GEOTECHNICAL DESIGN REPORT

17-0733

November 21, 2017

Explorations and Geotechnical Engineering Services

Route 4 Slope Stabilization Project
Turner, Maine
WIN 023026.00

PREPARED FOR:
Maine Department of Transportation
Attention: Kate Maguire, P.E.
State House Station 16
Augusta, ME 04333-0016

PREPARED BY:
S. W. Cole Engineering, Inc.
26 Coles Crossing Drive
Sidney, ME 04330
T: (207) 626-0600

S.W.COLE ENGINEERING, INC.

- *Geotechnical Engineering*
- *Construction Materials Testing and Special Inspections*
- *GeoEnvironmental Services*
- *Test Boring Explorations*

www.swcole.com

TABLE OF CONTENTS

17-0733

November 21, 2017

Maine Department of Transportation
Attention: Kate Maguire, P.E.
State House Station 16
Augusta, ME 04333-0016

Subject: Geotechnical Design Report
Explorations and Geotechnical Engineering Services
Route 4 Slope Stabilization Project
Turner, Maine
WIN 023026.00

Dear Kate:

In accordance with our Proposal dated July 25, 2017 and project Assignment Letter #3 dated July 26, 2017, we have made the requested subsurface explorations for the subject project. The purpose of our services was to obtain subsurface information in order to provide geotechnical recommendations for slope stability and earthwork associated with the proposed construction. The services provided by S. W. Cole Engineering, Inc. (S.W.COLE) were conducted in accordance with our Multi-PIN Agreement with the Maine Department of Transportation (MaineDOT), No. 20150720000000000085, dated July 20, 2015. The contents of this report are subject to the limitations in Appendix A.

1.0 INTRODUCTION

1.1 Site and Proposed Construction

The project is located on the east side of Route 4 approximately 1 mile north of the intersection with Harlow Hill Road in Turner, Maine. The project location is shown on the *Site Location Map* attached as Appendix B. The existing slope consists of a ±160-foot long, 3 to 7-foot tall gabion basket retaining wall located near the head of slope before steeply sloping (±1H:1V to 1.5H:1V) downward to a wet area about 10 to 15 feet below the wall. The wall embedment depth is unknown.

Based on the provided information and observations made during a site visit on July 18, 2017, we understand the existing gabion wall has bulged and shifted outward about 6 to 10 inches within a 50 to 60 foot portion on the southern end of the wall. Water seepage was not observed during our visit; however, a 3 to 4-foot wide erosion channel was observed corresponding to the prominently bulged section of wall. The erosion channel appears to have been temporarily repaired at the toe of wall with granular fill underlain by a non-woven geotextile fabric. The geotextile fabric is underlain by several boulders with limited soil matrix between the boulders. Silt fencing was observed to be pushed over and retaining up to 12 inches of sediment at the toe of the slope.

We understand wall replacement options under consideration include:

- Construction of a Reinforced Segmental Retaining Wall (SRW) at the head of slope with 1.5H:1V or flatter foreslope;
- Construction of a Reinforced SRW at the toe of slope with 1.5H:1V or flatter backslope; or
- Construction of a 1.5H:1V or flatter Unreinforced Soil Slope.

We understand replacement options will be constructed within the existing construction limit line shown on the *Boring Location Plan* attached as Appendix B.

2.0 EXPLORATIONS AND TESTING

2.1 Explorations

Two test borings (HB-TUR-101 and HB-TUR-102) were made at the site between August 7 and 8, 2017 by S. W. Cole Explorations, LLC using a track-mounted CME 850 drill rig. The exploration locations were selected and established in the field by S.W.COLE using taped measurements from existing site features. The exploration locations are shown on the *Boring Location Plan* attached as Appendix B. Logs of the test borings and a Key to Soil and Rock Descriptions and Terms used on the logs are attached as Appendix C.

2.2 Testing

The test borings were drilled using a combination of solid-stem auger and cased-wash boring drilling techniques. The soils were sampled at 2 to 5-foot intervals using a split-spoon sampler and Standard Penetration Testing (SPT) methods. Upon encountering refusal boring HB-TUR-101 was advanced about 5 feet into a granite boulder using a NQ2 rock coring. The hammer efficiency factor (0.813), uncorrected SPT blow counts, raw field N-values, corrected N-values (N_{60}) and rock core intervals are shown on the logs in Appendix C. The drill rig was equipped with a calibrated automatic hammer to drive the split-spoon. Corrected N-values in this report were computed by applying an average energy transfer of 0.813 for the calibrated automatic hammer to the raw field N-values.

Soils samples recovered from the test borings were visually classified in our laboratory and transported to the MaineDOT Laboratory in Bangor, Maine.

3.0 SUBSURFACE CONDITIONS

<u>3.1 Surficial and Bedrock Geology</u>

According to the Maine Geological Survey (MGS) mapping of the Buckfield Quadrangle (Open-File 08-68, 2008), mapped surficial geology units within the site vicinity consists of the following:

- Glaciomarine delta (regressive) deposits of sand, gravel and silt.
- Wetland deposits of peat, muck, silt and clay are mapped to the east at the toe of slope.
- Glacial till deposits of poorly sorted to stratified mixture of sand, silt, gravel with rock debris (cobbles and boulders) are mapped to the west of the slope.

The subsurface conditions encountered were generally consistent with the mapped surficial geology; however, the explorations also encountered a surface deposit of fill soils from previous site development. Wetland deposits of peat, muck and clay were not encountered in the explorations.

According to MGS mapping of the Buckfield Quadrangle (Open-File 78-18, 1978), mapped bedrock geology units within the site vicinity consists of thinly interlaminated, grey metasiltstone, metapelite and metasandstone. Bedrock was not encountered at the test boring locations within the depths drilled.

3.2 Soil and Bedrock

The test borings encountered a soils profile generally consisting of a surface layer of pavement overlying fill overlying glaciomarine delta sands overlying glacial till. The principal strata encountered in the explorations are summarized below. Refer to the attached logs for more detailed descriptions of the subsurface findings at the exploration locations.

Pavement: Bituminous concrete pavement was encountered at the ground surface in borings HB-TUR-101 and HB-TUR-102 made through the paved shoulders of Route 4. The bituminous concrete pavement thickness was 3.5 inches (0.3 feet).

Fill: Below the pavement, the borings encountered fills extending to depths of about 10 to 11 feet below ground surface (bgs), corresponding to Elevation (El.) 351.4 to 349.6 feet. The fill soils generally consisted of sand with varying amounts of gravel and silt.

The fill was generally loose to medium dense with SPT N_{60} values ranging from 5 to 26 blows per foot (bpf).

Glaciomarine Delta Deposit: Below the fill, the borings encountered glaciomarine delta deposits extending to depths of about 31 to 37 feet bgs, corresponding to El. 330.4 to 323.6 feet. The glaciomarine delta deposit generally consisted of sand with varying amounts of silt and trace gravel.

The glaciomarine delta deposit was generally very loose to medium dense with SPT N_{60} values ranging from 3 to 20 bpf.

Glacial Till: Below the glaciomarine delta deposits, the borings encountered glacial till generally consisting of sand with varying amounts of gravel and silt and occasional cobbles and boulders.

The glacial till was generally medium dense to very dense with SPT N_{60} values ranging from 15 bpf to refusal (e.g. greater than 50 blows per 6 inch increment of drive). Upon encountering refusal at about 37 feet bgs, boring HB-TUR-101 was advanced 5 feet into a granite boulder by rock coring. The borings were terminated in the glacial till at depths of about 42 to 44 feet bgs, corresponding to El. 317.4 to 318.6 feet.

3.3 Groundwater

The soils encountered at the test borings were damp to moist from the ground surface. The measured water level in HB-TUR-101 immediately after drilling was about 23 feet below ground surface, which is approximately 3 feet above the elevation of the wet area at the toe of the embankment slope. Long term groundwater information is not available. It should be anticipated that groundwater levels will fluctuate seasonally, particularly in response to periods of snowmelt and precipitation, changes in site use and the water level of adjacent waterways or wet areas.

4.0 GEOTECHNICAL EVALUATION AND RECOMMENDATIONS

S.W.COLE conducted geotechnical engineering evaluations in accordance with 2014 AASHTO LRFD Bridge Design Specifications, 7th Edition with 2016 interim revisions (AASHTO) and the MaineDOT Bridge Design Guide, 2003 Edition with revisions through August 2014 (MaineDOT BDG) and offers the following:

4.1 Slope Stabilization Options

Following removal of the existing gabion basket wall, we understand proposed slope stabilization alternatives under consideration include:

- Construction of a Reinforced SRW at head of slope with 1.5H:1V or flatter foreslope;
- Construction of a Reinforced SRW at toe of slope with 1.5H:1V or flatter backslope; or
- Construction of a 1.5H:1V or flatter Unreinforced Soil Slope.

The slope area is underlain by loose to medium dense glaciomarine delta deposits consisting of sand with varying amounts of silt and gravel overlying medium dense to very dense glacial till.

4.2 Global Stability Analyses

We performed global slope stability analyses for the proposed slope stabilization options through the section of slope where predominant wall bulging was observed. Our slope stability analyses were made using the computer software SLOPE/W (GeoStudio 2012) software by Geo-Slope. The purpose of the global stability analyses was to aid in an evaluation of geotechnical feasibility of the proposed stabilization options.

We evaluated global stability considering the resistance factors outlined in AASHTO LRFD 7th Edition with 2016 interim revisions, Section 11.6.2.3 and guidance in Section C11.6.2.3 as follows:

Global Stability for Static Conditions

FS ≥ 1.3 (ϕ =0.75) for slopes or walls not supporting a structural element

Our global stability analyses utilized a method of slices assuming moment equilibrium and was based on: 1) our current understanding of the project; 2) subsurface information from the explorations; 3) estimated soil parameters; and 4) existing site topography shown on Boring Location Plan attached in Appendix B.

The loose to medium dense glaciomarine delta sands soils that form a portion of the roadway embankment were modeled as two layers. The following soils values were used in our model runs:

Layer	Friction Angle	Cohesion	Unit Weight
Existing Granular Fill	30 deg	0 psf	122 pcf
Loose Glaciomarine Delta Sand	29 deg	0 psf	118 pcf
Med. Dense Glaciomarine Delta Sand	33 deg	0 psf	125 pcf
Glacial Till	36 deg	0 psf	130 pcf
New Granular Fill (Gravel Borrow)	34 deg	0 psf	125 pcf
	32 deg	0 psf	125 pcf

Results of our global stability model runs are summarized in the following table and included as Appendix C.

Model	Static Conditions FS ≥ 1.3
Unreinforced Soil Slope	
1.5H:1V Unreinforced Slope	1.14
1.75H:1V Unreinforced Slope	1.31
2H:1V Unreinforced Slope – New fills with friction angle of 34 deg	1.39
2H:1V Unreinforced Slope – New fills with friction angle of 32 deg	1.32
Segmental Retaining Wall (SRW) at toe of slope	
10.5-ft SRW; 2:1 backslope; 12.5 ft long reinforcement geotextile	1.30
13.5-ft SRW; 2:1 backslope; 13.5 ft long reinforcement geotextile	1.39
13.5-ft SRW; 2:1 backslope; 22.5 ft long reinforcement geotextile	1.50
Segmental Retaining Wall (SRW) at head of slope	
12-ft SRW; 3:1 foreslope; 15 ft long reinforcement geotextile	1.34

Notes: Reinforcement Geotextile: Mirafi Miragrid 5XT with a 2700 lb/ft long-term design strength.
Toe of 2H:1V unreinforced slope exceeds construction limit line.

4.2 Reinforced Segmental Retaining Wall

We anticipate the proposed SRW options will consist of embankment cuts, over-excavation to remove the existing wall structure and to bench new embankment fills, construction of a reinforced SRW (in accordance with MaineDOT Standard Specification 672), placement of new embankment fills, and surface slope erosion control over portions of exposed soil slopes.

Based on our analysis, we anticipate SRW construction will require excavation of the northbound travel lane to accommodate 1H:1V or flatter construction slopes as required by OSHA. Alternatively, temporary shoring, such as sheetpiling, could be used to reduce excavation.

4.2.1 Retaining Wall Design

The proposed retaining wall shall be supplier designed by a Professional Engineer licensed in the State of Maine in accordance with:

- Standard Specification Standard Specification 672 Precast Concrete Block Gravity Wall;
- AASHTO LRFD Bridge Design Specifications 7th Edition with 2016 interim revisions; and
- Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Slopes (FHWA-NHI-10-024 and FHWA-NHI-10-025).

The retaining wall shall be designed to withstand lateral earth pressures. Earth loads shall be calculated using an active earth pressure coefficient, K_a, in accordance with LRFD Article 3.11.5.3 and Equations 3.11.5.3-1 and -2 to calculate the Coulomb active earth pressure coefficient. Lateral earth pressure distributions for design of SRW walls are provided in LRFD Figures 3.11.5.8.1-1, -2 and -3. Passive earth pressure in front of the wall should be neglected in the design.

The factored bearing pressure for the strength limit state for the wall shall not exceed the calculated factored bearing resistance of 3 kips per square foot (ksf). A factored bearing resistance of 3 ksf shall be used to control settlement when analyzing the service limit state per LRFD C10.6.2.6.1. In no instance shall the bearing stress exceed the nominal resistance of the structural concrete which may be taken as 0.3f'c.

Additionally, the following considerations should be addressed in the wall design:

- A traffic surcharge estimated as a uniform horizontal earth pressure will be required in the design of the wall based on LRFD Table 3.11.6.4-1.
- Drainage system shall be included in the design of the wall.
- A minimum embedment of 2.0 feet is required for the wall design.
- The retaining wall design shall include a drainage system (swale) at the top of the wall to carry surface water runoff away from the face of the wall.
- If a fence is constructed at the top of the wall, fence posts shall not be driven or augered through the wall reinforcement layers.

4.3 Unreinforced Soil Slope

The proposed unreinforced soil slope option will consist of over-excavation to remove the existing gabion wall, embankment cuts to bench new embankment fills, placement and compaction of embankment fills, and provide surface slope erosion control.

The unreinforced soil slope shall be constructed with 1.75H:1V or flatter riprap slopes. The soil slope steeper than 2H:1V shall be constructed with Gravel Borrow with a minimum friction angle of 34 degrees and faced with riprap and geotextile fabric for erosion control. The riprap slopes should not exceed 1.75:1(H:V) in accordance with MaineDOT Standard Detail 610(03). The soil slope or 2H:1V or flatter shall be constructed with Gravel Borrow with a minimum friction angle of 32 degrees and faced with 6 inches of Dirty Borrow and seeded. Additionally, we recommend the use of riprap underlain by geotextile fabric in areas where surface water discharge will be localized on the slope.

The toe of the new fill and riprap slopes should be keyed into the existing soils at least 2 feet. The new fills should be benched into the native soils and existing fills, compacted and the face trimmed back with a smooth edged bucket.

4.4 Excavations and Dewatering

Excavations will generally encounter existing surficial topsoil with organics, granular fills and glaciomarine delta sands.

Earth support systems, such as sheetpiles, will be required if laying back construction slopes is not feasible. Regardless of excavation methods, excavations and earth support systems must be properly shored or sloped in accordance with OSHA regulations to prevent sloughing and caving of slopes during construction. The design and planning of excavations, excavation support systems, and dewatering is the responsibility of the Contractor.

The Contractor shall control groundwater and surface water infiltration using temporary ditches, sumps, granular drainage blankets, stone ditch protection or hand-laid riprap with geotextile underlayment to divert groundwater and surface water as necessary.

4.5 Settlement Considerations

The site is underlain by granular fills overlying glaciomarine sands and glacial till. Post-construction settlement of new embankment fills and retaining walls should be elastic and occur during construction with proper compaction.

4.6 Backfill and Compaction

Fill and backfill should be placed in horizontal lifts and compacted such that the desired density is achieved throughout the lift thickness with 3 to 5 passes of the compaction equipment. Loose lift thicknesses for grading, fill and backfill activities should be limited to 12 inches. Small, hand operated or walk-behind compaction equipment shall be used within 3 feet of the retaining wall to avoid over-compaction of material adjacent to the proposed wall. We recommend fill and backfill be compacted to at least 95 percent of its maximum dry density as determined by AASHTO T-180. Crushed Stone bedding material shall be compacted with 3 to 5 passes of a vibratory plate compactor having a static weight of at least 500 pounds.

5.0 CLOSURE

We trust this information meets your present needs. Please contact us if you have any questions or need further assistance.

Sincerely,

S. W. Cole Engineering, Inc.

Michael A. St. Pierre, P.E.
Geotechnical Engineer

STATE OF MAINE
MICHAEL A.
ST. PIERRE
No. 12519
LICENSED
PROFESSIONAL ENGINEER

Timothy J. Boyce, P.E.
Senior Geotechnical Engineer

MAS/ajh:tjb

APPENDIX A
Limitations

This report has been prepared for the exclusive use of the Maine Department of Transportation for specific application to the Route 4 Slope Stabilization Project (MaineDOT WIN 023026.00) in Turner, Maine. S. W. Cole Engineering, Inc. (S.W.COLE) has endeavored to conduct our services in accordance with generally accepted soil and foundation engineering practices. No warranty, expressed or implied, is made.

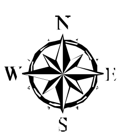
The soil profiles described in the report are intended to convey general trends in subsurface conditions. The boundaries between strata are approximate and are based upon interpretation of exploration data and samples.

The analyses performed during this investigation and recommendations presented in this report are based in part upon the data obtained from subsurface explorations made at the site. Variations in subsurface conditions may occur between explorations and may not become evident until construction. If variations in subsurface conditions become evident after submission of this report, it will be necessary to evaluate their nature and to review the recommendations of this report.

Observations have been made during exploration work to assess site groundwater levels. Fluctuations in water levels will occur due to variations in rainfall, temperature, and other factors.

Recommendations contained in this report are based substantially upon information provided by others regarding the proposed project. In the event that any changes are made in the design, nature, or location of the proposed project, S.W.COLE should review such changes as they relate to analyses associated with this report. Recommendations contained in this report shall not be considered valid unless the changes are reviewed by S.W.COLE.

APPENDIX B
Figures



S.W.COLE ENGINEERING,INC.

MAINE DEPARTMENT OF TRANSPORTATION

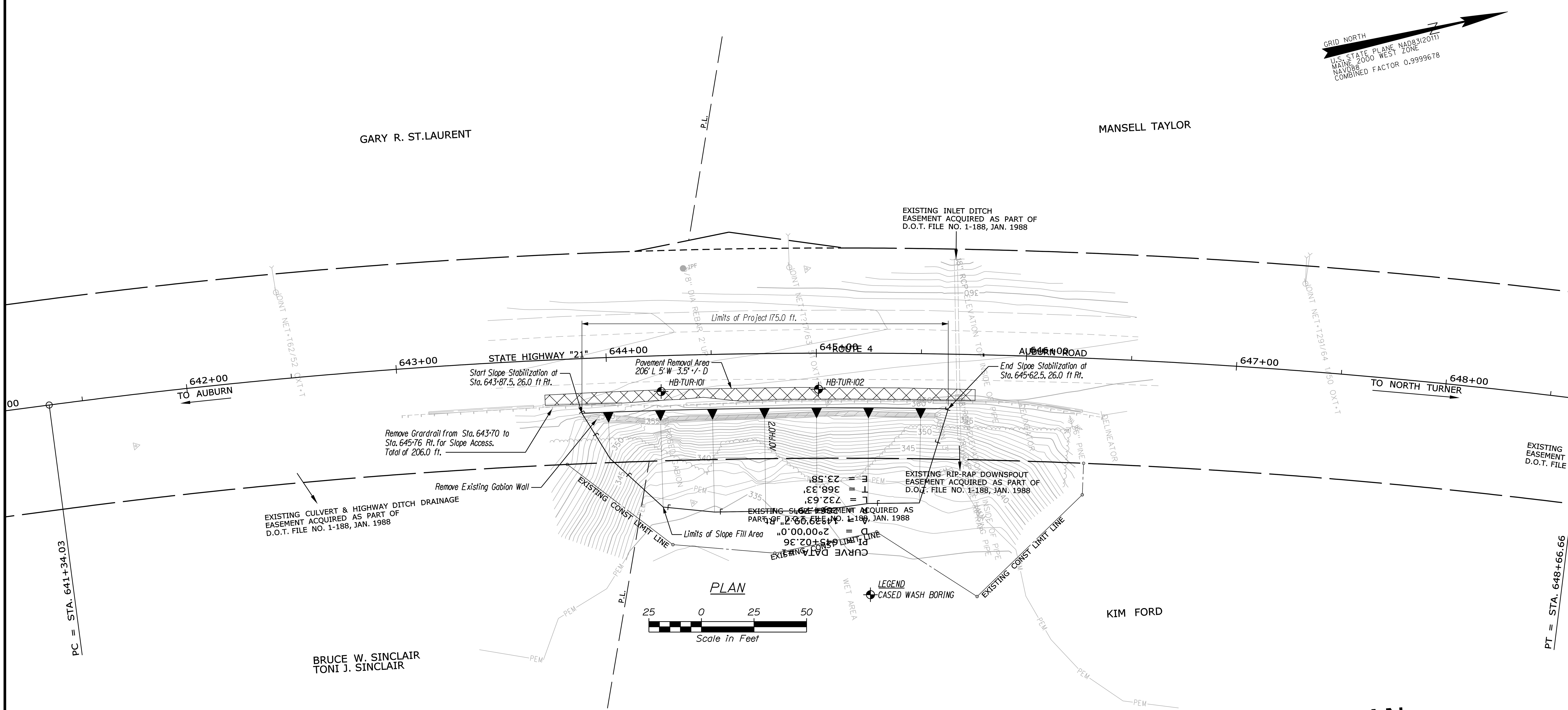
SITE LOCATION MAP

ROUTE 4 SLOPE STABILIZATION PROJECT
TURNER, MAINE
WIN 023026.00

Job No.	17-0733	Scale	1:24000
Date:	09/15/2017	Sheet	1

NOTE:
SITE LOCATION MAP PREPARED FROM ESRI ArcGIS ONLINE AND DATA PARTNERS INCLUDING USGS AND © 2007 NATIONAL GEOGRAPHIC SOCIETY.

R:\2017\17-0733\GIS\MXDs\17-0733 SLM.mxd, 9/15/2017 1:03:23 PM, 1:24,000, Filename: ...\GEOTECH\MSTA\001_BLP SS1.dgn Division: GEOTECH Username: Terry.White Date:11/29/2017



PRELIMINARY PLAN

EXISTING RIGHT OF WAY REFERENCES

OXFORD COUNTY COMMISSIONERS RECORDS
VOL. 2 PAGE 198
1836, 4 RODS WIDE

D.O.T. FILE NO. 1-188, SHEETS 6 & 7, DATED JANUARY 1988

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

023026.00

WIN
23026.00

HIGHWAY PLANS

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED		
CHECKED-REVIEWED	M.ST.PIERRE	
DESIGN2-DETAILED2	T.WHITE	AUG 2017
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

TURNER
ROUTE 4 SLOPE STABILIZATION

BORING LOCATION PLAN & SLOPE STABILIZATION

SHEET NUMBER
1
OF _

APPENDIX C

Boring Logs & Key to Soil and Rock Descriptions and Terms

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, if applicable):
Color (Munsell color chart)
Moisture (dry, damp, moist, wet)
Density/Consistency (from above right hand side)
Texture (fine, medium, coarse, etc.)
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.,)
Cementation (weak, moderate, or strong)
Geologic Origin (till, marine clay, alluvium, etc.)
Groundwater level

MODIFIED BURMISTER SYSTEM

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

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. Density is rated according to standard penetration resistance (N-value).

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 undrained shear strength as indicated.

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

Rock Quality Designation (RQD):

$$\text{RQD (\%)} = \frac{\text{sum of the lengths of intact pieces of core* > 4 inches}}{\text{length of core advance}}$$

*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, if applicable):
Color (Munsell color chart)
Texture (aphanitic, fine-grained, etc.)
Rock Type (granite, schist, sandstone, 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 deg., low angle - 5-35 deg., mod. dipping - 35-55 deg., steep - 55-85 deg., vertical - 85-90 deg.)
- -spacing (very close - <2 inch, close - 2-12 inch, mod. close - 1-3 feet, wide - 3-10 feet, very wide >10 feet)
- -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: ASTM D6032 and AASHTO Standard Specification for Highway Bridges, 17th Ed. Table 4.4.8.1.2A
Recovery (inch/inch and percentage)
Rock Core Rate (X.X ft - Y.Y ft (min:sec))

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

Sample Container Labeling Requirements:

WIN	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: Route 4 Slope Stabilization Project Location: Turner, Maine	Boring No.: HB-TUR-101 WIN: 23026.00

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	361.3	**Auger ID/OD:**	5" Solid-Stem
Operator:	J. Lee	**Datum:**	NAVD88	**Sampler:**	Standard Split-Spoon
Logged By:	N. Strout	**Rig Type:**	CME 850	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/8/2017	**Drilling Method:**	Cased Wash	**Core Barrel:**	NQ2
Boring Location:	Sta. 644+25.7, 17' Rt.	**Casing ID/OD:**	HW 4.0"/4.5"	**Water Level*:**	23.0' (casing removed)
Hammer Efficiency Factor:	0.813	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/18	0.7 - 2.7	4/6/4/4	10	14	SSA	361.00		3.5" of Pavement — 0.30	
										Brown, dry, medium dense, SAND, some gravel, trace silt, (Fill).	
	2D	24/16	2.7 - 4.7	4/3/3/2	6	8				Brown, dry, loose, SAND, trace gravel, trace silt, (Fill).	
5	3D	24/15	5.0 - 7.0	2/2/2/3	4	5				Similar to above.	
10	4D	24/10	10.0 - 12.0	1/1/1/2	2	3	14	351.30		10.00 Brown, moist, very loose, SAND, little silt, trace gravel, (Glacial Delta Deposit).	
							16				
	MD	24/0	12.0 - 14.0	2/3/2/3	5	7	17			No recovery.	
							19				
							28				
15	5D	24/6	15.0 - 17.0	1-24"	- -		7			Brown, wet, very loose, SAND, little silt, trace gravel, (Glacial Delta Deposit).	
							6				
	6D	24/18	17.0 - 19.0	2/2/3/4	5	7	8			Similar to above except reddish-brown and loose.	
							20				
							13				
20	7D	24/13	20.0 - 22.0	3/5/6/10	11	15	17			Similar to above except light brown and medium dense.	
							29				
							16				
							61				
25							72				

Remarks:

HW casing driven using 140# automatic hammer with 30" drop.

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

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

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Route 4 Slope Stabilization Project Location: Turner, Maine	Boring No.: HB-TUR-101 WIN: 23026.00

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	361.3	**Auger ID/OD:**	5" Solid-Stem
Operator:	J. Lee	**Datum:**	NAVD88	**Sampler:**	Standard Split-Spoon
Logged By:	N. Strout	**Rig Type:**	CME 850	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/8/2017	**Drilling Method:**	Cased Wash	**Core Barrel:**	NQ2
Boring Location:	Sta. 644+25.7, 17' Rt.	**Casing ID/OD:**	HW 4.0"/4.5"	**Water Level*:**	23.0' (casing removed)
Hammer Efficiency Factor:	0.813	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	8D	24/16	25.0 - 27.0	5/5/6/6	11	15	38			Similar to above except brown.	
							46				
							56				
							65				
							64				
30	9D	24/16	30.0 - 32.0	2/3/8/13	11	15	41			9D(A) Similar to above.	
							60	330.30		31.00 9D(B) Brown, wet, medium dense, SAND, little gravel, trace silt, (Glacial Till).	
							105				
							107				
							103				
35	10D	14/10	35.0 - 36.2	18/29/100-2"	- -		100			Similar to above except brown-grey and very dense.	
							104			Cobble.	
							OPEN				
	R1	60/57	39.0 - 44.0				NQ2			R1:Light grey, Biotite GRANITE boulder from 39 to 44 ft bgs.	
40											
45								316.30		45.00 **Bottom of Exploration at 45.00 feet below ground surface.** No Refusal.	
50											

Remarks:

HW casing driven using 140# automatic hammer with 30" drop.

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 4 Slope Stabilization Project
Location: Turner, Maine

Boring No.: HB-TUR-102
WIN: 23026.00

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	360.6	**Auger ID/OD:**	5" Solid-Stem
Operator:	J. Lee	**Datum:**	NAVD88	**Sampler:**	Standard Split-Spoon
Logged By:	N. Strout	**Rig Type:**	CME 850	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/7/2017	**Drilling Method:**	Cased Wash	**Core Barrel:**	NQ2
Boring Location:	Sta. 645+00.8, 17.1' Rt.	**Casing ID/OD:**	HW 4.0"/4.5"	**Water Level*:**	Not observed.
Hammer Efficiency Factor:	0.813	**Hammer Type:**	Automatic ☒	Hydraulic ☐	Rope & Cathead ☐

Definitions:

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/14	0.8 - 2.8	5/10/9/5	19	26	SSA	360.30		3.5" of Pavement 0.30 Brown, dry, medium dense, SAND, some gravel, trace silt, (Fill).	
	2D	24/6	2.8 - 4.8	4/5/8/7	13	18				Similar to above except little gravel.	
5	3D	24/20	5.0 - 7.0	3/3/3/8	6	8				Brown, dry, loose, SAND, trace gravel, trace silt, (Fill).	
10	4D	24/16	10.0 - 12.0	2/2/2/2	4	5	26			4D(A) Similar to above.	
							25	349.60		11.00 4D(B) Brown, moist, loose, SAND, little silt, trace gravel, trace organics (rootlets), (Glacial Delta Deposit). Similar to above.	
	5D	24/17	12.0 - 14.0	2/2/2/2	4	5	31				
							43				
							32				
15	6D	24/9	15.0 - 17.0	2/2/2/3	4	5	13			Similar to above.	
							23				
		24/18	17.0 - 19.0	2/2/3/4	5	7	21				
							45			Cobbles from 18 to 20 ft bgs.	
							81				
20	7D	24/10	20.0 - 22.0	5/4/3/4	7	9	38			Brown, wet, loose, SAND, trace silt, trace gravel, (Glacial Delta Deposit).	
							31				
							35				
							40				
25							44				

Remarks:

-bgs = below ground surface
HW casing driven using 140# automatic hammer with 30" drop.
Borehole caved at 19 ft 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.

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Route 4 Slope Stabilization Project Location: Turner, Maine	Boring No.: HB-TUR-102 WIN: 23026.00

Driller:	S. W. Cole Explorations, LLC	Elevation (ft.)	360.6	Auger ID/OD:	5" Solid-Stem
Operator:	J. Lee	Datum:	NAVD88	Sampler:	Standard Split-Spoon
Logged By:	N. Strout	Rig Type:	CME 850	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	8/7/2017	Drilling Method:	Cased Wash	Core Barrel:	NQ2
Boring Location:	Sta. 645+00.8, 17.1' Rt.	Casing ID/OD:	HW 4.0"/4.5"	Water Level*:	Not observed.
Hammer Efficiency Factor:	0.813	Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	8D	24/18	25.0 - 27.0	5/7/8/6	15	20	41			Brown, wet, medium dense, SAND, some silt, trace gravel, (Glacial Delta Deposit).	
							65				
							77				
							80				
							78				
30	9D	24/19	30.0 - 32.0	3/3/3/3	6	8	37			Brown, wet, loose, SAND, little silt, trace gravel, (Glacial Delta Deposit).	
							61				
							63				
							74				
							77				
35	10D	24/14	35.0 - 37.0	2/3/3/9	6	8	58			Similar to above.	
							62			Cobble.	
							94	323.60		37.00	
							146				
							175				
40	11D	24/1	40.0 - 42.0	63/40/55/43	95	129				Brown, wet, very dense, Gravelly SAND, little silt, (Glacial Till).	
										Sampler shoe bent/broken and left in borehole.	
								317.60		43.00 **Bottom of Exploration at 43.00 feet below ground surface.** No Refusal.	
45											
50											

Remarks:

-bgs = below ground surface
HW casing driven using 140# automatic hammer with 30" drop.
Borehole caved at 19 ft 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.

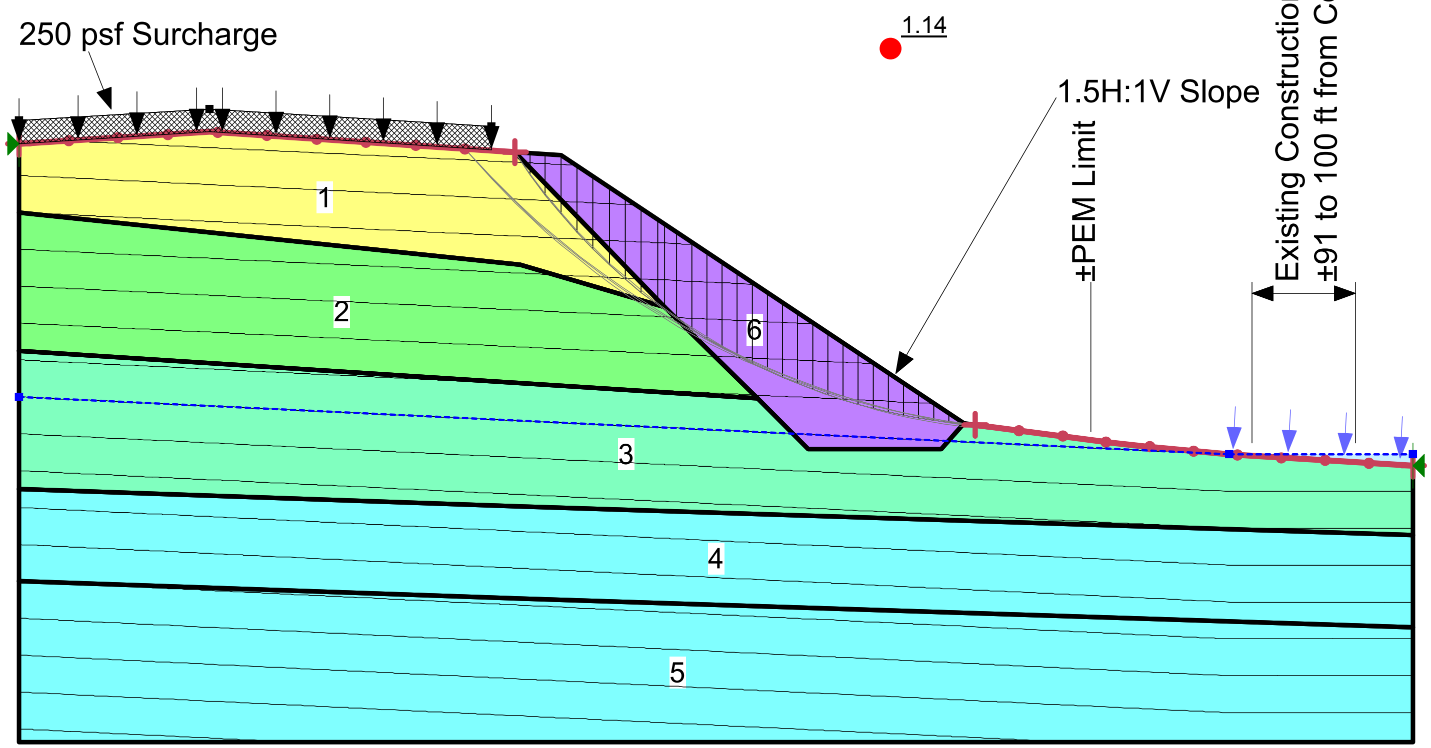
APPENDIX D

Evaluations

WIN 023026
Route 4 Slope Stabilization
Turner, ME

1.5:1 Unreinforced Slope

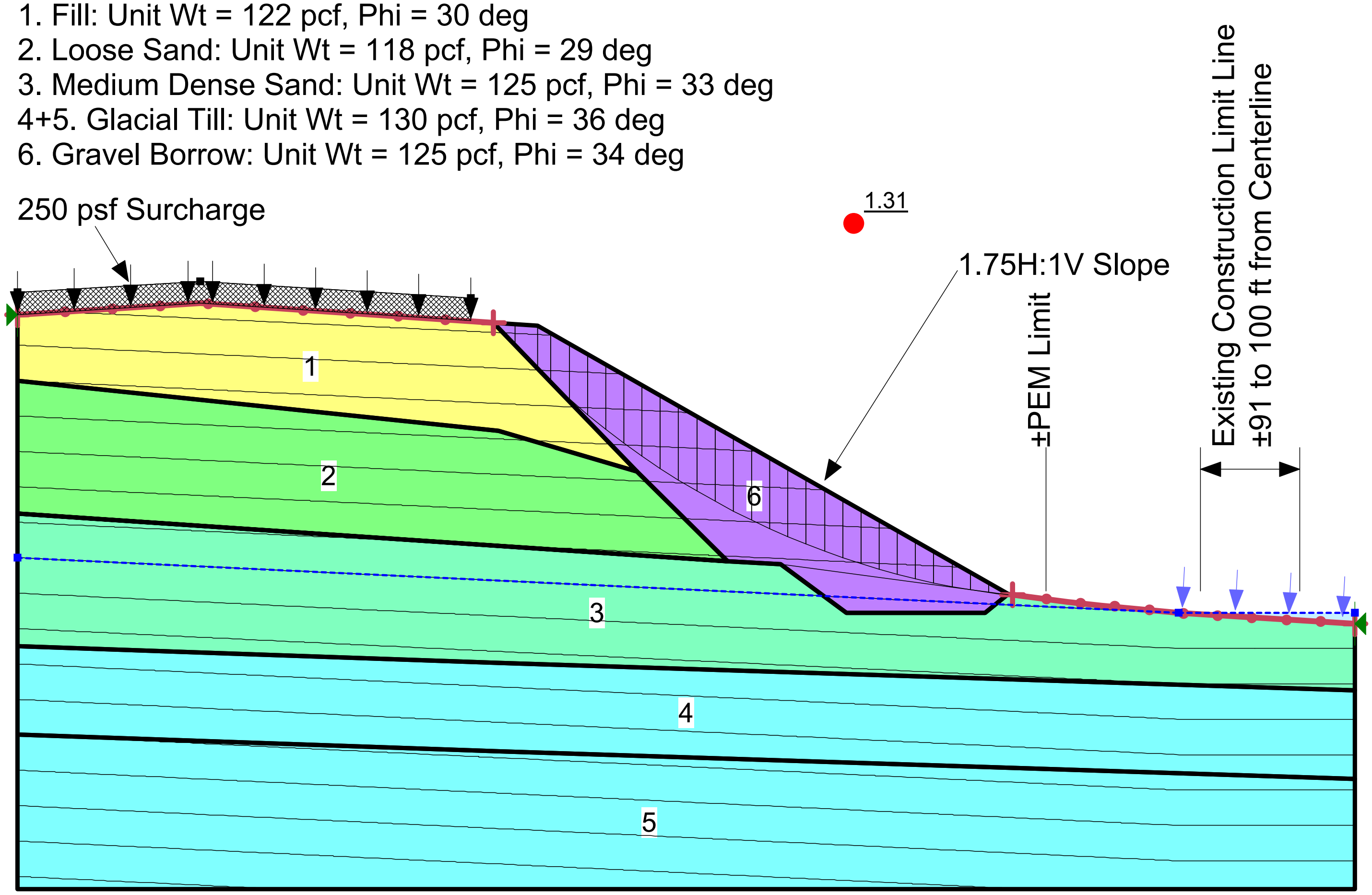
Materials:
1. Fill: Unit Wt = 122 pcf, Phi = 30 deg
2. Loose Sand: Unit Wt = 118 pcf, Phi = 29 deg
3. Medium Dense Sand: Unit Wt = 125 pcf, Phi = 33 deg
4+5. Glacial Till: Unit Wt = 130 pcf, Phi = 36 deg
6. Unreinforced Fill: Unit Wt = 125 pcf, Phi = 34 deg



WIN 023026
Route 4 Slope Stabilization
Turner, ME

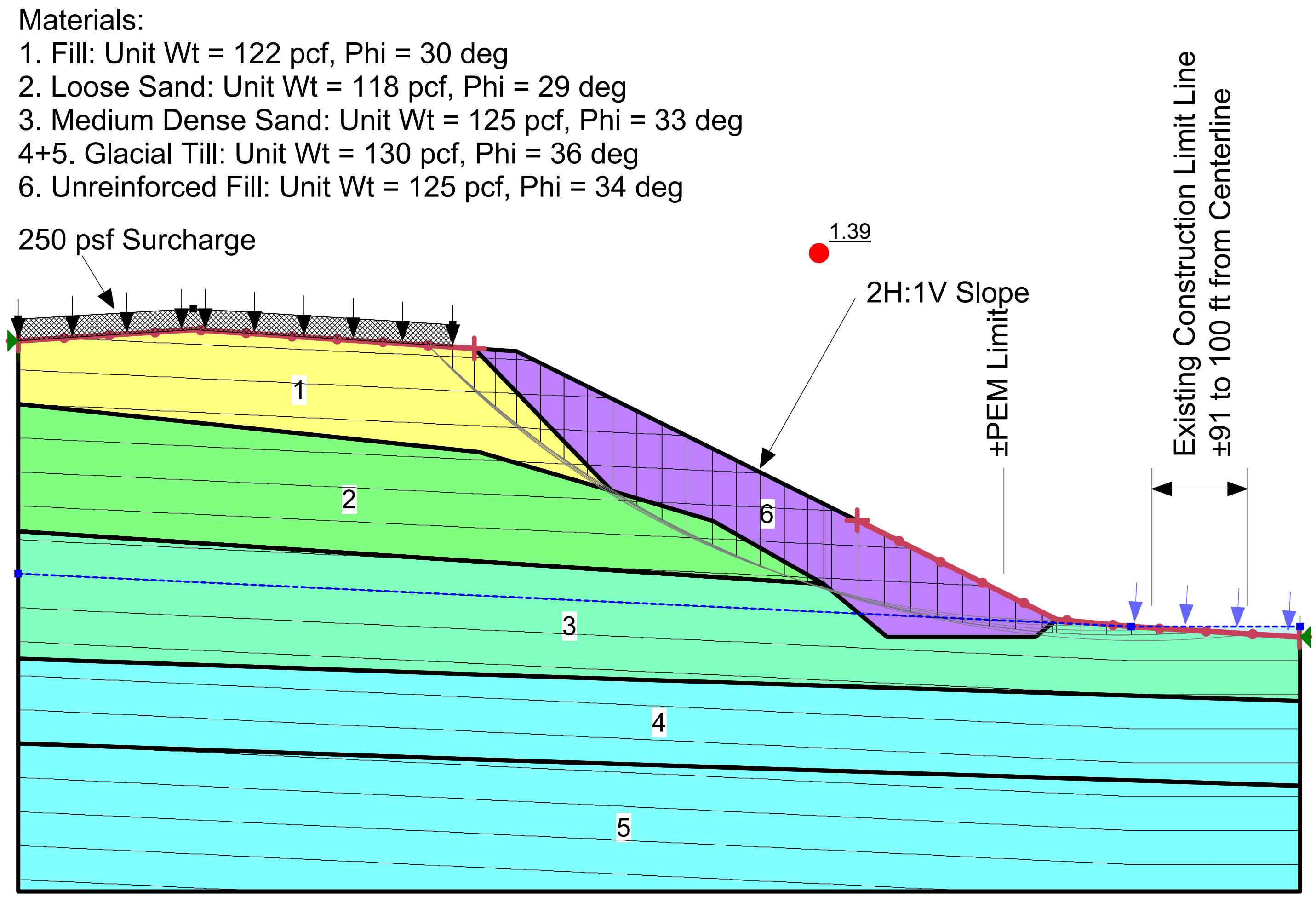
1.75:1 Unreinforced Slope

Materials:
1. Fill: Unit Wt = 122 pcf, Phi = 30 deg
2. Loose Sand: Unit Wt = 118 pcf, Phi = 29 deg
3. Medium Dense Sand: Unit Wt = 125 pcf, Phi = 33 deg
4+5. Glacial Till: Unit Wt = 130 pcf, Phi = 36 deg
6. Gravel Borrow: Unit Wt = 125 pcf, Phi = 34 deg



WIN 023026
Route 4 Slope Stabilization
Turner, ME

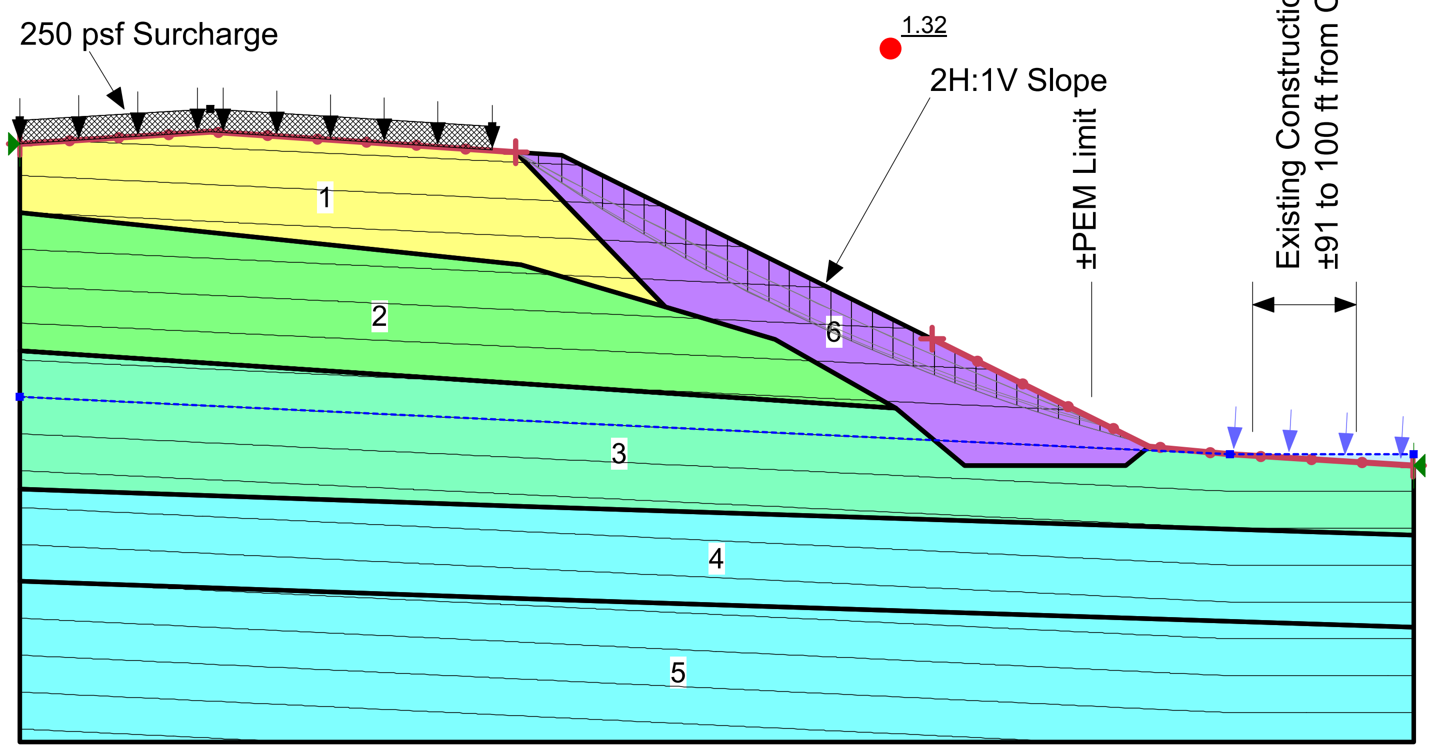
2:1 Unreinforced Slope



WIN 023026
Route 4 Slope Stabilization
Turner, ME

2:1 Unreinforced Slope

Materials:
1. Fill: Unit Wt = 122 pcf, Phi = 30 deg
2. Loose Sand: Unit Wt = 118 pcf, Phi = 29 deg
3. Medium Dense Sand: Unit Wt = 125 pcf, Phi = 33 deg
4+5. Glacial Till: Unit Wt = 130 pcf, Phi = 36 deg
6. Unreinforced Fill: Unit Wt = 125 pcf, Phi = 32 deg

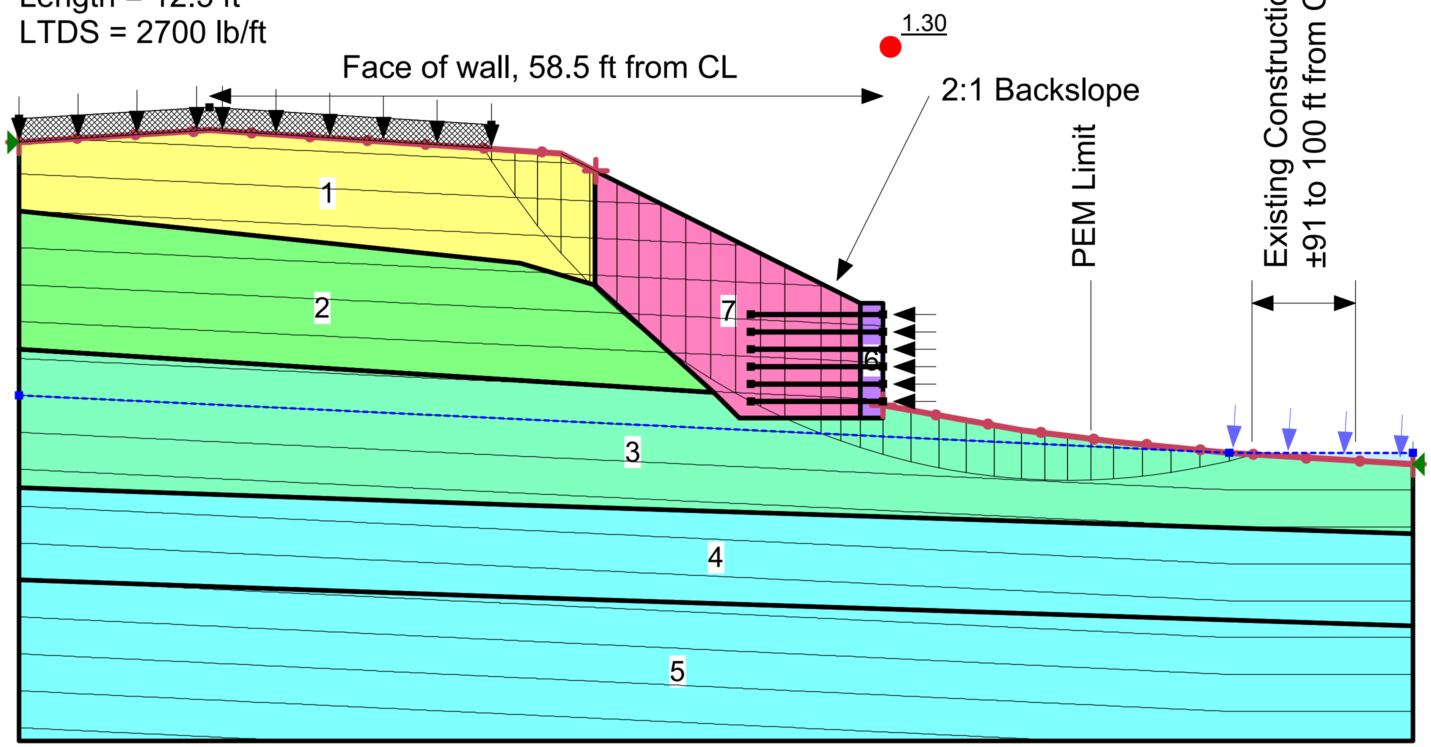


WIN 023026
Route 2 Slope Stabilization
Turner, ME

10.5 ft High SRW

Materials:
1. Fill: Unit Wt = 122 pcf, Phi = 30 deg
2. Loose Sand: Unit Wt = 118 pcf, Phi = 29 deg
3. Medium Dense Sand: Unit Wt = 125 pcf, Phi = 33 deg
4+5. Glacial Till: Unit Wt = 130 pcf, Phi = 36 deg
6. SRW Units: Unit Wt = 145 pcf, High Strength
7. Reinforced Fill: Unit Wt = 125 pcf, Phi = 34 deg

Reinforcement Geotextile: Mirafi Miragrid 5XT
Length = 12.5 ft
LTDS = 2700 lb/ft

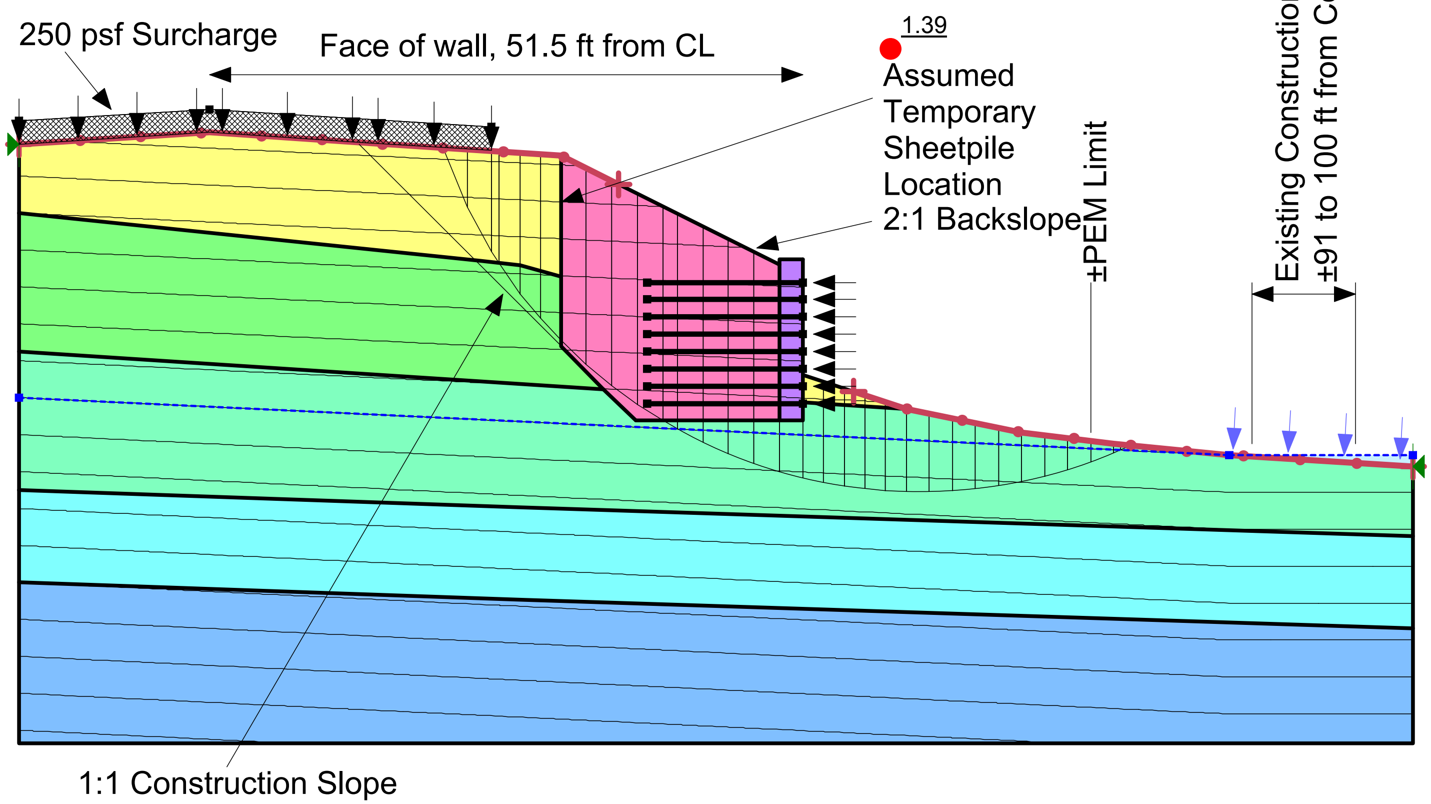


Materials:
1. Fill: Unit Wt = 122 pcf, Phi = 30 deg
5. Loose Sand: Unit Wt = 118 pcf, Phi = 29 deg
2. Medium Dense Sand: Unit Wt = 125 pcf, Phi = 33 deg
3. Glacial Till: Unit Wt = 130 pcf, Phi = 36 deg
4. Bedrock: Impenetrable
6. SRW Units: Unit Wt = 145 pcf, High Strength
7. Reinforced Fill: Unit Wt = 125 pcf, Phi = 34 deg

WIN 023026
Route 2 Slope Stabilization
Turner, ME

13.5 ft High SRW

Reinforcement Geotextile: Mirafi Miragrid 5XT
Length = 13.5 ft
LTDS = 2700 lb/ft

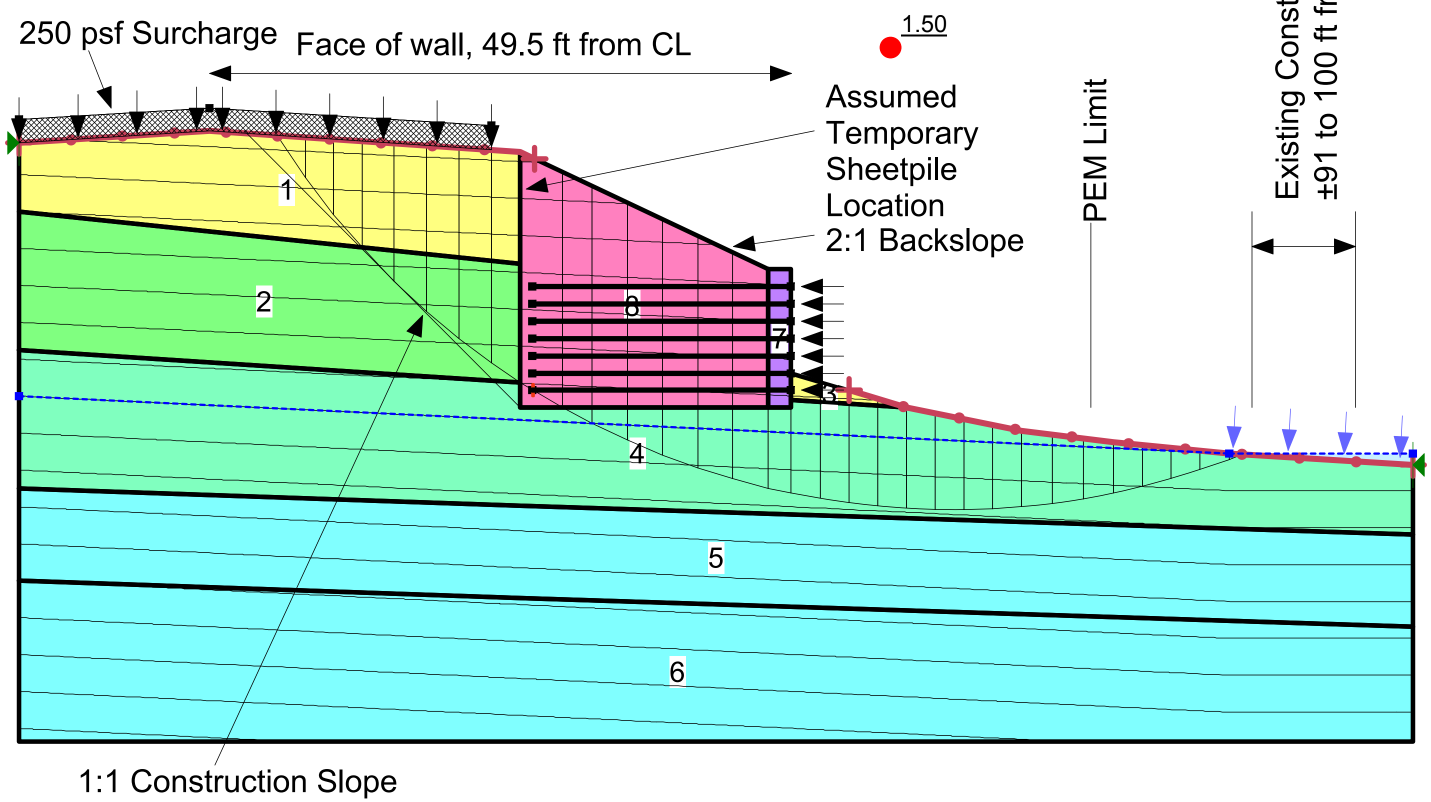


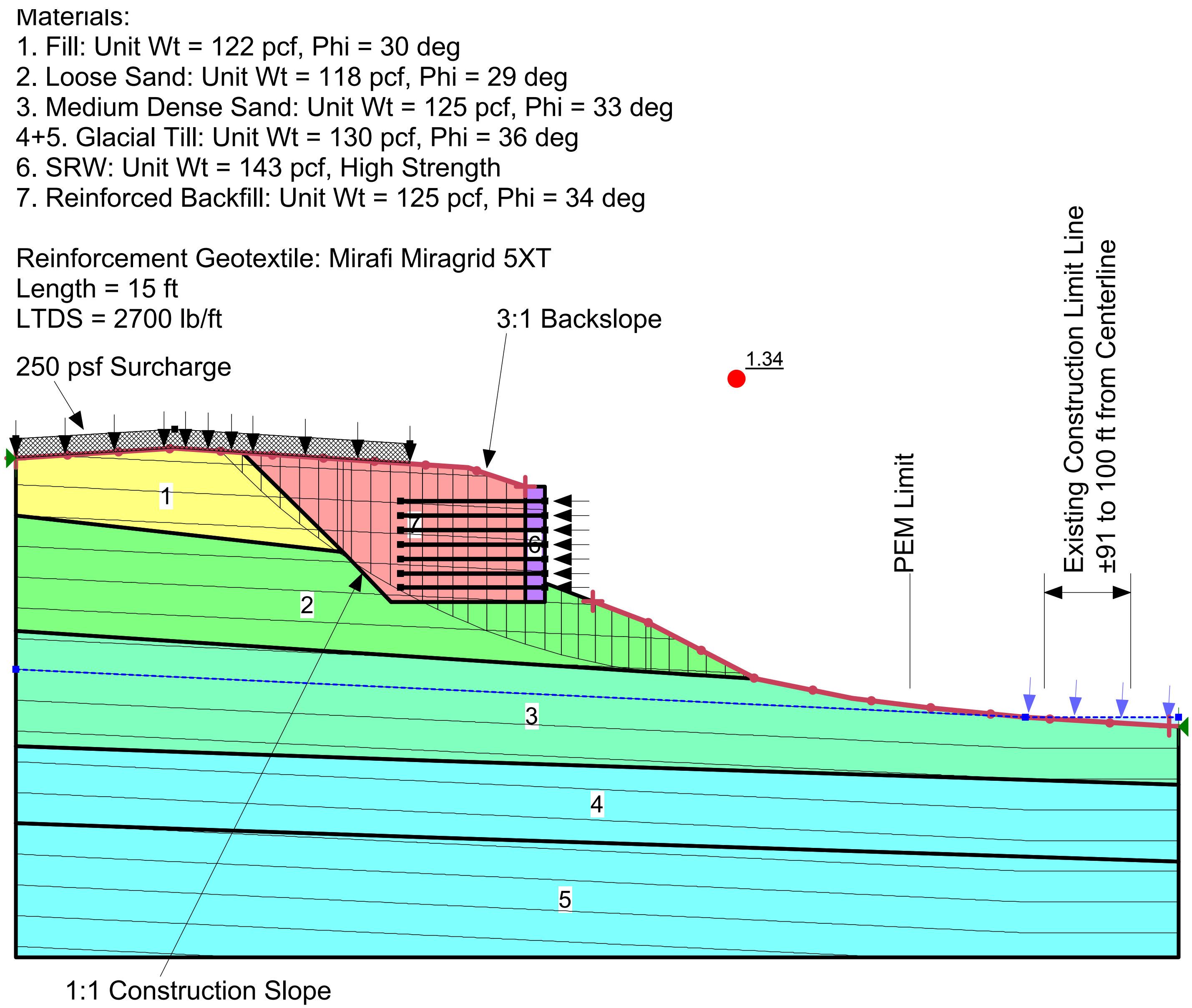
WIN 023026
Route 4 Slope Stabilization
Turner, ME

13.5 ft High SRW

Materials:
1+3. Fill: Unit Wt = 122 pcf, Phi = 30 deg
2. Loose Sand: Unit Wt = 118 pcf, Phi = 29 deg
4. Medium Dense Sand: Unit Wt = 125 pcf, Phi = 33 deg
5+6. Glacial Till: Unit Wt = 130 pcf, Phi = 36 deg
7. SRW Units: Unit Wt = 145 pcf, High Strength
8. Reinforced Fill: Unit Wt = 125 pcf, Phi = 34 deg

Reinforcement Geotextile: Mirafi Miragrid 5XT
Length = 22.5 ft
LTDS = 2700 lb/ft




Materials:
1. Fill: Unit Wt = 122 pcf, Phi = 30 deg
2. Loose Sand: Unit Wt = 118 pcf, Phi = 29 deg
3. Medium Dense Sand: Unit Wt = 125 pcf, Phi = 33 deg
4+5. Glacial Till: Unit Wt = 130 pcf, Phi = 36 deg
6. SRW: Unit Wt = 143 pcf, High Strength
7. Reinforced Backfill: Unit Wt = 125 pcf, Phi = 34 deg
Reinforcement Geotextile: Mirafi Miragrid 5XT
Length = 15 ft
LTDS = 2700 lb/ft
3:1 Backslope
250 psf Surcharge
1.34
Existing Construction Limit Line
±91 to 100 ft from Centerline
PEM Limit
1
7
6
2
3
4
5
1:1 Construction Slope

Bearing Resistance

Foundation Soil Parameters: med. dense to very dense, SAND, some gravel, little silt (SW-SM)

$\gamma_{sat} := 120\ pcf$ Saturated Unit Weight

$\gamma_{moist} := 118\ pcf$ Moist Unit Weight

$\phi := 29\ deg$ Undrained Friction Angle

$c_s := 0\ psf$ Undrained Shear Strenght

$\gamma_w := 62.4\ pcf$ Unit Weight of Water

Foundation Patameters:

$$B := \begin{bmatrix} 9.0 \\ 10.5 \\ 12.0 \\ 13.5 \\ 15.0 \\ 16.5 \end{bmatrix} ft$$ Foundation Width(s)

$D_f := 2\ ft$ Embedment Depth 2 ft (minimum)

$D_w := 0\ ft$ Depth of Water Below Foundation

Nominal Bearing Resistance - Service Limit State

From AASHTO LRFD Table 10.6.2.6.1-1, Presumptive Bearing Resistance for Spread Footing Foundations at the Service Limit State Modified after U.S. Department of the Navy (1982)

Bearing Material: fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)

Consistency in Place: loose

Bearing Resistance Range: 2 to 4 ksf

Recommended Bearing Resistance: 3 ksf

For loose, SAND, some to trace silt, trace gravel
Use Bearing Resistance of 3 ksf to limit settlement for Service Limit

Calculated by: MAS
Date: 09/13/2017
Checked by: TJB
Date: 09/15/2017

Nominal Bearing Resistance - Strength Limit State

Use Terzaghi Bearing Method for Strip Foundations since L>B

Reference: Bowles (1996) Foundaton Analysis and Design, 5th Ed.

Shape Factors for strip footing (From Table 4-1, page 220):

$s_c := 1$ $\qquad$ $s_\gamma := 1$

Meyerhof Bearing Factors (From Table 4-4, page 223):

$N_c := 27.96$ $\qquad$ $N_q := 16.55$ $\qquad$ $N_\gamma := 13.45$

Nominal Bearing Resistance (Terzaghi Equation, Bowles Table 4-1, page 220)

$q := D_f \cdot (\gamma_{moist}) = 236 \ psf$

$q_n := c_s \cdot N_c \cdot s_c + q \cdot N_q + 0.5 \cdot (\gamma_{sat} - \gamma_w) \cdot B \cdot N_\gamma \cdot s_\gamma$

$$q_n = \begin{bmatrix} 7.4 \\ 8 \\ 8.6 \\ 9.1 \\ 9.7 \\ 10.3 \end{bmatrix} ksf \quad \text{for} \quad B = \begin{bmatrix} 9 \\ 10.5 \\ 12 \\ 13.5 \\ 15 \\ 16.5 \end{bmatrix} ft$$

Nominal Bearing Resistance(s)

Factored Bearing Resistance

From AASHTO LRFD Table 10.5.5.2.2-1, Resistance Factor for Geotechnical Resistance of Shallow Foundations at the Strength Limit State

$\varphi_b := 0.45$

$$q_r := \varphi_b \cdot q_n = \begin{bmatrix} 3.3 \\ 3.6 \\ 3.8 \\ 4.1 \\ 4.4 \\ 4.6 \end{bmatrix} ksf \quad \text{for} \quad B = \begin{bmatrix} 9 \\ 10.5 \\ 12 \\ 13.5 \\ 15 \\ 16.5 \end{bmatrix} ft$$

Use 3 ksf for Strength Limit Factored Bearing Resistance

Calculated by: MAS
Date: 09/13/2017
Checked by: TJB
Date: 09/15/2017