Terzaghi Strip Footing Equation to Calculate Q_{nom}.

Assumptions:

1. Footings only embedded by riprap layer 3.0 feet.

$$D_f := 3.0\text{ft}$$

2. Assumed parameters for soils:
   Assume granular fill

Moist unit weight: $\gamma_m := 145pcf$

Saturated unit weight: $\gamma_{sat} := 150pcf$

Soil angle of internal friction: $\phi_{ns} := 36$ Assume similar to dense till

Undrained shear strength (cohesion): $c_{ns} := 0psf$

3. Use Terzaghi strip equations as L > B

Depth to Groundwater table based on boring data: $D_w := 0 \cdot ft$

Unit weight of water: $\gamma_w := 62.4pcf$

Effective Stress at the footing bearing level: $q_{eff\_str} := D_w \cdot \gamma_m + (D_f - D_w) \cdot (\gamma_{sat} - \gamma_w)$

$q_{eff\_str} = 0.26 \cdot ksf$

Look at several typical footing widths:

$$B := \begin{pmatrix} 12 \\ 14 \\ 16 \end{pmatrix} ft$$

Terzaghi Shape Factors from Bowles 5th Ed.,Table 4-1, p. 220, for strip footing:

$s_c := 1.0$

$s_\gamma := 1.0$

Meyerhof Bearing Capacity Factors For ϕ = 36 deg Bowles 5th Ed. Table 4-4 pg. 223

$N_c := 50.55$ $N_q := 37.7$ $N_\gamma := 44.4$

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

$$Q_{nom} := c_{ns} \cdot N_c \cdot s_c + q_{eff\_str} \cdot N_q + 0.5(\gamma_{sat} - \gamma_w) \cdot B \cdot N_\gamma \cdot s_\gamma$$

$$Q_{nom} = \begin{pmatrix} 33.2 \\ 37.1 \\ 41 \end{pmatrix} \cdot ksf$$

Resistance Factor from LRFD Table 10.5.5.2.2-1 pg. 10-39: $\phi_b := 0.45$

$q_{fac} := Q_{nom} \cdot \phi_b$

Factored Bearing Resistance

$$q_{fac} = \begin{pmatrix} 15 \\ 16.7 \\ 18.5 \end{pmatrix} \cdot ksf$$

Use a **Strength Limit State** Factored Bearing Resistance of **15 ksf.**

SEISMIC SITE CLASS:

Calculation

Determination of site class for Jefferson, Davis #2 Bridge substructures

Method

Use Shear wave velocity to determine site class per LRFD Table 3.10.3.1-1

Reference: Das, Fundamentals of Soil Dynamics, (1983) page 286.

Shear modulus for sands, sands and gravels, based on Seed and Idriss (1970), provided in Das (1983), Equation 8.48:

$G := 1000 \cdot K_2 \cdot \sqrt{\sigma_o}$ G and effective overburden stress in lb/ft^2

Estimate K2 from Das (1983) Figure 8.16 and 8.15

Use Curve from Figure 8.16 for "sand, gravel, cobbles with little clay"

$K_2 := 90$

Assumed Unit Weight Values for Soils

		Report as unitless (lb/sf)
Sand and Gravel Fill	$\rho_{dry1} := 110$	$\rho_{sat1} := 125$
Morraine/Till Silty Sand and Gravel	$\rho_{dry2} := 115$	$\rho_{sat2} := 135$

Determination of G based on Bowles Eq. 20.15

$$V_s := \sqrt{\frac{G}{\rho}}$$

Groundwater conditons: $\gamma_w := 62.4$ $D_w := 4 \cdot ft$

Average Soil Profile

Layer 1 - 4 feet of fill
Layer 2 - 4 feet of morraine
Layer 3 - 4 feet of morraine

Layer 1

Thickess Layer $H_1 := 4$ Remove units - report in ft

Effective overburden stress at midpoint of layer

$$\sigma'_{v1} := \frac{H_1}{2} \cdot \rho_{dry1} \qquad \sigma'_{v1} = 220$$

Spring constant $K := 90$ Unitless

Shear Modulus $G_1 := 1000 \cdot K \cdot \sqrt{\sigma'_{v1}}$ $G_1 = 1 \times 10^6$

Determination of Shear Velociy based on Bowles Eq. 20.14

$$V_{s\_1} := \sqrt{\frac{G_1}{\rho_{dry1}}} \qquad V_{s\_1} = 110.16$$ in ft/sec

Ratio of di / Vsi

$$\frac{H_1}{V_{s\_1}} = 0.04$$

Layer 2

Thickess Layer $H_2 := 4$ groundwater at top of layer

Effective overburden stress at midpoint of layer

$$\sigma'_{v2} := 4 \cdot \rho_{dry1} + 2 \cdot (\rho_{sat2} - 62.4)$$

$$\sigma'_{v2} = 585.2$$

Spring constant $K := 90$ Unitless

Shear Modulus $G_2 := 1000 \cdot K \cdot \sqrt{\sigma'_{v2}}$ $G_2 = 2177182$

Determination of Shear Velociy based on Bowles Eq. 20.14

$V_{s\_2} := \sqrt{\dfrac{G_2}{\rho_{dry2}}}$ $V_{s\_2} = 137.59$ in ft/sec

Ratio of di / Vsi

$$\frac{H_2}{V_{s\_2}} = 0.03$$

Layer 3

Thickess Layer $H_3 := 4$

Effective overburden stress at midpoint of layer

$$\sigma'_{v3} := 4 \cdot \rho_{dry1} + 6 \cdot (\rho_{sat2} - 62.4)$$

$$\sigma'_{v3} = 875.6$$

Spring constant $K := 90$ Unitless

Shear Modulus $G_3 := 1000 \cdot K \cdot \sqrt{\sigma'_{v3}}$ $G_3 = 2663149$

Determination of Shear Velociy based on Bowles Eq. 20.14

$V_{s\_3} := \sqrt{\dfrac{G_3}{\rho_{dry2}}}$ $V_{s\_3} = 152.18$ in ft/sec

Ratio of di / Vsi

$$\frac{H_3}{V_{s\_3}} = 0.03$$

Layer 4 - Bedrock - Interbedded Slate and Siltstone

$$H_4 := 100 - (H_1 + H_2 + H_3) \qquad H_4 = 88$$

Shear wave velocity

$$V_{s\_4} := 2000$$ ft/sec

$$\frac{H_4}{V_{s\_4}} = 0.04$$

Average Vs for the top 100 ft is determined per LRFD Table C3.10.3.1-1, Method A

$$v_s := \frac{100}{\frac{H_1}{V_{s\_1}} + \frac{H_2}{V_{s\_2}} + \frac{H_3}{V_{s\_3}} + \frac{H_4}{V_{s\_4}}} \qquad v_s = 737.1$$

Site Class D - 600 ft/s < vs < 1,200 ft/s

SEISMIC DESIGN PARAMETERS:

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years
State - Maine
Zip Code - 04348
Zip Code Latitude = 44.177800
Zip Code Longitude = -069.503200
Site Class B
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.069	PGA - Site Class B
0.2	0.147	Ss - Site Class B
1.0	0.043	S1 - Site Class B

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Spectral Response Accelerations SDs and SD1
State - Maine
Zip Code - 04348
Zip Code Latitude = 44.177800
Zip Code Longitude = -069.503200
As = FpgaPGA, SDs = FaSs, and SD1 = FvS1
Site Class D - Fpga = 1.60, Fa = 1.60, Fv = 2.40
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.111	As - Site Class D
0.2	0.235	SDs - Site Class D
1.0	0.103	SD1 - Site Class D