MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

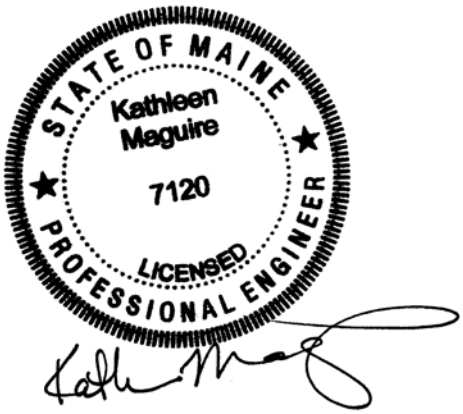
GEOTECHNICAL DESIGN RECOMMENDATIONS

For the Replacement of:

MILL BRIDGE
OVER ST. GEORGE RIVER
STATE ROUTES 131 AND 173
SEARSMONT, MAINE

Prepared by:

Kathleen Maguire, P.E.
Geotechnical Engineer



Reviewed by:

Laura Krusinski, P.E.
Senior Geotechnical Engineer

Waldo County
PIN 12637.00

Soils Report No. 2009-106
Bridge No. 2555

Fed No. AC-BH-1263(700)X
February 2, 2009

TEDOCS No. 933167

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
16 STATE HOUSE STATION
AUGUSTA, MAINE 04333-0016

MEMORANDUM

TO:	Robert S. Blunt, PE; Jim Wentworth, PE
CC:	TEDOCS
AUTHOR:	Kate Maguire, PE
DATE:	February 2, 2009
SUBJECT MATTER:	Subsurface Exploration Summary and Geotechnical Design Recommendations
PIN:	12637.00
PROJECT NAME:	Proposed Replacement of Mill Bridge (#2555) Over St. George River, Searsmont, Maine

INTRODUCTION

This memorandum provides the results of a subsurface exploration program and makes geotechnical design recommendations for the replacement of Mill Bridge which carries Routes 131 and 173 over the St. George River in Searsmont, Maine. See Sheet 1 – Location Map for site location. The purpose of the subsurface investigation was to identify soil and bedrock conditions in order to develop geotechnical design recommendations.

EXISTING SUBSTRUCTURES AND CONDITION

Mill Bridge was built in 1930 and is a 29 foot long, single span concrete T-beam superstructure supported on mass concrete abutments founded on bedrock. Maine Department of Transportation (MaineDOT) maintenance inspection reports indicate that the bridge deck is in "satisfactory" condition (rating of 6) the bridge superstructure is in "good" condition (rating of 7) and the substructure its in "poor" condition (rating of 4). Year 2007 MaineDOT Bridge Maintenance inspection reports indicate a Bridge Sufficiency Rating of 45.6. The inspection report notes that both abutments show scaling, spalling and leakage throughout. Some undermining of the abutments was noted. The bridge has a Scour Critical Rating of 8, meaning that the bridge foundations have been determined to be stable for the assessed or calculated scour condition.

PROPOSED STRUCTURE

The proposed replacement structure is a buried precast concrete arch with a 28 foot span, an 11 foot rise and a 20 degree skew. The arch will be founded on spread footings on bedrock. The proposed alignment is shifted 17 feet to the northwest of the existing alignment to accommodate a wider proposed bridge, improve site distances and allow for staged construction. Three of the wingwalls will be precast concrete walls. The fourth wall (on the southeast corner) will be a cast-in-place jacket rehabilitation of an existing wingwall which transitions into a precast concrete wall to retain the adjacent driveway to the south.

GEOLOGIC SETTING

According to the Reconnaissance Surficial Geologic Map of the Searsmont Quadrangle, Maine published by the Maine Geological Survey (1986) the surficial soils in the vicinity of the site consist of till, glaciomarine deposits and bedrock outcrops. Till soils in the site area are generally comprised of a mixture of sand, silt, clay and stones. The till generally overlies bedrock. The unit is commonly a blanket deposit that conforms to the bedrock surface. These soils were deposited directly by glacial ice. Immediately to the north of the site glaciomarine deposits are mapped. These soils are generally silts and clays of the Presumpscot Formation. Immediately to the south of the site an area of bedrock outcrop is mapped. This area is generally one of many bedrock outcrops with thin (<10 feet) surficial deposits.

According to the Surficial Bedrock Map of Maine, published by the Maine Geological Survey (1985), the site is located at the interface of bedrock identified as Devonian-Precambrian pelite of the Appleton Ridge Formation to the southeast and Devonian-Ordovician calcareous sandstone, interbedded sandstone and impure sandstone of the Bucksport Formation to the northwest.

SUBSURFACE INVESTIGATION

Subsurface conditions were explored by drilling two (2) test borings at the site. Test borings BB-SStGR-101 and BB-SStGR-102 were drilled behind the location of the existing eastern abutment. The exploration locations and boring logs are shown on Sheet 2 - Boring Location Plan and Boring Logs found at the end of this memorandum.

The borings were drilled between April 4 and 5, 2006 using the Maine Department of Transportation (MaineDOT) drill rig. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided at the end of this memorandum.

The borings were drilled using driven cased wash boring and solid stem auger techniques. Soil samples were obtained where possible at 5-foot intervals using Standard Penetration Test (SPT) methods. The bedrock was cored in the borings using an NQ core barrel. The MaineDOT Geotechnical Team member selected the boring locations and drilling

methods, designated type and depth of sampling techniques and identified field testing requirements. A Certified Subsurface Inspector logged the subsurface conditions encountered. The borings were located in the field by use of a tape after completion if the drilling program.

SUBSURFACE CONDITIONS

Subsurface conditions encountered at the test borings generally consisted of granular fill, (sands, silts, and cobbles and boulders) underlain by bedrock. The boring logs are provided at the end of this memorandum. The following paragraphs are a brief summary description of the strata encountered during exploration activities:

Fill. Beneath the pavement, fill was encountered in both of the borings. The layer was found to be approximately 13.0 feet thick in boring BB-SStGR-101 and approximately 12.2 feet thick in boring BB-SStGR-102. The fill generally consisted of layers of:

- Brown, damp, medium dense, fine to coarse sand with some gravel and trace silt,
- Brown, moist, stiff, silt with little fine to medium sand,
- Brown, damp, loose, silty fine sand with little medium sand,
- Gravel and Cobbles, and
- Sand and Gravel with frequent boulders.

SPT N-values in the fill ranged from 3 to 11 blows per foot (bpf) indicating that the fill is very loose to medium dense in consistency.

Bedrock. Bedrock was encountered and cored at a depth of approximately 13.0 feet below ground surface (bgs) and Elevation 201.40 feet in boring BB-SStGR-101 and a depth of approximately 12.2 feet bgs and Elevation 202.20 feet in boring BB-SStGR-102.

The bedrock is described as light grey, fine grained, SCHIST, moderately hard, moderately weathered, chaotic foliation, joints along bedding are tight to open. The rock quality designation (RQD) of the bedrock ranges from 30 to 72 percent indicating a rock mass quality of poor to fair.

FOUNDATION ALTERNATIVES AND RECOMMENDATIONS

Due to the presence of shallow bedrock at the site and the decision to replace the existing bridge with a buried structure, spread footings founded on bedrock are the most viable foundation alternative. The following paragraphs discuss the geotechnical recommendations for the project.

Frost Protection. It is anticipated that the buried structure footings and precast concrete walls will be founded on bedrock. Therefore, heave due to frost is not a design issue and no requirements for minimum depth of embedment are necessary.

In the event that any foundation is placed on granular subgrade soils it should be designed with an appropriate embedment for frost protection. According to the MaineDOT frost depth maps for the State of Maine (MaineDOT Bridge Design Guide [BDG] Figure 5-1), the site has a design-freezing index of approximately 1450 F-degree days. This correlates to a frost depth of 5.6 feet. Therefore, any foundations placed on granular soils should be founded a minimum of 5.6 feet below finished exterior grade for frost protection. This minimum embedment depth applies only to foundations placed on subgrade soils and not those founded on bedrock.

Bearing Resistance. It is anticipated that the buried structure footings and retaining walls will be founded on the bedrock surface. Applicable permanent and transient loads are specified in AASHTO LFRD Bridge Design Specifications 4th Edition (LRFD) Article 11.5.5. Buried structure footings and retaining walls shall be proportioned to provide stability against bearing capacity failure.

As the arch wall footings and retaining walls are to be supported on bedrock the vertical stress shall be calculated assuming a linearly distributed pressure over an effective base area as shown in LRFD Figure 11.6.3.2-2. Bearing resistance for any structure founded on bedrock shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 11 ksf for footings 7 feet wide or less and 15 ksf for footings 8 feet wide of more. The bearing resistance factor, ϕ_b, for spread footings on bedrock is 0.45. A factored bearing resistance of 20 ksf may be used when analyzing the service limit state and for preliminary footing sizing based on presumptive bearing resistance values. See the project calculations attached at the end of this memorandum for details.

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

Settlement. As the spread footings for the buried structure and the wingwalls will be founded on bedrock, settlements are anticipated to be negligible.

Arch Stem Wall and Footing Design. Arch stem walls and spread footings shall be proportioned for all applicable load combinations in LRFD Articles 3.4.1, 12.5 and 11.5.5 and shall be designed for all relevant strength and service limit states. In addition to the typical loads, spread footing loads should consider all reactions transferred to the footings through the arch walls. The design of arch stem walls founded on spread footings at the strength limit state shall consider nominal bearing resistance, eccentricity (overturning), lateral sliding and structural failure. In accordance with LRFD Article 12.5.5, the resistance factor values for geotechnical design of foundations for buried structures shall be as specified in LRFD Section 10, Foundations.

Design of buried structures shall also consider foundation resistance after scour due to the design flood. In accordance with LRFD Article 10.5.5.3.2 the foundation resistance after

scour shall be adequate to support the unfactored strength limit state loads with a resistance factor of 1.0.

A sliding resistance factor, ϕ_τ, of 0.9 shall be applied to the nominal sliding resistance of arch wall spread footings on bedrock. Sliding computations for resistance to lateral reactions from arch thrust shall assume a maximum frictional coefficient of 0.70 at the bedrock-concrete interface.

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

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

Arch stem walls shall be designed as restrained rigid frames. Earth pressures shall be calculated using an at-rest earth pressure coefficient, K_o, of 0.50. The designer may assume Soil Type 4 (MaineDOT BDG Section 3.6.1) for backfill material soil properties. The backfill properties are as follows: ϕ = 32 degrees, γ = 125 pcf.

Additional lateral earth pressure due to construction surcharge or live load surcharge is required per Section 3.6.8 of the MaineDOT BDG for the arch stem wall. The live load surcharge on arch stem walls may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) taken form the table below:

Arch Stem Wall Height (feet)	h_{eq} (feet)
5.0	4.0
10.0	3.0
≥20.0	2.0

Backfill within 10 feet of the arch walls shall conform to MaineDOT Specification 709.19, Granular Borrow for Underwater Backfill. This gradation specified 10 percent or less of material passing the No. 200 sieve. This material is specified in order to reduce the amount of fines and to minimize frost action behind the structure. The Contractor shall compact the backfill in the controlled backfill zone in accordance with the manufacturer's recommendations and the MaineDOT Standard Specifications but no less than 92% of the AASHTO T-180 maximum dry density.

Retaining Walls. Three of the wingwall retaining walls will be Precast Concrete Modular Gravity (PCMG) Walls. The fourth retaining wall (on the southeast corner) will be a cast-in-place jacket rehabilitation of an existing wingwall. The cast-in-place wall will transition into an L-shaped PCMG wall constructed to retain the driveway to the south of the bridge.

PCMG Wingwalls. The PCMG walls shall be designed in accordance with Special Provision 635 by a Professional Engineer subcontracted by the Contractor as a design-build item. Special Provision 635 is attached at the end of this memorandum. The PCMG walls shall be founded on bedrock.

The PCMG walls shall be designed for all relevant strength, service and extreme limit states and load combinations specified in LRFD Articles 3.4.1 and 11.5.5. The design of wingwalls at the strength limit state shall consider nominal bearing resistance, overturning, lateral sliding and structural failure. Strength limit state design shall also consider foundation resistance after scour due to the design flood. The PCMG walls shall be designed considering a traffic surcharge equal to 2 feet of fill placed on the backfill surface.

A resistance factor of ϕ= 1.0 shall be used to assess spread footing design at the service limit state including: settlement, horizontal movement, overall stability and scour at the design flood. Extreme limit state design checks for spread footings shall include bearing resistance, eccentricity, sliding and overall stability. A resistance factor of ϕ=1.0 shall be used for the extreme limit state. Furthermore, PCMG wall footings 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 PCMG wall system should be investigated at the Service I Load Combination with a resistance factor, ϕ, of 0.65.

The designer shall apply at sliding resistance factor, ϕ_τ, of 0.9 to the nominal sliding resistance of precast concrete wall units constructed on bedrock. For footings on bedrock, the eccentricity of loading as the strength limit state, based on factored loads, shall not exceed three-eights ($3/8^{ths}$) of the footing dimensions, in either direction. Sliding computations for resistance to lateral loads shall assume a maximum frictional coefficient of 0.56 (0.80 x tan 35º) at he foundation bedrock to precast concrete interfaces and a maximum frictional coefficient of 0.58(tan 30º) at he foundation rock to soil in-fill interfaces. Recommended values of sliding frictional coefficients are based on LRFD Articles 10.6.3.4, 11.11.4.2 and Table 3.11.5.3-1.

Any irregularities in the existing bedrock surface or irregularities created during the excavation process will be backfilled with un-reinforced Class S fill concrete to the bearing elevation.

The high water elevation shall be indicated on the retaining wall plans per the design requirements for hydrostatic conditions in Special Provision 635 - Prefabricated Bin Type Retaining Wall. The Special Provision reads: Hydrostatic forces - Unless specified otherwise, when a design high water surface is shown on the plans, the design stresses calculated from that elevation to the bottom of wall must include a 3 foot minimum

differential head of flow able fill or saturated backfill. In addition, the buoyant weight of saturated soil shall be used in the calculation of pullout resistance.

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

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

The Designer may assume Soil Type 4 (MaineDOT BDG Section 3.6.1) for backfill material soil properties. The backfill properties are as follows: ϕ = 32 degrees, γ = 125 pcf. Sliding computations for resistance to lateral loads shall assume a maximum allowable frictional coefficient of 0.70 at the bedrock-concrete interface. A sliding resistance factor of ϕ_τ=0.9 shall be applied to the nominal sliding resistance of walls founded on spread footings on bedrock.

The wingwalls will be independent of the buried structure and will be supported on spread footings. The design of walls founded on spread footings at the strength limit state shall consider nominal bearing resistance, overturning, lateral sliding and structural failure. Strength limit state design shall also consider foundation resistance after scour due to the design flood.

Drainage. The precast arch shall include a drainage system behind the arch stem walls to intercept any water. Drainage behind the structure shall be in accordance with Section 5.4.1.4 Drainage, of the MaineDOT BDG. French drains or geocomposite drainage board applied to the backsides of the arch stem walls with weep holes will provide adequate drainage.

Construction Considerations. Construction activities may include rock excavation, construction of cofferdams and earth support systems. Excavation of bedrock materials may require drilling and blasting techniques. Blasting should be done in accordance with Section 105.2.6 of the MaineDOT Standard Specifications. The Contractor may need to conduct pre-and post-blast surveys in accordance with industry standards. All loose rock and soil debris found at the substructure location shall be removed from bearing surfaces before concrete is placed for footings. The prepared bedrock surface should be benched to create level steps if the bedrock surface is steeper than 4H:1V in any direction. It is

likely that there will be seepage of water from fractures and joints exposed in the bedrock surface and cut slopes. Water should be controlled by pumping from sumps. The contractor should maintain the excavation so that all foundations are constructed in the dry.

Construction activities should not be permitted to disturb the rock mass or to create any rock falls or any open fissures. Any irregularities in the existing bedrock surface or irregularities created during the excavation process should be backfilled with un-reinforced concrete to the bearing elevation.

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

CLOSURE

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

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

SHEETS

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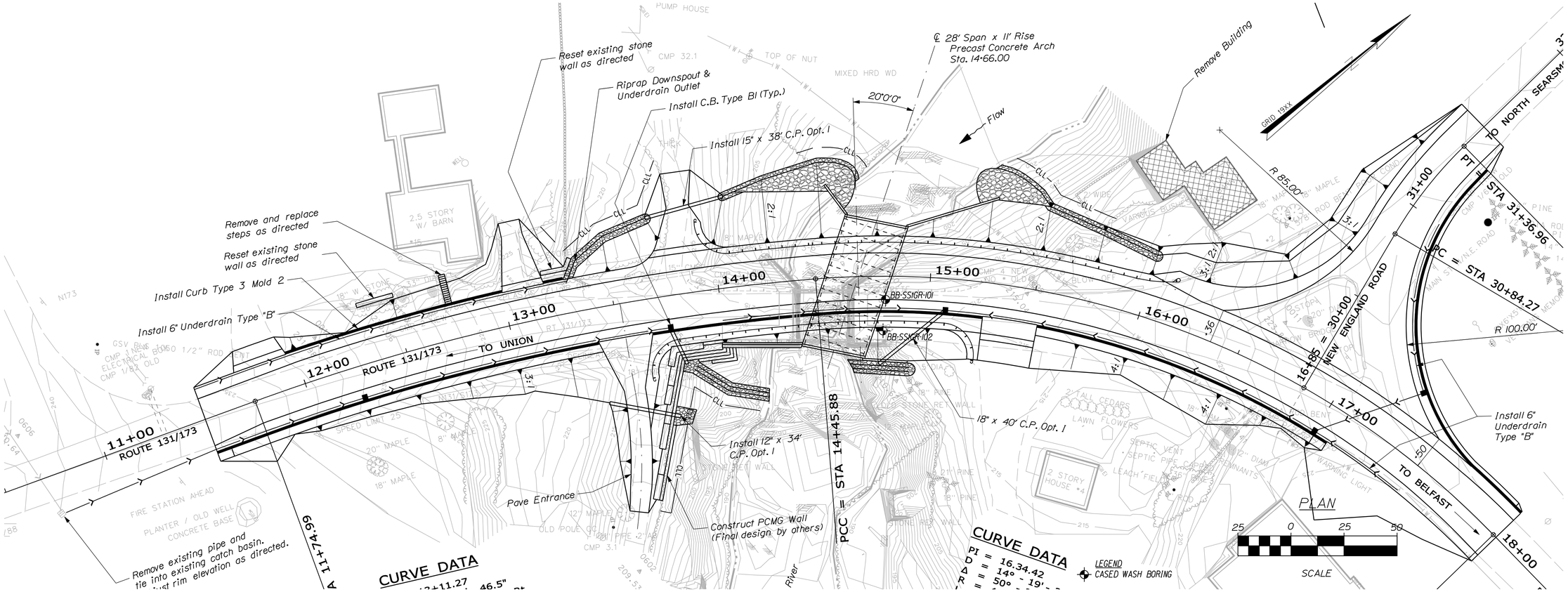
www.delorme.com

TN
MN (16.7°W)

Scale 1 : 25,000
1" = 2,083.3 ft
Data Zoom 13-0



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

Project: Mill Bridge #2555 over St. George River
Location: Searsmont, Maine

Boring No.: BB-SS1GR-101
PIN: 12637.00

Driller: MaineDOT	Elevation (ft.) 214.4	Auger ID/OD: 5" Solid Stem Auger	
Operator: E. Giguere/K. Speed	Datum: NAVD 88	Sampler: Standard Split Spoon	
Logged By: G. Lidstone	Rig Type: CME 45C	Hammer Wt./Fall: 140#/30"	
Date Start/Finish: 4/5/06-4/5/06	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"	
Boring Location: 14+79. 10.9 Rt.	Casing ID/OD: NW	Water Level*: None Observed	

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_{vu} = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{vu(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods WOC = weight of casing

Definitions:
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 (%)	Hammer	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
						SSA	213.80	Pavement 0.600	
								Brown, damp, medium dense, silty fine to coarse SAND, some gravel, (Fill). (1D/A) 2.0-3.0' bgs.	
	1D/AB	24/12	2.00 - 4.00	6/8/3/3	11		211.40	3.000	
								(1D/B) 3.0-4.0' bgs. Brown, moist, stiff, SILT, little fine to medium sand. (Fill).	
							210.40	4.000	
5								Brown, damp, loose, silty fine SAND, little medium sand. (Fill).	
	2D	24/11	5.00 - 7.00	4/3/2/2	5	SPLIT NW			
10	MD	24/0	10.00 - 12.00	6/1/2/8	3			Similar to above, but very loose.	
							202.40	12.000	
	R1	67.2/57	12.00 - 17.60	RQD = N/A%		MD	201.40	R1: 12.0-13.0' bgs mixed with Cobbles and Soil. R1: Core Times (minisec) (12.0-13.0' (1:30)) 13.000	
15								R1: Bedrock: Light grey, fine grained, SCHIST, moderately hard, moderately weathered, chaotic foliation, joints along bedding and tight to slightly open. R1: Core Times (minisec) 13.0-14.0' (2:15) 14.0-15.0' (2:30) 15.0-16.0' (2:40) 16.0-17.0' (2:00) 17.0-17.6' (1:50) 85% Recovery	
	R2	60/60	17.60 - 22.60	RQD = 30%				R2: Core Times (minisec) 17.6-18.6' (2:08) 18.6-19.6' (1:45) 19.6-20.6' (1:30) 20.6-21.6' (1:32) 21.6-22.6' (2:30) 100% Recovery	
20							191.80	22.600	
								Bottom of Exploration at 22.60 feet below ground surface.	
25									

Remarks:

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

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

Page 1 of 1 — Boring No.: BB-SS1GR-101

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

Project: Mill Bridge #2555 over St. George River
Location: Searsmont, Maine

Boring No.: BB-SS1GR-102
PIN: 12637.00

Driller: MaineDOT	Elevation (ft.) 214.4	Auger ID/OD: 5" Solid Stem Auger	
Operator: E. Giguere/K. Speed	Datum: NAVD 88	Sampler: Standard Split Spoon	
Logged By: G. Lidstone	Rig Type: CME 45C	Hammer Wt./Fall: 140#/30"	
Date Start/Finish: 4/4/06-4/5/06	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"	
Boring Location: 14+73. 25.7 Rt.	Casing ID/OD: NW	Water Level*: None Observed	

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_{vu} = Insitu Field Vane Shear Strength (psf)
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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (#6 in) Shear Strength (psf) or RQD (%)	Hammer	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
						SSA	213.60	Pavement 0.750	
								Brown, damp, fine to coarse SAND, some gravel, trace silt. (Fill).	
	1D	13.2/7	2.00 - 3.10	6/10/25(1.2")	---		211.30	3.100	
	R1	2.4/2.4	3.40 - 3.50			SPLIT NW	210.50	GRAVEL and COBBLES Spun Casing to 7.0' bgs, then Roller Coned ahead to 7.2' bgs.	
	R2	10.8/7	3.50 - 4.50				210.30	NQ-2" Core R1-R3 (3.4-6.7' bgs).	
	R3	26.4/18	4.50 - 6.70					SAND and GRAVEL 4.900	
5								GRAVEL and COBBLES	
	R4	62.4/59	7.20 - 12.40			OPEN HOLE		SAND and GRAVEL, with frequent boulders Spun Casing to 7.4' bgs. Roller Coned ahead to 22.4' bgs. R4: Core Times (minisec) 7.2-8.2' (4:30) 8.2-9.2' (3:30) 9.2-10.2' (3:00) 10.2-11.2' (2:00) 11.2-12.2' (2:30) 12.2-12.4' (0:45) 95% Recovery Boulder from 10.6-12' bgs	
10									
	R5	60/59	12.40 - 17.40	RQD = 48%		NW CASING	202.20	12.200	
								Bedrock: Light grey, fine grained, SCHIST, moderately hard, moderately weathered, chaotic foliation, joints along bedding and tight to slightly open. R5: Core Times (minisec) 12.4-13.4' (3:30) 13.4-14.4' (3:50) 14.4-15.4' (2:00) 15.4-16.4' (2:00) 16.4-17.4' (3:00) 98% Recovery	
15									
	R6	60/57	17.40 - 22.40	RQD = 72%				R6: Bedrock: R6: Core Times (minisec) 17.4-18.4' (3:10) 18.4-19.4' (2:50) 19.4-20.4' (2:00) 20.4-21.4' (2:10) 21.4-22.4' (2:00) 95% Recovery	
20							192.00	22.400	
								Bottom of Exploration at 22.40 feet below ground surface.	
25									

Remarks:
4/4/06: 08:15-11:00. (Rained Out)
4/5/06: 08:00-11:30. Started at 7.2' bgs.

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-SS1GR-102

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

AC-BH-1263(700)X

PIN 12637.00
BRIDGE NO. 2555
BRIDGE PLANS

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

SIGNATURE
P.E. NUMBER
DATE

MILL BRIDGE
ST. GEORGE RIVER
SEARSMONT WALDO COUNTY

BORING LOCATION PLAN & BORING LOGS

SHEET NUMBER
2
OF 2

Filename: ...\MSTA\006_BLP&BORING LOGS1.dgn Username: Kate.maguire Date:2/2/2009 Division: GEOTECH

BORING LOGS

UNIFIED SOIL CLASSIFICATION SYSTEM

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

Desired Soil Observations: (in this order)

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

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

TERMS DESCRIBING DENSITY/CONSISTENCY

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

Modified Burmister System

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

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

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

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

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: Mill Bridge #2555 over St. George River Routes 131/173
Location: Searsmont, Maine

Boring No.: BB-SStGR-101
PIN: 12637.00

Driller:	MaineDOT	**Elevation (ft.)**	214.4	**Auger ID/OD:**	5" Solid Stem Auger
Operator:	E. Giguere/K. Speed	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	G. Lidstone	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/5/06-4/5/06	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	14+79, 10.9 Rt.	**Casing ID/OD:**	NW	**Water Level*:**	None Observed

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods WOC = weight of casing

Definitions:
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-value	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA	213.8	Pavement — 0.6	
								Brown, damp, medium dense, silty fine to coarse SAND, some gravel, (Fill).	
	1D/AB	24/12	2.0 - 4.0	6/8/3/3	11			(1D/A) 2.0-3.0' bgs.	
							211.4	3.0	
								(1D/B) 3.0-4.0' bgs.	
							210.4	Brown, moist, stiff, SILT, little fine to medium sand, (Fill). — 4.0	
5	2D	24/11	5.0 - 7.0	4/3/2/2	5	SPUN NW		Brown, damp, loose, silty fine SAND, little medium sand, (Fill).	
10	MD	24/0	10.0 - 12.0	6/1/2/8	3			Similar to above, but very loose.	
							202.4	12.0	
	R1	67.2/57	12.0 - 17.6	RQD = N/A%		NQ		R1: 12.0-13.0' bgs mixed with Cobbles and Soil.	
							201.4	R1: Core Times (min:sec) 12.0-13.0' (1:30) — 13.0	
15								R1: Bedrock: Light grey, fine grained, SCHIST, moderately hard, moderately weathered, chaotic foliation, joints along bedding and tight to slightly open. R1: Core Times (min:sec) 13.0-14.0' (2:15) 14.0-15.0' (2:30) 15.0-16.0' (2:40) 16.0-17.0' (2:00) 17.0-17.6' (1:50) 85% Recovery	
	R2	60/60	17.6 - 22.6	RQD = 30%				R2: Core Times (min:sec) 17.6-18.6' (2:08) 18.6-19.6' (1:45) 19.6-20.6' (1:30) 20.6-21.6' (1:32) 21.6-22.6' (2:30) 100% Recovery	
20									
							191.8	22.6	
								Bottom of Exploration at 22.60 feet below ground surface.	
25									

Remarks:

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

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

Boring No.: BB-SStGR-101

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Mill Bridge #2555 over St. George River Routes 131/173
Location: Searsmont, Maine

Boring No.: BB-SStGR-102
PIN: 12637.00

Driller:	MaineDOT	**Elevation (ft.)**	214.4	**Auger ID/OD:**	5" Solid Stem Auger
Operator:	E. Giguere/K. Speed	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	G. Lidstone	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/4/06-4/5/06	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	14+77.7, 25.7 Rt.	**Casing ID/OD:**	NW	**Water Level*:**	None Observed

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods WOC = weight of casing

Definitions:
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA	213.7		Pavement — 0.8	
									Brown, damp, fine to coarse SAND, some gravel, trace silt, (Fill).	
	1D	13.2/7	2.0 - 3.1	6/10/25(1.2")	---					
							211.3		3.1	
	R1	2.4/2.4	3.4 - 3.6			SPUN NW			GRAVEL and COBBLES	
	R2	10.8/7	3.6 - 4.5				210.5		Spun Casing to 7.0' bgs, then Roller Coned ahead to 7.2' bgs.	
	R3	26.4/18	4.5 - 6.7				210.3		[a]NQ-2" Core R1-R3 (3.4-6.7' bgs).	
5									3.9	
									SAND and GRAVEL	
									4.1	
									GRAVEL and COBBLES	
									SAND and GRAVEL, with frequent boulders Boulder from 6.9-10.4' bgs [a]NQ-2" Core R4-R6 (7.2-22.4' bgs).	
	R4	62.4/59	7.2 - 12.4			OPEN HOLE			R4: Core Times (min:sec) 7.2-8.2' (4:30) 8.2-9.2' (3:30) 9.2-10.2' (3:00) 10.2-11.2' (2:00) 11.2-12.2' (2:30) 12.2-12.4' (0:45) 95% Recovery Boulder from 10.6-12' bgs	
10										
	R5	60/59	12.4 - 17.4	RQD = 48%		[a]NQ CORE	202.2		12.2	
									Bedrock: Light grey, fine grained, SCHIST, moderately hard, moderately weathered, chaotic foliation, joints along bedding and tight to slightly open. R5: Core Times (min:sec) 12.4-13.4' (3:30) 13.4-14.4' (3:50) 14.4-15.4' (2:30) 15.4-16.4' (2:00) 16.4-17.4' (3:00) 98% Recovery	
15										
	R6	60/57	17.4 - 22.4	RQD = 72%					R6: Bedrock: R6: Core Times (min:sec) 17.4-18.4' (3:10) 18.4-19.4' (2:50) 19.4-20.4' (2:00) 20.4-21.4' (2:10) 21.4-22.4' (2:00) 95% Recovery	
20										
							192.0		22.4	
									Bottom of Exploration at 22.40 feet below ground surface.	
25										

Remarks:

4/4/06; 08:15-11:00, (Rained Out)
4/5/06: 08:00-11:30, Started at 7.2' bgs.

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.

CALCULATIONS

Frost Protection:

Method 1 - MaineDOT Design Freezing Index (DFI) Map and Depth of Frost Penetration Table are in BDG Section 5.2.1.

From the Design Freezing Index Map:
Searsmont, Maine
DFI = 1450 degree-days

Soils are coarse grained, assume a water content = ~20%

By interpolation:
From Table 5-1 MaineDOT BDG for Design Freezing Index of 1450 frost penetration = 66.7 inches

$Frost_depth := 66.7in$ $\quad$ $Frost_depth = 5.558 \cdot ft$

Method 2 - Check Frost Depth using Modberg Software

Closest Station is Belfast

ModBerg Results

Project Location: Belfast, Maine

Air Design Freezing Index	=	1188 F-days
N-Factor	=	0.80
Surface Design Freezing Index	=	950 F-days
Mean Annual Temperature	=	45.5 deg F
Design Length of Freezing Season	=	118 days

Layer #:Type	t	w%	d	Cf	Cu	Kf	Ku	L
1-Coarse	67.5	20.0	125.0	34	46	3.8	1.9	3,600

t = Layer thickness, in inches.
w% = Moisture content, in percentage of dry density.
d = Dry density, in lbs/cubic ft.
Cf = Heat Capacity of frozen phase, in BTU/(cubic ft degree F).
Cu = Heat Capacity of thawed phase, in BTU/(cubic ft degree F).
Kf = Thermal conductivity in frozen phase, in BTU/(ft hr degree).
Ku = Thermal conductivity in thawed phase, in BTU/(ft hr degree).
L = Latent heat of fusion, in BTU / cubic ft.

****
Total Depth of Frost Penetration = 5.63 ft = 67.5 in.
****

Use BDG Calc Frost Depth = 5.6 feet for design

Bearing Resistance - BEDROCK:

Part 1 - Service Limit State

Nominal and Factored Bearing Resistance - Spread footing on bedrock

Presumptive Bearing Resistance for Service Limit State ONLY

Bedrock at the site is Schist.

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

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

Consistency In Place: Medium hard, rock

Bearing Resistance: Ordinary Range (ksf) 16 - 24

Recommended Value of Use (ksf): 20 ksf

Based on RQD values ranging from 30% to 72%

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

Part 2 - Strength Limit State

Nominal and Factored Bearing Resistance - spread footing on bedrock

Nominal Bearing Resistance for Strength Limit State

Bedrock at the site is Schist which is "poor to fair" in quality.
RQD = 30 to 72%

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

Due to competency of bedrock (RQD 30 to 72%), RMR method is not required.

From Foundation Analysis and Design by JE Bowles Fifth Edition

Section 4-16 pg 277 Bearing Capacity of Rock

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

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

Bearing Capacity factors by Stagg and Zienkiewicz 1968

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

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

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

Terzaghi Shape factors from Bowles Table 4-1 pg 220

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

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

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

$$B := \begin{pmatrix} 6 \\ 8 \\ 10 \\ 12 \\ 15 \end{pmatrix} \cdot ft$$ Look at several footing widths

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

$$q_{ult} = \begin{pmatrix} 99 \\ 131 \\ 164 \\ 197 \\ 246 \end{pmatrix} \cdot ksf$$

Reduce ultimate bearing based on average RQD = 50%

$$q_{reduced} := q_{ult} \cdot 0.50^2 \qquad q_{reduced} = \begin{pmatrix} 25 \\ 33 \\ 41 \\ 49 \\ 62 \end{pmatrix} \cdot ksf$$

Assume this ultimate load is a nominal load. Resistance factor Φ = 0.45 from LRFD Table 10.5.5.2.2-1

$$q_{factored} := q_{reduced} \cdot 0.45 \qquad q_{factored} = \begin{pmatrix} 11.1 \\ 14.8 \\ 18.5 \\ 22.2 \\ 27.7 \end{pmatrix} \cdot ksf$$

Footing Width: 6 feet
Footing Width: 8 feet
Footing Width: 10 feet
Footing Width: 12 feet
Footing Width: 15 feet

At the Strength Limit State:

Recommend a limiting factored bearing resistance of 11 ksf for footings 7 feet wide or less on bedrock
Recommend a limiting factored bearing resistance of 15 ksf for footings 8 feet wide or wider on bedrock

Passive and Active Earth Pressure:

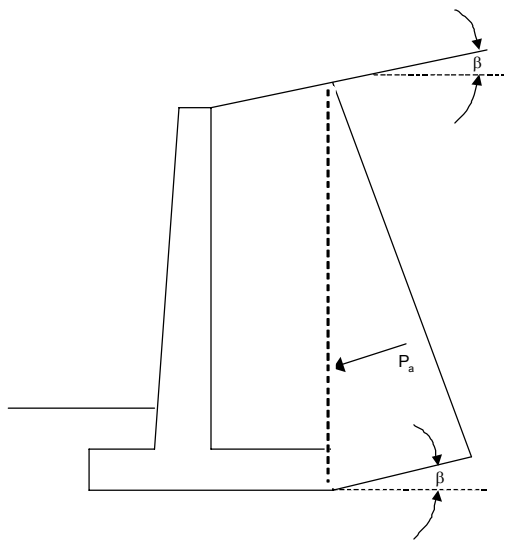
Soil Type 4 Properties from MaineDOT Bridge Design Guide (BDG)

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

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

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

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



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

For cantilever walls with horizontal backfill surface:

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

For cantilever walls with sloped backfill surface:

β = Angel of fill slope to the horizontal

$\beta := 0 \cdot deg$

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

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

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

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

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

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

Choosing Friction Angle between fill and wall:

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

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

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

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

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

$K_{a_coulomb} = 0.276$

Orientation of Coulomb Pa :

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

At-Rest Lateral Earth Pressure
from LRFD Article 3.11.5.2 pg 3-71

Effective friction angle of soil $\phi_f := 30 \cdot deg$

$K_o := 1 - \sin(\phi_f)$ $K_o = 0.5$

SPECIAL PROVISION 635

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 inch wide, by 0.5 inch thick preformed expansion joint filler shall be placed in all horizontal joints between facing units. In all vertical joints, a space of 0.25 inch shall be provided. All Preformed Expansion Joint Material shall meet the requirements of subsection 502.03.

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

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

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

Backfill and Bedding Material. Bedding and backfill material placed behind and within the reinforced concrete modules shall be gravel borrow conforming to the requirements of Subsection 703.20. The backfill materials shall conform to the following additional requirements: 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 and Extreme Limit States. Thirty days prior to beginning construction of the wall, the design computations shall be submitted to the Resident for review by the Department. Design calculations that consist of computer generated output shall be supplemented with at least one hand calculation and graphic demonstrating the design methodology used. Design calculations shall provide thorough documentation of the sources of equations used and material properties. The design by the wall system supplier shall consider the stability of the wall as outlined below:

A. Stability Analysis:

1. Overturning: Location of the resultant of the reaction forces shall be within the middle one-half of the base width.

2. Sliding: $R_R \geq \gamma_{p(max)} \cdot (EH+ES)$

   Where: R_R = Factored Sliding Resistance

   $\gamma_{p(max)}$ = Maximum Load Factor

   EH = Horizontal Earth Pressure

   ES = Earth Surcharge (as applicable)

4. Bearing Pressure: $q_R \geq$ Factored Bearing Pressure

   Where: q_R = Factored Bearing Resistance, as shown on the plans

   Factored Bearing Pressure = Determined considering the applicable loads and load factors which result in the maximum calculated bearing pressure.

5. Pullout Resistance: Pullout resistance shall be determined using nominal resistances and forces. The ratio of the sum of the nominal resistances to the sum of the nominal forces shall be greater than, or equal to, 1.5.

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. External loads which affect the internal stability such as those applied through piling, bridge footings, traffic, slope surcharge, hydrostatic and seismic loads shall be accounted for in the design.

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

F. Stability During Construction. Stability during construction shall be considered during design, and shall meet the requirements of the AASHTO LRFD Bridge Design Specifications, Extreme Limit State.

G. Hydrostatic forces. Unless specified otherwise, when a design high water surface is shown on the plans at the face of the wall, the design stresses calculated from that elevation to the bottom of wall must include a 3 feet minimum differential head of saturated backfill. In addition, the buoyant weight of saturated soil shall be used in the calculation of pullout resistance.

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

I. Not more than two vertically consecutive units shall have the same stem length, or the same unit depth. Walls with units with extended height curbs shall be designed for the added earth pressure. A separate computation for pullout of each unit with extended height curbs, or extended height coping, shall be prepared and submitted in the design package described above.

<u>635.04 Submittals</u>. The Contractor shall supply wall design computations, wall details, dimensions, quantities, and cross sections necessary to construct the wall. Thirty (30) days prior to beginning construction of the wall, the design computations and wall details shall be submitted to the Resident for review. The fully detailed plans shall be prepared in conformance with Subsection 105.7 of the Standard Specifications and shall include, but not be limited to the following items:

A. A plan and elevation sheet or sheets for each wall, containing the following: elevations at the top of leveling pads, the distance along the face of the wall to all steps in the leveling pads, the designation as to the type of prefabricated module, the distance along the face of the wall to where changes in length of the units occur, the location of the original and final ground line.

B. All details, including reinforcing bar bending details, shall be provided. Bar bending details shall be in accordance with Department standards.

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

D. All prefabricated modules shall be detailed. The details shall show all dimensions necessary to construct the element, and all reinforcing steel in the element.

E. The wall plans shall be prepared and stamped by a Professional Engineer. Four sets of design drawings and detail design computations shall be submitted to the Resident.

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

635.05 Construction Requirements

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

Foundation. The area upon which the modular gravity wall structure is to rest, and within the limits shown on the submitted plans, shall be graded for a width equal to, or exceeding, the length of the module. Prior to wall and leveling pad construction, this foundation material shall be compacted to at least 95 percent of maximum laboratory dry density, determined using AASHTO T180, Method C or D. Frozen soils and soils unsuitable or incapable of sustaining the required compaction, shall be removed and replaced.

A concrete leveling pad shall be constructed as indicated on the plans. The leveling pad shall be cast to the design elevations as shown on the plans, or as required by the wall supplier upon written approval of the Resident. Allowable elevation tolerances are +0.01 feet and -0.02 feet from the design elevations. Leveling pads which do not meet this requirement shall be repaired or replaced as directed by the Resident at no additional cost to the Department. Placement of wall units may begin after 24 hours curing time of the concrete leveling pad.

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

Installation of Wall Units. A field representative from the wall system being used shall be available, as needed, during the erection of the wall. The services of the representative shall be at no additional cost to the Department. Vertical and horizontal joint fillers shall be installed as shown on the plans.

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

Select Backfill Placement. Backfill placement shall closely follow the erection of each row of prefabricated wall units. The Contractor shall decrease the lift thickness if necessary to obtain the specified density. The maximum lift thickness shall be 8 inches (loose). Gravel borrow backfill shall be compacted in accordance with Subsection 203.12 except that the minimum required compaction shall be 92 percent of maximum density as determined by AASHTO T180 Method C or D. Backfill compaction shall be accomplished without disturbance or displacement of the wall units. Sheepsfoot rollers will not be allowed. Whenever a compaction test fails, no additional backfill shall be placed over the area until the lift is recompacted and a passing test achieved.

The moisture content of the backfill material prior to and during compaction shall be uniform throughout each layer. Backfill material shall have a placement moisture content less than or equal to the optimum moisture content. Backfill material with a placement moisture content in excess of the optimum moisture content shall be removed and reworked until the moisture content is uniform and acceptable throughout the entire lift. The optimum moisture content shall be determined in accordance with AASHTO T180, Method C or D. At the end of the day's operations, the Contractor shall shape the last level of backfill so as to direct runoff of rain water away from the wall face.

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

635.07 Basis of Payment. The accepted quantity of Prefabricated Concrete Modular Gravity Retaining Wall will be paid for at the contract unit price per square meter complete in place. Payment shall be full compensation for furnishing all labor, equipment and materials including excavation, foundation material, backfill material, pre-cast concrete units hardware, joint fillers, woven drainage geotextile, cast-in-place coping or traffic barrier and technical field representative. Cost of cast-in-place concrete for leveling pad will not be paid for separately, but will be considered incidental to the Prefabricated Concrete Modular Gravity Wall.

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

Payment will be made under:

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