MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

WEBB RIVER BRIDGE
OVER WEBB RIVER
AND
RECONSTRUCTION OF US ROUTE 2/STATE ROUTE 17
AND LEAVITT STREET
MEXICO AND DIXFIELD, MAINE

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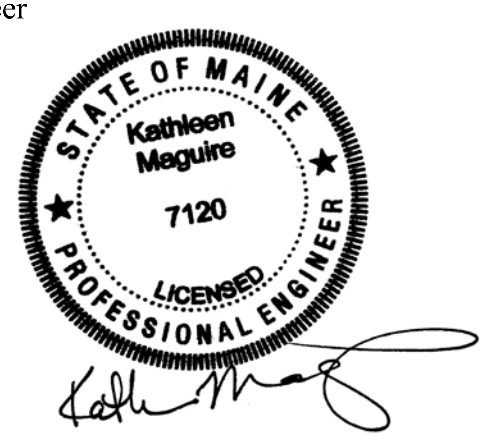

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GEOTECHNICAL DESIGN SUMMARY

The purpose of this design report is to make geotechnical recommendations for the replacement of Webb River Bridge over Webb River and reconstruction of 0.22 miles of US Route 2/State Route 17 and Leavitt Street in Mexico and Dixfield, Maine. The proposed replacement bridge will consist of steel superstructure on semi-integral stub abutments founded on soil behind the existing abutments (to remain in place). Cantilever retaining walls to the south of the existing bridge will be used to retain the soils to support the proposed abutments. The following design recommendations are discussed in detail in the attached report:

Frost Protection - Foundations placed on the bedrock surface will not require a minimum depth of embedment for frost protection. Any foundations placed on native subgrade soil should be founded a minimum of 6.0 feet below finished exterior grade for frost protection.

Abutment Subgrade Preparation - Abutment spread footings shall be constructed on a bed of select granular borrow 2.0 feet thick, placed in 8-inch maximum lifts. Backfill material shall meet the requirements of MaineDOT 703.19 Granular Borrow Material for Underwater Backfill. Granular borrow shall be placed in 8-inch lifts and compacted to 95% of AASHTO T-180.

Semi-integral Stub Abutment Bearing Resistance – The semi-integral stub abutments will be founded on granular fill soils behind the existing abutments which will remain in place. Bearing resistance for any structure founded on granular soils shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 14 ksf. A factored bearing resistance of 6 ksf may be used when analyzing the service limit state and for preliminary footing sizing. In no instance shall the factored bearing stress exceed the nominal resistance of the footing concrete, which is taken as $0.3f'c$. No footing shall be less than 2 feet wide regardless of the applied bearing pressure.

Repointing and Repair of Existing Abutments - The existing abutments are to be left in place as protection for the abutments on spread footings with concrete slopes constructed to the tops of the partially demolished, existing abutments. The condition of the existing concrete and granite masonry abutments should be improved. The Project Plan Notes should include repairing and patching areas of old concrete substructures that are spalling or cracked. Requirements for lateral support and global stability of foundations on spread footings also dictate that the existing dry laid granite block masonry be repointed or blocks reset, as required, to ensure serviceability. The interface contact of the bottom course of granite blocks and concrete footings with the subgrade bedrock should be examined and improved, if necessary.

Settlement – Due to the granular nature of the fill soils, settlements are anticipated to occur during construction and be less than 1.0 inch. The cantilever retaining walls are anticipated to be founded on bedrock and will not experience post-construction settlements.

Scour – The consequences of changes in foundation conditions resulting from the design flood for scour shall be considered at the strength and service limit states. The bedrock at the site is not anticipated to be erodible.

Semi-integral Stub Abutments - The bottom of footing elevation for Abutment No. 1 is anticipated to be approximately 407.5 feet. The bottom of footing elevation for Abutment No. 2 is anticipated to be approximately 407.0 feet. The footings on granular fill soils shall be designed for all applicable load combinations specified in AASHTO LRFD Bridge Design Specifications Fourth Edition (LRFD) Articles 3.4.1 and 11.5.5. The design of abutments founded on spread footings at the strength limit state shall consider factored bearing resistance, overturning (eccentricity), lateral sliding and structural failure. Strength limit state design shall also consider foundation resistance after scour due to the design flood. At the service limit state spread footing design shall be assessed for: settlement, horizontal movement, overall stability and scour at the design flood. The overall stability of the foundation should be investigated at the Service I Load Combination. Abutments shall be designed as unrestrained meaning that they are free to rotate at the top in an active state of earth pressure. Additional lateral earth pressure due to construction surcharge or live load surcharge is required for the return wings when traffic loads are located within a horizontal distance equal to one-half of the wall height behind the back of the wall. Use of an approach slab may be required. All abutment and return wingwall designs shall include a drainage system behind the abutments to intercept any groundwater. To avoid water intrusion behind the abutment the approach slab should be connected directly to the abutment.

Cantilever Type Retaining Walls - Cantilever type retaining walls founded on bedrock as extensions from the existing gravity abutments will be used on the south side of the bridge (downstream) to retain the earth supporting the semi-integral stub abutments. Concrete slope paving will be placed between the proposed and existing abutments to minimize scour potential. Cast-in-place retaining walls shall be designed as unrestrained meaning 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 walls (K_a = 0.307) and Coulomb Theory for gravity shaped structures (K_a = 0.276). Additional lateral earth pressure due to construction surcharge or live load surcharge is required. Bearing resistance for any structure founded on bedrock shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 16 ksf. A factored bearing resistance of 20 ksf may be used when analyzing the service limit state and for preliminary footing sizing. In no instance shall the factored bearing stress exceed the nominal resistance of the footing concrete, which is taken as $0.3f'c$. No footing shall be less than 2 feet wide regardless of the applied bearing pressure. For footings on bedrock, the eccentricity of loading at the strength limit state, based on factored loads shall not exceed three-eighths ($3/8^{ths}$) of the footing dimensions in either direction. The design of walls founded on spread footings at the strength limit state shall consider nominal bearing resistance, overturning (eccentricity), lateral sliding and structural failure. Strength limit state design shall also consider foundation resistance after scour due to the design flood.

Precast Concrete Block Gravity Retaining Walls - Two Precast Concrete Block Gravity retaining walls are proposed for the project. The walls shall be designed in accordance with the relevant Special Provision 635 by a Professional Engineer subcontracted by the Contractor as a design-build item. The walls 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 walls at the strength limit state shall consider nominal bearing resistance, overturning (eccentricity), lateral sliding and structural failure. The design of spread footings at the service limit state shall include settlement, horizontal movement and overall stability. Extreme limit state design checks for spread footings shall include bearing resistance, eccentricity, sliding and overall stability. The overall stability of the wall system should be investigated at the Service I Load Combination. For the bottom concrete block unit on leveling pad on bedrock, the eccentricity of loading as the strength limit state, based on factored loads, shall not exceed three-eights ($3/8^{ths}$) of the footing dimensions, in either direction.

Seismic Design - Webb River Bridge on US Route 2/State Route 17 is on the National Highway System (NHS) and is considered to be functionally important. The site is assigned to Site Class D and Seismic Zone 1. The LRFD code states that single span bridges need not be analyzed for seismic loads regardless of their seismic zone. The minimum requirements as specified in LRFD Articles 4.7.4.2 and 3.10.9.2 apply.

Construction Considerations - Boulders and cobbles were encountered within the existing abutment backfill in both of the borings. There is potential for these obstructions to impact excavation efforts for construction of the semi-integral stub abutments. If the abutment footing subgrade soil is found to contain cobbles or boulders, the Contractor shall remove any cobbles or boulders larger than 6 inches in diameter and replace with compacted gravel borrow. If encountered, unsuitable soils should also be excavated from the footing subgrade to a depth if 1.0 foot and replaced with compacted gravel borrow. The gravel borrow should be compacted, along with the entire footing subgrade, to 95% of AASHTO T-180. Construction activities may include rock excavation in the retaining walls areas. Excavation of bedrock materials may require drilling and blasting techniques.

1.0 INTRODUCTION

A subsurface investigation and geotechnical design for the replacement of Webb River Bridge over the Webb River and reconstruction of 0.22 miles of US Route 2/State Route 17 and Leavitt Street in Mexico and Dixfield, 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 and reconstruction of US Route 2/State Route 17 and Leavitt Street. This report presents the soils information obtained at the site, geotechnical design recommendations, and foundation recommendations.

The existing bridge was constructed in 1931 with portions of the substructure predating 1931 and consists of a 90 foot long, two-span, concrete T-beam superstructure supported on mass abutments and a cast-in-place concrete pier all founded on bedrock. The east abutment is comprised of mortared granite masonry which was widened with cast-in-place concrete in 1931. The west abutment is a cast-in-place concrete abutment constructed in 1931. Maine Department of Transportation (MaineDOT) maintenance inspection reports indicate that the bridge deck is in “satisfactory” condition (rating of 6), the bridge superstructure is in “fair” condition (rating of 5) and the substructure is in “fair” condition (rating of 5). Year 2007 MaineDOT Bridge Maintenance inspection reports indicate a Bridge Sufficiency Rating of 49.1. The bridge has a scour critical rating of 8 meaning that the bridge foundations have been determined to be stable for the assessed or calculated scour condition. It is understood that the existing bridge superstructure and substructures will be completely removed and replaced.

The proposed bridge has been designed by HNTB, Inc. of Westbrook, Maine and will consist of a 115 foot long, single-span, steel, welded plate girder superstructure with a composite structural concrete slab supported on semi-integral stub abutments founded on soil behind the location of the existing abutments which will remain in place. Concrete slope paving will be placed between the proposed and existing abutments to minimize scour potential. Cantilever retaining walls will be constructed as extensions of the existing abutments to retain the earth supporting the semi-integral abutments. The proposed horizontal alignment of the bridge will be located approximately 12 feet downstream (south) of the current alignment. Two Precast Concrete Block Gravity retaining walls are proposed as a part of the project one along US Route 2/State Route 17 and one along Leavitt Street.

2.0 GEOLOGIC SETTING

Webb River Bridge on US Route 2/State Route 17 in Mexico and Dixfield crosses the Webb River at the town line as shown on Sheet 1 - Location Map found at the end of this report. The Webb River flows in a southerly direction to the Androscoggin River just south of the bridge location.

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 glacial outwash deposits. Soils in the site area are generally comprised of sand and gravel. The unit generally is deposited in areas where the topography is flat to gently sloping. These soils were generally deposited by glacial meltwater streams in front of the receding late Wisconsinan ice margin.

According to the Surficial Bedrock Map of Maine, published by the Maine Geological Survey (1985), the bedrock at the site is identified as Devonian muscovite-diorite granodiorite.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions were explored by drilling ten (10) test borings at the site. Bridge test boring BB-MDWR-101 was drilled behind the location of Abutment No. 1 (west). Bridge test borings BB-MDWR-102 and BB-MDWR-102A were drilled behind the location of Abutment No. 2 (east). The bridge exploration locations and an interpretive subsurface profile depicting the site stratigraphy at the bridge location are shown on Sheet 2 - Boring Location Plan and Interpretive Subsurface Profile found at the end of this report. Highway test borings HB-MDR2-101 through HB-MDR2-105 were drilled along US Route 2/State Route 17. Highway test borings HB-MDLS-101 and HB-MDLS-102 were drilled on Leavitt Street. The highway exploration locations are shown on Sheets 4 through 6 - Geoplans found at the end of this report. The borings were drilled between March 3 and 12, 2009 using the MaineDOT drill rig and Northern Test Boring (NTB) 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 3 and 7 - Boring Logs found at the 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 5-foot intervals using Standard Penetration Test (SPT) methods. During SPT sampling, the sampler is driven 24 inches and the hammer blows for each 6 inch interval of penetration are recorded. The standard penetration resistance, N-value, is the sum of the blows for the second and third intervals. Both of drill rigs used at the site are equipped with automatic hammers to drive the split spoon. The hammers were calibrated February of 2009. The MaineDOT automatic hammer was found to deliver approximately 40 percent more energy during driving than the standard rope and cathead system. The NTB automatic hammer was found to deliver approximately 13 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 to the raw field N-values. These hammer efficiency factors (0.84 for MaineDOT and 0.68 for NTB) 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 and/or a Certified Subsurface Inspector selected the boring locations and drilling methods, designated type and depth of sampling techniques, identified field testing requirements and logged the subsurface conditions encountered. The borings were located in the field by survey during drilling activities.

4.0 SUBSURFACE CONDITIONS AT ABUTMENTS

The general soil stratigraphy encountered at the abutments consisted of fill materials overlying a thin veneer of sand over bedrock. An interpretive subsurface profile depicting the bridge site stratigraphy is show on Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile found at the end of this report. The following paragraphs discuss the subsurface conditions encountered at each abutment:

Abutment No. 1 (Boring BB-MDWR-101) - Beneath the pavement, a layer of fill materials was encountered behind Abutment No. 1. This layer was found to be light brown, damp to moist, fine and fine to medium SAND, with little silt, and trace to little gravel. The thickness of the fill layer was approximately 20.8 feet at the boring location. Corrected SPT N-values in the fill layer ranged from 3 to >50 blows per foot (bpf) indicating that the soil is loose to very dense in consistency. Underlying the fill material a layer of cobbles and boulders within a soil matrix was encountered. The thickness of the cobbles, boulders and soil was approximately 2.5 feet at the boring location. Bedrock was encountered beneath the cobbles and boulders at a depth of 23.3 feet below ground surface (bgs). The bedrock details are presented below.

Abutment No. 2 (Borings BB-MDWR-102 and BB-MDWR-102A) - Beneath the pavement, a layer of fill materials was encountered behind Abutment No. 2. This layer was found to be brown, dry to moist, fine to coarse SAND, with some gravel and trace silt with occasional layers of cobbles and boulders. The thickness of the fill layer was approximately 19.2 feet at the boring location. Corrected SPT N-values in the fill layer ranged from 24 to >50 blows per foot (bpf) indicating that the soil is medium dense to very dense in consistency. Underlying the fill material, a layer of sand was encountered. This layer was found to be brown, wet, very dense fine SAND with trace silt and gravel. The thickness of the sand was approximately 1.6 feet at the boring location. Bedrock was encountered beneath the sand at a depth of 20.8 feet bgs. The bedrock details are presented below.

Bedrock. Bedrock was encountered and cored in the borings that reached bedrock. Table 1 below presents the bedrock findings:

Boring Number/ Location	Depth to Bedrock	Bedrock Elevation	RQD
BB-MDWR-101/ Abutment No. 1	23.3 feet	397.40 feet	38 – 73%
BB-MDWR-102A Abutment No. 2	20.8 feet	399.6 feet	88%

Table 1 – Summary of Bedrock Depths, Elevations and RQD at Abutment Locations

The bedrock at the site can be identified as grey and white GNEISS with some banding and mica. The RQD of the bedrock was determined to range from 38 to 88% indicating a rock mass quality of poor to good.

5.0 SUBSURFACE CONDITIONS ALONG US ROUTE 2/STATE ROUTE 17

A total of 5 borings were drilled along US Route 2/State Route 17 to investigate the depth to bedrock along the roadway. The subsurface conditions encountered along US Route 2/State Route 17 consisted of brown, damp to wet, medium dense to very dense, fine to coarse SAND, with trace to some gravel and trace to some silt. Corrected SPT N-values in the sand along US Route2/State Route 17 ranged from 11 to >50 blows per foot (bpf) indicating that the soil is medium dense to very dense in consistency. Bedrock was encountered below the sand at varying depths ranging from 3.5 to 19.8 feet bgs. The bedrock was not cored in boring BB-MDR2-103 due to the depth of the bedrock. It was determined in the field that the roadway reconstruction would not encounter bedrock at this depth, therefore coring was not necessary. The bedrock along the roadway is identified as grey and white GNEISS with some banding and mica. The RQD of the bedrock along the roadway was determined to range from 0 to 96% indicating a rock mass quality of very poor to excellent. Table 2 below presents a summary of the boring information.

Boring Number	Roadway Elevation	Depth to Bedrock	Bedrock Elevation	RQD
HB-MDR2-101	432.0 feet	6.5 feet	425.5 feet	0%
HB-MDR2-102	435.8 feet	3.5 feet	432.3 feet	68%
HB-MDR2-103	438.0 feet	19.8 feet	418.2 feet	N/A
HB-MDR2-104	439.3 feet	5.9 feet	433.4 feet	96%
HB-MDR2-105	433.2 feet	3.9 feet	429.3 feet	69%

Table 2 - Summary of Roadway Elevations, Bedrock Depths, Bedrock Elevations and RQD along US Route 2/State Route 17

6.0 SUBSURFACE CONDITIONS ALONG LEAVITT STREET

A total of 2 borings were drilled along Leavitt Street to investigate the depth to bedrock along the side road. The subsurface conditions encountered along Leavitt Street consisted of brown , damp to wet, medium dense to very dense, fine to coarse SAND, with trace to some gravel and trace to some silt. Corrected SPT N-values in the sand along Leavitt Street ranged from 19 to >50 blows per foot (bpf) indicating that the soil is medium dense to very dense in consistency. Bedrock was encountered below the sand at depths ranging from 9.8 to 14.4 feet bgs. The bedrock along the side road is identified as grey and white GNEISS with some banding and mica. The RQD of the bedrock along the side road ranged from 71 to 79% indicating a rock mass quality of fair to good. Table 3 below presents a summary of the boring information.

Boring Number	Roadway Elevation	Depth to Bedrock	Bedrock Elevation	RQD
HB-MDLS-101	441.8 feet	9.8 feet	432.0 feet	79%
HB-MDLS-102	450.1 feet	14.4 feet	435.7 feet	71%

Table 3 - Summary of Roadway Elevations, Bedrock Depths, Bedrock Elevations and RQD along Leavitt Street

7.0 BRIDGE FOUNDATION ALTERNATIVES

MaineDOT has contracted HNTB, Inc. of Westbrook, Maine to design the replacement structure for the Webb River Bridge. During the Preliminary Design Report (PDR) development phase of the project, HNTB, Inc. evaluated a total of four foundation alternatives for this project:

- Reuse of the existing abutments
- Pile supported integral abutments
- Full height cantilever abutments founded on bedrock
- Semi-integral stub abutments founded on soil behind the existing abutments to remain in place

The first three alternatives listed were eliminated by HNTB, Inc. during the PDR phase. The use of semi-integral stub abutments founded on soil behind the existing abutments to remain in place was chosen as the most viable foundation for the site. Cantilever retaining walls founded on bedrock as extensions from the existing gravity abutments will be used on the south side of the bridge (downstream) to retain the earth supporting the semi-integral stub abutments. Concrete slope paving will be placed between the proposed and existing abutments to minimize scour potential. This report addresses only these foundation types.

8.0 FOUNDATION CONSIDERATIONS AND RECOMMENDATIONS

The following sections will discuss geotechnical design recommendations for semi-integral stub abutments founded on soil behind the existing abutments to remain in place and cantilever retaining walls founded on bedrock which have been identified as the optimal foundation types for the project.

8.1 Frost Protection

It is anticipated that the semi-integral stub abutments will be founded on fill soil behind the existing abutments which will remain in place. All foundations 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 Bridge Design Guide [BDG] Figure 5-1); the site has a design-freezing index of approximately 1700 F-degree days. This correlates to a frost depth of 6.0 feet. Therefore, any foundations placed on granular soils should be founded a minimum of 6.0 feet below finished exterior grade for frost protection. This minimum embedment depth applies only to foundations placed on subgrade soils and not those founded on bedrock. See Appendix B- Calculations at the end of this report for supporting documentation.

It is anticipated that the cantilever retaining wall footings will be founded on bedrock. For foundations on bedrock, heave due to frost is not a design issue and no requirements for minimum depth of embedment are necessary.

8.2 Abutment Subgrade Preparation

Abutment spread footings shall be constructed on a bed of select gravel borrow, 2.0 feet thick, placed in 8-inch maximum lifts. Backfill material shall meet the requirements of MaineDOT 703.19 Granular Borrow Material for Underwater Backfill. Granular borrow shall be placed in 8-inch lifts and compacted to 95% of AASHTO T-180.

8.3 Abutment Bearing Resistance

It is anticipated that the semi-integral stub abutments at the site will be founded on granular fill soils behind the existing abutments which will remain in place. Applicable permanent and transient loads are specified in AASHTO LFRD Bridge Design Specifications Fourth Edition (LRFD) Article 11.5.5. Abutment footings shall be proportioned to provide stability against bearing capacity failure.

As the semi-integral stub abutments are to be supported on granular soils the vertical stress shall be calculated assuming a linearly distributed pressure over an effective base area as shown in LRFD Figure 11.6.3.2-1. Bearing resistance for any structure founded on granular soils shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 14 ksf. The bearing resistance factor, ϕ_b, for spread footings on soil is 0.45. A factored bearing resistance of 6 ksf may be used when analyzing the service limit state and for preliminary footing sizing assuming a resistance factor of 1.0. See Appendix B - Calculations for supporting documentation.

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

8.4 Repointing and Repair of Existing Abutments

The existing abutments are to be left in place as protection for the proposed abutments on spread footings with concrete slopes constructed to the tops of the partially demolished, existing abutments. The proposed bridge design will rely on the existing abutments and wingwalls to provide lateral support and scour protection for the abutment spread footings constructed in the approach fills.

The condition of the existing concrete and granite masonry abutments should be improved. The Project Plan Notes should include repairing and patching areas of old concrete substructures that are spalling or cracked. Requirements for lateral support and global stability of foundations on spread footings also dictate that the existing dry laid granite block masonry be repointed or blocks reset, as required, to ensure serviceability.

The interface contact of the bottom course of granite blocks and concrete footings with the subgrade bedrock should be examined and improved, if necessary. Contract Documents should include a contingency item for injection grouting at the toe of the existing abutments if any portion is undermined or compromised.

8.5 Settlement

As the spread footings for the abutments will be founded on compacted granular soil, post-construction settlements are anticipated to be less than 1.0 inch. Widening of the existing roadway is anticipated to the south of the structure. Due to the granular nature of the fill soils settlements are anticipated to occur during construction having negligible effect on the finished bridge structure. The cantilever retaining walls are anticipated to be founded on bedrock and will not experience post-construction settlements.

8.6 Scour

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.

The bedrock at the site is not anticipated to be erodable. For scour protection, any footings constructed on granular deposits should be embedded a minimum of 2.0 feet below the design scour depth and armored with 3.0 feet of riprap underlain by an erosion control geotextile. Refer to MaineDOT BDG Section 2.3.11 for information regarding scour design.

8.7 Semi-integral Stub Abutments

The cast-in-place, semi-integral stub abutments will be placed on spread footings on granular fill soils behind the existing abutments (to remain). The bottom of footing elevation for Abutment No. 1 is anticipated to be approximately 407.5 feet. The bottom of footing elevation for Abutment No. 2 is anticipated to be approximately 407.0 feet.

The footings on granular fill soils shall be designed for all applicable load combinations specified in LRFD Articles 3.4.1 and 11.5.5. The design of abutments founded on spread footings at the strength limit state shall consider factored bearing resistance, overturning (eccentricity), lateral sliding and structural failure. Strength limit state design shall also consider foundation resistance after scour due to the design flood.

Per LRFD Table 10.5.5.2.2-1, a sliding resistance factor, ϕ_τ, of 0.80 shall be applied to the nominal sliding resistance of cast-in-place concrete footings on sand. Sliding computations for resistances to lateral loads shall assume a maximum frictional coefficient of 0.45 at the footing-soil interface.

For spread footings on soil, the eccentricity of loading at the strength limit state shall not exceed one-fourth ($1/4^{th}$) of the effective footing dimensions.

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

Semi-integral abutments should typically be designed for active earth pressure over the abutment height and a uniform pressure distribution due to the height of soil behind the superstructure. The superstructure backwall should typically be designed for full passive pressure only. However, the Designer may elect a more conservative approach and design the abutment stem wall to withstand a passive earth pressure state. In designing for active pressure, a Rankine active earth pressure coefficient, K_a, of 0.307 is recommended. In designing for passive earth pressure, 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 semi-integral abutments. Additional lateral earth pressure due to construction surcharge or live load surcharge is required per Section 3.6.8 of the MaineDOT BDG for the return wings when traffic loads are located within a horizontal distance equal to one-half of the wall height behind the back of the wall. 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 on abutments is permitted per LRFD Article 3.11.6.5. The live load surcharge may be estimated as a uniform horizontal earth pressure due to an equivalent height (h_{eq}) taken from Table 4 below:

Abutment Height	h_{eq}
5 feet	4.0 feet
10 feet	3.0 feet
≥20 feet	2.0 feet

Table 4 – Equivalent Height of Soil for Vehicular Loading on Abutments

The Designer may assume Soil Type 4 (MaineDOT BDG Section 3.6.1) for abutment backfill material soil properties. The backfill properties are as follows: ϕ = 32 degrees, γ = 125 pcf.

All abutment designs shall include a drainage system behind the abutments to intercept any groundwater. Drainage behind structure shall be in accordance with Section 5.4.1.4 of the MaineDOT BDG. To avoid water intrusion behind the abutment the approach slab should be connected directly to the abutment.

8.8 Cantilever Type Retaining Walls

Cantilever type retaining walls founded on bedrock as extensions from the existing gravity abutments will be used on the south side of the bridge (downstream) to retain the earth supporting the semi-integral stub abutments. Concrete slope paving will be placed between the proposed and existing abutments with new wingwalls to minimize scour potential.

Cast-in-place retaining walls shall be designed as unrestrained meaning free to rotate at the top in an active state of earth pressure. Earth loads shall be calculated using as active earth pressure coefficient, K_a, calculated using Rankine Theory for cantilever walls (K_a = 0.307) and Coulomb Theory for gravity shaped structures (K_a = 0.276). Additional lateral earth pressure due to construction surcharge or live load surcharge is required per Section 3.6.8 of the MaineDOT BDG. The live load surcharge may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) taken from Table 5 below:

Wall Height (feet)	h_{eq} (feet)	
	Distance from wall backface to edge of traffic = 0 feet	Distance from wall backface to edge of traffic ≥ 1 foot
5	5.0	2.0
10	3.5	2.0
≥20	2.0	2.0

Table 5 – Equivalent Height of Soil for Vehicular Loading on Retaining Walls

Bearing resistance for any structure founded on bedrock shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 16 ksf. The bearing resistance factor, ϕ_b, for spread footings on bedrock is 0.45. A factored bearing resistance of 20 ksf may be used when analyzing the service limit state and for preliminary footing sizing assuming a resistance factor of 1.0. See Appendix B - Calculations for supporting documentation. In no instance shall the factored bearing stress exceed the nominal resistance of the footing concrete, which is taken as 0.3*f'c*. No footing shall be less than 2 feet wide regardless of the applied bearing pressure.

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

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 = 125 pcf. Sliding computations for resistance to lateral loads shall assume a maximum allowable frictional coefficient of 0.70 at the bedrock-concrete interface. A sliding resistance factor of ϕ_τ=0.9 shall be applied to the nominal sliding resistance of walls founded on spread footings on bedrock.

The design of walls founded on spread footings at the strength limit state shall consider nominal bearing resistance, overturning (eccentricity), lateral sliding and structural failure. Strength limit state design shall also consider foundation resistance after scour due to the design flood.

8.9 Precast Concrete Block Gravity Retaining Walls

Two Precast Concrete Block Gravity retaining walls are proposed for the project. Retaining Wall #1 is planned on the north side of US Route 2 from Station 0+92.32 to Station 1+73.00. Retaining Wall #2 is planned on the east side of Leavitt Street from Station 1+97.43 to Station 2+93.53. The project plans will allow either a solid block wall or an aggregate filled block wall. The walls shall be designed in accordance with Special Provision 635 for the relevant wall system by a Professional Engineer subcontracted by the Contractor as a design-build item. Special Provisions for both wall systems are included in Appendix C found at the end of this report.

The PCBG walls 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 walls at the strength limit state shall consider nominal bearing resistance, overturning (eccentricity), lateral sliding and structural failure.
A resistance factor of ϕ= 1.0 shall be used to assess spread footing design at the service limit state including: settlement, horizontal movement and overall stability. Extreme limit state design checks for spread footings shall include bearing resistance, eccentricity, sliding and overall stability. A resistance factor of ϕ=1.0 shall be used for the extreme limit state. The overall stability of the wall system should be investigated at the Service I Load Combination with a resistance factor, ϕ, of 0.65.

The bearing resistance for the block wall founded on a leveling pad founded on bedrock shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 16 ksf. The stress distribution may be assumed to be a linear distribution over the effective footing base as shown in LRFD Figure 11.6.3.2-2. Based on presumptive bearing resistances values, a factored bearing resistance of 20 ksf may be used to control settlement when analyzing service limit state load combinations and for preliminary footing sizing. See Appendix B – Calculations for supporting documentation.

Failure by sliding shall be investigated by the wall subcontractor. A sliding resistance factor, ϕ_τ, of 0.90 shall be applied to the nominal sliding resistance of the portion of precast concrete blocks founded on leveling pads cast on bedrock and the aggregate within the precast concrete blocks in contact with leveling pads cast on bedrock. Sliding computations for resistance to lateral loads shall assume a maximum frictional coefficient of 0.46 (0.80 x tan 30^o) at the leveling pad to concrete block interfaces and a maximum frictional coefficient of 0.58(tan 30^o) at the leveling pad to aggregate in-fill interfaces. Recommended values of sliding frictional coefficients are based on LRFD Articles 10.6.3.4, 11.11.4.2 and Table 3.11.5.3-1.

For the lowest block unit on bedrock or leveling pad, the eccentricity of loading at the strength limit state, based on factored loads, shall not exceed three-eights ($3/8^{ths}$) of the footing dimensions, in either direction.

Any irregularities in the existing bedrock surface or irregularities created during the excavation process will be backfilled with un-reinforced Fill Concrete during the concrete placement for the wall leveling pad.

8.10 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) = 0.086g
- Short-term (0.2-second period) spectral acceleration coefficient = 0.177g
- Long-term (1.0-second period) spectral acceleration coefficient = 0.049g

According to Figure 2-2 of the MaineDOT BDG, Webb River Bridge on US Route 2/State Route 17 is on the National Highway System (NHS) and is therefore considered to be functionally important. Per LRFD Article 3.10.3.1 the site is assigned to Site Class D due to the presence of soils 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 0.118 (LRFD Eq. 3.10.4.2-6). Per LRFD Article 4.7.4.2 single span bridges need not be analyzed for seismic loads regardless of their seismic zone. However, the minimum requirements for superstructure connections and bridge seat dimensions as specified in LRFD Articles 4.7.4.2 and 3.10.9.2 apply.

8.11 Backfill Material

Backfill within 10 feet of the structure and fill materials shall conform to MaineDOT Specification 703.19 - Granular Borrow for Underwater Backfill. This gradation specifies that 10 percent or less of the material may pass the No. 200 sieve. This material is also specified in order to reduce the amount of fines and to minimize frost action behind the structure.

8.12 Construction Considerations

Boulders and cobbles were encountered within the existing abutment backfill in both of the borings. There is potential for these obstructions to impact excavation efforts for construction of the semi-integral stub abutments. Obstructions may be cleared by conventional excavation methods. Care should be taken replace any materials with compacted structural fill.

If the abutment footing subgrade soil is found to contain cobbles or boulders, the Contractor shall remove any cobbles or boulders larger than 6 inches in diameter and replace with compacted gravel borrow. If encountered, unsuitable soils should also be excavated from the footing subgrade to a depth if 1.0 foot and replaced with compacted gravel borrow. The gravel borrow should be compacted, along with the entire footing subgrade, to 95% of AASHTO T-180.

Construction activities may include rock excavation in the retaining walls areas. Excavation of bedrock materials may require drilling and blasting techniques. Blasting should be done in accordance with Section 105.2.6 of the MaineDOT Standard Specifications. The Contractor may need to conduct pre-and post-blast surveys in accordance with industry standards. All loose and fractured rock and soil debris should be removed from bearing surfaces before concrete is placed. It is likely that there will be seepage of water from fractures and joints exposed in the bedrock surface and cut slopes. Water should be controlled by pumping from sumps. The Contractor should maintain the excavation so that all foundations are constructed in the dry.

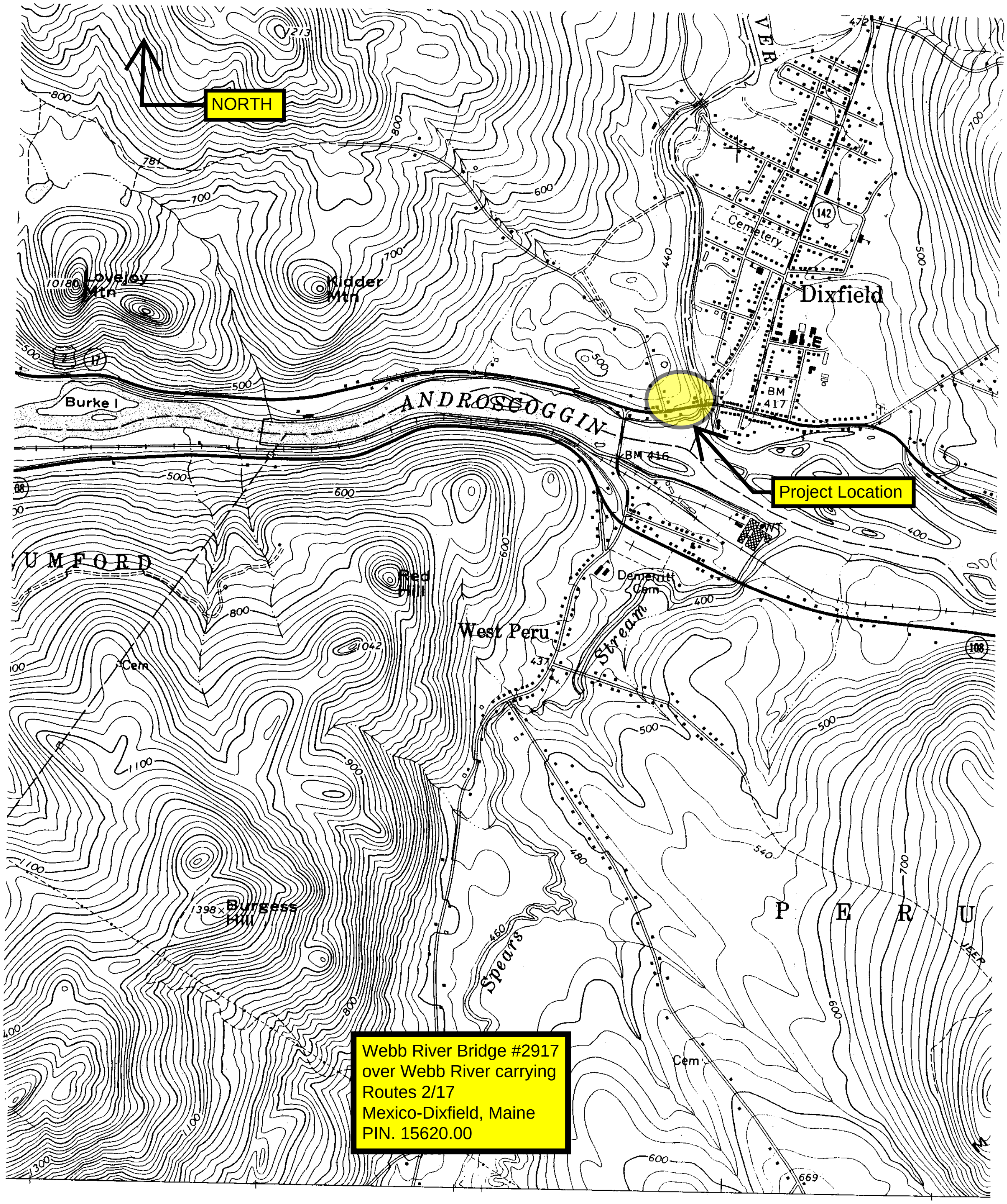
It is recommended that a person qualified by training and experience be present to inspect the condition of the bedrock bearing surfaces prior to pouring of the seal concrete.

9.0 CLOSURE

This report has been prepared for the use of the HNTB, Inc. and the MaineDOT Bridge Program for specific application to the proposed replacement of Webb River Bridge and reconstruction of 0.22 miles of US Route 2/State Route 17 and Leavitt Street in Mexico and Dixfield, 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



DIXFIELD QUADRANGLE

MAINE

7.5 MINUTE SERIES (TOPOGRAPHIC)

SW/4 DIXFIELD 15' QUADRANGLE

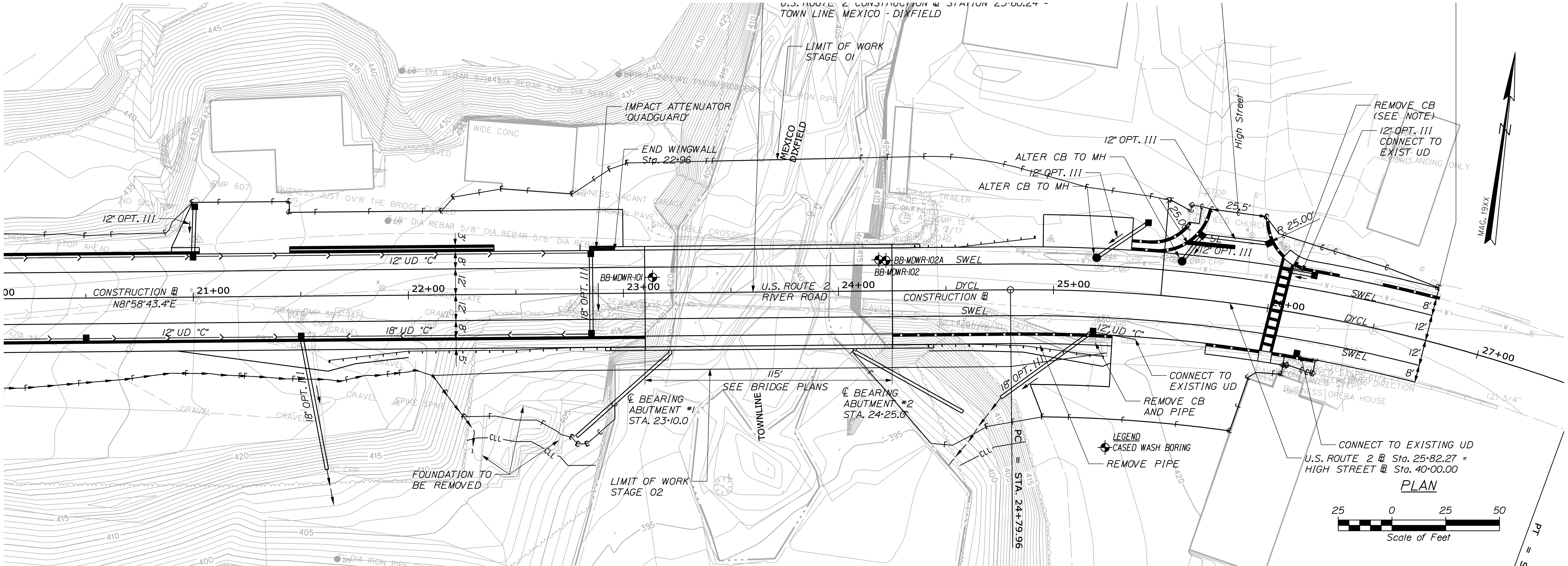
SCALE 1:24000

1 ½ 0 1 MILE

1000 0 1000 2000 3000 4000 5000 6000 7000 FEET

1 .5 0 1 KILOMETER

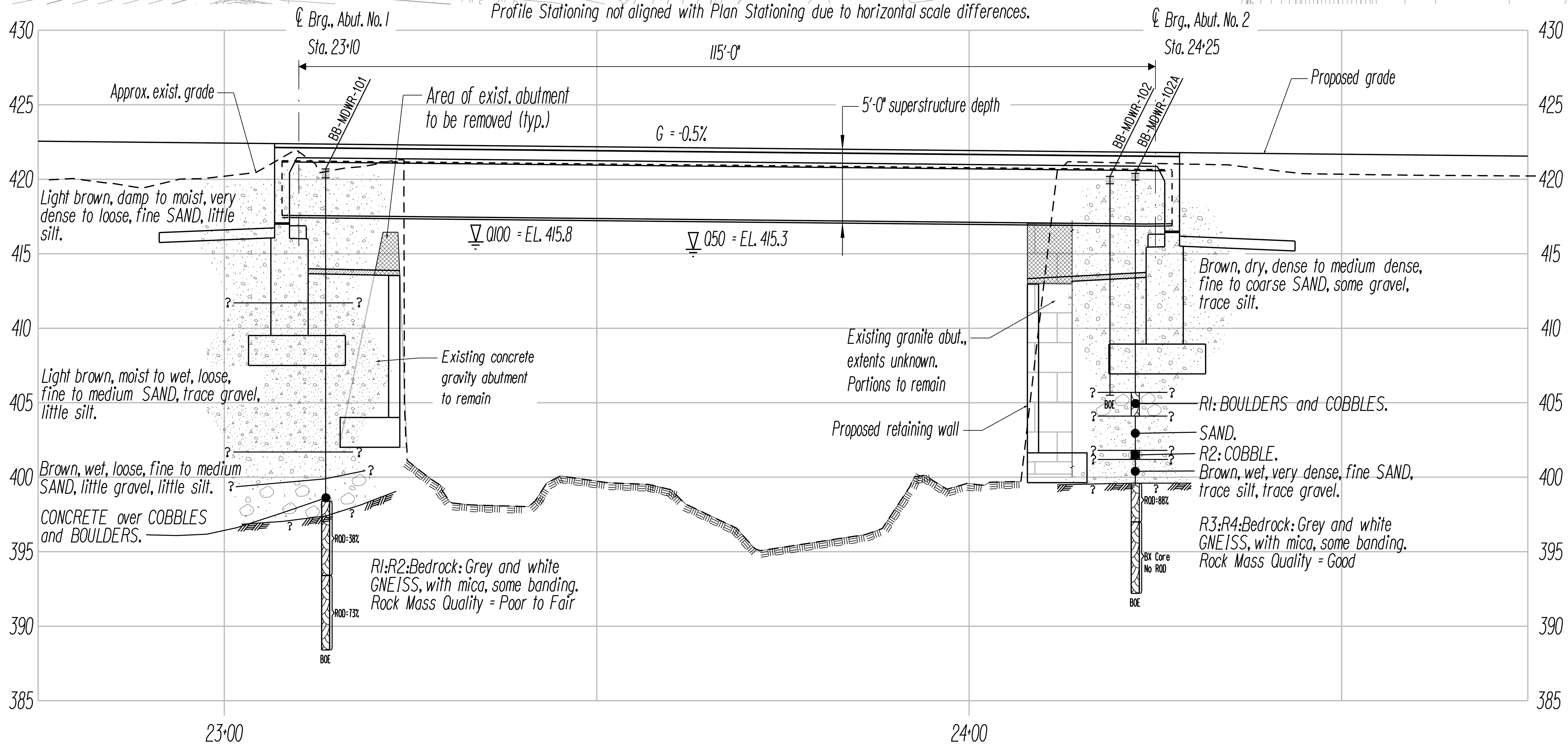
Filename: ...\GEOTECH\MSTA\002_BLP&SP1.dgn Division: GEOTECH Username: terry.white Date:4/28/2009



PLAN

25 0 25 50

Scale of Feet



PROFILE

HORIZ 10 0 10 20

VERT 5 0 5 10

SCALE

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

BR-A562(000)E

PIN 15620.00

BRIDGE NO. 2917

BRIDGE PLANS

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED	K. MAGUIRE	MAR 2009
CHECKED-REVIEWED	T. WHITE	
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

WEBB RIVER BRIDGE
WEBB RIVER
MEXICO-DIXFIELD OXFORD COUNTY

BORING LOCATION PLAN &
INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER

2

OF 7

Filename: ...\msta\003_BORING LOGS B1.dgn   Division: GEOTECH   Username: terry.white   Date:4/28/2009

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

Project: Webb River Bridge #2917 over Webb River carring Routes 2/17	Location: Mexico-Dixfield, Maine	Boring No.: BB-MDWR-101	PIN: 15620.00

Driller:	MaineDOT	Elevation (ft.)	420.7	Auger ID/OD:	5" Solid Stem
Operator:	E. Giguere/C. Giles	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	3/10/09, 3/12/09	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	23+13.6, 7.4 Lt.	Casing ID/OD:	HW	Water Level*:	15.0' bgs.
Hammer Efficiency Factor: 0.84		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	420.10		Pavement (0.60)	
	1D	12/12	1.00 - 2.00	20/53	---					Light brown, damp (frozen), very dense, fine SAND, little silt.	
5	2D	24/22	5.00 - 7.00	1/2/2/2	4	6				Light brown, moist, loose, fine SAND, little silt.	
								411.70		9.00	
10	3D	24/22	10.00 - 12.00	2/2/3/3	5	7	25			Light brown, moist, loose, fine to medium SAND, trace gravel, little silt.	
							24				
							22				
							22				
	4D	24/14	14.00 - 16.00	3/2/3/3	5	7	10			Similar to above, but wet.	
15							14				
							15				
							12				
							12				
	5D	21.6/16	19.00 - 20.80	2/1/1/40(3.6")	2	3	10	401.70		19.00 Brown, wet, loose, fine to medium SAND, little gravel, little silt. a40 blows for 0.8'.	
20	R1	78/60	20.80 - 27.30	RQD = 38%			a40 NQ-2	399.90		20.80 CONCRETE over COBBLES and BOULDERS. R1:Core Times (min:sec) 20.8-21.8' (2:25) 21.8-22.8' (0:15) 22.8-23.8' (0:56)	
								397.40		23.30 Top of Bedrock at Elev. 397.4'. Bedrock: Grey and white, GNEISS with mica, some banding. 23.8-24.8' (2:49) 24.8-25.8' (2:56) 25.8-26.8' (3:40) 26.8-27.3' (3:10) 76% Recovery Rock Mass Quality = Poor.	
25	R2	60/60	27.30 - 32.30	RQD = 73%						Bedrock: Grey and white, GNEISS with mica, some banding. R2:Core Times (min:sec) 27.3-28.3' (3:15) 28.3-29.3' (3:22) 29.3-30.3' (3:20) 30.3-31.3' (3:15) 31.3-32.3' (2:50) 100% recovery Rock Mass Quality = Fair.	
30								388.40		32.30 Bottom of Exploration at 32.30 feet below ground surface.	
35											
40											
45											
50											

Remarks:

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

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

Page 1 of 1

Boring No.: BB-MDWR-101

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

Project: Webb River Bridge #2917 over Webb River carring Routes 2/17	Location: Mexico-Dixfield, Maine	Boring No.: BB-MDWR-102	PIN: 15620.00

Driller:	Northern Test Boring; Inc.	Elevation (ft.)	420.2	Auger ID/OD:	5" Solid Stem
Operator:	Nick V./Mike B.	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	E. Giguere	Rig Type:	Dietrich D50	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	3/3/09; 09:30-10:45	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	24+18.9, 14.6 Lt.	Casing ID/OD:	HW	Water Level*:	None Observed
Hammer Efficiency Factor: 0.84		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	419.70		Pavement (0.50)	
	1D	24/13	2.00 - 4.00	12/14/10/3	24	34				Brown, dry, dense, fine to coarse SAND, some gravel, trace silt.	
5	2D	24/8	5.00 - 7.00	3/8/9/4	17	24	4			Similar to above, medium dense.	
							27				
							43				
							41				
							49				
10	3D	24/4	10.00 - 12.00	3/8/12/14	20	28	51			Similar to above.	
							43				
							101				
	4D	11/0	13.80 - 14.72	32/42/50(0")	---		a125			a125 blows for 0.7'. No sample recovery.	
15								405.50		14.70 Bottom of Exploration at 14.70 feet below ground surface. CASING BROKE, MOVED TO BB-MDWR-102A.	
20											
25											

Remarks:

Left 5.0 feet of casing in ground.

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-MDWR-102

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

Project: Webb River Bridge #2917 over Webb River carring Routes 2/17	Location: Mexico-Dixfield, Maine	Boring No.: BB-MDWR-102A	PIN: 15620.00

Driller:	MaineDOT/Northern Test Boring	Elevation (ft.)	420.4	Auger ID/OD:	5" Solid Stem
Operator:	E. Giguere/C. Giles	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	3/10/09, 3/12/09	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2" & BX
Boring Location:	24+22.3, 14.2 Lt.	Casing ID/OD:	HW & NW	Water Level*:	19.0' bgs.
Hammer Efficiency Factor: 0.68		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	419.95		Pavement (0.45) See Boring BB-MDWR-102 for material description in upper 14.7 feet of boring.	
5											
10											
	R1	19.2/ 19.2	14.70 - 16.30				27 NQ-2	405.70		14.70 COBBLES and BOULDERS within SAND matrix. R1:Core Times (min:sec) 14.7-15.7' (8:00) 15.7-16.3' (3:00) 100% Recovery	
15							8				
							19 RC	404.10		16.30 Roller Coned ahead to 18.6' bgs.	
							57				
	R2	7.2/0	18.60 - 19.20				63 NQ-2	401.80		18.60 COBBLE. R2:Core Times (min:sec) 18.6-19.2' (5:00) 0% Recovery Changed to NW Casing at 19.0' bgs.	
	1D	13.2/12	19.20 - 20.30	16/18/30(1.2")	---		47	401.20		19.20 Brown, wet, very dense, fine SAND, trace silt, trace gravel. Roller Coned ahead to 20.8' bgs.	
20	R3	31.2/ 31.2	20.80 - 23.40	RQD = 88%			NQ-2	399.60		20.80 Top of Bedrock at Elev. 399.6'. Bedrock: Grey and white, GNEISS with mica, some banding. R3:Core Times (min:sec) 20.8-21.8' (3:10) 21.8-22.8' (3:35) 22.8-23.4' (4:41) 100% Recovery Rock Mass Quality = Good.	
	R4	57.6/ 57.6	23.40 - 28.20	RQD = N/A%			BX			Bedrock: Grey and white, GNEISS with mica, some banding. R4:Core Times (min:sec) 23.4-24.4' (4:25) 24.4-25.4' (3:15) 25.4-26.4' (4:40) 26.4-27.4' (4:00) 27.4-28.2' (3:38) 100% Recovery Core Blocked BX core used - No RQD Calculated.	
25								392.20		28.20 Bottom of Exploration at 28.20 feet below ground surface.	
30											
35											
40											
45											
50											

Remarks:

All samples were with Auto Hammer #149.

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-MDWR-102A

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BR-A562(000)E

BRIDGE NO. 2917   **PIN 15620.00**   BRIDGE PLANS

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED	K. MAGUIRE / T. WHITE	MAR 2009
CHECKED-REVIEWED		
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

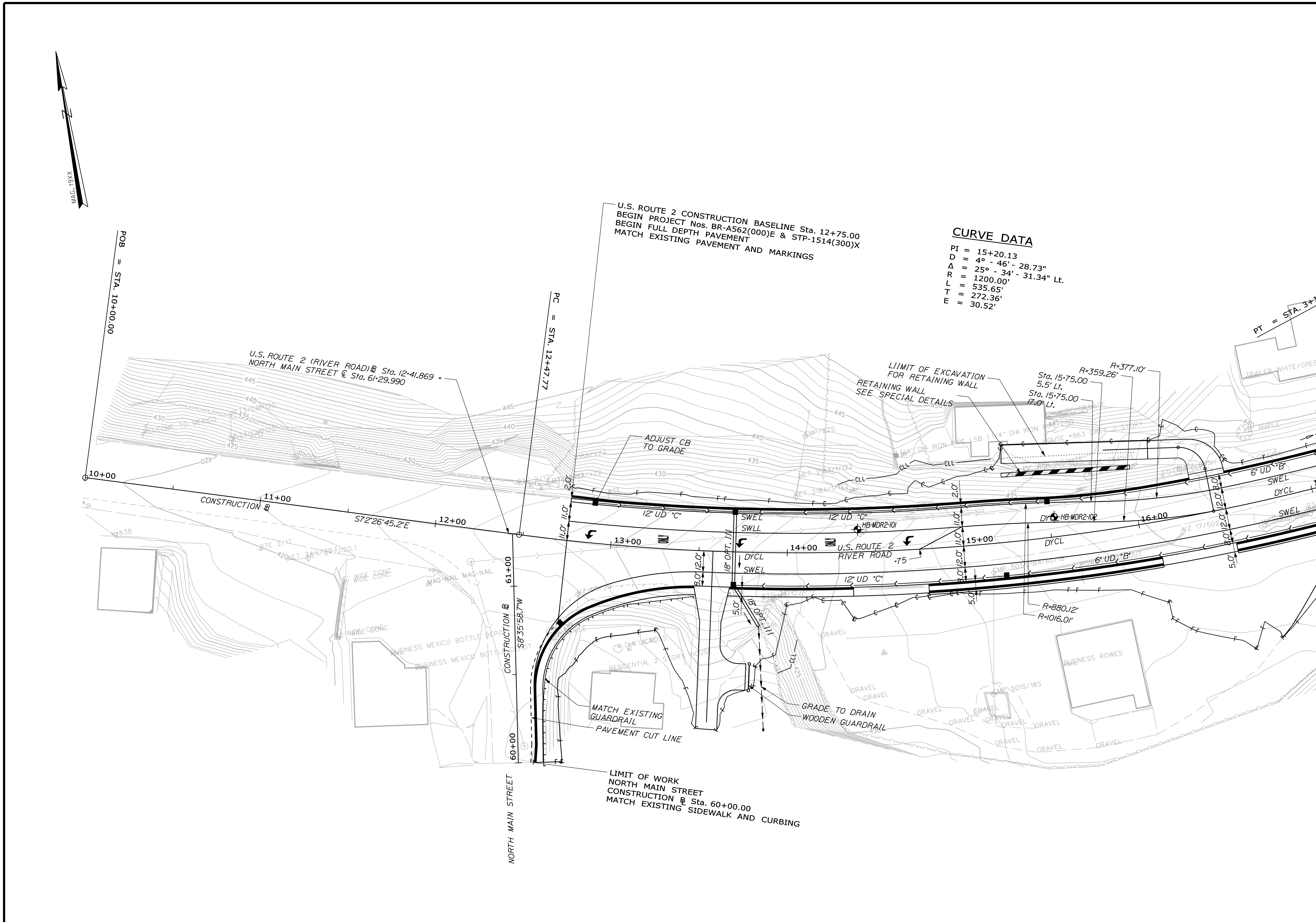
DATE

WEBB RIVER BRIDGE
WEBB RIVER
MEXICO-DIXFIELD   OXFORD COUNTY

BORING LOGS

SHEET NUMBER
3
OF 7

Filename: ...\geotech\vmsto\004_Geoplan1.dgn Division: GEOTECH Username: terry.white Date:4/28/2009



STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BR-A562(000)E

PIN 15620.00

HIGHWAY PLANS

	BY	DATE	SIGNATURE
PROJ. MANAGER			
DESIGN-DETAILED	K. MAGUIRE		
CHECKED-REVIEWED	T. WHITE	MAR 2009	
DESIGN2-DETAILED2			P.E. NUMBER
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			DATE
REVISIONS 4			
FIELD CHANGES			

MEXICO-DIXFIELD
ROUTES 2/17

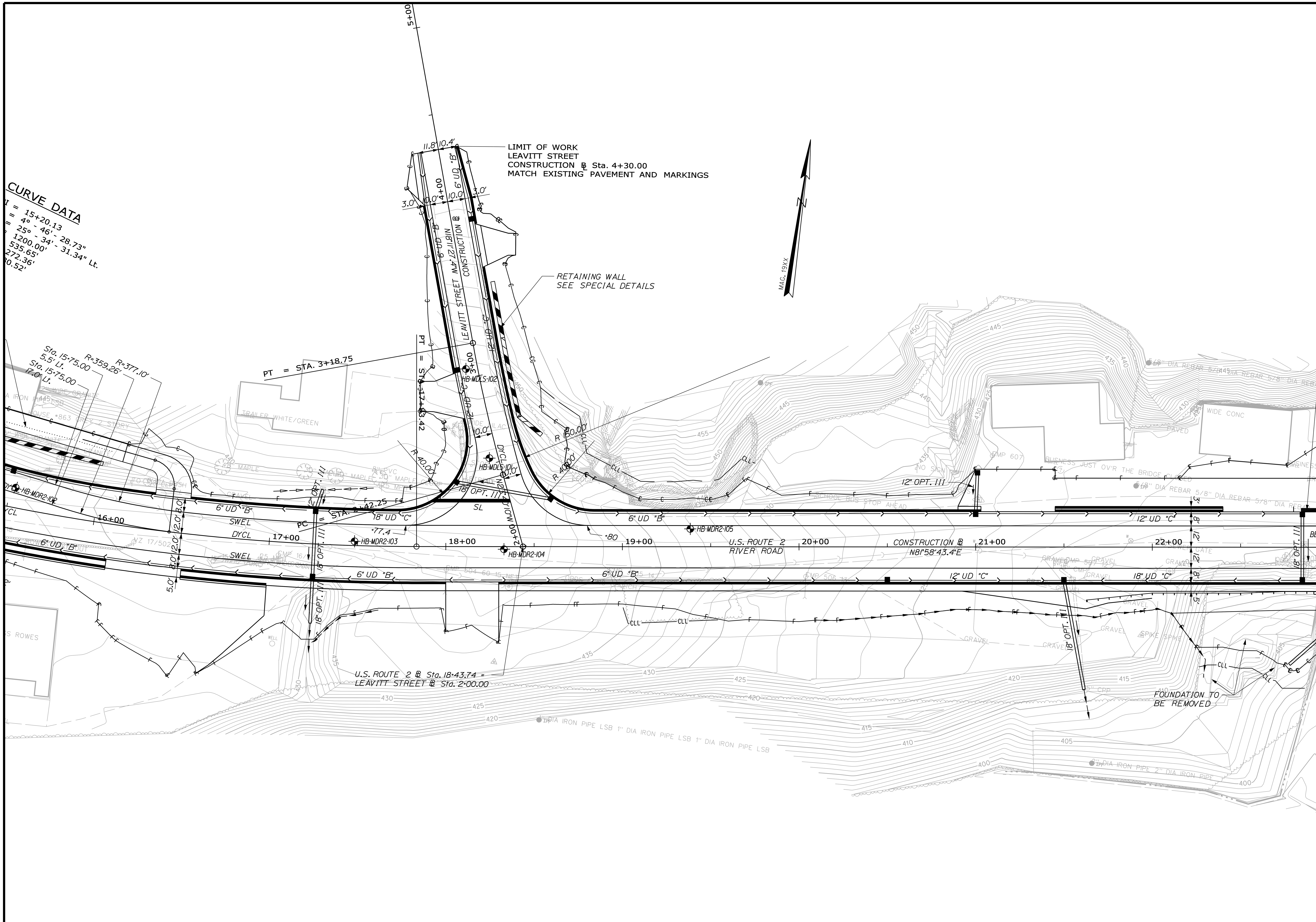
GEOPLANS

SHEET NUMBER

4

OF 7

Filename: ...\geotech\msta\005_Geoplan2.dgn Division: GEOTECH Username: terry.white Date:4/28/2009



STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BR-A562(000)E

PIN
15620.00

HIGHWAY PLANS

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED	K. MAGUIRE	
CHECKED-REVIEWED	T. WHITE	MAR 2009
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

MEXICO-DIXFIELD
ROUTES 2/17

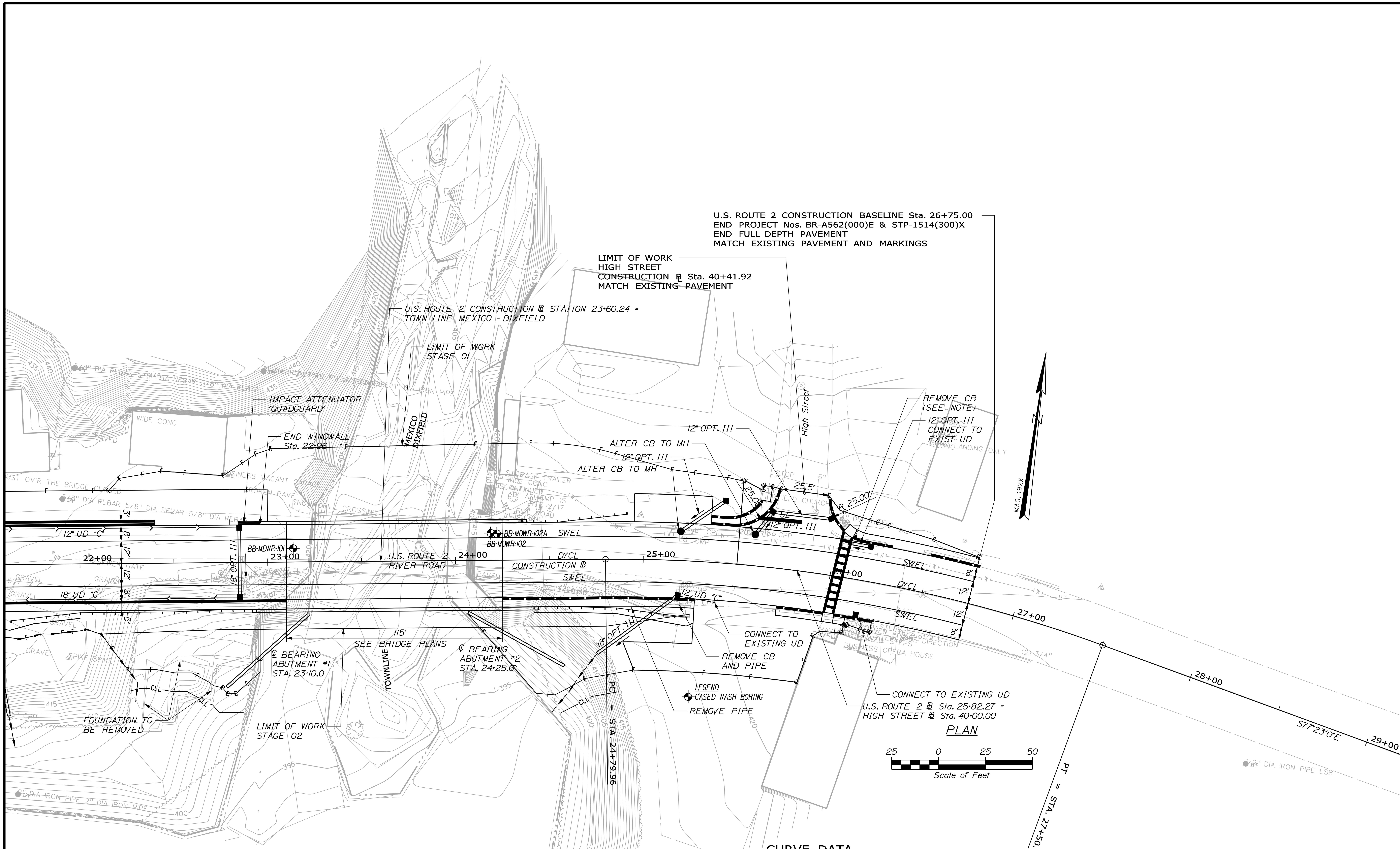
GEOPLANS

SHEET NUMBER

5

OF 7

Filename: ...\geotech\msta\006_Geoplan3.dgn Division: GEOTECH Username: terry.white Date:4/28/2009



CURVE DATA

PI = 26+16.51
D = 7° - 38' - 21.97"
Δ = 20° - 38' - 16.56" Rt.
R = 750.00'
L = 270.15'
T = 136.55'
E = 12.33'

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BR-A562(000)E

PIN
15620.00

HIGHWAY PLANS

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED	K. MAGUIRE	
CHECKED-REVIEWED	T. WHITE	MAR 2009
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

MEXICO-DIXFIELD
ROUTES 2/17

GEOPLANS

SHEET NUMBER
6
OF 7

Filename: ...\msta\007_BORING LOGS H1.dgn   Division: GEOTECH   Username: terry.white   Date:4/28/2009

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

Project:	Routes 2/17 and Leavitt Street	Location:	Mexico, Maine	Boring No.:	HB-MDR2-101
				PIN:	15143.00
Driller:	Northern Test Boring; Inc.	Elevation (ft.)	432.0	Auger ID/OD:	5" Solid Stem
Operator:	Nick V./Mike B.	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	K. Maguire	Rig Type:	Dietrich D50	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	3/9/09; 09:00-12:00	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	14+40.2, 12.7 Lt.	Casing ID/OD:	HW	Water Level*:	None Observed
Hammer Efficiency Factor:	0.68	Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Sample Information

Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows
						SSA
1D	6/6	2.00 - 2.50	50	---		
2D	18/16	5.00 - 6.50	36/48/50	98	111	120 RC
R1	60/53	6.50 - 11.50	RQD = 0%			NQ-2

Elevation (ft.)	Visual Description and Remarks	Depth (ft.)	Laboratory Testing Results/ AASHTO and Unified Class.
431.60	Pavement	0.40	
	Brown, damp, very dense, fine to coarse SAND, some gravel, trace silt.		
425.50	Brown, damp, very dense, fine to coarse SAND, some gravel, trace silt with rock fragments. Roller Coned ahead to 6.5' bgs.	6.50	
420.50	Top of Bedrock at Elev. 425.5'. Bedrock: Grey and white GNEISS with mica, some banding. R1:Core Times (min:sec) 6.5-7.5' (3:38) 7.5-8.5' (3:20) 8.5-9.5' (3:39) 9.5-10.5' (5:50) 10.5-11.5' (4:33) 88% Recovery Rock Mass Quality = Very Poor.	11.50	
	Bottom of Exploration at 11.50 feet below ground surface.		

Remarks:
Auto Hammer #149

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.: HB-MDR2-101

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

Project:	Routes 2/17 and Leavitt Street	Location:	Mexico, Maine	Boring No.:	HB-MDR2-102
				PIN:	15143.00
Driller:	Northern Test Boring; Inc.	Elevation (ft.)	435.8	Auger ID/OD:	5" Solid Stem
Operator:	Nick V./Mike B.	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	K. Maguire	Rig Type:	Dietrich D50	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	3/9/09; 12:30-15:30	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	15+52.9, 11.3 Lt.	Casing ID/OD:	HW	Water Level*:	None Observed
Hammer Efficiency Factor:	0.68	Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Sample Information

Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows
						SSA
1D	12/12	2.00 - 3.00	26/55	---		
R1	60/60	4.00 - 9.00	RQD = 68%			

Elevation (ft.)	Visual Description and Remarks	Depth (ft.)	Laboratory Testing Results/ AASHTO and Unified Class.
435.35	Pavement	0.45	
432.30	Brown, moist, very dense, fine to coarse SAND, some gravel, trace silt, occasional cobbles.	3.50	
426.80	Top of Bedrock at Elev. 432.3'. Auger into Bedrock to 4.0' bgs. Bedrock: Grey and white GNEISS with mica, some banding. R1:Core Times (min:sec) 4.0-5.0' (5:23) 5.0-6.0' (5:25) 6.0-7.0' (5:20) 7.0-8.0' (5:20) 8.0-9.0' (5:18) 100% Recovery Rock MAss Quality = Fair.	9.00	
	Bottom of Exploration at 9.00 feet below ground surface.		

Remarks:
Auto Hammer #149

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* 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.: HB-MDR2-102

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

Project:	Routes 2/17 and Leavitt Street	Location:	Mexico, Maine	Boring No.:	HB-MDR2-103
				PIN:	15143.00
Driller:	Northern Test Boring; Inc.	Elevation (ft.)	438.0	Auger ID/OD:	5" Solid Stem
Operator:	Nick V./Mike B.	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	Dietrich D50	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	3/12/09; 07:00-10:30	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	17+48.4, 2.4 Lt.	Casing ID/OD:	HW	Water Level*:	None Observed
Hammer Efficiency Factor:	0.68	Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Sample Information

Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows
						SSA
1D	24/24	2.00 - 4.00	17/21/23/12	44	50	
2D	24/20	5.00 - 7.00	4/6/7/7	13	15	
3D	24/24	10.00 - 12.00	4/5/5/8	10	11	
4D	15.6/12	15.00 - 16.30	4/6/40(3.6")	---		
						RC

Elevation (ft.)	Visual Description and Remarks	Depth (ft.)	Laboratory Testing Results/ AASHTO and Unified Class.
437.50	Pavement	0.50	
432.00	Light brown, damp, dense, coarse SAND, some silt.	6.00	
	Brown, damp, medium dense, fine to coarse SAND, trace gravel, trace silt.		
	Similar to above.		
418.20	Brown, wet, dense, fine to coarse SAND, some gravel, trace silt.	19.80	
416.50	Top of Bedrock at Elev. 418.2'. AUGER REFUSAL, Roller Coned ahead to 21.5' bgs.	21.50	
	Bottom of Exploration at 21.50 feet below ground surface. ROLLER CONE REFUSAL		

Remarks:
Auto Hammer #149

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Page 1 of 1
Boring No.: HB-MDR2-103

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

Project:	Routes 2/17 and Leavitt Street	Location:	Mexico, Maine	Boring No.:	HB-MDR2-104
				PIN:	15143.00
Driller:	Northern Test Boring; Inc.	Elevation (ft.)	439.3	Auger ID/OD:	5" Solid Stem
Operator:	Nick V./Mike B.	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	K. Maguire	Rig Type:	Dietrich D50	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	3/10/09; 12:45-?	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	18+32.9, 1.5 Rt.	Casing ID/OD:	HW	Water Level*:	None Observed
Hammer Efficiency Factor:	0.68	Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Sample Information

Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows
						SSA
1D	24/24	1.50 - 3.50	10/14/14/7	28	32	
2D	9.6/6	5.00 - 5.80	7/22(3.6")	---		
R1	60/58	6.00 - 11.00	RQD = 96%			NQ-2

Elevation (ft.)	Visual Description and Remarks	Depth (ft.)	Laboratory Testing Results/ AASHTO and Unified Class.
438.80	Pavement	0.50	
	Brown, damp, dense, fine to coarse SAND, some silt, trace gravel, (Fill).		
433.40	Brown, damp, dense, fine to coarse SAND, trace silt, trace gravel, (Fill).	5.90	
428.30	Top of Bedrock at Elev. 433.4'. Roller Coned ahead to 6.0' bgs. Bedrock: Grey and white, GNEISS with mica, no banding, no visible bedding. R1:Core Times (min:sec) 6.0-7.0' (3:58) 7.0-8.0' (3:22) 8.0-9.0' (5:02) 9.0-10.0' (4:25) 10.0-11.0' (5:29) 96% Recovery Rock Mass Quality = Excellent.	11.00	
	Bottom of Exploration at 11.00 feet below ground surface.		

Remarks:
Auto Hammer #149

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Boring No.: HB-MDR2-104

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

Project:	Routes 2/17 and Leavitt Street	Location:	Mexico, Maine	Boring No.:	HB-MDR2-105
				PIN:	15143.00
Driller:	Northern Test Boring; Inc.	Elevation (ft.)	433.2	Auger ID/OD:	5" Solid Stem
Operator:	Nick V./Mike B.	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder/K. Maguire	Rig Type:	Dietrich D50	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	3/10/09; 08:00-12:30	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	19+38.4, 10.1 Lt.	Casing ID/OD:	HW	Water Level*:	None Observed
Hammer Efficiency Factor:	0.68	Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Sample Information

Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows
						SSA
1D	12/12	2.00 - 3.00	30/50	---		
						105
						58
R1	54/45.6	5.00 - 9.50	RQD = 69%			NQ-2

Elevation (ft.)	Visual Description and Remarks	Depth (ft.)	Laboratory Testing Results/ AASHTO and Unified Class.
432.50	Pavement	0.70	
429.30	Brown, damp, very dense, fine to coarse SAND, some gravel, little silt.	3.90	
428.40	Weathered ROCK.	4.80	
423.70	Top of Bedrock at Elev. 429.4'. Roller Coned ahead to 5.0' bgs. R1:Bedrock: Grey and white, banded GNEISS with mica. Rock Quality Fair R1:Core Times (min:sec) 5.0-6.0' (4:45) 6.0-7.0' (6:34) 7.0-8.0' (4:13) 8.0-9.0' (4:38) 9.0-9.5' (3:50) 79% Recovery Core Blocked	9.50	
	Bottom of Exploration at 9.50 feet below ground surface.		

Remarks:
Auto Hammer #149

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Page 1 of 1
Boring No.: HB-MDR2-105

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

Project:	Routes 2/17 and Leavitt Street	Location:	Mexico, Maine	Boring No.:	HB-MDLS-101
				PIN:	15143.00
Driller:	Northern Test Boring; Inc.	Elevation (ft.)	441.8	Auger ID/OD:	5" Solid Stem
Operator:	Nick V./Mike B.	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	K. Maguire	Rig Type:	Dietrich D50	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	3/5/09; 08:00-12:00	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	2+53.4, 4.96 Lt. (Leavitt St.)	Casing ID/OD:	HW	Water Level*:	8.0' bgs.
Hammer Efficiency Factor:	0.68	Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Sample Information

Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows
1D	4.8/2	0.50 - 0.90	50(4.8")	---		SSA
2D	24/10	5.00 - 7.00	7/12/12/16	24	27	
R1	56.4/49	9.80 - 14.50	RQD = 79%			NQ-2

Elevation (ft.)	Visual Description and Remarks	Depth (ft.)	Laboratory Testing Results/ AASHTO and Unified Class.
441.59	Pavement	0.21	
	Brown, moist, medium dense, fine to coarse SAND, some silt, trace gravel.		
432.00	Brown, wet, medium dense, fine to coarse gravelly SAND, trace silt.	9.80	
427.30	Top of Bedrock at Elev. 432.0'. Bedrock: Grey and white, GNEISS with mica, some banding. R1:Core Times (min:sec) 9.8-10.8' (8:44) 10.8-11.8' (3:16) 11.8-12.8' (3:52) 12.8-13.8' (9:27) 13.8-14.5' (4:05) 87% Recovery Rock Mass Quality = Good.	14.50	
	Bottom of Exploration at 14.50 feet below ground surface.		

Remarks:
Auto Hammer #149

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Page 1 of 1
Boring No.: HB-MDLS-101

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

Project:	Routes 2/17 and Leavitt Street	Location:	Mexico, Maine	Boring No.:	HB-MDLS-102
				PIN:	15143.00
Driller:	Northern Test Boring; Inc.	Elevation (ft.)	450.1	Auger ID/OD:	5" Solid Stem
Operator:	Nick V./Mike B.	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	K. Maguire	Rig Type:	Dietrich D50	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	3/5/09; 12:00-?	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	3+05.1, 6.6 Lt. (Leavitt St.)	Casing ID/OD:	HW	Water Level*:	5.0' bgs.
Hammer Efficiency Factor:	0.68	Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Sample Information

Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows
1D	18/18	0.50 - 2.00	60/57/47	104	118	SSA
2D	24/12	5.00 - 7.00	6/9/8/8	17	19	
3D	24/24	10.00 - 12.00	10/13/28/24	41	46	
R1	60/58	14.40 - 19.40	RQD = 71%			NQ-2

Elevation (ft.)	Visual Description and Remarks	Depth (ft.)	Laboratory Testing Results/ AASHTO and Unified Class.
449.89	Pavement	0.21	
	Brown, wet, very dense, fine to coarse SAND, some silt, trace gravel.		
	Brown, wet, medium dense, fine to coarse SAND, some gravel, some silt.		
435.70	Brown, wet, dense, fine to coarse SAND, some gravel, some silt with broken rock fragments in nose of spoon.	14.40	
430.70	Top of Bedrock at Elev. 435.7'. Bedrock: Grey and white GNEISS with mica, some banding. R1:Core Times (min:sec) 14.4-15.4' (4:46) 15.4-16.4' (4:26) 16.4-17.4' (4:29) 17.4-18.4' (4:33) 18.4-19.4' (4:03) 96% Recovery Rock Mass Quality = Fair.	19.40	
	Bottom of Exploration at 19.40 feet below ground surface.		

Remarks:
Auto Hammer #149

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Page 1 of 1
Boring No.: HB-MDLS-102

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
BR-A562(000)E
PIN 15620.00
HIGHWAY PLANS

	BY	DATE
PROJ. MANAGER ______		
DESIGN-DETAILED	K. MAGUIRE	T. WHITE	MAR 2009
CHECKED-REVIEWED		
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE
P.E. NUMBER
DATE

MEXICO-DIXFIELD
ROUTES 2/17
BORING LOGS

SHEET NUMBER
7
OF 7

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

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

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

Sample Container Labeling Requirements:

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Webb River Bridge #2917 over Webb River carring Routes 2/17
Location: Mexico-Dixfield, Maine

Boring No.: BB-MDWR-101
PIN: 15620.00

Driller:	MaineDOT	**Elevation (ft.)**	420.7	**Auger ID/OD:**	5" Solid Stem
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/10/09, 3/12/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	23+13.6, 7.4 Lt.	**Casing ID/OD:**	HW	**Water Level*:**	15.0' bgs.
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒	Hydraulic ☐	Rope & Cathead ☐

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	420.10		Pavement 0.60	
	1D	12/12	1.00 - 2.00	20/53	---					Light brown, damp (frozen), very dense, fine SAND, little silt.	
5	2D	24/22	5.00 - 7.00	1/2/2/2	4	6				Light brown, moist, loose, fine SAND, little silt.	
								411.70		9.00	
10	3D	24/22	10.00 - 12.00	2/2/3/3	5	7	25			Light brown, moist, loose, fine to medium SAND, trace gravel, little silt.	
							24				
							22				
							22				
	4D	24/14	14.00 - 16.00	3/2/3/3	5	7	10			Similar to above, but wet.	
15							14				
							15				
							12				
							12				
	5D	21.6/16	19.00 - 20.80	2/1/1/40(3.6")	2	3	10	401.70		19.00 Brown, wet, loose, fine to medium SAND, little gravel, little silt. [a]40 blows for 0.8'.	
20	R1	78/60	20.80 - 27.30	RQD = 38%			[a]40 NQ-2	399.90		20.80 CONCRETE over COBBLES and BOULDERS. R1:Core Times (min:sec) 20.8-21.8' (2:25) 21.8-22.8' (0:15) 22.8-23.8' (0:56)	
								397.40		23.30 Top of Bedrock at Elev. 397.4'. Bedrock: Grey and white, GNEISS with mica, some banding. 23.8-24.8' (2:49)	
25											

Remarks:

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Webb River Bridge #2917 over Webb River carring Routes 2/17	**Boring No.:**	BB-MDWR-101		
Location:	Mexico-Dixfield, Maine	**PIN:**	15620.00		
Driller:	MaineDOT	**Elevation (ft.)**	420.7	**Auger ID/OD:**	5" Solid Stem
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/10/09, 3/12/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	23+13.6, 7.4 Lt.	**Casing ID/OD:**	HW	**Water Level*:**	15.0' bgs.
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25										24.8-25.8' (2:56) 25.8-26.8' (3:40) 26.8-27.3' (3:10) 76% Recovery Rock Mass Quality = Poor.	
	R2	60/60	27.30 - 32.30	RQD = 73%						Bedrock: Grey and white, GNEISS with mica, some banding. R2:Core Times (min:sec) 27.3-28.3' (3:15) 28.3-29.3' (3:22) 29.3-30.3' (3:20) 30.3-31.3' (3:15) 31.3-32.3' (2:50) 100% recovery Rock Mass Quality = Fair.	
30											
								388.40		32.30 **Bottom of Exploration at 32.30 feet below ground surface.**	
35											
40											
45											
50											

Remarks:

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

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Webb River Bridge #2917 over Webb River carring Routes 2/17 Location: Mexico-Dixfield, Maine	Boring No.: BB-MDWR-102 PIN: 15620.00

Driller:	Northern Test Boring; Inc.	**Elevation (ft.)**	420.2	**Auger ID/OD:**	5" Solid Stem
Operator:	Nick V./Mike B.	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	E. Giguere	**Rig Type:**	Dietrich D50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/3/09; 09:30-10:45	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	24+18.9, 14.6 Lt.	**Casing ID/OD:**	HW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	419.70		Pavement	
										0.50	
	1D	24/13	2.00 - 4.00	12/14/10/3	24	34				Brown, dry, dense, fine to coarse SAND, some gravel, trace silt.	
5	2D	24/8	5.00 - 7.00	3/8/9/4	17	24	4			Similar to above, medium dense.	
							27				
							43				
							41				
							49				
10	3D	24/4	10.00 - 12.00	3/8/12/14	20	28	51			Similar to above.	
							43				
							101				
	4D	11/0	13.80 - 14.72	32/42/50(0")	---		a125			a125 blows for 0.7'. No sample recovery.	
								405.50		14.70	
15										**Bottom of Exploration at 14.70 feet below ground surface.** CASING BROKE, MOVED TO BB-MDWR-102A.	
20											
25											

Remarks:

Left 5.0 feet of casing in ground.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Webb River Bridge #2917 over Webb River carring Routes 2/17
Location: Mexico-Dixfield, Maine

Boring No.: BB-MDWR-102A
PIN: 15620.00

Driller:	MaineDOT/Northern Test Boring	**Elevation (ft.)**	420.4	**Auger ID/OD:**	5" Solid Stem
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/10/09, 3/12/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2" & BX
Boring Location:	24+22.3, 14.2 Lt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	19.0' bgs.
Hammer Efficiency Factor:	0.68	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	419.95		Pavement —0.45— See Boring BB-MDWR-102 for material description in upper 14.7 feet of boring.	
5											
10											
	R1	19.2/19.2	14.70 - 16.30				27 NQ-2 8	405.70		—14.70— COBBLES and BOULDERS within SAND matrix. R1:Core Times (min:sec) 14.7-15.7' (8:00) 15.7-16.3' (3:00) 100% Recovery	
15							19 RC 57	404.10		—16.30— Roller Coned ahead to 18.6' bgs.	
	R2	7.2/0	18.60 - 19.20				63 NQ-2	401.80		—18.60— COBBLE. R2:Core Times (min:sec) 18.6-19.2' (5:00) 0% Recovery Changed to NW Casing at 19.0' bgs.	
	1D	13.2/12	19.20 - 20.30	16/18/30(1.2")	---		47	401.20		—19.20— Brown, wet, very dense, fine SAND, trace silt, trace gravel. Roller Coned ahead to 20.8' bgs.	
20	R3	31.2/31.2	20.80 - 23.40	RQD = 88%			NQ-2	399.60		—20.80— Top of Bedrock at Elev. 399.6'. Bedrock: Grey and white, GNEISS with mica, some banding. R3:Core Times (min:sec) 20.8-21.8' (3:10) 21.8-22.8' (3:35) 22.8-23.4' (4:41) 100% Recovery	
	R4	57.6/57.6	23.40 - 28.20	RQD = N/A%			BX				
25											

Remarks:

All samples were with Auto Hammer #149.

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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Webb River Bridge #2917 over Webb River carring Routes 2/17 Location: Mexico-Dixfield, Maine	Boring No.: BB-MDWR-102A PIN: 15620.00

Driller:	MaineDOT/Northern Test Boring	**Elevation (ft.)**	420.4	**Auger ID/OD:**	5" Solid Stem
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/10/09, 3/12/09	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2" & BX
Boring Location:	24+22.3, 14.2 Lt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	19.0' bgs.
Hammer Efficiency Factor:	0.68	**Hammer Type:**	Automatic ☒	Hydraulic ☐	Rope & Cathead ☐

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25								392.20		Rock Mass Quality = Good. Bedrock: Grey and white, GNEISS with mica, some banding. R4:Core Times (min:sec) 23.4-24.4' (4:25) 24.4-25.4' (3:15) 25.4-26.4' (4:40) 26.4-27.4' (4:00) 27.4-28.2' (3:38) 100% Recovery Core Blocked BX core used - No RQD Calculated. 28.20	
30										**Bottom of Exploration at 28.20 feet below ground surface.**	
35											
40											
45											
50											

Remarks:

All samples were with Auto Hammer #149.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Routes 2/17 and Leavitt Street
Location: Mexico, Maine

Boring No.: HB-MDR2-101
PIN: 15143.00

Driller:	Northern Test Boring; Inc.	**Elevation (ft.)**	432.0	**Auger ID/OD:**	5" Solid Stem
Operator:	Nick V./Mike B.	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	K. Maguire	**Rig Type:**	Dietrich D50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/9/09; 09:00-12:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	14+40.2, 12.7 Lt.	**Casing ID/OD:**	HW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.68	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	431.60		Pavement — 0.40	
	1D	6/6	2.00 - 2.50	50	---					Brown, damp, very dense, fine to coarse SAND, some gravel, trace silt.	
5	2D	18/16	5.00 - 6.50	36/48/50	98	111	120			Brown, damp, very dense, fine to coarse SAND, some gravel, trace silt with rock fragments. Roller Coned ahead to 6.5' bgs.	
							RC				
	R1	60/53	6.50 - 11.50	RQD = 0%			NQ-2	425.50		6.50 — Top of Bedrock at Elev. 425.5'. Bedrock: Grey and white GNEISS with mica, some banding. R1:Core Times (min:sec) 6.5-7.5' (3:38) 7.5-8.5' (3:20) 8.5-9.5' (3:39) 9.5-10.5' (5:50) 10.5-11.5' (4:33) 88% Recovery Rock Mass Quality = Very Poor.	
10								420.50		11.50 — **Bottom of Exploration at 11.50 feet below ground surface.**	
15											
20											
25											

Remarks:

Auto Hammer #149

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Routes 2/17 and Leavitt Street			**Boring No.:**	HB-MDR2-102
Location:	Mexico, Maine			**PIN:**	15143.00
Driller:	Northern Test Boring; Inc.	**Elevation (ft.)**	435.8	**Auger ID/OD:**	5" Solid Stem
Operator:	Nick V./Mike B.	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	K. Maguire	**Rig Type:**	Dietrich D50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/9/09; 12:30-15:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	15+52.9, 11.3 Lt.	**Casing ID/OD:**	HW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.68	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	435.35		Pavement ——0.45	
	1D	12/12	2.00 - 3.00	26/55	---			432.30		Brown, moist, very dense, fine to coarse SAND, some gravel, trace silt, occasional cobbles. ——3.50	
5	R1	60/60	4.00 - 9.00	RQD = 68%				426.80		Top of Bedrock at Elev. 432.3'. Auger into Bedrock to 4.0' bgs. Bedrock: Grey and white GNEISS with mica, some banding. R1:Core Times (min:sec) 4.0-5.0' (5:23) 5.0-6.0' (5:25) 6.0-7.0' (5:20) 7.0-8.0' (5:20) 8.0-9.0' (5:18) 100% Recovery Rock MAss Quality = Fair. ——9.00	
10										**Bottom of Exploration at 9.00 feet below ground surface.**	
15											
20											
25											

Remarks:

Auto Hammer #149

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

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Routes 2/17 and Leavitt Street
Location: Mexico, Maine
Boring No.: HB-MDR2-103
PIN: 15143.00

Driller:	Northern Test Boring; Inc.	**Elevation (ft.)**	438.0	**Auger ID/OD:**	5" Solid Stem
Operator:	Nick V./Mike B.	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Dietrich D50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/12/09; 07:00-10:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	17+48.4, 2.4 Lt.	**Casing ID/OD:**	HW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.68	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	437.50		Pavement 0.50	
	1D	24/24	2.00 - 4.00	17/21/23/12	44	50				Light brown, damp, dense, coarse SAND, some silt.	
5	2D	24/20	5.00 - 7.00	4/6/7/7	13	15		432.00		6.00 Brown, damp, medium dense, fine to coarse SAND, trace gravel, trace silt.	
10	3D	24/24	10.00 - 12.00	4/5/5/8	10	11				Similar to above.	
15	4D	15.6/12	15.00 - 16.30	4/6/40(3.6")	---					Brown, wet, dense, fine to coarse SAND, some gravel, trace silt.	
							RC				
20								418.20		19.80 Top of Bedrock at Elev. 418.2'. AUGER REFUSAL, Roller Coned ahead to 21.5' bgs.	
								416.50		21.50 **Bottom of Exploration at 21.50 feet below ground surface.** ROLLER CONE REFUSAL	
25											

Remarks:

Auto Hammer #149

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Routes 2/17 and Leavitt Street
Location: Mexico, Maine

Boring No.: HB-MDR2-104
PIN: 15143.00

Driller:	Northern Test Boring; Inc.	**Elevation (ft.)**	439.3	**Auger ID/OD:**	5" Solid Stem
Operator:	Nick V./Mike B.	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	K. Maguire	**Rig Type:**	Dietrich D50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/10/09; 12:45-?	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	18+32.9, 1.5 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.68	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	438.80		Pavement — 0.50	
	1D	24/24	1.50 - 3.50	10/14/14/7	28	32				Brown, damp, dense, fine to coarse SAND, some silt, trace gravel, (Fill).	
5	2D	9.6/6	5.00 - 5.80	7/22(3.6")	---			433.40		Brown, damp, dense, fine to coarse SAND, trace silt, trace gravel, (Fill). — 5.90	
	R1	60/58	6.00 - 11.00	RQD = 96%			NQ-2			Top of Bedrock at Elev. 433.4'. Roller Coned ahead to 6.0' bgs. Bedrock: Grey and white, GNEISS with mica, no banding, no visible bedding. R1:Core Times (min:sec) 6.0-7.0' (3:58) 7.0-8.0' (3:22) 8.0-9.0' (5:02) 9.0-10.0' (4:25) 10.0-11.0' (5:29) 96% Recovery Rock Mass Quality = Excellent.	
10								428.30		— 11.00 **Bottom of Exploration at 11.00 feet below ground surface.**	
15											
20											
25											

Remarks:

Auto Hammer #149

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Routes 2/17 and Leavitt Street		**Boring No.:**	HB-MDR2-105
Location: Mexico, Maine		**PIN:**	15143.00

Driller:	Northern Test Boring; Inc.	**Elevation (ft.)**	433.2	**Auger ID/OD:**	5" Solid Stem
Operator:	Nick V./Mike B.	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder/K. Maguire	**Rig Type:**	Dietrich D50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/10/09; 08:00-12:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	19+38.4, 10.1 Lt.	**Casing ID/OD:**	HW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.68	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	432.50		Pavement — 0.70	
	1D	12/12	2.00 - 3.00	30/50	---					Brown, damp, very dense, fine to coarse SAND, some gravel, little silt.	
							105	429.30		— 3.90	
							58	428.40		Weathered ROCK. — 4.80	
5	R1	54/45.6	5.00 - 9.50	RQD = 69%			NQ-2			Top of Bedrock at Elev. 429.4'. Roller Coned ahead to 5.0' bgs. R1:Bedrock: Grey and white, banded GNEISS with mica. Rock Quality Fair R1:Core Times (min:sec) 5.0-6.0' (4:45) 6.0-7.0' (6:34) 7.0-8.0' (4:13) 8.0-9.0' (4:38) 9.0-9.5' (3:50) 79% Recovery Core Blocked	
								423.70		— 9.50	
10										**Bottom of Exploration at 9.50 feet below ground surface.**	
15											
20											
25											

Remarks:

Auto Hammer #149

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

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

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Routes 2/17 and Leavitt Street Location: Mexico, Maine	Boring No.: HB-MDLS-101 PIN: 15143.00

Driller:	Northern Test Boring; Inc.	**Elevation (ft.)**	441.8	**Auger ID/OD:**	5" Solid Stem
Operator:	Nick V./Mike B.	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	K. Maguire	**Rig Type:**	Dietrich D50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/5/09; 08:00-12:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	2+53.4, 4.96 Lt. (Leavitt St.)	**Casing ID/OD:**	HW	**Water Level*:**	8.0' bgs.
Hammer Efficiency Factor:	0.68	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	4.8/2	0.50 - 0.90	50(4.8")	---		SSA	441.59		Pavement — 0.21	
										Brown, moist, medium dense, fine to coarse SAND, some silt, trace gravel.	
5	2D	24/10	5.00 - 7.00	7/12/12/16	24	27				Brown, wet, medium dense, fine to coarse gravelly SAND, trace silt.	
	R1	56.4/49	9.80 - 14.50	RQD = 79%			NQ-2	432.00		9.80 Top of Bedrock at Elev. 432.0'. Bedrock: Grey and white, GNEISS with mica, some banding. R1:Core Times (min:sec) 9.8-10.8' (8:44) 10.8-11.8' (3:16) 11.8-12.8' (3:52) 12.8-13.8' (9:27) 13.8-14.5' (4:05) 87% Recovery Rock Mass Quality = Good.	
10											
								427.30		14.50 **Bottom of Exploration at 14.50 feet below ground surface.**	
15											
20											
25											

Remarks:

Auto Hammer #149

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

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

Boring No.: HB-MDLS-101

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Routes 2/17 and Leavitt Street
Location: Mexico, Maine
Boring No.: HB-MDLS-102
PIN: 15143.00

Driller:	Northern Test Boring; Inc.	**Elevation (ft.)**	450.1	**Auger ID/OD:**	5" Solid Stem
Operator:	Nick V./Mike B.	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	K. Maguire	**Rig Type:**	Dietrich D50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/5/09; 12:00-?	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	3+05.1, 6.6 Lt. (Leavitt St.)	**Casing ID/OD:**	HW	**Water Level*:**	5.0' bgs.
Hammer Efficiency Factor:	0.68	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	18/18	0.50 - 2.00	60/57/47	104	118	SSA	449.89		Pavement (0.21)	
										Brown, wet, very dense, fine to coarse SAND, some silt, trace gravel.	
5	2D	24/12	5.00 - 7.00	6/9/8/8	17	19				Brown, wet, medium dense, fine to coarse SAND, some gravel, some silt.	
10	3D	24/24	10.00 - 12.00	10/13/28/24	41	46				Brown, wet, dense, fine to coarse SAND, some gravel, some silt with broken rock fragments in nose of spoon.	
14.40	R1	60/58	14.40 - 19.40	RQD = 71%			NQ-2	435.70		Top of Bedrock at Elev. 435.7'. Bedrock: Grey and white GNEISS with mica, some banding. R1:Core Times (min:sec) 14.4-15.4' (4:46) 15.4-16.4' (4:26) 16.4-17.4' (4:29) 17.4-18.4' (4:33) 18.4-19.4' (4:03) 96% Recovery Rock Mass Quality = Fair.	
19.40								430.70		**Bottom of Exploration at 19.40 feet below ground surface.**	
25											

Remarks:

Auto Hammer #149

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

Calculations

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:
Mexico/Dixfield, Maine
DFI = 1700 degree-days

Soils are coarse grained. Assume a water content = ~20%

From MaineDOT BDG Table 5-1:
Depth of frost penetration = 72.4 inches

$\text{Frost\_depth} := 72.4\text{in} \qquad \text{Frost\_depth} = 6.033 \cdot \text{ft}$

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	82.4	20.0	125.0	34	46	3.8	1.9	3,600

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 = 6.86 ft = 82.4 in.

Use BDG Calculated Frost Depth = 6.0 feet for design

Bearing Resistance - Fill Soils:

Part 1 - Service Limit State

Nominal and factored Bearing Resistance - spread footing on fill soils

Presumptive Bearing Resistance for Service Limit State ONLY

Reference: AASHTO LRFD Bridge Design Specifications Third Edition
Table C10.6.2.6.1-1 "Presumptive Bearing Resistances for Spread Footings at the Service Limit State Modified after US Department of Navy *(1982)*"

Type of Bearing Material: Coarse to medium sand, with little gravel (SW, SP)

Based on corrected N-values ranging from 6 to 24 - Soils are loose to medium dense

Consistency In Place: Loose to Medium Dense

Bearing Resistance: Ordinary Range (ksf) 2 - 8

Recommended Value of Use (ksf): 6 ksf

Recommended Value: $q_{nom} := 6 \cdot ksf$

Resistance factor at the **service limit state** Φ=1.0 (LRFD Article 10.5.5.1)

$q_{factored\_bc} := q_{nom} \cdot 1.0$ $\quad q_{factored\_bc} = 6 \cdot ksf$

Note: This bearing resistance is settlement limited (1 inch) and applies only at the service limit state.

Part 2 - Strength Limit State

Nominal and factored Bearing Resistance - spread footing on fill soils

Reference: Foundation Analysis and Design by JE Bowles Fifth Edition

Section 4-2 Bearing Capacity

Assumptions:

1. Footings will be embedded 6.0 feet for frost protection. $D_f := 6.0 \cdot ft$
2. Assumed parameters for fill soils: (Ref: Bowles 5th Ed Table 3-4, pg 163)
   - Saturated unit weight: $\gamma_s := 125 \cdot pcf$
   - Dry unit weight: $\gamma_d := 120 \cdot pcf$
   - Internal friction angle: $\phi_{ns} := 32 \cdot deg$
   - Undrained shear strength: $c_{ns} := 0 \cdot psf$
3. Use Terzaghi strip equations as L>B
4. Effective stress analysis footing on ϕ-c soil (Bowles 5th Ed. Example 4-1 pg 231)

Depth to Groundwater table: $D_w := 15 \cdot ft$ Based on boring logs

Unit Weight of water: $\gamma_w := 62.4 \cdot pcf$

Look at several footing widths

$$B := \begin{pmatrix} 5 \\ 8 \\ 10 \\ 12 \end{pmatrix} \cdot ft$$

Terzaghi Shape factors from Table 4-1 pg 220

For a strip footing: $s_c := 1.0$ $s_\gamma := 1.0$

Meyerhof Bearing Capacity Factors - Bowles 5th Ed. table 4-4 pg 223

For ϕ=32 deg

$N_c := 35.47$ $N_q := 23.2$ $N_\gamma := 22.0$

Nominal Bearing Resistance per Terzaghi equation (Bowles 5th Ed. Table 4-1 pg 220)

$$q := D_w \cdot \gamma_d + (D_f - D_w) \cdot (\gamma_s - \gamma_w) \qquad q = 1.237 \cdot ksf$$

$$q_{ult} := c_{ns} \cdot N_c \cdot s_c + q \cdot N_q + 0.5(\gamma_s - \gamma_w) B \cdot N_\gamma \cdot s_\gamma \qquad q_{ult} = \begin{pmatrix} 32 \\ 34 \\ 36 \\ 37 \end{pmatrix} \cdot ksf$$

Assume this ultimate load is a nominal load. Apply 0.45 resistance factor to get factored resistance.

Resistance Factor: $\phi_b := 0.45$ AASHTO LRFD Table 10.5.5.2.2-1

$$q_{factored} := q_{ult} \cdot \phi_b$$

$$q_{factored} = \begin{pmatrix} 14.5 \\ 15.4 \\ 16 \\ 16.6 \end{pmatrix} \cdot ksf$$

Based on these footing widths: $B = \begin{pmatrix} 5 \\ 8 \\ 10 \\ 12 \end{pmatrix} \cdot ft$

At the Strength Limit State:

Recommend a limiting factored bearing resistance of 14 ksf

Bearing Resistance - Bedrock:

Part 1 - Service Limit State

Nominal and factored Bearing Resistance - spread footing on bedrock

Presumptive Bearing Resistance for Service Limit State ONLY

Bedrock at the site is GNEISS which is "poor" to "good" in quality.
RQD = 38 to 88%

Reference: AASHTO LRFD Bridge Design Specifications Third Edition
Table C10.6.2.6.1-1 "Presumptive Bearing Resistances for Spread Footings at the Service Limit State Modified after US Department of Navy *(1982)*"

Due to RQD look at "medium hard rock"

Type of Bearing Material: Weathered or broken rock of any kind except highly argillaceous rock (shale)

Consistency In Place: Medium hard, rock

Bearing Resistance: Ordinary Range (ksf) 16 - 24

Recommended Value of Use (ksf): 20 ksf

Based on RQD values ranging from 38% to 88%

Recommended Value: $q_{pres} := 20 \cdot ksf$

Note: This bearing resistance is settlement limited (1 inch) and applies only at the service limit state.

Part 2 - Strength Limit State

Nominal and Factored Bearing Resistance - spread footing on bedrock

Nominal Bearing Resistance for Strength Limit State

Bedrock at the site is GNEISS which is "poor" to "good" in quality.
RQD = 38 to 88%

Reference: AASHTO LRFD Bridge Design Specifications Third Edition Article 10.6.3.2:
For footings on competent rock, reliance on simple and direct analyses based on uniaxial compressive rock strengths and RQD may be applicable. Where engineering judgment does not verify the presence of competent rock, the competency of the rock mass should be verified using the procedures for RMR rating in Article 10.4.6.4.

Due to competency of bedrock (RQD 38 to 88%), RMR method is not required.

Reference: <u>Foundation Analysis and Design</u> by JE Bowles Fifth Edition

Section 4-16 pg 277 Bearing Capacity of Rock

Assume: $\phi := 45 \cdot deg$ internal friction angle rock

$c_r := 0 \cdot psi$ cohesion (rock)

Bearing Capacity factors by Stagg and Zienkiewicz 1968

$$N_c := 5 \cdot \left(\tan\left(45 \cdot deg + \frac{\phi}{2}\right)^4 \right) \qquad N_c = 170$$

$$N_q := \tan\left(45 \cdot deg + \frac{\phi}{2}\right)^6 \qquad N_q = 198$$

$$N_\gamma := N_q + 1 \qquad N_\gamma = 199$$

Terzaghi Shape factors from Table 4-1 pg 220

For a strip footing: $s_c := 1.0$ $s_\gamma := 1.0$

Assume $\gamma_r := 165 \cdot pcf$ for the rock

$D_f := 0 \cdot ft$ footing placed on bedrock surface - no embedment $q := \gamma_r \cdot D_f$ $q = 0 \cdot psf$

$$B := \begin{pmatrix} 6 \\ 8 \\ 10 \\ 12 \end{pmatrix} \cdot ft$$ Look at several footing widths

$$q_{ult} := c_r \cdot N_c \cdot s_c + q \cdot N_q + 0.5 \cdot \gamma_r \cdot B \cdot N_\gamma \cdot s_\gamma$$

$$q_{ult} = \begin{pmatrix} 99 \\ 131 \\ 164 \\ 197 \end{pmatrix} \cdot ksf$$

Reduce ultimate bearing based on average RQD = 60%

$$q_{reduced} := q_{ult} \cdot (0.6)^2$$

$$q_{reduced} = \begin{pmatrix} 35 \\ 47 \\ 59 \\ 71 \end{pmatrix} \cdot ksf$$

Assume this ultimate load is a nominal load. Apply 0.45 resistance factor to get factored resistance.

$$q_{factored} := q_{reduced} \cdot 0.45$$

$$q_{factored} = \begin{pmatrix} 16 \\ 21 \\ 27 \\ 32 \end{pmatrix} \cdot ksf \qquad B := \begin{pmatrix} 6 \\ 8 \\ 10 \\ 12 \end{pmatrix} \cdot ft$$

At the Strength Limit State:

Recommend a limiting factored bearing resistance of 16 ksf

Active Earth Pressures:

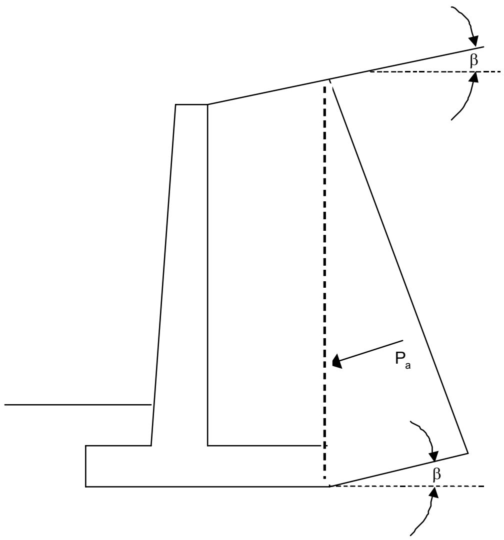
Soil Type 4 Properties from MaineDOT Bridge Design Guide (BDG)

unit weight: $\gamma_{type4} := 125 \cdot pcf$

Internal Friction Angle: $\phi_{type4} := 32 \cdot deg$

Cohesion: $c_{sand} := 0 \cdot psf$

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



Generally use Rankine for long heeled cantilever walls where the failure surface is un interrupted by the top of the wall system. The earth pressure is applied to a plane extending vertically up from the heel of the wall base and the weight of the soil on the inside of the vertical plane is considered as part of the wall weight. The failure sliding surface is not restricted by the top of the wall or the backface of the wall.

For cantilever walls with horizontal backfill surface:

$$K_{a\_rankine} := \tan\left(45 \cdot deg - \frac{\phi_{type4}}{2}\right)^2 \qquad K_{a\_rankine} = 0.307$$

For cantilever walls with sloped backfill surface:

β = Angel of fill slope to the horizontal

$\beta := 0 \cdot deg$

$$K_{a\_rankine\_slope} := \frac{\cos(\beta) - \sqrt{\cos(\beta)^2 - \cos(\phi_{type4})^2}}{\cos(\beta) + \sqrt{\cos(\beta)^2 - \cos(\phi_{type4})^2}} \qquad K_{a\_rankine\_slope} = 0.307$$

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

Active Earth Pressure - Coulomb Theory
from MaineDOT Bridge Design Guide Section 3.6.5.2 pg 3-5

For cases where the backface of the wall interferes with the development of a full sliding surface in the backfill use Coulomb Theory.

- Coulomb Theory applies for gravity, semi-gravity, and prefab modular walls with steep back faces
- Coulomb Theory applies to concrete cantilever wall with short heels where the sliding surface is restricted by the top of the wall - the wedge of soil does not move.
- Inter face friction is considered in Coulomb Theory

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

Choosing Friction Angle between fill and wall:

i.) From LRFD Table 3.11.5.3-1 range from 17 to 22 - choose δ = 20 degrees
ii.) From MaineDOT BDG Table 3-3 δ = 24 degrees
iii.) From LRFD Figure C3.11.5.3-1 - δ = 1/3 to 2/3 * Internal Friction Angle = 21.33 degrees

Use Friction Angle between fill and wall = $\delta := 20 \cdot deg$

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

Internal Friction Angle: $\phi_{type4} := 32 \cdot deg$

$$K_{a\_coulomb} := \frac{\sin(\alpha + \phi_{type4})^2}{\sin(\alpha)^2 \cdot \sin(\alpha - \delta) \cdot \left(1 + \sqrt{\frac{\sin(\phi_{type4} + \delta) \cdot \sin(\phi_{type4} - \beta)}{\sin(\alpha - \delta) \cdot \sin(\beta + \alpha)}}\right)^2}$$

$$K_{a\_coulomb} = 0.276$$

Orientation of Coulomb Pa :

- In the case of gravity shaped walls and prefab walls - Pa is oriented δ degrees up from a perpendicular line to the backface.
- In the case of short heeled cantilever walls where the top of the wall interferes with the failure surface - Pa is oriented at an angle of 1/3 to 2/3 Φ to the normal of a vertical line extending up from the heel of the wall.

Passive Earth Pressure:

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 $\beta > 0$.

Seismic:

Mexico Dixfield Webb River Bridge PIN 15620.00
Date and Time: 4/13/2009 3:56:32 PM

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years
State - Maine
Zip Code - 04257
Zip Code Latitude = 44.559500
Zip Code Longitude = -070.544600
Site Class B
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.086	PGA - Site Class B
0.2	0.177	Ss - Site Class B
1.0	0.049	S1 - Site Class B

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Spectral Response Accelerations SDs and SD1
State - Maine
Zip Code - 04257
Zip Code Latitude = 44.559500
Zip Code Longitude = -070.544600
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	0.137	As - Site Class D
0.2	0.282	SDs - Site Class D
1.0	0.118	SD1 - Site Class D

Appendix C

Special Provisions

SPECIAL PROVISION
SECTION 635
PRECAST AGGREGATE-FILLED, CONCRETE BLOCK GRAVITY WALL

The following replaces Section 635 in the Standard Specifications in its entirety:

635.01 Description The work under this item shall consist of the design, fabrication, furnishing and construction of a Precast Aggregate-filled Concrete Block Gravity Wall in accordance with these specifications and in conformance with the lines and grades shown on the Plans, or established by the Resident. The Precast Aggregate-filled Concrete Block Gravity Wall shall consist of blocks made of Structural Precast concrete made from Portland cement, water, chemical admixtures, and aggregates, supported on concrete leveling pads, and if required, geosynthetic reinforced backfill.

Included in the scope of the precast gravity wall construction are**:** geotechnical design of any wall with an exposed height greater than 4.5 ft or as specified on the Plans, all grading necessary for wall construction, compaction of the wall foundation soil, backfill, piped drainage, construction of leveling pads, and concrete wall unit installation. The top of the upper row of concrete wall units shall be at or above the top of the face elevation shown on the Plans.

635.02 Quality Assurance The wall system shall be one of the approved combinations of facing block and soil reinforcement systems noted in the Plans or on the Department's Qualified Products List (QPL). Alternate wall systems will not be considered for this Item.

All design calculations and Shop Drawings shall be signed and sealed by a Professional Engineer licensed in the State of Maine.

The Contractor shall require the wall design-supplier to provide an on-site, qualified experienced technical representative to advise the Contractor concerning proper installation procedures. The technical representative shall be on-site during initial stages of installation and thereafter shall remain available for consultation as necessary for the Contractor or as required by the Resident.

635.03 Materials Materials for walls shall meet the requirements of the following sections of Division 700:

Gravel Borrow	703.20
Underdrain Backfill Type C	703.22
Underdrain Pipe	706.06 or 706.09
Reinforcing Steel	709.01
Structural Precast Concrete Units	712.061
Reinforcement Geotextile	722.01
Drainage Geosynthetic	722.02

The Contractor is cautioned that all of the materials listed are not required for every Precast Aggregate-filled Concrete Block Gravity Wall. The Contractor shall furnish the Resident a Materials Certification Letter certifying that the applicable materials comply with this section of the specifications. Materials shall meet the following additional requirements:

635.031 Concrete Units The Materials Certification Letter described above shall contain the date of concrete casting, a lot identification number, compressive strength results, and entrained air results. All prefabricated concrete units shall conform to the requirements of 712.061 with the following exceptions:

A. Materials. Materials are modified as follows: the maximum water cement ratio shall be 0.42, use of calcium nitrite is not required, and the minimum 28 day compressive strength shall be 4600 psi.

B. Quality Control and Quality Assurance. Quality Control and Quality Assurance is modified as follows: delete the second and third paragraphs.

C. Construction. Construction requirements are modified as follows:
Delete the second paragraph and replace it with the following:
All units for a designated wall system, including end blocks, steps, caps and other wall units shall be manufactured from the same material sources of aggregates, brand and type of cement and color pigment.

Replace the first sentence in the paragraph which begins "The forms shall remain …" with the following:
The forms shall remain in place until the concrete has gained sufficient strength such that removal of the forms and subsequent handling will not damage the units.

Replace the paragraph which begins "A minimum of 8 …." With the following:
The Contractor shall make and test at least one set of cylinders for every 50 yd^3 of production concrete used to cast the concrete units.

Replace the paragraph which begins "At least once …" with the following:
The Contractor shall make four cylinders for use by the Department for every 200 yd^3.

Add the following paragraph at the end of the Construction section:
Face texture of the units shall be a formed finish on all exposed surfaces. Pigment shall be added during the casting process of the concrete unit to achieve a consistent shade of gray or other color as determined by the Resident.

D. Tolerances. Maximum dimensional deviation of formed unit dimensions shall not vary more than ½-inch or 2 percent of the unit dimension or the manufacturer's published tolerances, whichever is less. All units not meeting the specified tolerances will be rejected.

635.032 Geosynthetic Reinforcement Geosynthetic Reinforcement shall be as required by the proprietary wall system manufacturer or wall designer. Geosynthetic reinforcement shall consist of a geotextile or geogrid approved by the Geotechnical Engineer. Substitution of a geosynthetic other than that required by the proprietary wall system manufacturer shall not be allowed unless approved by the Geotechnical Engineer after submittal of shop drawings and pullout and interface friction test data.

A. Geotextiles and Thread for Sewing. Woven or nonwoven geotextiles shall consist of long chain polymeric filaments or yarns formed into a stable network such that the filaments or yarns retain their position relative to each other during handling, placement, and design life. At least 95 percent by weight of the long chain polymer shall be polyolefin or polyester. The material shall be free of defects and tears. Geotextiles used for reinforcement shall conform as a minimum to the properties indicated for 722.01, Stabilization/Reinforcement Geotextile and shall meet the requirements of part D and E below. Geotextiles shall have a minimum permeability greater or equal to that shown on the Shop Drawings and the reinforced soil permeability.

B. Geogrids. The geogrid shall be a regular network of integrally connected polymer tensile elements with aperture geometry sufficient to permit significant mechanical interlock with the surrounding soil or rock. The geogrid structure shall be dimensionally stable and able to retain its geometry under manufacture, transport and installation. Geogrids shall conform as a minimum to the criteria specified in part D and E below.

C. Required Properties. The specific geosynthetic materials shall be preapproved and shall the have the ultimate tensile strength (T_{ult}) shown on the approved Shop Drawings for the geosynthetic specified and for the fill type shown. T_{ult} shall be determined from wide width tests specified in ASTM D 4595 for geotextiles and ASTM D 6637 or GRI:GG1 for geogrids. The ultimate tensile strength value is based on the minimum average roll values (MARV) for the product.

D. The geosynthetic shall conform to the following criteria:
    1. PP and HDPE: Min. retained strength of 70 percent after 150 hours, per ASTM D-4355.
    2. HDPE: Grade = E-4, E-5, E-8, E-9, E-10, E-11, J-3, J-4, or J-5, per ASTM D-1248.
    3. PET: Molecular weight (Mn) > 25,000, per GRI:GG8 and ASTM D-4603.
    4. PET: Carboxyl end group (CEG) ≥ 15 mmol/kg, GRI:GG7.
    5. All polymers: Minimum Weight per Unit Area of 8 oz/yd^2, per ASTM D-5261.
    6. All Polymers: Maximum 0 percent post consumer recycled material by weight.
    7. A default total reduction factor for creep, durability, and installation damage of RF = 7 may be used in design, provided the criteria of 2 through 6 are satisfied and 1 is adjusted to 70 percent after 500 hours is satisfied.

E. Manufacturer Quality Control. The geosynthetic reinforcements shall be manufactured with a high degree of quality control. The Manufacturer is responsible for establishing and maintaining a quality control program to ensure compliance with the requirements of the specification. The purpose of the QC testing program is to verify that the reinforcement geosynthetic being supplied to the project is representative of the material used for performance testing and approval. Conformance testing shall be performed as part of the manufacturing process and may vary for each type of product. As a minimum the following index tests shall be considered as applicable for an acceptable QA/QC program:

Property	Test Procedure
1. Specific Gravity (HDPE only)	ASTM D-1505
2. Ultimate Tensile Strength	ASTM D-4595 GRI:GG1
3. Melt Flow (HDPE and PP only)	ASTM D-1238
4. Intrinsic Viscosity (PET only)	ASTM D-4603
5. Carboxyl End Group (PET only)	ASTM D-2455

F. Sampling Testing and Acceptance. Sampling and conformance testing shall be in accordance with ASTM D-4354. Conformance testing procedures are established above.

Geosynthetic product acceptance shall be based on ASTM D-4759. The quality control certificate shall include:

1. Roll numbers and identification
2. Sampling procedures
3. Results of quality control tests, including a description of test methods used.

G. Certification. The Contractor shall submit a manufacturer's certification that the geosynthetics supplied meet the respective index criteria set when the geosynthetic was approved, measured in full accordance with all test methods and standards specified, or referenced, in this specification.

The manufacturer's certificate shall state that the furnished geosynthetic meets the requirements of these specifications as evaluated by the manufacturer's quality control program. The values submitted shall be certified by a person having legal authority to bond the manufacturer. In case of dispute over validity of values, the Resident can require the Contractor to supply test data from an agency approved laboratory to support the values submitted, at the Contractor's cost.

635.033 Concrete Leveling Pad Concrete for leveling pads shall be Fill Concrete conforming to the requirements of Section 502 Structural Concrete. Unless otherwise specified, concrete for leveling pads shall be accepted under Method "C" requirements.

635.034 Drainage Stone Fill Concrete wall unit voids shall be filled with drainage stone material that conforms to the requirements of 703.22, Underdrain Backfill Material, Type C.

635.035 Backfill Material Backfill material placed behind the concrete wall units shall meet the requirements of Section 703.20 Gravel Borrow, except that the backfill material shall only contain particles that will pass the 3-inch square mesh sieve. The contractor is required to submit a grain size distribution curve (ASTM D 422) and a moisture-density relationship curve (AASHTO T-180) for acceptance of the proposed backfill material and determination of the appropriate installation damage reduction factor (RF_{ID}).

Walls with reinforced backfill also require that the backfill material be subjected to pH testing to determine the appropriate durability reduction factor (RF_D).

635.036 Materials Certificate Letter The Contractor, or the supplier as their agent, shall furnish the Resident a Materials Certificate Letter for the above materials, including the backfill material, in accordance with Section 700 of the Standard Specifications. A copy of all test results performed by the Contractor or their supplier necessary to assure contract compliance shall also be furnished to the Resident. The Resident will base acceptance upon the materials Certificate Letter, accompanying test reports, and visual inspection.

635.04 Design Requirements The wall shall be designed with a service life of not less than 75 years. The Precast Aggregate-filled Concrete Block Gravity Wall shall be designed and sealed by a Professional Engineer licensed in the State of Maine. The wall shall be designed in accordance with the following:

1. AASHTO LRFD Bridge Design Specifications, current edition, herein referred to as LRFD
2. FHWA-NHI-00-043 Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines, 2001

3. FHWA-NHI-00-044 Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, 2000
4. The Contract Plans
5. The requirements specified herein
6. The manufacturer's requirements

Where conflicting requirements occur, the more stringent requirements shall govern.

Forty-five days prior to beginning construction of the wall, the design computations shall be submitted to the Resident for review by the Geotechnical Engineer. Any additional design or costs arising as a result of rejection of a wall design by the Geotechnical Engineer shall be borne by the Contractor.

Design calculations that consist of computer program generated output shall be supplemented with at least one hand calculation and graphic demonstrating the design methodology used. Design calculations shall provide thorough documentation of the sources of equations used and material properties. The design by the wall system supplier shall consider the stability of the wall as outlined below and in the Contract Documents:

A. Failure Plane The theoretical failure plane within the reinforced soil mass shall be determined in accordance with LRFD Article 11 and be analyzed so that the soil stabilizing components extend sufficiently beyond the failure plane within the reinforced soil mass to stabilize the material.

B. External Loads External loads which affect the internal and external stability such as those applied through traffic loadings, impact on traffic barrier posts, slope surcharge, hydrostatic, and seismic loads shall be accounted for in the design. Traffic surcharge and traffic impact loads shall be calculated and applied in compliance with LRFD Section 11.

C. External Stability Loads and load combinations selected for design shall be consistent with LRFD. Application of load factors shall be taken as specified in LRFD Section 11. Sliding resistance factors and bearing resistance factors shall be consistent with LRFD. Overturning and sliding provisions of LRFD shall apply.

D. Internal Stability Evaluation of reinforcement pullout, reinforcement rupture and reinforcement/block connection pullout or rupture shall be consistent with LRFD Section 11, and checked at each level. Loads, load combinations and load factors shall be as specified in LRFD Section 11. Resistance factors for internal design are specified in LRFD Section 11. Maximum reinforcement loads shall be calculated using the Simplified Method approach. Calculations for factored stresses and resistances shall be based upon assumed conditions at the end of the design life.

a. Geosynthetic Reinforcement Design Tensile Resistance The nominal long term reinforcement design strength (T_{al}) shall be determined by reducing T_{ult} by reduction factors (RF) in accordance with the documents referenced above. The designer shall procure and use the manufacturers tested and certified geosynthetic reinforcement reduction factors for creep (RF_{CR}), durability (RF_D), and installation damage (RF_{ID}) to determine T_{al}. In absence of manufacturers tested and certified reduction factors, a combined default reduction factor RF = 7 shall be used in accordance with the referenced documents. For RF_{ID}, the installation damage reduction factor shall be checked in accordance with LRFD and FHWA-NHI-00-044.

b. Reinforcement/Facing Connection Design Strength The nominal long-term connection strength between the geosynthetic reinforcement and the concrete blocks shall be checked in accordance with LRFD and FHWA-NHI-00-043.

c. Reinforcement Pullout The pullout resistance factor, (F*), and scale effect correction factor (α) used in pullout design, shall be determined from project specific pullout tests using the proposed geosynthetic in the specified project backfill material or equivalent soil. The pullout resistance factors shall be determined in accordance with LRFD and FHWA-NHI-00-043. In the absence of test data, empirical relationships may be used to determine the pullout resistance factors, any empirical relationships used in design shall be referenced in the design calculations.

E. Backfill and Foundation Soils Parameters The friction angle of the backfill used in the reinforced fill zone for internal stability design shall be assumed have a friction angle of 34 degrees unless specific project select backfill is tested for frictional strength. The friction angle of the foundation soils and random backfill shall be assumed to be 30 degrees unless otherwise shown on the plans.

F. Reinforcement Length The soil reinforcement shall be the same length from the bottom to the top of each wall section. The reinforcement length defining the width of the entire reinforced soil mass may vary with wall height. The minimum length of the soil reinforcement shall be 8 ft, but shall not be less than 70 percent of the wall height, H, for walls with level surcharges, or 70 percent of H1 for walls with a sloped surcharge or walls supporting an abutment. The mechanical wall height, H or H1, shall be the vertical difference between the top of the leveling footing and the elevation at which the failure surface, as described above, intercepts the ground surface supported by the wall.

G. Bearing Resistance The factored bearing pressures under the Precast Aggregate-filled Concrete Block Gravity Wall shall be clearly indicated on the Shop Drawings. Walls shall be dimensioned so that the factored bearing resistance of the foundation soils, as noted on the Plans, is not exceeded.

H. Facing Stability Stability calculations for the concrete facing blocks shall be in accordance with LRFD, and shall include an evaluation of the maximum vertical spacing between reinforcement layers.

I. Stability During Construction Walls shall be designed to resist failure by instability of temporary construction slope. Passive pressure in front of the wall mass shall be assumed to be zero for design purposes.

J. Design Life The wall design life shall be a minimum of 75 years.

K. Depth of Embedment The depth of embedment for frost protection and stability shall be at or below the elevation shown on the Plans and the approved Shop Drawings.

L. Drainage System Piped drainage shall be designed to collect and dispose of water from the base of the reinforced soil zone and backfill soil. This shall outlet into surrounding drainage systems or ditches.

635.05 Submittals. The Contractor shall supply wall design computations, wall details, dimensions, quantities, and cross sections necessary to construct the wall. A sample hand calculation including all equations, parameter values used, units, theory, free-body diagram, comparison to design requirements, etc. shall be provided. Spreadsheet calculations alone are not acceptable.

Forty-five (45) days prior to beginning construction of the wall, four (4) sets of the wall design computations and Shop Drawings shall be submitted to the Resident for review by the Geotechnical Engineer. Mix design information shall be submitted at the same time, including aggregate source, current gradation, aggregate quality information and concrete unit weight.

The contractor shall also submit backfill material test results as part of the wall submittal package. Backfill material test results shall include grain size distribution curve, moisture-density relationship curve, and pH test results required for reinforced backfill only.

If geotechnical design is required, the fully detailed plans shall be prepared in conformance with Section 105 and shall include, but not be limited to the following items:

A. A plan and elevation sheet or sheets for each wall, containing the following: elevations at the top of leveling pads, the distance along the face of the wall to all steps in the leveling pads, the location of the original and final ground line.

B. All details for foundations and leveling pads, including details for steps in the leveling pads, as well as allowable and actual maximum bearing pressures shall be provided.

C. Details for the barriers, posts, curbs and facing as required by the project conditions.

D. Design computations prepared and sealed by a licensed Professional Engineer.

E. Prior to the beginning of construction, the contractor shall supply the Resident with two copies of the design-supplier's Installation Manual. In addition, the Contractor shall have two copies of the Installation Manual on the project site.

635.06 Construction Requirements The Precast Aggregate-Filled Concrete Block Gravity Wall shall have the following construction requirements:

A. Excavation. The excavation and use as fill disposal of all excavated material shall meet the requirements of Section 203 -- Excavation and Embankment, except as modified herein.

B. Foundation. The area upon which the prefabricated, aggregate-filled concrete block gravity wall structure is to rest, and within the limits shown on the submitted plans, shall be graded for a width equal to, or exceeding, the length of the blocks. Prior to wall and leveling pad construction, this foundation material shall be compacted to at least 95 percent of maximum laboratory dry density (AASHTO T-180 Method C or D). Frozen and unsuitable soil shall be removed and replaced with gravel borrow compacted to 95 percent of AASHTO T-180, or as shown on the plans.

A concrete leveling pad shall be constructed a minimum of 6 inches beyond the front and back of the concrete wall units, or as indicated on the plans. Dimensions may be modified per the wall supplier's recommendations, with written approval of the Geotechnical Engineer. The leveling pad shall be cast to the design elevations as shown on the plans, or as required by the wall supplier upon written approval of the Geotechnical Engineer.

The allowable elevation tolerances from the design elevations are +0.01 ft and -0.02 ft. Leveling pads which do not meet this requirement shall be repaired or replaced as directed by the Resident at no additional cost to the Department. Placement of wall units may begin after the strength of the concrete leveling pad reaches 1000 psi or is adequate to support the proposed loads. Contractor may begin placement of concrete block units after 12 hours at their own risk.

C. Method and Equipment. Prior to erection of the wall, the Contractor shall furnish the Resident with detailed information concerning the proposed construction method and equipment to be used. The erection procedure shall be in accordance with the manufacturer's instructions. Any units that are damaged due to handling will be replaced at the Contractor's expense.

D. Installation of Concrete Wall Units. A field representative from the wall system being used shall be available, as needed, during the erection of the wall. The services of the representative shall be at no additional cost to the project.

The contractor shall place the first course of wall units directly on the leveling pad and check for level and alignment. Adjacent units should be in contact. The prefabricated concrete wall units shall be installed to a tolerance of plus or minus 3/4 inch in 10 ft in vertical and horizontal alignment.

Fill all voids between and within the wall units with drainage stone as described in this specification. The drainage stone fill shall extend a minimum of 6 in behind the tails of the wall units unless a geotextile filter is placed over the inside joint at the back of adjacent wall units. If used, the drainage geotextile shall conform to the requirements of Section 722.02.

E. Backfill Placement. Backfill placement shall closely follow the erection of each row of prefabricated wall units. The maximum lift thickness shall be 8 inches loose. The Contractor shall decrease the lift thickness if necessary to obtain the specified density. The backfill shall be compacted in accordance with Section 203.12 except that the minimum required compaction shall be at least 92 percent of maximum density as determined by AASHTO T-180 Method C or D. Backfill compaction shall be accomplished without disturbance or displacement of the concrete wall units. Sheepsfoot rollers will not be allowed. Whenever a compaction test fails, no additional backfill shall be placed over the area until the lift is recompacted and a passing test achieved.

The moisture content of the backfill material prior to and during compaction shall be uniform throughout each layer. Backfill material shall have a placement moisture content less than or equal to the optimum moisture content. Backfill material with placement moisture content in excess of the optimum moisture content shall be removed and reworked until the moisture content is uniform and acceptable throughout the entire lift. The optimum moisture content shall be determined in accordance with AASHTO T-180, Method C or D. At the end of the day's operations, the Contractor shall shape the last level of backfill so as to direct runoff of rainwater away from the wall face.

635.07 Method of Measurement Precast Aggregate-filled Concrete Block Gravity Wall will be measured by the square foot of front surface not to exceed the dimensions shown on the Contract Plans unless authorized by the Resident. Vertical and horizontal dimensions will be from the edges of the blocks. No field measurements for computations will be made unless the Resident specifies, in writing, a change in the limits indicated on the Plans.

635.08 Basis of Payment The accepted quantity of Precast Aggregate-Filled Concrete Block Gravity Wall will be paid for at the contract unit price per square foot complete in place. Payment shall be full compensation for furnishing geotechnical design as required, all labor, equipment and materials including all precast concrete units, hardware, joint fillers, geosynthetic, drainage pipe, and technical field representative.

Cost of cast-in-place concrete for leveling pad will not be paid for separately, but will be considered incidental to the Precast Aggregate-Filled Concrete Block Gravity Wall. Excavation, foundation material and backfill material will all be incidental to the Precast Aggregate-Filled Concrete Block Gravity Wall.

There will be no allowance for excavating and backfilling for the Precast Aggregate-Filled Concrete Block Gravity Wall beyond the limits shown on the approved submitted plans, except for excavation required to remove unsuitable subsoil in preparation for the foundation. Payment for excavating unsuitable subsoil shall be full compensation for all costs of pumping, drainage, sheeting, bracing and incidentals for proper execution of the work, and will be paid as common excavation in accordance with Section 203.

Payment will be made under:

Pay Item	Pay Unit
635.40 Precast Aggregate-Filled Concrete Block Gravity Wall	Square Foot

SPECIAL PROVISION
SECTION 635
PRECAST CONCRETE BLOCK GRAVITY WALL

The following replaces Section 635 in the Standard Specifications in its entirety:

635.01 Description The work under this item shall consist of design, fabrication, furnishing and construction of a Precast Concrete Block Gravity Wall in accordance with these specifications and in close conformance with the lines and grades shown on the Plans, or established by the Resident. The Precast Concrete Block Gravity Wall shall consist of facing blocks made of wet cast concrete made from Portland cement, water, chemical admixtures, and aggregates, supported on concrete leveling pads, and if required, geosynthetic-reinforced backfill.

Included in the scope of the precast gravity wall construction are: geotechnical design of any wall with a exposed height greater than 4.5 feet or as specified on the Plans, all grading necessary for wall construction, compaction of the wall foundation soil, backfill, piped drainage, construction of leveling pads, and block wall installation. The top of the upper row of blocks shall be at or above the top of the face elevation shown on the Plans.

635.02 Quality Assurance The wall system shall be one of the approved combinations of facing block and soil reinforcement systems noted in the Plans or on the Department's Qualified Products List (QPL). Alternate wall systems will not be considered for this Item.

All design calculations and Shop Drawings shall be signed and sealed by a Professional Engineer licensed in the State of Maine.

The Contractor shall require the wall design-supplier to provide an on-site, qualified experienced technical representative to advise the Contractor concerning proper installation procedures. The technical representative shall be on-site during initial stages of installation and thereafter shall remain available for consultation as necessary for the Contractor or as required by the Resident.

635.03 Materials Materials for walls shall meet the requirements of the following sections of Division 700:

Gravel Borrow	703.20
Underdrain Backfill Type C	703.22
Underdrain Pipe	706.06 or 706.09
Reinforcing Steel	709.01
Structural Precast Concrete Units	712.061
Reinforcement Geotextile	722.01
Drainage Geotextile	722.02

The Contractor is cautioned that all of the materials listed are not required for every Precast Concrete Block Gravity Wall. The Contractor shall furnish the Resident a Materials

Certification Letter certifying that the applicable materials comply with this section of the specifications. Materials shall meet the following additional requirements:

635.031 Concrete Units The Materials Certification Letter described above shall contain the date of concrete casting, a lot identification number, compressive strength results, and entrained air results. All prefabricated concrete units shall conform to the requirements of 712.061 with the following exceptions:

A. Materials. Materials are modified as follows: the maximum water cement ratio shall be 0.42, use of calcium nitrite is not required, and the minimum 28 day compressive strength shall be 4600 psi.

B. Quality Control and Quality Assurance. Quality Control and Quality Assurance is modified as follows: delete the second and third paragraphs.

C. Construction. Construction requirements are modified as follows:

Delete the second paragraph and replace it with the following:
All units for a designated wall system, including end blocks, steps, caps and other wall units shall be manufactured from the same material sources of aggregates, brand and type of cement and color pigment.

Replace the first sentence in the paragraph which begins "The forms shall remain …" with the following:
The forms shall remain in place until the concrete has gained sufficient strength such that removal of the forms and subsequent handling will not damage the units.

Replace the paragraph which begins "A minimum of 8 …." with the following:
The Contractor shall make and test at least one set of cylinders for every 50 yd^3 of production concrete used to cast the concrete units.

Replace the paragraph which begins "At least once …" with the following:
The Contractor shall make four cylinders for use by the Department for every 200 yd^3.

Add the following paragraph at the end of the Construction section:
Face texture of the units shall be a formed finish on all exposed surfaces. Pigment shall be added during the casting process of the concrete unit to achieve a consistent shade of gray or other color as determined by the Resident.

D. Tolerances. Maximum dimensional deviation of formed unit dimensions shall be ½ - inch or 2 percent or the manufacturer's published tolerances, whichever is less. Units not meeting the specified tolerances will be rejected.

635.032 Geosynthetic Reinforcement Geosynthetic reinforcement shall be as required by the proprietary wall system manufacturer or wall designer. Geosynthetic reinforcement shall consist of a geotextile or geogrid approved by the Geotechnical Engineer. Substitution of a geosynthetic other than that required by the proprietary wall system manufacturer shall not be allowed unless approved by the Geotechnical Engineer after submittal of shop drawings and pullout and interface friction test data.

A. Geotextiles and Thread for Sewing. Woven or nonwoven geotextiles shall consist of long chain polymeric filaments or yarns formed into a stable network such that the filaments or yarns retain their position relative to each other during handling, placement, and design life. At least 95 percent by weight of the long chain polymer shall be polyolefin or polyester. The material shall be free of defects and tears. Geotextiles used for reinforcement shall conform as a minimum to the properties indicated for 722.01, Stabilization/Reinforcement Geotextile and shall meet the requirements of part D and E below. Geotextiles shall have a minimum permeability greater or equal to that shown on the Shop Drawings and the reinforced soil permeability.

B. Geogrids. The geogrid shall be a regular network of integrally connected polymer tensile elements with aperture geometry sufficient to permit significant mechanical interlock with the surrounding soil or rock. The geogrid structure shall be dimensionally stable and able to retain its geometry under manufacture, transport and installation. Geogrids shall conform as a minimum to the criteria specified in part D and E below.

C. Required Properties. The specific geosynthetic materials shall be preapproved and shall the have the ultimate tensile strength (T_{ult}) shown on the approved Shop Drawings for the geosynthetic specified and for the fill type shown. T_{ult} shall be determined from wide width tests specified in ASTM D 4595 for geotextiles and ASTM D 6637 or GRI:GG1 for geogrids. The ultimate tensile strength value is based on the minimum average roll values (MARV) for the product.

D. The geosynthetic shall conform to the following criteria:
    1. PP and HDPE: Min. retained strength of 70 percent after 150 hours, per ASTM D-4355.
    2. HDPE: Grade = E-4, E-5, E-8, E-9, E-10, E-11, J-3, J-4, or J-5, per ASTM D-1248.
    3. PET: Molecular weight (Mn) > 25,000, per GRI:GG8 and ASTM D-4603.
    4. PET: Carboxyl end group (CEG) ≥ 15 mmol/kg, GRI:GG7.
    5. All polymers: Minimum Weight per Unit Area of 8 oz/yd^2, per ASTM D-5261.
    6. All Polymers: Maximum 0 percent post consumer recycled material by weight.
    7. A default total reduction factor for creep, durability, and installation damage of RF = 7 may be used in design, provided the criteria of 2 through 6 are satisfied and 1 is adjusted to 70 percent after 500 hours is satisfied.

E. Manufacturer Quality Control. The geosynthetic reinforcements shall be manufactured with a high degree of quality control. The Manufacturer is responsible for establishing and maintaining a quality control program to ensure compliance with the requirements of the specification. The purpose of the QC testing program is to verify that the reinforcement geosynthetic being supplied to the project is representative of the material used for performance testing and approval. Conformance testing shall be performed as part of the manufacturing process and may vary for each type of product. As a minimum the following index tests shall be considered as applicable for an acceptable QA/QC program:

Property	Test Procedure
1. Specific Gravity (HDPE only)	ASTM D-1505
2. Ultimate Tensile Strength	ASTM D-4595 GRI:GG1
3. Melt Flow (HDPE and PP only)	ASTM D-1238
4. Intrinsic Viscosity (PET only)	ASTM D-4603
5. Carboxyl End Group (PET only)	ASTM D-2455

F. Sampling Testing and Acceptance. Sampling and conformance testing shall be in accordance with ASTM D-4354. Conformance testing procedures are established above. Geosynthetic product acceptance shall be based on ASTM D-4759. The quality control certificate shall include:

1. Roll numbers and identification
2. Sampling procedures
3. Results of quality control tests, including a description of test methods used.

G. Certification. The Contractor shall submit a manufacturer's certification that the geosynthetics supplied meet the respective index criteria set when the geosynthetic was approved, measured in full accordance with all test methods and standards specified, or referenced, in this specification.

The manufacturer's certificate shall state that the furnished geosynthetic meets the requirements of these specifications as evaluated by the manufacturer's quality control program. The values submitted shall be certified by a person having legal authority to bond the manufacturer. In case of dispute over validity of values, the Resident can require the Contractor to supply test data from an agency approved laboratory to support the values submitted, at the Contractor's cost.

635.033 Geosynthetic Connection Reinforcing bar used in the geosynthetic connection shall be ½-inch diameter epoxy coated reinforcing bar, coated on the ends and meeting the requirements of Section 503, Reinforcing Steel. Installation shall be in accordance with manufacturer's recommendations.

635.034 Concrete Leveling Pad Concrete for leveling pads shall be Fill Concrete conforming to the requirements of Section 502 Structural Concrete. Unless otherwise specified, concrete for leveling pads shall be accepted under Method "C" requirements.

635.035 Backfill Material Backfill material placed behind the concrete units shall meet the requirements of Section 703.20 Gravel Borrow, except that the backfill material shall only contain particles that will pass the 3-inch square mesh sieve. The contractor is required to submit a grain size distribution curve (ASTM D 422) and a moisture-density relationship curve

(AASHTO T-180) for acceptance of the proposed backfill material and determination of the appropriate installation damage reduction factor (RF_{ID}).

Walls with reinforced backfill require that the backfill material be subjected to pH testing to determine the appropriate durability reduction factor (RF_D).

Material between blocks must be Gravel Borrow, or Underdrain Backfill Material meeting the requirements of Section 703.22, Type C.

635.036 Materials Certification Letter The Contractor, or the supplier as his agent, shall furnish the Resident a Materials Certification Letter for the above materials, including the backfill material, in accordance with Section 700 of the Standard Specifications. A copy of all test results performed by the Contractor or his supplier necessary to assure contract compliance shall also be furnished to the Resident. The Resident will base acceptance upon the materials Certificate Letter, accompanying test reports, and visual inspection.

635.04 Design Requirements The wall shall be designed with a service life of not less than 75 years. The Precast Concrete Block Gravity Wall shall be designed and sealed by a Professional Engineer licensed in the State of Maine. The wall shall be designed in accordance with the following:

1. AASHTO LRFD Bridge Design Specifications, current edition, herein referred to as LRFD
2. FHWA-NHI-00-043 Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines, 2001
3. FHWA-NHI-00-044 Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, 2000
4. The Contract Plans
5. The requirements specified herein
6. The manufacturer's requirements

Where conflicting requirements occur, the more stringent requirements shall govern.

Forty-five days prior to beginning construction of the wall, the design computations shall be submitted to the Resident for review by the Geotechnical Engineer. Any additional design or costs arising as a result of rejection of a wall design by the Geotechnical Engineer shall be borne by the Contractor.

Design calculations that consist of computer program generated output shall be supplemented with at least one hand calculation and graphic demonstrating the design methodology used. Design calculations shall provide thorough documentation of the sources of equations used and material properties. The design by the wall system supplier shall consider the stability of the wall as outlined below and in the Contract Documents:

A. Failure Plane The theoretical failure plane within the reinforced soil mass shall be determined in accordance with LRFD Article 11 and be analyzed so that the soil stabilizing components extend sufficiently beyond the failure plane within the reinforced soil mass to stabilize the material.

B. External Loads External loads which affect the internal and external stability such as those applied through traffic loadings, impact on traffic barrier posts, slope surcharge, hydrostatic, and seismic loads shall be accounted for in the design. Traffic surcharge and traffic impact loads shall be calculated and applied in compliance with LRFD Section 11.

C. External Stability Loads and load combinations selected for design shall be consistent with LRFD. Application of load factors shall be taken as specified in LRFD Section 11. Sliding resistance factors and bearing resistance factors shall be consistent with LRFD. Overturning and sliding provisions of LRFD shall apply.

D. Internal Stability Evaluation of reinforcement pullout, reinforcement rupture and reinforcement/block connection pullout or rupture shall be consistent with LRFD Section 11, and checked at each level. Loads, load combinations and load factors shall be as specified in LRFD Section 11. Resistance factors for internal design are specified in LRFD Section 11. Maximum reinforcement loads shall be calculated using the Simplified Method approach. Calculations for factored stresses and resistances shall be based upon assumed conditions at the end of the design life.

a. Geosynthetic Reinforcement Design Tensile Resistance The nominal long term reinforcement design strength (T_{al}) shall be determined by reducing T_{ult} by reduction factors (RF) in accordance with the documents referenced above. The designer shall procure and use the manufacturers tested and certified geosynthetic reinforcement reduction factors for creep (RF_{CR}), durability (RF_D), and installation damage (RF_{ID}) to determine T_{al}. In absence of manufacturers tested and certified reduction factors, a combined default reduction factor RF = 7 shall be used in accordance with the referenced documents. For RF_{ID}, the installation damage reduction factor shall be checked in accordance with LRFD and FHWA-NHI-00-044.

b. Reinforcement/Facing Connection Design Strength The nominal long-term connection strength between the geosynthetic reinforcement and the concrete blocks shall be checked in accordance with LRFD and FHWA-NHI-00-043.

c. Reinforcement Pullout The pullout resistance factor, (F*), and scale effect correction factor (α) used in pullout design, shall be determined from project specific pullout tests using the proposed geosynthetic in the specified project backfill material or equivalent soil. The pullout resistance factors shall be determined in accordance with LRFD and FHWA-NHI-00-043. In the absence of test data, empirical relationships may be used to determine the pullout resistance factors, any empirical relationships used in design shall be referenced in the design calculations.

E. Backfill and Foundation Soils Parameters The friction angle of the backfill used in the reinforced fill zone for internal stability design shall be assumed have a friction angle of 34 degrees unless specific project select backfill is tested for frictional strength. The friction angle of the foundation soils and random backfill shall be assumed to be 30 degrees unless otherwise shown on the plans.

F. Reinforcement Length The soil reinforcement shall be the same length from the bottom to the top of each wall section. The reinforcement length defining the width of the entire reinforced soil mass may vary with wall height. The minimum length of the soil reinforcement shall be 8 ft, but shall not be less than 70 percent of the wall height, H, for walls with level surcharges, or 70 percent of H1 for walls with a sloped surcharge or walls supporting an abutment. The mechanical wall height, H or H1, shall be the vertical difference between the top of the leveling footing and the elevation at which the failure surface, as described above, intercepts the ground surface supported by the wall.

G. Bearing Resistance The factored bearing pressures under the Precast Concrete Block Gravity Wall shall be clearly indicated on the Shop Drawings. Walls shall be dimensioned so that the factored bearing resistance of the foundation soils, as noted on the Plans, is not exceeded.

H. Facing Stability Stability calculations for the concrete facing blocks shall be in accordance with LRFD, and shall include an evaluation of the maximum vertical spacing between reinforcement layers.

I. Stability During Construction Walls shall be designed to resist failure by instability of temporary construction slope. Passive pressure in front of the wall mass shall be assumed to be zero for design purposes.

J. Design Life The wall design life shall be a minimum of 75 years.

K. Depth of Embedment The depth of embedment for frost protection and stability shall be at or below the elevation shown on the Plans and the approved Shop Drawings.

L. Drainage System Piped drainage shall be designed to collect and dispose of water from the base of the reinforced soil zone and backfill soil. This shall outlet into surrounding drainage systems or ditches.

635.05 Submittals The Contractor shall supply wall design computations, wall details, dimensions, quantities, and cross sections necessary to construct the wall. A sample hand calculation including all equations, parameter values used, units, theory, free-body diagram, comparison to design requirements, etc. shall be provided. Spread sheet calculations alone are not acceptable.

Forty-five days prior to beginning construction of the wall, four (4) sets of the wall design computations and Shop Drawings shall be submitted to the Resident for review by the

Geotechnical Engineer. Mix design information shall be submitted at the same time, including aggregate source, current gradation, aggregate quality information and concrete unit weight.

The contractor shall also submit backfill material test results as part of the wall submittal package. Backfill material test results shall include grain size distribution curve, moisture-density relationship curve, and pH test results required for reinforced backfill only.

If geotechnical design is required, the fully detailed plans shall be prepared in conformance with Section 105 and shall include, but not be limited to the following items:

A. A plan and elevation sheet or sheets for each wall, containing the following: elevations at the top of leveling pads, the distance along the face of the wall to all steps in the leveling pads, the location of the original and final ground line.

B. All details for foundations and leveling pads, including details for steps in the leveling pads, as well as allowable and actual maximum bearing pressures shall be provided.

C. Details for the barriers, posts, curbs and facing as required by the project conditions.

D. Design computations prepared and sealed by a licensed Professional Engineer.

E. Prior to the beginning of construction, the contractor shall supply the Resident with two copies of the design-supplier's Installation Manual. In addition, the Contractor shall have two copies of the Installation Manual on the project site.

635.06 Construction Requirements The Precast Concrete Block Gravity Wall shall have the following construction requirements:

A. Excavation. The excavation and use as fill disposal of all excavated material shall meet the requirements of Section 203 - Excavation and Embankment, except as modified herein.

B. Foundation. The area upon which the prefabricated block gravity wall structure is to rest, and within the limits shown on the submitted plans, shall be graded for a width equal to, or exceeding, the length of the blocks. Prior to wall and leveling pad construction, this foundation material shall be compacted to at least 95 percent of maximum laboratory dry density (AASHTO T-180 Method C or D). Frozen and unsuitable soil shall be removed and replaced with gravel borrow compacted to 95 percent of AASHTO T-180.

   A concrete leveling pad shall be constructed as indicated on the plans. Dimensions may be modified per the wall supplier's recommendations, with written approval of the Geotechnical Engineer. The leveling pad shall be cast to the design elevations as shown on the plans, or as required by the wall supplier upon written approval of the Geotechnical Engineer. The allowable elevation tolerances from the design elevations are +0.01 feet and -0.02 feet. Leveling pads which do not meet this requirement shall be repaired or replaced as directed by the Resident at no additional cost to the Department. Placement of wall units may begin after the strength of the concrete leveling pad reaches 1000 psi or is adequate to support the proposed loads. Contractor may begin placement of concrete block units after 12 hours at his own risk.

C. Method and Equipment. Prior to erection of the prefabricated concrete block wall, the Contractor shall furnish the Resident with detailed information concerning the proposed construction method and equipment to be used. The erection procedure shall be in accordance with the manufacturer's instructions. Any units that are damaged due to handling will be replaced at the Contractor's expense.

D. Installation of Wall Units. A field representative from the wall system being used shall be available, as needed, during the erection of the wall. The services of the representative shall be at no additional cost to the project. Horizontal joint fillers shall be installed as needed.

The maximum offset in any unit horizontal joint shall be 1/4 inch. The prefabricated wall blocks shall be installed to a tolerance of plus or minus 3/4 inch in 10 feet in vertical alignment and horizontal alignment.

E. Backfill Placement. Backfill placement shall closely follow the erection of each row of prefabricated wall units. The Contractor shall decrease the lift thickness if necessary to obtain the specified density. The maximum lift thickness shall be 8 inches loose. Gravel borrow backfill shall be compacted in accordance with Section 203.12 except that the minimum required compaction shall be at least 92 percent of maximum density as determined by AASHTO T-180 Method C or D. Backfill compaction shall be accomplished without disturbance or displacement of the wall blocks. Sheepsfoot rollers will not be allowed. Whenever a compaction test fails, no additional backfill shall be placed over the area until the lift is recompacted and a passing test achieved.

The moisture content of the backfill material prior to and during compaction shall be uniform throughout each layer. Backfill material shall have a placement moisture content less than or equal to the optimum moisture content. Backfill material with a placement moisture content in excess of the optimum moisture content shall be removed and reworked until the moisture content is uniform and acceptable throughout the entire lift. The optimum moisture content shall be determined in accordance with AASHTO T-180, Method C or D. At the end of the day's operations, the Contractor shall shape the last level of backfill so as to direct runoff of rain water away from the wall face.

Material between blocks must be Gravel Borrow or Underdrain Backfill Material meeting the requirements of Section 703.22, Type C. If Gravel Borrow is used between blocks, 722.02 drainage geotextile shall be placed behind vertical joints to prevent loss of granular material between blocks. Compliance with the gradation requirements shall be the responsibility of the Contractor, who shall furnish a copy of the backfill test results prior to construction. If Underdrain Backfill Material is used between blocks, no geotextile is required behind vertical joints.

635.07 Method of Measurement Precast Concrete Block Gravity Wall will be measured by the square foot of front surface not to exceed the dimensions shown on the Contract Plans unless authorized by the Resident. Vertical and horizontal dimensions will be from the edges of the blocks. No field measurements for computations will be made unless the Resident specifies, in writing, a change in the limits indicated on the Plans.

635.08 Basis of Payment The accepted quantity of Precast Concrete Block Gravity Wall will be paid for at the contract unit price per square foot complete in place. Payment shall be full compensation for furnishing geotechnical design as required, all labor, equipment and materials including all precast concrete units, hardware, joint fillers, geosynthetics, reinforcing steel, drainage pipe, backfill materials and technical field representative.

Cost of cast-in-place concrete for leveling pad will not be paid for separately, but will be considered incidental to the Precast Concrete Block Gravity Wall. Excavation, foundation material and backfill material will all be incidental to the Precast Concrete Block Gravity Wall.

There will be no allowance for excavating and backfilling for the Precast Concrete Block Gravity Wall beyond the limits shown on the approved submitted plans, except for excavation required to remove unsuitable subsoil in preparation for the foundation. Payment for excavating unsuitable subsoil shall be full compensation for all costs of pumping, drainage, sheeting, bracing and incidentals for proper execution of the work, and will be paid as Common Excavation in accordance with Section 203.

Payment will be made under:

Pay Item	Pay Unit
635.31 Precast Concrete Block Gravity Wall	square foot