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BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

JACKSONVILLE BRIDGE
ROUTE 191 OVER EAST MACHIAS RIVER
EAST MACHIAS, MAINE

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GEOTECHNICAL DESIGN SUMMARY

The purpose of this report is to present subsurface information and make geotechnical recommendations for the replacement of Jacksonville Bridge over East Machias River on State Route 191 in East Machias, Maine. The proposed replacement bridge is a 130-foot single span, weathering steel plate girder bridge on H-pile supported integral abutments. The new bridge alignment will match the horizontal alignment of the existing bridge. The following design recommendations are discussed in detail in this report:

Integral Abutment H-piles - H-piles for support of the integral abutments should be end bearing and driven to the required resistance on or within bedrock. The H-piles shall be designed for all relevant strength, service and extreme limit state load groups. Due to shallow bedrock at Abutments No. 1 and No. 2 there is inadequate pile embedment to provide fixity or a pinned condition at the pile tip. Therefore, the piles should be fitted with Rock Injector Pile Points to improve penetration and friction at the pile tips.

Lateral loads will be reacted by plumb piles. A series of lateral pile resistance analyses using LPILE Plus 5.0 software (herein referred to as LPILE) have been completed for both abutments. LPILE analyses confirm the pile lengths are inadequate to prevent translation of the pile at both abutments. The results for both abutments have been provided to the structural engineer for evaluation. The structural engineer is responsible for assessing the pile for compliance with the interaction equation and for combined axial and flexure loads due to pile head moments, loads and thermal displacements.

The contractor is required to perform a wave equation analysis. The first pile driven at each abutment shall be dynamically tested to confirm nominal pile resistance and verify the stopping criteria developed by the contractor in the wave equation analysis. With this level of quality control, the pile should be driven to a nominal resistance equal to the factored axial pile load divided by a resistance factor, φ_{dyn}, of 0.65. Restrike tests may be required as part of the pile field quality control program if pile behavior indicates a pile is refusing on a boulder or cobble above bedrock, is not seated firmly within bedrock, or if piles “walk” out of position.

Integral Abutment Design - Integral abutments shall be designed for all relevant strength, service and extreme limit states and load combinations. Calculation of passive earth pressures for integral abutment design should assume a Coulomb passive earth pressure coefficient, K_p, of 6.73. If the ratio of the calculated lateral abutment movement to abutment height (y/H) is less than 0.005, the designer may consider using the Rankine passive earth pressure coefficient of 3.25. For purposes of the integral abutment backwall reinforcing steel design, use a maximum load factor (γ_{EH}) of 1.50 to calculate factored passive earth pressures.

The abutment design shall include a drainage system behind the abutment to intercept any groundwater. The approach slab should be positively connected to the integral abutment.

Additional lateral earth pressure due to construction surcharge or live load surcharge is required if an approach slab is not specified. When a structural approach stab is specified, reduction, not elimination of the surcharge load is permitted.

Return (U-Shaped) Wingwalls - U-shaped return wingwalls will be constructed monolithically with the integral abutments. Monolithic U-shaped wingwalls that are not cantilevered will be designed to be pile supported. The wingwalls shall be designed to resist lateral earth pressures, vehicular loads, collision loads, and creep and temperature and shrinkage deformations.

For design of the wingwalls, two (2) load cases shall be considered. The first load case is where the wingwall is subjected to passive earth pressure to account for the abutment moving and pushing the wingwall into the fill. A Rankine passive earth pressure coefficient, K_p, of 3.25 and a load factor (γ_{EH}) of 1.50 may be used to calculate factored passive earth pressures. The second load case considers that the wingwall is subjected to active pressure and to collision loads. Calculation of active earth pressures shall use the Rankine active earth pressure coefficient, K_a, of 0.31.

There are no bearing resistance considerations for slope-tapered, cantilevered U-shaped wingwalls, however, the walls shall be embedded a minimum of 5.7 feet for frost protection. Alternatively, monolithic U-shaped wingwalls may be designed to be pile supported.

Settlement - The new vertical alignment of the replacement bridge will require 300 feet approach work at each end and a raise in grade of approximately 2 feet. The proposed raise in grade will result in some elastic compression of the loose to medium dense consistency granular fills and alluvial soils encountered below the bridge approaches. Elastic settlements due to the proposed raise in grade are estimated to range from approximately ½ to 1 inch. These settlements are anticipated to occur relatively quickly during construction. Any settlement of the bridge abutments will be due to axial compression of the foundation piles and is anticipated to be less than 0.5 inch.

Frost Protection – Integral abutments shall be embedded a minimum of 4 feet for frost protection. Foundations placed on or in granular soils should be founded a minimum of 5.7 feet below finished exterior grade for frost protection.

Scour and Riprap – The consequences of changes in foundation conditions resulting from the design and check floods shall be considered at the strength and extreme limit states. Preliminary scour depths for the design flood (Q_{100}) event indicate scour will expose bedrock at Abutment No. 1 and leave 4 feet of soil overlying bedrock at Abutment No. 2, the consequence of this potential scour being that the abutment pile groups will become unstable. The abutment slopes will require special scour countermeasures.

For scour protection and protection of pile supported integral abutments the bridge approach slopes and slopes at abutments shall be armored at a minimum with 4 feet of heavy riprap. The top of the riprap should be located at a minimum elevation of the Q_{100} elevation. The

riprap shall be underlain by Class 1 nonwoven erosion control geotextile and 1 foot thick layer of bedding material.

Seismic Design Considerations – Jacksonville Bridge is in Seismic Zone 1 and the response acceleration coefficient is less than 0.10g, therefore no consideration for seismic forces is required except that superstructure connections and minimum support length requirements shall be satisfied.

Construction Considerations – The new integral abutments will be constructed behind the existing abutments, therefore avoiding constructing fills or cofferdams in the river. Construction of the abutments will require pile driving. Temporary lateral earth support systems may be required to permit construction of driven pile foundations at the proposed abutments.

A Special Provision is provided for Rock Injector pile tips to provide a pinned or fixed condition at the pile tips. This item may also require detailing on the construction plans and construction notes. We can assist in preparation of the construction plans and notes.

Excavations for the proposed abutments will expose soils which may become saturated and water seepage may be encountered during construction. There may be localized sloughing and instability in some excavations and cut slopes. The contractor should control groundwater, surface water infiltration and soil erosion. Water should be controlled by pumping from sumps.

Cobbles were encountered in the fill soils underlying the bridge approaches and in the glacial till deposit at the location of the proposed Abutment No. 2. Cobbles may also be encountered in the glacial till at proposed Abutment No. 1. There is potential for these obstructions to impact construction activities. These impacts include but are not limited to impeding the driving of sheet piles for temporary earth support systems and driving H-piles for abutment foundations. Obstructions may be cleared by conventional excavation methods, pre-augering, predrilling, spudding, use of rock chisels, or down-hole hammers. Alternative methods to clear obstructions may be used as approved by the Resident. Care should take to drive piles within allowable tolerances.

1.0 INTRODUCTION

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

The existing Jacksonville Bridge was constructed in 1937. The bridge is a 100-foot single span pony truss. The abutments are unreinforced, mass concrete supported on timber piles. The mass concrete abutments are in "fair" condition and the concrete deck and steel thru truss are in "poor" condition according to a 2012 Maine Department of Transportation (MaineDOT) Bridge Inspection Report. Both abutments exhibit moderate cracking and scaling with undermining of the north bearings. Scour countermeasures are present at both abutments. The superstructure is in overall poor condition with moderate to heavy deterioration. The bridge has a Sufficiency Rating of 21.8 and is fracture critical.

The replacement structure will be a 130-foot single span, weathering steel plate girder bridge. The proposed abutments will be H-pile supported integral abutments. Estimated pile lengths are approximately 16 feet at Abutment No. 1 and approximately 20 feet at Abutment No. 2. Return (U-shaped) wingwalls will be used to minimize the span length. The new integral abutments will be constructed behind the existing abutments, avoiding fills or cofferdams in the river. The existing abutments will be removed to approximately 1 to 2 feet above the $Q_{1.1}$ elevation. The remaining portion of the existing abutments will be left in place as scour protection. Preliminary plans indicate the abutment sideslopes will be constructed at 1.75H:1V and be armored with heavy riprap to resist the calculated Q_{100} and Q_{500} discharge velocities and prevent soil loss and foundation instability after the Q_{100} and Q_{500} flood events.

The new Jacksonville Bridge will be located on the same horizontal alignment as the existing bridge. The vertical alignment of the replacement bridge will require 300 feet of approach work on each end and an approximate 2-foot raise in roadway grade. The new bridge will accommodate two (2) 11-foot lanes with 4-foot shoulders. A temporary bridge will be constructed to maintain traffic during construction.

2.0 GEOLOGIC SETTING

Jacksonville Bridge on State Route 191 in East Machias crosses the East Machias River as shown on Sheet 1 – Location Map. The East Machias River flows in an easterly direction at the bridge site.

The Maine Geological Survey (MGS) Surficial Geology Map of the Gardiner Lake Quadrangle, Maine, Open-file No. 82-4 (1982), indicates the surficial soils in the vicinity of the bridge project consist of glacial marine 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.

The Bedrock Geologic Map of Maine, MGS (1985), cites the bedrock at the proposed bridge site as plutonic igneous rock, comprised of Devonian quartz diorite to gabbro, diorite and ultramafic rocks. The Reconnaissance Bedrock Geology Map of the Gardiner Lake Quadrangle, Maine, MGS (1978), indicates the bedrock consists of hornblende-biotite quartz diorite with a northerly contact to quartz monzonite.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions at the site were explored by drilling two (2) test borings. Both of the borings were terminated with bedrock cores. Test boring BB-EMR-101 was drilled behind the existing southerly abutment and test boring BB-EMR-102 was drilled behind the existing northerly abutment. The test boring locations are shown on Sheet 2 - Boring Location Plan and Interpretive Subsurface Profile.

Test borings BB-EMR-101 and BB-EMR-102 were drilled on October 6 and 7, 2004 by the MaineDOT Drill Crew. 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.

All the borings were drilled using cased wash boring, solid stem auger and rock coring techniques. Soil samples were typically obtained at 5-foot intervals using Standard Penetration Test (SPT) methods. During SPT sampling, the sampler is driven 24 inches and the hammer blows for each 6-inch interval of penetration are recorded. The sum of the blows for the second and third intervals is the N-value, or standard penetration resistance. The borings were drilled using a rope and cathead hammer system to drive the split spoon. No correction is required for N-values obtained with a rope and cathead system based on the assumption that the average energy transfer of the rope and cathead hammer is sixty (60) percent.

The bedrock was cored in two (2) borings using an NQ-1.88” core barrel and the Rock Quality Designation (RQD) of the cores was calculated. A Northeast Transportation Technician Certification Program (NETTCP) Certified Subsurface Inspector logged the subsurface conditions encountered. The MaineDOT geotechnical engineer selected the boring locations and drilling methods, designated type and depth of sampling techniques, reviewed draft boring logs and identified field and laboratory testing requirements. The as-drilled boring locations were located in the field by use of a tape at the completion of the drilling program.

4.0 LABORATORY TESTING

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

Soil laboratory testing consisted of one (1) standard grain size analyses with natural water content and three (3) grain size analyses with hydrometer and natural water content. The results of soil laboratory tests are included as Appendix B – Laboratory Test Results. Laboratory test information is also shown on the boring logs provided in Appendix A – Boring Logs and on Sheet 3 - Boring Logs.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered in the test borings generally consisted of granular fill overlying stream alluvium and glacial till with embedded boulders and cobbles, all underlain by igneous bedrock. The boring logs are provided in Appendix A – Boring Logs and on Sheet 3 – Boring Logs. A generalized subsurface profile is shown on Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile. The following paragraphs discuss the subsurface conditions encountered in detail:

5.1 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 15.0 to 15.2 feet thick at the boring locations. The fill soils behind the south abutment consist of brown, dry to wet, sandy gravel, with trace silt and cobbles. The 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 fills ranged from 8 to 27 blows per foot (bpf) indicating that the fill soils are loose to medium dense in consistency.

One (1) grain size analysis of a sample of the fill soils resulted in the soil being classified as A-1-a under the AASHTO Soil Classification System and SW-SM under the Unified Soil Classification System (USCS). The natural water content of the sample tested was approximately 5 percent.

5.2 Stream Alluvium

Stream alluvium was encountered below the fill unit in the borings. The stream alluvium was approximately 2.5 to 2.8 feet thick at the boring locations. The deposit consisted of brown, moist, silty sand, trace gravel, with iron staining; and sand, some silt, little clay, trace gravel, with iron staining.

SPT N-values in the stream alluvium ranged from 11 to 21 bpf indicating the stream alluvium is medium dense in consistency.

One (1) grain size analysis of a sample of the stream alluvium resulted in the soil being classified as A-4 under the AASHTO Soil Classification System and SC-SM under the Unified Soil Classification System (USCS). The natural water content of the sample tested was approximately 14 percent.

5.3 Glacial Till

Underlying the fill and stream alluvium, a deposit of glacial till was encountered. The glacial till was approximately 9.6 to 13.6 feet thick at the boring locations. The glacial till consisted of olive-grey and olive-brown, moist, fine to coarse sand, some silt, little gravel, trace clay and fractured rock fragments; and dark grey, moist, silty, fine to medium sand, little coarse sand, trace gravel. Cobbles were encountered in the lower approximately 2.1 feet of the glacial till deposit, directly overlying bedrock, in boring BB-EMR-102.

SPT N-values in the glacial till ranged from 40 to 66 bpf indicating the glacial till deposit is dense to very dense in consistency.

Two (2) grain size analyses of samples from the glacial till deposit resulted in the soil being classified as A-2-4 and A-4 under the AASHTO Soil Classification System and SC-SM under the Unified Soil Classification System (USCS). The natural water contents of the samples tested were approximately 10 percent.

5.4 Bedrock

Bedrock was encountered and cored in the two (2) borings. Table 5-1 summarizes approximate depths to bedrock and corresponding approximate top of bedrock elevations.

Proposed Substructure	Boring	Station	Offset (feet)	Approximate Depth to Bedrock (feet)	Approximate Elevation of Bedrock Surface (feet)
Abutment No. 1	BB-EMR-101	6+26.8	9.9 ft Left	27.1	17.9
Abutment No. 2	BB-EMR-201	7+45.6	2.8 ft Right	31.6	13.4

Table 5-1 Summary of Approximate Bedrock Elevations

The bedrock in the borings is identified as white, medium grained, very hard, fresh, monzonite and dark grey, fine grained, very hard, fresh, diorite. The RQD of the bedrock ranged from 85 to 100% correlating to a Rock Mass Quality of good to excellent.

5.5 Groundwater

Groundwater was measured at depths ranging from approximately 9 feet to 10 feet bgs in the borings. The water levels measured upon completion of drilling are indicated on the boring logs found in Appendix A. Note that water was introduced into the boreholes during the drilling operations. Therefore, the water levels indicated on the boring logs may not represent stabilized groundwater conditions. Groundwater levels will fluctuate with changes in the water levels in the river, seasonal changes, precipitation, runoff, and construction activities.

6.0 FOUNDATION ALTERNATIVES

This project was originally scoped as a bridge replacement candidate in the MaineDOT 2006-2007 Workplan before the decision was made to defer the project to a future date. Prior to deferment, the following foundation alternatives were considered at a concept level based on the subsurface conditions encountered during the geotechnical investigation:

- **Full height abutments on spread footings founded on bedrock**. It was considered feasible that cofferdams, tremie seals and spread footings could be constructed to bear on bedrock, although with associated higher costs and constructability issues.

- **Integral abutments founded on driven H-piles**. Given the limited depth to bedrock at both abutments, it was anticipated that the piles would not achieve fixity with only the lateral support of the surrounding soils and might require bedrock sockets or special pile tips.

- **Full height abutments founded on driven pile groups.** A traditional streambed elevation pile cap founded on a pile group was not considered practical.

The Jacksonville Bridge project was programed again for replacement in the MaineDOT 2014-2016 Workplan. Foundation concepts under consideration were broadened to include semi-integral abutments supported by spread footings constructed on bedrock. Pile-supported integral abutments require less structural concrete and fewer piles, therefore pile-supported integral abutments have been judged the most cost effective option and preferred substructure type. Given the limited depth to bedrock at both abutments, the use of special pile tips will be specified to prevent pile tip displacement. The proposed use of short piles (i.e. < 12 feet) will require that the piles be designed in accordance with Technical Report ME-01-7, University of Maine, June 2005, *Behavior of Pile Supported Integral Abutments at Bridge Sites with Shallow Bedrock – Phase I.* Monolithic return (U-shaped) wingwalls will be used to retain the fill slopes.

7.0 GEOTECHNICAL DESIGN CONSIDERATIONS AND RECOMMENDATIONS

The following sections provide geotechnical design considerations and recommendations for H-pile supported integral bridge abutments, which have been selected for the substructures for the Jacksonville Bridge replacement project.

7.1 Integral Abutment H-Piles

Abutments No. 1 and No. 2 will be integral abutments founded on a single row of H-piles. The piles should be end bearing and driven to the required resistance on bedrock or within bedrock. Piles may be HP 12x53, 12x74, 14x73, 14x89, or 14x117 depending on the factored design axial loads. H-piles should be 50 ksi, Grade A572 steel. The piles should be oriented for weak axis bending. Abutment No. 1 and Abutment No. 2 piles should be fitted with Rock Injector HP-80500 Pile Points manufactured by Associated Pile and Fitting (APF), LLC, or equivalent, to protect the tips, improve penetration and improve friction at the pile tip to support pinned pile tip assumptions. Special Provision 501 Foundation Piles – Rock Injector Pile Tip is provided in Appendix D – Special Provision.

At Abutment No. 1 bedrock was encountered at approximate Elev. 17.9 feet at the proposed centerline. At Abutment No. 2 bedrock was encountered at approximate Elev. 13.4 feet at the proposed centerline. Pile lengths at the proposed abutments may be estimated based on Table 7-1 below:

Location	Approximate Bottom Elevation of Proposed Abutment (feet)	Approximate Top of Bedrock Elevation at Proposed Centerline (feet)	Estimated Pile Lengths [1] (feet)
Abutment No. 1	34.5	17.9	16.6
Abutment No. 2	34.0	13.4	20.6

Table 7-1 Estimated Pile Lengths for Integral Abutments No. 1 and No. 2

The estimated pile lengths do not take into account locations where bedrock may be deeper or shallower than that encountered in the test borings, damaged pile, or the additional five (5) feet of pile required for dynamic testing instrumentation (per ASTM D4945) or additional pile length needed to accommodate leads and driving equipment.

[1] Pile lengths do not include embedment into the pile cap.

To accommodate integral abutment piles at the proposed Abutments No. 1 and 2, the following are recommended:

- Piles supporting integral abutments should be designed in accordance with AASHTO LRFD Bridge Design Specifications, 6th Edition, 2012 (herein referred to LRFD); the design example in *Integral Abutment Bridge Design Guidelines,* VTrans Structures Section, 2008; the design example found in Appendix B of Technical Report ME-01-7, University of Maine, June 2005, *Behavior of Pile Supported Integral Abutments at Bridge Sites with Shallow Bedrock – Phase I;* and Chapter 5 of Technical Report ME-01-7

- Soil/structure analyses using LPILE® Plus 5.0 software (herein referred to as LPILE) indicate the H-pile design at both abutments do not achieve fixity. To provide a fixed or pinned boundary condition at the pile tip, the piles should be fitted with Rock Injector HP-80500 Pile Points. Special Provision 501 Foundation Piles – Rock Injector Pile Tip is provided in Appendix D – Special Provision.

- The abutment slopes must provide lateral support to the piles supporting the abutments and will require special scour countermeasures, consisting of thicker riprap, heavy riprap, concrete cable mats or toe berms to satisfy the need for foundation stability after the Q_{100} and Q_{500} flood events.

7.1.1 Strength Limit State Design

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

- compressive axial geotechnical resistance of individual piles bearing on bedrock
- drivability resistance of individual piles driven to bedrock
- structural resistance of individual piles in axial compression
- structural resistance of individual piles in combined axial loading and flexure

The pile groups should be designed to resist all lateral earth loads, vehicular loads, dead and live loads, and lateral forces transferred through the pile caps. The pile group resistance after scour due to the design flood shall provide adequate foundation resistance using the resistance factors given in this section.

Per LRFD Article 6.5.4.2, at the strength limit state, the axial resistance factor $\phi_c = 0.50$ (severe driving conditions) shall be applied to the structural compressive resistance of the pile. Since the H-piles will be subjected to lateral loading, the piles shall also be checked for resistance against combined axial compression and flexure as prescribed in LRFD Articles 6.9.2.2 and 6.15.2. This design axial load may govern the design. Per LRFD Article 6.5.4.2,

at the strength limit state, the axial resistance factor ϕ_c = 0.70 and the flexural resistance factor ϕ_f = 1.0 shall be applied to the combined axial and flexural resistance of the pile in the interaction equation (LRFD Eq. 6.9.2.2-1 or -2).

Abutment H-piles were analyzed for determination of unbraced lengths and fixity using LPILE. The estimated unbraced lengths should be used to analyze the piles in combined axial compression and flexure resistance as prescribed in LRFD Articles 6.9.2.2 and 6.15.2. Supporting calculations are provided in Appendix C – Calculations.

Structural Resistance. The nominal axial compressive structural resistance (P_n) for piles loaded in compression shall be as specified in LRFD Article 6.9.4.1. Preliminary estimates of the structural axial resistance of five (5) H-pile sections were calculated for upper and lower unbraced pile segments, and for the lower braced pile segment. The controlling resistance shown in Table 7-2 is for the lower braced pile segment, using a resistance factor, ϕ_c =0.50 for severe driving conditions. The factored structural resistances for the approximated upper unbraced segments use an axial resistance factor ϕ_c = 0.70 for combined axial and flexure are not provided in Table 7-2 because these did not govern. Supporting calculations are provided in Appendix C - Calculations. The unbraced pile lengths and effective length factors (K) assumed in these evaluations have been determined from LPILE analyses for a 14x117 pile section, which are also provided in Appendix C – Calculations.

Geotechnical Resistance. The nominal axial geotechnical resistance in the strength limit state was calculated using the guidance in LRFD Article 10.7.3.2.3 which states the nominal bearing resistance of piles driven to point bearing on hard rock shall not exceed the structural pile resistances obtained from LRFD Article 6.9.4.1 with a resistance factor ϕ_c, of 0.50, for severe driving conditions applied. The resulting, limiting factored geotechnical compressive resistances for piles driven to rock are provided below in Table 7-2 below.

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

A summary of the calculated factored axial compressive structural, geotechnical and drivability resistances of five (5) H-piles sections for each abutment for the strength limit state is provided in Table 7-2 below.

Pile Section	Strength Limit State Factored Axial Pile Resistance			
	Structural Resistance[2] ϕ_c=0.50 (kips)	Controlling Geotechnical Resistance[3] (kips)	Drivability Resistance φ_{dyn} = 0.65 (kips)	Governing Axial Pile Resistance (kips)
	Abutment No. 1			
HP 12 x 53	387	387	202	202
HP 12 x 74	545	545	345	345
HP 14 x 73	535	535	345	345
HP 14 x 89	652	652	455	455
HP 14 x 117	860	860	540 (416)[4]	540 (416)[4]
	Abutment No. 2			
	Structural Resistance[4] ϕ_c=0.50 (kips)	Controlling Geotechnical Resistance[6] (kips)	Drivability Resistance φ_{dyn} = 0.65 (kips)	Governing Axial Pile Resistance (kips)
HP 12 x 53	387	387	273	273
HP 12 x 74	545	545	390	390
HP 14 x 73	535	535	371	371
HP 14 x 89	652	652	455	455
HP 14 x 117	860	860	546 (436)[4]	546 (436)[4]

Table 7-2 Factored Axial Resistances for H-Piles for Strength Limit State

LRFD Article 10.7.3.2.3 states that the nominal axial compressive resistance of piles driven to hard rock is typically controlled by the structural resistance with a resistance factor for severe driving conditions applied. However, at both Abutments No. 1 and No. 2, the estimated factored axial pile resistances from the drivability analyses for the H-pile sections are less than the controlling factored axial structural resistance per LRFD Article 10.7.3.2.3. Therefore, drivability controls, and the recommended governing resistances for pile design are the drivability resistances provided in the rightmost column "Governing Axial Pile Resistance (kips)" in Table 7-2, above. The maximum applied factored axial pile load should not exceed the governing factored pile resistance shown in Table 7-2 above.

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[2] Structural resistances were calculated for approximated upper and lower unbraced pile segments and the lower braced pile segment. Controlling value shown here is for a braced pile segment, using a resistance factor, ϕ_c, for severe driving conditions. The factored structural resistances for the upper segments use a resistance factor of ϕ_c = 0.70 for combined axial loading and bending are not shown here but are provided in Appendix C - Calculations.

[3] Based on guidance in LRFD Article 10.7.3.2.3., *Piles Driven to Hard Rock*

[4] Drivability resistance based on an APE D19-42. Drivability resistance with a Delmag D36-32 shown in parentheses.

7.1.2 Service and Extreme Limit State Design

The design of H-piles at the service limit state shall consider tolerable transverse and longitudinal movement of the piles, overall global stability of the pile group and pile group movements/stability considering changes in soil conditions due to scour due to the design flood event. For the service limit state, resistance factors of $\phi = 1.0$ should be used in accordance with LRFD Article 10.5.5.1. The exception is the overall global stability of the foundation which should be investigated at the Service I load combination and a resistance factor, ϕ, of 0.65.

Extreme limit state design checks for the H-piles shall include pile axial compressive resistance, overall global of the pile group, pile failure by uplift in tension and structural failure. The extreme event load combinations are those related to earthquake/seismic forces, ice loads, debris loads and certain hydraulic events. Extreme limit state design shall also check that the nominal pile foundation resistance remaining after scour due to the check flood can support the extreme limit state loads. Resistance factors for extreme limits state, per LRFD Article 10.5.5.3 shall be taken as $\phi = 1.0$, with the exception of uplift of piles, for which the resistance factor, φ_{up}, shall be 0.80 or less per LRFD Article 10.5.5.3.2.

Preliminary scours depths calculated for the 100-year flood event are 14.9 feet below the streambed at Abutment No. 1 and 13.4 feet below the streambed at Abutment No. 2.[5] These potential scour depths will expose bedrock at Abutment No. 1 and leave approximately 4 feet of soil overlying bedrock at Abutment No. 2, the consequence of this potential scour being that the abutment pile groups will be unstable. Abutment slopes will therefore require special scour countermeasures; (refer to Section 7.6 Scour and Riprap).

The nominal axial geotechnical piles resistance in the service and extreme limit state was calculated using the guidance in LRFD Article 10.7.3.2.3. The calculated factored axial structural, geotechnical and drivability resistances of five (5) H-pile sections for the service and extreme limit states are provided in Table 7-3. Supporting documentation is provided in Appendix C – Calculations.

[5] Preliminary Design Report, June 30, 2014, Appendix F – Hydrology/Hydraulics Report.

Pile Section	Service and Extreme Limit State Factored Axial Pile Resistance			
	Structural Resistance (normal conditions)[6] ϕ_c=1.0 (kips)	Controlling Geotechnical Resistance ϕ_c=1.0[7] (kips)	Drivability Resistance φ = 1.0 (kips)	Governing Axial Pile Resistance (kips)
	Abutment No. 1			
HP 12 x 53	649	775	310	310
HP 12 x 74	919	1090	530	530
HP 14 x 73	950	1070	530	530
HP 14 x 89	1161	1305	700	700
HP 14 x 117	1537	1720	830 (640)[8]	830 (640)[8]
	Abutment No. 2			
Pile Section	Structural Resistance (normal conditions)[6] ϕ_c=1.0 (kips)	Controlling Geotechnical Resistance ϕ_c=1.0[7] (kips)	Drivability Resistance φ= 1.0 (kips)	Governing Axial Pile Resistance (kips)
HP 12 x 53	630	775	420	420
HP 12 x 74	893	1090	600	600
HP 14 x 73	931	1070	570	570
HP 14 x 89	1139	1305	700	700
HP 14 x 117	1508	1720	840 (670)[8]	840 (670)[8]

Table 7-3 Factored Axial Resistances for H-Piles for Service and Extreme Limit States

LRFD Article 10.7.3.2.3 states that the nominal axial compressive resistance of piles driven to hard rock is typically controlled by the structural resistance with a resistance factor for severe driving conditions applied. However, for the site conditions at Abutments No. 1 and No. 2, the estimated factored axial pile resistances from the drivability analyses for the H-pile sections are less than the controlling factored axial structural resistance per LRFD Article 10.7.3.2.3 and the nominal structural resistances. Therefore, drivability controls, and the recommended governing resistances for pile design are the resistances provided in the rightmost column "Governing Axial Pile Resistance (kips)" in Table 7-3, above. The

[6] Normal conditions consider no soil loss due to scour. Nominal structural resistances were calculated for approximated upper and lower unbraced pile segments and the lower braced pile segment. Controlling value shown here is for the lower unbraced pile segment, using a resistance factor, ϕ = 1.0.
[7] Based on guidance in LRFD Article 10.7.3.2.3., *Piles Driven to Hard Rock.*
[8] Drivability resistance based on an APE D19-42. Drivability resistance with a Delmag D36-32 shown in parentheses.

maximum applied factored axial pile load for the extreme and service limits states should not exceed the governing factored pile resistance shown in Table 7-3 above.

7.1.3 Lateral Pile Resistance/Behavior

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

A series of lateral pile resistance analyses were performed to evaluate pile behavior at both abutments using LPILE software with pile head deflections, moments and axial loads supplied by the structural engineer. The results of the LPILE analyses for a 14x117 section (the pile section selected for final design) were provided to the structural designer to evaluate the associated pile stresses, bending moments and fixity due to factored pile head loads and displacements.

Geotechnical parameters used for generation of soil-resistance (p-y) curves in lateral pile analyses are provided in Tables 7-4 and 7-5 below. In general, the models developed emulate the soil at the site by using the soil layers (referenced in Tables 7-4 and 7-5 by elevations) and using appropriate structural parameters and pile-head boundary conditions for the pile section being analyzed. The results of the LPILE analyses for the Abutment No. 1 and Abutment No. 2 pile section selected for final design are provided in Appendix C – Calculations.

Soil Layer	Approx. Elevation of Soil Layer (feet)	Water Table Condition	Effective Unit Weight lbs/in^3 (lbs/ft^3)	k_s (lb/in^3)	Internal Angle of Friction
Loose, sandy GRAVEL, (Fill)	36 – 30	Below	0.033 (57.6)	20	30°
Medium dense SAND, (Alluvium)	30 – 27.5	Below	0.036 (62.6)	60	32°
Very dense, SAND, some silt, (Glacial Till)	27.5 – 17.9	Below	0.039 (67.6)	120	34°

Table 7-4 Soil Parameters for Generation of Soil-Resistance (p-y) Curves at Abutment No. 1

Soil Layer	Approx. Elevation of Soil Layer (feet)	Water Table Condition	Effective Unit Weight lbs/in^3 (lbs/ft^3)	k_s (lb/in^3)	Internal Angle of Friction
Medium dense, SAND, some gravel, trace silt, (Fill).	34 – 29.8	Below	0.036 (62.6)	50	30°
Medium dense, SAND, some silt, trace clay & gravel, (Alluvium)	29.8 - 27	Below	0.036 (62.6)	60	32°
Dense to very dense, silty SAND, trace gravel, (Glacial Till).	27 – 13.4	Below	0.039 (67.6)	120	34°

Table 7-5 Soil Parameters for Generation of Soil-Resistance (p-y) Curves at Abutment No. 2

7.1.4 Driven Pile Resistance and Pile Quality Control

The contract plans shall require the contractor to perform a wave equation analysis of the proposed pile-hammer system and a dynamic pile test with signal matching at each abutment. The first pile driven at each abutment should be dynamically tested to confirm nominal pile resistance and verify the stopping criteria developed by the contractor in the wave equation analysis. Restrikes or additional dynamic tests may be required as part of the pile field quality control program should pile behavior vary radically between adjacent piles, should pile behavior indicates a pile is refusing on a boulder or in a cobble layer above bedrock, should the special pile tip be not firmly embedded in bedrock, or if the pile "walk" out of position.

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

Piles should be driven to an acceptable penetration resistance as determined by the contractor based on the results of a wave equation analysis and as approved by the Resident. Driving stresses in the pile determined in the drivability analysis shall be less than 45 ksi, in accordance with LRFD Article 10.7.8. A hammer should be selected which provides the required pile resistance when the penetration resistance for the final 3 to 6 inches is 3 to 15 blows per inch (bpi). If an abrupt increase in driving resistance is encountered, the driving could be terminated when the penetration is less than 0.5-inch in 10 consecutive blows.

7.2 Integral Abutment Design

Integral abutment sections shall be designed for all relevant strength, service and extreme limit states and load combinations specified in LRFD Articles 3.4.1 and 11.5.5. Stub abutments shall be designed to resist all lateral earth loads, vehicular loads, dead and live loads, and lateral forces transferred through the integral superstructure. The design of the integral abutment at the strength limit state shall consider reinforced-concrete structural design. Strength limit state design shall also consider changes in foundation conditions and foundation resistance after scour due to the design (Q_{100}) flood.

A resistance factor (ϕ) of 1.0 shall be used to assess abutment design at the service limit state, including: settlement, excessive horizontal movement, and movement resulting after scour due to the design (Q_{100}) flood. The overall stability of the foundation should be investigated at the Service I Load Combination and a resistance factor, ϕ, of 0.65.

Extreme limit state design of integral abutment supported on H-piles shall include pile structural resistance, pile geotechnical resistance, pile resistance in combined axial and flexure, and overall stability. Resistance factors for extreme limit state shall be taken as 1.0. Extreme limit state design shall also check that the nominal foundation resistance remaining after scour due to the check (Q_{500}) flood can support the extreme limit state loads with a resistance factor of 1.0.

The designer may assume Soil Type 4 (MaineDOT Bridge Design Guide (BDG) Section 3.6.1) for abutment backfill material soil properties. The backfill properties are as follows: $\phi = 32°$, $\gamma = 125$ pcf and a soil-concrete interface friction angle of 20 degrees.

Integral abutment sections shall be designed to withstand a lateral earth load equal to the passive pressure state. Calculation of passive earth pressures should assume a Coulomb passive earth pressure coefficient, K_p, of 6.73. Developing full passive pressure assumes that the ratio of lateral abutment movement to abutment height (y/H) exceeds 0.005. If the calculated displacements are significantly less than that required to develop full passive pressure the designer may consider using the Rankine passive earth pressure coefficient of 3.25. A load factor for passive earth pressure is not specified in LRFD. For purposes of the integral abutment backwall reinforcing steel design, use a maximum load factor (γ_{EH}) of 1.50 to calculate factored passive earth pressures.

Additional lateral earth pressure due to live load surcharge is required per Section 3.6.8 of the MaineDOT BDG for abutments if an approach slab is not specified. When a structural approach slab is specified, reduction, not elimination of the surcharge load is permitted per LRFD Article 3.11.6.5. The live load surcharge may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) taken from Table 7-6 below:

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

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

The abutment design shall include a drainage system behind the abutment to intercept any groundwater. We recommend weep holes be constructed approximately 6 inches above $Q_{1.1}$ (normal high water). Drainage behind the structure shall be in accordance with MaineDOT BDG Section 5.4.1.4.

Backfill within 10 feet of the abutments 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.

Slopes in front of the pile supported integral abutments should be set back from the riverbank and should be constructed with riprap and erosion control geotextile. The slopes should not exceed 1.75H:1V unless project specific slope stability analyses are performed.

7.3 Return (U-Shaped) Wingwalls

U-shaped return wingwalls will be used in conjunction with the integral abutments. The wingwalls will be approximately 10 feet long and be constructed monolithically with the abutments. Monolithic U-shaped wingwalls that are not cantilevered may be designed to be pile supported. A chamfer, typically 1 foot, should be used between the abutment and the wingwalls to minimize concrete shrinkage cracking caused by the abrupt change in thickness at the connection. The wingwalls are essentially retaining walls and shall be designed for all relevant strength, service limit and extreme states and load combinations specified in LRFD Articles 3.4.1, 11.5.5 and 11.6. The walls shall be designed to resist lateral earth pressures, vehicular loads, collision loads, and creep and temperature and shrinkage deformations. The wingwalls shall be designed considering a live load surcharge equal to a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) per LRFD Article 3.11.6.4.

For design of the wingwalls, two (2) load cases shall be considered. The first load case is where the wingwall is subjected to passive earth pressure to account for the bridge moving laterally and pushing the wingwall into the fill. This load case is considered under strength limit state. Calculation of passive earth pressures may assume a Rankine passive earth pressure coefficient, K_p, of 3.25 assuming small wingwall movements. A load factor for passive earth pressure is not specified in LRFD; use a maximum load factor (γ_{EH}) of 1.50 to calculate factored passive earth pressures.

The second load case considers that the wingwall is subjected to active pressure and to collision loads on wall mounted bridge rail. This load case is considered under the extreme limit state. Calculation of active earth pressure shall use the Rankine active earth pressure coefficient, K_a, of 0.33 assuming a level backslope. See Appendix C – Calculations for supporting documentation.

There are no bearing resistance considerations for U-shaped wingwalls that are slope-tapered and designed as cantilevered walls. However, the wall sections shall be embedded a minimum of 5.7 feet for frost protection as discussed in Section 7.6, below. Alternatively, U-shaped wingwalls constructed monolithically with the abutments may be designed to be pile supported. Refer the Sections 7.1.1. and 7.1.2. for pile design recommendations.

7.4 Settlement

The fill soils, alluvium and glacial till deposits encountered at the site are loose to very dense in consistency. These cohesionless materials undergo elastic compression where a load greater than the existing overburden pressure is being applied. A raise in grade up to 2 feet is planned. Elastic settlements due to the proposed raise in grade are estimated to range from approximately ½ to 1 inch and are anticipated to occur relatively quickly during construction. Post construction settlement will be minimal.

Any settlement of the bridge abutments will be due to axial compression of the foundation piles and is anticipated to be less than 0.5 inch.

7.5 Frost Protection

Pile-supported integral abutments shall be embedded a minimum of 4.0 feet for frost protection per Figure 5-2 of the MaineDOT BDG.

Foundations placed on fill side slopes should be designed with an appropriate embedment for frost protection. According to BDG Figure 5-1, Maine Design Freezing Index Map, East Machias has a design freezing index of approximately 1150 F-degree days. An assumed water content of 10% was used for coarse grained soils. These components correlate to a frost depth of approximately 6.0 feet. A similar analysis was performed using Modberg software by the US Army Cold Regions Research and Engineering Laboratory (CRREL). For the Modberg analysis, East Machias was assigned a design freezing index of approximately 1102 F-degree days, for Eastport, the closest location in the Modberg database. An assumed water content of 10% was used for coarse grained fill soils above the water table. These components correlate to a frost depth of approximately 5.3 feet. Based on an average of these results, we recommend foundations be designed with an embedment of approximately 5.7 feet for frost protection. See Appendix C – Calculations for supporting documentation.

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

7.6 Scour and Riprap

Grain size analyses were performed a soil sample of alluvium taken at the approximate streambed elevation to generate grain size curves for determining parameters to be used in scour analyses. The sample was assumed to be similar in nature to the soils likely to be exposed to scour conditions. The following streambed grain size parameters can be used in scour analyses:

- Average diameter of particle at 50 percent passing, D_{50} = 0.10 mm (fine sand)
- Average diameter of particle at 95 percent passing, D_{95} = 3.7 mm (coarse sand)
- Soil Classification AASHTO Soil Type A-4.

The grain size curves are included in Appendix B – Laboratory Test Results.

The consequences of changes in foundation conditions resulting from the design (Q_{100}) and check (Q_{500}) floods for scour shall be considered at the strength and extreme limit states, respectively. Design at the strength limit state should consider loss of lateral and vertical support due to scour. Design at the extreme limit state should check that the nominal foundation resistance due to the check flood (Q_{500}) event is no less that the extreme limit state loads. At the service limit state, the design shall limit movements and overall stability considering scour at the design load.

Preliminary scour depths for the design flood (Q_{100}) event indicate scour will expose bedrock at Abutment No. 1 and removal all but 4 feet of soil overlying bedrock at Abutment No. 2[9], the consequence of this potential scour being that the abutment pile groups will become unstable. The slope at the abutments will require special scour countermeasures, consisting of thicker riprap, heavy riprap, concrete cable mats or toe berms, to satisfy the need for foundation stability during the design and check floods. The final type and size of riprap, or other scour countermeasures, shall be evaluated and detailed in final design.

For scour protection of the pile supported abutments, the PDR indicates the bridge approach slopes will be armored with plain riprap and the abutment slopes will be armored with a minimum of 4.0 feet of heavy riprap. Refer to MaineDOT BDG Section 2.3.11. for information regarding scour design. Typically the top of the riprap is located at the Q_{50} elevation. For this project, the top of the riprap will be located at a minimum elevation of Q_{100}

Plain and heavy riprap shall conform to item numbers 703.26 and 703.28, respectively, of Supplemental Specification Section 703 and shall be placed at a maximum slope of 1.75H:1V. The toe of the riprap section shall be constructed 1 foot below the streambed elevation. The riprap section shall be underlain by a 1 foot thick layer of bedding material conforming to item number 703.19 of the Standard Specification and Class 1 nonwoven erosion control geotextile per Standard Details 610(02) through 610(04).

[9] Preliminary Design Report, June 30, 2014, Appendix F – Hydrology/Hydraulics Report.

7.7 Seismic Design Considerations

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) = 0.077g
- Site Class D (based on an average N-value for the upper 100 ft of the soil profile greater than 15 bpf and less than 50 bpf, using steps in LRFD Table C3.10.3.1-1)
- Acceleration coefficient (As) = 0.123g
- Design spectral acceleration coefficient at 0.2-second period (S_{DS}) = 0.242g
- Design spectral acceleration coefficient at 1.0-second period, (S_{D1})= 0.095g
- Seismic Zone 1, based on a $S_{D1} \leq 0.15$g

See Appendix C – Calculations for supporting documentation.

In conformance with LRFD 5.10.11.2, no consideration for seismic forces is required except that superstructure connections and minimum support length requirements shall be satisfied per LRFD Articles 3.10.9.2 and 4.7.4.4, respectively.

7.8 Construction Considerations

The new integral abutments will be constructed behind the existing abutments, therefore avoiding constructing fills or cofferdams in the river. The existing abutments will be removed to 1 to 2 feet above the $Q_{1.1}$ elevation. Construction of the abutments and will require pile driving. Temporary lateral earth support systems may be required to permit construction of driven pile foundations at the proposed abutments.

Based on recommendations made in the preceding sections, a special provision is provided in Appendix D for Rock Injector pile tips to provide fixity or a pinned condition. This item may also require detailing on the construction plans and require construction notes. We can assist in preparation of the construction plans and notes.

Exposed soils may become saturated and water seepage may be encountered during construction. There may be localized sloughing and instability in some excavations and cut slopes. The contractor should control groundwater, surface water infiltration and soil erosion. Water should be controlled by pumping from sumps.

Cobbles were encountered in the fill soils underlying the bridge approaches and in the glacial till deposit at the location of the proposed Abutment No. 2. Cobbles may likely be encountered in the glacial till deposit at proposed Abutment No. 1. There is potential for these obstructions to impact construction activities. These impacts include but are not limited to impeding the driving of sheet piles for temporary earth support systems and driving H-piles for abutment foundations. Obstructions may be cleared by conventional excavation methods, pre-augering, predrilling, spudding, use of rock chisels, or down-hole

hammers. Alternative methods to clear obstructions may be used as approved by the Resident. Care should take to drive piles within allowable tolerances.

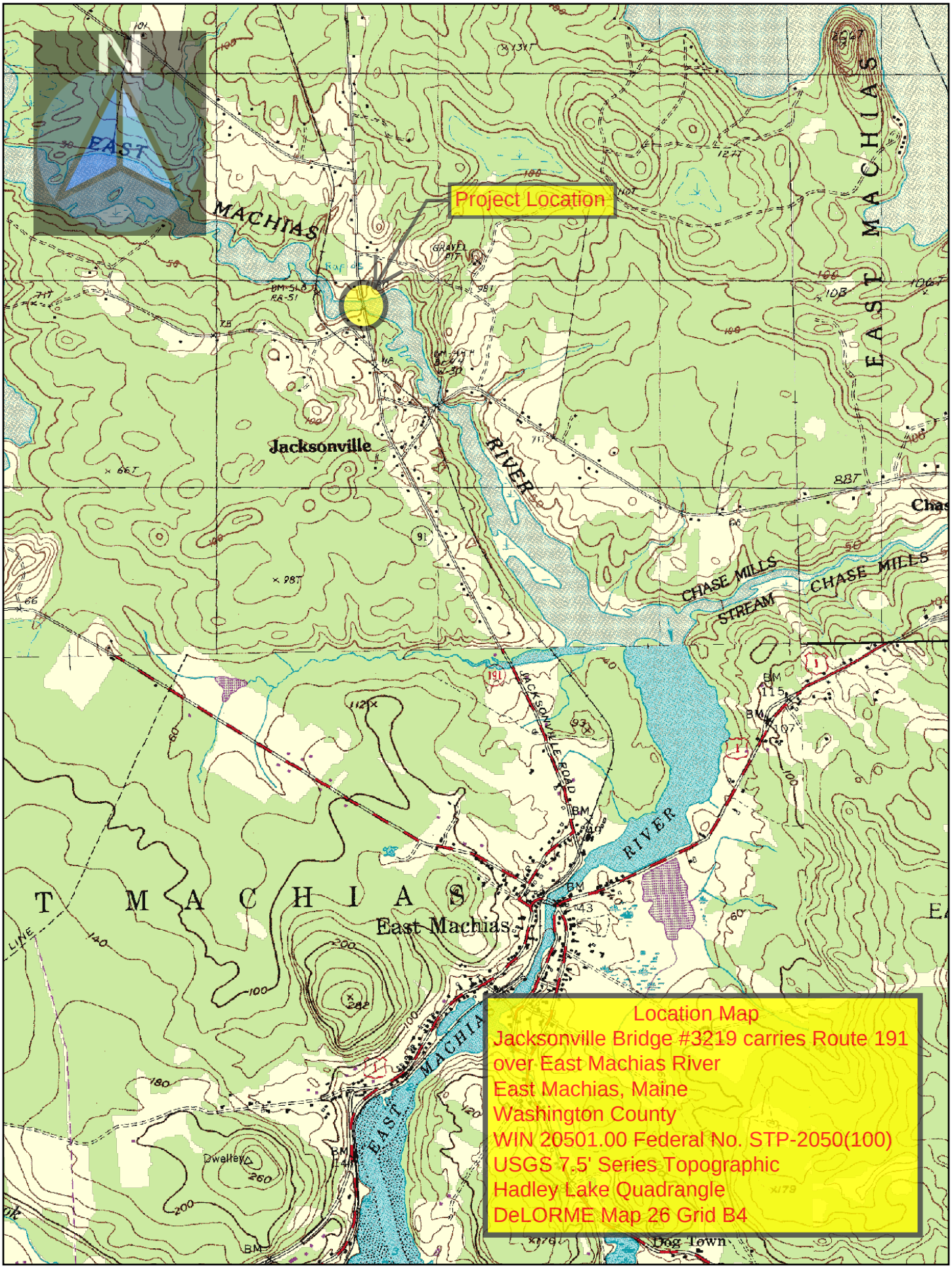
8.0 CLOSURE

This report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed replacement of Jacksonville Bridge in East Machias, Maine in accordance with generally accepted geotechnical 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 at discrete locations completed at the site. If variations from the conditions encountered during the investigation appear evident during construction, it may also become necessary to re-evaluate the recommendations made in this report.

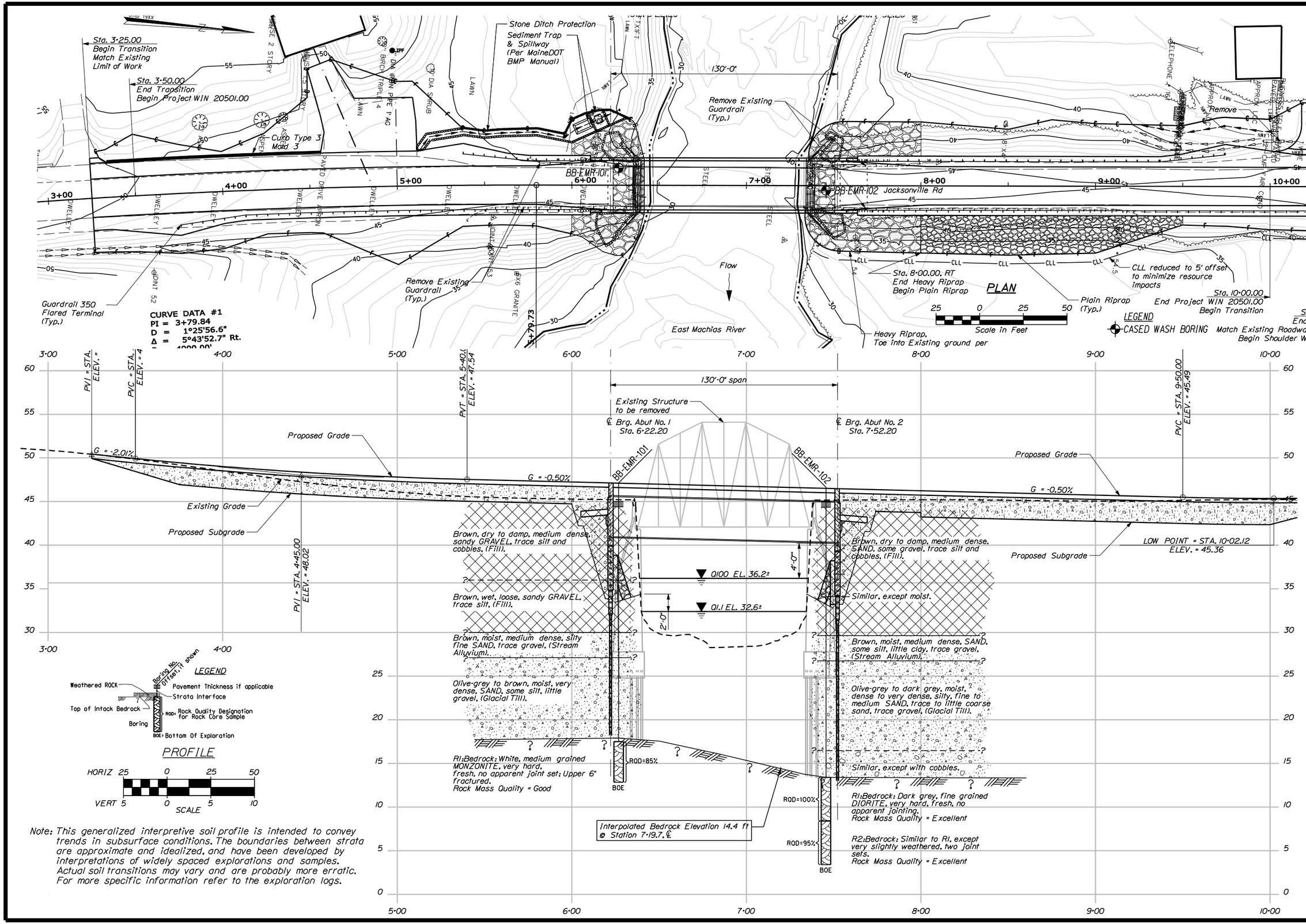
We also recommend that we be provided the opportunity for a general review of the final design and specifications in order that the earthwork and foundation recommendations may be properly interpreted and implemented in the design.

Sheets



Map Scale 1:24000

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



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

STP-2050(100)

PIN 20501.00

BRIDGE NO. 3219

BRIDGE PLANS

	BY	DATE
PROJ. MANAGER	D. ANDERSON	
DESIGN-DETAILED	L. KRUSINSKI	T. WHITE	APR 2014
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 3

Filename: ...\geotech\msto\006_BLP&ISP1.dgn Division: GEOTECH Username: terry.white Date:10/7/2014

Filename: ...\msta\007_BORING LOGS1.dgn Division: GEOTECH Username: terry.white Date:10/7/2014

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
WIN: 20501.00

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:	6+26.8, 9.9 ft Lt.	Casing ID/OD:	HW	Water Level*:	9.0' bgs.

Definitions:
D = Split Spoon Sample S = Sample off Auger Flight
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 WOC = weight of casing

Definitions:
WC = water content, percent ≥ = Similar or Equal too
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.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0						SSA	44.40		7.2" PAVEMENT. 0.60	
									Brown, dry, sandy GRAVEL, trace silt and cobbles. (Fill).	
5	1D	24/9	5.00 - 7.00	9/12/12/13	24	38			Brown, dry to damp, medium dense, sandy GRAVEL, trace silt. (Fill).	
						35				
						30				
						31				
						22	36.00		Brown, wet, loose, sandy GRAVEL, trace silt. (Fill). 9.00	
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 gravel, some iron staining. (Stream 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 gravel, trace clay. (Glacial Till).	GM182502 A-2-4, SC-SM WC=9.5%
						135				
						172				
						288				
						OPEN HOLE			Washed ahead from 24.0-25.0 ft bgs.	
25	5D	18/11	25.00 - 26.50	31/62/55	117				Similar to 4D, except olive-brown.	GM182504 A-4, SC-SM WC=9.7%
									Washed ahead from 25.0-27.5 ft bgs.	
	R1	56/54	27.50 - 32.17	RQD = 85%		WOR ND	11.90		27.10	
							13.50		Top of Bedrock at Elev. 17.9 ft. *Roller coned into bedrock from 27.1 to 27.5' bgs. 27.50	
									R1: Bedrock: White, medium grained, MONZONITE, very hard, fresh, no joint set evident. Upper 6-inches fractured and surfaces oxidized. Rock Mass Quality = Good. Core Times (min:sec) 27.5-28.5 ft (5:09) 28.5-29.5 ft (5:48) 29.5-30.5 ft (4:29) 30.5-31.5 ft (5:02) 31.5-32.17 ft (31:54) 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
WIN: 20501.00

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:	7+45.6, 2.8 ft Rt.	Casing ID/OD:	HW	Water Level*:	10.0' bgs.

Definitions:
D = Split Spoon Sample S = Sample off Auger Flight
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 WOC = weight of casing

Definitions:
WC = water content, percent ≥ = Similar or Equal too
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.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0						SSA	44.35		7.8" PAVEMENT. 0.65	
									Brown, dry, fine to coarse SAND, some gravel, trace silt, trace cobbles (Fill).	
5	1D	24/9	5.00 - 7.00	9/14/13/12	27	42			Brown, dry to damp, medium dense, fine to coarse SAND, some gravel, trace silt. (Fill).	GM182501 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			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		Brown, moist, medium dense, SAND, some silt, little clay, trace gravel, some iron staining. (Stream Alluvium). 15.20	GM182503 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 gravel. (Glacial Till).	
						20			Washed ahead from 20.0-25.0 ft bgs.	
						19				
						16				
						50				
25	5D	24/4	25.00 - 27.00	24/23/17/14	40	76	20.00		Dark, moist, dense, silty fine to medium SAND, little coarse sand, trace gravel, fractured rock fragments. (Glacial Till). 25.00	
						78				
						62				
						117	16.50		Similar to above, with cobbles. (Glacial Till). 28.50	
						310				
30	MD	1/0	30.00 - 30.08	53(1")	---	178			Washed ahead from 30.0-31.6 ft bgs.	
	R1	60/60	31.60 - 36.60	RQD = 100%		9100 ND	13.40		Q100 blows for 0.6 ft.	
									Top of Bedrock at Elev. 13.4 ft. R1:Bedrock: Dark grey, fine grained DIORITE, very hard, fresh, no apparent joint set. Rock Mass Quality = Excellent. R1:Core Times (min:sec) 31.6-32.6 ft (6:48) 32.6-33.6 ft (4:45) 33.6-34.6 ft (5:09) 34.6-35.6 ft (4:32) 35.6-36.6 ft (4:47) 100% Recovery	31.60
35										
	R2	60/58	36.60 - 41.60	RQD = 95%					R2: Similar to R1, except very slighty weathered, two joint sets, chlorite on joint surfaces. Rock Mass Quality = Excellent. Core Times (min:sec) 36.6-37.6 ft (3:33) 37.6-38.6 ft (3:20) 38.6-39.6 ft (3:29) 39.6-40.6 ft (3:16) 40.6-41.6 ft (3:14) 98% Recovery	
40							3.40		Bottom of Exploration at 41.60 feet below ground surface. 41.60	
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-102

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
STP-2050(100)
PIN 20501.00
BRIDGE NO. 3219
BRIDGE PLANS

		BY	DATE
PROJ. MANAGER	D. ANDERSON		
DESIGN-DETAILED	L. KRUSINSKI	T. WHITE	OCT 2014
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 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

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 thumbnai
Hard	>30	over 4000	Indented by thumbnail with difficulty

Rock Quality Designation (RQD):

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

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

Correlation of RQD to Rock Mass Quality

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

Desired Rock Observations: (in this order)

Color (Munsell color chart)
Texture (aphanitic, fine-grained, etc.)
Lithology (igneous, sedimentary, metamorphic, etc.)
Hardness (very hard, hard, mod. hard, etc.)
Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.)
Geologic discontinuities/jointing:
- -dip (horiz - 0-5, low angle - 5-35, mod. dipping - 35-55, steep - 55-85, vertical - 85-90)
- -spacing (very close - <5 cm, close - 5-30 cm, mod. close 30-100 cm, wide - 1-3 m, very wide >3 m)
- -tightness (tight, open or healed)
- -infilling (grain size, color, etc.)

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

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

Sample Container Labeling Requirements:

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

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: 20501.00

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:	6+26.8, 9.9 ft 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 WOC = weight of casing

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	7.2" PAVEMENT. 0.60	
								Brown, dry, sandy GRAVEL, trace silt and cobbles, (Fill).	
								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			
15	3D	24/6	15.00 - 17.00	7/5/6/4	11	15	30.00	15.00	
								Brown, moist, medium dense, silty fine SAND, trace gravel, some iron staining, (Stream 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 gravel, trace clay, (Glacial Till).	G#182502 A-2-4, SC-SM WC=9.5%
						135			
						172			
						288			
25						OPEN		Washed ahead from 24.0-25.0 ft 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: 20501.00

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:	6+26.8, 9.9 ft 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 WOC = weight of casing

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.)	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.	G#182504 A-4, SC-SM WC=9.7%
									Washed ahead from 25.0-27.5 ft bgs.	
							17.90		27.10	
	R1	56/54	27.50 - 32.17	RQD = 85%		~~aRC~~ NQ	17.50		Top of Bedrock at Elev. 17.9 ft. [a]Roller coned into bedrock from 27.1 to 27.5' bgs.	
									27.50 R1: Bedrock: White, medium grained, MONZONITE, very hard, fresh, no joint set evident. Upper 6-inches fractured and surfaces oxidized. Rock Mass Quality = Good. Core Times (min:sec) 27.5-28.5 ft (5:09) 28.5-29.5 ft (5:48) 29.5-30.5 ft (4:29) 30.5-31.5 ft (5:02) 31.5-32.17 ft (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: 20501.00

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:	7+45.6, 2.8 ft 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 WOC = weight of casing

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	7.8" PAVEMENT. 0.65	
								Brown, dry, fine to coarse SAND, some gravel, trace silt, trace cobbles (Fill).	
5	1D	24/9	5.00 - 7.00	9/14/13/12	27	42		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		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 gravel, some iron staining, (Stream Alluvium).	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 gravel, (Glacial Till).	
						20		Washed ahead from 20.0-25.0 ft bgs.	
						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.

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: 20501.00

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:	7+45.6, 2.8 ft 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 WOC = weight of casing

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.
25	5D	24/4	25.00 - 27.00	24/23/17/14	40	76	20.00	Dark grey, moist, dense, silty fine to medium SAND, little coarse sand, trace gravel, fractured rock fragments, (Glacial Till).	
						78			
						62			
						117	16.50	Similar to above, with cobbles, (Glacial Till).	
						310			
30	MD	1/0	30.00 - 30.08	53(1")	---	178		Washed ahead from 30.0-31.6 ft bgs.	
	R1	60/60	31.60 - 36.60	RQD = 100%		[a]100 NQ	13.40	[a]100 blows for 0.6 ft.	
								Top of Bedrock at Elev. 13.4 ft. R1:Bedrock: Dark grey, fine grained DIORITE, very hard, fresh, no apparent joint set. Rock Mass Quality = Excellent. R1:Core Times (min:sec) 31.6-32.6 ft (6:48) 32.6-33.6 ft (4:45) 33.6-34.6 ft (5:09) 34.6-35.6 ft (4:32) 35.6-36.6 ft (4:47) 100% Recovery	
35	R2	60/58	36.60 - 41.60	RQD = 95%				R2: Similar to R1, except very slighty weathered, two joint sets, chlorite on joint surfaces. Rock Mass Quality = Excellent. Core Times (min:sec) 36.6-37.6 ft (3:33) 37.6-38.6 ft (3:20) 38.6-39.6 ft (3:29) 39.6-40.6 ft (3:16) 40.6-41.6 ft (3:14) 98% Recovery	
40							3.40	**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.

Appendix B

Laboratory Test Results

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): East Machias Project Number: 20501.00

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	6+26.8	9.9 Lt.	20.0-22.0	182502	1	9.5			SC-SM	A-2-4	III
BB-EMR-101, 5D	6+26.8	9.9 Lt.	25.0-26.5	182504	1	9.7			SC-SM	A-4	III
BB-EMR-102, 1D	7+45.6	2.8 Rt.	5.0-7.0	182501	1	5.0			SW-SM	A-1-a	0
BB-EMR-102, 3D	7+45.6	2.8 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.

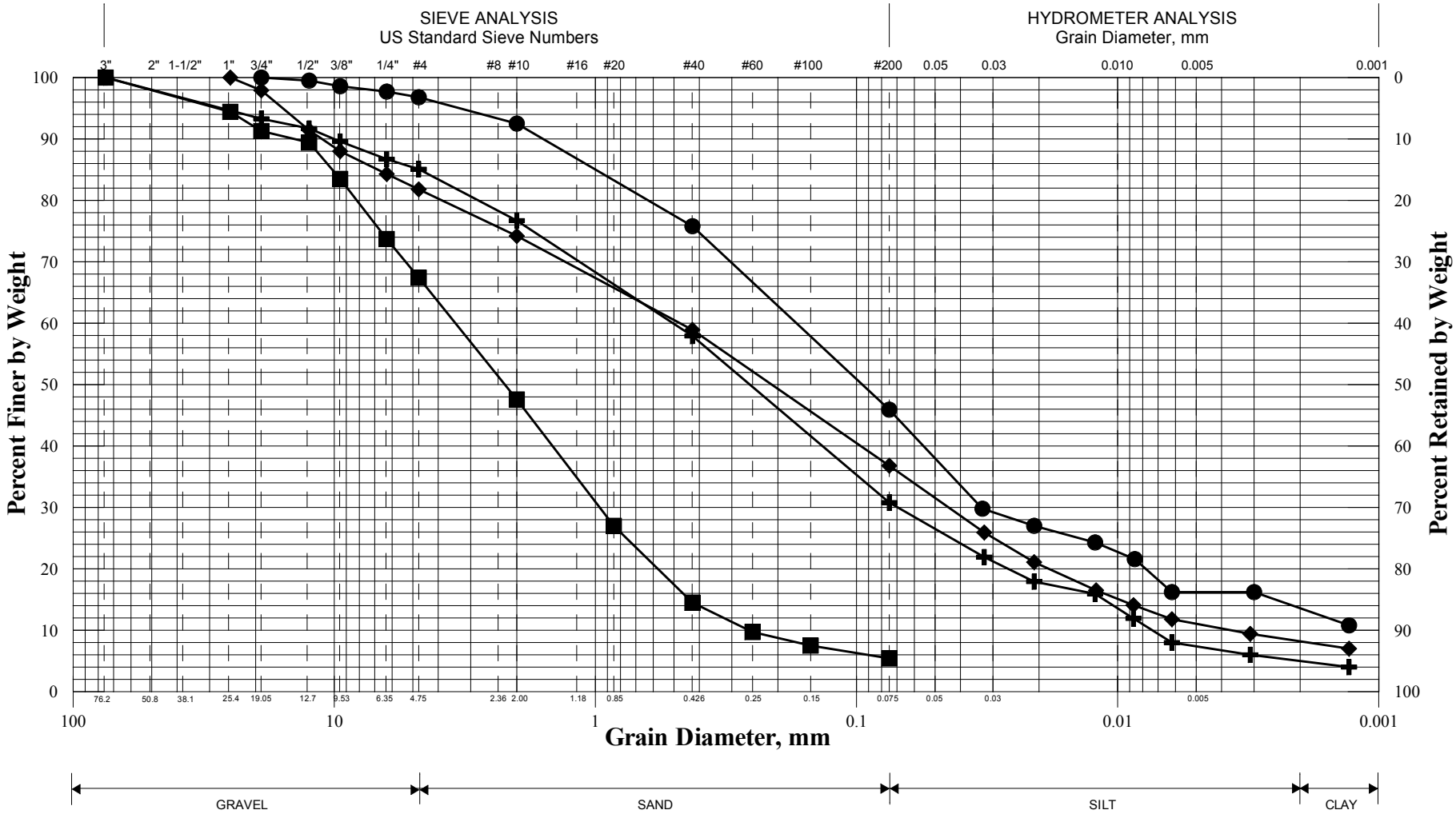
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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: 20501.00
Town: East Machias
Reported by: T. White
Date: 11/01/04

SHEET NO.1

Appendix C

Calculations

Abutment No. 1 H-Piles

Design of H-piles

Reference: AASHTO LRFD Bridge Design Specifications, 6th Edition, 2012

Bedrock Properties at Abutment 1

BB-EMR-101, R1=85%
Rock Type: Monzonite (granite; igneous), very hard, fresh.

ϕ = 34-40 (AASHTO LRFD Table C.10.4.6.4-1);
Co = 2,100 - 49,000 psi (AASHTO Standard Specifications for Bridges 17th Edition, Table 4.4.8.1.2B)

For Design Purposes, use bedrock data from BB-EMR-101: RQD = 85% and an Unconfined Compressive Strength of 15,000 psi.

Pile Properties

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

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

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

Pile yield strength $F_y := 50 \cdot ksi$

1. Nominal and Factored Structural Compressive Resistance of H-piles

Use LRFD Equation 6.9.2.1-1 Pr = ϕ Pn

Nominal Axial Structural Resistance

Determine equivalent yield resistance Po=QFyAs (LRFD 6.9.4.1.1)

$Q := 1.0$ LRFD Article 6.9.4.2

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

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

Only the extreme tip of the lower portion of the Abutment 1 pile is in pure axial compression per Lpile Analysis for a 14x117 pile (selected for final design). The top ubraced pile segment is 2.58 ft per Bending Moment vs Depth curve, with a K of 2.1 since a plastic hinge develops. The second unbraced length is 11.75 ft per the Moment vs Depth curves with a K of 1.0 where the second segment is pinned at the top and bottom. Use structural resistance factors of 0.7 for both segments since the factored resistances are being computed for pile segments in combined axial and flexure.

A. Structural Resistance of upper unbraced segment of pile

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

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

K = effective length factor $K_{eff} := 2.1$ LRFD Table C4.6.2.5-1 (plastic hinge develops, K selected per VTrans Design Procedure)

l = unbraced length $l_{unbraced_top} := 2.58 \cdot ft$

r s = radius of gyration

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

radius of gyration about the Y-Y or weak axis per LRFD Article C6.9.4.1.2.

LRFD eq. 6.9.4.1.2-1

$$P_e := \left[\frac{\pi^2 \cdot E}{\left(\frac{K_{eff} \cdot l_{unbraced_top}}{r_s} \right)^2} \cdot A_s \right] \qquad P_e = \begin{pmatrix} 8585 \\ 12586 \\ 17649 \\ 22022 \\ 30020 \end{pmatrix} \cdot kip$$

LRFD Article 6.9.4.1.1

$$\frac{P_e}{P_o} = \begin{pmatrix} 11.077 \\ 11.547 \\ 16.494 \\ 16.875 \\ 17.453 \end{pmatrix}$$

If Pe/Po > or = 0.44, then:

$$P_n := \overrightarrow{\begin{pmatrix} \frac{P_o}{P_e} \\ 0.658^{\frac{P_o}{P_e}} \cdot P_o \end{pmatrix}}$$

LRFD Eq. 6.9.4.1.1-1

then:

$$\boxed{P_n = \begin{pmatrix} 746 \\ 1051 \\ 1043 \\ 1273 \\ 1679 \end{pmatrix} \cdot kip}$$

this applies to all pile sizes

Factored Axial Structural Resistance at the Strength Limit State

Resistance factor for upper unbraced segements of H-pile in combined compression and flexure per LRFD 6.5.4.2:

$$\phi_{cu} := 0.7$$

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

$$P_r := \phi_{cu} \cdot P_n$$

Factored structural compressive resistance, Pr

this applies to all pile sizes

$$\boxed{P_r = \begin{pmatrix} 522 \\ 736 \\ 730 \\ 891 \\ 1175 \end{pmatrix} \cdot kip}$$

B. Structural Resistance of lower unbraced segment of pile

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

K = effective length factor $K_{eff} := 1.0$

Per VTtrans Integral Abutment Design Guideline, use K=1.0 for segment pinned at the top and bottom as shown by LPile analysis for 2nd unbraced segment (see also LRFD Table C4.6.2.5-1)

l = unbraced length $l_{unbraced_bot} := 11.75 \cdot ft$ Ref : LPile

LRFD eq. 6.9.4.1.2-1

$$P_e := \overrightarrow{\left[\frac{\pi^2 \cdot E}{\left(\frac{K_{eff} \cdot l_{unbraced_bot}}{r_s}\right)^2} \cdot A_s\right]} \qquad P_e = \begin{pmatrix} 1825 \\ 2676 \\ 3753 \\ 4682 \\ 6383 \end{pmatrix} \cdot kip$$

LRFD Article 6.9.4.1.1

$$\frac{P_e}{P_o} = \begin{pmatrix} 2.355 \\ 2.455 \\ 3.507 \\ 3.588 \\ 3.711 \end{pmatrix}$$

If Pe/Po > or = 0.44, then:

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

LRFD Eq. 6.9.4.1.1-1

then

$$P_n = \begin{pmatrix} 649 \\ 919 \\ 950 \\ 1161 \\ 1537 \end{pmatrix} \cdot kip$$

These nominal pile resistance will govern Structural Resistance for Extreme Event and Service Limit States with a ϕ = 1.0 - normal conditions (no scour).

Factored Axial Structural Resistance for the Strength Limit State

Resistance factor for lower unbraced segments of H-pile in combined compression and flexure:

$$\phi_{cu} := 0.7$$

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

$$P_r := \phi_{cu} \cdot P_n$$

Factored structural compressive resistance, Pr

$$P_r = \begin{pmatrix} 454 \\ 643 \\ 665 \\ 813 \\ 1076 \end{pmatrix} \cdot kip$$

C. Structural Resistance of braced segment of pile

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

K = effective length factor $K_{eff} := 2.0$ LRFD Table C4.6.2.5-1

l = unbraced length $l_{unbraced_tip} := .1 \cdot ft$ Ref : LPile

LRFD eq. 6.9.4.1.2-1

$$P_e := \overrightarrow{\left[\frac{\pi^2 \cdot E}{\left(\frac{K_{eff} \cdot l_{unbraced_tip}}{r_s} \right)^2} \cdot A_s \right]} \qquad P_e = \begin{pmatrix} 6299978 \\ 9236288 \\ 1 \times 10^7 \\ 2 \times 10^7 \\ 2 \times 10^7 \end{pmatrix} \cdot kip$$

LRFD Article 6.9.4.1.1

$$\frac{P_e}{P_o} = \begin{pmatrix} 8129 \\ 8474 \\ 12105 \\ 12384 \\ 12808 \end{pmatrix}$$

If Pe/Po > or = 0.44, then:

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

LRFD Eq. 6.9.4.1.1-1

then

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

Factored Axial Structural Resistance for the Strength Limit State

Resistance factor for lower portion of H-pile in pure compression, severe driving conditions, per LRFD 6.5.4.2 for the case where pile tip is necessary

$$\phi_c := 0.5$$

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

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

Factored structural compressive resistance, Pr

$$P_r = \begin{pmatrix} 387 \\ 545 \\ 535 \\ 652 \\ 860 \end{pmatrix} \cdot kip$$

These factored pile structural resistances will govern Strength Limit States with a ϕ = 0.50

LRFD 10.7.3.2.3 - Piles Driven to Hard Rock -

Article 10.7.3.2.3 states "The nominal resistance of piles driven to point bearing on hard rock where pile penetration into the rock formation is minimal is controlled by the structural limit state. The nominal bearing resistance shall not exceed the values obtained from Article 6.9.4.1 with the resistance factors specified in Article 6.5.4.2 and Article 6.15 for severe driving conditions. A pile driving acceptance criteria shall be developed that will prevent pile damage."

Therefore limit the nominal axial geotechnical pile resistance to the nominal structural resistance with a resistance factor for severe driving conditions of 0.50 applied per 10.7.3.2.3.

Nominal Structural Resistance Previously Calculated (lower braced segment):

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

The factored geotechnical compressive resistance (Pr) for the **Strength Limit State**, per LRFD 6.9.2.1-1 is

$$\phi_c := 0.5$$

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

$$\boxed{P_r = \begin{pmatrix} 387 \\ 545 \\ 535 \\ 652 \\ 860 \end{pmatrix} \cdot kip}$$

12x53
12x74
14x73
14x89
14x117

The factored geotechnical compressive resistance (Pr) for the **Extreme and Service Limit States**, per LRFD 6.9.2.1-1 is

$$\phi_c := 1.0$$

$$P_{r_ee} := \phi_c \cdot P_n$$

$$\boxed{P_{r_ee} = \begin{pmatrix} 775 \\ 1090 \\ 1070 \\ 1305 \\ 1720 \end{pmatrix} \cdot kip}$$

12x53
12x74
14x73
14x89
14x117

Based on past practice, limit the factored geotechnical resistance to:

Based on recent LRFD driven pile projects, limit the factored geotechnical resistance to 75% of the nominal structural pile resistance with a resistance factor for severe driving conditions, of 0.50, applied. Then compute the nominal resistance by dividing by a resistance factor of 0.65 for a pile resistance determination method consisting of at least 2 dynamic load tests with signal matching

$$R_{geo} := P_r \cdot 0.75 \qquad R_{geo} = \begin{pmatrix} 291 \\ 409 \\ 401 \\ 489 \\ 645 \end{pmatrix} \cdot kip$$

Pile Load Test to the following "nominal" resistance

$$\phi_{dyn} := 0.65$$

Reference LRFD Table 10.5.5.2.3-1

$$R_{nominal} := \frac{1}{\phi_{dyn}} \cdot R_{geo} \qquad R_{nominal} = \begin{pmatrix} 447 \\ 629 \\ 617 \\ 753 \\ 992 \end{pmatrix} \cdot kip$$

12x53
12x74
14x73
14x89
14x117

Nominal and Factored Axial Geotechnical Resistance of HP piles

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

Nominal unit bearing resistance of pile point, qp

Design value of compressive strength of rock core

$$q_{u_1} := 15000 \cdot psi$$

Spacing of discontinuities $\quad s_d := 12 \cdot in$

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

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

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

Diameter of socket:

$$D_s := 12 \cdot in$$

Depth factor

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

$$dd = 1$$ OK

Ksp

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

$$K_{sp} = \begin{pmatrix} 0.339 \\ 0.338 \\ 0.324 \\ 0.324 \\ 0.323 \end{pmatrix}$$

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

Geotechnical tip resistance.

$$q_{p_1} := 3 \cdot q_{u_1} \cdot K_{sp} \cdot dd$$

$$\boxed{q_{p_1} = \begin{pmatrix} 2196 \\ 2188 \\ 2101 \\ 2097 \\ 2092 \end{pmatrix} \cdot ksf}$$

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

Case I

$$R_{p_1} := \overrightarrow{\left(q_{p_1} \cdot A_s\right)}$$

$$\boxed{R_{p_1} = \begin{pmatrix} 236 \\ 331 \\ 312 \\ 380 \\ 500 \end{pmatrix} \cdot kip}$$

$\phi = 1.0$

Factored Axial Geotechnical Compressive Resistance - *Strength Limit States*

Resistance factor, end bearing on rock Canadian Geotechnical Society method

$\phi_{stat} := 0.45$ LRFD Table 10.5.5.2.3-1

Factored Geotechnical Tip Resistance (Rr)

$R_{r_p1} := \phi_{stat} \cdot R_{p_1}$

$$R_{r_p1} = \begin{pmatrix} 106 \\ 149 \\ 140 \\ 171 \\ 225 \end{pmatrix} \cdot kip$$

CGS method is superceded by LRFD 10.7.3.2.3 - Piles Driven to hard rock.

Nominal and Factored Axial Geotechnical Resistance of HP piles

Geotechncial axial pile resistance for pile end bearing on rock determined by Intact Rock Method, proposed by Sandford, MaineDOT Transportation Research Division Technical Report 14-01, Phase 2 (January 2014), based on Rowe and Armitage (1987) equation cited by NCHRP Synthesis 360, Turner, (2006).

Nominal unit bearing resistance of pile point, qp

Design value of compressive strength of rock core

$q_{u_1} := 15000 \cdot psi$

Geotechnical tip resistance.

$q_{p_2} := 2.5 \cdot q_{u_1}$

$$q_{p_1} = \begin{pmatrix} 2196 \\ 2188 \\ 2101 \\ 2097 \\ 2092 \end{pmatrix} \cdot ksf$$

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

$$R_{p_2} := \overrightarrow{\left(q_{p_2} \cdot A_s\right)} \qquad R_{p_2} = \begin{pmatrix} 581 \\ 818 \\ 803 \\ 979 \\ 1290 \end{pmatrix} \cdot kip \qquad \phi = 1.0$$

Factored Axial Geotechnical Compressive Resistance - *Strength Limit States*

Resistance factor, end bearing on rock Canadian Geotechnical Society method

$\phi_{stat} := 0.45$ LRFD Table 10.5.5.2.3-1

Factored Geotechnical Tip Resistance (Rr)

$$R_{r_p2} := \phi_{stat} \cdot R_{p_2} \qquad R_{r_p2} = \begin{pmatrix} 262 \\ 368 \\ 361 \\ 440 \\ 581 \end{pmatrix} \cdot kip$$

Drivability Analyses

Ref: LRFD Article 10.7.8

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

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

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

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

Limit driving stress to 45 ksi or limit blow count to 5-15 blows per inch (bpi) per Section 501 (Note: 6-10 bpi is considered optimal for diesel hammers).

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

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

$\phi_{dyn} := 0.65$ Reference LRFD Table 10.5.5.2.3-1 - for Strength Limit State

$\phi := 1.0$ For Extreme and Service Limit States

GRLWeap Soil and Pile Model Assumptions

Based on Table 7-1 of this Report, estmated pile lengths at Abutment 1 will be approx. 16 feet.

Minimum length of pile embedment in soil is 16 ft.

Assume the Contractor drives pile lengths of 20 ft.

Use constant shaft resistance at 10% - use min. ultimate capacity of 300 kips, so GRLWeap will assign 30 kip shaft resistance to all ultimate capacities analyzed.

Pile Size is 12 x 53

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

State of Maine Dept. Of Transportation
Abut 1 12x53 Delmag D19-42

22-Jul-2014
GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
300.0	38.75	0.92	3.9	7.17	13.94
310.0	39.78	1.01	4.0	7.22	13.98
320.0	40.78	1.09	4.2	7.29	14.05
340.0	42.82	1.19	4.5	7.42	14.25
350.0	43.75	1.49	4.7	7.48	14.34
360.0	44.79	1.99	4.9	7.56	14.50
370.0	45.75	2.38	5.0	7.65	14.63
400.0	48.32	3.09	5.7	7.86	14.98

Rounding up blow counts >4 to 5 will overstress the piles

$R_{ndr} := 310 \cdot kip$

Strength Limit State

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

$R_{fdr} = 202 \cdot kip$

Extreme and Service Limit States

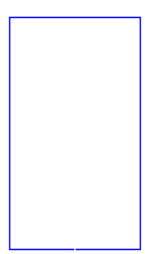
$R_{dr} := R_{ndr} \cdot \phi$

$R_{dr} = 310 \cdot kip$

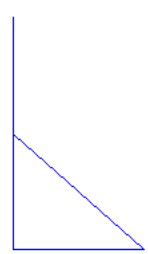
DELMAG D 19-42

Efficiency	0.800	
Helmet	2.70	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	20.00	ft
Pile Penetration	16.00	ft
Pile Top Area	15.50	in2

Pile Model



Skin Friction Distribution



Res. Shaft = 10 %
(Constant Res. Shaft)

Pile Size is 12 x 74

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

State of Maine Dept. Of Transportation — 21-Jul-2014
Abut 1 12x 74 Delmag D19-42 — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
300.0	30.17	0.39	4.0	7.08	13.40
400.0	35.46	1.00	5.6	7.54	13.78
520.0	42.48	2.93	7.6	8.39	15.23
530.0	42.86	3.01	8.0	8.37	15.11
540.0	43.48	3.09	8.2	8.44	15.20
550.0	44.13	3.18	8.4	8.52	15.37
570.0	45.42	3.26	8.8	8.67	15.63
600.0	47.37	2.79	9.4	8.96	16.18

Limit driving stress < 45 ksi stress;
8 bpi.

$R_{ndr} := 530 \cdot kip$

Strength Limit State

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

$R_{fdr} = 345 \cdot kip$

Extreme and Service Limit States

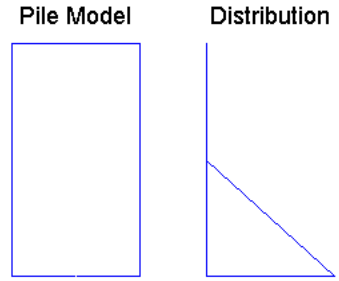
$R_{dr} := R_{ndr} \cdot \phi$

$R_{dr} = 530 \cdot kip$

DELMAG D 19-42

Efficiency	0.800	
Helmet	2.70	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	20.00	ft
Pile Penetration	16.00	ft
Pile Top Area	21.80	in2

Pile Model — Skin Friction Distribution



Res. Shaft = 10 %
(Constant Res. Shaft)

Pile Size is 14 x 73

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

State of Maine Dept. Of Transportation
14 x 73 Delmag D19-42

21-Jul-2014
GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
300.0	30.46	0.38	4.0	7.08	13.46
450.0	38.55	1.73	6.4	7.88	14.37
500.0	41.88	2.79	7.3	8.26	15.02
520.0	43.32	3.02	7.6	8.42	15.31
530.0	43.73	3.10	8.0	8.40	15.21
540.0	44.33	3.21	8.2	8.46	15.28
550.0	45.00	3.31	8.4	8.55	15.46
580.0	46.81	3.29	9.1	8.77	15.84

Blow counts >8 bpi round up to 9 bpi will result in a driving stresses that exceed 45 ksi. Use bpi of 8.

$R_{ndr} := 530 \cdot kip$

Strength Limit State

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

$R_{fdr} = 345 \cdot kip$

Extreme and Service Limit States

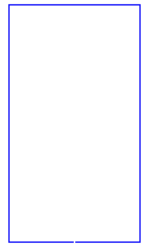
$R_{dr} := R_{ndr} \cdot \phi$

$R_{dr} = 530 \cdot kip$

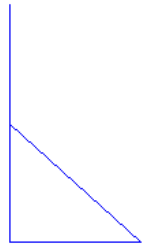
DELMAG D 19-42

Efficiency	0.800	
Helmet	2.70	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	20.00	ft
Pile Penetration	16.00	ft
Pile Top Area	21.40	in2

Pile Model



Skin Friction Distribution



Res. Shaft = 10 %
(Constant Res. Shaft)

Pile Size is 14 x 89

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

State of Maine Dept. Of Transportation — 10-Apr-2014
Ab1 14x89 Delmag D19-42 2.7 kip helmet — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
300.0	26.90	0.20	4.1	7.05	13.21
500.0	36.72	1.71	7.2	8.02	14.20
650.0	43.03	3.38	10.3	8.94	15.70
690.0	44.30	3.76	11.7	9.09	15.84
700.0	44.68	3.85	11.9	9.16	15.98
710.0	45.03	3.88	12.3	9.21	16.04
720.0	45.43	3.92	12.5	9.28	16.16

Limiting driving stress to 45 ksi

$R_{ndr} := 700 \cdot kip$

Strength Limit State

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

$R_{fdr} = 455 \cdot kip$

Extreme and Service Limit States

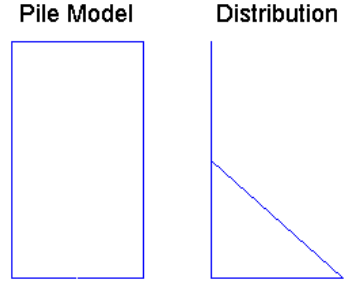
$R_{dr} := R_{ndr} \cdot \phi$

$R_{dr} = 700 \cdot kip$

DELMAG D 19-42

Efficiency	0.800	
Helmet	2.70	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	20.00	ft
Pile Penetration	16.00	ft
Pile Top Area	26.10	in2

Pile Model — Skin Friction Distribution



Res. Shaft = 10 %
(Constant Res. Shaft)

Pile Size is 14 x 117

WEAP runs indicated a 14 x117 pile cannot be driven to a driving resistances of 5 bpi with a D 36-32 at any fuel setting or either helmet (2.7 kip or 3.2 kip) without overstressing the pile. Should be 5-15 bpi per 501. Try 19-42. Helmet is 2.7 kip. See GRLWEAP results below:

State of Maine Dept. Of Transportation
14 x 117 APE D 19-42 Abut 1

10-Apr-2014
GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
300.0	24.42	0.06	3.4	7.38	16.87
400.0	29.10	0.87	4.6	7.73	16.82
700.0	40.54	3.39	8.6	9.14	18.46
800.0	43.89	3.12	10.4	9.71	19.27
830.0	44.83	2.71	11.0	9.88	19.49
840.0	45.13	2.71	11.2	9.93	19.60
850.0	45.38	2.55	11.5	9.96	19.61

APE D 19-42

Efficiency	0.800	
Helmet	2.70	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	20.00	ft
Pile Penetration	16.00	ft
Pile Top Area	34.40	in2

Limit driving stress to 45 ksi

$R_{ndr} := 830 \cdot kip$

Strength Limit State

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

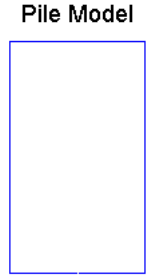
$R_{fdr} = 540 \cdot kip$

Extreme and Service Limit States

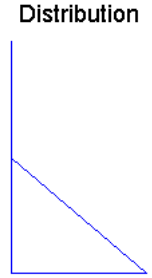
$R_{dr} := R_{ndr} \cdot \phi$

$R_{dr} = 830 \cdot kip$

Pile Model



Skin Friction Distribution



Res. Shaft = 10 %
(Constant Res. Shaft)

Pile Size is 14 x 117

WEAP runs indicated a 14 x117 pile can be driven to a driving resistances of 5 bpi with a D 36-32 at a lowered fuel setting of 3 (1174 psi) with a helmet 2.7 kip, without overstressing the pile. Should be 5-15 bpi per 501.

State of Maine Dept. Of Transportation
14 x 117 Delmag D36-32 Abut 1

10-Apr-2014
GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
300.0	27.05	0.00	2.0	6.05	25.86
400.0	32.08	0.00	2.9	6.51	24.71
500.0	36.78	0.11	3.7	6.84	24.61
600.0	41.23	0.88	4.5	7.19	25.24
630.0	42.58	1.10	4.7	7.33	25.60
640.0	42.84	1.11	4.9	7.30	25.35
670.0	43.93	1.15	5.2	7.40	25.57
700.0	45.05	1.17	5.5	7.50	25.76

DELMAG D 36-32

Efficiency	0.800	
Helmet	2.70	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	20.00	ft
Pile Penetration	16.00	ft
Pile Top Area	34.40	in2

$R_{ndr} := 640 \cdot kip$

Strength Limit State

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

$R_{fdr} = 416 \cdot kip$

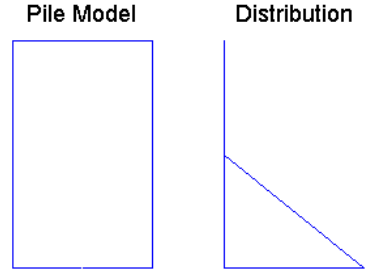
Extreme and Service Limit States

$R_{dr} := R_{ndr} \cdot \phi$

$R_{dr} = 640 \cdot kip$

Pile Model

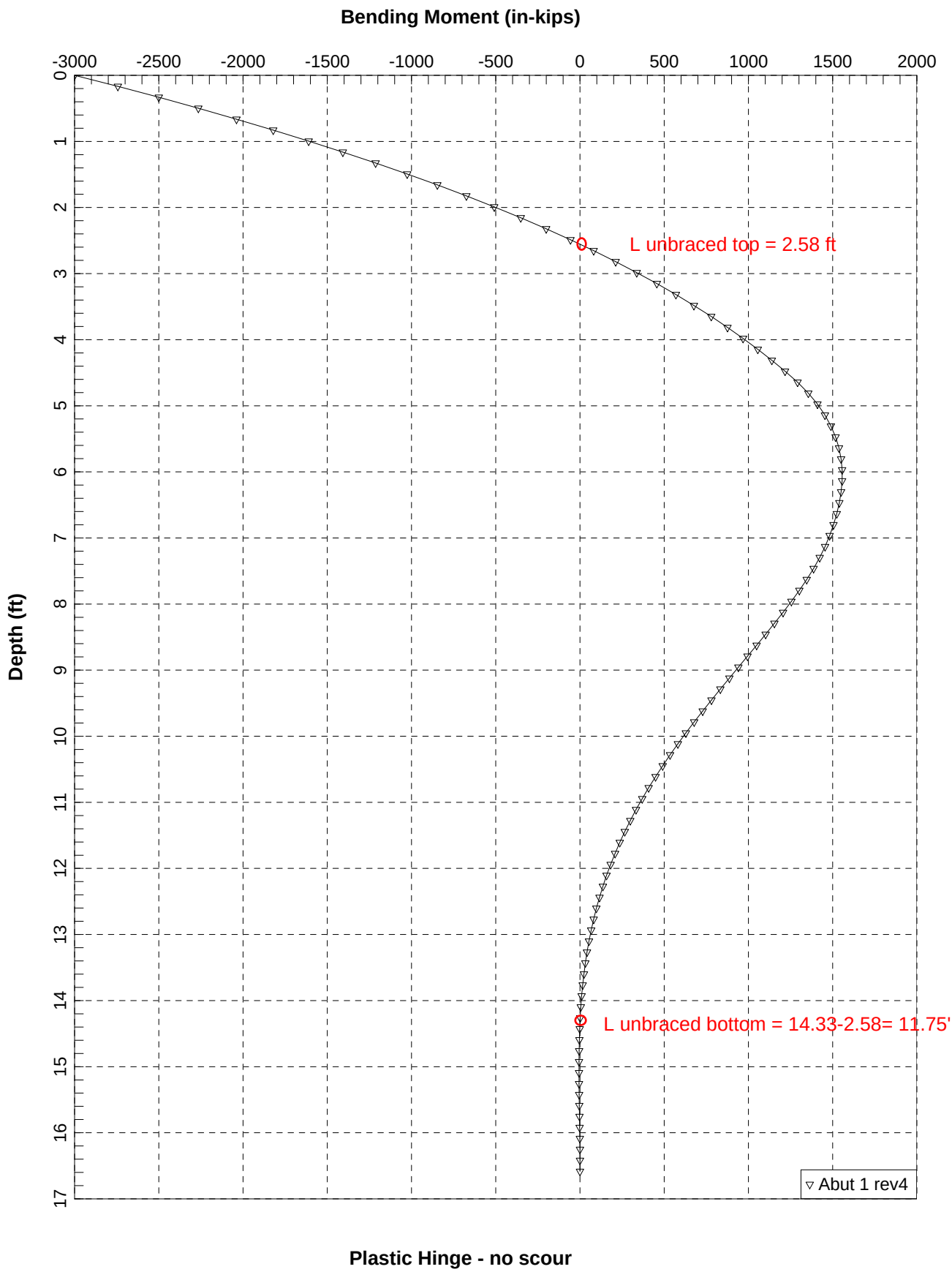
Skin Friction Distribution

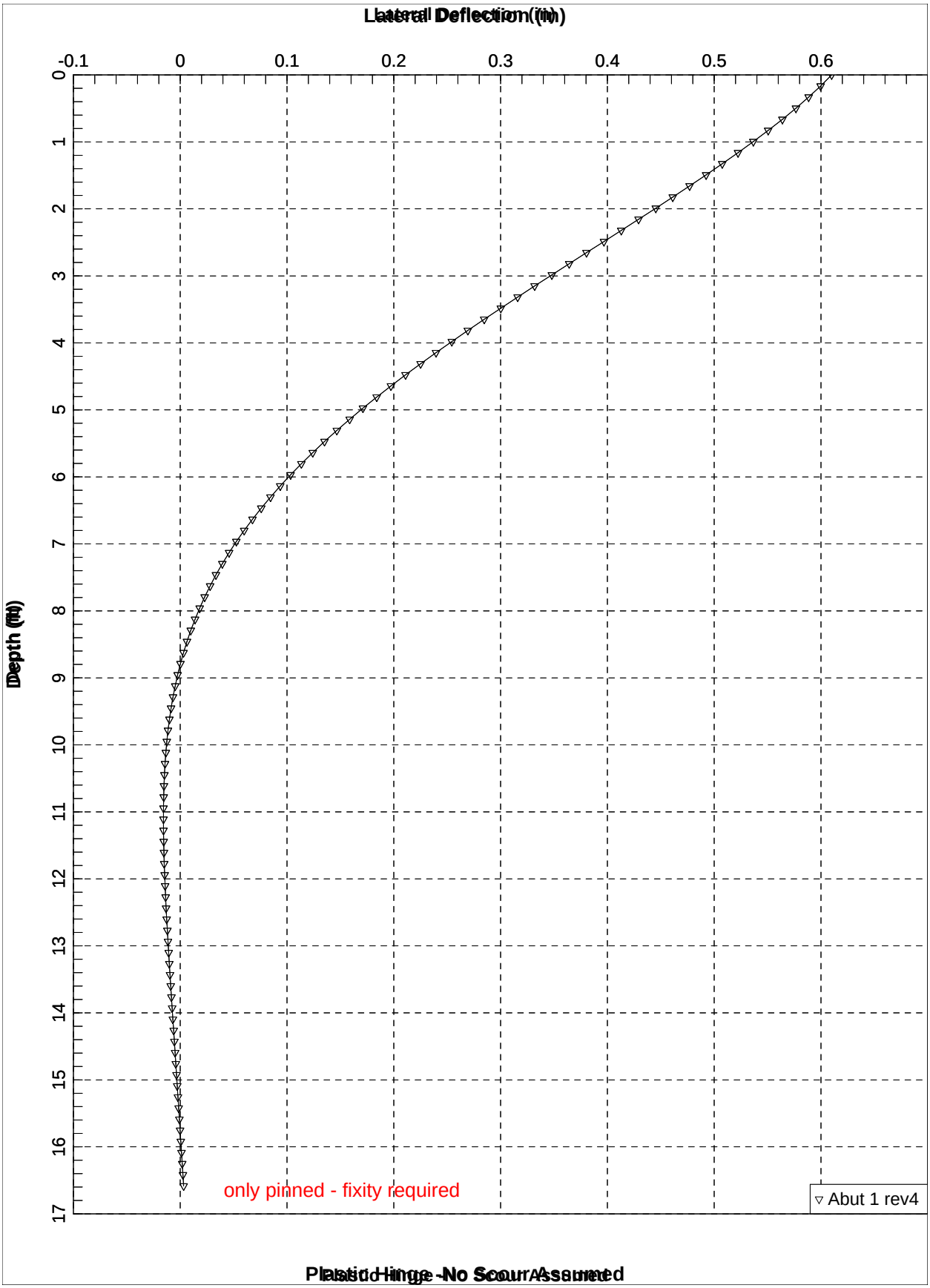


Res. Shaft = 10 %
(Constant Res. Shaft)

Abutment No. 1 Lpile Analyses

Abut 1 rev4 14x117 reduced plastic moment of 250 ft-kip applied




Lateral Deflection (in)
-0.1
0
0.1
0.2
0.3
0.4
0.5
0.6
Depth (ft)
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
only pinned - fixity required
Abut 1 rev4
Plastic Hinge - No Scour Assumed

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LPILE Plus for Windows, Version 5.0 (5.0.39)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

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This program is licensed to:

L. Krusinski
Maine Department of Transportation

Path to file locations: C:\Program Files\Ensoft\Lpilep5\
Name of input data file: EMachias_Ab1_117_rev4.lpd
Name of output file: EMachias_Ab1_117_rev4.lpo
Name of plot output file: EMachias_Ab1_117_rev4.lpp
Name of runtime file: EMachias_Ab1_117_rev4.lpr

Time and Date of Analysis

Date: April 25, 2014 Time: 11:40:37

Problem Title

Abut 1 14x117_16 foot pile - factored loads - 250 ft-kip Plastic moment applied

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Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis does not use p-y multipliers (individual pile or shaft action only)
- Analysis assumes no shear resistance at pile tip
- Analysis for fixed-length pile or shaft only
- No computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 100
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and
  soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) = 1

Pile Structural Properties and Geometry

Pile Length = 199.00 in

Depth of ground surface below top of pile = -150.00 in

Slope angle of ground surface = .00 deg.

Structural properties of pile defined using 2 points

Point	Depth X in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	14.88500000	443.0000	34.4000	29000000.
2	199.0000	14.88500000	443.0000	34.4000	29000000.

Soil and Rock Layering Information

The soil profile is modelled using 4 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = -150.000 in
Distance from top of pile to bottom of layer = 2.000 in
p-y subgrade modulus k for top of soil layer = 60.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 60.000 lbs/in**3

Layer 2 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 2.000 in
Distance from top of pile to bottom of layer = 54.000 in
p-y subgrade modulus k for top of soil layer = 20.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 20.000 lbs/in**3

Layer 3 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 54.000 in
Distance from top of pile to bottom of layer = 84.000 in
p-y subgrade modulus k for top of soil layer = 60.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 60.000 lbs/in**3

Layer 4 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 84.000 in
Distance from top of pile to bottom of layer = 199.000 in
p-y subgrade modulus k for top of soil layer = 120.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 120.000 lbs/in**3

(Depth of lowest layer extends .00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Effective unit weight of soil with depth defined using 8 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	-150.00	.07000
2	2.00	.07000
3	2.00	.03330
4	54.00	.03330
5	54.00	.03600
6	84.00	.03600
7	84.00	.03900
8	199.00	.03900

Shear Strength of Soils

Shear strength parameters with depth defined using 8 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	-150.000	.00000	32.00	------	------
2	2.000	.00000	32.00	------	------
3	2.000	.00000	30.00	------	------
4	54.000	.00000	30.00	------	------
5	54.000	.00000	32.00	------	------
6	84.000	.00000	32.00	------	------
7	84.000	.00000	34.00	------	------
8	199.000	.00000	34.00	------	------

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

Loading Type

Static loading criteria was used for computation of p-y curves.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load Case Number 1

Pile-head boundary conditions are Displacement and Moment (BC Type 4)
Deflection at pile head = .610 in
Bending moment at pile head = -3000000.000 in-lbs
Axial load at pile head = 478000.000 lbs

Reduced plastic moment of 250 kip*ft

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Displacement and Moment (BC Type 4)
Specified deflection at pile head = .610000 in
Specified moment at pile head = -3000000.000 in-lbs
Specified axial load at pile head = 478000.000 lbs

Depth X in	Deflect. y in	Moment M lbs-in	Shear V lbs	Slope S Rad.	Stress lbs/in**2	Total p lbs/in	Soil Res. F/L lbs/in	Es*h
0.000	.610000	-3000000.	130170.	-.0049651	64296.0260	-3567.3075	5818.8049	
1.990	.599657	-2743081.	122982.	-.0054100	59979.7368	-3657.1387	12136.4486	
3.980	.588468	-2500241.	117194.	-.0058160	55899.9689	-2159.6905	7303.3387	
5.970	.576509	-2265586.	112917.	-.0061852	51957.6973	-2138.7447	7382.5407	
7.960	.563851	-2039065.	108685.	-.0065186	48152.1093	-2114.2284	7461.7427	
9.950	.550565	-1820618.	104506.	-.0068175	44482.1347	-2086.3227	7540.9447	
11.940	.536718	-1610163.	100385.	-.0070832	40946.4575	-2055.2105	7620.1467	
13.930	.522374	-1407611.	96328.8540	-.0073169	37543.5288	-2021.0757	7699.3487	
15.920	.507596	-1212855.	92343.7012	-.0075199	34271.5790	-1984.1030	7778.5507	
17.910	.492445	-1025777.	88434.7632	-.0076933	31128.6305	-1944.4779	7857.7527	
19.900	.476977	-846248.	84607.1341	-.0078383	28112.5099	-1902.3855	7936.9547	

21.890	.461249	-674129.	80865.5393	-.0079560	25220.8614	-1858.0112	8016.1567
23.880	.445312	-509268.	77214.3364	-.0080477	22451.1586	-1811.5395	8095.3587
25.870	.429219	-351505.	73657.5160	-.0081143	19800.7179	-1763.1544	8174.5607
27.860	.413017	-200674.	70198.7036	-.0081571	17266.7109	-1713.0389	8253.7627
29.850	.396754	-56596.1229	66841.1620	-.0081770	14846.1765	-1661.3747	8332.9647
31.840	.380473	80910.4976	63587.7937	-.0081751	15254.6635	-1608.3422	8412.1667
33.830	.364217	212036.	60441.1436	-.0081524	17457.6015	-1554.1202	8491.3687
35.820	.348026	336976.	57403.4029	-.0081099	19556.6182	-1498.8856	8570.5707
37.810	.331939	455930.	54476.4120	-.0080485	21555.0790	-1442.8138	8649.7727
39.800	.315993	569104.	51661.6648	-.0079691	23456.4199	-1386.0779	8728.9747
41.790	.300222	676704.	48960.3123	-.0078726	25264.1355	-1328.8492	8808.1767
43.780	.284660	778943.	46373.1671	-.0077599	26981.7669	-1271.2967	8887.3787
45.770	.269338	876032.	43900.7074	-.0076317	28612.8900	-1213.5874	8966.5807
47.760	.254286	968187.	41543.0815	-.0074889	30161.1039	-1155.8859	9045.7827
49.750	.239532	1055621.	39300.1121	-.0073321	31630.0194	-1098.3547	9124.9847
51.740	.225104	1138550.	37171.3007	-.0071622	33023.2476	-1041.1542	9204.1867
53.730	.211026	1217188.	35155.8321	-.0069798	34344.3888	-984.4424	9283.3887
55.720	.197324	1291749.	31558.1772	-.0067854	35597.0215	-2631.2911	26536.3585
57.710	.184020	1355699.	26476.5647	-.0065804	36671.3953	-2475.8572	26773.9645
59.700	.171134	1409644.	21701.7825	-.0063662	37577.6929	-2322.9188	27011.5705
61.690	.158683	1454183.	17228.4894	-.0061444	38325.9560	-2172.8531	27249.1765
63.680	.146680	1489903.	13050.6250	-.0059164	38926.0582	-2026.0055	27486.7825
65.670	.135136	1517380.	9161.4730	-.0056835	39387.6810	-1882.6900	27724.3885
67.660	.124059	1537178.	5553.7225	-.0054469	39720.2912	-1743.1899	27961.9945
69.650	.113457	1549847.	2219.5278	-.0052078	39933.1208	-1607.7595	28199.6005
71.640	.103332	1555920.	-849.4338	-.0049673	40035.1487	-1476.6241	28437.2065
73.630	.093687	1555916.	-3661.9068	-.0047263	40035.0845	-1349.9819	28674.8125
75.620	.084522	1550337.	-6227.0036	-.0044857	39941.3536	-1228.0047	28912.4185
77.610	.075834	1539666.	-8554.1533	-.0042464	39762.0852	-1110.8391	29150.0245
79.600	.067621	1524370.	-10653.0529	-.0040090	39505.1007	-998.6077	29387.6305
81.590	.059878	1504894.	-12533.6209	-.0037744	39177.9048	-891.4103	29625.2365
83.580	.052599	1481666.	-14205.9527	-.0035431	38787.6776	-789.3251	29862.8425
85.570	.045777	1455095.	-16287.6230	-.0033157	38341.2685	-1302.8058	56635.5731
87.560	.039403	1423149.	-18709.0736	-.0030928	37804.5810	-1130.8129	57110.7851
89.550	.033467	1386516.	-20797.8604	-.0028751	37189.1369	-968.4704	57585.9971
91.540	.027960	1345844.	-22573.1727	-.0026635	36505.8278	-815.7631	58061.2091
93.530	.022867	1301742.	-24054.1223	-.0024585	35764.9139	-672.6284	58536.4211
95.520	.018175	1254785.	-25259.6532	-.0022605	34976.0274	-538.9604	59011.6331
97.510	.013870	1205509.	-26208.4596	-.0020699	34148.1774	-414.6139	59486.8451
99.500	.009937	1154414.	-26918.9117	-.0018871	33289.7588	-299.4083	59962.0571
101.490	.006359	1101962.	-27408.9887	-.0017124	32408.5627	-193.1314	60437.2691
103.480	.003121	1048584.	-27696.2200	-.0015458	31511.7897	-95.5433	60912.4811
105.470	.000207	994672.	-27797.6334	-.0013876	30606.0644	-6.3798	61387.6931
107.460	-.002401	940589.	-27729.7102	-.0012377	29697.4525	74.6443	61862.9051
109.450	-.004719	886662.	-27508.3470	-.0010962	28791.4783	147.8314	62338.1171
111.440	-.006764	833191.	-27148.8237	-.0009630	27893.1446	213.4985	62813.3291
113.430	-.008552	780442.	-26665.7784	-.0008380	27006.9526	271.9741	63288.5411
115.420	-.010099	728655.	-26073.1869	-.0007211	26136.9233	323.5952	63763.7531
117.410	-.011422	678043.	-25384.3485	-.0006122	25286.6192	368.7047	64238.9651
119.400	-.012535	628790.	-24611.8765	-.0005109	24459.1668	407.6491	64714.1771
121.390	-.013455	581059.	-23767.6937	-.0004172	23657.2790	440.7759	65189.3891
123.380	-.014196	534989.	-22863.0320	-.0003308	22883.2780	468.4319	65664.6011
125.370	-.014772	490694.	-21908.4364	-.0002514	22139.1176	490.9606	66139.8131
127.360	-.015196	448271.	-20913.7731	-.0001786	21426.4065	508.7011	66615.0251
129.350	-.015483	407797.	-19888.2396	-.0001123	20746.4299	521.9858	67090.2371
131.340	-.015644	369330.	-18840.3802	-5.2150E-05	20100.1719	531.1393	67565.4491
133.330	-.015690	332911.	-17778.1020	2.2388E-06	19488.3366	536.4770	68040.6611
135.320	-.015635	298569.	-16708.6950	5.1147E-05	18911.3694	538.3039	68515.8731
137.310	-.015487	266314.	-15638.8538	9.4897E-05	18369.4767	536.9134	68991.0851
139.300	-.015257	236145.	-14574.7011	.0001338	17862.6459	532.5867	69466.2971
141.290	-.014954	208052.	-13521.8135	.0001682	17390.6639	525.5919	69941.5091
143.280	-.014588	182009.	-12485.2474	.0001984	16953.1348	516.1831	70416.7211
145.270	-.014165	157983.	-11469.5678	.0002248	16549.4972	504.6005	70891.9331
147.260	-.013693	135932.	-10478.8763	.0002475	16179.0398	491.0694	71367.1451
149.250	-.013179	115806.	-9516.8408	.0002670	15840.9170	475.8004	71842.3571
151.240	-.012630	97547.1441	-8586.7256	.0002835	15534.1629	458.9888	72317.5691
153.230	-.012051	81091.4826	-7691.4207	.0002974	15257.7041	440.8151	72792.7811
155.220	-.011447	66369.5447	-6833.4725	.0003088	15010.3722	421.4444	73267.9931
157.210	-.010822	53306.7895	-6015.1136	.0003181	14790.9149	401.0269	73743.2051
159.200	-.010181	41824.2864	-5238.2921	.0003254	14598.0063	379.6982	74218.4171
161.190	-.009527	31839.2636	-4504.7009	.0003311	14430.2557	357.5794	74693.6291
163.180	-.008863	23265.5996	-3815.8058	.0003354	14286.2162	334.7775	75168.8411
165.170	-.008192	16014.2598	-3172.8733	.0003385	14164.3920	311.3859	75644.0531
167.160	-.007516	9993.6797	-2576.9968	.0003405	14063.2449	287.4850	76119.2651

M=0 at D=31" 2.58'

169.150	-.006837	5110.0962	-2029.1224	.0003416	13981.1996	263.1426	76594.4771
171.140	-.006156	1267.8309	-1530.0726	.0003421	13916.6487	238.4150	77069.6891
173.130	-.005475	-1630.4741	-1080.5695	.0003421	13922.7412	213.3470	77544.9011
175.120	-.004794	-3683.6634	-681.2556	.0003417	13957.2352	187.9735	78020.1131
177.110	-.004115	-4991.9163	-332.7139	.0003410	13979.2142	162.3197	78495.3251
179.100	-.003437	-5656.6313	-35.4852	.0003402	13990.3815	136.4025	78970.5371
181.090	-.002761	-5780.3452	209.9154	.0003393	13992.4599	110.2312	79445.7491
183.080	-.002087	-5466.6806	402.9850	.0003384	13987.1903	83.8086	79920.9611
185.070	-.001414	-4820.3202	543.2208	.0003376	13976.3313	57.1319	80396.1731
187.060	-.000743	-3947.0013	630.1105	.0003370	13961.6594	30.1944	80871.3851
189.050	-7.30E-05	-2953.5280	663.1252	.0003364	13944.9688	2.9861	81346.5971
191.040	.000596	-1947.7941	641.7141	.0003360	13928.0722	-24.5047	81821.8091
193.030	.001264	-1038.8145	565.3031	.0003358	13912.8012	-52.2903	82297.0211
195.020	.001933	-336.7567	433.2943	.0003357	13901.0064	-80.3818	82772.2331
197.010	.002601	47.0308	245.0704	.0003357	13896.1390	-108.7880	83247.4451
199.000	.003269	0.0000	0.0000	.0003357	13895.3488	-137.5139	41861.3286

M=0 at D=172" = 14.3'

not 0

Output Verification:

Computed forces and moments are within specified convergence limits.

Output Summary for Load Case No. 1:

Pile-head deflection = .61000000 in
Computed slope at pile head = -.00496515
Maximum bending moment = -3000000. lbs-in
Maximum shear force = 130169.85288 lbs
Depth of maximum bending moment = 0.00000 in
Depth of maximum shear force = 0.00000 in
Number of iterations = 5
Number of zero deflection points = 2

Summary of Pile Response(s)

Definition of Symbols for Pile-Head Loading Conditions:

Type 1 = Shear and Moment, y = pile-head displacment in
Type 2 = Shear and Slope, M = Pile-head Moment lbs-in
Type 3 = Shear and Rot. Stiffness, V = Pile-head Shear Force lbs
Type 4 = Deflection and Moment, S = Pile-head Slope, radians
Type 5 = Deflection and Slope, R = Rot. Stiffness of Pile-head in-lbs/rad

Load Type	Pile-Head Condition 1	Pile-Head Condition 2	Axial Load lbs	Pile-Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
4	y= .610000	M= -3.00E+06	478000.	.6100000	-3000000.	130170.

The analysis ended normally.

Abutment No. 2 H-Piles

Design of H-piles

Reference: AASHTO LRFD Bridge Design Specifications, 6th Edition, 2012

Bedrock Properties at Abutment 2

BB-EMR-102, R1=100% and R2=95%

Rock Type: DIORITE, very hard, fresh.

ϕ = 34-40 (AASHTO LRFD Table C.10.4.6.4-1);

Co = 2,100 - 49,000 psi (AASHTO Standard Specifications 17th Edition, Table 4.4.8.1.2B)

For Design Purposes: RQD = 95% and an Unconfined Compressive Strength of 15,000 psi

Pile Properties

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

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

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

Pile yield strength $F_y := 50 \cdot ksi$

1. Nominal and Factored Structural Compressive Resistance of HP piles

Use LRFD Equation 6.9.2.1-1 Pr = ϕ Pn

Nominal Axial Structural Resistance

Determine equivalent yield resistance Po=QFyAs (LRFD 6.9.4.1.1)

$Q := 1.0$ LRFD Article 6.9.4.2

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

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

Only the pile tip of the Abutment 2 pile is in only axial compression per LPILE analysis. Top unbraced pile segment is 1.75 ft per Bending Moment vs Depth curve, with a K of 2.1 since a plastic hinge develops.

The second unbraced length is 10.95 ft per the Moment vs Depth curves with a K of 1.0 where the second segment is pinned at the top and bottom. Use structural resistance factors of 0.7 for both segments since the factored resistances are being computed for pile segments in combined axial and flexure.

A. Structural Resistance of upper unbraced segment of pile

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

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

K = effective length factor $K_{eff} := 2.1$ LRFD Table C4.6.2.5-1 (plastic hinge develops, K=2.1 selected per VTrans Design Example)

l = unbraced length $l_{unbraced_top} := 1.75 \cdot ft$ Ref: LPile Analysis

r s = radius of gyration

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

radius of gyration about the Y-Y or weak axis per LRFD Article C6.9.4.1.2.

LRFD eq. 6.9.4.1.2-1

$$P_e := \overrightarrow{\left[\frac{\pi^2 \cdot E}{\left(\frac{K_{eff} \cdot l_{unbraced_top}}{r_s}\right)^2} \cdot A_s\right]} \qquad P_e = \begin{pmatrix} 18659 \\ 27355 \\ 38361 \\ 47864 \\ 65248 \end{pmatrix} \cdot kip$$

LRFD Article 6.9.4.1.1

$$\frac{P_e}{P_o} = \begin{pmatrix} 24.076 \\ 25.097 \\ 35.851 \\ 36.678 \\ 37.935 \end{pmatrix}$$

If Pe/Po > or = 0.44, then:

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

LRFD Eq. 6.9.4.1.1-1

then

$$\boxed{P_n = \begin{pmatrix} 762 \\ 1072 \\ 1058 \\ 1290 \\ 1701 \end{pmatrix} \cdot kip}$$

this applies to all the pile sizes

Factored Axial Structural Resistance for the Strength Limit State

Resistance factor for upper unbraced segement of H-pile in combined compression and flexure per LRFD 6.5.4.2

$$\phi_{cu} := 0.70$$

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

$$P_r := \phi_{cu} \cdot P_n$$

Factored structural compressive resistance, Pr

$$\boxed{P_r = \begin{pmatrix} 533 \\ 750 \\ 740 \\ 903 \\ 1191 \end{pmatrix} \cdot kip}$$

B. Structural Resistance of lower unbraced segment of pile

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

K = effective length factor $K_{eff} := 1.0$ Per VTrans Integral Abutment Design Guideline, use K=1.0 for segement pinned at top and bottom as shown by LPile analysis for second unbraced length (also see LRFD Table C4.6.2.5-1)

l = unbraced length $l_{unbraced_bot} := 12.7 \cdot ft$ Ref : LPILE

LRFD eq. 6.9.4.1.2-1

$$P_e := \left[\frac{\pi^2 \cdot E}{\left(\frac{K_{eff} \cdot l_{unbraced_bot}}{r_s}\right)^2} \cdot A_s\right] \qquad P_e = \begin{pmatrix} 1562 \\ 2291 \\ 3212 \\ 4008 \\ 5464 \end{pmatrix} \cdot kip$$

LRFD Article 6.9.4.1.1

$$\frac{P_e}{P_o} = \begin{pmatrix} 2.016 \\ 2.101 \\ 3.002 \\ 3.071 \\ 3.176 \end{pmatrix}$$

If Pe/Po > or = 0.44, then:

LRFD Eq. 6.9.4.1.1-1

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

$$P_n = \begin{pmatrix} 630 \\ 893 \\ 931 \\ 1139 \\ 1508 \end{pmatrix} \cdot kip$$

These values will govern for Service and Extreme Limit States with a ϕ = 1.0 applied (normal conditions).

Factored Axial Structural Resistance for the Strength Limit State

Resistance factor for lower unbraced segments of H-pile in combined compression and flexure:

$$\phi_{cu} := 0.7$$

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

$$P_r := \phi_{cu} \cdot P_n$$

Factored structural compressive resistance, Pr

$$P_r = \begin{pmatrix} 441 \\ 625 \\ 652 \\ 797 \\ 1055 \end{pmatrix} \cdot kip$$

C. Structural Resistance of tip of pile if in axial compression

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

K = effective length factor $K_{eff} := 1.0$ LRFD Table C4.6.2.5-1

l = unbraced length $l_{unbraced_tip} := .1 \cdot ft$ Ref : LPile

LRFD eq. 6.9.4.1.2-1

$$P_e := \overrightarrow{\left[\frac{\pi^2 \cdot E}{\left(\frac{K_{eff} \cdot l_{unbraced_tip}}{r_s} \right)^2} \cdot A_s \right]} \qquad P_e = \begin{pmatrix} 3 \times 10^7 \\ 4 \times 10^7 \\ 5 \times 10^7 \\ 6 \times 10^7 \\ 9 \times 10^7 \end{pmatrix} \cdot kip$$

LRFD Article 6.9.4.1.1

$$\frac{P_e}{P_o} = \begin{pmatrix} 32516 \\ 33894.6 \\ 48419 \\ 49535.3 \\ 51233.5 \end{pmatrix}$$

If Pe/Po > or = 0.44, then:

$$P_n := \overrightarrow{\begin{pmatrix} \frac{P_o}{P_e} \\ 0.658 \cdot P_o \end{pmatrix}}$$

LRFD Eq. 6.9.4.1.1-1

then

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

Factored Axial Structural Resistance for the Strength Limit State

Resistance factor for lower portion of H-pile in pure compression, severe driving conditions, per LRFD 6.5.4.2 for the case where pile tip is necessary

$$\phi_c := 0.5$$

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

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

Factored structural compressive resistance, Pr

$$P_r = \begin{pmatrix} 387 \\ 545 \\ 535 \\ 652 \\ 860 \end{pmatrix} \cdot kip$$

These values will govern Abutment 2 structural resistance for Strength Limit State

LRFD 10.7.3.2.3 - Piles Driven to Hard Rock -

Article 10.7.3.2.3 states "The nominal resistance of piles driven to point bearing on hard rock where pile penetration into the rock formation is minimal is controlled by the structural limit state. The nominal bearing resistance shall not exceed the values obtained from Article 6.9.4.1 with the resistance factors specified in Article 6.5.4.2 and Article 6.15 for severe driving conditions. A pile driving acceptance criteria shall be developed that will prevent pile damage."

Therefore limit the nominal axial geotechnical pile resistance to the nominal structural resistance with a resistance factor for severe driving conditions of 0.50 applied per 10.7.3.2.3.

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

$$\phi_c := 0.5 \qquad P_r := \phi_c \cdot P_n$$

Nominal Structural Resistance Previously Calculated (lower braced segment):

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

The factored **geotechnical compressive resistance** (Pr) for the **Strength Limit State**, per LRFD 6.9.2.1-1 is

$$\phi_c := 0.5$$

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

$$P_r = \begin{pmatrix} 387 \\ 545 \\ 535 \\ 652 \\ 860 \end{pmatrix} \cdot kip$$

12x53
12x74
14x73
14x89
14x117

The factored **geotechnical compressive resistance** (Pr) for the **Extreme and Service Limit States**, per LRFD 6.9.2.1-1 is

$$\phi_c := 1.0$$

$$P_{r_ee} := \phi_c \cdot P_n$$

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

12x53
12x74
14x73
14x89
14x117

Based on past practice, limit the factored geotechnical resistance to:

Based on recent LRFD driven pile projects, limit the factored geotechnical resistance to 75% of the nominal structural pile resistance with a resistance factor for severe driving conditions, of 0.50, applied. Then compute the nominal resistance by dividing by a resistance factor of 0.65 for a pile resistance determination method consisting of at least 2 dynamic load tests with signal matching

$$R_{geo} := P_r \cdot 0.75 \qquad R_{geo} = \begin{pmatrix} 291 \\ 409 \\ 401 \\ 489 \\ 645 \end{pmatrix} \cdot kip$$

Pile Load Test to the following "nominal" resistance

$$\phi_{dyn} := 0.65$$

Reference LRFD Table 10.5.5.2.3-1

$$R_{nominal} := \frac{1}{\phi_{dyn}} \cdot R_{geo} \qquad R_{nominal} = \begin{pmatrix} 447 \\ 629 \\ 617 \\ 753 \\ 992 \end{pmatrix} \cdot kip$$

12x53
12x74
14x73
14x89
14x117

Nominal and Factored Axial Geotechnical Resistance of HP piles

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

Nominal unit bearing resistance of pile point, qp

Design value of compressive strength of rock core

$$q_{u_1} := 15000 \cdot psi$$

Spacing of discontinuities $\quad s_d := 12 \cdot in$

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

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

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

Diameter of socket:

$$D_s := 12 \cdot in$$

Depth factor

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

$$dd = 1$$ OK

Ksp

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

$$K_{sp} = \begin{pmatrix} 0.339 \\ 0.338 \\ 0.324 \\ 0.324 \\ 0.323 \end{pmatrix}$$

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

Geotechnical tip resistance.

$$q_{p_1} := 3 \cdot q_{u_1} \cdot K_{sp} \cdot dd$$

$$\boxed{q_{p_1} = \begin{pmatrix} 2196 \\ 2188 \\ 2101 \\ 2097 \\ 2092 \end{pmatrix} \cdot ksf}$$

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

Case I

$$R_{p_1} := \overrightarrow{\left(q_{p_1} \cdot A_s\right)}$$

$$\boxed{R_{p_1} = \begin{pmatrix} 236 \\ 331 \\ 312 \\ 380 \\ 500 \end{pmatrix} \cdot kip}$$

$\phi = 1.0$

Factored Axial Geotechnical Compressive Resistance - *Strength Limit States*

Resistance factor, end bearing on rock Canadian Geotechnical Society method

$\phi_{stat} := 0.45$ LRFD Table 10.5.5.2.3-1

Factored Geotechnical Tip Resistance (Rr)

$R_{r_p1} := \phi_{stat} \cdot R_{p_1}$

$$R_{r_p1} = \begin{pmatrix} 106 \\ 149 \\ 140 \\ 171 \\ 225 \end{pmatrix} \cdot kip$$

CGS method is superceded by LRFD 10.7.3.2.3 - Piles Driven to Hard Rock

Nominal and Factored Axial Geotechnical Resistance of HP piles

Geotechncial axial pile resistance for pile end bearing on rock determined by Intact Rock Method, proposed by Sandford, MaineDOT Transportation Research Division Technical Report 14-01, Phase 2, (January 2014), in turn based on Rowe and Armitage (1987) equation cited by NCHRP Synthesis 360, Turner, (2006).

Nominal unit bearing resistance of pile point, qp

Design value of compressive strength of rock core

$q_{u_1} := 15000 \cdot psi$

Geotechnical tip resistance.

$q_{p_2} := 2.5 \cdot q_{u_1}$

$$q_{p_1} = \begin{pmatrix} 2196 \\ 2188 \\ 2101 \\ 2097 \\ 2092 \end{pmatrix} \cdot ksf$$

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

$$R_{p_2} := \overrightarrow{\left(q_{p_2} \cdot A_s\right)} \qquad R_{p_2} = \begin{pmatrix} 581 \\ 818 \\ 803 \\ 979 \\ 1290 \end{pmatrix} \cdot kip \qquad \phi = 1.0$$

Factored Axial Geotechnical Compressive Resistance - *Strength Limit States*

Resistance factor, end bearing on rock Canadian Geotechnical Society method

$\phi_{stat} := 0.45$ LRFD Table 10.5.5.2.3-1

Factored Geotechnical Tip Resistance (Rr)

$$R_{r_p2} := \phi_{stat} \cdot R_{p_2} \qquad R_{r_p2} = \begin{pmatrix} 262 \\ 368 \\ 361 \\ 440 \\ 581 \end{pmatrix} \cdot kip$$

Drivability Analyses

Ref: LRFD Article 10.7.8

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

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

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

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

Limit driving stress to 45 ksi or limit blow count to 5-15 blows per inch (bpi) per Section 501 (Note: 6-10 bpi is considered optimal for diesel hammers).

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

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

$\phi_{dyn} := 0.65$ For Strength Limit State, Reference LRFD Table 10.5.5.2.3-1

$\phi := 1.0$ For Extreme Event and Service limit states

GRLWeap Soil and Pile Model Assumptions

Bottom of integral abutment is Elevation 34 ft

Depth of Soil Present at time of driving is Elev. 34 - Elev. 13.4 (approx. top of bedrock) = 20.6 ft

Length of pile required is Elev. 36 - Elev. 13.4 = 22.6 ft

Assume the Contractor drives pile lenghts of 30 ft to account for bedrock uncertainty

Assume side resistance along pile shaft in only the Glacial Till deposit, from El. 27 to El. 13.4 = Lower 13.6 ft.

Use constant shaft resistance at 20% - use min. ultimate capacity of 300 kips, so GRLWeap will assign 60 kip shaft resistance to all ultimate capacities analyzed.

Pile Size is 12 x 53

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

State of Maine Dept. Of Transportation
Abut 2 12x53 Delmag D19-42

22-Jul-2014
GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
300.0	35.78	2.65	3.7	7.20	15.02
400.0	42.27	3.98	5.6	7.94	16.38
420.0	43.97	4.57	6.0	8.16	16.79
450.0	46.38	5.31	6.8	8.44	17.34
460.0	47.14	5.43	7.0	8.53	17.51
470.0	47.66	5.53	7.5	8.53	17.46
500.0	49.68	5.91	8.5	8.76	17.90

Limit stresses to <45 ksi

$R_{ndr} := 420 \cdot kip$

Strength Limit State

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

$R_{fdr} = 273 \cdot kip$

Extreme and Service Limit States

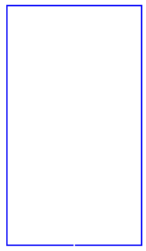
$R_{dr} := R_{ndr} \cdot \phi$

$R_{dr} = 420 \cdot kip$

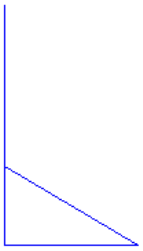
DELMAG D 19-42

Efficiency	0.800	
Helmet	2.70	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	30.00	ft
Pile Penetration	20.00	ft
Pile Top Area	15.50	in2

Pile Model



Skin Friction Distribution



Res. Shaft = 20 %
(Constant Res. Shaft)

Pile Size is 12 x 74

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

State of Maine Dept. Of Transportation — 22-Jul-2014
Abut2 12x 74 Delmag D19-42 — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
300.0	29.05	0.87	3.8	7.05	14.04
400.0	34.42	3.23	5.4	7.65	14.94
520.0	40.93	4.95	8.0	8.53	16.34
530.0	41.48	4.98	8.3	8.62	16.45
540.0	41.99	5.04	8.6	8.70	16.62
550.0	42.51	5.07	8.9	8.78	16.73
570.0	43.26	5.17	9.7	8.85	16.79
600.0	44.66	5.34	11.0	9.05	17.08

Limit driving stresses to < 45 ksi

$R_{ndr} := 600 \cdot kip$

Strength Limit State

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

$R_{fdr} = 390 \cdot kip$

Extreme and Service Limit States

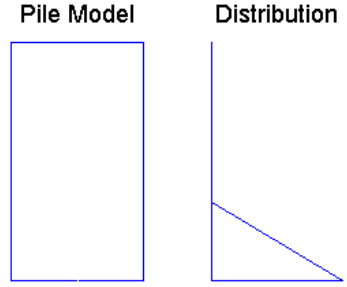
$R_{dr} := R_{ndr} \cdot \phi$

$R_{dr} = 600 \cdot kip$

DELMAG D 19-42

Efficiency	0.800	
Helmet	2.70	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	30.00	ft
Pile Penetration	20.00	ft
Pile Top Area	21.80	in2

Pile Model — Skin Friction Distribution



Res. Shaft = 20 %
(Constant Res. Shaft)

Pile Size is 14 x 73

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

State of Maine Dept. Of Transportation
Abut 2 14 x 73 Delmag D19-42

22-Jul-2014
GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
300.0	29.37	0.94	3.8	7.05	14.06
520.0	41.34	5.22	8.1	8.56	16.44
530.0	41.92	5.31	8.3	8.65	16.60
540.0	42.45	5.36	8.6	8.73	16.74
550.0	42.96	5.40	8.9	8.81	16.87
570.0	43.72	5.50	9.8	8.87	16.92
580.0	44.18	5.54	10.1	8.94	17.01
590.0	44.63	5.57	10.6	9.01	17.11

Limit driving stress to <45 ksi and 10 bpi
(15 bpi will overstress the pile).

$R_{ndr} := 570 \cdot kip$

Strength Limit State

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

$R_{fdr} = 371 \cdot kip$

Extreme and Service Limit States

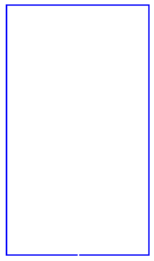
$R_{dr} := R_{ndr} \cdot \phi$

$R_{dr} = 570 \cdot kip$

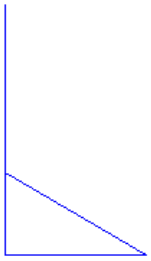
DELMAG D 19-42

Efficiency	0.800	
Helmet	2.70	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	30.00	ft
Pile Penetration	20.00	ft
Pile Top Area	21.40	in2

Pile Model



Skin Friction Distribution



Res. Shaft = 20 %
(Constant Res. Shaft)

Pile Size is 14 x 89

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

State of Maine Dept. Of Transportation
14 x 89 Delmag D19-42 2.7 kip helmet

11-Apr-2014
GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
300.0	25.49	0.49	4.0	7.02	13.69
400.0	30.52	2.03	5.5	7.51	14.29
600.0	39.91	3.94	10.7	8.74	16.35
680.0	43.21	4.45	14.0	9.26	17.57
700.0	43.92	4.59	14.8	9.38	17.87
710.0	44.34	4.62	15.3	9.45	18.01
720.0	44.74	4.66	15.7	9.52	18.19

Limiting driving resistance to 15 bpi and driving stress to <45 ksi

$R_{ndr} := 700 \cdot kip$

DELMAG D 19-42

Efficiency	0.800	
Helmet	2.70	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	30.00	ft
Pile Penetration	20.00	ft
Pile Top Area	26.10	in2

Strength Limit State

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

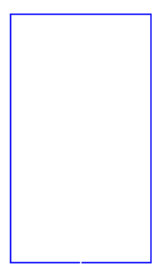
$R_{fdr} = 455 \cdot kip$

Extreme and Service Limit States

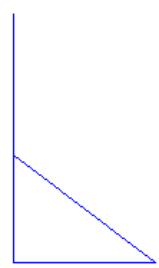
$R_{dr} := R_{ndr} \cdot \phi$

$R_{dr} = 700 \cdot kip$

Pile Model



Skin Friction Distribution



Res. Shaft = 20 %
(Constant Res. Shaft)

Pile Size is 14 x 117

The 14 x 117 pile can be driven to the resistances below with a D 19-42 at a reasonable blow count and level of driving stress. See GRLWEAP results below:

State of Maine Dept. Of Transportation
14 x 117 APE D 19-42 Abut 2

11-Apr-2014
GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
300.0	23.40	0.30	3.3	7.35	17.31
400.0	27.76	0.78	4.5	7.73	17.59
700.0	39.68	3.83	9.7	9.23	20.45
800.0	42.94	4.37	12.6	9.75	21.76
830.0	43.95	4.50	13.5	9.94	22.28
840.0	44.26	4.50	13.9	10.00	22.40
850.0	44.65	4.53	14.2	10.07	22.62

APE D 19-42

Blow counts are rounded up to the nearest whole number. Select resistance achievable at 14 bpi and limit driving stress to <45 ksi

Efficiency	0.800	
Helmet	2.70	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	30.00	ft
Pile Penetration	20.00	ft
Pile Top Area	34.40	in2

$R_{ndr} := 840 \cdot kip$

Strength Limit State

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

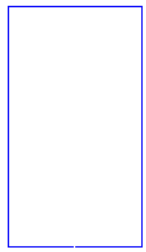
$R_{fdr} = 546 \cdot kip$

Extreme and Service Limit States

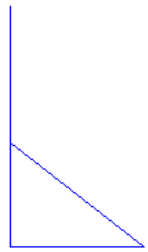
$R_{dr} := R_{ndr} \cdot \phi$

$R_{dr} = 840 \cdot kip$

Pile Model



Skin Friction Distribution



Res. Shaft = 20 %
(Constant Res. Shaft)

Pile Size is 14 x 117

The 14 x 117 pile can be driven to the resistances below with a D 36-32 at Fuel Setting 3 (1174 psi) and a 2.7 kip helmet, at a reasonable blow count and level of driving stress. See GRLWEAP results below:

State of Maine Dept. Of Transportation
14 x 117 Delmag D36-32 setting 3

11-Apr-2014
GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
300.0	26.56	0.00	1.9	5.99	26.47
400.0	30.03	0.00	2.7	6.44	25.69
500.0	34.76	0.77	3.5	6.77	25.93
600.0	39.55	0.92	4.3	7.20	27.19
660.0	41.85	1.20	4.9	7.40	27.69
670.0	42.36	1.27	5.0	7.43	27.78
720.0	44.19	1.78	5.6	7.60	28.16
740.0	45.04	2.15	5.8	7.70	28.55

Blow count of 6 bpi will overstress the pile. Select 5 bpi.

$R_{ndr} := 670 \cdot kip$

Strength Limit State

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

$R_{fdr} = 436 \cdot kip$

Extreme and Service Limit States

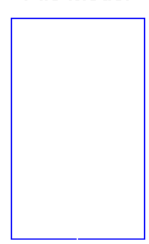
$R_{dr} := R_{ndr} \cdot \phi$

$R_{dr} = 670 \cdot kip$

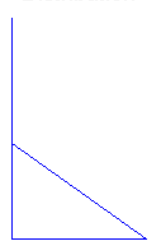
DELMAG D 36-32

Efficiency	0.800	
Helmet	2.70	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	30.00	ft
Pile Penetration	20.00	ft
Pile Top Area	34.40	in2

Pile Model



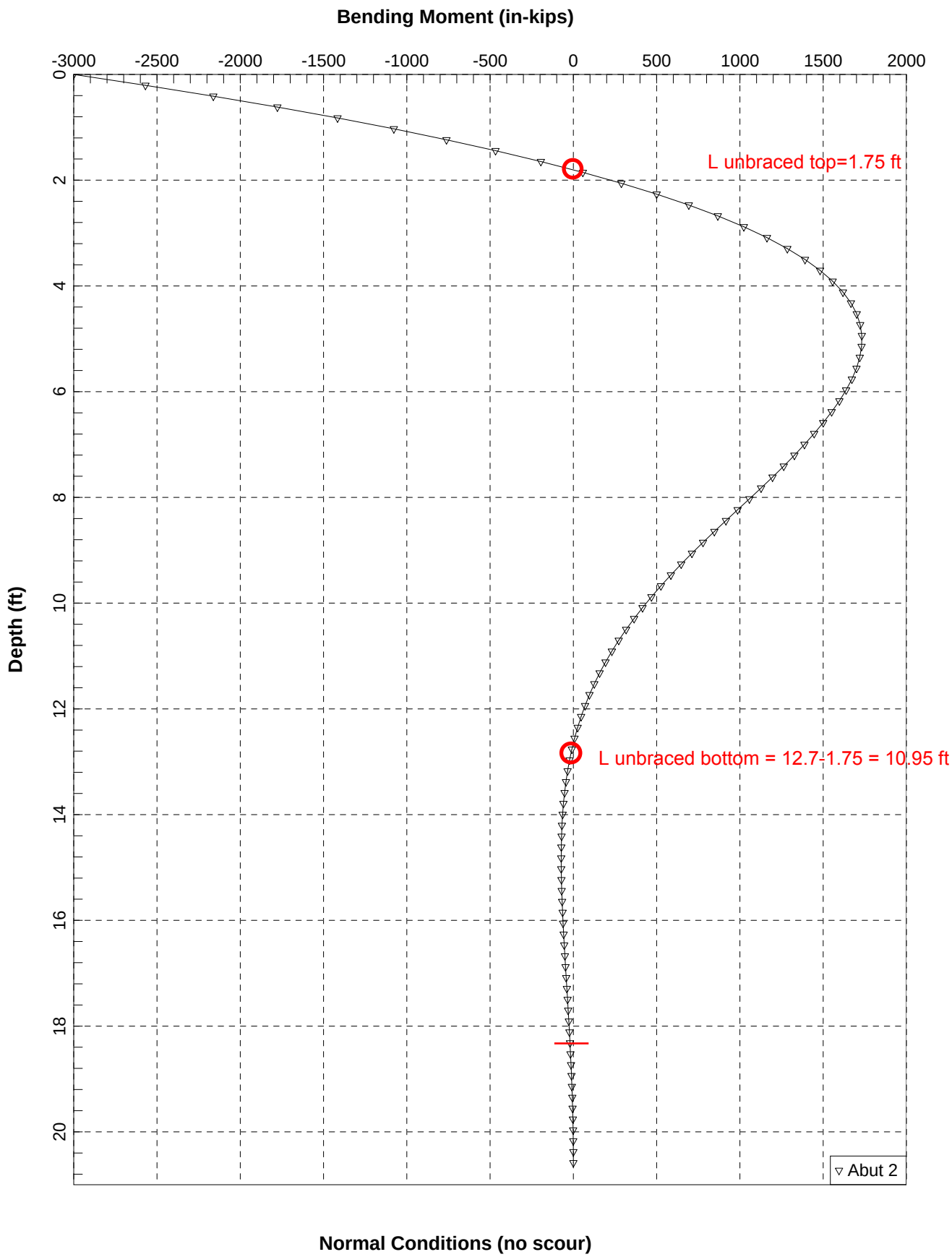
Skin Friction Distribution

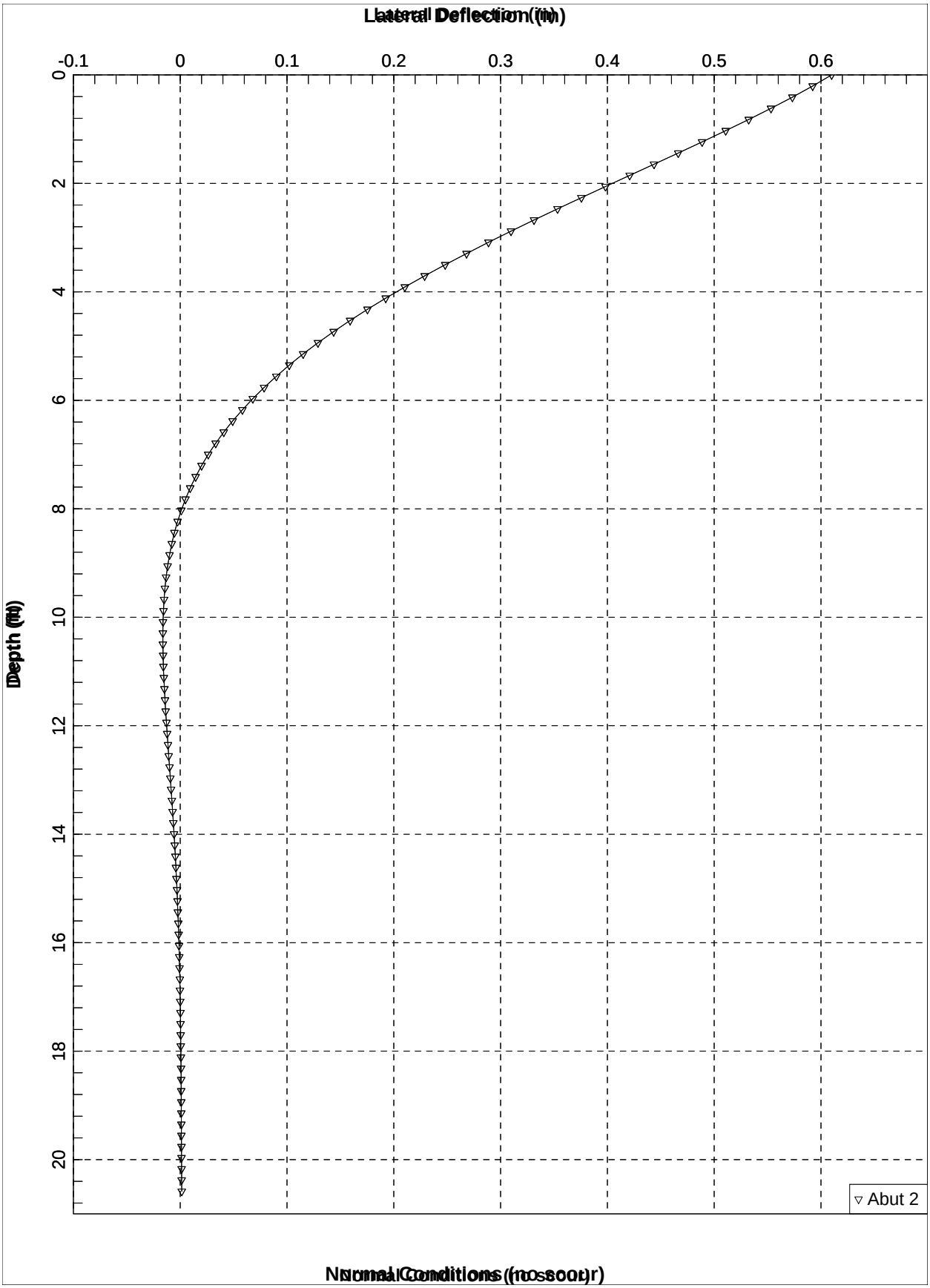


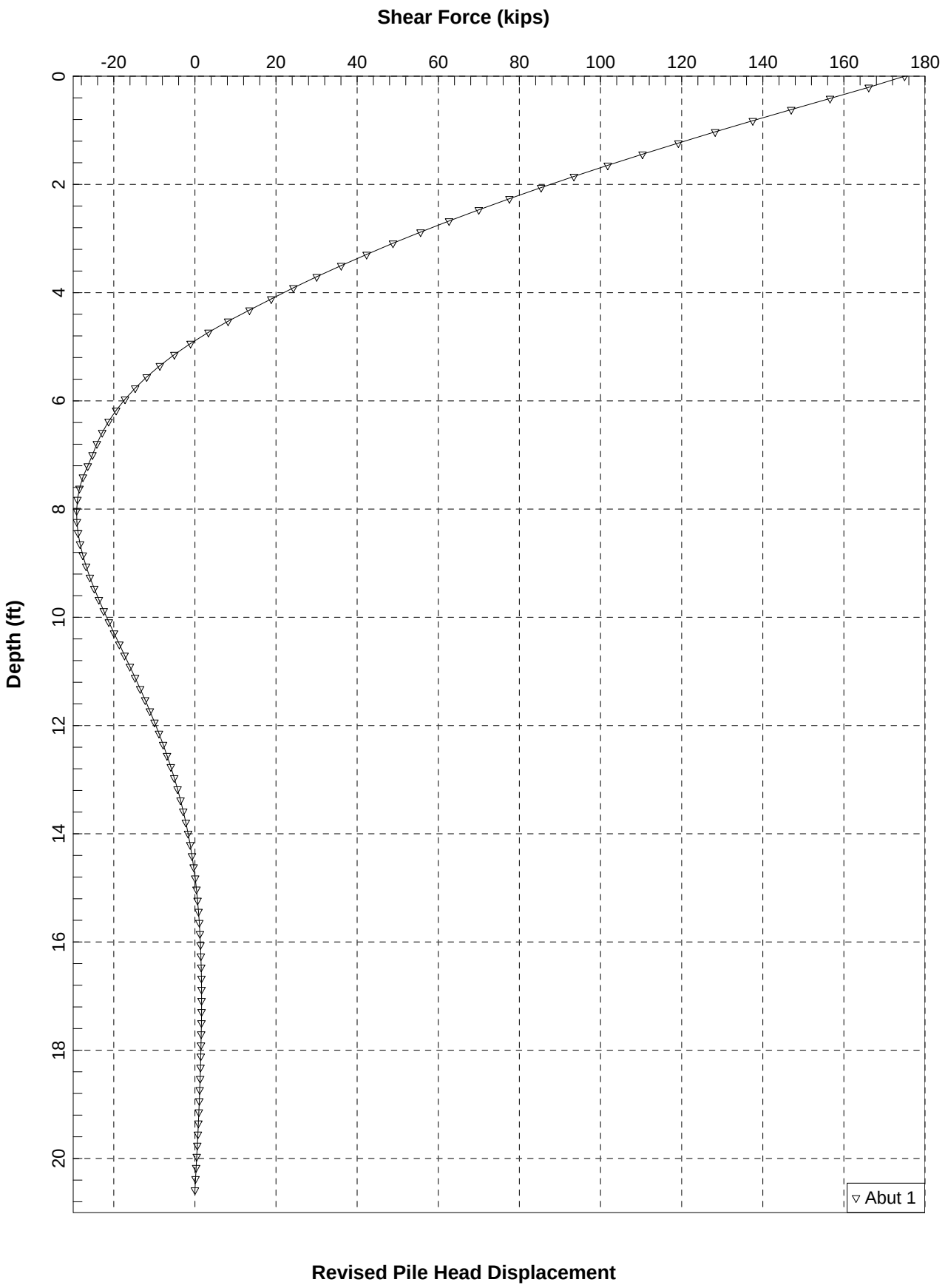
Res. Shaft = 20 %
(Constant Res. Shaft)

Abutment No. 2 Lpile Analyses

Abut 2 rev 3 - 14x117 - reduced plastic moment of 250 ft-kip applied




Lateral Deflection (in)
-0.1
0
0.1
0.2
0.3
0.4
0.5
0.6
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
Abut 2
Normal Conditions (no scour)


Shear Force (kips)
-20
0
20
40
60
80
100
120
140
160
180
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
Abut 1
Revised Pile Head Displacement

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LPILE Plus for Windows, Version 5.0 (5.0.39)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

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==========

This program is licensed to:

L. Krusinski
Maine Department of Transportation

Path to file locations: C:\Program Files\Ensoft\Lpilep5\
Name of input data file: EMachias_Ab2_117_rev3.lpd
Name of output file: EMachias_Ab2_117_rev3.lpo
Name of plot output file: EMachias_Ab2_117_rev3.lpp
Name of runtime file: EMachias_Ab2_117_rev3.lpr

Time and Date of Analysis

Date: July 23, 2014 Time: 15:10:52

Problem Title

Abut 1 14x117_16 foot pile - factored loads

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis does not use p-y multipliers (individual pile or shaft action only)
- Analysis assumes no shear resistance at pile tip
- Analysis for fixed-length pile or shaft only
- No computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 100
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and
  soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) = 1

Pile Structural Properties and Geometry

Pile Length = 247.00 in

Depth of ground surface below top of pile = -144.00 in

Slope angle of ground surface = .00 deg.

Structural properties of pile defined using 2 points

Point	Depth X in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	14.88500000	443.0000	34.4000	29000000.
2	247.0000	14.88500000	443.0000	34.4000	29000000.

Soil and Rock Layering Information

The soil profile is modelled using 4 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = -144.000 in
Distance from top of pile to bottom of layer = 1.000 in
p-y subgrade modulus k for top of soil layer = 60.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 60.000 lbs/in**3

Layer 2 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 1.000 in
Distance from top of pile to bottom of layer = 50.000 in
p-y subgrade modulus k for top of soil layer = 50.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 50.000 lbs/in**3

Layer 3 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 50.000 in
Distance from top of pile to bottom of layer = 84.000 in
p-y subgrade modulus k for top of soil layer = 60.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 60.000 lbs/in**3

Layer 4 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 84.000 in
Distance from top of pile to bottom of layer = 247.000 in
p-y subgrade modulus k for top of soil layer = 120.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 120.000 lbs/in**3

(Depth of lowest layer extends .00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Effective unit weight of soil with depth defined using 8 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	-144.00	.07000
2	1.00	.07000
3	1.00	.03600
4	50.00	.03600
5	50.00	.03600
6	84.00	.03600
7	84.00	.03900
8	247.00	.03900

Shear Strength of Soils

Shear strength parameters with depth defined using 8 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	-144.000	.00000	32.00	------	------
2	1.000	.00000	32.00	------	------
3	1.000	.00000	30.00	------	------
4	50.000	.00000	30.00	------	------
5	50.000	.00000	32.00	------	------
6	84.000	.00000	32.00	------	------
7	84.000	.00000	34.00	------	------
8	247.000	.00000	34.00	------	------

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

Loading Type

Static loading criteria was used for computation of p-y curves.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load Case Number 1

Pile-head boundary conditions are Displacement and Moment (BC Type 4)
Deflection at pile head = .610 in
Bending moment at pile head = -3000000.000 in-lbs
Axial load at pile head = 478000.000 lbs

plastic moment (combined flexure and axial loading) = 250 ft-kip

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Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Displacement and Moment (BC Type 4)
Specified deflection at pile head = .610000 in
Specified moment at pile head = -3000000.000 in-lbs
Specified axial load at pile head = 478000.000 lbs

Depth X in	Deflect. y in	Moment M lbs-in	Shear V lbs	Slope S Rad.	Stress lbs/in**2	Total p lbs/in	Soil Res. F/L lbs/in	Es*h
0.000	.610000	-3000000.	174944.	-.0069101	64296.0260	-3303.1968	6687.6197	
2.470	.592220	-2569465.	166120.	-.0074455	57062.9347	-3841.6727	16022.6548	
4.940	.573219	-2161784.	156590.	-.0079003	50213.8024	-3875.1546	16698.0324	
7.410	.553192	-1777254.	147011.	-.0082790	43753.6157	-3881.4446	17330.6342	
9.880	.532321	-1416001.	137524.	-.0085860	37684.4858	-3800.1234	17632.7952	
12.350	.510777	-1077611.	128245.	-.0088257	31999.4552	-3713.1334	17955.8486	
14.820	.488722	-761630.	119187.	-.0090025	26690.8981	-3621.2778	18301.9325	
17.290	.466305	-467568.	110361.	-.0091207	21750.6048	-3525.3297	18673.5409	
19.760	.443666	-194909.	101776.	-.0091843	17169.8623	-3426.0310	19073.5801	
22.230	.420934	56893.0575	93439.7818	-.0091976	14851.1650	-3324.0918	19505.4373	
24.700	.398230	288402.	85357.5952	-.0091644	18740.5699	-3220.1889	19973.0645	

64.2 ksi

X=approx. 21"

27.170	.375662	500200.	77533.6797	-.0090886	22298.8169	-3114.9654	20481.0827	
29.640	.353332	692880.	69970.5450	-.0089739	25535.8833	-3009.0303	21034.9101	
32.110	.331331	867045.	62669.2403	-.0088240	28461.8922	-2902.9573	21640.9212	
34.580	.309742	1023302.	55629.4409	-.0086422	31087.0516	-2797.2851	22306.6452	
37.050	.288638	1162261.	48849.5356	-.0084321	33421.5966	-2692.5167	23041.0150	
39.520	.268087	1284530.	42326.7152	-.0081969	35475.7357	-2589.1192	23854.6830	
41.990	.248145	1390711.	36057.0611	-.0079398	37259.6005	-2487.5237	24760.4226	
44.460	.228864	1481400.	30037.9320	-.0076637	38783.1992	-2386.2650	25753.5690	
46.930	.210287	1557194.	24270.4596	-.0073715	40056.5646	-2283.7532	26824.6390	
49.400	.192449	1618702.	18830.9109	-.0070662	41089.9147	-2120.7398	27218.7912	
51.870	.175380	1666905.	13455.4901	-.0067504	41899.7234	-2231.8277	31432.4621	
54.340	.159102	1701112.	8169.5795	-.0064266	42474.4207	-2048.2619	31798.5161	
56.810	.143632	1722438.	3330.0430	-.0060975	42832.6910	-1870.3911	32164.5701	
59.280	.128980	1731961.	-1077.7944	-.0057654	42992.6840	-1698.7080	32530.6241	
61.750	.115151	1730727.	-5069.7396	-.0054326	42971.9604	-1533.6363	32896.6781	
64.220	.102143	1719744.	-8662.5649	-.0051009	42787.4441	-1375.5340	33262.7321	
66.690	.089953	1699979.	-11873.8480	-.0047721	42455.3820	-1224.6953	33628.7861	
69.160	.078569	1672356.	-14721.8193	-.0044479	41991.3099	-1081.3543	33994.8401	
71.630	.067980	1637756.	-17225.2157	-.0041297	41410.0251	-945.6873	34360.8941	
74.100	.058168	1597015.	-19403.1428	-.0038188	40725.5649	-817.8165	34726.9481	
76.570	.049115	1550922.	-21274.9448	-.0035161	39951.1909	-697.8127	35093.0021	
79.040	.040798	1500220.	-22860.0815	-.0032228	39099.3791	-585.6989	35459.0561	
81.510	.033194	1445603.	-24178.0143	-.0029397	38181.8139	-481.4533	35825.1101	
83.980	.026277	1387722.	-25248.0997	-.0026673	37209.3884	-385.0126	36191.1641	
86.450	.020018	1327176.	-26414.8392	-.0024063	36192.2084	-559.7157	69062.9674	
88.920	.014390	1262915.	-27608.2468	-.0021573	35112.5981	-406.6062	69795.0754	
91.390	.009361	1195886.	-28440.5024	-.0019209	33986.4958	-267.2850	70527.1834	
93.860	.004900	1126954.	-28945.1877	-.0016976	32828.4371	-141.3670	71259.2914	
96.330	.000975	1056905.	-29154.8539	-.0014877	31651.5911	-28.4032	71991.3994	
98.800	-.002449	986442.	-29100.8759	-.0012913	30467.8032	72.1101	72723.5074	
101.270	-.005404	916196.	-28813.3289	-.0011084	29287.6444	160.7215	73455.6154	
103.740	-.007925	846722.	-28320.8872	-.0009389	28120.4651	238.0167	74187.7234	
106.210	-.010043	778508.	-27650.7440	-.0007827	26974.4525	304.6093	74919.8314	
108.680	-.011791	711975.	-26828.5509	-.0006394	25856.6924	361.1341	75651.9394	
111.150	-.013201	647484.	-25878.3746	-.0005087	24773.2321	408.2394	76384.0474	
113.620	-.014304	585337.	-24822.6719	-.0003902	23729.1457	446.5806	77116.1554	
116.090	-.015129	525782.	-23682.2786	-.0002834	22728.6000	476.8147	77848.2634	
118.560	-.015704	469016.	-22476.4139	-.0001877	21774.9210	499.5941	78580.3714	
121.030	-.016056	415192.	-21222.6956	-.0001027	20870.6600	515.5624	79312.4794	
123.500	-.016211	364418.	-19937.1688	-2.7790E-05	20017.6590	525.3500	80044.5874	
125.970	-.016193	316768.	-18634.3426	3.7693E-05	19217.1147	529.5700	80776.6954	
128.440	-.016025	272276.	-17327.2366	9.4318E-05	18469.6415	528.8154	81508.8034	
130.910	-.015727	230948.	-16027.4349	.0001427	17775.3317	523.6557	82240.9114	
133.380	-.015320	192763.	-14745.1453	.0001834	17133.8140	514.6354	82973.0194	
135.850	-.014821	157674.	-13489.2653	.0002171	16544.3095	502.2715	83705.1274	
138.320	-.014247	125614.	-12267.4507	.0002443	16005.6848	487.0521	84437.2354	
140.790	-.013614	96495.9238	-11086.1882	.0002657	15516.5021	469.4357	85169.3434	
143.260	-.012935	70220.3773	-9950.8700	.0002817	15075.0670	449.8502	85901.4514	
145.730	-.012222	46673.3822	-8865.8698	.0002930	14679.4722	428.6925	86633.5594	
148.200	-.011488	25731.2022	-7834.6192	.0002999	14327.6388	406.3283	87365.6674	
150.670	-.010741	7262.1498	-6859.6840	.0003031	14017.3546	383.0928	88097.7754	
153.140	-.009990	-8871.3400	-5942.8405	.0003029	14044.3894	359.2905	88829.8834	X=approx. 12.7 ft
155.610	-.009244	-22810.8203	-5085.1494	.0002999	14278.5758	335.1962	89561.9914	
158.080	-.008509	-34700.1242	-4287.0286	.0002944	14478.3188	311.0555	90294.0994	
160.550	-.007790	-44683.8327	-3548.3235	.0002867	14646.0473	287.0862	91026.2074	
163.020	-.007092	-52905.9137	-2868.3753	.0002774	14784.1801	263.4791	91758.3154	
165.490	-.006420	-59508.5256	-2246.0856	.0002665	14895.1055	240.3992	92490.4234	
167.960	-.005776	-64630.9775	-1679.9782	.0002546	14981.1638	217.9873	93222.5314	
170.430	-.005162	-68408.8397	-1168.2582	.0002418	15044.6328	196.3609	93954.6394	
172.900	-.004581	-70973.1949	-708.8664	.0002284	15087.7145	175.6162	94686.7474	
175.370	-.004034	-72450.0224	-299.5314	.0002146	15112.5256	155.8291	95418.8554	
177.840	-.003521	-72959.7061	62.1828	.0002007	15121.0884	137.0568	96150.9634	
180.310	-.003043	-72616.6581	378.8323	.0001867	15115.3251	119.3395	96883.0714	
182.780	-.002599	-71529.0477	653.0530	.0001728	15097.0530	102.7015	97615.1794	
185.250	-.002189	-69798.6287	887.5233	.0001592	15067.9816	87.1529	98347.2874	
187.720	-.001812	-67520.6542	1084.9306	.0001460	15029.7111	72.6910	99079.3954	
190.190	-.001468	-64783.8722	1247.9413	.0001333	14983.7325	59.3015	99811.5034	
192.660	-.001154	-61670.5920	1379.1743	.0001211	14931.4287	46.9600	100544.	
195.130	-.000869	-58256.8145	1481.1769	.0001096	14874.0765	35.6333	101276.	
197.600	-.000612	-54612.4179	1556.4053	9.8766E-05	14812.8498	25.2804	102008.	
200.070	-.000381	-50801.3914	1607.2061	8.8633E-05	14748.8237	15.8539	102740.	
202.540	-.000174	-46882.1099	1635.8027	7.9242E-05	14682.9789	7.3012	103472.	
205.010	1.03E-05	-42907.6423	1644.2827	7.0611E-05	14616.2069	-.4348680	104204.	
207.480	.000175	-38926.0879	1634.5885	6.2744E-05	14549.3159	-7.4147	104936.	

209.950	.000320	-34980.9337	1608.5105	5.5639E-05	14483.0364	-13.7011	105668.
212.420	.000449	-31111.4279	1567.6823	4.9286E-05	14418.0278	-19.3582	106400.
214.890	.000564	-27352.9624	1513.5777	4.3665E-05	14354.8848	-24.4511	107133.
217.360	.000665	-23737.4616	1447.5107	3.8754E-05	14294.1436	-29.0445	107865.
219.830	.000755	-20293.7701	1370.6357	3.4521E-05	14236.2888	-33.2024	108597.
222.300	.000836	-17048.0364	1283.9518	3.0931E-05	14181.7597	-36.9871	109329.
224.770	.000908	-14024.0874	1188.3062	2.7944E-05	14130.9567	-40.4587	110061.
227.240	.000974	-11243.7897	1084.4016	2.5515E-05	14084.2470	-43.6745	110793.
229.710	.001034	-8727.3936	972.8036	2.3596E-05	14041.9710	-46.6882	111525.
232.180	.001090	-6493.8566	853.9503	2.2132E-05	14004.4471	-49.5493	112257.
234.650	.001143	-4561.1405	728.1632	2.1070E-05	13971.9770	-52.3026	112989.
237.120	.001194	-2946.4820	595.6598	2.0348E-05	13944.8504	-54.9877	113722.
239.590	.001244	-1666.6290	456.5669	1.9904E-05	13923.3486	-57.6381	114454.
242.060	.001293	-738.0423	310.9366	1.9673E-05	13907.7481	-60.2811	115186.
244.530	.001341	-177.0567	158.7630	1.9585E-05	13898.3234	-62.9365	115918.
247.000	.001389	0.0000	0.0000	1.9568E-05	13895.3488	-65.6166	58324.9937

no portion of pile is in full axial compression

Output Verification:

Computed forces and moments are within specified convergence limits.

Output Summary for Load Case No. 1:

Pile-head deflection = .61000000 in
Computed slope at pile head = -.00691012
Maximum bending moment = -3000000. lbs-in
Maximum shear force = 174944.37616 lbs
Depth of maximum bending moment = 0.00000 in
Depth of maximum shear force = 0.00000 in
Number of iterations = 6
Number of zero deflection points = 2

Summary of Pile Response(s)

Definition of Symbols for Pile-Head Loading Conditions:

Type 1 = Shear and Moment, y = pile-head displacment in
Type 2 = Shear and Slope, M = Pile-head Moment lbs-in
Type 3 = Shear and Rot. Stiffness, V = Pile-head Shear Force lbs
Type 4 = Deflection and Moment, S = Pile-head Slope, radians
Type 5 = Deflection and Slope, R = Rot. Stiffness of Pile-head in-lbs/rad

Load Type	Pile-Head Condition 1	Pile-Head Condition 2	Axial Load lbs	Pile-Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
4	y= .610000	M= -3.00E+06	478000.	.6100000	-3000000.	174944.

The analysis ended normally.

Earth Pressure Coefficients

Frost Depth

Seismic Parameters

Backfill engineering strength parameters

Soil Type 4 Properties from Bridge Design Guide (BDG)

Unit weight $\gamma_1 := 125 \cdot pcf$

Internal friction angle $\phi_1 := 32 \cdot deg$

Cohesion $c_1 := 0 \cdot psf$

Active Earth Pressure for Return Wingwalls

Rankine Theory

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. The earth pressure is applied to a plane extending vertically up from the heel of the wall base, and the weight of the soil on the inside of the vertical plane is considered as part of the wall weight. The failure sliding surface is not restricted by the top of the wall or back face of wall.

- For cantilever walls with horizontal backslope

$$K_a := \tan\left(45 \cdot deg - \frac{\phi_1}{2}\right)^2 \qquad K_a = 0.307$$

- For a sloped backfill

β = Angle of fill slope to the horizontal

$\beta := 0 \cdot deg$

$$K_{aslope} := \frac{\cos(\beta) - \sqrt{\cos(\beta)^2 - \cos(\phi_1)^2}}{\cos(\beta) + \sqrt{\cos(\beta)^2 - \cos(\phi_1)^2}} \qquad K_{aslope} = 0.307$$

- Pa is oriented at an angle of β to the vertical plane

Coulomb Theory

- Coulomb theory applies for gravity, semigravity and prefab modular walls with steep back faces
- Coulomb theory also applies to concrete cantilever walls with short heels where the sliding surface in restricted by the top of wall - the wedge of soil does not move.

Angle of back face of wall to the horizontal, θ :

$\theta := 90 \cdot deg$

Friction angle between fill and wall, δ :

Per LRFD Table 3.11.5.3-1, for "Clean sand, silty sand-gravel mixture, single-size hard rock fill against Formed or precast concrete" δ = 17 to 22 degrees; select 20 degrees.

$\delta := 20 \cdot deg$ for a gravity shaped wall where the interface friction is between soil and concrete

to $\delta := 24 \cdot deg$ per BDG Table 3-3

Per LRFD Figure C3.11.5.3-1, for a cantilever wall where the sliding surface is a plane from the footing heel to the top of the wall, δ=1/3 to 2/3 Φ

$$\delta := \frac{2}{3} \cdot \phi_1$$

$$\delta = 21.333 \cdot deg$$

(If δ is taken as 0 and the slope of the backslope is horizontal, there is no difference in the active earth pressure coefficient when using either Rankine or Coulomb)

$$K_{ac} := \frac{\sin(\theta + \phi_1)^2}{\sin(\theta)^2 \cdot \sin(\theta - \delta) \cdot \left(1 + \sqrt{\frac{\sin(\phi_1 + \delta) \cdot \sin(\phi_1 - \beta)}{\sin(\theta - \delta) \cdot \sin(\theta + \beta)}}\right)^2} \qquad K_{ac} = 0.275$$

Orientation of Coulomb Pa

- In the case of gravity shaped walls and prefab walls, Pa is oriented δ degrees up from a perpendicular line to the backface or 'pressure surface.'

At-rest Earth Pressure

Reference LRFD Article 3.11.5.2

There is no estimation of at-rest earth pressure which considers sloped backfill.

- For vertical walls with level backslope

$$K_o := 1 - \sin(\phi_1) \qquad K_o = 0.47$$

Integral Abutments Passive Earth Pressure - Coulomb Theory

β = Angle of fill slope to the horizontal $\beta := 0 \cdot deg$

$\phi_1 := 32 \cdot deg$

θ = Angle of back face of wall to the horizontal $\theta := 90 \cdot deg$

For cases where interface friction is considered (this is for gravity shaped structures), use Coulomb.

For IAB abutment against clean sand, silty sand-gravel mixture use δ = 17 - 22, per LRFD Table 3.11.5.3-1 - because of the interface of the integral abutment backface and backfill soil

δ = friction angle between fill and wall taken as specified in LRFD Table 3.11.5.3-1 (degrees)

$\delta := 19.5 \cdot deg$

$$K_{p_coul} := \frac{\sin(\theta - \phi_1)^2}{\sin(\theta)^2 \cdot \sin(\theta + \delta) \cdot \left(1 - \sqrt{\frac{\sin(\phi_1 + \delta) \cdot \sin(\phi_1 + \beta)}{\sin(\theta + \delta) \cdot \sin(\theta + \beta)}}\right)^2}$$

$K_{p_coul} = 6.73$

Integral Abutment and Wingwall - Passive Earth Pressure - Rankine Theory

Bowles does not recommend use of Rankine method for Kp when B>0.

β = Angle of fill slope to the horizontal

$\beta := 0 \cdot deg$

$$K_{p_rank} := \frac{\cos(\beta) + \sqrt{\cos(\beta)^2 - \cos(\phi_1)^2}}{\cos(\beta) - \sqrt{\cos(\beta)^2 - \cos(\phi_1)^2}}$$

$K_{p_rank} = 3.255$

Pp is oriented at an angle of β to the vertical plane

Method 1 - 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 degree-days.
Case 1 - coarse grained (sands) W=10% .

Interpolate between frost depth of 69.8 inches at 1100 DFI and 73.1 inches at 1200 DFI

Depth of Frost Penetration =

$$d := \frac{73.1 - 68.9}{100} \cdot 50 \cdot in + 69.8 \cdot in \qquad d = 5.992 \cdot ft$$

Method 2 - ModBerg Software

Examine potential foundations placed on coarse-grained glaciomarine outwash deposit; use ModBerg weather database information for Eastport which is on a DFI contour similar to East Machias

--- ModBerg Results ---

Project Location: Eastport, Maine

Air Design Freezing Index = 1102 F-days
N-Factor = 0.80
Surface Design Freezing Index = 882 F-days
Mean Annual Temperature = 43.5 deg F
Design Length of Freezing Season = 122 days

Layer #:Type	t	w%	d	Cf	Cu	Kf	Ku	L
1-Coarse	63.3	10.0	125.0	28	34	2.0	1.6	1,800

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

Total Depth of Frost Penetration = 5.27 ft = 63.3 in.

Recommendation: use 5.7 feet for design of foundations constructed on native soils or granular fill soils

East Machias
Jacksonville Bridge
20501.00

Site Classification

Date: July 23, 2014
By: L. Krusinski
Check by: KM
Date Reviewed: 9/16/2014

BB-EMR-101					BB-EMR-201				
Depth	**SPT N**		**di**	**di/N**	**Depth**	**SPT N**		**di**	**di/N**
6	24	Fill	9	0.38	1	27	fill	10	0.37
6	8	Fill	5	0.63	6	11	fill	5.2	0.47
11	11	Alluvium	2.5	0.23	11	21	sand	2.8	0.13
16	56	Till	9.6	0.17	15	66	till	7	0.11
					19	40	till	6.6	0.17
27.1	100	**bedrock**	72.9	0.73	31.6	100	**bedrock**	68.4	0.68
SUM			**99**	**2.13**				**100**	**1.93**
			di/di/N	**46.53**				**di/di/N**	**51.77**

Note: Weight of rod (WOR) and weight of hammer (WHO) values are taken as N=1.

SUM	Nav.	49.15

Conclusion: Site Class D

Site Classification per LRFD Table C310.3.1-1 - Method B

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years
State - Maine
Zip Code - 04630
Zip Code Latitude = 44.762900
Zip Code Longitude = -067.374000
Site Class B
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.077	PGA - Site Class B
0.2	0.152	Ss - Site Class B
1.0	0.040	S1 - Site Class B

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Spectral Response Accelerations SDs and SD1
State - Maine
Zip Code - 04630
Zip Code Latitude = 44.762900
Zip Code Longitude = -067.374000
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	0.123	As - Site Class D
0.2	0.242	SDs - Site Class D
1.0	0.095	SD1 - Site Class D

Appendix D

Special Provision

SPECIAL PROVISION
SECTION 501
FOUNDATION PILES
(Rock Injector Pile Tip)

Subsection 501.10 Prefabricated Pile Tips of the Standard Specifications is amended as follows:

Pile tips for use on all piles shall be Rock Injector HP-80500 Pile Point, manufactured by Associated Pile and Fitting or approved equal. Material specifications, attachment of pile tips and seating of the piles shall be in accordance with Manufacturer's recommendations and in accordance with the Standard Specifications.

Payment will be made under:

Pay Item	Description	Pay Unit
501.903	Pile Tips – Rock Injector Point	Each