GEOTECHNICAL DESIGN REPORT
IRON BRIDGE
ISLAND FALLS, MAINE

P.I.N. 15097.00

Prepared For:

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This report presents the results of the geotechnical investigation performed for the Maine Department of Transportation (MaineDOT) for the proposed replacement of the Iron Bridge (MaineDOT Bridge Identification Number 2403). The MaineDOT Project Identification Number (PIN) is 15097.00. The bridge is located in Island Falls, Maine, and crosses the West Branch of the Mattawamkeag River in Island Falls.

The objectives of the geotechnical investigation were to characterize the subsurface conditions at the bridge site and to develop foundation design recommendations to support the proposed replacement structure.

A total of three test borings have been drilled for the purposes of this report by URS Corporation (URS). At the abutment locations for the bridge replacement structure, approximately 14 to 17 feet of medium dense silty sand and/or gravel fill and alluvium overlies the bedrock surface. The entire site is underlain by competent phyllite bedrock. Estimated groundwater levels at the time of drilling indicate that groundwater was similar to the river elevation, at approximately elevation 441 feet.

URS understands that complete replacement of the existing, four-span bridge with a two-span structure at the same location and of the same length is proposed. We further understand that the existing abutments will rehabilitated and reused and a new center pier will be built. The existing abutments and center pier are supported by shallow foundations on bedrock.

Shallow spread footing foundations on bedrock can be used for the abutments, center pier and wingwalls. Bedrock is approximately 15 feet below finished grade. The abutments, pier and wingwalls shall be proportioned for all applicable load combinations in AASHTO LRFD (2008) Articles 3.4.1 and 11.5.5, and shall be designed for the relevant strength, service and extreme event limit states. The design of the abutments, pier and wingwalls founded on spread footings at the strength limit state shall consider nominal bearing resistance, eccentricity (i.e., overturning), lateral sliding and structural failure.

Substructure spread footings for the abutments, pier and wingwalls shall be proportioned to provide stability against bearing capacity failure. The factored bearing resistance for any structure founded on competent, sound bedrock shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 7 kips per square foot (ksf) for footings that are 4 to 9 feet wide and 10 ksf for footings that are 9 to 12 feet wide. This assumes a bearing resistance factor, φ_b, of 0.45 for spread footings on bedrock, based on a bearing resistance evaluation using semi-empirical methods. A factored bearing resistance of 16 ksf (based on a resistance factor of 1.0) may be used for preliminary footing sizing and to control settlements, when analyzing the service limit state load combination. In no instance shall the factored bearing stress exceed the factored compressive resistance of the footing concrete, which may be taken as $0.3f'_c$. No footing shall be less than 2 feet wide, regardless of the applied bearing pressure or the bearing material.

2.1 INTRODUCTION

This report presents the results of the geotechnical investigation performed for the Maine Department of Transportation (MaineDOT) for the proposed replacement of the Iron Bridge (MaineDOT Bridge Identification Number 2403). The MaineDOT Project Identification Number (PIN) is 15097.00. The bridge is located in Island Falls, Maine, and is the crossing for U.S. Route 2 over the West Branch of the Mattawamkeag Rive in Island Falls, Maine. This work was performed in accordance with our scope of work dated July 1, 2008 and authorized by MaineDOT through General Consultant Agreement Number U1210060665.

The objectives of the geotechnical investigation were to characterize the subsurface conditions at the bridge site and to develop foundation design recommendations for the proposed replacement structure. This report summarizes the investigation and provides geotechnical recommendations for shallow foundation design to support the proposed replacement for the Iron Bridge.

2.2 SCOPE

In accordance with the scope of services described in our proposal dated July 1, 2008, URS performed the following:

- Visited the site and reviewed readily available information provided by MaineDOT, as well as topographic and geologic maps for the site and surrounding area;
- Provided a field geologist for observation of the subsurface exploration program to evaluate soil/bedrock conditions at the site. This program consisted of three borings, one each at the eastern and western abutments, and one at the center of the bridge, advanced to bedrock;
- Conducted a limited laboratory-testing program of representative soil samples at the URS Regional Soils Laboratory to confirm field classification and evaluate soil-engineering parameters;
- Performed engineering analyses and provided geotechnical recommendations for the proposed replacement structures, including bearing resistances for shallow footings on rock and settlement analyses; and

- Prepared this geotechnical design report to be submitted to MaineDOT at the conclusion of the geotechnical investigation. This report includes the following:

  - subsurface conditions with boring logs, and engineering description and characterization of the subsurface stratigraphy and groundwater conditions at the time of field exploration;

  - results of laboratory and field testing, including soil properties relevant to development of scour conditions at the bridge site;

  - recommendations for foundations supported on bedrock, including applicable geotechnical design parameters;

  - summary of applicable geotechnical design parameters, based on cast-in-place concrete cantilever retaining structures, for external stability of abutments, and wingwalls;

  - summary of recommended seismic design parameters (as applicable to the structure), and potential susceptibility of site soils to liquefaction during an earthquake; and

  - recommendations for site preparation and earthwork construction including temporary excavations, construction dewatering, fill placement and compaction, protection of existing improvements, and special requirements for protection of soils at foundation subgrade, as necessary.

2.3 PROJECT BACKGROUND AND PROPOSED CONSTRUCTION

The site for the proposed replacement of the Iron Bridge is located at the existing bridge site on U.S. Route 2 in Island Falls, Maine. The limits of proposed construction start at approximately Station 1+030, and extend eastward to approximately Station 1+312. A site locus map is presented in Figure 1.

Based on drawings and other information provided by MaineDOT, the existing Iron Bridge was constructed in the mid-1930s as a replacement to an earlier bridge. The existing bridge is a 4-span structure with each span 54 feet in length. The bridge, abutments and piers are of reinforced concrete and founded on bedrock. The eastside abutment incorporates an existing cut-stone retaining wall into the upstream wingwall. This older, stone wall may be a remnant of the older bridge abutment and may extend behind the current abutment.

Based on the 30-percent design submittal by the project engineer, Erdman, Anthony and Associates of Rochester, New York (Erdman Anthony), the proposed replacement structure for the bridge will consists of a two-span structure with a new deck, utilizing rehabilitated existing abutments and center pier. The proposed replacement structure is to be supported by shallow foundations on bedrock.

The Iron Bridge is not considered to be a critical bridge. Specifically, the bridge is not classified as a major structure because construction costs are expected to be less than 10 million dollars ($10,000,000), and the bridge is not classified as "functionally important."

2.4 REPORT ORGANIZATION

This report is divided into six sections. The Executive Summary is the first section. Following this introduction (Section 2), is a description of the subsurface conditions at the proposed abutment and pier locations (Section 3). Our engineering evaluation and recommendations for foundation design are presented in Section 4, and construction considerations are presented in Section 5. Finally, the limitations of this study are described in Section 6. Supporting figures and data, including site plan, subsurface profile, site photographs, boring logs, and calculations, are appended at the end of this report

3.1 SITE DESCRIPTION

The Iron Bridge crosses the West Branch of the Mattawamkeag River at the upper end of a rock gorge and just upstream of an old dam site. The area on either side of the bridge abutments is vegetated with brush and trees. Earth-fill embankments form the approaches, with steep side slopes to the riverbed. Bedrock is exposed in the riverbed, upstream and downstream of this bridge. The topography of the bridge and surrounding area are shown in Figure 1 and on Sheet 1. Photographs of the site are in Appendix A (Photographs 1 through 4).

3.2 LOCAL GEOLOGY

Based upon the Bedrock Geologic Map of Maine[1], the Iron Bridge site is underlain by bedrock mapped as part of the Albany and unnamed Formations (the Formation). The Formation includes Silurian to Ordovician age metasedimentary rocks. The bedrock protolith is pelite.

The Surficial Geologic Map of Maine[2] indicates that subsurface soil deposits in the Island Fall area are glacial till. Glacial till is a heterogeneous mixture of sand, silt, clay, and stones that may include boulders, and generally conforms to the underlying bedrock surface. Stream deposits in the riverbed are generally thin with bedrock near the surface. However, URS observed under the existing bridge that the stream bed was generally intact bedrock with no overlying stream deposits, as shown in the photographs in Appendix A.

3.3 SUBSURFACE INVESTIGATION PROGRAM

The subsurface investigation program consisted of three test borings. Borings URS-B1, URS-B2, and URS-B3 were advanced under the direction of URS Corporation by Northern Test Boring Inc. of Gorham, Maine in October 2008. Drilling and sampling operations associated with these borings were performed in the presence of our field geologist. The boring locations

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1 Osberg, P.H., Hussey II, A.M., and Boone, G.M. (1985) Bedrock Geologic Map of Maine, Maine Geological Survey, Department of Conservation.

2 Thompson, W.B. and Borns Jr., H.W.(1985) Surficial Geologic Map of Maine, Maine Geological Survey, Department of Conservation.

are presented in Sheet 1. Boring logs from the URS Corporation investigation are presented in Appendix B and on Sheet 2

3.4 LABORATORY ANALYSES

Grain-size analysis tests were performed on soil samples selected as representative of soils at the site. These tests were performed by the URS Corporation Soil Laboratory located in Totowa, New Jersey. The results of these analyses are provided in Appendix C and summarized in the table below.

Grain-size Analysis						
Boring No.	Sample No.	Depth (feet)	Identification Tests			
			Water Content (%)	USCS Symbol	AASHTO Symbol	% Passing No. 200 Sieve
URS-B1	S-2	5.0-7.0	7.8	SM*	A-1-b	16.0
URS-B3	S-2	5.0-7.0	6.1	SM*	A-1-a	14.5

Note: * Plasticity of fines for USCS and AASHTO symbol based on visual observation.

3.4.1 Generalized Subsurface Profiles

A generalized subsurface profile (i.e., foundation survey) was developed for the bridge site, as presented in Sheet 1. The subsurface conditions at the Iron Bridge can generally be described from the ground surface to the limiting depth of the borings as follows:

Fill Material

Fill Material generally consists of medium dense, fine sand and fine to medium gravel with little silt. The thickness of this stratum is approximately 14 feet at the south abutment and 17 feet at the north abutment. The Standard Penetration Resistance N-values[3] in this stratum vary from 9 to 38, with an average value of about 23. The N-values of this layer indicate a medium dense soil density.

[3] N-value is defined as the number of blows required to advance a 50.8mm (2-in.) O.D standard split spoon sampler a distance of 12 inches after seating the spoon a distance of 6 inches using a 140-pound hammer falling freely a distance of 30 inches.

Stream Deposit

A thin sandy stream deposit may underlie the fill material, but was not encountered in the borings. This stratum, when present, generally consists of fine sand with trace to little gravel. The thickness of this layer, where present, may be approximately of one to two feet.

Bedrock

Bedrock underlying the entire proposed construction area is light gray, fine-grained phyllite. The bedrock is moderately hard, slightly to moderately fractured, with near vertical bedding planes and fractures. The upper one to two feet of the bedrock may be weathered and soft, as well as highly to very highly fractured (as observed in the abutment borings).

Bedrock was encountered in the abutment borings at depths varying from approximately 17 feet (elevation 441 feet) in URS-B1 and 14 feet (elevation 446) in URS-B3 below the road surface. Bedrock in the river is exposed at approximately elevation 440 feet. An N-size core barrel was used to obtain rock cores. Core recovery[4] was variable for the three core runs, ranging from 47 to 97 percent. The Rock Quality Designation[5] (RQD) value of the rock core could not be determined due to severe coring-induced fracturing. The vertically bedded phyllite caused coring difficulties due to frequent plugging of the core barrel that resulted in breakage and grinding of intact rock, which resulted in an unknown amount of rock loss. Also, re-assembling the highly fractured rock into “intact” lengths was problematic. Thus, an accurate RQD could not be determined in our opinion. Photographs of the rock cores are included in Appendix A.

Extensive exposures of bedrock exist at both abutments and beneath the bridge deck, as well as upstream and downstream of the bridge. The exposed bedrock (see photographs in Appendix A) is intact, and not fractured or weathered. The rock core from URS-B2 represents the exposed outcrop condition. While highly fractured, the fractures in the core from URS-B2 appeared fresh and unweathered and, thus, caused by the coring procedure (i.e., mechanical fractures). Based on the core recovery (97%) and adjacent bedrock exposures, URS-B2 core is estimated to have a very high RQD, in the range of 80% to 100%.

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[4] Core recovery is the ratio of total length of the recovered core to the length cored, in percent.

[5] The Rock Quality Designation (RQD) is defined as the ratio (expressed as a percentage) of the total length of recovered core samples having a length of at least twice the core diameter (e.g., about 4 in for NX-core) to the total length cored.

The two rock cores from the abutment borings had more rock loss, were less intact, and had some evidence of iron staining. While most fractures were fresh, there were some minor weathered rock fractures in the upper portion of the cores. These rock cores were judged to have moderate RQDs, in the 40% to 60% range.

The difference between the RQDs of the abutment bedrock and pier bedrock indicates that there may be up to several feet of very fractured, weathered bedrock overlying more competent rock behind the abutments. In the river bed, historic erosion has removed the weathered rock surface. It should be noted that the bridge abutments appear to be underlain by intact, unweathered bedrock.

Rock mass ratings (RMRs) can be approximated for the rock cores using reasonable estimates for the uniaxial compressive strength of the intact rock (3,500 to 35,000 pounds per square inch [psi][6]) and the estimated RQDs.

Rock Core	RMR	Class Number[7]	Description[7]
URS-B1	62	II	Good Rock
URS-B2	86	I	Very Good Rock
URS-B3	62	II	Good Rock

3.5 GROUNDWATER CONDITIONS

At the time of the URS field investigation, the bridge deck (at approximately elevation 459 feet) was about 17.4 feet above the water surface. Estimated groundwater levels in boring URS-B1, at the time of drilling, indicated that groundwater was at approximately the same elevation as the river approximately elevation 441 feet. Groundwater was not observed in boring URS-B3 as the groundwater level is likely below the bedrock surface elevation of 446 feet. Seasonal variations in the water surface and groundwater elevations will occur.

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[6] AASHTO, 2002. Standard Specifications for Highway Bridges, 17th Edition, American Association of State Highway and Transportation Officials, Washington, D.C

[7] Reference Table 3.1.4, LRFD for Highway Bridge Substructures and Earth Retaining Structures Publication No. FHWA-NH1-05-094, Federal Highway Administration, December 2005.

3.6 SEISMIC DESIGN PARAMETERS

Based on Horizontal Peak Ground Acceleration Coefficient map provided in AASHTO LRFD, 2008[8] (Figure 3.10.2.1-1), the Horizontal Peak Ground Acceleration Coefficient with a 7 percent chance probability of exceedance in 75 years for the Village Bridge site is 7 (i.e., 7 percent of gravitational acceleration [0.07 g]). Based on AASHTO LRFD Figures 3.10.2.1-2 and 3.10.2.1-3, the Horizontal Spectral Response Acceleration of 0.2 second period (S_s) and 1.0 second period (S_1) are 16 (0.16 g) and 5 (0.05g), respectively. Based on the soil type and profile, the Village Bridge is considered to be Site Class B (AASHTO LRFD, 2008, Table 3.10.3.1-1).

Liquefaction potential for soils below the groundwater table, where present, is considered negligible, as these soils are limited in thickness and generally in a medium dense to dense condition.

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[8] AASHTO LRFD. 2008 . *LRFD Bridge Design Specifications* , 4th edition, 2007, with 2008 Interim Revisions, American Association of State Highway and Transportation Officials, Washington D.C.

4.1 GENERAL DISCUSSION

The engineering evaluation presented herein is based on our current understanding of the project design requirements for the bridge center pier abutments, wingwalls, and retaining walls. The project design engineer, Erdman Anthony, is proposing to rehabilitate and reuse the existing abutments and construct a new center pier will be built. Therefore, the bridge abutments will be founded on the existing shallow footings on bedrock. Additionally, the center pier will be founded on the shallow footings on bedrock. The design methodology used in the following evaluations is based on AASHTO LRFD (2008).

The existing abutments appeared to be supported by nearly unweathered intact bedrock similar in nature to the bedrock cored at the center pier. The more weathered bedrock observed in the abutment borings represents a weathered upper zone of bedrock.

4.2 SHALLOW FOUNDATION

Borings URS-B1 and URS-B3 encountered bedrock approximately 14 to 17 feet below the existing eastern and western bridge approaches, respectively, and boring URS-B2 encountered bedrock at the surface of the stream bed. Therefore, it is considered feasible that a coffer dam, seals (if required) and spread footings can be practicably and economically constructed to bear on bedrock.

Foundations on bedrock have no minimum cover requirement for frost or scour. Our design recommendations for the spread footing foundation on bedrock are presented below.

Alternatively, the bearing resistance for reuse of the existing abutment spread footings can be estimated based on the current bridge configuration and applicable dead and live load combinations. By backing out the appropriate resistance factors, the resulting factored bearing resistance for foundation design can be determined. However, the applicable dead and live load combinations were not provided for the abutments, so the bearing resistance was not determined by this method.

4.2.1 Abutment, Pier and Wingwall Design

The abutments, pier and wingwalls shall be proportioned for all applicable load combinations in AASHTO LRFD (2008) Articles 3.4.1 and 11.5.5, and shall be designed for the relevant strength, service and extreme event limit states. The design of the project abutments, pier and wingwalls founded on spread footings at the strength limit state shall consider nominal bearing resistance, eccentricity (i.e., overturning), lateral sliding and structural failure. The extreme event limit state design shall confirm that the nominal foundation resistance remaining after scour due to the design flood event will support the unfactored strength limit state loads with a resistance factor of 1.0.

A sliding resistance factor, φ_τ, of 0.80 shall be applied to the nominal sliding resistance of cast-in-place concrete abutments, pier and wingwalls founded on spread footings supported on bedrock. Calculation of the sliding resistance to lateral loads shall assume a maximum frictional coefficient of 0.70 at the bedrock/concrete interface.

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

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

4.2.2 Lateral Earth Pressures at Abutments

Cantilever-type abutments and wingwalls shall be designed as unrestrained retaining walls, which means that these walls are free to rotate at the top in an active state of earth pressure. Earth loads shall be calculated using an active earth pressure coefficient of 0.33, (i.e., $K_a = 0.33$), calculated using the Rankine Theory for cantilever-type abutments and wingwalls, which assumes a level backfill surface. The earth pressure coefficient may change if the backfill surface conditions are different (e.g., sloping). See Appendix D – Calculations for supporting documentation for the active earth pressure coefficient.

Reuse of the existing abutments is proposed, and the existing backfill material is assumed to be consistent with the Fill Material encountered in borings URS-B1 and URS-B3. Assuming adequate drainage, the design soil properties for the existing backfill are summarized below.

Existing Backfill Properties for Lateral Earth Pressure Analyses	
Design Parameter	**Value**
Total unit weight of backfill (γ)	125 pcf
Angle of internal friction (ϕ)	30^{o}
Angle of wall interface friction (δ)	20^{o}
Coefficient of Friction, tan δ Soil to concrete	0.36

Additional lateral earth pressure due to construction surcharge or live load surcharge is required for the abutments and wingwalls if an approach slab is not specified, in accordance with MaineDOT BDG (2003) Section 3.6.8. The live load surcharge on abutments may be estimated as a uniform earth pressure due to an equivalent height of soil (h_{eq}) taken from the table below:

Height of Soil (h_{eq}) for Uniform Earth Pressure for Equivalent Live Load	
Abutment Height (feet)	**Height of Soil (h_{eq}, feet)**
5.0	4.0
10.0	3.0
≥ 20.0	2.0

Note: Linear interpolation should be used for intermediate wall heights.

In the case where a structural approach slab is specified, reduction of the surcharge loads is permitted, in accordance with AASHTO LRFD (2008) Article 3.11.6.5. Based on AASHTO LRFD (2008) Table 3.11.6.4-1, the live load surcharge on walls may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) of 2.0 feet.

The backfill behind the abutments and wingwalls shall be adequately drained in accordance with MaineDOT BDG (2003) Section 5.4.1.4 – Drainage. The designer shall confirm that there is adequate drainage, and the designer shall include a drainage system behind the abutments to intercept groundwater if the existing drainage is not adequate. Drainage behind the structure shall be designed in accordance with MaineDOT BDG (2003) Section 5.4.1.4 – Drainage. To prevent water intrusion behind the abutment, the approach slab should be connected directly to the abutment.

If during construction for the bridge replacement, backfill within 10 feet of the back of the abutments, wingwalls and side slope fills is removed, it shall be replaced with material that conforms to MaineDOT Standard Specification[9] 709.19: Granular Borrow for Underwater Backfill. The gradation for this material specifies 10 percent or less of material passing the No. 200 sieve. This backfill will be specified in order to reduce the amount of fine material in the backfill and minimize frost action behind the structure.

Slopes in front of, and sloping down to, the wingwalls should be constructed with riprap. The steepness of these slopes should not be steeper than 1.75 horizontal to 1 vertical (i.e., 1.75H:1V).

4.2.3 Factored Bedrock Bearing Resistance

Substructure spread footings for the abutments, pier and wingwalls shall be proportioned to provide stability against bearing capacity failure. Application of permanent and transient loads will be performed in accordance with AASHTO LRFD (2008) Article 11.5.5. The stress distribution may be assumed to be a triangular or trapezoidal distribution over the effective base, as shown in AASHTO LRFD (2008) Figure 11.6.3.2-2. The factored bearing resistance for any structure founded on competent, sound bedrock shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 7 kips per square foot (ksf) for footings that are 4 to 9 feet wide and 10 ksf for footings that are 9 to 12 feet wide (see Appendix D for supporting calculations). This assumes a bearing resistance factor, φ_b, of 0.45 for spread footings on bedrock, based on a bearing resistance evaluation using semi-empirical methods. A factored bearing resistance of 16 ksf may be used for preliminary footing sizing and to control settlements, when analyzing the service limit state load combination.

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

[9] MaineDOT. 2002. *Standard Specifications*, State of Maine Department of Transportation Revision of December 2002.

4.2.4 Abutment and Pier Settlement

URS understands that the current bridge replacement plans do not include changes to the profile. Additionally, the spread footings for the abutments and pier will be supported in bedrock. Therefore, settlements are expected to be negligible (i.e., less than ½ inch). Differential settlement is also expected to be on the order of ½ inch or less. It is expected that these settlements will occur due to elastic compression of the bedrock during construction, and will have minimal impact on the structure.

4.3 SCOUR

The designer shall consider the consequences of changes in foundation conditions at the service and extreme event limit states, resulting from scour due to the design flood event. The extreme event limit state shall determine that there is adequate foundation resistance to support the unfactored strength limit state loads with a resistance factor of 1.0, in accordance with AASHTO LRFD Article 10.5.2.1. Changes in foundation conditions due to scour shall be investigated at abutments, pier, wingwalls and retaining walls.

In general, for scour protection, any footing for wingwalls or retaining walls that are constructed on soil should be embedded at least 2 feet below the design scour depth and armored with at least 3 feet of riprap for scour protection. Refer to MaineDOT BDG (2003) for additional information regarding scour design. Scour protection for footings on bedrock is not required.

4.4 FROST PROTECTION

The proposed foundations for the abutments, pier and wingwalls are spread footings supported on bedrock. Therefore, heave due to frost action is not considered to be a design issue, and no requirements for embedment depth are necessary.

However, the potential frost depth has been evaluated for foundations for ancillary structures (e.g., light poles, retaining walls, etc.). Based on the State of Maine frost depth maps (MaineDOT BDG Figure 5-1), the site has a freezing index of approximately 2200 F-degree days. The water content of the soil was approximately 9 percent, which correlates to a frost depth of 100 inches (approximately 8.3 feet). Consequently, we recommend that any foundations or leveling pads constructed at the site should be founded a minimum of 8.3 feet below the finished exterior grade. This minimum embedment applies only to foundations constructed on soil, and not to foundations supported directly on bedrock.

4.5 SEISMIC DESIGN CONSIDERATIONS

the bridge is not classified as a major structure because construction costs are expected to be less than 10 million dollars ($10,000,000), and the bridge is not classified as "functionally important." Furthermore, in accordance with the guidance in AASHTO LRFD (2008) Article 3.10.6, local seismic activity deisgnates this area as Seismic Performance Zone. Consequently, seismic earth loads do not need to be considered in the design of the bridge substructure. However, superstructure connections and bridge seat dimensions shall be designed to satisfy the requirements of AASHTO LRFD (2008) Articles 3.10.9 and 4.7.4.4, respectively.

5.1 EXCAVATION FOR FOUNDATIONS

For shallow foundations supported on bedrock, the top of rock should be excavated to a firm surface, cleaned, and examined to verify that the quality of the rock is consistent with the recommended rock bearing capacity and ensure concrete is placed on clean and sound rock. The boring near the center pier (URS-B2) indicates that the bedrock is unweathered and not fractured. However, if the bedrock under the proposed new center pier location is found to be weathered and/or highly fractured, it will be necessary to excavate all dislodged, loose fractured or weathered bedrock before placing seal concrete or concrete for the spread footing. The full extent of rock excavation needed will not be known until the foundation excavation is made.

In accordance with MaineDOT (2002) Standard Specifications Subsection 206.02, the rock surface should level stepped or serrated. Additionally, preparation for the footings may require excavation of bedrock and/or placement of seal concrete to provide a level surface for the footings. Highly weathered or disintegrated rock encountered at the elevation of bottom of footings should be removed and replaced with seal concrete.

Since the groundwater level was measured at approximately 17 feet below the ground surface and is controlled by the river level, work at the abutment foundations may require excavation below the water level. The sides of the excavations should be supported or sloped (if site conditions permit) as per the relevant OSHA, local, and/or federal regulations (see MaineDOT Standard Specifications Section 203).

The contractor should also be prepared to control rainwater and surface water runoff and keep it away from prepared subgrades. Control of runoff should be performed in accordance with MaineDOT (2002) Standard Specifications Subsection 203.10.

Disturbed subgrade, unsuitable soil, or deleterious material encountered at the elevation of bottom of excavations for footings should be removed and replaced with concrete (if subject to scour) or granular or gravel borrow. Borrow material should be compacted to not less than 90 percent of the maximum dry density, as determined by AASHTO Standard Method of Test T-180, Methods C or D at optimum water content. Granular and gravel borrow should conform to the material specifications, Sections 703.19 and 703.20, respectively, in the MaineDOT (2002) Standard Specifications.

5.2 FILL PLACEMENT

Placement and compaction of the embankments shall be performed in accordance with MaineDOT (2002) Standard Specifications Sections 203.10, 203.11, and 203.12.

Abutments, wingwalls and retaining walls should be backfilled with granular borrow that meets the MaineDOT criteria for underwater backfill (Standard Specification 703.19). This backfill should be placed for a horizontal distance of at least 10 feet from the back of the wall (MaineDOT BDG, 2003). Placement and compaction of backfill behind abutments, wingwalls, and retaining walls shall be performed in accordance with MaineDOT (2002) Standard Specifications Section 206.

5.3 EMBANKMENT CONSTRUCTION ADJACENT TO FOUNDATIONS

Reconstruction of the existing embankments may be required for the bridge replacement. Since recommended support for the foundations for the replacement bridge structure is bedrock, additional settlements due to placing fill for the embankments will not be significant.

5.4 REUSE OF EXISTING EMBANKMENT SOILS

The existing embankment soils are silty sand which appear adequate for reuse as common borrow. Excavated embankment soils may be stockpiled and reused where appropriate after testing (e.g., gradation analysis and compaction testing) is performed on representative samples. Additionally, excavated embankment soil may also meet the criteria for granular fill, but this needs to be confirmed by laboratory testing of represent samples prior to use.

5.5 SCOUR PROTECTION

The abutment and pier foundations will be supported on bedrock, so scour protection is not required. However, any footing for wingwalls or retaining walls that are constructed on soil should be armored with riprap for scour protection, as noted above in Section 4.4. The riprap layer shall be at least 3 feet thick, and the stone shall conform to MaineDOT Standard Specification 703.26: Plain and Hand Laid Riprap. For wingwalls and retaining walls, the riprap shall extend outward at least 1.5 feet horizontally from the front of the structure before sloping at a maximum slope of 1.75H:1V to the existing ground surface. The toe of the riprap sections shall be constructed at least 1 foot below the streambed elevation. The riprap section shall be underlain by Class A erosion control geotextile and a 1-foot thick layer of bedding material conforming to MaineDOT Standard Specification 703.19: Granular Borrow for Underwater Backfill, as shown in Standard Detail 610 (03)[10].

5.6 EROSION AND SEDIMENTATION CONTROL

The erosion and sedimentation potential of soils along the alignment should be considered moderate due to the fines content and proximity to the river, so exposed soils need to be protected during construction. Erosion control should be provided for disturbed areas in accordance with MaineDOT Standard Specifications Section 656 and the MaineDOT Best Management Practices Handbook[11].

[10] MaineDOT Standard Details are at: http://www.maine.gov/mdot/contractor-consultant-information/ss_standard_details_updates.php.

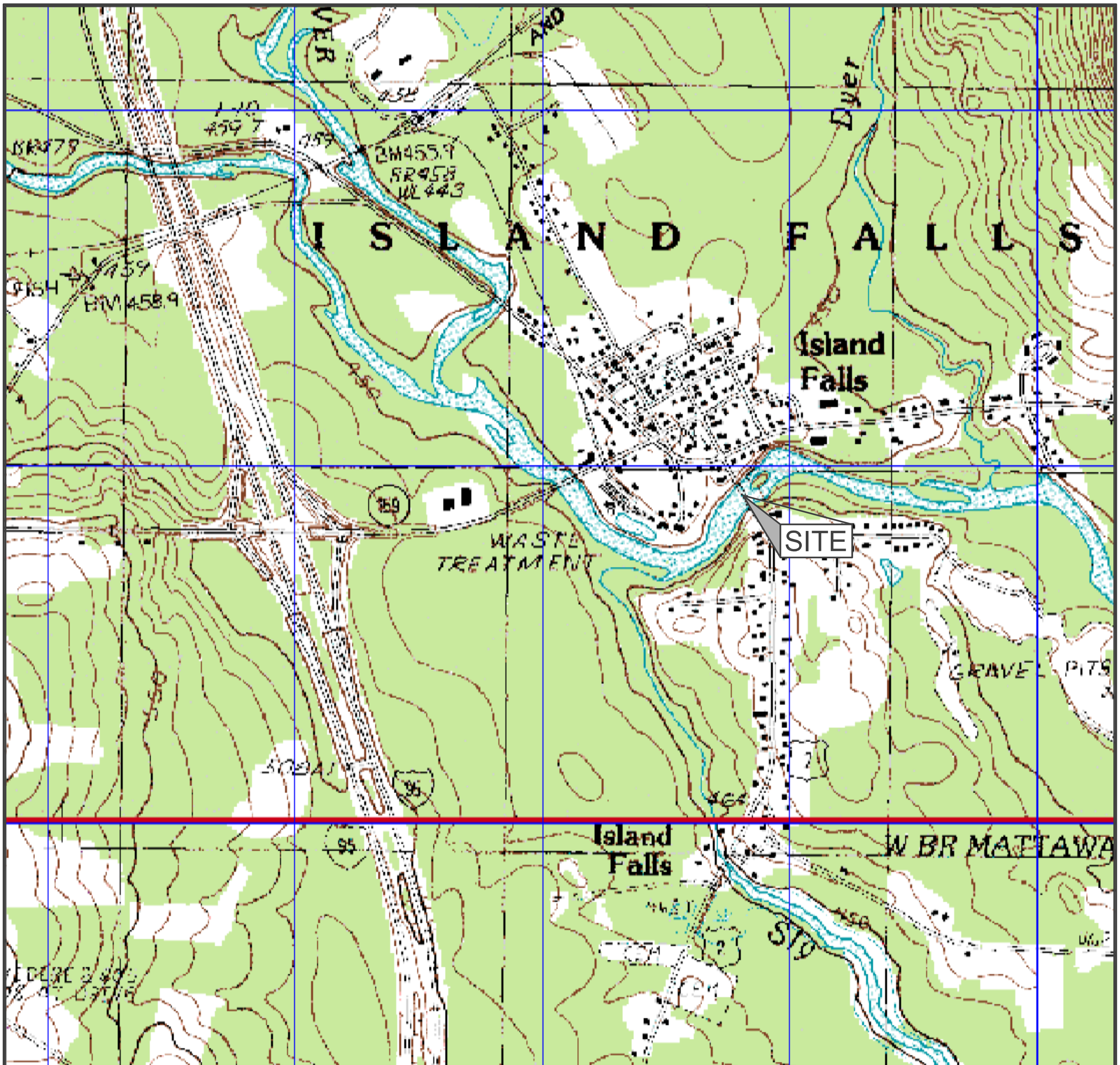
[11] MaineDOT. 2008. *MaineDOT Best Management Practices for Erosion and Sedimentation Control*. Maine Department of Transportation.

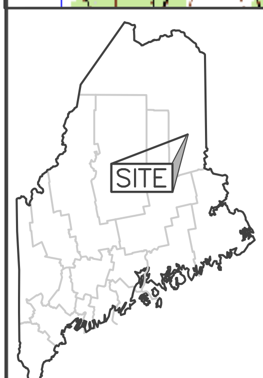
The results and recommendations presented in this report are largely based on subsurface information from a limited number of borings, laboratory tests, and our use of generally accepted analytical procedures. Subsurface conditions may vary from those presented in this report, and these variances may require a modification of the recommended foundation systems. If further investigation or construction activity reveals significant differences in the subsurface conditions, URS Corporation requests the opportunity to review and modify our recommendations, as appropriate. The recommendations presented in this report should not be extrapolated to other areas or used for other facilities without URS Corporation's prior review.

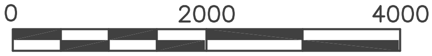
This report has been prepared by URS Corporation for the exclusive use of the Maine Department of Transportation and its designers, based on our understanding of the project as described in this report. Any modification or final decisions in the design concept from the descriptions in this report should be made known to URS Corporation for possible modifications of our recommendations.

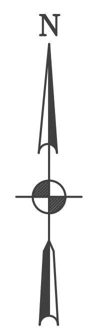
FIGURES

SHEETS





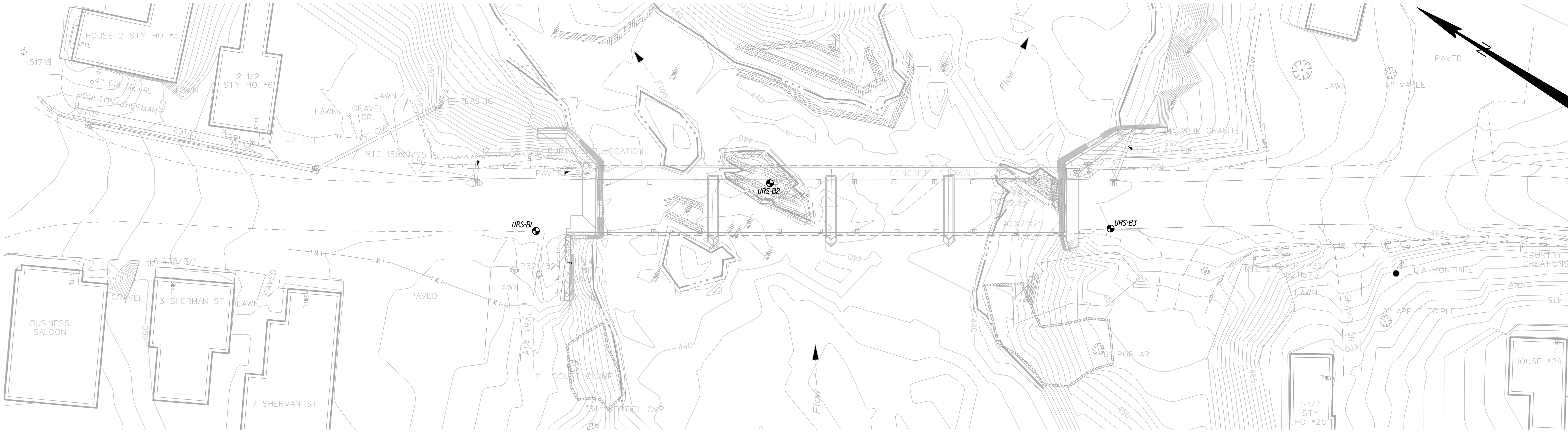




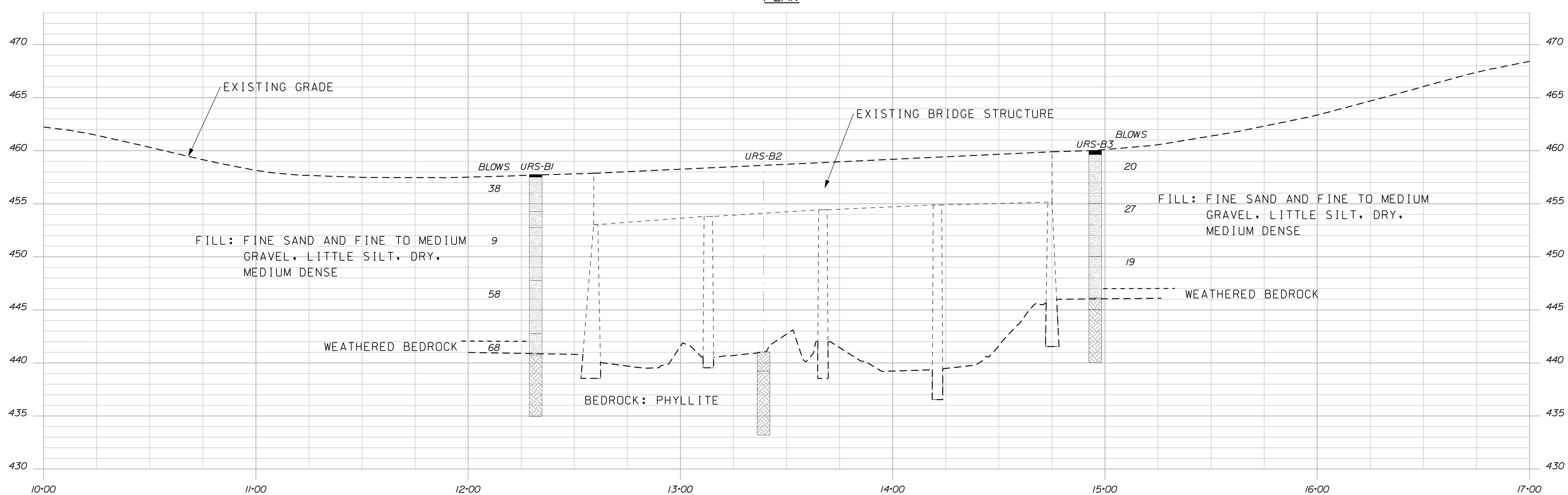
SOURCE:
USGS 7.5–MINUTE TOPOGRAPHIC MAP OF THE ISLAND FALLS, MAINE QUADRANGLE DATED 1986.

URS URS Corporation 115 Water Street, Suite 3 Hallowell, ME 04347 Tel: 207.623.9188 Fax: 207.622.6085 www.urscorp.com	PROJECT NO: 39460348		CLIENT: MAINE DEPARTMENT OF TRANSPORTATION	TITLE:	FIGURE NO:
	DESIGN: DWA	SCALE: AS SHOWN	PROJECT: IRON BRIDGE ISLAND FALLS, MAINE MDOT PIN D15097.00	SITE LOCUS	1
	APPROVED: GKT	DATE: 12/30/08			
	DRAWN: LRH	FILE NO: FIG1			

Filename: ...\Foundation_Survey.dgn   Division: HIGHWAY   Username: dennis_burnheimer   Date:3/14/2009



PLAN



PROFILE

Notes:

1. *Base drawings for plan and profile provided by Erdman Anthony, Inc. the project Structural Engineer.*
2. *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.*

HORIZ 25 0 25 50

VERT 5 0 5 10

SCALE (IN FEET)

Strata Symbols

- Silty Sand
- Rock
- Sand
- Weathered Rock (May include boulders)
- Estimated Groundwater Level
- Estimated strata interface.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BR-1509(700)X

PIN 015097.00

BRIDGE PLANS

BRIDGE NO. 2403

SIGNATURE 3601
P.E. NUMBER
JANUARY 2009
DATE

	BY	DATE
PROJ. MANAGER DEVIN ANDERSON		
DESIGN-DETAILED	D. ANDREWS	
CHECKED-REVIEWED	BURNHEIMER	02/16/09
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

IRON BRIDGE
WEST BRANCH MATTAWAMKEAG
ISLAND FALLS AROOSTOOK

BORING LOCATION PLAN AND
SUBSURFACE INTERPRETIVE PROFILE

SHEET NUMBER

1

1 OF 2

Filename: ...\Island Falls\BoringLog.dgn   Division: HIGHWAY   Username: dennis_burnheimer   Date:3/14/2009

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

Project: Iron Bridge Replacement
Location: Route 2, Island Falls, Maine
Boring No.: URS-B1
PIN: 15097.00

Driller:	Nothern Test Borings, Inc.	Elevation (ft.)	455	Auger ID/OD:	na
Operator:	M. Nadeau	Datum:		Sampler:	Standard Split Spoon
Logged By:	M. Reiter - URS Corp.	Rig Type:	Diedrich D-50	Hammer Wt./Fall:	140#/30 inches
Date Start/Finish:	10/7/08	Drilling Method:	Cased Boring	Core Barrel:	N size
Boring Location:	North Abutment	Casing ID/OD:	4-inch	Water Level*:	14 feet
Hammer Efficiency Factor: 0.63		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in. Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	S-1	24/18.5	0.25 - 2.25	9/18/20/38	38	40		454.75	Pavement	SM
									0.25 Brown, fine Sand and fine to medium subrounded to angular gravel, little silt and asphalt, dense, dry, {FILL}	
			3.50 3.54					451.50	3.50 Concrete	
	S-2	24/12	5.00 7.00	5/4/4/5	9	9		450.00	5.00 Brown, fine Sand and fine to medium subrounded to angular gravel, little silt loose, dry, {FILL}	WC 7.8%, 16.0% -#200 Sieve, SM
10	S-3	24/12	10.00 12.00	20/47/11/10	58	61		445.00	10.00 Cobble	
	S-4	21/14	15.00 16.75	16/28/40/50-3"	68	71		440.00	15.00 Tan fine Sand, some silt, very dense wet, and weathered bedrock	SM
	R	68/42	17.00 22.67					438.00	17.00 Phyllite, very fine grained, moderately hard, moderately strong, slight weathering, vertical fractures, light gray (see remarks)	
20								432.20	22.80 Bottom of Exploration at 22.80 feet below ground surface.	
30										
40										
50										
60										

Remarks:

The thin vertical foliation of phyllite caused extensive mechanical fracturing and crushing by coring, therefore, RQD could not be estimated. Outcrops adjacent to boring location appeared to be essentially unfractured and unweathered.

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.: URS-B1

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

Project: Iron Bridge Replacement
Location: Route 2, Island Falls, Maine
Boring No.: URS-B2
PIN: 15097.00

Driller:	Nothern Test Borings, Inc.	Elevation (ft.)	455	Auger ID/OD:	na
Operator:	M. Nadeau	Datum:		Sampler:	Standard Split Spoon
Logged By:	M. Reiter - URS Corp.	Rig Type:	Diedrich D-50	Hammer Wt./Fall:	140#/30 inches
Date Start/Finish:	10/7/08	Drilling Method:	Cased Boring	Core Barrel:	N size
Boring Location:	Center of Bridge	Casing ID/OD:	4-inch	Water Level*:	17.4 feet
Hammer Efficiency Factor: 0.63		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in. Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0									Air- drilled from bridge deck	
10										
								436.90	18.10 Bedrock surface - roller coned to set casing into bedrock	
20	R	60/58	19.90 24.90					435.10	19.90 Phyllite, very fine grained, moderately hard, moderately strong, slight to no weathering, vertical fractures, light gray (see remarks)	
								430.10	24.90 Bottom of Exploration at 24.90 feet below ground surface.	
30										
40										
50										
60										

Remarks:

The tin vertical foliation of phyllite caused extensive mechanical fracturing by coring, therefore, RQD could not be estimated. Outcrop at boring location appeared to be essentially unfractured and unweathered.

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.: URS-B2

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

Project: Iron Bridge Replacement
Location: Route 2, Island Falls, Maine
Boring No.: URS-B3
PIN: 15097.00

Driller:	Nothern Test Borings, Inc.	Elevation (ft.)	460	Auger ID/OD:	na
Operator:	M. Nadeau	Datum:		Sampler:	Standard Split Spoon
Logged By:	M. Reiter - URS Corp.	Rig Type:	Diedrich D-50	Hammer Wt./Fall:	140#/30 inches
Date Start/Finish:	10/7/08	Drilling Method:	Cased Boring	Core Barrel:	N size
Boring Location:	South Abutment	Casing ID/OD:	4-inch	Water Level*:	not observed
Hammer Efficiency Factor: 0.63		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in. Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	S-1	24/12	0.40 - 2.40	6/10/10/9	20	21		459.60	Pavement	SM
									0.40 Brown, fine Sand and fine to medium, subrounded to angular gravel, little silt, medium dense, dry, {FILL}	
	S-2	24/8	5.00 7.00	9/12/15/22	27	28		455.00	5.00 Brown, fine Sand and fine to medium, subrounded to angular gravel, little silt, medium dense, dry,{FILL}	WC 6.1 %, 14.5% -#200 Sieve, SM
10	S-3	24/11	10.00 12.00	6/7/12/18	19	20		450.00	10.00 Brown, fine Sand and fine to medium, subrounded to angular gravel, little silt, medium dense, dry, {FILL}	SM
								446.10	13.90 Weathered bedrock	
	R	60/28	15.00 20.00					445.00	15.00 Phyllite, very fine grained, moderately hard, moderately strong, slight to no weathering, vertical fractures, light gray (see remarks)	
20								440.00	20.00 Bottom of Exploration at 20.00 feet below ground surface.	
30										
40										
50										
60										

Remarks:

The thin vertical foliation of phyllite caused extensive mechanical fracturing and crushing by coring,therefore, RQD could not be estimated. Outcrops adjacent to boring location appeared to be essentially unfractured and unweathered.

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

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Page 1 of 1
Boring No.: URS-B3

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
BR-1509(700)X
BRIDGE NO. 2403
PIN 015097.00
BRIDGE PLANS

	BY	DATE
PROJ. MANAGER DEVIN ANDERSON		
DESIGN-DETAILED	D. ANDREWS	BURNHEIMER 02/16/09
CHECKED-REVIEWED		
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE
3601
P.E. NUMBER
JANUARY 2009
DATE

IRON BRIDGE
WEST BRANCH MATTAWAMKEAG
ISLAND FALLS AROOSTOOK

BORING LOGS

SHEET NUMBER
2
2 OF 2

APPENDIX A

SITE PHOTOGRAPHS

URS

PHOTOGRAPHIC LOG

Client Name:	Site Location:	Project No.
Maine DOT	Iron Bridge, Island Falls, Maine	39460348

Photo No. 1 | **Date:** 9/15/08

Direction Photo Taken:

Vertical, West Abutment

Description:

Intact phyllite bedrock at base of west abutment. Greenish color is from graffiti spray paint. Pocket notepad for scale.

Photo No. 2 | **Date:** 9/15/08

Direction Photo Taken:

Looking east, upstream of Bridge.

Description:

Note bedrock outcrop at center pier.

PHOTOGRAPHIC LOG

Client Name:	Site Location:	Project No.
Maine DOT	Iron Bridge, Island Falls, Maine	39460348

Photo No. 3 **Date:** 9/15/08

Direction Photo Taken:

Looking north, upstream.

Description:

Intact phyllite bedrock at base of east pier.

Photo No. 4 **Date:** 9/15/08

Direction Photo Taken:

Looking north, east abutment

Description:

Intact phyllite bedrock at base of west abutment.

PHOTOGRAPHIC LOG

Client Name:	**Site Location:**	**Project No.**
Maine DOT	Iron Bridge, Island Falls, Maine	39460348

Photo No.	**Date:**
5	na

Direction Photo Taken:

na

Description:

Upper portion of rock cores in core box.

Photo No.	**Date:**
6	na

Direction Photo Taken:

na

Description:

Lower portion of rock cores in core box.

APPENDIX B

BORING LOGS

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: Iron Bridge Replacement

Location: Route 2, Island Falls, Maine

Boring No.: URS-B1

PIN: 15097.00

Driller:	Nothern Test Borings, Inc.	**Elevation (ft.)**	455	**Auger ID/OD:**	na
Operator:	M. Nadeau	**Datum:**		**Sampler:**	Standard Split Spoon
Logged By:	M. Reiter - URS Corp.	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30 inches
Date Start/Finish:	10/7/08	**Drilling Method:**	Cased Boring	**Core Barrel:**	N size
Boring Location:	North Abutment	**Casing ID/OD:**	4-inch	**Water Level*:**	14 feet
Hammer Efficiency Factor:	0.63	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	S-1	24/18.5	0.25 - 2.25	9/18/20/38	38	40		454.75		Pavement 0.25	SM
										Brown, fine Sand and fine to medium subrounded to angular gravel, little silt and asphalt, dense, dry, {FILL}	
			3.50 - 3.54					451.50		3.50	
	S-2	24/12	5.00 - 7.00	5/4/4/5	9	9		450.00		Concrete 5.00	WC 7.8%, 16.0% -#200 Sieve, SM
										Brown, fine Sand and fine to medium subrounded to angular gravel, little silt loose, dry, {FILL}	
10	S-3	24/12	10.00 - 12.00	20/47/11/10	58	61		445.00		10.00 Cobble	
	S-4	21/14	15.00 - 16.75	16/28/40/50-3"	68	71		440.00		15.00 Tan fine Sand, some silt, very dense wet, and weathered bedrock	SM
	R	68/42	17.00 - 22.67					438.00		17.00 Phyllite, very fine grained, moderately hard, moderately strong, slight weathering, vertical fractures, light gray (see remarks)	
20											
								432.20		22.80 **Bottom of Exploration at 22.80 feet below ground surface.**	
30											
40											
50											
60											

Remarks:

The thin vertical foliation of phyllite caused extensive mechanical fracturing and crushing by coring, therefore, RQD could not be estimated. Outcrops adjacent to boring location appeared to be essentially unfractured and unweathered.

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Boring No.: URS-B1

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Iron Bridge Replacement
Location: Route 2, Island Falls, Maine

Boring No.: URS-B2
PIN: 15097.00

Driller:	Nothern Test Borings, Inc.	**Elevation (ft.)**	455	**Auger ID/OD:**	na
Operator:	M. Nadeau	**Datum:**		**Sampler:**	Standard Split Spoon
Logged By:	M. Reiter - URS Corp.	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30 inches
Date Start/Finish:	10/7/08	**Drilling Method:**	Cased Boring	**Core Barrel:**	N size
Boring Location:	Center of Bridge	**Casing ID/OD:**	4-inch	**Water Level*:**	17.4 feet
Hammer Efficiency Factor:	0.63	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks		Laboratory Testing Results/ AASHTO and Unified Class.
0										Air- drilled from bridge deck		
10												
								436.90			18.10	
										Bedrock surface - roller coned to set casing into bedrock		
20	R	60/58	19.90 - 24.90					435.10			19.90	
										Phyllite, very fine grained, moderately hard, moderately strong, slight to no weathering, vertical fractures, light gray (see remarks)		
								430.10			24.90	
										Bottom of Exploration at 24.90 feet below ground surface.		
30												
40												
50												
60												

Remarks:

The tin vertical foliation of phyllite caused extensive mechanical fracturing by coring, therefore, RQD could not be estimated. Outcrop at boring location appeared to be essentially unfractured and unweathered.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Iron Bridge Replacement
Location: Route 2, Island Falls, Maine

Boring No.: URS-B3
PIN: 15097.00

Driller:	Nothern Test Borings, Inc.	**Elevation (ft.)**	460	**Auger ID/OD:**	na
Operator:	M. Nadeau	**Datum:**		**Sampler:**	Standard Split Spoon
Logged By:	M. Reiter - URS Corp.	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30 inches
Date Start/Finish:	10/7/08	**Drilling Method:**	Cased Boring	**Core Barrel:**	N size
Boring Location:	South Abutment	**Casing ID/OD:**	4-inch	**Water Level*:**	not observed
Hammer Efficiency Factor:	0.63	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	S-1	24/12	0.40 - 2.40	6/10/10/9	20	21		459.60		Pavement (0.40)	SM
										Brown, fine Sand and fine to medium, subrounded to angular gravel, little silt, medium dense, dry, {FILL} (5.00)	
	S-2	24/8	5.00 - 7.00	9/12/15/22	27	28		455.00		Brown, fine Sand and fine to medium, subrounded to angular gravel, little silt, medium dense, dry,{FILL} (10.00)	WC 6.1 %, 14.5% -#200 Sieve, SM
10	S-3	24/11	10.00 - 12.00	6/7/12/18	19	20		450.00		Brown, fine Sand and fine to medium, subrounded to angular gravel, little silt, medium dense, dry, {FILL} (13.90)	SM
								446.10		Weathered bedrock (15.00)	
	R	60/28	15.00 - 20.00					445.00		Phyllite, very fine grained, moderately hard, moderately strong, slight to no weathering, vertical fractures, light gray (see remarks) (20.00)	
20								440.00		**Bottom of Exploration at 20.00 feet below ground surface.**	
30											
40											
50											
60											

Remarks:

The thin vertical foliation of phyllite caused extensive mechanical fracturing and crushing by coring,therefore, RQD could not be estimated. Outcrops adjacent to boring location appeared to be essentially unfractured and unweathered.

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 C

LABORATORY DATA

Project No.: 39460348.10002
File: Indexb.xls

MDOT Aroostook Bridges/Island Falls

LABORATORY TESTING DATA SUMMARY

BORING NO.	SAMPLE NO.	DEPTH (ft)	IDENTIFICATION TESTS			REMARKS
			WATER CONTENT (%)	USCS SYMB. (1)	SIEVE MINUS NO. 200 (%)	
URS-IF-B1	S-2	5-7	7.8	SM	16.0	
URS-IF-B3	S-2	5-7	6.1	SM	14.5	

Note: (1) USCS symbol based on visual observation and Sieve results reported.

Prepared by: JR
Reviewed by: GET
Date: 10/29/2008

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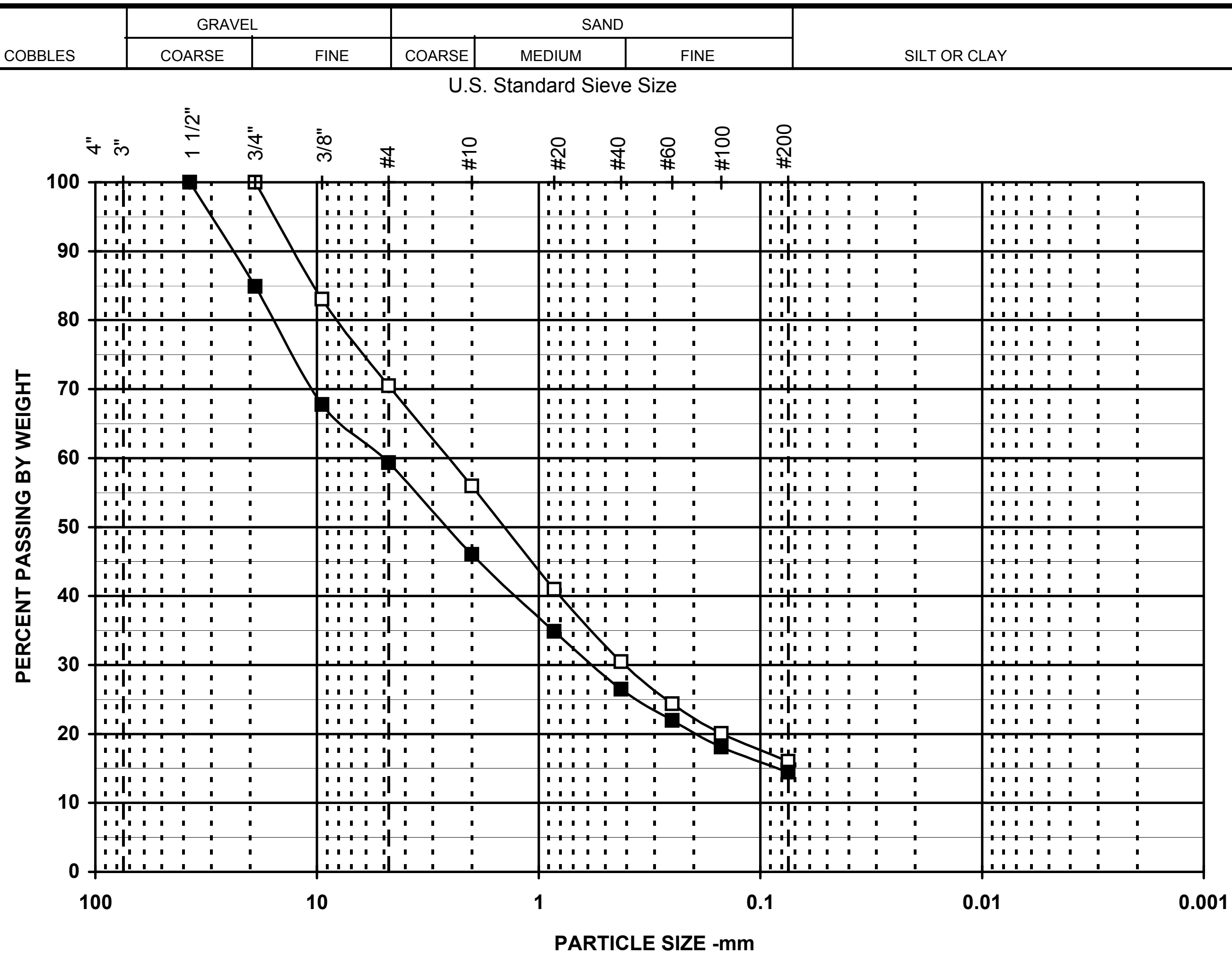


Symbol	□	■	○
Boring	URS-IF-B1	URS-IF-B3	
Sample	S-2	S-2	
Spec			
Depth	5-7	5-7	
% +3"			
% Gravel	29.5	40.6	
% SAND	54.5	44.9	
% FINES	16.0	14.5	
% -2µ			
Cc			
Cu			
LL			
PL			
PI			
USCS	SM	SM	
w (%)	7.8	6.1	

Particle Size (Sieve #)	PERCENT FINER □	■	○
4"			
3"			
1 1/2"		100.0	
3/4"	100.0	84.9	
3/8"	83.0	67.7	
4	70.5	59.4	
10	56.0	46.0	
20	41.0	34.9	
40	30.5	26.5	
60	24.4	22.0	
100	20.1	18.1	
200	16.0	14.5	

SYMBOL	DESCRIPTION AND REMARKS
□	lt. brown c-f SAND, some f. gravel, silt
■	lt. brown gravelly c-f SAND, some silt
○	

PARTICLE SIZE DISTRIBUTION
MDOT Aroostook Bridges/Island Falls

Project No. 39460348-10002 — October 2008

URS Corporation

sievB.xls 10/29/2008

APPENDIX D

SUPPORTING CALCULATIONS

Bearing Resistance - Footings on Bedrock

Determine the Factored Bearing Resistance and Nominal Bearing Resistance for spread footings supported on the bedrock surface.

Service Limit State:

Method A1: Based on AASHTO LRFD (2008) Table C.10.6.2.6.1-1 "Presumptive Bearing Resistances for Spread Footing Foundations at Service State Limit."

Description of Bedrock Material:

Boring URS-B1: Approximately 2 feet of weathered bedrock (rounded to subangular gravel, some fine sand) overlying highly fractured PHYLLITE, RQD = 40% (est.)
Boring URS-B2: Competent PHYLLITE, RQD = 90% (est.)
Boring URS-B3: Approximately 1 foot of weathered bedrock (rounded to subangular gravel, some fine sand) overlying highly fractured PHYLLITE, RQD = 40% (est.)

Bearing Material: Weathered bedrock
Consistency In-place: Medium hard rock
Bearing Resistance: Range = 16 to 24 kips per square foot (ksf)
Recommended Value: 16 ksf

Method A2: Based on AASHTO Standard Specifications - 17th Edition, 2002
Article 4.4.8.1.1 - Footings on Competent Rock
Figure 4.4.8.1.1A - Allowable Contact Stress for Footings on Rock with Tight Discontinuities
For Weathered Bedrock assume RQD = 0%
Allowable Contact Stress = 10 tons per square foot (10 tsf = 20 ksf)

Use a Factored Bearing Resistance of **16 ksf** for the **Service Limit State** analysis and preliminary sizing of the footings.

Strength Limit State:

Method B1: Based on AASHTO Standard Specifications - 17th Edition, 2002
Article 4.4.8.1.2 - Footings on Broken or Jointed Rock

$$q_{ult} = N_{ms}C_o$$

Estimated Rock Mass Quality: Very Poor - Highly weathered with joints spaced less than 2 inches apart.
N_{ms} from Table 4.4.8.1.2A = Use q_{ult} for an equivalent soil mass
Estimated C_o (Uniaxial Compressive Strength) from Table 4.4.8.1.2B
Rock Type - B, Lithified Argillaceous rock: Phyllite
C_o = 3,500 to 35,000 pounds per square inch (psi)
Therefore, $q_{ult} = q_{nom}$ = Use q_{ult} for an equivalent soil mass

Use AASHTO LRFD Theoretical Estimation: Basic Formulation (Article 10.6.3.1.2)
$q_{nom} = cN_m + \gamma D_f N_{qm} C_{wq} + 0.5\gamma B N_{\gamma m} C_{w\gamma}$, where:

$$N_{cm} = N_c s_c i_c$$
$$N_{qm} = N_q s_q d_q i_q$$
$$N_{\gamma m} = N_\gamma s_\gamma i_\gamma$$

1. Soil parameters for granular fill/rip rap assumed to be similar to dense till.

Moist unit weight, γ_m =	145	pounds per cubic foot (pcf)
Saturated unit weight, γ_{sat} =	150	pcf
Angle of internal friction, Φ_{ns} =	36	°
Undrained shear strength, c =	0	pounds per square foot (psf)
Unit weight of water, γ_w =	62.4	pcf

2. Footings will be founded on bedrock surface, so embedment due to riprap protection

Foundation depth, D_f = 3 feet

3. Bearing Capacity Factors from AASHTO LRFD Table 10.6.3.1.2a-1 for Φ_{ns} = 36°:

N_c = 50.6

N_q = 37.8

N_γ = 56.3

4. Assume strip footings (L > 5B) and no load inclination

s_c, s_q, s_γ = 1

i_c, i_q, i_γ = 1

5. Correction for depth to groundwater table (GWT) based on boring data.
(Based on AASHTO LRFD Table 10.6.3.1.2a-2)

Depth to GWT, D_w = D_f

Design unit weight, γ = $\gamma_{sat} - \gamma_w$ = 87.6 pcf

C_{wq} = 1

$C_{w\gamma}$ = 0.5

6. Foundation depth correction (AASHTO LRFD Table 10.6.3.1.2a-4).

d_q = 1

7. Evaluate nominal bearing resistance for footings from 4 to 12 feet wide.

$$\text{Footing width, } B = \begin{cases} 4 & \text{feet} \\ 6 & \text{feet} \\ 8 & \text{feet} \\ 10 & \text{feet} \\ 12 & \text{feet} \end{cases}$$

Therefore,

$$q_{nom} = \begin{cases} 15 & \text{ksf} \\ 17 & \text{ksf} \\ 20 & \text{ksf} \\ 22 & \text{ksf} \\ 25 & \text{ksf} \end{cases}$$

Resistance Factor, Φ_b, from AASHTO LRFD Table 10.5.5.2.2-1 (page 10-32)

For footings on rock, $\Phi_b =$ 0.45

$q_{fac} = q_{nom}\Phi_b$, so

Factored Bearing Resistance :

$$q_{fac} = \begin{cases} 7 & \text{ksf} \\ 8 & \text{ksf} \\ 9 & \text{ksf} \\ 10 & \text{ksf} \\ 11 & \text{ksf} \end{cases}$$

Recommended **Strength Limit State Factored Bearing Resistance** for wall bases and footings that are **4 to 9 feet wide is 7 ksf** and for wall bases and footings that are **9 to 12 feet wide is 10 ksf**.

References

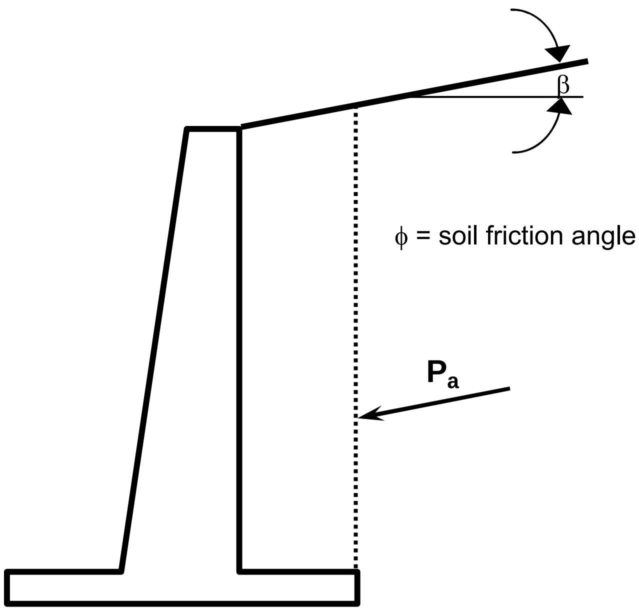
1 American Association of State Highway and Transportation Officials (AASHTO). 2008. *LRFD Bridge Design Specifications* , Customary U.S. Units, 4th Edition, with 2008 Interim Revisions, AASHTO, Washington, D.C.

2 American Association of State Highway and Transportation Officials (AASHTO). 2002. *Standard Specifications for Highway Bridges* , 17th Edition, AASHTO, Washington, D.C.

Lateral Earth Pressure on Abutments and Wingwalls

Determine the lateral earth pressure acting on abutments and wingwalls. Assume that the abutments and wingwalls are unrestrained (i.e., free to rotate at the top).

For unrestrained walls, use Rankine Earth Pressure Theory to determine the active and passive earth pressures (K_a and K_p, respectively).



Rankine Active Earth Pressure Theory

Applicable to cantilever retaining walls and cases where interface friction between the backfill and wall can be neglected.

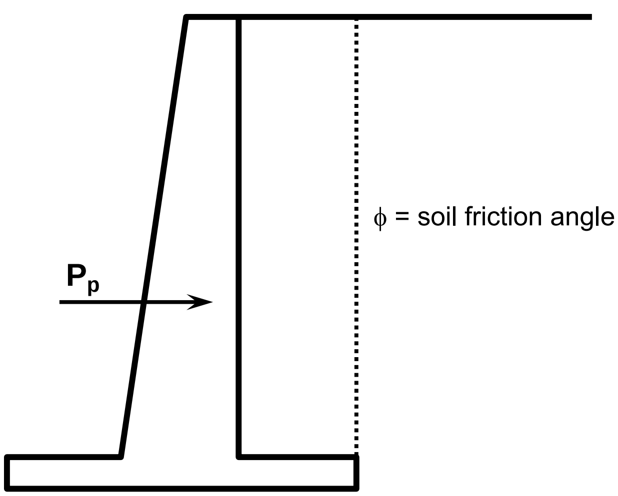
For horizontal backfill surface:

$$K_a = \tan^2 (45 - \phi/2)$$

For sloped backfill, $\beta > 0$:

$$K_a = \cos(\beta) * \left[\frac{\cos(\beta) - \sqrt{\cos^2(\beta) - \cos^2(\phi)}}{\cos(\beta) + \sqrt{\cos^2(\beta) - \cos^2(\phi)}} \right]$$

Active earth pressure, P_a is oriented at angle β



Rankine Passive Earth Pressure Theory

Applicable to cantilever retaining walls and cases where interface friction between the backfill and wall can be neglected. Note, only applicable where the backfill surface is horizontal.

For horizontal backfill surface:

$$K_p = \tan^2 (45 + \phi/2)$$

Passive earth pressure, P_p, is oriented horizontally into the soil mass

1. Soil Properties

Backfill for abutments and wingwalls is assumed to be similar to the soils (i.e., the Fill) observed in borings URS-B1 and URS-B3. Based on the soil encounterd in these borings, blow counts and labaoratory test results, the existing backfill is assumed to have the following properties:

Soil: fine sand and fine to medium gravel, little (14.5% to 16%) silt; water contrent (w_c) = 6% to 8%

Blow Counts (N_{60}) = 9 - 61; assume N_{60} = 20

Assumptions:

Backfill is adequately drained

Total unit weight, γ = 125 pounds per cubic foot (pcf)

Angle of internal friction, Φ = 30 ^{o}

2. Active Earth Pressure Coefficient

Assume that the backfill surface is horizontal, then:

$K_a = \tan^2(45 - \phi/2) = \tan^2(45 - 30/2) =$

$\mathbf{K_a = 0.33}$

3. Passive Earth Pressure Coefficient

Assume that the backfill surface is horizontal, then:

$K_p = \tan^2(45 + \phi/2) = \tan^2(45 + 30/2) =$

$\mathbf{K_p = 3.00}$

4. Soil/wall Interface Friction

Angle of wall interface friction (δ) is assumed to be 2/3 of the angle of internal friction, Φ, of the soil. So,

Angle of wall interface friction (d) = (2/3)(Φ) = 20 ^{o}

And, tan d (Coefficient of Friction for soil to concrete) = 0.36