MAINE DEPARTMENT OF TRANSPORTATION
HIGHWAY/BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

ROUTE 2 RETAINING WALL
SKOWHEGAN, MAINE

Prepared by:
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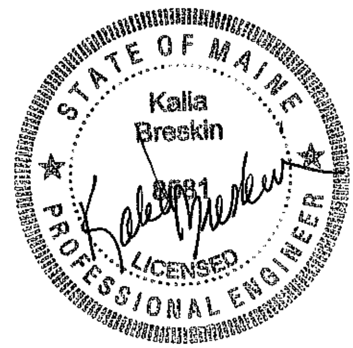
Senior Geotechnical Engineer



Reviewed by:
Karen Gross
Geotechnical Engineer

Penobscot County

Soils Report No. 2013-13

WIN 19153.00

March 5, 2013

DESIGN SUMMARY

The purpose of this report is to present subsurface information and make geotechnical recommendations for the replacement of a stone wall in Skowhegan which retains US Route 2 at a height of 6-feet above the Kennebec River. Sheet 1, the location map, shows the location of the wall. This wall appears to have been built before the 1954 reconstruction of Route 2 in Skowhegan. The wall is west of the intersection of Route 201, and approximately 600-feet upstream from a dam on the River. The speed limit is 25 mph in this location, and the wall is on the inside of a curve, behind a curb and sidewalk. There is presently no guardrail above the wall between Route 2 and the river, and stone posts of the pedestrian fence have fallen in many places. Sheets 2 and 3 show photographs of the wall taken in 2008 and 2010. The eastern half of the wall in this area has tipped outward, indicating instability. The western portion of this wall appears to have been rebuilt by MDOT Maintenance staff since the original construction, however MaineDOT has no records of the work. The altered portion of the wall will not be rebuilt as part of this project. The portion of the wall to be rebuilt extends from Station 213+07 to Station 215+25 at the eastern end of the existing wall, for a length of 218 feet.

Route 2 is a Priority 1 Highway Corridor. The existing highway in this area has 12-foot lanes and 8-foot shoulders. The distance from the existing curb to the face of the wall is approximately 8-feet in most places. In this construction, shoulders and sidewalks will be narrowed to 5-feet to allow the installation of a guardrail behind the sidewalk.

The existing wall appears to have been built on a stone base, and riprap covers the slope between the wall and the river. It is estimated that the base of the stone wall must be at least 6-feet wide for it to have had adequate stability to have lasted since construction.

A pedestrian fence will be built on top of the wall at a distance greater than 3-feet from the face of the guardrail. A reinforced concrete cap will be constructed on top of both existing and new walls to support the new pedestrian fence. The cap will be a uniform thickness over the existing wall, but both thickness and reinforcement will vary to match the elevation at the back of the sidewalk on the new section of wall.

Twenty-four stone fence posts remain on the portion of wall that will not be rebuilt; they will be removed for construction of the concrete cap described above, so that a fence meeting current code requirements can be built. In the area of new wall construction, the existing stone wall will be removed to approximately elevation 155.3-feet for construction of a concrete leveling pad for the new portion of retaining wall.

The water level in the Kennebec River below this wall is controlled by the dam downstream, with a minimum elevation of 155.3 feet according to the FERC permit held by Florida Power and Light. All construction will be above this elevation, and much of the leveling pad will be built on the lower portion of the existing stone wall in order to stay above the FERC water level. It is common for the dam operator to drop the water level in the late summer and fall to allow for maintenance activities, and this construction will be done while the water is low to prevent the need for a cofferdam for this construction.

Subsurface Investigation – A single boring to determine soil properties was drilled in February, 2011, at the edge of the existing roadway. A solid stem auger was used, and Standard Penetration Testing was done.

Existing Soils - Soils encountered included medium dense sands, with small amounts if silt and trace gravel. Wet soils were found 6.8-feet below ground surface, at Station 213+42, 15.7 feet Left. These appear to be the fill soils used when the existing wall was built. The boring was terminated 12-feet below ground surface; no refusal was encountered. Laboratory testing was not done on these samples.

Retaining Wall Design and Construction – A retaining wall meeting the requirements of Special Provision 635, Precast Concrete Block Gravity Wall will be built. The retaining wall has been designed by MaineDOT to support a 125 psf pedestrian load. The impact load from a vehicle collision with the guardrail was not considered in the design. There is no crash history in this location, and if the guardrail were to experience a direct impact, maintenance or repair of a portion of the wall may be needed. The new leveling pad will be built on the lower portion of the existing stone wall to stay above river level.

The manufacturer's standard 28-inch deep blocks will be used to allow space for installation of a guardrail behind the sidewalk. The face of curb will be 17-feet Left, the back of the guardrail posts will be 24-feet left, and the back of the upper 28" block will be approximately 25.25-feet left. Care will be needed during construction layout of the wall and guardrail to make sure this space is available. The block wall will have a maximum height of 6-feet.

Reinforcement geogrid will be placed at heights of 1.5-feet, 3-feet and 4.5-feet above the base of the wall. From Station 215+00to Station 215+25 where the cap thickness allows this, "middle" blocks should be used in place of the standard "top" block and reinforcement geotextile should be used above this block. The reinforcement geogrid will be embedded for a length of 6-feet measured from the face of the block. Upper layers of geogrid will be pierced by the guardrail posts, and added length has been included for loss of strength from guardrail installation.

The manufacturer's "Cobblestone" face pattern will be required on the blocks. The blocks should be tinted grey to minimize the visual difference between the existing stone wall and the new blocks.

Retaining Wall Cap and Fence – An 18-inch wide cap will be built on top of the new and existing walls to support a pedestrian fence. The cap will be a uniform thickness of 3-inches on top of the existing stone cap. The bulk of the cap on the new wall will be in the area of the top block intended for landscaping. This portion of the cap will have varying thicknesses to follow the profile at the back of the sidewalk. The height of the cap above the face of the wall is given in the following table; five additional inches of concrete will fill space in the top retaining wall blocks. A detail showing the cap is included as Sheet 5.

Station	Elevation at face of wall	Thickness at face of cap (inches)
213+25	161.85	3.6
213+50	161.74	2.3
213+75	161.70	1.8
214+00	161.65	1.2
214+25	161.72	2.1
214+50	161.86	3.8
214+75	162.22	8.1
215+00	162.57	12.3
215+25	162.99	17.3

Two longitudinal #4 steel reinforcing bars with 2-inches of cover at the top, spaced well apart will be required from the beginning of the wall to Station 214+70. Beyond this Station, an additional layer of steel will be required. This section will have two #4 bars near the bottom of the cap and two #4 bars with 2-inches of cover, transverse bars will be required spaced 2-feet on center.

Fence posts will be drilled and grouted for a depth of 12-inches into the cap and stones at the top of the existing wall, or into the cap and top retaining wall blocks as needed. These holes will be approximately 6-inches in diameter.

Construction Considerations – Contractor should coordinate work with the owners of the dam downstream to ensure that the work can be done while the water is low. Care will be needed during excavation of the existing wall to protect the lower wall that is to be used as a base for the new construction. Limited space is available for guardrail installation, so accurate layout of both the wall and the guardrail will be essential. Reinforcement steel within the concrete cap should be spaced to avoid fence post locations.

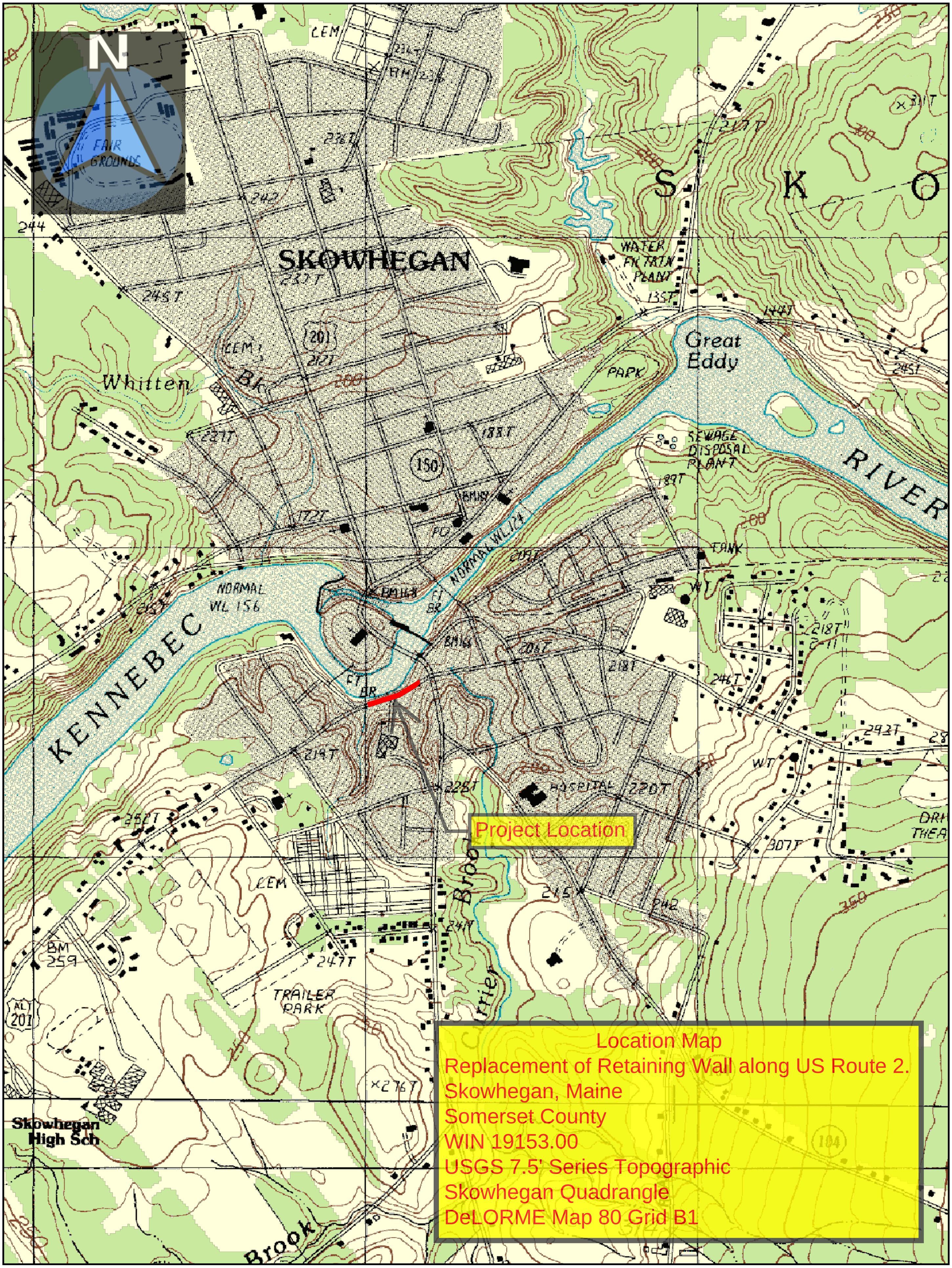
Attachments

Sheets

Sheet 1 - Location Map
Sheets 2 and 3 - Photographs
Sheet 4 - Boring Location Plan
Sheet 5 – Fence Post and Wall Cap Details

Appendices

Appendix A - Boring Log
Appendix B – Calculations
Appendix C – Retaining Wall Design



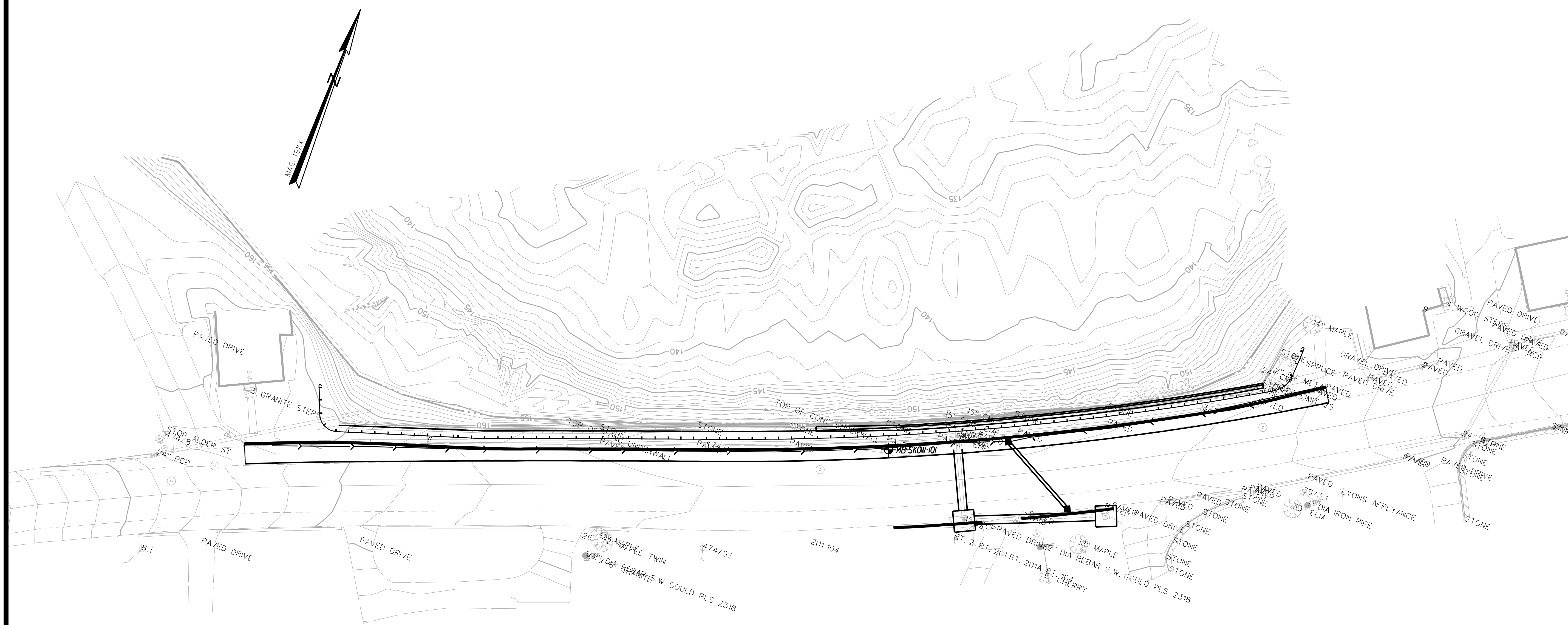
Map Scale 1:16000

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.

Sheet 2

Wall condition in 2008


Sheet 3
Wall condition in 2010
DEC 20 2010



LEGEND

SOLID STEM AUGER W/DESCRIPTIONS

PLAN

25 0 25 50

Scale in Feet

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

019153.00

WIN
19153.00

HIGHWAY PLANS

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED		
CHECKED-REVIEWED		
DESIGN2-DETAILED2	K.BRESKIN	
DESIGN3-DETAILED3	T.WHITE	FEB 2013
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

SKOWHEGAN
ROUTE 2 RETAINING WALL

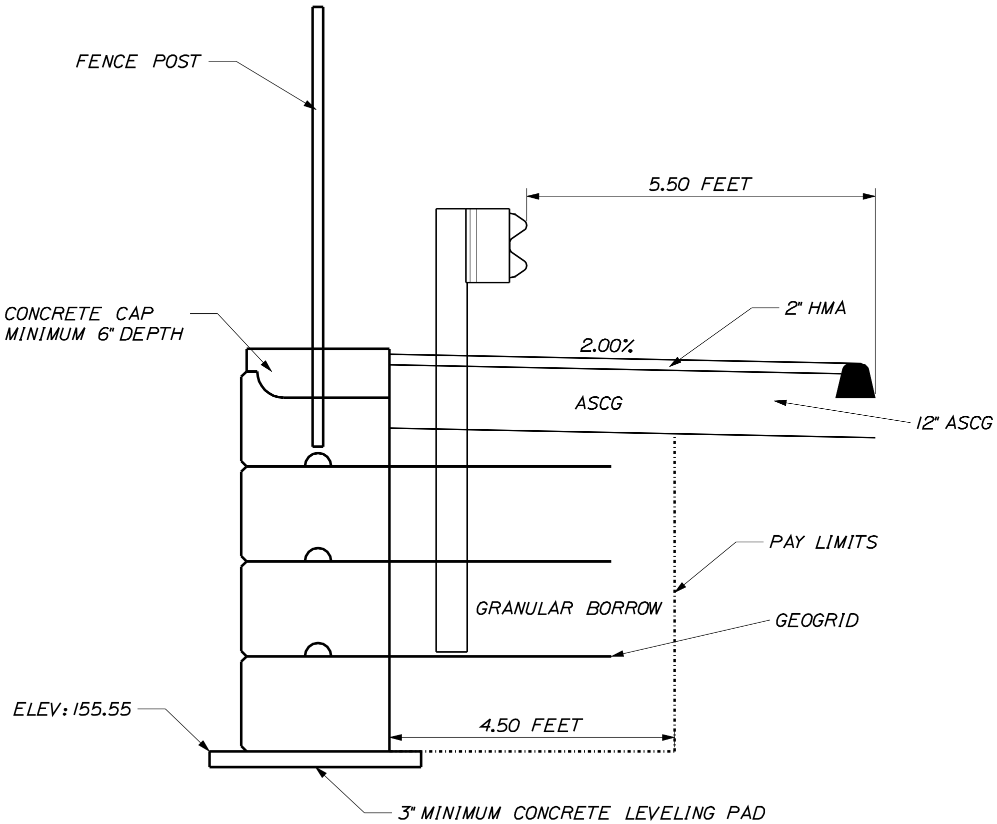
GEOPLANS

SHEET NUMBER

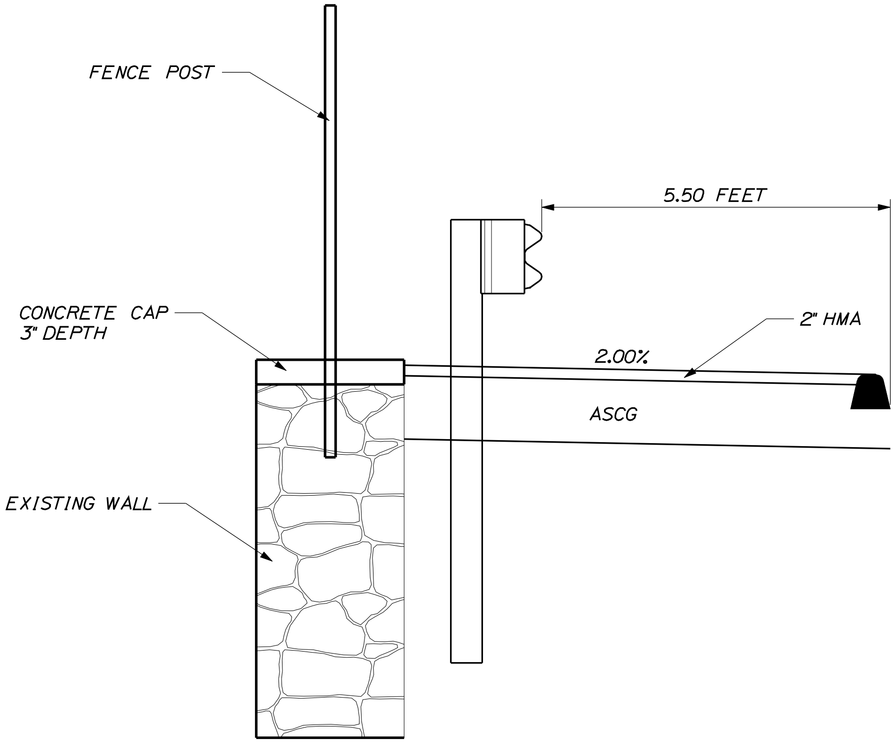
4

OF 5

Filename: ...\GEOTECH\MSTA\001_Geoplan1.dgn Division: GEOTECH Username: kitty.breskin Date:3/5/2013



CONCRETE BLOCK WALL DETAIL



EXISTING WALL DETAIL

Sheet 5 - Fence and Cap Detail

Appendix A
Boring Log

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Retaining Wall Investagation
Location: Skowhegan, Maine

Boring No.: HB-SKOW-101
PIN: 19153.00

Driller:	MaineDOT	**Elevation (ft.)**	161.0	**Auger ID/OD:**	5" Dia.
Operator:	Giguere/Giles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	K. Breskin	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	2/24/2011-2/24/2011	**Drilling Method:**	Solid Stem Auger	**Core Barrel:**	N/A
Boring Location:	213+42, 15.7 ft Lt.	**Casing ID/OD:**	N/A	**Water Level*:**	8.2 ft bgs.
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA			8" PAVEMENT.	
								160.33		Fine to coarse SAND, some gravel, off auger flight. (0.67)	
								158.50		(2.50)	
										Frost depth at 3.5 ft bgs.	
5	1D	24/12	5.00 - 7.00	9/10/15/8	25	35				Brown, damp, medium dense, SAND, little silt, trace gravel.	
								154.20		Dark brown, wet, SAND, some silt, trace gravel, off auger flight. (6.80)	
								153.00		(8.00)	
10	2D	24/6	10.00 - 12.00	2/3/3/2	6	8				Wet SAND, little silt, trace gravel.	
								149.00		(12.00) **Bottom of Exploration at 12.00 feet below ground surface.** NO REFUSAL	
15											
20											
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.

Appendix B
Calculations

Retaining Wall Active Earth Pressures:

Effective active earth pressure coefficient:

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

$\beta := 0 \cdot deg$ slope angle of backfill soils from horizontal

$\phi_a := 34 \cdot deg$ assumed effective friction angle for dense gravelly sand

$$K_a := \tan\left(45 \cdot deg - \frac{\phi_a}{2}\right)^2 \qquad K_a = 0.28$$

Service Limit State Bearing Resistance - Native Granular Soils:

Nominal and factored Bearing Resistance

Presumptive Bearing Resistance for Service Limit State ONLY

Reference: AASHTO LRFD Bridge Design Specifications 5th 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: SAND, little silt, trace gravel (Fill)

Consistency In Place: medium dense

Bearing Resistance: Ordinary Range (ksf)
4 - 8 medium dense to dense

Recommended Value of Use: 5 ksf

Recommended Value: $5 \cdot ksf = 2.5 \cdot tsf$ $\qquad tsf := g \cdot \left(\frac{ton}{ft^2}\right)$

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

Characteristics of Native Soils at foundation level

Gray-brown, wet, loose gravelly fine SAND (fill)

$wc := 0.225$ estimated water content

$e := 0.6$ estimated void ratio

$\gamma_w := 62.4pcf$ unit weight of water

$sp := 2.65$ estimated specific gravity

saturation $sat := \frac{(wc \cdot sp)}{e}$ $sat = 0.9938$

total unit weight

$$\gamma_t := \frac{(sp + sat \cdot e) \cdot \gamma_w}{(1 + e)} \qquad \gamma_t = 126.6 \cdot pcf$$

saturated unit weight

$$\gamma_{sat} := \frac{(sp + e) \cdot \gamma_w}{(1 + e)} \qquad \gamma_{sat} = 127 \cdot pcf$$

Strength Limit State Bearing Resistance - Native Granular Soils:

Nominal and factored Bearing Resistance for box culvert and wingwall base slab

Assumptions

1. Box culvert will be embedded 1.0 feet below streambed
   $D_f := 1.0ft$

2. Assumed parameters based on granular fill

   Moist unit weight $\gamma_m := 124pcf$

   Saturated unit weight $\gamma_s := 127pcf$

   Angle of internal friction $\phi_b := 34deg$

   Undrained shear strength $c := 0$

   Depth to groundwater $D_w := 0ft$

3. L > B, so use Terzaghi strip foundation equations

   Terzaghi shape factors $B := 12ft$

   $s_c := 1.0$

   $s_\gamma := 1.0$

   Meyerhof bearing capacity factors $N_c := 35.47$

   $N_q := 23.2$

$N_{\gamma} := 22.0$

Effective stress at footing level

$$q_{eff} := D_w \cdot \gamma_m + (D_f - D_w) \cdot (\gamma_s - \gamma_w)$$

$$q_{eff} = 65 \cdot psf$$

Nominal Bearing Resistance

$$q_{nom} := c \cdot N_c \cdot s_c + q_{eff} \cdot N_q + 0.5 \cdot (\gamma_s - \gamma_w) \cdot B \cdot N_{\gamma} \cdot s_{\gamma}$$

$$q_{nom} = 10.026 \cdot ksf$$

Resistance Factor from LRFD Table 10.5.5.2.2-1, pg 10-32 $\Phi_b := 0.45$

$q_{factored} := q_{nom} \cdot \Phi_b$ $q_{factored} = 4.512 \cdot ksf$

For this project, the Strength limit state controls.
Factored Bearing Resistance of 4.5 ksf should be used.

Appendix C
Retaining Wall Design

AASHTO 98 ASD DESIGN METHOD
skowhegan wall

MSEW(3.0): Update # 10.2

PROJECT IDENTIFICATION

Title: skowhegan wall
Project Number:
Client:
Designer: klb
Station Number:

Description:

Company's information:

Name:
Street:

,
Telephone #:
Fax #:
E-Mail:

Original file path and name: D:\skowhegan 019153\MSEW1.BEN
Original date and time of creating this file: Wed Feb 13 15:27:34 2013

PROGRAM MODE: DESIGN
of a SIMPLE STRUCTURE
using GEOTEXTILE as reinforcing material.

DESIGN DATA

DESIGN OBJECTIVES

Minimum factor of safety against pullout, Fs-po	1.50
Minimum factor of safety against direct sliding, Fs-sliding	1.50
Maximum allowable eccentricity ratio at each reinforcement level, e/L	0.1667
Minimum factor of safety against compound and overall failure, Fs-comp-static	1.30
Minimum factor of safety against compound and overall failure, Fs-comp-seismic	1.10

Prescribed minimum resistive length to prevent pullout, Le = 3.28 ft.
Prescribed minimum normalized length of each layer is: L/Hd = 0.70 --> L = 4.20 ft.
Prescribed minimum absolute total length of each layer is: L = 6.00 ft.

BEARING CAPACITY

Bearing capacity is controlled by general shear.

Maximum permissible eccentricity ratio (soil), e/L	0.1667
Minimum factor of safety with respect to ultimate bearing capacity (Meyerhof approach)	2.00

Bearing capacity coefficients: N_c = 55.63 N_γ = 66.19

SOIL DATA

REINFORCED SOIL

Unit weight, γ	120.0 lb/ft ^{3}
Design value of internal angle of friction, ϕ	34.0 °

RETAINED SOIL

Unit weight, γ	120.0 lb/ft ^{3}
Design value of internal angle of friction, ϕ	34.0 °

FOUNDATION SOIL (Considered as an equivalent uniform soil)

Equivalent unit weight, $\gamma_{equiv.}$	135.0 lb/ft ^{3}
Equivalent internal angle of friction, $\phi_{equiv.}$	37.0 °
Equivalent cohesion, $c_{equiv.}$	0.0 lb/ft ^{2}

Water table does not affect bearing capacity

LATERAL EARTH PRESSURE COEFFICIENTS

Ka (internal stability) = 0.2827 (if batter is less than 10°, Ka is calculated from eq. 15. Otherwise, eq. 38 is utilized)
Inclination of internal slip plane, ψ = 62.00° (see Fig. 28 in DEMO 82).
Ka (external stability) = 0.2827 (if batter is less than 10°, Ka is calculated from eq. 16. Otherwise, eq. 17 is utilized)

SEISMICITY

Not Applicable

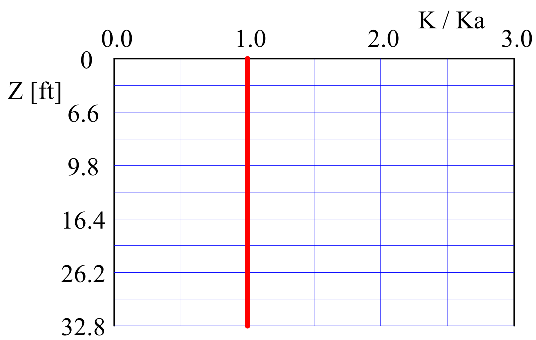
INPUT DATA: Geotextiles
(Equally spaced single type reinforcement)

D A T A	Geotextile type #1	Geotextile type #2	Geotextile type #3	Geotextile type #4	Geotextile type #5
Tult [lb/ft]	68519.0	N/A	N/A	N/A	N/A
Durability reduction factor, RFd	2.00	N/A	N/A	N/A	N/A
Installation-damage reduction factor, RFid	3.00	N/A	N/A	N/A	N/A
Creep reduction factor, RFc	5.00	N/A	N/A	N/A	N/A
Fs-overall for strength	1.50	N/A	N/A	N/A	N/A
Coverage ratio, Rc	1.000	N/A	N/A	N/A	N/A

Friction angle along geotextile-soil interface, ρ	24.22
Pullout resistance factor, F*	0.67·tanϕ
Scale-effect correction factor, α	0.6

Variation of Lateral Earth Pressure Coefficient With Depth

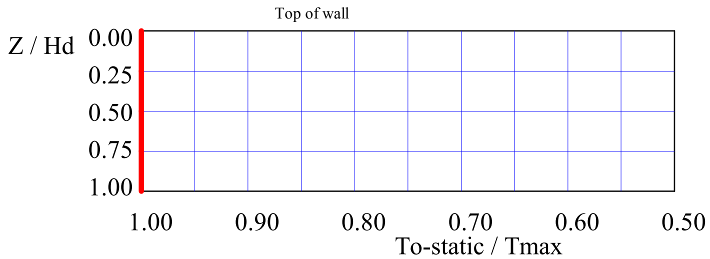
Z	K / Ka
0 ft	1.00
3.3 ft	1.00
6.6 ft	1.00
9.8 ft	1.00
13.1 ft	1.00
16.4 ft	1.00
19.7 ft	1.00



INPUT DATA: Facia and Connection (Design)

FACIA type: Facing enabling frictional connection of reinforcement (e.g., modular concrete blocks, gabions)
Depth/height of block is 2.33/1.50 ft. Horizontal distance to Center of Gravity of block is 1.20 ft.
Average unit weight of block is γ_f = 140.00 lb/ft ^{3}

Z / Hd	To-static / Tmax
0.00	1.00
0.25	1.00
0.50	1.00
0.75	1.00
1.00	1.00



To-static, To-seismic = connection force, static and superimposed dynamic component, respectively.

Geotextile Type #1 σ [1]	CRu [2]	Geotextile Type #2 σ	CRu	Geotextile Type #3 σ	CRu	Geotextile Type #4 σ	CRu	Geotextile Type #5 σ	CRu
1044.2	0.90								
2506.1	0.90	N/A		N/A		N/A		N/A	

Geotextile Type #1 σ	CRs [3]	Geotextile Type #2 σ	CRs	Geotextile Type #3 σ	CRs	Geotextile Type #4 σ	CRs	Geotextile Type #5 σ	CRs
0.0	0.00								
1044.2	0.90	N/A		N/A		N/A		N/A	

(1) σ = Confining stress in between stacked blocks [lb/ft ^{2}]
(2) CRu = Tult-connection / Tult-geotextile
(3) CRs = Tpo-connection / Tult-geotextile

D A T A (for connection only)	Type #1	Type #2	Type #3	Type #4	Type #5
Product Name	---	N/A	N/A	N/A	N/A
Durability reduction factor, RFd	2.00	N/A	N/A	N/A	N/A
Creep reduction factor, RFc	5.00	N/A	N/A	N/A	N/A
Overall factor of safety: connection break, Fs	1.50	N/A	N/A	N/A	N/A
Overall factor of safety: connection pullout, Fs	1.50	N/A	N/A	N/A	N/A

INPUT DATA: Geometry and Surcharge loads (of a SIMPLE STRUCTURE)

Design height, Hd	6.00	[ft]	{ Embedded depth is E = 0.00 ft, and height above top of finished bottom grade is H = 6.00 ft }
Batter, ω	3.8	[deg]	
Backslope, β	0.0	[deg]	
Backslope rise	0.0	[ft]	Broken back equivalent angle, I = 0.00° (see Fig. 25 in DEMO 82)

U N I F O R M   S U R C H A R G E

Uniformly distributed dead load is 0.0 [lb/ft ²], and live load is 125.0 [lb/ft ²]

NOTE: Tmax is NOT affected by live loads.

DESIGNED REINFORCEMENT LAYOUT:

SCALE:

0 2 4 6 [ft]

REINFORCEMENT LAYOUT AND DESIGN CRITERIA

LEGEND:
(1) Connection strength
(2) Geotextile strength
(3) Pullout resistance
(4) Direct sliding
(5) Eccentricity

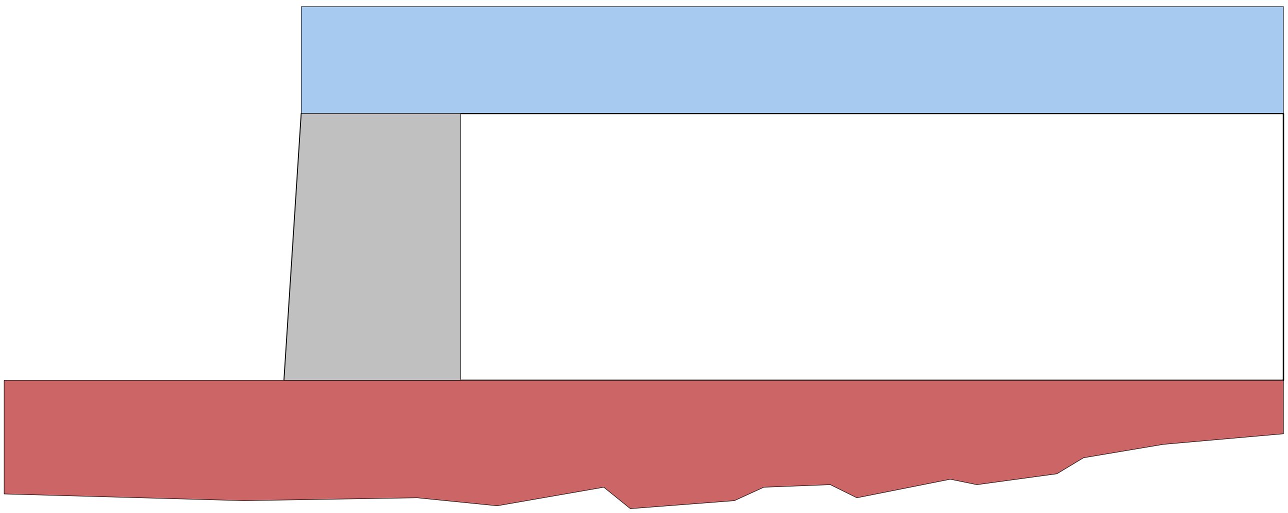
√ Satisfactory
⊗ Unsatisfactory

Bearing capacity: √
Foudation Interface: Direct sliding √ Eccentricity √

#	Geotextile Elevation [ft]	Geotextile Length [ft]	Geotextile Type #	(1)	(2)	(3)	(4)	(5)
1	1.50	6.00	N/A	√	√	√	√	√
2	3.00	6.00	N/A	√	√	√	√	√
3	4.50	6.00	N/A	√	√	√	√	√

BEARING CAPACITY for DESIGNED LAYOUT

	STATIC	SEISMIC	UNITS
(Water table does not affect bearing capacity)			
Ultimate bearing capacity, q-ult	13541	N/A	[lb/ft ²]
Meyerhof stress, σ_V	1046.3	N/A	[lb/ft ²]
Eccentricity, e	0.47	N/A	[ft]
Eccentricity, e/L	0.119	N/A	
Fs calculated	12.94	N/A	
Base length	3.98	N/A	[ft]



SCALE:

0 2 4 6 [ft]

DIRECT SLIDING for DESIGNED LAYOUT
(for GEOTEXTILE reinforcements)

Specified Fs-static = 1.500

Along reinforced and foundation soils interface: Fs-static = 2.232

#	Geotextile Elevation [ft]	Geotextile Length [ft]	Fs Static	Fs Seismic	Geotextile Type #	Product name
1	1.50	6.00	2.829	N/A	N/A	---
2	3.00	6.00	3.695	N/A	N/A	---
3	4.50	6.00	5.285	N/A	N/A	---

ECCENTRICITY for DESIGNED LAYOUT

At interface with foundation: e/L static = 0.1470; Overturning: Fs-static = 3.06

#	Geotextile Elevation [ft]	Geotextile Length [ft]	e / L Static	e / L Seismic	Geotextile Type #	Product name
1	1.50	6.00	0.0338	N/A	N/A	---
2	3.00	6.00	0.0163	N/A	N/A	---
3	4.50	6.00	0.0050	N/A	N/A	---

RESULTS for STRENGTH Live Load NOT included in calculating Tmax

#	Geotextile Elevation [ft]	Tavailable [lb/ft]	Tmax [lb/ft]	Tmd [lb/ft]	Specified minimum Fs-overall static	Actual calculated Fs-overall static	Specified minimum Fs-overall seismic	Actual calculated Fs-overall seismic	Product name
1	1.50	2284	372.1	N/A	1.500	6.138	N/A	N/A	---
2	3.00	2284	152.7	N/A	1.500	14.961	N/A	N/A	---
3	4.50	2284	85.9	N/A	1.500	26.597	N/A	N/A	---

RESULTS for PULLOUT Live Load NOT included in calculating Tmax

NOTE: Live load is not included in calculating the overburden pressure used to assess pullout resistance.

#	Geotextile Elevation [ft]	Coverage Ratio	Tmax [lb/ft]	Tmd [lb/ft]	Le [ft] (see NOTE)	La [ft]	Avail.Static Pullout, Pr [lb/ft]	Specified Static Fs	Actual Static Fs	Avail.Seism. Pullout, Pr [lb/ft]	Specified Seismic Fs	Actual Seismic Fs
1	1.50	1.000	372.1	N/A	5.30	0.70	1545.3	1.500	4.153	N/A	N/A	N/A
2	3.00	1.000	152.7	N/A	4.60	1.40	894.3	1.500	5.858	N/A	N/A	N/A
3	4.50	1.000	85.9	N/A	3.90	2.10	379.4	1.500	4.418	N/A	N/A	N/A

RESULTS for CONNECTION (static conditions)

Live Load NOT included in calculating Tmax

#	Geotextile Elevation [ft]	Connection force, To [lb/ft]	Reduction factor for connection break, CRu	Reduction factor for connection pullout, CRs	Available connection strength, Tc-break criterion [lb/ft]	Available connection strength, Tc-pullout criterion [lb/ft]	Available Geotextile strength, Tavailable [lb/ft]	Fs-overall connection strength Specified	Fs-overall connection strength Actual	Fs-overall connection pullout Specified	Fs-overall connection pullout Actual	Fs-overall Geotextile strength Specified	Fs-overall Geotextile strength Actual	Product name
1	1.50	372	0.90	0.54	6167	37205	2284	1.50	16.57	1.50	99.98	1.50	6.14	---
2	3.00	153	0.90	0.36	6167	24803	2284	1.50	40.39	1.50	162.47	1.50	14.96	---
3	4.50	86	0.90	0.18	6167	12402	2284	1.50	71.81	1.50	144.41	1.50	26.60	---