

Soils Report 2010-01 Addendum #1

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Since the January 22, 2010 publication of the Geotechnical Design Report for the Replacement of Underwitted Road Bridge over Piscataqua River, Falmouth, Maine, Soils Report No. 2010-01, Bridge Program Management has made the decision that the existing twin, 16-foot diameter, steel culverts will not be replaced but will be invert lined. No geotechnical recommendations are necessary for the invert lining application. The subsurface data reported in the Geotechnical Design Report will remain in the MaineDOT archives to assist in future work at the project site.

**MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE**

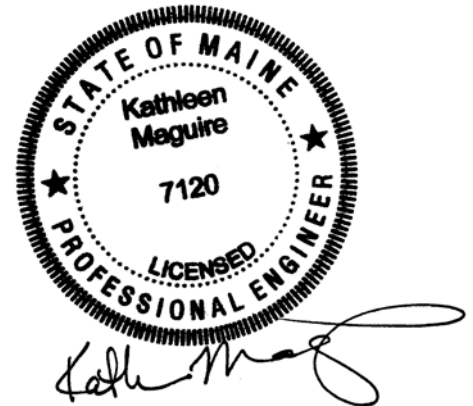
GEOTECHNICAL DESIGN REPORT

For the Replacement of:

**UNDERWITTED ROAD BRIDGE
OVER PISCATAQUA RIVER
FALMOUTH, MAINE**

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Soils Report No. 2010-01
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GEOTECHNICAL DESIGN SUMMARY

The purpose of this report is to present subsurface information and make geotechnical recommendations for the replacement of the Underwitted Road Bridge over the Piscataqua River in Falmouth, Maine. The MaineDOT Bridge Program has selected the Underwitted Road Bridge site as a location to install a rigidified, inflatable, composite, tubular arch bridge structure. The proposed 38 foot, single span, replacement structure will be founded on reinforced concrete spread footings cast on concrete seals on bedrock. The following design recommendations are discussed in detail in the attached report:

Arch and Wingwall Spread Footings and Concrete Seals – Arch concrete footings, seals and wingwalls shall be designed to resist all lateral earth loads, vehicular loads, arch dead and live loads, and lateral thrust forces transferred through the bridge arches. The design of arch and wingwall spread footings at the strength limit state shall consider factored bearing resistance, eccentricity (overturning), lateral sliding and reinforced-concrete structural design. Spread footings shall be designed at the strength limit state considering the loss of lateral support due to scour from the design flood event. The design of spread footings at the service limit state shall consider tolerable horizontal movement of the footing, and overall stability of the footing considering changes in the foundation conditions due to the scour resulting from the design flood event. Extreme limit state design shall check that the nominal footing resistance remaining after scour due to the check flood can support the factored extreme limit state loads. Spread footing design at the service limit state shall be assessed for: settlement, excessive horizontal movement, and movement resulting after scour due to the design flood. The overall stability of the foundation should be investigated.

Calculation of earth pressures on concrete seals or spread footings resisting lateral thrust forces from the arches should assume an at rest earth pressure coefficient, K_o , of 0.47, assuming the arch footings are to be prevented from movement. A resistance factor, ϕ , of 0.50 for at rest earth pressures mobilized to resist lateral sliding forces is recommended. The design of arch footing reinforcing steel for at rest earth pressure shall be assumed a maximum load factor, γ_{EH} of 1.50.

Independent Wingwalls – Independent wingwalls shall be designed as free to rotate at the top in a state of active earth pressure. An active earth pressure coefficient, K_a , of 0.31 is recommended.

Bearing Resistance – The bearing resistance for any structure founded on competent, sound bedrock shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 8 ksf. A factored bearing resistance of 20 ksf may be used and for preliminary footing sizing, and to control settlements when analyzing the service limit state load combination. Bearing resistance for Prefabricated Concrete Modular Gravity (PCMG) walls founded on a leveling slab on fill soils shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 5 ksf for wall system bases less than 8 feet wide and 6.5 ksf for bases from 10 to 12 feet wide. Based on presumptive bearing resistance values a factored bearing resistance of 6 ksf may be used to control settlement when analyzing the service limit state and for preliminary footing sizing.

Prefabricated Concrete Modular Gravity Wall - Precast Concrete Modular Gravity (PCMG) walls may be constructed on all four corners of the bridge to retain the roadway section and minimize impacts. In general PCMG walls should only be used above the ordinary high water elevation (Q1.1). Should PCMG wingwalls be used at stream crossings below Q1.1, the flow velocities should be low and the potential for severe ice or wave action should be minimal. These walls shall be designed by a Professional Engineer subcontracted by the Contractor as a design-build item. The walls shall be designed in accordance with LRFD and Special Provision 635 and plan notes.

Frost Protection - Any foundation placed on granular subgrade soils including the PCMG wall base shall be founded a minimum of 5.0 feet below finished exterior grade for frost protection. For foundations on bedrock, heave due to frost is not a design issue and no requirements for minimum depth of frost embedment are necessary.

Scour and Riprap - The consequences of changes in foundation conditions resulting from the design flood for scour shall be considered at the strength and service limit states. For scour protection and protection of spread footings, the bridge approach slopes and slopes at arch footings should be armored with 3 feet of riprap. The riprap shall be underlain by a Class 1 nonwoven erosion control geotextile and a 1 foot thick layer of bedding material.

Settlement - The grade of the existing bridge approaches will be lowered slightly in the replacement of the structure. Post-construction settlements are anticipated to be negligible.

Seismic Design Considerations – Seismic analysis is not required for buried structures, except where they cross active faults. There are no known active faults in Maine; therefore seismic analysis is not required.

Construction Considerations – Construction of the arch spread footings and concrete seals will require soil excavation and removal of the existing twin steel culverts. Construction activities may require cofferdams and earth support systems. The nature, slope and degree of fracturing in the bedrock bearing surfaces will not be evident until the foundation excavation is made. The bedrock surface shall be cleared of all loose fractured bedrock, loose decomposed bedrock and soil. The final bedrock surface shall be approved by the Resident prior to placement of the footing or seal concrete.

1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present geotechnical recommendations for the replacement of the Underwitted Road Bridge over the Piscataqua River in Falmouth, Maine. A subsurface investigation at the site has been completed. The purpose of the investigation was to explore subsurface conditions at the site in order to develop geotechnical recommendations for the bridge replacement. This report presents the soils information obtained at the site, geotechnical design recommendations, and foundation recommendations.

The existing Underwitted Road Bridge carries Leighton Road over the Piscataqua River and was constructed in 1957. The bridge consists of twin, 16 foot diameter steel culverts with a total span of approximately 34 feet. The 2007 Maine Department of Transportation (MaineDOT) maintenance inspection reports indicate that the culverts are in fair to poor condition with “considerable damage” (rating of 4). The Bridge Sufficiency Rating is 69.6. The structure has a scour critical rating of “8 Stable Above Footing” meaning that the foundations have been determined to be stable for the assessed or calculated scour condition. Inspection records note that the culverts show moderate pitting and rust below the flow line and scattered pin holes. Embankment erosion was noted between the pipes at the inlet.

The MaineDOT Bridge Program has selected the Underwitted Road Bridge site as a location to install a rigidified, inflatable, composite, tubular arch bridge structure developed by the University of Maine’s Advance Engineering Wood Composites (AECW) Center in Orono, Maine. The carbon fiber tubes are inflated and then infused with resin. After hardening, the tubes are transported to the bridge site and are lowered into place and filled with concrete. The proposed arch structure will have a span length of approximately 38 feet and will be founded on spread footings constructed on concrete seals founded on bedrock. The proposed bridge alignment will closely match the existing alignment. The proposed roadway grade will be slightly lower than the existing grade. The roadway will be widened on both sides of the road to accommodate a wider roadway section. The bridge will be closed to traffic during the replacement.

2.0 GEOLOGIC SETTING

The Underwitted Road Bridge in Falmouth carries Leighton Road over the Piscataqua River approximately 0.2 miles east of Route 100 as shown on Sheet 1 - Location Map found at the end of this report. The Piscataqua River flows in a southeasterly direction into the Presumpscot River.

According to the Surficial Geologic Map of Maine published by the Maine Geological Survey (1985) the surficial soils in the vicinity of the site consist of glaciomarine deposits. Soils in the site area are generally comprised of silt, clay, sand and minor amounts of gravel. Sand is dominant in some areas, but may be underlain by finer-grained sediments. The unit contains small areas of till not completely covered by marine sediments. The unit generally is deposited in areas where the topography is gently sloping except where dissected by modern streams and commonly has a branching network of steep-walled stream gullies.

These soils were generally deposited as glacial sediments that accumulated on the ocean floor during the late-glacial marine submergence of lowland areas in southern Maine.

According to the Bedrock Geologic Map of Maine, published by the Maine Geological Survey (1985), the bedrock at the site is identified as calcareous sandstone and interbedded sandstone and impure limestone of the Vassalboro Formation.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions were explored by drilling two (2) test borings at the site. Test boring BB-FPR-101 was drilled at the east end of the existing structure. Test boring BB-FPR-102 was drilled at the west end of the existing structure.

The exploration locations and an interpretive subsurface profile depicting the site stratigraphy are shown on Sheet 2 - Boring Location Plan and Interpretive Subsurface Profile found at the end of this report. The borings were drilled on July 12 and August 10, 2009 by the MaineDOT drill crew. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A - Boring Logs and on Sheet 3 - Boring Logs found end of this report.

The borings were drilled using solid stem auger and driven cased wash boring drilling 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. MaineDOT drill rig is equipped with an automatic hammer to drive the split spoon. The hammer was calibrated in February of 2009 and was found to deliver approximately 40 percent more energy during driving than the standard rope and cathead system. All N-values discussed in this report are corrected values computed by applying an average energy transfer factor of 0.84 to the raw field N-values. This hammer efficiency factor (0.84) 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-2" core barrel and the Rock Quality Designation (RQD) of the core was calculated.

The MaineDOT geotechnical team member selected the boring locations and drilling methods, designated type and depth of sampling techniques and identified field and laboratory testing requirements. A Northeast Transportation Technician Certification Program (NETTCP) Certified Subsurface Inspector logged the subsurface conditions encountered. The borings were located in the field by use of a tape after completion of the drilling program.

4.0 LABORATORY TESTING

Laboratory testing for samples obtained in the borings consisted of ten (10) standard grain size analyses with water content. The results of these laboratory tests are provided in Appendix B - Laboratory Data at the end of this report. Moisture content information and

other soil test results are included on the Boring Logs in Appendix A and on Sheet 3 - Boring Logs found at the end of this report.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered at the test borings generally consisted of fill sands, underlain by a thin layer of native sand, underlain by bedrock. An interpretive subsurface profile depicting the site stratigraphy is shown 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 in detail:

5.1 Sand Fill

Several layers of sand fill, silty sand and silt were encountered beneath the pavement. The thickness of the sand fill layer was approximately 18.5 feet in both borings. The soil generally consisted of:

- Brown, moist, fine to coarse sand with trace to some gravel and trace silt.
- Brown, damp, gravelly fine to coarse sand with trace silt.
- Light brown, moist, fine to coarse sand with trace gravel and trace silt.
- Grey, wet, silt, with little fine to coarse sand, little gravel and occasional cobbles.
- Brown, wet, fine to coarse sand with little silt and little gravel.
- Grey wet, silty, fine to coarse sand with trace gravel and little organics.

Corrected SPT N-values in the sand fill unit ranged from 7 to 28 blows per foot (bpf) indicating that the sand fill is loose to medium dense in consistency. The corrected SPT N-value in the silt unit was >50 blows per foot indicating that the silt is hard in consistency. Water contents from eight (8) samples obtained within the fill layer range from approximately 3% to 21%. Eight (8) grain size analyses conducted on samples of the fill indicate that the soil is classified as an A-1-b, A-3 or A-4 by the AASHTO Classification System and a SW-SM, SP-SM, SP, SM or ML by the Unified Soil Classification System. Organics were noted in both borings at the bottom of the fill layer (approximately 18.5 feet below ground surface) indicating the old ground surface.

5.2 Native Sand

A thin layer of native sand was encountered beneath the fill. The thickness of the native sand layer ranged from approximately 4.0 feet in boring BB-FPR-101 to approximately 5.8 feet thick boring BB-FPR-102. The native sand generally consisted of grey and brown, wet, gravelly fine to coarse sand and fine to coarse sand, with little silt, and trace to little gravel. Corrected SPT N-values in the native sand layer ranged from 27 to 57 bpf indicating that the soil is medium dense to very dense in consistency. Water contents from two (2) samples obtained within the native sand layer range from approximately 10% to 16%. Two (2) grain size analyses conducted on samples from the native sand layer indicate that the soil is

classified as an A-1-b or A-2-4 by the AASHTO Classification System and a SW-SM or SM by the Unified Soil Classification System.

5.3 Bedrock

Bedrock was encountered and cored in both of the borings. The Table 5-1 summarizes the depths to bedrock and corresponding elevations of the top of bedrock:

Boring Number	Depth to Bedrock	Bedrock Elevation	RQD
BB- FPR -101	23.0 feet	13.5 feet	0 – 80%
BB- FPR -102	24.3 feet	12.2 feet	67 – 83%

Table 5-1 - Summary of Bedrock Depths, Elevations and RQD

The bedrock is identified as violet and green, banded, meta-sandstone/siltstone, that has been re-crystallized to quartz, biotite (chlorite in the green bands), amphibole (hornblende) and feldspar with traces of pyrite and calcite cement. The rock quality designation (RQD) of the bedrock was determined to range from 0 to 83 percent indicating a rock mass quality of very poor to good quality.

5.4 Groundwater

Groundwater was observed at a depths ranging from approximately 15.0 feet to 16.0 feet below the existing ground surface. The water levels measured upon completion of drilling are indicated on the boring logs found in Appendix A. Note that water was introduced into the boreholes during the drilling operations. It is likely that the water levels indicated on the boring logs do not represent stabilized groundwater conditions. Additionally, groundwater levels are expected to fluctuate seasonally depending upon the local precipitation magnitudes.

6.0 FOUNDATION ALTERNATIVES

The MaineDOT Bridge Program has selected the Underwitted Road Bridge site as a location to install a rigidified, inflatable, composite, tubular arch bridge structure developed by the University of Maine’s AEWCA Advanced Structures & Composites Center in Orono, Maine. AEWCA’s tubular arches are made of Fiber Reinforced Polymer (FRP) composite materials. The carbon fiber tubes are inflated off-site and infused with resin. After hardening, the tubes are transported to the bridge site, lowered into place and filled with concrete. The tubular arches are covered with a corrugated, FRP composite deck material and backfill is placed over the tubular structure.

The following foundation alternatives may be considered for the bridge replacement:

- Spread footings founded on soil,
- Spread footings founded on bedrock, or
- Spread footings founded on concrete seals on bedrock.

Due to the shallow depth of overburden at the site the use spread footings founded on concrete seals on bedrock are recommended. Prefabricated Concrete Modular Gravity (PCMG) Walls will be required to support the bridge approaches.

The design of the FRP tubular arches and associated headwalls is the responsibility of the AEW and will be supplied to the designer and Contractor prior to construction of the structure.

7.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

The following sections will discuss geotechnical design recommendations for spread footings founded on concrete seals on bedrock to support the tubular arches which will make up the replacement structure.

7.1 Arch and Wingwall Spread Footings and Concrete Seals

The use of spread footings founded on concrete seals on bedrock is recommended to support the tubular arches which will make up the replacement structure. The concrete seals and spread footings shall be proportioned for all applicable load combinations specified in AASHTO LRFD Bridge Design Specifications 4th Edition (LRFD) Articles 3.4.1, 11.5.5 and 12.5. Arch spread footings and concrete seals shall be designed to resist all lateral earth loads, vehicular loads, arch dead and live loads, and lateral thrust forces transferred through the bridge arches. The design of arch and wingwall spread footings at the strength limit state shall consider factored bearing resistance, eccentricity (overturning), lateral sliding and reinforced-concrete structural design.

In accordance with LRFD Article 12.5.5, the resistance factor values for the geotechnical design of foundations for buried structures shall be as specified in LRFD Section 10 – Foundations. Spread footings shall be designed at the strength limit state considering the loss of lateral support due to scour from the design flood event. The design of spread footings at the service limit state shall consider tolerable horizontal movement of the footing, and overall stability of the footing considering changes in the foundation conditions due to the scour resulting from the design flood event. Extreme limit state design shall check that the nominal footing resistance remaining after scour due to the check flood can support the extreme limit state loads with a resistance factor of 1.0.

Failure by sliding shall be investigated. A sliding resistance factor, ϕ_{τ} , of 0.9 shall be applied to the nominal sliding resistance of cast-in-place arch spread footings constructed on seal concrete. A sliding resistance factor, ϕ_{τ} , of 0.9 shall also be applied to the nominal sliding resistance of concrete seals bearing on bedrock. Sliding computations for resistance to lateral loads shall assume maximum frictional coefficients of 0.70 at the level bedrock-seal concrete interfaces and 0.60 at cast-in-place arch footings to seal interfaces. Anchorage of the arch footings to seals or of seals to bedrock may be required to resist sliding forces and improve stability.

Dowels should be #9 reinforcing bars or larger and be embedded into footings and bedrock by depths determined by the designer. If bedrock is observed to slope steeper than 4H:1V at the arch subgrade elevation, the bedrock should be benched to create level steps.

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

A resistance factor of 1.0 shall be used to assess spread footing design at the service limit state, including: settlement, excessive horizontal movement, and movement resulting after scour due to 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.

Calculation of earth pressures on concrete seals or spread footings resisting lateral thrust forces from the arches should assume an at rest earth pressure coefficient, K_o , of 0.47, assuming the arch footings are to be prevented from movement. A resistance factor for at rest earth pressures mobilized to resist lateral sliding forces is not specified in LRFD, therefore use the resistance for passive pressure, ϕ_{ep} , of 0.50 per LRFD Table 10.5.5.2.2-1. For designing the arch footing reinforcing steel for at rest earth pressure, a maximum load factor, γ_{EH} , of 1.50 is recommended.

The live load surcharge on arch footings may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) taken from the Table 7-1 below:

Arch Height (feet)	h_{eq} (feet)
5	4.0
10	3.0
≥ 20	2.0

Table 7-1 - Equivalent Height of Soil for Vehicular Loading Perpendicular to Traffic

The designer may assume Soil Type 4 (MaineDOT Bridge Design Guide (BDG) Section 3.6.1) for arch and arch footing backfill material soil properties. The backfill properties are as follows: $\phi = 32$ degrees, $\gamma = 125$ pcf.

Arch foundations and wingwall designs shall include a drainage system behind the arch or wall to intercept any groundwater. Drainage behind the structure shall be in accordance with Section 5.4.1.4 Drainage, of the MaineDOT BDG.

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

7.2 Independent Wingwalls

If used, independent wingwalls shall be designed as unrestrained meaning that they are free to rotate at the top in an active state of earth pressure. Earth loads shall be calculated using an active earth pressure coefficient, K_a , of 0.31, calculated using Rankine Theory for cantilever-type walls. The designer may assume Soil Type 4 (BDG Section 3.6.1) for backfill material soil properties. The backfill properties are as follows: $\phi = 32$ degrees, $\gamma = 125$ pcf.

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 on wingwalls may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) taken from the Table 7-2 below:

Retaining Wall Height (feet)	h_{eq} (feet)	
	Distance from wall backface to edge of traffic	
	0.0 feet	1.0 feet or Further
5	5.0	2.0
10	3.5	2.0
≥ 20	2.0	2.0

Table 7-2 - Equivalent Height of Soil for Vehicular Loading on Retaining Walls Parallel to traffic

Slopes above the wingwalls should be constructed with riprap and not exceed 1.75H:1V.

7.3 Bearing Resistance

Concrete seals and spread footings shall be proportioned to provide stability against bearing capacity failure. Application of permanent and transient loads is specified in LRFD Article 11.5.5. The stress distribution may be assumed to be a triangular or trapezoidal distribution over the effective base as shown in LRFD Figure 11.6.3.2-2. The bearing resistance for any structure founded on competent, sound bedrock shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 8 ksf. This assumes a bearing resistance factor, ϕ_b , for spread footings on bedrock of 0.45, based on bearing resistance evaluation using semi-empirical methods. A factored bearing resistance of 20 ksf may be used and for preliminary footing sizing, and to control settlements when analyzing the service limit state load combination.

Bearing resistance for foundations on fill soils shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 5 ksf for wall system bases less than 8 feet wide and 6.5 ksf for bases from 10 to 12 feet wide. Based on presumptive bearing resistance values a factored bearing resistance of 6 ksf may be used to control settlement when analyzing the service limit state and for preliminary footing sizing.

See Appendix C – Calculations, for supporting documentation.

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

7.4 Precast Concrete Modular Gravity Retaining Wall

Precast Concrete Modular Gravity (PCMG) walls may be constructed on all four corners of the bridge to retain the roadway section and minimize impacts. In general, PCMG wingwalls should be used only at stream crossings where the flow velocities are low, and the potential for severe ice or wave action is low. PCMG walls should also only be used above the ordinary mean high water elevation (Q1.1). These walls shall be designed by a Professional Engineer subcontracted by the Contractor as a design-build item in accordance with Special Provision 635 which is included in Appendix D found at the end of this report. PCMG walls shall be designed considering a live load surcharge equal to a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) per LRFD Table 3.11.6.4-2.

Bearing resistance for PCMG walls founded on a leveling slab on fill sand shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 5 ksf for wall system bases less than 8 feet wide and 6.5 ksf for bases from 10 to 12 feet wide. The bearing resistance factor, ϕ_b , for spread footings on soil is 0.45. The stress distribution may be assumed to be a uniform distribution over the effective footing base as shown in LRFD Figure 11.6.3.2-1. Based on presumptive bearing resistance values a factored bearing resistance of 6 ksf may be used to control settlement when analyzing the service limit state and for preliminary footing sizing assuming a resistance factor of 1.0. See Appendix C – Calculations, for supporting documentation.

The bearing resistance for PCMG bottom unit of the PCMG wall shall be checked for the extreme limit state with a resistance factor of 1.0. The PCMG units shall be designed so that the nominal bearing resistance after the design scour event provides adequate resistance to support the factored strength limit state loads with strength limit state resistance factors. In general, spread footings at stream crossings should be founded a minimum of 2 feet below the calculated design scour depth. The overall stability of the wall system should be investigated at the Service I Load Combination with a resistance factor ϕ , of 0.65.

Failure by sliding shall be investigated by the wall designer-supplier. A sliding resistance factor, ϕ_r , of 0.90 shall be applied to the nominal sliding resistance of precast concrete wall segments founded on bedrock, concrete fill or granular borrow. For the lowest PCMG wall unit on bedding material the eccentricity of loading at the strength limit state, based on factored loads, shall not exceed one-fourth (1/4) of the footing dimensions in either direction (LRFD Article 10.6.3.3). Sliding computations for resistance to lateral loads shall assume a maximum frictional coefficient of $\tan 30^\circ$ at the foundation soil to soil infill interface and a maximum frictional coefficient of $0.8x(\tan 30^\circ)$ at the foundation soil to concrete module interface. Recommended values of sliding frictional coefficients are based on LRFD Articles 11.11.4.2 and 10.6.3.4, and Table 10.5.5.2.2-1.

The ordinary mean high water elevation shall be indicated on the retaining wall plans per the design requirements for hydrostatic conditions in Special Provision 635.

7.5 Frost Protection

Any foundation placed on granular subgrade soils should be designed with an appropriate embedment for frost protection. According to the Modberg Software by the US Army Cold Regions Research and Engineering Laboratory the site has an air design-freezing index of approximately 1195 F-degree days. In a granular soil with a water content of approximately 5%, this correlates to a frost depth of approximately 5.0 feet. Therefore, any foundations placed on granular soils should be founded a minimum of 5.0 feet below finished exterior grade for frost protection. See Appendix C- Calculations at the end of this report for supporting documentation.

It is anticipated that the foundation seals will be constructed directly on the prepared bedrock surface. For foundations on bedrock, heave due to frost is not a design issue and no requirements for minimum depth of frost embedment are necessary.

7.6 Scour and Riprap

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

- Average diameter of particle at 50 percent passing, $D_{50} = 0.6$ mm
- Average diameter of particle at 95 percent passing, $D_{95} = 10.9$ mm
- Soil Classification AASHTO Soil Type A-4, A-1-b or A-2-4

The grain size curves are included in Appendix B- Laboratory Data found at the end of this report.

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

Riprap conforming to Special Provisions 610 and 703 shall be placed at the toes of arch footing and wingwalls. Special Provisions 610 and 703 are provided in Appendix D – Special Provisions found at the end of this report. Stone riprap shall conform to item number 703.26 of the MaineDOT Special Provision 703 and shall be placed at a maximum slope of 1.75H:1V. The toe of the riprap section shall be constructed 1 foot below the streambed

elevation. The riprap section shall be underlain by a 1 foot thick layer of bedding material conforming to item number 703.19 of the Standard Specification and Class "1" Erosion Control Geotextile per Standard Details 610(02) through 610(04). Riprap shall be 3 feet thick.

7.7 Settlement

The grade of the existing bridge approaches will be lowered slightly in the replacement of the structure. Post-construction settlements are anticipated to be negligible.

7.8 Seismic Design Considerations

In conformance with LRFD Article 3.10.1, seismic analysis is not required for buried structures, except where they cross active faults. There are no known active faults in Maine, therefore seismic analysis is not required.

7.9 Construction Considerations

Construction of the arch concrete spread footings and seals will require soil excavation and removal of the existing twin pipe arches. Construction activities will require cofferdams to permit construction of the arch footings in the dry and earth support systems.

The nature, slope and degree of fracturing in the bedrock bearing surfaces will not be evident until the seal excavation is made. The bedrock surface shall be cleared of all loose fractured bedrock, loose decomposed bedrock and soil. The final bearing surface shall be solid. The bedrock surface slope shall be less than 4 horizontal to 1 vertical (4H:1V) or it shall be benched in level steps or excavated to be completely level. Anchoring, doweling or other means of improving sliding resistance may also be employed where the prepared bedrock surface is steeper than 4H:1V in any direction.

Excavation of highly sloped and loose bedrock material may be done using conventional excavation methods, but may require drilling and blasting techniques. Blasting should be conducted in accordance with Section 105.2.6 of the MaineDOT Standard Specifications. It is also recommended that the contractor conduct pre-and post-blast surveys, as well as blast vibration monitoring at nearby residences and bridge structures in accordance with industry standards at the time of the blast.

The final bedrock surface shall be approved by the Resident prior to placement of the footing/seal concrete.

In some locations the native soils may be saturated and significant water seepage may be encountered during construction. There may be localized sloughing and surface instability in some soil slopes. The Contractor should control groundwater, surface water infiltration and soil erosion during construction.

Using the excavated native soils as structural backfill should not be permitted. The native soils may only be used as common borrow in accordance with MaineDOT Standard Specifications 203 and 703.

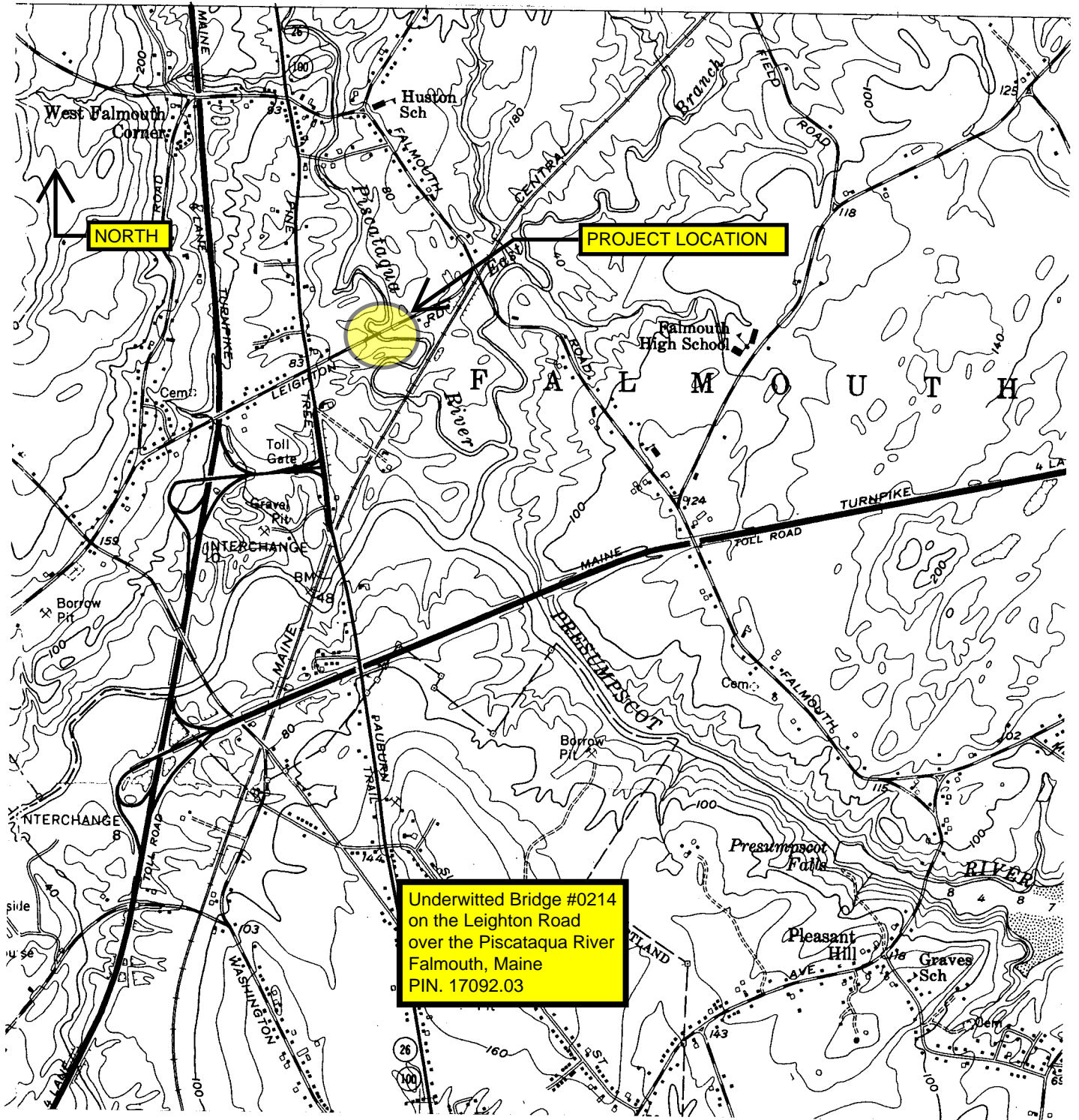
The Contractor will have to excavate the existing subbase and subgrade fill soils in the bridge approaches. These materials should not be used to re-base the new bridge approaches. Excavated subbase sand and gravel may be used as fill below subgrade level in fill areas provided all other requirements of MaineDOT Standard Specifications 203 and 703 are met.

8.0 CLOSURE

This report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed replacement of the Underwitted Road Bridge in Falmouth in accordance with generally accepted geotechnical and foundation engineering practices. No other intended use or warranty 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.

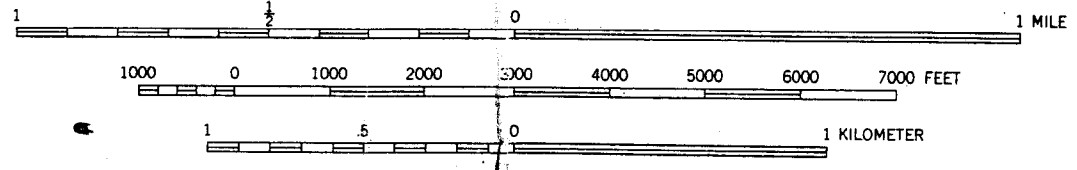
We also recommend that we be provided the opportunity for a general review of the final design and specifications in order that the earthwork and foundation recommendations may be properly interpreted and implemented in the design.

Sheets



PORTLAND WEST QUADRANGLE
MAINE - CUMBERLAND CO.
7.5 MINUTE SERIES (TOPOGRAPHIC)
NE/4 PORTLAND 15' QUADRANGLE

SCALE 1:24000 1" = 2000'

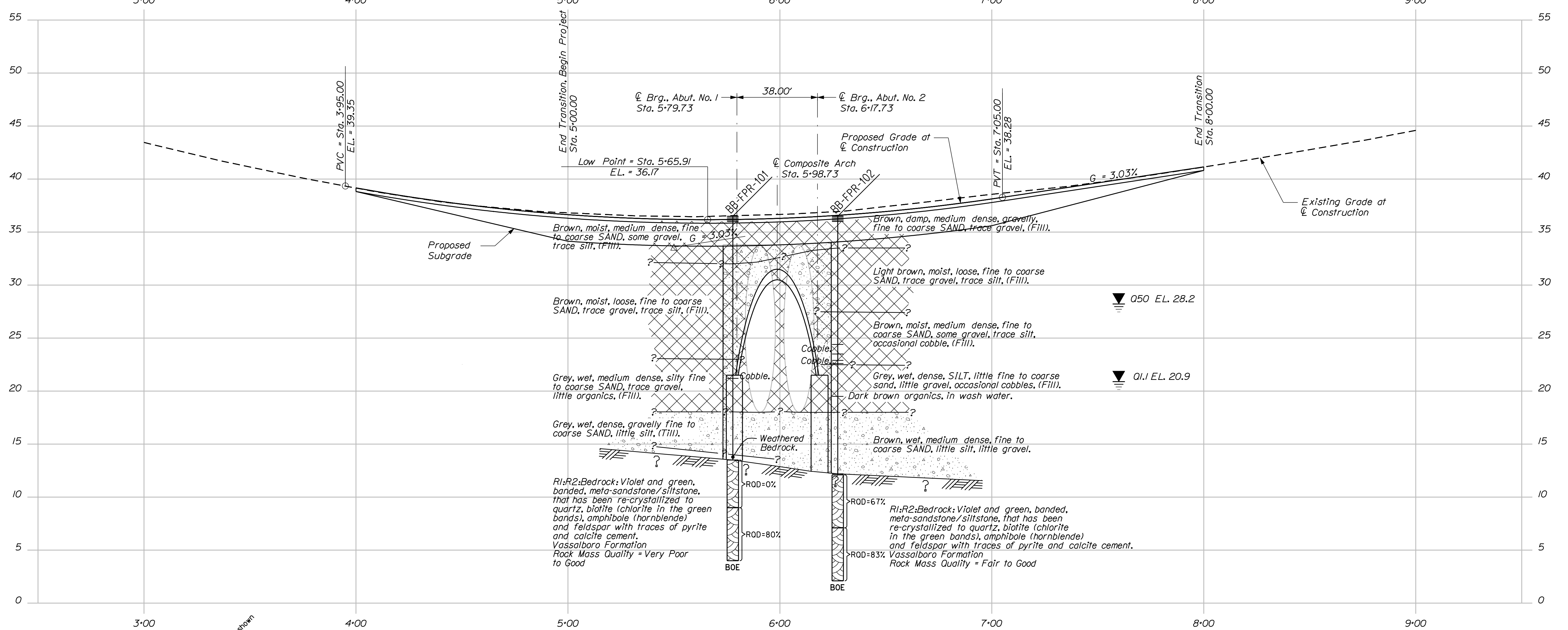
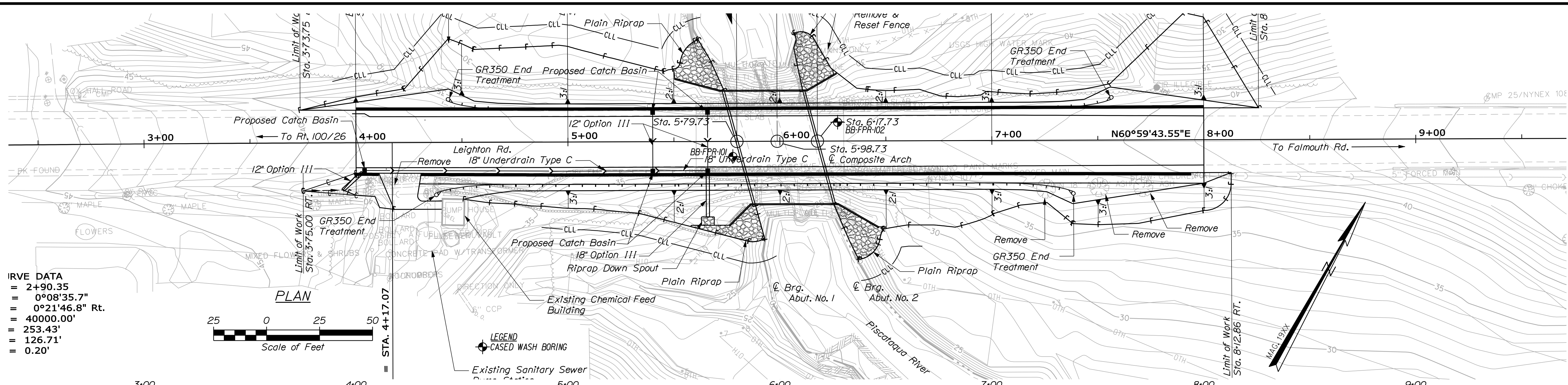


CONTOUR INTERVAL 20 FEET

Date: 1/20/2010

Username: terry.white

Filename: ... \GEOTECH\MSTA\006_BLP&SP1.dgn Division: GEOTECH



IRVE DATA
 = 2+90.35
 = 0°08'35.7"
 = 0°21'46.8" Rt.
 = 40000.00'
 = 253.43'
 = 126.71'
 = 0.20'

PLAN
 Scale of Feet
 25 0 25 50

LEGEND
 Boring No. 011501.11 shown
 Strata Interface
 Boring
 RQD - Rock Quality Designation for Rock Core Sample
 BOE - Bottom Of Exploration

PROFILE
 HORIZ 25 0 25 50
 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	
UNDERWITTED ROAD BRIDGE		AC-BR-1709(203)X	
PISCATAQUA RIVER		BRIDGE NO. 0214	
CUMBERLAND COUNTY		PIN 17092.03	
FALMOUTH		BRIDGE PLANS	
BORING LOCATION PLAN & INTERPRETIVE SUBSURFACE PROFILE		SHEET NUMBER	
2		OF 3	
PROJ. MANAGER	DATE	CHECKED	SIGNATURE
K. MAGUIRE	OCT 2009	T. WHITE	
DESIGNED	DATE	DRAWN	P.E. NUMBER
REVISIONS	DATE	BY	DATE
1			
2			
3			
4			
FIELD CHANGES			

Maine Department of Transportation		Project: Underwritten Bridge #0214 on the Leighton Road over the Piscataqua US CUSTOMARY UNITS		Location: Falmouth, Maine		Boring No.: BB-FPR-101	
Driller: MalmDOT		Elevation (ft.): 36.5		Auger ID/OD: 5" Solid Stem		Operator: Ciguere/Wright	
Logged By: B. Wilder		Rig Type: CME 45C		Hammer Wt./Fall: 140#/30"		Date Start/Finish: 7/29/09: 07:00-13:00	
Boring Location: 5477.8, T.1 Rt.		Casing ID/OD: HW		Water Level#: 16.9' bgs.		Hammer Efficiency Factor: 0.84	
Depth (ft.)	Sample No.	Pen./Rec. (ft.)	Sample Depth (ft.)	Shear (1/8 in. Strength or RCU (1.5)	Non-corroded	Moisture (%)	Grain Size Analysis
0			1.00 - 3.00	7/11/9/8	20	28	GS246310 A-1-b, SM-5M WC=3.4%
5			5.00 - 7.00	4/4/3/3	7	10	GS246312 A-3, SP-5M WC=4.9%
10			10.00 - 12.00	2/2/3/3	5	7	GS246313 A-3, SP WC=3.8%
15			15.50 - 17.50	10/5/6/7	11	15	GS246314 A-4, SM WC=21.2%
20			20.00 - 22.00	16/18/23/17	41	57	GS246315 A-1-b, SM-5M WC=9.7%
23			23.00 - 27.50	ROD = 0%			Weathered ROCK. Top of Intract Bedrock at Elev. 13.5'. Bedrock: Violet and green, banded, meta-sandstone/siltstone, that has been re-crystallized to quartz, biotite chlorite in the green bands, amphibole (hornblende) and feldspar with traces of pyrite and calcite cement. Rock Mass Quality = Very Poor R1: Core Times (min:sec) 23.0-24.0' (2:50) 24.0-25.0' (2:50) 25.0-26.0' (2:31) 26.0-27.0' (3:03) 27.0-27.5' (2:32) 98% Recovery Core blocked, no water return. Bedrock: Violet and green, banded, meta-sandstone/siltstone, that has been re-crystallized to quartz, biotite chlorite in the green bands, amphibole (hornblende) and feldspar with traces of pyrite and calcite cement. Rock Mass Quality = Good R2: Core Times (min:sec) 27.5-28.5' (3:20) 28.5-29.5' (3:15) 29.5-30.5' (3:25) 30.5-31.5' (3:35) 31.5-32.5' (3:20) 100% Recovery Bottom of Exploration at 32.50 feet below ground surface.

Maine Department of Transportation		Project: Underwritten Bridge #0214 on the Leighton Road over the Piscataqua US CUSTOMARY UNITS		Location: Falmouth, Maine		Boring No.: BB-FPR-102	
Driller: MalmDOT		Elevation (ft.): 36.5		Auger ID/OD: 5" Solid Stem		Operator: Ciguere/Wright	
Logged By: B. Wilder		Rig Type: CME 45C		Hammer Wt./Fall: 140#/30"		Date Start/Finish: 8/10/09: 09:00-15:00	
Boring Location: 6427.3, B.5 Lt.		Casing ID/OD: HW		Water Level#: 15.0' bgs.		Hammer Efficiency Factor: 0.84	
Depth (ft.)	Sample No.	Pen./Rec. (ft.)	Sample Depth (ft.)	Shear (1/8 in. Strength or RCU (1.5)	Non-corroded	Moisture (%)	Grain Size Analysis
0			1.00 - 3.00	7/7/4/5	11	15	GS246316 A-1-b, SM-5M WC=5.5%
5			5.00 - 7.00	2/3/2/4	5	7	GS246317 A-3, SP WC=6.0%
10			10.00 - 12.00	9/7/3/7	10	14	GS246318 A-1-b, SM-5M WC=8.2%
15			15.00 - 16.10	10/13/50(1.2")		57	GS246319 A-4, ML WC=18.5%
20			20.00 - 22.00	7/7/12/22	19	27	GS246320 A-2-b, SM WC=15.5%
25			24.40 - 25.40	ROD = 67%			960 blows for 0.3'. Top of Bedrock at Elev. 12.2'. Bedrock: Violet and green, banded, meta-sandstone/siltstone, that has been re-crystallized to quartz, biotite chlorite in the green bands, amphibole (hornblende) and feldspar with traces of pyrite and calcite cement. Rock Mass Quality = Fair R1: Core Times (min:sec) 24.4-25.4' (3:30) 24.4-25.4' (2:20) 26.4-27.4' (3:32) 27.4-28.4' (3:20) 28.4-29.4' (2:50) 98% Recovery Bedrock: Violet and green, banded, meta-sandstone/siltstone, that has been re-crystallized to quartz, biotite chlorite in the green bands, amphibole (hornblende) and feldspar with traces of pyrite and calcite cement. Rock Mass Quality = Good R2: Core Times (min:sec) 29.4-30.4' (3:38) 30.4-31.4' (2:20) 31.4-32.4' (3:32) 32.4-33.4' (2:20) 33.4-34.4' (2:30) 95% Recovery Bottom of Exploration at 34.40 feet below ground surface.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
AC-BR-1709(203)X

UNDERWRITTEN ROAD BRIDGE
PISCATAQUA RIVER
FALMOUTH CUMBERLAND COUNTY

BORING LOGS

BRIDGE NO. 0214 PIN 17092.03 BRIDGE PLANS

PROJ. MANAGER	DATE	BY	DATE
K. MAGUIRE	JAN 2010	T. WHITE	
CHECKED-REVIEWED	SIGNATURE		
DESIGNS DETAILED	P.E. NUMBER		
REVISIONS 1	DATE		
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SHEET NUMBER
3
OF 3

Appendix A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM				TERMS DESCRIBING DENSITY/CONSISTENCY																												
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	GW	Well-graded gravels, gravel-sand mixtures, little or no fines	<p>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.</p> <p style="text-align: center;">Modified Burmister System</p> <table border="1"> <thead> <tr> <th>Descriptive Term</th> <th>Portion of Total</th> </tr> </thead> <tbody> <tr> <td>trace</td> <td>0% - 10%</td> </tr> <tr> <td>little</td> <td>11% - 20%</td> </tr> <tr> <td>some</td> <td>21% - 35%</td> </tr> <tr> <td>adjective (e.g. sandy, clayey)</td> <td>36% - 50%</td> </tr> </tbody> </table> <table border="1"> <thead> <tr> <th>Density of Cohesionless Soils</th> <th>Standard Penetration Resistance N-Value (blows per foot)</th> </tr> </thead> <tbody> <tr> <td>Very loose</td> <td>0 - 4</td> </tr> <tr> <td>Loose</td> <td>5 - 10</td> </tr> <tr> <td>Medium Dense</td> <td>11 - 30</td> </tr> <tr> <td>Dense</td> <td>31 - 50</td> </tr> <tr> <td>Very Dense</td> <td>> 50</td> </tr> </tbody> </table>	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					
		Descriptive Term	Portion of Total																													
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Very Dense	> 50																															
(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.																														
SANDS (more than half of coarse fraction is smaller than No. 4 sieve size)	CLEAN SANDS	SW	Well-graded sands, gravelly sands, little or no fines																													
	(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																													
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.	<p>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.</p> <table border="1"> <thead> <tr> <th>Consistency of Cohesive soils</th> <th>SPT N-Value blows per foot</th> <th>Approximate Undrained Shear Strength (psf)</th> <th>Field Guidelines</th> </tr> </thead> <tbody> <tr> <td>Very Soft</td> <td>WOH, WOR, WOP, <2</td> <td>0 - 250</td> <td>Fist easily Penetrates</td> </tr> <tr> <td>Soft</td> <td>2 - 4</td> <td>250 - 500</td> <td>Thumb easily penetrates</td> </tr> <tr> <td>Medium Stiff</td> <td>5 - 8</td> <td>500 - 1000</td> <td>Thumb penetrates with moderate effort</td> </tr> <tr> <td>Stiff</td> <td>9 - 15</td> <td>1000 - 2000</td> <td>Indented by thumb with great effort</td> </tr> <tr> <td>Very Stiff</td> <td>16 - 30</td> <td>2000 - 4000</td> <td>Indented by thumb nail</td> </tr> <tr> <td>Hard</td> <td>>30</td> <td>over 4000</td> <td>Indented by thumb nail with difficulty</td> </tr> </tbody> </table>	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 thumb nail	Hard	>30	over 4000	Indented by thumb nail with difficulty
		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 thumb nail																													
Hard	>30	over 4000	Indented by thumb nail with difficulty																													
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.																														
<p>Desired Soil Observations: (in this order)</p> <p>Color (Munsell color chart)</p> <p>Moisture (dry, damp, moist, wet, saturated)</p> <p>Density/Consistency (from above right hand side)</p> <p>Name (sand, silty sand, clay, etc., including portions - trace, little, etc.)</p> <p>Gradation (well-graded, poorly-graded, uniform, etc.)</p> <p>Plasticity (non-plastic, slightly plastic, moderately plastic, highly plastic)</p> <p>Structure (layering, fractures, cracks, etc.)</p> <p>Bonding (well, moderately, loosely, etc., if applicable)</p> <p>Cementation (weak, moderate, or strong, if applicable, ASTM D 2488)</p> <p>Geologic Origin (till, marine clay, alluvium, etc.)</p> <p>Unified Soil Classification Designation</p> <p>Groundwater level</p>				<p>Rock Quality Designation (RQD):</p> <p>RQD = $\frac{\text{sum of the lengths of intact pieces of core}^* > 100 \text{ mm}}{\text{length of core advance}}$</p> <p>*Minimum NQ rock core (1.88 in. OD of core)</p> <p style="text-align: center;">Correlation of RQD to Rock Mass Quality</p> <table border="1"> <thead> <tr> <th>Rock Mass Quality</th> <th>RQD</th> </tr> </thead> <tbody> <tr> <td>Very Poor</td> <td><25%</td> </tr> <tr> <td>Poor</td> <td>26% - 50%</td> </tr> <tr> <td>Fair</td> <td>51% - 75%</td> </tr> <tr> <td>Good</td> <td>76% - 90%</td> </tr> <tr> <td>Excellent</td> <td>91% - 100%</td> </tr> </tbody> </table> <p>Desired Rock Observations: (in this order)</p> <p>Color (Munsell color chart)</p> <p>Texture (aphanitic, fine-grained, etc.)</p> <p>Lithology (igneous, sedimentary, metamorphic, etc.)</p> <p>Hardness (very hard, hard, mod. hard, etc.)</p> <p>Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.)</p> <p>Geologic discontinuities/jointing:</p> <ul style="list-style-type: none"> -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.) <p>Formation (Waterville, Ellsworth, Cape Elizabeth, etc.)</p> <p>RQD and correlation to rock mass quality (very poor, poor, etc.)</p> <p>ref: AASHTO Standard Specification for Highway Bridges</p> <p>17th Ed. Table 4.4.8.1.2A</p> <p>Recovery</p>		Rock Mass Quality	RQD	Very Poor	<25%	Poor	26% - 50%	Fair	51% - 75%	Good	76% - 90%	Excellent	91% - 100%															
Rock Mass Quality	RQD																															
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<p>Maine Department of Transportation</p> <p>Geotechnical Section</p> <p>Key to Soil and Rock Descriptions and Terms</p> <p>Field Identification Information</p>				<p>Sample Container Labeling Requirements:</p> <table border="1"> <tbody> <tr> <td>PIN</td> <td>Blow Counts</td> </tr> <tr> <td>Bridge Name / Town</td> <td>Sample Recovery</td> </tr> <tr> <td>Boring Number</td> <td>Date</td> </tr> <tr> <td>Sample Number</td> <td>Personnel Initials</td> </tr> <tr> <td>Sample Depth</td> <td></td> </tr> </tbody> </table>		PIN	Blow Counts	Bridge Name / Town	Sample Recovery	Boring Number	Date	Sample Number	Personnel Initials	Sample Depth																		
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Driller: MaineDOT	Elevation (ft.): 36.5	Auger ID/OD: 5" Solid Stem
Operator: Giguere/Wright	Datum: NAVD 88	Sampler: Standard Split Spoon
Logged By: B. Wilder	Rig Type: CME 45C	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 7/29/09; 07:00-13:00	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"
Boring Location: 5+77.8, 7.1 Rt.	Casing ID/OD: HW	Water Level*: 16.0' bgs.

Hammer Efficiency Factor: 0.84 **Hammer Type:** Automatic Hydraulic Rope & Cathead

Definitions: R = Rock Core Sample S_u = Insitu Field Vane Shear Strength (psf) S_{u(lab)} = Lab Vane Shear Strength (psf)
 D = Split Spoon Sample SSA = Solid Stem Auger T_v = Pocket Torvane Shear Strength (psf) WC = water content, percent
 MD = Unsuccessful Split Spoon Sample attempt HSA = Hollow Stem Auger q_p = Unconfined Compressive Strength (ksf)
 U = Thin Wall Tube Sample RC = Roller Cone N-uncorrected = Raw field SPT N-value
 MU = Unsuccessful Thin Wall Tube Sample attempt WOH = weight of 140lb. hammer Hammer Efficiency Factor = Annual Calibration Value
 V = Insitu Vane Shear Test, PP = Pocket Penetrometer WOR/C = weight of rods or casing N₆₀ = SPT N-uncorrected corrected for hammer efficiency
 MV = Unsuccessful Insitu Vane Shear Test attempt WO1P = Weight of one person N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected
 LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index
 G = Grain Size Analysis C = Consolidation Test

Depth (ft.)	Sample Information							Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows				
0								36.00	Pavement		
	1D	24/18	1.00 - 3.00	7/11/9/8	20	28			Brown, moist, medium dense, fine to coarse SAND, some gravel, trace silt, (Fill).	G#246311 A-1-b, SW-SM WC=3.4%	
5								32.00	Brown, moist, loose, fine to coarse SAND, trace gravel, trace silt, (Fill).	G#246312 A-3, SP-SM WC=4.9%	
	2D	24/12	5.00 - 7.00	4/4/3/3	7	10					
10									Brown, moist, loose, fine to coarse SAND, trace gravel, trace silt, (Fill).	G#246313 A-3, SP WC=3.8%	
	3D	24/6	10.00 - 12.00	2/2/3/3	5	7					
15								23.00	Cobble from 15.0-15.3' bgs. Grey, wet, medium dense, silty fine to coarse SAND, trace gravel, little organics, (Fill).	G#246314 A-4, SM WC=21.2%	
	4D	24/15	15.50 - 17.50	10/5/6/7	11	15	52				
							65				
							107				
							199				
							148				
20								18.00	Grey, wet, very dense, gravelly fine to coarse SAND, little silt, (Till).	G#246315 A-1-b, SW-SM WC=9.7%	
	5D	24/18	20.00 - 22.00	16/18/23/17	41	57	81				
							163				
							133				
	R1	54/52	23.00 - 27.50	RQD = 0%				14.00	Weathered ROCK.		
								13.50	Top of Intact Bedrock at Elev. 13.5'. Bedrock: Violet and green, banded, meta-sandstone/siltstone, that has been re-crystallized to quartz, biotite (chlorite in the green bands),		

Remarks:
300-400 lbs down pressure on Core Barrel.

Driller: MaineDOT	Elevation (ft.): 36.5	Auger ID/OD: 5" Solid Stem
Operator: Giguere/Wright	Datum: NAVD 88	Sampler: Standard Split Spoon
Logged By: B. Wilder	Rig Type: CME 45C	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 7/29/09; 07:00-13:00	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"
Boring Location: 5+77.8, 7.1 Rt.	Casing ID/OD: HW	Water Level*: 16.0' bgs.

Hammer Efficiency Factor: 0.84 **Hammer Type:** Automatic Hydraulic Rope & Cathead

Definitions: R = Rock Core Sample S_u = Insitu Field Vane Shear Strength (psf) S_{u(lab)} = Lab Vane Shear Strength (psf)
 D = Split Spoon Sample SSA = Solid Stem Auger T_v = Pocket Torvane Shear Strength (psf) WC = water content, percent
 MD = Unsuccessful Split Spoon Sample attempt HSA = Hollow Stem Auger q_p = Unconfined Compressive Strength (ksf) LL = Liquid Limit
 U = Thin Wall Tube Sample RC = Roller Cone N-uncorrected = Raw field SPT N-value PL = Plastic Limit
 MU = Unsuccessful Thin Wall Tube Sample attempt WOH = weight of 140lb. hammer Hammer Efficiency Factor = Annual Calibration Value PI = Plasticity Index
 V = Insitu Vane Shear Test, PP = Pocket Penetrometer N₆₀ = SPT N-uncorrected corrected for hammer efficiency G = Grain Size Analysis
 MV = Unsuccessful Insitu Vane Shear Test attempt WO1P = Weight of one person N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected C = Consolidation Test

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
25									4.00		amphibole (hornblende) and feldspar with traces of pyrite and calcite cement. Rock Mass Quality = Very Poor R1: Core Times (min:sec) 23.0-24.0' (2:30) 24.0-25.0' (2:50) 25.0-26.0' (2:31) 26.0-27.0' (3:03) 27.0-27.5' (2:32) 96% Recovery Core Blocked, no water return. Bedrock: Violet and green, banded, meta-sandstone/siltstone, that has been re-crystallized to quartz, biotite (chlorite in the green bands), amphibole (hornblende) and feldspar with traces of pyrite and calcite cement. Rock Mass Quality = Good R2: Core Times (min:sec) 27.5-28.5' (3:20) 28.5-29.5' (3:15) 29.5-30.5' (3:25) 30.5-31.5' (3:35) 31.5-32.5' (3:20) 100% Recovery	
	R2	60/60	27.50 - 32.50	RQD = 80%								
30												
35												
40												
45												
50												

Remarks:
300-400 lbs down pressure on Core Barrel.

Driller: MaineDOT	Elevation (ft.): 36.5	Auger ID/OD: 5" Solid Stem
Operator: Giguere/Giles/Wright	Datum: NAVD 88	Sampler: Standard Split Spoon
Logged By: B. Wilder	Rig Type: CME 45C	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 8/10/09; 09:00-15:00	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"
Boring Location: 6+27.3, 8.9 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 R = Rock Core Sample S_u = Insitu Field Vane Shear Strength (psf) S_{u(lab)} = Lab Vane Shear Strength (psf)
MD = Unsuccessful Split Spoon Sample attempt SSA = Solid Stem Auger T_v = Pocket Torvane Shear Strength (psf) WC = water content, percent
U = Thin Wall Tube Sample HSA = Hollow Stem Auger q_p = Unconfined Compressive Strength (ksf) LL = Liquid Limit
MU = Unsuccessful Thin Wall Tube Sample attempt RC = Roller Cone N-uncorrected = Raw field SPT N-value PL = Plastic Limit
V = Insitu Vane Shear Test, PP = Pocket Penetrometer WOH = weight of 140lb. hammer Hammer Efficiency Factor = Annual Calibration Value PI = Plasticity Index
MV = Unsuccessful Insitu Vane Shear Test attempt WOR/C = weight of rods or casing N₆₀ = SPT N-uncorrected corrected for hammer efficiency G = Grain Size Analysis
WO1P = Weight of one person N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected C = Consolidation Test

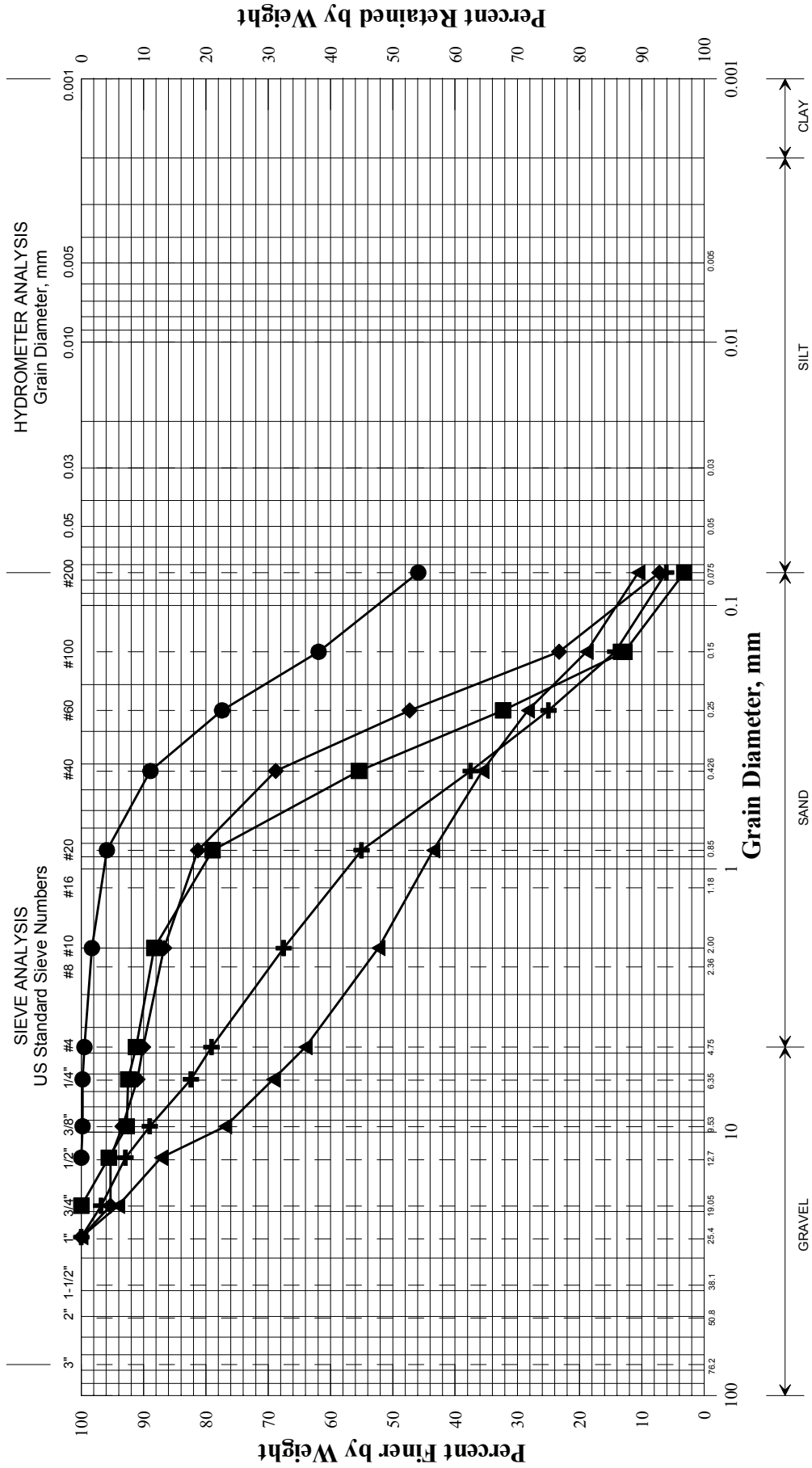
Depth (ft.)	Sample Information							Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.		
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows						
0								36.00	Pavement	Brown, damp, medium dense, gravelly fine to coarse SAND, trace silt, (Fill).	G#246316 A-1-b, SW-SM WC=3.5%		
	1D	24/19	1.00 - 3.00	7/7/4/5	11	15		33.50					
5									Light brown, moist, loose, fine to coarse SAND, trace gravel, trace silt, (Fill).	G#246317 A-3, SP WC=6.0%			
	2D	24/14	5.00 - 7.00	2/3/2/4	5	7		27.50					
10									Brown, moist, medium dense, fine to coarse SAND, some gravel, trace silt, occasional cobbles, (Fill). Cobble from 12.1-13.0' bgs. Cobble from 13.6-13.9' bgs.	G#246318 A-1-b, SW-SM WC=8.2%			
	3D	24/15	10.00 - 12.00	9/7/3/7	10	14		22.50					
15									Grey, wet, hard, SILT, little fine to coarse sand, little gravel, occasional cobbles, (Fill). Dark brown, organics in wash from 17.0 18.5' bgs.	G#246319 A-4, ML WC=18.5%			
	4D	13.2/12.2	15.00 - 16.10	10/13/50(1.2")	---			18.00					
20									Brown, wet, medium dense, fine to coarse SAND, little silt, little gravel. Roller Coned ahead to 24.3' bgs.	G#246320 A-2-4, SM WC=15.5%			
	5D	24/18	20.00 - 22.00	7/7/12/22	19	27		36					
								41					
								46					
								73					
25									a60 blows for 0.3'.				
	R1	60/59	24.40 - 29.40	RQD = 67%				12.20					

Remarks:
500-600 lbs down pressure on Core Barrel.

Appendix B

Laboratory Data

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE

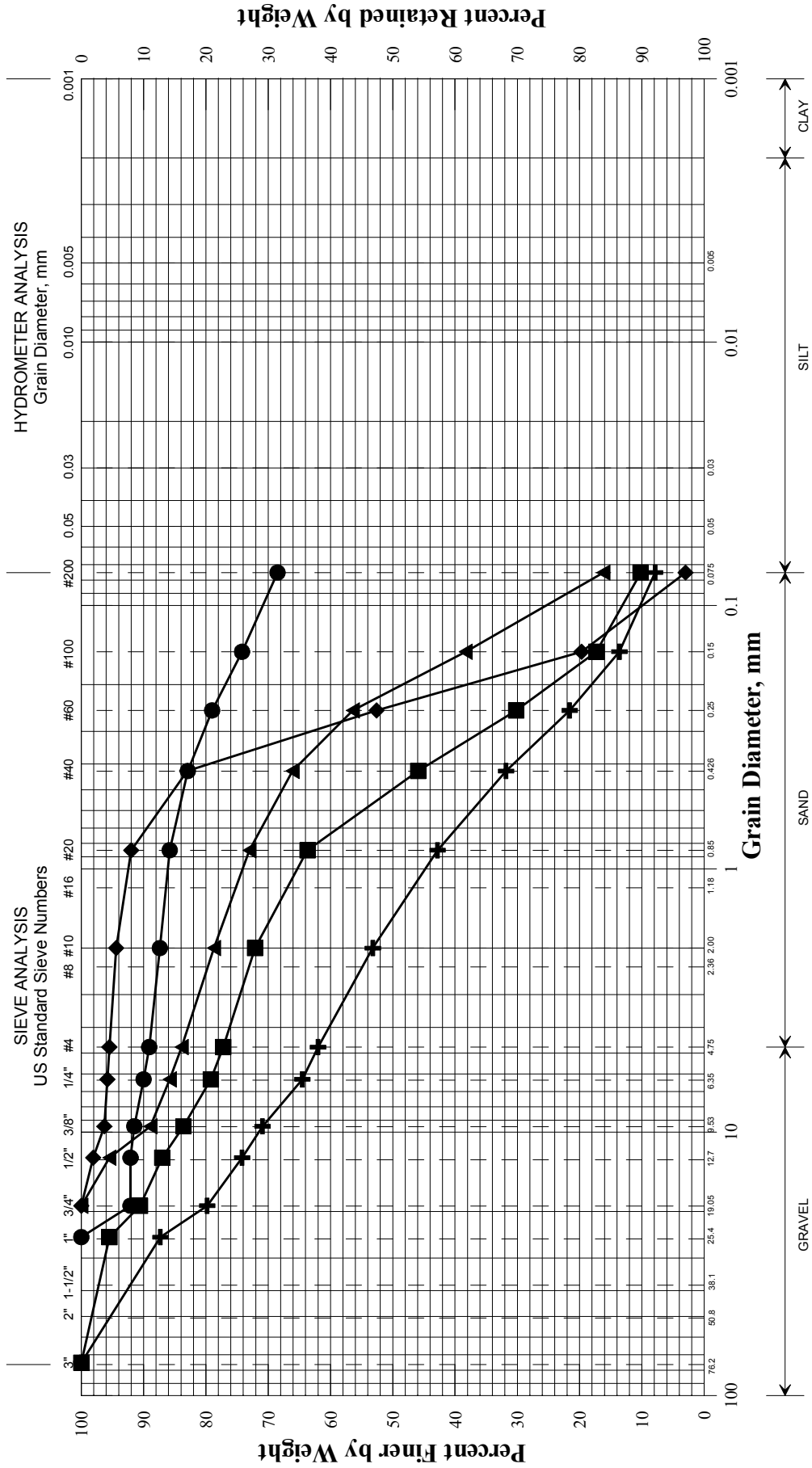


UNIFIED CLASSIFICATION

Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
+	5+77.8	7.1 RT	1.0-3.0	SAND, some gravel, trace silt.	3.4			
◆	5+77.8	7.1 RT	5.0-7.0	SAND, trace gravel, trace silt.	4.9			
■	5+77.8	7.1 RT	10.0-12.0	SAND, trace gravel, trace silt.	3.8			
●	5+77.8	7.1 RT	15.5-17.5	Silty SAND, trace gravel.	21.2			
▲	5+77.8	7.1 RT	20.0-22.0	Gravelly SAND, little silt.	9.7			
×								

PIN	017092.03
Town	Falmouth
Reported by/Date	WHITE, TERRY A 10/6/2009

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



UNIFIED CLASSIFICATION

Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
+	6+27.3	8.9 LT	1.0-3.0	Gravelly SAND, trace silt.	3.5			
◆	6+27.3	8.9 LT	5.0-7.0	SAND, trace gravel, trace silt.	6.0			
■	6+27.3	8.9 LT	10.0-12.0	SAND, some gravel, trace silt.	8.2			
●	6+27.3	8.9 LT	15.0-16.1	SILT, little sand, little gravel.	18.5			
▲	6+27.3	8.9 LT	20.0-22.0	SAND, little gravel, little silt.	15.5			
×								

PIN	017092.03
Town	Falmouth
Reported by/Date	WHITE, TERRY A 10/6/2009

Appendix C

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:
 Falmouth, Maine
 DFI = 1250 degree-days

From the lab testing: soils are coarse grained with a water content = ~5%

From Table 5-1 MaineDOT BDG for Design Freezing Index of 1250 and wc = 5%
 Frost Penetration = 79.0 inches (by interpolation)

$$\text{Frost_depth} := 79.0\text{in} \quad \text{Frost_depth} = 6.6 \cdot \text{ft}$$

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 Portland

ModBerg Results

Project Location: Portland Wsfo Airport, Maine

Air Design Freezing Index = 1195 F-days
 N-Factor = 0.80
 Surface Design Freezing Index = 956 F-days
 Mean Annual Temperature = 45.5 deg F
 Design Length of Freezing Season = 118 days

Layer #	Type	t	w%	d	Cf	Cu	Kf	Ku	L
1	Coarse	58.6	5.0	125.0	24	28	1.2	1.3	900

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 = 4.88 ft = 58.6 in.

$$\text{Frost_depth}_{\text{modberg}} := 58.6 \cdot \text{in}$$

$$\text{Frost_depth}_{\text{modberg}} = 4.9\text{ft}$$

Use Frost Depth = 5.0 feet for design

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 Sandstone which is "very poor" to "good" in quality.
RQD = 0 to 83%

Reference: AASHTO LRFD Bridge Design Specifications 4th 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 0% to 83%

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

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 Sandstone which is "very poor" to "good" in quality.
RQD = 0 to 83%

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, RMR method is not required.

Reference: Foundation Analysis and Design by JE Bowles Fifth Edition

Section 4-16 pg 277 Bearing Capacity of Rock

Assume: $\phi := 45 \cdot \text{deg}$ internal friction angle rock
 $c_r := 0 \cdot \text{psi}$ cohesion (rock)

Bearing Capacity factors by Stagg and Zienkiewicz 1968

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

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

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

Terzaghi Shape factors from Table 4-1 pg 220

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

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

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

$$B := \begin{pmatrix} 3 \\ 4 \\ 5 \\ 6 \end{pmatrix} \cdot \text{ft} \quad \text{Look at several footing widths}$$

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

$$q_{ult} = \begin{pmatrix} 49 \\ 66 \\ 82 \\ 99 \end{pmatrix} \cdot \text{ksf}$$

Reduce ultimate bearing based on average RQD = 60%

$$q_{\text{reduced}} := q_{\text{ult}} \cdot (0.6)^2$$
$$q_{\text{reduced}} = \begin{pmatrix} 18 \\ 24 \\ 30 \\ 35 \end{pmatrix} \cdot \text{ksf}$$

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

$$q_{\text{factored}} := q_{\text{reduced}} \cdot 0.45$$
$$q_{\text{factored}} = \begin{pmatrix} 8 \\ 11 \\ 13 \\ 16 \end{pmatrix} \cdot \text{ksf} \quad B = \begin{pmatrix} 3 \\ 4 \\ 5 \\ 6 \end{pmatrix} \text{ ft}$$

At the Strength Limit State:

Recommend a limiting factored bearing resistance of 8 ksf

Bearing Resistance - Native 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 4th 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 7 to 28 - Soils are loose to medium dense

Consistency In Place: Medium dense

Bearing Resistance: Ordinary Range (ksf) 4 to 8

Recommended Value of Use: 6 ksf

$$\text{tsf} := g \cdot \left(\frac{\text{ton}}{\text{ft}^2} \right)$$

Recommended Value: $6 \cdot \text{ksf} = 3 \cdot \text{tsf}$

Therefore: $q_{\text{nom}} := 3 \cdot \text{tsf}$

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

$$q_{\text{factored_bc}} := 3 \cdot \text{tsf} \quad \text{or} \quad q_{\text{factored_bc}} = 6 \cdot \text{ksf}$$

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 native soils

Reference: **Foundation Engineering and Design** by JE Bowles Fifth Edition

Assumptions:

1. Footings will be embedded 5.0 feet for frost protection. $D_f := 5.0 \cdot \text{ft}$
2. Assumed parameters for fill soils: (Ref: Bowles 5th Ed Table 3-4)
 - Saturated unit weight: $\gamma_s := 125 \cdot \text{pcf}$
 - Dry unit weight: $\gamma_d := 120 \cdot \text{pcf}$
 - Internal friction angle: $\phi_{\text{ns}} := 32 \cdot \text{deg}$
 - Undrained shear strength: $c_{\text{ns}} := 0 \cdot \text{psf}$
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: $D_w := 15 \cdot \text{ft}$ Based on boring logs

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

Look at several footing widths

$$B := \begin{pmatrix} 5 \\ 8 \\ 10 \\ 12 \\ 15 \end{pmatrix} \cdot \text{ft}$$

Terzaghi Shape factors from Table 4-1

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 $\phi=32$ 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)

$q := D_f \cdot (\gamma_s - \gamma_w)$ $q = 0.1565 \cdot \text{tsf}$

$q_{\text{nominal}} := c_{\text{ns}} \cdot N_c \cdot s_c + q \cdot N_q + 0.5(\gamma_s - \gamma_w)B \cdot N_\gamma \cdot s_\gamma$

$$q_{\text{nominal}} = \begin{pmatrix} 5.4 \\ 6.4 \\ 7.1 \\ 7.8 \\ 8.8 \end{pmatrix} \cdot \text{tsf}$$

Resistance Factor:

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

$q_{\text{factored}} := q_{\text{nominal}} \cdot \phi_b$

$$q_{\text{factored}} = \begin{pmatrix} 2.4 \\ 2.9 \\ 3.2 \\ 3.5 \\ 4 \end{pmatrix} \cdot \text{tsf}$$

Based on these footing widths

$$q_{\text{factored}} = \begin{pmatrix} 4.8 \\ 5.7 \\ 6.4 \\ 7 \\ 7.9 \end{pmatrix} \cdot \text{ksf}$$

$$B := \begin{pmatrix} 5 \\ 8 \\ 10 \\ 12 \\ 15 \end{pmatrix} \cdot \text{ft}$$

At Strength Limit State:

Recommend a limiting factored bearing resistance of 5 ksf for walls less than 8 feet wide.
 Recommend a limiting factored bearing resistance of 6.5 ksf for walls between 10 and 12 feet wide.

Earth Pressure:

At Rest Earth Pressure:

Reference: Das Principles of Foundation Engineering Fourth Edition Equation 6.3 pg 336

$$\phi_{\text{type4}} := 32 \cdot \text{deg} \quad K_o := 1 - \sin(\phi_{\text{type4}}) \quad K_o = 0.47$$

Active Earth Pressures:

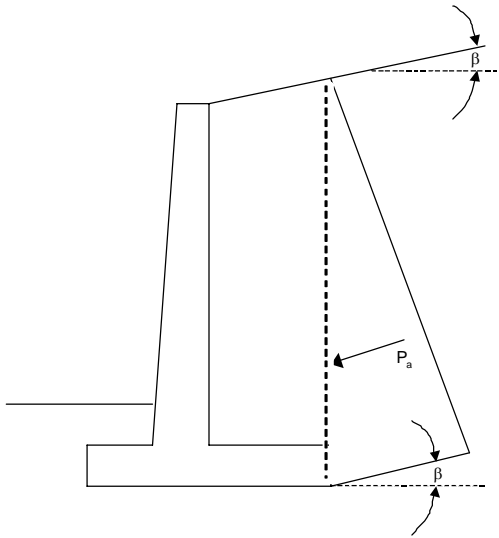
Soil Type 4 Properties from MaineDOT Bridge Design Guide (BDG)

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

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

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

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 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:

$$K_{a_rankine} := \tan\left(45 \cdot \text{deg} - \frac{\phi_{\text{type4}}}{2}\right)^2 \quad K_{a_rankine} = 0.31$$

Appendix D

Special Provisions

SPECIAL PROVISION
SECTION 635
PREFABRICATED CONCRETE MODULAR GRAVITY WALL

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

635.01 Description. This work shall consist of the construction of a prefabricated modular reinforced concrete gravity wall in accordance with these specifications and in reasonably close conformance with the lines and grades shown on the plans, or established by the Resident.

Included in the scope of the Prefabricated Concrete Modular Gravity Wall construction are: all grading necessary for wall construction, excavation, compaction of the wall foundation, backfill, construction of leveling pads, placement of geotextile, segmental unit erection, and all incidentals necessary to complete the work.

The Prefabricated Concrete Modular Gravity Wall design shall follow the general dimensions of the wall envelope shown in the contract plans. The top of the leveling pad shall be located at or below the theoretical leveling pad elevation. The minimum wall embedment shall be at or below the elevation shown on the plans. The top of the face panels shall be at or above the top of the panel elevation shown on the plans.

The Contractor shall require the design-supplier to supply 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. The work done by this representative is incidental.

635.02 Materials. Materials shall meet the requirements of the following subsections of Division 700 - Materials:

Gravel Borrow	703.20
Preformed Expansion Joint Material	705.01
Reinforcing Steel	709.01
Structural Pre-cast Concrete Units	712.061
Drainage Geotextile	722.02

The Contractor is cautioned that all of the materials listed are not required for every Prefabricated Concrete Modular Gravity Wall. The Contractor shall furnish the Resident a Certificate of Compliance certifying that the applicable materials comply with this section of the specifications. Materials shall meet the following additional requirements:

Concrete Units:

Tolerances. In addition to meeting the requirements of 712.061, all prefabricated units shall be manufactured with the following tolerances. All units not meeting the listed tolerances will be rejected.

1. All dimensions shall be within (edge to edge of concrete) $\pm 3/16$ inch.

2. Squareness. The length differences between the two diagonals shall not exceed 5/16 inch.
3. Surface Tolerances. For steel formed surfaces, and other formed surface, any surface defects in excess of 0.08 inch in 4 feet will be rejected. For textured surfaces, any surface defects in excess of 5/16 inch in 5 feet shall be rejected.

Joint Filler. (where applicable) Joints shall be filled with material approved by the Resident and supplied by the approved Prefabricated Concrete Modular Gravity Wall supplier. 4 inch wide, by 0.5 inch thick preformed expansion joint filler shall be placed in all horizontal joints between facing units. In all vertical joints, a space of 0.25 inch shall be provided. All Preformed Expansion Joint Material shall meet the requirements of subsection 502.03.

Woven Drainage Geotextile. Woven drainage geotextile 12 inches wide shall be bonded with an approved adhesive compound to the back face, covering all joints between units, including joints abutting concrete structures. Geotextile seam laps shall be 6 inches minimum. The fabric shall be secured to the concrete with an adhesive satisfactory to the Resident. Dimensions may be modified per the wall supplier's recommendations, with written approval of the Resident.

Concrete Shear Keys. (where applicable) Shear keys shall have a thickness at least equal to the pre-cast concrete stem.

Concrete Leveling Pad. Cast-in-place concrete shall be Fill Concrete conforming to the requirements of Section 502 Structural Concrete. The horizontal tolerance on the surface of the pad shall be 0.25 inch in 10 feet. Dimensions may be modified per the wall supplier's recommendations, with written approval of the Resident.

Backfill and Bedding Material. Bedding and backfill material placed behind and within the reinforced concrete modules shall be gravel borrow conforming to the requirements of Subsection 703.20. The backfill materials shall conform to the following additional requirements: the plasticity index (PI) as determined by AASHTO T90 shall not exceed 6. Compliance with the gradation and plasticity requirements shall be the responsibility of the Contractor, who shall furnish a copy of the backfill test results prior to construction.

The backfilling of the interior of the wall units and behind the wall shall progress simultaneously. The material shall be placed in layers not over 8 inches in depth, loose measure, and thoroughly compacted by mechanical or vibratory compactors. Puddling for compaction will not be allowed.

Materials Certificate Letter. The Contractor, or the supplier as his 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 his supplier necessary to assure contract compliance shall also be furnished to the Resident. Acceptance will be based upon the materials Certificate Letter, accompanying test reports, and visual inspection by the Resident.

635.03 Design Requirements. The Prefabricated Concrete Modular Gravity Wall shall be designed and sealed by a licensed Professional Engineer registered in accordance with the laws

of the State of Maine. The design to be performed by the wall system supplier shall be in accordance with AASHTO LRFD Bridge Design Specifications, current edition, except as required herein. Design shall consider Strength and Extreme Limit States. Thirty days prior to beginning construction of the wall, the design computations shall be submitted to the Resident for review by the Department. Design calculations that consist of computer 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:

A. Stability Analysis:

1. Overturning: Location of the resultant of the reaction forces shall be within the middle one-half of the base width.
2. Sliding: $R_R \geq \gamma_{p(\max)} \cdot (EH + ES)$
Where: R_R = Factored Sliding Resistance
 $\gamma_{p(\max)}$ = Maximum Load Factor
EH = Horizontal Earth Pressure
ES = Earth Surcharge (as applicable)
4. Bearing Pressure: $q_R \geq$ Factored Bearing Pressure
Where: q_R = Factored Bearing Resistance, as shown on the plans
Factored Bearing Pressure = Determined considering the applicable loads and load factors which result in the maximum calculated bearing pressure.
5. Pullout Resistance: Pullout resistance shall be determined using nominal resistances and forces. The ratio of the sum of the nominal resistances to the sum of the nominal forces shall be greater than, or equal to, 1.5.

Live load surcharge on PCMG walls shall be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) taken from LRFD Table 3.11.6.4-2 with consideration for the distance from the wall pressure surface to the edge of traffic. Traffic impact loads transmitted to the wall through guardrail posts shall be calculated and applied in compliance with LRFD Section 11, where Article 11.10.10.2 is modified such that the upper 3.5 feet of concrete modular units shall be designed for an additional horizontal load of γP_{HI} , where $\gamma P_{HI} = 300$ lbs per linear foot of wall.

- B. Backfill and Wall Unit Soil Parameters. For overturning and sliding stability calculations, earth pressure shall be assumed acting on a vertical plane rising from the back of the lowest wall stem. For overturning, the unit weight of the backfill within the wall units shall be limited to 96 pcf. For sliding analyses, the unit weight of the backfill within the wall units can be assumed to be 120 pcf. Both analyses may assume a friction angle of 34 degrees for backfill within the wall units.

These unit weights and friction angles are based on a wall unit backfill meeting the requirements for select backfill in this specification. Backfill behind the wall units shall be assumed to have a unit weight of 120 pcf and a friction angle of 30 degrees. The friction angle of the foundation soils shall be assumed to be 30 degrees unless otherwise noted on the plans.

- C. Internal Stability. Internal stability of the wall shall be demonstrated using accepted methods, such as Elias' Method, 1991. Shear keys shall not contribute to pullout resistance. Soil-to-soil frictional component along stem shall not contribute to pullout resistance. The failure plane used to determine pullout resistance shall be found by the Rankine theory only for vertical walls with level backfills. When walls are battered or with backslopes > 0 degrees are considered, the angle of the failure plane shall be per Jumikus Method. For computation of pullout force, the width of the backface of each unit shall be no greater than 4.5 feet. A unit weight of the soil inside the units shall be assumed no greater than 120 pcf when computing pullout. Coulomb theory may be used.
- D. External loads which affect the internal stability such as those applied through piling, bridge footings, traffic, slope surcharge, hydrostatic and seismic loads shall be accounted for in the design.
- E. The maximum calculated factored bearing pressure under the Prefabricated Concrete Modular Gravity block wall shall be clearly indicated on the design drawings.
- F. Stability During Construction. Stability during construction shall be considered during design, and shall meet the requirements of the AASHTO LRFD Bridge Design Specifications, Extreme Limit State.
- G. Hydrostatic forces. Unless specified otherwise, when a design high water surface is shown on the plans at the face of the wall, the design stresses calculated from that elevation to the bottom of wall must include a 3 feet minimum differential head of saturated backfill. In addition, the buoyant weight of saturated soil shall be used in the calculation of pullout resistance.
- H. Design Life. The wall design life shall be a minimum of 75 years.
- I. Not more than two vertically consecutive units shall have the same stem length, or the same unit depth. Walls with units with extended height curbs shall be designed for the added earth pressure. A separate computation for pullout of each unit with extended height curbs, or extended height coping, shall be prepared and submitted in the design package described above.

635.04 Submittals. The Contractor shall supply wall design computations, wall details, dimensions, quantities, and cross sections necessary to construct the wall. Thirty (30) days prior to beginning construction of the wall, the design computations and wall details shall be submitted to the Resident for review. The fully detailed plans shall be prepared in conformance with Subsection 105.7 of the Standard Specifications 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 designation as to the type of prefabricated module, the distance along the face of the wall to where changes in length of the units occur, the location of the original and final ground line.

- B. All details, including reinforcing bar bending details, shall be provided. Bar bending details shall be in accordance with Department standards.
- C. 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.
- D. All prefabricated modules shall be detailed. The details shall show all dimensions necessary to construct the element, and all reinforcing steel in the element.
- E. The wall plans shall be prepared and stamped by a Professional Engineer. Four sets of design drawings and detail design computations shall be submitted to the Resident.
- F. Four weeks 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.

635.05 Construction Requirements

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.

Foundation. The area upon which the modular 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 module. Prior to wall and leveling pad construction, this foundation material shall be compacted to at least 95 percent of maximum laboratory dry density, determined using AASHTO T180, Method C or D. Frozen soils and soils unsuitable or incapable of sustaining the required compaction, shall be removed and replaced.

A concrete leveling pad shall be constructed as indicated on the plans. 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 Resident. Allowable elevation tolerances are +0.01 feet and -0.02 feet from the design elevations. 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 24 hours curing time of the concrete leveling pad.

Method and Equipment. Prior to erection of the Prefabricated Concrete Modular Gravity 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 pre-cast units that are damaged due to handling will be replaced at the Contractor's expense.

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 Department. Vertical and horizontal joint fillers shall be installed as shown on the plans.

The maximum offset in any unit joint shall be 3/4 inch. The overall vertical tolerance of the wall, plumb from top to bottom, shall not exceed 1/2 inch per 10 feet of wall height. The prefabricated wall units shall be installed to a tolerance of plus or minus 3/4 inch in 10 feet in vertical alignment and horizontal alignment.

Select 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 Subsection 203.12 except that the minimum required compaction shall be 92 percent of maximum density as determined by AASHTO T180 Method C or D. Backfill compaction shall be accomplished without disturbance or displacement of the 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 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 T180, 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.

635.06 Method of Measurement. Prefabricated Concrete Modular Gravity Wall will be measured by the square meter of front surface not to exceed the dimensions shown on the contract plans or authorized by the Resident. Vertical and horizontal dimensions will be from the edges of the facing units. No field measurements for computations will be made unless the Resident specifies, in writing, a change in the limits indicated on the plans.

635.07 Basis of Payment. The accepted quantity of Prefabricated Concrete Modular Gravity Retaining Wall will be paid for at the contract unit price per square meter complete in place. Payment shall be full compensation for furnishing all labor, equipment and materials including excavation, foundation material, backfill material, pre-cast concrete units hardware, joint fillers, woven drainage geotextile, cast-in-place coping or traffic barrier 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 Prefabricated Concrete Modular Gravity Wall.

There will be no allowance for excavating and backfilling for the Prefabricated Concrete Modular Gravity Wall beyond the limits shown on the approved submitted plans, except for excavation required to remove unsuitable subsoil in preparation for the foundation, as approved by the Resident. Payment for excavating unsuitable material shall be full compensation for all costs of pumping, drainage, sheeting, bracing and incidentals for proper execution of the work.

Payment will be made under:

<u>Pay Item</u>	<u>Pay Unit</u>
635.14 Prefabricated Concrete Modular Gravity Wall	Square Foot

SPECIAL PROVISION
SECTION 610
STONE FILL, RIPRAP, STONE BLANKET,
AND STONE DITCH PROTECTION

Add the following paragraph to Section 610.02:

Materials shall meet the requirements of the following Sections of Special Provision 703:

Stone Fill	703.25
Plain and Hand Laid Riprap	703.26
Stone Blanket	703.27
Heavy Riprap	703.28
Definitions	703.32

Add the following paragraph to Section 610.032.a.

Stone fill and stone blanket shall be placed on the slope in a well-knit, compact and uniform layer. The surface stones shall be chinked with smaller stone from the same source.

Add the following paragraph to Section 610.032.b:

Riprap shall be placed on the slope in a well-knit, compact and uniform layer. The surface stones shall be chinked with smaller stone from the same source.

Add the following to Section 610.032:

Section 610.032.d. The grading of riprap, stone fill, stone blanket and stone ditch protection shall be determined by the Resident by visual inspection of the load before it is dumped into place, or, if ordered by the Resident, by dumping individual loads on a flat surface and sorting and measuring the individual rocks contained in the load. A separate, reference pile of stone with the required gradation will be placed by the Contractor at a convenient location where the Resident can see and judge by eye the suitability of the rock being placed during the duration of the project. The Resident reserves the right to reject stone at the job site or stockpile, and in place. Stone rejected at the job site or in place shall be removed from the site at no additional cost to the Department.

SPECIAL PROVISION
SECTION 703
AGGREGATES

Replace subsections 703.25 through 703.28 with the following:

703.25 Stone Fill Stones for stone fill shall consist of hard, sound, durable rock that will not disintegrate by exposure to water or weather. Stone for stone fill shall be angular and rough. Rounded, subrounded, or long thin stones will not be allowed. Stone for stone fill may be obtained from quarries or by screening oversized rock from earth borrow pits. The maximum allowable length to thickness ratio will be 3:1. The minimum stone size (10 lbs) shall have an average dimension of 5 inches. The maximum stone size (500 lbs) shall have a maximum dimension of approximately 36 inches. Larger stones may be used if approved by the Resident. Fifty percent of the stones by volume shall have an average dimension of 12 inches (200 lbs).

703.26 Plain and Hand Laid Riprap Stone for riprap shall consist of hard, sound durable rock that will not disintegrate by exposure to water or weather. Stone for riprap shall be angular and rough. Rounded, subrounded or long thin stones will not be allowed. The maximum allowable length to width ratio will be 3:1. Stone for riprap may be obtained from quarries or by screening oversized rock from earth borrow pits. The minimum stone size (10 lbs) shall have an average dimension of 5 inches. The maximum stone size (200 lbs) shall have an average dimension of approximately 12 inches. Larger stones may be used if approved by the Resident. Fifty percent of the stones by volume shall have an average dimension greater than 9 inches (50 lbs).

703.27 Stone Blanket Stones for stone blanket shall consist of sound durable rock that will not disintegrate by exposure to water or weather. Stone for stone blanket shall be angular and rough. Rounded or subrounded stones will not be allowed. Stones may be obtained from quarries or by screening oversized rock from earth borrow pits. The minimum stone size (300 lbs) shall have minimum dimension of 14 inches, and the maximum stone size (3000 lbs) shall have a maximum dimension of approximately 66 inches. Fifty percent of the stones by volume shall have average dimension greater than 24 inches (1000 lbs).

703.28 Heavy Riprap Stone for heavy riprap shall consist of hard, sound, durable rock that will not disintegrate by exposure to water or weather. Stone for heavy riprap shall be angular and rough. Rounded, subrounded, or thin, flat stones will not be allowed. The maximum allowable length to width ratio will be 3:1. Stone for heavy riprap may be obtained from quarries or by screening oversized rock from earth borrow pits. The minimum stone size (500 lbs) shall have minimum dimension of 15 inches, and at least fifty percent of the stones by volume shall have an average dimension greater than 24 inches (1000 lbs).

Add the following paragraph:

703.32 Definitions (ASTM D 2488, Table 1).

Angular: Particles have sharp edges and relatively plane sides with unpolished surfaces

Subrounded: Particles have nearly plane sides but have well-rounded corners and edges

Rounded: Particles have smoothly curved sides and no edges