Maine Department of Transportation
Bridge Program
Geotechnical Section
Augusta, Maine

Geotechnical Design Report

For the Replacement of:

**Jacksonville Bridge
Over East Machias River
State Route 191
East Machias, Maine**

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Soils Report No. 2006-09

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GEOTECHNICAL DESIGN SUMMARY

The purpose of this report is to present the results of a subsurface investigation program and make geotechnical recommendations for the replacement of Jacksonville Bridge over the East Machias River on State Route 191 in East Machias, Maine. The proposed bridge alignment will be similar to the existing alignment. The following design recommendations are discussed in detail in the attached report:

Driven Integral Abutment H-piles - The use of stub integral abutments founded on a single row of driven H-piles is a viable foundation system for use at the site. Piles should be driven to bedrock. For the three pile sections evaluated, the allowable structural capacity of the piles exceeds the allowable geotechnical capacity and therefore the geotechnical capacity governs. The allowable capacity for three pile sections is shown below.

Pile Type	Maximum Allowable Axial Design Load (kips)
HP 12 x 53	149
HP 14 x 73	198
HP 14 x 89	231

The piles should be oriented for weak axis bending and driven to refusal on or within the bedrock the bedrock. Driven piles should be fitted with pile driving points to protect the tips and to improve penetration. Design axial loads should be shown on the plans.

Abutments spread footings founded on Bedrock - The use of full height cast-in-place cantilever abutments founded on spread footings is a viable foundation system for use at the site. In consideration of requirements for scour and bearing capacity, bedrock is recommended for supporting spread footing foundations. The borings encountered bedrock 13 to 17 ft below the streambed. It is therefore considered feasible that cofferdams, tremie seals and spread footings could be practically constructed to bear on bedrock, but will likely be more costly than a pile supported integral bridge abutment.

Bearing Capacity - The applied bearing pressure for any structure founded on bedrock should not exceed the calculated allowable bearing capacity of 30 ksf. The applied bearing pressure for any structure founded on the fill should not exceed the calculated allowable bearing capacity of 6 kips per square foot (ksf).

Scour and Riprap - For scour protection, any retaining wall footing constructed on native subgrade soils or on fill soils should be embedded for scour protection and armored with 3 feet of riprap per Section 2.3.11.3 of the Maine DOT Bridge Design Guide (BDG).

Stub Abutments – The integral stub abutments may be constructed behind the existing concrete abutments. The existing gravity abutments may be partially demolished and the remaining portion left in place as protection for the new integral abutments.

Settlement - Settlement of abutments founded directly on bedrock or piles driven to bedrock is anticipated to be negligible. The roadway will be raised approximately 2 ft at the center of the existing bridge resulting in a 1 to 2-ft raise in grade at the approaches. Due to the presence of dense alluvial and glacial till soils underlying the approaches, settlements in the widening area are anticipated to be less than ½-inch and will occur during construction having minimal effect on the finished structure. Any settlement of pile supported bridge abutments will be due to the elastic compression of the piling.

Cast-in-place Wingwalls - The bridge approaches may be retained using cast-in-place concrete abutment wingwalls. These walls shall be designed as unrestrained meaning that they are free to rotate at the top in an active state of earth pressure. All walls should be designed to achieve a minimum factor of safety of 2.0 against overturning and a minimum factor of safety of 1.5 against sliding. An active earth pressure coefficient, K_a, shall be calculated using Rankine Theory for cantilever wingwalls and Coulomb Theory for gravity shaped structures.

Prefabricated Concrete Modular Gravity Walls - The bridge approaches may be retained using Prefabricated Concrete Modular Gravity (PCMG) walls functioning as wingwalls. These walls are to be designed by a Professional Engineer subcontracted by the Contractor as a design-build item. The allowable bearing capacity for the PCMG walls founded on granular soils should not exceed 6 ksf. It is anticipated that the walls will be placed on granular soils and that any settlement of the walls will be immediate (occurring during construction) and negligible.

Frost Protection - It is anticipated that foundations for the abutments and wingwalls at the site will be founded either on piles or on granular fills. For integral abutment foundations placed directly on the granular fills, requirements for minimum depth of embedment for frost protection are necessary and shown in BDG Figures 5-2 and 5-3. In the event that a wingwall foundation is placed on the native granular subgrade or fill soils it should be founded a minimum of 6.0 ft below finished exterior grade for frost protection.

Seismic Design Considerations – The horizontal bedrock acceleration coefficient (A) for East Machias is less than 0.09g. Furthermore, the bridge is not on the National Highway System (NHS) nor is it classified as a major structure. As a result, the bridge substructures will not be designed for seismic earth loads.

Construction Considerations - There is a potential that cobbles may be encountered in the lower glacial till unit during pile driving operations. Obstructions may be cleared by conventional excavation methods, predrilling or spudding. Alternative methods to clear obstructions may be used as approved by the Resident.

1.0 INTRODUCTION

The purpose this Geotechnical Design Report is to present geotechnical recommendations for replacement of the Jacksonville Bridge over the East Machias River on State Route 191 in East Machias, Maine. This report presents the soils information obtained at the site during the subsurface investigation, foundation alternatives for bridge replacement, and geotechnical design recommendations.

2.0 PROJECT BACKGROUND

The existing bridge was built in the 1930's and is a single 98-ft span riveted pony truss. The superstructure is supported on abutments consisting of unreinforced concrete. The existing bridge plans indicate that the abutments are supported on timber piles. Both abutment backwalls are cracked and the breastwalls show extensive "map cracks". The age of the existing timber piles are approximately 70 years. Treated marine piles will generally last about 50 years in northern climates[1].

The proposed bridge will be a single span structure with an approximate span length of 110 ft. The superstructure curb-to-curb width will be increased slightly to 32.5 ft and will be centered on the existing alignment. The proposed abutments can be full height cast-in-place cantilever abutments founded on bedrock or H-pile supported integral stub abutments. Either cast-in-place concrete wingwalls or prefabricated concrete modular gravity (PCMG) wingwalls can be used.

3.0 GEOLOGICAL SETTING

The Jacksonville Bridge on State Route 191 in East Machias, Maine crosses the East Machias River as shown on *Sheet 1 - Location Map* presented at the end of this Report. The East Machias River flows in an easterly direction at the bridge site.

According to the Maine Geological Survey map "Surficial Geology of Gardiner Lake Quadrangle, Maine", Open-file No. 82-4 (1982), the surficial soils in the vicinity of the Jacksonville Bridge consist of glaciomarine deposits, which accumulated on the ocean floor during the late-glacial marine submergence of lowland areas in southern and eastern Maine. These soils are generally comprised of silt, clay, sand and minor amounts of gravel. The most common component is the clayey silt known as the Presumpscot Formation, but sand is very abundant in some areas. The unit also may contain small areas of till, sand and gravel that are not completely covered by the marine sediment.

According to the Bedrock Geologic Map of Maine, Maine Geological Survey, 1985, the bedrock in the vicinity of the site consists of plutonic igneous rock, mostly granite, with smaller areas of quartz diorite, gabbro, diorite and ultramaphic rocks.

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[1] Hannigan, P.J. et al, Design and Construction of Driven Pile Foundations, FHWA-HI-97-013, December 1996.

4.0 SUBSURFACE INVESTIGATION

Subsurface conditions at the site were explored by drilling two (2) cased wash borings BB-EMR-101 and BB-EMR-102. The test borings were drilled behind the existing south and north abutments and were terminated with bedrock cores.

The boring locations are shown on *Sheet 2 - Boring Location Plan and Interpretive Subsurface Profile* found at the end of this Report. The test borings were drilled by the Maine Department of Transportation (MaineDOT) Drill Crew, on October 6 and October 7, 2004. Borehole logging was completed by the MaineDOT inspector. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A – Boring Logs and on *Sheet 3 – Boring Logs* found at the end of this Report.

The borings were drilled using cased wash boring techniques. Soil samples were obtained at 5-ft intervals using Standard Penetration Test (SPT) methods. Bedrock was cored in borings using NQ core barrel and the Rock Quality Designation (RQD) of the core sample was calculated. The Maine DOT Geotechnical Team Member selected the boring locations and drilling methods, designated type and depth of sampling techniques, identified field and lab testing requirements and reviewed for accuracy the subsurface conditions encountered as reported on the field logs. The borings were located in the field by use of a tape after completion of the drilling program.

5.0 LABORATORY TESTING

Laboratory testing consisted of four (4) standard grain size analyses with natural water content. The results of these laboratory tests are provided in Appendix B – Laboratory Data at the end of this Report. Moisture content information and other soil test results are also included on the Boring Logs in Appendix A and on *Sheet 3 - Boring Logs*, found at the end of this Report.

6.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered in the explorations generally consisted of **granular fill** overlying **stream alluvium** and **glacial till** all of which is underlain by **bedrock**. An interpretive subsurface profile depicting the 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 summary description of each strata encountered is as follows:

6.1 Fill

A layer of granular fill was encountered behind the existing abutments in borings BB-EMR-101 and BB-EMR-102. The fill layer is approximately 15 to 15.2 ft thick. The fill soils behind the south abutment consist of brown, dry to moist, sandy GRAVEL, with trace of silt and cobbles. SPT N-values in the fill unit ranged from 8 to 24 blows per foot (bpf) indicating that the layer is loose to medium dense in consistency. Fill soils behind the north abutment consist of brown, dry to moist, fine to coarse SAND, some gravel, trace silt and cobbles. SPT N-values in the fill layer ranged from 11 to 27 bpf indicating that the layer is medium dense in consistency.

One sample of the fill unit from BB-EMR-102 was tested and is classified as Unified SW-SM (AASHTO A-1-a) and described as fine to coarse sand, some gravel, trace silt. The measured water content was 5.0 percent.

6.2 Alluvium

Stream alluvium was encountered below the fill unit in the borings. The thickness of the unit is 2.5 to 2.8 ft thick. The layer is described as brown, moist SAND, some silt, little clay, trace small rounded gravel, with iron staining. SPT N-values taken in the layer ranged from 11 to 21 bpf indicating that the stream alluvium is medium dense.

One sample of the alluvium unit was tested and is classified as Unified SC-SM (AASHTO A-4) and described as fine to coarse SAND, little clay, with trace of gravel. The measured water content was 13.5 percent.

6.3 Glacial Till

Underlying the fill and alluvium, a deposit of fine-grained glacial till was encountered. The glacial till unit encountered is described as olive-grey and olive-brown, moist, fine to coarse SAND, some silt, little gravel, trace to no clay and dark grey, moist, silty fine to medium SAND, little coarse sand, trace gravel and cobbles. Cobbles increased with depth in the glacial till unit in boring BB-EMR-102.

Based on the data gathered, the glacial till unit is 9.6 to 13.6 ft thick. SPT N-values taken in the layer range from 54 to 90 bpf indicating that the glacial till unit is dense to very dense in consistency.

Two soil samples from the till unit from BB-EMR-101 were tested and classified as Unified SW-SM (AASHTO A-2-4 and A-4). Measured water contents for samples of the till unit ranged from 9.5 to 9.7 percent.

6.4 Bedrock

The bedrock at the site was encountered and cored at a depth of 27.1 ft bgs (elevation 17.90 ft) in boring BB-EMR-101 and a depth of 31.6 ft bgs (elevation 13.4 ft) in boring BB-EMR-102. The bedrock at the site is identified as white, medium grained, very hard, fresh MONZONITE (granite) and dark grey, fine grained, very hard, fresh DIORTE. The rock quality designation (RQD) of the bedrock was determined to range from 85 to 100 percent, indicating a rock of good to excellent quality.

6.5 Groundwater

Water depths were measured to range between 9 to 10 ft bgs in the borings, however, subsurface water levels will vary with season, precipitation, infiltration, and construction activity.

7.0 Foundation Alternatives

Spread footings founded on glacial till. In consideration of requirements for scour and bearing capacity, the use of spread footings founded on bedrock is a viable abutment foundation alternative for this site. The borings encountered bedrock 13 to 17 ft below the streambed. Therefore, it is considered feasible that cofferdams, tremie seals and spread footings could be practically constructed to bear on bedrock, but this foundation alternative may not be as economical as a pile supported abutment.

Integral abutments founded on driven H-piles. The use of integral abutments founded on driven H-piles is a viable foundation type for use at the site. The piles should be end bearing, driven to refusal on or within the bedrock. In consideration of the potential difficulty and cost of constructing of a spread footing to meet embedment requirements, a pile supported integral abutment bridge (IAB) is likely the more practicable and economical foundation alternative.

Full height abutments founded on driven pile groups. A traditional streambed elevation pile cap founded on a pile group is not considered practical as the estimated pile lengths would be only 8.5 to 12.5 ft long, assuming a pile cap embedment of 5 ft below the streambed to meet the potential requirements for scour. We do not believe that 8.5 to 12.5-ft long piles are practicable.

8.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

The following subsections will discuss foundation considerations and recommendations for both full height cast-in-place cantilever abutments founded on bedrock and an H-pile supported integral abutment bridge.

8.1 Abutments Founded on Spread Footings

If economically viable, full height cast-in-place cantilever abutments shall be constructed on competent sound bedrock. Footings shall be designed in accordance with AASHTO Standard Specifications Section 4.4.8. Bridge abutments shall be designed as unrestrained meaning that they are free to rotate at the top in an active state of earth pressure. The Rankine active earth pressure coefficient of $K_a = 0.31$ is recommended. The Designer may assume Soil Type 4 (BDG Section 3.6.1) for abutment backfill material soil properties. The backfill properties are as follows: $\phi = 32°$, $\gamma = 125$ pcf. Sliding computations for resistance to lateral loads shall assume a maximum allowable frictional coefficient of 0.70 at the bedrock-concrete interface.

All abutments should be designed to achieve a minimum factor of safety of 2.0 against overturning and a minimum factor of safety of 1.5 against sliding. All abutment designs shall include a drainage system behind the abutments to intercept any groundwater. Drainage behind structure shall be in accordance with BDG Section 5.4.1.4.

8.2 Bearing Capacity

The applied bearing pressure for abutment footings founded on bedrock should not exceed the calculated allowable bearing capacity of 30 ksf. See Appendix C - Calculations for supporting documentation. In no instance shall the maximum allowable contact stress exceed the allowable bearing stress of the concrete, per the AASHTO Standard Specification for Highway Bridges, 17^{th} Edition, 2002, Section 4.4.8.1.1.

The applied bearing pressure for any retaining wall structure founded on compacted granular borrow or the native sand layer should not exceed the calculated allowable bearing capacity of 6 ksf. See Appendix C- Calculations for supporting documentation. No footing shall be less than 2 ft wide regardless of the applied bearing pressure or bearing material. Any organic material encountered shall be removed to the full depth and replaced with compacted granular fill.

8.3 Driven H-Pile Integral Abutments

The use of integral abutments founded on a single row of driven H-piles is a viable foundation system at the site and is considered the most practicable. The piles should be end bearing and driven to refusal on bedrock. The following piles were considered for use at the site: HP 12 x 53, HP 14 x 73 or HP 14 x 89. Driven piles should be Grade A572 steel H-piles. Pile lengths at the abutments can be estimated based on the following data:

Location	Estimate abutment/pile Cap bottom elevation (ft)	Top of Rock Elevation (ft)	Estimated Pile Length (ft)
Abutment #1 BB-EMR-101	38	17.9	20.1 feet
Abutment #2 BB-EMR-102	38	13.4	24.6 feet

Table 1. Estimated Pile Lengths

For integral abutment piles the MaineDOT Bridge Design Guide (BDG) recommends a factor of safety of 4.0 for computing the allowable structural capacity. Assuming that 50 ksi steel will be used, the allowable structural capacity of the piles exceeds the allowable geotechnical capacity, and therefore the geotechnical capacity governs. Calculations can be found in Appendix C at the end of this report. Design axial loads should be shown on the plans. The computed allowable geotechnical and structural capacities of the H-piles are summarized in the following table:

Pile Type	Allowable pile static capacity FS = 2.25 (kips)	Allowable pile structural capacity 50 ksi steel, FS = 4 (kips)
HP 12 x 53	149 [a]	194
HP 14 x 73	198 [a]	268
HP 14 x 89	231 [a]	326
[a] governs pile axial working load		

Table 2. Allowable pile axial capacities

The first pile driven should be dynamically tested to confirm capacity and verify the stopping criteria developed by the Contractor. With this level of quality control, the piles shall be driven to an ultimate capacity of 2.25 times the design load. No downdrag should be considered. The piles should be driven to refusal on or within the bedrock. The piles should be fitted with pile driving points to protect the tips and to improve penetration.

Piles should be driven to an acceptable penetration resistance as determined by the Contractor based on the results of a wave equation analysis and as approved by the Resident. Contract documents should require that the contractor perform a wave equation analysis of the proposed pile driving system, and the piles be driven to 2.25 times the design (working) load. This factor of safety assumes that a field dynamic testing will be performed. A hammer should be selected which provides the required geotechnical capacity when the penetration resistance for the final 3 to 6 inches is 8 to 13 blows per 1 inch. If an abrupt increase in driving resistance is encountered, the driving could be terminated when the penetration is less than 0.5-inch in 10 consecutive blows. Allowable pile stresses during driving shall be less than $0.90F_y$, per AASHTO Section 4.5.11.

The soils encountered at the site will provide sufficient lateral support to assume the H-piles are fully braced against Euler buckling. The Designer should check that pile axial stresses from the dead loads, live loads, pile dead load and secondary thermal forces do not exceed the allowable axial pile loads shown in the Table 2. The Designer should also check the live load rotation demand in accordance with BDG Design Procedure 5-4.

8.4 Stub Integral Abutments

It is anticipated that integral abutments might be constructed behind the existing bridge abutments. It is practicable to partially demolish the existing abutments, allowing the remaining lower portion to remain in place as protection for the new pile-supported abutments.

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 and a soil-concrete friction coefficient of 0.45. Cast-in-place integral abutment sections shall be designed to withstand a maximum applied lateral load equal to the passive earth pressure. The Coulomb passive case is recommended, $K_p = 7.3$. If an approach slab is not specified, additional lateral earth pressure due to traffic surcharge is required and shall be approximated by an additional 2 feet of earth fill. This results in a traffic surcharge of 250 psf. Use of an approach slab may be required per BDG Sections 5.4.2.10 and 5.4.4.

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

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

8.5 Settlement

Settlement of integral abutments founded on driven piles or abutment spread footings founded directly on bedrock is anticipated to be negligible. Any settlement of the pile supported bridge abutments will be due to the elastic compression of the piling. The roadway and sideslopes will be raised in the construction of the proposed bridge. Due to the presence of medium dense to dense cohesionless soils, settlements in this area are anticipated to be less than ½ inch and will occur during construction having minimal effect on the finished structure.

8.6 Wingwalls

Cast-in-place concrete abutment wingwalls shall be designed as unrestrained meaning that they are free to rotate at the top in an active state of earth pressure. All walls should be designed to achieve minimum factors of safety of 2.0 against overturning and 1.5 against sliding. An active earth pressure coefficient, K_a, shall be calculated using Rankine Theory for cantilever wingwalls and Coulomb Theory for gravity shaped structures. See *Sheet 4 - Rankine and Coulomb Active Earth Pressure Coefficients* at the end of this report for guidance in calculating these values. Additional lateral earth pressure due to construction surcharge or traffic surcharge is required per Section 3.6.8 of the BDG for the wingwalls if an approach slab is not specified.

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. Sliding computations for resistance to lateral loads shall assume a maximum allowable frictional coefficient of 0.45 at the soil-concrete interface.

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

8.7 Prefabricated Concrete Modular Gravity Walls

The bridge approaches may be retained using Prefabricated Concrete Modular Gravity (PCMG) walls functioning as return wings. These walls are to be designed by a Professional Engineer subcontracted by the Contractor as a design-build item.

The allowable bearing capacity for the PCMG walls founded on granular soils should not exceed 6 ksf. It is anticipated that the walls will be placed on granular soils and that any settlement of the walls will be immediate (occurring during construction) and negligible.

8.8 Scour and Riprap

For scour protection, slopes in front of pile-supported stub abutments should be armored with 3.0 ft of riprap per BDG 5.4.2.6.

Stone riprap shall be placed at the toe of PCMG walls or other retaining wall spread footings which are constructed on native soil deposits or fills below the Q50 elevation. Footings should be embedded a minimum of 2 ft below the design flood scour level for scour protection and also armored with 3 ft of riprap. Refer to BDG Section 2.3.11 for information regarding scour design.

Stone riprap shall conform to Item Number 703.26 of the MaineDOT Standard Specifications. The toe of the riprap section shall be constructed 1-ft below the streambed elevation. The riprap shall be underlain by a 1-ft thick layer of bedding material conforming to Item Number 703.19 of the MaineDOT Standard Specifications. For retaining wall applications, the riprap shall extend 1.5 ft horizontally in front of the wall before sloping at 2:1 to the existing ground surface

8.9 Frost Protection

It is anticipated that foundations for integral abutments and wingwall footings at the site will be founded on compacted fill. Foundations placed directly on the fill or alluvium should be designed with an appropriate embedment for frost protection. According to the MaineDOT design freezing index maps for the State of Maine, the site has a design-freezing index of approximately 1150 F-degree days. An assumed water content of 10 percent was used for coarse grained soils. These components correlate to a frost depth of 6.0 ft. Therefore, any foundations placed on soil should be founded a minimum of 6.0 ft below finished exterior grade for frost protection.

8.10 Seismic Design Considerations

The horizontal bedrock acceleration coefficient (A) for East Machias is approximately 0.04g, based on Figure 3-4 of the BDG, Seismic Performance Categories for Maine, August 2003. Per Section 3.5 of Division 1-A of the AASHTO Standard Specifications for Highway Bridges, Soil Profile Type I is applicable to the site and a site coefficient (S) of 1.0 would be used.

The bridge is not located on the National Highway System (NHS), and is not classified as functionally important bridge. Moreover, per BDG Section 3.7.1.1, bridges located in areas where the horizontal acceleration coefficient is less than or equal to 0.09 are designated a Seismic Performance Category (SPC) classification of A. As a result, design should be in conformance with Section 5 of Division 1-A of the AASHTO Standard Specifications for Highway Bridges, 17th Edition, 2002. For SPC A, no detailed seismic analysis is required other than connection design and bearing seat length.

8.11 Construction Considerations

It is anticipated that obstructions (cobbles) may be encountered during pile driving operations in the fill and glacial till unit. Obstructions may be cleared using the following methods:

- Excavation – Obstructions may be cleared using conventional excavation methods as approved by the Resident.

- Predrilling - The pile locations may be predrilled with a solid stem auger to clear obstructions. The diameter of the auger shall not exceed the diagonal dimension of the pile minus 4 inches.

- Spudding - Spudding methods may be used to clear obstructions. The spud shall consist of an H-pile section of the same size or smaller than the production piles. Other spuds may be accepted as approved by the Resident. Spuds shall be driven in the production pile locations to the elevation approved by the Resident. Once driven, the spud shall be removed. In the event that the spud cannot be removed, the Contractor shall bear the cost for all additional piles and associated design changes.

Alternative methods to clear obstructions may be used as approved by the Resident. The cost for clearing obstructions shall be considered incidental to related contract pay items.

9.0 CLOSURE

This report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed replacement of Jacksonville Bridge in the town of East Machias, 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.

Appendix A

Boring Logs

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Jacksonville Bridge over East Machias River on Route 191
Location: East Machias, Maine

Boring No.: BB-EMR-101
PIN: 11036.00

Driller:	MaineDOT	**Elevation (ft.)**	45.0	**Auger ID/OD:**	5" SSA
Operator:	C. Mann	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	G. Lidstone	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/7/04-10/7/04	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ 1.88 in ID
Boring Location:	14+39.6, 5.9 Lt.	**Casing ID/OD:**	HW	**Water Level*:**	9.0' bgs.

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods

Definitions:
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA	44.40		PAVEMENT. 0.60	
									Brown, dry, sandy GRAVEL, trace of cobbles and silt, (Fill).	
							41.00		4.00	
									Brown, dry to damp, medium dense, sandy GRAVEL, trace silt, (Fill).	
5	1D	24/9	5.00 - 7.00	9/12/12/13	24	38				
						35				
						30				
						31				
						22	36.00		9.00	
									Brown, wet, loose, sandy GRAVEL, trace silt, (Fill).	
10	2D	24/1	10.00 - 12.00	4/3/5/4	8	15				
						13				
						12				
						13				
						11	30.00		15.00	
15	3D	24/6	15.00 - 17.00	7/5/6/4	11	15			Brown, moist, medium dense, silty fine SAND, trace small rounded gravel, some iron staining, changing to same but dark grey. (Alluvium)	
						18				
						85	27.50		17.50	
						117				
						120				
20	4D	24/11	20.00 - 22.00	32/27/29/28	56	68			Olive-grey, moist, very dense, fine to coarse SAND, some silt, little small round gravel, trace of clay (Till).	G#182502 A-2-4, SC-SM WC=9.5%
						135				
						172				
						288				
25						OPEN			Washed ahead from 24.0-25.0' bgs.	

Remarks:

Borings BB-EMR-101 and BB-EMR-102 marked on pavement with white paint for possible location by Survey.

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: Jacksonville Bridge over East Machias River on Route 191 Location: East Machias, Maine	Boring No.: BB-EMR-101 PIN: 11036.00

Driller:	MaineDOT	Elevation (ft.)	45.0	Auger ID/OD:	5" SSA
Operator:	C. Mann	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	G. Lidstone	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	10/7/04-10/7/04	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ 1.88 in ID
Boring Location:	14+39.6, 5.9 Lt.	Casing ID/OD:	HW	Water Level*:	9.0' bgs.

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods

Definitions:
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	5D	18/11	25.00 - 26.50	31/62/55	117	HOLE			Similar to 4D, except Olive-brown. Washed ahead from 25.0-27.5' bgs.	G#182504 A-4, SC-SM WC=9.7
							17.90		27.10	
	R1	56/54	27.50 - 32.17	RQD = 85%		aRC NQ	17.50		aRoller coned into bedrock from 27.1 to 27.5' bgs. 27.50 R1: Bedrock: White, medium grained GRANITE (monzonite), very hard, fresh, no joint set evident. Topmost 4 to 6 in fractured and surfaces oxidized. Good rock quality. Core Times (min:sec) 27.5-28.5' (5:09) 28.5-29.5' (5:48) 29.5-30.5' (4:29) 30.5-31.5' (5:02) 31.5-32.17' (3:54) 96% Recovery	
30										
							12.83		32.17	
									Bottom of Exploration at 32.17 feet below ground surface.	
35										
40										
45										
50										

Remarks:

Borings BB-EMR-101 and BB-EMR-102 marked on pavement with white paint for possible location by Survey.

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: Jacksonville Bridge over East Machias River on Route 191
Location: East Machias, Maine

Boring No.: BB-EMR-102
PIN: 11036.00

Driller:	MaineDOT	**Elevation (ft.)**	45.0	**Auger ID/OD:**	5" SSA
Operator:	C. Mann	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	G. Lidstone	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/6/04-10/6/04	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-1.88" I.D.
Boring Location:	15+58.3, 7.1 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	10.0' bgs.

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods

Definitions:
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA	44.35		PAVEMENT. —0.65— Brown, dry, fine to coarse SAND, some gravel, trace silt, trace of cobbles (Fill).	
5	1D	24/9	5.00 - 7.00	9/14/13/12	27	42	40.00		—5.00— Brown, dry to damp, medium dense, fine to coarse SAND, some gravel, trace silt, (Fill).	G#182501 A-1-a, SW-SM WC=5.0%
						19				
						21				
						18				
						16				
10	2D	24/2	10.00 - 12.00	7/6/5/5	11	13	35.00		—10.00— Similar to above, but moist, (Fill).	
						23				
						25				
						28				
						33				
15	3D	24/11	15.00 - 17.00	12/10/11/13	21	21	29.80		—15.20— Brown, moist, medium dense SAND, some silt, little clay, trace of small rounded gravel, some iron staining.	G#182503 A-4, SC-SM WC=13.5%
						75				
						86	27.00		—18.00—	
						143				
						148				
20	4D	24/13	20.00 - 22.00	21/33/33/32	66	25			Olive-grey, moist, very dense, silty fine to medium SAND, trace coarse sand, trace small rounded gravel, (Till) Washed ahead from 20.0-25.0' bgs.	
						20				
						19				
						16				
25						50				

Remarks:

Borings BB-EMR-101 and BB-EMR-102 marked on pavement with white paint for possible location by Survey.

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

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

Boring No.: BB-EMR-102

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Jacksonville Bridge over East Machias River on Route 191
Location: East Machias, Maine

Boring No.: BB-EMR-102
PIN: 11036.00

Driller:	MaineDOT	**Elevation (ft.)**	45.0	**Auger ID/OD:**	5" SSA
Operator:	C. Mann	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	G. Lidstone	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/6/04-10/6/04	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-1.88" I.D.
Boring Location:	15+58.3, 7.1 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	10.0' bgs.

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods

Definitions:
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-value	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	5D	24/4	25.00 - 27.00	24/23/17/14	40	76	20.00	25.00 Dark grey, moist, dense, silty fine to medium SAND, little coarse sand, trace of rounded gravel; one 1 1/2 in dia fractured rock fragment (Till).	
						78			
						62			
						117	16.50	28.50 Similar to above, with cobbles, (Till).	
						310			
30	MD	1/0	30.00 - 30.08	53(1")	---	178		Washed ahead from 30.0-31.6' bgs.	
	R1	60/60	31.60 - 36.60	RQD = 100%		a100 NQ	13.40	a100 blows for 0.6'. 31.60 R1: Bedrock: Dark grey, fine grained DIORITE, very hard, fresh, no jointing. Excellent rock quality. R1:Core Times (min:sec) 31.6-32.6' (6:48) 32.6-33.6' (4:45) 33.6-34.6' (5:09) 34.6-35.6' (4:32) 35.6-36.6' (4:47) 100% Recovery	
35									
	R2	60/58	36.60 - 41.60	RQD = 95%			8.40	36.60 R2: Same as R1, only very slighty weathered, 2 joint sets evident, chlorite on joint surfaces. Excellent rock quality. Core Times (min:sec) 36.6-37.6' (3:33) 37.6-38.6' (3:20) 38.6-39.6' (3:29) 39.6-40.6' (3:16) 40.6-41.6' (3:14) 98% Recovery	
40									
							3.40	41.60 **Bottom of Exploration at 41.60 feet below ground surface.**	
45									
50									

Remarks:

Borings BB-EMR-101 and BB-EMR-102 marked on pavement with white paint for possible location by Survey.

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

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

Boring No.: BB-EMR-102

Appendix B

Laboratory Data

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): East Machias   Project Number: 11036.00

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-EMR-101, 4D	14+39.6	5.9 Lt.	20.0-22.0	182502	1	9.5			SC-SM	A-2-4	III
BB-EMR-101, 5D	14+39.6	5.9 Lt.	25.0-26.5	182504	1	9.7			SC-SM	A-4	III
BB-EMR-102, 1D	15+58.3	7.1 Rt.	5.0-7.0	182501	1	5.0			SW-SM	A-1-a	0
BB-EMR-102, 3D	15+58.3	7.1 Rt.	15.0-17.0	182503	1	13.5			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)

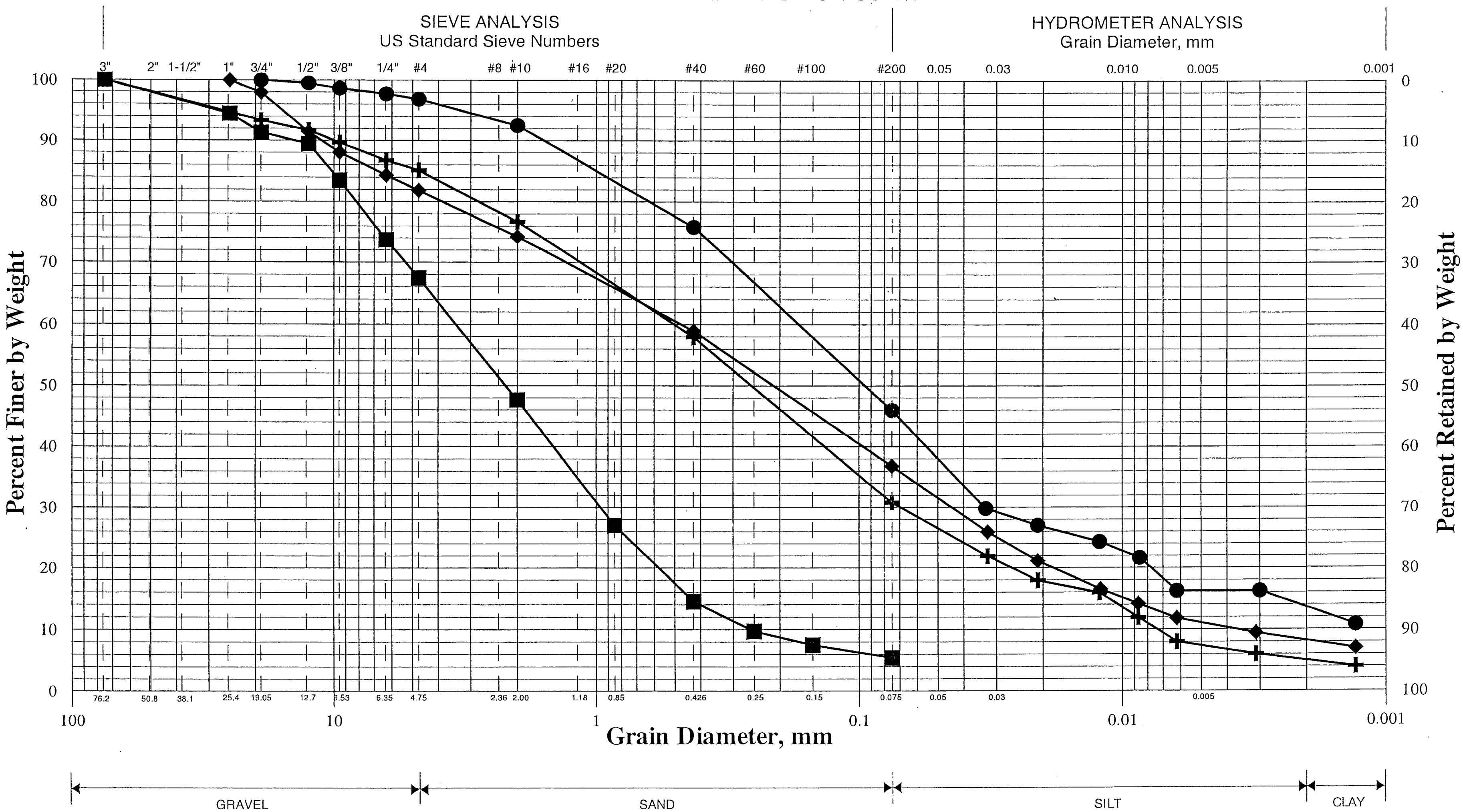
WC = water content as determined by AASHTO T 265-93 and/or ASTM D 2216-98

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



UNIFIED CLASSIFICATION

	Boring No.	Sample No.	Depth (ft)	Description	w%	LL	PL	PI
+	BB-EMR-101	4D	20.0-22.0	SAND, some silt, little gravel, trace clay.	9.5			
◆	BB-EMR-101	5D	25.0-26.5	SAND, some silt, little gravel, trace clay.	9.7			
■	BB-EMR-102	1D	5.0-7.0	SAND, some gravel, trace silt.	5.0			
●	BB-EMR-102	3D	15.0-17.0	SAND, some silt, little clay, trace gravel.	13.5			
▲								
×								

PIN: 11036.00
Town: East Machias
Reported by: T. White
Date: 11/01/04

SHEET NO.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
182502	BB-EMR-101/4D	GEOTECHNICAL (DISTURBED)	10/7/2004	10/15/2004

Sample Type: **GEOLOGY** Location: **OTHER** Station: **14+39.6** Offset, ft: **5.9 LT** Dbfg, ft: **20.0-22.0**

PIN: **011036.00** Town: **East Machias** Sampler: **MANN, CLYDE S**

TEST RESULTS

Sieve Analysis (T-88)

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	100.0
1 in. [25.0 mm]	93.3
¾ in. [19.0 mm]	93.3
½ in. [12.5 mm]	91.7
⅜ in. [9.5 mm]	89.6
¼ in. [6.3 mm]	86.7
No. 4 [4.75 mm]	85.1
No. 10 [2.00 mm]	76.7
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	57.9
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	30.8
[0.0324 mm]	21.9
[0.0209 mm]	17.9
[0.0122 mm]	15.9
[0.0087 mm]	11.9
[0.0062 mm]	8.0
[0.0031 mm]	6.0
[0.0013 mm]	4.0

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)

Plastic Limit (T 90)

Plasticity Index (T 90)

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

2.81

Loss on Ignition (T 267)

Loss, % H2O, %

Water Content (T 265), %

9.5

Vane Shear Test on Shelby Tubes (Maine DOT)

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

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **10/28/2004**

Paper Copy: Lab File; Project File; Geotech File

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
182504	BB-EMR-101/5D	GEOTECHNICAL (DISTURBED)	10/7/2004	10/15/2004

Sample Type: **GEOLOGY** Location: **OTHER** Station: **14+39.6** Offset, ft: **5.9 LT** Dbfg, ft: **25.0-26.5**

PIN: **011036.00** Town: **East Machias** Sampler: **MANN, CLYDE S**

TEST RESULTS

Sieve Analysis (T-88)

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	100.0
¾ in. [19.0 mm]	97.9
½ in. [12.5 mm]	91.4
⅜ in. [9.5 mm]	88.0
¼ in. [6.3 mm]	84.3
No. 4 [4.75 mm]	81.8
No. 10 [2.00 mm]	74.2
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	58.9
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	36.8
[0.0324 mm]	25.9
[0.0209 mm]	21.1
[0.0121 mm]	16.5
[0.0087 mm]	14.1
[0.0062 mm]	11.8
[0.0031 mm]	9.4
[0.0013 mm]	7.0

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)

Plastic Limit (T 90)

Plasticity Index (T 90)

Specific Gravity, Corrected to 20°C (T 100)
2.81

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

Water Content (T 265), %
9.7

Vane Shear Test on Shelby Tubes (Maine DOT)

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

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **10/28/2004**

Paper Copy: Lab File; Project File; Geotech File

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
182501	BB-EMR-102/1D	GEOTECHNICAL (DISTURBED)	10/6/2004	10/15/2004

Sample Type: **GEOLOGY** Location: **OTHER** Station: **15+58.3** Offset, ft: **7.1** **RT** Dbfg, ft: **5.0-7.0**

PIN: **011036.00** Town: **East Machias** Sampler: **MANN, CLYDE S**

TEST RESULTS

Sieve Analysis (T 27, T 11)

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	100.0
1 in. [25.0 mm]	94.4
¾ in. [19.0 mm]	91.3
½ in. [12.5 mm]	89.4
⅜ in. [9.5 mm]	83.5
¼ in. [6.3 mm]	73.7
No. 4 [4.75 mm]	67.4
No. 10 [2.00 mm]	47.6
No. 20 [0.850 mm]	27.0
No. 40 [0.425 mm]	14.5
No. 60 [0.250 mm]	9.7
No. 100 [0.150 mm]	7.5
No. 200 [0.075 mm]	5.4

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), %
5.0

Vane Shear Test on Shelby Tubes (Maine DOT)

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

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **10/25/2004**

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
182503	BB-EMR-102/3D	GEOTECHNICAL (DISTURBED)	10/6/2004	10/15/2004

Sample Type: **GEOLOGY** Location: **OTHER** Station: **15+58.3** Offset, ft: **7.1** **RT** Dbfg, ft: **15.0-17.0**

PIN: **011036.00** Town: **East Machias** Sampler: **MANN, CLYDE S**

TEST RESULTS

Sieve Analysis (T-88)

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	100.0
½ in. [12.5 mm]	99.5
⅜ in. [9.5 mm]	98.6
¼ in. [6.3 mm]	97.7
No. 4 [4.75 mm]	96.8
No. 10 [2.00 mm]	92.5
No. 20 [0.850 mm]	
No[illegible] [0.425 mm]	75.8
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	45.9
[0.0329 mm]	29.8
[0.0209 mm]	27.0
[0.0122 mm]	24.3
[0.0086 mm]	21.6
[0.0062 mm]	16.2
[0.0030 mm]	16.2
[0.0013 mm]	10.8

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)

Plastic Limit (T 90)

Plasticity Index (T 90)

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

2.75

Loss on Ignition (T 267)

Loss, % H2O, %

Water Content (T 265), %

13.5

Vane Shear Test on Shelby Tubes (Maine DOT)

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

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **10/28/2004**

Paper Copy: Lab File; Project File; Geotech File

Appendix C

Calculations

Bearing Capacity - Abutment spread footing foundations

Definition of units

$psf := \frac{lbf}{ft^2}$ $pcf := \frac{lbf}{ft^3}$ $Mg := 1000 \cdot kg$ $kN := 1000 \cdot newton$ $kPa := \frac{kN}{m^2}$ $ton := 2000 \cdot lbf$ $tsf := \frac{ton}{ft^2}$ $kip := 1000 \cdot lbf$

$ksf := \frac{kip}{ft^2}$ $i := 1..5$

Method 1

Method: *NavFac DM 7.2, May 1983, Foundations and Earth Structures*, Table 1 7.2-142, "Presumptive Values of Allowable Bearing Pressures for Spread Foundations".

Description of Bearing Material:	Fresh, hard, medium grained IGNEOUS (Diorite and Monzonite). RQD's recorded are 100%, 95%, 85%.
Consistency in Place:	Hard, fresh.
Allowable Bearing Pressure Tons Per sq ft	Range: 60 tsf - 100 tsf. Recommended Value for use is 80 tsf

Method 2

Method: *Rock Engineering Applications by Franklin, Dusseault*, Chapter 3.6.4.

RQD 85% (lowest value)

Allowable Bearing Pressure 15 MPa

$q_2 := 15000 \cdot kPa$ $q_2 = 313\,ksf$

Method 3

Method: *AASHTO Standard Specifications - 17th Edition, 2002*

Section 4.4.8.1.1 - Competent Rock

Figure 4.4.8.1.1.A - for footings suported on compent rock.

Low RQD of rock is 85%

Allowable contact stress 150 tsf

Method 4

AASHTO Standard Specifications - 17th Edition, 2002

Section 4.4.8.1.2. Footings on Broken or Jointed Rock

Table 4.4.8.1.2.A - for footings supported on jointed rock.

a. estimated RMR, Rock Mass Rating,	Low value is 85% - Good for RQD = 75-90
b. Rock Category per 4.4.8.1.2B	E, Granite
c. Unconfined compressive strength, Co	5,000 psi estimated (range given 3500 to 49,000 psi)
d. NMS, per Table 4.4.8.1.2A	0.46
e. Q ult	NMS x Co

$Q_{ult} := 0.46 \cdot 5000 \cdot psi$ $Q_{ult} = 331.2\,ksf$ $Q_{allowable} := \frac{Q_{ult}}{3}$ $\boxed{Q_{allowable} = 110\,ksf}$

$Q_{allowable} = 766.667\,psi$

Recommend limiting allowable bearing pressure of 30 ksf to control eccentricity - the location of the resultant pressure on the base of the footing shall be within B/4 of the center of the footing.

Method 5

Reference: Canadian Foundation Manual (1985)

Type and condition of rocks:	Massive igneous
Presumed allowable bearing pressure	10 MPa

$q_{allowable} := 10000 \cdot kPa$ $\boxed{q_{allowable} = 209\,ksf}$

Analysis: ALLOWABLE BEARING CAPACITY - Compacted granular fill
Structure: Spread footing on fill 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 footing founded with 5.9 ft embedment for frost.

2. Assumed pamaters for compacted granular backfill
   saturated unit weight = 130 pcf
   dry unit weight = 125 pcf
   internal friction angle of 32 degree
   undrained shear strength (c) 0 psf

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

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 1977 Table 3-4
ϕ: Bowles (4 th Ed) Table 2-6
Mass Highway unnamed Table for γsat
γ sat : Holtz, Kovacs, Table 2-1 1981

Footing Width and Depth

$$B := \begin{pmatrix} 5 \\ 8 \\ 10 \\ 12 \\ 15 \\ 20 \end{pmatrix} \cdot ft \qquad D_f := 5.9 \cdot ft \qquad D_w := 0 \cdot ft \qquad \gamma_w := 62.4 \cdot pcf$$

Foundation Soil (Granular Fill):

$\gamma 1_{sat} := 20.5 \cdot \frac{kN}{m^3}$ $\gamma 1_{sat} = 130.501\,pcf$ $\gamma 1_d := 19.7 \cdot \frac{kN}{m^3}$ $\gamma 1_d = 125.408\,pcf$

$\gamma 1_t := \gamma 1_{sat}$ $\phi := 32 \cdot deg$ $c_1 := 0 \cdot psf$

Method 1 : Presumptive Bearing Capacity

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:
Coarse to medium sand, little gravel	Very compact	4 to 6	4 tsf
	Medium to compact	2 to 4	3 tsf
	Loose	1 to 3	1.5 tsf

Recommend 3 tsf / 6 ksf

Method 2: Terzaghi Method - ϕ and c soil.

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

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

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

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

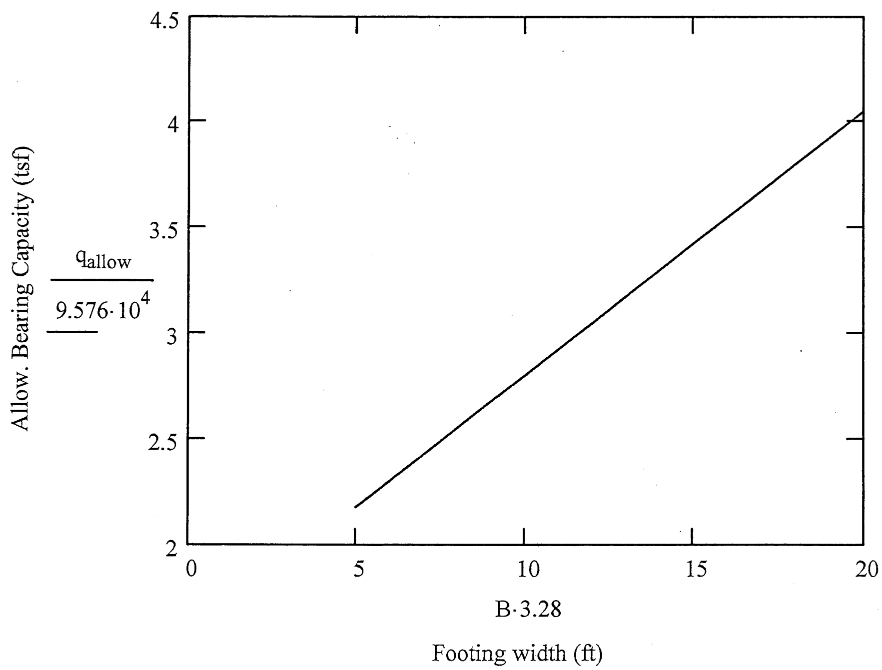
Terzaghi equation (Bowles, Table 4-1, 5th Ed., pg 220)

$q := D_w \cdot \gamma 1_d + (D_f - D_w) \cdot (\gamma 1_{sat} - \gamma_w)$ $q = 0.201\,tsf$

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

$$q_u = \begin{pmatrix} 6.5 \\ 7.7 \\ 8.4 \\ 9.2 \\ 10.3 \\ 12.2 \end{pmatrix} \text{tsf} \qquad q_{allow} := \frac{q_u}{3} \qquad \boxed{q_{allow} = \begin{pmatrix} 2.2 \\ 2.6 \\ 2.8 \\ 3.1 \\ 3.4 \\ 4.1 \end{pmatrix} \text{tsf}}$$

Recommend a limiting value of 6 ksf or 3 tsf, for footings 10 ft or greater on compacted granular fill.



Frost Protection

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

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

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

From Design Freezing Index Map:
East Machias, Maine
DFI =1150

Case I - Soils at elevation of footings of are native fill - fine to coarse SAND, some gravel, trace silt - WC=5%, assume WC=10%

Result:

Depth of Frost Penetration = 79.95 in or 5.9 ft

Frost Protection

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

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

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

From Design Freezing Index Map:
East Machias, Maine
DFI =1150

Case I - Soils at elevation of possible footings of are native fill - fine to coarse SAND, some gravel, trace silt - WC=5%, assume WC=10%

Result:

Depth of Frost Penetration = 71.5 in or 6.0 ft

Pile Input Parameters

HP Pile	area of steel (sq. in) HP Tip	perimeter (ft2/ln ft)	Web depth (in)	flange width (in)	Area of 100% Plugged Toe (ft2)
14x73	21.4	6.96	13.61	14.585	1.37848507
14x89	26.1	7.01	13.83	14.695	1.41133229
12x53	15.5	5.82	11.78	12.045	0.98534792

Overburden Stress

Depth to pile top (ft)	Unit weight (pcf)	Q (psf)
12	125	1500

Ground surface elev 47
Water table elev 30
Depth to Water 17

Table 1. Static Capacity of Abutment Piles in Glacial Till and Fill - 12x53 HP-Piles
Effective stress method, use effective stress at midpoint of layer in these formulas

Effective Stress Method, per PDCA , last revised 1998 Hannigan, et al
Bjerrum-Burland beta coefficient

unit pile leng	2

Top Elev of Stratum (ft)	Soil Stratum	Density (SPT)	Internal friction Angle	Depth to bottom of layer (ft)	Unit weight (pcf)	Midpoint Total Stress (psf)	Midpoint Pore water pressure (psf)	Effective stress (psf)	B coef-ficient	Unit shaft resistance (ksf)	Cumm-ulative shaft resistance (kips)	Toe bearing coefficient	Steel toe resistance (no plug) (kips)	Shaft + steel toe (no plug) kips	Steel toe resistance (30% plug) (kips)	Shaft + steel toe (30% plug) (kips)	Steel toe resistance 100% plug (kips)	Shaft + steel toe 100% plug (kips)
35	fill	loose	28	14	120	1620	0	1620	**0.28**	0.45	5	**30**	5	11	14	20	48	53
33	fill	loose	28	16	120	1860	0	1860	**0.28**	0.52	11	**30**	6	17	16	28	55	66
31	fill	loose	28	18	120	2100	0	2100	**0.28**	0.59	18	**30**	7	25	19	37	62	80
29	alluvium	med dense	32	20	125	2345	0	2345	**0.3**	0.70	26	**30**	8	34	21	47	69	96
27	Till	very dense	36	22	125	2595	0	2595	**0.42**	1.09	39	**30**	8	47	23	62	77	116
25	Till	very dense	36	24	125	2845	62.4	2782.6	**0.42**	1.17	53	**70**	21	74	58	110	192	245
23	Till	very dense	36	26	125	3095	187.2	2907.8	**0.42**	1.22	67	**70**	22	89	60	127	201	267
21	Till	very dense	36	28	125	3345	312	3033	**0.42**	1.27	82	**70**	23	105	63	144	209	291
19	Till	very dense	36	30	125	3595	436.8	3158.2	**0.42**	1.33	97	**70**	24	121	65	162	218	315

Pile Input Parameters

HP Pile	area of steel (sq. in) HP Tip	perimeter (ft2/ln ft)	Web depth (in)	flange width (in)	Area of 100% Plugged Toe (ft2)
14x73	21.4	6.96	13.61	14.585	1.37848507
14x89	26.1	7.01	13.83	14.695	1.41133229
12x53	0	0			

Overburden Stress

Depth to pile top (ft)	Unit weight (pcf)	Q (psf)
12	125	1500

Ground surface elev	47
Water table elev i	30
Depth to Water	17

Table 1. Static Capacity of Abutment Piles in Glacial Till and Fill - 14x73 HP-Piles
Effective stress method, use effective stress at midpoint of layer in these formulas

Effective Stress Method, per PDCA , last revised 1998 Hannigan, et al
Bjerrum-Burland beta coefficient

unit pile leng	2

Top Elev of Stratum (ft)	Soil Stratum	Density (SPT)	Internal friction Angle	Depth to bottom of layer (ft)	Unit weight (pcf)	Midpoint Total Stress (psf)	Midpoint Pore water pressure (psf)	Effective stress (psf)	B coef-ficient	Unit shaft resistance (ksf)	Cumm-ulative shaft resistance (kips)	Toe bearing coefficient	Steel toe resistance (no plug) (kips)	Shaft + steel toe (no plug) kips	Steel toe resistance (30% plug) (kips)	Shaft + steel toe (30% plug) (kips)	Steel toe resistance 100% plug (kips)	Shaft + steel toe 100% plug (kips)
35	fill	loose	28	14	120	1620	0	1620	**0.28**	0.45	6.3	**30**	7	14	20	26	67	73
33	fill	loose	28	16	120	1860	0	1860	**0.28**	0.52	13.6	**30**	8	22	23	37	77	90
31	fill	loose	28	18	120	2100	0	2100	**0.28**	0.59	21.7	**30**	9	31	26	48	87	109
29	alluvium	med dense	32	20	125	2345	0	2345	**0.3**	0.70	31.5	**30**	10	42	29	61	97	129
27	Till	very dense	36	22	125	2595	0	2595	**0.42**	1.09	46.7	**30**	12	58	32	79	107	154
25	Till	very dense	36	24	125	2845	62.4	2782.6	**0.42**	1.17	63.0	**70**	29	92	81	144	269	331
23	Till	very dense	36	26	125	3095	187.2	2907.8	**0.42**	1.22	80.0	**70**	30	110	84	164	281	361
21	Till	very dense	36	28	125	3345	312	3033	**0.42**	1.27	97.7	**70**	32	129	88	186	293	390
19	Till	very dense	36	30	125	3595	436.8	3158.2	**0.42**	1.33	116.2	**70**	33	149	91	208	305	421

Pile Input Parameters

HP	area of steel	perimeter	Web	flange	Area of 100%
Pile	(sq. in)	(ft2/ln ft)	depth	width	Plugged Toe
	HP Tip		(in)	(in)	(ft2)
14x73	21.4	6.96	13.61	14.585	1.37848507
14x89	26.1	7.01	13.83	14.695	1.41133229
12x53	0	0			

Overburden Stress

Depth to pile top (ft)	Unit weight (pcf)	Q (psf)
12	125	1500

Ground surface elev	47
Water table elev i	30
Depth to Water	17

Table 1. Static Capacity of Abutment Piles in Glacial Till and Fill - 14x89 HP-Piles
Effective stress method, use effective stress at midpoint of layer in these formulas

Effective Stress Method, per PDCA , last revised 1998 Hannigan, et al
Bjerrum-Burland beta coefficient

unit pile leng	2

Top Elev of Stratum (ft)	Soil Stratum	Density (SPT)	Internal friction Angle	Depth to bottom of layer (ft)	Unit weight (pcf)	Midpoint Total Stress (psf)	Midpoint Pore water pressure (psf)	Effective stress (psf)	B coef-ficient	Unit shaft resistance (ksf)	Cumm-ulative shaft resistance (kips)	Toe bearing coefficient	Steel toe resistance (no plug) (kips)	Shaft + steel toe (no plug) kips	Steel toe resistance (30% plug) (kips)	Shaft + steel toe (30% plug) (kips)	Steel toe resistance 100% plug (kips)	Shaft + steel toe 100% plug (kips)
35	fill	loose	28	14	120	1620	0	1620	**0.28**	0.45	6.4	**30**	9	15	21	27	69	75
33	fill	loose	28	16	120	1860	0	1860	**0.28**	0.52	13.7	**30**	10	24	24	37	79	92
31	fill	loose	28	18	120	2100	0	2100	**0.28**	0.59	21.9	**30**	11	33	27	49	89	111
29	alluvium	med dense	32	20	125	2345	0	2345	**0.3**	0.70	31.8	**30**	13	45	30	62	99	131
27	till	very dense	36	22	125	2595	0	2595	**0.42**	1.09	47.0	**30**	14	61	33	80	110	157
25	till	very dense	36	24	125	2845	62.4	2782.6	**0.42**	1.17	63.4	**70**	35	99	82	146	275	338
23	till	very dense	36	26	125	3095	187.2	2907.8	**0.42**	1.22	80.6	**70**	37	117	86	167	287	368
21	till	very dense	36	28	125	3345	312	3033	**0.42**	1.27	98.4	**70**	38	137	90	188	300	398
19	till	very dense	36	30	125	3595	436.8	3158.2	**0.42**	1.33	117.0	**70**	40	157	94	211	312	429

$psf := \frac{lbf}{ft^2}$ $pcf := \frac{lbf}{ft^3}$ $tonf := g \cdot ton$ $tsf := \frac{tonf}{ft^2}$

$psi := \frac{lbf}{in^2}$ $kip := 1000 \cdot lbf$ $kN := 10^3 \cdot newton$ $kPa := 10^3 \cdot Pa$

$MN := 10^6 \cdot newton$ $ksf := \frac{kip}{ft^2}$ $MPa := 10^6 \cdot Pa$ $ksi := \frac{kip}{in^2}$

Pile Properties

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

$i := 1, 2 .. 4$

$$A_s := \begin{pmatrix} 15.5 \\ 21.4 \\ 26.1 \\ 34.4 \end{pmatrix} \cdot in^2 \qquad d_i := \begin{pmatrix} 11.78 \\ 13.61 \\ 13.83 \\ 14.21 \end{pmatrix} \qquad b_i := \begin{pmatrix} 12.045 & 14.585 & 14.695 & 14.885 \end{pmatrix}$$

$$A_{box} := \begin{pmatrix} 11.78 \cdot 12.045 \\ 13.61 \cdot 14.585 \\ 13.83 \cdot 14.695 \\ 14.21 \cdot 14.885 \end{pmatrix} \cdot in^2 \qquad A_{box} = \begin{pmatrix} 141.89 \\ 198.502 \\ 203.232 \\ 211.516 \end{pmatrix} in^2$$

Structural Capacity

Geotechnical design capacity shall not exceed the pile structural allowable design load , based on allowable steel stress for integral piles, use 50 ksi steel, therefore 0.25Fy is the allowable stress.

For 36 ksi steel $F_y := 36 \cdot ksi$ $\sigma_a := \frac{F_y}{4}$ $Q_{all} := \sigma_a \cdot A_s$ $Q_{all} = \begin{pmatrix} 140 \\ 193 \\ 235 \\ 310 \end{pmatrix} kip$

For 50 ksi steel $F_y := 50 \cdot ksi$ $\sigma_a := \frac{F_y}{4}$ $Q_{all} := \sigma_a \cdot A_s$ $Q_{all} = \begin{pmatrix} 194 \\ 268 \\ 326 \\ 430 \end{pmatrix} kip$

Geotechnical Capacity

HP pile capacity based on end bearing on bedrock - assume driven thru overlying medium consistency fills and glacial till. RQD ranges from 85% at Abutment #1, and 100-95% at the Abutment 2.

Rock Type: Hard, fresh IGNEOUS (Monzonite, Diorite). Use RQD of 85% for design.

ϕ = 34 - 40 (Tomlinson, pg. 139)

Method 1

based on (UCT'S) unconfined compressive strength of bedrock. Foundation bedrock is granite (monzonite) and diorite of excellent quality.

$t := 0 \cdot in$

$q_{uc} := 10000 \cdot psi$ Reference AASHTO Table 4.4.8.1.2B, granite Quc is 2,100 to 49,000 psi

$Q_{ult} := q_{uc} \cdot A_s$ $\qquad Q_{ult} = \begin{pmatrix} 155 \\ 214 \\ 261 \\ 344 \end{pmatrix} kip$

$Q_{all\_tip\_method1} := \dfrac{Q_{ult}}{2.25}$ $\qquad Q_{all\_tip\_method1} = \begin{pmatrix} 69 \\ 95 \\ 116 \\ 153 \end{pmatrix} kip$

2. Goodman's Method

Also based on unconfined compressive strength of bedrock (Ref: DAS, 2nd Ed., Principles of Foundation Engineering, page 473). Per DAS, reduce lab unconfined compressive strength by 4 to 5 for scale effect due to small dia. lab specimens.

Qu_design = Qu_lab/5 ***(Equation 8.56)***

$\phi := 36 \cdot deg$ $\qquad q_{uc\_lab} := 10000 \cdot psi$

$N_\phi := \tan\left(45 \cdot deg + \dfrac{\phi}{2}\right)^2$ $\qquad N_\phi = 3.852$

$q_{point\_ult} := \dfrac{q_{uc\_lab}}{5} \cdot (N_\phi + 1)$ $\qquad q_{point\_ult} = 9.704\,ksi$

$Q_{point\_ult} := q_{point\_ult} \cdot A_s$ $\qquad Q_{point\_ult} = \begin{pmatrix} 150.407 \\ 207.659 \\ 253.266 \\ 333.807 \end{pmatrix} kip$

$Q_{point\_all\_Das} := \dfrac{Q_{point\_ult}}{2.25}$ $\qquad Q_{point\_all\_Das} = \begin{pmatrix} 67 \\ 92 \\ 113 \\ 148 \end{pmatrix} kip$

Method 3 - Goodman's Method

For factured rock. Reference: "Pile Design and Construction Practice", 4th Edition, Tomlinson.

low friction: 20-27 (schist, shale)
medium freciton 27-34 (sandstone, siltstone, gneiss, slate)
high friction 34-40 (granite)

$\phi := 36 \cdot deg$ $\qquad N_\phi := \tan\left(45 \cdot deg + \dfrac{\phi}{2}\right)^2$ $\qquad N_\phi = 3.852$

$q_b := 2 \cdot N_\phi \cdot \dfrac{q_{uc\_lab}}{5}$ $\qquad q_b = 15.407\,ksi$

$Q_{ult} := q_b \cdot A_s$ $\qquad Q_{ult} = \begin{pmatrix} 238.814 \\ 329.718 \\ 402.132 \\ 530.013 \end{pmatrix} kip$

$Q_{all\_tip\_Goodman} := \dfrac{Q_{ult}}{2.25}$ $\qquad Q_{all\_tip\_Goodman} = \begin{pmatrix} 106 \\ 147 \\ 179 \\ 236 \end{pmatrix} kip$

Max. design end bearing - side resistance ignored

Method 4 - Kulhawy and Goodman - RQD method

Ref: "Pile Design & Construction Practice", Tomlinson, page 139

For calculating maximum end bearing allowable load -this method ignores side resistance - use Driven to calculate that.

Correct for wedge failure under strip footing:

multiply cNc by 1.25 - square piles,
multiply $\gamma B N\gamma$ by 0.8 - square pile.

For RQD 0 -70 %

qc=0.33 x Quc
c=0.1 x Quc
ϕ=30 degrees

For RQD 70 - 100 %

qc=0.33 to 0.88 x Quc
c=0.10 x Quc
ϕ=30 to 60 degrees

Calculation

assume RQD =85	
ϕ = 36	***PELLS & TURNER, TOMLINSON, PAGE 140***
c= .10 x Quc	*Assume pile penetrates 1 inches into bedrock*
qc = 0.33 x Quc	*Assume fresh granite has an unconfined Qu of 10000 psi and a phi of 36.*

$q_{uc} := (10000)\cdot psi$ $\quad c := 0.1\cdot q_{uc}$ $\quad c = 1 \times 10^{3}\,psi$

$D := .1\cdot in$

$B_{min} := 12.045\cdot in$

$\gamma := 150\cdot\frac{lbf}{ft^3}$ $\quad q_c := 0.33\cdot q_{uc}$ $\quad q_c = 3300\,psi$

$N_c := 19.04$ $\quad N_q := 14.84$ $\quad N_\gamma := 27.16$ ***(BASED ON PELLS & TURNER, TOMLINSON, PAGE 140)***

$$q_{ub} := 1.25 \cdot c \cdot N_c + \frac{\gamma \cdot B_{min} \cdot N_\gamma}{2} \cdot 0.8 + \gamma \cdot D \cdot N_q$$

This is ultimate base resistance, settlement up to 20% of base dia. needed to mobilize. So to ensure settlements at working load are within allowable limits, limit working load to q ub divided by 2.5

$$q_{ub} = 23.811\,\text{ksi}$$

$$Q_{ult} := q_{ub} \cdot A_s$$

$$x := \gamma \cdot D \cdot N_q$$

$$x = 1.288 \times 10^{-4}\,\text{ksi}$$

$$Q_{ult} = \begin{pmatrix} 369.078 \\ 509.566 \\ 621.48 \\ 819.115 \end{pmatrix} \text{kip}$$

$$Q_{all\_tip\_Kulhawy} := \frac{Q_{ult}}{2.25} \qquad Q_{all\_tip\_Kulhawy} = \begin{pmatrix} 164 \\ 226 \\ 276 \\ 364 \end{pmatrix} \text{kip}$$

<u>Pile Shaft Resistance</u>

Applying FS of 2.25 to skin friction. Beta - Effective Stress Method, PDCA, (analysis included in this Appendix) is used for estimation of pile shaft resistance capacity.

$$Q_{side} := \begin{pmatrix} 97 \\ 116 \\ 117 \\ 0 \end{pmatrix} \cdot \text{kip}$$

<u>Total Pile Capacity - Results</u>

1. Method 1

$$Q_{all\_tip\_method1} + \frac{Q_{side}}{2.25} = \begin{pmatrix} 112 \\ 147 \\ 168 \\ 153 \end{pmatrix} \text{kip}$$

2. Goodmans Method, (DAS)

$$Q_{point\_all\_Das} + \frac{Q_{side}}{2.25} = \begin{pmatrix} 110 \\ 144 \\ 165 \\ 148 \end{pmatrix} kip$$

3. Goodmans Method, Tomlinson, 4th Ed.

$$Q_{all\_tip\_Goodman} + \frac{Q_{side}}{2.25} = \begin{pmatrix} 149 \\ 198 \\ 231 \\ 236 \end{pmatrix} kip$$

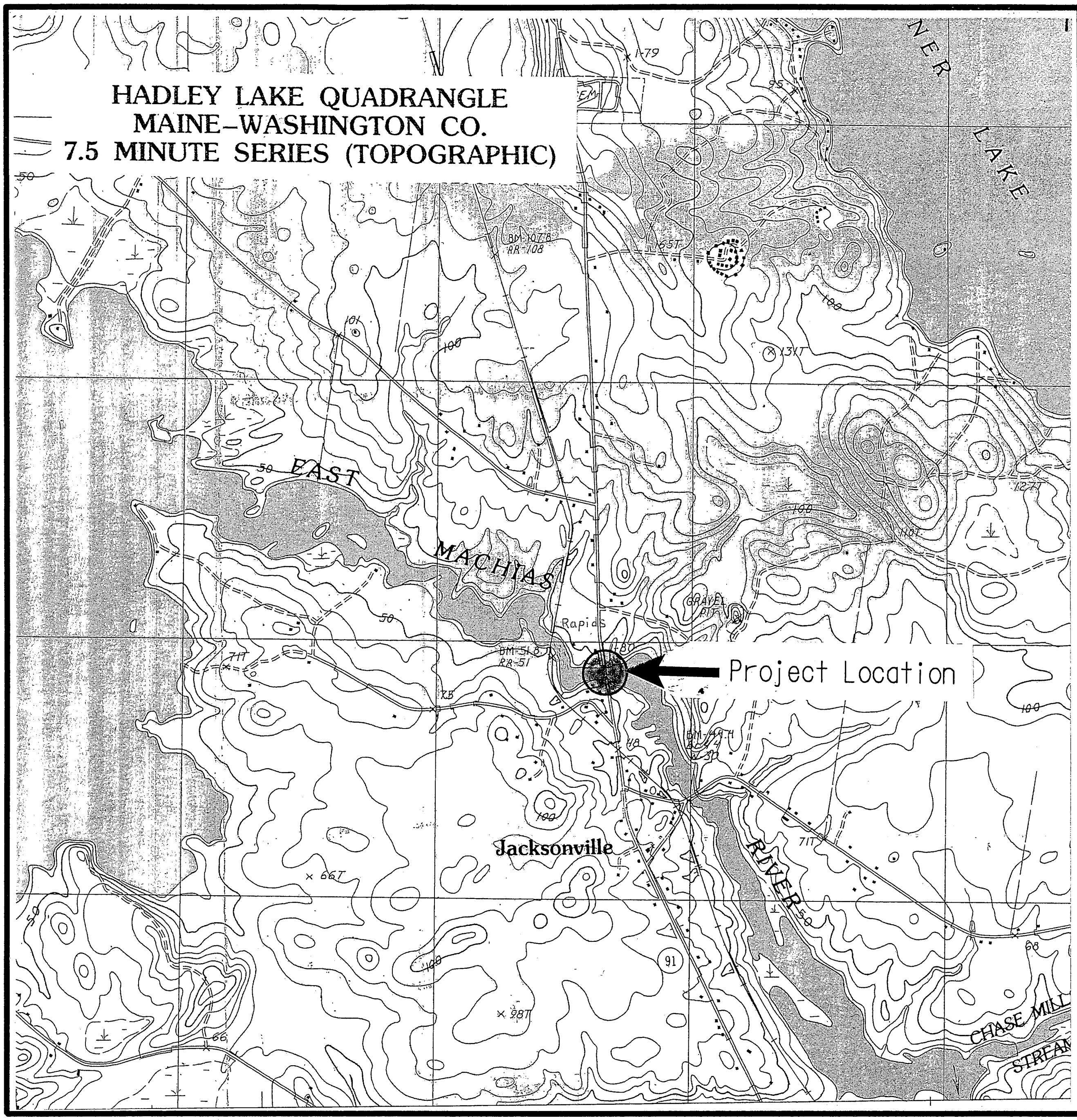
4. Kulhawy and Goodman, Tomlinson, 4th Ed.

$$Q_{all\_tip\_Kulhawy} + \frac{Q_{side}}{2.25} = \begin{pmatrix} 207 \\ 278 \\ 328 \\ 364 \end{pmatrix} kip$$

Sheets

Location Map

Sheet 1



East Machias, Maine, Jacksonville Bridge over East Machias River,
PIN. 11036.00

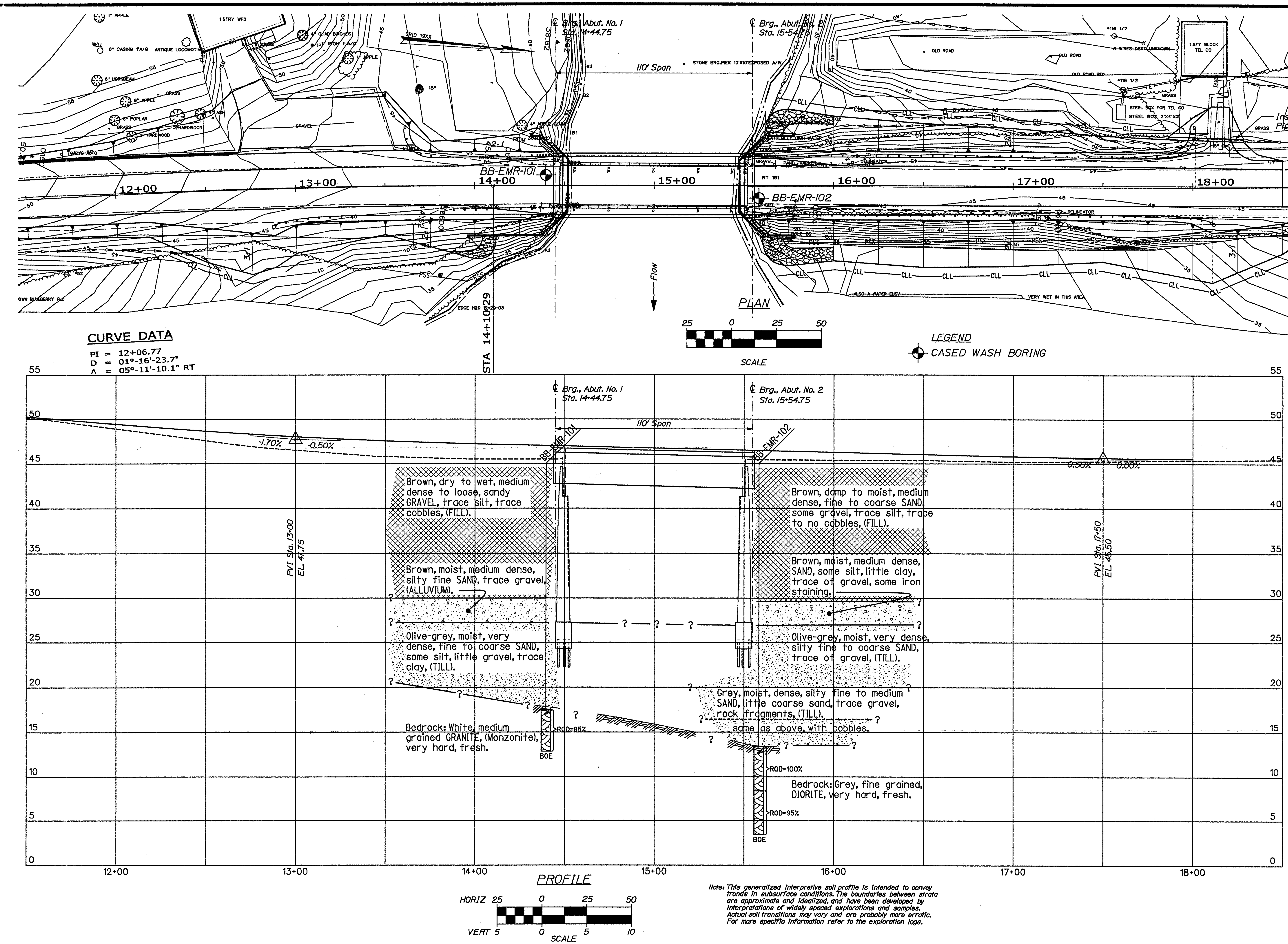
1 .5 0 1 MILE

1000 0 1000 2000 3000 4000 5000 6000 7000 FEET

1 .5 0 1 KILOMETER

1:24000, 1''= 2000', 1 cm = 240 m

N



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

BH-1103(600)X

PIN 11036.00

BRIDGE PLANS

BRIDGE NO. 3219

	BY	DATE
PROJ. MANAGER	D. ANDERSON	
DESIGN-DETAILED	L. KRUSINSKI	T. WHITE	OCT 2004
CHECKED-REVIEWED		
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

JACKSONVILLE BRIDGE
EAST MACHIAS RIVER
EAST MACHIAS WASHINGTON COUNTY

BORING LOCATION PLAN &
INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
2
OF 4

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

Project: Jacksonville Bridge over East Machias River on Route 191
Location: East Machias, Maine

Boring No.: BB-EMR-101
PIN: 11036.00

Driller:	MaineDOT	Elevation (ft.)	45.0	Auger ID/OD:	5" SSA
Operator:	C. Mann	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	G. Lidstone	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	10/7/04-10/7/04	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ 1.88 in ID
Boring Location:	14+39.6, 5.9 Lt.	Casing ID/OD:	HW	Water Level*:	9.0' bgs.

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods

Definitions:
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-value	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0						SSA	44.40	PAVEMENT. —0.60— Brown, dry, sandy GRAVEL, trace of cobbles and silt, (Fill).	
							41.00	—4.00— Brown, dry to damp, medium dense, sandy GRAVEL, trace silt, (Fill).	
5	1D	24/9	5.00 - 7.00	9/12/12/13	24	38			
						35			
						30			
						31	36.00	—9.00— Brown, wet, loose, sandy GRAVEL, trace silt, (Fill).	
						22			
10	2D	24/1	10.00 - 12.00	4/3/5/4	8	15			
						13			
						12			
						13			
						11	30.00		
15	3D	24/6	15.00 - 17.00	7/5/6/4	11	15		—15.00— Brown, moist, medium dense, silty fine SAND, trace small rounded gravel, some iron staining, changing to same but dark grey. (Alluvium)	
						18			
						85	27.50	—17.50—	
						117			
						120			
20	4D	24/11	20.00 - 22.00	32/27/29/28	56	68		Olive-grey, moist, very dense, fine to coarse SAND, some silt, little small round gravel, trace of clay (Till).	G#182502 A-2-4, SC-SM WC=9.5%
						135			
						172			
						288		Washed ahead from 24.0-25.0' bgs.	
						OPEN HOLE			
25	5D	18/11	25.00 - 26.50	31/62/55	117			Similar to 4D, except Olive-brown.	G#182504 A-4, SC-SM WC=9.7
								Washed ahead from 25.0-27.5' bgs.	
	R1	56/54	27.50 - 32.17	RQD = 85%		~~SSC~~ NQ	17.90 / 17.50	—27.10— ªRoller coned into bedrock from 27.1 to 27.5' bgs. —27.50— R1: Bedrock: White, medium grained GRANITE (monzonite), very hard, fresh, no joint set evident. Topmost 4 to 6 in fractured and surfaces oxidized. Good rock quality. Core Times (min:sec) 27.5-28.5' (5:09) 28.5-29.5' (5:48) 29.5-30.5' (4:29) 30.5-31.5' (5:02) 31.5-32.17' (3:54) 96% Recovery	
30									
							12.83	—32.17— Bottom of Exploration at 32.17 feet below ground surface.	
35									
40									
45									
50									

Remarks:
Borings BB-EMR-101 and BB-EMR-102 marked on pavement with white paint for possible location by Survey.

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-EMR-101

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

Project: Jacksonville Bridge over East Machias River on Route 191
Location: East Machias, Maine

Boring No.: BB-EMR-102
PIN: 11036.00

Driller:	MaineDOT	Elevation (ft.)	45.0	Auger ID/OD:	5" SSA
Operator:	C. Mann	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	G. Lidstone	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	10/6/04-10/6/04	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-1.88" I.D.
Boring Location:	15+58.3, 7.1 Rt.	Casing ID/OD:	HW	Water Level*:	10.0' bgs.

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods

Definitions:
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-value	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0						SSA	44.35	PAVEMENT. —0.65— Brown, dry, fine to coarse SAND, some gravel, trace silt, trace of cobbles (Fill).	
5	1D	24/9	5.00 - 7.00	9/14/13/12	27	42	40.00	—5.00— Brown, dry to damp, medium dense, fine to coarse SAND, some gravel, trace silt, (Fill).	G#182501 A-1-a, SW-SM WC=5.0%
						19			
						21			
						18			
						16			
10	2D	24/2	10.00 - 12.00	7/6/5/5	11	13	35.00	—10.00— Similar to above, but moist, (Fill).	
						23			
						25			
						28			
						33			
15	3D	24/11	15.00 - 17.00	12/10/11/13	21	21	29.80	—15.20— Brown, moist, medium dense SAND, some silt, little clay, trace of small rounded gravel, some iron staining.	G#182503 A-4, SC-SM WC=13.5%
						75			
						86			
						143	27.00	—18.00—	
						148			
20	4D	24/13	20.00 - 22.00	21/33/33/32	66	25		Olive-grey, moist, very dense, silty fine to medium SAND, trace coarse sand, trace small rounded gravel. (Till) Washed ahead from 20.0-25.0' bgs.	
						20			
						19			
						16			
						50			
25	5D	24/4	25.00 - 27.00	24/23/17/14	40	76	20.00	—25.00— Dark grey, moist, dense, silty fine to medium SAND, little coarse sand, trace of rounded gravel; one 1 1/2 in dia fractured rock fragment (Till).	
						78			
						62			
						117	16.50	—28.50— Similar to above, with cobbles, (Till).	
						310			
30	MD	1/0	30.00 - 30.08	53(1")	—	178		Washed ahead from 30.0-31.6' bgs. ª100 blows for 0.6'.	
	R1	60/60	31.60 - 36.60	RQD = 100%		ª100 NQ	13.40	—31.60— R1: Bedrock: Dark grey, fine grained DIORITE, very hard, fresh, no jointing. Excellent rock quality. R1:Core Times (min:sec) 31.6-32.6' (6:48) 32.6-33.6' (4:45) 33.6-34.6' (5:09) 34.6-35.6' (4:32) 35.6-36.6' (4:47) 100% Recovery	
35									
	R2	60/58	36.60 - 41.60	RQD = 95%			8.40	—36.60— R2: Same as R1, only very slighty weathered, 2 joint sets evident, chlorite on joint surfaces. Excellent rock quality. Core Times (min:sec) 36.6-37.6' (3:33) 37.6-38.6' (3:20) 38.6-39.6' (3:29) 39.6-40.6' (3:16) 40.6-41.6' (3:14) 98% Recovery	
40									
							3.40	—41.60— Bottom of Exploration at 41.60 feet below ground surface.	
45									
50									

Remarks:
Borings BB-EMR-101 and BB-EMR-102 marked on pavement with white paint for possible location by Survey.

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.

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Boring No.: BB-EMR-102

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BH-1103(600)X

PIN 11036.00

BRIDGE NO. 3219

BRIDGE PLANS

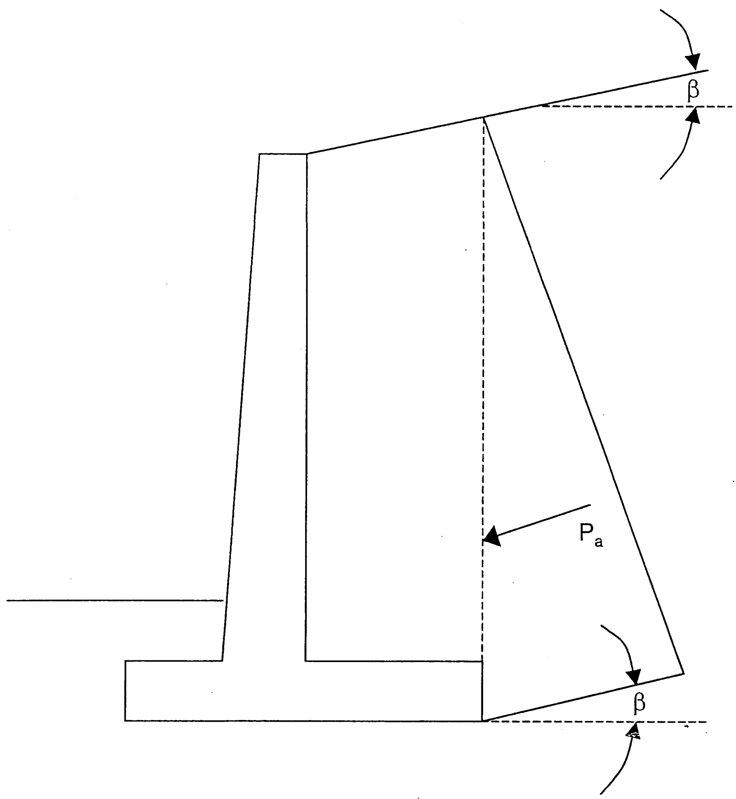
	BY	DATE
PROJ. MANAGER	D. ANDERSON	
DESIGN-DETAILED	L. KRUSINSKI	
CHECKED-REVIEWED	T. WHITE	OCT 2004
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE
P.E. NUMBER
DATE

JACKSONVILLE BRIDGE
EAST MACHIAS RIVER
EAST MACHIAS WASHINGTON COUNTY

BORING LOGS

SHEET NUMBER
3
OF 4



For cases where interface friction between the backfill and wall are 0 or not considered, use Rankine.

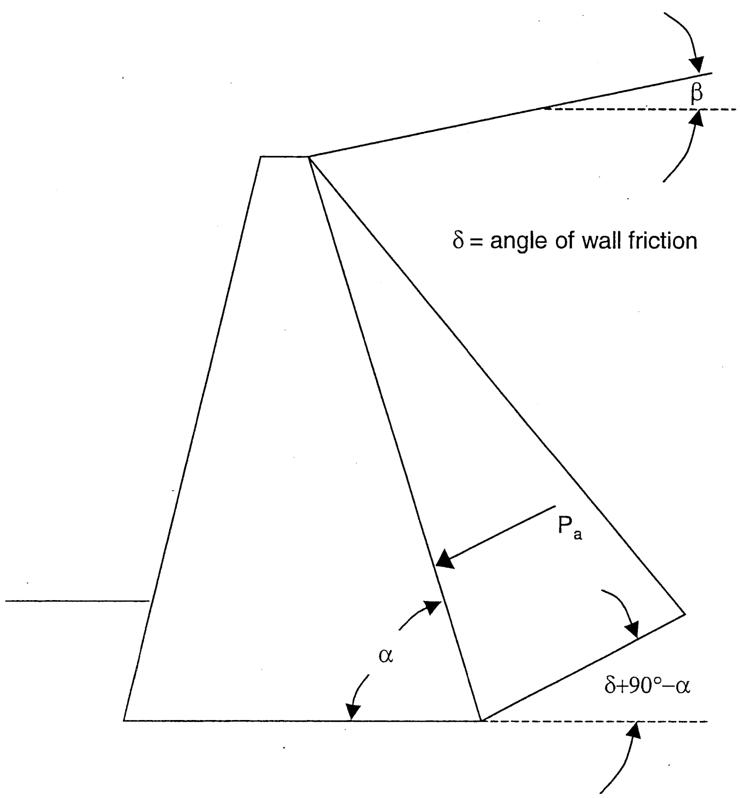
For a horizontal backfill surface, $\beta = 0°$:

$$K_a = \tan^2\left(45° - \frac{\phi}{2}\right)$$

For a sloped backfill surface, $\beta > 0°$:

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

P_a is oriented at β



For cases where interface friction is considered, use Coulomb.

For horizontal or sloped backfill surfaces:

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

P_a is oriented at $\delta + 90° - \alpha$

Rankine and Coulomb Active Earth Pressure Coefficients