MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

WILD RIVER BRIDGE
OVER WILD RIVER
STATE ROUTE 2
GILEAD, MAINE

Prepared by:

Kathleen Maguire, P.E.
Geotechnical Engineer

STATE OF MAINE
Kathleen Maguire
7120
LICENSED
PROFESSIONAL ENGINEER

Reviewed by:

Laura Krusinski, P.E.
Senior Geotechnical Engineer

Oxford County
PIN 15619.00

Soils Report No. 2008-15
Bridge No. 2948

Fed No. BH-1561(900)E
December 1, 2008

Table of Contents

GEOTECHNICAL DESIGN SUMMARY

The purpose of this report is to make geotechnical recommendations for the replacement of the Wild River Bridge on a new alignment for State Route 2 over Wild River in Gilead, Maine. The proposed replacement bridge will consist of a 79 meter (260 foot) long, two span welded plate girder superstructure with integral abutments supported on driven H-piles and a center pile supported pier with a full height curtain wall. The following design recommendations are discussed in detail in the attached report:

Integral Abutment H-piles - Stub abutments founded on driven integral end bearing H-piles may be HP 310x79 (HP 12x53), HP 360x108 (HP 14x73), HP 360x132 (HP 14x89), or HP 360x174 (HP 14x117). Piles should be 345 MPa (50 ksi), Grade A572 steel H-piles. Piles should be fitted with driving points. The designer shall design the H-piles at the strength limit state considering the structural resistance of the piles, the geotechnical resistance of the pile and loss of the lateral support due to scour at the design flood event. The structural resistance check should include checking axial, lateral, and flexural resistance. The design of the H-piles at the service limit state shall consider tolerable horizontal movement of the piles, overall stability of the pile group and scour at the design flow event. Since the abutment piles will be subjected to lateral loading, piles should be analyzed for axial loading and combined axial and flexure. The Contractor is required to perform a wave equation analysis of the proposed pile-hammer system and a dynamic pile test at each abutment. The first pile driven at the each abutment should be dynamically tested to confirm capacity and verify the stopping criteria developed by the Contractor in the wave equation analysis. The ultimate pile resistance that must be achieved in the wave equation analysis and dynamic testing will be the factored axial pile load divided by a resistance factor of 0.65. The factored pile load should be shown on the plans.

Stub Abutments and Wingwalls - Integral stub abutments and wingwalls shall be designed for all relevant strength, service and extreme limit states and load combinations. The design of pile supported abutments and wingwalls at the strength limit state shall consider pile stability and structural resistance. Strength limit state design shall also consider foundation resistance after scour due to the design flood. Abutment design at the service limit state shall include: settlement, horizontal movement, overall stability and scour at the design flood. Extreme limit and strength limit state design checks for abutments shall include pile structural resistance, pile geotechnical resistance, pile resistance in combined axial and flexure, and overall stability. In designing for passive earth pressure associated with integral abutments, the Rankine state is recommended. All abutment designs shall include a drainage system to intercept any water. To avoid water intrusion behind the abutment, the approach slab should connect directly to the abutment.

Pile Supported Pier with Curtain Wall – A pile supported pier with a full height curtain wall was selected for intermediate structure support. Piles for the pier may consist of end-bearing concrete filled pipe piles driven to bedrock or end-bearing H-piles driven to bedrock. The designer shall design the piles at the strength limit state considering the structural, geotechnical and drivability resistance of the pile. The structural resistance check should include checking axial, lateral, and flexural resistance. The design of the piles at the service limit state shall consider tolerable horizontal movement of the piles and overall stability of the pile group. Since the pier piles will be subjected to lateral loading and have a substantial unbraced length, piles should be analyzed for axial loading and combined axial and lateral loading. The Contractor is required to perform a wave equation analysis of the proposed pile-hammer system and a dynamic pile test at the pier. The first pile driven at the pier should be dynamically tested to confirm capacity and verify the stopping criteria developed by the Contractor in the wave equation analysis. The ultimate pile resistance that must be achieved in the wave equation analysis and dynamic testing will be the factored axial pile load divided by a resistance factor of 0.65. The factored pile load should be shown on the plans.

Scour and Riprap- The consequences of changes in foundation conditions resulting from the design flood for scour shall be considered at the strength and service limit states. These changes in foundation conditions shall be investigated at the abutments, wingwalls and pier. For scour protection, any footings which are constructed on granular deposits, should be embedded a minimum of 0.9 meters (3 feet) below the design scour depth and at least 0.6 meters (2.0 feet) below the super flood scour event and armored with 0.9 meters (3 feet) of riprap. Riprap, 0.9 meters (3 feet) thick, conforming to item number 703.26 of the Standard Specification, shall be placed at the toes of abutments and wingwalls. The riprap shall extend 0.5 meters (1.5 feet) horizontally in front of the wall before sloping at a maximum 1.75H:1V slope to the existing ground surface. The toe of the riprap section shall be constructed 0.3 meters (1 foot) below the streambed elevation. The riprap section shall be underlain by a 0.3 meters (1 foot) thick layer of bedding material.

Settlement - Large amounts of fill will be place behind both abutments in order to raise the existing grade to accommodate the new roadway approaches to the bridge. Settlements due to the addition of this fill have been calculated to range between 20 and 40 mm (1 and 2 inches). Due to the granular nature of the subsurface soils present at the site all settlement associated with this fill occur will during construction having negligible effect on the finished bridge structure. Any settlement of the bridge abutments and pier will be due to the elastic compression of the piling and will be negligible.

Frost Protection - Any foundation placed on granular soils should be founded a minimum of 2 meters (6.5 feet) below finished exterior grade for frost protection. Integral abutments shall be embedded a minimum of 1.2 meters (4.0 feet) for frost protection.

Seismic Design Considerations - The Wild River Bridge on Route 2 is on the National Highway System (NHS) and is therefore considered to be functionally important. Consequently, a detailed seismic analysis is required. The minimum seismic analysis requirements are defined in LFRD Article 4.7.4.1. The designer shall determine the specific analysis method using LRFD Tables 4.7.4.3.1-1. Seismic design requirements for Seismic Zone 1 are found in LRFD Article 3.10.9.2.

Construction Considerations - There is potential for boulders and cobbles to impact the pile installation operations. These impacts include, but are not limited to, driving the piles and cleaning out pipe piles. Obstruction may be cleared by conventional excavation methods, pre-augering, pre-drilling, or down-hole hammers or as approved by the Resident.

1.0 INTRODUCTION

A subsurface investigation for the replacement of the Wild River Bridge on a new alignment for State Route 2 over Wild River in Gilead, Oxford County, Maine has been completed. The purpose of the investigation was to explore subsurface conditions at the site in order to develop geotechnical recommendations for the bridge replacement. This report presents the soils information obtained at the site, geotechnical design recommendations, and foundation recommendations.

The existing bridge Wild River Bridge was constructed in 1928 and consists of a 68.8 meter (216 foot) long four-span, concrete T-beam structure supported on mass concrete piers and concrete abutments on spread footings. The bridge was widened in 1953. The abutments and wingwalls have moderate to severe cracking and spalling. The piers show significant signs of deterioration and are susceptible to scour. Maine Department of Transportation (MaineDOT) maintenance inspection reports indicate that the bridge superstructure is in “fair” condition while the deck and substructures are in “poor” condition. Year 2007 MaineDOT Bridge Maintenance inspection reports indicate a Bridge Sufficiency Rating of 36.1.

The only option for this bridge replacement is a new bridge built on the new alignment of State Route 2. The new bridge will be a 79 meter (260 foot), two span, welded plate girder superstructure on integral abutments supported on driven H-piles and a center pile supported pier. The pier will have a full height curtain wall from the underside of the deck to just below the river mud line. The curtain wall will help to minimize deterioration of the pier piles during high water events.

2.0 GEOLOGIC SETTING

The Wild River Bridge on State Route 2 in Gilead crosses the Wild River approximately 0.16 km (0.1 miles) west of the intersection of State Routes 2 and 113 as shown on Sheet 1 - Location Map found at the end of this report. The Wild River flows in a northerly direction to the Androscoggin River.

According to the Surficial Geologic Map of Maine published by the Maine Geological Survey (1985) the surficial soils in the vicinity of the site consist of ice-contact glaciofluvial deposits. Soils in the site area are generally comprised of sand, gravel, and silt. The unit generally is deposited in areas where the topography is flat-topped kame terraces and deltas which are locally kettled and bounded by steep sides or hummocky terrain with numerous kames and kettles. These soils were generally deposited by meltwater streams adjacent to stagnant glacial ice. Additional geologic units mapped nearby the site are till deposits which are generally sand, silt, clay and stones, thin drift which is generally thin surficial deposits over bedrock and exposed bedrock.

According to the Surficial Bedrock Map of Maine, published by the Maine Geological Survey (1985), the bedrock at the site is identified as interbedded petite and sandstone. This rock is identified as the Littleton Formation.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions were explored by drilling three (3) test borings at the site. Test borings BB-GWR-101 was drilled at the location of Abutment No. 1 (west). Test boring BB-GWR-102 was drilled at the center pier location. Test boring BB-GWR-103 was drilled at the location of Abutment No. 2 (east). The exploration locations are shown on Sheet 2 - Boring Location Plan found at the end of this report. An interpretive subsurface interpretive subsurface profile depicting the site stratigraphy is shown on Sheet 3 - Interpretive Subsurface Profile found at the end of this report. Borings BB-GWR-101 and BB-GWR-103 were drilled between March 20 and April 8, 2008 by Northern Test Boring of Gorham, Maine. Boring BB-GWR-102 was drilled between September 15 and 19, 2008 also by Northern Test Boring of Gorham, Maine. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A - Boring Logs and on Sheets 4 and 5 - Boring Logs found end of this report.

The borings were drilled using driven cased wash boring and solid stem auger techniques. Soil samples were obtained where possible at 1.5 meter (5-foot) intervals using Standard Penetration Test (SPT) methods. During SPT sampling, the sampler is driven 60 cm (24 inches) and the hammer blows for each 15 cm (6 inch) interval of penetration are recorded. The standard penetration resistance, N-value, is the sum of the blows for the second and third intervals. The Northern Test Boring drill rig is equipped with an automatic hammer to drive the split spoon. The hammer was calibrated in February of 2008 and was found to deliver approximately 6 percent more energy during driving than the standard rope and cathead system. All N-values discussed in this report are corrected values computed by applying an average energy transfer factor of 0.633 to the raw field N-values. This hammer efficiency factor (0.633) and both the raw field N-value and the corrected N-value are shown on the boring logs.

The bedrock was cored in the borings using an NQ core barrel and the Rock Quality Designation (RQD) of the core was calculated. The MaineDOT geotechnical team member selected the boring locations and drilling methods, designated type and depth of sampling techniques and identified field and laboratory testing requirements. The MaineDOT Certified Subsurface Inspector logged the subsurface conditions encountered. The borings were located in the field by the survey crew prior to drilling.

4.0 LABORATORY TESTING

Laboratory testing for samples obtained in the borings consisted of 29 standard grain size analyses. The results of these laboratory tests are provided in Appendix B - Laboratory Data at the end of this report. Moisture content information and other soil test results are included on the Boring Logs in Appendix A and on Sheets 4 and 5 - Boring Logs found at the end of this report.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered at the test borings generally consisted of interbedded sands and gravels underlain by metamorphic gneiss. An interpretive subsurface profile depicting the site stratigraphy is show on Sheet 3 - Interpretive Subsurface Profile found at the end of this report. The following paragraphs discuss the subsurface conditions encountered in detail:

Interbedded Sand and Gravel. Interbedded layers of sand and gravel were encountered in all of the borings. The layers vary in their grain size content and are comprised of:

- Sandy GRAVEL
- SAND
- SAND with cobbles
- Gravelly SAND

<u>Sandy GRAVEL</u>: Several layers of sandy gravel were encountered in all of the borings. The layers ranged from approximately 0.97 meters (3.2 feet) to approximately 4.88 meters (16.0 feet) thick. The soil generally consisted of brown, damp to wet, fine to coarse sandy gravel with trace silt and occasional cobbles. Corrected SPT N-values in the sandy gravel ranged from 24 to >50 blows per foot (bpf) indicating that the soil is medium dense to very dense in consistency. Water contents from seven (7) samples obtained within the sandy gravel layers range from approximately 3% to 11%. Seven (7) grain size analyses conducted on samples from these layers indicate that the soil is classified as an A-1-a by the AASHTO Classification System and a GW-GM, GW, or GP-GM by the Unified Soil Classification System.

<u>SAND:</u> Several layers of sand were encountered in all of the borings. The layers ranged from approximately 0.9 meters (3.0 feet) to approximately 5.79 meters (19.0 feet) thick. The soil generally consisted of brown, damp to wet, fine sand, fine to medium sand, fine to coarse sand with trace to some gravel, and trace to some silt. The sand layer in the upper portion of boring BB-GWR-101 had trace organics. Corrected SPT N-values in the sand layers ranged from 3 to 55 bpf indicating that the soil is very loose to very dense in consistency. Water contents from fourteen (14) samples obtained within the sand layers range from approximately 11% to 26%. Fourteen (14) grain size analyses conducted on samples from the sand layers indicate that the soil is classified as an A-3, A-2-4, or A-1-b by the AASHTO Classification System and a SP-SM, SM, SP or SW-SM by the Unified Soil Classification System.

<u>SAND with cobbles:</u> Several layers of sand with cobbles were encountered in the borings. The layers ranged from approximately 0.4 meters (1.3 feet) to approximately 7.5 meters (25.0 feet) thick. The soil generally consisted of brown, wet, fine to coarse sand, little to some gravel and trace to little silt with occasional cobbles. The layer in the upper portion of boring BB-GWR-103 had trace organics. Corrected SPT N-values in the layers ranged from 9 to 78 bpf indicating that the soil is loose to very dense in consistency. Water contents from two (2) samples obtained within the layers range from approximately 9% to 16%. Two (2) grain size analyses conducted on samples from these layers indicate that the soil is classified

as an A-1-b by the AASHTO Classification System and a SM or SW by the Unified Soil Classification System.

Gravelly SAND: A layer of gravelly sand was encountered at the bottom of boring BB-GWR-103. The layer was approximately 0.37 meters (1.2 feet) thick. The gravelly sand generally consisted of brown, wet, fine to coarse gravelly sand with trace silt. One corrected SPT N-values in the gravelly sand was 30 bpf indicating that the soil is medium dense in consistency. One (1) water content from a sample of the gravelly sand was approximately 11%. One (1) grain size analysis conducted on a sample from this layer indicates that the soil is classified as an A-1-b by the AASHTO Classification System and a SW-SM by the Unified Soil Classification System.

Bedrock. Bedrock was encountered and cored in all of the borings. The following table summarizes the depths to bedrock and corresponding elevations of the top of bedrock:

Boring Number/ Location	Depth to Bedrock	Bedrock Elevation	RQD
BB-GWR-101/ Abutment No. 1	12.8 meters (42.0 feet)	197.95 meters (649.44 feet)	85 – 93%
BB-GWR-102/ Center Pier	25.5 meters (83.7 feet)	182.88 meters (600.0 feet)	95%
BB-GWR-103/ Abutment No. 2	26.82 meters (88.0 feet)	182.23 meters (597.87 feet)	65 – 87%

The bedrock is identified as black, grey and white metamorphic GNEISS with 70 degree banding. The rock quality designation (RQD) of the bedrock was determined to range from 65 to 95 percent indicating a rock mass quality of fair to excellent quality.

6.0 FOUNDATION ALTERNATIVES

Based on the subsurface conditions encountered during the subsurface exploration program, the following foundation alternatives, with varying levels of risk and durability, may be considered for the bridge replacement:

- Cast-in-place concrete or precast concrete integral abutments supported on driven steel H-piles
- Pile bent pier
- Pile supported pier with curtain wall with pipe piles or H-piles

Due to the high scour susceptibility of the Wild River, the use of spread footings is not a viable foundation alternative for the site. The use of drilled shafts, although a viable foundation type for the site, would not be an economical alternative.

The Preliminary Design Report (PDR) for this project recommends that the replacement bridge be supported on H-pile supported integral abutments and a pile supported pier with a curtain wall. This report addresses only those foundation types.

7.0 FOUNDATION CONSIDERATIONS AND RECOMMENDATIONS

The following sections will discuss geotechnical design recommendations for cast-in-place concrete or precast concrete integral abutments supported on driven steel H-piles and a center pile supported pier with a curtain wall on pipe piles or H-piles.

7.1 Integral Abutment H-piles

The use of stub abutments founded on a single row of driven integral H-piles is a viable foundation system for use at the site. The piles should be end bearing, driven to the required resistance on or within the bedrock. Piles may be HP 310x79 (HP 12x53), HP 360x108 (HP 14x73), HP 360x132 (HP 14x89), or HP 360x174 (HP 14x117) depending on the design axial loads. Piles should be 345 MPa (50 ksi), Grade A572 steel H-piles. Piles should be fitted with driving points to protect the tips and improve penetration.

Pile lengths at the proposed abutments may be estimated based on the table below:

Location	Estimated Pile Cap Bottom Elevation	Depth to Bedrock From Ground Surface	Top of Rock Elevation	Estimated Pile Length
Abutment No.1 BB-GWR-101	214.5 meters (703.74 feet)	12.8 meters (42.0 feet)	197.95 meters (649.44 feet)	17 meters (56 feet)
Abutment No.2 BB-GWR-103	215.3 meters (706.36 feet)	26.82 meters (88.0 feet)	181.31 meters (594.85 feet)	34 meters (112 feet)

These pile lengths do not take into account the additional 1.5 meters (5 feet) of pile required for dynamic testing instrumentation or any additional pile length needed to accommodate the Contractor's leads and driving equipment.

The designer shall design the H-piles at the strength limit state considering the structural resistance of the piles, the geotechnical resistance of the pile and loss of the lateral support due to scour at the design flood event. The structural resistance check should include checking 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 the H-piles at the service limit state shall consider tolerable horizontal movement of the piles, overall stability of the pile group and scour at the design flow event. The design flood scour is defined in AASHTO LRFD Bridge Design Specifications 4th Edition (LRFD) Articles 2.6.4.4.2 and 3.7.5. Since the abutment piles will be subjected to lateral loading, piles should be analyzed for axial loading and combined axial and flexure as defined in LRFD Article 6.15.2 and specified in LRFD Article 6.9.2.2.

7.1.1 Strength Limit State

The nominal structural compressive resistance (P_n) in the strength limit state for piles loaded in compression shall be as specified in LRFD Article 6.9.4.1. The H-piles are assumed fully embedded and λ shall be taken as 0. It is the responsibility of the structural engineer to recalculate the column slenderness factor (λ) for the upper and lower portions of the H-pile based on unbraced lengths and K-values from project specific L-Pile® analyses and determine structural pile resistances. The factored structural axial compressive resistances of the four proposed H-pile sections were calculated using a resistance factor, ϕ_c, of 0.50 (severe driving conditions) and a λ of 0.

The nominal geotechnical compressive resistance in the strength limit state was calculated using Canadian Foundation Engineering Manual methods. The factored geotechnical compressive resistances of the four proposed H-pile sections were calculated using a resistance factor, ϕ_{stat}, of 0.45.

The drivability of the four proposed H-pile sections was considered. The maximum driving stresses in the pile, assuming the use of 345 MPa (50 ksi) steel, shall be less than 310 MPa (45 ksi). As the piles will be driven to refusal on bedrock a drivability analysis to determine the resistance that must be achieved was conducted. The resistance factor for a single pile in axial compression when a dynamic test is done, given in LRFD Table 10.5.5.2.3-1, is ϕ_{dyn}= 0.65.

The calculated factored axial compressive structural, geotechnical and drivability resistances of the four proposed H-pile sections for the abutments are summarized in the table below. Supporting calculations are included in Appendix C- Calculations found at the end of this report.

Factored Axial Resistances for Abutment Piles at the Strength Limit State

Pile Section	Factored Resistance			
	Structural Resistance*	Geotechnical Resistance	Drivability Resistance	Design Resistance
HP 310 x 79 (HP 12 x 53)	1724 kN (388 kips)	1311 kN (295 kips)	1359 kN (306 kips)	1311 kN (295 kips)
HP 360 x 108 (HP 14 x 73)	2380 kN (535 kips)	1653 kN (372 kips)	1749 kN (393 kips)	1653 kN (372 kips)
HP 360 x 132 (HP 14 x 89)	2902 kN (653 kips)	2009 kN (452 kips)	1966 kN (442 kips)	2009 kN (452 kips)
HP 360 x 174 (HP 14 x 117)	3825 kN (860 kips)	2632 kN (592 kips)	2414 kN (543 kips)	2632 kN (592 kips)

* based on preliminary assumption of λ=0 for the lower portion of the pile in only axial compression (no flexure)

LRFD Article 10.7.8 states that for routine pile installation applications where significant local experience can be applied to keep the risk of pile installation problems low, a project specific drivability analysis using the wave equation may be waived. In light of this, it is recommended that the governing resistance used in design be the factored geotechnical resistance indicated in the table above.

Per LRFD Article 6.5.4.2, at the strength limit state, for H-piles in compression and bending, the axial resistance factor ϕ_c=0.7 and the flexural resistance factor ϕ_f =1.0 shall be applied to the combined axial and flexural resistance of the pile in the interaction equation (LFRD Eq. 6.12.2.2.1-1 or -2). The combined axial compression and flexure should be evaluated in accordance with the applicable sections of LRFD Articles 6.9.2.2 and 6.12.2.

7.1.2 Service and Extreme Limit States

For the service and extreme limit states resistance factors, ϕ, of 1.0 are recommended for structural and geotechnical pile resistances. For preliminary analysis, the H-piles can be assumed fully embedded and λ can be taken as 0. It is the responsibility of the structural engineer to recalculate the column slenderness factor (λ) for the upper and lower portions of the H-pile based on unbraced lengths and K-values from project specific L-Pile® analyses and determine structural pile resistances.

The calculated factored axial structural, geotechnical and drivability resistances of the four proposed H-pile sections for each abutment are summarized in the table below. Supporting calculations are included in Appendix C- Calculations found at the end of this report.

Factored Axial Resistances for Abutment Piles at the Service and Extreme Limit States

Pile Section	Factored Resistance			
	Structural Resistance*	Geotechnical Resistance	Drivability Resistance	Design Resistance
HP 310 x 79 (HP 12 x 53)	3447 kN (775 kips)	2913 kN (655 kips)	2091 kN (470 kips)	2913 kN (655 kips)
HP 360 x 108 (HP 14 x 73)	4760 kN (1070 kips)	3672 kN (826 kips)	2691 kN (605 kips)	3672 kN (826 kips)
HP 360 x 132 (HP 14 x 89)	5805 kN (1305 kips)	4464 kN (1003 kips)	3025 kN (680 kips)	4464 kN (1003 kips)
HP 360 x 174 (HP 14 x 117)	7651 kN (1720 kips)	5849 kN (1315 kips)	3714 kN (835 kips)	5849 kN (1315 kips)

*based on preliminary assumption of λ=0 for the lower portion of the pile in only axial compression (no flexure)

Although the factored axial drivability resistance is less than both the factored axial structural and geotechnical resistances, LRFD Article 10.7.8 states that for routine pile installation applications where significant local experience can be applied to keep the risk of pile installation problems low, a project specific drivability analysis using the wave equation may be waived. In light of this, it is recommended that the governing resistance used in design be the factored geotechnical resistance in the table above.

7.1.3 Pile Resistance and Pile Quality Control

Based on the anticipated depth to bedrock at the site, pile splices will be required. The location and number of pile splices shall be in conformance with MaineDOT Standard Specification 501 and be subject to the approval of the Resident. The splices shall be the Champion HP-30000, or approved equivalent, mechanical splicer. Evaluation of equivalent products will be based on the submission of data demonstrating the capability of transferring the full pile strength in compression and tension and developing the bending moment capacity of the pile in both the x-x and y-y axes. The splicers shall be installed and welded as recommended by the manufacturer. Welding shall not be done when the temperature in the immediate vicinity of the weld is below 18ºC (0°F); when the surfaces are damp or exposed to rain, snow, or high wind; or when the welders or welding operators are exposed to inclement conditions. The pile shall be preheated to and maintained at 66ºC (150°F) minimum within 15 cm (6 inches) from the weld during welding. Formal welding procedures are not required. Welders shall be prequalified in accordance with Section 504 - Structural Steel.

The Contractor is required to perform a wave equation analysis of the proposed pile-hammer system and a dynamic pile test at each abutment. The first pile driven at each abutment should be dynamically tested to confirm capacity and verify the stopping criteria developed by the Contractor in the wave equation analysis. The ultimate pile resistance that must be achieved in the wave equation analysis and dynamic testing will be the factored axial pile load divided by a resistance factor of 0.65. The factored pile load should be shown on the plans.

Piles should be driven to an acceptable penetration resistance as determined by the Contractor based on the results of a wave equation analysis and as approved by the Resident. Driving stresses in the pile determined in the drivability analysis shall be less than 310 MPa (45 ksi) in accordance with LRFD Article 10.7.8. A hammer should be selected which provides the required resistance when the penetration resistance for the final 76 mm to 152 mm (3 to 6 inches) is 8 to 15 blows per 25 mm (1 inch). If an abrupt increase in driving resistance is encountered, the driving could be terminated when the penetration is less than 12 mm (0.5-inch) in 10 consecutive blows.

7.2 Stub Abutments and Wingwalls

Integral stub abutments and wingwalls shall be designed for all relevant strength, service and extreme limit states and load combinations specified in LRFD Articles 3.4.1 and 11.5.5. The design of pile supported abutments and wingwalls at the strength limit state shall consider pile stability and structural failure. Strength limit state design shall also consider foundation resistance after scour due to the design flood.

A resistance factor of ϕ= 1.0 shall be used to assess abutment design at the service limit state including: settlement, horizontal movement, overall stability and scour at the design flood. Extreme limit and strength limit state design checks for abutments shall include pile structural resistance, pile geotechnical resistance, pile resistance in combined axial and flexure, and overall stability. A resistance factor of ϕ=1.0 shall be used for the extreme limit state.

Conventional wingwalls shall be designed as unrestrained meaning that they are free to rotate at the top in an active state of earth pressure. Earth loads shall be calculated using an active earth pressure coefficient, K_a, calculated using Rankine Theory for cantilever wingwalls and Coulomb Theory for gravity shaped structures. See Sheet 6 - Rankine and Coulomb Active Earth Pressure Coefficients at the end of this report for guidance in calculating these values. Additional lateral earth pressure due to construction surcharge or live load surcharge is required per section 3.6.8 of the MaineDOT Bridge Design Guide (BDG) for the abutments and walls if an approach slab is not specified. Use of an approach slab may be required per the MaineDOT BDG Sections 5.4.2.10 and 5.4.4. When a structural approach slab is specified, reduction, not elimination, of the surcharge loads is permitted per LRFD Article 3.11.6.2. 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 0.6 meters (2.0 feet) per LRFD Table 3.11.6.4-2. The live load surcharge on abutments may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (H_{eq}) taken form the table below:

Abutment Height	H_{eq}
1.5 meters (5 feet)	1.2 meters (4.0 feet)
3.0 meters (10 feet)	0.9 meters (3.0 feet)
≥6 meters (≥20 feet)	0.6 meters (2.0 feet)

The Designer may assume Soil Type 4 (MaineDOT BDG Section 3.6.1) for backfill material soil properties. The backfill properties are as follows: ϕ = 32 degrees, γ = 19.6 kN/m^3 (125 pcf). Sliding computations for resistance to lateral loads shall assume a maximum allowable frictional coefficient of 0.45 at the soil-concrete interface. A sliding resistance factor of ϕ_τ=0.8 shall be applied to the nominal sliding resistance of walls found on spread footings on sand.

Integral abutments and wingwall sections that are integral with the abutment should be designed to withstand a passive earth pressure state. In designing for passive earth pressure associated with integral abutments, the Coulomb state is recommended. Experience in designing wingwalls for integral abutments has shown that the use of the Coulomb passive earth pressure K_p=6.89 may result in uneconomical wall sections. For this reason, consideration may be given to using a Rankine passive earth pressure, K_p=3.25 when designing integral abutments and integral wingwall extensions.

All abutment designs shall include a drainage system behind the abutments to intercept any water. Drainage behind the structure shall be in accordance with Section 5.4.1.4 Drainage, of the MaineDOT BDG. Geocomposite drainage board applied to the backsides of the abutments and wingwalls with weep holes will provide adequate drainage. To avoid water intrusion behind the abutment, the approach slab should connect directly to the abutment.

Backfill within 3 meters (10 feet) of the abutments and wingwalls and side slope fill shall conform to Granular Borrow for Underwater Backfill - MaineDOT Specification 709.19. This gradation specifies 10 percent or less of the material passing the No. 200 sieve. This material is specified in order to reduce the amount of fines and to minimize frost action behind the structure.

7.3 Pile Supported Pier with Curtain Wall

A pile supported pier with a full height curtain wall was selected for intermediate support. Piles for the pier may consist of concrete filled pipe piles driven to bedrock or H-piles driven to bedrock.

Pipe piles with diameters ranging from 609 to 762 mm (24 to 30 inches) and wall thicknesses of 13 to 16 mm (½ to 5/8 inch) are recommended. Pipe piles should be fabricated in accordance with ASTM A252, Grade 3, with a minimum yield strength of 310 MPa (45 ksi). Open ended piles should be equipped with a cutting shoe, constructed from Grade ASTM A148 90/60 steel, and driven open ended. Pier piles should be end bearing and driven to the required resistance on or within the bedrock.

H-piles may be HP 310x79 (HP 12x53), HP 360x108 (HP 14x73), HP 360x132 (HP 14x89), or HP 360x174 (HP 14x117) depending on the design axial loads and design scour depth. H-piles should be 345 MPa (50 ksi), Grade A572 steel. Piles should be fitted with driving points to protect the tips, improve penetration and improve friction at the pile tip to support a pinned pile tip assumption. Pier piles should be end bearing and driven to the required resistance on or within the bedrock.

A full height curtain wall will be constructed from the under side of the bridge deck to just below the river mud line to protect the piles from large stones transported by the water.

Pile lengths at the proposed pier may be estimated based on the table below:

Location	Estimated Curtain Wall Bottom Elevation	Depth to Bedrock From Ground Surface	Top of Rock Elevation	Estimated Pile Length
Center Pier BB-GWR-102	208.0 meters (682.41 feet)	25.5 meters (83.7 feet)	182.88 meters (600.0 feet)	26 meters (85 feet)

This pile length does not take into account the additional 1.5 to 2.4 meters (5 to 8 feet) of pile required for dynamic testing instrumentation or any additional pile length needed to accommodate the Contractor’s leads and driving equipment.

The designer shall design the piles at the strength limit state considering the structural, geotechnical and drivability resistance of the pile. The structural resistance check should include checking 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 the piles at the service limit state shall consider tolerable horizontal movement of the piles and overall stability of the pile group. Since the pier piles will be subjected to lateral loading and have a substantial unbraced length, piles should be analyzed for axial loading and combined axial and lateral loading as defined in LRFD Article 6.15.2.

7.3.1 Strength Limit State

The nominal compressive structural resistance (P_n) for piles in the strength limit state loaded in compression shall be as specified in LRFD Article 6.9.4.1 for non-composite members (H-pile) and Article 6.9.5.1 for composite members (pipe pile). The piles have an unbraced length and require calculation of the λ-factor as specified in LRFD Article 6.9.

For the strength limit state, the factored axial compressive structural resistance of the pile (P_r) shall be calculated using the resistance factors (ϕ_c) of 0.6 for pipe pile in severe driving conditions and 0.5 for H-pile in severe driving conditions as specified in LRFD Article 6.5.4.2. The proposed pier pipe piles will have an unbraced pile length ranging from 7.6 to 8.2 meters (25 to 27 feet). The proposed pier H-piles will have an unbraced pile length ranging from 6.4 to 7.0 meters (21 to 23 feet).

Per LRFD Article 6.5.4.2, at the strength limit state, for piles in compression and bending, the axial resistance factor ϕ_c=0.8 and the flexural resistance factor ϕ_f =1.0 shall be applied to the combined nominal axial and flexural resistance of the pile in the interaction equation, (LRFD Eq. 6.9.2.2-1 or -2) with flexural resistance determined as specified in LRFD 6.12. The factored structural resistance for pile sections in combined axial compression and flexure are not provided in this report as these analyses are considered part of the structural design and the responsibility of the structural designer.

The nominal geotechnical compressive resistance in the strength limit state was calculated using Canadian Foundation Engineering Manual methods. The factored geotechnical compressive resistances of the eight (8) proposed pipe pile sections and four (4) proposed H-pile sections were calculated using a resistance factor, ϕ_{stat}, of 0.45 for end bearing piles on bedrock.

The drivability of the eight (8) proposed pipe pile sections and four (4) proposed H-pile sections was considered. The maximum driving stresses in the pipe pile, assuming the use of 310 MPa (45 ksi) steel, shall be less than 275 MPa (40 ksi). The maximum driving stresses in the H-pile, assuming the use of 345 MPa (50 ksi) steel, shall be less than 310 MPa (45 ksi). As the piles will be driven to refusal on bedrock a drivability analysis to determine the resistance that could potentially be achieved was conducted. The resistance factor for a single pile in axial compression when a dynamic test is done given in LRFD Table 10.5.5.2.3-1 is ϕ_{dyn}= 0.65.

Factored axial compressive structural, geotechnical and drivability resistances for eight (8) pipe pile sections and four (4) proposed H-pile sections are summarized in the table below. Supporting calculations are included in Appendix C- Calculations found at the end of this report.

Factored Axial Resistances for Pipe Piles and H-Piles at the Strength Limit State

Pipe Pile		Factored Resistance			
Diameter	Wall thickness	Structural Resistance	Geotechnical Resistance	Drivability Resistance	Governing Resistance
609 mm (24-in)	13 mm (1/2–in)	3031 kN (681 kips)	1756 kN (395 kips)	1691 kN (380 kips)	1756 kN (395 kips)
660 mm (26-in)	13 mm (1/2–in)	3321 kN (746 kips)	1857 kN (417 kips)	1822 kN (409 kips)	1857 kN (417 kips)
711 mm (28-in)	13 mm (1/2–in)	3609 kN (811 kips)	1957 kN (440 kips)	1986 kN (447 kips)	1957 kN (440 kips)
762 mm (30-in)	13 mm (1/2–in)	3897 kN (876 kips)	2057 kN (463 kips)	2145 kN (482 kips)	2057 kN (463 kips)
609 mm (24-in)	16 mm (5/8–in)	4007 kN (901 kips)	2329 kN (524 kips)	2299 kN (517 kips)	2329 kN (524 kips)
660 m (26-in)	16 mm (5/8–in)	4394 kN (988 kips)	2463 kN (554 kips)	2544 kN (572 kips)	2463 kN (554 kips)
711 mm (28-in)	16 mm (5/8–in)	4780 kN (1074 kips)	2598 kN (584 kips)	2776 kN (624 kips)	2598 kN (584 kips)
762 mm (30-in)	16 mm (5/8–in)	5164 kN (1161 kips)	2732 kN (614 kips)	3050 kN (686 kips)	2732 kN (614 kips)
H-pile Section		Structural Resistance	Geotechnical Resistance	Drivability Resistance	Governing Resistance
HP 310 x 79 (HP 12 x 53)		1433 kN (322 kips)	1311 kN (295 kips)	1301 kN (293 kips)	1311 kN (295 kips)
HP 360 x 108 (HP 14 x 73)		2049 kN (461 kips)	1653 kN (372 kips)	1613 kN (363 kips)	1653 kN (372 kips)
HP 360 x 132 (HP 14 x 89)		2493 kN (560 kips)	2009 kN (452 kips)	1778 kN (400 kips)	2009 kN (452 kips)
HP 360 x 174 (HP 14 x 117)		3275 kN (736 kips)	2632 kN (592 kips)	2148 kN (483 kips)	2632 kN (592 kips)

Although the factored axial drivability resistance is less than both the factored axial structural and geotechnical resistances for the first two pile sections analyzed, LRFD Article 10.7.8 states that for routine pile installation applications where significant local experience can be applied to keep the risk of pile installation problems low, a project specific drivability analysis using the wave equation may be waived. In light of this, it is recommended that the governing resistance in the lower portion of the pile used in design be the factored geotechnical resistance in the table above. The upper portion of the pile may be governed by a lesser axial pile load in order to satisfy the interaction equation (LRFD Article 6.9.2.2).

7.3.2 Service Limit and Extreme Limit State Designs

Per LRFD Article 10.5.5.1 the ability of the pier piles to meet defection criteria at the service limit state shall be investigated using a resistance factor of 1.0. Per LRFD Article 10.5.5.3.3

the ability of the pier piles at the extreme limit state shall be investigated using a resistance factor of 1.0.

The axial structural resistance of eight (8) proposed pipe pile sections and four (4) proposed H-pile sections was investigated using a resistance factor of 1.0. The piles have an unbraced length and require calculation of the λ factor as specified in LRFD Article 6.9. The axial geotechnical compressive resistance of eight (8) proposed pipe pile sections and four (4) proposed H-pile sections was calculated using Canadian Foundation Engineering Manual methods and a resistance factor of 1.0. The drivability of the eight (8) proposed pipe pile sections and four (4) proposed H-pile sections was considered. The maximum driving stresses in the pipe pile, assuming the use of 310 MPa (45 ksi) steel, shall be less than 275 MPa (40 ksi). The maximum driving stresses in the H-pile, assuming the use of 345 MPa (50 ksi) steel, shall be less than 310 MPa (45 ksi). The resistance factor for a single pile in axial compression for the service and extreme limit states of 1.0 was used.

The calculated factored axial structural, geotechnical and drivability resistances for the eight (8) pipe pile sections and four (4) proposed H-pile sections are summarized in the table below. Supporting calculations are included in Appendix C- Calculations found at the end of this report.

Factored Axial Resistances for Pipe Piles and H-Piles at the Service and Extreme Limit States

Pipe Pile		Factored Resistance			
Diameter	Wall thickness	Structural Resistance	Geotechnical Resistance	Drivability Resistance	Governing Resistance
609 mm (24-in)	13 mm (1/2–in)	5051 kN (1136 kips)	3902 kN (877 kips)	2602 kN (585 kips)	3902 kN (877 kips)
660 mm (26-in)	13 mm (1/2–in)	5534 kN (1244 kips)	4126 kN (928 kips)	2802 kN (630 kips)	4126 kN (928 kips)
711 mm (28-in)	13 mm (1/2–in)	6016 kN (1352 kips)	4349 kN (978 kips)	3056 kN (687 kips)	4349 kN (978 kips)
762 mm (30-in)	13 mm (1/2–in)	6496 kN (1460 kips)	4572 kN (1028 kips)	3301 kN (742 kips)	4572 kN (1028 kips)
609 mm (24-in)	16 mm (5/8–in)	6679 kN (1501 kips)	5175 kN (1163 kips)	3536 kN (795 kips)	5175 kN (1163 kips)
660 m (26-in)	16 mm (5/8–in)	7324 kN (1646 kips)	5474 kN (1231 kips)	3914 kN (880 kips)	5474 kN (1231 kips)
711 mm (28-in)	16 mm (5/8–in)	7966 kN (1791 kips)	5772 kN (1298 kips)	4270 kN (960 kips)	5772 kN (1298 kips)
762 mm (30-in)	16 mm (5/8–in)	8607 kN (1935 kips)	6070 kN (1365 kips)	4693 kN (1055 kips)	6070 kN (1365 kips)

H-pile Section	Structural Resistance	Geotechnical Resistance	Drivability Resistance	Governing Resistance
HP 310 x 79 (HP 12 x 53)	2867 kN (644 kips)	2913 kN (655 kips)	2202 kN (450 kips)	2867 kN (644 kips)
HP 360 x 108 (HP 14 x 73)	4098 kN (921 kips)	3672 kN (826 kips)	2482 kN (558 kips)	3672 kN (826 kips)
HP 360 x 132 (HP 14 x 89)	4986 kN (1121 kips)	4464 kN (1003 kips)	2736 kN (615 kips)	4464 kN (1003 kips)
HP 360 x 174 (HP 14 x 117)	6549 kN (1472 kips)	5849 kN (1315 kips)	3305 kN (743 kips)	5849 kN (1315 kips)

Although the factored axial drivability resistance is less than both the factored axial structural and geotechnical resistances for the first two pile sections analyzed, LRFD Article 10.7.8 states that for routine pile installation applications where significant local experience can be applied to keep the risk of pile installation problems low, a project specific drivability analysis using the wave equation may be waived. In light of this, it is recommended that the governing resistance in the lower portion of the pile used in design be the resistance shown in the last column of in the table above. For the H-piles, It should be noted that the governing resistance for the HP 310 x 79 (HP 12 x 53) pile is the structural resistance while the remaining H-pile sections are governed by the geotechnical resistance. The upper portion of the pile may be governed by a lesser axial pile load in order to satisfy the interaction equation (LRFD Article 6.9.2.2).

7.3.3 Estimated Depths to Pile Fixity

Stability of the piles shall be evaluated in accordance with the provisions in LRFD Article 6.9 using an equivalent pile length of the pile that accounts for the laterally supported length of the exposed pile extending through the air and/or water plus the embedment depth to pile fixity.

All piles should be designed to achieve a fixed condition for the design scour event. Preliminary depths to fixity for eight (8) proposed pipe pile sections and four (4) proposed H-pile sections were calculated, assuming only axial loading and without consideration of lateral loads, using the buckling methodology in LRFD Article 10.7.3.13.4. The table below summarizes the calculated depths to fixity for the eight (8) proposed pile sections and four (4) proposed H-pile sections and the estimated design scour depth. The design scour depth provided by the Structural Designer was estimated to be approximately 4.3 meters (14 feet). Supporting calculations are included in Appendix C- Calculations found at the end of this report.

Preliminary Estimates of Depth to Fixity

Outside Pipe Pile Diameter	Wall thickness	Preliminary Estimates of Depth to Fixity w/ no lateral loads applied	Estimated Exposed Pile Length Due to Scour
609 mm (24-in)	13 mm (1/2–in)	3.41 meters (11 feet)	4.3 meters (14 feet)
660 mm (26-in)	13 mm (1/2–in)	3.61 meters (12 feet)	4.3 meters (14 feet)
711 mm (28-in)	13 mm (1/2–in)	3.81 meters (13 feet)	4.3 meters (14 feet)
762 mm (30-in)	13 mm (1/2–in)	4.01 meters (13 feet)	4.3 meters (14 feet)
609 mm (24-in)	16 mm (5/8–in)	3.5 meters (11 feet)	4.3 meters (14 feet)
660 mm (26-in)	16 mm (5/8–in)	3.71 meters (12 feet)	4.3 meters (14 feet)
711 mm (28-in)	16 mm (5/8–in)	3.91 meters (13 feet)	4.3 meters (14 feet)
762 mm (30-in)	16 mm (5/8–in)	4.11 meters (13 feet)	4.3 meters (14 feet)
H-pile Section		Preliminary Estimates of Depth to Fixity w/ no lateral loads applied	Estimated Exposed Pile Length Due to Scour
HP 310 x 79 (HP 12 x 53)		2.18 meters (7 feet)	4.3 meters (14 feet)
HP 360 x 108 (HP 14 x 73)		2.47 meters (8 feet)	4.3 meters (14 feet)
HP 360 x 132 (HP 14 x 89)		2.57 meters (8 feet)	4.3 meters (14 feet)
HP 360 x 174 (HP 14 x 117)		2.73 meters (9 feet)	4.3 meters (14 feet)

In general it is recommended that piles be designed to achieve a fixed condition below the design scour depth. Due to the depth of the overburden at the site, it is anticipated that the pile sections will all achieve a fixed condition assuming a pile penetration to the top of bedrock.

When the lateral and axial pile load groups are known, this data should be provided to the geotechnical engineer. A more refined analysis of pile fixity can then be performed using LPile or FBPier software.

7.3.4 Buckling and Combined Axial and Flexure

Pile group design shall consider loading effects due to combined axial and flexural loading, as outlined in LRFD Article 6.15. In designing piles for the bent group the group effects of soil-structure interaction shall be considered in conformance with LRFD Article 10.7.3.9. The recommended design approach considers the non-linear response of soil with lateral displacement. Soil-structure interaction considering the non-linear response of soil can be modeled using computer software supplied by the geotechnical engineer.

The factored structural resistances for pipe pile sections in combined axial compression and flexure are not provided in this report as these analyses are considered part of the structural design and the responsibility of the structural engineer.

7.3.5 Pile Resistance and Pile Quality Control

Based on the anticipated depth to bedrock at the site, pile splices will be required. The location and number of pile splices shall be in conformance with MaineDOT Standard Specification 501 and be subject to the approval of the Resident.

The Contractor is required to perform a wave equation analysis of the proposed pile-hammer system and a dynamic pile test at the pier. The first pile driven at the pier should be dynamically tested to confirm capacity and verify the stopping criteria developed by the Contractor in the wave equation analysis. The ultimate pile resistance that must be achieved in the wave equation analysis and dynamic testing will be the factored axial pile load divided by a resistance factor of 0.65. The factored pile load should be shown on the plans per LRFD Article 3.6.5.2.

Piles should be driven to an acceptable penetration resistance as determined by the Contractor based on the results of a wave equation analysis and as approved by the Resident. Driving stresses in the pipe pile determined in the drivability analysis shall be less than 280 MPa (40 ksi) in accordance with LRFD Article 10.7.8. Driving stresses in the H-pile determined in the drivability analysis shall be less than 310 MPa (45 ksi) in accordance with LRFD Article 10.7.8. A hammer should be selected which provides the required resistance when the penetration resistance for the final 76 mm to 152 mm (3 to 6 inches) is 8 to 15 blows per 25 mm (1 inch). If an abrupt increase in driving resistance is encountered, the driving could be terminated when the penetration is less than 12 mm (0.5-inch) in 10 consecutive blows.

7.4 Scour and Riprap

The consequences of changes in foundation conditions resulting from the design flood for scour shall be considered at the strength and service limit states. These changes in foundation conditions shall be investigated at the abutments and wingwalls. For scour protection, any non critical retaining wall footings which are constructed on granular deposits, should be embedded a minimum of 0.9 meters (3 feet) below the design scour depth and at least 0.6 meters (2.0 feet) below the super flood scour event and armored with 0.9 meters (3 feet) of riprap. Refer to MaineDOT BDG Section 2.3.11 for information regarding scour design.

Riprap conforming to item number 703.26 of the Standard Specification shall be placed at the toes of abutments and wingwalls. Riprap shall be 0.9 meters (3 feet) thick. In front of the wingwalls, the bottom of the riprap section shall be constructed 2 meters (6.5 feet) above the bottom of the structures for frost protection. The riprap shall extend 0.5 meters (1.5 feet) horizontally in front of the wall before sloping at a maximum 1.75H:1V slope to the existing ground surface. The toe of the riprap section shall be constructed 0.3 meters (1 foot) below the streambed elevation. The riprap section shall be underlain by a 0.3 meters (1 foot) thick layer of bedding material conforming to item number 703.19 of the Standard Specification.

7.5 Settlement

Large amounts of fill will be placed behind both abutments in order to raise the existing grade to accommodate the new roadway approaches to the bridge. Settlements due to the addition of this fill have been calculated to range between 20 and 50 mm (1 and 2 inches). Due to the granular nature of the subsurface soils present at the site all settlement associated with this fill occur will during construction having negligible effect on the finished bridge structure. Any settlement of the bridge abutments and pier will be due to the elastic compression of the piling and will be negligible.

7.6 Frost Protection

Any foundation placed on granular subgrade soils should be designed with an appropriate embedment for frost protection. According to the MaineDOT frost depth maps for the State of Maine (MaineDOT BDG Figure 5-1) the site has a design-freezing index of approximately 1550 F-degree days. This correlates to a frost depth of 2 meters (6.5 feet). Therefore, any foundations placed on granular soils should be founded a minimum of 2 meters (6.5 feet) below finished exterior grade for frost protection. Integral abutments shall be embedded a minimum of 1.2 meters (4.0 feet) for frost protection per Figure 5-2 of the MaineDOT BDG. See Appendix D- Calculations at the end of this report for supporting documentation.

7.7 Seismic Design Considerations

The following parameters were determined for the site from the USGS Seismic Parameters CD provided with the LRFD manual:

- Peak Ground Acceleration coefficient (PGA) = 0.090g
- Short-term (0.2-second period) spectral acceleration coefficient = 0.183g
- Long-term (1.0-second period) spectral acceleration coefficient = 0.050g

Per LRFD Article 3.10.3.1 the site is assigned to Site Class D due to the presence of soils in the upper 30 meters (100 feet) of the soil profile with an average N-value between 15 and 50 blows per foot at the site. Per LRFD Article 3.10.6 the site is assigned to Seismic Zone 1 based on a calculated S_{D1} of 0.0.119 (LRFD Eq. 3.10.4.2-6). Per LRFD Article 4.7.4.1, bridges in Seismic Zone 1 need not be analyzed for seismic loads regardless of their importance. However the minimum requirements as specified in LRFD Articles 4.7.4.4 and 3.10.9 apply.

According to Figure 2-2 of the MaineDOT BDG, the Wild River Bridge on Route 2 is on the National Highway System (NHS) and is therefore considered to be functionally important. Consequently, a detailed seismic analysis is required. The minimum seismic analysis requirements are defined in LFRD Article 4.7.4.1. The designer shall determine the specific analysis method using LRFD Tables 4.7.4.3.1-1. Seismic design requirements for Seismic Zone 1 are found in LRFD Article 3.10.9.2.

7.8 Construction Considerations

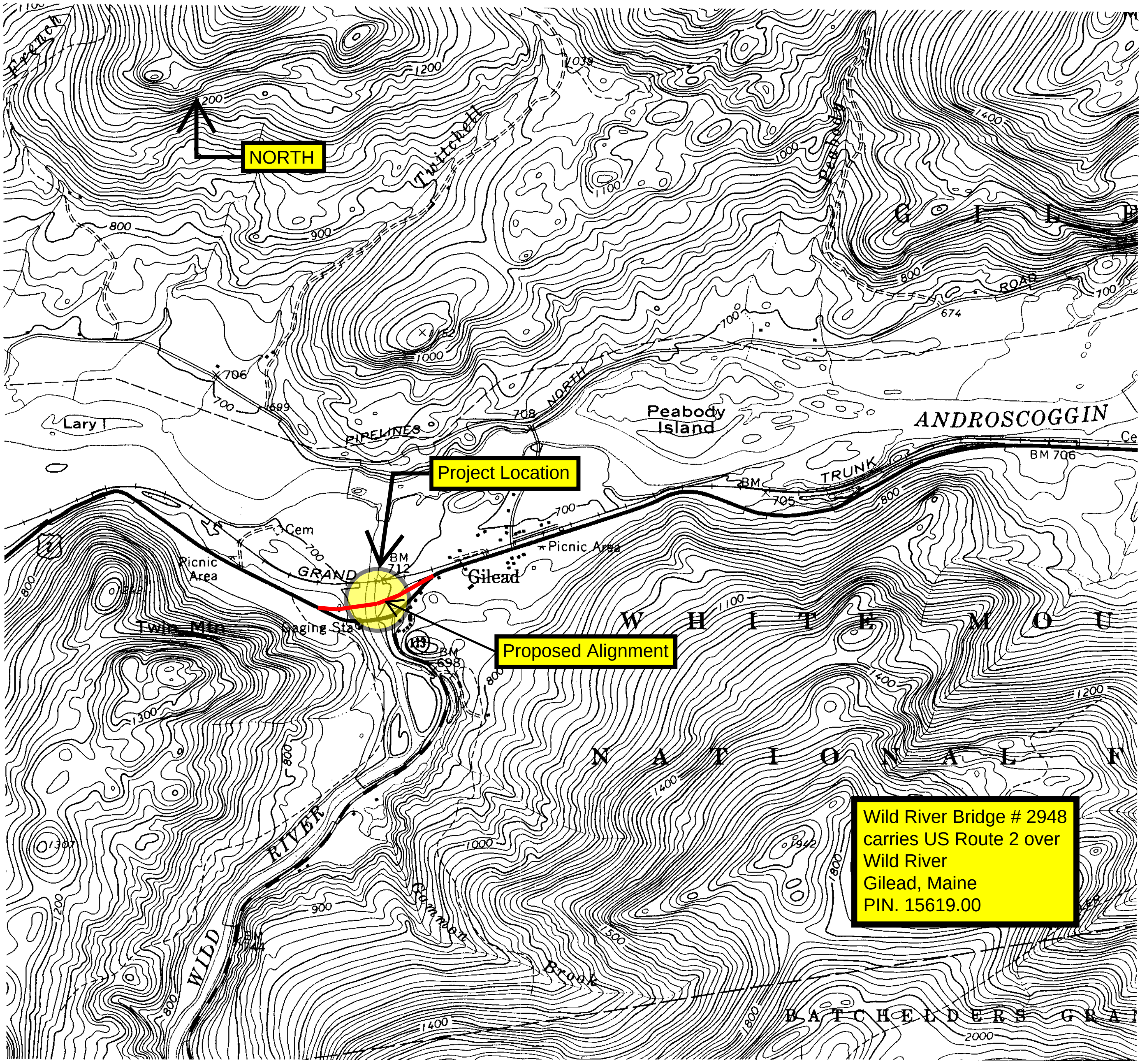
Boulders and cobbles were encountered within the interbedded sand and gravel layers in all of the borings. There is potential for these obstructions to impact the pile installation operations. These impacts include, but are not limited to, driving the piles and cleaning out pipe piles. Obstructions may be cleared by conventional excavation methods, pre-augering, pre-drilling, or down-hole hammers. Care should be taken to drive piles within allowable tolerances. Alternative methods to clear obstructions may be used as approved by the Resident.

8.0 CLOSURE

This report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed replacement of Wild River Bridge in Gilead, Maine in accordance with generally accepted geotechnical and foundation engineering practices. No other intended use is implied. In the event that any changes in the nature, design, or location of the proposed project are planned, this report should be reviewed by a geotechnical engineer to assess the appropriateness of the conclusions and recommendations and to modify the recommendations as appropriate to reflect the changes in design. Further, the analyses and recommendations are based in part upon limited soil explorations at discrete locations completed at the site. If variations from the conditions encountered during the investigation appear evident during construction, it may also become necessary to re-evaluate the recommendations made in this report.

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

Sheets



GILEAD QUADRANGLE
MAINE–OXFORD CO.
7.5 MINUTE SERIES (TOPOGRAPHIC)
NW/4 BETHEL 15' QUADRANGLE

SCALE 1:24000

1 ½ 0 1 MILE

1000 0 1000 2000 3000 4000 5000 6000 7000 FEET

1 5 0 1 KILOMETER

CONTOUR INTERVAL 20 FEET
DATUM IS MEAN SEA LEVEL

Filename: ...\00\GEOTECH\MSTA\006_BLP1.dgn Division: GEOTECH Username: terry.white Date:11/19/2008

PROJECT DESIGN ENGINEER	K. MAGUIRE		
DESIGN-DETAILED	T. WHITE	APR 2008	
CHECKED			
REVISIONS			
FIELD CHANGES			

PLANS

METRIC
1. All dimensions are in millimeters unless otherwise noted.
2. All elevations and stations are in meters.

F.H.W.A. REG. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	BH-1561(900)E	2	6
		15619.00		

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

WILD RIVER BRIDGE
OVER
WILD RIVER
IN THE TOWN OF
GILEAD
OXFORD COUNTY

BORING LOCATION PLAN

SHEET OF AUGUSTA, MAINE

METRIC
1. All dimensions are in millimeters unless otherwise noted.
2. All elevations and stations are in meters.

F.H.W.A. REG. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	BH-1561(900)E	3	6

BRIDGE NO. 2948

PROFILE

HORIZ. 20 0 20 40
VERT. 4 0 4 8
SCALE (in meters)

Boring No. Offset, if shown
Strata Interface
Boring
RQD= Rock Quality Designation for Rock Core Sample
BOE= Bottom Of Exploration

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

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

WILD RIVER BRIDGE
OVER
WILD RIVER
IN THE TOWN OF
GILEAD
OXFORD COUNTY

INTERPRETIVE SUBSURFACE PROFILE

SHEET OF AUGUSTA, MAINE

		BY	DATE
PROJECT DESIGN ENGINEER			
DESIGN-DETAILED	K. MAGUIRE	T. WHITE	APR 2008
CHECKED			
REVISIONS			
FIELD CHANGES			

PLANS

Filename: ...\00\geotech\msta\007_ISP1.dgn Division: GEOTECH Username: terry.white Date:11/19/2008

METRIC	1. All dimensions are in millimeters unless otherwise noted. 2. All elevations and stations are in meters.	F.H.W.A. REG. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
		1	MAINE	BH-1561(900)E	4	6

15619.00

Maine Department of Transportation
Soil/Rock Exploration Log
METRIC UNITS

Project: Wild River Bridge #2948 over Wild River, Route 2	Location: Gilead, Maine	Boring No.: BB-GWR-101	PIN: 15619.00
Driller: Northern Test Boring	Elevation (m): 210.75	Auger ID/OD: 5" Solid Stem	
Operator: Mike/Nick	Datum: NAVD 88	Sampler: Standard Split Spoon	
Logged By: B. Wilder	Rig Type: Diedrich D-50	Hammer Wt./Fall: 140#/30"	
Date Start/Finish: 4/7/08-4/8/08	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"	
Boring Location: 3+410, CL	Casing ID/OD: HW	Water Level*: 2.44 m bgs.	
Hammer Efficiency Factor: 0.633	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 Walled Tube Sample attempt
V = Insitu Vane Shear Test
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 64 kg hammer
WOR/C = weight of rods or casing
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (kPa)
T_v = Pocket Torvane Shear Strength (kPa)
q_p = Unconfined Compressive Strength (Pa)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer effeciency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (kPa)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Sample No.	Pen/Rec (cm)	Sample Depth (m)	Blows (150 mm) Shear Strength (kPa) or RQD (%)	N-value	N60	Casing Blows	Elevation (m)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
1D	61.0/35.6	0.00 - 0.61	2/1/2/2	3	3	SSA		Brown, damp, very loose, fine to medium SAND, trace silt, trace coarse sand, trace gravel, trace organics.	G#209912 A-3, SP-SM WC=15.0%
							209.84	0.91	
2D	61.0/38.1	1.22 - 1.83	10/16/23/40	39	41			Brown, damp, dense, fine to coarse Sandy GRAVEL, trace silt, occasional cobbles.	G#209913 A-1-a, GW-GM WC=2.9%
3D	55.9/40.6	2.74 - 3.30	18/31/21/50(100)	52	55			Brown, wet, very dense, fine to coarse Sandy GRAVEL, trace silt, occasional cobbles.	G#209914 A-1-a, GW WC=10.2%
4D	30.5/20.3	4.27 - 4.57	18/60	---		50		Similar to above. Large cobble from 4.57-4.85 m bgs. Roller Coned ahead to 5.79 m bgs.	
						183			
						143			
						126			
						111			
							204.96	5.79	
5D	61.0/40.6	5.79 - 6.40	9/8/8/8	16	17	113		Brown, wet, medium dense, fine to medium SAND, some silt, trace coarse sand.	G#209915 A-2-4, SM WC=23.4%
						103			
						107			
						126			
						131			
6D	61.0/61.0	7.32 - 7.92	6/7/8/7	15	16	92		Brown, wet, medium dense, fine SAND, some silt, trace medium sand.	G#209916 A-2-4, SM WC=24.3%
						112			
						129			
						142			
						153			
7D	61.0/35.6	8.84 - 9.45	4/7/13/16	20	21	97		Brown, wet, medium dense, fine to coarse SAND, little gravel, little silt.	G#209917 A-2-4, SP-SM WC=17.5%
						162			
						173			
							201.00	9.75	
						192			
						191			
8D	61.0/33.0	10.36 - 10.97	3/7/31/48	38	40	127		Brown, wet, dense, fine to coarse SAND, some gravel, little silt, occasional cobbles.	G#209918 A-1-b, SM WC=9.2%
						213			
						217			
						234			
						373			
							198.93	11.83	
						b310		Large cobble from 11.83-12.04 m bgs. b310 blows for 0.15 m.	
9D	61.0/35.6	12.04 - 12.65	22/16/27/24	43	45			Brown, wet, dense, fine to coarse Sandy GRAVEL, trace silt.	G#209919 A-1-a, GP-GM WC=9.0%
						aRC		aRoller Coned ahead to 13.11 m bgs.	
							197.95	12.80 Top of Bedrock at Elev. 197.95 m.	
R1	152.4/149.9	13.11 - 14.63	RQD = 85%			NQ CORE		Bedrock: Black, white and grey, coarse grained, metamorphic, GNEISS, hard, fresh, with banding at 70 degrees. Rock Mass Quality = Good. R1:Core Times (min:sec) 600-700 psi down pressure 13.11-13.41 m (2:30) 13.41-13.72 m (2:40) 13.72-14.02 m (3:04) 14.02-14.33 m (3:07) 14.33-14.63 m (3:23) 98% Recovery	
R2	152.4/152.4	14.63 - 16.15	RQD = 93%					R2: Rock Mass Quality = Excellent. Core Times (min:sec) 14.63-14.94 m (2:46) 14.94-15.24 m (2:36) 15.24-15.54 m (2:34) 15.54-15.85 m (2:00) 15.85-16.15 m (2:20) 100% Recovery	
							194.60	16.15 Bottom of Exploration at 16.15 m below ground surface.	

Remarks:
Auto Hammer #283

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Page 1 of 1
Boring No.: BB-GWR-101

Maine Department of Transportation
Soil/Rock Exploration Log
METRIC UNITS

Project: Wild River Bridge #2948 over Wild River, Route 2	Location: Gilead, Maine	Boring No.: BB-GWR-102	PIN: 15619.00
Driller: Northern Test Boring	Elevation (m): 208.39	Auger ID/OD: 5" Solid Stem	
Operator: Mike/Nick	Datum: NAVD 88	Sampler: Standard Split Spoon	
Logged By: B. Wilder	Rig Type: Diedrich D-50	Hammer Wt./Fall: 140#/30"	
Date Start/Finish: 9/15,16,18,19/08	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"	
Boring Location: 3+450, CL	Casing ID/OD: HW & NW	Water Level*: River Boring	
Hammer Efficiency Factor: 0.633	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 Walled Tube Sample attempt
V = Insitu Vane Shear Test
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 64 kg hammer
WOR/C = weight of rods or casing
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (kPa)
T_v = Pocket Torvane Shear Strength (kPa)
q_p = Unconfined Compressive Strength (Pa)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer effeciency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (kPa)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Sample No.	Pen/Rec (cm)	Sample Depth (m)	Blows (150 mm) Shear Strength (kPa) or RQD (%)	N-value	N60	Casing Blows	Elevation (m)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
1D	27.4/27.4	0.00 - 0.27	10/50(120)	---		SSA		Brown, wet, very dense, fine to coarse Sandy GRAVEL, trace silt, with cobbles and boulders.	
							207.47	0.91	
2D	61.0/35.6	1.52 - 2.13	3/6/9/10	15	16	132		Brown, wet, medium dense, fine to coarse SAND, some silt, trace gravel. Roller Coned ahead to 2.74 m bgs.	G#208757 A-2-4, SM WC=26.3%
						152			
						170			
						156			
3D	61.0/25.4	2.74 - 3.35	10/11/12/9	23	24	148		Brown, wet, medium dense Gravelly fine to coarse SAND, trace silt. Roller Coned ahead to 4.27 m bgs.	G#208758 A-1-a, SW WC=14.3%
						148			
						112			
						122			
						122			
4D	61.0/22.9	4.27 - 4.88	4/4/21/20	25	26	19		Brown, wet, medium dense, fine to coarse SAND, some gravel, trace silt. Roller Coned ahead to 5.18 m bgs.	G#208759 A-3, SP WC=15.7%
						91			
						136			
						150			
							202.90	5.49	
						167			
5D	61.0/38.1	5.79 - 6.40	16/18/16/15	34	36	128		Brown, wet, dense, fine to coarse Sandy GRAVEL, trace silt.	G#208760 A-1-a, GW WC=8.7%
						140			
						145			
						139			
						152			
6D	61.0/22.9	7.32 - 7.92	17/12/11/15	23	24	131		Similar to above, medium dense.	
						105			
						140			
						143			
						183			
7D	61.0/40.6	8.84 - 9.45	10/11/20/19	31	33	108		Brown, wet, dense, Gravelly fine to coarse SAND, trace silt.	G#208761 A-1-a, SW WC=10.6%
						230			
						216			
						173			
						182			
							198.03	10.36	
8D	61.0/35.6	10.36 - 10.97	5/5/4/5	9	9	127		Brown, wet, loose, fine to coarse SAND, little gravel, trace silt.	G#208762 A-1-b, SP WC=15.3%
						146			
						184			
						234			
						268			
9D	61.0/33.0	11.89 - 12.50	7/7/7/10	14	15	127		Similar to above, but medium dense.	G#208763 A-1-b, SP WC=19.0%
						192			
						226			
							195.59	12.80	
						226			
						311			
10D	61.0/30.5	13.41 - 14.02	9/14/10/7	24	25	146		Brown, wet, medium dense, fine to coarse Sandy GRAVEL, trace silt.	G#208764 A-1-a, GW-GM WC=4.9%
						256			
						261			
						312			
						309			
MD	61.0/0.0	14.94 - 15.54	18/21/14/12	35	37	184		Failed sample attempt.	
						252			

Remarks:
Auto Hammer #283
0.3 m water at boring location.

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 2
Boring No.: BB-GWR-102

Maine Department of Transportation
Soil/Rock Exploration Log
METRIC UNITS

Project: Wild River Bridge #2948 over Wild River, Route 2	Location: Gilead, Maine	Boring No.: BB-GWR-102	PIN: 15619.00
Driller: Northern Test Boring	Elevation (m): 208.39	Auger ID/OD: 5" Solid Stem	
Operator: Mike/Nick	Datum: NAVD 88	Sampler: Standard Split Spoon	
Logged By: B. Wilder	Rig Type: Diedrich D-50	Hammer Wt./Fall: 140#/30"	
Date Start/Finish: 9/15,16,18,19/08	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"	
Boring Location: 3+450, CL	Casing ID/OD: HW & NW	Water Level*: River Boring	
Hammer Efficiency Factor: 0.633	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 Walled Tube Sample attempt
V = Insitu Vane Shear Test
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 64 kg hammer
WOR/C = weight of rods or casing
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (kPa)
T_v = Pocket Torvane Shear Strength (kPa)
q_p = Unconfined Compressive Strength (Pa)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer effeciency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (kPa)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Sample No.	Pen/Rec (cm)	Sample Depth (m)	Blows (150 mm) Shear Strength (kPa) or RQD (%)	N-value	N60	Casing Blows	Elevation (m)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
						309			
						347		Boulder from 16.12-16.46 m bgs.	
						400			
R1	396.2/152.4	16.46 - 20.42				a275 NQ-2		aChanged/telescoped to NW Casing. Roller Coned ahead with H Roller Cone to 22.56 m bgs. R1:Cobbles and Boulders. R1:Core Times (min:sec) 16.46-16.76 m (3:01) 16.76-17.07 m (1:27) 17.07-17.37 m (1:10) 17.37-17.68 m (1:22) 17.68-17.98 m (3:05) 17.98-18.29 m (2:06) 18.29-18.59 m (0:41) 18.59-18.9 m (0:40) 18.9-19.2 m (0:52) 19.2-19.51 m (0:53) 19.51-19.8 m (0:29) 19.8-20.12 m (0:26) 20.12-20.42 m (0:45) 38% Recovery	
						130			
						110			
						164			
						387			
						121			
						99			
						158			
						264			
						247			
						162			
						196			
						161			
						209			
						432			
						121			
						112			
						194			
						201			
						336			
11D	30.5/25.4	22.56 - 22.86	26/50	---		96		Brown, wet, very dense, fine to coarse SAND, trace silt, trace gravel. Roller Coned ahead every 1.52 m. Boulder from 22.68-23.32 m bgs.	G#208765 A-3, SP-SM WC=19.3%
						205			
						792			
						408			
						b250		b250 blows for 0.03 m.	
12D	42.7/38.1	24.08 - 24.51	41/60/50(75)	---				Brown, wet, very dense, fine to coarse SAND, little silt, trace gravel. Cobble from 24.5-24.69 m bgs. Roller Coned ahead to 25.97 m bgs., Hit Bedrock at 25.51 m bgs.	G#208766 A-2-4, SM WC=15.1%
							182.88	25.51 Top of Bedrock at Elev. 182.88 m.	
R2	152.4/144.8	25.97 - 27.49	RQD = 95%			NQ-2		Bedrock: Black, white and grey, coarse grained, metamorphic, GNEISS, hard, fresh, with banding at 70 degrees. Rock Mass Quality = Excellent. R2:Core Times (min:sec) 25.97-26.27 m (1:45) 26.27-26.58 m (2:31) 26.58-26.88 m (3:06) 26.88-27.19 m (3:45) 27.19-27.49 m (4:20) 95% Recovery	
							180.90	27.49 Bottom of Exploration at 27.49 m below ground surface. Bent Casing and bottom of Drive Shoe, could not get back down hole.	

Remarks:
Auto Hammer #283
0.3 m water at boring location.

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 2 of 2
Boring No.: BB-GWR-102

BRIDGE NO. 2948

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

WILD RIVER BRIDGE
OVER
WILD RIVER
IN THE TOWN OF
GILEAD
OXFORD COUNTY

BORING LOGS

SHEET 1 OF 2
AUGUSTA, MAINE

	BY	DATE
PROJECT DESIGN ENGINEER	K. MAGUIRE	APR 2008
DESIGN-DETAILED	T. WHITE	
CHECKED		
REVISIONS		
FIELD CHANGES		

PLANS

Filename: ...\msta\008_BORING LOGS1.dgn Division: GEOTECH Username: terry.white Date:11/19/2008

Maine Department of Transportation

Soil/Rock Exploration Log

METRIC UNITS

Project: Wild River Bridge #2948 over Wild River, Route 2	Location: Gilead, Maine	Boring No.: BB-GWR-103	PIN: 15619.00
Driller: Northern Test Boring	Elevation (m): 209.05	Auger ID/OD: 5" Solid Stem	
Operator: Mike/Nick	Datum: NAVD 88	Sampler: Standard Split Spoon	
Logged By: B. Wilder	Rig Type: Diedrich D-50	Hammer Wt./Fall: 140#/30"	
Date Start/Finish: 3/20-21/08, 4/8/08	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"	
Boring Location: 3+489, CL	Casing ID/OD: HW	Water Level*: 0.97 m bgs.	
Hammer Efficiency Factor: 0.633	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 Walled Tube Sample attempt
V = Insitu Vane Shear Test
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 64 kg hammer
WOR/C = weight of rods or casing
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (kPa)
T_v = Pocket Torvane Shear Strength (kPa)
q_p = Unconfined Compressive Strength (Pa)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer effeciency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (kPa)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (m)	Sample No.	Pen/Rec (cm)	Sample Depth (m)	Blows (150 mm) Shear Strength (kPa) or RQD (%)	N-value	N_{60}	Casing Blows	Elevation (m)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	40.6/10.2	0.00 - 0.41	7/11/50(100 mm)	---		SSA		Brown, wet, dense, fine to coarse SAND, some gravel, little silt, trace organics. Cobble from 0.4-0.46 m bgs.	
	2D	61.0/43.2	1.52 - 2.13	2/10/18/33	28	30			Brown, wet, medium dense, fine to coarse Sandy GRAVEL, trace silt, occasional cobble.	G#210091 A-1-a, GW-GM WC=11.3%
	3D	61.0/45.7	3.05 - 3.66	17/38/36/30	74	78	101	205.85	3.20 Brown, wet, very dense, fine to coarse SAND, some gravel, trace silt, occasional cobble.	
							304			
							206			
							138			
							118			
	4D	61.0/35.6	4.57 - 5.18	10/13/23/23	36	38	59		Brown, wet, dense, fine to coarse SAND, some gravel, trace silt. Roller Coned ahead to 6.1 m bgs.	G#210092 A-1-b, SW-SM WC=12.9%
							103			
							134			
							156			
							147			
	5D	61.0/33.0	6.10 - 6.71	13/15/15/17	30	32	73		Similar to above. Roller Coned ahead to 7.62 m bgs.	
							133			
							213			
							166			
							135			
	6D	61.0/33.0	7.62 - 8.23	10/10/13/16	23	24	89		Brown, wet, medium dense, fine to medium SAND, trace gravel, trace coarse sand, trace silt. Roller Coned ahead to 9.14 m bgs.	G#210093 A-3, SP-SM WC=20.2%
							64			
							210			
							318	200.52	8.53	
							168			
	7D	51.8/25.4	9.14 - 9.66	12/14/12/30(60)	26	27	131		Brown, wet, medium dense, fine to coarse Sandy GRAVEL, occasional cobbles. Roller Coned ahead to 10.67 m bgs.	
							148			
							462			
							380			
							264			
	8D	61.0/35.6	10.67 - 11.28	8/26/17/12	43	45	187		Brown, wet, dense, fine to coarse Sandy GRAVEL, trace silt. Roller Coned ahead to 12.19 m bgs.	G#210094 A-1-a, GW WC=9.6%
							240			
							369			
							315			
							316			
	9D	61.0/38.1	12.19 - 12.80	40/11/13/13	24	25	135	196.86	12.19 Brown, wet, medium dense, fine to coarse SAND, some gravel, trace silt. Roller Coned ahead to 13.72 m bgs.	
							26			
							43			
							48			
							54			
	10D	51.8/38.1	13.72 - 14.23	26/17/22/30(60)	39	41	168		Brown, wet, dense, fine to coarse SAND, some gravel, trace silt. Cobble ftom 14.23-14.33 m bgs.	G#210095 A-1-b, SW-SM WC=11.2%
							215			
							331			
							353			
							487			
	11D	61.0/33.0	15.24 - 15.85	14/13/11/11	24	25	217		Brown, wet, medium dense, fine to coarse SAND, little gravel, trace silt. Roller Coned ahead to 16.76 m bgs.	

Remarks:

Auto Hammer #283

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
METRIC UNITS
Project: Wild River Bridge #2948 over Wild River, Route 2
Location: Gilead, Maine
Boring No.: BB-GWR-103
PIN: 15619.00
Driller: Northern Test Boring | Elevation (m): 209.05 | Auger ID/OD: 5" Solid Stem
Operator: Mike/Nick | Datum: NAVD 88 | Sampler: Standard Split Spoon
Logged By: B. Wilder | Rig Type: Diedrich D-50 | Hammer Wt./Fall: 140#/30"
Date Start/Finish: 3/20-21/08, 4/8/08 | Drilling Method: Cased Wash Boring | Core Barrel: NQ-2"
Boring Location: 3+489, CL | Casing ID/OD: HW | Water Level*: 0.97 m bgs.
Hammer Efficiency Factor: 0.633 | Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐

Depth (m)	Sample No.	Pen/Rec (cm)	Sample Depth (m)	Blows (150 mm) Shear Strength (kPa) or RQD (%)	N-value	N_{60}	Casing Blows	Elevation (m)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
							267		0.61 m running sand in casing.	
							271			
							323			
							449			
	12D	61.0/55.9	16.76 - 17.37	6/13/17/21	30	32	224		Brown, wet, dense, fine to coarse SAND, little gravel, trace silt. Roller Coned ahead to 18.29 m bgs.	G#210096 A-1-b, SW WC=15.4%
							295			
							262			
							258			
							286	191.07	17.98	
	13D	48.8/38.1	18.29 - 18.78	19/27/34/30(30)	61	64	266		Brown, wet, very dense, fine to coarse SAND, some gravel, trace silt, occasional cobbles. Cobble from 18.78-18.84 m bgs. Roller Coned ahead to 19.2 m then to 19.81 m bgs.	
							350			
							600			
							350			
							312			
	14D	61.0/45.7	19.81 - 20.42	9/5/4/8	9	9	202		Brown, wet, loose, fine to coarse SAND, little gravel, trace silt, occasional cobble. Roller Coned ahead to 21.34 m bgs.	G#210097 A-1-b, SW WC=15.6%
							416			
							327			
							330			
							424			
	15D	36.6/35.6	21.34 - 21.70	7/14/50(60)	₵50		225		Brown, wet, very dense, fine to coarse SAND, some gravel, trace silt. Roller Coned ahead to 22.86 m bgs.	
							334			
							387			
							447			
							507			
	16D	61.0/40.6	22.86 - 23.47	25/23/16/16	39	41	237		Brown, wet, dense, fine to coarse SAND, some gravel, trace silt.	G#210098 A-1-b, SW-SM WC=12.8%
							405			
							286			
							355			
							437		Boulder from 24.08-24.54 m bgs.	
							408	184.51	24.54	
	17D	61.0/43.2	24.54 - 25.15	5/12/16/15	28	30	467		Brown, wet, medium dense, Gravelly fine to coarse SAND, trace silt. Roller Coned ahead to 25.91 m bgs.	G#210099 A-1-b, SW-SM WC=11.2%
							625			
							550			
							981			
	18D	61.0/40.6	25.91 - 26.52	11/24/28/36	52	55	450	183.14	25.91 Light brown, wet, very dense, fine to medium SAND, little silt, trace coarse sand, trace gravel. aRoller Coned ahead to 27.74 m bgs.	G#210100 A-2-4, SM WC=22.1%
							500			
							478 aRC	182.23	26.82 Soft weathered Bedrock.	
	R1	152.4/152.4	27.74 - 29.26	RQD = 65%			NQ CORE	181.31	27.74 Top of intact Bedrock at Elev. 181.31 m. Bedrock: Black, white and grey, coarse grained, metamorphic, GNEISS, hard, fresh, with banding at 70 degrees. Rock Mass Quality = Fair. R1:Core Times (min:sec) 800-1000 psi down pressure 27.74-28.04 m (2:13) 28.04-28.35 m (4:18) 28.35-28.65 m (2:37) 28.65-28.96 m (1:30) 28.96-29.26 m (1:45) 100% Recovery	
	R2	152.4/147.3	29.26 - 30.78	RQD = 87%					R2: Rock Quality = Good. Core Times (min:sec) 29.26-29.57 m (3:20) 29.57-29.87 m (3:25) 29.87-30.18 m (2:57) 30.18-30.48 m (3:09) 30.48-30.78 m (2:40) 97% Recovery	
								178.27	30.78 Bottom of Exploration at 30.78 m below ground surface.	

Remarks:

Auto Hammer #283

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.

BRIDGE NO. 2948

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

WILD RIVER BRIDGE
OVER
WILD RIVER
IN THE TOWN OF
GILEAD
OXFORD COUNTY

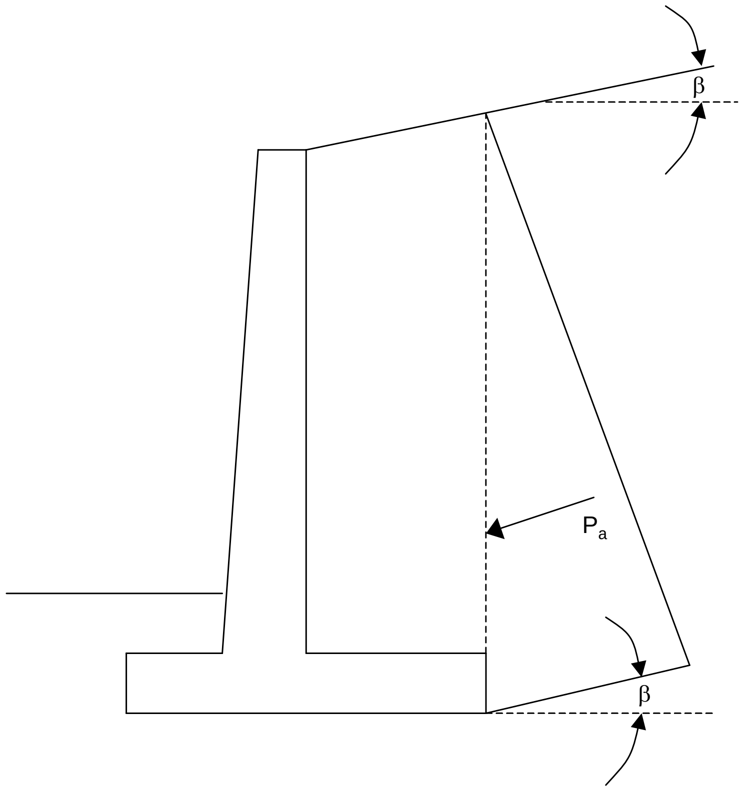
BORING LOGS

SHEET 2 OF 2 AUGUSTA, MAINE

	BY	DATE
PROJECT DESIGN ENGINEER	K. MAGUIRE	
DESIGN-DETAILED	T. WHITE	APR 2008
CHECKED	-----	-----
REVISIONS	-----	-----
FIELD CHANGES	-----	-----

PLANS

Filename: ...\msta\009_BORING LOGS2.dgn Division: GEOTECH Username: terry.white Date:11/19/2008



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

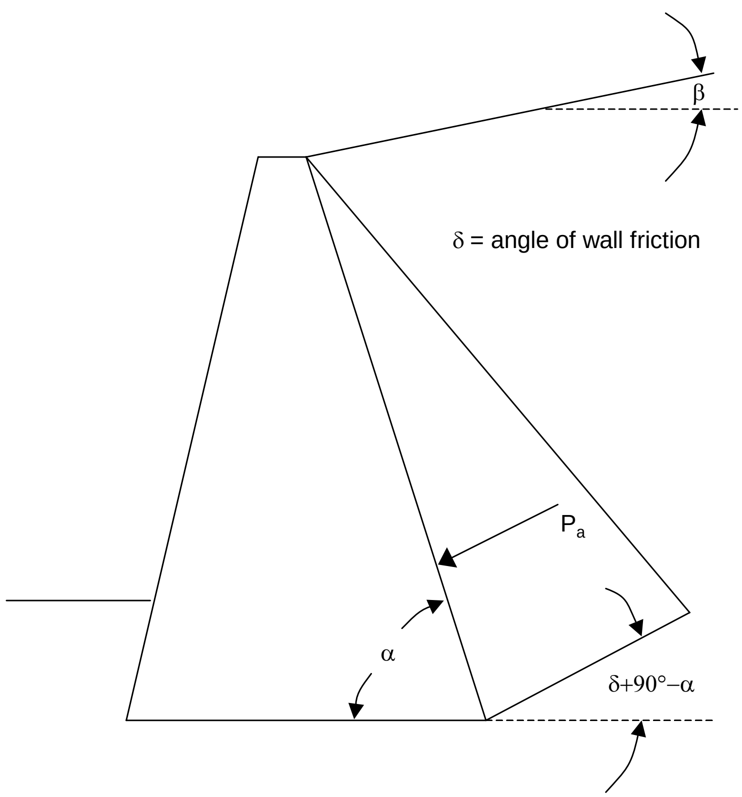
For a horizontal backfill surface, $\beta = 0°$:

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

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

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

P_a is oriented at β



For cases where interface friction is considered, use Coulomb.

For horizontal or sloped backfill surfaces:

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

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

Rankine and Coulomb Active Earth Pressure Coefficients

Appendix A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM

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

Desired Soil Observations: (in this order)

Color (Munsell color chart)
Moisture (dry, damp, moist, wet, saturated)
Density/Consistency (from above right hand side)
Name (sand, silty sand, clay, etc., including portions - trace, little, etc.)
Gradation (well-graded, poorly-graded, uniform, etc.)
Plasticity (non-plastic, slightly plastic, moderately plastic, highly plastic)
Structure (layering, fractures, cracks, etc.)
Bonding (well, moderately, loosely, etc., if applicable)
Cementation (weak, moderate, or strong, if applicable, ASTM D 2488)
Geologic Origin (till, marine clay, alluvium, etc.)
Unified Soil Classification Designation
Groundwater level

TERMS DESCRIBING DENSITY/CONSISTENCY

Coarse-grained soils (more than half of material is larger than No. 200 sieve): Includes (1) clean gravels; (2) silty or clayey gravels; and (3) silty clayey or gravelly sands. Consistency is rated according to standard penetration resistance.

Modified Burmister System

Descriptive Term	Portion of Total
trace	0% - 10%
little	11% - 20%
some	21% - 35%
adjective (e.g. sandy, clayey)	36% - 50%

Density of Cohesionless Soils	Standard Penetration Resistance N-Value (blows per foot)
Very loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

Fine-grained soils (more than half of material is smaller than No. 200 sieve): Includes (1) inorganic and organic silts and clays; (2) gravelly, sandy or silty clays; and (3) clayey silts. Consistency is rated according to shear strength as indicated.

Consistency of Cohesive soils	SPT N-Value blows per foot	Approximate Undrained Shear Strength (psf)	Field Guidelines
Very Soft	WOH, WOR, WOP, <2	0 - 250	Fist easily Penetrates
Soft	2 - 4	250 - 500	Thumb easily penetrates
Medium Stiff	5 - 8	500 - 1000	Thumb penetrates with moderate effort
Stiff	9 - 15	1000 - 2000	Indented by thumb with great effort
Very Stiff	16 - 30	2000 - 4000	Indented by thumbnail
Hard	>30	over 4000	Indented by thumbnail with difficulty

Rock Quality Designation (RQD):

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

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

Correlation of RQD to Rock Mass Quality

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

Desired Rock Observations: (in this order)

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

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

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

Sample Container Labeling Requirements:

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

Maine Department of Transportation

Soil/Rock Exploration Log

METRIC UNITS

Project: Wild River Bridge #2948 over Wild River, Route 2

Location: Gilead, Maine

Boring No.: BB-GWR-101

PIN: 15619.00

Driller:	Northern Test Boring	**Elevation (m):**	210.75	**Auger ID/OD:**	5" Solid Stem
Operator:	Mike/Nick	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/7/08-4/8/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	3+410, CL	**Casing ID/OD:**	HW	**Water Level*:**	2.44 m bgs.

Hammer Efficiency Factor: 0.633 **Hammer Type:** Automatic ☒ Hydraulic ☐ Rope & Cathead ☐

Definitions:

D = Split Spoon Sample	R = Rock Core Sample	S_u = Insitu Field Vane Shear Strength (kPa)	$S_{u(lab)}$ = Lab Vane Shear Strength (kPa)
MD = Unsuccessful Split Spoon Sample attempt	SSA = Solid Stem Auger	T_v = Pocket Torvane Shear Strength (kPa)	WC = water content, percent
U = Thin Wall Tube Sample	HSA = Hollow Stem Auger	q_p = Unconfined Compressive Strength (Pa)	LL = Liquid Limit
MU = Unsuccessful Thin Walled Tube Sample attempt	RC = Roller Cone	N-uncorrected = Raw field SPT N-value	PL = Plastic Limit
V = Insitu Vane Shear Test	WOH = weight of 64 kg hammer	Hammer Efficiency Factor = Annual Calibration Value	PI = Plasticity Index
MV = Unsuccessful Insitu Vane Shear Test attempt	WOR/C = weight of rods or casing	N_{60} = SPT N-uncorrected corrected for hammer effeciency	G = Grain Size Analysis
	WO1P = Weight of one person	N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected	C = Consolidation Test

Depth (m)	Sample No.	Pen/Rec (cm)	Sample Depth (m)	Blows (150 mm) Shear Strength (kPa) or RQD (%)	N-value	N_{60}	Casing Blows	Elevation (m)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	61.0/35.6	0.00 - 0.61	2/1/2/2	3	3	SSA			Brown, damp, very loose, fine to medium SAND, trace silt, trace coarse sand, trace gravel, trace organics.	G#209912 A-3, SP-SM WC=15.0%
								209.84		0.91	
1	2D	61.0/38.1	1.22 - 1.83	10/16/23/40	39	41				Brown, damp, dense, fine to coarse Sandy GRAVEL, trace silt, occasional cobbles.	G#209913 A-1-a, GW-GM WC=2.9%
2											
3	3D	55.9/40.6	2.74 - 3.30	18/31/21/50(100)	52	55				Brown, wet, very dense, fine to coarse Sandy GRAVEL, trace silt, occasional cobbles.	G#209914 A-1-a, GW WC=10.2%
4	4D	30.5/20.3	4.27 - 4.57	18/60	---		50			Similar to above.	
							183			Large cobble from 4.57-4.85 m bgs. Roller Coned ahead to 5.79 m bgs.	
5							143				
							126				
							111				
								204.96		5.79	
6	5D	61.0/40.6	5.79 - 6.40	9/8/8/8	16	17	113			Brown, wet, medium dense, fine to medium SAND, some silt, trace coarse sand.	G#209915 A-2-4, SM WC=23.4%
							103				
							107				
7							126				
							131				
	6D	61.0/61.0	7.32 - 7.92	6/7/8/7	15	16	92			Brown, wet, medium dense, fine SAND, some silt, trace medium sand.	G#209916 A-2-4, SM WC=24.3%
							112				

Remarks:

Auto Hammer #283

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
METRIC UNITS

Project: Wild River Bridge #2948 over Wild River, Route 2
Location: Gilead, Maine

Boring No.: BB-GWR-101
PIN: 15619.00

Driller:	Northern Test Boring	**Elevation (m):**	210.75	**Auger ID/OD:**	5" Solid Stem
Operator:	Mike/Nick	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/7/08-4/8/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	3+410, CL	**Casing ID/OD:**	HW	**Water Level*:**	2.44 m bgs.

Hammer Efficiency Factor: 0.633 **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 Walled Tube Sample attempt
V = Insitu Vane Shear Test
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 64 kg hammer
WOR/C = weight of rods or casing
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (kPa)
T_v = Pocket Torvane Shear Strength (kPa)
q_p = Unconfined Compressive Strength (Pa)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer effeciency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (kPa)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (m)	Sample No.	Pen/Rec (cm)	Sample Depth (m)	Blows (150 mm) Shear Strength (kPa) or RQD (%)	N-value	N_{60}	Casing Blows	Elevation (m)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
8							129			
							142			
							153			
9	7D	61.0/35.6	8.84 - 9.45	4/7/13/16	20	21	97		Brown, wet, medium dense, fine to coarse SAND, little gravel, little silt.	G#209917 A-2-4, SP-SM WC=17.5%
							162			
							173			
								201.00	9.75	
10							192			
							191			
	8D	61.0/33.0	10.36 - 10.97	3/7/31/48	38	40	127		Brown, wet, dense, fine to coarse SAND, some gravel, little silt, occasional cobbles.	G#209918 A-1-b, SM WC=9.2%
							213			
11							217			
							234			
							373			
								198.93	11.83	
12							b310		Large cobble from 11.83-12.04 m bgs. b310 blows for 0.15 m.	
	9D	61.0/35.6	12.04 - 12.65	22/16/27/24	43	45			Brown, wet, dense, fine to coarse Sandy GRAVEL, trace silt.	G#209919 A-1-a, GP-GM WC=9.0%
							aRC		aRoller Coned ahead to 13.11 m bgs.	
13								197.95	12.80 Top of Bedrock at Elev. 197.95 m.	
	R1	152.4/149.9	13.11 - 14.63	RQD = 85%			NQ CORE		Bedrock: Black, white and grey, coarse grained, metamorphic, GNEISS, hard, fresh, with banding at 70 degrees. Rock Mass Quality = Good. R1:Core Times (min:sec) 600-700 psi down pressure 13.11-13.41 m (2:30) 13.41-13.72 m (2:40) 13.72-14.02 m (3:04) 14.02-14.33 m (3:07) 14.33-14.63 m (3:23) 98% Recovery	
14										
15	R2	152.4/152.4	14.63 - 16.15	RQD = 93%					R2: Rock Mass Quality = Excellent. Core Times (min:sec) 14.63-14.94 m (2:46) 14.94-15.24 m (2:36) 15.24-15.54 m (2:34) 15.54-15.85 m (2:00)	

Remarks:

Auto Hammer #283

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
METRIC UNITS

Project:	Wild River Bridge #2948 over Wild River, Route 2	**Location:**	Gilead, Maine	**Boring No.:**	BB-GWR-101
				PIN:	15619.00
Driller:	Northern Test Boring	**Elevation (m):**	210.75	**Auger ID/OD:**	5" Solid Stem
Operator:	Mike/Nick	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/7/08-4/8/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	3+410, CL	**Casing ID/OD:**	HW	**Water Level*:**	2.44 m bgs.
Hammer Efficiency Factor:	0.633	**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 Walled Tube Sample attempt
V = Insitu Vane Shear Test
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 64 kg hammer
WOR/C = weight of rods or casing
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (kPa)
T_v = Pocket Torvane Shear Strength (kPa)
q_p = Unconfined Compressive Strength (Pa)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer effeciency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (kPa)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (m)	Sample No.	Pen/Rec (cm)	Sample Depth (m)	Blows (150 mm) Shear Strength (kPa) or RQD (%)	N-value	N_{60}	Casing Blows	Elevation (m)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
16										15.85-16.15 m (2:20) 100% Recovery	
								194.60		16.15	
										Bottom of Exploration at 16.15 m below ground surface.	
17											
18											
19											
20											
21											
22											
23											

Remarks:

Auto Hammer #283

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

METRIC UNITS

Project: Wild River Bridge #2948 over Wild River, Route 2

Location: Gilead, Maine

Boring No.: BB-GWR-102

PIN: 15619.00

Driller:	Northern Test Boring	**Elevation (m):**	208.39	**Auger ID/OD:**	5" Solid Stem
Operator:	Mike/Nick	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	9/15,16,18,19/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	3+450, CL	**Casing ID/OD:**	HW & NW	**Water Level*:**	River Boring
Hammer Efficiency Factor:	0.633	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:

D = Split Spoon Sample	R = Rock Core Sample	S_u = Insitu Field Vane Shear Strength (kPa)	$S_{u(lab)}$ = Lab Vane Shear Strength (kPa)
MD = Unsuccessful Split Spoon Sample attempt	SSA = Solid Stem Auger	T_v = Pocket Torvane Shear Strength (kPa)	WC = water content, percent
U = Thin Wall Tube Sample	HSA = Hollow Stem Auger	q_p = Unconfined Compressive Strength (Pa)	LL = Liquid Limit
MU = Unsuccessful Thin Walled Tube Sample attempt	RC = Roller Cone	N-uncorrected = Raw field SPT N-value	PL = Plastic Limit
V = Insitu Vane Shear Test	WOH = weight of 64 kg hammer	Hammer Efficiency Factor = Annual Calibration Value	PI = Plasticity Index
MV = Unsuccessful Insitu Vane Shear Test attempt	WOR/C = weight of rods or casing	N_{60} = SPT N-uncorrected corrected for hammer effeciency	G = Grain Size Analysis
	WO1P = Weight of one person	N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected	C = Consolidation Test

Depth (m)	Sample No.	Pen/Rec (cm)	Sample Depth (m)	Blows (150 mm) Shear Strength (kPa) or RQD (%)	N-value	N_{60}	Casing Blows	Elevation (m)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	27.4/27.4	0.00 - 0.27	10/50(120)	---		SSA			Brown, wet, very dense, fine to coarse Sandy GRAVEL, trace silt, with cobbles and boulders.	
								207.47		0.91	
1											
	2D	61.0/35.6	1.52 - 2.13	3/6/9/10	15	16	132			Brown, wet, medium dense, fine to coarse SAND, some silt, trace gravel. Roller Coned ahead to 2.74 m bgs.	G#208757 A-2-4, SM WC=26.3%
							152				
2											
							170				
							156				
	3D	61.0/25.4	2.74 - 3.35	10/11/12/9	23	24	148			Brown, wet, medium dense Gravelly fine to coarse SAND, trace silt. Roller Coned ahead to 4.27 m bgs.	G#208758 A-1-a, SW WC=14.3%
3											
							148				
							112				
							122				
4							122				
	4D	61.0/22.9	4.27 - 4.88	4/4/21/20	25	26	19			Brown, wet, medium dense, fine to coarse SAND, some gravel, trace silt. Roller Coned ahead to 5.18 m bgs.	G#208759 A-3, SP WC=15.7%
							91				
5							136				
							150				
								202.90		5.49	
							167				
	5D	61.0/38.1	5.79 - 6.40	16/18/16/15	34	36	128			Brown, wet, dense, fine to coarse Sandy GRAVEL, trace silt.	G#208760 A-1-a, GW WC=8.7%
6											
							140				
							145				
							139				
7											
							152				
	6D	61.0/22.9	7.32 - 7.92	17/12/11/15	23	24	131			Similar to above, medium dense.	
							105				

Remarks:

Auto Hammer #283
0.3 m water at boring location.

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
METRIC UNITS

Project: Wild River Bridge #2948 over Wild River, Route 2
Location: Gilead, Maine

Boring No.: BB-GWR-102
PIN: 15619.00

Driller:	Northern Test Boring	**Elevation (m):**	208.39	**Auger ID/OD:**	5" Solid Stem
Operator:	Mike/Nick	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	9/15,16,18,19/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	3+450, CL	**Casing ID/OD:**	HW & NW	**Water Level*:**	River Boring
Hammer Efficiency Factor:	0.633	**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 Walled Tube Sample attempt
V = Insitu Vane Shear Test
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 64 kg hammer
WOR/C = weight of rods or casing
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (kPa)
T_v = Pocket Torvane Shear Strength (kPa)
q_p = Unconfined Compressive Strength (Pa)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer effeciency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (kPa)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (m)	Sample No.	Pen/Rec (cm)	Sample Depth (m)	Blows (150 mm) Shear Strength (kPa) or RQD (%)	N-value	N_{60}	Casing Blows	Elevation (m)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
8							140				
							143				
							183				
	7D	61.0/40.6	8.84 - 9.45	10/11/20/19	31	33	108			Brown, wet, dense, Gravelly fine to coarse SAND, trace silt.	G#208761 A-1-a, SW WC=10.6%
9							230				
							216				
10							173				
							182				
	8D	61.0/35.6	10.36 - 10.97	5/5/4/5	9	9	127	198.03		10.36 Brown, wet, loose, fine to coarse SAND, little gravel, trace silt.	G#208762 A-1-b, SP WC=15.3%
							146				
11							184				
							234				
							268				
12	9D	61.0/33.0	11.89 - 12.50	7/7/7/10	14	15	127			Similar to above, but medium dense.	G#208763 A-1-b, SP WC=19.0%
							192				
							226				
13							226	195.59		12.80	
							311				
	10D	61.0/30.5	13.41 - 14.02	9/14/10/7	24	25	146			Brown, wet, medium dense, fine to coarse Sandy GRAVEL, trace silt.	G#208764 A-1-a, GW-GM WC=4.9%
14							256				
							261				
							312				
							309				
15	MD	61.0/0.0	14.94 - 15.54	18/21/14/12	35	37	184			Failed sample attempt.	
							252				

Remarks:

Auto Hammer #283
0.3 m water at boring location.

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
METRIC UNITS

Project: Wild River Bridge #2948 over Wild River, Route 2
Location: Gilead, Maine

Boring No.: BB-GWR-102
PIN: 15619.00

Driller:	Northern Test Boring	**Elevation (m):**	208.39	**Auger ID/OD:**	5" Solid Stem
Operator:	Mike/Nick	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	9/15,16,18,19/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	3+450, CL	**Casing ID/OD:**	HW & NW	**Water Level*:**	River Boring
Hammer Efficiency Factor:	0.633	**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 Walled Tube Sample attempt
V = Insitu Vane Shear Test
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 64 kg hammer
WOR/C = weight of rods or casing
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (kPa)
T_v = Pocket Torvane Shear Strength (kPa)
q_p = Unconfined Compressive Strength (Pa)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer effeciency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (kPa)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (m)	Sample No.	Pen/Rec (cm)	Sample Depth (m)	Blows (150 mm) Shear Strength (kPa) or RQD (%)	N-value	N_{60}	Casing Blows	Elevation (m)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
							309				
16							347				
							400			Boulder from 16.12-16.46 m bgs.	
	R1	396.2/152.4	16.46 - 20.42				a275 NQ-2			aChanged/telescoped to NW Casing. Roller Coned ahead with H Roller Cone to 22.56 m bgs. R1:Cobbles and Boulders. R1:Core Times (min:sec) 16.46-16.76 m (3:01) 16.76-17.07 m (1:27) 17.07-17.37 m (1:10) 17.37-17.68 m (1:22) 17.68-17.98 m (3:05) 17.98-18.29 m (2:06) 18.29-18.59 m (0:41) 18.59-18.9 m (0:40) 18.9-19.2 m (0:52) 19.2-19.51 m (0:53) 19.51-19.8 m (0:29) 19.8-20.12 m (0:26) 20.12-20.42 m (0:45) 38% Recovery	
17							130				
							110				
							164				
18							387				
							121				
							99				
19							158				
							264				
							247				
							162				
20							196				
							161				
							209				
21							432				
							121				
							112				
22							194				
							201				
							336				
	11D	30.5/25.4	22.56 - 22.86	26/50	---		96			Brown, wet, very dense, fine to coarse SAND, trace silt, trace gravel. Roller Coned ahead every 1.52 m. Boulder from 22.68-23.32 m bgs.	G#208765 A-3, SP-SM WC=19.3%
23							205				
							792				

Remarks:

Auto Hammer #283
0.3 m water at boring location.

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
METRIC UNITS

Project: Wild River Bridge #2948 over Wild River, Route 2
Location: Gilead, Maine

Boring No.: BB-GWR-102
PIN: 15619.00

Driller:	Northern Test Boring	**Elevation (m):**	208.39	**Auger ID/OD:**	5" Solid Stem
Operator:	Mike/Nick	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	9/15,16,18,19/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	3+450, CL	**Casing ID/OD:**	HW & NW	**Water Level*:**	River Boring
Hammer Efficiency Factor:	0.633	**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 Walled Tube Sample attempt
V = Insitu Vane Shear Test
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 64 kg hammer
WOR/C = weight of rods or casing
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (kPa)
T_v = Pocket Torvane Shear Strength (kPa)
q_p = Unconfined Compressive Strength (Pa)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer effeciency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (kPa)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (m)	Sample No.	Pen/Rec (cm)	Sample Depth (m)	Blows (150 mm) Shear Strength (kPa) or RQD (%)	N-value	N_{60}	Casing Blows	Elevation (m)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
							408				
							b250			b250 blows for 0.03 m.	
24	12D	42.7/38.1	24.08 - 24.51	41/60/50(75)	---					Brown, wet, very dense, fine to coarse SAND, little silt, trace gravel.	G#208766 A-2-4, SM WC=15.1%
										Cobble from 24.5-24.69 m bgs. Roller Coned ahead to 25.97 m bgs., Hit Bedrock at 25.51 m bgs.	
25											
								182.88		25.51 Top of Bedrock at Elev. 182.88 m.	
26	R2	152.4/144.8	25.97 - 27.49	RQD = 95%			NQ-2			Bedrock: Black, white and grey, coarse grained, metamorphic, GNEISS, hard, fresh, with banding at 70 degrees. Rock Mass Quality = Excellent. R2:Core Times (min:sec) 25.97-26.27 m (1:45) 26.27-26.58 m (2:31) 26.58-26.88 m (3:06) 26.88-27.19 m (3:45) 27.19-27.49 m (4:20) 95% Recovery	
27											
								180.90		27.49 **Bottom of Exploration at 27.49 m below ground surface.** Bent Casing and bottom of Drive Shoe, could not get back down hole.	
28											
29											
30											
31											

Remarks:

Auto Hammer #283
0.3 m water at boring location.

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

METRIC UNITS

Project: Wild River Bridge #2948 over Wild River, Route 2
Location: Gilead, Maine

Boring No.: BB-GWR-103
PIN: 15619.00

Driller:	Northern Test Boring	**Elevation (m):**	209.05	**Auger ID/OD:**	5" Solid Stem
Operator:	Mike/Nick	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/20-21/08, 4/8/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	3+489, CL	**Casing ID/OD:**	HW	**Water Level*:**	0.97 m bgs.
Hammer Efficiency Factor:	0.633	**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 Walled Tube Sample attempt
V = Insitu Vane Shear Test
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 64 kg hammer
WOR/C = weight of rods or casing
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (kPa)
T_v = Pocket Torvane Shear Strength (kPa)
q_p = Unconfined Compressive Strength (Pa)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer effeciency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (kPa)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (m)	Sample No.	Pen/Rec (cm)	Sample Depth (m)	Blows (150 mm) Shear Strength (kPa) or RQD (%)	N-value	N_{60}	Casing Blows	Elevation (m)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	40.6/10.2	0.00 - 0.41	7/11/50(100 mm)	---		SSA			Brown, wet, dense, fine to coarse SAND, some gravel, little silt, trace organics. Cobble from 0.4-0.46 m bgs.	
1											
	2D	61.0/43.2	1.52 - 2.13	2/10/18/33	28	30				Brown, wet, medium dense, fine to coarse Sandy GRAVEL, trace silt, occasional cobble.	G#210091 A-1-a, GW-GM WC=11.3%
2											
3											
	3D	61.0/45.7	3.05 - 3.66	17/38/36/30	74	78	101	205.85		3.20 Brown, wet, very dense, fine to coarse SAND, some gravel, trace silt, occasional cobble.	
							304				
							206				
4							138				
							118				
	4D	61.0/35.6	4.57 - 5.18	10/13/23/23	36	38	59			Brown, wet, dense, fine to coarse SAND, some gravel, trace silt. Roller Coned ahead to 6.1 m bgs.	G#210092 A-1-b, SW-SM WC=12.9%
5							103				
							134				
							156				
							147				
6	5D	61.0/33.0	6.10 - 6.71	13/15/15/17	30	32	73			Similar to above. Roller Coned ahead to 7.62 m bgs.	
							133				
							213				
7							166				
							135				
	6D	61.0/33.0	7.62 - 8.23	10/10/13/16	23	24	89			Brown, wet, medium dense, fine to medium SAND, trace gravel,	G#210093

Remarks:

Auto Hammer #283

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 METRIC UNITS	Project: Wild River Bridge #2948 over Wild River, Route 2 Location: Gilead, Maine	Boring No.: BB-GWR-103 PIN: 15619.00

Driller:	Northern Test Boring	**Elevation (m):**	209.05	**Auger ID/OD:**	5" Solid Stem
Operator:	Mike/Nick	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/20-21/08, 4/8/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	3+489, CL	**Casing ID/OD:**	HW	**Water Level*:**	0.97 m bgs.
Hammer Efficiency Factor:	0.633	**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 Walled Tube Sample attempt
V = Insitu Vane Shear Test
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 64 kg hammer
WOR/C = weight of rods or casing
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (kPa)
T_v = Pocket Torvane Shear Strength (kPa)
q_p = Unconfined Compressive Strength (Pa)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer effeciency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (kPa)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (m)	Sample No.	Pen/Rec (cm)	Sample Depth (m)	Blows (150 mm) Shear Strength (kPa) or RQD (%)	N-value	N_{60}	Casing Blows	Elevation (m)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
										trace coarse sand, trace silt.	A-3, SP-SM
8							64			Roller Coned ahead to 9.14 m bgs.	WC=20.2%
							210				
								200.52		8.53	
							318				
							168				
9											
	7D	51.8/25.4	9.14 - 9.66	12/14/12/30(60)	26	27	131			Brown, wet, medium dense, fine to coarse Sandy GRAVEL, occasional cobbles. Roller Coned ahead to 10.67 m bgs.	
							148				
							462				
10							380				
							264				
	8D	61.0/35.6	10.67 - 11.28	8/26/17/12	43	45	187			Brown, wet, dense, fine to coarse Sandy GRAVEL, trace silt. Roller Coned ahead to 12.19 m bgs.	G#210094 A-1-a, GW WC=9.6%
11							240				
							369				
							315				
12							316				
								196.86		12.19	
	9D	61.0/38.1	12.19 - 12.80	40/11/13/13	24	25	135			Brown, wet, medium dense, fine to coarse SAND, some gravel, trace silt. Roller Coned ahead to 13.72 m bgs.	
							26				
							43				
13							48				
							54				
	10D	51.8/38.1	13.72 - 14.23	26/17/22/30(60)	39	41	168			Brown, wet, dense, fine to coarse SAND, some gravel, trace silt.	G#210095 A-1-b, SW-SM WC=11.2%
14							215			Cobble ftom 14.23-14.33 m bgs.	
							331				
							353				
15							487				
	11D	61.0/33.0	15.24 - 15.85	14/13/11/11	24	25	217			Brown, wet, medium dense, fine to coarse SAND, little gravel, trace silt.	

Remarks:

Auto Hammer #283

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

METRIC UNITS

Project: Wild River Bridge #2948 over Wild River, Route 2
Location: Gilead, Maine

Boring No.: BB-GWR-103

PIN: 15619.00

Driller:	Northern Test Boring	**Elevation (m):**	209.05	**Auger ID/OD:**	5" Solid Stem
Operator:	Mike/Nick	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/20-21/08, 4/8/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	3+489, CL	**Casing ID/OD:**	HW	**Water Level*:**	0.97 m bgs.
Hammer Efficiency Factor:	0.633	**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 Walled Tube Sample attempt
V = Insitu Vane Shear Test
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 64 kg hammer
WOR/C = weight of rods or casing
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (kPa)
T_v = Pocket Torvane Shear Strength (kPa)
q_p = Unconfined Compressive Strength (Pa)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer effeciency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (kPa)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (m)	Sample No.	Pen/Rec (cm)	Sample Depth (m)	Blows (150 mm) Shear Strength (kPa) or RQD (%)	N-value	N_{60}	Casing Blows	Elevation (m)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
							267			Roller Coned ahead to 16.76 m bgs. 0.61 m running sand in casing.	
16							271				
							323				
							449				
17	12D	61.0/55.9	16.76 - 17.37	6/13/17/21	30	32	224			Brown, wet, dense, fine to coarse SAND, little gravel, trace silt. Roller Coned ahead to 18.29 m bgs.	G#210096 A-1-b, SW WC=15.4%
							295				
							262				
							258				
18							286	191.07		17.98	
	13D	48.8/38.1	18.29 - 18.78	19/27/34/30(30)	61	64	266			Brown, wet, very dense, fine to coarse SAND, some gravel, trace silt, occasional cobbles.	
							350			Cobble from 18.78-18.84 m bgs.	
19							600			Roller Coned ahead to 19.2 m then to 19.81 m bgs.	
							350				
							312				
20	14D	61.0/45.7	19.81 - 20.42	9/5/4/8	9	9	202			Brown, wet, loose, fine to coarse SAND, little gravel, trace silt, occasional cobble. Roller Coned ahead to 21.34 m bgs.	G#210097 A-1-b, SW WC=15.6%
							416				
							327				
							330				
21							424				
	15D	36.6/35.6	21.34 - 21.70	7/14/50(60)	>50		225			Brown, wet, very dense, fine to coarse SAND, some gravel, trace silt. Roller Coned ahead to 22.86 m bgs.	
							334				
22							387				
							447				
							507				
23	16D	61.0/40.6	22.86 - 23.47	25/23/16/16	39	41	237			Brown, wet, dense, fine to coarse SAND, some gravel, trace silt.	G#210098 A-1-b, SW-SM WC=12.8%
							405				

Remarks:

Auto Hammer #283

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 METRIC UNITS	**Project:** Wild River Bridge #2948 over Wild River, Route 2 **Location:** Gilead, Maine	**Boring No.:** BB-GWR-103 **PIN:** 15619.00

Driller:	Northern Test Boring	**Elevation (m):**	209.05	**Auger ID/OD:**	5" Solid Stem
Operator:	Mike/Nick	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/20-21/08, 4/8/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	3+489, CL	**Casing ID/OD:**	HW	**Water Level*:**	0.97 m bgs.
Hammer Efficiency Factor:	0.633	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:

D = Split Spoon Sample	R = Rock Core Sample	S_u = Insitu Field Vane Shear Strength (kPa)	$S_{u(lab)}$ = Lab Vane Shear Strength (kPa)
MD = Unsuccessful Split Spoon Sample attempt	SSA = Solid Stem Auger	T_v = Pocket Torvane Shear Strength (kPa)	WC = water content, percent
U = Thin Wall Tube Sample	HSA = Hollow Stem Auger	q_p = Unconfined Compressive Strength (Pa)	LL = Liquid Limit
MU = Unsuccessful Thin Walled Tube Sample attempt	RC = Roller Cone	N-uncorrected = Raw field SPT N-value	PL = Plastic Limit
V = Insitu Vane Shear Test	WOH = weight of 64 kg hammer	Hammer Efficiency Factor = Annual Calibration Value	PI = Plasticity Index
MV = Unsuccessful Insitu Vane Shear Test attempt	WOR/C = weight of rods or casing	N_{60} = SPT N-uncorrected corrected for hammer effeciency	G = Grain Size Analysis
	WO1P = Weight of one person	N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected	C = Consolidation Test

Depth (m)	Sample No.	Pen/Rec (cm)	Sample Depth (m)	Blows (150 mm) Shear Strength (kPa) or RQD (%)	N-value	N_{60}	Casing Blows	Elevation (m)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
							286				
							355				
24							437			Boulder from 24.08-24.54 m bgs.	
							408	184.51		24.54	
	17D	61.0/43.2	24.54 - 25.15	5/12/16/15	28	30	467			Brown, wet, medium dense, Gravelly fine to coarse SAND, trace silt. Roller Coned ahead to 25.91 m bgs.	G#210099 A-1-b, SW-SM WC=11.2%
25							625				
							550				
							981	183.14		25.91	
26	18D	61.0/40.6	25.91 - 26.52	11/24/28/36	52	55	450			Light brown, wet, very dense, fine to medium SAND, little silt, trace coarse sand, trace gravel.	G#210100 A-2-4, SM WC=22.1%
							500				
							478 aRC			aRoller Coned ahead to 27.74 m bgs.	
27								182.23		26.82 Soft weathered Bedrock.	
								181.31		27.74	
28	R1	152.4/152.4	27.74 - 29.26	RQD = 65%			NQ CORE			Top of intact Bedrock at Elev. 181.31 m. Bedrock: Black, white and grey, coarse grained, metamorphic, GNEISS, hard, fresh, with banding at 70 degrees. Rock Mass Quality = Fair. R1:Core Times (min:sec) 800-1000 psi down pressure 27.74-28.04 m (2:13) 28.04-28.35 m (4:18) 28.35-28.65 m (2:37) 28.65-28.96 m (1:30) 28.96-29.26 m (1:45) 100% Recovery	
29	R2	152.4/147.3	29.26 - 30.78	RQD = 87%						R2: Rock Quality = Good. Core Times (min:sec) 29.26-29.57 m (3:20) 29.57-29.87 m (3:25) 29.87-30.18 m (2:57) 30.18-30.48 m (3:09) 30.48-30.78 m (2:40) 97% Recovery	
30								178.27		30.78 **Bottom of Exploration at 30.78 m below ground surface.**	
31											

Remarks:

Auto Hammer #283

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

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

Appendix B

Laboratory Data

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Gilead   Project Number: 15619.00

Boring & Sample Identification Number	Station (Meter)	Offset (Meter)	Depth (Meter)	Reference Number	G.S.D.C. Sheet	W.C.	L.L.	P.I.	Classification Unified	AASHTO	Frost
BB-GWR-101, 1D	3+410	CL	0.0-0.61	209912	1	15.0			SP-SM	A-3	0
BB-GWR-101, 2D	3+410	CL	1.22-1.83	209913	1	2.9			GW-GM	A-1-a	0
BB-GWR-101, 3D	3+410	CL	2.74-3.3	209914	1	10.2			GW	A-1-a	0
BB-GWR-101, 5D	3+410	CL	5.79-6.4	209915	1	23.4			SM	A-2-4	II
BB-GWR-101, 6D	3+410	CL	7.32-7.92	209916	2	24.3			SM	A-2-4	II
BB-GWR-101, 7D	3+410	CL	8.84-9.45	209917	2	17.5			SP-SM	A-2-4	0
BB-GWR-101, 8D	3+410	CL	10.36-10.97	209918	2	9.2			SM	A-1-b	II
BB-GWR-101, 9D	3+410	CL	12.04-12.65	209919	2	9.0			GP-GM	A-1-a	0
BB-GWR-102, RB	3+450	CL	SURFACE	208756	3	11.8			GP	A-1-a	0
BB-GWR-102, 2D	3+450	CL	1.52-2.13	208757	3	26.3			SM	A-2-4	II
BB-GWR-102, 3D	3+450	CL	2.74-3.35	208758	3	14.3			SW	A-1-a	0
BB-GWR-102, 4D	3+450	CL	4.27-4.88	208759	3	15.7			SP	A-3	0
BB-GWR-102, 5D	3+450	CL	5.79-6.4	208760	3	8.7			GW	A-1-a	0
BB-GWR-102, 7D	3+450	CL	8.84-9.45	208761	4	10.6			SW	A-1-a	0
BB-GWR-102, 8D	3+450	CL	10.36-10.97	208762	4	15.3			SP	A-1-b	0
BB-GWR-102, 9D	3+450	CL	11.89-12.5	208763	4	19.0			SP	A-1-b	0
BB-GWR-102, 10D	3+450	CL	13.41-14.02	208764	4	4.9			GW-GM	A-1-a	0
BB-GWR-102, 11D	3+450	CL	22.56-22.86	208765	4	19.3			SP-SM	A-3	0
BB-GWR-102, 12D	3+450	CL	24.08-24.51	208766	4	15.1			SM	A-2-4	II
BB-GWR-103, 2D	3+489	CL	1.52-2.13	210091	5	11.3			GW-GM	A-1-a	0
BB-GWR-103, 4D	3+489	CL	4.57-5.18	210092	5	12.9			SW-SM	A-1-b	0
BB-GWR-103, 6D	3+489	CL	7.62-8.23	210093	5	20.2			SP-SM	A-3	0
BB-GWR-103, 8D	3+489	CL	10.67-11.28	210094	5	9.6			GW	A-1-a	0
BB-GWR-103, 10D	3+489	CL	13.72-14.23	210095	5	11.2			SW-SM	A-1-b	0
BB-GWR-103, 12D	3+489	CL	16.76-17.37	210096	6	15.4			SW	A-1-b	0
BB-GWR-103, 14D	3+489	CL	19.81-20.42	210097	6	15.6			SW	A-1-b	0
BB-GWR-103, 16D	3+489	CL	22.86-23.47	210098	6	12.8			SW-SM	A-1-b	0
BB-GWR-103, 17D	3+489	CL	24.54-25.15	210099	6	11.2			SW-SM	A-1-b	0
BB-GWR-103, 18D	3+489	CL	25.91-26.52	210100	6	22.1			SM	A-2-4	II

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

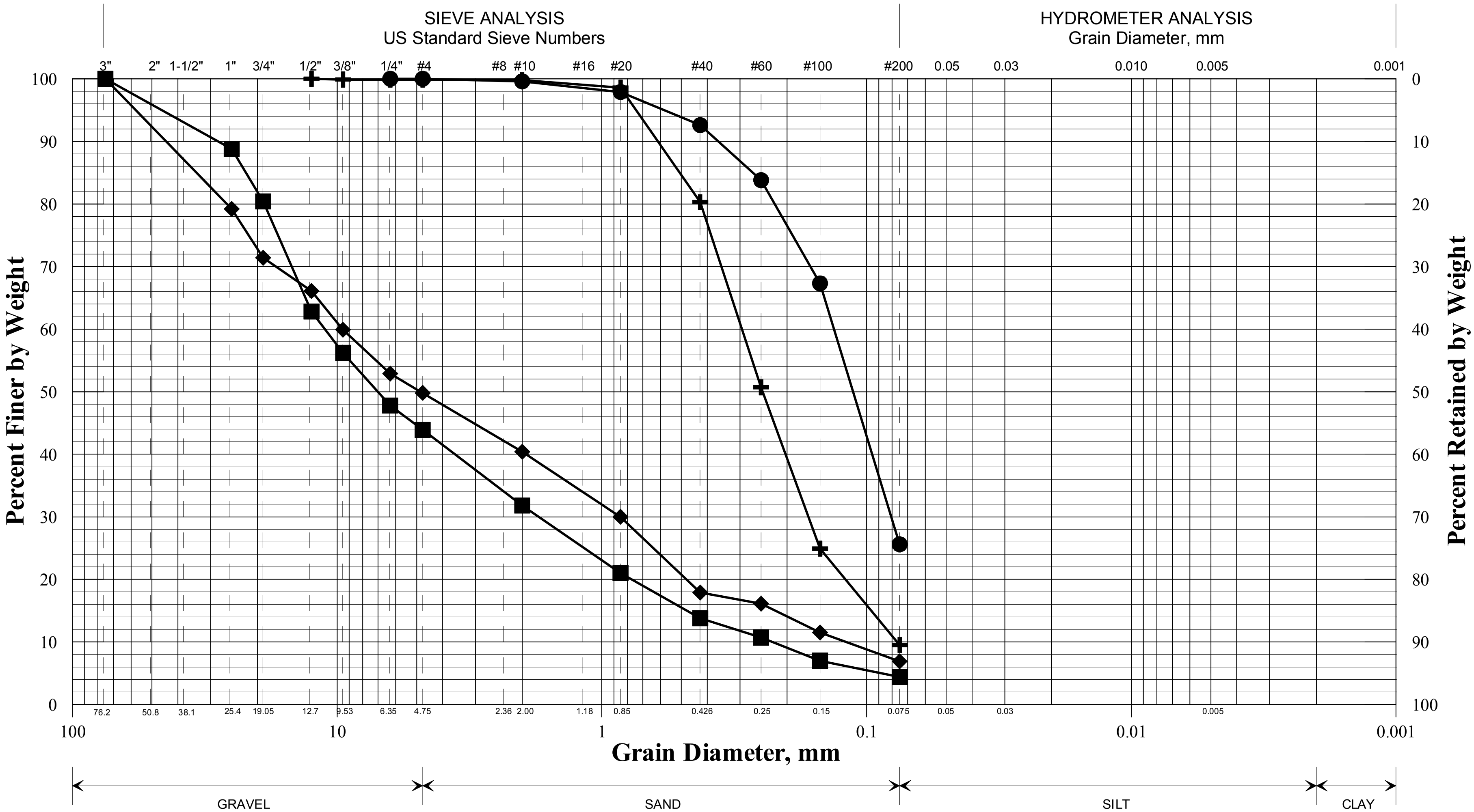
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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

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

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

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE

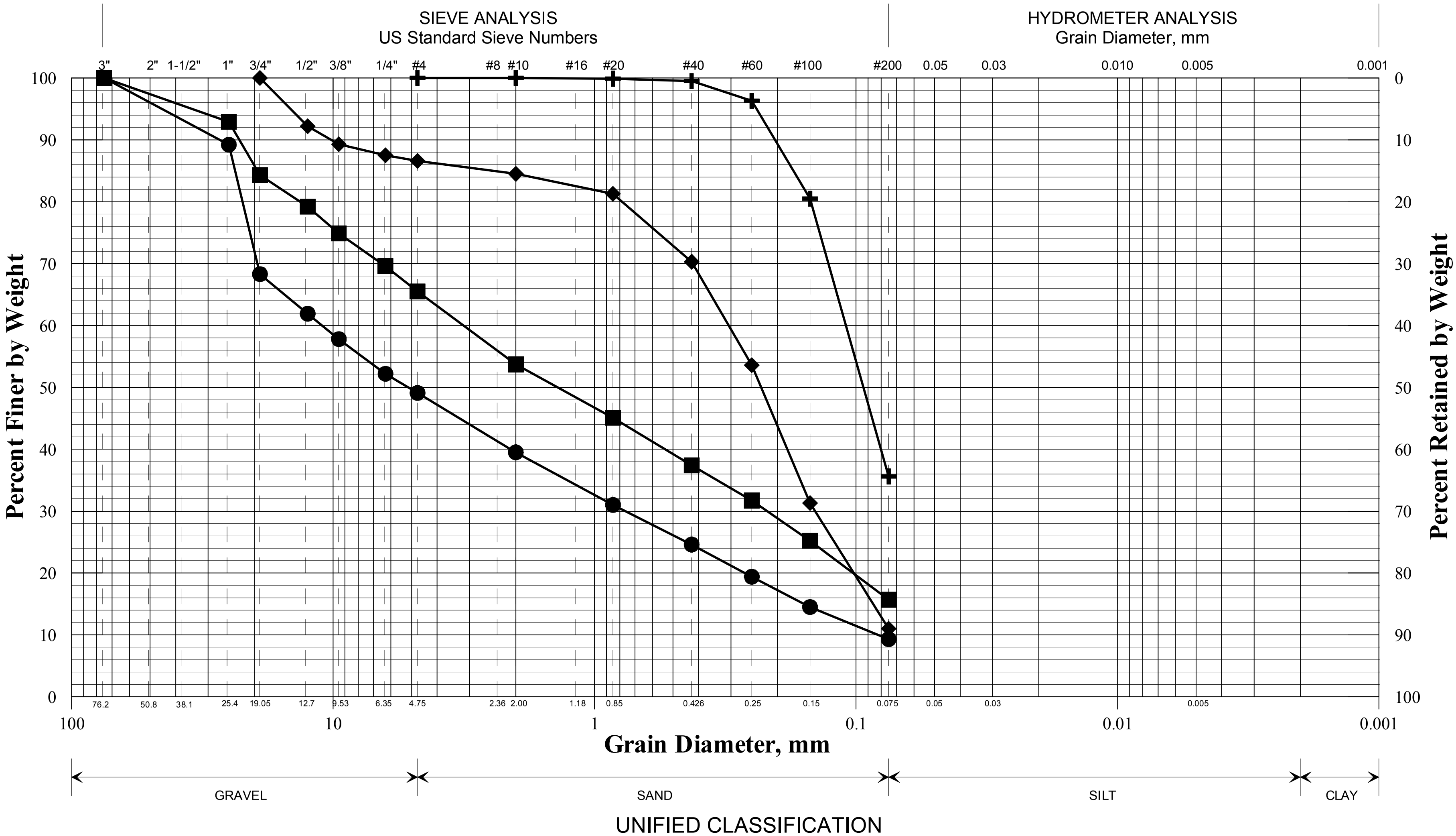


	Boring/Sample No.	Station	Offset, m	Depth, m	Description	W, %	LL	PL	PI
✚	BB-GWR-101/1D	3+410	CL	0.0-0.61	SAND, trace silt, trace gravel.	15.0			
◆	BB-GWR-101/2D	3+410	CL	1.22-1.83	Sandy GRAVEL, trace silt.	2.9			
■	BB-GWR-101/3D	3+410	CL	2.74-3.3	Sandy GRAVEL, trace silt.	10.2			
●	BB-GWR-101/5D	3+410	CL	5.79-6.4	SAND, some silt.	23.4			
▲									
✖									

PIN
015619.00
Town
Gilead
Reported by/Date
WHITE, TERRY A 5/27/2008

SHEET 1

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE

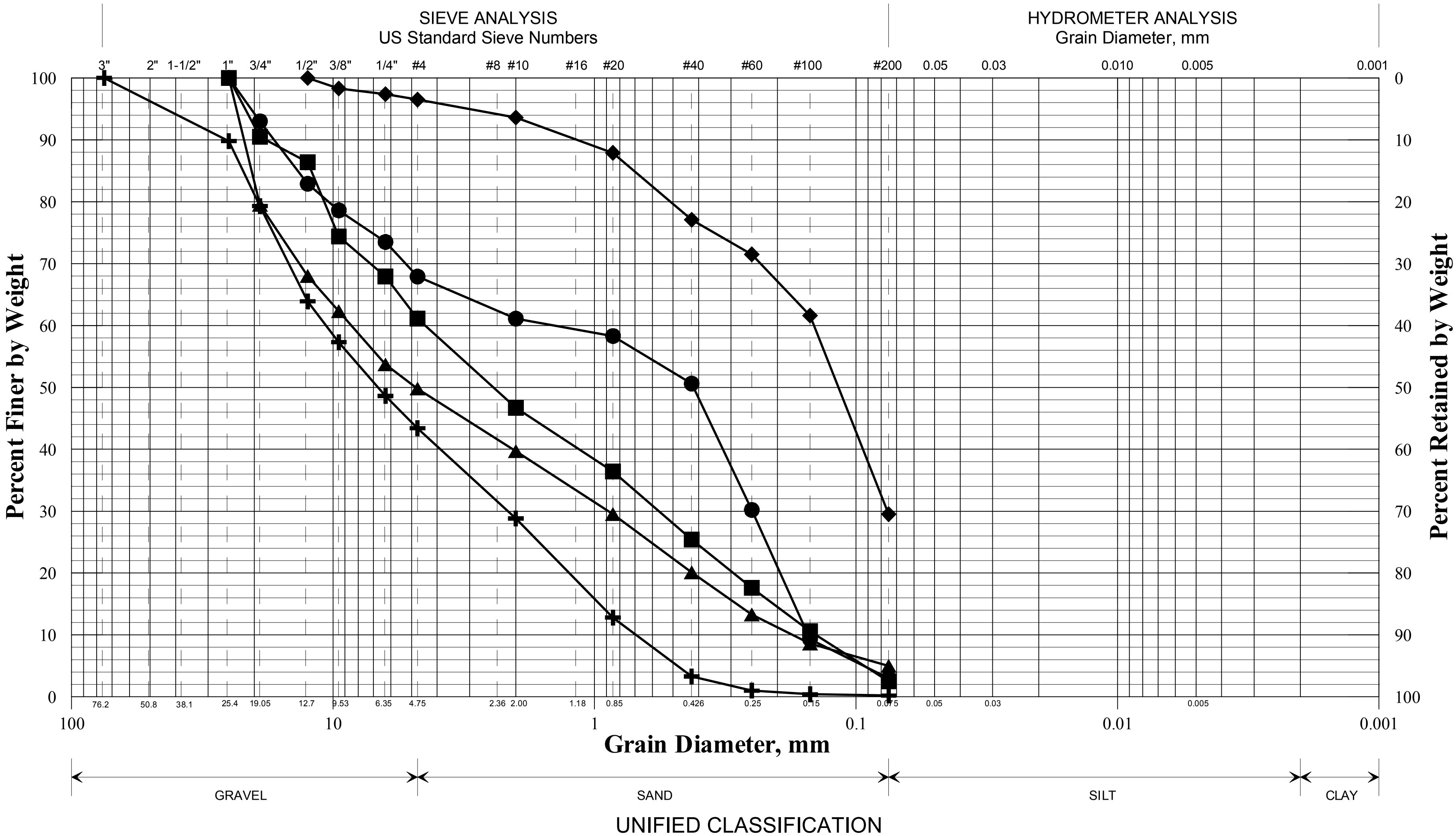


	Boring/Sample No.	Station	Offset, m	Depth, m	Description	W, %	LL	PL	PI
✚	BB-GWR-101/6D	3+410	CL	7.32-7.92	SAND, some silt.	24.3			
◆	BB-GWR-101/7D	3+410	CL	8.84-9.45	SAND, little gravel, little silt.	17.5			
■	BB-GWR-101/8D	3+410	CL	10.36-10.97	SAND, some gravel, little silt.	9.2			
●	BB-GWR-101/9D	3+410	CL	12.04-12.65	Sandy GRAVEL, trace silt.	9.0			
▲									
✖									

PIN	
015619.00	
Town	
Gilead	
Reported by/Date	
WHITE, TERRY A	5/27/2008

SHEET 2

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE

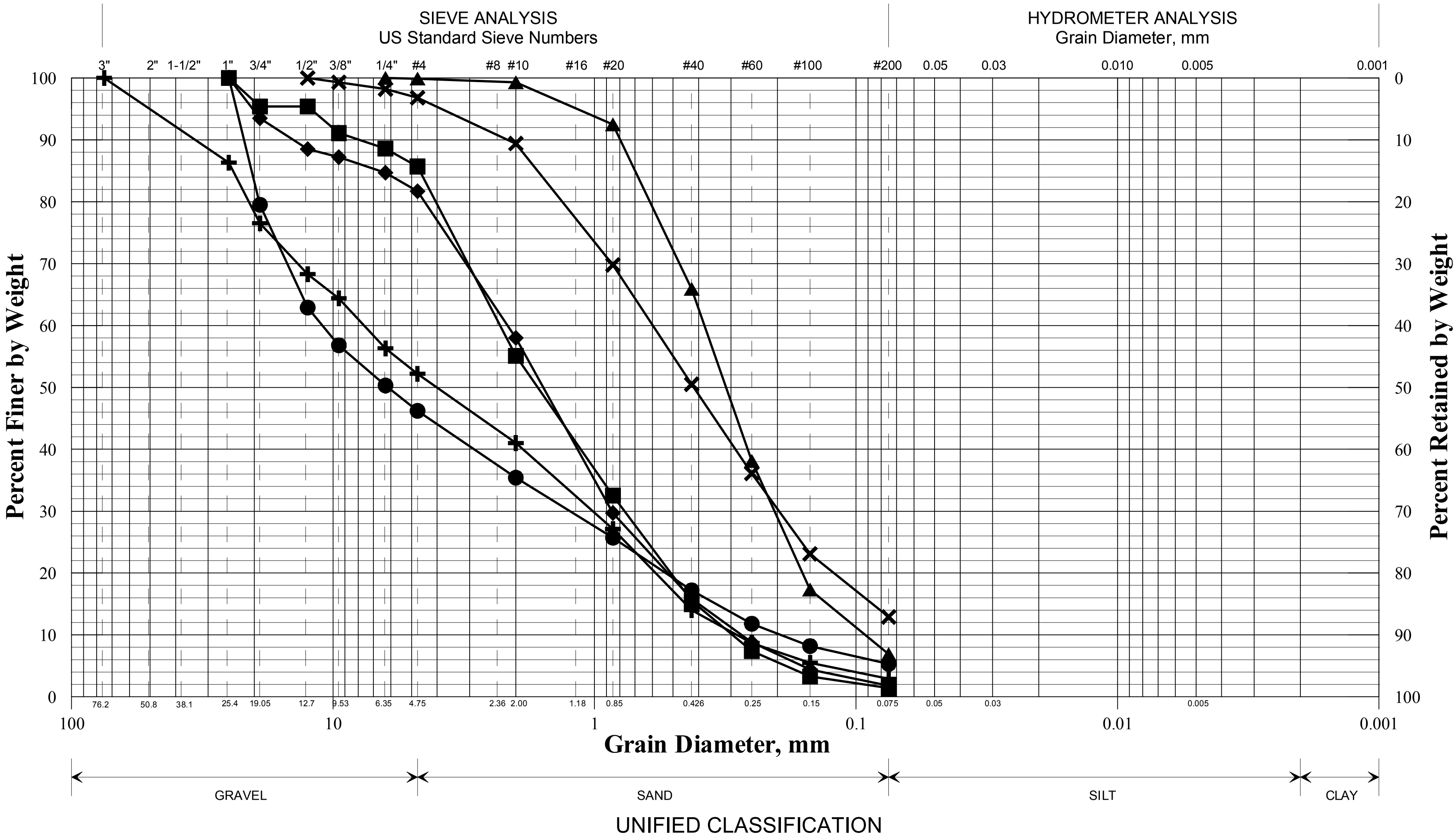


	Boring/Sample No.	Station	Offset, m	Depth, m	Description	W, %	LL	PL	PI
✚	BB-GWR-102/RIVER BED	3+450	CL	SURFACE	Sandy GRAVEL, trace silt.	11.8			
◆	BB-GWR-102/2D	3+450	CL	1.52-2.13	SAND, some silt, trace gravel.	26.3			
■	BB-GWR-102, 3D	3+450	CL	2.74-3.35	Gravelly SAND, trace silt.	14.3			
●	BB-GWR-102/4D	3+450	CL	4.27-4.88	SAND, some gravel, trace silt.	15.7			
▲	BB-GWR-102/5D	3+450	CL	5.79-6.4	Sandy GRAVEL, trace silt.	8.7			
✖									

PIN
015619.00
Town
Gilead
Reported by/Date
WHITE, TERRY A 9/29/2008

SHEET 3

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE

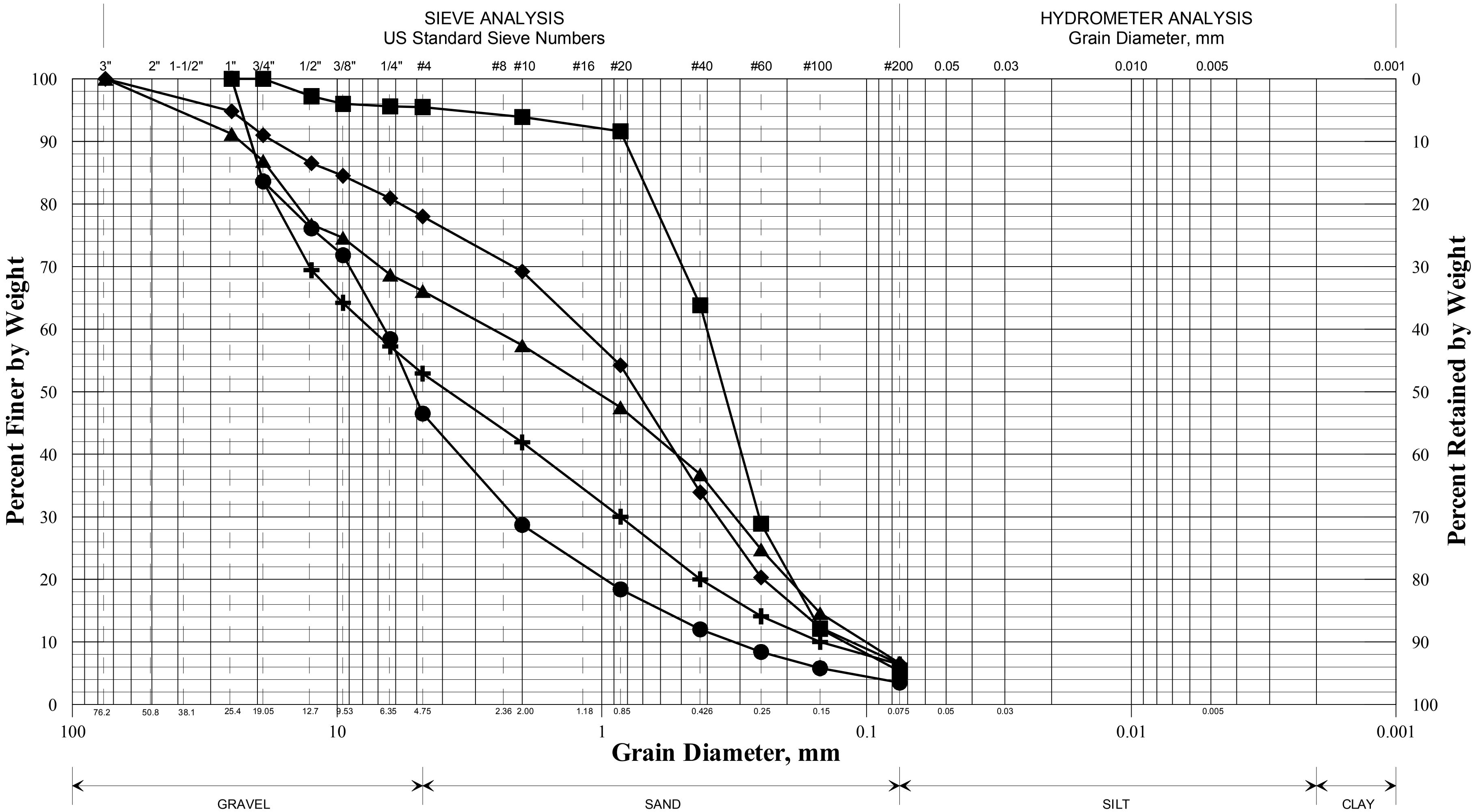


	Boring/Sample No.	Station	Offset, m	Depth, m	Description	W, %	LL	PL	PI
✚	BB-GWR-102/7D	3+450	CL	8.84-9.45	Gravelly SAND, trace silt.	10.6			
◆	BB-GWR-102/8D	3+450	CL	10.36-10.97	SAND, little gravel, trace silt.	15.3			
■	BB-GWR-102/9D	3+450	CL	11.89-12.5	SAND, little gravel, trace silt.	19.0			
●	BB-GWR-102/10D	3+450	CL	13.41-14.02	Sandy GRAVEL, trace silt.	4.9			
▲	BB-GWR-102/11D	3+450	CL	22.56-22.86	SAND, trace silt, trace gravel.	19.3			
✖	BB-GWR-102/12D	3+450	CL	24.08-24.51	SAND, little silt, trace gravel.	15.1			

PIN	
015619.00	
Town	
Gilead	
Reported by/Date	
WHITE, TERRY A	9/29/2008

SHEET 4

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



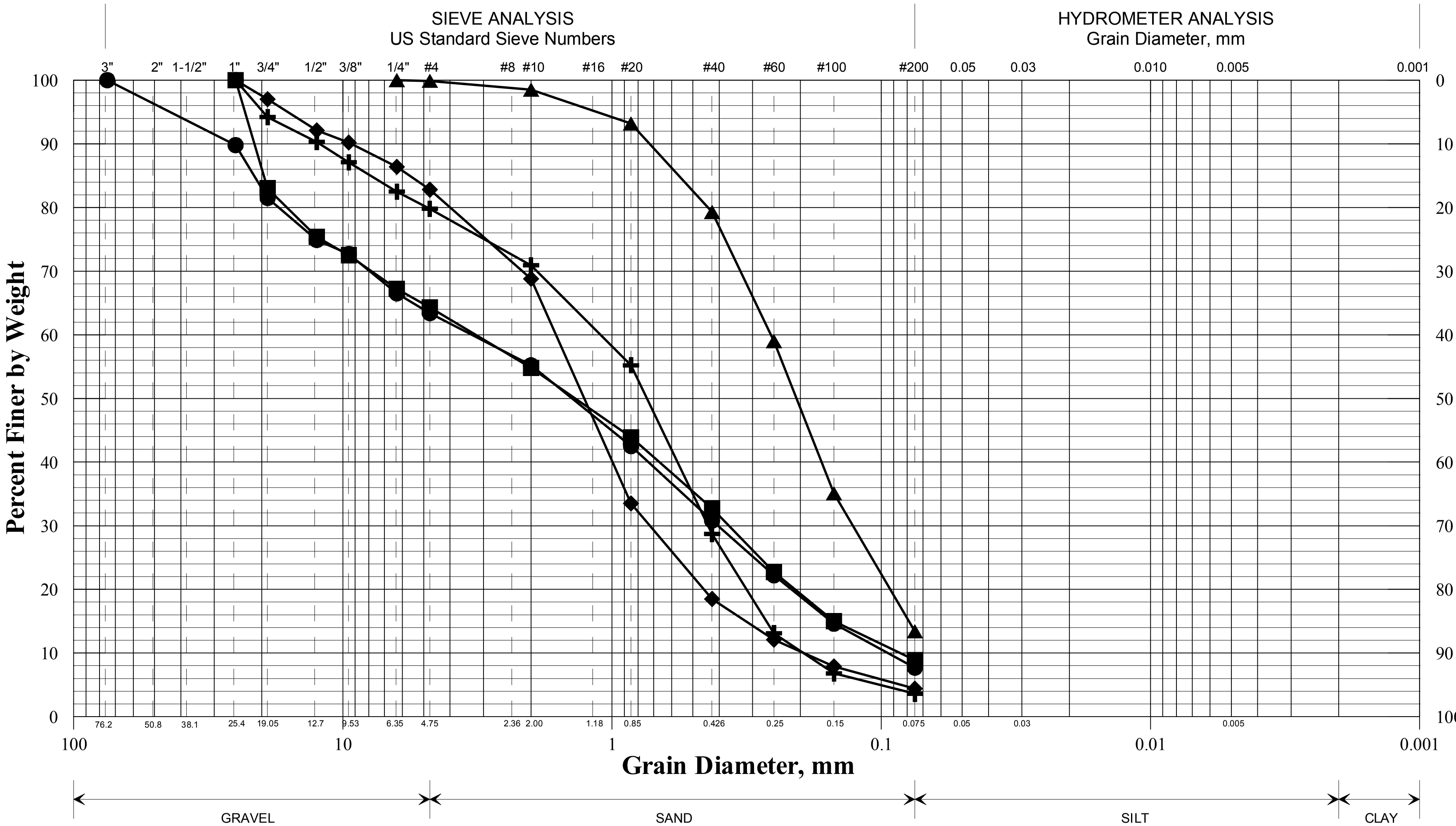
UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, m	Depth, m	Description	W, %	LL	PL	PI
✚	BB-GWR-103/2D	3+489	CL	1.52-2.13	Sandy GRAVEL, trace silt.	11.3			
◆	BB-GWR-103/4D	3+489	CL	4.57-5.18	SAND, some gravel, trace silt.	12.9			
■	BB-GWR-103/6D	3+489	CL	7.62-8.23	SAND, trace silt, trace gravel.	20.2			
●	BB-GWR-103/8D	3+489	CL	10.67-11.28	Sandy GRAVEL, trace silt.	9.6			
▲	BB-GWR-103/10D	3+489	CL	13.72-14.23	SAND, some gravel, tarce silt.	11.2			
✖									

PIN
015619.00
Town
Gilead
Reported by/Date
WHITE, TERRY A 4/29/2008

SHEET 5

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, m	Depth, m	Description	W, %	LL	PL	PI
✚	BB-GWR-103/12D	3+489	CL	16.76-17.37	SAND, little gravel, trace silt.	15.4			
◆	BB-GWR-103/14D	3+489	CL	19.81-20.42	SAND, little gravel, trace silt.	15.6			
■	BB-GWR-103/16D	3+489	CL	22.86-23.47	SAND, some gravel, trace silt.	12.8			
●	BB-GWR-103/17D	3+489	CL	24.54-25.15	Gravelly SAND, trace silt.	11.2			
▲	BB-GWR-103/18D	3+489	CL	25.91-26.52	SAND, little silt, trace gravel.	22.1			
✖									

PIN
015619.00
Town
Gilead
Reported by/Date
WHITE, TERRY A 4/29/2008

SHEET 6

<u>Appendix C</u>

Calculations

Abutment Foundations: Integral driven H-piles

Axial Structural Resistance of H-piles

Ref: AASHTO LRFD Bridge Design Specifications 4th Edition 2007

Look at the following piles:

HP 12 x 53
HP 14 x 73
HP 14 x 89
HP 14 x 117

Note: All matrices set up in this order

H-pile Steel area: $A_s := \begin{pmatrix} 15.5 \\ 21.4 \\ 26.1 \\ 34.4 \end{pmatrix} \cdot in^2$

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

Nominal Compressive Resistance $P_n=0.66^{\lambda} * F_y * A_s$: eq. 6.9.4.1-1

Where λ=normalized column slenderness factor

$\lambda=(Kl/r_s\pi)2*F_y/E$ eq. 6.9.4.1-3

$\lambda := 0$ as l = unbraced length = 0

$P_n := 0.66^{\lambda} \cdot F_y \cdot A_s$

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

HP 12 x 53
HP 14 x 73
HP 14 x 89
HP 14 x 117

STRENGTH LIMIT STATE:

Factored Resistance:

Driving conditions are assumed "severe" due to the presence of cobbles and boulders.

Strength Limit State Axial Resistance factor for piles in compression under severe driving conditions:

From Article 6.5.4.2 $\phi_c := 0.5$

Factored Compressive Resistance: eq. 6.9.2.1-1

$P_f := \phi_c \cdot P_n$

$P_f = \begin{pmatrix} 388 \\ 535 \\ 653 \\ 860 \end{pmatrix} \cdot kip$

$P_f = \begin{pmatrix} 1724 \\ 2380 \\ 2902 \\ 3825 \end{pmatrix} \cdot kN$

HP 12 x 53
HP 14 x 73
HP 14 x 89
HP 14 x 117

Strength Limit State

SERVICE/EXTREME LIMIT STATES:

Service and Extreme Limit States Axial Resistance

Nominal Compressive Resistance $P_n=0.66^{\lambda}*F_y*A_s$: eq. 6.9.4.1-1

Where λ=normalized column slenderness factor

$\lambda=(Kl/r_s\pi)2*F_y/E$ eq. 6.9.4.1-3

$\lambda := 0$ as l unbraced length is 0

$$P_n := 0.66^{\lambda} \cdot F_y \cdot A_s \qquad P_n = \begin{pmatrix} 775 \\ 1070 \\ 1305 \\ 1720 \end{pmatrix} \cdot \text{kip}$$

HP 12 x 53
HP 14 x 73
HP 14 x 89
HP 14 x 117

Resistance Factors for Service and Extreme Limit States ϕ = 1.0 LRFD 10.5.5.1 and 10.5.8.3

$\phi := 1.0$

Factored Compressive Resistance for Service and Extreme Limit States:

eq. 6.9.2.1-1

$$P_f := \phi \cdot P_n \qquad P_f = \begin{pmatrix} 775 \\ 1070 \\ 1305 \\ 1720 \end{pmatrix} \cdot \text{kip} \qquad P_f = \begin{pmatrix} 3447 \\ 4760 \\ 5805 \\ 7651 \end{pmatrix} \cdot \text{kN}$$

HP 12 x 53
HP 14 x 73
HP 14 x 89
HP 14 x 117

Service/Extreme Limit States

Geotechnical Resistance

Assume piles will be end bearing on bedrock driven through overlying sand with cobbles and boulders.

Bedrock Type:

Gneiss RQD ranges from 65 to 93%

Use RQD = 80% and ϕ = 27 to 34 deg (Tomlinson 4th Ed. pg 139)

Axial Geotechnical Resistance of H-piles

Ref: AASHTO LRFD Bridge Design Specifications 4th Edition 2007

Look at these piles:

HP 12 x 53
HP 14 x 73
HP 14 x 89
HP 14 x 117

Note: All matrices set up in this order

Steel area: $A_s = \begin{pmatrix} 15.5 \\ 21.4 \\ 26.1 \\ 34.4 \end{pmatrix} \cdot in^2$ Pile depth: $d := \begin{pmatrix} 11.78 \\ 13.61 \\ 13.83 \\ 14.21 \end{pmatrix} \cdot in$ Pile width: $b := \begin{pmatrix} 12.045 \\ 14.585 \\ 14.695 \\ 14.885 \end{pmatrix} \cdot in$

End bearing resistance of piles on bedrock - LRFD code specifies Canadian Geotech Method 1985 (LRFD Table 10.5.5.2.3-1) Canadian Foundation Manual 4th Edition (2006) Section 18.6.3.3.

Average compressive strength of rock core
from AASHTO Standard Spec for Highway Bridges 17 Ed.
Table 4.4.8.1.2B pg 64

q_u for gneiss compressive strength ranges from 3500 to 45000 psi

use $\sigma_c := 25000 \cdot psi$

Determine K_{sp}: From Canadian Foundation Manual 4th Edition (2006) Section 9.2

Spacing of discontinuities: $c := 36 \cdot in$ Assumed based on rock core

Aperture of discontinuities: $\delta := \frac{1}{64} \cdot in$ joints are tight

Footing width, b: $b = \begin{pmatrix} 12.045 \\ 14.585 \\ 14.695 \\ 14.885 \end{pmatrix} \cdot in$

HP 12 x 53
HP 14 x 73
HP 14 x 89
HP 14 x 117

$$K_{sp} := \frac{3 + \frac{c}{b}}{10 \cdot \left(1 + 300 \cdot \frac{\delta}{c}\right)^{0.5}} \qquad K_{sp} = \begin{pmatrix} 0.5633 \\ 0.5144 \\ 0.5126 \\ 0.5097 \end{pmatrix}$$

K_{sp} includes a factor of safety of 3

Length of rock socket, L_s: $L_s := 0 \cdot in$ Pile is end bearing on rock

Diameter of socket, B_s: $B_s := 1 \cdot ft$

depth factor, d_f: $d_f := 1 + 0.4\left(\frac{L_s}{B_s}\right)$ $d_f = 1$ should be < or = 3 OK

$$q_a := \sigma_c \cdot K_{sp} \cdot d_f \qquad q_a = \begin{pmatrix} 2028 \\ 1852 \\ 1845 \\ 1835 \end{pmatrix} \cdot ksf$$

Nominal Geotechnical Tip Resistance, R_p:

Multiply by 3 to take out FS=3 on K_{sp}

$$R_p := \overrightarrow{\left(3q_a \cdot A_s\right)} \qquad R_p = \begin{pmatrix} 655 \\ 826 \\ 1003 \\ 1315 \end{pmatrix} \cdot kip$$

HP 12 x 53
HP 14 x 73
HP 14 x 89
HP 14 x 117

STRENGTH LIMIT STATE:

Factored Geotechnical Resistance at Strength Limit State:

Resistance factor, end bearing on rock (Canadian Geotech. Society, 1985 method):

Nominal resistance of Single Pile in Axial Compression - Static Analysis Methods, ϕ_{stat} $\phi_{stat} := 0.45$ LRFD Table 10.5.5.2.3-1

$$R_f := \phi_{stat} \cdot R_p \qquad R_f = \begin{pmatrix} 295 \\ 372 \\ 452 \\ 592 \end{pmatrix} \cdot kip \qquad R_f = \begin{pmatrix} 1311 \\ 1653 \\ 2009 \\ 2632 \end{pmatrix} \cdot kN$$

HP 12 x 53
HP 14 x 73
HP 14 x 89
HP 14 x 117

Strength Limit State

SERVICE/EXTREME LIMIT STATES:

Factored Geotechnical Resistance at the Service/Extreme Limit States:

Resistance Factors for Service and Extreme Limit States ϕ = 1.0 LRFD 10.5.5.1 and 10.5.8.3

$\phi := 1.0$

$$R_{fse} := \phi \cdot R_p \qquad R_{fse} = \begin{pmatrix} 655 \\ 826 \\ 1003 \\ 1315 \end{pmatrix} \cdot kip \qquad R_{fse} = \begin{pmatrix} 2913 \\ 3672 \\ 4464 \\ 5849 \end{pmatrix} \cdot kN$$

HP 12 x 53
HP 14 x 73
HP 14 x 89
HP 14 x 117

Service/Extreme Limit States

DRIVABILITY ANALYSIS

Ref: LRFD Article 10.7.8

For steel piles in compression or tension
$\sigma_{dr} = 0.9 \times \phi_{da} \times f_y$ (eq. 10.7.8-1)

$f_y := 50 \cdot ksi$ yield strength of steel

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

$\sigma_{dr} := 0.9 \cdot \phi_{da} \cdot f_y$ $\sigma_{dr} = 45 \cdot ksi$ driving stresses in pile can not exceed 45 ksi

Compute Resistance that can be achieved in a drivability analysis:

The resistance that must be achieved in a drivability analysis will be the maximum applied pile axial load (must be less than the the factored geotechnical resistance from above as this governs)
divided by the appropriate resistance factor for wave equation analysis and dynamic test which will be required for construction.

Table 10.5.5.2.3-1 pg 10-38 gives resistance factor for dynamic test, ϕ_{dyn}:

$\phi_{dyn} := 0.65$

There are 5 piles at each abutment. No reduction of Φ_{dyn} is necessary.

Pile Size = 12 x 53

Assume Contractor will use a Delmag D19-42 hammer to install 12 x 53 piles

State of Maine Dept. Of Transportation — 27-Sep-2008
Gilead Wild River Bridge — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
300.0	35.00	6.34	3.8	7.73	21.58
350.0	37.75	7.42	5.5	7.97	22.24
400.0	40.37	8.17	7.9	8.29	23.08
450.0	42.24	8.73	12.5	8.45	23.57
460.0	42.53	8.72	13.8	8.48	23.59
470.0	42.90	8.79	15.0	8.52	23.74
480.0	43.20	8.80	16.6	8.55	23.80
490.0	43.47	8.87	18.5	8.58	23.88
500.0	43.75	8.94	20.7	8.60	23.95
510.0	44.01	9.00	23.3	8.63	24.02

Limited to blow count to 15 blows per inch

Strength Limit State:

$$R_{dr\_12x53\_factored} := 470 \cdot kip \cdot \phi_{dyn}$$

$$R_{dr\_12x53\_factored} = 306 \cdot kip$$

$$R_{dr\_12x53\_factored} = 1359 \cdot kN$$

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

$$R_{dr\_12x53\_servext} := 470 \cdot kip$$

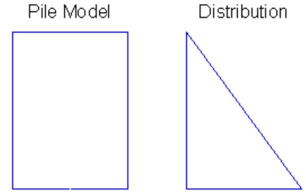
$$R_{dr\_12x53\_servext} = 2091 \cdot kN$$

DELMAG D 19-42

Efficiency	0.800
Helmet	3.20 kips
Hammer Cushion	109975 kips/in
Skin Quake	0.100 in
Toe Quake	0.040 in
Skin Damping	0.050 sec/ft
Toe Damping	0.150 sec/ft
Pile Length	110.00 ft
Pile Penetration	110.00 ft
Pile Top Area	15.50 in2

Pile Model

Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Pile Size = 14 x 73

Assume Contractor will use a Delmag D36-32 hammer on second fuel setting to install 14 x 73 piles

State of Maine Dept. Of Transportation — 27-Sep-2008
Gilead Wild River Bridge — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
570.0	43.62	6.56	6.0	7.07	39.76
580.0	44.00	6.64	6.3	7.11	40.00
590.0	44.42	6.72	6.6	7.15	40.26
600.0	44.80	6.79	7.0	7.20	40.50
605.0	45.00	6.82	7.2	7.22	40.60
610.0	45.19	6.85	7.4	7.24	40.73
620.0	45.56	6.89	7.9	7.28	40.89
630.0	45.92	6.93	8.3	7.32	41.15
640.0	46.25	6.95	8.7	7.35	41.32
650.0	46.54	6.99	9.2	7.38	41.50

Limit to driving stress to 45 ksi

Strength Limit State:

$$R_{dr\_14x73\_factored} := 605 \cdot kip \cdot \phi_{dyn}$$

$R_{dr\_14x73\_factored} = 393 \cdot kip$ $\qquad$ $R_{dr\_14x73\_factored} = 1749 \cdot kN$

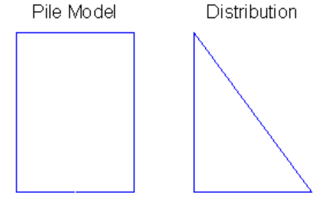
Service and Extreme Limit States: $\qquad \phi := 1.0$

$R_{dr\_14x73\_servext} := 605 \cdot kip$ $\qquad$ $R_{dr\_14x73\_servext} = 2691 \cdot kN$

DELMAG D 36-32

Efficiency	0.800
Helmet	3.20 kips
Hammer Cushion	109975 kips/in
Skin Quake	0.100 in
Toe Quake	0.040 in
Skin Damping	0.050 sec/ft
Toe Damping	0.150 sec/ft
Pile Length	110.00 ft
Pile Penetration	110.00 ft
Pile Top Area	21.40 in2

Pile Model $\qquad$ Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Pile Size = 14 x 89

Assume Contractor will use a Delmag D36-32 hammer on third fuel setting to install 14 x 89 piles

State of Maine Dept. Of Transportation — 27-Sep-2008
Gilead Wild River Bridge — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
600.0	42.43	6.17	4.7	7.62	43.27
610.0	42.55	6.24	4.9	7.59	43.10
620.0	42.95	6.29	5.1	7.63	43.26
630.0	43.34	6.36	5.3	7.67	43.51
640.0	43.66	6.42	5.6	7.70	43.68
650.0	44.00	6.48	5.8	7.74	43.86
660.0	44.37	6.55	6.1	7.78	44.12
670.0	44.74	6.59	6.3	7.82	44.27
680.0	45.05	6.63	6.6	7.85	44.44
690.0	45.36	6.67	6.9	7.88	44.63

Limit to driving stress to 45 ksi

Strength Limit State:

$$R_{dr\_14x89\_factored} := 680 \cdot kip \cdot \phi_{dyn}$$

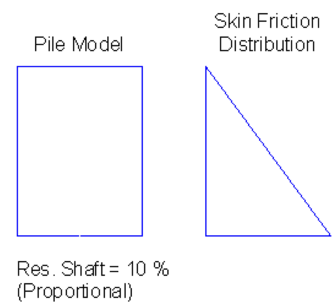
$$R_{dr\_14x89\_factored} = 442 \cdot kip \qquad R_{dr\_14x89\_factored} = 1966 \cdot kN$$

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

$$R_{dr\_14x89\_servext} := 680 \cdot kip \qquad R_{dr\_14x89\_servext} = 3025 \cdot kN$$

DELMAG D 36-32

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	110.00	ft
Pile Penetration	110.00	ft
Pile Top Area	26.10	in2



Pile Size = 14 x 117

Assume Contractor will use a Delmag D36-32 hammer on highest fuel setting to install 14 x 117 piles

State of Maine Dept. Of Transportation — 27-Sep-2008
Gilead Wild River Bridge — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
770.0	43.55	5.01	5.7	8.68	48.05
780.0	43.58	5.07	6.0	8.62	47.81
790.0	43.85	5.12	6.2	8.64	47.84
800.0	44.15	5.17	6.3	8.67	48.02
810.0	44.40	5.22	6.5	8.69	48.19
820.0	44.65	5.27	6.7	8.71	48.26
830.0	44.86	5.32	7.0	8.73	48.29
835.0	44.99	5.34	7.1	8.74	48.34
840.0	45.12	5.37	7.2	8.75	48.47
850.0	45.34	5.42	7.4	8.77	48.54

Limit to driving stress to 45 ksi

Strength Limit State:

$$R_{dr\_14x117\_factored} := 835 \cdot kip \cdot \phi_{dyn}$$

$$R_{dr\_14x117\_factored} = 543 \cdot kip \qquad R_{dr\_14x117\_factored} = 2414 \cdot kN$$

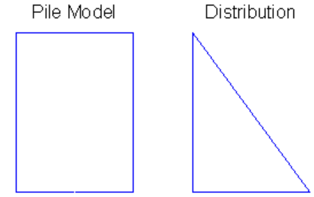
Service and Extreme Limit States: $\phi := 1.0$

$$R_{dr\_14x117\_servext} := 835 \cdot kip \qquad R_{dr\_14x117\_servext} = 3714 \cdot kN$$

DELMAG D 36-32

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	110.00	ft
Pile Penetration	110.00	ft
Pile Top Area	34.40	in2

Pile Model — Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Pipe Pile Supported Pier

Calculate Depth to Fixity for pipe piles:

Soil conditions at boring BB-GWR-102:
84 ft of fill sand, gravel, cobbles and boulders over bedrock

Consider Pile sizes:

24 in diameter 1/2 in wall
26 in diameter 1/2 in wall
28 in diameter 1/2 in wall
30 in diameter 1/2 in wall

24 in diameter 5/8 in wall
26 in diameter 5/8 in wall
28 in diameter 5/8 in wall
30 in diameter 5/8 in wall

Diameter of piles:

$$dia_{steel} := \begin{pmatrix} 24 \\ 26 \\ 28 \\ 30 \end{pmatrix} \cdot in$$

Pipe pile wall thickness:

$$wall_t := \begin{pmatrix} \frac{1}{2} \\ \frac{5}{8} \end{pmatrix} \cdot in$$

Corrosion loss per MaineDOT BDG:

$$cor := \frac{1}{8} in$$

$$dia_{steelcor} := dia_{steel} - 2 \cdot cor \qquad dia_{steelcor} = \begin{pmatrix} 23.75 \\ 25.75 \\ 27.75 \\ 29.75 \end{pmatrix} \cdot in \qquad wall_{cor} := wall_t - cor \qquad wall_{cor} = \begin{pmatrix} 0.375 \\ 0.5 \end{pmatrix} \cdot in$$

$$dia_{conccore\_0.5} := dia_{steel} - 2 \cdot \frac{1}{2} \cdot in \qquad dia_{conccore\_0.5} = \begin{pmatrix} 23 \\ 25 \\ 27 \\ 29 \end{pmatrix} \cdot in$$

Diameter concrete core for 1/2" thick wall

$$dia_{conccore\_0.625} := dia_{steel} - 2 \cdot \frac{5}{8} \cdot in \qquad dia_{conccore\_0.625} = \begin{pmatrix} 22.75 \\ 24.75 \\ 26.75 \\ 28.75 \end{pmatrix} \cdot in$$

Diameter concrete core for 5/8" thick wall

$$A_{0.5} := \pi \cdot \left(\frac{dia_{steelcor}}{2}\right)^2 - \pi \cdot \left(\frac{dia_{conccore\_0.5}}{2}\right)^2 \qquad A_{0.5} = \begin{pmatrix} 27.54 \\ 29.89 \\ 32.25 \\ 34.61 \end{pmatrix} \cdot in^2$$

STEEL AREA FOR 1/2" PILES with 1/8" corrosion loss

$$A_{0.625} := \pi \cdot \left(\frac{dia_{steelcor}}{2}\right)^2 - \pi \cdot \left(\frac{dia_{conccore\_0.625}}{2}\right)^2 \qquad A_{0.625} = \begin{pmatrix} 36.52 \\ 39.66 \\ 42.8 \\ 45.95 \end{pmatrix} \cdot in^2$$

STEEL AREA FOR 5/8" PILES with 1/8" corrosion loss

Transformed pile properties of 1/2 inch wall pile:

unit weight of concrete: $wc := 0.15$ in kips per cubic foot

compressive strength of concrete: $f_c := 4.45$ in ksi

Modulus of elasticity of concrete: $E_c := 33000 \cdot wc^{1.5} \cdot \sqrt{f_c} \cdot 1000 \cdot psi$ $\quad E_c = 4044 \cdot ksi$

Steel modulus: $E_{steel} := 29000 \cdot ksi$

$$n := \frac{E_{steel}}{E_c} \qquad n = 7.17$$

MaineDOT Structural engineers routinely use:

$$n := 7.6$$

Moment of inertia of concrete core:

$$I_{c\_0.5} := \frac{\pi \cdot dia_{conccore\_0.5}{}^4}{64} \qquad I_{c\_0.5} = \begin{pmatrix} 0.662 \\ 0.925 \\ 1.258 \\ 1.674 \end{pmatrix} ft^4$$

Moment of inertia of steel pipe:

$$I_{s\_0.5} := \frac{\pi \cdot \overrightarrow{\left(dia_{steelcor}{}^4 - dia_{conccore\_0.5}{}^4\right)}}{64} \qquad I_{s\_0.5} = \begin{pmatrix} 0.091 \\ 0.116 \\ 0.146 \\ 0.18 \end{pmatrix} ft^4$$

Composite Moment of Inertia:

$$I_{t\_0.5} := \overrightarrow{\left(\frac{I_{c\_0.5}}{n} + I_{s\_0.5}\right)} \qquad I_{t\_0.5} = \begin{pmatrix} 0.178 \\ 0.238 \\ 0.311 \\ 0.4 \end{pmatrix} ft^4$$

Transformed Area:

$$A_{conc\_0.5} := \pi \cdot \frac{dia_{conccore\_0.5}{}^2}{4} \qquad A_{conc\_0.5} = \begin{pmatrix} 415.48 \\ 490.87 \\ 572.56 \\ 660.52 \end{pmatrix} \cdot in^2$$

$$A_{t\_0.5} := A_{0.5} + \frac{A_{conc\_0.5}}{n} \qquad A_{t\_0.5} = \begin{pmatrix} 0.571 \\ 0.656 \\ 0.747 \\ 0.844 \end{pmatrix} \cdot ft^2$$

LRFD Eq.10.7.3.13.4-2 for fixity in feet: $1.8*(E_pI_w/n_h)^{0.2}$ (in sands)

E_p Young's modulus of pile in ksi

I_w moment of inertia of pile in ft^4

n_h= rate of increase of soil modulus with depth for sands as specified in Table C10.4.6.3-2 in ksi/ft

Use equation for sands in NCHRP#343 pg 61:

$L_{eq}=L_u+1.8T$ where:

L_{eq} = equivalent free standing length of pile

L_u = unsupported length of pile extending above ground

$T=(E_p*I_p/n_h)^{0.2}$

Rate of increase of soil modulus with depth: for submerged medium dense sand

$$n_h := 0.556 \cdot \frac{ksi}{ft}$$

T parameter:

$$T_{0.5} := \left(\frac{E_{steel} \cdot I_{t\_0.5}}{n_h}\right)^{0.2} \qquad T_{0.5} = \begin{pmatrix} 6.22 \\ 6.59 \\ 6.95 \\ 7.31 \end{pmatrix} ft$$

Depth of Fixity:

$$D_{fix\_0.5} := 1.8 \cdot T_{0.5}$$

$$D_{fix\_0.5} = \begin{pmatrix} 11 \\ 12 \\ 13 \\ 13 \end{pmatrix} ft$$

Depth to fixity for 1/2" wall pipe piles

$$D_{fix\_0.5} = \begin{pmatrix} 3.41 \\ 3.61 \\ 3.81 \\ 4.01 \end{pmatrix} \cdot m$$

Check with LRFD Eq. 10.7.3.13.4-2

$$E_{steel} = 29000 \cdot ksi$$

$$I_{t\_0.5} = \begin{pmatrix} 0.1779 \\ 0.2377 \\ 0.3113 \\ 0.4003 \end{pmatrix} ft^4 \qquad Check := 1.8 \cdot \left(\frac{E_{steel} \cdot I_{t\_0.5}}{n_h}\right)^{0.2} \qquad Check = \begin{pmatrix} 11.19 \\ 11.86 \\ 12.51 \\ 13.16 \end{pmatrix} ft$$

OK

Transformed pile properties of 5/8 inch wall pile:

$$n = 7.6$$

Diameter of concrete core:

$$dia_{conccore\_0.625} = \begin{pmatrix} 22.75 \\ 24.75 \\ 26.75 \\ 28.75 \end{pmatrix} \cdot in$$

Diameter concrete core for 5/8" thick wall

Diameter of steel pipe

$$dia_{steelcor} = \begin{pmatrix} 23.75 \\ 25.75 \\ 27.75 \\ 29.75 \end{pmatrix} \cdot in$$

Moment of inertia of concrete core:

$$I_{c\_0.625} := \frac{\pi \cdot dia_{conccore\_0.625}^{4}}{64} \qquad I_{c\_0.625} = \begin{pmatrix} 0.634 \\ 0.888 \\ 1.212 \\ 1.617 \end{pmatrix} ft^4$$

Moment of inertia of steel pipe:

$$I_{s\_0.625} := \frac{\pi \cdot \overrightarrow{\left(dia_{steelcor}^{4} - dia_{conccore\_0.625}^{4}\right)}}{64} \qquad I_{s\_0.625} = \begin{pmatrix} 0.119 \\ 0.152 \\ 0.192 \\ 0.237 \end{pmatrix} ft^4$$

Composite Moment of Inertia:

$$I_{t\_0.625} := \frac{I_{c\_0.625}}{n} + I_{s\_0.625} \qquad I_{t\_0.625} = \begin{pmatrix} 0.202 \\ 0.269 \\ 0.351 \\ 0.45 \end{pmatrix} ft^4$$

Transformed Area:

$$A_{conc\_0.625} := \pi \cdot \frac{dia_{conccore\_0.625}^{2}}{4}$$

$$A_{conc\_0.625} = \begin{pmatrix} 406.49 \\ 481.11 \\ 562 \\ 649.18 \end{pmatrix} \cdot in^2$$

$$A_{t\_0.625} := A_{0.625} + \frac{A_{conc\_0.625}}{n} \qquad A_{t\_0.625} = \begin{pmatrix} 0.625 \\ 0.715 \\ 0.811 \\ 0.912 \end{pmatrix} \cdot ft^2$$

LRFD Eq.10.7.3.13.4-2 for fixity in feet: $1.8*(E_p I_w/n_h)^{0.2}$ (in sands)

E_p Young's modulus of pile in ksi

I_w moment of inertia of pile in ft^4

n_h= rate of increase of soil modulus with depth for sands as specified in Table C10.4.6.3-2 in ksi/ft

Use equation for sands in NCHRP#343 pg 61:

$L_{eq}=L_u+1.8T$ where:

L_{eq} = equivalent free standing length of pile

L_u = unsupported length of pile extending above ground

$T=(E_p*I_p/n_h)^{0.2}$

Rate of increase of soil modulus with depth: for submerged medium dense sand

$$n_h := 0.556 \cdot \frac{ksi}{ft}$$

T parameter:

$$T_{0.625} := \left(\frac{E_{steel} \cdot I_{t\_0.625}}{n_h}\right)^{0.2} \qquad T_{0.625} = \begin{pmatrix} 6.38 \\ 6.75 \\ 7.12 \\ 7.48 \end{pmatrix} ft$$

Depth of Fixity:

$$D_{fix\_0.625} := 1.8 \cdot T_{0.625}$$

$$D_{fix\_0.625} = \begin{pmatrix} 11 \\ 12 \\ 13 \\ 13 \end{pmatrix} ft$$

Depth to fixity for 5/8" wall pipe piles

$$D_{fix\_0.625} = \begin{pmatrix} 3.5 \\ 3.71 \\ 3.91 \\ 4.11 \end{pmatrix} \cdot m$$

Check with LRFD Eq. 10.7.3.13.4-2

$$E_{steel} = 29000 \cdot ksi$$

$$I_{t\_0.625} = \begin{pmatrix} 0.2025 \\ 0.2694 \\ 0.3512 \\ 0.4498 \end{pmatrix} ft^4 \qquad Check := 1.8 \cdot \left(\frac{E_{steel} \cdot I_{t\_0.625}}{n_h}\right)^{0.2} \qquad Check = \begin{pmatrix} 11.48 \\ 12.16 \\ 12.82 \\ 13.47 \end{pmatrix} ft$$

OK

Nominal Axial Structural Resistance of pipe piles

Ref: AASHTO LRFD Bridge Design Specifications 4th Edition 2007

Pier - Pipe Pile driven to bedrock, assume driven through cohesionless soils to bedrock (refusal)

Axial pile resistance may be controlled by structural resistance if piles are driven to bedrock.
Check concurrent axial loading and moments with LRFD Equation 6.9.2.2-1 or 6.9.2.2-2.
Use LRFD Equation 6.9.5.1-1 or 6.9.5.1-2 to compute the nominal compressive structural resistance for pipe pile sections.

λ in Equation 6.9.5.1-2 has to be computed for the pipe piles since they have an unbraced length.

Yield strength of steel shell: $F_y := 45 \cdot ksi$

Compressive strength of concrete core: $f_c := 4000 \cdot psi$

Yield strength of longitudinal reinforcement: $F_{yr} := 60 \cdot ksi$

Assume unsupported length is from bottom of curtain wall including 14 feet of scour plus depth to fixity.

Compute λ per 6.9.5.1-3 for composite members:

Effective length factor per LRFD Article 4.6.2.5:

Use case (c) in table C4.6.2.5-1

$K := 1.0$ Because piles are fixed at the end

Exposed length of pile:

Scour depth calculated to be approximately 14 feet

$L_{ex} := 14 \cdot ft$

Unbraced length of column:

$$L_{UB\_0.5} := L_{ex} + D_{fix\_0.5} \qquad L_{UB\_0.5} = \begin{pmatrix} 25.19 \\ 25.86 \\ 26.51 \\ 27.16 \end{pmatrix} ft$$

$$L_{UB\_0.625} := L_{ex} + D_{fix\_0.625} \qquad L_{UB\_0.625} = \begin{pmatrix} 25.48 \\ 26.16 \\ 26.82 \\ 27.47 \end{pmatrix} ft$$

Longitudinal reinforcement:

Assume longitudinal reinforcement of 12 - #8 bars (1-inch) bars equally spaced for all pile sections.

$$A_r := 12 \cdot \frac{\pi \cdot (1 \cdot in)^2}{4} \qquad A_r = 9.42 \cdot in^2$$

Composite Column Constant per Table 6.9.5.1.1

for tube filled sections: $C1 := 1.0$ $C2 := 0.85$ $C3 := 0.40$

Variable Fe:

$$F_{e\_0.5} := F_y + C1 \cdot F_{yr} \cdot \frac{A_r}{A_{0.5}} + C2 \cdot f_c \cdot \frac{A_{conc\_0.5}}{A_{0.5}} \qquad F_{e\_0.5} = \begin{pmatrix} 116.83 \\ 119.75 \\ 122.9 \\ 126.23 \end{pmatrix} \cdot ksi$$

for 1/2" walls

$$F_{e\_0.625} := F_y + C1 \cdot F_{yr} \cdot \frac{A_r}{A_{0.625}} + C2 \cdot f_c \cdot \frac{A_{conc\_0.625}}{A_{0.625}} \qquad F_{e\_0.625} = \begin{pmatrix} 98.33 \\ 100.5 \\ 102.85 \\ 105.35 \end{pmatrix} \cdot ksi$$

for 5/8" walls

Radius of gyration of both sets of steel sections:

$$r_{s\_0.5} := \overrightarrow{\sqrt{\frac{I_{s\_0.5}}{A_{0.5}}}} \qquad r_{s\_0.5} = \begin{pmatrix} 0.6888 \\ 0.7477 \\ 0.8066 \\ 0.8655 \end{pmatrix} ft$$

for 1/2" walls

$$r_{s\_0.625} := \overrightarrow{\sqrt{\frac{I_{s\_0.625}}{A_{0.625}}}} \qquad r_{s\_0.625} = \begin{pmatrix} 0.6852 \\ 0.7441 \\ 0.803 \\ 0.8619 \end{pmatrix} ft$$

for 5/8" walls

E_e term:

$$E_{e\_0.5} := E_{steel} \cdot \left(1 + \frac{C3}{n} \cdot \overrightarrow{\frac{A_{conc\_0.5}}{A_{0.5}}}\right) \qquad E_{e\_0.5} = \begin{pmatrix} 52028 \\ 54063 \\ 56097 \\ 58132 \end{pmatrix} \cdot ksi$$

for 1/2" walls

$$E_{e\_0.625} := E_{steel} \cdot \left(1 + \frac{C3}{n} \cdot \overrightarrow{\frac{A_{conc\_0.625}}{A_{0.625}}}\right) \qquad E_{e\_0.625} = \begin{pmatrix} 45988 \\ 47514 \\ 49040 \\ 50566 \end{pmatrix} \cdot ksi$$

for 5/8" walls

Lamda (λ) term for composite members LRFD Eq. 6.9.5.1-3

$$\lambda_{0.5} := \overrightarrow{\left[\left(\frac{K \cdot L_{UB\_0.5}}{r_{s\_0.5} \cdot \pi}\right)^2 \cdot \frac{F_{e\_0.5}}{E_{e\_0.5}}\right]} \qquad \lambda_{0.5} = \begin{pmatrix} 0.3043 \\ 0.2684 \\ 0.2398 \\ 0.2166 \end{pmatrix}$$

for 1/2" walls

$$\lambda_{0.625} := \overrightarrow{\left[\left(\frac{K \cdot L_{UB\_0.625}}{r_{s\_0.625} \cdot \pi}\right)^2 \cdot \frac{F_{e\_0.625}}{E_{e\_0.625}}\right]} \qquad \lambda_{0.625} = \begin{pmatrix} 0.2996 \\ 0.2648 \\ 0.237 \\ 0.2144 \end{pmatrix}$$

for 5/8" walls

Lamda (λ) term for non composite members LRFD Eq. 6.9.4.1-3

$$\lambda_{0.5\_tip} := \overrightarrow{\left[\left(\frac{K \cdot L_{UB\_0.5}}{r_{s\_0.5} \cdot \pi}\right)^2 \cdot \frac{F_y}{E_{steel}}\right]} \qquad \lambda_{0.5\_tip} = \begin{pmatrix} 0.2103 \\ 0.188 \\ 0.1699 \\ 0.1548 \end{pmatrix}$$

for 1/2" walls

$$\lambda_{0.625\_tip} := \overrightarrow{\left[\left(\frac{K \cdot L_{UB\_0.625}}{r_{s\_0.625} \cdot \pi}\right)^2 \cdot \frac{F_y}{E_{steel}}\right]} \qquad \lambda_{0.625\_tip} = \begin{pmatrix} 0.2175 \\ 0.1943 \\ 0.1754 \\ 0.1597 \end{pmatrix}$$

for 5/8" walls

Nominal Axial Structural Resistance of 1/2-inch wall

Since λ<2.25 use LRFD Eq. 6.9.5.1-1

$$P_{n\_0.5} := \overrightarrow{\left(0.66^{\lambda_{0.5}} \cdot F_{e\_0.5} \cdot A_{0.5}\right)} \qquad P_{n\_0.5} = \begin{pmatrix} 2835 \\ 3202 \\ 3588 \\ 3993 \end{pmatrix} \cdot kip$$

At the bottom of open-ended piles, or closed ended piles where the conical tip or closed tip experiences breeching, the nominal compressive resistance is a function of only the steel pipe.

$$P_{n\_0.5tip} := \overrightarrow{\left(0.66^{\lambda_{0.5\_tip}} \cdot F_y \cdot A_{0.5}\right)} \qquad P_{n\_0.5tip} = \begin{pmatrix} 1136 \\ 1244 \\ 1352 \\ 1460 \end{pmatrix} \cdot kip$$

USE THIS FOR DESIGN

for 1/2" walls

Nominal Axial Structural Resistance of 5/8-inch wall

Since λ<2.25 use LRFD Eq. 6.9.5.1-1

$$P_{n\_0.625} := \overrightarrow{\left(0.66^{\lambda_{0.625}} \cdot F_{e\_0.625} \cdot A_{0.625}\right)} \qquad P_{n\_0.625} = \begin{pmatrix} 3171 \\ 3571 \\ 3990 \\ 4428 \end{pmatrix} \cdot kip$$

At the bottom of open-ended piles, or closed ended piles where the conical tip or closed tip experiences breeching, the nominal compressive resistance is a function of only the steel pipe.

$$P_{n\_0.625tip} := \overrightarrow{\left(0.66^{\lambda_{0.625\_tip}} \cdot F_y \cdot A_{0.625}\right)} \qquad P_{n\_0.625tip} = \begin{pmatrix} 1501 \\ 1646 \\ 1791 \\ 1935 \end{pmatrix} \cdot kip$$

USE THIS FOR DESIGN

for 5/8" walls

Factored Axial Structural Resistance of a single Pipe Pile:

Strength limit state resistance factor for pipe piles in compression, severe driving conditions - LRFD 6.5.4.2

$$\phi_c := 0.6$$

Factored Structural Resistance (Pr):

$$P_{r\_0.5} := \phi_c \cdot P_{n\_0.5} \qquad P_{r\_0.5} = \begin{pmatrix} 1701 \\ 1921 \\ 2153 \\ 2396 \end{pmatrix} \cdot kip$$

for 1/2" walls

$$P_{r\_0.625} := \phi_c \cdot P_{n\_0.625} \qquad P_{r\_0.625} = \begin{pmatrix} 1902 \\ 2142 \\ 2394 \\ 2657 \end{pmatrix} \cdot kip$$

for 5/8" walls

Factored Structural Resistance (Pr) for the lower portion of open-ended piles or breached close-ended piles is a function of only the steel shell.

$$P_{r\_0.5tip} := \phi_c \cdot P_{n\_0.5tip} \qquad P_{r\_0.5tip} = \begin{pmatrix} 681 \\ 746 \\ 811 \\ 876 \end{pmatrix} \cdot kip$$

for 1/2" walls

$$P_{r\_0.5tip} = \begin{pmatrix} 3031 \\ 3321 \\ 3609 \\ 3897 \end{pmatrix} \cdot kN$$

$$P_{r\_0.625tip} := \phi_c \cdot P_{n\_0.625tip} \qquad P_{r\_0.625tip} = \begin{pmatrix} 901 \\ 988 \\ 1074 \\ 1161 \end{pmatrix} \cdot kip$$

for 5/8" walls

$$P_{r\_0.625tip} = \begin{pmatrix} 4007 \\ 4394 \\ 4780 \\ 5164 \end{pmatrix} \cdot kN$$

Service and Extreme Limit States Axial Structural Resistance

Resistance Factors for Service and Extreme Limit States $\phi = 1.0$ LRFD 10.5.5.1 and 10.5.8.3

$$\phi := 1.0$$

Factored Compressive Resistance for Service and Extreme Limit States:

$$P_{\_0.5tipf} := \phi \cdot P_{n\_0.5tip} \qquad P_{\_0.5tipf} = \begin{pmatrix} 1136 \\ 1244 \\ 1352 \\ 1460 \end{pmatrix} \cdot kip$$

for 1/2" walls

$$P_{\_0.5tipf} = \begin{pmatrix} 5051 \\ 5534 \\ 6016 \\ 6496 \end{pmatrix} \cdot kN$$

$$P_{\_0.625tipf} := \phi \cdot P_{n\_0.625tip} \qquad P_{\_0.625tipf} = \begin{pmatrix} 1501 \\ 1646 \\ 1791 \\ 1935 \end{pmatrix} \cdot kip$$

for 5/8" walls

$$P_{\_0.625tipf} = \begin{pmatrix} 6679 \\ 7324 \\ 7966 \\ 8607 \end{pmatrix} \cdot kN$$

COMPUTE GEOTECHNICAL RESISTANCE OF PIPE PILES

Pipe pile capacity based on steel shell end bearing on bedrock - driven through sand, gravel, cobbles and boulders.

Pipe piles evaluated:
24 in diameter 1/2 in wall
26 in diameter 1/2 in wall
28 in diameter 1/2 in wall
30 in diameter 1/2 in wall

RQD of bedrock in pier location= 95%.
Bedrock is identified as: GNEISS

24 in diameter 5/8 in wall
26 in diameter 5/8 in wall
28 in diameter 5/8 in wall
30 in diameter 5/8 in wall

Uniaxial Compressive Strength of GNEISS from AASHTO Standard Spec for Highway Bridges 17th Ed. Table 4.4.8.1.2B pg 64
Granite 3500 - 45000 psi   Use 22000 psi

Reference: Pile Design and Construction Practice, M.J. Tomlinson, Fourth Edition pg 139
Friction angle = 27 to 34 degrees

$$Q_{uc} := 25000 \cdot psi$$

$$\phi_1 := 32 \cdot deg$$

Diameter of piles:

$$dia_{steel} := \begin{pmatrix} 24 \\ 26 \\ 28 \\ 30 \end{pmatrix} \cdot in$$

Pipe pile wall thickness:

$$wall_t := \begin{pmatrix} \frac{1}{2} \\ \frac{5}{8} \end{pmatrix} \cdot in$$

Corrosion loss per MaineDOT BDG:

$$cor := \frac{1}{8} in$$

$$A_{0.5} = \begin{pmatrix} 27.54 \\ 29.89 \\ 32.25 \\ 34.61 \end{pmatrix} \cdot in^2$$

STEEL AREA FOR 1/2" PILES with 1/8" corrosion loss

$$A_{0.625} = \begin{pmatrix} 36.52 \\ 39.66 \\ 42.8 \\ 45.95 \end{pmatrix} \cdot in^2$$

STEEL AREA FOR 5/8" PILES with 1/8" corrosion loss

LRFD Code specifies Canadian Geotechnical Society Method 1985 for resistance determination of end bearing piles on bedrock. (LRFD Table 10.5.5.2.3-1)
Use Canadian Foundation Manual 4th Edition 2006 Section 18.6.3.3.

Determine K_{sp}: From Canadian Foundation Manual 4th Edition (2006) Section 9.2

Spacing of discontinuities: $c := 36 \cdot in$ Assumed based on rock core

Aperture of discontinuities: $\delta := \frac{1}{64} \cdot in$ joints are tight

Footing width, b:

$$b := dia_{steelcor} \qquad b = \begin{pmatrix} 23.75 \\ 25.75 \\ 27.75 \\ 29.75 \end{pmatrix} \cdot in$$

$$K_{sp} := \frac{3 + \frac{c}{b}}{10 \cdot \left(1 + 300 \cdot \frac{\delta}{c}\right)^{0.5}}$$

K_{sp} includes a factor of safety of 3

Length of rock socket, L_s: $L_s := 0 \cdot in$ Pile is end bearing on rock

Diameter of socket, B_s: $B_s := 0 \cdot ft$

depth factor, d_f: $d_f := 1 + 0.4\left(\frac{L_s}{B_s}\right)$ $d_f = 1$ should be < or = 3 OK

$$q_{aA} := Q_{uc} \cdot K_{sp} \cdot d_f \qquad q_{aA} = \begin{pmatrix} 1529 \\ 1489 \\ 1455 \\ 1426 \end{pmatrix} \cdot ksf$$

Nominal Geotechnical Tip Resistance, R_p:

Multiply by 3 to take out FS=3 on K_{sp}

$$R_{pA0.5} := \overrightarrow{\left(3q_{aA} \cdot A_{0.5}\right)} \qquad R_{pA0.5} = \begin{pmatrix} 877 \\ 928 \\ 978 \\ 1028 \end{pmatrix} \cdot kip$$

for 1/2" walls

$$R_{pA0.625} := \overrightarrow{\left(3q_{aA} \cdot A_{0.625}\right)} \qquad R_{pA0.625} = \begin{pmatrix} 1163 \\ 1231 \\ 1298 \\ 1365 \end{pmatrix} \cdot kip$$

for 5/8" walls

STRENGTH LIMIT STATE:

Factored Geotechnical Resistance at Strength Limit State:

Resistance factor, end bearing on rock (Canadian Geotech. Society, 1985 method):

Nominal resistance of Single Pile in Axial Compression - Static Analysis Methods, ϕ_{stat} $\phi_{stat} := 0.45$ LRFD Table 10.5.5.2.3-1

$R_{f0.5} := \phi_{stat} \cdot R_{pA0.5}$ $R_{f0.5} = \begin{pmatrix} 395 \\ 417 \\ 440 \\ 463 \end{pmatrix} \cdot kip$ Strength Limit State for 1/2" walls $R_{f0.5} = \begin{pmatrix} 1756 \\ 1857 \\ 1957 \\ 2057 \end{pmatrix} \cdot kN$

$R_{f0.625} := \phi_{stat} \cdot R_{pA0.625}$ $R_{f0.625} = \begin{pmatrix} 524 \\ 554 \\ 584 \\ 614 \end{pmatrix} \cdot kip$ Strength Limit State for 5/8" walls $R_{f0.625} = \begin{pmatrix} 2329 \\ 2463 \\ 2598 \\ 2732 \end{pmatrix} \cdot kN$

SERVICE/EXTREME LIMIT STATES:

Factored Geotechnical Resistance at the Service/Extreme Limit States:

Resistance Factors for Service and Extreme Limit States ϕ = 1.0 LRFD 10.5.5.1 and 10.5.8.3

$\phi := 1.0$

$R_{fse0.5} := \phi \cdot R_{pA0.5}$ $R_{fse0.5} = \begin{pmatrix} 877 \\ 928 \\ 978 \\ 1028 \end{pmatrix} \cdot kip$ Service/Extreme Limit States for 1/2" walls $R_{fse0.5} = \begin{pmatrix} 3902 \\ 4126 \\ 4349 \\ 4572 \end{pmatrix} \cdot kN$

$R_{fse0.625} := \phi \cdot R_{pA0.625}$ $R_{fse0.625} = \begin{pmatrix} 1163 \\ 1231 \\ 1298 \\ 1365 \end{pmatrix} \cdot kip$ Service/Extreme Limit States for 5/8" walls $R_{fse0.625} = \begin{pmatrix} 5175 \\ 5474 \\ 5772 \\ 6070 \end{pmatrix} \cdot kN$

DRIVABILITY ANALYSIS

Ref: LRFD Article 10.7.8

For steel piles in compression or tension
σ_{dr} = 0.9 x ϕ_{da} x f_y (eq. 10.7.8-1)

$f_y := 45 \cdot ksi$ yield strength of steel

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

$\sigma_{dr} := 0.9 \cdot \phi_{da} \cdot f_y$ $\sigma_{dr} = 40.5 \cdot ksi$ driving stresses in pile cannot exceed 40 ksi $\sigma_{dr} = 279.2377 \cdot MPa$

Compute Resistance that can be achieved in a drivability analysis:

The resistance that must be achieved in a drivability analysis will be the maximum applied pile axial load (must be less than the the factored geotechnical resistance from above as this governs) divided by the appropriate resistance factor for wave equation analysis and dynamic test which will be required for construction.

Table 10.5.5.2.3-1 pg 10-38 gives resistance factor for dynamic test, ϕ_{dyn}:

$\phi_{dyn} := 0.65$

Greater than 5 piles in pier, no reduction to Φ_{dyn} necessary.

Assume Contractor will use a Delmag D 36-32 hammer on the third fuel setting to install:
24-in Dia. pile with 1/2-in wall thickness

Pile Size = 24"D x 1/2"W

Pier with curtain wall: Unsupported length = preliminary scour depth = 14 ft +11 ft to fixity = 25 ft.

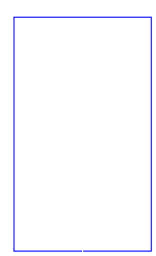
State of Maine Dept. Of Transportation 14-Oct-2008
Gilead Wild River Bridge Pipe Pile GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
570.0	39.28	4.40	3.7	7.58	39.81
575.0	39.48	4.50	3.8	7.61	40.00
580.0	39.71	4.57	3.9	7.64	40.05
585.0	39.95	4.67	3.9	7.68	40.34
590.0	40.23	4.77	4.0	7.71	40.54
595.0	40.44	4.85	4.0	7.75	40.69
600.0	40.72	4.93	4.1	7.79	40.87
605.0	40.98	5.04	4.2	7.83	41.17
610.0	41.17	5.11	4.2	7.86	41.23
615.0	41.40	5.20	4.3	7.89	41.44

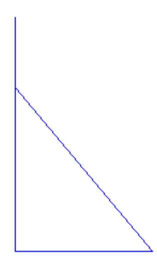
DELMAG D 36-32

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	84.00	ft
Pile Penetration	59.00	ft
Pile Top Area	27.54	in2

Pile Model



Skin Friction Distribution



Res. Shaft = 15 %
(Proportional)

Limit driving stress to 40 ksi

Strength Limit State:

$$R_{dr\_24x0.5\_factored} := 585 \cdot kip \cdot \phi_{dyn}$$

$$R_{dr\_24x0.5\_factored} = 380 \cdot kip$$

$$R_{dr\_24x0.5\_factored} = 1691 \cdot kN$$

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

$$R_{dr\_24x0.5\_servext} := 585 \cdot kip$$

$$R_{dr\_24x0.5\_servext} = 2602 \cdot kN$$

Assume Contractor will use a Delmag D 36-32 hammer on the third fuel setting to install:
26-in Dia. pile with 1/2-in wall thickness

Pile Size = 26"D x 1/2"W

Pier with curtain wall: Unsupported length = preliminary scour depth = 14 ft +12 ft to fixity = 26 ft.

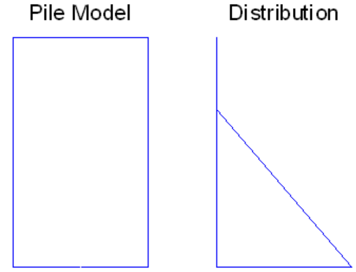
State of Maine Dept. Of Transportation — 14-Oct-2008
Gilead Wild River Bridge Pipe Pile — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
580.0	37.90	4.44	3.8	7.59	39.10
590.0	38.35	4.64	3.9	7.66	39.50
600.0	38.79	4.82	4.0	7.72	39.79
610.0	39.19	5.00	4.2	7.77	40.08
620.0	39.61	5.16	4.3	7.83	40.37
630.0	40.07	5.33	4.4	7.89	40.69
640.0	40.20	5.48	4.7	7.86	40.50
643.0	40.54	5.51	4.6	7.95	40.95
650.0	40.54	5.62	4.8	7.91	40.69
660.0	40.93	5.77	5.0	7.95	40.95

DELMAG D 36-32

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	84.00	ft
Pile Penetration	58.00	ft
Pile Top Area	29.89	in2

Pile Model   Skin Friction Distribution



Res. Shaft = 15 %
(Proportional)

Limit driving stress to 40 ksi

Strength Limit State:

$$R_{dr\_26x0.5\_factored} := 630 \cdot kip \cdot \phi_{dyn}$$

$$R_{dr\_26x0.5\_factored} = 409 \cdot kip$$

$$R_{dr\_26x0.5\_factored} = 1822 \cdot kN$$

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

$$R_{dr\_26x0.5\_servext} := 630 \cdot kip$$

$$R_{dr\_26x0.5\_servext} = 2802 \cdot kN$$

Assume Contractor will use a Delmag D 36-32 hammer on the third fuel setting to install:
28-in Dia. pile with 1/2-in wall thickness

Pile Size = 28"D x 1/2"W

Pier with curtain wall: Unsupported length = preliminary scour depth = 14 ft +13 ft to fixity = 27 ft.

State of Maine Dept. Of Transportation — 14-Oct-2008
Gilead Wild River Bridge Pipe Pile — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
680.0	39.77	5.73	5.2	7.94	40.23
687.0	40.06	5.80	5.4	7.99	40.38
690.0	40.13	5.83	5.4	8.00	40.44
700.0	40.42	5.90	5.6	8.02	40.49
710.0	40.71	5.97	5.8	8.06	40.78
720.0	41.09	6.02	5.9	8.10	41.05
730.0	41.39	6.08	6.1	8.13	41.21
740.0	41.62	6.13	6.3	8.15	41.27
750.0	41.94	6.18	6.5	8.19	41.54
760.0	42.20	6.23	6.7	8.21	41.62

Limit driving stress to 40 ksi

Strength Limit State:

$$R_{dr\_28x0.5\_factored} := 687 \cdot kip \cdot \phi_{dyn}$$

$$R_{dr\_28x0.5\_factored} = 447 \cdot kip$$

$$R_{dr\_28x0.5\_factored} = 1986 \cdot kN$$

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

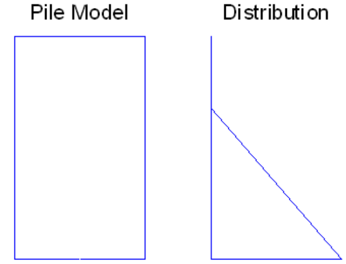
$$R_{dr\_28x0.5\_servext} := 687 \cdot kip$$

$$R_{dr\_28x0.5\_servext} = 3056 \cdot kN$$

DELMAG D 36-32

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	84.00	ft
Pile Penetration	57.00	ft
Pile Top Area	32.25	in2

Pile Model — Skin Friction Distribution



Res. Shaft = 15 %
(Proportional)

Assume Contractor will use a Delmag D 36-32 hammer on the third fuel setting to install:
30-in Dia. pile with 1/2-in wall thickness

Pile Size = 30"D x 1/2"W

Pier with curtain wall: Unsupported length = preliminary scour depth = 14 ft +13 ft to fixity = 27 ft.

State of Maine Dept. Of Transportation — 14-Oct-2008
Gilead Wild River Bridge Pipe Pile — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
680.0	38.31	5.13	5.1	7.95	39.42
687.0	38.52	5.18	5.1	7.98	39.70
690.0	38.61	5.18	5.2	7.99	39.73
700.0	38.99	5.23	5.3	8.03	39.90
710.0	39.10	5.27	5.6	7.99	39.69
720.0	39.41	5.31	5.7	8.03	39.88
730.0	39.67	5.34	5.9	8.06	40.10
742.0	40.02	5.38	6.1	8.09	40.28
750.0	40.24	5.40	6.2	8.11	40.30
760.0	40.52	5.43	6.4	8.15	40.51

DELMAG D 36-32

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	84.00	ft
Pile Penetration	57.00	ft
Pile Top Area	34.61	in2

Limit driving stress to 40 ksi

Strength Limit State:

$$R_{dr\_30x0.5\_factored} := 742 \cdot kip \cdot \phi_{dyn}$$

$$R_{dr\_30x0.5\_factored} = 482 \cdot kip$$

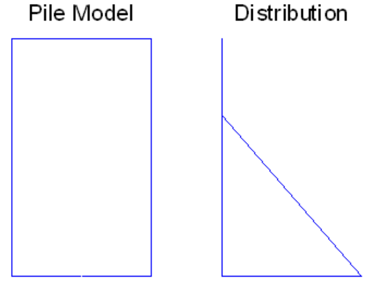
$$R_{dr\_30x0.5\_factored} = 2145 \cdot kN$$

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

$$R_{dr\_30x0.5\_servext} := 742 \cdot kip$$

$$R_{dr\_30x0.5\_servext} = 3301 \cdot kN$$

Pile Model — Skin Friction Distribution



Res. Shaft = 15 %
(Proportional)

Assume Contractor will use a Delmag D 36-32 hammer on the third fuel setting to install:
24-in Dia. pile with 5/8-in wall thickness

Pile Size = 24"D x 5/8"W

Pier with curtain wall: Unsupported length = preliminary scour depth = 14 ft +11 ft to fixity = 25 ft.

State of Maine Dept. Of Transportation — 14-Oct-2008
Gilead Wild River Bridge Pipe Pile — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
750.0	38.92	4.77	6.1	8.05	39.49
760.0	39.14	4.78	6.3	8.07	39.52
770.0	39.40	4.80	6.5	8.09	39.57
780.0	39.67	4.81	6.6	8.12	39.75
790.0	39.94	4.82	6.8	8.15	39.90
795.0	40.03	4.83	6.9	8.16	40.00
810.0	40.42	4.89	7.2	8.20	40.25
820.0	40.67	4.95	7.5	8.22	40.31
830.0	40.93	5.00	7.7	8.25	40.47
840.0	41.12	5.05	8.0	8.28	40.50

Limit driving stress to 40 ksi

Strength Limit State:

$$R_{dr\_24x0.625\_factored} := 795 \cdot kip \cdot \phi_{dyn}$$

$$R_{dr\_24x0.625\_factored} = 517 \cdot kip$$

$$R_{dr\_24x0.625\_factored} = 2299 \cdot kN$$

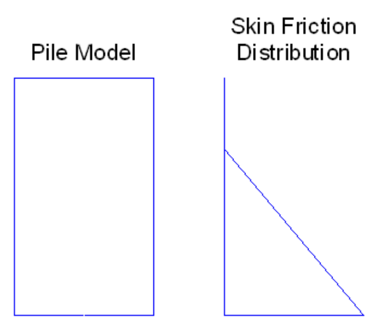
Service and Extreme Limit States: $\phi := 1.0$

$$R_{dr\_24x0.625\_servext} := 795 \cdot kip$$

$$R_{dr\_24x0.625\_servext} = 3536 \cdot kN$$

DELMAG D 36-32

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	84.00	ft
Pile Penetration	59.00	ft
Pile Top Area	36.52	in2



Res. Shaft = 15 %
(Proportional)

Assume Contractor will use a Delmag D 36-32 hammer on the third fuel setting to install:
26-in Dia. pile with 5/8-in wall thickness

Pile Size = 26"D x 5/8"W

Pier with curtain wall: Unsupported length = preliminary scour depth = 14 ft +12 ft to fixity = 26 ft.

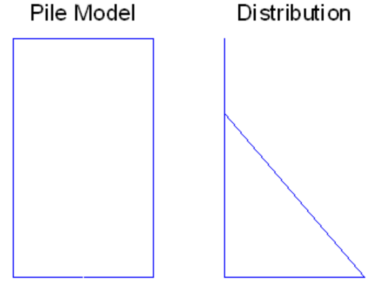
State of Maine Dept. Of Transportation — 14-Oct-2008
Gilead Wild River Bridge Pipe Pile — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
850.0	39.41	4.26	7.8	8.22	39.43
860.0	39.68	4.30	8.0	8.24	39.53
870.0	39.88	4.36	8.2	8.27	39.73
880.0	40.09	4.41	8.5	8.29	39.72
890.0	40.36	4.46	8.7	8.31	39.94
900.0	40.52	4.52	9.0	8.34	40.01
910.0	40.77	4.58	9.3	8.35	40.14
920.0	40.99	4.63	9.6	8.38	40.22
930.0	41.17	4.68	9.8	8.40	40.33
940.0	41.33	4.72	10.1	8.42	40.45

DELMAG D 36-32

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	84.00	ft
Pile Penetration	58.00	ft
Pile Top Area	39.66	in2

Pile Model — Skin Friction Distribution



Res. Shaft = 15 %
(Proportional)

Limit driving stress to 40 ksi

Strength Limit State:

$$R_{dr\_26x0.625\_factored} := 880 \cdot kip \cdot \phi_{dyn}$$

$$R_{dr\_26x0.625\_factored} = 572 \cdot kip$$

$$R_{dr\_26x0.625\_factored} = 2544 \cdot kN$$

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

$$R_{dr\_26x0.625\_servext} := 880 \cdot kip$$

$$R_{dr\_26x0.625\_servext} = 3914 \cdot kN$$

Assume Contractor will use a Delmag D 36-32 hammer on the third fuel setting to install:
28-in Dia. pile with 5/8-in wall thickness

Pile Size = 28"D x 5/8"W

Pier with curtain wall: Unsupported length = preliminary scour depth = 14 ft +13 ft to fixity = 27 ft.

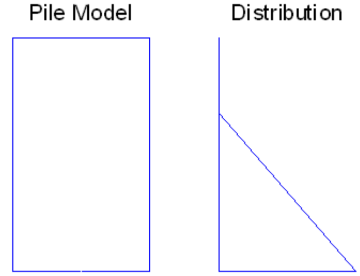
State of Maine Dept. Of Transportation — 14-Oct-2008
Gilead Wild River Bridge Pipe Pile — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
950.0	39.85	4.19	9.6	8.39	39.49
960.0	40.01	4.22	9.9	8.40	39.53
970.0	40.22	4.27	10.1	8.42	39.58
980.0	40.37	4.33	10.4	8.43	39.63
990.0	40.55	4.40	10.6	8.45	39.81
1000.0	40.70	4.44	11.0	8.46	39.74
1010.0	40.88	4.50	11.2	8.48	39.90
1020.0	41.07	4.55	11.5	8.49	39.96
1030.0	41.22	4.62	11.8	8.52	40.11
1040.0	41.38	4.66	12.1	8.52	40.06

DELMAG D 36-32

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	84.00	ft
Pile Penetration	57.00	ft
Pile Top Area	42.80	in2

Pile Model — Skin Friction Distribution



Res. Shaft = 15 %
(Proportional)

Limit driving stress to 40 ksi

Strength Limit State:

$$R_{dr\_28x0.625\_factored} := 960 \cdot kip \cdot \phi_{dyn}$$

$$R_{dr\_28x0.625\_factored} = 624 \cdot kip$$

$$R_{dr\_28x0.625\_factored} = 2776 \cdot kN$$

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

$$R_{dr\_28x0.625\_servext} := 960 \cdot kip$$

$$R_{dr\_28x0.625\_servext} = 4270 \cdot kN$$

Assume Contractor will use a Delmag D 36-32 hammer on the third fuel setting to install:
30-in Dia. pile with 5/8-in wall thickness

Pile Size = 30"D x 5/8"W

Pier with curtain wall: Unsupported length = preliminary scour depth = 14 ft +13 ft to fixity = 27 ft.

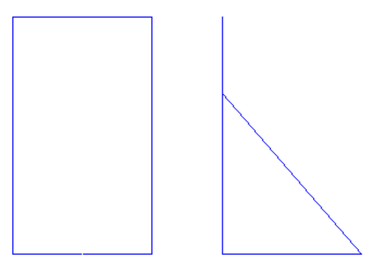
State of Maine Dept. Of Transportation — 14-Oct-2008
Gilead Wild River Bridge Pipe Pile — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
1020.0	39.45	4.07	10.8	8.44	38.89
1025.0	39.53	4.11	10.8	8.45	39.03
1030.0	39.59	4.14	11.0	8.45	39.04
1035.0	39.68	4.16	11.1	8.46	39.03
1040.0	39.78	4.19	11.2	8.47	39.12
1045.0	39.85	4.22	11.4	8.47	39.14
1050.0	39.91	4.23	11.5	8.49	39.12
1055.0	40.04	4.26	11.6	8.50	39.23
1060.0	40.10	4.30	11.8	8.50	39.25
1065.0	40.18	4.32	11.9	8.50	39.26

DELMAG D 36-32

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	84.00	ft
Pile Penetration	57.00	ft
Pile Top Area	45.95	in2

Pile Model — Skin Friction Distribution



Res. Shaft = 15 %
(Proportional)

Limit driving stress to 40 ksi

Strength Limit State:

$$R_{dr\_30x0.625\_factored} := 1055 \cdot kip \cdot \phi_{dyn}$$

$$R_{dr\_30x0.625\_factored} = 686 \cdot kip$$

$$R_{dr\_30x0.625\_factored} = 3050 \cdot kN$$

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

$$R_{dr\_30x0.625\_servext} := 1055 \cdot kip$$

$$R_{dr\_30x0.625\_servext} = 4693 \cdot kN$$

H-pile supported Pier

Calculate Depth to Fixity for H-piles:

Soil conditions at boring BB-GWR-102:
84 ft of fill sand, gravel, cobbles and boulders over bedrock

Consider Pile sizes:
HP 12x53
HP 14x73
HP 14x 89
HP 14x117

H-pile Steel area: $$A_s := \begin{pmatrix} 15.5 \\ 21.4 \\ 26.1 \\ 34.4 \end{pmatrix} \cdot in^2$$

LRFD Eq.10.7.3.13.4-2 for fixity in feet: $1.8*(E_p I_w / n_h)^{0.2}$ (in sands)

E_p Young's modulus of pile in ksi

I_w moment of inertia of pile in ft^4

n_h= rate of increase of soil modulus with depth for sands as specified in Table C10.4.6.3-2 in ksi/ft

Steel modulus: $E_{steel} := 29000 \cdot ksi$

Moment of Inertia: $$I_w := \begin{pmatrix} 393 \\ 729 \\ 904 \\ 1220 \end{pmatrix} \cdot in^4$$

Rate of increase of soil modulus with depth:
for submerged medium dense sand $$n_h := 0.556 \cdot \frac{ksi}{ft}$$

T parameter: $$T_H := \left(\frac{E_{steel} \cdot I_w}{n_h}\right)^{0.2} \qquad T_H = \begin{pmatrix} 3.97 \\ 4.49 \\ 4.69 \\ 4.98 \end{pmatrix} ft$$

Depth of Fixity: $$D_{fixH} := 1.8 \cdot T_H$$

$$D_{fixH} = \begin{pmatrix} 7 \\ 8 \\ 8 \\ 9 \end{pmatrix} ft$$

Depth to fixity for H-piles $$D_{fixH} = \begin{pmatrix} 2.18 \\ 2.47 \\ 2.57 \\ 2.73 \end{pmatrix} \cdot m$$

HP 12 x 53
HP 14 x 73
HP 14 x 89
HP 14 x 117

Check with LRFD Eq. 10.7.3.13.4-2 $\quad E_{steel} = 29000 \cdot ksi$

$$I_w = \begin{pmatrix} 0.019 \\ 0.0352 \\ 0.0436 \\ 0.0588 \end{pmatrix} ft^4 \qquad Check := 1.8 \cdot \left(\frac{E_{steel} \cdot I_w}{n_h}\right)^{0.2} \qquad Check = \begin{pmatrix} 7.15 \\ 8.09 \\ 8.45 \\ 8.97 \end{pmatrix} ft$$

OK

For total unbraced length of H-pile:
Use equation for sands in NCHRP#343 pg 61:

$L_{eq}=L_u+1.8T$ where:

L_{eq} = equivalent free standing length of pile

L_u = unsupported length of pile extending above ground

$T=(E_p*I_p/n_h)^{0.2}$

For H-piles supporting a curtain wall in the Wild River the unsupported length = scour depth

Total unbraced length L_{eqH}:

$L_{UBH} := 14 \cdot ft$ $\qquad L_{eqH} := L_{UBH} + D_{fixH}$

$$L_{eqH} = \begin{pmatrix} 21 \\ 22 \\ 22 \\ 23 \end{pmatrix} ft$$ Total unbraced length

Nominal Axial Structural Resistance of H-piles

Ref: AASHTO LRFD Bridge Design Specifications 4th Edition 2007

Pier - H-Pile driven to bedrock, assume driven through cohesionless soils to bedrock (refusal)

Axial pile resistance may be controlled by structural resistance if piles are driven to bedrock.
Check concurrent axial loading and moments with LRFD Equation 6.9.2.2-1 or 6.9.2.2-2.
Use LRFD Equation 6.9.4.1-1 or 6.9.4.1-2 to compute the nominal compressive structural resistance for pile sections.

λ in Equation 6.9.4.1-1 and -2 has to be computed for the H-piles since they have an unbraced length.

$\lambda = (Kl/r_s\pi)^{2(}F_y/E)$ Compute λ per 6.9.4.1-3 for noncomposite members:

Effective length factor, K, per LRFD Article 4.6.2.5:

Use case (c) in table C4.6.2.5-1

$K := 1.0$ Piles are fixed at the end

Unbraced length is from bottom of curtain wall including 14 feet of scour plus depth to fixity.

$$L_{eqH} = \begin{pmatrix} 21 \\ 22 \\ 22 \\ 23 \end{pmatrix} ft$$

Radius of gyration, r_s:

$$r_{sH} := \overrightarrow{\sqrt{\frac{I_w}{A_s}}} \qquad r_{sH} = \begin{pmatrix} 0.4196 \\ 0.4864 \\ 0.4904 \\ 0.4963 \end{pmatrix} ft$$

Yield strength of steel: $F_y := 50 \cdot ksi$

Steel modulus of elasticity: $E_{steel} := 29000 \cdot ksi$

Lamda (λ) term for noncomposite members LRFD Eq. 6.9.4.1-3

$$\lambda_H := \overrightarrow{\left[\left(\frac{K \cdot L_{eqH}}{r_{sH} \cdot \pi}\right)^2 \cdot \frac{F_y}{E_{steel}}\right]} \qquad \lambda_H = \begin{pmatrix} 0.4438 \\ 0.3603 \\ 0.3659 \\ 0.3742 \end{pmatrix}$$

Nominal Axial Compressive Structural Resistance of H-pile

Since λ<2.25 use LRFD Eq. 6.9.4.1-1

$$P_{nH} := \overrightarrow{\left(0.66^{\lambda_H} \cdot F_y \cdot A_s\right)} \qquad P_{nH} = \begin{pmatrix} 644 \\ 921 \\ 1121 \\ 1472 \end{pmatrix} \cdot kip$$

<u>Factored Axial Structural Resistance of a single H-Pile:</u>

Strength limit state resistance factor for H-piles in compression, severe driving conditions - LRFD 6.5.4.2 $\phi_c := 0.5$

Factored Structural Resistance (Pr):

$$P_{rH} := \phi_c \cdot P_{nH} \qquad P_{rH} = \begin{pmatrix} 322 \\ 461 \\ 560 \\ 736 \end{pmatrix} \cdot kip \qquad P_{rH} = \begin{pmatrix} 1433 \\ 2049 \\ 2493 \\ 3275 \end{pmatrix} \cdot kN$$

HP 12 x 53
HP 14 x 73
HP 14 x 89
HP 14 x 117

Service and Extreme Limit States Axial Structural Resistance

Resistance Factors for Service and Extreme Limit States $\phi = 1.0$ LRFD 10.5.5.1 and 10.5.8.3

$\phi := 1.0$

Factored Compressive Resistance for Service and Extreme Limit States:

$$P_{r\_servext} := \phi \cdot P_{nH} \qquad P_{r\_servext} = \begin{pmatrix} 644 \\ 921 \\ 1121 \\ 1472 \end{pmatrix} \cdot kip \qquad P_{r\_servext} = \begin{pmatrix} 2867 \\ 4098 \\ 4986 \\ 6549 \end{pmatrix} \cdot kN$$

HP 12 x 53
HP 14 x 73
HP 14 x 89
HP 14 x 117

Geotechnical Resistance for H-pile supported pier

Assume piles will be end bearing on bedrock driven through overlying sand with cobbles and boulders.

Bedrock Type:

Gneiss RQD = 95% at boring BB-GWR-102

Use RQD = 95% and ϕ = 27 to 34 deg (Tomlinson 4th Ed. pg 139)

Axial Geotechnical Resistance of H-piles

Ref: AASHTO LRFD Bridge Design Specifications 4th Edition 2007

Look at these piles:

HP 12 x 53
HP 14 x 73
HP 14 x 89
HP 14 x 117

Note: All matrices set up in this order

Steel area: $A_s = \begin{pmatrix} 15.5 \\ 21.4 \\ 26.1 \\ 34.4 \end{pmatrix} \cdot in^2$

Pile depth: $d := \begin{pmatrix} 11.78 \\ 13.61 \\ 13.83 \\ 14.21 \end{pmatrix} \cdot in$

Pile width: $b := \begin{pmatrix} 12.045 \\ 14.585 \\ 14.695 \\ 14.885 \end{pmatrix} \cdot in$

End bearing resistance of piles on bedrock - LRFD code specifies Canadian Geotech Method 1985 (LRFD Table 10.5.5.2.3-1) Canadian Foundation Manual 4th Edition (2006) Section 18.6.3.3.

Average compressive strength of rock core
from AASHTO Standard Spec for Highway Bridges 17 Ed.
Table 4.4.8.1.2B pg 64

q_u for gneiss compressive strength ranges from 3500 to 45000 psi

use $\sigma_c := 25000 \cdot psi$

Determine K_{sp}: From Canadian Foundation Manual 4th Edition (2006) Section 9.2

Spacing of discontinuities: $c := 36 \cdot in$ Assumed based on rock core

Aperture of discontinuities: $\delta := \frac{1}{64} \cdot in$ joints are tight

Footing width, b: $b = \begin{pmatrix} 12.045 \\ 14.585 \\ 14.695 \\ 14.885 \end{pmatrix} \cdot in$

HP 12 x 53
HP 14 x 73
HP 14 x 89
HP 14 x 117

$$K_{sp} := \frac{3 + \frac{c}{b}}{10 \cdot \left(1 + 300 \cdot \frac{\delta}{c}\right)^{0.5}}$$

$$K_{sp} = \begin{pmatrix} 0.5633 \\ 0.5144 \\ 0.5126 \\ 0.5097 \end{pmatrix}$$

K_{sp} includes a factor of safety of 3

Length of rock socket, L_s: $L_s := 0 \cdot in$ Pile is end bearing on rock

Diameter of socket, B_s: $B_s := 1 \cdot ft$

depth factor, d_f: $d_f := 1 + 0.4\left(\frac{L_s}{B_s}\right)$ $d_f = 1$ should be < or = 3 OK

$$q_a := \sigma_c \cdot K_{sp} \cdot d_f \qquad q_a = \begin{pmatrix} 2028 \\ 1852 \\ 1845 \\ 1835 \end{pmatrix} \cdot ksf$$

Nominal Geotechnical Tip Resistance, R_p:

Multiply by 3 to take out FS=3 on K_{sp}

$$R_p := \overrightarrow{\left(3q_a \cdot A_s\right)} \qquad R_p = \begin{pmatrix} 655 \\ 826 \\ 1003 \\ 1315 \end{pmatrix} \cdot kip$$

HP 12 x 53
HP 14 x 73
HP 14 x 89
HP 14 x 117

STRENGTH LIMIT STATE:

Factored Geotechnical Resistance at Strength Limit State:

Resistance factor, end bearing on rock (Canadian Geotech. Society, 1985 method):

Nominal resistance of Single Pile in Axial Compression - Static Analysis Methods, ϕ_{stat} $\phi_{stat} := 0.45$ LRFD Table 10.5.5.2.3-1

$$R_f := \phi_{stat} \cdot R_p \qquad R_f = \begin{pmatrix} 295 \\ 372 \\ 452 \\ 592 \end{pmatrix} \cdot kip \qquad R_f = \begin{pmatrix} 1311 \\ 1653 \\ 2009 \\ 2632 \end{pmatrix} \cdot kN$$

HP 12 x 53
HP 14 x 73
HP 14 x 89
HP 14 x 117

Strength Limit State

SERVICE/EXTREME LIMIT STATES:

Factored Geotechnical Resistance at the Service/Extreme Limit States:

Resistance Factors for Service and Extreme Limit States ϕ = 1.0 LRFD 10.5.5.1 and 10.5.8.3

$$\phi := 1.0$$

$$R_{fse} := \phi \cdot R_p \qquad R_{fse} = \begin{pmatrix} 655 \\ 826 \\ 1003 \\ 1315 \end{pmatrix} \cdot kip \qquad R_{fse} = \begin{pmatrix} 2913 \\ 3672 \\ 4464 \\ 5849 \end{pmatrix} \cdot kN$$

HP 12 x 53
HP 14 x 73
HP 14 x 89
HP 14 x 117

Service/Extreme Limit States

DRIVABILITY ANALYSIS

Ref: LRFD Article 10.7.8

For steel piles in compression or tension
$\sigma_{dr} = 0.9 \times \phi_{da} \times f_y$ (eq. 10.7.8-1)

$f_y := 50 \cdot ksi$ yield strength of steel

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

$\sigma_{dr} := 0.9 \cdot \phi_{da} \cdot f_y$ $\sigma_{dr} = 45 \cdot ksi$ driving stresses in pile cannot exceed 45 ksi $\sigma_{dr} = 310.2641 \cdot MPa$

Compute Resistance that can be achieved in a drivability analysis:

The resistance that must be achieved in a drivability analysis will be the maximum applied pile axial load (must be less than the the factored geotechnical resistance from above as this governs)
divided by the appropriate resistance factor for wave equation analysis and dynamic test which will be required for construction.

Table 10.5.5.2.3-1 pg 10-38 gives resistance factor for dynamic test, ϕ_{dyn}:

$\phi_{dyn} := 0.65$

Greater than 5 piles in pier, no reduction to Φ_{dyn} necessary.

Pile Size = 12 x 53

Assume Contractor will use a Delmag D19-42 hammer to install 12 x 53 piles

State of Maine Dept. Of Transportation — 15-Oct-2008
Gilead Wild River Bridge — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
300.0	36.06	4.91	3.3	8.07	21.45
350.0	39.49	6.21	4.4	8.55	22.46
400.0	42.72	6.93	6.0	9.07	23.86
450.0	45.00	7.69	8.4	9.36	24.67
460.0	45.45	7.84	9.1	9.42	24.85
470.0	45.87	7.95	9.8	9.48	24.97
480.0	46.31	8.04	10.5	9.55	25.14
490.0	46.69	8.15	11.3	9.60	25.33
500.0	47.06	8.24	12.1	9.65	25.47
510.0	47.39	8.32	13.1	9.70	25.60

Limited driving stresses to 45 ksi

Strength Limit State:

$$R_{drpier\_12x53\_factored} := 450 \cdot kip \cdot \phi_{dyn}$$

$$R_{drpier\_12x53\_factored} = 293 \cdot kip$$

$$R_{drpier\_12x53\_factored} = 1301 \cdot kN$$

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

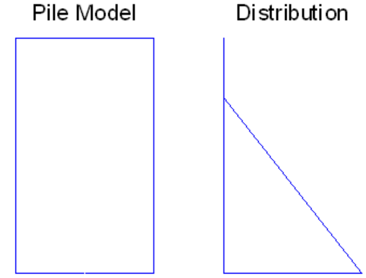
$$R_{drpier\_12x53\_servext} := 450 \cdot kip$$

$$R_{drpier\_12x53\_servext} = 2002 \cdot kN$$

DELMAG D 19-42

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	84.00	ft
Pile Penetration	63.00	ft
Pile Top Area	15.50	in2

Pile Model — Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Pile Size = 14 x 73

Assume Contractor will use a Delmag D36-32 hammer to install 14 x 73 piles

State of Maine Dept. Of Transportation 15-Oct-2008
Gilead Wild River Bridge GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
500.0	41.67	5.51	3.7	6.88	36.13
510.0	42.34	5.62	3.8	6.97	36.61
520.0	43.00	5.72	4.0	7.07	37.09
530.0	43.63	5.81	4.1	7.16	37.57
540.0	44.27	5.90	4.3	7.25	37.97
550.0	44.55	5.98	4.5	7.26	37.96
558.0	45.00	6.05	4.6	7.32	38.27
560.0	45.12	6.06	4.7	7.33	38.33
570.0	45.60	6.14	4.9	7.39	38.53
580.0	46.05	6.23	5.1	7.45	38.84

Limited driving stresses to 45 ksi

Strength Limit State:

$$R_{drpier\_14x73\_factored} := 558 \cdot kip \cdot \phi_{dyn}$$

$$R_{drpier\_14x73\_factored} = 363 \cdot kip$$

$$R_{drpier\_14x73\_factored} = 1613 \cdot kN$$

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

$$R_{drpier\_14x73\_servext} := 558 \cdot kip$$

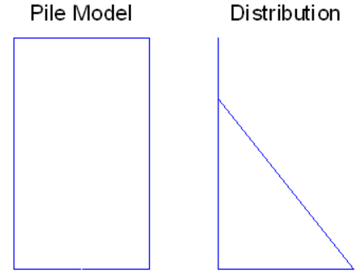
$$R_{drpier\_14x73\_servext} = 2482 \cdot kN$$

DELMAG D 36-32

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	84.00	ft
Pile Penetration	62.00	ft
Pile Top Area	21.40	in2

Pile Model

Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Pile Size = 14 x 89

Assume Contractor will use a Delmag D36-32 hammer to install 14 x 89 piles

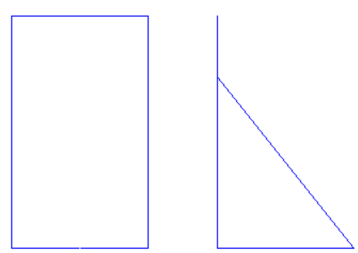
State of Maine Dept. Of Transportation — 15-Oct-2008
Gilead Wild River Bridge — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
600.0	44.30	4.83	4.1	7.95	42.27
610.0	44.83	5.00	4.2	8.02	42.66
615.0	45.02	5.08	4.3	8.05	42.79
620.0	45.31	5.17	4.3	8.09	43.07
630.0	45.48	5.33	4.6	8.07	42.92
640.0	45.85	5.47	4.7	8.11	43.15
650.0	46.27	5.60	4.9	8.17	43.43
660.0	46.69	5.75	5.1	8.22	43.79
670.0	47.14	5.88	5.3	8.27	44.09
680.0	47.51	6.00	5.5	8.32	44.29

DELMAG D 36-32

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	84.00	ft
Pile Penetration	62.00	ft
Pile Top Area	26.10	in2

Pile Model — Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Limited driving stresses to 45 ksi

Strength Limit State:

$$R_{drpier\_14x89\_factored} := 615 \cdot kip \cdot \phi_{dyn}$$

$$R_{drpier\_14x89\_factored} = 400 \cdot kip$$

$$R_{drpier\_14x89\_factored} = 1778 \cdot kN$$

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

$$R_{drpier\_14x89\_servext} := 615 \cdot kip$$

$$R_{drpier\_14x89\_servext} = 2736 \cdot kN$$

Pile Size = 14 x 117

Assume Contractor will use a Delmag D36-32 hammer to install 14 x 117 piles

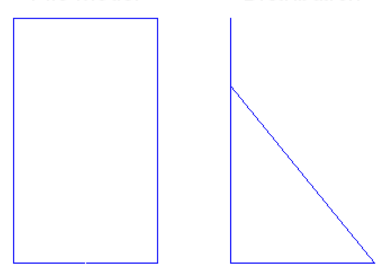
State of Maine Dept. Of Transportation — 15-Oct-2008
Gilead Wild River Bridge — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count bl/ft	Stroke feet	Energy kips-ft
700.0	43.45	5.33	51.2	8.87	46.80
710.0	43.86	5.42	52.5	8.93	47.14
720.0	44.20	5.52	53.9	8.97	47.42
730.0	44.59	5.61	55.5	9.02	47.60
740.0	44.94	5.68	57.0	9.06	47.86
743.0	45.01	5.71	57.6	9.08	47.91
750.0	45.31	5.77	58.7	9.12	48.06
760.0	45.62	5.84	60.3	9.15	48.33
770.0	45.96	5.88	61.8	9.19	48.59
780.0	46.30	5.91	63.4	9.24	48.82

DELMAG D 36-32

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	84.00	ft
Pile Penetration	61.00	ft
Pile Top Area	34.40	in2

Pile Model — Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Limited driving stresses to 45 ksi

Strength Limit State:

$$R_{drpier\_14x117\_factored} := 743 \cdot kip \cdot \phi_{dyn}$$

$$R_{drpier\_14x117\_factored} = 483 \cdot kip$$

$$R_{drpier\_14x117\_factored} = 2148 \cdot kN$$

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

$$R_{drpier\_14x117\_servext} := 743 \cdot kip$$

$$R_{drpier\_14x117\_servext} = 3305 \cdot kN$$

Abutment and Wingwall Passive and Active Earth Pressure:

For cases where interface friction is considered (for gravity structures) use Coulomb Theory

Coulomb Theory - Passive Earth Pressure from Maine DOT Bridge Design Guide Section 3.6.6 pg 3-8

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

Angle of internal soil friction: $\phi := 32 \cdot deg$

Friction angle between fill and wall:
From LRFD Table 3.11.5.3-1 range from 17 to 22 $\delta := 20 \cdot deg$

Angle of backfill to the horizontal $\beta := 0 \cdot deg$

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

$K_p = 6.89$

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

Angle of backfill to the horizontal $\beta := 0 \cdot deg$

Angle of internal soil friction: $\phi := 32 \cdot deg$

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

$K_{p\_rank} = 3.25$

Bowles does not recommend the use of the Rankine Method for K_p when $\beta>0$.

Rankine Theory - Active Earth Pressure from Maine DOT Bridge Design Guide Section 3.6.5.2 pg 3-7

For a horizontal backfill surface:

$\phi := 32 \cdot deg$

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

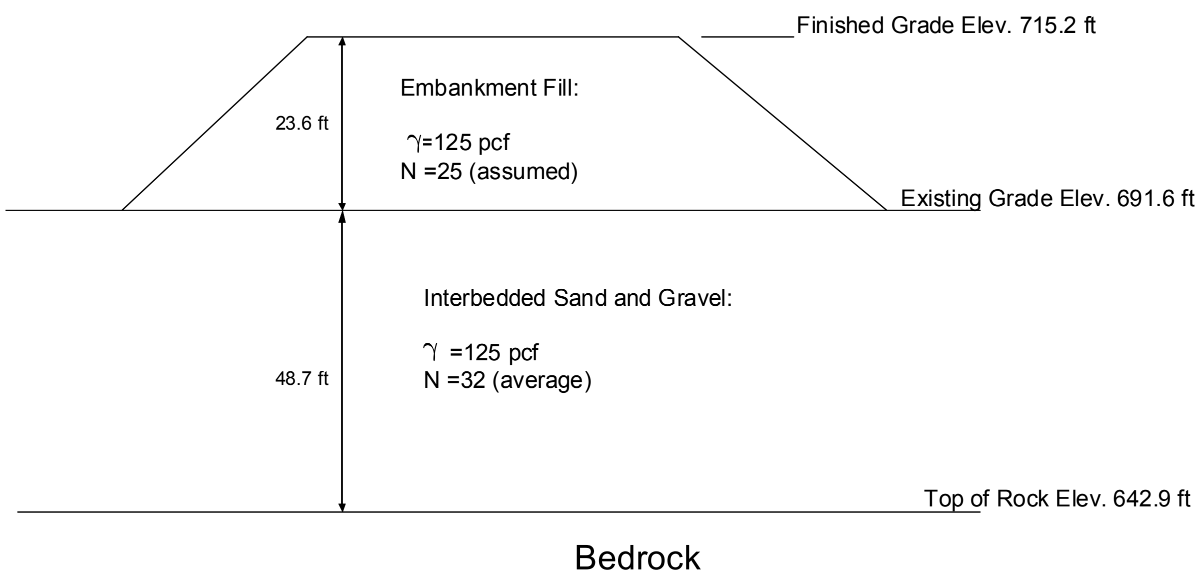
$K_a = 0.307$

Settlement Analysis:

Reference: FHWA Soils and Foundation Workshop Manual (FHWA HI-88-009) Bazaraa 1967 pg 168

Look at fill behind abutments to bring site to bridge deck elevation.
Maximum of ~24 feet of fill behind Abutment No. 1
Maximum of ~33 feet of fill behind Abutment No. 2

Abutment No. 1
Boring BB-GWR-101



Divide sand and gravel layer up into 10 ' layers:

Layer 1:	$H_1 := 10 \cdot ft$	$N_1 := 48$
Layer 2:	$H_2 := 10 \cdot ft$	$N_2 := 17$
Layer 3:	$H_3 := 10 \cdot ft$	$N_3 := 19$
Layer 4:	$H_4 := 10 \cdot ft$	$N_4 := 43$
Layer 5:	$H_5 := 8.7 \cdot ft$	$N_5 := 45$

LOADING ON AN INFINITE STRIP - VERTICAL EMBANKMENT LOADING

Project Name: Wild River Bridge Client: Gilead Project Number: 15619.00

Project Manager: JWentworth Date: 10/16/08 Computed by: km

Embank. slope a = 48.00(ft)
Embank. width b = 68.00(ft)
p load/unit area = 3000.00(psf)

INCREMENT OF STRESSES FOR Z-DIRECTION
X = 50.00(ft)

Z (ft)	Vert. Δz (psf)
0.00	3000.00
2.00	2990.62
4.00	2958.05
6.00	2910.13
8.00	2849.97
10.00	2779.44
12.00	2700.86
14.00	2616.81
16.00	2529.76
18.00	2441.83
20.00	2354.65
22.00	2269.41
24.00	2186.90
26.00	2107.62
28.00	2031.83
30.00	1959.64
32.00	1891.05
34.00	1825.97
36.00	1764.29
38.00	1705.85
40.00	1650.48
42.00	1598.03
44.00	1548.31
46.00	1501.17
48.00	1456.44
50.00	1413.97

at 5.0 ft $\Delta\sigma_{z1} := 2934.09 \cdot psf$

at 15.0 ft $\Delta\sigma_{z2} := 2573.29 \cdot psf$

at 25.0 ft $\Delta\sigma_{z3} := 2147.26 \cdot psf$

at 35.0 ft $\Delta\sigma_{z4} := 1795.13 \cdot psf$

at 44.4 ft $\Delta\sigma_{z5} := 1538.88 \cdot psf$

Height of Layer 1: $H_1 := 10 \cdot ft$

Unit weight of sand and gravel: $\gamma_{sagr} := 125 \cdot pcf$

Determine corrected SPT value N': N'/N - Ratio of Corrected blow count to SPT Value

$\sigma_{1o} := \frac{H_1}{2} \cdot \gamma_{sagr}$ $\sigma_{1o} = 625 \cdot psf$ at mid-point

SPT N-value (bpf) $N_1 = 48$ At P_o = 625 psf N'/N = $r1 := 1.75$

Corrected Blow Count $N'_1 := r1 \cdot N_1$ $N'_1 = 84$

From Figure 13 using the "well graded silty SAND & GRAVEL" curve

Bearing Capacity Index: $C1 := 300$

Use STRESS to determine the change in stress at the mid point of the layer under consideration (above)

$\Delta\sigma_{z1} = 2934.09 \cdot psf$

Height of Layer 2: $H_2 := 10 \cdot ft$

Unit weight of sand and gravel: $\gamma_{sagr} := 125 \cdot pcf$

Determine corrected SPT value N': N'/N - Ratio of Corrected blow count to SPT Value

$\sigma_{2o} := H_1 \cdot \gamma_{sagr} + \frac{H_2}{2} \cdot \gamma_{sagr}$ $\sigma_{2o} = 1875 \cdot psf$ at mid-point

SPT N-value (bpf) $N_2 = 17$ At P_o = 1875 psf N'/N = $r2 := 0.96$

Corrected Blow Count $N'_2 := r2 \cdot N_2$ $N'_2 = 16$

From Figure 13 using the "well graded silty SAND & GRAVEL" curve

Bearing Capacity Index: $C2 := 60$

Use STRESS to determine the change in stress at the mid point of the layer under consideration (above)

$\Delta\sigma_{z2} = 2573.29 \cdot psf$

Height of Layer 3: $H_3 := 10 \cdot ft$

Unit weight of sand and gravel: $\gamma_{sagr} := 125 \cdot pcf$

Determine corrected SPT value N': N'/N - Ratio of Corrected blow count to SPT Value

$\sigma_{3o} := (H_1 + H_2) \cdot \gamma_{sagr} + \frac{H_3}{2} \cdot \gamma_{sagr}$ $\sigma_{3o} = 3125 \cdot psf$ at mid-point

SPT N-value (bpf) $N_3 = 19$ At P_o = 3125 psf N'/N = $r3 := 0.83$

Corrected Blow Count $N'_3 := r3 \cdot N_3$ $N'_3 = 16$

From Figure 13 using the "well graded silty SAND & GRAVEL" curve

Bearing Capacity Index: $C3 := 60$

Use STRESS to determine the change in stress at the mid point of the layer under consideration (above)

$\Delta\sigma_{z3} = 2147.26 \cdot psf$

Height of Layer 4: $H_4 := 10 \cdot ft$

Unit weight of sand and gravel: $\gamma_{sagr} := 125 \cdot pcf$

Determine corrected SPT value N': N'/N - Ratio of Corrected blow count to SPT Value

$\sigma_{4o} := (H_1 + H_2 + H_3) \cdot \gamma_{sagr} + \frac{H_4}{2} \cdot \gamma_{sagr}$ $\sigma_{4o} = 4375 \cdot psf$ at mid-point

SPT N-value (bpf) $N_4 = 43$ At P_o = 4375 psf N'/N = $r4 := 0.72$

Corrected Blow Count $N'_4 := r4 \cdot N_4$ $N'_4 = 31$

From Figure 13 using the "well graded silty SAND & GRAVEL" curve

Bearing Capacity Index: $C4 := 105$

Use STRESS to determine the change in stress at the mid point of the layer under consideration (above)

$\Delta\sigma_{z4} = 1795.13 \cdot psf$

Height of Layer 5: $H_5 = 8.7\,ft$

Unit weight of sand and gravel: $\gamma_{sagr} := 125 \cdot pcf$

Determine corrected SPT value N': N'/N - Ratio of Corrected blow count to SPT Value

$\sigma_{5o} := (H_1 + H_2 + H_3 + H_4) \cdot \gamma_{sagr} + \frac{H_5}{2} \cdot \gamma_{sagr}$ $\sigma_{5o} = 5543.75 \cdot psf$ at mid-point

SPT N-value (bpf) $N_5 = 45$ At P_o = 5544 psf N'/N = $r5 := 0.66$

Corrected Blow Count $N'_5 := r5 \cdot N_5$ $N'_5 = 30$

From Figure 13 using the "well graded silty SAND & GRAVEL" curve

Bearing Capacity Index: $C5 := 103$

Use STRESS to determine the change in stress at the mid point of the layer under consideration (above)

$\Delta\sigma_{z5} = 1538.88 \cdot psf$

Settlement at each layer Intebedded sand and gravel:

$$\Delta H_1 := H_1 \cdot \frac{1}{C1} \cdot \log\left(\frac{\sigma_{1o} + \Delta\sigma_{z1}}{\sigma_{1o}}\right) \qquad \Delta H_1 = 0.3 \cdot in$$

$$\Delta H_2 := H_2 \cdot \frac{1}{C2} \cdot \log\left(\frac{\sigma_{2o} + \Delta\sigma_{z2}}{\sigma_{2o}}\right) \qquad \Delta H_2 = 0.75 \cdot in$$

$$\Delta H_3 := H_3 \cdot \frac{1}{C3} \cdot \log\left(\frac{\sigma_{3o} + \Delta\sigma_{z3}}{\sigma_{3o}}\right) \qquad \Delta H_3 = 0.45 \cdot in$$

$$\Delta H_4 := H_4 \cdot \frac{1}{C4} \cdot \log\left(\frac{\sigma_{4o} + \Delta\sigma_{z4}}{\sigma_{4o}}\right) \qquad \Delta H_4 = 0.17 \cdot in$$

$$\Delta H_5 := H_5 \cdot \frac{1}{C5} \cdot \log\left(\frac{\sigma_{5o} + \Delta\sigma_{z5}}{\sigma_{5o}}\right) \qquad \Delta H_5 = 0.11 \cdot in$$

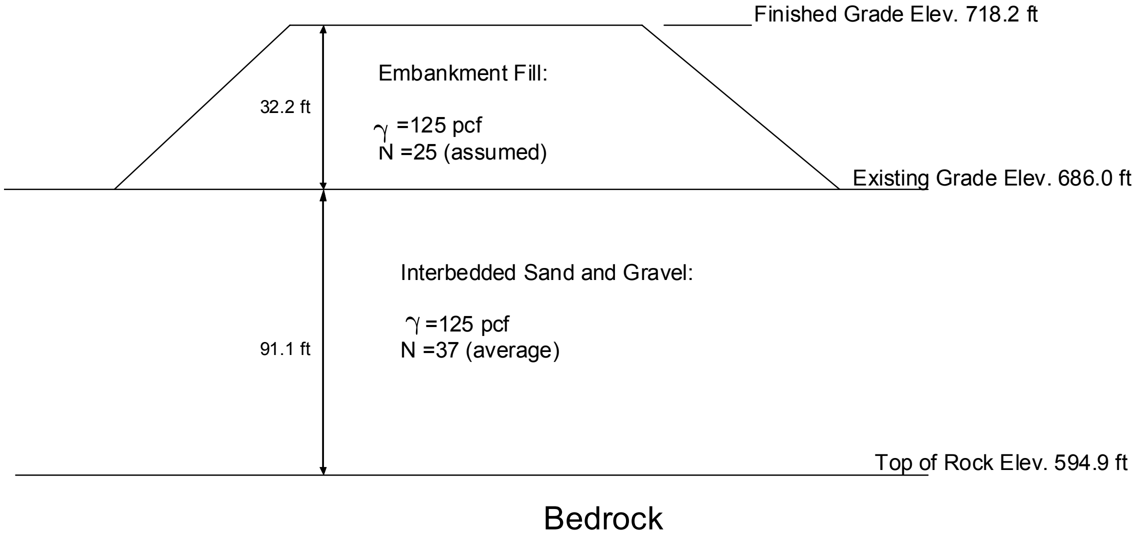
Total settlement =

$\Delta H_{A1} := \Delta H_1 + \Delta H_2 + \Delta H_3 + \Delta H_4 + \Delta H_5$ $\Delta H_{A1} = 1.7853 \cdot in$ At Abutment No. 1

$\Delta H_{A1} = 45.3478 \cdot mm$

Abutment No. 2

Boring BB-GWR-103



Divide sand and gravel layer up into 10 ' layers:

Layer 1:	$H_1 := 10 \cdot ft$	$N_1 := 28$
Layer 2:	$H_2 := 10 \cdot ft$	$N_2 := 58$
Layer 3:	$H_3 := 10 \cdot ft$	$N_3 := 28$
Layer 4:	$H_4 := 10 \cdot ft$	$N_4 := 26$
Layer 5:	$H_5 := 10 \cdot ft$	$N_5 := 33$
Layer 6:	$H_6 := 10 \cdot ft$	$N_6 := 29$
Layer 7:	$H_7 := 10 \cdot ft$	$N_7 := 37$
Layer 8:	$H_8 := 10 \cdot ft$	$N_8 := 45$
Layer 9:	$H_9 := 11.1 \cdot ft$	$N_9 := 43$

LOADING ON AN INFINITE STRIP - VERTICAL EMBANKMENT LOADING

Project Name: Wild River Bridge   Client: Gilead   Project Number: 15619.00
Project Manager : JWentworth   Date: 10/18/08   Computed by : km

Embank. slope a = 48.00(ft)
Embank. width b = 68.00(ft)
p load/unit area = 3000.00(psf)

INCREMENT OF STRESSES FOR Z-DIRECTION
X = 50.00(ft)

Z (ft)	Vert. Δz (psf)		
0.00	3000.00		
2.00	2990.62		
4.00	2958.05	at 5.0 ft	$\Delta\sigma_{z1} := 2934.09 \cdot psf$
6.00	2910.13		
8.00	2849.97		
10.00	2779.44		
12.00	2700.86		
14.00	2616.81	at 15.0 ft	$\Delta\sigma_{z2} := 2573.29 \cdot psf$
16.00	2529.76		
18.00	2441.83		
20.00	2354.65		
22.00	2269.41		
24.00	2186.90	at 25.0 ft	$\Delta\sigma_{z3} := 2147.26 \cdot psf$
26.00	2107.62		
28.00	2031.83		
30.00	1959.64		
32.00	1891.05		
34.00	1825.97	at 35.0 ft	$\Delta\sigma_{z4} := 1795.13 \cdot psf$
36.00	1764.29		
38.00	1705.85		
40.00	1650.48		
42.00	1598.03		
44.00	1548.31	at 45.0 ft	$\Delta\sigma_{z5} := 1524.74 \cdot psf$
46.00	1501.17		
48.00	1456.44		
50.00	1413.97		
52.00	1373.63		
54.00	1335.27	at 55.0 ft	$\Delta\sigma_{z6} := 1317.02 \cdot psf$
56.00	1298.77		
58.00	1264.01		
60.00	1230.89		
62.00	1199.30		
64.00	1169.15		
66.00	1140.36	at 65.0 ft	$\Delta\sigma_{z7} := 1154.76 \cdot psf$
68.00	1112.84		
70.00	1086.51		
72.00	1061.30		
74.00	1037.16		
76.00	1014.02	at 75.0 ft	$\Delta\sigma_{z8} := 1025.59 \cdot psf$
78.00	991.81		
80.00	970.50		
82.00	950.03		
84.00	930.35		
86.00	911.43		
88.00	893.21		
90.00	875.68	at 88.2 ft	$\Delta\sigma_{z9} := 891.46 \cdot psf$
92.00	858.78		

Height of Layer 1: $H_1 := 10 \cdot ft$

Unit weight of sand and gravel: $\gamma_{sagr} := 125 \cdot pcf$

Determine corrected SPT value N': N'/N - Ratio of Corrected blow count to SPT Value

$\sigma_{1o} := \frac{H_1}{2} \cdot \gamma_{sagr}$ $\sigma_{1o} = 625 \cdot psf$ at mid-point

SPT N-value (bpf) $N_1 = 28$ At P_o = 625 psf N'/N = $r1 := 1.75$

Corrected Blow Count $N'_1 := r1 \cdot N_1$ $N'_1 = 49$

From Figure 13 using the "well graded silty SAND & GRAVEL" curve

Bearing Capacity Index: $C1 := 165$

Use STRESS to determine the change in stress at the mid point of the layer under consideration (above)

$\Delta\sigma_{z1} = 2934.09 \cdot psf$

Height of Layer 2: $H_2 := 10 \cdot ft$

Unit weight of sand and gravel: $\gamma_{sagr} := 125 \cdot pcf$

Determine corrected SPT value N': N'/N - Ratio of Corrected blow count to SPT Value

$\sigma_{2o} := H_1 \cdot \gamma_{sagr} + \frac{H_2}{2} \cdot \gamma_{sagr}$ $\sigma_{2o} = 1875 \cdot psf$ at mid-point

SPT N-value (bpf) $N_2 = 58$ At P_o = 1875 psf N'/N = $r2 := 0.96$

Corrected Blow Count $N'_2 := r2 \cdot N_2$ $N'_2 = 56$

From Figure 13 using the "well graded silty SAND & GRAVEL" curve
Bearing Capacity Index: $C2 := 200$

Use STRESS to determine the change in stress at the mid point of the layer under consideration (above)

$\Delta\sigma_{z2} = 2573.29 \cdot psf$

Height of Layer 3: $H_3 := 10 \cdot ft$

Unit weight of sand and gravel: $\gamma_{sagr} := 125 \cdot pcf$

Determine corrected SPT value N': N'/N - Ratio of Corrected blow count to SPT Value

$\sigma_{3o} := (H_1 + H_2) \cdot \gamma_{sagr} + \frac{H_3}{2} \cdot \gamma_{sagr}$ $\sigma_{3o} = 3125 \cdot psf$ at mid-point

SPT N-value (bpf) $N_3 = 28$ At P_o = 3125 psf N'/N = $r3 := 0.83$

Corrected Blow Count $N'_3 := r3 \cdot N_3$ $N'_3 = 23$

From Figure 13 using the "well graded silty SAND & GRAVEL" curve

Bearing Capacity Index: $C3 := 85$

Use STRESS to determine the change in stress at the mid point of the layer under consideration (above)

$\Delta\sigma_{z3} = 2147.26 \cdot psf$

Height of Layer 4: $H_4 := 10 \cdot ft$

Unit weight of sand and gravel: $\gamma_{sagr} := 125 \cdot pcf$

Determine corrected SPT value N': N'/N - Ratio of Corrected blow count to SPT Value

$$\sigma_{4o} := (H_1 + H_2 + H_3) \cdot \gamma_{sagr} + \frac{H_4}{2} \cdot \gamma_{sagr} \qquad \sigma_{4o} = 4375 \cdot psf \qquad \text{at mid-point}$$

SPT N-value (bpf) $N_4 = 26$ At P_o = 4375 psf N'/N = $r4 := 0.72$

Corrected Blow Count $N'_4 := r4 \cdot N_4$ $N'_4 = 19$

From Figure 13 using the "well graded silty SAND & GRAVEL" curve

Bearing Capacity Index: $C4 := 75$

Use STRESS to determine the change in stress at the mid point of the layer under consideration (above)

$$\Delta\sigma_{z4} = 1795.13 \cdot psf$$

Height of Layer 5: $H_5 = 10\,ft$

Unit weight of sand and gravel: $\gamma_{sagr} := 125 \cdot pcf$

Determine corrected SPT value N': N'/N - Ratio of Corrected blow count to SPT Value

$$\sigma_{5o} := (H_1 + H_2 + H_3 + H_4) \cdot \gamma_{sagr} + \frac{H_5}{2} \cdot \gamma_{sagr} \qquad \sigma_{5o} = 5625 \cdot psf \qquad \text{at mid-point}$$

SPT N-value (bpf) $N_5 = 33$ At P_o = 5625 psf N'/N = $r5 := 0.65$

Corrected Blow Count $N'_5 := r5 \cdot N_5$ $N'_5 = 21$

From Figure 13 using the "well graded silty SAND & GRAVEL" curve

Bearing Capacity Index: $C5 := 80$

Use STRESS to determine the change in stress at the mid point of the layer under consideration (above)

$$\Delta\sigma_{z5} = 1524.74 \cdot psf$$

Height of Layer 6: $H_6 = 10\,ft$

Unit weight of sand and gravel: $\gamma_{sagr} := 125 \cdot pcf$

Determine corrected SPT value N': N'/N - Ratio of Corrected blow count to SPT Value

$$\sigma_{6o} := (H_1 + H_2 + H_3 + H_4 + H_5) \cdot \gamma_{sagr} + \frac{H_6}{2} \cdot \gamma_{sagr} \qquad \sigma_{6o} = 6875 \cdot psf \qquad \text{at mid-point}$$

SPT N-value (bpf) $N_6 = 29$ At P_o = 6875 psf N'/N = $r6 := 0.60$

Corrected Blow Count $N'_6 := r6 \cdot N_6$ $N'_6 = 17$

From Figure 13 using the "well graded silty SAND & GRAVEL" curve

Bearing Capacity Index: $C6 := 70$

Use STRESS to determine the change in stress at the mid point of the layer under consideration (above)

$$\Delta\sigma_{z6} = 1317.02 \cdot psf$$

Height of Layer 7: $H_7 = 10\,ft$

Unit weight of sand and gravel: $\gamma_{sagr} := 125 \cdot pcf$

Determine corrected SPT value N': N'/N - Ratio of Corrected blow count to SPT Value

$$\sigma_{7o} := (H_1 + H_2 + H_3 + H_4 + H_5 + H_6) \cdot \gamma_{sagr} + \frac{H_7}{2} \cdot \gamma_{sagr} \qquad \sigma_{7o} = 8125 \cdot psf$$ at mid-point

SPT N-value (bpf) $N_7 = 37$ At P_o = 8125 psf N'/N = $r7 := 0.60$

Corrected Blow Count $N'_7 := r7 \cdot N_7$ $N'_7 = 22$

From Figure 13 using the "well graded silty SAND & GRAVEL" curve

Bearing Capacity Index: $C7 := 82$

Use STRESS to determine the change in stress at the mid point of the layer under consideration (above)

$$\Delta\sigma_{z7} = 1154.76 \cdot psf$$

Height of Layer 8: $H_8 = 10\,ft$

Unit weight of sand and gravel: $\gamma_{sagr} := 125 \cdot pcf$

Determine corrected SPT value N': N'/N - Ratio of Corrected blow count to SPT Value

$$\sigma_{8o} := (H_1 + H_2 + H_3 + H_4 + H_5 + H_6 + H_7) \cdot \gamma_{sagr} + \frac{H_8}{2} \cdot \gamma_{sagr} \qquad \sigma_{8o} = 9375 \cdot psf$$ at mid-point

SPT N-value (bpf) $N_8 = 45$ At P_o = 9375 psf N'/N = $r8 := 0.60$

Corrected Blow Count $N'_8 := r8 \cdot N_8$ $N'_8 = 27$

From Figure 13 using the "well graded silty SAND & GRAVEL" curve

Bearing Capacity Index: $C8 := 95$

Use STRESS to determine the change in stress at the mid point of the layer under consideration (above)

$$\Delta\sigma_{z8} = 1025.59 \cdot psf$$

Height of Layer 9: $H_9 = 11.1\,ft$

Unit weight of sand and gravel: $\gamma_{sagr} := 125 \cdot pcf$

Determine corrected SPT value N': N'/N - Ratio of Corrected blow count to SPT Value

$$\sigma_{9o} := (H_1 + H_2 + H_3 + H_4 + H_5 + H_6 + H_7 + H_8) \cdot \gamma_{sagr} + \frac{H_9}{2} \cdot \gamma_{sagr} \qquad \sigma_{9o} = 10693.75 \cdot psf$$ at mid-point

SPT N-value (bpf) $N_9 = 43$ At P_o = 10690 psf N'/N = $r9 := 0.60$

Corrected Blow Count $N'_9 := r9 \cdot N_9$ $N'_9 = 26$

From Figure 13 using the "well graded silty SAND & GRAVEL" curve

Bearing Capacity Index: $C9 := 93$

Use STRESS to determine the change in stress at the mid point of the layer under consideration (above)

$$\Delta\sigma_{z9} = 891.46 \cdot psf$$

Settlement at each layer Intebedded sand and gravel:

$$\Delta H_1 := H_1 \cdot \frac{1}{C1} \cdot \log\left(\frac{\sigma_{1o} + \Delta\sigma_{z1}}{\sigma_{1o}}\right) \qquad \Delta H_1 = 0.55 \cdot in$$

$$\Delta H_2 := H_2 \cdot \frac{1}{C2} \cdot \log\left(\frac{\sigma_{2o} + \Delta\sigma_{z2}}{\sigma_{2o}}\right) \qquad \Delta H_2 = 0.23 \cdot in$$

$$\Delta H_3 := H_3 \cdot \frac{1}{C3} \cdot \log\left(\frac{\sigma_{3o} + \Delta\sigma_{z3}}{\sigma_{3o}}\right) \qquad \Delta H_3 = 0.32 \cdot in$$

$$\Delta H_4 := H_4 \cdot \frac{1}{C4} \cdot \log\left(\frac{\sigma_{4o} + \Delta\sigma_{z4}}{\sigma_{4o}}\right) \qquad \Delta H_4 = 0.24 \cdot in$$

$$\Delta H_5 := H_5 \cdot \frac{1}{C5} \cdot \log\left(\frac{\sigma_{5o} + \Delta\sigma_{z5}}{\sigma_{5o}}\right) \qquad \Delta H_5 = 0.16 \cdot in$$

$$\Delta H_6 := H_6 \cdot \frac{1}{C6} \cdot \log\left(\frac{\sigma_{6o} + \Delta\sigma_{z6}}{\sigma_{6o}}\right) \qquad \Delta H_6 = 0.13 \cdot in$$

$$\Delta H_7 := H_7 \cdot \frac{1}{C7} \cdot \log\left(\frac{\sigma_{7o} + \Delta\sigma_{z7}}{\sigma_{7o}}\right) \qquad \Delta H_7 = 0.08 \cdot in$$

$$\Delta H_8 := H_8 \cdot \frac{1}{C8} \cdot \log\left(\frac{\sigma_{8o} + \Delta\sigma_{z8}}{\sigma_{8o}}\right) \qquad \Delta H_8 = 0.06 \cdot in$$

$$\Delta H_9 := H_9 \cdot \frac{1}{C9} \cdot \log\left(\frac{\sigma_{9o} + \Delta\sigma_{z9}}{\sigma_{9o}}\right) \qquad \Delta H_9 = 0.05 \cdot in$$

Total settlement =

$$\Delta H_{A2} := \Delta H_1 + \Delta H_2 + \Delta H_3 + \Delta H_4 + \Delta H_5 + \Delta H_6 + \Delta H_7 + \Delta H_8 + \Delta H_9$$

$\Delta H_{A2} = 1.8121 \cdot in$ At Abutment No. 2

$\Delta H_{A2} = 46.0268 \cdot mm$

Frost Protection:

Method 1 - MaineDOT Design Freezing Index (DFI) Map and Depth of Frost Penetration Table are in BDG Section 5.2.1.

From the Design Freezing Index Map:
Gilead, Maine
DFI = 1550 degree-days

From the lab testing: soils are coarse grained assume a water content = ~10%

From Table 5-1 MaineDOT BDG for Design Freezing Index of 1550 frost penetration = 83.5 inches

$\text{Frost\_depth} := 83.5\text{in}$ $\text{Frost\_depth} = 6.9583 \cdot \text{ft}$ $\text{Frost\_depth} = 2.1209 \cdot \text{m}$

Note: The final depth of footing embedment may be controlled by the scour susceptibility of the foundation material and may, in fact, be deeper than the depth required for frost protection.

Method 2 - Check Frost Depth using Modberg Software

Closest Station is Rumford

ModBerg Results

Project Location: Rumford 1 SSE, Maine

Air Design Freezing Index = 1631 F-days
N-Factor = 0.80
Surface Design Freezing Index = 1305 F-days
Mean Annual Temperature = 43.5 deg F
Design Length of Freezing Season = 136 days

Layer #:Type	t	w%	d	Cf	Cu	Kf	Ku	L
1-Coarse	78.5	10.0	125.0	28	34	2.0	1.6	1,800

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

Total Depth of Frost Penetration = 6.54 ft = 78.5 in.

$\text{Frost\_depth}_{\text{modberg}} := 78.5 \cdot \text{in}$ $\text{Frost\_depth}_{\text{modberg}} = 6.5417\,\text{ft}$

$\text{Frost\_depth}_{\text{modberg}} = 1.9939 \cdot \text{m}$

Use Modberg Frost Depth = 2.0 meters for design

Seismic:

Gilead Wild River Bridge PIN 15619.00
Date and Time: 11/17/2008 4:22:07 PM

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years
State - Maine
Zip Code - 04217
Zip Code Latitude = 44.407000
Zip Code Longitude = -070.790000
Site Class B
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.090	PGA - Site Class B
0.2	0.183	Ss - Site Class B
1.0	0.050	S1 - Site Class B

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Spectral Response Accelerations SDs and SD1
State - Maine
Zip Code - 04217
Zip Code Latitude = 44.407000
Zip Code Longitude = -070.790000
As = FpgaPGA, SDs = FaSs, and SD1 = FvS1
Site Class D - Fpga = 1.60, Fa = 1.60, Fv = 2.40
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.144	As - Site Class D
0.2	0.293	SDs - Site Class D
1.0	0.119	SD1 - Site Class D