GEOTECHNICAL DESIGN REPORT
VILLAGE BRIDGE
OAKFIELD, MAINE

P.I.N. 15630.00

Prepared For:

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TABLE OF CONTENTS

TABLE OF CONTENTS

Figures

Sheets

Appendices

This report presents the results of the geotechnical investigation performed for the Maine Department of Transportation (MaineDOT) for the proposed replacement of the Village Bridge (Bridge #2898). The MaineDOT Project Identification Number (PIN) is 15630.00. The bridge is located in Oakfield, Maine and forms the crossing over the East Branch of the Mattawamkeag River.

The objectives of the geotechnical investigation were to characterize the subsurface conditions in the proposed construction area and to develop foundation design recommendations for the proposed replacement structures.

Two test borings were drilled for the purposes of this investigation by URS Corporation (URS). At the location of the Village Bridge replacement structure, approximately 10 feet of medium dense sand fill overlies the stream deposit, which consists of approximately 5 to 10 feet of sand that, in turn, overlies the bedrock surface. The bedrock surface at the bridge location was encountered at depths varying from 15 to 20 feet below the ground surface (bgs), which correspond to approximately elevations 535 to 530 feet. The site is underlain by phyllite bedrock. Estimated groundwater levels at the time of drilling indicate that groundwater was similar to the stream elevation, at approximately elevation 540 feet.

URS understands that complete replacement of the existing, two-span bridge with a longer single span structure is proposed. The project design engineer is proposing to support the west abutment and wingwalls for the replacement bridge structure on a shallow spread footing founded on bedrock, and to support the east abutment and wingwalls on integral piles founded on bedrock.

A shallow spread footing foundation on bedrock can be used for the west abutment and wingwalls. Bedrock is approximately 15 feet below finished grade. The abutment 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 west project abutment 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 west abutment 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.

An integral abutment pile foundation on bedrock is proposed for the east abutment and wingwalls. Bedrock is approximately 20 feet below finished grade. For the integral abutment, the piles will be at least 12 feet in length, and minimal penetration into the bedrock is anticipated. The piles should be end-bearing, and driven to the required resistance on or within the bedrock. Consistent with MaineDOT experience, the piles are expected to be HP 12x53, HP 14x73, HP 14x89 or HP 14x117, depending on the factored design axial loads. Foundation piles should be 50 kips per square inch (ksi), Grade A572 steel H-piles that are fitted with driving points to protect the tips, improve penetration and improve friction at the pile tip.

The strength limit state factored axial compressive resistances of four proposed H-pile sections were calculated. A resistance factor (ϕ_c) of 0.60 was used for the factored structural axial compressive resistances, a resistance factor (ϕ_{stat}) of 0.45 for end-bearing piles was used for the factored geotechnical compressive resistance, and a reduced resistance factor ($\phi_{dyn,80\%}$) of 0.52 was used for the factored driving compressive resistance. The evaluation of the geotechnical bearing resistance considered skin friction to be negligible due to insufficient overburden, therefore it was not incorporated into the total factored geotechnical compressive resistance. For the strength limit state, the factored axial compressive structural, geotechnical and driving resistances for the four proposed H-pile sections are summarized below.

Factored Axial Resistances for Abutment Piles at the Strength Limit State				
Pile Section	Factored Resistance (kips)			
	Structural Resistance	Geotechnical Resistance	Driving Resistance	Design Resistance
HP 12x53	465	217	180	180
HP 14x73	642	314	307	307
HP 14x89	783	385	348	348
HP 14x117	1,032	512	325	325

For the service and extreme event limit states, resistance factors of 1.0 are recommended for structural, geotechnical and driving piles resistances. The factored axial structural, geotechnical and driving resistances for the four proposed H-pile sections are summarized below.

Factored Axial Resistances for Abutment Piles at the Service and Extreme Event Limit States				
Pile Section	**Factored Resistance (kips)**			
	Structural Resistance	**Geotechnical Resistance**	**Driving Resistance**	**Design Resistance**
HP 12x53	775	482	346	346
HP 14x73	1,070	698	591	591
HP 14x89	1,305	855	669	669
HP 14x117	1,720	1,138	625	625

For the strength, service and extreme event limits states, the factored axial driving resistance is less than either the factored axial geotechnical resistance or the factored axial structural resistance, and so the factored axial driving resistance governs the design.

2.1 INTRODUCTION

This report presents the results of the geotechnical investigation performed for the MaineDOT for the proposed replacement of the Village Bridge (MaineDOT Bridge #2898). The MaineDOT Project Identification Number (PIN) is 15630.00. The bridge is located in Oakfield, Maine, and forms the crossing over the East Branch of the Mattawamkeag River. This work was performed in accordance with our scope of work dated July 1, 2008, and authorized by MaineDOT through General Consultant Agreement Number U01210060665.

The objectives of the geotechnical investigation were to characterize the subsurface conditions in the proposed construction area and to develop foundation design recommendations for the proposed replacement structure. This report summarizes the investigation and provides geotechnical recommendations for foundation designs to support the proposed replacement for the Village 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 two borings, one at each abutment location, 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 structure, including bearing resistances for shallow footings on rock, bearing resistances for integral abutments (end-bearing piles) 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 Village Bridge is located approximately 0.2 miles east of US Route 2, on Main Street in Oakfield, Maine. The limits of proposed construction start at approximately Station 12+50, and extend eastward to approximately 500 feet to Station 17+50. The bridge proper extends from approximately Station 14+45 to Station 15+05. A site locus map is presented in Figure 1.

Based on the information provided by MaineDOT, the existing 1930's-era bridge consists of two spans with lengths of approximately 30 feet each. The bridge is constructed of reinforced concrete. The east abutment and center pier for the bridge are concrete and the west abutment is dressed stone.

Information from the project design engineering firm, Erdman, Anthony and Associates, Inc. (Erdman Anthony) of Albany, New York, and MaineDOT indicated that the replacement bridge design will be a 90-foot long single span structure with new abutments located at Station 14+33 and 15+23. The new bridge will be at the same location as the existing bridge. Grades will not significantly change from existing grades. The proposed abutments are to be supported by either a spread footing foundation on bedrock or integral abutments founded on bedrock.

The Village 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 laboratory testing results, are appended to this report.

3.1 SITE DESCRIPTION

The Village Bridge crosses the East Branch of the Mattawamkeag River at point where two channels upstream merge into a single downstream channel. The area on either side of the bridge abutments is heavily vegetated with brush and trees. Earth-fill embankments form the approaches. The river bank has steep side slopes immediately upstream and downstream of the bridge. The topography of the bridge site and surrounding area is shown in Figure 1 and on Sheet 1. Photographs of the sites are in Appendix A.

3.2 LOCAL GEOLOGY

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

The Surficial Geologic Map of Maine[2] indicates that subsurface soil deposits in the Oakfield area are primarily 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. An esker deposit, which is a narrow ridge of glacially deposited sand and gravel, is also identified on the Surficial Geologic Map in the vicinity of the project. Recent stream deposits in the riverbed and flood plain are also present.

3.3 SUBSURFACE INVESTIGATION PROGRAM

The subsurface investigation program for the bridge consisted of two test borings. Borings URS B1 and URS B2 were advanced under the direction of URS Corporation by Northern Test Borings, Inc. of Gorham, Maine in October, 2008. Drilling and sampling activities associated with these borings were performed in the presence of our field geologist. The boring location plan for the bridge is presented on Sheet 1. Boring logs from the URS Corporation investigation are presented in Appendix B and on Sheet 2.

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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.

3.4 LABORATORY ANALYSES

Grain-size analysis tests were performed on soil samples selected as representative of soils at the site and a grab sample of finer grained river sediment. 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	5-7	8.6	SM*	A-1b*	13.7
URS B2	S-2	5-7	9.6	SM*	A-1b*	18.7
URS SED1	-	Stream bed 0-0.5	NA	SM*	A-2-4*	25.1

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

3.5 GENERALIZED SUBSURFACE PROFILES

A generalized interpretive subsurface profile was developed for the bridge site, as presented on Sheet 1. The subsurface conditions can generally be described from the ground surface to the limiting depth of the borings as follows:

Fill Material

Beneath a dense granular pavement subbase, the embankment fill material generally consists of medium dense, fine to coarse sand with little silt and trace to little gravel. The thickness of this stratum is approximately 10 feet at both abutments. The Standard Penetration Resistance N-values[3] in this stratum vary from 9 to 15, with an average value of about 10. The N-values in 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 300 mm (12 inches) after seating the spoon a distance of 150 mm (6-inches) using a 0.62 kN (140 lb) hammer falling freely a distance of 760 mm (30 inches).

Alluvium

Alluvium (i.e., a stream deposit) underlies the fill material. This stratum generally consists of sand with trace to little silt and gravel. Cobbles (URS B1) and limited amounts of organics (URS B2) were also encountered. The alluvium was judged to be a medium density soil. The alluvium was approximately 10 feet thick on east site of the bridge and approximately 5 feet thick on the west side.

Bedrock

Bedrock underlying the entire proposed construction area is light gray, fine-grained phyllite. The bedrock is moderately hard to hard and slightly to moderately fractured, with moderately to steeply dipping bedding planes and fractures. The upper 1 to 4 feet of the bedrock may be weathered and soft, as well as highly to very highly fractured.

Bedrock was encountered in the borings at depths varying from approximately 14.9 feet (west abutment) to 19.8 feet (east abutment) below the ground surface (bgs), which correspond to approximately elevations 535 to 530 feet). Approximately 5 feet of rock coring using an NX-core size core barrel was performed in borings. Core recovery[4] was close to 100 percent for both core runs. The Rock Quality Designation[5] (RQD) value of the rock core was about 36.6 percent in URS B1 (east abutment), and 90 percent in URS B2 (west abutment). Note that the shallower bedrock depth beneath the west abutment corresponds with the more competent, higher RQD, bedrock.

Rock Mass Rating (RMR) of the in-place bedrock can be estimated from the RQD, joint conditions and general observations of the rock core and site conditions assuming a uniaxial compression strength of intact bedrock of 3,500 to 35,000 pounds per square inch[6].

Rock Core	RMR	Class Number[7]	Description[7]
URS B1	49	III	Fair Rock
URS B2	61	II	Good Rock

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 100 mm (4 in) for NX-core) to the total length cored.

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 GROUNDWATER CONDITIONS

Estimated groundwater levels in the borings at the time of drilling, indicated that the groundwater elevation was similar to the river elevation, at a depth of approximately 10 feet bgs, which corresponds to approximately elevation 540 feet. Groundwater and river levels are shown on the subsurface profiles in Sheet 1. Seasonal variations in the water surface and groundwater elevations will occur, as indicated by water stains on the existing bridges structures (see the photographs in Appendix A).

3.7 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 is considered negligible as these soils are generally in a medium 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 abutments, and wingwalls. The project design engineer, Erdman Anthony, is proposing to support the west abutment and wingwalls for the replacement bridge structure on a shallow spread footing founded on bedrock. The project design engineer is also proposing to support the east abutment and wingwalls on integral piles founded on bedrock. The design methodology used in the following evaluations is based on AASHTO LRFD (2008).

4.2 SHALLOW FOUNDATION

Boring URS-B2 encountered bedrock approximately 15 feet below the existing western bridge approach. 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.

4.2.1 West Abutment and Wingwall Design

The abutment 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 west project abutment 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 strength limit state design shall also consider foundation resistance after scour due to the design flood event. 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 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 West Abutment

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 0.31, (i.e., $K_a = 0.31$), 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. The designer may assume Soil Type 4 (MaineDOT BDG, 2003[9]) for backfill material soil properties. The backfill soil properties for Soil Type 4 are summarized below.

Backfill Properties for Active Lateral Earth Pressure Analyses	
Design Parameter	**Value**
Total unit weight of backfill (γ)	125 pcf
Angle of internal friction (ϕ)	32^{o}
Angle of wall interface friction (δ)	24^{o}
Coefficient of Friction, tan δ Soil to concrete	0.45

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:

[9] MaineDOT BDG. 2003. *Bridge Design Guide*, prepared by Guertin Elkerton and Associates for Maine Department of Transportation, Section 3.6.1.

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.

Abutment designs shall include a drainage system behind the abutments to intercept groundwater. 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.

Backfill within 10 feet of the back of the abutments, wingwalls and side slope fills shall conform to MaineDOT Standard Specification[10] 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 West Abutment Factored Bedrock Bearing Resistance

Substructure spread footings for the west abutment 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

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

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.

4.2.4 West Abutment Settlement

URS understands that the current bridge replacement plans do not include changes to the profile. Additionally, the spread footings 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 INTEGRAL ABUTMENT PILE FOUNDATION

An integral abutment pile foundation on bedrock is proposed for the east abutment and wingwalls. The piles should be end-bearing, and driven to the required resistance on or within the bedrock. Piles are expected to be HP 12x53, HP 14x73, HP 14x89 or HP 14x117, depending on the factored design axial loads. Foundation piles should be 50 kips per square inch (ksi), Grade A572 steel H-piles that are fitted with driving points to protect the tips, improve penetration and improve friction at the pile tip. Since H-piles are typically used on MaineDOT projects, these types of piles were evaluated. However, other pile types or sections may be used after review and approval by the Geotechnical Engineer.

Bedrock is approximately 20 feet below finished grade. For the integral abutment, the piles will be at least 12 feet in length, and minimal penetration into the bedrock is anticipated. The minimum pile length does not include embedment in the pile cap or lead length required for installation (if applicable).

4.3.1 Pile Design

Design of the H-piles at the strength limit state should consider the combined axial and flexural structural resistance of the piles, and the axial geotechnical resistance of the piles. The structural resistance evaluation should include confirming the axial, lateral and flexural resistance. Resistance factors for use in the design of piles at the strength limit state are discussed below.

The design of H-piles at the service limit state should consider tolerable horizontal movement of the piles and overall stability of the pile group. Since the east abutment piles will be subject to lateral loading, the piles should be analyzed for axial loading and combined axial and lateral loading, in accordance with AASHTO LRFD (2008) Articles 6.15.2 and 6.15.3, respectively.

4.3.1.1 Strength Limit State

The nominal structural compressive resistance (Pn) in the strength limit state for piles loaded in compression shall be in accordance with the requirements of AASHTO LRFD (2008) Article 6.9.4.1. The H-piles are assumed to be fully embedded and the normalized column slenderness factor (λ) shall be taken as 0. The factored structural axial compressive resistances of the four proposed H-pile sections were calculated using a resistance factor (ϕ_c) of 0.60.

The nominal geotechnical strength in the strength limit state was calculated using the Pell, Turner, Tomlinson Method[11] for estimating the nominal tip resistance of end-bearing piles founded on rock. The factored geotechnical compressive resistance for the four proposed H-pile sections were calculated using a resistance factor (ϕ_{stat}) of 0.45 for end-bearing. Skin friction was considered to be negligible due to insufficient overburden, and not incorporated into the total factored geotechnical compressive resistance.

The maximum driving stresses in the piles, assuming the use of 50 ksi steel, shall be less than 45 ksi. The resistance factor, ϕ_{dyn}, for a single pile in axial compression with the driving resistance established by a dynamic load test, in accordance with AASHTO LRFD Table 10.5.5.2.3-1, is 0.65 (i.e., ϕ_{dyn} = 0.65). However, AASHTO LRFD Table 10.5.5.2.3-1 requires no less than 3 or 4 dynamic tests be conducted for sites with low or medium variability, respectively. Since one dynamic load test is typically conducted for each abutment at a site (i.e., one dynamic load test is anticipated for this site), the resistance factor shall be reduced by 20 percent, resulting in a resistance factor of 0.52 (i.e., ϕ_{dyn} = 0.52).

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[11] Tomlinson, M. J. 1994. *Pile design and Construction Practice*, 4th Edition, Taylor & Francis, Inc. New York, NY, 411 pages

The drivability analysis was performed using a wave equation analysis program (the GRLWEAP® software) to determine the ultimate pile resistance (R_{ult}) for four typical H-Piles. This analysis assumed that a Delmag D19-42 hammer would be used to drive the piles, and the input data for the hammer, hammer cushion and pile were based on typical values used by MaineDOT. For each pile, R_{ult} was estimated from the GRLWEAP output by linear interpolation, based on either the maximum allowable driving stress in a pile (i.e., 45 ksi) or the maximum allowable blowcounts per foot (180 blows per foot).

For the strength limit state, the factored axial compressive structural, geotechnical and driving resistances for the four proposed H-pile sections are summarized below. Supporting calculations are provided in Appendix D.

Factored Axial Resistances for Abutment Piles at the Strength Limit State				
Pile Section	Factored Resistance (kips)			
	Structural Resistance	Geotechnical Resistance	Driving Resistance	Design Resistance
HP 12x53	465	217	180	180
HP 14x73	642	314	307	307
HP 14x89	783	385	348	348
HP 14x117	1,032	512	325	325

The factored axial driving resistance is less than either the factored axial structural resistance or the factored axial geotechnical resistance. Therefore, **the factored axial driving resistance governs the strength limit state design**.

In accordance with AASHTO LRFD (2008) Article 6.5.4.2, at the strength limit state the axial resistance factor, ϕ_c, is 0.7, (i.e., ϕ_c = 0.7) and the flexural resistance factor, ϕ_f, is 1.0, (i.e., ϕ_f = 1.0) for H-piles in compression and flexure. These resistance factors shall be applied to the combined axial and flexural resistance of the piles in the interaction equation. For the strength limit state, the combined axial compression and flexure should be evaluated as shown in AASHTO LRFD (2008) Article 6..9.2.2. The structural designer should evaluate the capacity of the piles in combined axial load and flexure when the loads and moments are calculated. Moments resulting from the abutment wingwalls must also be considered in the design of the piles.

4.3.1.2 Service and Extreme Event Limit States

For the service and extreme event limit states, resistance factors of 1.0 are recommended for structural and geotechnical piles resistances. The factored axial structural, geotechnical and driving resistances for the four proposed H-pile sections are summarized below. Supporting calculations are provided in Appendix D.

Factored Axial Resistances for Abutment Piles at the Service and Extreme Event Limit States				
Pile Section	Factored Resistance (kips)			
	Structural Resistance	Geotechnical Resistance	Driving Resistance	Design Resistance
HP 12x53	775	482	346	346
HP 14x73	1,070	698	591	591
HP 14x89	1,305	855	669	669
HP 14x117	1,720	1,138	625	625

The factored axial driving resistance is less than either the factored axial structural resistance or the factored axial geotechnical resistance. Therefore, **the factored axial driving resistance governs the service and extreme event limit state design.**

4.3.2 Integral Stub Abutments and Wingwalls

Integral stub abutments and wingwalls should be designed for all relevant strength, service and extreme event limit states and load combinations specified in AASHTO LRFD (2008) Articles 3.4.1, 11.5.5 and 11.6.1.3. The design of abutments and wingwalls at the strength limit state shall consider structural failure.

Integral abutments and wingwalls shall be designed to resist and/or absorb lateral earth loads, vehicular load, superstructure loads, creep and temperature and shrinkage deformations of teh superstructure. the integral abutments and wingwalls shall be designed for all relevant service and strength limit states. If the plans call for stub abutments and “butterfly” wingwalls, the design should size the piles to account for the additional bending moment stress resulting from the wingwall configuration,

4.3.3 Integral abutment and Wingwall Lateral Earth Pressures

Integral abutment and integral wingwall sections shall be designed to resist passive earth pressure using a Rankine earth pressure coefficient, K_p, of 3.25. Wingwall sections that are independent of the abutment should be designed using the Rankine active earth pressure coefficient, K_a, of 0.31. Both earth pressure coefficients (i.e., K_p and K_a) assume a level backfill surface. One or both earth pressure coefficients may change if the backfill surface conditions are different (e.g., sloping). See Appendix D – Calculations for supporting documentation for the passive and active earth pressure coefficients.

4.4 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, 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.

Specifically, the pile foundation will require protection from scour, therefore, a scour analysis must be performed. The river bed sediment sample obtained by URS (see Section 3.3) indicates that the finer fraction of the visible stream bed is a silty sand.

4.5 FROST PROTECTION

The proposed foundations for the west abutment and wingwalls are spread footings supported on bedrock and the proposed foundations for the west abutment and wingwalls are integral abutments. 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. Furthermore, the base of the pile cap for the integral abutment should be at least 4 feet below the finished grade.

4.6 SEISMIC DESIGN CONSIDERATIONS

In accordance with the guidance in AASHTO LRFD (2008) Article 4.7.4.2, seismic analysis is not required for single-span bridges, regardless of the seismic zone. 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. Furthermore, 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.” Consequently, seismic earth loads do not need to be considered for design of the bridge substructure.

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 indicates that the upper approximately 1 foot of the bedrock surface is weathered and highly fractured, so 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 10 feet bgs and is controlled by the river level, installation of 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 abutment placed on piles should be removed and replaced with 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 RE-USE OF EXISTING EMBANKMENT SOILS

The existing embankment soils are silty sand which appear adequate for re-use as common borrow. Excavated embankment soils may be stockpiled and re-used 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 PILE RESISTANCE AND QUALITY CONTROL

The contractor is required to perform a wave equation analysis of the proposed pile-hammer system and a dynamic pile test at the integral abutment. The first pile driven at the integral abutment should be dynamically tested to confirm the capacity and verify the stopping criteria developed by the contractor from the wave equation analysis. The nominal pile resistance that must be achieved in the wave equation analysis and dynamic testing will be the maximum factored axial pile load divided by a resistance factor of 0.52. The maximum factored pile load should be shown on the plans. If three or four piles are dynamically tested (at sites with low or medium variability, respectively), the resistance factor may be increased by 20 percent to 0.65.

Piles should be driven to an acceptable penetration resistance as determined by the contractor, based on the results of a wave equation analysis, the dynamic test results and as approved by the MaineDOT resident. Driving stresses in the pile determined in the drivability analysis shall be less than 45 ksi, in accordance with AASHTO LRFD (2008) Article 10.7.8. The contractor should select a hammer that provides the required nominal resistance when the penetration for the final 3 to 6 inches is 8 to 13 blows per inch. If an abrupt increase in driving resistance is encountered, the driving could be terminated when the pile penetration is less than ½-inch in 10 consecutive blows.

5.6 SCOUR PROTECTION

As noted above in Section 4.4, any footing for wingwalls or retaining walls that are constructed on soil should be armored with riprap for scour protection. 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)[12].

5.7 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[13].

---

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

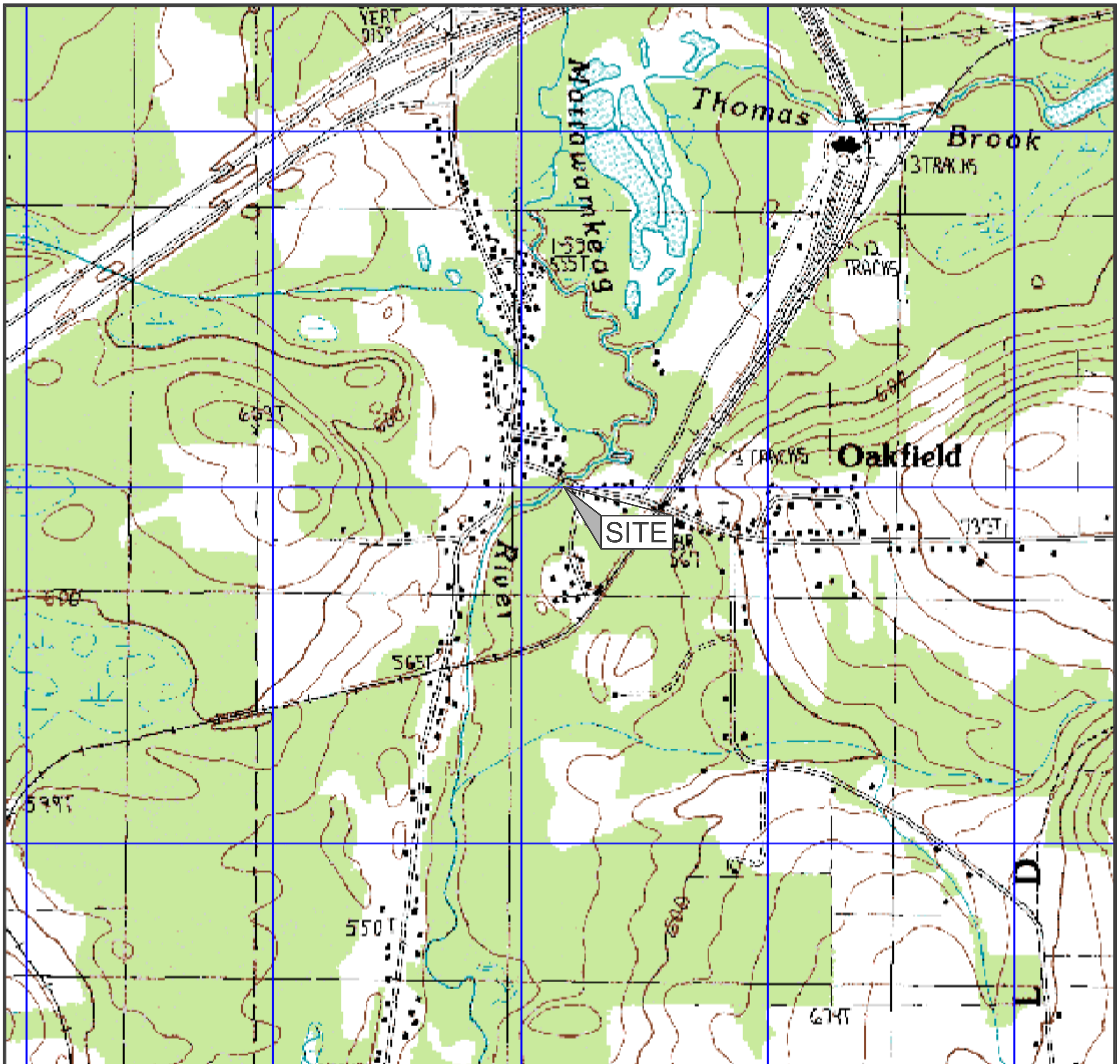
[13] MaineDOT. 2008. *MaineDOT Best Management Practices for Erosion and Sedimentation Control*. Maine Department of Transportation.

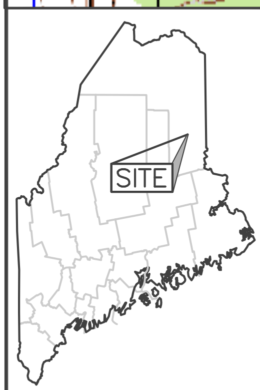
The results and recommendations presented in this report are largely based on subsurface information from a limited number of borings, laboratory tests, and our use of generally accepted analytical procedures. Subsurface conditions may vary from those presented in this report, and these variances may 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





0 2000 4000

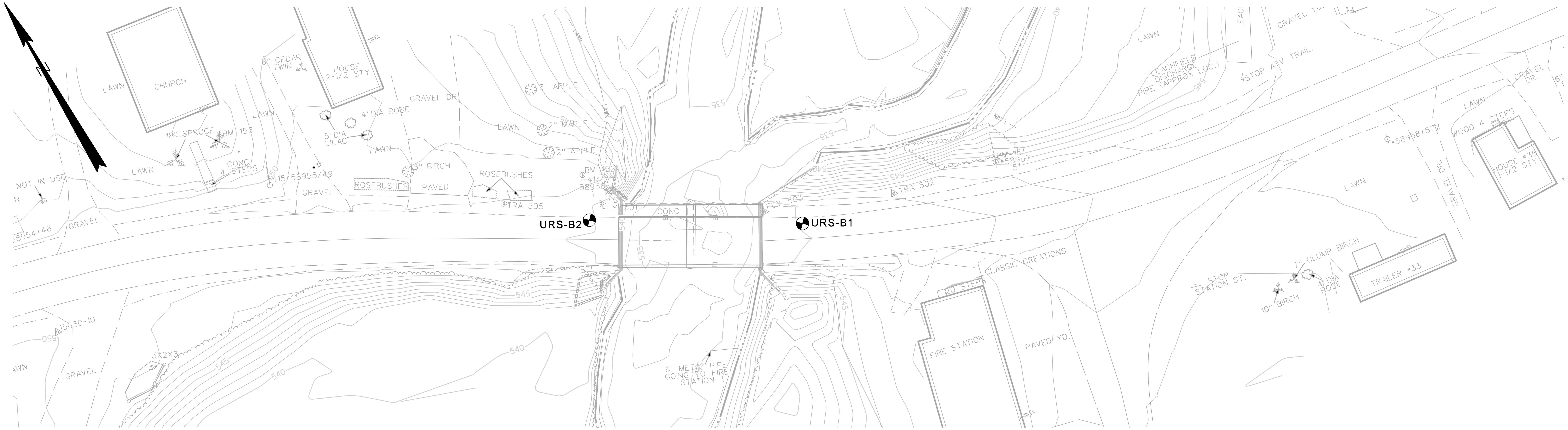
SCALE, FEET

N

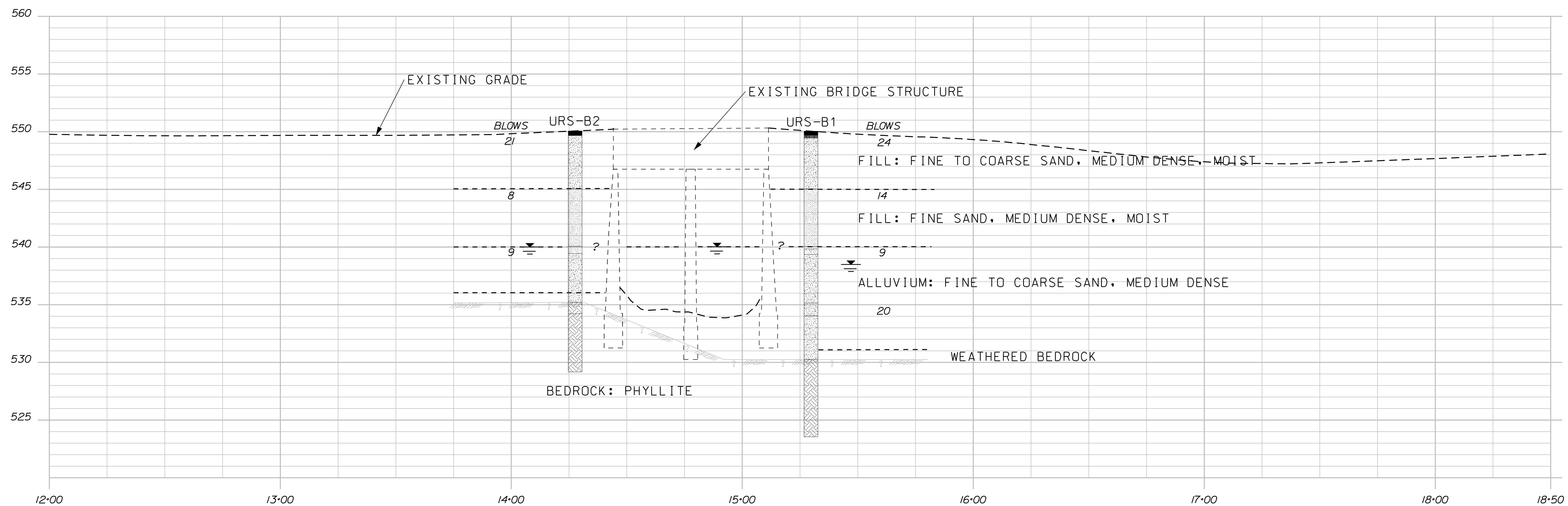
SOURCE:
USGS 7.5–MINUTE TOPOGRAPHIC MAP OF THE OAKFIELD, MAINE QUADRANGLE DATED 1986.

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: VILLAGE BRIDGE OAKFIELD, MAINE MDOT PIN D15630.00		SITE LOCUS	1
APPROVED: GKT	DATE: 12/30/08				
DRAWN: LRH	FILE NO: FIG1				



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-A530(000)X

PIN 015630.00
BRIDGE NO. 2898
BRIDGE PLANS

	BY	DATE	
PROJ. MANAGER DEVIN ANDERSON			SIGNATURE
DESIGN-DETAILED	BURNHEIMER	02/12/09	3601
CHECKED-REVIEWED	D. ANDREWS		P.E. NUMBER
DESIGN2-DETAILED2			JANUARY 2009
DESIGN3-DETAILED3			DATE
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

VILLAGE BRIDGE
EAST BRANCH MATTAWAMKEAG
OAKFIELD AROOSTOOK

BORING LOCATION PLAN AND
SUBSURFACE INTERPRETIVE PROFILE

SHEET NUMBER
1
1 OF 2

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

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

Project: Village Bridge Replacement
Location: Oakfield, Maine
Boring No.: URS-B1
PIN: 15630.00

Driller:	Northern Test Borings, Inc	Elevation (ft.)	549.9	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:	10/6/08	Drilling Method:	Cased Boring	Core Barrel:	N size
Boring Location:	East Abutment	Casing ID/OD:	4 inch	Water Level*:	10'
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	24/17	0.00 - 2.00	31/8/16/7	24	25		549.55		Bituminous Pavement 0.35	SP
								549.30		Pulverized Pavement 0.60	
										Brown fine to coarse Sand, little subangular to angular gravel, medium dense, dry, {FILL} 5.00	
	S-2	24/12	5.00 - 7.00	5/9/5/4	14	15		544.90		Brown fine Sand, little silt and subangular to angular gravel, medium dense, dry,{FILL} 10.00	WC 8.6%, 13.7% -#200 Sieve, USCS SM
10	S-3	24/18	10.00 - 12.00	3/4/5/5	9	9		539.90		Brown fine Sand, little silt, wet, {Alluvium?} 10.20	SP
								539.70		Fine to coarse Sand and Gravel, wet, {Alluvium} 10.70	SP
								539.20		Dark brown, fine Sand, trace silt, wet,{Alluvium} 14.90	SP
								535.00		Cobble 16.00	
	S-4	24/12	16.00 - 18.00	4/8/12/34	20	21		533.90		Rounded to subangular Gravel, some fine Sand, weathered rock, wet 19.80	SP
20	R	60/56	19.80 - 24.80	RQD = 36.3%				530.10		Phyllite, very fine grained, hard, slight to moderate weathering, vertical(70 degree) fractures, closely fractured, light gray, difficult coring due to fracture orientation 26.50	
								523.40		Bottom of Exploration at 26.50 feet below ground surface.	
30											
40											
50											
60											

Remarks:

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 ??

Boring No.: URS-B1

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

Project: Village Bridge Replacement
Location: Oakfield, Maine
Boring No.: URS-B2
PIN: 15630.00

Driller:	Northern Test Borings, Inc	Elevation (ft.)	549.9	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:	10/6/08	Drilling Method:	Cased Boring	Core Barrel:	N size
Boring Location:	West Abutment	Casing ID/OD:	4 inch	Water Level*:	10'
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	24/6	0.40 - 2.40	5/12/9/7	21	22		549.50		Bituminous Pavement 0.40	SP
										Brown fine Sand and subangular to angular fine gravel, medium dense, dry, {FILL} 5.00	
	S-2	24/17	5.00 - 7.00	3/4/4/5	8	8		544.90		Brown fine to coarse Sand, little silt and subangular to angular gravel, loose, dry, {FILL} 10.00	WC 9.6%, 18.7% -#200 Sieve, USCS SM
10	S-3	24/15	10.00 - 12.00	3/6/3/4	9	9		539.90		Brown fine to coarse Sand, little silt and subangular to angular gravel, loose, dry, {FILL} 10.70	SP
								539.20		Dark brown, fine Sand, little Gravel, trace silt and organics, wet, {Alluvium} 14.90	SP
	R	60/60	15.90 - 20.90	RQD = 90%				535.00		Very dense soil and/or weathered bedrock 15.90	
								534.00		Phyllite, very fine grained, hard, slight weathering, near vertical (70 deg.) fractures, little fracturing, light gray, 20.90	
20								529.00		Bottom of Exploration at 20.90 feet below ground surface.	
30											
40											
50											
60											

Remarks:

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

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BR-A530(000)X

BRIDGE PLANS

PIN
015630.00

BRIDGE NO. 2898

SIGNATURE
3601
P.E. NUMBER
JANUARY 2009
DATE

		BY	DATE
PROJ. MANAGER	DEVIN ANDERSON		
DESIGN-DETAILED	D. ANDREWS	BURNHEIMER	02/12/09
CHECKED-REVIEWED			
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

VILLAGE BRIDGE
EAST BRANCH MATTAWAMKEAG
OAKFIELD AROOSTOOK

BORING LOGS

SHEET NUMBER
2
2 OF 2

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

APPENDIX A

SITE PHOTOGRAPHS

PHOTOGRAPHIC LOG

Client Name:	Site Location:	Project No.
Maine DOT	Village Bridge, Oakfield, Maine	39460348

Photo No.	Date:
1	9/15/08

Direction Photo Taken:

Upstream side of bridge, looking west

Description:

Photo No.	Date:
2	9/15/08

Direction Photo Taken:

West abutment looking downstream.

Description:

.

URS	PHOTOGRAPHIC LOG	
Client Name: Maine DOT	**Site Location:** Village Bridge, Oakfield, Maine	**Project No.** 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: Village Bridge Replacement		Boring No.:	URS-B1
Location: Oakfield, Maine		PIN:	15630.00

Driller:	Northern Test Borings, Inc	Elevation (ft.)	549.9	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:	10/6/08	Drilling Method:	Cased Boring	Core Barrel:	N size
Boring Location:	East Abutment	Casing ID/OD:	4 inch	Water Level*:	10'
Hammer Efficiency Factor:	0.63	Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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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
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WC = water content, percent
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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	24/17	0.00 - 2.00	31/8/16/7	24	25		549.55		Bituminous Pavement — 0.35	SP
								549.30		Pulverized Pavement — 0.60	
	S-2	24/12	5.00 - 7.00	5/9/5/4	14	15		544.90		Brown fine to coarse Sand, little subangular to angular gravel, medium dense, dry, {FILL} — 5.00	WC 8.6%, 13.7% -#200 Sieve, USCS SM
								539.90		Brown fine Sand, little silt and subangular to angular gravel, medium dense, dry, {FILL} — 10.00	
10	S-3	24/18	10.00 - 12.00	3/4/5/5	9	9		539.70		Brown fine Sand, little silt, wet, {Alluvium?} — 10.20	SP
								539.20		Fine to coarse Sand and Gravel, wet, {Alluvium} — 10.70	SP
								535.00		Dark brown, fine Sand, trace silt, wet, {Alluvium} — 14.90	SP
	S-4	24/12	16.00 - 18.00	4/8/12/34	20	21		533.90		Cobble — 16.00	SP
								530.10		Rounded to subangular Gravel, some fine Sand, weathered rock, wet — 19.80	
20	R	60/56	19.80 - 24.80	RQD = 36.3%				523.40		Phyllite, very fine grained, hard, slight to moderate weathering, vertical(70 degree) fractures, closely fractured, light gray, difficult coring due to fracture orientation — 26.50	
										Bottom of Exploration at 26.50 feet below ground surface.	
30											
40											
50											
60											

Remarks:

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:	Village Bridge Replacement			Boring No.:	URS-B2
Location:	Oakfield, Maine			PIN:	15630.00
Driller:	Northern Test Borings, Inc	Elevation (ft.)	549.9	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:	10/6/08	Drilling Method:	Cased Boring	Core Barrel:	N size
Boring Location:	West Abutment	Casing ID/OD:	4 inch	Water Level*:	10'
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	24/6	0.40 - 2.40	5/12/9/7	21	22		549.50		Bituminous Pavement 0.40 Brown fine Sand and subangular to angular fine gravel, medium dense, dry, {FILL}	SP
	S-2	24/17	5.00 - 7.00	3/4/4/5	8	8		544.90		5.00 Brown fine to coarse Sand, little silt and subangular to angular gravel, loose, dry, {FILL}	WC 9.6%, 18.7% -#200 Sieve, USCS SM
10	S-3	24/15	10.00 - 12.00	3/6/3/4	9	9		539.90 539.20		10.00 Brown fine to coarse Sand, little silt and subangular to angular gravel, dry, {FILL} 10.70 Dark brown, fine Sand, little Gravel, trace silt and organics, wet, {Alluvium}	SP SP
	R	60/60	15.90 - 20.90	RQD = 90%				535.00 534.00		14.90 Very dense soil and/or weathered bedrock 15.90 Phyllite, very fine grained, hard, slight weathering, near vertical (70 deg.) fractures, little fracturing, light gray,	
20								529.00		20.90 **Bottom of Exploration at 20.90 feet below ground surface.**	
30											
40											
50											
60											

Remarks:

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

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

Boring No.: URS-B2

APPENDIX C

LABORATORY DATA

Project No.: 39460348.20002
File: IndexA.xls

MDOT Aroostook Bridges/Oackfield

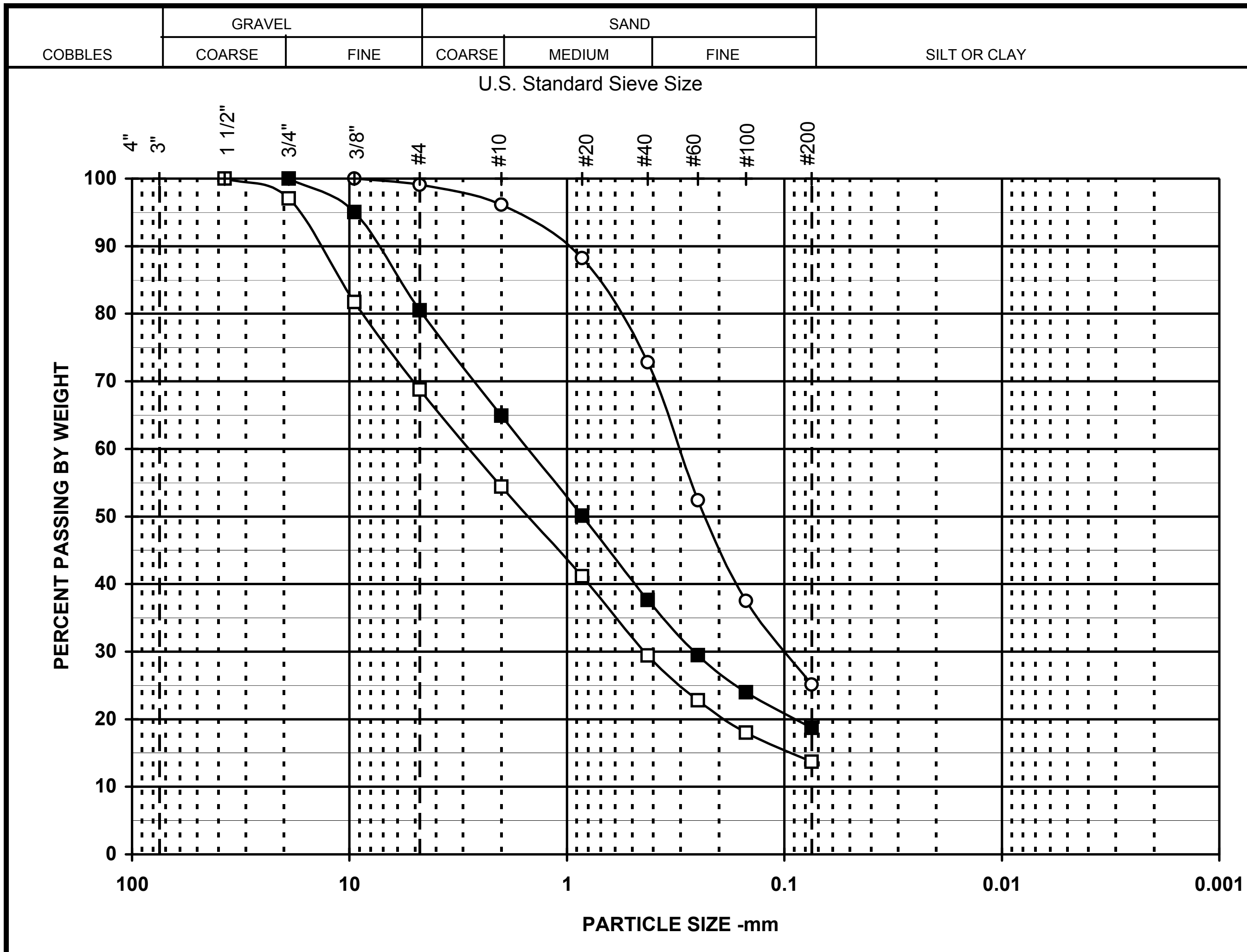
LABORATORY TESTING DATA SUMMARY

BORING NO.	SAMPLE NO.	DEPTH (ft)	IDENTIFICATION TESTS			REMARKS
			WATER CONTENT (%)	USCS SYMB. (1)	SIEVE MINUS NO. 200 (%)	
URS-OF-B1	S-2	5-7	8.6	SM	13.7	
URS-OF-B2	S-2	5-7	9.6	SM	18.7	
URS-SED1			113.8	SM	25.1	

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

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

URS Corporation
45 J Commerce Way
Totowa, NJ 07512



Symbol	□	■	○
Boring	URS-OF-B1	URS-OF-B2	URS-SED1
Sample	S-2	S-2	
Spec			
Depth	5-7	5-7	
% +3"			
% Gravel	31.2	19.5	0.9
% SAND	55.1	61.8	74.0
% FINES	13.7	18.7	25.1
% -2μ			
Cc			
Cu			
LL			
PL			
PI			
USCS	SM	SM	SM
w (%)	8.6	9.6	113.8

Particle Size (Sieve #)	PERCENT FINER □	■	○
4"			
3"			
1 1/2"	100.0		
3/4"	97.0	100.0	
3/8"	81.7	95.0	100.0
4	68.8	80.5	99.1
10	54.4	64.9	96.2
20	41.2	50.1	88.2
40	29.4	37.6	72.8
60	22.8	29.5	52.4
100	18.0	24.0	37.5
200	13.7	18.7	25.1

SYMBOL	DESCRIPTION AND REMARKS
□	brown gravelly c-f SAND, some silt
■	brown c-f SAND, some f. gravel, silt
○	dark brown m-f. SAND, some silt, tr. c. sand

PARTICLE SIZE DISTRIBUTION

MDOT Aroostook Bridges/Oackfield

Project No. 39460348-20002 | October 2008

URS Corporation

sievA.xls 10/29/2008

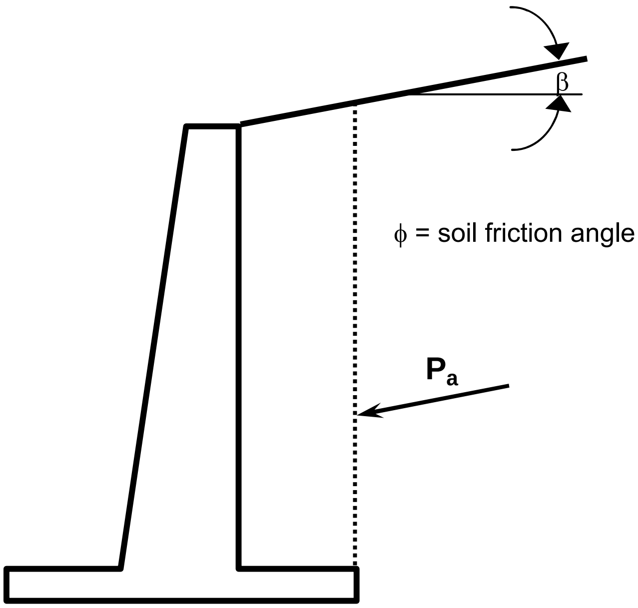
APPENDIX D

SUPPORTING CALCULATIONS

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.

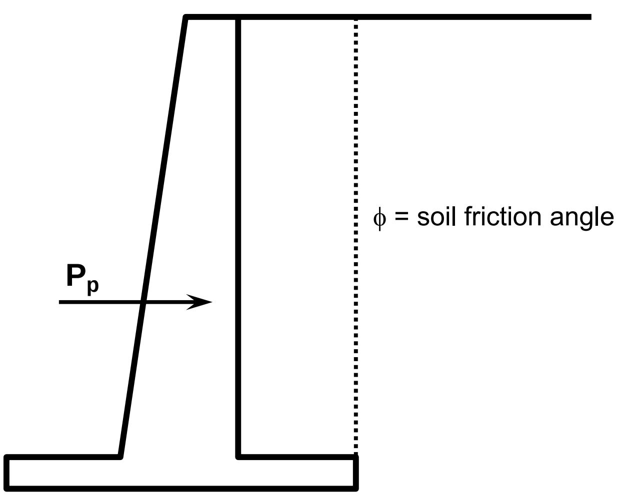
For horizontal backfill surface:

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

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:

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

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

1. Soil Properties

Backfill for abutments and wingwalls to meet MaineDOT Specification for underwater backfill. In the MaineDOT *Bridge Design Guide* (2003) this is Soil Type 4, which has the following properties:

Total unit weight, γ =	125 pounds per cubic foot (pcf)
Angle of internal friction, Φ =	32 ^{o}

2. Active Earth Pressure Coefficient

Assume that the backfill surface is horizontal, then:

$K_a = \tan^2(45 - \phi/2) = \quad \tan^2(45 - 32/2) =$

$\mathbf{K_a = 0.31}$

3. Passive Earth Pressure Coefficient

Assume that the backfill surface is horizontal, then:

$K_p = \tan^2(45 + \phi/2) = \quad \tan^2(45 + 32/2) =$

$\mathbf{K_p = 3.25}$

Integral Abutment and Traditional End-bearing Driven H-Piles

Determine the Factored Compressive Resistance for integral abutments and traditional end-bearing piles supported on the bedrock surface.

1. Structural Axial Resistance of Individual H-Piles

Strength Limit State:

Evaluate the following H-piles:

Section	Steel Area, A_s (in^2)	Pile Depth, d (in)	Pile Width, w (in)	"Box" Area, A_{box} (in^2)	"Box" Area, A_{box} (ft^2)
HP 12x53	15.5	11.78	12.05	141.9	0.99
HP 14x73	21.4	13.61	14.59	198.6	1.38
HP 14x89	26.1	13.83	14.70	203.3	1.41
HP 14x117	34.4	14.21	14.89	211.6	1.47

Yield Strength, f_y = 50 kips per square inch (ksi)

Nominal Compressive Resistance: Based on AASHTO LRFD (2008) Article 6.9.4.1 (page 6-73)

Nominal Compressive Resistance, $P_n = 0.66^{\lambda} f_y A_s$, where

λ = normalized column slenderness factor = $([K\ell]/[r_s\pi])^2(f_y/E)$

$\lambda = 0$ when the unbraced length, $\ell = 0$, so $P_n = f_y A_s$

Section	P_n
HP 12x53	775 kips
HP 14x73	1,070 kips
HP 14x89	1,305 kips
HP 14x117	1,720 kips

Factored Compressive Resistance: Based on AASHTO LRFD (2008) Article 6.5.4.2 (page 6-28)

Factored Compressive Resistance, $P_f = \Phi_c P_n$

Resistance factor, Φ_c, for axial resistance of piles in compression under good driving conditions.

Φ_c = 0.60

Section	P_f
HP 12x53	465 kips
HP 14x73	642 kips
HP 14x89	783 kips
HP 14x117	1,032 kips

Strength Limit State Factored Compressive Resistance

Service/Extreme Limit State:

Nominal Compressive Resistance: As determined for the Strength Limit State

Factored Compressive Resistance: Based on AASHTO LRFD (2008) Articles 6.5.5 (page 6-29), 10.5.5.1 (page 10-32), and 10.5.5.3 (page 10-43).

Factored Compressive Resistance, $P_f = \Phi_c P_n$

Resistance factor, Φ_c, shall be taken to be 1.0 for the extreme event limit state.

Φ_c = 1.00

Section	P_f
HP 12x53	775 kips
HP 14x73	1,070 kips
HP 14x89	1,305 kips
HP 14x117	1,720 kips

Service and Extreme Event Limit State Factored Compressive Resistance

2. Geotechnical Axial Resistance of Individual H-Piles

Nominal Tip Resistance, $R_{t,nom}$:

Based on Pell, Turner, Tomlinson Method for estimating the nominal tip resistance of end-bearing piles founded on rock presented in Tomlinson (1994)

Assume piles will be driven trhough overlying fill and alluvium deposits and end-bearing on bedrock.

Description of Bedrock Material:

Boring URS-B1: Approximately 4 feet of weathered bedrock (rounded to subangular gravel, some fine sand) overlying highly fractured PHYLLITE, RQD = 36%

Boring URS-B2: Approximately 1 foot of weathered bedrock overlying highly fractured PHYLLITE, RQD = 90%

Summary of Assumed Design Parameters

Bearing Material: Phyllite, RQD = 36%; RQD improves with depth

Consistency In-place: Medium hard rock, tight joints, good core recovery; Upper zone weathered.

Estimated q_{uc} (i.e., Uniaxial Compressive Strength, C_o) from AASHTO 2002 Table 4.4.8.1.2B

Rock Type - B, Lithified Argillaceous rock: Phyllite

q_{uc} = 3,500 to 35,000 pounds per square inch (psi); use q_{uc} = 12,500 psi

Nominal Tip Resistance per unit area, q_p, is estimated as follows:

$q_p = F_{wc}cN_c + 0.5F_{w\gamma}\gamma B_{min}N_\gamma + \gamma DN_q$, where:

1. Rock parameters for analysis.
For RQD = 0 to 70 %

$q_c = 0.33q_{uc}$ =	(0.33)(12,500) =	4,125	psi
$c = 0.1q_{uc}$ =	(0.1)(12,500) =	1,250	psi
Unit weight of rock, γ =		145	pcf
Angle of internal friction, Φ =		30	°

2. Corrections for wedge failure under strip footing

F_{wc}, $F_{w\gamma}$ = corrections for wedge failure

F_{wc} = 1.25 for square piles, such as H-piles

$F_{w\gamma}$ = 0.8 for square piles, such as H-piles

3. Bearing Capacity Factors from Tomlinson (1994) page 139

N_c = 13.86

N_q = 9.00

N_γ = 13.86

4. Pile will be founded on bedrock surface, assume no penetration.

Embedment depth, D = 0 inches

5. Width of H-pile.

Use minimum dimension for H-Piles shown above

6. Calculate nominal tip resistance per unit area for four H-Piles shown above.

q_p =

HP 12x53	31 kips per square inch (ksi)
HP 14x73	33 ksi
HP 14x89	33 ksi
HP 14x117	33 ksi

7. Calculate nominal tip resistance, $R_{t,nom}$, for four typical H-Piles (summarized above).

HP 12x53	$R_{t,nom}$ =	482 kips
HP 14x73		698 kips
HP 14x89		855 kips
HP 14x117		1,138 kips

8. Calculate Factored Geotechnical Tip Resistance, $R_{tip,f}$, for four typical H-Piles.

$R_{tip,f} = \Phi_{stat} R_{t,nom}$

Resistance factor, Φ_{stat}, for single pile in axial compression, end-bearing on rock:

Φ_{stat} = 0.45 from AASHTO LRFD Table 10.5.5.2.3-1

HP 12x53	$R_{tip,f}$ =	217 kips
HP 14x73		314 kips
HP 14x89		385 kips
HP 14x117		512 kips

Nominal Skin Resistance, $R_{s,nom}$:

There is not sufficient soil in some locations to develop significant skinw friction.
Therefore, $R_{s,nom}$ is consdiered to be negligible, and

$R_{s,nom}$ = 0 kips

And, $R_{skin,f} = \Phi_{stat} R_{s,nom}$ = 0 kips

Strength Limit State:

Total Factored Geotechnical Resistance, R_{gf} :

$R_{gf} = R_{tip,f} + R_{skin,f}$

Strength Limit State

Total Factored Geotechnical Tip Resistances, R_{gf} :

HP 12x53	R_{gf} =	217 kips
HP 14x73		314 kips
HP 14x89		385 kips
HP 14x117		512 kips

Service/Extreme Limit State:

Resistance factor, Φ, for Service and Extreme Event Limit States:

Φ = 1.00 Based on AASHTO LRFD (2008) Articles 6.5.5 (page 6-29), 10.5.5.1 (page 10-32), and 10.5.5.3 (page 10-43).

Nominal tip resistance, $R_{t,nom}$, for four typical H-Piles (as above).

HP 12x53	$R_{t,nom}$ =	482 kips
HP 14x73		698 kips
HP 14x89		855 kips
HP 14x117		1,138 kips

Skin friction, $R_{s,nom}$ is consdiered to be negligible.

So, $R_{s,nom}$ = 0 kips

Total Factored Geotechnical Resistance, R_{gf} :

$R_g = (R_{t,nom} + R_{s,nom})\Phi$

Service/Extreme Event Limit State

Total Factored Geotechnical Tip Resistances, R_g :

Pile		Resistance
HP 12x53	R_{gf} =	482 kips
HP 14x73		698 kips
HP 14x89		855 kips
HP 14x117		1,138 kips

3. Drivability Analysis for Axial Resistance of Individual H-Piles

Drivability analysis conducted to establish installation criteria for driven piles, in accordance with AASHTO LRFD (2008) Article 10.7.8 (page 10-121)

For steel piles (e.g., H-Piles) in compression: $\sigma_{dr} = 0.9\Phi_{da}f_y$, where:

σ_{dr} = allowable driving stresses anywhere in the pile.

Φ_{da} = resistance factor = 1.00 Based on AASHTO LRFD (2008) Article 6.5.4.2 (page 6-2) and Table 10.5.5.2.3-1 (page 10-38).

f_y = Yield Strength = 50 ksi

So, σ_{dr} = 45 ksi

Use GRLWEAP to determine the ultimate pile resistance (R_{ult}) for four typical H-Piles, based on the limiting the allowable driving stress to 45 ksi.

The input paraemters for the hammer/driver system are shown below:

```
                Hammer Model:   D 19-42             Made by:      DELMAG

                No.     Weight    Stiffn       CoR      C-Slk      Dampg
                          kips    k/inch                   ft     k/ft/s
                  1      0.800
                  2      0.800  140046.7     1.000     0.0100
                  3      0.800  140046.7     1.000     0.0100
                  4      0.800  140046.7     1.000     0.0100
                  5      0.800  140046.7     1.000     0.0100
          Imp Block      0.753   70735.6     0.900     0.0100
          Helmet         3.200  109975.0     0.800     0.0100        5.8
          Combined Pile Top      12329.5

HAMMER OPTIONS:
  Hammer File ID No.                 41   Hammer Type                OE Diesel
  Stroke Option               FxdP-VarS   Stroke Convergence Crit.      0.010
  Fuel Pump Setting             Maximum

HAMMER DATA:
  Ram Weight          (kips)       4.00   Ram Length          (inch)    129.10
  Maximum Stroke        (ft)      11.86
  Rated Stroke          (ft)      10.81   Efficiency                     0.800

  Maximum Pressure     (psi)    1520.00   Actual Pressure      (psi)   1520.00
  Compression Exponent            1.350   Expansion Exponent             1.250
  Ram Diameter        (inch)      12.60
  Combustion Delay       (s)    0.00200   Ignition Duration      (s)   0.00200

          The Hammer Data Includes Estimated (NON-MEASURED) Quantities


  HAMMER CUSHION                          PILE CUSHION
  Cross Sect. Area     (in2)       0.00   Cross Sect. Area     (in2)      0.00
  Elastic-Modulus      (ksi)        0.0   Elastic-Modulus      (ksi)       0.0
  Thickness           (inch)       0.00   Thickness           (inch)      0.00
  Coeff of Restitution              0.8   Coeff of Restitution             1.0
  RoundOut              (ft)        0.0   RoundOut              (ft)       0.0
  Stiffness        (kips/in)   109975.0   Stiffness        (kips/in)       0.0
```

A copy of the GRLWEAP output for a HP 12x53 pile is shown below:

```
                          SUMMARY OVER DEPTHS

                    G/L at Shaft and Toe:  1.000  1.000
 Depth      Rut   Frictn   End Bg   Bl Ct Com Str Ten Str  Stroke   ENTHRU
    ft     kips     kips     kips   bl/ft     ksi     ksi      ft   kip-ft
   5.0      8.2      3.7      4.5  Hammer did not run
  10.0     23.6     14.6      9.0     1.6   9.591   0.000    3.80     25.0
  10.0     19.0     14.7      4.3     1.4   6.928   0.000    3.58     23.7
  12.5     24.8     20.1      4.7     1.6   9.644   0.000    3.77     24.9
  15.0     31.1     25.9      5.2     1.9  11.655   0.000    3.94     24.4
  15.0     58.4     26.1     32.3     5.0  17.955   0.000    4.90     20.6
  16.5     67.9     33.4     34.5     5.9  18.859  -0.244    5.09     20.1
  18.0     77.8     41.1     36.7     6.8  19.659  -0.261    5.28     19.6
  18.0     72.7     43.7     29.1     6.2  19.169  -0.268    5.16     19.9
  19.0    190.3    161.3     29.1    26.1  28.264  -0.309    7.12     16.6
  20.0    307.9    278.9     29.1    48.6  42.445  -1.894    7.94     16.8
  20.0    475.0    281.3    193.8    97.1  53.607  -5.039    9.14     18.9
  21.0    475.0    281.3    193.8    97.9  53.403  -4.642    9.13     18.8
  22.0    475.0    281.3    193.8   100.0  51.514  -4.818    9.06     18.2
```

As shown in this output, R_{ult} for σ_{dr} = 45 ksi is between 307.9 and 475 kips. Determine the approximate R_{ult} for σ_{dr} = 45 ksi by linear interpolation.

The relevant outputs for the H-pile sections evaluated are summarized below:

Pile Section	R_{ult} (kips)	Blows per foot	Compressive Stress (ksi)
HP 12x53	307.6	49	42.45
	475.0	97	53.61
Interpolated	*345.9*	*60*	*45.00*
HP 14x73	373.6	64	34.29
	603.8	144	45.63
Interpolated	*591.0*	*140*	*45.00*
HP 14x89	668.5	172	40.15
	668.5	173	30.70
Interpolated	*No Interpolation possible*		
HP 14x117	412.3	76	26.42
	780.6	256	36.57
Interpolated	*625.1*	*180*	*32.28*

Note: *Copies of the entire input and output files for all of the piles evaluated are in the URS project file.*

Strength Limit State:

Total Factored Drivign Resistance, $R_{dr,f}$:

Compute resistance that must be achieved in drivability analyis:

The resistance that must be achieved in a drivability analysis will be the maximum applied axial stress multiplied by the appropriate resistance factor for wave equation analysis and the dynamic test that will be required for construction.

Φ_{dyn} = dynamic resistance factor = 0.65

Based on AASHTO LRFD (2008) Table 10.5.5.2.3-1 (page 10-37).

However, AASHTO LRFD (2008) Table 10.5.5.2.3-3 (page 10-39) requires at least 3 or 4 piles to be dynamically tested at a site with low to medium variablity. Since only one (1) pile is typically dynamically tested at each abutment at a small bridge site (one test per abutment is typically specified), reduce the dynamic resistance factor (Φ_{dyn}) by 20 percent.
Therefore, use $\Phi_{dyn,80\%}$ to determine the maximum applied stress in the drivability analysis.

$\Phi_{dyn,80\%}$ = (80%)(0.65) = 0.52

$R_{dr,f} = R_{ult}\Phi_{dyn,80\%}$

From GRLWeap Output:

	q_{ult} =
HP 12x53	346 kips
HP 14x73	591 kips
HP 14x89	669 kips
HP 14x117	625 kips

Factored Driving Resist

Strength Limit State

Total Factored Driving Resistances, $R_{dr,f}$:

	$R_{dr,f}$ =
HP 12x53	180 kips
HP 14x73	307 kips
HP 14x89	348 kips
HP 14x117	325 kips

Note: The factored driving resistance is less than the factored compressive (structural) resistance and the factored geotechnical resistance.
Therefore, the driving resistance governs design for the Strength Limit State.

Service/Extreme Limit State:

Resistance factor, Φ, for Service and Extreme Event Limit States:

Φ = 1.00 Based on AASHTO LRFD (2008) Articles 6.5.5 (page 6-29), 10.5.5.1 (page 10-32), and 10.5.5.3 (page 10-43).

$R_{dr,fx} = R_{ult}\Phi$

Factored Driving Resistance for Service and Extreme Event Limits, $R_{dr,fx}$:

Service/Extreme Event Limit State

Factored Driving Resistances, $R_{dr,fx}$:

	$R_{dr,fx}$ =
HP 12x53	346 kips
HP 14x73	591 kips
HP 14x89	669 kips
HP 14x117	625 kips

Note: The factored driving resistance is less than the factored compressive (structural) resistance and the factored geotechnical resistance.
Therefore, driving resistance governs design for the Service/Extreme Event Limit States.

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.
3 Tomlinson, M. J. 1994. *Pile design and construction practice* , 4th Edition, Taylor & Francis, Inc. New York, NY, 411 pages

Bearing Resistance - Footings on Weathered 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 4 feet of weathered bedrock (rounded to subangular gravel, some fine sand) overlying highly fractured PHYLLITE, RQD = 36%

Boring URS-B2: Approximately 1 foot of weathered bedrock overlying highly fractured PHYLLITE, RQD = 90%

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 $\Phi_{ns} = 36^o$:

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.

Footing width, B = $\left\{ \begin{array}{ll} 4 & \text{feet} \\ 6 & \text{feet} \\ 8 & \text{feet} \\ 10 & \text{feet} \\ 12 & \text{feet} \end{array} \right.$

Therefore,

q_{nom} = $\left\{ \begin{array}{ll} 15 & \text{ksf} \\ 17 & \text{ksf} \\ 20 & \text{ksf} \\ 22 & \text{ksf} \\ 25 & \text{ksf} \end{array} \right.$

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

For footings on rock, Φ_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.