MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL REPORT

For the Rehabilitation of

CATHANCE BRIDGE
OVER CATHANCE RIVER
TOPSHAM, MAINE

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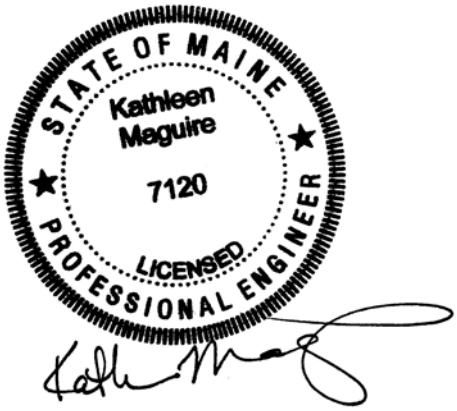

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GEOTECHNICAL SUMMARY

The purpose of this report is to present subsurface information and make geotechnical recommendations for the rehabilitation of the east abutment wingwalls of the Cathance Bridge over Cathance River in Topsham, Maine. The MaineDOT Bridge Program is currently proposing to rehabilitate or replace the east abutment wingwalls to extend the life of the existing bridge for a period of 10 to 15 years. The east abutment, west abutment and wingwalls, bridge superstructure and deck will not be affected by this rehabilitation project. The following wingwall stabilization/replacement options are discussed in detail in the attached report:

Sheet Pile Wingwalls – With this rehabilitation alternative sheet pile walls will be driven in front of the existing east abutment wingwalls and the annular space between the back of the sheet pile and the front of the existing block wall will be filled with concrete or grout. The final design of the chosen rehabilitation method will be the responsibility of the Contractor. The Contractor shall provide the proposed design and construction plans and methods for review by the Department prior to construction. The sheet pile walls shall be designed to withstand lateral earth pressures. The selected sheet pile section should consider a sacrificial steel loss. The use of hot-rolled sheets is recommended. The roadway curbing should be extended off the deck and back towards the approaches beyond the wings to direct surface water runoff away from the wingwalls.

Restack the Existing Granite Block Wingwalls - With this rehabilitation alternative the existing east abutment granite block wingwalls shall be dismantled and restacked and the roadway approach backfill shall be removed and replaced using geosynthetic reinforcement layers within the backfill and between the blocks. A geotextile fabric shall be placed between the back of the block wall and the backfill material to prevent the loss of material through any voids in the wall face. Surface drainage from the roadway shall be intercepted and directed away from the wall to a positive and permanent discharge well below and beyond the toe of the wall. The final design of the chosen rehabilitation method will be the responsibility of the Contractor. The design of the restacked wingwalls shall meet current AASHTO LRFD standards for stability. The Contractor shall provide the proposed design and construction plans and methods for review by the Department prior to construction.

Prefabricated Concrete Modular Block Gravity Wall – With this rehabilitation alternative the existing east abutment granite block wingwalls shall be removed and replaced with Precast Concrete Modular Gravity (PCMG) walls. 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.

Gabion Walls - MaineDOT experience has shown that construction of gabion walls correctly can be costly and time consuming. Disadvantages include corrosion when exposed to water and the potential for vandalism (cut wires) in more urban areas. The MaineDOT BDG recommends that gabion walls should only be used in non-critical situations, in dry environments (to minimize corrosion) and in rural areas (to minimize vandalism opportunities). The MaineDOT BDG also recommends that gabion wall heights be limited to 6 feet or less.

Gabion walls consist of stacked 3 foot cubed wire baskets which are filled with stone. Gabions are designed as mass gravity structures. Gabion walls should be backfilled with granular or gravel borrow. Gabion walls should be designed as specified in MaineDOT BDG Section 3.6.7.2 Prefabricated Modular Walls. The face of the gabion wall should be battered at an angle of about 6 degrees.

Gabions shall be constructed in accordance with MaineDOT Standard Specification 601 – Gabions and Mattresses. Materials for gabion baskets shall conform to MaineDOT Standard Specification 711.02 – Gabions. Wire for gabion baskets should be either PVC-coated or galvanized. Fill materials for the gabion baskets shall conform to MaineDOT Standard Specification 711.03 – Stones for Gabions. A filter fabric is recommended behind the gabion baskets to prevent loss of backfill materials through the gabion baskets.

The Contractor shall provide the proposed design and construction plans for review by the Department.

Riprap – Riprap shall be placed at the bottom of the rehabilitated wingwalls. Stone riprap shall be placed at a maximum slope of 1.75H:1V. The riprap section shall be underlain by a 1 foot thick layer of bedding material and Class "1" Erosion Control Geotextile. Riprap shall be 3 feet thick.

Construction Considerations – Care should be taken in construction of the riprap slopes to assure that they are constructed in accordance with MaineDOT Supplemental Specifications 610 and 703 and the Plans.

Rehabilitation of the wingwalls will require soil excavation and partial or full removal of the roadway approach fill. Construction activities may require earth support systems.

The roadway approach fill 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.

The Contractor will have to excavate the existing subbase and subgrade fill soils in the east bridge approach. These materials should not be used to re-base the newly constructed approach. 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.

1.0 INTRODUCTION

The purpose of this Geotechnical Report is to present geotechnical recommendations for the rehabilitation of the east abutment wingwalls of the Cathance Bridge over Cathance River in Topsham, Maine. A subsurface investigation has been completed at the site. The initial purpose of the subsurface investigation was to provide data and recommendations for the replacement of the existing bridge. A decision was made by the Maine Department of Transportation (MaineDOT) Bridge Program during the development of the Preliminary Design Report (PDR) to rehabilitate or replace the east abutment wingwalls rather than replace the structure. This report presents the geotechnical information obtained at the site and geotechnical recommendations for rehabilitation of the east abutment wingwalls.

The existing Cathance Bridge carries Cathance Road over Cathance River and was constructed in 1920. The bridge consists of a single span, concrete T-beam superstructure founded on concrete capped, stacked stone abutments with stacked granite block return wings and wingwalls. The existing abutments are believed to be founded on soil. The existing structure has a total length of approximately 55 feet. The 2010 Maine Department of Transportation (MaineDOT) maintenance inspection reports indicate that the bridge deck is in satisfactory condition (rating of 6), the superstructure is in fair condition (rating of 5) and the substructure is in serious condition (rating of 3). The Bridge Sufficiency Rating is 36.7. 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. The scour is determined to be above the top of the footings. Inspection records note that the bridge is in overall poor condition with moderate to heavy isolated areas of deterioration and that the east abutment wingwalls are both in need of repair. The south wingwall of the east abutment is partially collapsed and is heavily voided. The north wingwall of the east abutment is bulging and is separated from the abutment. Roadway settlement is noted at both ends of the bridge. The 2012 MaineDOT Underwater Dive Inspection Report shows that there is no loss of support at the existing abutments due to scour. Surface water runoff was determined to be a contributing factor to the near failure of the east abutment wingwalls. The west abutment and wingwalls show no signs of distress.

The MaineDOT Bridge Program is currently proposing to rehabilitate or replace the east abutment wingwalls to extend the life of the existing bridge for a period of 10 to 15 years. Four (4) rehabilitation alternatives have been proposed for the east abutment wingwalls:

- Jacket wingwalls with concrete,
- Install a sheet pile wall in front of the existing wingwalls with or without anchors,
- Restack the existing wingwalls, or
- Reconstruct the wingwalls with prefabricated wall elements or gabions.

A short road closure will be required for the rehabilitation activities. The east abutment, west abutment and wingwalls, bridge superstructure and deck will not be affected by this rehabilitation project.

2.0 GEOLOGIC SETTING

Cathance Bridge in Topsham carries Cathance Road over the Cathance River 2.5 miles south of the Topsham – Bowdoinham town line as shown on Sheet 1 - Location Map found at the end of this report.

According to the Surficial Geology map entitled Brunswick Quadrangle, Maine, Open File No. 01-484 (2001) published by the Maine Geological Survey the surficial soils in the vicinity of the site consist of massive to laminated silty clays with rare dropstones and occasional shelly horizons (Presumpscot Formation) which overlie rock and till and are interbedded with and overlie end moraines and marine fan deposits. The soils may also include sand deposited as a distal unit of submarine fans.

According to the Bedrock Geologic Map of Maine (1985) published by the Maine Geologic Survey, the bedrock in the vicinity of the site consists of mafic and felsic volcanic rocks identified the Cushing Formation.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions at the site were explored by drilling two (2) test borings. Test boring BB-TCR-101 was drilled behind the west abutment and test boring BB-TCR-102 was drilled behind the east abutment. The exploration locations are shown on Sheet 2 - Boring Location Plan and a generalized interpretive subsurface profile depicting the soil stratigraphy across the site is shown on Sheet 3 - Interpretive Subsurface Profile both found at the end of this report. The borings were drilled between January 11 and 18, 2012 by Northern Test Boring (NTB) of Gorham, Maine. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A – Boring Logs and on Sheet 4 – Boring Logs found end of this report.

The borings were drilled using solid stem auger and driven cased wash boring 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. The borings were drilled using an automatic hammer to drive the split spoon. The automatic hammer was calibrated in September of 2011 and was found to deliver approximately 28 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 0.768 to the raw field N-values. This hammer efficiency factor (0.768) and both the raw field N-value and the corrected N-value (N_{60}) are shown on the boring logs. The bedrock was cored in the borings using an NQ-2 inch 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 New England 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 exploration programs.

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 4 – Boring Logs, found at the end of this report.

4.0 LABORATORY TESTING

A laboratory testing program was conducted on selected samples recovered from test borings to assist in soil classification, evaluation of engineering properties of the soils, and geologic assessment of the project site. Laboratory testing consisted of seven (7) standard grain size analyses with hydrometer and natural water content and three (3) standard grain size analyses with natural water content. The tests were performed in the MaineDOT Materials and Testing Laboratory in Bangor, Maine. 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 4 – Boring Logs found at the end of this report.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered at the borings generally consisted of fill with occasional cobbles, underlain by silt, sand, clayey silt and glacial till all underlain by bedrock. The exploration locations are shown on Sheet 2 - Boring Location Plan and an interpretive subsurface profile depicting the generalized soil stratigraphy across the site is shown on Sheet 3 - Interpretive Subsurface Profile found at the end of this report. A brief summary description of the strata encountered is as follows:

5.1 Fill Material

Fill material was encountered beneath the pavement in both of the borings. The fill material consisted of:

- Brown, moist, silty fine to coarse sand, trace gravel, and
- Brown, moist, fine to coarse sand, some gravel, trace to little silt, trace to little clay.

A cobble was encountered at the top of the fill in boring BB-TCR-101.

The thickness of the fill ranged from approximately 4.6 feet in boring BB-TCR-101 to approximately 8.1 feet in boring BB-TCR-102. Corrected SPT N-values in the fill ranged from 9 to 17 blows per foot (bpf) indicating that the fill is loose to medium dense in consistency. One corrected N-value in boring BB-TCR-102 was 74 bpf. This value was influenced by the presence of frozen soil and is not indicative of the actual density of the fill layer. Water contents obtained from fill samples ranged from approximately 7% to 10%. Grain size analyses conducted on samples of the fill indicate that the soil is classified as an A-4 or A-1-b by the American Association of State Highway and Transportation Officials (AASHTO) Classification System and an SM or SW-SM by the Unified Soil Classification System.

5.2 Interbedded Silt, Silty Sand, Sandy Silt, Clayey Silt and Sand

Layers of interbedded silt, silty sand, sandy silt, clayey silt and sand were encountered beneath the fill in both of the borings. The soils consisted of:

- Brown, olive and grey, moist to wet, silt, some sand, little to some clay, trace gravel;
- Olive, wet, silty fine sand, little clay;
- Grey, wet, fine to medium sandy silt, little clay;
- Dark brown, silty fine sand, little clay;
- Grey, wet, clayey silt, trace to little fine sand; and
- Grey, wet, fine to medium sand, little silt.

An approximately 0.5 foot thick layer of wood was encountered at a depth of approximately 14.5 feet bgs in boring BB-TCR-101. An approximately 1.2 foot thick boulder was encountered at a depth of approximately 14.8 feet bgs in boring BB-TCR-102.

The total thickness of the layer ranged from approximately 31.0 feet in boring BB-TCR-101 to approximately 14.5 feet in boring BB-TCR-102. Corrected SPT N-values in the fine grained layers ranged from weight of rods (WOR) to 17 bpf indicating that these layers are very soft to very stiff in consistency. One (1) SPT N-value in the coarse grained soils was 5 bpf indicating that the granular layer is loose in consistency. Water contents from samples obtained within the layer ranged from approximately 12% to 48%. Grain size analyses conducted on samples indicate that the soil is classified as an A-4 by the AASHTO Classification System and a CL or CL-ML by the Unified Soil Classification System.

5.3 Glacial Till

A layer of glacial till was encountered beneath the interbedded silt, silty sand, sandy silt, clayey silt and sand in both of the borings. This glacial till soil consisted of:

- Brown, wet, gravel with cobbles, and
- Grey, wet, sandy gravel, trace silt, trace clay.

The thickness of the glacial till ranged from approximately 4.9 feet in boring BB-TCR-101 to approximately 1.3 feet in boring BB-TCR-102. One (1) corrected SPT N-value in the glacial till was 104 bpf indicating that the gravel is very dense in consistency. One (1) water content obtained from glacial till samples was approximately 8%. Grain size analyses conducted on a sample of the glacial till indicate that the soil is classified as an A-1-a by the AASHTO Classification System and a GW-GC by the Unified Soil Classification System.

5.4 Bedrock

Bedrock was encountered and cored in both of the borings. The Table 5.1 summarizes the depths to bedrock corresponding elevations of the top of bedrock and RQD for the borings:

Boring Number	Approximate Depth to Bedrock	Approximate Bedrock Elevation	RQD
BB-TCR-101	40.9 feet	-12.5 feet	15-60%
BB-TCR-102	24.3 feet	6.1 feet	39-42%

Table 5.1 - Summary of Bedrock Depths, Elevations and RQD

The bedrock is identified as light grey to purplish black, banded gneiss, with quartz, feldspar, biotite and muscovite mica and garnet with joints parallel and planar, healed joints at 60 degrees and foliation varying from 0 to 30 degrees. The RQD of the bedrock was determined to range from 15 to 60 percent indicating a rock mass quality of very poor to fair.

5.5 Groundwater

Groundwater was observed in boring BB-TCR-101 at a depth of approximately 16.0 feet below the existing ground surface. Note that water was introduced into the boreholes during the drilling operations. It is likely that the stabilized groundwater conditions differ from this estimate. Additionally, groundwater levels are expected to fluctuate seasonally depending upon the local precipitation magnitudes.

6.0 FOUNDATION ALTERNATIVES

Four (4) rehabilitation alternatives have been proposed by the MaineDOT Bridge Program for the east abutment wingwalls:

- Jacket wingwalls with a cast-in-place reinforced concrete jacket,
- Install a sheet pile wall in front of the existing wingwalls with or without anchors,
- Restack the existing granite block wingwalls, or
- Reconstruct the wingwalls with prefabricated wall elements or gabions.

Jacketing the wingwalls with concrete and the anchored sheet pile wall alternatives were eliminated from consideration during the PDR process. Three rehabilitation alternatives will be presented on the final construction plans: sheet pile wall in front of the existing wingwalls, restacking of the existing granite blocks and replacement with prefabricated wall elements or gabions. If a sheet pile wall is installed short road closures may be required to accommodate construction equipment in the roadway. Due to excavation needs, a short road closure will be required if the existing wall is restacked or if the existing granite blocks are replaced with prefabricated wall elements or gabions. The west abutment and wingwalls, bridge superstructure and deck will not be affected by this project.

7.0 GEOTECHNICAL RECOMMENDATIONS

The following sections will discuss geotechnical recommendations for the rehabilitation alternatives for the east abutment wingwalls. Three rehabilitation alternatives will be presented on the final construction plans: sheet pile wall in front of the existing wingwalls, restacking of the existing granite blocks and replacement with prefabricated wall elements or gabions. The final design of the chosen rehabilitation method will be the responsibility of the Contractor and subject to Department review.

7.1 Sheet Pile Wall

With this rehabilitation alternative a sheet pile wall will be driven in front of the existing east abutment wingwalls and the annular space between the back of the sheet pile and the front of the existing block wall will be filled with concrete or grout. The final design of the chosen rehabilitation method will be the responsibility of the Contractor. The Contractor shall provide the proposed design and construction plans and methods for review by the Department prior to construction. Based on the subsurface conditions encountered during the subsurface exploration program the following recommendations are made:

Unanchored cantilever sheet pile walls shall be designed to meet the requirements of AASHTO LRFD Bridge Design Specifications 6th Edition (LRFD) Article 11.8 and to withstand lateral earth pressures. The design of the sheet pile wall shall be consistent with the apparent earth pressure diagrams provided in LRFD Article 3.11.5.6. Earth loads shall be calculated using an active earth pressure coefficient, K_a, calculated using Rankine Theory. Where passive earth pressure in front of the wall can be considered, a passive earth pressure coefficient, K_p, calculated using Rankine Theory may be used. Table 7.1 presents the recommended earth pressure coefficients:

Internal Friction Angle ϕ	K_a Rankine	K_p Rankine
32 degrees	0.307	3.25
34 degrees	0.283	3.54

Table 7.1 – Recommended Earth Pressure Coefficients

Anchored sheet pile walls shall be designed to meet the requirements of LRFD Article 11.9 using the apparent earth pressure diagrams provided in LRFD Article 3.11.5.7.

A live load surcharge shall be applied where vehicular load is expected to act on the surface of the backfill within a distance equal to one-half the wall height behind the backface of the wall as required by section 3.6.8 of the MaineDOT Bridge Design Guide (BDG) and LRFD Article 3.11.6.4. The live load surcharge may be estimated as a uniform horizontal earth pressure due to an equivalent height (h_{eq}) taken from Table 7.2 (LRFD Table 3.11.6.4-2) below:

Wall Height (feet)	h_{eq} (feet)	
	Distance from wall pressure surface to edge of traffic = 0 feet	Distance from wall pressure surface to edge of traffic ≥ 0 feet
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

The selected sheet pile section should consider a sacrificial steel loss per the MaineDOT BDG. Due to the apparent amount of water moving through in the retained approach evidenced by the erosion of the existing soils, water induced corrosion of the steel is likely.

The use of hot-rolled sheets is recommended. Cold rolled sheet piles are not recommended for permanent applications. Cold rolled piles are typically thinner for the same section modulus. Section loss from corrosion could have a greater effect on cold rolled steel. The use of a ball and socket interlock system is recommended over the hook-type interlock system as the ball and socket system is less likely to unhook and separate underground due to driving pressure or obstructions. The use of American Society for Testing and Materials (ASTM) A 572 Grade 50 steel is recommended.

The roadway curbing should be extended off the deck and back towards the approaches beyond the wingwalls to direct surface water runoff away from the wingwalls.

7.2 Restack Existing Granite Block Wingwalls

With this rehabilitation alternative the existing east abutment granite block wingwalls shall be dismantled and restacked and the roadway approach backfill shall be removed and replaced using geosynthetic reinforcement layers within the backfill and between the blocks. Any voids in the wall facing shall be filled with grout. As the wall is reconstructed, the blocks shall be placed so that there are no continuous joint planes in the vertical direction. Each block should bear on at least two blocks below it. A geotextile fabric shall be placed between the back of the block wall and the backfill material to prevent the loss of material through any voids in the wall face. The fabric shall be a geotextile meeting the requirements of MDOT Standard Specification Subsection 722.02, Drainage Geotextile. Surface drainage from the roadway shall be intercepted and directed away from the wall to a positive and permanent discharge well below and beyond the toe of the wall. Roadway backfill shall be compacted to 95% of the maximum density as determined by AASHTO T-180, Method C or D.

The final design of the chosen rehabilitation method will be the responsibility of the Contractor. The design of the restacked wingwalls shall meet current AASHTO LRFD standards for stability. The Contractor shall provide the proposed design and construction plans and methods for review by the Department prior to construction. Design calculations shall provide thorough documentation of the sources of equations used and materials properties.

7.3 Precast Concrete Modular Block Retaining Wall

With this rehabilitation alternative the existing east abutment granite block wingwalls shall be removed and replaced with Precast Concrete Modular Gravity (PCMG) walls. 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 which is included in Appendix D found at the end of this report.

The PCMG wall designs shall consider a live load surcharge estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) taken from Table 7.2 (LRFD Table 3.11.6.4-2) in Section 7.1 above.

The PCMG walls may be designed to bear on the lower courses of the granite block walls which remain. The factored bearing resistance at the strength limit state for PCMG walls founded on the lower portion of the granite block wingwall to remain vs. foundation width is shown by the dashed line in Figure 7.1 below.

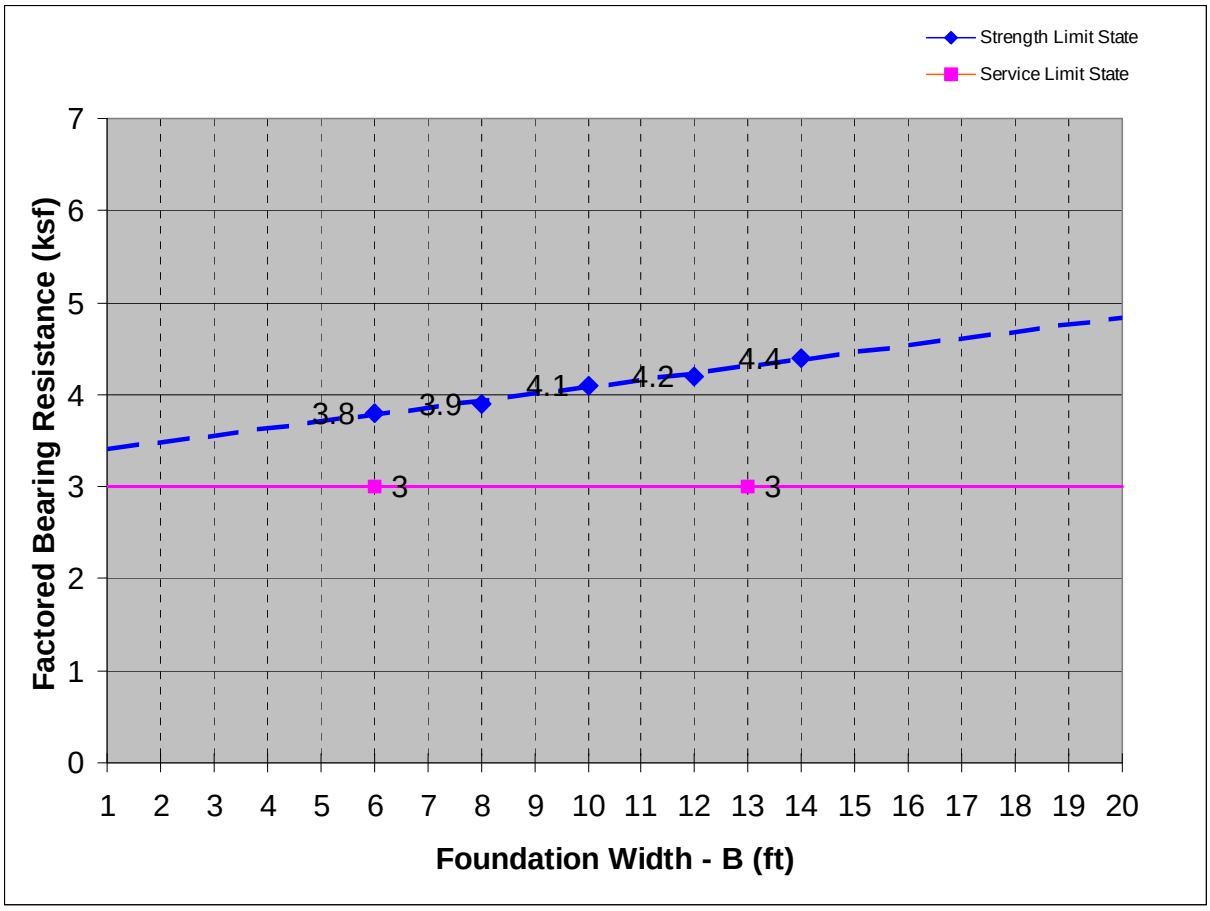


Figure 7.1 – Factored Bearing Resistance PCMG Wall Bearing on Lower Course of Existing Granite Block Walls vs. Foundation Width

Once the dimensions of the PCMG wall are determined, a factored bearing resistance can be determined from the figure. This factored bearing resistance must be greater than the applied factored vertical bearing pressure determined by the structural designer. The factored bearing resistance at the service limit state is shown by the solid line in Figure 7.1. A factored bearing resistance of 3 ksf may be used to control settlement when analyzing the service limit state and for preliminary footing sizing as allowed in LRFD C10.6.2.1. See Appendix C - Calculations for supporting calculations.

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 unfactored strength limit state loads with a resistance factor of 1.0. The overall stability of the wall system should be demonstrated at the Service I Load Combination with a resistance factor, ϕ, of 0.65.

The designer shall apply a sliding resistance factor φ_τ of 0.90 to the nominal sliding resistance of precast concrete wall segments founded on sand or the lower courses of granite block to remain. The eccentricity of loading at the strength limit state, based on factored loads, shall not exceed 0.45 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 30º at the foundation material to soil infill interface and a maximum frictional coefficient of 0.8x(tan 35º) at the granite block to concrete module interface. Recommended values of sliding frictional coefficients are based on LRFD Article 11.11.4.2, Table 10.5.5.2.2-1 and Table 3.11.5.3-1.

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

7.4 Gabion Wall

MaineDOT experience has shown that construction of gabion walls correctly can be costly and time consuming. Disadvantages include corrosion when exposed to water and the potential for vandalism (cut wires) in more urban areas. Section 5.6.7 of the MaineDOT BDG recommends that gabion walls should only be used in non-critical situations, in dry environments (to minimize corrosion) and in rural areas (to minimize vandalism opportunities). The MaineDOT BDG also recommends that gabion wall heights be limited to 6 feet or less.

The Contractor shall provide the proposed gabion wall design and construction plans and methods for review by the Department prior to construction.

Gabion walls consist of stacked 3 foot cubed wire baskets which are filled with stone. Gabions are designed as mass gravity structures. In designing gabion walls a unit weight, γ, of 100 pounds per cubic foot (pcf) should be used for the weight of the stone inside the baskets. Gabion walls should be backfilled with granular or gravel borrow. An angle of wall friction, δ, of 24 degrees should be used for design. Gabion walls should be designed as specified in MaineDOT BDG Section 3.6.7.2 Prefabricated Modular Walls. The face of the gabion wall should be battered at an angle of about 6 degrees to keep the resultant force toward the back of

the wall and increase wall stability. The coefficient of friction between the base of the gabion wall and cohesionless soil can be taken as tanϕ for the soil.

Gabions shall be constructed in accordance with MaineDOT Standard Specification 601 – Gabions and Mattresses. Materials for gabion baskets shall conform to MaineDOT Standard Specification 711.02 – Gabions. Wire for gabion baskets should be either PVC-coated or galvanized. The PVC coating is preferred as it does not flake off. Fill materials for the gabion baskets shall conform to MaineDOT Standard Specification 711.03 – Stones for Gabions. Gabions shall be well packed and full without excessive bulging. At no time shall stones be placed in the gabion baskets by machine. A filter fabric is recommended behind the gabion baskets to prevent loss of backfill materials through the gabion baskets.

7.5 Riprap

Riprap conforming to Supplemental Specifications 610 and 703 shall be placed at the bottom of the rehabilitated wingwalls. Stone riprap shall conform to item number 703.26 of the MaineDOT Supplemental Specification 703 and shall be placed at a maximum slope of 1.75H:1V. 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.

7.6 Construction Considerations

Care should be taken in construction of the riprap slopes to assure that they are constructed in accordance with MaineDOT Supplemental Specifications 610 and 703 and the Plans.

Rehabilitation or replacement of the wingwalls will require soil excavation and partial or full removal of the roadway approach fill. Construction activities may require earth support systems.

The roadway approach fill 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.

The Contractor will have to excavate the existing subbase and subgrade fill soils in the east bridge approach. These materials should not be used to re-base the newly constructed approach. 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.

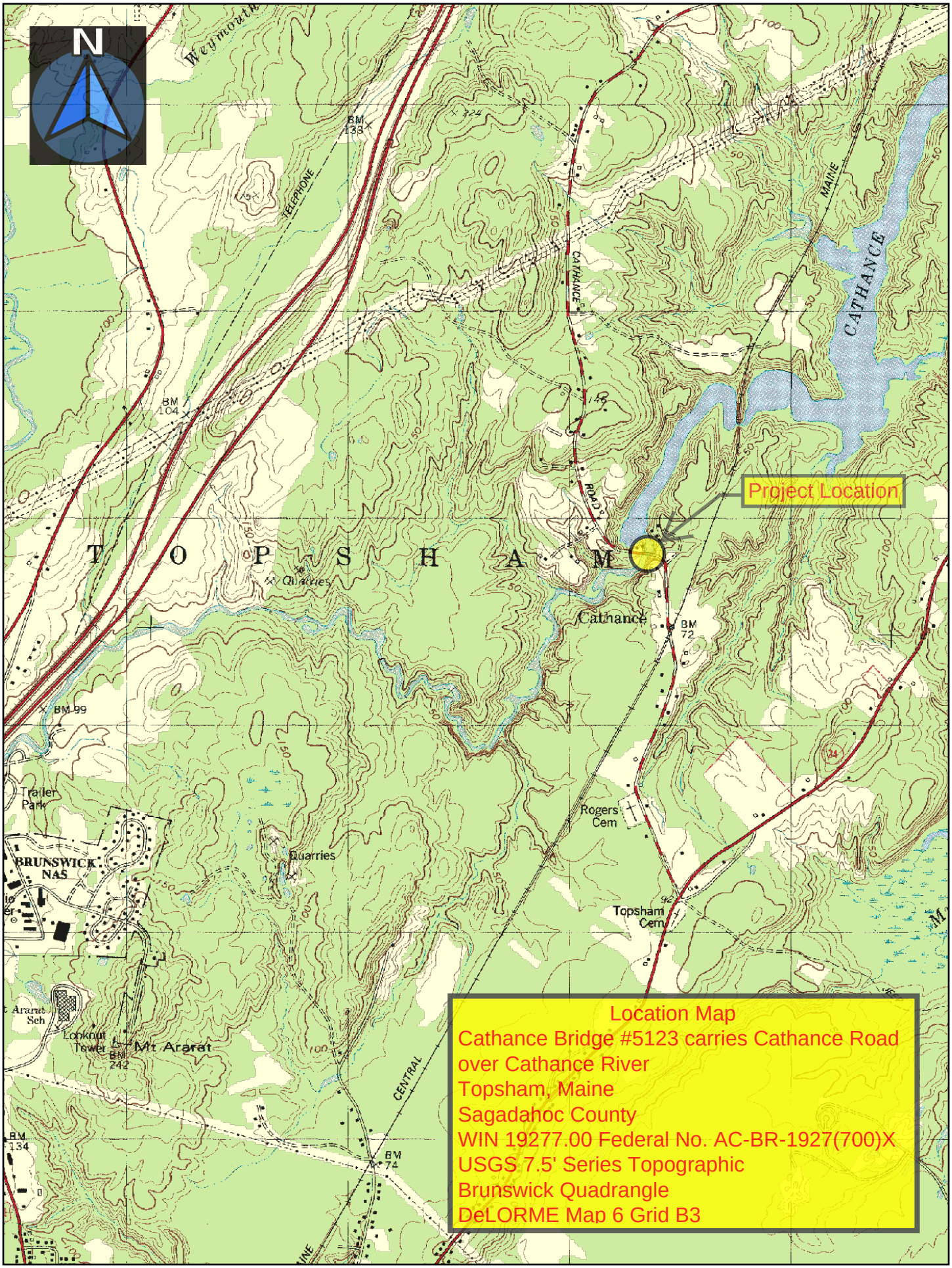
8.0 CLOSURE

This report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed rehabilitation of Cathance Bridge in Topsham in accordance with generally accepted geotechnical and foundation engineering practices. No other intended use or warranty is expressed or 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.

It is also recommended that the geotechnical engineer be provided the opportunity for a general review of the final design plans and specifications in order to verify that the earthwork and foundation recommendations have been properly interpreted and implemented in the design.

Sheets



Map Scale 1:24000

The Maine Department of Transportation provides this publication for information only. Reliance upon this information is at user risk. It is subject to revision and may be incomplete depending upon changing conditions. The Department assumes no liability if injuries or damages result from this information. This map is not intended to support emergency dispatch. Road names used on this map may not match official road names.

CURVE DATA #1
PI = 2+83.18
D = 2°17'30.6"
Δ = 5°36'35.8" Rt.
R = 2500.00'
L = 244.78'
T = 122.49'
E = 3.00'

CURVE DATA #2
PI = 4+53.37
D = 24°04'29.8"
Δ = 22°45'24.3" Rt.
R = 237.99'
L = 94.52'
T = 47.89'
E = 4.77'

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

PIN
19277.00

BRIDGE NO. 5123

BRIDGE PLANS

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED	K.MCGUIRE	
CHECKED-REVIEWED		
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

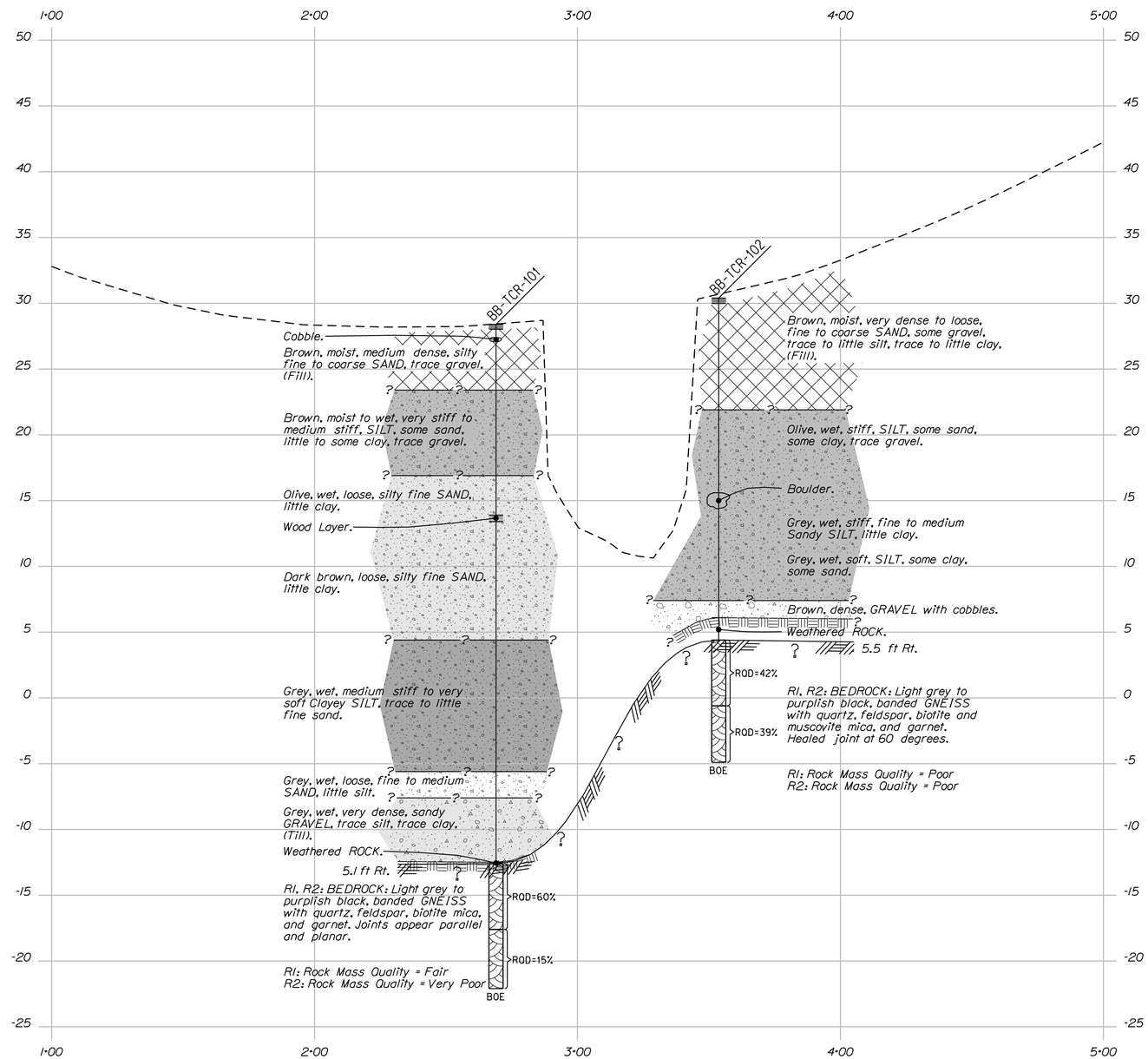
DATE

CATHANCE BRIDGE
CATHANCE RIVER
SAGADAHOC COUNTY
TOPSHAM

BORING LOCATION PLAN

SHEET NUMBER
2
OF 4

Filename: ...\00\geotech\msto\006_BLP1.dgn Division: GEOTECH Username: terry.white Date:8/9/2012



LEGEND

Boring No.
Offset, if shown
Pavement Thickness if applicable
Strata Interface
Boring
RQD= Rock Quality Designation for Rock Core Sample
BOE= Bottom Of Exploration
Top of Intact Bedrock
Weathered ROCK

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

PIN 19277.00

BRIDGE NO. 5123

BRIDGE PLANS

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED	T. WHITE	JAN 2012
CHECKED-REVIEWED	K.MCGUIRE	
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

CATHANCE BRIDGE
CATHANCE RIVER
TOPSHAM SAGADAHOC COUNTY

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
3
OF 4

Filename: ...\00\geotech\msta\007_ISP1.dgn Division: GEOTECH Username: terry.white Date:8/9/2012

Filename: ...\msta\008_BORING LOGS1.dgn Division: GEOTECH Username: terry.white Date:8/9/2012

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Cathance Bridge #5123 carries Cathance Road over Cathance River
Location: Topsham, Maine

Boring No.: BB-TCR-101
WIN: 19277.00

Driller:	Northern Test Boring	Elevation (ft.)	28.4	Auger ID/OD:	2.75/6.75 HSA
Operator:	Mike/Adam	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	Diedrich D-50 Track	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	1/11/2012; 08:00-15:00	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	2+69. 5.1 ft Rt.	Casing ID/OD:	HW	Water Level*:	16.0 ft bgs.
Hammer Efficiency Factor: 0.768		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
							HSA	28.03	4.5" Pavement — 0.38	
									Cobble from 1.0-1.3 ft bgs.	
	10	24/20	2.00 - 4.00	7/8/5/4	13	17			Brown, moist, medium dense, Silty, fine to coarse SAND, trace gravel. (Fill).	GM244713 A-4, SM WC=9.5%
5								23.40	— 5.00	
	20	24/17	5.00 - 7.00	8/6/7/7	13	17			Brown, moist, very stiff, SILT, some sand, little clay, trace gravel.	GM244714 A-4, CL WC=12.1%
10	3D/A	24/16	10.00 - 12.00	2/3/2/2	5	6			3D (10.0-11.5 ft bgs) Brown, wet, medium stiff, SILT, some sand, some clay, trace gravel.	GM244715 A-4, CL WC=18.8%
								16.90	— 11.50	
									3D/A (11.5-12.0 ft bgs) Olive, wet, loose, Silty fine SAND, little clay.	
15	40	6/6	14.50 - 15.00	50	---				Wood layer from 14.5-15.0 ft bgs.	
20	50	24/20	20.00 - 22.00	3/2/2/2	4	5	0WP		Dark brown, loose, Silty fine SAND, little clay. 0WP = Hydraulic Push	GM244716 A-4, CL-ML WC=48.0%
							33			
							36			
							47			
								4.40	— 24.00	
25	60	24/6	24.00 - 26.00	3/3/2/2	5	6	80		Grey, wet, medium stiff, Clayey SILT, little fine sand.	
							60			
							44			
							50			
							46			
30	70	24/24	29.00 - 31.00	WOR/WOR/WOR/WOR	---		0WP		Grey, wet, very soft, Clayey SILT, trace fine sand.	GM244717 A-4, CL WC=34.0%
								-5.60	— 34.00	
35	1U	24/18	34.00 - 36.00	Piston Sample			11		Grey, wet, loose, fine to medium SAND, little silt. Sand sample in tube.	
							50			
							91			
							89			
							80	-7.60	— 36.00	
40	8D	22.8/17	39.00 - 40.90	18/45/36/50(4.8)	81	104			Grey, wet, very dense, Sandy GRAVEL, trace silt, trace clay. (Till). Weathered rock in spoon tip.	GM244718 A-1-a, GW-GC WC=8.2%
								-12.50	— 40.90	
	R1	60/60	41.00 - 46.00	RQD = 60%			NQ-2	-12.60	Roller Coned ahead to 41.0 ft bgs. — 41.00	
									Top of Intact Bedrock at Elev. -12.6 ft. R1:Bedrock: Light grey to purplish black, banded GNEISS with quartz, feldspar, biotite mica and garnet. Joints appear parallel and planar. (Cushing Formation). Rock Mass Quality = Fair. R1:Core Times (min:sec) 41.0-42.0 ft (2:10) 42.0-43.0 ft (2:00) 43.0-44.0 ft (2:00) 44.0-45.0 ft (2:15) 45.0-46.0 ft (2:00) 100% Recovery		
45										
	R2	54/38	46.00 - 50.50	RQD = 15%					R2:Bedrock: Similar to above. Rock Mass Quality = Very poor. R2:Core Times (min:sec) 46.0-47.0 ft (3:00) 47.0-48.0 ft (3:00) 48.0-49.0 ft (3:00) 49.0-50.0 ft (6:30) 50.0-50.5 ft (6:46) 70% Recovery	
50								-22.10	Bit plugged and ground into bedrock at 50.5 ft bgs. — 50.50	
									Bottom of Exploration at 50.50 feet below ground surface.	
55										
60										
65										
70										
75										

Remarks:
Auto Hammer # 283

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

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Cathance Bridge #5123 carries Cathance Road over Cathance River
Location: Topsham, Maine

Boring No.: BB-TCR-102
WIN: 19277.00

Driller:	Northern Test Boring	Elevation (ft.)	30.4	Auger ID/OD:	2.75/6.75 HSA
Operator:	Mike/Adam	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	Diedrich D-50 Track	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	1/18/2012; 08:00-13:00	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	3+53.7. 5.5 ft Rt.	Casing ID/OD:	HW	Water Level*:	None Observed
Hammer Efficiency Factor: 0.768		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
							HSA	29.98	5" Pavement — 0.42	
	1D	24/18	1.00 - 3.00	43/43/15/6	58	74			Frost depth approximately 2.0 ft bgs. Brown, moist, very dense, fine to coarse SAND, some gravel, little silt. (Fill).	GM244719 A-1-b, SW-SM WC=6.2%
5	2D	24/16	5.00 - 7.00	5/4/3/3	7	9			Brown, moist, loose, fine to coarse SAND, some gravel, little silt. (Fill).	GM244720 A-1-b, SW WC=7.3%
								21.90	— 8.50	
10	3D	24/15	10.00 - 12.00	2/4/4/4	8	10	24		Olive, wet, stiff, SILT, some sand, some clay, trace gravel.	GM244721 A-4, CL WC=27.1%
							33			
							32			
							50			
							10		Boulder or Block from 14.8-16.0 ft bgs. Roller Cone through.	
15							21			
	4D	24/13	16.00 - 18.00	4/4/4/4	8	10	32		Grey, wet, stiff, fine to medium Sandy SILT, little clay. Roller Coned ahead to 19.0 ft bgs.	
							43			
							39			
20	5D	24/24	19.00 - 21.00	WOH/1/2/2	3	4	43		Grey, wet, soft, SILT, some clay, some sand.	GM244722 A-4, CL WC=26.4%
							26			
							31			
							31	7.40	— 23.00	
	6D	9.6/1	23.50 - 24.30	14/40(3.6")	---		050		050 blows for 0.5 ft. Brown, dense, GRAVEL with cobbles.	
								6.10	— 24.30	
25									Roller Cone through Weathered ROCK to 26.0 ft bgs.	
								4.40	— 26.00	
	R1	60/60	26.00 - 31.00	RQD = 42%			NQ-2		Top of Intact Bedrock at Elev. 4.4 ft. R1:Bedrock: Light grey to purplish black, banded GNEISS with quartz, feldspar, biotite and muscovite mica, and garnet. Healed joint at 60 degrees. (Cushing Formation) Rock Mass Quality = Poor. R1:Core Times (min:sec) 26.0-27.0 ft (2:20) 27.0-28.0 ft (2:30) 28.0-29.0 ft (2:30) 29.0-30.0 ft (2:15) 30.0-31.0 ft (2:30) 100% Recovery	
30										
	R2	51.6/51.6	31.00 - 35.30	RQD = 39%					R2:Bedrock: Similar to above with foliation varying from 0 to 30 degrees. Rock Mass Quality = Poor. R2:Core Times (min:sec) 31.0-32.0 ft (3:00) 32.0-33.0 ft (3:00) 33.0-34.0 ft (3:00) 34.0-35.0 ft (3:00) 35.0-35.3 ft (2:45) 100% Recovery	
35								-4.90	— 35.30	
									Bottom of Exploration at 35.30 feet below ground surface.	
40										
45										
50										

Remarks:
Auto Hammer # 283

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

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

Page 1 of 1
Boring No.: BB-TCR-102

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

PIN 19277.00

BRIDGE NO. 5123

BRIDGE PLANS

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED	T. WHITE	AUG 2012
CHECKED-REVIEWED	K.MCGUIRE	
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

CATHANCE BRIDGE
CATHANCE RIVER
TOPSHAM SAGADAHOC COUNTY

BORING LOGS

SHEET NUMBER
4
OF 4

Appendix A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM

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

Desired Soil Observations: (in this order)

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

TERMS DESCRIBING DENSITY/CONSISTENCY

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

Modified Burmister System

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

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

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

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

Rock Quality Designation (RQD):

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

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

Correlation of RQD to Rock Mass Quality

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

Desired Rock Observations: (in this order)

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

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

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

Sample Container Labeling Requirements:

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Cathance Bridge #5123 carries Cathance Road over Cathance River
Location: Topsham, Maine

Boring No.: BB-TCR-101
WIN: 19277.00

Driller:	Northern Test Boring	**Elevation (ft.)**	28.4	**Auger ID/OD:**	2.75/6.75 HSA
Operator:	Mike/Adam	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrich D-50 Track	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	1/11/2012; 08:00-15:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	2+69, 5.1 ft Rt.	**Casing ID/OD:**	HW	**Water Level*:**	16.0 ft bgs.
Hammer Efficiency Factor:	0.768	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							HSA	28.03	4½" Pavement — 0.38	
									Cobble from 1.0-1.3 ft bgs.	
	1D	24/20	2.00 - 4.00	7/8/5/4	13	17			Brown, moist, medium dense, Silty, fine to coarse SAND, trace gravel, (Fill).	G#244713 A-4, SM WC=9.5%
5	2D	24/17	5.00 - 7.00	8/6/7/7	13	17		23.40	5.00 — Brown, moist, very stiff, SILT, some sand, little clay, trace gravel.	G#244714 A-4, CL WC=12.1%
10	3D/A	24/16	10.00 - 12.00	2/3/2/2	5	6			3D (10.0-11.5 ft bgs) Brown, wet, medium stiff, SILT, some sand, some clay, trace gravel.	G#244715 A-4, CL WC=18.8%
								16.90	11.50 — 3D/A (11.5-12.0 ft bgs) Olive, wet, loose, Silty fine SAND, little clay.	
15	4D	6/6	14.50 - 15.00	50	---				Wood layer from 14.5-15.0 ft bgs.	
20	5D	24/20	20.00 - 22.00	3/2/2/2	4	5	aHP		Dark brown, loose, Silty fine SAND, little clay. aHP = Hydraulic Push	G#244716 A-4, CL-ML WC=48.0%
							33			
							36			
							47			
25	6D	24/6	24.00 - 26.00	3/3/2/2	5	6	80	4.40	24.00 — Grey, wet, medium stiff, Clayey SILT, little fine sand.	

Remarks:

Auto Hammer # 283

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

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Cathance Bridge #5123 carries Cathance Road over Cathance River
Location: Topsham, Maine

Boring No.: BB-TCR-101
WIN: 19277.00

Driller:	Northern Test Boring	**Elevation (ft.)**	28.4	**Auger ID/OD:**	2.75/6.75 HSA
Operator:	Mike/Adam	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrich D-50 Track	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	1/11/2012; 08:00-15:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	2+69, 5.1 ft Rt.	**Casing ID/OD:**	HW	**Water Level*:**	16.0 ft bgs.
Hammer Efficiency Factor:	0.768	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25							60			
							44			
							50			
							46			
	7D	24/24	29.00 - 31.00	WOR/WOR/WOR/ WOR	---		[a]HP		Grey, wet, very soft, Clayey SILT, trace fine sand.	G#244717 A-4, CL WC=34.0%
30										
								-5.60	34.00	
	1U	24/18	34.00 - 36.00	Piston Sample			11		Grey, wet, loose, fine to medium SAND, little silt. Sand sample in tube.	
35							50			
							91	-7.60	36.00	
							89			
							80			
	8D	22.8/17	39.00 - 40.90	18/45/36/50(4.8)	81	104			Grey, wet, very dense, Sandy GRAVEL, trace silt, trace clay, (Till). Weathered rock in spoon tip.	G#244718 A-1-a, GW-GC WC=8.2%
40										
								-12.50	40.90	
	R1	60/60	41.00 - 46.00	RQD = 60%			NQ-2	-12.60	Roller Coned ahead to 41.0 ft bgs. 41.00	
									Top of Intact Bedrock at Elev. -12.6 ft. R1:Bedrock; Light grey to purplish black, banded GNEISS with quartz, feldspar, biotite mica and garnet. Joints appear parallel and planar, (Cushing Formation). Rock Mass Quality = Fair. R1:Core Times (min:sec) 41.0-42.0 ft (2:10) 42.0-43.0 ft (2:00) 43.0-44.0 ft (2:00) 44.0-45.0 ft (2:15) 45.0-46.0 ft (2:00) 100% Recovery	
45										
	R2	54/38	46.00 - 50.50	RQD = 15%					R2:Bedrock: Similar to above. Rock Mass Quality = Very poor. R2:Core Times (min:sec) 46.0-47.0 ft (3:00) 47.0-48.0 ft (3:00) 48.0-49.0 ft (3:00)	
50										

Remarks:

Auto Hammer # 283

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Cathance Bridge #5123 carries Cathance Road over Cathance River
Location: Topsham, Maine

Boring No.: BB-TCR-101
WIN: 19277.00

Driller:	Northern Test Boring	**Elevation (ft.)**	28.4	**Auger ID/OD:**	2.75/6.75 HSA
Operator:	Mike/Adam	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrich D-50 Track	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	1/11/2012; 08:00-15:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	2+69, 5.1 ft Rt.	**Casing ID/OD:**	HW	**Water Level*:**	16.0 ft bgs.
Hammer Efficiency Factor:	0.768	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50								-22.10		49.0-50.0 ft (6:30) 50.0-50.5 ft (6:46) 70% Recovery Bit plugged and ground into bedrock at 50.5 ft bgs. 50.50 **Bottom of Exploration at 50.50 feet below ground surface.**	
55											
60											
65											
70											
75											

Remarks:

Auto Hammer # 283

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Cathance Bridge #5123 carries Cathance Road over Cathance River
Location: Topsham, Maine

Boring No.: BB-TCR-102
WIN: 19277.00

Driller:	Northern Test Boring	**Elevation (ft.)**	30.4	**Auger ID/OD:**	2.75/6.75 HSA
Operator:	Mike/Adam	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrich D-50 Track	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	1/18/2012; 08:00-13:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	3+53.7, 5.5 ft Rt.	**Casing ID/OD:**	HW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.768	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							HSA	29.98	5" Pavement (0.42)	
	1D	24/18	1.00 - 3.00	43/43/15/6	58	74			Frost depth approximately 2.0 ft bgs. Brown, moist, very dense, fine to coarse SAND, some gravel, little silt, (Fill).	G#244719 A-1-b, SW-SM WC=8.2%
5	2D	24/16	5.00 - 7.00	5/4/3/3	7	9			Brown, moist, loose, fine to coarse SAND, some gravel, little silt, (Fill).	G#244720 A-1-b, SM WC=7.3%
								21.90	(8.50)	
10	3D	24/15	10.00 - 12.00	2/4/4/4	8	10	24		Olive, wet, stiff, SILT, some sand, some clay, trace gravel.	G#244721 A-4, CL WC=27.1%
							33			
							32			
							50			
							10			
15							21		Boulder or Block from 14.8-16.0 ft bgs. Roller Cone through.	
	4D	24/13	16.00 - 18.00	4/4/4/4	8	10	32		Grey, wet, stiff, fine to medium Sandy SILT, little clay. Roller Coned ahead to 19.0 ft bgs.	
							43			
							39			
	5D	24/24	19.00 - 21.00	WOH/1/2/2	3	4	43		Grey, wet, soft, SILT, some clay, some sand.	G#244722 A-4, CL WC=26.4%
20							26			
							31			
							31	7.40	(23.00)	
	6D	9.6/1	23.50 - 24.30	14/40(3.6")	---		a50		a50 blows for 0.5 ft. Brown, dense, GRAVEL with cobbles.	
								6.10	(24.30)	
25									Roller Cone through Weathered ROCK to 26.0 ft bgs.	

Remarks:

Auto Hammer # 283

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Cathance Bridge #5123 carries Cathance Road over Cathance River
Location: Topsham, Maine

Boring No.: BB-TCR-102
WIN: 19277.00

Driller:	Northern Test Boring	**Elevation (ft.)**	30.4	**Auger ID/OD:**	2.75/6.75 HSA
Operator:	Mike/Adam	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrich D-50 Track	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	1/18/2012; 08:00-13:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	3+53.7, 5.5 ft Rt.	**Casing ID/OD:**	HW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.768	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25											
								4.40		26.00	
	R1	60/60	26.00 - 31.00	RQD = 42%			NQ-2			Top of Intact Bedrock at Elev. 4.4 ft. R1:Bedrock: Light grey to purplish black, banded GNEISS with quartz, feldspar, biotite and muscovite mica, and garnet. Healed joint at 60 degrees. (Cushing Formation) Rock Mass Quality = Poor. R1:Core Times (min:sec) 26.0-27.0 ft (2:20) 27.0-28.0 ft (2:30) 28.0-29.0 ft (2:30) 29.0-30.0 ft (2:15) 30.0-31.0 ft (2:30) 100% Recovery	
30											
	R2	51.6/51.6	31.00 - 35.30	RQD = 39%						R2:Bedrock: Similar to above with foliation varying from 0 to 30 degrees. Rock Mass Quality = Poor. R2:Core Times (min:sec) 31.0-32.0 ft (3:00) 32.0-33.0 ft (3:00) 33.0-34.0 ft (3:30) 34.0-35.0 ft (3:00) 35.0-35.3 ft (2:45) 100% Recovery	
35								-4.90		35.30	
										Bottom of Exploration at 35.30 feet below ground surface.	
40											
45											
50											

Remarks:

Auto Hammer # 283

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

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

Appendix B

Laboratory Data

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Topsham Work Number: 19277.00

Boring & Sample Identification Number	Station (Feet)	Offset (Feet)	Depth (Feet)	Reference Number	G.S.D.C. Sheet	W.C. %	L.L.	P.I.	Classification		
									Unified	AASHTO	Frost
BB-TCR-101, 1D	2+69	5.1 Rt.	2.0-4.0	244713	1	9.5			SM	A-4	III
BB-TCR-101, 2D	2+69	5.1 Rt.	5.0-7.0	244714	1	12.1			CL	A-4	IV
BB-TCR-101, 3D	2+69	5.1 Rt.	10.0-12.0	244715	1	18.8			CL	A-4	IV
BB-TCR-101, 5D	2+69	5.1 Rt.	20.0-22.0	244716	1	48.0			CL-ML	A-4	IV
BB-TCR-101, 7D	2+69	5.1 Rt.	29.0-31.0	244717	1	34.0			CL	A-4	IV
BB-TCR-101, 8D	2+69	5.1 Rt.	39.0-40.9	244718	1	8.2			GW-GC	A-1-a	0
BB-TRC-102, 1D	3+53.7	5.5 Rt.	1.0-3.0	244719	2	8.2			SW-SM	A-1-b	0
BB-TRC-102, 2D	3+53.7	5.5 Rt.	5.0-7.0	244720	2	7.3			SM	A-1-b	II
BB-TRC-102, 3D	3+53.7	5.5 Rt.	10.0-12.0	244721	2	27.1			CL	A-4	IV
BB-TRC-102, 5D	3+53.7	5.5 Rt.	19.0-21.0	244722	2	26.4			CL	A-4	IV

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

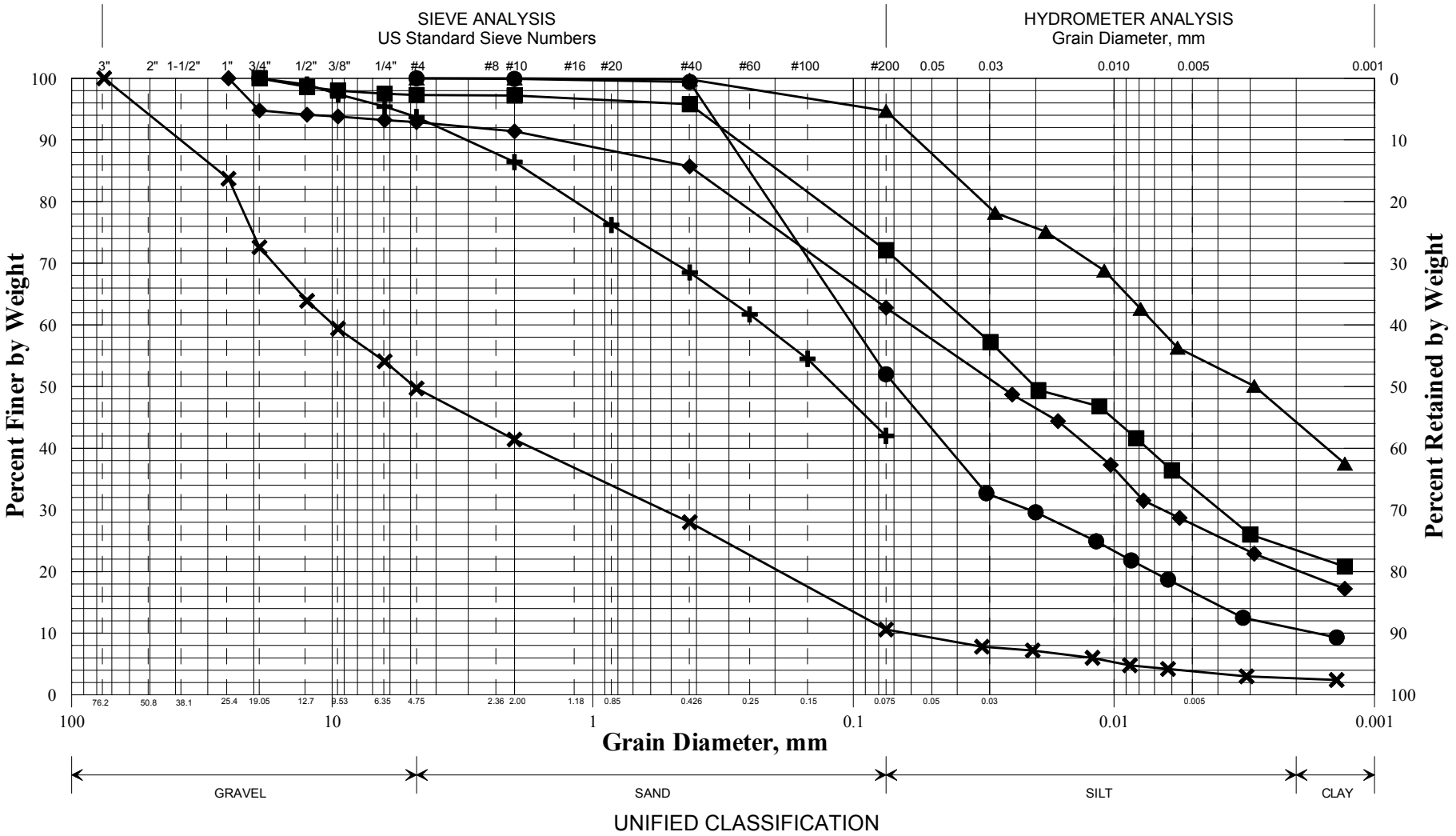
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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

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

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

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE

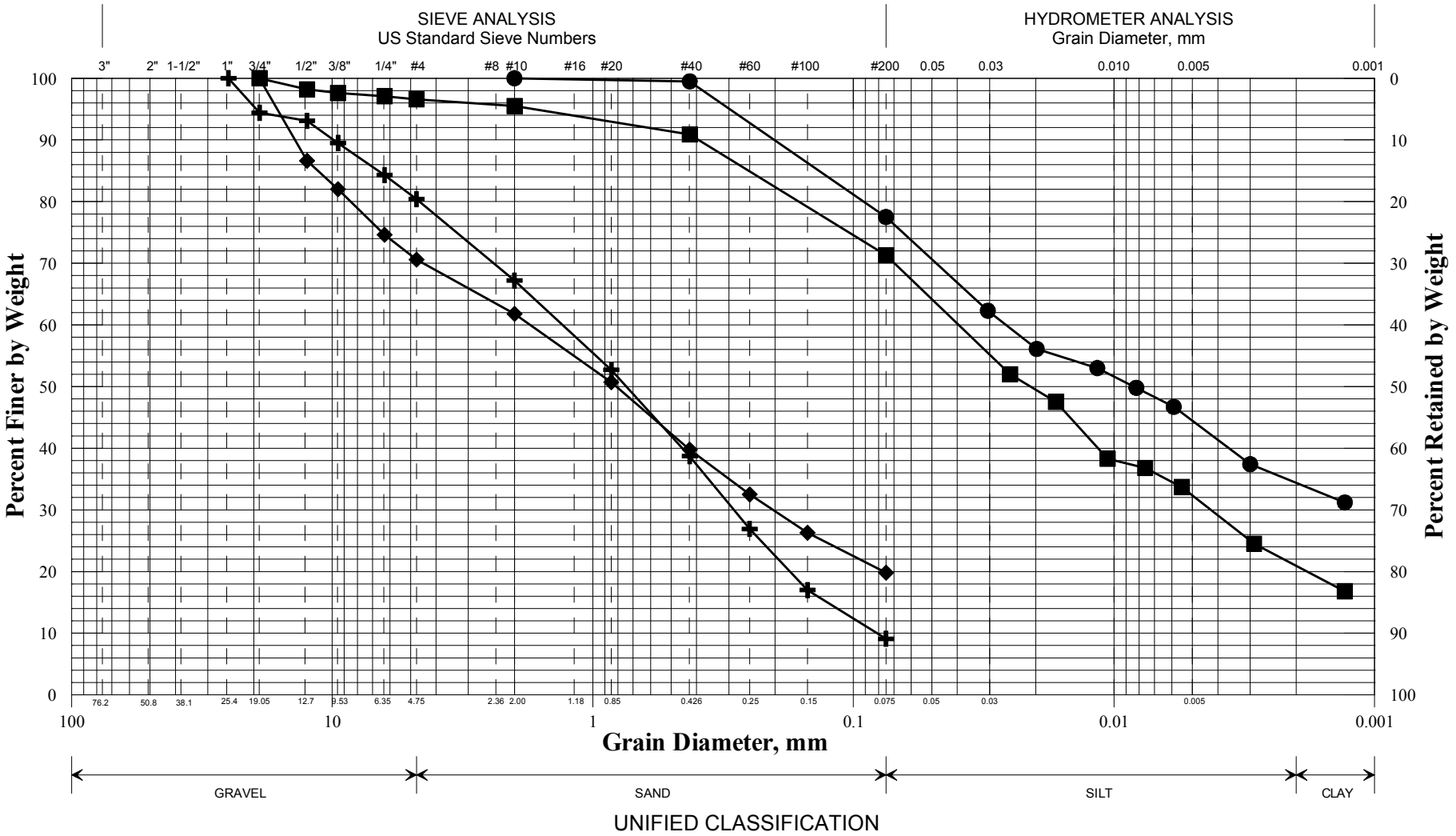


	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-TCR-101/1D	2+69	5.1 RT	2.0-4.0	Silty SAND, trace gravel.	9.5			
◆	BB-TCR-101/2D	2+69	5.1 RT	5.0-7.0	SILT, some sand, little clay, trace gravel.	12.1			
■	BB-TCR-101/3D	2+69	5.1 RT	10.0-12.0	SILT, some sand, some clay, trace gravel.	18.8			
●	BB-TCR-101/5D	2+69	5.1 RT	20.0-22.0	Silty SAND, little clay.	48.0			
▲	BB-TCR-101/7D	2+69	5.1 RT	29.0-31.0	Clayey SILT, trace sand.	34.0			
✖	BB-TCR-101/8D	2+69	5.1 RT	39.0-40.9	Sandy GRAVEL, trace silt, trace clay.	8.2			

WIN
019277.00
Town
Topsham
Reported by/Date
WHITE, TERRY A 4/12/2012

SHEET 1

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
✚	BB-TCR-102/1D	3+53.7	5.5 RT	1.0-3.0	SAND, some gravel, little silt.	8.2			
◆	BB-TCR-102/2D	3+53.7	5.5 RT	5.0-7.0	SAND, some gravel, little silt.	7.3			
■	BB-TCR-102/3D	3+53.7	5.5 RT	10.0-12.0	SILT, some sand, some clay, trace gravel.	27.1			
●	BB-TCR-102/5D	3+53.7	5.5 RT	19.0-21.0	SILT, some clay, some sand.	26.4			
▲									
✖									

WIN
019277.00
Town
Topsham
Reported by/Date
WHITE, TERRY A 4/12/2012

Appendix C

Calculations

Earth Pressure:

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

For a horizontal backfill surface:

$\phi := 32 \cdot deg$

$K_a := \tan\left(45 \cdot deg - \frac{\phi}{2}\right)^2$ $\quad K_a = 0.307$

$\phi := 34 \cdot deg$

$K_a := \tan\left(45 \cdot deg - \frac{\phi}{2}\right)^2$ $\quad K_a = 0.283$

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

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

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

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

$K_{p_rank} = 3.25$

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

Angle of internal soil friction: $\phi := 34 \cdot deg$

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

$K_{p_rank} = 3.54$

Bearing Resistance - PCMG Wall:

Part 1 - Service Limit State

Nominal and factored Bearing Resistance - spread footing on silt

Presumptive Bearing Resistance for Service Limit State ONLY

Reference: AASHTO LRFD Bridge Design Specifications 6th 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: Silt under granite block to remain

Consistency In Place: stiff

Bearing Resistance: Ordinary Range (ksf) 2 to 6

Recommended Value of Use: 3 ksf

Recommended Value: $3 \cdot ksf = 1.5 \cdot tsf$ $\quad tsf := g \cdot \left(\frac{ton}{ft^2}\right)$

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

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

$q_{factored_bc} := 1.5 \cdot tsf$ or $q_{factored_bc} = 3 \cdot ksf$

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

Part 2 - Strength Limit State

Nominal and factored Bearing Resistance - spread footing on remaining granite block on silt

Reference: Foundation Engineering and Design by JE Bowles Fifth Edition

Assumptions:
1. Walls will be on granite block to remain. $D_f := 0 \cdot ft$
2. Assumed parameters for foundation soils: (Ref: Bowles 5th Ed Table 3-4)
   - Saturated unit weight: $\gamma_s := 115 \cdot pcf$
   - Dry unit weight: $\gamma_d := 110 \cdot pcf$
   - Internal friction angle: $\phi_{ns} := 20 \cdot deg$
   - Undrained shear strength: $c_{ns} := 500 \cdot psf$
3. Use Terzaghi strip equations as L>B
4. Effective stress analysis footing on ϕ-c soil (Bowles 5th Ed. Example 4-1 pg 231)

Look at several stem lengths

$$B := \begin{pmatrix} 6 \\ 8 \\ 10 \\ 12 \\ 14 \end{pmatrix} \cdot ft$$

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 ϕ=20 deg $N_c := 14.83$ $N_q := 6.4$ $N_\gamma := 2.9$

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

$q := D_f \cdot (\gamma_s)$ $q = 0 \cdot tsf$

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

$$q_{nominal} = \begin{pmatrix} 4.2 \\ 4.4 \\ 4.5 \\ 4.7 \\ 4.9 \end{pmatrix} \cdot tsf$$

Resistance Factor: $\phi_b := 0.45$

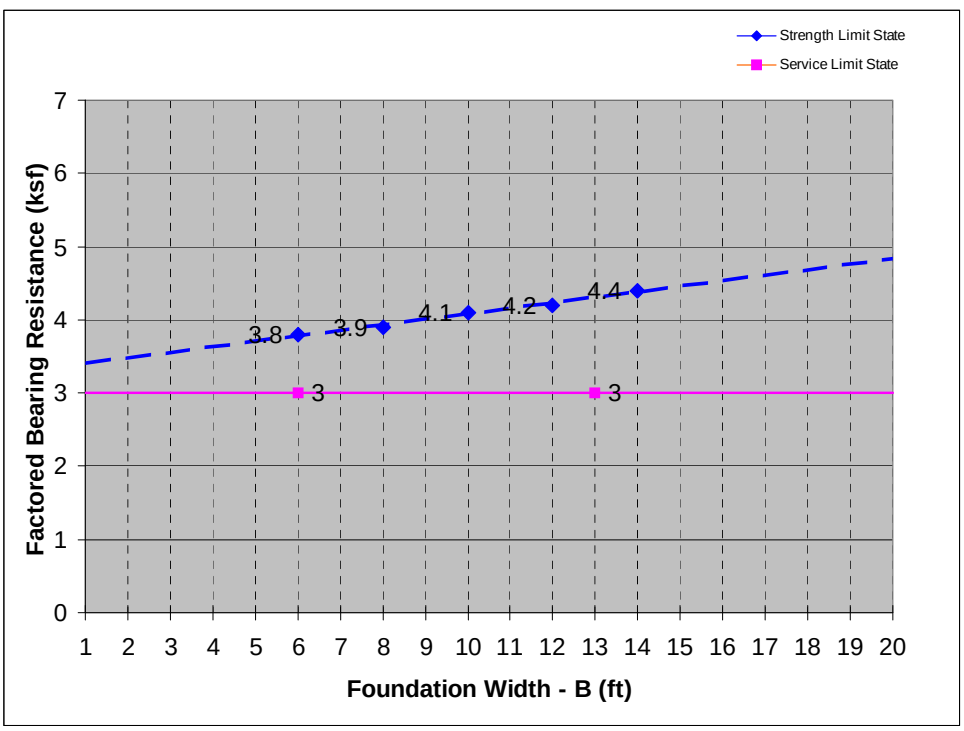
AASHTO LRFD Table 10.5.5.2.2-1

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

Based on these footing widths

$$q_{factored} = \begin{pmatrix} 1.9 \\ 2 \\ 2 \\ 2.1 \\ 2.2 \end{pmatrix} \cdot tsf \qquad q_{factored} = \begin{pmatrix} 3.8 \\ 3.9 \\ 4.1 \\ 4.2 \\ 4.4 \end{pmatrix} \cdot ksf \qquad B = \begin{pmatrix} 6 \\ 8 \\ 10 \\ 12 \\ 14 \end{pmatrix} \cdot ft$$

At Strength Limit State:



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 necessity 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) ±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 inches wide, by 0.5 inch 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: backfill and bedding material shall only contain particles that will pass the 3-inch square mesh sieve and 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, Service 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)
3. 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.
4. 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 Prefabricated Concrete Modular Gravity 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_{H1}, where γP_{H1}=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. Safety Against Structural Failure. Prefabricated units shall be designed for all strength and reinforcement requirements in accordance with LRFD Section 5 and LRFD Article 11.11.5.

E. 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.

F. The maximum calculated factored bearing pressure under the Prefabricated Concrete Modular Gravity block wall shall be clearly indicated on the design drawings.

G. 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.

H. 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.

I. Design Life. Design life shall be in accordance with AASHTO requirements or 75 years; the more stringent requirements apply.

J. 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 or 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 foot 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 foot 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:

Pay Item	Pay Unit
635.14 Prefabricated Concrete Modular Gravity Wall	Square Foot