April 10, 2025
Subject: Eastern Trail WIN 019426.00 & 019426.10
Location: Scarborough and South Portland
Amendment No. 1

Dear Sir/Ms.:

Please make the following changes to bid documents:

In the Bid Book:

REMOVE section 530.08 Warranty and **REPLACE** with section 530.08 "The bridge manufacturer shall warranty their steel structure(s) to be free of design, material and workmanship defects for a period of ten years from the date of delivery. Wood decking and wood attachments shall carry a five year warranty against rot, termite damage, or fungal decay.

In the Plans:

Remove 48 hours under Pile Notes note 7, **Replace** with 24 hours.

The following questions have been received:

Question: Can the project geotechnical report be provided?

Response: Yes, the geotechnical report will be added to the bid documents.

Question: Can the Exploration Location Plan as referenced in the Boring Logs be provided?

Response: Yes, the Exploration Location Plan will be provided and is included in the project geotechnical report.

Question: Would 24 hour restrikes on the test piles be acceptable?

Response: A minimum of 24 hours is acceptable for restrike tests.

Question: Can you provide existing and design elevations for the site from station 104+50 to 116+00 and 126+50 to 183+00?

Response: Design profiles for the station ranges referenced above were not developed for the plans. Intent is for the Contractor to follow parameters shown on Trail Typical Sections sheet 3 of the plans.

Question: Special provision section 530.032 (Bridge Requirements, bullet one) indicates that the Nonesuch Crossing should have a top chord that is cambered along a circular arch, however the plans show the bridge with parallel top and bottom chords. Please clarify.

Response: The top and bottom chord shall be cambered along a circular arch, following the camber requirements of Special Provision Section 530.032.

Question: Special provision section 530.032 (Bridge Requirements, bullet one) require verticals to be plumb. The bridge is on a slope, is it acceptable for the verticals to be perpendicular to the bottom chord? This is customary in prefabricated truss design and fabrication, but can be built plumb if you would like.

Response: Yes, it is acceptable for the verticals to be perpendicular to the bottom chord.

Question: Special provision section 530.032 (Bridge Requirements, bullet four) indicates that the bridges are to be designed utilizing a nested floor beam (floor beam to fit within the trusses) as shown on the plans. The plans show the floor beams framing into the bottom chords for both structures. This will work for the CSX railroad structures as they are designed as through trusses, but not the Nonesuch Crossing structure. For the Nonesuch Crossing structure, we can nest the floor beams as shown on the plans, but would also prefer to have the option to situate them below the bottom chord in an underhung configuration as it would result in a similar depth and also may reduce price and weight. Please confirm that both of these conditions are acceptable.

Response: The floor beams shall remain nested as required in Special Provision 530.

Question: Special provision section 530.038 (Warranty) indicates that wood decking shall carry a ten-year warranty against rot, termite damage, or fungal decay. The industry standard is to carry a 10-year warranty for hardwood materials, but not softwood materials like what is required per this project. Can the term of this warranty be reduced or eliminated entirely?

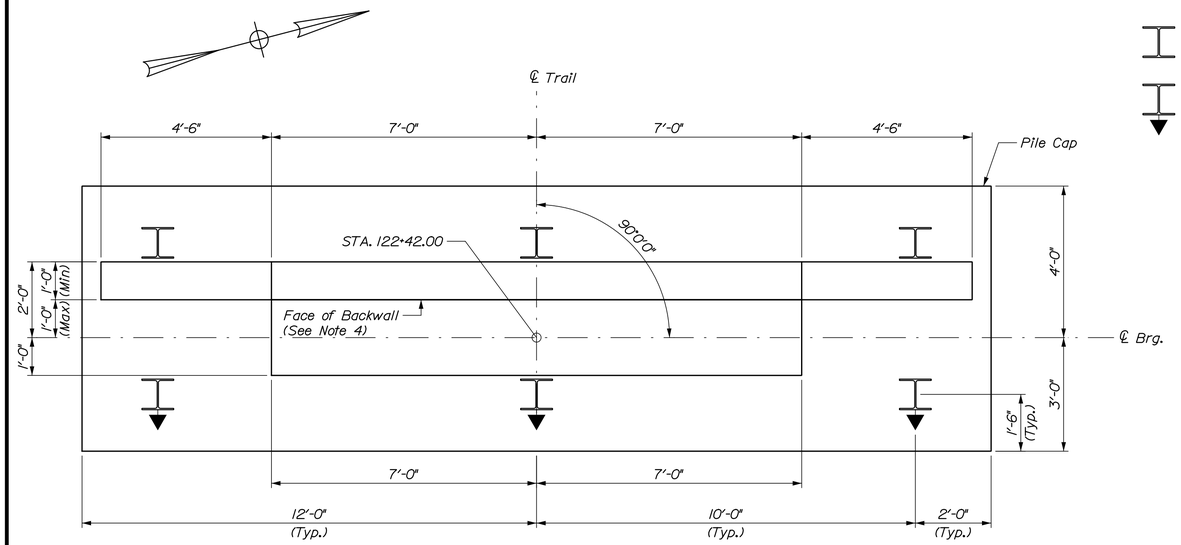
Response: The warranty for timber against rot, termite damage, and fungal decay shall be reduced from 10-years to 5-years.

Question: Sheet 29 of 69 shows 22’-6” track clearance is required over the track, what is the width from centerline of track the clearance needs to be maintained, as well as the track elevation?

Response: The required clearance envelope is depicted in Figure 2-1 of the MaineDOT Bridge Design Guide located on their website. In general, the 22’-6” vertical clearance is required for a 10’-0” horizontal dimension, centered on each track (5’-0” each side of the track). The track elevation beneath the bridge is EL 34.1 (western-most rail) and EL 34.5 (eastern-most rail).

Question: Does 107’-0” length of protective screening on the CSX Railroad crossing need to be installed on only the sides of the truss, or will it be required over the top as well?

Response: Protective screening shall be provided over the top as well.

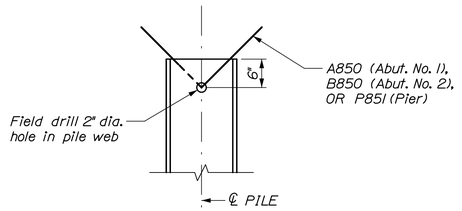


ABUTMENT NO. 1 PLAN

LEGEND

= Plumb Pile

= Battered Pile (1H:4V)



PILE ANCHORAGE DETAIL

ABUTMENT NOTES:

1. Reinforcing steel shall have a minimum concrete cover of 2 inches in the walls and 3 inches in the footings unless otherwise noted.

2. Place 4-in. diameter drains in the breastwall and wingwalls at 10-ft maximum spacing. The exact location will be determined by the Resident.

3. Cover joints where waterstops are not required in accordance with Standard Details Section 502.

4. The face of backwall shall be shifted to accomodate the Prefabricated Steel Truss dimensions and joint opening, as determined by the manufacturer. Location of piles, footing, ℄ bearing, face of seat, and back of backwall shall not be adjusted.

5. All concrete shall be Class A, and shall be paid under Item 502.219, Structural Concrete, Abutments and Retaining Walls.

6. Bridge seat elevations are approximate and shall be confirmed by the Contractor based on final bridge and bearing details. Refer to Special Provision 530 for additional guidance.

PILE NOTES:

1. The maximum factored pile load is:

   Abutment No. 1 & 2: 365 kips (including 190 kips allowed for downdrag)
   Pier: 145 kips

2. Piles Shall be driven to the following nominal driving resistances:

   Abutment No. 1 & 2: 560 kips
   Pier: 225 kips

3. H-pile material shall be ASTM A572, Grade 50.

4. Estimate of piles required:

   Abutment No. 1: 6 ~ HP 12x74 @ 130 feet
   Abutment No. 2: 6 ~ HP 12x74 @ 110 feet
   Pier: 8 ~ HP 12x74 @ 95 feet

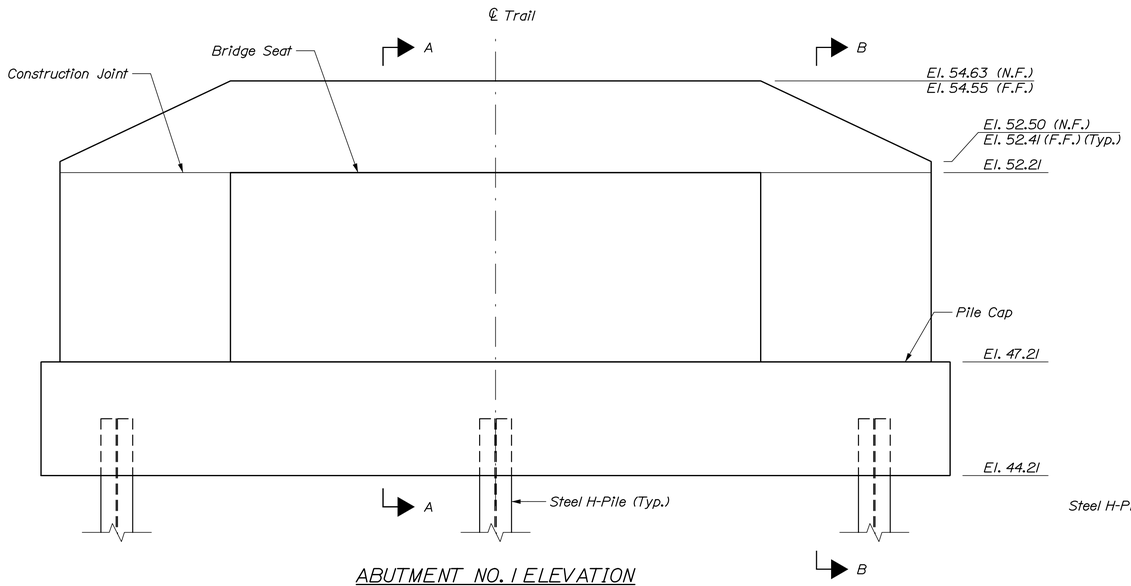
5. All piles shall be equipped with a pile tip in accordance with Standard Specifications Section 501.048, Prefabricated Pile Tips.

6. The Contractor shall perform and submit a wave equation analysis for review and acceptance by the Resident. The maximum allowable driving stress is 0.90 times Fy. The submittal analyses shall include the proposed stopping criteria based on the wave equation analysis and the proposed driving system. The stopping criteria shall include the blows per inch and the number of 1-in. intervals at which pile installation may be terminated. The cost of performing the wave equation analysis will be considered incidental to Item No. 501.92, Pile Driving Equipment Mobilization.

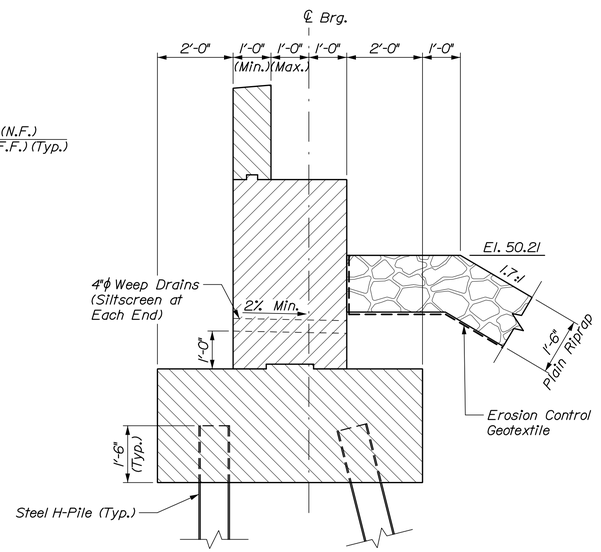
7. The Contractor shall perform 3 dynamic load tests, one at each substructure location, to confirm the nominal driving resistances have been met. The dynamic tests shall be performed on the first production pile driven at each substructure. Minimum 24 hour pile restrikes shall be conducted on all test piles in order to ensure the required nominal resistance has been achieved and verify pile relaxation has not occurred. The Contractor may drive production piles to the preliminary driving criteria, however pile cut-off will not be permitted until completion of final driving criteria.

8. All pile heads to be constructed with an anchorage dowel in accordance with the Pile Anchorage Detail, this sheet. Payment for drilling through the pile webs will not be made separately and will be considered incidental to Item 501.481, Steel H-Beam Piles, 74 lbs/ft, in place.

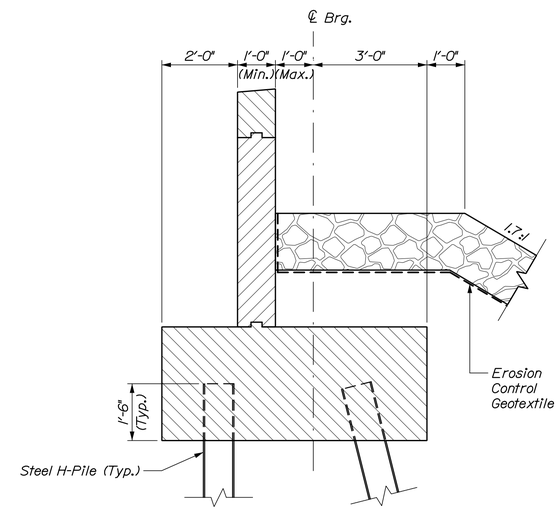
9. Abutment No. 2 piles may be driven before or after partial construction of the MSE Retaining Wall. If the piles are to be driven after partial construction of the MSE Retaining Wall, blockouts for pile installation shall be provided through the use of 2-ft diameter corrugated PVC sleeves and the annulus between the pile and sleeve shall be backfilled with material meeting the requirements of 703.02, Coarse Aggregate for Concrete, Class Latex. Payment for sleeves and sleeve backfill will not be made separately and will be considered incidental to Item 501.481, Steel H-Beam Piles, 74 lbs/ft, in place.



ABUTMENT NO. 1 ELEVATION



SECTION A-A



SECTION B-B

HNTB

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

AC-STP-1942(600)X & CMAQ-1942(610)

WIN
19426.00 & 019426.10 BRIDGE PLANS

	BIL. REICHL	BY	DATE	SIGNATURE
PROJ. MANAGER				
DESIGN-DETAILED	J. Olund	P. Bishop	3\2025	
CHECKED-REVIEWED	G. Standley	A. Stephens	3\2025	
DESIGN2-DETAILED2	-	-	-	-
DESIGN3-DETAILED3	-	-	4/9/2025	P.E. NUMBER
REVISIONS 1	Abutment Notes			
REVISIONS 2	-	-	-	DATE
REVISIONS 3	-	-	-	
REVISIONS 4	-	-	-	
FIELD CHANGES	-	-	-	

TOWN OF SCARBOROUGH
EASTERN TRAIL CONNECTOR
SCARBOROUGH TO SOUTH PORTLAND, MAINE

CSX RAILROAD
ABUT. NO. 1 PLAN & ELEV.

SHEET NUMBER
47
OF 69

Date:4/9/2025
Username:
Division:
Filename: 047_CSX Abutment 1.dgn

R.W. Gillespie & Associates, Inc.
Geotechnical Engineering • Environmental Consulting • Materials Testing Services

08 February 2019

Timothy Cote, P.E.
HNTB Corporation
340 Country Road, Suite 6-C
Westbrook, Maine 04092

VIA EMAIL: tcote@hntb.com

Subject: Report of Geotechnical Evaluation
Scarborough Eastern Trail, MaineDOT Project No. STP-1942(610)
Scarborough, Maine
RWG&A Project No. 0600-102

Dear Mr. Cote:

R. W. Gillespie & Associates, Inc., (RWG&A) is pleased to present the results of the geotechnical evaluation related to a proposed Eastern Trail pedestrian-bicycle bridge over the Nonesuch River and the above grade Pan-Am Railroad crossing in Scarborough, Maine. This work was performed in general accordance with Task Order Nos. 1 and 2 dated 25 April 2016 and 30 May 2018, respectively, associated with the Master/Task Order Agreement dated 25 April 2016 (HNTB Project 66807). The purpose of this geotechnical evaluation was to explore and evaluate subsurface conditions in the area of the planned Nonesuch River approach embankment and Pan-Am Railroad crossing, and to provide evaluations and recommendations for use in design of the embankments and the bridge foundations.

We have enjoyed working with HNTB Corporation on this project. Please contact us if you have any questions or if we may be of further service.

Sincerely,
R. W. GILLESPIE & ASSOCIATES, INC.

Serena Pape
Geotechnical Engineer

Erik J. Wiberg, P.E.
Chief Geotechnical Engineer

Marc R. Grenier, P.E.
Technical Reviewer

EJW/SNP:ks

G:\PROJECTS\0600\0600-102-16\Reports\2019-02-08 Final GI Report 0600-102.docx

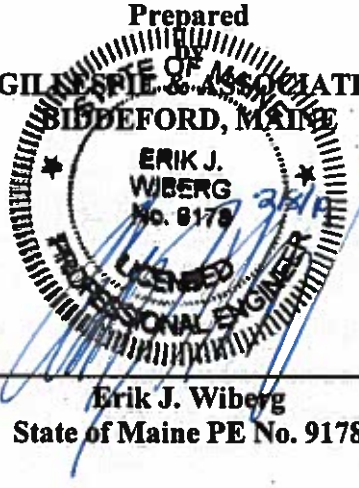
R.W. Gillespie & Associates, Inc.

Report
of
GEOTECHNICAL EVALUATION

for
SCARBOROUGH EASTERN TRAIL,
MAINEDOT PROJECT NO. STP-1942(610)
SCARBOROUGH, MAINE

Prepared
for
HNTB CORPORATION
WESTBROOK, MAINE

Prepared
by
R. W. GILLESPIE & ASSOCIATES, INC.
BIDDEFORD, MAINE



Erik J. Wiberg
State of Maine PE No. 9178

RWG&A Project No. 0600-102

08 February 2019

EXECUTIVE SUMMARY

This report presents the results of a geotechnical evaluation for the Scarborough Eastern Trail project in Scarborough, Maine (MaineDOT Project No. 1942(610)). The purpose of this geotechnical evaluation was to make a visual condition assessment of the existing Nonesuch River approach south embankment, to explore and evaluate subsurface conditions in the area of the planned Nonesuch River north embankment and the planned Pan-Am Railroad crossing, and to provide evaluations and recommendations for use in design of the approach embankments and railroad crossing bridge foundations.

R.W. Gillespie & Associates, Inc.'s scope of services and understanding of the proposed project are summarized in Section 1 of this report. The field exploration and laboratory testing programs are presented in Sections 2 and 3, respectively. Section 4 presents a summary of subsurface conditions encountered at the planned Pan-Am approach embankment and bridge substructures. Sections 5 and 6 summarize evaluations and design recommendations for the Pan-Am Railroad crossing approach embankments and bridge substructures, respectively. Section 7 presents a summary of subsurface conditions encountered, evaluations, and recommendations for the Nonesuch River embankment.

The primary geotechnical considerations for the proposed project are slope stability of the approach embankments and settlement of the Pan-Am Railroad crossing bridge pier. Lower weight fill (i.e., open-graded crushed rock) with granular lightweight coarse aggregate such as expanded shale aggregate is recommended for portions of the railroad crossing approach embankments. Geofoam fill with granular borrow embankment fill is a lightweight fill alternative. Recommendations for the use of expanded shale and geofoam fills are provided in the report. The Nonesuch River approach embankment may be constructed using granular borrow.

Recommendations for end-bearing, pile-support of the bridge's stub abutments and pier are provided. Piles supporting the stub abutments may be located within the reinforced zone of mechanically stabilized earth fill. Design and construction of reinforced zones and pile-supported abutment will need to be coordinated with one another. Pipe sleeves around piles through reinforced zones of mechanically stabilized earth fill will be needed.

TABLE OF CONTENTS

1.0 INTRODUCTION 1

1.01 Background 1
1.02 Scope of Services 2

2.0 SUBSURFACE EXPLORATION 3

3.0 LABORATORY TESTING 4

4.0 PAN-AM RAILROAD CROSSING SUBSURFACE CONDITIONS 5

4.01 Subsurface Soils 5
4.02 Groundwater 6
4.03 Bedrock 6
4.04 Frost Depth 6
4.05 Seismic Design Parameters 7

5.0 PAN-AM RAILROAD CROSSING APPROACH EMBANKMENTS 7

5.01 General 7
5.02 Existing and Proposed Conditions 7
5.03 Overall Stability Evaluations 8
5.03.01 Service Limit State Stability Evaluations 8
5.03.02 Extreme Limit State Stability Evaluations 9
5.03.03 Summary of Stability Evaluations 9
5.04 Strength and Extreme Limit State Evaluations 10
5.04.01 Liquefaction Potential 10
5.04.02 Bearing Resistance 10
5.04.03 Sliding Resistance 11
5.05 Service Limit State Evaluation of Embankment Settlement and Lateral Movement 11
5.05.01 Settlement 11
5.05.02 Lateral Movement 12
5.06 Design Recommendations 12
5.06.01 Embankment - General 12
5.06.01.01 Lightweight Coarse Aggregate 12
5.06.01.02 Geofoam 13
5.06.02 Embankment Settlement and Pile Reliability Considerations 14
5.06.03 Embankment at West Bridge Stub Abutment 14
5.06.04 Retaining Wall 15
5.06.04.01 Wall Units and Backfill 15
5.06.04.02 Frost Protection 16
5.06.04.03 Wall Embedment 17
5.06.04.04 Wall Drainage 17
5.07 Construction Considerations 17
5.07.01 Site Preparation 17
5.07.02 Construction Dewatering 17
5.07.03 Temporary Excavations 18
5.07.04 Material Utilization and Site Filling 18

6.0 PAN-AM RAILROAD CROSSING BRIDGE FOUNDATIONS 19

6.01 General 19
6.02 Service Limit State Evaluation of Abutment and Pier Settlements 19
6.03 Driven Piles 20
6.03.01 Factored Abutment and Pier Bridge Reactions 20
6.03.02 Factored Pile Axial Compressive Resistance 20
6.03.03 Factored Pile Axial Uplift Resistance 22
6.03.04 Negative Skin Friction (Downdrag) 22
6.03.05 Lateral Loads Resistance 23
6.04 Seismic Design 23

6.05 Frost Protection 23
6.06 Construction Considerations 24

7.0 NONESUCH RIVER CROSSING 24

7.01 Subsurface Information 24
7.02 Observed Slope Conditions – South Embankment 25
7.03 Subsurface Conditions 25
7.03.01 Subsurface Soils 25
7.03.02 Groundwater 26
7.04 Overall Stability Evaluations 26
7.05 Embankment Settlement Evaluations 26
7.06 Construction Considerations 26

8.0 CLOSURE 27

FIGURES:

Figure 1. Locus Map
Figure 2. Exploration Location Plan – Pan-Am Railroad Crossing
Figure 3. Exploration Location Plan – Nonesuch River Embankment
Figure 4. Interpretive Subsurface Profile – Pan-Am Railroad Crossing
Figure 5. Recommended Embankment Fill Materials
Figure 6. Lightweight Embankment Fill Alternatives
Figure 7. Interpretive Subsurface Profile – Nonesuch River

APPENDICES:

Appendix A. Limitations
Appendix B. Exploration Logs
Appendix C. Laboratory Testing
Appendix D. Geotechnical Engineering Calculations

1.0 INTRODUCTION

1.01 Background

The proposed Eastern Trail project consists of a new recreational trail extending north from the east side of the Nonesuch River in Scarborough, Maine toward the Wainwright Recreational Complex in South Portland, Maine. The trail project will use a section of a former railroad track right-of-way to cross the Nonesuch River with a new bridge. The project will also require an above-grade crossing of the active Pan-Am Railroad right-of-way. Approach embankments and a two-span bridge are proposed. This report addresses geotechnical conditions related to the Nonesuch River approach embankments, and to the Pan-Am Railroad crossing bridge foundations and approach embankments.

R.W. Gillespie & Associates, Inc.'s (RWG&A's) understanding of the project is based on communications with HNTB Corporation (HNTB), site visits, and review of the following drawings:

- Preliminary plan set titled *STP-1938(600) Eastern Trail Multi-Use Recreational Trail Development, Project Length 1.6 mil., A Grading, Base, Surface, Trail and Bridge Project* provided by HNTB 31 July 2018.

- Sheet Nos. 22, 37, 38, and 39 of the plan set titled *Town of Scarborough, Eastern Trail Connector, Scarborough to South Portland, Maine*, *State of Maine Department of Transportation STP-1942600 and STP-1942610,* provided by HNTB 14 November 2018. The sheets show the Pan-Am railroad crossing abutment and pier plans.

The planned Pan-Am Railroad crossing will be located within the Central Maine Power transmission line easement off of Parkway Drive. Based on conceptual design information provided to RWG&A, the crossing will consist of a 220-foot long, 10-foot wide, two-span bridge with a pier at the east side of the railroad tracks. The minimum design vertical clearance between the railway and pedestrian-bicycle bridge is 22.5 feet. Approach embankments would have a top width of about 14 feet and would be up to 22 feet above current ground surface.

The railroad crossing's west approach embankment would be about 475 feet long and have side slopes pitched at 2 units horizontal to 1 unit vertical (2H:1V). The east embankment would have a Mechanically Stabilized Earth (MSE) Retaining Wall about 210 feet long to laterally support the embankment's north side. The top of the wall would be about 16 feet above adjacent ground surface at the east abutment. The south side (note: transmission line side) of the east embankment would be a fill slope pitched at 2H:1V.

The existing Nonesuch River embankments and abutments would be used to support a new pedestrian-bicycle bridge over the river. The new bridge would span about 34.5 feet. The south embankment is full height and would be used without modification, excepting trail base construction. The railroad's north embankment had been removed and will be rebuilt to accommodate the grade change between the abutment and new trail over a distance of about 200 feet. Up to about 10 feet of fill above current ground surface would be needed to construct the north embankment.

1.02 Scope of Services

The subject evaluation was performed to develop site-specific soil and laboratory data, and to make geotechnical evaluations for planning and final design of the proposed construction. Refer to Appendix A for use and limitations of this report. RWG&A conducted a preliminary geotechnical evaluation for use in planning of the project and summarized its findings and preliminary recommendations in the *Report of Preliminary Geotechnical Evaluation for Scarborough Eastern Trail, MaineDOT Project No. 019386.00, Scarborough, Maine*, dated 06 November 2013, RWG&A Project No. 0600-092. As performed, RWG&A's scope of services for the subject services included the following items:

Nonesuch River Crossing

- Made a site visit to observe conditions exposed at ground surface and conducted a visual observation of the south approach embankment.
- Reviewed readily available, published soils and geologic mapping information.
- Prepared a program of soil explorations and sampling. Marked explorations in the field and notified DigSafe. Arranged for and monitored soil borings drilled on the planned embankment at the north side of the Nonesuch River.
- Conducted geotechnical laboratory tests and evaluated data relative to the proposed embankment construction.

Pan-Am Railroad Crossing

- Made a site visit, reviewed project information and previous subsurface explorations, and reviewed readily available soils and geologic mapping information.
- Prepared a program of soil explorations and sampling. Marked proposed exploration locations in the field. Contacted DigSafe, OK-TO-DIG registered member public entities, and Central Maine Power to locate public utilities as-needed to conduct explorations.
- Arranged to have soil borings performed by a local drilling contractor. RWG&A provided technical monitoring of the exploration activities, sampling, and testing.
- Performed laboratory tests on selected soil samples recovered from the subsurface explorations to aid in soil description, and for determination of engineering properties needed for railroad foundation design and site development analysis.
- Evaluated acquired field, laboratory, and engineering information with respect to the proposed construction.
- Prepared this report presenting the findings, conclusions, and recommendations of the geotechnical evaluations.

RWG&A's scope of services excluded an environmental site assessment relative to oil and hazardous materials or evidence of a potential release or threat of oil or hazardous materials on, below, or around the site. Any statement in this report, or on the exploration logs, regarding odors or unusual or suspicious conditions is for informational purposes only and is not intended to constitute an environmental assessment. RWG&A's scope of services also excluded preparing contract drawings or technical specifications for construction.

Elevations used in this report were obtained or interpolated from the above noted drawings. It is understood elevations presented on the drawings are in units of feet and referenced to North American Vertical Datum of 1988 (NAVD 88).

2.0 SUBSURFACE EXPLORATION

Test borings were drilled as part of the preliminary geotechnical evaluation in 2013 and in 2016 and 2018 for the final geotechnical evaluation. As performed, the subsurface exploration program for the Scarborough Eastern Trail project consisted of the following test borings:

Boring Designation	Depth Advanced Below Local Ground Surface (feet)	Date Drilled	Termination Stratum
PAN-AM RAILROAD CROSSING			
BB-SPC-101	91.2	13 June 2013	Silty sand with gravel (SM)
BB-SPC-102	101.5	12 June 2013	Silty sand with gravel (SM)
BB-SPC-201	52.0	07 October 2016	Silty clay (CL)
BB-SPC-201A	133.8	15-18 June 2018	Bedrock
BB-SPC-202	136.0	04-06 October 2016	Bedrock
BB-SPC-203	151.2	27-30 September 2016	Bedrock
BB-SPC-203A	62.0	19 June 2018	Possible sand
EB-SPC-202	60.8	07 October 2016	Silty clay (CL)
EB-SPC-203	16.0	27-September 2016	Silty clay (CL)
EB-SPC-203A	65.0	19 June 2018	Possible sand
EB-SPC-204	60.0	20 June 2018	Possible sand
EB-SPC-205	57.0	20 June 2018	Sand with silt (SP-SM)
NONESUCH RIVER CROSSING			
EB-SNR-101	62.0	12 July 2016	Sand
EB-SNR-102	62.0	13 July 2016	Sand
EB-SNR-103	62.0	12 July 2016	Sand

Boring locations for the Pan-Am Railroad Crossing and Nonesuch River Crossing are shown on Figure 2 and 3, respectively.

The BB-series borings were drilled at the approximate locations of the anticipated pedestrian-bicycle bridge substructure locations when the borings were drilled. The EB-series borings were drilled along the alignment of approach embankments. The boring locations were marked in the field prior to drilling by tape survey methods from features exposed at ground surface including flagged trail centerline and shown on provided plans. Elevations were interpolated from

topographic mapping and centerline survey data. Exploration locations shown on Figures 2 and 3 are accurate only to the degree implied by the methods used to locate them.

The June 2013 and July 2016 borings were drilled by Great Works Pump and Test Boring, Inc. of Rollinsford, New Hampshire. The October 2016 and June 2018 borings were drilled by Northern Test Boring, Inc. of Gorham, Maine. The borings were advanced using hollow stem augers and rotary wash drilling methods. Standard penetration tests (ASTM D1586, *Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils*) were typically performed at 2 to 5-foot intervals. Closer interval spacing was used in the 2016 and 2018 borings in the upper part of the soil profile. In lieu of standard penetration test sampling, field vane shear strength tests were conducted in silty clay deposits in general accordance with ASTM D2573, *Standard Test Method for Field Vane Shear Test in Cohesive Soil.*

Field vane shear strength tests were conducted using an Acker field vane in the 2013 Pan-Am crossing borings and in the Nonesuch River embankment borings; the Maine Department of Transportation's Geonor field vane was used in the 2016 and 2018 Pan-Am railroad crossing borings. Thin-walled tube samples of silty clay soils were taken at using a piston sampler (ASTM D1587, *Standard Practice for Thin-Walled Tube Geotechnical Sampling of Soils*). Upon completion of drilling and sampling, boreholes were backfilled to ground surface with cuttings extracted from the boreholes.

Exploration activities were observed by an RWG&A representative who prepared the exploration logs presented in Appendix B, *Exploration Logs*. The soils were described in general accordance with ASTM D2488, *Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)*. Stratification lines shown on the exploration logs represent the estimated boundaries between the different soil types encountered and approximate refusal depths; the actual transitions may be more gradual and can vary over short distances.

3.0 LABORATORY TESTING

Laboratory testing was performed to assist in soil description and estimation of engineering properties of encountered soils. The laboratory testing program included moisture content determinations, grain-size analysis, liquid and plastic Atterberg Limits tests, measurements of undrained shear strength using a Geonor vane apparatus, and consolidation tests using thin-walled tube samples. The tests were performed in general accordance with the following methods and procedures:

- ASTM D2216, *Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass*.
- ASTM D1140 – 17, *Standard Test Methods for Determining the Amount of Material Finer than 75-µm (No. 200) Sieve in Soils by Washing.*
- ASTM D6913/6913M – 17, *Standard Test Methods for Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis.*

- ASTM D4318, *Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils.*

- ASTM D2435, *Standard Test Method for One-Dimensional Consolidation Properties of Soils.*

The laboratory Geonor vane shear tests were performed in accordance with the manufacturer's recommended procedures. Moisture content test results are presented on the soil boring logs. Results of the other laboratory tests are presented in Appendix C, *Laboratory Testing*. All tests were conducted at the RWG&A soil and materials testing laboratory, which is accredited by the American Association of State Highway and Transportation Officials (AASHTO) for the tests performed.

4.0 PAN-AM RAILROAD CROSSING SUBSURFACE CONDITIONS

4.01 Subsurface Soils

Soil borings were drilled at the planned east and west abutment substructures and along the planned embankment alignments. Five distinct soil units were encountered: fill, near surface silty sand alluvium, silty clay glacial marine deposits, silty sand marine deposits, and silty sand with gravel glacial till deposits. Generalized interpreted descriptions with encountered thicknesses are summarized below and illustrated on Figure 4, *Interpreted Subsurface Profile – Pan-Am Railroad Crossing.* Refer to the exploration logs in Appendix B for detailed descriptions at specific locations.

Soil Unit	Encountered Thickness (feet)	Generalized description	Comments
Fill	3	Fine to medium sand with little to some silt, trace gravel, gray to dark reddish brown	Encountered at boring BB-101
Topsoil	0.2 to 0.3	Silty sand with organic matter	
Alluvium Deposits	10.5 to 14.8	Very loose to medium dense, fine to medium sand, little to some silt, orange-brown to gray	
Glacial Marine Deposits	35 to 56.5	Very soft to medium, silty clay, trace fine sand, gray	Contains occasional seams of silt and fine sand
Marine Deposits	18-29.5	Medium dense to dense, fine to medium sand with little silt and trace coarse sand	
Glacial Till Deposits	31.2 to 49	Dense to very dense, silty sand with gravel	Total thickness determined at BB-201A, BB-202 and BB-203

The upper part of the silty clay deposits was very soft to soft and became medium stiff with increasing depth. Undrained shear strengths determined from field vane tests ranged from

approximately 800 pounds per square foot (psf) in the upper few feet of the deposits, about 400 psf at a depth of 30 feet below ground surface, and about 800 psf at a depth of 65 feet.

4.02 Groundwater

Free water levels were observed approximately 5 to 8 feet below ground surface about one hour after removal of drilling equipment, and might not be representative of stabilized groundwater levels when the borings were drilled. The *Soil Survey of Cumberland County Maine* by the United States Department of Agriculture Soil Conservation Service indicates seasonal high groundwater level of nearby mapped soil units is 1.5 to 2 feet below ground surface. In general, groundwater levels will fluctuate due to water level in streams, season, temperature, rainfall, and elevation, and will vary from conditions observed on the dates the explorations were made.

4.03 Bedrock

Bedrock was encountered at depths of about 124 feet, 126 feet, and 141 feet in borings BB-201A, BB-202, and BB-203, respectively; the depth to bedrock increased from east to west. Based on recovered rock core samples, the encountered bedrock generally consisted of dark gray, micaceous, highly to slightly weathered aphanitic to fine-grained pelite. The rock ranged from slightly weathered to fresh, soft to hard, and moderately to extremely fractured. Joint surfaces ranged from low to high angle and weathered to fresh. The unconfined compressive strength of intact rock core samples from borings BB-SPC-203 and BB-SPC-201A ranged from about 500 to 6,000 pounds per square inch.

The rock quality designation (RQD) of a core sample provides an estimate of the in-place bedrock quality; see table below developed by Deere (1968). RQD is calculated by dividing the sum of core pieces 4 inches, or greater, in length by the total length of core run. The RQD of the cored rock ranged between 0 to 25 percent. Please refer to the exploration logs for RQD values at specific locations.

RQD (%)	Bedrock Quality
90 to 100	Excellent
75 to 90	Good
50 to 75	Fair
25 to 50	Poor
0 to 25	Very Poor
Source: Engineer Manual 1110-1-2908, Engineering and Design, Rock Foundations	

4.04 Frost Depth

According to Modberg Software, the site has a design-freezing index of approximately 1,200 F-degree days. For snow-free turf ground conditions and granular soil backfill, the corresponding freezing temperature penetration for the design freezing index is approximately 5 feet.

4.05 Seismic Design Parameters

With reference to the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*, seismic design parameters for the site are as follows:

- Peak Ground Acceleration coefficient (PGA) = 0.087g
- Short-period (0.2 second) spectral acceleration coefficient = 0.173g
- Long-period (1.0 second) spectral acceleration coefficient = 0.044g

RWG&A estimated site shear wave velocities using site boring data and geophysical shear wave velocity test results relationships with undrained shear strength and standard penetration test data for Presumpscot Formation deposits and glacial till deposits at sites in southern Maine. RWG&A's evaluation indicates the average shear wave velocity in the upper 100 feet of the soil profile could range from about 600 to 625 feet per second, which corresponds to a Site Class D. Based on the above and S_{D1} = 0.11, the site is assigned to Seismic Zone 1.

5.0 PAN-AM RAILROAD CROSSING APPROACH EMBANKMENTS

5.01 General

Engineering evaluations and recommendations are based on the subsurface explorations, laboratory testing data, and design information currently available to RWG&A. The engineering evaluations should be reviewed by RWG&A to confirm their continued applicability after proposed railroad crossing design and site grading have been finalized.

This report addresses settlement based on the existing and proposed grades shown on drawings provided to RWG&A. Evaluation of settlement is sensitive to fill thicknesses and/or excavation depths. RWG&A should be advised if the current or constructed ground surface elevations vary or are changed from those depicted on the referenced plans.

Geotechnical engineering analyses were performed in general accordance with *AASHTO LRFD Bridge Design Specifications* (AASHTO LRFD). It is recommended the pedestrian-bicycle bridge design and construction comply with the requirements of all applicable ordinances, regulations, and codes. Supporting calculations are attached in Appendix D of this report.

5.02 Existing and Proposed Conditions

The two-span bridge will be supported on stub abutments within approach embankments at each end of the bridge, and on a pier at the east side of the railroad right-of-way. Maximum design embankment height above current ground surface occurs at the abutments, and is about 22 feet at the west abutment at Sta. 142+42 and 17 feet at the east abutment at Sta. 123+74. The approach embankments profile pitches down and away from the bridge at a slope of 8.33 percent. The west approach embankment will have side slopes pitched at 2H:1V. The east approach embankment would have a Mechanically Stabilized Earth (MSE) Retaining Wall at the north side to maintain the trail within its easement. The east embankment south side slope would be pitched at 2H:1V.

5.03 Overall Stability Evaluations

Overall stability analyses of the proposed embankments were performed with the computer software model *Slide 2018* produced by RocScience, Inc. Rotational arc stability calculations used Spencer's method of slices and non-circular slide surface calculations used the Janbu method of slices. Stability of temporary construction conditions were outside the scope of RWG&A's services and should be evaluated by a qualified Maine-licensed engineer retained by the Contractor.

The factor of safety of slope stability is defined as the ratio of the resisting forces to the driving forces causing failure. When the driving and resisting forces are the same, the factor of safety is equal to 1.0, and failure is considered imminent. When resisting forces are greater than driving forces, the factor of safety is greater than 1. It should be noted that embankment failure might occur even with a calculated factor of safety greater than 1; in general, the likelihood of failure occurring becomes less with increasing factors of safety.

Short-term variations in factor of safety may occur due to seasonal changes in conditions such as groundwater levels, surface water levels, and soil erosion and deposition. Longer term trends may reflect the influences of weathering, geomorphologic processes, or longer-term changes in groundwater conditions. Human activity such as excavating, filling, irrigation, draining, or altering vegetative cover can also influence stability. It is important to note that slope instability may not be attributable to a single causal factor.

Based on the sensitive behavior of silty clay deposits, it is accepted engineering practice to evaluate the stability of natural and man-made slopes using undrained shear strength in cohesive soil deposits. Analyses using undrained shear strengths in silty clay deposits were conducted for service and extreme limit states. Other engineering soil parameters used in stability analyses are discussed below.

Analyses were conducted using two-dimensional method of slices. In general, the calculated factors of safety might be 5 to 10 percent higher if three-dimensional effects are considered. Given the level of subsurface, laboratory, and design information currently available, RWG&A recommends that design and construction be based on the results of two-dimensional analyses.

5.03.01 Service Limit State Stability Evaluations

Overall stability of the proposed embankments and retaining walls at the service limit state were evaluated with live load surcharge of 90 pounds per square foot for pedestrian loads. A load factor of 1.0 was applied to the live load surcharges in accordance with AASHTO LRFD. The effect of the live load surcharges used in the evaluations had a small effect on calculated factors of safety due to the narrow embankment crest width relative to total embankment width.

Article 11.6.2.3 LRFD resistance factors at service limit states depend in part on how well the geotechnical parameters are known, and proximity to structures. A resistance factor of 0.65, which is equivalent to a factor of safety of 1.5, was used for the portion of the embankment near the abutments and railroad tracks, and for the retaining wall on the north side of the east approach embankment that is adjacent to an existing off-site building. A resistance factor of 0.75, which is equivalent to a factor of safety of 1.3, was used for other portions of the east and west approach embankments. In consideration of the potential direct impact of material

sloughing into the railroad right-of-way and to reduce embankment maintenance needs at a difficult to access location, a resistance factor of 0.5 (i.e., factor of safety of 2) was used for evaluation of shallow slope failures at the west abutment slope facing the rail road tracks. RWG&A should be advised if different resistance factors (i.e., higher or lower factors of safety) should be used for HNTB's, MaineDOT's, or the Owner's assessment of risk of embankment instability relative to off-site or non-controlled features.

5.03.02 Extreme Limit State Stability Evaluations

Pseudo-static method of slices was used in extreme limit state stability evaluations. An equivalent horizontal destabilizing seismic ground acceleration of 0.07 g was used in the model. A load factor of 1.0 was applied to earth loads in accordance with AASHTO LRFD and live load surcharges were not applied in extreme limit state stability evaluations. A resistance factor of 1.0, which is equivalent to a factor of safety of 1.0, was used to assess extreme limit states.

5.03.03 Summary of Stability Evaluations

Undrained stability analyses indicate lightweight fill materials are required to provide an adequate factor of safety for failure through embankment foundation soils at service limit and/or extreme limit states where the embankment height is more than about 16 feet above current ground surface. Fill types and factors of safety against deep seated bearing capacity-type failures for the sections analyzed are summarized in the table below.

Uniformly graded crushed stone and with granular lightweight coarse aggregate fills or, alternatively, geofoam with granular borrow are recommended within about 150 feet of the west abutment. Uniformly graded crushed stone is recommended within about 65 feet of the east abutment. The amount of material needed varies with embankment height. Figure 5 illustrates the lateral limits of graded crushed stone with granular, expanded shale lightweight coarse aggregate. West embankment cross sections on Figure 6 illustrate the use of crushed stone with lightweight coarse aggregate and the geofoam alternative for use in planning and budgeting.

Location	Analyzed Embankment Height (feet)	Embankment Fill Material		Minimum Factor of Safety	
				Service Limit State	Extreme Limit State
West Embankment					
< Sta. 121+42	16	Granular Borrow		≥1.3	1.0
Sta. 121+92	18	Expanded Shale, Crushed Stone & Granular Borrow	Geofoam and Granular Borrow	1.5	1.1
Sta. 122+42	22	Expanded Shale Aggregate over Crushed Stone	Geofoam and Granular Borrow	1.5	1.1

East Embankment				
Sta. 124+62 to Sta. 125+25	17	Crushed Stone	Retaining Wall: 1.4 South Slope: 1.9	1.1
>Sta. 125+25	12	Granular Borrow	Retaining Wall: 1.5 South Slope: 1.8	1.2

Stability analyses of the west abutment indicate minimum reinforcement lengths of 15 feet extending back from the face of the slope would be needed to provide a factor of safety of 2 for local stability under service limit state load conditions. The factor of safety would be about 1.5 without soil reinforcement. The height of east embankment at the abutment is lower than the west embankment and the embankment fill width is narrower; by inspection the factors of safety at the east abutment are greater than the west embankment.

Compacted granular fill, lower weight coarse aggregate, lightweight coarse aggregate can be placed using conventional earth moving equipment such as dump trucks, bulldozers, and roller compactors in embankment construction with each material. Other combinations of normal weight granular fill and lower to lightweight fills could be used. Examples include preformed cellular concrete or low-density controlled low strength material generally with a design density of 35 to 50 pcf and ultra-light foamed glass at 12 to 15 pcf. In general, any combination of materials that result in equivalent or lower ground pressures needed for slope stability could be used; the strength of the materials would also have an effect on stability and would need to be considered prior to selecting alternatives.

5.04 Strength and Extreme Limit State Evaluations

AASHTO LRFD resistance factors for MSE retaining walls are provided in AASHTO LRFD Table 11.5.7-1. Resistance factors at the service and strength limit states for bearing and sliding are 0.65 and 1.0, respectively. For the extreme limit state, resistance factors should be taken as 1.0 for sliding and overturning analyses and 0.9 for bearing resistance,. In accordance with AASHTO LRFD, safety against overturning is satisfied if the resultant of the reaction forces is in the middle two-thirds of the base width. AASHTO LRFD specifications and MaineDOT design guidelines recommend neglecting resistance to passive earth pressures at the toe of retaining walls for stability evaluations. The retaining walls should be designed to withstand lateral, uplift, and overturning forces due to earthquake. Refer to Section 4.05 above for seismic acceleration parameters.

5.04.01 Liquefaction Potential

Liquefaction potential of naturally deposited was evaluated in accordance with the MaineDOT Bridge Design Guide Section 3.7.4.2. The soils encountered in the borings are not considered susceptible to liquefaction.

5.04.02 Bearing Resistance

Nominal (i.e., unfactored) bearing resistance for the retaining wall design is influenced by subsurface conditions and soil reinforcement length if used. Recommended bearing resistances for MSE retaining walls are summarized below:

Limit State	Wall Height (feet)	
	20	≤10
Service and Strength	3,400	2,700
Extreme	6,800	5,500

Interpolated values may be used for wall heights between 10 and 20 feet.

5.04.03 Sliding Resistance

Based on the anticipated foundation subgrade material, a friction angle of 30 degrees is recommended for analysis of failure by sliding. AASHTO LRFD Article 11.10.2 specifies a minimum reinforcement length of 70 percent of the wall height. MaineDOT, Standard Specification, November 2014 Edition, Section 677, *Mechanically Stabilized Earth Retaining Wall*, calls for a minimum 8 feet reinforcement length, or 70 percent of the wall height, whichever is greater. Reinforcement length should be the same from the bottom to the top of the wall section. Reinforcement lengths less than 8 feet or 70 percent of the wall height should be approved by MaineDOT.

5.05 Service Limit State Evaluation of Embankment Settlement and Lateral Movement

5.05.01 Settlement

Settlement of the embankments will vary over the length of the embankments due to variable heights, fill materials, side slopes, and subsurface conditions. Settlement calculations were performed using the computer program Settle3D produced by Rocscience, Inc., and lightweight fills required for embankment stability (note: refer to Section 5.03). In accordance with AASHTO LRFD, elastic settlements include dead and live loads, and consolidation settlements considered dead load only. Estimated settlements are summarized below.

Settlement Type	Estimated Settlement Amounts (inches)				
	West Embankment		East Embankment		Pan-Am Railroad Tracks
	Location of Maximum Settlement (Sta. 122+30)	Abutment	Location of Maximum Settlement (Sta. 124+75)	Abutment	
Elastic Settlement	2	1.5	1	1.5	0.1
Primary Consolidation	3	2.5	2	1	0.1
Secondary Consolidation	-	-	-	-	-
Total	5	4	3	2.5	0.2
Anticipated Range	3 to 6	3 to 5	3 to 4	2 to 3	¼ to 1/2

Secondary consolidation of silty clay deposits is expected to be negligible due the slightly over consolidated condition. While settlement is expected to be within the above anticipated ranges based on the currently available information, actual settlements could be more (or less) due to

variability in fill unit weights, subsurface conditions, engineering properties, actual loads, and changes in subsurface conditions such as prolonged lowering of groundwater.

Elastic settlement is expected to occur primarily during construction as fill is placed. Post construction elastic settlement will also occur due to loads applied during the facility's service life loads. Ninety percent of the primary consolidation is expected to occur over a period of about 15 to 20 years after fill and the bridge structure are in place. (Note: evaluation of estimated settlement amounts relative to the bridge structure and foundations is discussed in Section 6.02.)

Differential settlements along the embankment alignment are anticipated to be gradual and more-or-less proportional to embankment fill height. Angular distortion along the embankments alignment is expected to be less than 1/300 and tolerable for the proposed east embankment wall type. MaineDOT and the retaining wall designer should review the estimated settlements and verify they are tolerable for the retaining wall. Based on the distance from the railroad tracks, embankment construction is expected to cause of settlement of the railroad tracks near the west abutment.

5.05.02 Lateral Movement

Lateral displacements of the east embankment wall are dependent in part on wall length, overall structure stiffness, soil conditions, reinforcement lengths (if needed), and contractor workmanship. Lateral displacements of the walls were estimated using AASHTO LRFD first order estimating methodology. In general, lateral displacement of about 1 inch should be expected during construction of east embankment retaining wall.

5.06 Design Recommendations

5.06.01 Embankment - General

Fill used in embankment construction should consist of MaineDOT 703.19 Granular Borrow – Material for Embankment Construction and granular lightweight coarse aggregate. Granular borrow should be compacted to at least 95 percent of the maximum dry density as determined by ASTM D1557. Recommended limits of granular borrow and lower to lightweight coarse aggregate are provided in Section 5.03.03. The composition of Common Borrow can run the gamut of soil types including materials which are frost susceptible and sensitive to disturbance when wet. Due to its more-or-less uncontrolled composition potential from moisture sensitive clays and silts to coarse-grained soils, and the potentially variable unit weights and shear strengths, Common Borrow should not be used in embankment construction where embankment heights are more than 8 feet above pre-construction ground surface

5.06.01.01 Lightweight Coarse Aggregate

Granular lightweight coarse aggregate should be produced from naturally occurring materials such as pumice, scoria, and tuff or from expanded clay, shale, or slate fired in a rotary kiln. The aggregate should be reasonably uniform in quality and density, and free of deleterious substances. Lightweight coarse aggregate should meet the following criteria:

- Material passing the No. 200 Sieve (ASTM D422): maximum 3.0 percent
- Dry loose weight (ASTM C29/C29M): 33 to 50 pcf

- Los Angeles Abrasion (ASTM C131): maximum 35 percent
- Ferric Oxide (ASTM C 641): maximum 1.5 milligrams per 200 grams

The bulk density of lightweight aggregate should be tested by *ASTM Designation: C 29/C 29M Standard Test Method for Bulk Density ("Unit Weight") and Voids in Aggregate*. The bulk density for "oven dry" and "saturated-surface-dry" conditions shall be determined by the "rodding procedure" and "jigging procedure"; maximum acceptable oven-dry bulk density is 50 pcf by both procedures.

In open areas, lightweight coarse aggregate should be placed in level, uniform lifts not exceeding 12 inches in loose lift thickness and be compacted with a minimum of four complete coverages, two each in mutually perpendicular directions, of a single- or double-drum roller with an operating weight of about 1.5 to 3 tons. The recommended number of coverages and roller operating weight is intended to reduce under and over compaction of the aggregate.

In confined areas, lightweight coarse aggregate should be placed in lifts not exceeding 6 inches in loose lift thickness and be compacted with a minimum of four complete coverages, two each in mutually perpendicular directions, with a reversible vibratory plate compactor with an operating weight of 1,000 pounds plus or minus 100 pounds. The recommended number of coverages and compactor operating weight is intended to reduce under and over compaction of the crushed stone.

Construction inspections for the project should include full-time observation of placement and compaction of lightweight coarse aggregate to document the aggregate isn't under- or over-compacted.

5.06.01.02 Geofoam

Geofoam should consist of block-molded expanded polystyrene (EPS) conforming to ASTM D6817 Type EPS29, formulated to resist pest infestation, and manufactured for use in geotechnical applications. The nominal density of the EPS geofoam should not exceed 2 pounds per cubic foot. The flexural strength of the EPS geofoam should not be less than 50 pounds per square inch, and the compressive resistance at 1 percent deformation should be greater than 10 pounds per square inch. Blocks of EPS geofoam should be placed on a level, compacted, dewatered and fully drained subgrade not less than one foot above pre-construction ground surface below limits of geofoam fill or adjacent grade at the bottom of the embankment slope, whichever is higher. Manufacturer's certifications, laboratory test results, and shop drawings illustrating the block layout and orientation should be submitted by the Contractor to verify the above criteria are complied with.

Blocks should be oriented 90 degrees to underlying layers geofoam block. In addition, block joints should be offset between successive layers. Blocks should be restrained during installation to prevent horizontal separation between blocks. Gripper plates designed for specific use with geofoam blocks should be installed to prevent lateral movement between layers.

The top of the geofoam should be placed no closer than 2 feet below final grades. The geofoam blocks should be field trimmed to accommodate underground utilities that pass through the geofoam fill. The top and sides of the geofoam should be covered with a geomembrane cover that is resistant to chemical degradation (e.g., petroleum substances and solvents). The

geomembrane should be placed above a sand cover and graded at 2 percent to drain toward the edges of the geofoam.

5.06.02 Embankment Settlement and Pile Reliability Considerations

To the extent sequence and schedule can be accommodated in design and construction, RWG&A recommends embankments at the bridge abutments be constructed early in construction and prior to pile installation to reduce downdrag forces on piles and to reduce post-construction settlements. Settlement amounts and pile downdrag loads discussed in this report include settlement that occurs during and after construction. Downdrag loads herein are based on total settlement amounts and exclude the aspects of construction including sequence and schedule that might in theory reduce downdrag loads since construction sequence, means, and methods are the responsibility of the contractor and cannot be relied upon for design purposes. Nonetheless settlement that occurs during construction and pile driving reduces post-construction settlement and downdrag forces and increases embankment and abutment reliability.

5.06.03 Embankment at West Bridge Stub Abutment

Geosynthetic-reinforced fill would be needed for the portion of the embankment facing the rail road tracks below the west stub abutment in order to increase the factor of safety to about 2 for shallow slope failures (see Section 5.03.01). Geogrid reinforcement extending 15 feet horizontally from the slope face in 5-foot vertical spacing and having a minimum factored tensile strength of 1,000 pounds per foot is recommended to provide a factor of safety of 2 for the slope face. Internal stability should be evaluated for vertical capacity, deformations, and reinforcement strength. Recommended resistance factors for use in design are as follows:

Failure Mode	Resistance Factor
Direct Sliding – Soil on Soil	1.00
Bearing Capacity	0.65
Global Stability	0.65
Vertical Load Carrying Capacity	0.45
Reinforcement Strength (ultimate)	0.40

The stub abutment subgrade and backfill should consist of non-frost susceptible, crushed stone such as MaineDOT 703.22, Underdrain Backfill Material, Type C or lightweight expanded shale aggregate to provide resistance to frost action. Crushed stone should extend vertically and laterally a minimum of 3 feet beyond the design frost penetration depth as measured from the nearest surface exposed to freezing temperatures. Crushed stone should be placed in 3-inch lifts and compacted with a minimum of 3 passes of hand-guided plate vibratory compaction equipment or until the compacted surface is firm, stable, and unyielding to the compaction equipment.

The pile-supported bridge abutment might be located within the geosynthetic-reinforced fill zones. If piles would be located within the reinforced soil zone, then corrugated plastic pipe sleeves should be provided within the reinforced zone at the planned pile locations to allow construction of deep foundations after the embankment is constructed.

5.06.04 Retaining Wall

The retaining wall should be designed by a qualified Maine-licensed Professional Engineer. It is anticipated that design of the wall would occur during final site design and be revised, as needed, during construction. RWG&A requests the opportunity to review the geotechnical aspects of the retaining walls design and shop drawings prior to construction to confirm the recommendations herein are interpreted as intended.

5.06.04.01 Wall Units and Backfill

Retaining walls should be designed in accordance with MaineDOT Standard Specification *Section 677 - Mechanically Stabilized Earth Retaining Wall*, as applicable, and AASHTO LRFD. Additional recommended parameters for use in design of the retaining walls follow. It is recommended the designer review these recommendations for compliance with the project-specific wall unit and geogrid manufacturer recommendations.

MSE Wall Facing Units
- Unit Weight = 143 pounds per cubic foot
- Block/Leveling Pad Interface Friction Angle = 29 degrees
- Block/Backfill Interface Friction Angle = 17 degrees

Reinforcement Backfill
- MaineDOT 703.20 Gravel Borrow (note: 3-inch minus)
- Unit Weight = 135 pounds per cubic foot
- Friction Angle = 35 degrees
- Active Earth Pressure Coefficient, Ka = 0.3 for level retained fills
- At-rest Earth Pressure Coefficient, Ko = 0.43

Embankment Fill
- MaineDOT 703.22, Underdrain Backfill Material, Type C
- Unit Weight = 105 pounds per cubic foot
- Friction Angle = 45 degrees
- Active Earth Pressure Coefficient, Ka = 0.2 for level retained fills
- At-rest Earth Pressure Coefficient, Ko = 0.3

Expanded Shale Embankment Fill
- Unit Weight = 50 pounds per cubic foot
- Friction Angle = 45 degrees
- Active Earth Pressure Coefficient, Ka = 0.2 for level retained fills
- At-rest Earth Pressure Coefficient, Ko = 0.3

Existing Foundation and Retained Soils

Silty Sand
- Unit Weight = 125 pounds per cubic foot
- Friction Angle = 30 degrees
- Cohesion = 0 pounds per square foot
- Ka = 0.3 for level retained fills

Equipment/Pedestrian Surcharge
Live Load = 125 pounds per square foot

Factor of Safety
Geogrid Pullout = 1.5
Construction Uncertainties = 1.5

Reinforcement layout, resistance to sliding, and resistance to pullout should consider the stub abutment and bridge load reactions carried by the stub abutment and supporting piles that are transferred to the reinforced earth mass. Methods of analysis, design, and construction recommendations for bridge foundations located above and within geosynthetic-reinforced embankments are provided in Chapter 6 of Federal Highway Administration's (FHWA) *Geotechnical Engineering Circular 11, Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Volumes I and II* dated November 2009. The retaining walls should be designed to withstand lateral, uplift, and overturning forces due to earthquake.

Wall facing units should be installed on a minimum 6-inch thick cast-in-place leveling pad constructed of Fill Concrete conforming to the requirements of MaineDOT Section 502 - Structural Concrete. The leveling pad should extend a minimum of 6 inches beyond the outside edges of the lowest wall block. Final subgrade preparation should include compaction of soil subgrades with hand-guided, vibratory compaction equipment. Following recompaction and prior to placement of concrete, care should be taken to limit disturbance of the bearing surfaces. Any loose, softened, or disturbed material due to construction traffic should be removed prior to placement of concrete.

Retaining wall backfill at locations where lightweight fill is unnecessary for global stability should be MaineDOT 703.20 Gravel Borrow with a maximum particle size passing a 3-inch square mesh sieve. Backfill should be placed in 6-inch lifts, not exceeding 8 inches in uncompacted thickness, and be compacted with a vibratory plate compactor to achieve a minimum of 92 percent maximum dry density as determined by ASTM D1557, *Test Method for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbf/ft3 (2,700 kN-m/m3))*. The Contractor should provide test results of proposed backfill material including: gradation, maximum dry density, and pH.

5.06.04.02 Frost Protection

In accordance with AASHTO LRFD Article 11.10.2.2, the bottom of the retaining walls should be located a minimum of 5 feet below ground surfaces exposed to freezing temperatures. Alternatively, frost susceptible material should be removed within the depth and lateral extent of frost penetration below the wall (5 feet below finished grade at the toe of the wall) and be replaced with non-frost susceptible material. MaineDOT 703.06 Aggregate for Base, Type A or Type B, or MaineDOT 703.22 Underdrain Backfill Material, Type C may be used as non-frost susceptible material. Non-woven filter fabric meeting AASHTO M288-06 Class 3 geotextile properties should be provided between the interface stone fills and soil fills and naturally deposited soils.

It is understood that MaineDOT might accept lesser frost protection for retaining walls supporting flexible structures such as the trail path. If full excavation depth for removal of frost

susceptible material is not practicable, the designers should obtain approval from MaineDOT for lesser frost protection.

5.06.04.03 Wall Embedment

In accordance with AASHTO LRFD Article 11.10.2.2, the required embedment depth should be taken as the frost protection depth (5 feet) per Table C11.10.2.2-1. As mentioned above, MaineDOT might accept lesser frost protection for retaining walls supporting flexible structures. A design exception should be obtained from MaineDOT if embedment depths less than 5 feet are proposed. Minimum recommended embedment depth is 2 feet.

5.06.04.04 Wall Drainage

Drains should be provided at the base of the wall to prevent hydrostatic pressure buildup behind the retaining wall. Drains should consist of 4-inch (minimum) diameter perforated pipes bedded in 2 cubic feet of MaineDOT 703.22 Underdrain Backfill material Type C per linear foot. The drainage stone should be completely wrapped in a non-woven filter fabric meeting AASHTO M288-06 Class 3 geotextile properties.

Flow from the foundation drains should be conveyed by gravity to surface drainage features or storm drains that will be free flowing at all times and under all conditions. Multiple outlets should be provided for drainage so as not to be reliant upon a single flow path.

Vegetated swales or stone lined ditches should be provided to divert surface runoff around and away from the retaining walls. Swales should be lined with protected, relatively impervious soils or geomembrane if located above or within 15 feet of a reinforced zone.

5.07 Construction Considerations

5.07.01 Site Preparation

It is anticipated the surficial material will be stripped and be either incorporated into proposed landscaped areas, where practicable, or hauled off site. Topsoil and organic materials are not considered suitable for use as common fill. Existing fills will need to be evaluated during construction to determine if they can be reused.

After unsuitable fill and materials have been removed from the embankment areas and prior to placement of fill, the exposed subgrade should be compacted with several overlapping passes, with a smooth drum, minimum 10-ton vibratory roller. If high groundwater is present during subgrade preparation, the vibratory roller should be operated in the static mode to avoid pumping up groundwater and disturbing the subgrade soils. The exposed subgrade should then be observed during compaction for areas of soil rutting or weaving indicative of unsuitable underlying materials. Soft areas or areas that yield more than 1 inch during proof-rolling should be overexcavated and replaced with structural fill.

5.07.02 Construction Dewatering

The on-site soils are sensitive to disturbance when wet. To reduce disturbance of exposed subgrade soils, it will be important to divert runoff, provide positive grading to shed seepage and

runoff from flat areas, and compact exposed soils to reduce rutting, ponding, and surface water infiltration. Site grading should provide positive drainage away from constructed facilities both during and after construction.

It should be practicable to accomplish construction dewatering from within excavations to depths of about one foot below groundwater levels by open pumping methods. Excavations extending below ground water will be susceptible to sloughing and caving of excavation side walls. Dewatering to depths of more than one foot below groundwater might require side trenches, wells or wellpoints. Temporary detention ponds, trenches, ditches, and dewatering sumps should not be made within or near the retaining walls and areas to be filled. Surface runoff and infiltration of groundwater should be controlled so that excavation, retaining wall construction, backfilling, and compaction can be completed in-the-dry.

5.07.03 Temporary Excavations

Design and construction options to address temporary excavations are provided below for the engineer's planning and preliminary design purposes only and not for construction. Means and methods of construction, including the design of temporary excavations, are typically the responsibility of the Contractor. It is recommended the Contractor be responsible for design of temporary excavations and that the final excavation design be based on subsurface conditions exposed during construction.

The Contractor should be aware that slope height, slope inclination, and excavation depths should in no case exceed those specified in local, state, or federal safety regulations (e.g., OSHA Health and Safety Standards for Excavations, 29 CFR Part 1926, or successor regulations). Such regulations are strictly enforced and, if they are not followed, the Owner, Contractor, and/or earthwork and utility subcontractors could be liable for substantial penalties.

Temporary excavations not meeting OSHA standards would need to be designed by a qualified Maine Licensed Professional Engineer. The type of temporary excavation would depend, in part, on the soil and groundwater conditions at the time of construction.

As a safety measure, it is recommended all vehicles and spoil piles be kept a minimum lateral distance from the top of excavations equal to no less than 100 % of the slope height. Exposed slope faces should be protected against the elements.

5.07.04 Material Utilization and Site Filling

The subsurface soils from site work excavations will generally consist of silty sands that might not be suitable for wall backfill but, with proper moisture conditioning and earthwork handling, might be used as common fill and granular borrow (Maine DOT 703.19 Granular Borrow – Material for Embankment Construction). Silt and other fine-grained soils such as sandy silt and clayey silt should not be used as fill below planned construction.

If on-site soil is proposed for use other than common fill, the soil should be stockpiled separately and tested to determine if it meets specification requirements for its intended use. Moisture-density relationships should be established during construction to provide guidance for appropriate working moisture contents. Working moisture content for moisture sensitive soils typically ranges from about minus three to plus one percent of optimum moisture content.

The integrity of natural soils and compacted fill must be maintained during cold weather conditions. Footing and slab subgrades should not be allowed to freeze. The naturally deposited soils are considered moderately to highly frost susceptible. Freezing of subgrade soils beneath wall leveling pads may result in frost heaving and post-construction settlement. The Contractor should make every effort to prevent freezing of subgrade soils. In the event frost penetration occurs, frozen and previously frozen fill or naturally deposited soils should be completely removed and replaced with compacted granular borrow fill. At no time should frozen material be placed as fill.

6.0 PAN-AM RAILROAD CROSSING BRIDGE FOUNDATIONS

6.01 General

This section provides recommendations for design and construction of the bridge foundations. If the bridge abutment is located within geosynthetic-reinforced embankments, then their design and construction will need to be coordinated with one other. Engineering evaluations and recommendations are based on the subsurface explorations, laboratory testing data, and the preliminary construction information currently available to RWG&A. The engineering evaluations should be reviewed by RWG&A to confirm their continued applicability after proposed railroad crossing design and site grading have been finalized.

6.02 Service Limit State Evaluation of Abutment and Pier Settlements

Spread Footing Evaluation: Estimated embankment settlements at stub abutment locations are provided in Section 5. A stub abutment supported on a spread footing would settle the same amount and at the same rate as the embankment. As discussed in Section 5, up to about 5 inches of settlement is anticipated at the abutments. RWG&A estimated pier settlement for spread footing support based on a factored bearing pressure of 1,600 pounds per square foot and loads provided by HNTB. Estimated elastic settlement, consolidation, and total settlements at the pier are provided below.

Estimated Pier Settlement if Supported on Spread Footing	
Settlement Type	**Settlement (inches)**
Elastic Settlement	1
Consolidation	1
Secondary	-
Total	2
Anticipated Range	1 to 3

Most elastic settlement is anticipated to occur in cohesionless soils during construction due to dead weight of the foundation and bridge; additional elastic settlement would occur during service life live loading.

Pile Supported Foundation Settlements: Estimated total settlements for pile supported abutments and pier are less than 1 inch. Pile support of the pier and abutments is recommended based on the need to maintain minimum clearance over the railroad, reliability, resources of the bridge owner/operator, and potential difficulties and expense to maintain railroad clearances and/or reduce distortion in the bridge if excessive amounts of settlement occur.

6.03 Driven Piles

6.03.01 Factored Abutment and Pier Bridge Reactions

Based on design information provided, piles supporting the bridge pier would be embedded in below-grade pile caps. The abutment piles and pier piles are considered fully embedded (i.e., $\lambda = 0$) for purposes of calculating the nominal compressive structural resistance. Based on subsurface conditions encountered in the subsurface explorations, the piles would be driven to end bearing in glacial till deposits or on bedrock. Factored bridge pile demand loads at the abutment and piers provided by HNTB are summarized below:

Location	Strength Limit State (kips)		Service Limit State (kips)	
	Minimum	Maximum	Minimum	Maximum
West Abutment Piles	-46	161	-7	132
Pier Piles	-55	138	14	77
East Abutment Piles	-47	171	-13	130

A number of pile types and sizes are technically practicable. Pile sizes provided below were selected based on anticipated bridge design loading, local practice, and drivability using locally available hammer capacities. Piles evaluated for this geotechnical investigation would be 50 kips per square inch, Grade A572 H-piles fitted with drive points to reduce damage during installation.

6.03.02 Factored Pile Axial Compressive Resistance

Based on driving the piles to end bearing resistance in glacial till or on bedrock, the factored structural axial resistance of the H-pile sections in the strength limit state were calculated using a resistance factor, φ, of 0.5 due to the potential severe driving conditions. The nominal geotechnical compressive resistance in the strength limit state was calculated using Nordlund/Thurman method for piles bearing in marine sand and glacial till deposits and Canadian Geotechnical Society method for end bearing resistance in rock. Static analyses indicate that the H-pile would bear in glacial till deposits with most resistance provided in side resistance (i.e., skin friction). A resistance factor of 0.45 was applied the nominal resistances for strength limit state conditions.

Yield strength of steel used for preliminary calculations was 50 kips per square inch. Therefore, maximum pile driving stresses of 45 kips per square inch were used for preliminary drivability analyses. The resistance factor for a driven single pile in axial compression when based on the recommended resistance determination method, φ_{dyn}, is 0.65, when verified in the field with

dynamic pile testing. Preliminary wave equation analysis for this report used pile lengths of 105 feet, a Delmag D30 pile hammer with hammer energy per blow of 59,730 foot-pounds maximum and terminal driving criteria of 10 and 15 blows per inch (bpi); other hammers would have different resistances.

Factored axial structural, geotechnical and drivability resistances calculated for the two pile sections at the strength limit state are summarized below:

Pile Section	Factored Compressive Resistance (kips) at Strength Limit State				
	Structural	Geotechnical		Drivability	Governing Resistance
		Resistance at Minimum Pile Tip Penetration Into Glacial Till Deposits	Pile Tip at Bottom of Glacial Till Deposits		
HP 10x42	308	>316 at 10 ft (Abutment 1) >325 at 30 ft (Abutment 2)	645 (Abutment 1) 520 (Abutment 2)	260 at 10 bpi 275 at 15 bpi	260 at 10 bpi 275 at 15 bpi
HP 12x53	385	>350 at 5 ft (Abutment 1) >360 at 25 ft (Abutment 2)	780 (Abutment 1) 630 (Abutment 2)	290 at 10 bpi 310 at 15 bpi	290 at 10 bpi 310 at 15 bpi
HP 12x74	547	>350 at 5 ft (Abutment 1) >360 at 25 ft (Abutment 2)	800 (Abutment 1) 650 (Abutment 2)	345 at 10 bpi 385 at 15 bpi	345 at 10 bpi 385 at 15 bpi

Per commentary in LRFD Article 10.5.5.2.3 the resistance factor should be reduced by 20% for nonredundant pile groups (i.e., less than three to five piles in the group). The above factored resistances would be lower if the design pile groups are non-redundant.

The structural and drivability resistance factors, φ, for the service and extreme limit states are 1.0. Geotechnical resistance factors for the service and extreme limit states are 1.0 for compressive pile resistance. Factored axial resistances for structural, geotechnical and drivability calculated for the two pile sections at the service and extreme limit states are summarized below:

Pile Section	Factored Compressive Resistance (kips) at Service and Extreme Limit States				
	Structural	Geotechnical		Drivability	Governing Resistance
		Resistance at Minimum Pile Tip Penetration Into Glacial Till Deposits	Pile Tip at Bottom of Glacial Till Deposits		
HP 10x42	616	>700 at 10 ft (Abutment 1) >720 at 30 ft (Abutment 2)	1,400 (Abutment 1) 1,200 (Abutment 2)	400 at 10 bpi 425 at 15 bpi	400 at 10 bpi 425 at 15 bpi
HP 12x53	771	>780 at 5 ft (Abutment 1) >800 at 25 ft (Abutment 2)	1,700 (Abutment 1) 1,400 (Abutment 2)	445 at 10 bpi 475 at 15 bpi	445 at 10 bpi 475 at 15 bpi
HP 12x74	1,094	>780 at 5 ft (Abutment 1) >800 at 25 ft (Abutment 2)	1,800 (Abutment) 1,400 (Abutment 2)	530 at 10 bpi 590 at 15 bpi	530 at 10 bpi 590 at 15 bpi

Calculated pile stresses from the WEAP analysis were less than 36 kips per square inch (ksi) which is less than the maximum allowable driving stress of 45 ksi. Pile hammer size and driving criteria should be based on actual structural capacities required based on design pile layout including an allowance for negative skin friction. Piles should be driven into the dense to very dense glacial till deposits, or to the minimum depth in glacial till deposits required design resistance as demonstrated by dynamical pile analysis, whichever is greater.

6.03.03 Factored Pile Axial Uplift Resistance

The nominal geotechnical uplift resistance in the strength limit state was calculated using Nordlund/Thurman method for piles bearing in marine sand and glacial till deposits. The factored structural uplift resistance of the H-pile sections in the strength limit state were calculated using a resistance factor, φ, of 0.35 for the sections of pile penetrating through marine sand and embedded in glacial till, and 0.25 for the sections of pile in silty clay and embankment fill in accordance with AASHTO Table 10.2.2.2.3-1. A resistance factor of 0.8 was used for the service limit state.

Pile Section	Factored Uplift Resistance (kips) at Strength and Service Limit States		
	Location	Strength Limit State	Service Limit State
HP 10x42	Abutment 1 Pier Abutment 2	255 90 260	605 200 630
HP 12x53	Abutment 1 Pier Abutment 2	280 100 290	670 235 695
HP 12x74	Abutment 1 Pier Abutment 2	270 105 290	655 240 690

6.03.04 Negative Skin Friction (Downdrag)

Pile supported stub abutments will be located within the approach embankments, which will settle due to compression of near surface sand deposits and consolidation of silty clay deposits. Silty clay consolidation will occur post-construction. LRFD Article 3.11.8 states that if the settlement in the soil layer is 0.4 inches or greater relative to the pile or shaft, downdrag can be assumed to fully develop. RWG&A estimates about 3 inches of consolidation settlement might occur. Due to the finite extent of the embankment in length and width, stresses due to embankment loading diminish with increasing depth. The evaluations indicate that consolidation would result in negative skin friction due to skin friction in the embankment, near surface sand deposits, and through most of the silty clay deposits. Based on the above and downdrag load factors of 1.4 at strength and extreme limit states and 1.0 at the service limit state per AASHTO LRFD Table 3.4.1-1, the calculated factored downdrag loads are:

Pile Section	Downdrag Load (Kips)			
	Strength and Extreme Limit States		Service Limit State	
	Abutments	Pier	Abutments	Pier
HP 10x42	150	0	110	0
HP 12x53	190	0	130	0
HP 12x74	190	0	140	0

The above downdrag loads should be added to the factored structural reaction at the pile heads at the abutments.

6.03.05 Lateral Loads Resistance

Lateral loads from wind and earthquake may be resisted by a combination of batter piles, lateral pile capacity, and passive earth pressure on the side of the pile cap. The pile batter should be no flatter than 3H:12V. (Note: battered piles might extend beyond the Eastern Trail easement boundary, and potentially across property boundaries. The bridge Owner should obtain permission to install battered piles from the appropriate parties).

Analysis for piles supporting the abutments considered soil settlement below the stub-abutment which could result in a free head condition. Lateral pile capacity of piles in groups should be reduced based on location in proximity to adjacent piles in accordance with the reduction factors in AASHTO LRFD Table 10.7.2.4-1. Non-linear analysis using p-y curves is recommended to evaluate the soil pile interaction for combined axial and flexure, with factored axial loads, movements, and tolerable pile head displacements. Recommended soil properties for use is lateral load resistance are summarized below:

Soil Type	Unit Weight (pounds per cubic foot)		Cohesion (pounds per cubic foot)	Internal Angle of Friction (degrees)	Static Soil Modulus (pounds per cubic inch)	Soil Strain Parameter, E50
	Total (moist)	Buoyant				
Expanded Shale	40	40	0	40	125	-
Crushed Rock	105	40	0	45	125	-
Granular Borrow	135	73	0	32	125	-
Near Surface Sand Deposits	125	63	0	30	90	-
Silty Clay (Upper 10 feet)	110	48	800	-	100	0.01
Silty Clay	110	48	450	-	30	0.02
Marine Sand	135	73	0	32	60	-
Silty Sand with Gravel (Glacial Till)	140	78	0	40	125	-

6.04 Seismic Design

The foundations should be designed to withstand lateral, uplift, and overturning forces due to earthquake. Seismic site design parameters are provided in Section 5.

6.05 Frost Protection

The bottom of pile caps at piers should be located a minimum of 5 feet below the lowest adjacent surface exposed to ground surface. Stub abutment frost protection recommendations are provided in Section 5.

6.06 Construction Considerations

Stub abutment pile installation will need to be coordinated with geosynthetic reinforced earth embankment and retaining wall construction as discussed in Section 5. RWG&A suggests that abutment piles be driven prior to embankment construction and that unfilled pipe sleeves be provided around the piles to reduce down drag forces on the piles during construction. The annulus between the sleeves and pile should be filled with silica sand. Filling the annulus should be delayed as long as practicable to allow settlement to occur.

The Contractor will need to perform a wave equation analysis of the pile hammer system(s) planned to install the driven piles. The Contractor's wave equation analyses should indicate the Contractor's pile hammer system(s) are capable of driving the new piles to the required minimum ultimate capacity without over stressing or damaging the piles. Cast steel points should be provided on H-Piles to protect the piles from driving damage and reduce the tendency of the tips to translate if boulders or sloping bedrock is encountered. The analyses should demonstrate that the hammer is capable of driving the piles to the required resistance without exceeding a driving stress of 45 kips per square inch. The ultimate drivability resistance that must be achieved with the Contractor's pile driving system is the geotechnical compressive strength limit resistance divided by a resistance factor, φ_{dyn}, of 0.65 based on load testing in the field with dynamic pile analysis and use of redundant pile groups.

Pile leads should be fixed at two points to control the location and alignment of the piles during driving; hanging or "flying" leads are not recommended. Jetting and/or use of a vibratory hammer to assist pile installation is not recommended for this project.

Based on achieving geotechnical resistance below the bottom of silty clay deposits, pile tip depths might vary from 90 to 120 feet below current ground surface at the abutments and about 80 to 100 feet below current ground surface at the pier. The piles will need to be spliced so the final 40 feet, or more, can be driven in a continuous manner using the same equipment and minimum penetration resistance prior to developing soil setup against the side of the piles. Any sudden decrease in driving resistance should be investigated with regard to the possibility of damage. If the sudden decrease in driving resistance cannot be correlated to load bearing data or subsurface conditions, the pile should be removed for inspection or rejected.

Restriking after pile setup to demonstrate pile capacity should be avoided unless dynamic pile testing is performed after full pile setup has developed. Post-setup dynamic pile testing would be needed so that the contributing setup effects of the silty clay deposits and overlying materials, which would not be available during the structure service life due to downdrag, can be accounted for in determining actual service life geotechnical capacity from the test results.

7.0 NONESUCH RIVER CROSSING

7.01 Subsurface Information

The *Surficial Geology Map of the Prouts Neck Quadrangle* (Maine Geological Survey, Open-File No. 99-97) indicates the railroad embankment overlies swamp deposits. The swamp deposits likely overlie coarse- to fine sand alluvium deposits, which in turn overlie silty clay deposits of the Presumpscot Formation.

7.02 Observed Slope Conditions – South Embankment

An RWG&A engineer observed conditions exposed at ground surface of the former railroad embankment at the south side of the Nonesuch River in December 2011, November 2013, and August 2018. There is not a railroad embankment at the east side of the Nonesuch River except an area directly behind the abutment that pitches down steeply toward the river's flood plain. The existing abutments are constructed of cut granite blocks with bridge seats elevated approximately 16 feet above the water surface.

The crest of the north embankment is about 15 feet wide and about 20 feet above natural ground surface. Ground surface at the bottom of the embankment is at about the same elevation as the water level in the Nonesuch River and was inundated in some areas at the time of the visits. The slopes on both sides of the embankment are pitched at about 1.5H:1V.

The slopes are vegetated with young and mature tree growth; the larger trees are generally about 8 to 15 inches in diameter at breast height. The tree trunks of mature and younger trees were generally more-or-less vertical, although a few curved tree trunk shapes and some tilted trees on the northwest side of the southern approach embankment suggests longer term slope movement or slow-developing instability. The ground was covered with leaves, branches, and other vegetation, which obscured direct observation of soil except at the embankment crest. No scarps, sloughs, longitudinal cracks in the slope crest, or heaved conditions were noted. The railroad embankment appeared stable in its current condition.

7.03 Subsurface Conditions

7.03.01 Subsurface Soils

Soil borings were drilled at the planned north embankment. Four distinct soil units were encountered: fill, near surface alluvium composed of clayey silt with organic matter, silty clay glacial marine deposits, and marine sand deposits. Generalized interpreted descriptions with encountered thicknesses are summarized below and illustrated on Figure7 *Interpreted Subsurface Profile – Nonesuch River.* Refer to the exploration logs in Appendix B for detailed descriptions at specific locations.

Soil Unit	Encountered Thickness (feet)	Generalized description	Comments
Fill	3 to 5	Fine to medium sand with little gravel, tan	Encountered at boring EB-101 and EB-102
Alluvium Deposits	11.5 to 15	Interbedded layers of very loose to loose silt, clayey silt, and silty sand with gravel and very soft to soft silty clay	
Glacial Marine Deposits	35 to 45	Very soft to medium, silty clay, trace fine sand, gray. Grades to very loose clayey silt to sandy silt in lower part of deposit.	Contains occasional seams of silt and fine sand
Marine Deposits	7 to 12	Very loose to medium dense, fine to medium sand with trace to few silt	Total thickness undetermined

The silty clay with organic matter alluvium deposits and the upper part of the silty clay deposits were very soft to soft. Undrained shear strengths determined from field vane tests ranged from approximately 650 psf in the upper few feet of the deposits and about 800 pounds per square foot (psf) at a depth of 30 feet below ground surface.

7.03.02 Groundwater

Free water levels were observed approximately 3 to 7 feet below ground surface about one hour after removal of drilling equipment, and might not be representative of stabilized groundwater levels when the borings were drilled. The groundwater level is influenced by the Nonesuch River water level, season, temperature, rainfall, and elevation, and will vary from conditions observed on the dates the explorations were made. The embankments are located within the Nonesuch River floodplain and the bottom of the embankment slopes are subject to inundation.

7.04 Overall Stability Evaluations

Stability of the proposed north embankment was evaluated using the method and approach described in Section 5.03. Undrained stability analyses at the maximum abutment height indicate the planned embankment can be constructed using compacted granular borrow. Minimum calculated factors of safety for deep seated circular arc and non-circular failure surfaces were 2.3 and 2.1, respectively for service limit state conditions. The minimum calculated factor of safety for extreme limit state conditions was 1.4. Naturally deposited soils encountered in soil boring are considered not susceptible to liquefaction. RWG&A's scope of serviced excluded stability assessments of the existing embankment and abutments.

7.05 Embankment Settlement Evaluations

The planned embankment follows the alignment of a former railroad embankment that was equivalent to or greater in height to the proposed embankment. The former embankment would have consolidated and preloaded the silty clay deposits under the weight of the fill that has since been removed. Recompression settlement of the silty clay deposits under the weight of new fill is expected to be less than 2 inches.

7.06 Construction Considerations

Fill from the former railroad embankment is several feet thick within the planned embankment alignment. The composition of the fill and the methods used to place and compact the fill are uncertain. It appears that remnants of the former embankment are mounded along the west side of the north embankment. It is recommended that the existing fill be evaluated during construction for reuses as fill to remain in place or as fill to removed and used as compacted fill at other locations. The elevated portions of the fill berm should be removed down to the level of adjacent stripped subgrade in the east part of the trial alignment, and the exposed surface evaluated as described below.

Following removal of vegetation and other unsuitable materials, and prior to placement of new fill, test pits should be conducted to verify material proposed to remain in place meets composition requirements for granular borrow. Material not meeting granular borrow composition requirements should be removed and replaced. The exposed surface should be compacted with multiple passes of a minimum 10-ton smooth-drum vibratory compactor.

Rolling of subgrades should be observed for areas of soil rutting or weaving indicative of deleterious conditions. Areas identified should be further explored with test pits to determine the nature of the condition. Unsuitable materials and any soft, loose, and/or disturbed native soil materials should be excavated and replaced with compacted granular borrow. Test strip procedures should be used to determine the number of passes required to achieve maximum density of compacted subgrade surface.

8.0 CLOSURE

This report has been prepared for specific application to the embankments at the Nonesuch River crossing and the proposed pedestrian-bicycle bridge and associated proposed approach embankments at the Pan-Am Railroad crossing associated with MaineDOT Project No. STP-1942(610) in Scarborough, Maine, for the exclusive use of HNTB Corporation. This work has been completed in accordance with generally accepted soil and foundation engineering practices. No other warranty, expressed or implied, is made. In the event any changes are made in the nature, design, or location of the proposed construction, the conclusions and recommendations of this report should be reviewed by RWG&A.

The recommendations presented are based on the results of widely spaced explorations. The nature of variations between the explorations may not become evident until construction has begun. If variations are encountered, it will be necessary for RWG&A to re-evaluate the recommendations presented in this report. RWG&A requests an opportunity for a general review of the final design and specifications in order to determine that earthwork and foundation recommendations have been interpreted in the manner in which they were intended.

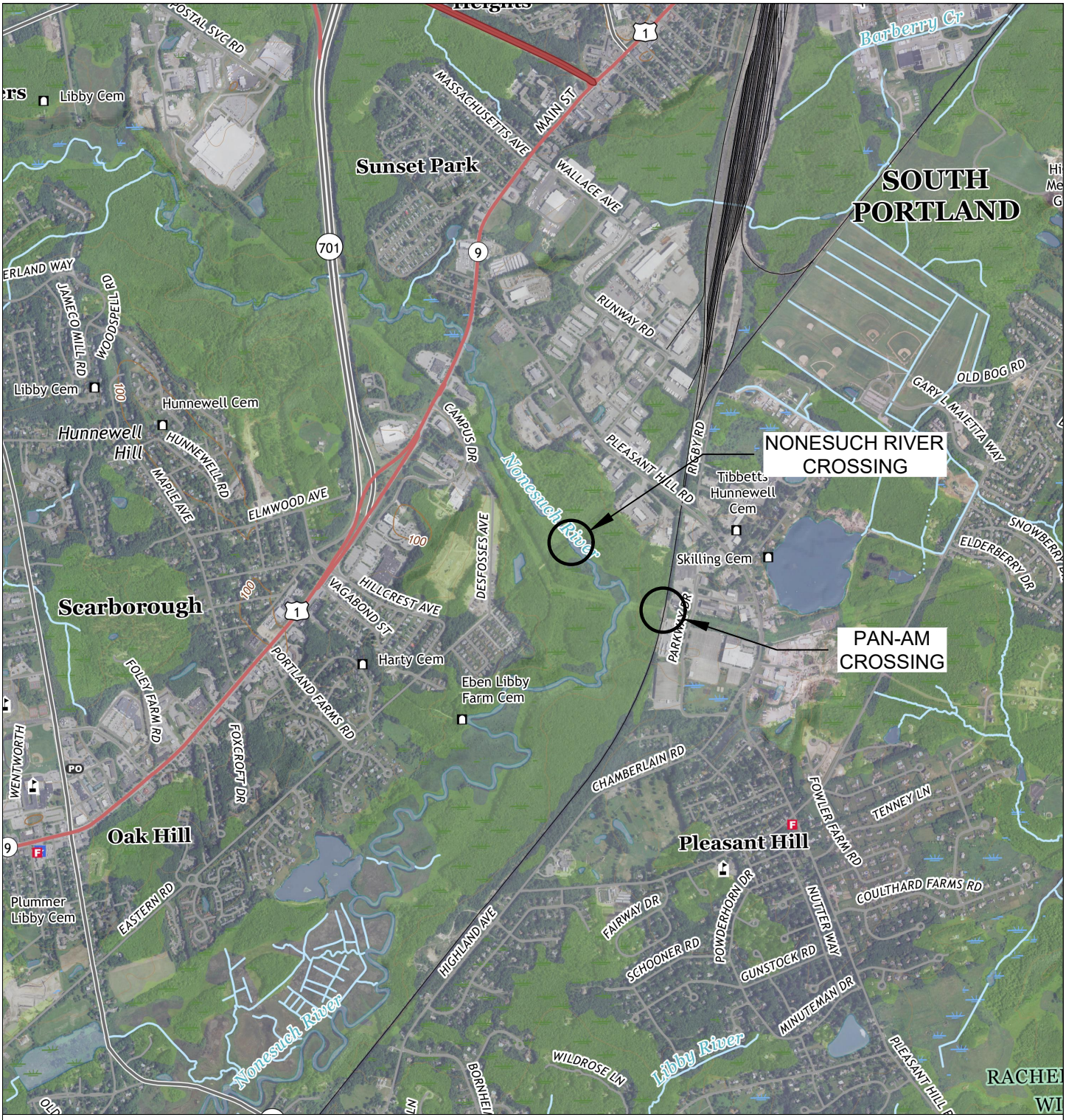


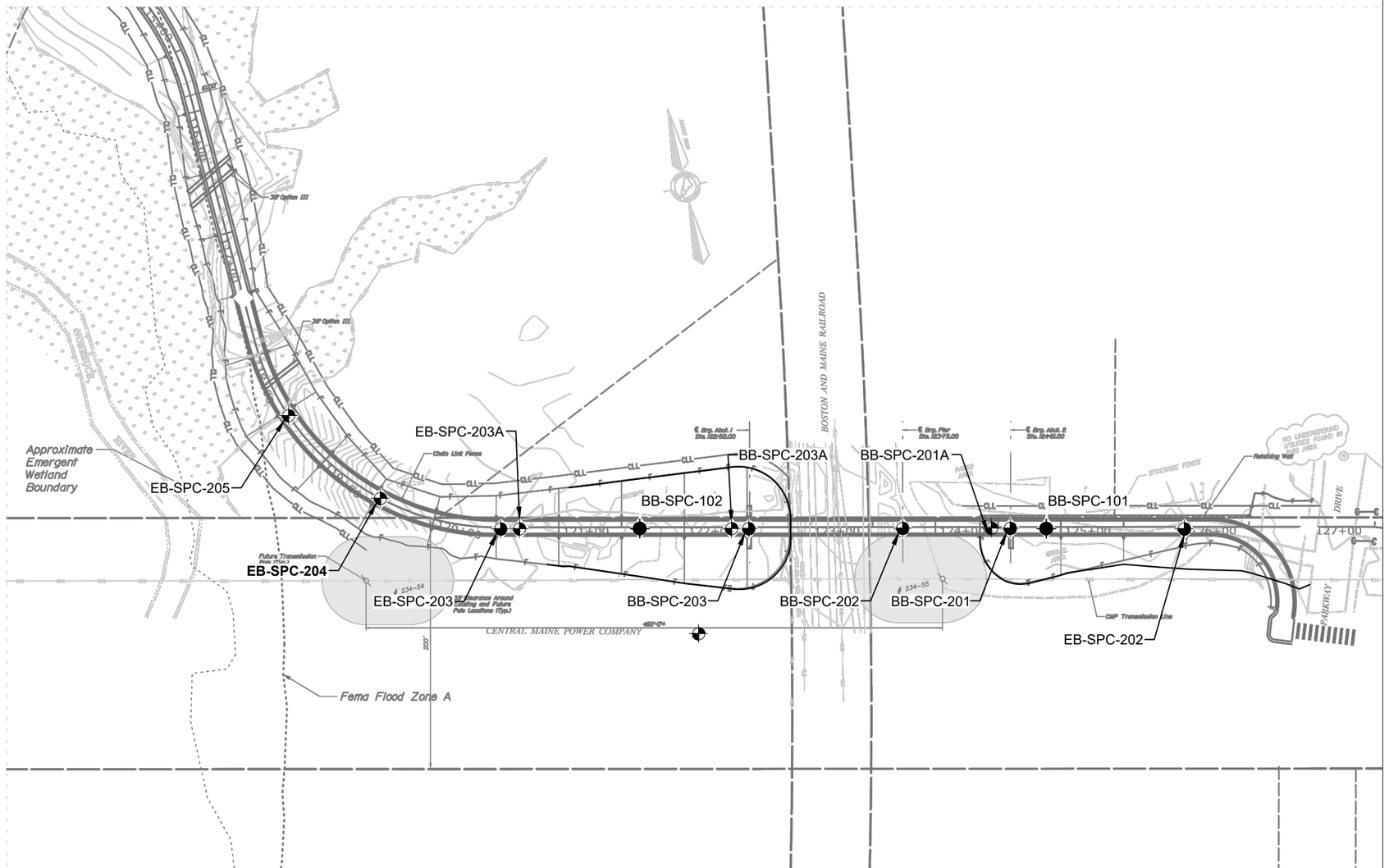
FIGURE 1
LOCUS MAP
GEOTECHNICAL EVALUATION
SCARBOROUGH EASTERN TRAIL PROJECT
SCARBOROUGH, MAINE

FEBRUARY 2019 PROJECT NO. 0600-102

SOURCE:
USGS 7.5-MINUTE TOPOGRAPHIC QUADRANGLE OF PROUTS NECK, ME, DATED 2018.

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Environmental Consulting ● www.rwgillespie.com

G:\Autocad\600\600\600-102\FIG 1, LOCUS MAP 0600-102.dwg, 2/4/2019 1:12:46 PM



LEGEND:

BB-SPC-101 — APPROXIMATE LOCATION OF SOIL BORING DRILLED JUNE 2013.

BB-SPC-201 — APPROXIMATE LOCATION OF SOIL BORINGS DRILLED FALL 2016

BB-SPC-201 — APPROXIMATE LOCATION OF SOIL BORINGS DRILLED JULY 2018

SOURCES:

- SKETCH TITLED "2016-09-19 EASTERN TRAIL ALIGNMENT AND SUBSTRUCTURE LOCATIONS 206-09-19.PDF.", UNDATED, RECEIVED VIA EMAIL FROM HNTB CORPORATION.
- SKETCH TITLED "PAN AM SPAN CONFIGURATION 08-11-2016.DGN", BY HNTB CORPORATION.

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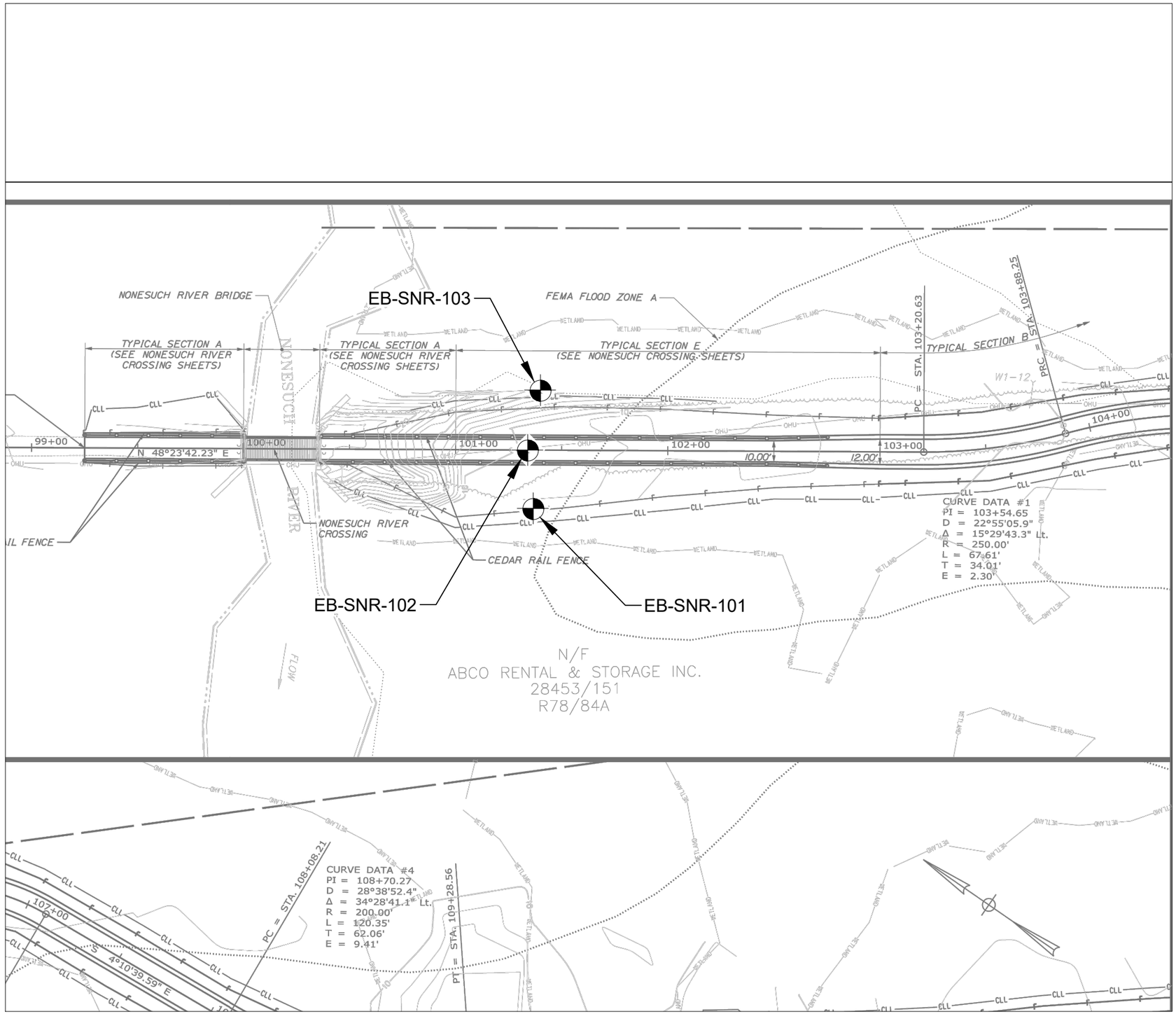
SCALE, FEET

FIGURE 2
EXPLORATION LOCATION PLAN
PAN AM CROSSING
GEOTECHNICAL EVALUATION
SCARBOROUGH EASTERN TRAIL
SCARBOROUGH, MAINE

FEBRUARY 2019 PROJECT NO. 0600-102

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G:\Autocad\600\600-102\FIG 2. PANAM CROSSING ELP 600-102 AS DRILLED.dwg, 2/4/2019 1:16:23 PM



LEGEND:

EB-SNR-101

APPROXIMATE LOCATION OF BORINGS DRILLED FALL 2016.

SOURCE:
SHEET 8 OF 42 TITLED "TRAIL PLAN 1" BY STATE OF MAINE DEPARTMENT OF TRANSPORTATION, DATED "PROGRESS PRINT JULY 27, 2018".

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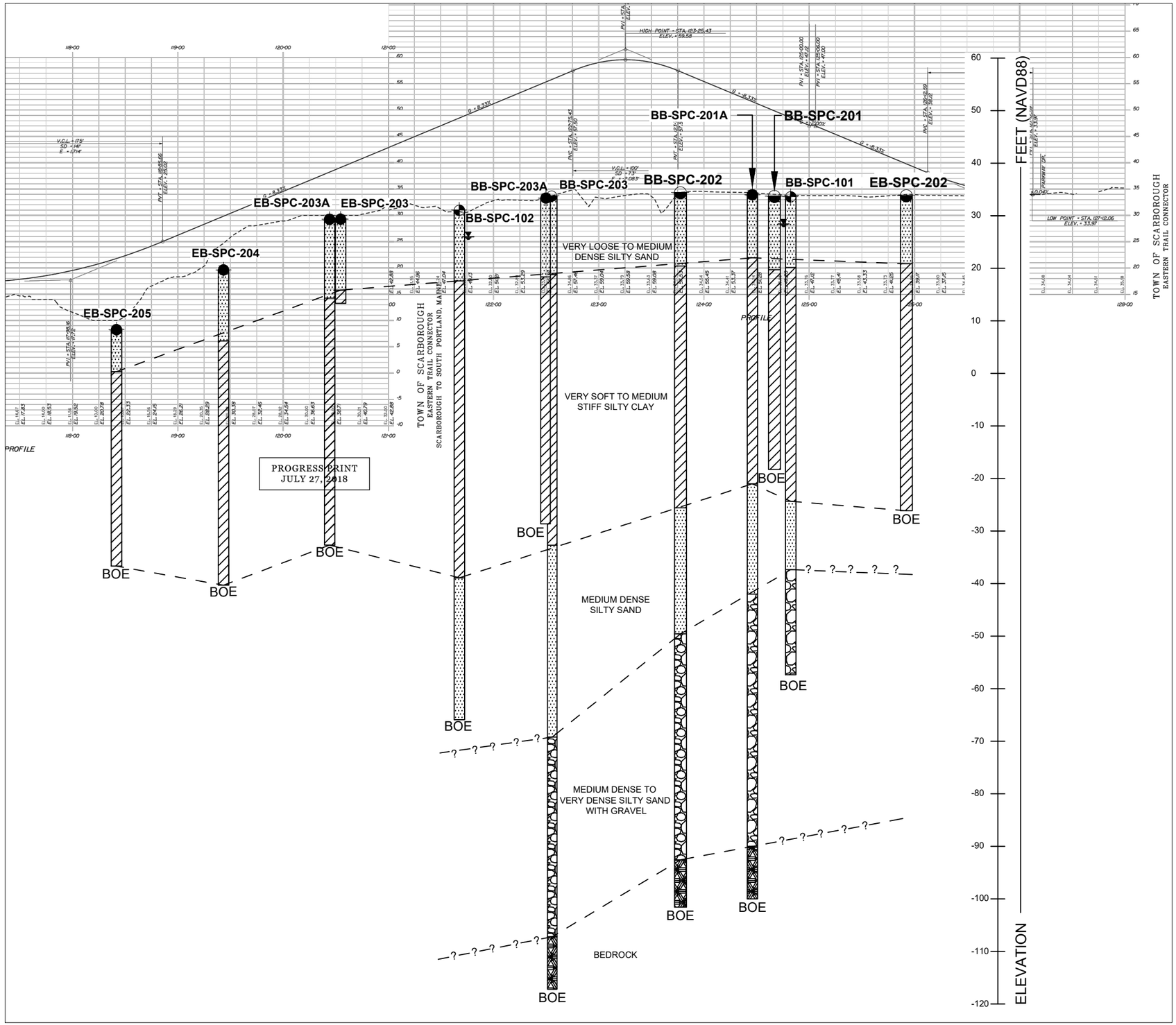
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FIGURE 3
EXPLORATION LOCATION PLAN
NONESUCH RIVER CROSSING
GEOTECHNICAL EVALUATION
SCARBOROUGH EASTERN TRAIL
SCARBOROUGH, MAINE

FEBRUARY 2019 PROJECT NO. 0600-102

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G:\Autocad\600\600-102\FIG 3, NONESUCH RIVER CROSSING ELP 600-102 AS DRILLED.dwg, 2/4/2019 1:15:49 PM



LEGEND:

BB-SPC-101	APPROXIMATE LOCATION OF SOIL BORING DRILLED OCTOBER 2013
BB-SPC-201	APPROXIMATE LOCATION OF SOIL BORING DRILLED SEPTEMBER & OCTOBER 2016
BB-SPC-203A	APPROXIMATE LOCATION OF SOIL BORING DRILLED JUNE 2018

NOTES:

1. REFER TO FIGURE 2, "EXPLORATION LOCATION PLAN" FOR BORING LOCATIONS.
2. LINES REPRESENTING CHANGES BETWEEN STRATA ARE BASED UPON INTERPOLATION BETWEEN SUBSURFACE EXPLORATIONS AND MAY NOT REPRESENT ACTUAL FIELD CONDITIONS AT OTHER THAN SPECIFIC EXPLORATION LOCATIONS.
3. REFER TO APPENDIX B FOR LOGS OF TEST BORINGS.

SOURCE:
DRAWING TITLED "PANAM CROSSING PROFILE - TOWN OF SCARBOROUGH, EASTERN TRAIL CONNECTOR, SCARBOROUGH TO SOUTH PORTLAND" DATED 27 JULY 2018.

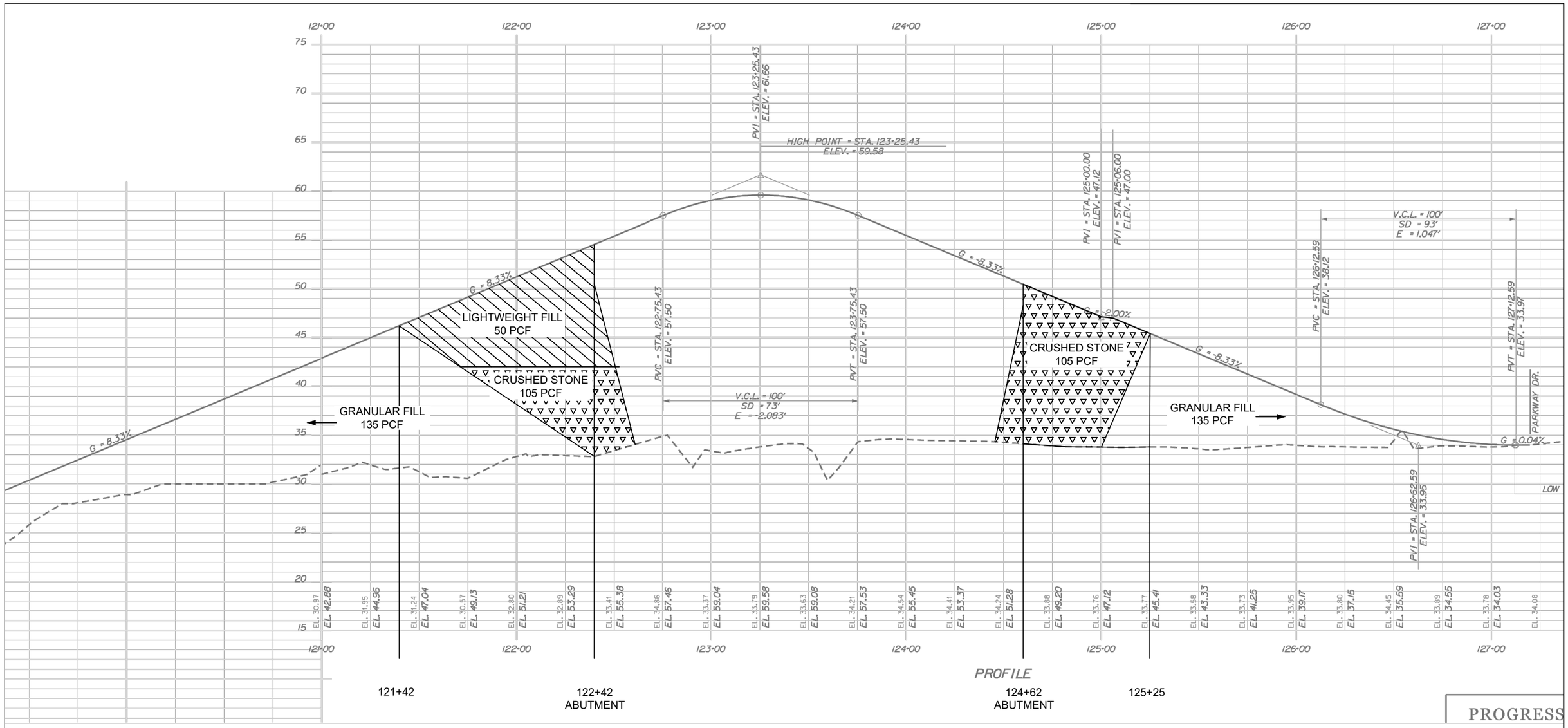
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VERTICAL 0 20 30 40
SCALE, FEET

FIGURE 4
INTERPRETIVE SUBSURFACE PROFILE
PAN AM CROSSING
GEOTECHNICAL EVALUATION
SCARBOROUGH EASTERN TRAIL
SCARBOROUGH, MAINE

FEBRUARY 2019 PROJECT NO. 0600-102

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G:\Autocad\600\600-102\SNP FIG 4, INTERPRETIVE SUBSURFACE PROFILE 0600-102.dwg, 2/4/2019 1:14:47 PM



PROFILE

121+42

122+42
ABUTMENT

124+62
ABUTMENT

125+25

PROGRESS

SOURCE:

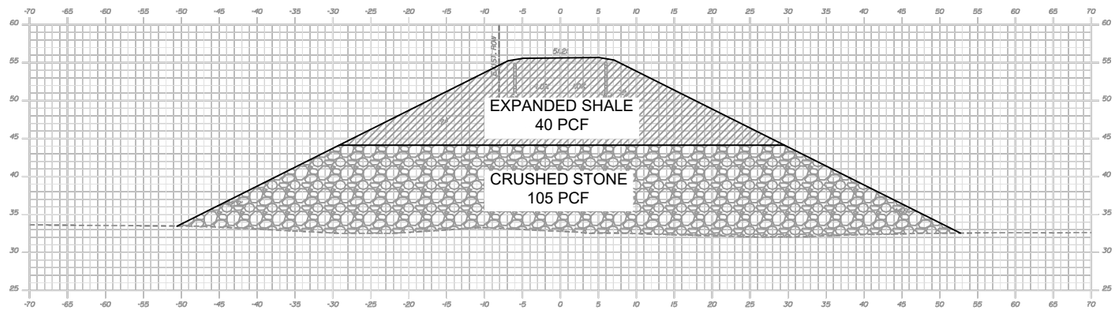
1. DRAWING SHEET NO. 26 OF 42, TITLED "PROFILE I", PREPARED BY STATE OF MAINE DEPARTMENT OF TRANSPORTATION, DATED "PROGRESS PRINT JULY 27, 2018".
2. DRAWING SHEET NO. 27 OF 42, TITLED "PROFILE II", PREPARED BY STATE OF MAINE DEPARTMENT OF TRANSPORTATION, DATED "PROGRESS PRINT JULY 27, 2018".

FIGURE 5
EMBANKMENT MATERIAL COMPOSITION
PANAM CROSSING
GEOTECHNICAL EVALUATION
SCARBOROUGH EASTERN TRAIL
SCARBOROUGH, MAINE

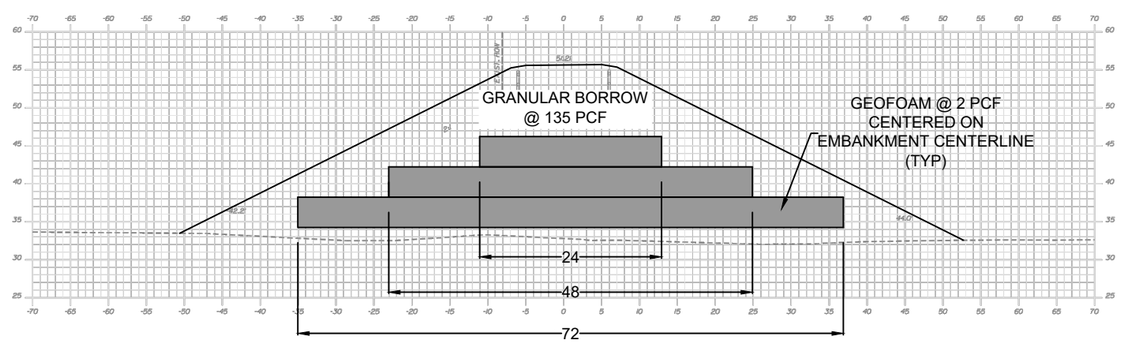
FEBRUARY 2019 PROJECT NO. 0600-102

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Geotechnical Engineering ● Materials Testing Servies
Environmental Consulting ● www.rwgillespie.com

G:\Autocad\600\600-102\FIG 5, EMBANKMENT MATERIAL COMPOSITION 0600-102.dwg, 2/4/2019 1:15:20 PM

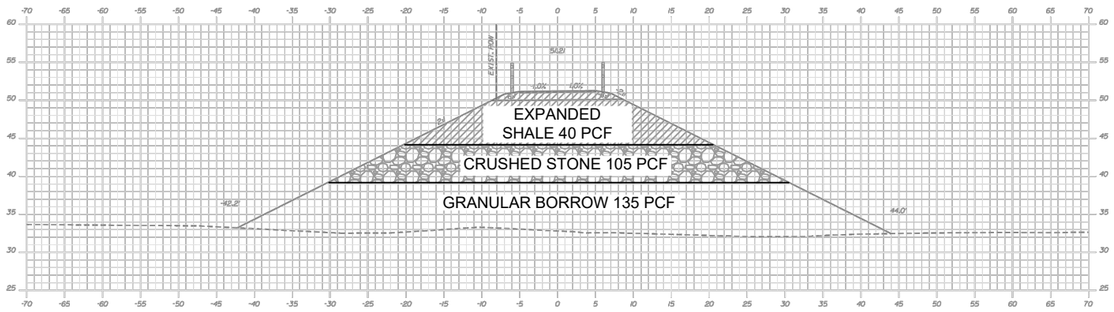


LIGHTWEIGHT AGGREGATE FILL ALTERNATIVE

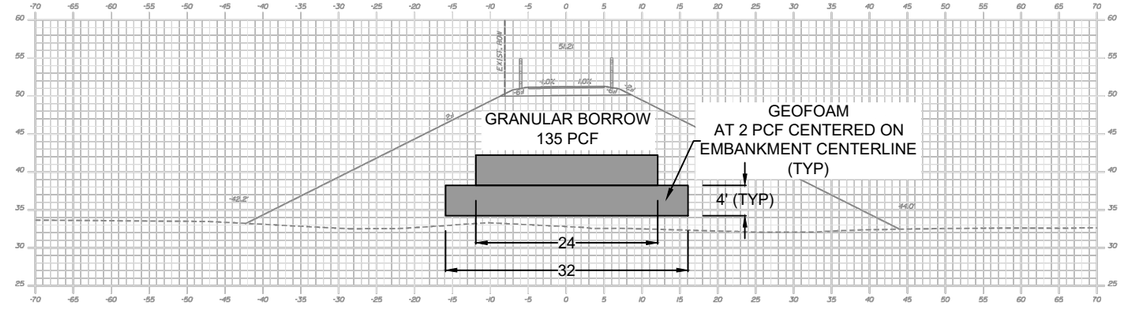


GEOFOAM ALTERNATIVE

STA. 122+42



LIGHTWEIGHT AGGREGATE FILL ALTERNATIVE



GEOFOAM ALTERNATIVE

STA. 121+92

NOT FOR CONSTRUCTION

0 20 30 40
SCALE, FEET

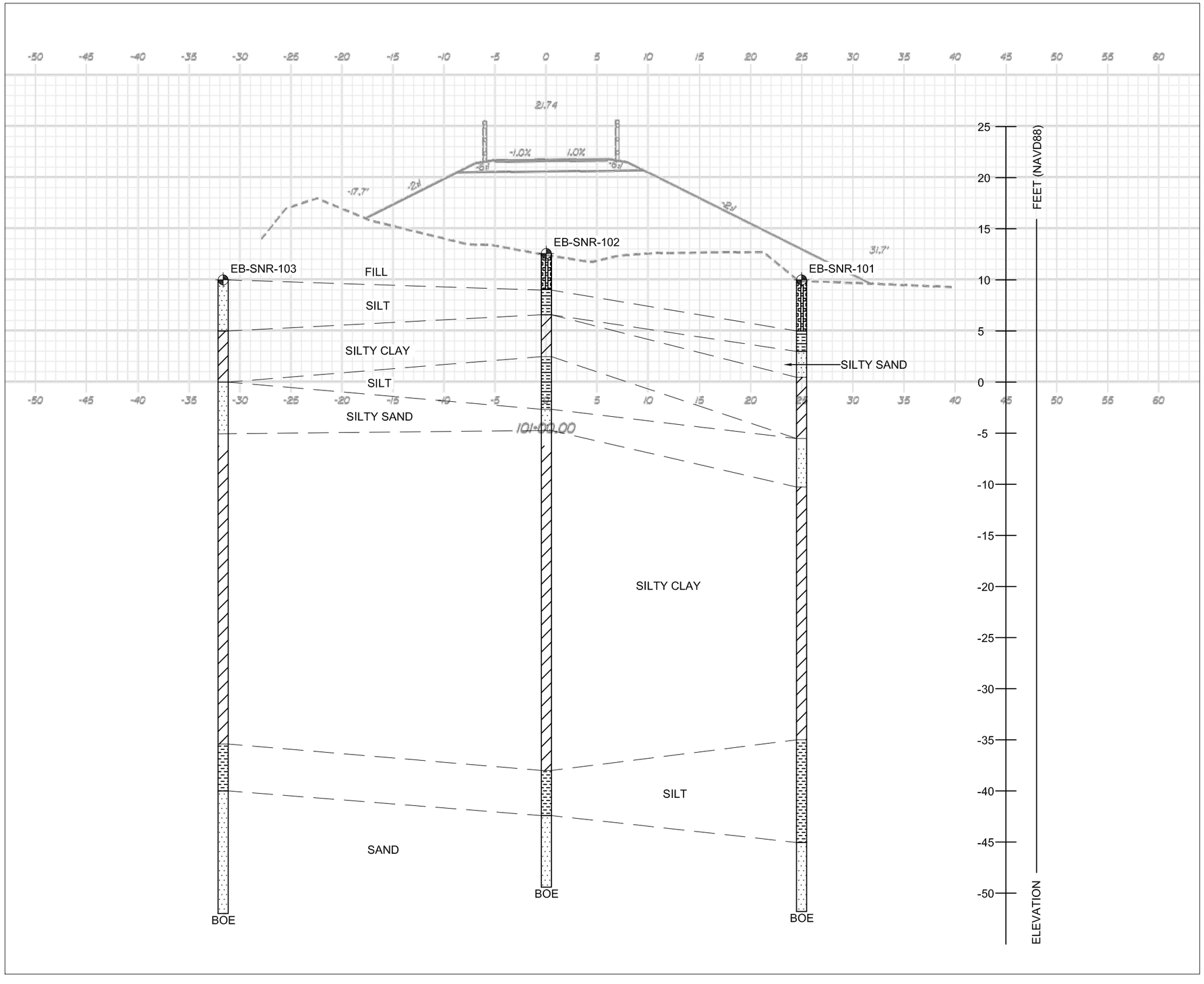
SOURCE:
DRAWING SHEET NO. 32 OF42, TITLED "PAN AM CROSSING CROSS SECTIONS", PREPARED BY STATE OF MAINE DEPARTMENT OF TRANSPORTATION, DATED "PROGRESS PRINT" JULY 27, 2018.

FIGURE 6
LIGHTWEIGHT EMBANKMENT FILL ALTERNATIVES
GEOTECHNICAL EVALUATION
SCARBOROUGH EASTERN TRAIL
SCARBOROUGH, MAINE

FEBRUARY 2019 PROJECT NO. 0600-102

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G:\Autocad\600\600-102\FIG 6, LIGHTWEIGHT EMBANKMENT FILL ALTERNATIVE 0600-102.dwg, 2/8/2019 11:11:13 AM



LEGEND:

EB-SNR-101 APPROXIMATE LOCATION OF SOIL BORING DRILLED JULY 2016.

SOURCE:
DRAWING SHEET NO. 18 OF 42, TITLED "NONESUCH RIVER CROSSING CROSS SECTIONS", PREPARED BY STATE OF MAINE DEPARTMENT OF TRANSPORTATION, DATE "PROGRESS PRINT JULY 27, 2018".

0 10 15 20
SCALE, FEET

FIGURE 7
INTERPRETIVE SUBSURFACE PROFILE
NONESUCH RIVER CROSSING
GEOTECHNICAL EVALUATION
SCARBOROUGH EASTERN TRAIL
SCARBOROUGH, MAINE

FEBRUARY 2019 PROJECT NO. 0600-102

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G:\Autocad\600\600\600-102\FIG 7. NONSUCH RIVER CROSSING INTERPRETIVE SUBSURFACE PROFILE 0600-102.dwg, 2/4/2019 1:09:39 PM

APPENDIX A

LIMITATIONS

Report of Geotechnical Evaluation
Scarborough Eastern Trail, MaineDOT Project No. STP-1942(610)
Scarborough, Maine

LIMITATIONS AND USE OF REPORT

This evaluation has been limited to consideration of the geotechnical aspects of the proposed Scarborough Eastern Trail, MaineDOT Project No. STP-1942(610) in Scarborough, Maine. The purpose of the evaluation was to obtain information regarding subsurface conditions on which to base recommendations about the geotechnical aspects of design and construction of embankments, foundations, and seismic considerations. This report identifies construction considerations intended to solely assist engineers that will design the project and monitor its construction, and not to the benefit of others including but not limited to the Contractor. This report is not a technical specification nor is it intended to be used as a specification for bidding or building the project.

This geotechnical evaluation might also aid Contractors responsible for construction of the planned building. However, the recommendations and comments provided hereinafter are not intended to be instructions or directives to the project Contractors. The project Contractors must evaluate construction issues encountered in the work on the basis of their experience with similar projects taking in to account their own methods and procedures.

In particular, the Contractor should be advised that the project site is underlain by medium to soft silty clay that is vulnerable to instability and landslides when subject to unbalanced loading from stockpiles of excavated soils and offsite fill. RWG&A's scope of services for this project excludes stability evaluations of the Contractor excavations and stockpiles.

RWG&A has not considered the construction from a worker safety perspective. Construction safety is the responsibility of the project Contractor, who is also solely responsible for the means, methods, and sequencing of construction operations. RWG&A is providing this information as a service to HNTB Corporation. Under no circumstances should this information be interpreted to mean that RWG&A and/or the HNTB Corporation are assuming responsibility for construction site safety or the Contractor's activities; such responsibility is not being implied and should not be inferred.
RWG&A's services excluded any environmental site assessment relative to oil and hazardous materials or evidence of a potential release or threat of oil or hazardous materials on, below, or around the site. (Note: any statement in this report, or on the exploration logs, regarding odors or unusual or suspicious conditions is for informational purposes only and is not intended to constitute an environmental assessment.)

APPENDIX B

EXPLORATION LOGS

Report of Geotechnical Evaluation
Scarborough Eastern Trail, MaineDOT Project No. STP-1942(610)
Scarborough, Maine

RWG&A, Inc. soil descriptions are based on the following criteria. Descriptive terminology is used to denote the grain size and percentage of each component. The soil descriptions are based on visual-manual classification procedures, Standard Penetration Test results, and the results of laboratory testing on selected soil samples, where available. The Unified Soil Classification Group Symbol will be indicated in capital letters.

COMPONENT DEFINITIONS BY GRADATION SIEVE LIMITS

Materials	Definitions	Fractions	Upper	Lower
Boulders	Material too large to pass through an opening 12 in. square.			
Cobbles	Material passing through a 12 in. opening and retained on the 3 in. sieve.			
Gravel	Material passing the 3 in. sieve and retained on 1/4" (No. 4 sieve).	Coarse Fine	3 in. 3/4 in.	3/4 in. 1/4 in.
Sand	Material passing the No. 4 sieve and retained on the No. 200 sieve.	Coarse Medium Fine	No. 4 (1/4") No. 10 (1/8") No. 40 (1/32")	No. 10 (1/8") No. 40 (1/32") No. 200
Silt	Material passing the No. 200 sieve which is usually non-plastic in character and exhibits little or no strength when air dried.		No. 200	
Clay	Material passing the No. 200 sieve which can also be made to exhibit plasticity within a certain range of moisture contents and which exhibits considerable strength when air dried.		No. 200	

SOIL DESCRIPTION

General

Soils are described as to the Unified Soil Classification Systems Group Symbol, density or consistency, color, grain size distribution and other pertinent properties such as plasticity and dry strength. The RWG&A order of descriptors is as follows:

1. USCS Group Name and Symbol, or Fill
2. Density or Consistency
3. Moisture
4. Grain Size & Constituent percentages
5. Other pertinent descriptors
6. Color

DESCRIPTIVE TERMINOLOGY DENOTING COMPONENT PROPORTIONS

Descriptive Terms	Range of Proportions
Noun (major component)	∃50%
Adjective (secondary component)	20 - 50%
Some (third component)	25 - 45%
Little (second or third component)	15 - 25%
Few (second or third component)	5 - 15%
Trace	0 - 5%
With	Amount of component not determined. Used as a conjunction only. Does not indicate component percentile

OTHER DESCRIPTIVE TERMS

Where appropriate, geological classifications are also used (Glacial Till, etc.)

TYPICAL DESCRIPTIONS

SAND WITH SILT (SP-SM): Medium dense, moist, coarse to medium sand, few silt, brown.
FILL; Loose, dry, fine sand, some gravel and silt, with brick and concrete fragments, dark brown.
SILTY CLAY (CL); Very stiff, moist, silty clay, olive-brown.

DENSITY OR CONSISTENCY OF SOILS

COHESIVE SOILS

Consistency of Cohesive Soils	Standard Penetration Test (Blows Per Foot) (N)	Undrained Shear Strength (TSF)
Very Soft	0 - 2	Below 0.13 (250 psf)
Soft	2 - 4	0.13 to 0.25 (to 500 psf)
Medium	4 - 8	0.25 to 0.5 (to 1,000 psf)
Stiff	8 - 15	0.5 to 1.0 (to 2,000 psf)
Very Stiff	15 - 30	1.0 to 2.0 (to 4,000 psf)
Hard	Over 30	over 2.0 (over 4,000 psf)

Consistency of cohesive soils is based upon field vane shear, torvane, or pocket penetrometer, or laboratory vane shear or Unconsolidated-Undrained Triaxial Compression tests. Consistency of cohesive soils is based upon the Standard Penetration test when no other data is available.

COHESIONLESS SOILS

Density of Cohesionless Soils	Standard Penetration Test (Blows per Foot) (in)
Very Loose	0 - 4
Loose	4 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	over 50

PENETRATION RESISTANCE

STANDARD PENETRATION TEST (ASTM D1586) - a 2.0-inch diameter, 1-3/8 inch inside diameter split barrel sample is driven into soil by means of a 140-pound weight falling freely through a vertical distance of 30 inches. The total number of blows required for penetration from 6 to 18 inches is the Standard Penetration Resistance (N).

COBBLES AND BOULDERS

The percentage of cobbles and boulders is estimated visually where possible.

Descriptive Term	Estimated Percentage
Very Few	0 - 10%
Few	10 - 25%
Common	25 - 40%
Numerous	40 - 50%

If the percentage cannot be determined, as in a typical test boring, then use "with" to indicate the presence of cobbles and/or boulders. (i.e., gravelly sand with cobbles and boulders).

FILLS

The following terminology is used to denote size range of man-made materials within fill deposits:

Size Range	Comparative Soil Terms
<No. 200 Sieve	Silt - size
No. 200 to 1/4 in.	Sand - size
1/4 in. to 3 in.	Gravel - size
3 in. to 12 in.	Cobble - size
>12 in.	Boulder - size

SUPPLEMENTAL SOIL DESCRIPTION TERMINOLOGY

Term	Example	
Seam	Typically 1/16 to 1/2 inch thick	1/4 inch sand seams
Layer	Greater than 1/2 inch thick	2-inch sand layers
Occasional	One or less per foot of thickness	
Frequent	More than one per foot of thickness	
Interbedded	Alternating soil layers of different composition	
Varved	Alternating thin seams of silt and clay	
Mottled	Variations in color	

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PAN-AM EXPLORATION LOGS

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- Environmental Consulting
- Materials Testing Services

Boring Log: BB-SPC-101
Total Depth (ft): 91.2
Sheet 1 of 4

Project Name: Scarborough Eastern Trail Project
RWG&A Project No. 0600-092
Location: Scarborough, Maine
Client: HNTB Corporation
RWG&A Representative: C. Morrell
Boring Location: See Exploration Location Plan
Boring Abandonment Method: Backfilled with cuttings
Observed Water Depth: 5.0'

Drilling Contractor: Great Works Test Boring
Drill Rig: CME 850
Driller Rep.: J. Lee
Date Started: 06/13/2013
Date Completed: 06/13/2013
Surface Elevation: 33.5 (FEET)
Drilling Method: Rotary Wash
Casing Type: 4" Steel

DEPTH, FT.	SYMBOL	SAMPLES	SAMPLE NUMBER	DESCRIPTION OF MATERIAL	SAMPLE RECOVERY, IN.	BLOWS PER 6"	SPT-N BLOWS PER FT.	MOISTURE CONTENT %	LAB TESTS
0			S-1	FILL; Silty sand, moist, fine sand, some silt, trace gravel, gray and black. FILL; Silty sand, moist, fine to medium sand, little silt, dark red-brown.	15	3 5 7 7	12	14.0	MC
				SILTY SAND (SP/SM); Loose, wet, fine to medium sand, little silt, light brown.					
5			S-2		18	4 4 4 6	8	27.4	MC GS
				SILTY SAND (SM); Loose to medium dense, wet, fine sand, some silt, gray.					
10			S-3	Silty sand becomes brown around 10 ft.	20	1 5 6 8	11	25.1	MC
				SILTY CLAY (CL); Very soft to medium stiff, wet, silty clay, trace fine sand, gray.					
15			S-4		1	WOH/24"	WOH	42.1	MC
20			FV	Field Vane Raw Data: T=74ft-lb, Residual T=5ft-lb					
			FV	Undrained Shear Strength: Su=0.81 ksf, Residual Su=0.05 ksf Field Vane Raw Data: T=45ft-lb, Residual T=4ft-lb Undrained Shear Strength: Su=0.49 ksf, Residual Su=0.04 ksf					
25			U-1	Thin-walled tube sample from 25' - 27'.	24				
			FV	Field Vane Raw Data: T=45ft-lb, Residual T=5ft-lb					
			FV	Undrained Shear Strength: Su=0.49 ksf, Residual Su=0.05 ksf Field Vane Raw Data: T=45ft-lb, Residual T=5ft-lb Undrained Shear Strength: Su=0.49 ksf, Residual Su=0.05 ksf					
30									

Notes: 1. MaineDOT Project No. 019386.00
2. SPT N-valves are uncorrected.
3. Shear Strength data collected using an Acker 3-5/8" x 9" Field vane and torque wrench.
4. Hydraulically pushed casing from 15' to 60' to continue drilling below 60', no hammer used to drive casing.

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Boring Log: BB-SPC-101
Total Depth: 91.2
Sheet 2 of 4

Project Name: Scarborough Eastern Trail Project
Location: Scarborough Maine
Client: HNTB Corporation
Observed Water Depth: 5.0'

RWG&A Project No. 0600-092
Surface Elevation: 33.5 (FEET)
Casing Type: 4" Steel

DEPTH, FT.	SYMBOL	SAMPLES	SAMPLE NUMBER	DESCRIPTION OF MATERIAL	SAMPLE RECOVERY, IN.	BLOWS PER 6"	SPT-N BLOWS PER FT.	MOISTURE CONTENT %	LAB TESTS
30			FV	Field Vane Raw Data: T=39ft-lb, Residual T=5ft-lb					
			FV	Undrained Shear Strength: Su=0.43 ksf, Residual Su=0.05 ksf Field Vane Raw Data: T=40ft-lb, Residual T=5ft-lb Undrained Shear Strength: Su=0.44 ksf, Residual Su=0.05 ksf					
35			U-2	Thin-walled tube sample from 35' - 37'.	22				
			FV	Field Vane Raw Data: T=65ft-lb, Residual T=4ft-lb					
			FV	Undrained Shear Strength: Su=0.71 ksf, Residual Su=0.04 ksf Field Vane Raw Data: T=61ft-lb, Residual T=5ft-lb Undrained Shear Strength: Su=0.66 ksf, Residual Su=0.05 ksf					
40			FV	Field Vane Raw Data: T=50ft-lb, Residual T=3ft-lb.					
			FV	Undrained Shear Strength: Su=0.55 ksf, Residual Su=0.03 ksf Field Vane Raw Data: T=54ft-lb, Residual T=3ft-lb Undrained Shear Strength: Su=0.59 ksf, Residual Su=0.03 ksf					
45			U-3	Thin-walled tube sample from 45' - 47'.	23				
			FV	Field Vane Raw Data: T=72fft-lb, Residual T=9ft-lb					
			FV	Undrained Shear Strength: Su=0.78 ksf, Residual Su=0.10 ksf Field Vane Raw Data: T=74ft-lb, Residual T=11ft-lb Undrained Shear Strength: Su=0.81 ksf, Residual Su=0.12 ksf					
50			S-5	Unable to push field vane at 50'. Occasional fine sand seams from 50' - 51'.	21	3 1 1 1	2	33.5	MC
55			FV	Field Vane Raw Data: T=79ft-lb, Residual T=0ft-lb Undrained Shear Strength: Su=0.86 ksf, Residual Su=0.00 ksf Unable to push field vane below 55.9'.					
			S-6	SILTY SAND (SM); Medium dense, wet, fine to medium sand, little silt, trace coarse sand, gray with occasional red-brown oxidized layers.	16	9 13 13	26	20.7	MC
60									

Notes: 1. MaineDOT Project No. 019386.00
2. SPT N-valves are uncorrected.
3. Shear Strength data collected using an Acker 3-5/8" x 9" Field vane and torque wrench.
4. Hydraulically pushed casing from 15' to 60' to continue drilling below 60', no hammer used to drive casing.

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Boring Log: BB-SPC-101
Total Depth: 91.2
Sheet 3 of 4

Project Name: Scarborough Eastern Trail Project
Location: Scarborough Maine
Client: HNTB Corporation
Observed Water Depth: 5.0'

RWG&A Project No. 0600-092
Surface Elevation: 33.5 (FEET)
Casing Type: 4" Steel

DEPTH, FT.	SYMBOL	SAMPLES	SAMPLE NUMBER	DESCRIPTION OF MATERIAL	SAMPLE RECOVERY, IN.	BLOWS PER 6"	SPT-N BLOWS PER FT.	MOISTURE CONTENT %	LAB TESTS
60						14			
65									
70									
			S-7	SILTY SAND WITH GRAVEL (SM); Medium dense to very dense, wet, fine to medium sand, some silt, few fine gravel, gray. [Glacial Till]	20	6 9 10 30	19	9.6	MC
75									
			S-8	Increased drilling resistance, possible cobbles from 78' - 79'.	8	30 50 30/0"	80+	8.3	MC
80									
85									
90									

Notes: 1. MaineDOT Project No. 019386.00
2. SPT N-valves are uncorrected.
3. Shear Strength data collected using an Acker 3-5/8" x 9" Field vane and torque wrench.
4. Hydraulically pushed casing from 15' to 60' to continue drilling below 60', no hammer used to drive casing.

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Boring Log: BB-SPC-101
Total Depth: 91.2
Sheet 4 of 4

Project Name: Scarborough Eastern Trail Project
Location: Scarborough Maine
Client: HNTB Corporation
Observed Water Depth: 5.0'

RWG&A Project No. 0600-092
Surface Elevation: 33.5 (FEET)
Casing Type: 4" Steel

DEPTH, FT.	SYMBOL	SAMPLES	SAMPLE NUMBER	DESCRIPTION OF MATERIAL	SAMPLE RECOVERY, IN.	BLOWS PER 6"	SPT-N BLOWS PER FT.	MOISTURE CONTENT %	LAB TESTS
90				Possible bedrock at 91', able to drill 2 - 3" into rock until tri-cone bit continually plugged.					
				Bottom of Exploration at 91.2'; tri-cone bit refusal, possible boulder or bedrock.					
95									
100									
105									
110									
115									
120									

Notes: 1. MaineDOT Project No. 019386.00
2. SPT N-valves are uncorrected.
3. Shear Strength data collected using an Acker 3-5/8" x 9" Field vane and torque wrench.
4. Hydraulically pushed casing from 15' to 60' to continue drilling below 60', no hammer used to drive casing.

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Boring Log: BB-SPC-102
Total Depth (ft): 101.5
Sheet 1 of 4

Project Name: Scarborough Eastern Trail Project
RWG&A Project No. 0600-092
Location: Scarborough, Maine
Client: HNTB Corporation
RWG&A Representative: C. Morrell
Boring Location: See Exploration Location Plan
Boring Abandonment Method: Backfilled with cuttings
Observed Water Depth: 5.0'

Drilling Contractor: Great Works Test Boring
Drill Rig: CME 850
Driller Rep.: J. Lee
Date Started: 06/12/2013
Date Completed: 06/12/2013
Surface Elevation: 31.5 (FEET)
Drilling Method: Rotary Wash
Casing Type: 4" Steel

DEPTH, FT.	SYMBOL	SAMPLES	SAMPLE NUMBER	DESCRIPTION OF MATERIAL	SAMPLE RECOVERY, IN.	BLOWS PER 6"	SPT-N BLOWS PER FT.	MOISTURE CONTENT %	LAB TESTS
0			S-1	TOPSOIL AND ORGANIC MATERIAL (4 inches).	14	1 1 2 5	3	21.0	MC
				SILTY SAND (SP/SM); Very loose to medium dense, moist, fine sand, little silt, orange-brown.					
5			S-2	SILTY SAND (SP/SM); Medium dense, wet, fine to medium sand, little silt, orange-brown. Becomes gray-brown around 6ft.	24	4 7 7 10	14	22.6	MC
10			S-3	SILTY SAND (SM); Loose, wet, fine sand, some silt, gray.	13	4 6 4 5	10	26.1	MC
15			S-4	SILTY CLAY WITH SAND (CL-ML); Very soft to medium stiff, wet, silty clay, occasional fine sand seams, gray.	22	WOH/ 18" 6	WOH	41.8	MC
20			S-5		24	WOH/ 24"	WOH	46.0	MC
25			FV	Field Vane Raw Data: T=50ft-lb, Residual T=4ft-lb					
			FV	Undrained Shear Strength: Su=0.55 ksf, Residual Su=0.04 ksf Field Vane Raw Data: T=39ft-lb, Residual T=2ft-lb Undrained Shear Strength: Su=0.43 ksf, Residual Su=0.02 ksf					
30									

Notes: 1. MaineDOT Project No. 019386.00
2. SPT N-valves are uncorrected.
3. Shear Strength data collected using an Acker 3-5/8" x 9" Field vane and torque wrench.
4. Hydraulically pushed casing from 15' to 60' to continue drilling below 60', no hammer used to drive casing.

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- Geotechnical Engineering
- Environmental Consulting
- Materials Testing Services

Boring Log: BB-SPC-102
Total Depth: 101.5
Sheet 2 of 4

Project Name: Scarborough Eastern Trail Project
Location: Scarborough Maine
Client: HNTB Corporation
Observed Water Depth: 5.0'

RWG&A Project No. 0600-092
Surface Elevation: 31.5 (FEET)
Casing Type: 4" Steel

DEPTH, FT.	SYMBOL	SAMPLES	SAMPLE NUMBER	DESCRIPTION OF MATERIAL	SAMPLE RECOVERY, IN.	BLOWS PER 6"	SPT-N BLOWS PER FT.	MOISTURE CONTENT %	LAB TESTS
30			U-1	Thin-walled tube sample from 30' - 32'.	20			50.2	C AL GV MC
35			FV	Field Vane Raw Data: T=39ft-lb, Residual T=1ft-lb					
			FV	Undrained Shear Strength: Su=0.40 ksf, Residual Su=0.01 ksf Field Vane Raw Data: T=42ft-lb, Residual T=1ft-lb Undrained Shear Strength: Su=0.46 ksf, Residual Su=0.01 ksf					
40			U-2	Thin-walled tube sample from 40' - 42'.	12				
45			FV	Field Vane Raw Data: T=66ft-lb, Residual T=1ft-lb					
			FV	Undrained Shear Strength: Su=0.72 ksf, Residual Su=0.01 ksf Field Vane Raw Data: T=75ft-lb, Residual T=0ft-lb Undrained Shear Strength: Su=0.82 ksf, Residual Su=0.00 ksf					
50			U-3	Thin-walled tube sample from 50' - 52'.	24			40.9	C GV MC
55			FV	Field Vane Raw Data: T=66ft-lb, Residual T=0ft-lb					
			FV	Undrained Shear Strength: Su=0.72 ksf, Residual Su=0.00 ksf Field Vane Raw Data: T=90ft-lb, Residual T=0ft-lb Undrained Shear Strength: Su=0.98 ksf, Residual Su=0.00 ksf					
60									

Notes: 1. MaineDOT Project No. 019386.00
2. SPT N-valves are uncorrected.
3. Shear Strength data collected using an Acker 3-5/8" x 9" Field vane and torque wrench.
4. Hydraulically pushed casing from 15' to 60' to continue drilling below 60', no hammer used to drive casing.

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- Environmental Consulting
- Materials Testing Services

Boring Log: BB-SPC-102
Total Depth: 101.5
Sheet 3 of 4

Project Name: Scarborough Eastern Trail Project
Location: Scarborough Maine
Client: HNTB Corporation
Observed Water Depth: 5.0'

RWG&A Project No. 0600-092
Surface Elevation: 31.5 (FEET)
Casing Type: 4" Steel

DEPTH, FT.	SYMBOL	SAMPLES	SAMPLE NUMBER	DESCRIPTION OF MATERIAL	SAMPLE RECOVERY, IN.	BLOWS PER 6"	SPT-N BLOWS PER FT.	MOISTURE CONTENT %	LAB TESTS
60			FV	Field Vane Raw Data: T=50ft-lb, Residual T=0ft-lb					
			FV	Undrained Shear Strength: Su=0.55 ksf, Residual Su=0.00 ksf Field Vane Raw Data: T=79ft-lb, Residual T=0ft-lb Undrained Shear Strength: Su=0.86 ksf, Residual Su=0.00 ksf					
65			FV	Field Vane Raw Data: T=88ft-lb, Residual T=12ft-lb					
			FV	Undrained Shear Strength: Su=0.96 ksf, Residual Su=0.13 ksf Field Vane Raw Data: T=87ft-lb, Residual T=13ft-lb Undrained Shear Strength: Su=0.95 ksf, Residual Su=0.14 ksf					
70			S-6	SILTY SAND (SM); Medium dense to dense, wet, fine sand, some silt, trace medium sand, gray-brown.	14	10 11 18 18	29	24.1	MC
75			S-7		12	12 17 18 22	35	20.0	MC
80			S-8		14	12 15 20 31	35	21.3	MC
85			S-9	Occasional medium to coarse sand from 85' - 86'.	20	15 13 19 26	32	22.2	MC
90									

Notes: 1. MaineDOT Project No. 019386.00
2. SPT N-valves are uncorrected.
3. Shear Strength data collected using an Acker 3-5/8" x 9" Field vane and torque wrench.
4. Hydraulically pushed casing from 15' to 60' to continue drilling below 60', no hammer used to drive casing.

R.W. Gillespie & Associates

- Geotechnical Engineering
- Environmental Consulting
- Materials Testing Services

Boring Log: BB-SPC-102
Total Depth: 101.5
Sheet 4 of 4

Project Name: Scarborough Eastern Trail Project
Location: Scarborough Maine
Client: HNTB Corporation
Observed Water Depth: 5.0'

RWG&A Project No. 0600-092
Surface Elevation: 31.5 (FEET)
Casing Type: 4" Steel

DEPTH, FT.	SYMBOL	SAMPLES	SAMPLE NUMBER	DESCRIPTION OF MATERIAL	SAMPLE RECOVERY, IN.	BLOWS PER 6"	SPT-N BLOWS PER FT.	MOISTURE CONTENT %	LAB TESTS
90			S-10		15	10 14 17 24	31	26.2	MC
				SILTY SAND WITH GRAVEL (SM); Dense, wet, fine to medium sand, little silt, few fine gravel, orange-brown.					
95			S-11		12	13 20 30 31	50	13.5	MC
				SILTY SAND WITH GRAVEL (SM); Dense to very dense, wet, fine sand, some silt, little gravel, gray. [Glacial Till]					
100			S-12		14	17 32 50/5"	82	10.3	MC
				Bottom of Exploration at 101.5'; Terminated boring in Glacial Till.					
105									
110									
115									
120									

Notes: 1. MaineDOT Project No. 019386.00
2. SPT N-valves are uncorrected.
3. Shear Strength data collected using an Acker 3-5/8" x 9" Field vane and torque wrench.
4. Hydraulically pushed casing from 15' to 60' to continue drilling below 60', no hammer used to drive casing.

Maine Department of Transportation

Soil/Rock Exploration Log
US UNITS

Project: Scarborough Eastern Trail Project
Location: Scarborough, Maine

Boring No.: BB-SPC-201
PIN: 1942(610)

Driller:	R.W. Gillespie & Associates, Inc.	**Elevation (ft.)**	N/A	**Auger ID/OD:**	4.25" ID / 7.5" OD
Operator:	M. Nadeau	**Datum:**		**Sampler:**	SPT Split-Barrel
Logged By:	D. Walker	**Rig Type:**	Diedrich D50	**Hammer Wt./Fall:**	140 lbs. 30"
Date Start/Finish:	10/07/2016 / 10/07/2016	**Drilling Method:**	HSA, 3" roller bit, drive + wash	**Core Barrel:**	NQ2
Boring Location:	See Exploration Location Plan	**Casing ID/OD:**	4" ID	**Water Level*:**	6.5'

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

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 (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	S-1	24/10	0.0 - 2.0		22				SAND WITH SILT (SP-SM); Brown dry, medium dense, fine sand, some silt, little gravel, reddish band at 2'.	
5	S-2	24/14	5.0 - 7.0		2				5.0 SAND (SP); Brown, moist to wet, very loose, medium to coarse sand. 6.5 SILTY SAND (SM); Gray, wet, very loose, fine sand, some silt. 1" silty clay lense at 6.5'.	
10	S-3	24/14	10.0 - 12.0		3				Grades to include trace clay. 11.0 SAND WITH SILT (SP-SM); Tan, wet, very loose, fine sand, little silt.	
15	FV-1		16.0 - 16.8	Su=659 psf Residual Su=91 psf					14.0 SILTY CLAY (CL); Gray, wet, very soft to medium, clay with silt. 65x130 mm raw torque readings: FV-1: 24.0/3.3 ft-lbs	
	FV-2		17.0 - 17.8	Su=637 psf Residual Su=82 psf					FV-2: 23.2/3.0 ft-lbs	
20	U	24/0	20.0 - 22.0						Attempted Shelby Tube, no recovery.	
	U-1	24/24	22.0 - 24.0							
25	FV-3		26.0 - 26.8	Su=519 psf Residual Su=96 psf					65x130 mm raw torque readings: FV-3: 18.9/3.5 ft-lbs	
	FV-4		27.0 - 27.8	Su=483 psf Residual Su=69 psf					FV-4: 17.6/2.5 ft-lbs	
30										

Remarks:

1. RWG&A Project No. 0600-102.
2. In-situ vane shear tests conducted using MaineDOT provided Geonor 65x130mm rectangular vane.
3. Hammer Serial No. 283.
4. SPT N-Values are uncorrected.

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 UNITS

Project: Scarborough Eastern Trail Project
Location: Scarborough, Maine

Boring No.: BB-SPC-201
PIN: 1942(610)

Driller:	R.W. Gillespie & Associates, Inc.	**Elevation (ft.)**	N/A	**Auger ID/OD:**	4.25" ID / 7.5" OD
Operator:	M. Nadeau	**Datum:**		**Sampler:**	SPT Split-Barrel
Logged By:	D. Walker	**Rig Type:**	Diedrich D50	**Hammer Wt./Fall:**	140 lbs. 30"
Date Start/Finish:	10/07/2016 / 10/07/2016	**Drilling Method:**	HSA, 3" roller bit, drive + wash	**Core Barrel:**	NQ2
Boring Location:	See Exploration Location Plan	**Casing ID/OD:**	4" ID	**Water Level*:**	6.5'

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

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample Information: 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.
30	U-2	24/24	30.0 - 32.0							14430 C
35	FV-5		36.0 - 36.8	Su=475 psf Residual Su=33 psf					65x130 mm raw torque readings: FV-5: 17.3/1.2 ft-lbs FV-6: 18.6/1.2 ft-lbs	
	FV-6		37.0 - 37.8	Su=511 psf Residual Su=33 psf						
40	S-4	24/24	40.0 - 42.0		WOR					
45	U-3	24/24	45.0 - 47.0							
50	S-5	24/24	50.0 - 52.0		2				Could not advance field vane. Silty sand lense at 51'. Becomes soft, dark gray.	
									52.0 **Bottom of Exploration at 52.0 feet below ground surface.** Not refusal.	
55										
60										

Remarks:

1. RWG&A Project No. 0600-102.
2. In-situ vane shear tests conducted using MaineDOT provided Geonor 65x130mm rectangular vane.
3. Hammer Serial No. 283.
4. SPT N-Values are uncorrected.

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Maine Department of Transportation

Soil/Rock Exploration Log
US UNITS

Project: Scarborough Eastern Trail Project
Location: Scarborough, Maine

Boring No.: BB-SPC-202
PIN: 1942(610)

Driller:	R.W. Gillespie & Associates, Inc.	**Elevation (ft)**	N/A	**Auger ID/OD:**	4.25" ID / 7.5" OD
Operator:	M. Nadeau	**Datum:**		**Sampler:**	SPT Split-Barrel
Logged By:	D. Walker	**Rig Type:**	Diedrich D50	**Hammer Wt./Fall:**	140lbs. 30"
Date Start/Finish:	10/04/2016 / 10/06/2016	**Drilling Method:**	HSA, 3" roller bit, drive + wash	**Core Barrel:**	NQ2
Boring Location:	See Exploration Location Plan	**Casing ID/OD:**	3" and 4" ID	**Water Level*:**	7'

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

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 (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	S-1	24/15	0.0 - 2.0		13				SANDY SILT (ML): Brown to dark brown, dry, medium dense, silt, little fine to medium sand, little gravel.	
5	S-2	24/23	5.0 - 7.0		15				5.0 SAND (SP): Tan to reddish brown, moist, medium dense, fine sand, trace silt, grading to fine to coarse sand.	
	S-3	24/20	7.0 - 9.0		13				7.0 SANDY SILT (ML): Gray, wet, stiff, silt with some fine sand. SILTY SAND (SM): Gray, wet, medium dense, fine sand, some silt.	
10	S-4	24/18	10.0 - 12.0		9	P			10.0 SAND (SP): Gray, wet, loose, fine sand, little silt.	
	S-5	24/24	12.0 - 14.0		7				Occasional silt lenses.	
15	S-7	24/24	15.0 - 17.0		WOH				14.0 SILTY CLAY (CL): Gray, wet, very soft to medium, clay, some silt. Occasional silt lenses.	
20	U-1	24/24	20.0 - 22.0							
25	FV-1		25.0 - 25.8	Su=445 psf Residual=52 psf					65x130 mm raw torque readings: FV-1: 16.2/1.9 ft-lbs FV-2: 21.5/2.3 ft-lbs	
	FV-2		26.0 - 26.8	Su=590 psf Residual=63 psf						
30										

Remarks:

1. RWG&A Project No. 0600-102.
2. In-situ vane shear tests conducted using MaineDOT provided Geonor 65x130mm rectangular vane.
3. Hammer Serial No. 283.
4. SPT N-Values are uncorrected.

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Maine Department of Transportation

Soil/Rock Exploration Log
US UNITS

Project: Scarborough Eastern Trail Project
Location: Scarborough, Maine

Boring No.: BB-SPC-202
PIN: 1942(610)

Driller:	R.W. Gillespie & Associates, Inc.	**Elevation (ft.)**	N/A	**Auger ID/OD:**	4.25" ID / 7.5" OD
Operator:	M. Nadeau	**Datum:**		**Sampler:**	SPT Split-Barrel
Logged By:	D. Walker	**Rig Type:**	Diedrich D50	**Hammer Wt./Fall:**	140lbs. 30"
Date Start/Finish:	10/04/2016 / 10/06/2016	**Drilling Method:**	HSA, 3" roller bit, drive + wash	**Core Barrel:**	NQ2
Boring Location:	See Exploration Location Plan	**Casing ID/OD:**	3" and 4" ID	**Water Level*:**	7'

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
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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

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 (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
30	U-2	24/24	30.0 - 32.0							
35	FV-3		36.0 - 36.8	Su=563 psf Residual=52 psf					65x130 mm raw torque readings: FV-3: 20.5/1.9 ft-lbs FV-4: 18.5/1.2 ft-lbs	
	FV-4		37.0 - 37.8	Su=508 psf Residual=33 psf						
40	U-3	24/24	40.0 - 42.0							
45	FV-5		46.0 - 46.8	Su=794 psf Residual Su=107 psf					65x130 mm raw torque readings: FV-5: 28.9/3.9 ft-lbs FV-6: 31.3/3.5 ft-lbs	
	FV-6		47.0 - 47.8	Su=860 psf Residual Su=96 psf						
									49.0 SANDY SILT (ML): Dark gray, wet, loose, silt, with fine sand.	
50	U-4	12/12	50.0 - 51.0							
55	S-9	24/12	55.0 - 57.0		9				55.0 SILTY SAND (SM); Gray, wet, loose to medium dense, fine sand, little silt, little clay, few fine to medium gravel.	14380A WC=13.2% G
60										

Remarks:

1. RWG&A Project No. 0600-102.
2. In-situ vane shear tests conducted using MaineDOT provided Geonor 65x130mm rectangular vane.
3. Hammer Serial No. 283.
4. SPT N-Values are uncorrected.

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 UNITS

Project: Scarborough Eastern Trail Project
Location: Scarborough, Maine

Boring No.: BB-SPC-202
PIN: 1942(610)

Driller:	R.W. Gillespie & Associates, Inc.	**Elevation (ft.)**	N/A	**Auger ID/OD:**	4.25" ID / 7.5" OD
Operator:	M. Nadeau	**Datum:**		**Sampler:**	SPT Split-Barrel
Logged By:	D. Walker	**Rig Type:**	Diedrich D50	**Hammer Wt./Fall:**	140lbs. 30"
Date Start/Finish:	10/04/2016 / 10/06/2016	**Drilling Method:**	HSA, 3" roller bit, drive + wash	**Core Barrel:**	NQ2
Boring Location:	See Exploration Location Plan	**Casing ID/OD:**	3" and 4" ID	**Water Level*:**	7'

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

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 (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
60	S-10	24/24	60.0 - 62.0		5					14380B WC=11.2% G
						25				
						25				
						58				
65	S-11	24/7	65.0 - 67.0		13	39			Becomes medium dense.	
						9				
						22				
						30			68.0	
						30			SAND WITH CLAY (SP-SC); Brownish gray, wet, medium dense, fine to medium sand, little coarse sand, few clay, few fine gravel, trace silt.	
70	S-12	24/14	70.0 - 72.0		26	15				12380C WC=14.4% G
						23				
						39				
						18			73.0	
						19			SILTY SAND (SM); Gray, wet, medium dense, fine to medium sand, some silt, few clay, few fine gravel.	
75	S-13	24/12	75.0 - 77.0		18	81				
						114				
						130				
						130				
						100				
80	S-14	24/13	80.0 - 82.0		47	25			Becomes dense.	12380D WC=9.6% G
						25				
						40				
						50				
						51			Drilling resistance at 84'. Rock fragments and rounded gravel in cuttings.	
85	S-15	5/6	85.0 - 85.4		50/5"				85.0	
									GRAVEL WITH SILT AND SAND (GM); Gray, wet, very dense, medium to coarse gravel, some silt, little fine to coarse sand.	
									Drilling resistance at 88'.	
90										

Remarks:

1. RWG&A Project No. 0600-102.
2. In-situ vane shear tests conducted using MaineDOT provided Geonor 65x130mm rectangular vane.
3. Hammer Serial No. 283.
4. SPT N-Values are uncorrected.

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

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Maine Department of Transportation

Soil/Rock Exploration Log
US UNITS

Project: Scarborough Eastern Trail Project
Location: Scarborough, Maine

Boring No.: BB-SPC-202
PIN: 1942(610)

Driller:	R.W. Gillespie & Associates, Inc.	**Elevation (ft.)**	N/A	**Auger ID/OD:**	4.25" ID / 7.5" OD
Operator:	M. Nadeau	**Datum:**		**Sampler:**	SPT Split-Barrel
Logged By:	D. Walker	**Rig Type:**	Diedrich D50	**Hammer Wt./Fall:**	140lbs. 30"
Date Start/Finish:	10/04/2016 / 10/06/2016	**Drilling Method:**	HSA, 3" roller bit, drive + wash	**Core Barrel:**	NQ2
Boring Location:	See Exploration Location Plan	**Casing ID/OD:**	3" and 4" ID	**Water Level*:**	7'

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

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample Information: 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.
90	S-16	2/1	90.0 - 90.2		50/2"				Black, wet, very dense, broken rock, fine to medium gravel.	
95	S-17	3/0	95.0 - 95.3		50/3"				No sample recovery.	
100	S-18	3/3	100.0 - 100.3		50/3"					
105										
110	S-19	10/4	110.0 - 110.8		86/10"					
115										
120										

Remarks:

1. RWG&A Project No. 0600-102.
2. In-situ vane shear tests conducted using MaineDOT provided Geonor 65x130mm rectangular vane.
3. Hammer Serial No. 283.
4. SPT N-Values are uncorrected.

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 UNITS

Project: Scarborough Eastern Trail Project
Location: Scarborough, Maine

Boring No.: BB-SPC-202
PIN: 1942(610)

Driller:	R.W. Gillespie & Associates, Inc.	**Elevation (ft.)**	N/A	**Auger ID/OD:**	4.25" ID / 7.5" OD
Operator:	M. Nadeau	**Datum:**		**Sampler:**	SPT Split-Barrel
Logged By:	D. Walker	**Rig Type:**	Diedrich D50	**Hammer Wt./Fall:**	140lbs. 30"
Date Start/Finish:	10/04/2016 / 10/06/2016	**Drilling Method:**	HSA, 3" roller bit, drive + wash	**Core Barrel:**	NQ2
Boring Location:	See Exploration Location Plan	**Casing ID/OD:**	3" and 4" ID	**Water Level*:**	7'

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:
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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

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample Information: 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.
120	S-20	11/7	120.0 - 120.9		92/11"				120.0 SILT (ML): Gray, wet, very dense, silt with little fine to medium gravel, trace fine sand.	
125									126.0 Increased drilling resistance, possible weathered rock.	
									127.0 Top of Bedrock.	
	R-1		128.0 - 128.5	RQD<25%					Gray to black, aphanitic to fine grained PHYLLITE. Rock is very soft to hard, slightly to highly weathered, thinly bedded, moderate to extremely fractured with low to vertical jointing. Cleavage planes are slightly rough to rough with tight to very wide joint wall separations and slight to moderately weathered joints. Some clay infilling in fractures. Characteristics similar to rock types of the Scarboro formation.	
130	R-2		129.0 - 133.5	RQD<25%					R-1: Recovery=6"/6" R-2: Recovery=42"/54" R-3: Recovery=19"/36"	
	R-3		133.0 - 136.0	RQD<25%					136.0 **Bottom of Exploration at 136.0 feet below ground surface.**	
135										
140										
145										
150										

Remarks:

1. RWG&A Project No. 0600-102.
2. In-situ vane shear tests conducted using MaineDOT provided Geonor 65x130mm rectangular vane.
3. Hammer Serial No. 283.
4. SPT N-Values are uncorrected.

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 UNITS

Project: Scarborough Eastern Trail Project
Location: Scarborough, Maine

Boring No.: BB-SPC-203
PIN: 1942(610)

Driller:	R.W. Gillespie & Associates, Inc.	**Elevation (ft.)**	N/A	**Auger ID/OD:**	4.25" ID / 7.5" OD
Operator:	M. Nadeau	**Datum:**		**Sampler:**	SPT Split-Barrel
Logged By:	D. Walker	**Rig Type:**	Diedrich D50	**Hammer Wt./Fall:**	140 lbs / 30 in.
Date Start/Finish:	09/27/2016 / 09/30/2016	**Drilling Method:**	HSA, 3" roller bit, drive + wash	**Core Barrel:**	NQ2
Boring Location:	See Exploration Location Plan	**Casing ID/OD:**	3- and 4-inch ID	**Water Level*:**	8'

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

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 (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	S-1	24/15	0.0 - 2.0		4				TOPSOIL AND ORGANIC MATERIAL (2 inches). 0.2	
									SAND WITH SILT (SP-SM); Reddish brown, dry, loose, fine to medium sand, little silt. 2.0	
	S-2	24/15	2.0 - 4.0		26				SAND (SP); Tan, dry, medium dense, fine to coarse sand, trace silt.	
5	S-3	24/15	5.0 - 7.0		15				Becomes reddish brown, moist.	
	S-4	24/20	7.0 - 9.0		13				2" lense of gray, wet, stiff, silty clay. 8.5	
									SILT (ML); Gray, wet, stiff, silt, trace fine sand. 10.0	
10	S-5	24/12	10.0 - 12.0		5				SILTY SAND (SM); Tan to gray, wet, loose, fine sand, some silt.	
	S-6	24/24	12.0 - 14.0		7					
15	S-7	24/24	15.0 - 17.0		WOH/ 24				SILTY CLAY (CL-ML); Gray, wet, very soft to medium, clay, with some silt, occasional silt lenses.	
20	FV-1		21.0 - 21.8	Su=714 psf, residual Su=41 psf					65x130 mm raw torque readings: FV-1: 26.0/1.5 ft-lbs	
	FV-2		22.0 - 22.8	Su=920 psf, residual=247 psf					FV-2: 33.5/9.0 ft-lbs	
25	U-1	24/24	25.0 - 27.0							14328A & 14429 WC=47.4% G & C
	FV-3		28.0 - 28.8	Su=434 psf, residual=33 psf					65x130 mm raw torque readings: FV-3: 15.8/1.2 ft-lbs	
30	FV-4		29.0 - 29.8	Su=563 psf,					FV-4: 20.5/1.1 ft-lbs	

Remarks:

1. RWG&A Project No. 0600-102.
2. In situ vane shear tests conducted using MaineDOT provided Geonor 65x130mm rectangular vane.
3. Hammer Serial No. 283.
4. SPT N-Values are uncorrected.

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

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Maine Department of Transportation

Soil/Rock Exploration Log
US UNITS

Project: Scarborough Eastern Trail Project
Location: Scarborough, Maine

Boring No.: BB-SPC-203
PIN: 1942(610)

Driller:	R.W. Gillespie & Associates, Inc.	**Elevation (ft.)**	N/A	**Auger ID/OD:**	4.25" ID / 7.5" OD
Operator:	M. Nadeau	**Datum:**		**Sampler:**	SPT Split-Barrel
Logged By:	D. Walker	**Rig Type:**	Diedrich D50	**Hammer Wt./Fall:**	140 lbs / 30 in.
Date Start/Finish:	09/27/2016 / 09/30/2016	**Drilling Method:**	HSA, 3" roller bit, drive + wash	**Core Barrel:**	NQ2
Boring Location:	See Exploration Location Plan	**Casing ID/OD:**	3- and 4-inch ID	**Water Level*:**	8'

Definitions:
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V = Insitu Vane Shear Test
SSA = Solid Stem Auger

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

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample Information: 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.
30				residual=30 psf						
35	FV-5		35.0 - 35.8	Su=409 psf, residual=25 psf					65x130 mm raw torque readings: FV-5: 14.9/0.9 ft-lbs FV-6: 15.3/0.5 ft-lbs	
	FV-6		36.0 - 36.8	Su=420 psf, residual=14 psf						
40	U-2	24/24	40.0 - 42.0							14328B & 14416 WC=40.9% G & C
	FV-7		43.0 - 43.8	Su=574 psf, residual=41 psf					65x130 mm raw torque readings: FV-7: 20.9/1.5 ft-lbs FV-8: 18.2/1.1 ft-lbs	
	FV-8		44.0 - 44.8	Su=500 psf, residual=30 psf						
45										
50	S-8	24/24	50.0 - 52.0		WOR/ 24					
	FV-9		51.0 - 51.8	Su=623 psf, residual=69 psf					65x130 mm raw torque readings: FV-9: 22.7/2.5 ft-lbs FV-10: 19.5/0.5 ft-lbs	
	FV-10		52.0 - 52.8	Su=536 psf, residual=19 psf						
55	S-9	24/16	55.0 - 57.0		WOR/ 24				Occasional silt lenses.	
60										

Remarks:

1. RWG&A Project No. 0600-102.
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3. Hammer Serial No. 283.
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Maine Department of Transportation

Soil/Rock Exploration Log
US UNITS

Project: Scarborough Eastern Trail Project
Location: Scarborough, Maine

Boring No.: BB-SPC-203
PIN: 1942(610)

Driller:	R.W. Gillespie & Associates, Inc.	**Elevation (ft.)**	N/A	**Auger ID/OD:**	4.25" ID / 7.5" OD
Operator:	M. Nadeau	**Datum:**		**Sampler:**	SPT Split-Barrel
Logged By:	D. Walker	**Rig Type:**	Diedrich D50	**Hammer Wt./Fall:**	140 lbs / 30 in.
Date Start/Finish:	09/27/2016 / 09/30/2016	**Drilling Method:**	HSA, 3" roller bit, drive + wash	**Core Barrel:**	NQ2
Boring Location:	See Exploration Location Plan	**Casing ID/OD:**	3- and 4-inch ID	**Water Level*:**	8'

Definitions:
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SSA = Solid Stem Auger

Definitions:
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T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
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Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample Information: 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.
60	U-3	24/18	60.0 - 62.0							
	FV-11		63.0 - 63.8	Su>1217 psf						
65	S-10	24/24	65.0 - 67.0		10				65.0' SILTY SAND (SM); Gray, wet, loose to medium dense, fine sand, some silt.	
70	S-11	24/16	70.0 - 72.0		24				Becomes brown, medium dense.	
75	S-12	24/12	75.0 - 77.0		28				Becomes gray to orange brown.	
80	S-13	24/13	80.0 - 82.0		33	30			Becomes brown to tan, dense.	
						30				
						30				
						30				
						30				
85	S-14	24/14	85.0 - 87.0		26	60			Becomes medium dense.	
						65				
						65				
						65				
90						65				

Remarks:

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3. Hammer Serial No. 283.
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Maine Department of Transportation

Soil/Rock Exploration Log
US UNITS

Project: Scarborough Eastern Trail Project
Location: Scarborough, Maine

Boring No.: BB-SPC-203
PIN: 1942(610)

Driller:	R.W. Gillespie & Associates, Inc.	**Elevation (ft.)**	N/A	**Auger ID/OD:**	4.25" ID / 7.5" OD
Operator:	M. Nadeau	**Datum:**		**Sampler:**	SPT Split-Barrel
Logged By:	D. Walker	**Rig Type:**	Diedrich D50	**Hammer Wt./Fall:**	140 lbs / 30 in.
Date Start/Finish:	09/27/2016 / 09/30/2016	**Drilling Method:**	HSA, 3" roller bit, drive + wash	**Core Barrel:**	NQ2
Boring Location:	See Exploration Location Plan	**Casing ID/OD:**	3- and 4-inch ID	**Water Level*:**	8'

Definitions:
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SSA = Solid Stem Auger

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

Definitions:
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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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
90	S-15	24/10	90.0 - 92.0		20	65				
						65				
						65				
						65			93.0	
						65			SAND (SP); Gray, wet, medium dense to dense, fine to coarse sand, trace silt.	
95	S-16	24/11	95.0 - 97.0		24	120				
						120			Becomes gray to brown, and grades to include some fine to coarse gravel.	
						120				
						120				
						125				
100	S-17	24/4	100.0 - 102.0		47	190				
						190				
						190				
						190			103.0	
						190			SILTY GRAVEL (GM); Olive gray, wet, very dense, fine to coarse gravel, some silt, little fine to coarse sand.	
105	S-18	5/5	105.0 - 105.4		50/5"	60				
						35				
						27				
						27				
						27				
110	S-19	16/12	110.0 - 111.3		85/10"	90			110.0	
						90			SILTY SAND WITH GRAVEL(SM); Olive gray, wet, very dense, fine sand, some medium to coarse gravel, little silt and coarse sand.	
						125				
						125				
						125				
115						125				
						125				
						125				
						125				
						125				
120						125				

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US UNITS

Project: Scarborough Eastern Trail Project
Location: Scarborough, Maine

Boring No.: BB-SPC-203
PIN: 1942(610)

Driller:	R.W. Gillespie & Associates, Inc.	**Elevation (ft.)**	N/A	**Auger ID/OD:**	4.25" ID / 7.5" OD
Operator:	M. Nadeau	**Datum:**		**Sampler:**	SPT Split-Barrel
Logged By:	D. Walker	**Rig Type:**	Diedrich D50	**Hammer Wt./Fall:**	140 lbs / 30 in.
Date Start/Finish:	09/27/2016 / 09/30/2016	**Drilling Method:**	HSA, 3" roller bit, drive + wash	**Core Barrel:**	NQ2
Boring Location:	See Exploration Location Plan	**Casing ID/OD:**	3- and 4-inch ID	**Water Level*:**	8'

Definitions:
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$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
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Definitions:
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C = Consolidation Test

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.
120	S-20	17/20	120.0 - 121.4		86/11"	120			Grades to little fine gravel, trace coarse gravel and clay.	
						120				
						120				
						120			Drilling resistance increase.	
						120				
125	S-21	24/8	125.0 - 127.0		59	175				
						175				
						225				
						225				
						225				
130										
135	S-22	24/24	135.0 - 137.0		69					
140	S-23	16/0	140.0 - 141.3		84/10"				Wash materials and large cobble in spoon.	
	R-1	18/14	141.2 - 142.7	RQD<25%					141.2'	
	R-2	42/15	142.0 - 145.5	RQD<25%					Roller bit refusal at 141.2', top of bedrock. Gray to black, aphanitic to medium grained biotite PHYLLITE. Rock is very soft to hard, slightly to highly weathered with low to vertical discontinuities and jointing. Cleavage planes have very close to close spacing. Highly fractured zones throughout, along with quartz feldspathic veins with pyritic mineralage. Some gneissic banding the rock type is similar to the Scarboro formation.	
145										
	R-3	60/24	146.0 - 151.0	RQD<25%					Grades to little fine gravel, trace coarse gravel and clay. R-1 Recovery=78% R-2 Recovery=36% R-3 Recovery=40%	
150										

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US UNITS

Project: Scarborough Eastern Trail Project
Location: Scarborough, Maine

Boring No.: BB-SPC-203
PIN: 1942(610)

Driller:	R.W. Gillespie & Associates, Inc.	**Elevation (ft.)**	N/A	**Auger ID/OD:**	4.25" ID / 7.5" OD
Operator:	M. Nadeau	**Datum:**		**Sampler:**	SPT Split-Barrel
Logged By:	D. Walker	**Rig Type:**	Diedrich D50	**Hammer Wt./Fall:**	140 lbs / 30 in.
Date Start/Finish:	09/27/2016 / 09/30/2016	**Drilling Method:**	HSA, 3" roller bit, drive + wash	**Core Barrel:**	NQ2
Boring Location:	See Exploration Location Plan	**Casing ID/OD:**	3- and 4-inch ID	**Water Level*:**	8'

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
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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

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample Information: 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.
150									151.2	
									Bottom of Exploration at 151.2 feet below ground surface.	
155										
160										
165										
170										
175										
180										

Remarks:

1. RWG&A Project No. 0600-102.
2. In situ vane shear tests conducted using MaineDOT provided Geonor 65x130mm rectangular vane.
3. Hammer Serial No. 283.
4. SPT N-Values are uncorrected.

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Maine Department of Transportation

Soil/Rock Exploration Log
US UNITS

Project: Scarborough Eastern Trail Project
Location: Scarborough, Maine

Boring No.: BB-SPC-201A
PIN: 1942(610)

Driller:	Northern Test Boring	**Elevation (ft.)**		**Auger ID/OD:**	
Operator:	Mike Nadeau	**Datum:**		**Sampler:**	SPT Split-Barrel
Logged By:	Serena Pape	**Rig Type:**	Diedrich D-50 Turbo	**Hammer Wt./Fall:**	140 lb/30in.
Date Start/Finish:	06-15-2018 / 06-18-2018	**Drilling Method:**	3" Tri cone Bit, Rotary Wash	**Core Barrel:**	
Boring Location:	East of Tracks	**Casing ID/OD:**	4" Steel Casing	**Water Level*:**	6'

Definitions:
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V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_U = Insitu Field Vane Shear Strength (psf)
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$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
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G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample Information: 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									Casing hydraulically pushed to 60 feet below ground surface.	
5										
10										
15										
20										
25										
30										

Remarks:

1. RWG&A Project No. 0600-102
2. Drive and wash to 55 feet without sampling. Refer to log BB-SPC-201 for subsurface materials from 0 to 55 feet below ground surface.
3. Hammer Serial No. 377.
4. SPT N-Values uncorrected.

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Maine Department of Transportation

Soil/Rock Exploration Log
US UNITS

Project: Scarborough Eastern Trail Project
Location: Scarborough, Maine

Boring No.: BB-SPC-201A
PIN: 1942(610)

Driller:	Northern Test Boring	**Elevation (ft.)**		**Auger ID/OD:**	
Operator:	Mike Nadeau	**Datum:**		**Sampler:**	SPT Split-Barrel
Logged By:	Serena Pape	**Rig Type:**	Diedrich D-50 Turbo	**Hammer Wt./Fall:**	140 lb/30in.
Date Start/Finish:	06-15-2018 / 06-18-2018	**Drilling Method:**	3" Tri cone Bit, Rotary Wash	**Core Barrel:**	
Boring Location:	East of Tracks	**Casing ID/OD:**	4" Steel Casing	**Water Level*:**	6'

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Depth (ft.)	Sample Information: 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.
55	S-1	24/24	55.0 - 57.0	3 / 4 / 4 / 3	8		-55.0		SILTY SAND (SM); Loose, wet, fine sand, some silt, tan. 55.0	WC=24.3%

Remarks:

1. RWG&A Project No. 0600-102
2. Drive and wash to 55 feet without sampling. Refer to log BB-SPC-201 for subsurface materials from 0 to 55 feet below ground surface.
3. Hammer Serial No. 377.
4. SPT N-Values uncorrected.

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Maine Department of Transportation

Soil/Rock Exploration Log
US UNITS

Project: Scarborough Eastern Trail Project
Location: Scarborough, Maine

Boring No.: BB-SPC-201A
PIN: 1942(610)

Driller:	Northern Test Boring	**Elevation (ft.)**		**Auger ID/OD:**	
Operator:	Mike Nadeau	**Datum:**		**Sampler:**	SPT Split-Barrel
Logged By:	Serena Pape	**Rig Type:**	Diedrich D-50 Turbo	**Hammer Wt./Fall:**	140 lb/30in.
Date Start/Finish:	06-15-2018 / 06-18-2018	**Drilling Method:**	3" Tri cone Bit, Rotary Wash	**Core Barrel:**	
Boring Location:	East of Tracks	**Casing ID/OD:**	4" Steel Casing	**Water Level*:**	6'

Definitions:
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S_u = Insitu Field Vane Shear Strength (psf)
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$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
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WOR = weight of rods

Definitions:
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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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
60	S-2	24/14	60.0 - 62.0	5 / 5 / 7 / 8	12				Becomes medium dense, medium to fine sand, little silt, slightly darker.	
						81				
						70				
						65				
						69				
65	S-3	24/8	65.0 - 67.0	4 / 5 / 7 / 9	12	70	-65.0		65.0 SAND WITH SILT (SP-SM);Medium dense, wet, coarse to fine sand, few silt, gray-brown.	15112F WC=21.6% G
						70				
						60				
						72				
						80				
70	S-4	24/24	70.0 - 72.0	3 / 2 / 3 / 5	5	88			SAND WITH SILT AND CLAY (SP-SM); Loose/soft, wet, coarse to fine sand, varying amounts of clay/silt, gray.	WC=13.3%
						58				
						60				
						71				
						89				
75	S-5	24/9	75.0 - 77.0	16 / 16 / 35 / 23	51	97				WC=7.8%
						61	-76.0		76.0 SANDY GRAVEL WITH SILT (GW-GM); Dense to very dense, wet, gravel, some sand, some silt, occasional cobbles. Chatter in auger.	
						67				
						73				
						70				
80	S-6	24/18	80.0 - 82.0	26 / 26 / 34 / 27	60	91			Probable till; angular aggregate.	WC=7.7%
						75				
						89				
						93			Chatter in auger at 83'.	
						97				
85	S-7	24/15	85.0 - 87.0	17 / 18 / 25 / 38	43	93				WC=7.1%
						86				
						75				
						96				
90						118				

Remarks:

1. RWG&A Project No. 0600-102
2. Drive and wash to 55 feet without sampling. Refer to log BB-SPC-201 for subsurface materials from 0 to 55 feet below ground surface.
3. Hammer Serial No. 377.
4. SPT N-Values uncorrected.

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

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Maine Department of Transportation

Soil/Rock Exploration Log
US UNITS

Project: Scarborough Eastern Trail Project
Location: Scarborough, Maine

Boring No.: BB-SPC-201A
PIN: 1942(610)

Driller:	Northern Test Boring	**Elevation (ft.)**		**Auger ID/OD:**	
Operator:	Mike Nadeau	**Datum:**		**Sampler:**	SPT Split-Barrel
Logged By:	Serena Pape	**Rig Type:**	Diedrich D-50 Turbo	**Hammer Wt./Fall:**	140 lb/30in.
Date Start/Finish:	06-15-2018 / 06-18-2018	**Drilling Method:**	3" Tri cone Bit, Rotary Wash	**Core Barrel:**	
Boring Location:	East of Tracks	**Casing ID/OD:**	4" Steel Casing	**Water Level*:**	6'

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

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample Information: 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.
90	S-8	23/16	90.0 - 91.9	23 / 19 / 21 / 50/5"	40	109			Switch to 3" casing at 90'. Chatter in auger at 91'.	WC=11.8%
95	S-9	24/15	95.0 - 97.0	40 / 50 / 50/3"	100+					WC=8.1%
100	S-10	24/17	100.0 - 102.0	39 / 45 / 50/5"	95+					WC=7.5%
105	S-11	24/21	105.0 - 107.0	32 / 37 / 45 / 50/5"	82					WC=8.4%
110	S-12	24/8	110.0 - 112.0	21 / 50/5"	100+					WC=9.8%
115	S-13	24/19	115.0 - 117.0	33 / 46 / 50/5"	86+					WC=9.0%
120										

Remarks:

1. RWG&A Project No. 0600-102
2. Drive and wash to 55 feet without sampling. Refer to log BB-SPC-201 for subsurface materials from 0 to 55 feet below ground surface.
3. Hammer Serial No. 377.
4. SPT N-Values uncorrected.

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Maine Department of Transportation

Soil/Rock Exploration Log
US UNITS

Project: Scarborough Eastern Trail Project
Location: Scarborough, Maine

Boring No.: BB-SPC-201A
PIN: 1942(610)

Driller:	Northern Test Boring	**Elevation (ft.)**		**Auger ID/OD:**	
Operator:	Mike Nadeau	**Datum:**		**Sampler:**	SPT Split-Barrel
Logged By:	Serena Pape	**Rig Type:**	Diedrich D-50 Turbo	**Hammer Wt./Fall:**	140 lb/30in.
Date Start/Finish:	06-15-2018 / 06-18-2018	**Drilling Method:**	3" Tri cone Bit, Rotary Wash	**Core Barrel:**	
Boring Location:	East of Tracks	**Casing ID/OD:**	4" Steel Casing	**Water Level*:**	6'

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

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample Information: 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.
120	S-14	24/4	120.0 - 122.0	16 / 25 / 50/4"	75+				Bedrock encountered about 123.8' below groud surface. Begin NQ2 rock core.	
	R-1	24/23	123.8 - 125.8	RQD = 0%			-123.8		Run 1 & Run 2 Dark-gray, highly micaceous, aphanitic to fine-grained PELITE. Rock is hard and fresh to slightly weathered. Fractures are highly to extremely spaced and tight to open, with vertical to moderate dipping angles. Surfaces are smooth and slightly undulating. Occasional quartzite inclusions and calcite intrusions. Characteristics similar to interbedded pelite and sandstone of the Cape Elizabeth Formation. 1' rock core lost due to drilling method. Adjustments made for R-3.	
125	R-2	24/23	125.8 - 127.8	RQD = 0%					Depth (ft.) Time(min.) 124.8 2 125.8 2.5 126.8 5* Plugged 127.8 3	
	R-3	60/48	128.8 - 133.8	RQD = 22%					Run 3 Dark-gray, highly micaceous, aphanitic to fine-grained PELITE. Rock is hard and fresh to slightly weathered. Hightly fractured with moderate to extreme fracture spacing. High to shallow dipping angles. Stepped and undulating fractures, tight to open, with smooth and polished faces. Occasional quartzite inclusions and calcite intrusions. Characteristics similar to those of interbedded pelite and sandstone of the Cape Elizabeth formation.	
130							-133.8		Depth (ft.) Time(min.) 129.8 3 130.8 1.25 131.8 2.25 132.8 2 133.8 1.75	
135									123.8 133.8	
140									**Bottom of Exploration at 133.8 feet below ground surface.**	
145										
150										

Remarks:

1. RWG&A Project No. 0600-102
2. Drive and wash to 55 feet without sampling. Refer to log BB-SPC-201 for subsurface materials from 0 to 55 feet below ground surface.
3. Hammer Serial No. 377.
4. SPT N-Values uncorrected.

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Maine Department of Transportation

Soil/Rock Exploration Log
US UNITS

Project: Scarborough Eastern Trail Project
Location: Scarborough, Maine

Boring No.: BB-SPC-203A
PIN: 1942(610)

Driller:	Northern Test Boring	**Elevation (ft.)**		**Auger ID/OD:**	
Operator:	Mike Nadeau	**Datum:**		**Sampler:**	Thin-Walled Tube
Logged By:	Serena Pape	**Rig Type:**	Diedrich D-50 Turbo	**Hammer Wt./Fall:**	
Date Start/Finish:	06-19-2018 / 06-19-2018	**Drilling Method:**	3" Tri cone Bit, Rotary Wash	**Core Barrel:**	
Boring Location:	East of Tracks	**Casing ID/OD:**	4" Steel Casing	**Water Level*:**	7'

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

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 (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							-0.2		TOPSOIL AND ORGANIC MATERIAL (2 inches).	
									0.2 SILTY SAND (SM); Loose, dry, medium to fine sand, some silt, brown.	
5									Becomes wet.	
10										
15	FV		15.0 - 15.4	Su=1210 psf Residual Su=110 psf			-15.0		15.0 SILTY CLAY (CL); Very stiff, wet, clay gray.	
	FV		16.0 - 16.4	Su=1210 psf Residual Su=160 psf					65x130 mm raw torque readings: FV-1: 37.0/4.0 ft-lbs FV-2: 42.0/6.0 ft-lbs	
20	U-1	24/21	20.0 - 22.0							15111A WC=45.6% A & C LL=40.9 PL=21.7 PI=12.2
25	FV		25.0 - 25.4	Su=630 psf Residual Su=60 psf					65x130 mm raw torque readings: FV-3: 23.0/2.0 ft-lbs FV-4:19.0/2.0 ft-lbs	
	FV		26.0 - 26.4	Su=520 psf Residual Su=50 psf						
30										

Remarks:

1. RWG&A Project No. 0600-102
2. Hammer Serial No. 377.
3. SPT N-Values uncorrected.
4. Casing hydraulically pushed.
5. MaineDOT field vanes

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Boring No.: BB-SPC-203A

Maine Department of Transportation

Soil/Rock Exploration Log
US UNITS

Project: Scarborough Eastern Trail Project
Location: Scarborough, Maine

Boring No.: BB-SPC-203A
PIN: 1942(610)

Driller:	Northern Test Boring	**Elevation (ft.)**		**Auger ID/OD:**	
Operator:	Mike Nadeau	**Datum:**		**Sampler:**	Thin-Walled Tube
Logged By:	Serena Pape	**Rig Type:**	Diedrich D-50 Turbo	**Hammer Wt./Fall:**	
Date Start/Finish:	06-19-2018 / 06-19-2018	**Drilling Method:**	3" Tri cone Bit, Rotary Wash	**Core Barrel:**	
Boring Location:	East of Tracks	**Casing ID/OD:**	4" Steel Casing	**Water Level*:**	7'

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

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample Information: 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.
30	U-2	24/22	30.0 - 32.0							15111B WC=39.0% C
35	FV		35.0 - 35.4	Su=470 psf Residual Su=30 psf					65x130 mm raw torque readings: FV-5: 17.0/1.0 ft-lbs FV-6: 18.0/0.0 ft-lbs	
	FV		36.0 - 36.4	Su=490 psf Residual Su=0 psf						
40	U-3		40.0 - 42.0						Significant water loss, possible sand seam allowing water to fill BB-SPC-203.	15111C WC=42.3% A & C LL=40.0 PL=22.5 PI=17.5
45	FV		45.0 - 45.4	Su=550 psf Residual Su=30 psf					65x130 mm raw torque readings: FV-7: 20.0/1.0 ft-lbs FV-8: 23.0/0.0 ft-lbs	
	FV		46.0 - 46.4	Su=630 psf Residual Su=0 psf						
50	U-4	24/20	50.0 - 52.0							
55	FV		55.0 - 55.4	Su=770 psf Residual Su=0 psf					65x130 mm raw torque readings: FV-9: 28.0/0.0 ft-lbs FV-10: 27.0/0.0 ft-lbs	
	FV		56.0 - 56.4	Su=470 psf Residual Su=0 psf						
60										

Remarks:

1. RWG&A Project No. 0600-102
2. Hammer Serial No. 377.
3. SPT N-Values uncorrected.
4. Casing hydraulically pushed.
5. MaineDOT field vanes

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Maine Department of Transportation

Soil/Rock Exploration Log
US UNITS

Project: Scarborough Eastern Trail Project
Location: Scarborough, Maine

Boring No.: BB-SPC-203A
PIN: 1942(610)

Driller:	Northern Test Boring	**Elevation (ft.)**		**Auger ID/OD:**	
Operator:	Mike Nadeau	**Datum:**		**Sampler:**	Thin-Walled Tube
Logged By:	Serena Pape	**Rig Type:**	Diedrich D-50 Turbo	**Hammer Wt./Fall:**	
Date Start/Finish:	06-19-2018 / 06-19-2018	**Drilling Method:**	3" Tri cone Bit, Rotary Wash	**Core Barrel:**	
Boring Location:	East of Tracks	**Casing ID/OD:**	4" Steel Casing	**Water Level*:**	7'

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

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample Information: 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.
60	U-5	24/21	60.0 - 62.0							15111D WC=37.4% A & C LL=29.9 PL=19.2 PI=10.7
							-62.0		62.0 **Bottom of Exploration at 62.0 feet below ground surface.** Not refusal.	
65										
70										
75										
80										
85										
90										

Remarks:

1. RWG&A Project No. 0600-102
2. Hammer Serial No. 377.
3. SPT N-Values uncorrected.
4. Casing hydraulically pushed.
5. MaineDOT field vanes

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Maine Department of Transportation

Soil/Rock Exploration Log
US UNITS

Project: Scarborough Eastern Trail Project
Location: Scarborough, Maine

Boring No.: EB-SPC-202
PIN: 1942(610)

Driller:	R.W. Gillespie & Associates, Inc.	**Elevation (ft.)**	N/A	**Auger ID/OD:**	4.25" ID / 7.5" OD
Operator:	M. Nadeau	**Datum:**		**Sampler:**	SPT Split-Barrel
Logged By:	D. Walker	**Rig Type:**	Diedrich D50	**Hammer Wt./Fall:**	140 lbs. 30"
Date Start/Finish:	10/07/2016 / 10/07/2016	**Drilling Method:**	HSA	**Core Barrel:**	N/A
Boring Location:	See Exploration Location Plan	**Casing ID/OD:**	N/A	**Water Level*:**	5.5'

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

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 (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	S-1	24/13	0.0 - 2.0		10				SAND (SP); Tan to orange, dry, medium dense, fine to medium sand, trace silt.	
5	S-2	24/14	5.0 - 7.0		10				Becomes wet.	
10	S-3	24/24	10.0 - 12.0		6				11.0	
	S-4	24/24	12.0 - 14.0		5				SILTY CLAY (CL); Gray, wet, medium, clay, some silt. Reddish band above clay.	
									11.5	
									SILTY SAND (SM); Gray, wet, loose, fine sand some silt.	
									13.0	
15									SILTY CLAY (CL); Gray, wet, medium, clay, some silt. Silty sand lense.	
	FV-1		16.0 - 16.8	Su=607 psf Residual Su=60 psf					65x130 mm raw torque readings: FV-1: 22.1/2.2 ft-lbs	
	FV-2		17.0 - 17.8	Su=505 psf Residual Su=33 psf					FV-2: 18.4/1.2 ft-lbs	
20	FV-3		21.0 - 21.8	Su=692 psf Residual Su=102 psf					65x130 mm raw torque readings: FV-3: 25.2/3.7 ft-lbs	
	FV-4		22.0 - 22.8	Su=549 psf Residual Su=69 psf					FV-4: 20.0/2.5 ft-lbs	
25	FV-5		26.0 - 26.8	Su=390 psf Residual Su=27 psf					65x130 mm raw torque readings: FV-5: 14.2/1.0 ft-lbs	
	FV-6		27.0 - 27.8	RQD = % Su=412 psf Residual Su=33 psf					FV-6: 15.0/1.2 ft-lbs	
30										

Remarks:

1. RWG&A Project No. 0600-102.
2. In-situ vane shear tests conducted using MaineDOT provided Geonor 65x130mm rectangular vane.
3. Hammer Serial No. 283.
4. SPT N-Values uncorrected.

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Maine Department of Transportation

Soil/Rock Exploration Log
US UNITS

Project: Scarborough Eastern Trail Project
Location: Scarborough, Maine

Boring No.: EB-SPC-202
PIN: 1942(610)

Driller:	R.W. Gillespie & Associates, Inc.	**Elevation (ft.)**	N/A	**Auger ID/OD:**	4.25" ID / 7.5" OD
Operator:	M. Nadeau	**Datum:**		**Sampler:**	SPT Split-Barrel
Logged By:	D. Walker	**Rig Type:**	Diedrich D50	**Hammer Wt./Fall:**	140 lbs. 30"
Date Start/Finish:	10/07/2016 / 10/07/2016	**Drilling Method:**	HSA	**Core Barrel:**	N/A
Boring Location:	See Exploration Location Plan	**Casing ID/OD:**	N/A	**Water Level*:**	5.5'

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)
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Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample Information: 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.
30										
	FV-7		31.0 - 31.8	Su=398 psf Residual Su=33 psf					65x130 mm raw torque readings: FV-7: 14.5/1.2 ft-lbs FV-8: 14.6/1.4 ft-lbs	
	FV-8		32.0 - 32.8	Su=401 psf Residual Su=38 psf						
35									Hydraulic push rod probe to 60.8'.	
40										
45										
50									Stiff seams encountered approximately 50'.	
55										
60										

Remarks:

1. RWG&A Project No. 0600-102.
2. In-situ vane shear tests conducted using MaineDOT provided Geonor 65x130mm rectangular vane.
3. Hammer Serial No. 283.
4. SPT N-Values uncorrected.

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Maine Department of Transportation

Soil/Rock Exploration Log
US UNITS

Project: Scarborough Eastern Trail Project
Location: Scarborough, Maine

Boring No.: EB-SPC-202
PIN: 1942(610)

Driller:	R.W. Gillespie & Associates, Inc.	**Elevation (ft.)**	N/A	**Auger ID/OD:**	4.25" ID / 7.5" OD
Operator:	M. Nadeau	**Datum:**		**Sampler:**	SPT Split-Barrel
Logged By:	D. Walker	**Rig Type:**	Diedrich D50	**Hammer Wt./Fall:**	140 lbs. 30"
Date Start/Finish:	10/07/2016 / 10/07/2016	**Drilling Method:**	HSA	**Core Barrel:**	N/A
Boring Location:	See Exploration Location Plan	**Casing ID/OD:**	N/A	**Water Level*:**	5.5'

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)
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$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
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Definitions:
WC = water content, percent
LL = Liquid Limit
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PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample Information: 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.
60									Increased resistance to probe penetration at 60.8'.	
									Bottom of Exploration at 60.8 feet below ground surface. Not refusal.	
65										
70										
75										
80										
85										
90										

Remarks:

1. RWG&A Project No. 0600-102.
2. In-situ vane shear tests conducted using MaineDOT provided Geonor 65x130mm rectangular vane.
3. Hammer Serial No. 283.
4. SPT N-Values uncorrected.

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 UNITS

Project: Scarborough Eastern Trail Project
Location: Scarborough, Maine

Boring No.: EB-SPC-203
PIN: 1942(610)

Driller:	R.W. Gillespie & Associates, Inc.	**Elevation (ft.)**	N/A	**Auger ID/OD:**	4.25" ID / 7.5 OD
Operator:	M. Nadeau	**Datum:**		**Sampler:**	SPT Split-Barrel
Logged By:	D. Walker	**Rig Type:**	Diedrich D50	**Hammer Wt./Fall:**	140 lbs. 30"
Date Start/Finish:	09/27/2016 / 09/27/2016	**Drilling Method:**	HSA	**Core Barrel:**	N/A
Boring Location:	See Exploration Location Plan	**Casing ID/OD:**	N/A	**Water Level*:**	5.5'

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

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 (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	S-1	24/15	0.0 - 2.0		3				TOPSOIL AND ORGANIC MATERIAL (2 inches).	
									0.2	
									SAND (SP); Reddish brown, dry to wet, very loose to medium dense, fine to medium sand, little coarse sand.	
	S-2	24/13	2.0 - 4.0		8				Becomes tan, loose, fine to coarse sand, little silt.	
									Becomes moist, medium dense.	
5	S-3	24/16	4.0 - 6.0		20					
									Becomes reddish brown, wet, fine to medium sand.	
	S-4	24/24	6.0 - 8.0		11					
									2" Clay layer at 7', then 2" medium to coarse sand layer. Grades to fine sand. Becomes very soft.	
	S-5	24/15	8.0 - 10.0		2					
									9.0	
10	S-6	24/15	10.0 - 12.0		5				SILTY CLAY (CL); Gray, wet, soft to medium, clay, some silt. Alternating clay and silt lenses.	
									11.5	
	S-7	24/24	12.0 - 14.0		2				SILTY SAND (SM); Gray, wet, very loose, fine sand, some silt. Alternating seams of fine sand and silt.	
									13.5	
	S-8	24/16	14.0 - 16.0		WOH				SILTY CLAY (CL); Gray, wet, very soft, silty clay. Alternating lenses of silty clay and silty sand.	
15									16.0	
									Bottom of Exploration at 16.0 feet below ground surface. Not refusal.	
20										
25										
30										

Remarks:

1. RWG&A Project No. 0600-102.
2. Hammer Serial No. 283.
3. SPT N-Values uncorrected.

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.: EB-SPC-203

Maine Department of Transportation

Soil/Rock Exploration Log
US UNITS

Project: Scarborough Eastern Trail Project
Location: Scarborough, Maine

Boring No.: EB-SPC-203A
PIN: 1942(610)

Driller:	Northern Test Boring	**Elevation (ft.)**		**Auger ID/OD:**	
Operator:	Mike Nadeau	**Datum:**		**Sampler:**	SPT Split-Barrel
Logged By:	Serena Pape	**Rig Type:**	Diedrich D-50 Turbo	**Hammer Wt./Fall:**	140lb/30in
Date Start/Finish:	06-19-2018 / 06-19-2018	**Drilling Method:**	3" Tri cone Bit, Rotary Wash	**Core Barrel:**	
Boring Location:	East of Tracks	**Casing ID/OD:**	4" Steel Casing	**Water Level*:**	6.5'

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

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample Information: 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	S-1	24/23	0.0 - 2.0	3 / 4 / 4 / 7	8		-0.2		TOPSOIL AND ORGANIC MATERIAL (3 inches).	
									0.2 SILTY SAND (SM): Loose, moist, medium to fine sand, little silt, tan. Becomes brown at 0.5'.	
5	S-2	24/17	5.0 - 7.0	5 / 6 / 3 / 4	9		-6.0		6.0 SANDY SILT (ML): Very loose, wet, silt, some fine sand, gray.	
									Becomes some clay at 9'.	
10	S-3	24/24	10.0 - 12.0	WOH						
15	FV-1 ~~S-4~~	24/24	15.0 - 15.4 ~~15.0 - 17.0~~	Su=1020 psf Residual Su=110 psf WOH			-15.0		15.0 SILTY CLAY WITH SAND (CL): Soft, wet, clay, little silt, few fine sand, gray. 65x130 mm raw torque reading: FV-1: 37.0/4.0 ft-lbs	
20	S-5 ~~FV-2~~ FV-3	24/24	20.0 - 22.0 ~~20.0 - 20.4~~ 21.0 - 21.4	WOH ~~Su=440 psf~~ Residual Su=30 psf ~~Su=550 psf~~ Residual Su=30 psf					Becomes trace fine sand. 65x130 mm raw torque readings: FV-2: 16.0/1.0 ft-lbs FV-3: 20.0/1.0 ft-lbs FV-3: 20.0/1.0 ft-lbs	
25	S-6 ~~FV-4~~	24/24	25.0 - 27.0 ~~25.0 - 25.4~~	WOH ~~Su=490 psf~~ Residual Su=50 psf					65x130 mm raw torque reading: FV-4: 18.0/2.0 ft-lbs	
30										

Remarks:

1. RWG&A Project No. 0600-102
2. SPT N-Values are uncorrected
3. Hammer Serial No. 377
4. MaineDOT field vanes
5. Field vanes conducted then samples with SPT Split-Barrel

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 UNITS

Project: Scarborough Eastern Trail Project
Location: Scarborough, Maine

Boring No.: EB-SPC-203A
PIN: 1942(610)

Driller:	Northern Test Boring	**Elevation (ft.)**		**Auger ID/OD:**	
Operator:	Mike Nadeau	**Datum:**		**Sampler:**	SPT Split-Barrel
Logged By:	Serena Pape	**Rig Type:**	Diedrich D-50 Turbo	**Hammer Wt./Fall:**	140lb/30in
Date Start/Finish:	06-19-2018 / 06-19-2018	**Drilling Method:**	3" Tri cone Bit, Rotary Wash	**Core Barrel:**	
Boring Location:	East of Tracks	**Casing ID/OD:**	4" Steel Casing	**Water Level*:**	6.5'

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

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample Information: 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.
30	FV-5	24/24	30.0 - 30.4	Su=440 psf					65x130 mm raw torque readings:	
	~~S-7~~		~~30.0 - 32.0~~	Residual Su=0 psf					FV-5: 16.0/0.0 ft-lbs	
	FV-6		31.0 - 31.4	WOH					FV-6: 12.0/0.0 ft-lbs	
				Su=330 psf						
				Residual Su=0 psf					Hydraulically push probe rod without sampling from 32 to 65 feet.	
35										
40										
45										
50										
55										
60										

Remarks:

1. RWG&A Project No. 0600-102
2. SPT N-Values are uncorrected
3. Hammer Serial No. 377
4. MaineDOT field vanes
5. Field vanes conducted then samples with SPT Split-Barrel

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Maine Department of Transportation

Soil/Rock Exploration Log
US UNITS

Project: Scarborough Eastern Trail Project
Location: Scarborough, Maine

Boring No.: EB-SPC-203A
PIN: 1942(610)

Driller:	Northern Test Boring	**Elevation (ft.)**		**Auger ID/OD:**	
Operator:	Mike Nadeau	**Datum:**		**Sampler:**	SPT Split-Barrel
Logged By:	Serena Pape	**Rig Type:**	Diedrich D-50 Turbo	**Hammer Wt./Fall:**	140lb/30in
Date Start/Finish:	06-19-2018 / 06-19-2018	**Drilling Method:**	3" Tri cone Bit, Rotary Wash	**Core Barrel:**	
Boring Location:	East of Tracks	**Casing ID/OD:**	4" Steel Casing	**Water Level*:**	6.5'

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

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample Information: 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.
60									Becomes more stiff at 62'.	
65							-65.0		65.0 **Bottom of Exploration at 65.0 feet below ground surface.** Hydraulic push probe refusal.	
70										
75										
80										
85										
90										

Remarks:

1. RWG&A Project No. 0600-102
2. SPT N-Values are uncorrected
3. Hammer Serial No. 377
4. MaineDOT field vanes
5. Field vanes conducted then samples with SPT Split-Barrel

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Maine Department of Transportation

Soil/Rock Exploration Log
US UNITS

Project: Scarborough Eastern Trail Project
Location: Scarborough, Maine

Boring No.: EB-SPC-204
PIN: 1942(610)

Driller:	Northern Test Boring	**Elevation (ft.)**		**Auger ID/OD:**	
Operator:	Mike Nadeau	**Datum:**		**Sampler:**	SPT Split-Barrel
Logged By:	Serena Pape	**Rig Type:**	Diedrich D-50 Turbo	**Hammer Wt./Fall:**	140lbs/30in
Date Start/Finish:	06-20-2018 / 06-20-2018	**Drilling Method:**	3" Tri cone Bit, Rotary Wash	**Core Barrel:**	
Boring Location:	East of Tracks	**Casing ID/OD:**	4" Steel Casing	**Water Level*:**	5'

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

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 (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	S-1	24/24	0.0 - 2.0	2 / 2 / 5 / 7	7		-0.2		TOPSOIL AND ORGANIC MATERIAL (3 inches).	
							-0.2		SILTY SAND (SM); Loose, moist, medium to fine sand, few silt, orange.	
	S-2	24/21	2.0 - 4.0	5 / 6 / 8 / 10	14		-2.0		SILTY CLAY (CL); Stiff, moist, clay, few silt, gray.	
							-3.0		SILTY SAND (SM); Loose, moist, medium to fine sand, few silt, tan.	
	S-3	24/14	4.0 - 6.0	5 / 4 / 2 / 4	6		-4.0		SANDY SILT (SM-ML); Loose, wet, silt, some fine sand, orange.	
5									Becomes gray, runny at 5'.	
	S-4	24/21	6.0 - 8.0	4 / 3 / 5 / 2	5		-6.7		SILTY CLAY (CL); Very soft, wet, clay, some silt, frequent silty sand seams, gray.	
	S-5	24/24	8.0 - 10.0	WOH	1					
10	S-6	24/24	10.0 - 12.0	WOH / 4 / 5	4					
	S-7	24/24	12.0 - 14.0	3 / 2 / 1 / 2	3		-11.5		SILTY SAND (SM); Very loose, wet, fine sand, some silt, few clay.	15112B WC=31.2% G
							-13.5		SILTY CLAY (CL); Very soft to medium, wet, clay, some silt, frequent silty sand seams, gray. Occasional silty sand seams.	
15	S-8	24/24	15.0 - 17.0	1 / 1 / WOR	1					
20	S-9	24/24	20.0 - 22.0	WOH					65x130 mm raw torque readings: FV-1: 20.0/0.0 ft-lbs FV-2: 18.0/0.0 ft-lbs	
	~~FV-1~~		~~20.0 - 20.4~~	~~Su=550 psf~~						
	FV-2		21.0 - 21.4	Residual Su=0 psf						
				~~Su=490 psf~~ Residual Su=0 psf						
25	S-10	24/24	25.0 - 27.0	WOH						15112A WC=4.5% A LL=33.5 PL=21.1 PI=12.4
30										

Remarks:

1. RWG&A Project No. 0600-102
2. SPT N-Values are uncorrected.
3. Hammer Serial No. 377
4. MaineDOT field vanes.
5. Field vane test conducted then sampled with SPT Split-Barrel.

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

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Maine Department of Transportation

Soil/Rock Exploration Log
US UNITS

Project: Scarborough Eastern Trail Project
Location: Scarborough, Maine

Boring No.: EB-SPC-204
PIN: 1942(610)

Driller:	Northern Test Boring	**Elevation (ft.)**		**Auger ID/OD:**	
Operator:	Mike Nadeau	**Datum:**		**Sampler:**	SPT Split-Barrel
Logged By:	Serena Pape	**Rig Type:**	Diedrich D-50 Turbo	**Hammer Wt./Fall:**	140lbs/30in
Date Start/Finish:	06-20-2018 / 06-20-2018	**Drilling Method:**	3" Tri cone Bit, Rotary Wash	**Core Barrel:**	
Boring Location:	East of Tracks	**Casing ID/OD:**	4" Steel Casing	**Water Level*:**	5'

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

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample Information: 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.
30	S-11	24/24	30.0 - 32.0	WOH					Hydraulically push probe from 32 to 60 feet.	
35										
40										
45										
50										
55									Becomes firmer at 55'.	
60										

Remarks:

1. RWG&A Project No. 0600-102
2. SPT N-Values are uncorrected.
Hammer Serial No. 377
4. MaineDOT field vanes.
5. Field vane test conducted then sampled with SPT Split-Barrel.

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 UNITS

Project: Scarborough Eastern Trail Project
Location: Scarborough, Maine

Boring No.: EB-SPC-204
PIN: 1942(610)

Driller:	Northern Test Boring	**Elevation (ft.)**		**Auger ID/OD:**	
Operator:	Mike Nadeau	**Datum:**		**Sampler:**	SPT Split-Barrel
Logged By:	Serena Pape	**Rig Type:**	Diedrich D-50 Turbo	**Hammer Wt./Fall:**	140lbs/30in
Date Start/Finish:	06-20-2018 / 06-20-2018	**Drilling Method:**	3" Tri cone Bit, Rotary Wash	**Core Barrel:**	
Boring Location:	East of Tracks	**Casing ID/OD:**	4" Steel Casing	**Water Level*:**	5'

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

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample Information: 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.
60							-60.0		Bottom of Exploration at 60.0 feet below ground surface. 60.0	
									Bottom of Exploration at 60.0 feet below ground surface. Hydraulic probe refusal.	
65										
70										
75										
80										
85										
90										

Remarks:

1. RWG&A Project No. 0600-102
2. SPT N-Values are uncorrected.
3. Hammer Serial No. 377
4. MaineDOT field vanes.
5. Field vane test conducted then sampled with SPT Split-Barrel.

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.: EB-SPC-204

Maine Department of Transportation

Soil/Rock Exploration Log
US UNITS

Project: Scarborough Eastern Trail Project
Location: Scarborough, Maine

Boring No.: EB-SPC-205
PIN: 1942(610)

Driller:	Northern Test Boring	**Elevation (ft.)**	N/A	**Auger ID/OD:**	
Operator:	Mike Nadeau	**Datum:**		**Sampler:**	SPT Split-Barrel
Logged By:	Serena Pape	**Rig Type:**	Diedrich D-50 Turbo	**Hammer Wt./Fall:**	140lbs/30in
Date Start/Finish:	06-20-2018 / 06-20-2018	**Drilling Method:**	3" Tri cone Bit, Rotary Wash	**Core Barrel:**	
Boring Location:	East of Tracks	**Casing ID/OD:**	4" Steel Casing	**Water Level*:**	2'

Definitions:
O = 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

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 (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	S-1	24/17	0.0 - 2.0	1 / 2 / 3 / 8	5				TOPSOIL AND ORGANIC MATERIAL (2 inches). 0.2	
	S-2	24/16	2.0 - 4.0	8 / 10 / 8 / 11	18				SAND WITH SILT (SP-SM); Loose to medium dense, moist, medium to fine sand, few silt, orangy brown. Becomes medium dense, tan at 2'.	
	S-3	24/19	4.0 - 6.0	5 / 5 / 5 / 5	10				Becomes wet, possibly due to roto- wash at 4'.	
5									Rust colored stripe at 5.2'	
	S-4	24/22	6.0 - 8.0	4 / 4 / 5 / 4	9				Becomes loose at 6'.	15112C WC=21.6% G
									Becomes gray at 7.2'.	
	S-5	24/24	8.0 - 10.0	1 / 1 / 1 / 1	2				8.2 SILTY CLAY (CL); Very soft to medium, wet, clay, some silt, gray.	
10	S-6	24/24	10.0 - 12.0	WOR						
15	FV-1 ~~S-7~~ FV-2	24/24	15.0 - 15.4 ~~15.0 - 17.0~~ 16.0 - 16.4	Su=360 psf Residual Su=0 psf WOR Su=490 psf Residual Su=0 psf					65x130 mm raw torque readings: FV-1: 13.0/0.0 ft-lbs FV-2: 15.0/0.0 ft-lbs	
20	FV-3 ~~S-8~~ FV-4	24/24	20.0 - 20.4 ~~20.0 - 22.0~~ 21.0 - 21.4	Su=380 psf Residual Su=50 psf WOR Su=630 psf Residual Su=30 psf					65x130 mm raw torque readings: FV-3: 14.0/2.0 ft-lbs FV-4: 23.0/1.0 ft-lbs	
25	S-9 ~~FV-5~~ FV-6	24/24	25.0 - 27.0 ~~25.0 - 25.4~~ 26.0 - 26.4	WOR ~~Su=490 psf~~ Residual Su=60 psf ~~Su=630 psf~~ Residual Su=80 psf					65x130 mm raw torque readings: FV-5: 18.0/2.0 ft-lbs FV-6: 23.0/3.0 ft-lbs	
30										

Remarks:

1. RWG&A Project No. 0600-102
2. SPT N-Values are uncorrected
3. Hammer Serial No. 377.
4. MaineDOT field vanes
5. Field vane test conducted then sampled with SPT Split-Barrel.

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 UNITS

Project: Scarborough Eastern Trail Project
Location: Scarborough, Maine

Boring No.: EB-SPC-205
PIN: 1942(610)

Driller:	Northern Test Boring	**Elevation (ft.)**	N/A	**Auger ID/OD:**	
Operator:	Mike Nadeau	**Datum:**		**Sampler:**	SPT Split-Barrel
Logged By:	Serena Pape	**Rig Type:**	Diedrich D-50 Turbo	**Hammer Wt./Fall:**	140lbs/30in
Date Start/Finish:	06-20-2018 / 06-20-2018	**Drilling Method:**	3" Tri cone Bit, Rotary Wash	**Core Barrel:**	
Boring Location:	East of Tracks	**Casing ID/OD:**	4" Steel Casing	**Water Level*:**	2'

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

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 (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
30	S-10	24/24	30.0 - 32.0	WOR						
35	S-11	24/24	34.0 - 36.0	WOR						
40	S-12	24/24	40.0 - 42.0	5 / 5 / 4 / 4	9				40.0 SANDY SILT (SM-ML); Loose, wet, silt, some fine sand, gray.	15112D WC=21.6% G
									41.5 SILTY CLAY (CL); Stiff, wet, clay, little silt, gray.	
45	S-13	24/24	45.0 - 47.0	2 / 3 / 5 / 4	8				45.0 SAND WITH SILT (SP-SM); Loose to medium dense, wet, fine sand, little silt, occasional clay seams, gray.	15112E WC=24.0% G
50	S-14	24/24	50.0 - 52.0	3 / 1 / 1 / 1	2				Becomes very loose at 50'.	
55	S-15	24/19	55.0 - 57.0	7 / 7 / 9 / 14	16				Becomes medium dense at 54'.	
60									57.0 **Bottom of Exploration at 57.0 feet below ground surface.**	

Remarks:

1. RWG&A Project No. 0600-102
2. SPT N-Values are uncorrected
3. Hammer Serial No. 377.
4. MaineDOT field vanes
5. Field vane test conducted then sampled with SPT Split-Barrel.

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.

NONESUCH RIVER EXPLORATION LOGS

Maine Department of Transportation

Soil/Rock Exploration Log
US UNITS

Project: Scarborough Eastern Trail Project
Location: Scarborough, Maine

Boring No.: EB-SNR-101
PIN: 1942(610)

Driller:	Great Works Test Boring	**Elevation (ft.)**		**Auger ID/OD:**	-
Operator:	Jeff Lee	**Datum:**		**Sampler:**	SPT Split-Barrel
Logged By:	D. Walker	**Rig Type:**	CME 75 Track Rig	**Hammer Wt./Fall:**	140lbs / 30 inches
Date Start/Finish:	07/12/2016	**Drilling Method:**	Drive + Wash	**Core Barrel:**	-
Boring Location:	See Expl Location Plan	**Casing ID/OD:**	4-inch	**Water Level*:**	7'

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

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 (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	S-1	24/12	0.0 - 2.0		4				FILL; Fine to medium sand, little fine gravel and silt.	
	S-2	24/8	2.0 - 4.0		4					
5	S-3	24/18	5.0 - 7.0		5		-5.0		5.0 SILT WITH SAND (SM); Very loose, moist, silt, little fine sand, gray.	
	S-4	24/14	7.0 - 9.0		4		-7.0		7.0 SILTY SAND (SM); Very loose, wet, fine to medium sand, some silt, tan-brown.	
10	S-5	24/14	9.0 - 11.0		3		-9.5		Becomes very loose. 9.5 SILTY CLAY (CL); Soft, moist, clay, some silt, gray.	
15	S-6	24/12	15.0 - 17.0		3		-15.5		15.5 SAND WITH GRAVEL (SP); Very loose, wet, fine to medium sand, little fine gravel, trace silt, gray.	
20	S-7	24/24	20.0 - 22.0		WOH		-20.0		20.0 SILTY CLAY (CL); Very soft to medium, wet, clay, some silt, dark gray.	
25	FV-1 ~~FV-2~~		25.0 - 25.8 ~~25.9 - 26.7~~	Su=660 psf Residual Su=50 psf Su=640 psf Residual Su=50 psf					65x130 mm raw torque readings: FV-1: 61.0/5.0 ft-lbs FV-2: 59.0/5.0 ft-lbs	
30										

Remarks:

1. RWG&A Project No. 0600-102
2. SPT N-Values are uncorrected.
3. Shear vane data collected using 3-5/8"x9" field vane and torque wrench.
4. Hydraulically pushed 4-inch diameter steel casing, no hammer used to drive casing.

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 UNITS

Project: Scarborough Eastern Trail Project
Location: Scarborough, Maine

Boring No.: EB-SNR-101
PIN: 1942(610)

Driller:	Great Works Test Boring	**Elevation (ft.)**		**Auger ID/OD:**	-
Operator:	Jeff Lee	**Datum:**		**Sampler:**	SPT Split-Barrel
Logged By:	D. Walker	**Rig Type:**	CME 75 Track Rig	**Hammer Wt./Fall:**	140lbs / 30 inches
Date Start/Finish:	07/12/2016	**Drilling Method:**	Drive + Wash	**Core Barrel:**	-
Boring Location:	See Expl Location Plan	**Casing ID/OD:**	4-inch	**Water Level*:**	7'

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

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample Information: 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.
30	FV-3		30.0 - 30.8	Su=780 psf					65x130 mm raw torque readings:	
	~~FV-4~~		~~30.9 - 31.7~~	Residual Su=110 psf Su=740 psf Residual Su=110 psf					FV-3: 72.0/10.0 ft-lbs FV-4: 68.0/10.0 ft-lbs	
35	FV-5		35.0 - 35.8	Su>1090 psf					Could not advance field vane. Possible sand seam.	
40	S-8	24/24	40.0 - 42.0		WOH				Occasional silt lenses. Pocket penetrometer: Undrained Shear Strength: Su=<0.25 ksf.	
45	S-9	24/24	45.0 - 47.0		1		-45.0		45.0 SANDY SILT (ML); Very loose, wet, silt with little fine sand, gray.	
							-46.0		46.0 CLAYEY SILT (ML); Very loose wet, silt, some clay, gray.	
50	S-10	24/24	50.0 - 52.0		1		-50.0		50.0 SANDY SILT (ML); Very loose, wet, silt, little fine sand.	
55	S-11	24/24	55.0 - 57.0		6		-55.0		55.0 SAND (SP); Loose, wet, fine sand, trace silt, gray.	
60										

Remarks:

1. RWG&A Project No. 0600-102
2. SPT N-Values are uncorrected.
3. Shear vane data collected using 3-5/8"x9" field vane and torque wrench.
4. Hydraulically pushed 4-inch diameter steel casing, no hammer used to drive casing.

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 UNITS

Project: Scarborough Eastern Trail Project
Location: Scarborough, Maine

Boring No.: EB-SNR-101
PIN: 1942(610)

Driller:	Great Works Test Boring	**Elevation (ft.)**		**Auger ID/OD:**	-
Operator:	Jeff Lee	**Datum:**		**Sampler:**	SPT Split-Barrel
Logged By:	D. Walker	**Rig Type:**	CME 75 Track Rig	**Hammer Wt./Fall:**	140lbs / 30 inches
Date Start/Finish:	07/12/2016	**Drilling Method:**	Drive + Wash	**Core Barrel:**	-
Boring Location:	See Expl Location Plan	**Casing ID/OD:**	4-inch	**Water Level*:**	7'

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

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample Information: 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.
60	S-12	24/24	60.0 - 62.0		9					
							-62.0		62.0	
									Bottom of Exploration at 62.0 feet below ground surface. Not refusal.	
65										
70										
75										
80										
85										
90										

Remarks:

1. RWG&A Project No. 0600-102
2. SPT N-Values are uncorrected.
3. Shear vane data collected using 3-5/8"x9" field vane and torque wrench.
4. Hydraulically pushed 4-inch diameter steel casing, no hammer used to drive casing.

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Boring No.: EB-SNR-101

Maine Department of Transportation

Soil/Rock Exploration Log
US UNITS

Project: Scarborough Eastern Trail Project
Location: Scarborough, Maine

Boring No.: EB-SNR-102
PIN: 1942(610)

Driller:	Great Works Test Boring	**Elevation (ft.)**		**Auger ID/OD:**	-
Operator:	Jeff Lee	**Datum:**		**Sampler:**	SPT Split-Barrel
Logged By:	D. Walker	**Rig Type:**	CME 75	**Hammer Wt./Fall:**	140lbs / 30 inches
Date Start/Finish:	07/13/2016	**Drilling Method:**	Drive + Wash	**Core Barrel:**	-
Boring Location:	See Expl. Location Plan	**Casing ID/OD:**	4-inch	**Water Level*:**	6'

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

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 (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	S-1	24/18	0.0 - 2.0		4		-0.3		TOPSOIL AND ORGANIC MATERIAL (2 inches), dark brown silt, fine sand.	
							0.3		FILL; Moist, fine sand, trace fine gravel, tan.	
	S-2	24/18	2.0 - 4.0		3					
							-3.5		SILT (ML); Very loose, moist, silt, tan.	
5	S-3	24/18	5.0 - 7.0		1				Becomes moist to wet, organic, brown.	
							-6.0		SILTY CLAY (CL); Very soft, wet, clay, some silt, gray.	
10	S-4	24/18	10.0 - 12.0		2		-10.0		CLAYEY SILT (ML); Very loose, wet, silt with some clay, interbedded silt lenses, gray.	
15	S-5	24/14	15.0 - 17.0		1		-15.0		SILTY SAND (SM); Very loose, wet, fine sand, some silt, occasional coarse sand and fine gravel lenses 1/4" to 1/2".	
							-17.0		SILTY CLAY (CL); Medium, wet, clay, some silt.	
20	FV-1		20.0 - 20.8	Su=770 psf Residual Su=90 psf					65x130 mm raw torque readings: FV-1: 71.0/8.0 ft-lbs FV-2: 55.0/6.0 ft-lbs	
	~~FV-2~~		~~20.9 - 21.7~~	~~Su=600 psf Residual Su=70 psf~~						
25	U-1		25.0 - 27.0							14285A MC=39.2% G & C
30										

Remarks:

1. RWG&A Project No. 0600-102
2. SPT N-Values are uncorrected.
3. Shear vane data collected using 3-5/8"x9" field vane and torque wrench.
4. Hydraulically pushed 4-inch diameter steel casing, no hammer used to drive casing.

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Maine Department of Transportation

Soil/Rock Exploration Log
US UNITS

Project: Scarborough Eastern Trail Project
Location: Scarborough, Maine

Boring No.: EB-SNR-102
PIN: 1942(610)

Driller:	Great Works Test Boring	**Elevation (ft.)**		**Auger ID/OD:**	-
Operator:	Jeff Lee	**Datum:**		**Sampler:**	SPT Split-Barrel
Logged By:	D. Walker	**Rig Type:**	CME 75	**Hammer Wt./Fall:**	140lbs / 30 inches
Date Start/Finish:	07/13/2016	**Drilling Method:**	Drive + Wash	**Core Barrel:**	-
Boring Location:	See Expl. Location Plan	**Casing ID/OD:**	4-inch	**Water Level*:**	6'

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

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample Information: 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.
30	FV-3		30.0 - 30.8	Su=760 psf					65x130 mm raw torque readings:	
	FV-4		30.9 - 31.7	Residual Su=100 psf Su=820 psf					FV-3: 70.0/9.0 ft-lbs	
				Residual Su=110 psf					FV-4: 75.0/10.0 fot-lbs	
35	U-2		35.0 - 37.0							14285B WC=29.9% G & C
40	U-3		40.0 - 42.0							
45	S-6	24/24	45.0 - 47.0		WOH				Becomes very soft.	
50	S-7	24/24	50.0 - 52.0		WOH				Some frequent silt and/or sand layers.	
55	S-8	24/24	55.0 - 57.0		4		-55.0		SAND (SP): Very loose to loose, wet, fine sand, trace silt, gray. -55.0	
60										

Remarks:

1. RWG&A Project No. 0600-102
2. SPT N-Values are uncorrected.
3. Shear vane data collected using 3-5/8"x9" field vane and torque wrench.
4. Hydraulically pushed 4-inch diameter steel casing, no hammer used to drive casing.

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 UNITS

Project: Scarborough Eastern Trail Project
Location: Scarborough, Maine

Boring No.: EB-SNR-102
PIN: 1942(610)

Driller:	Great Works Test Boring	**Elevation (ft.)**		**Auger ID/OD:**	-
Operator:	Jeff Lee	**Datum:**		**Sampler:**	SPT Split-Barrel
Logged By:	D. Walker	**Rig Type:**	CME 75	**Hammer Wt./Fall:**	140lbs / 30 inches
Date Start/Finish:	07/13/2016	**Drilling Method:**	Drive + Wash	**Core Barrel:**	-
Boring Location:	See Expl. Location Plan	**Casing ID/OD:**	4-inch	**Water Level*:**	6'

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

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample Information: 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.
60	S-9	24/24	60.0 - 62.0		10				Becomes medium dense.	
							-62.0		62.0	
									Bottom of Exploration at 62.0 feet below ground surface. Not refusal.	
65										
70										
75										
80										
85										
90										

Remarks:

1. RWG&A Project No. 0600-102
2. SPT N-Values are uncorrected.
3. Shear vane data collected using 3-5/8"x9" field vane and torque wrench.
4. Hydraulically pushed 4-inch diameter steel casing, no hammer used to drive casing.

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 UNITS

Project: Scarborough Eastern Trail Project
Location: Scarborough, Maine

Boring No.: EB-SNR-103
PIN: 1942(610)

Driller:	Great Works Test Boring	**Elevation (ft.)**		**Auger ID/OD:**	-
Operator:	Jeff Lee	**Datum:**		**Sampler:**	SPT Split-Barrel
Logged By:	D. Walker	**Rig Type:**	CME 75	**Hammer Wt./Fall:**	140lbs / 30 inches
Date Start/Finish:	07/12/2016	**Drilling Method:**	Drive + Wash	**Core Barrel:**	-
Boring Location:	See Expl Location Plan	**Casing ID/OD:**	4-inch	**Water Level*:**	3'

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

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 (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	S-1	24/14	0.0 - 2.0		WOH		-0.3		TOPSOIL AND ORGANIC MATERIAL (2 inches).	
							0.3		SANDY SILT (ML); Very loose, moist, silt, some fine sand, little clay, mottled near top.	
5	S-2	24/24	5.0 - 7.0		WOH		-5.0		SILTY CLAY (CL); Very soft , wet, clay with some silt, gray.	
10	S-3	24/12	10.0 - 12.0		3		-10.0		Attempted vane, too firm to advance, proceed with split-barrel sample. SILTY SAND (SM); Very loose, wet, coarse to fine sand, little silt, with fine gravel, organic odor, gray to dark gray.	
15	S-4	24/16	15.0 - 17.0		2		-15.0		SILTY CLAY (CL); Soft to medium, wet, clay, some silt.	
20	FV-1 ~~FV-2~~		20.0 - 20.8 ~~20.9 - 21.7~~	Su=720 psf Residual Su=70 psf ~~Su=580 psf Residual Su=40 psf~~					65x130 mm raw torque readings: FV-1: 66.0/6.0 ft-lbs FV-2: 53.0/4.0 ft-lbs	
25	S-5	24/24	25.0 - 27.0		WOH					
30										

Remarks:

1. RWG&A Project No. 0600-102
2. SPT N-Values are uncorrected.
3. Shear vane data collected using 3-5/8"x9" field vane and torque wrench.
4. Hydraulically pushed 4-inch diameter steel casing, no hammer used to drive casing.

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 UNITS

Project: Scarborough Eastern Trail Project
Location: Scarborough, Maine

Boring No.: EB-SNR-103
PIN: 1942(610)

Driller:	Great Works Test Boring	**Elevation (ft.)**		**Auger ID/OD:**	-
Operator:	Jeff Lee	**Datum:**		**Sampler:**	SPT Split-Barrel
Logged By:	D. Walker	**Rig Type:**	CME 75	**Hammer Wt./Fall:**	140lbs / 30 inches
Date Start/Finish:	07/12/2016	**Drilling Method:**	Drive + Wash	**Core Barrel:**	-
Boring Location:	See Expl Location Plan	**Casing ID/OD:**	4-inch	**Water Level*:**	3'

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

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample Information: 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.
30	FV-3		30.0 - 30.8	Su=840 psf					65x130 mm raw torque readings:	
	FV-4		30.9 - 31.7	Residual Su=110 psf Su=820 psf Residual Su=100 psf					FV-3: 77.0/10.0 ft-lbs FV-4: 75.0/10.0 ft-lbs	
35	S-6	24/24	35.0 - 37.0		WOH					
40	S-7	24/24	40.0 - 42.0		4				Too stiff for attempted vane test. Proceed with split-spoon sampling.	
							-41.0		41.0	
							-41.5		SILT (ML): Soft, wet, silt, few fine sand, trace clay, gray.	
									41.5	
									SILTY CLAY (CL): Very soft, silty clay, trace fine sand.	
45		24/6	45.0 - 47.0		WOH				Too stiff for attempted vane test. Proceed with split-spoon sample.	
	S-8						-46.0		46.0	
									Sand and silt layers.	
50	S-9	24/20	50.0 - 52.0		WOH		-50.0		50.0	
									SAND (SP): Very loose to medium dense, wet, fine sand, few silt, gray.	
55	S-10	24/24	55.0 - 57.0		10					
60										

Remarks:

1. RWG&A Project No. 0600-102
2. SPT N-Values are uncorrected.
3. Shear vane data collected using 3-5/8"x9" field vane and torque wrench.
4. Hydraulically pushed 4-inch diameter steel casing, no hammer used to drive casing.

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 UNITS

Project: Scarborough Eastern Trail Project
Location: Scarborough, Maine

Boring No.: EB-SNR-103
PIN: 1942(610)

Driller:	Great Works Test Boring	**Elevation (ft.)**		**Auger ID/OD:**	-
Operator:	Jeff Lee	**Datum:**		**Sampler:**	SPT Split-Barrel
Logged By:	D. Walker	**Rig Type:**	CME 75	**Hammer Wt./Fall:**	140lbs / 30 inches
Date Start/Finish:	07/12/2016	**Drilling Method:**	Drive + Wash	**Core Barrel:**	-
Boring Location:	See Expl Location Plan	**Casing ID/OD:**	4-inch	**Water Level*:**	3'

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

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample Information: 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.
60	S-11	18/18	60.0 - 61.5		12				Becomes medium dense, gray to brown.	
							-62.0		62.0	
									Bottom of Exploration at 62.0 feet below ground surface. Not refusal.	
65										
70										
75										
80										
85										
90										

Remarks:

1. RWG&A Project No. 0600-102
2. SPT N-Values are uncorrected.
3. Shear vane data collected using 3-5/8"x9" field vane and torque wrench.
4. Hydraulically pushed 4-inch diameter steel casing, no hammer used to drive casing.

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 UNITS

Project: Scarborough Eastern Trail Project
Location: Scarborough, Maine

Boring No.: FV-SNR-103
PIN: 1942(610)

Driller:	R.W. Gillespie & Associates, Inc.	**Elevation (ft.)**	N/A	**Auger ID/OD:**	-
Operator:	N/A	**Datum:**		**Sampler:**	-
Logged By:	D. Walker	**Rig Type:**	N/A	**Hammer Wt./Fall:**	-
Date Start/Finish:	07/19/2016	**Drilling Method:**	N/A	**Core Barrel:**	-
Boring Location:	See Notes	**Casing ID/OD:**	-	**Water Level*:**	Not Obs.

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

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 (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	FV-1		0.5 - 1.3	Su=800 psf					TOPSOIL AND ORGANIC MATERIAL (2 inches). 0.2	
	FV-2		1.3 - 2.1	Su=800 psf					SILTY CLAY (CL); Medium, wet, silty clay. Large Geonor vane raw readings: FV-1: 72 kPa FV-2: 72 kPa FV-3: 114/12 kPa FV-4: 88/10 kPa FV-5: 54/8 kPa FV-6: 94/12 kPa FV-7: 160/26 kPa FV-8: 68/22 kPa FV-9: 112/28 kPa FV-10: 100/22 kPa FV-11: 94/24 kPa	
	FV-3		2.2 - 3.0	Su=1200 psf Residual Su=100 psf						
	FV-4		3.0 - 3.8	Su=900 psf Residual Su=100 psf						
	~~FV-5~~		~~3.8 - 4.6~~							
	FV-6		4.6 - 5.4	Su=600 psf ~~Residual Su=100 psf~~						
5	FV-7		5.5 - 6.3	Su=1000 psf ~~Residual Su=100 psf~~						
	FV-8		6.3 - 7.1	Su=1700 psf ~~Residual Su=300 psf~~						
	FV-9		7.2 - 8.0	Su=700 psf Residual Su=200 psf						
	FV-10		8.0 - 8.8	Su=1200 psf						
	~~FV-11~~		~~8.8 - 9.6~~	Residual Su=300 psf Su=1000 psf						
10				Residual Su=200 psf Su=1000 psf						
				Residual Su=300 psf						
15										
20										
25										
30										

8.8
Bottom of Exploration at 8.8 feet below ground surface.
Not refusal.

Remarks:

1. RWG&A Project No. 0600-102
2. FV-SNR-103 is located approximately 5 feet west of EB-SNR-103.
3. Exploration conducted as probe with in-situ vane shear tests conducted with Geonor mini-vane shear tester.

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.

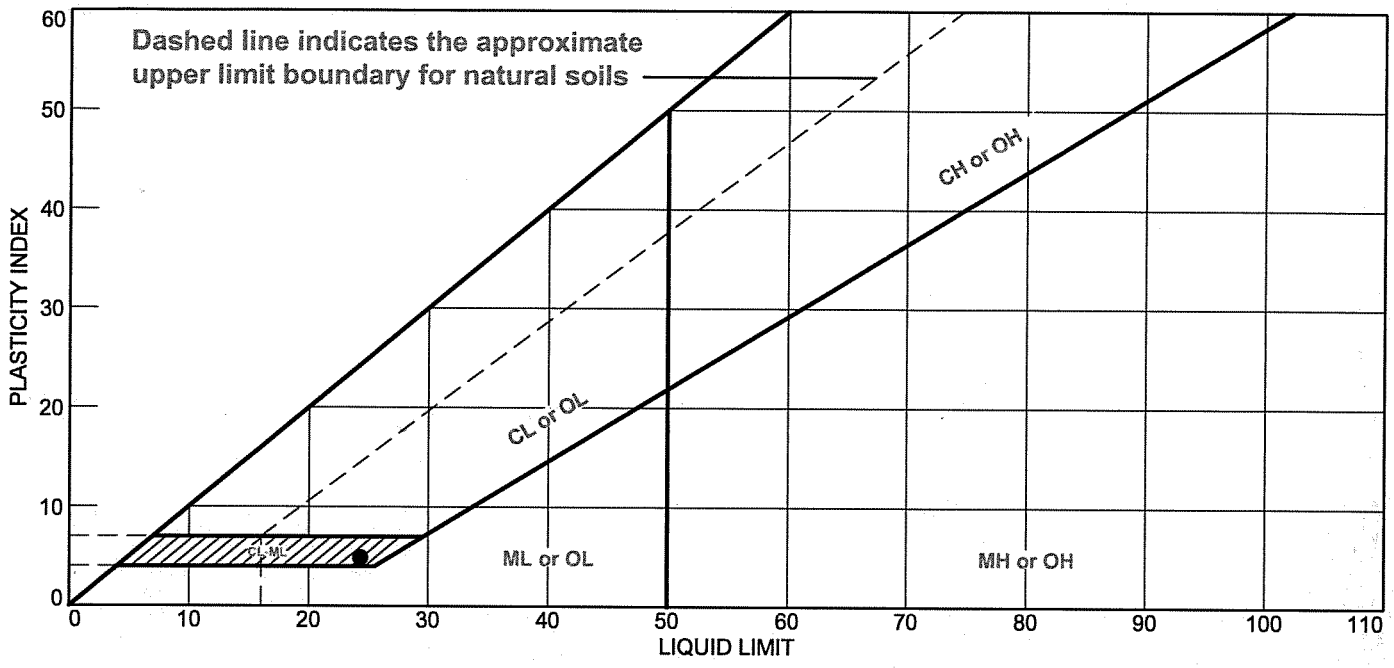
Boring No.: FV-SNR-103

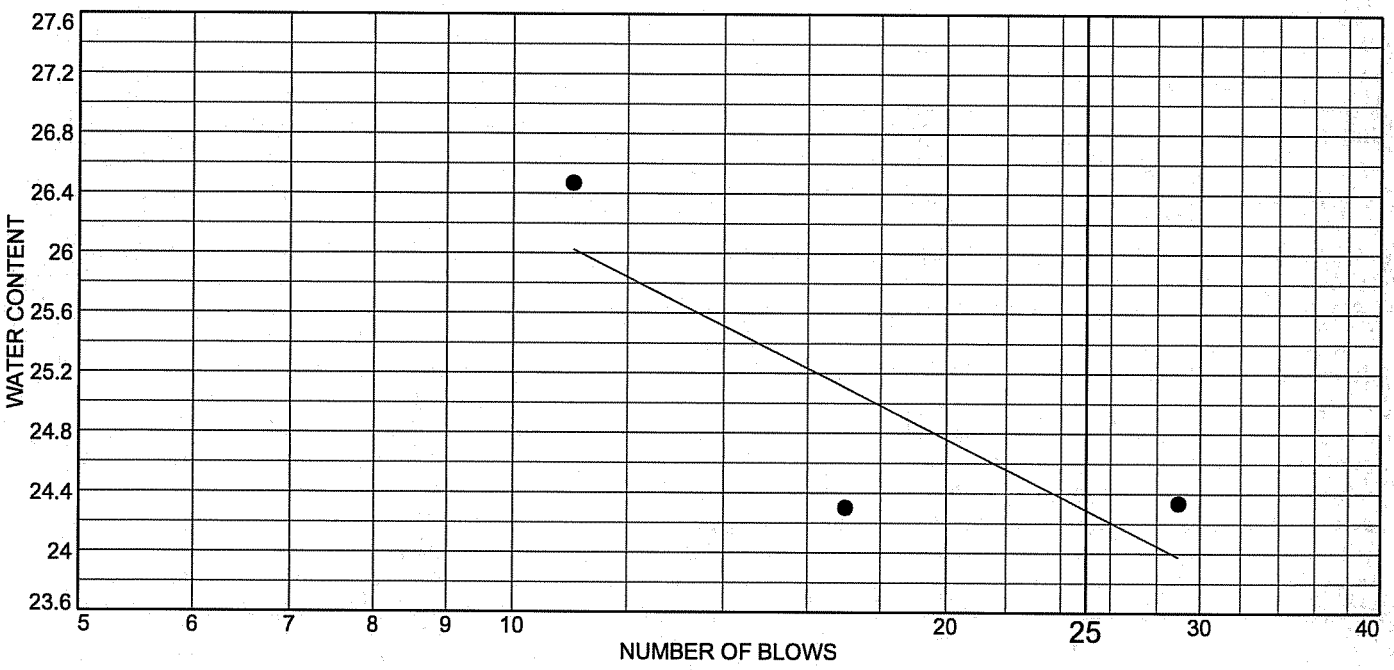
APPENDIX C

LABORATORY TESTING

Report of Geotechnical Evaluation
Scarborough Eastern Trail, MaineDOT Project No. STP-1942(610)
Scarborough, Maine

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●		24.3	19.4	4.9			

Project No. 600-092 **Client:** HNTB

Project: Scarborough Eastern Trail Project

Source of Sample: BB-SPC-102 **Depth:** 30-32'
Sample Number: U-1

R.W. Gillespie & Associates, Inc.

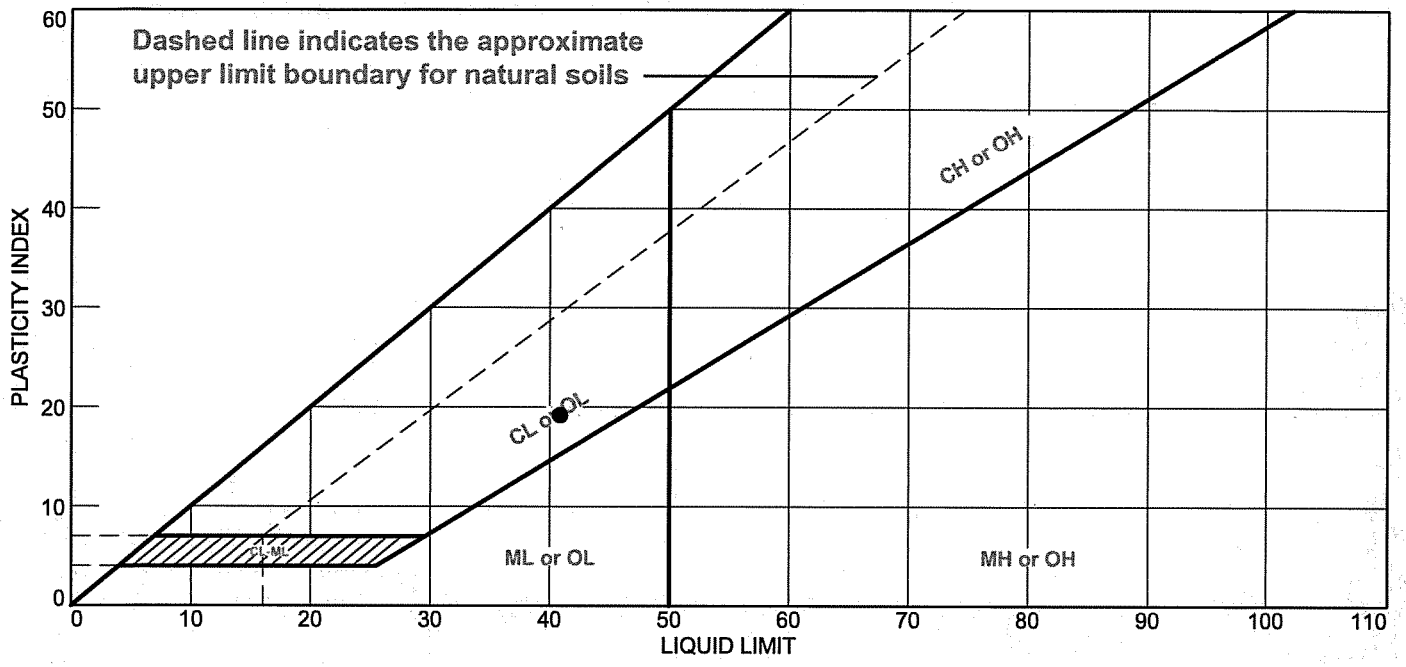
Biddeford, Maine

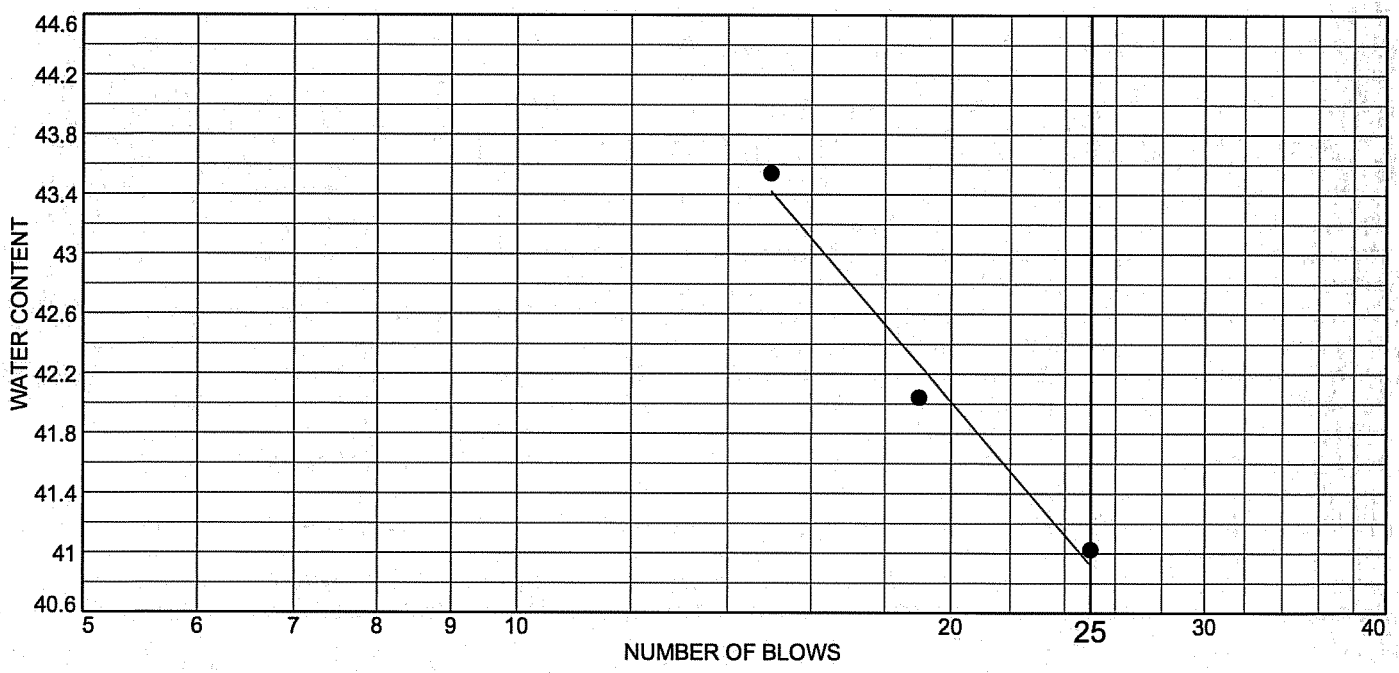
Remarks:

Lab No. 12793a

Tested By: JMB **Checked By:** MTG

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Lean Clay	40.9	21.7	19.2			

Project No. 0600-102 **Client:** HNTB Corporation

Project: Scarborough Eastern Trail Project

Scarborough, ME

Location: BB-SPC-203A

Sample Number: U-1 **Depth:** 20'-22'

R.W. Gillespie & Associates, Inc.

Biddeford, Maine

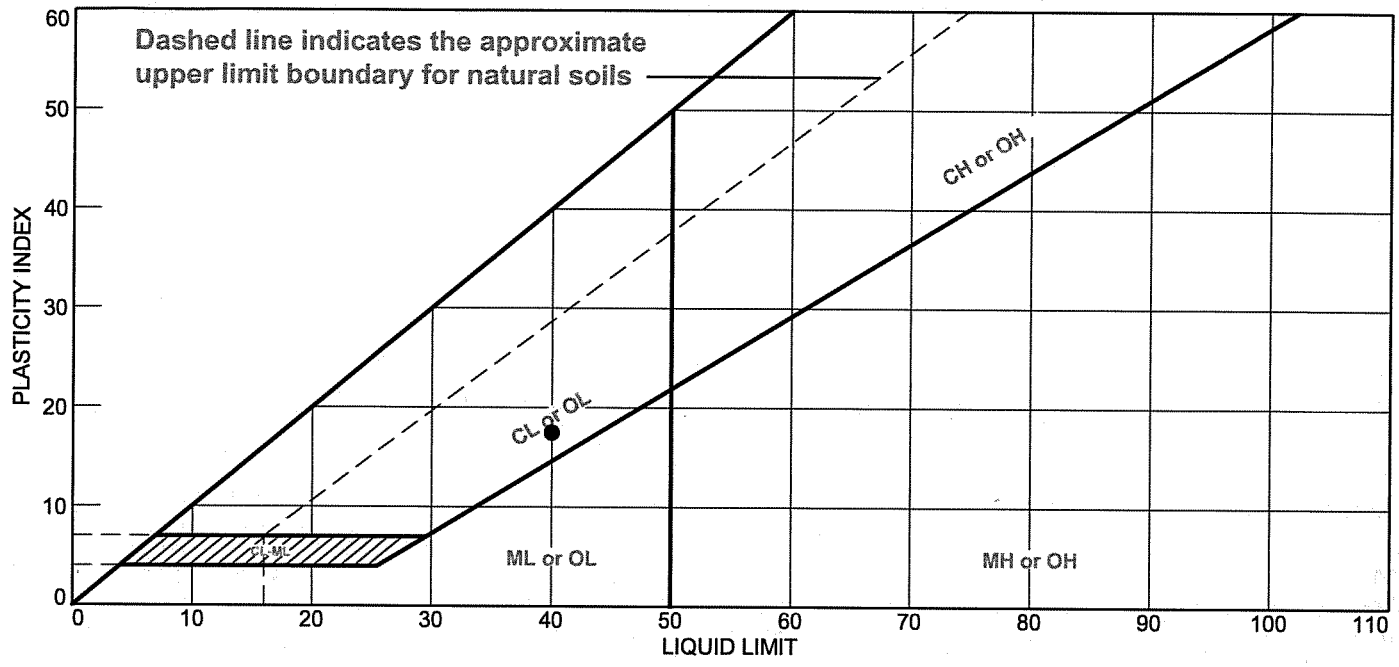
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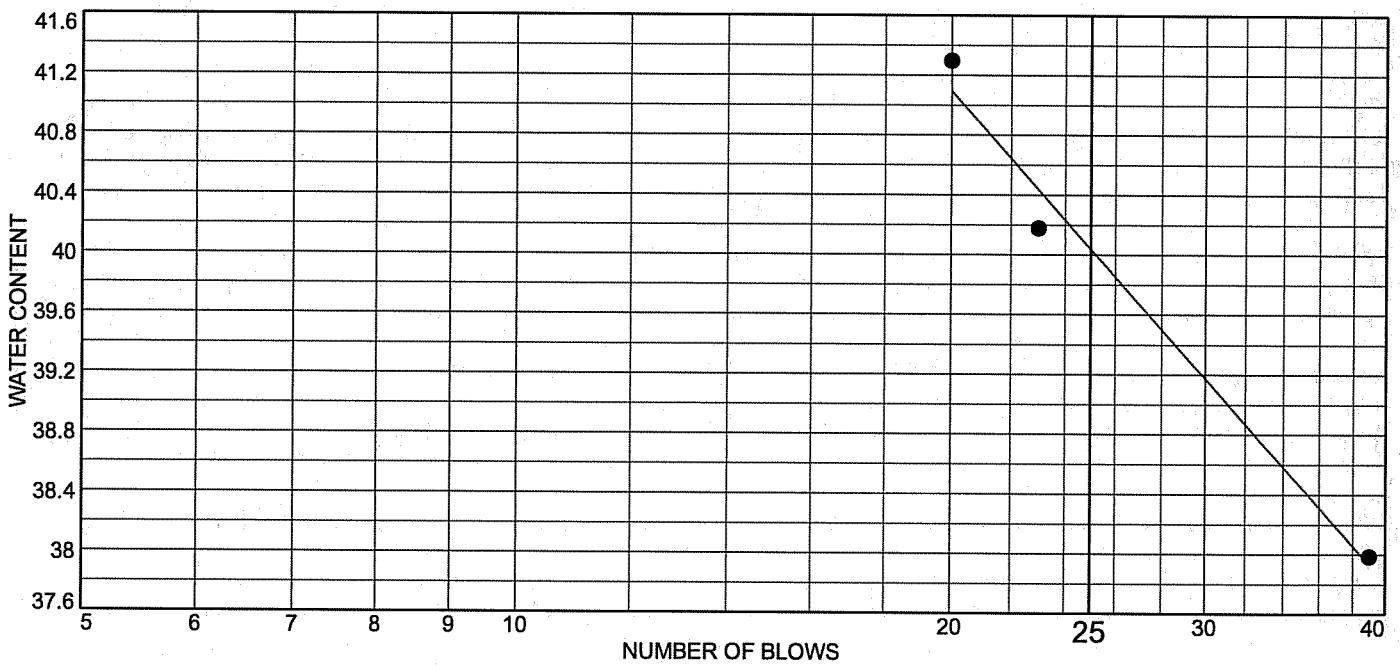
● Moisture Content: 45.6%

Lab No. 15111a

Tested By: AGS **Checked By:** EJW

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Lean Clay	40.0	22.5	17.5			

Project No. 0600-102 **Client:** HNTB Corporation

Project: Scarborough Eastern Trail Project

Scarborough, ME

Location: BB-SPC-203A

Sample Number: U-3 **Depth:** 40'-42'

R.W. Gillespie & Associates, Inc.

Biddeford, Maine

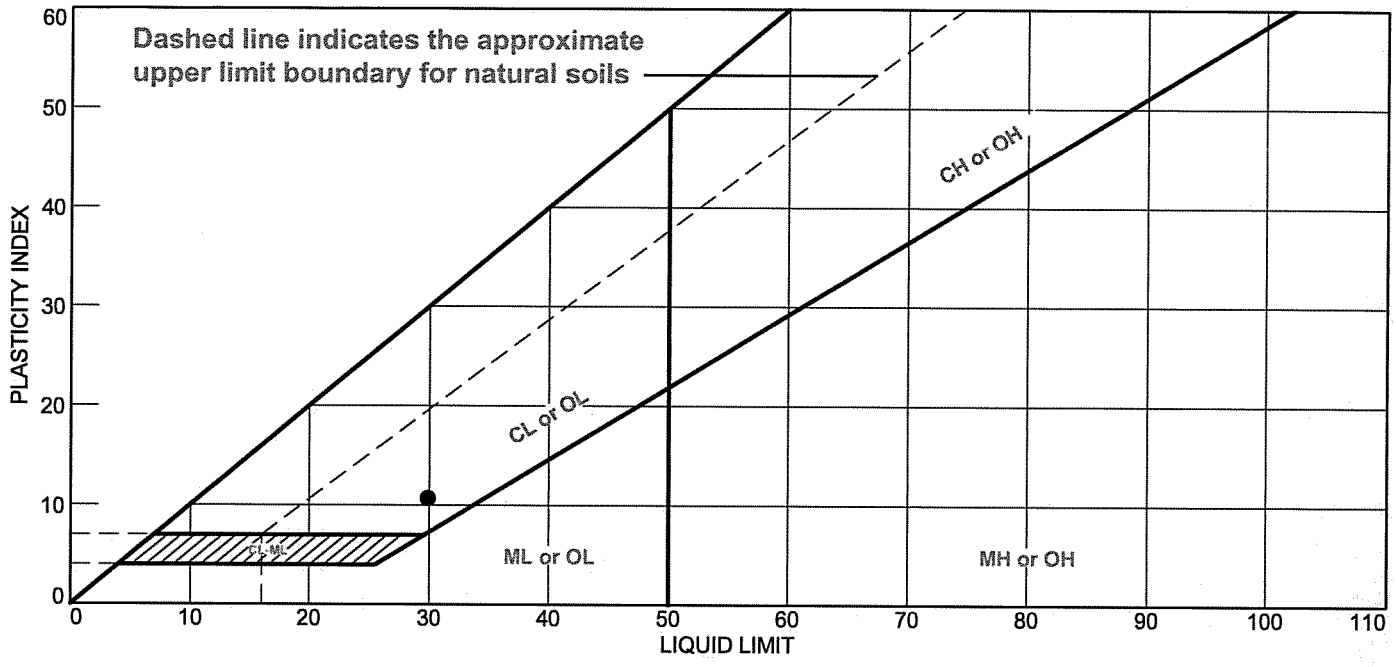
Remarks:

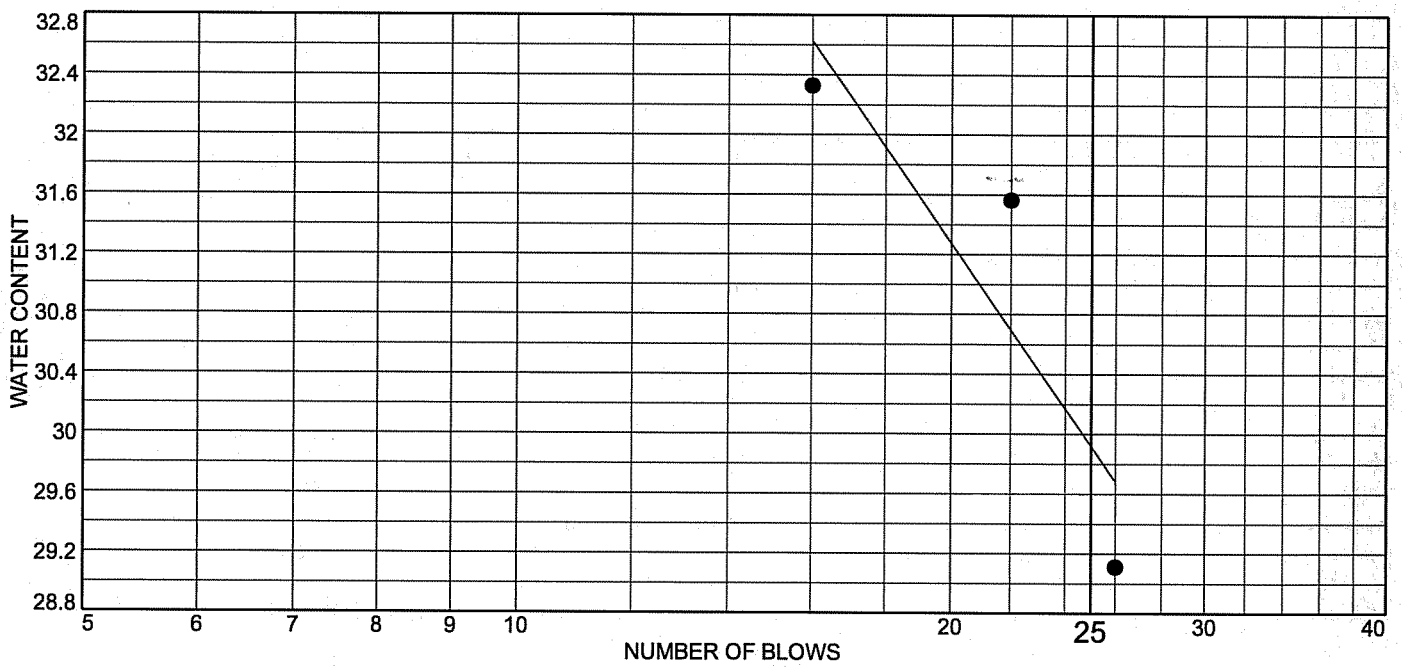
● Moisture Content: 42.3%

Lab No. 15111c

Tested By: AGS **Checked By:** EJW

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Lean Clay	29.9	19.2	10.7			

Project No. 0600-102 **Client:** HNTB Corporation

Project: Scarborough Eastern Trail Project

Scarborough, ME

Location: BB-SPC-203A

Sample Number: U-5 **Depth:** 60'-62'

R.W. Gillespie & Associates, Inc.

Biddeford, Maine

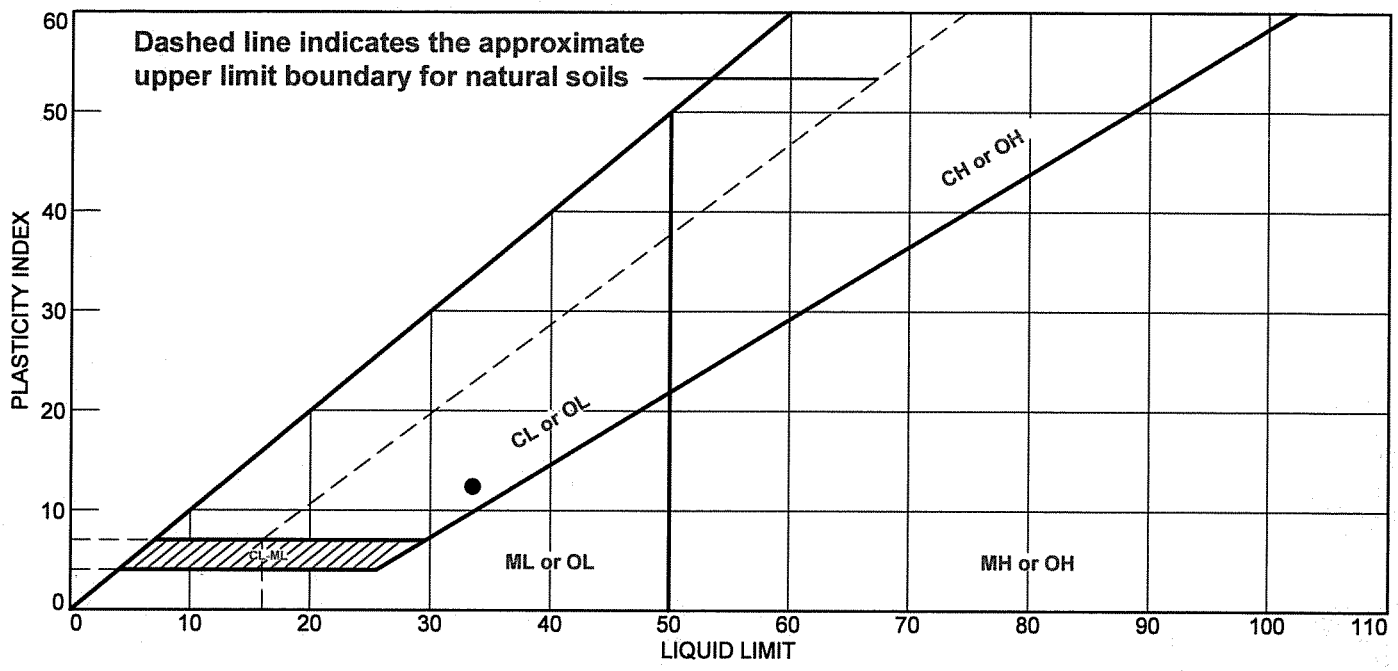
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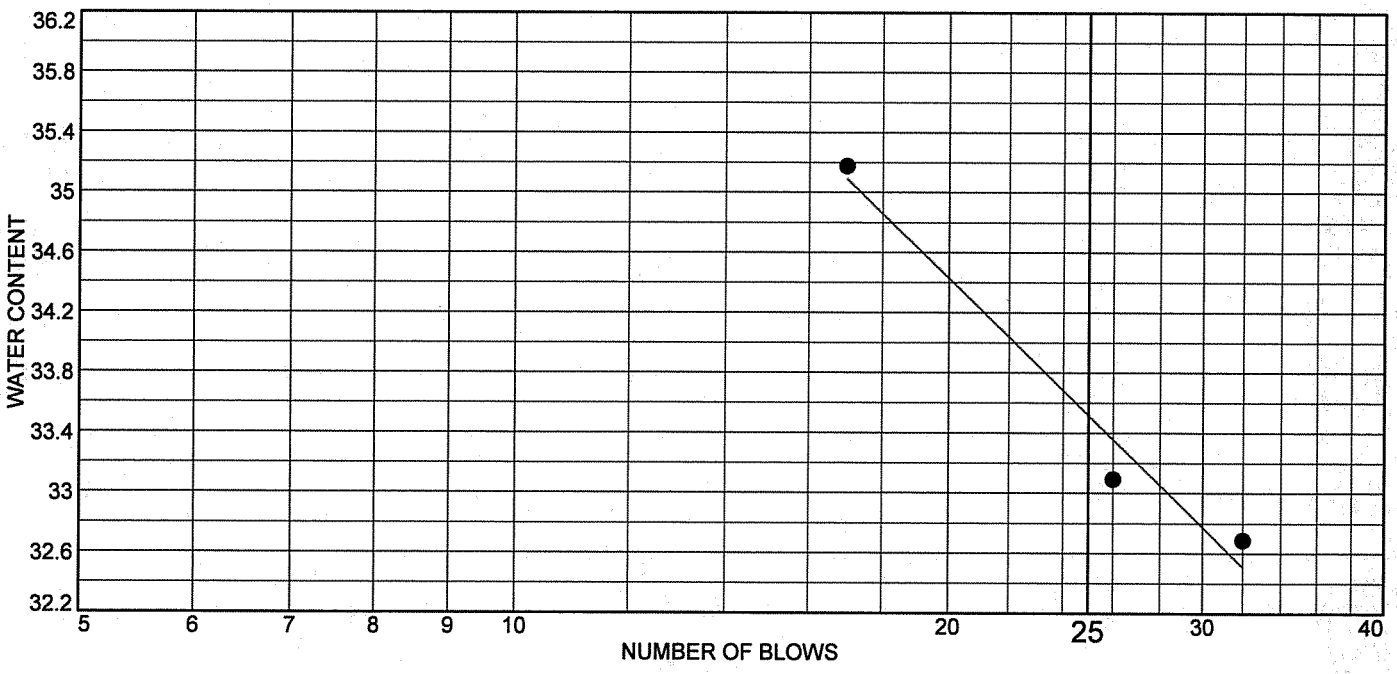
● Moisture Content: 32.3%

Lab No. 15111d

Tested By: AGS **Checked By:** EJW

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Lean Clay	33.5	21.1	12.4			

Project No. 0600-102 **Client:** HNTB Corporation

Project: Scarborough Eastern Trail Project

Scarborough, ME

Location: Boring: EB-SPC-204

Sample Number: S-10 **Depth:** 25'-27

R.W. Gillespie & Associates, Inc.

Biddeford, Maine

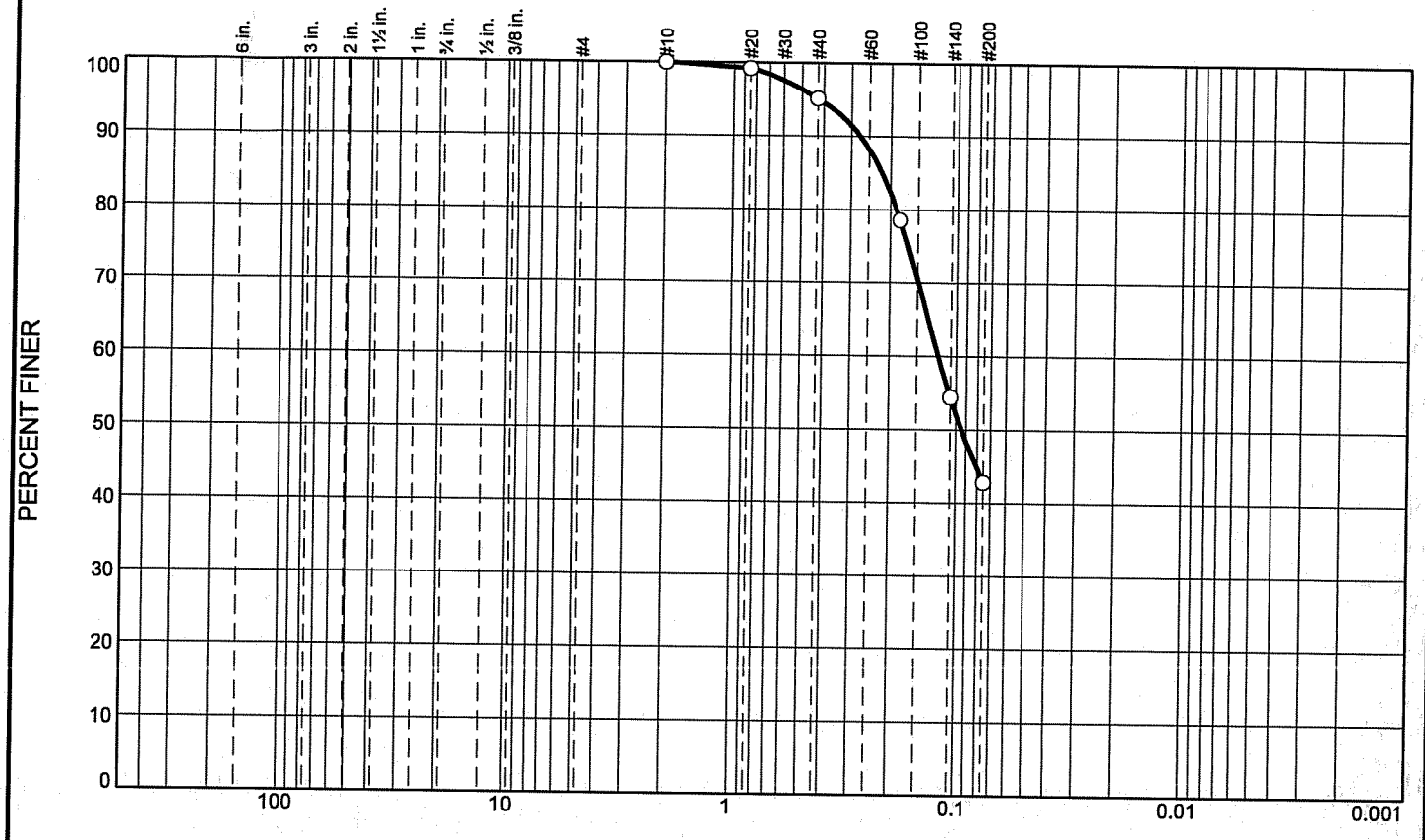
Remarks:

● Moisture Content: 42.5%

Lab No. 15112a

Tested By: JRF **Checked By:** SNP

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	4.9	52.2	42.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#20	99.2		
#40	95.1		
#80	78.6		
#140	54.5		
#200	42.9		

* (no specification provided)

Soil Description

silty sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.2730 D_{85}= 0.2181 D_{60}= 0.1203
D_{50}= 0.0941 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO= A-4(0)

Remarks

Moisture Content: 28.2%

Location: Boring: BB-SPC-101
Sample Number: S-2 **Depth:** 5'-7'

Date: 6/25/13

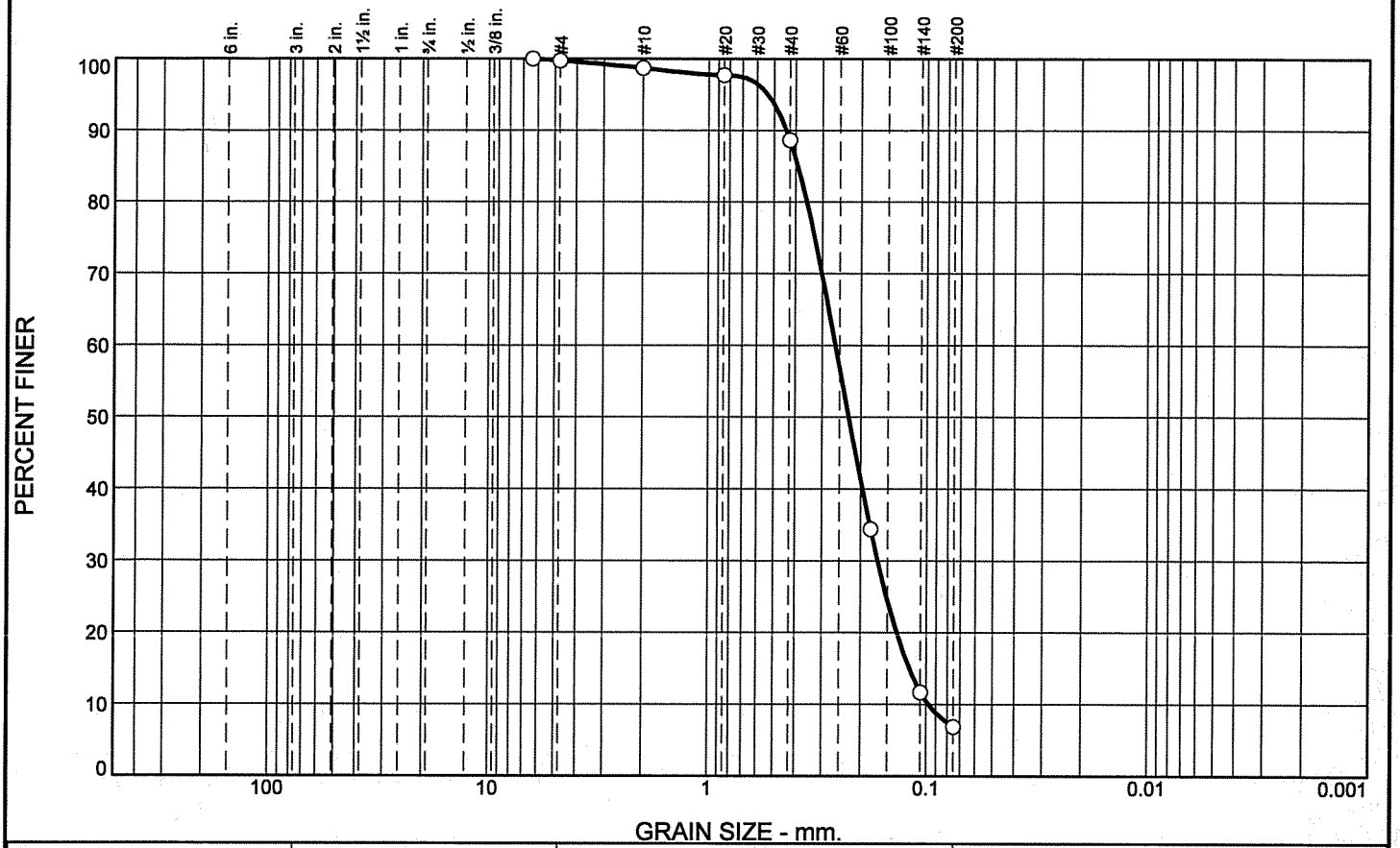
R.W. Gillespie & Associates, Inc.
Biddeford, Maine

Client: HNTB
Project: Scarborough Eastern Trail Project

Project No: 600-092 **Lab No.** 12791

Tested By: AWS **Checked By:** MTG

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.3	1.0	10.1	81.7	6.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1/4"	100.0		
#4	99.7		
#10	98.7		
#20	97.7		
#40	88.6		
#80	34.4		
#140	11.7		
#200	6.9		

* (no specification provided)

Material Description

poorly graded sand with silt

Atterberg Limits

PL= LL= PI=

Classification

USCS= SP-SM AASHTO= A-3

Remarks

Moisture Content: 21.6%

Location: Boring: BB-SPC-201A
Sample Number: S-3 **Depth:** 65'-67'

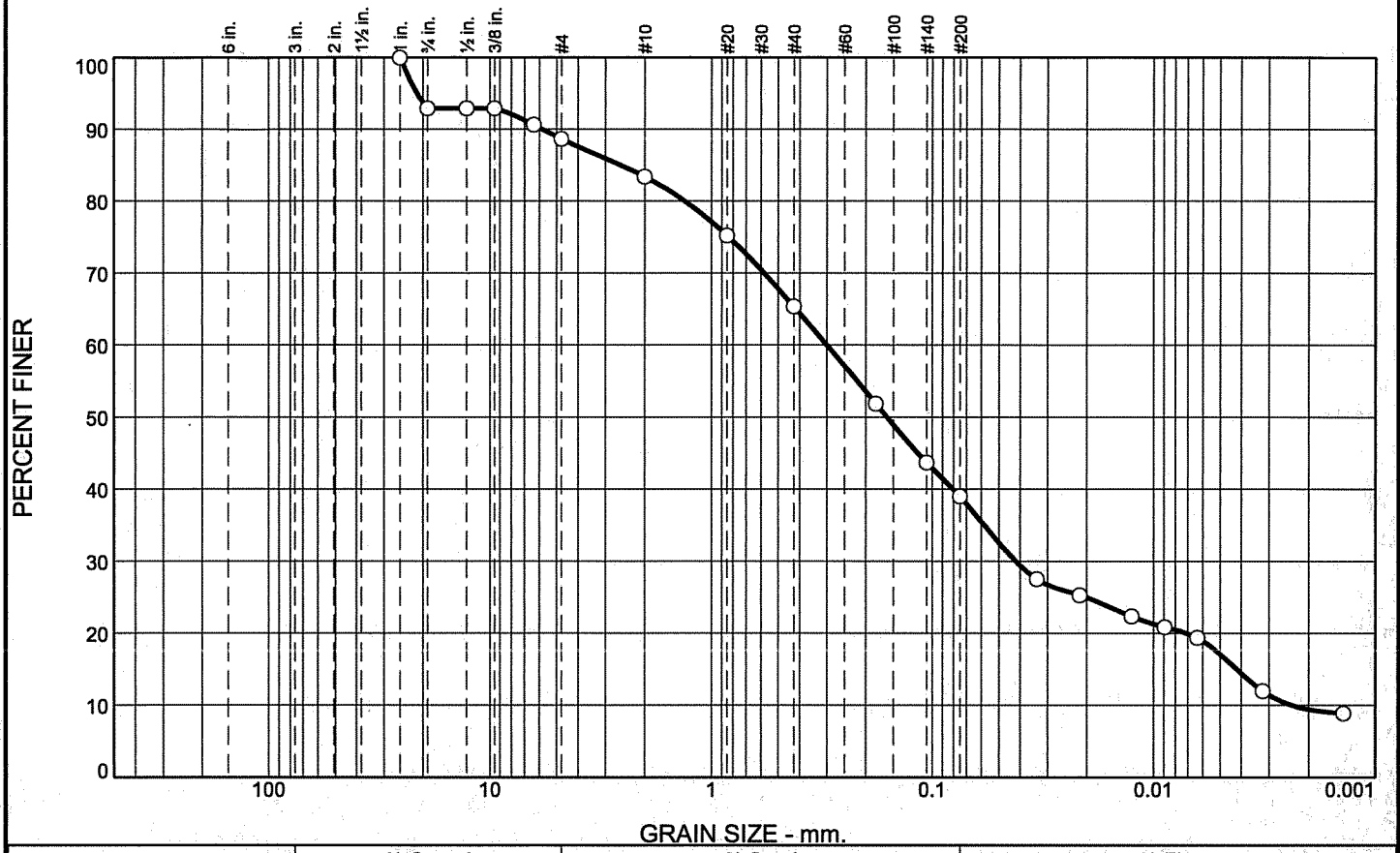
Date: 7/25/18

R.W. Gillespie & Associates, Inc.
Biddeford, Maine

Client: HNTB Corporation
Project: Scarborough Eastern Trail Project
Scarborough, ME
Project No: 0600-102 **Lab No.** 15112f

Tested By: JJB **Checked By:** SNP

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	7.1	4.3	5.2	18.0	26.4	21.9	17.1

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1"	100.0		
3/4"	92.9		
1/2"	92.9		
3/8"	92.9		
1/4"	90.6		
#4	88.6		
#10	83.4		
#20	75.2		
#40	65.4		
#80	51.9		
#140	43.7		
#200	39.0		

* (no specification provided)

Material Description

silty sand

Atterberg Limits

PL= LL= NV PI=

Classification

USCS= SM AASHTO= A-4(0)

Remarks

Moisture Content: 13.2%

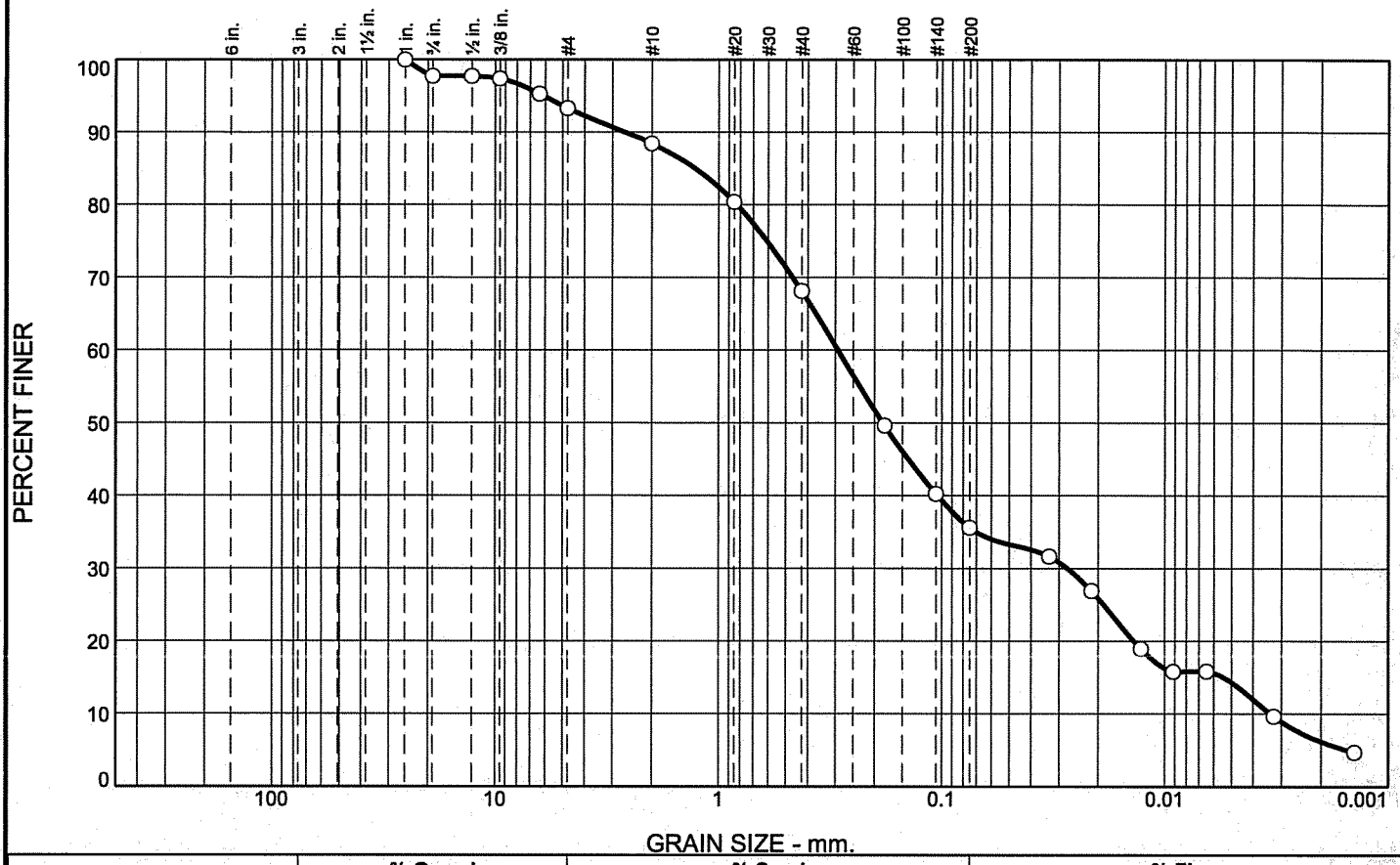
Location: BB-SPC-202
Sample Number: S-9 **Depth:** 55'-57' **Date:** 12/5/16

R.W. Gillespie & Associates, Inc.
Biddeford, Maine

Client: HNTB Corporation
Project: Scarborough Eastern Trail Project
Scarborough, ME
Project No: 0600-102 **Lab No.** 14380A

Tested By: JEL **Checked By:** MTG

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	2.2	4.5	4.9	20.3	32.5	21.2	14.4

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1"	100.0		
3/4"	97.8		
1/2"	97.8		
3/8"	97.4		
1/4"	95.3		
#4	93.3		
#10	88.4		
#20	80.4		
#40	68.1		
#80	49.6		
#140	40.3		
#200	35.6		

* (no specification provided)

Material Description

silty sand

Atterberg Limits

PL= LL= PI=

Classification

USCS= SM AASHTO= A-4(0)

Remarks

Moisture Content: 11.2%

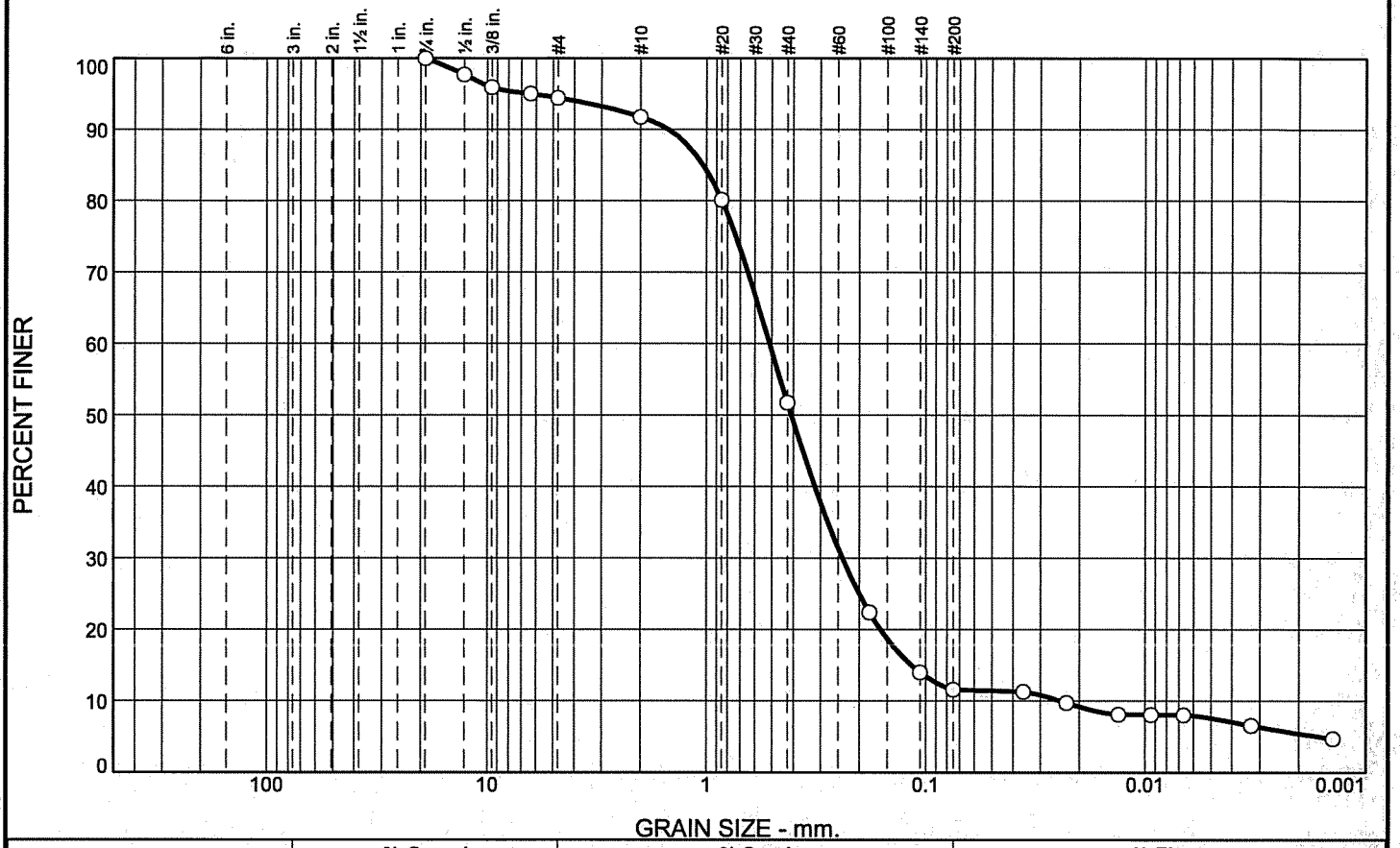
Location: BB-SPC-202
Sample Number: S-10 **Depth:** 60'-62' **Date:** 12/5/16

R.W. Gillespie & Associates, Inc.
Biddeford, Maine

Client: HNTB Corporation
Project: Scarborough Eastern Trail Project
Scarborough, ME
Project No: 0600-102 **Lab No.** 14380B

Tested By: JEL **Checked By:** MTG

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	5.6	2.6	40.0	40.2	4.0	7.6

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100.0		
1/2"	97.7		
3/8"	95.9		
1/4"	95.0		
#4	94.4		
#10	91.8		
#20	80.1		
#40	51.8		
#80	22.4		
#140	14.0		
#200	11.6		

* (no specification provided)

Material Description

poorly graded sand with silt

Atterberg Limits

PL= LL= NV PI=

Classification

USCS= SP-SM AASHTO= A-2-4(0)

Remarks

Moistrue Content: 14.4%

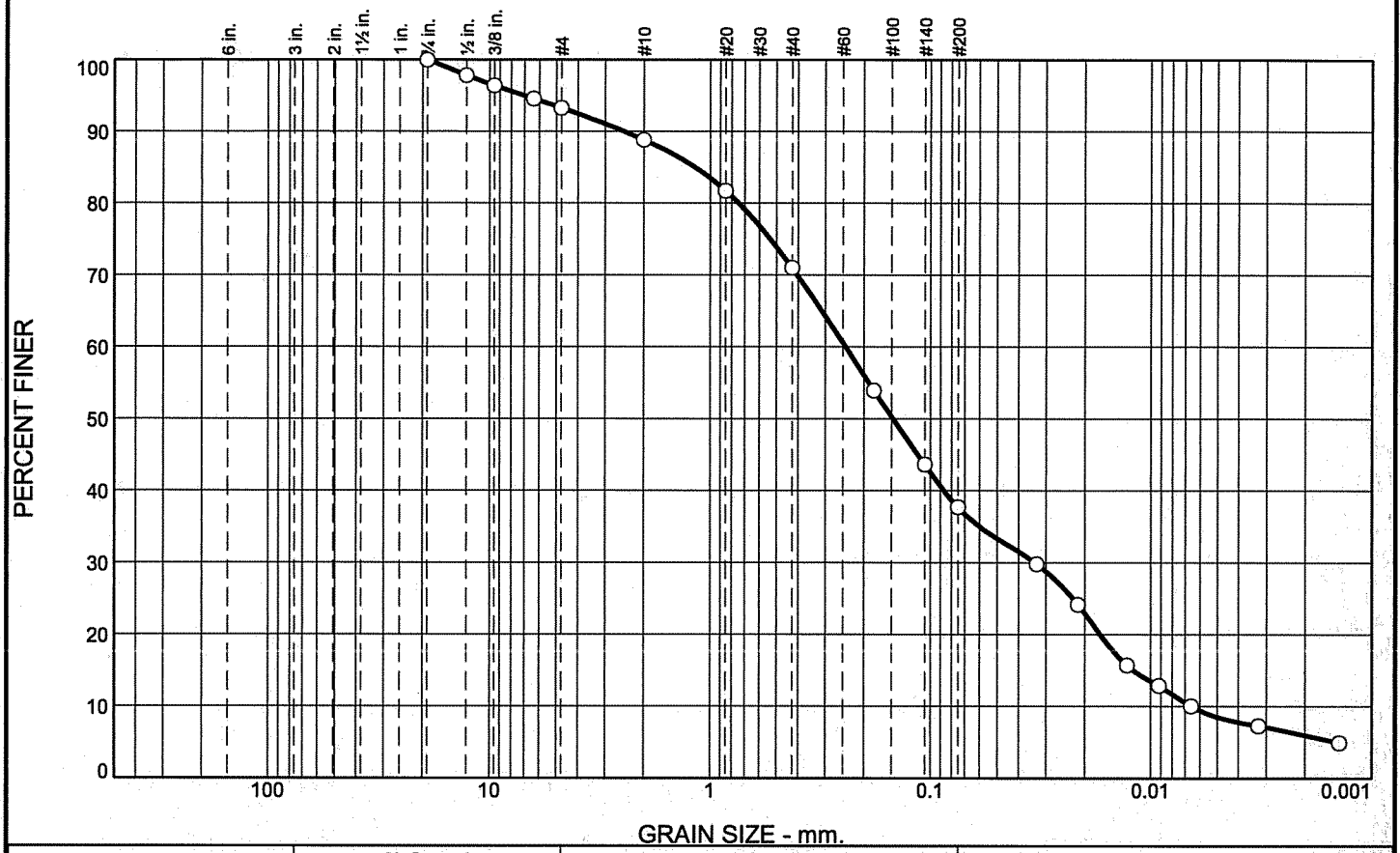
Location: BB-SPC-202
Sample Number: S-12 **Depth:** 70'-72' **Date:** 12/5/16

R.W. Gillespie & Associates, Inc.
Biddeford, Maine

Client: HNTB Corporation
Project: Scarborough Eastern Trail Project
Scarborough, ME
Project No: 0600-102 **Lab No.** 14380C

Tested By: JEL **Checked By:** MTG

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	6.7	4.5	17.8	33.3	29.2	8.5

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100.0		
1/2"	97.8		
3/8"	96.4		
1/4"	94.6		
#4	93.3		
#10	88.8		
#20	81.7		
#40	71.0		
#80	53.9		
#140	43.7		
#200	37.7		

* (no specification provided)

Material Description

silty sand

Atterberg Limits

PL= LL= PI=

Classification

USCS= SM AASHTO= A-4(0)

Remarks

Moisture Content: 9.6%

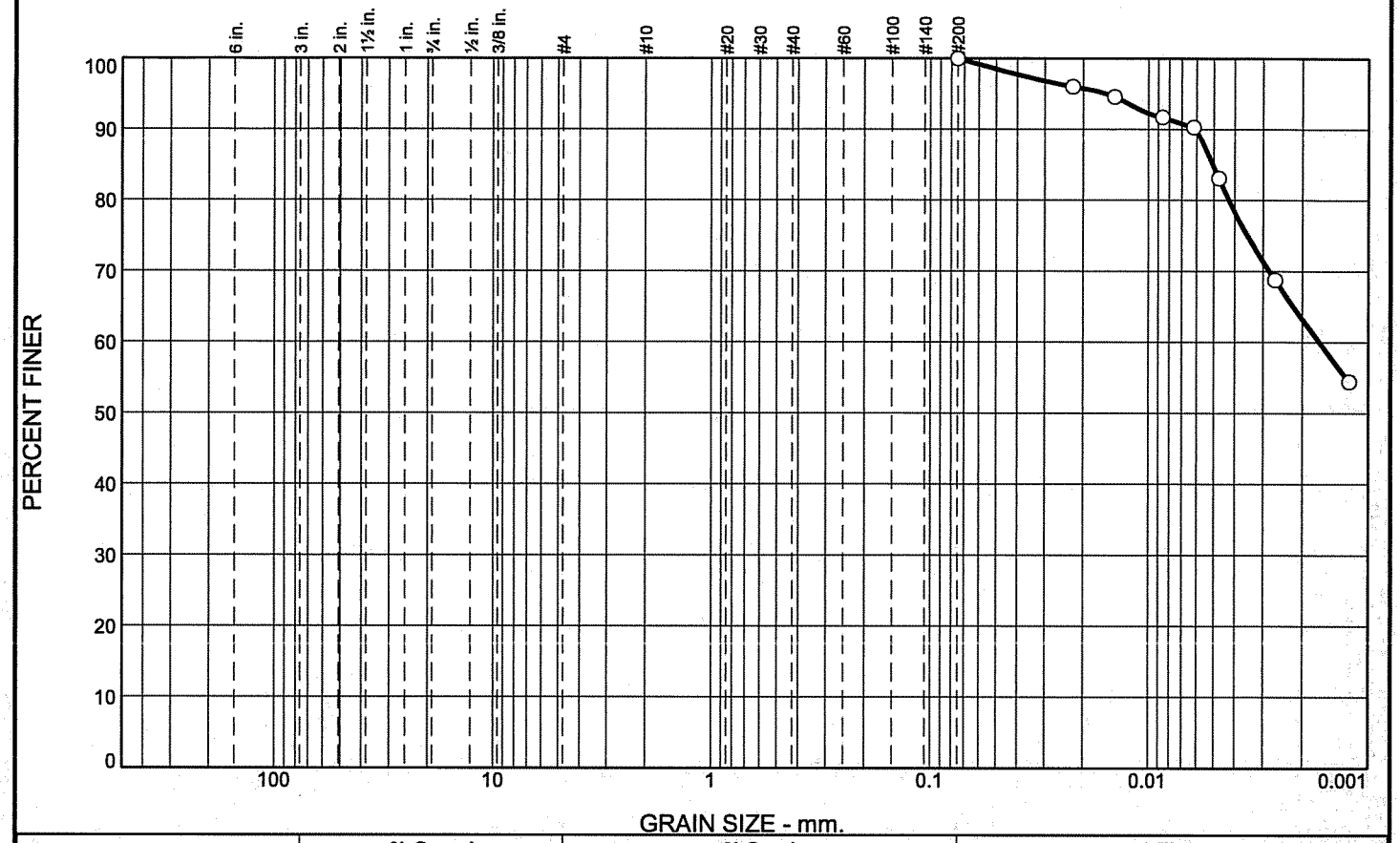
Location: BB-SPC-202
Sample Number: S-14 **Depth:** 80'-82' **Date:** 12/5/16

R.W. Gillespie & Associates, Inc.
Biddeford, Maine

Client: HNTB Corporation
Project: Scarborough Eastern Trail Project
Scarborough, ME
Project No: 0600-102 **Lab No.** 14380D

Tested By: JEL **Checked By:** MTG

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	0.0	0.0	15.2	84.8

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	100.0		

* (no specification provided)

Material Description

Silty Clay

Atterberg Limits

PL= LL= PI=

Classification

USCS= ML AASHTO= A-4(0)

Remarks

Moisture Content: 47.4%

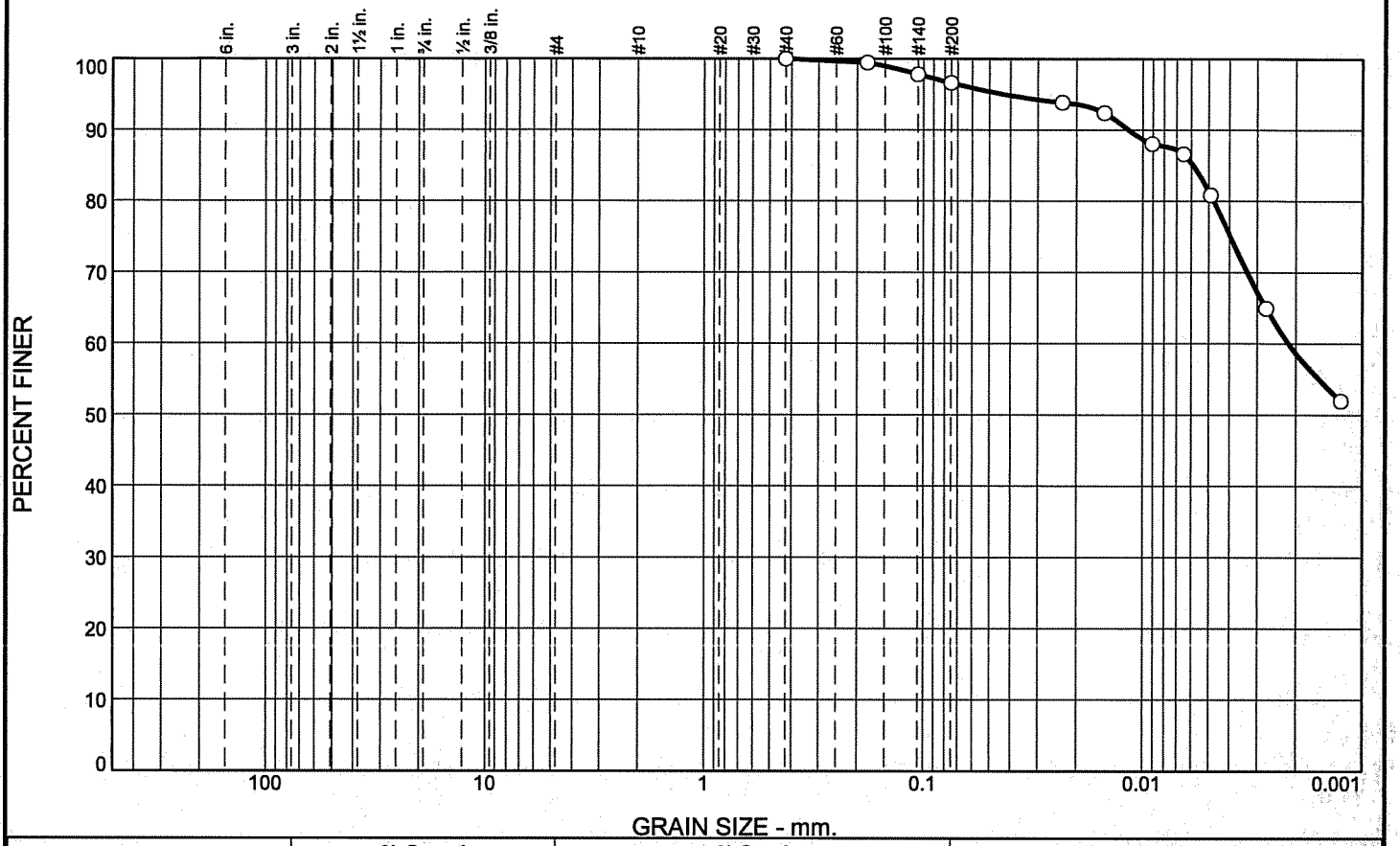
Location: BB-SPC-203
Sample Number: U-1 **Depth:** 25'-27' **Date:** 11/10/16

R.W. Gillespie & Associates, Inc.
Biddeford, Maine

Client: HNTB Corporation
Project: Scarborough Eastern Trail Project
Scarborough, ME
Project No: 0600-102 **Lab No.** 14328A

Tested By: JEL **Checked By:** MTG

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	0.0	3.4	15.2	81.4

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#40	100.0		
#80	99.5		
#140	97.8		
#200	96.6		

* (no specification provided)

Material Description

silt

Atterberg Limits

PL= LL= PI=

Classification

USCS= ML AASHTO= A-4(0)

Remarks

Moisture Content: 40.9%

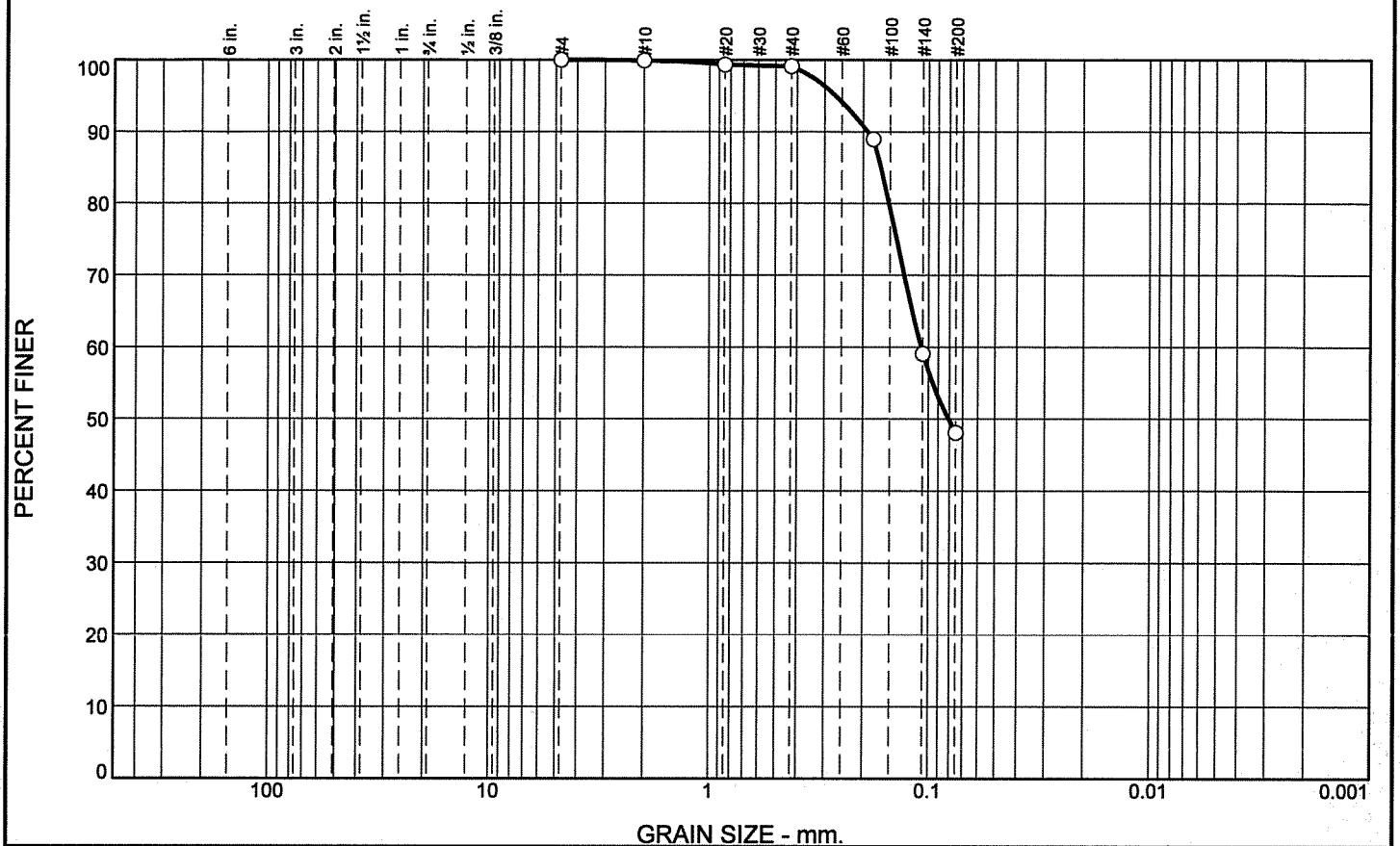
Location: BB-SPC 203
Sample Number: U-1 **Depth:** 40'-42' **Date:** 11/10/16

R.W. Gillespie & Associates, Inc.
Biddeford, Maine

Client: HNTB Corporation
Project: Scarborough Eastern Trail Project
Scarborough, ME
Project No: 0600-102 **Lab No.** 14328B

Tested By: JEL **Checked By:** MTG

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.1	0.8`	51.0	48.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	99.9		
#20	99.4		
#40	99.1		
#80	89.0		
#140	59.1		
#200	48.1		

* (no specification provided)

Material Description

silty sand

Atterberg Limits

PL= LL= PI=

Classification

USCS= SM AASHTO= A-4(0)

Remarks

Moisture Content: 31.2%

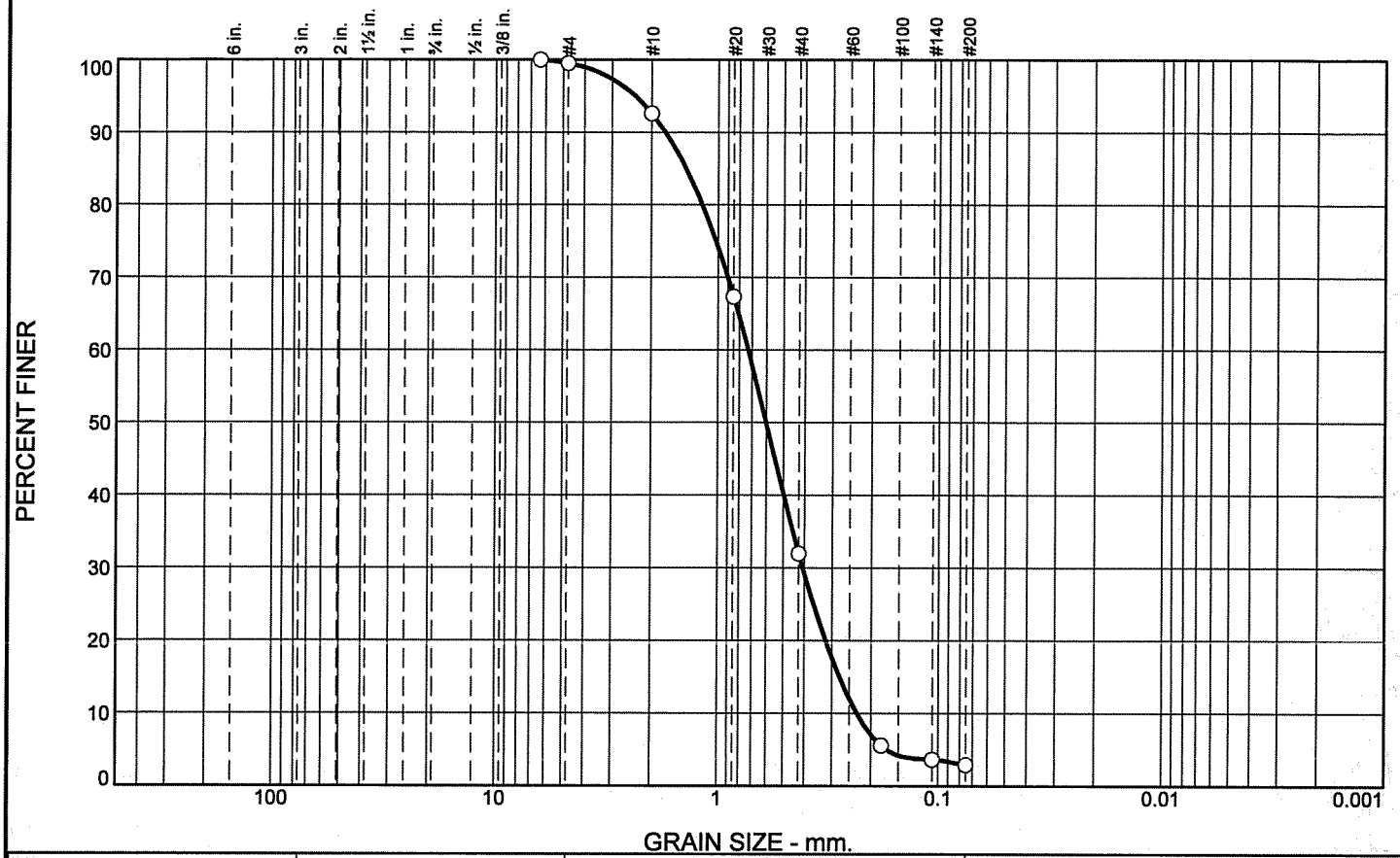
Location: Boring: EB-SPC-204
Sample Number: S-7 **Depth:** 12'-14' **Date:** 7/25/18

R.W. Gillespie & Associates, Inc.
Biddeford, Maine

Client: HNTB Corporation
Project: Scarborough Eastern Trail Project
Scarborough, ME
Project No: 0600-102 **Lab No.** 15112b

Tested By: JJB **Checked By:** SNP

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.5	6.9	60.6	29.1	2.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1/4"	100.0		
#4	99.5		
#10	92.6		
#20	67.3		
#40	32.0		
#80	5.6		
#140	3.6		
#200	2.9		

* (no specification provided)

Material Description

poorly graded sand

Atterberg Limits

PL= LL= PI=

Classification

USCS= SP AASHTO= A-1-b

Remarks

Moisture Content: 21.6%

Location: Boring: EB-SPC-205
Sample Number: S-4 **Depth:** 6'-8'

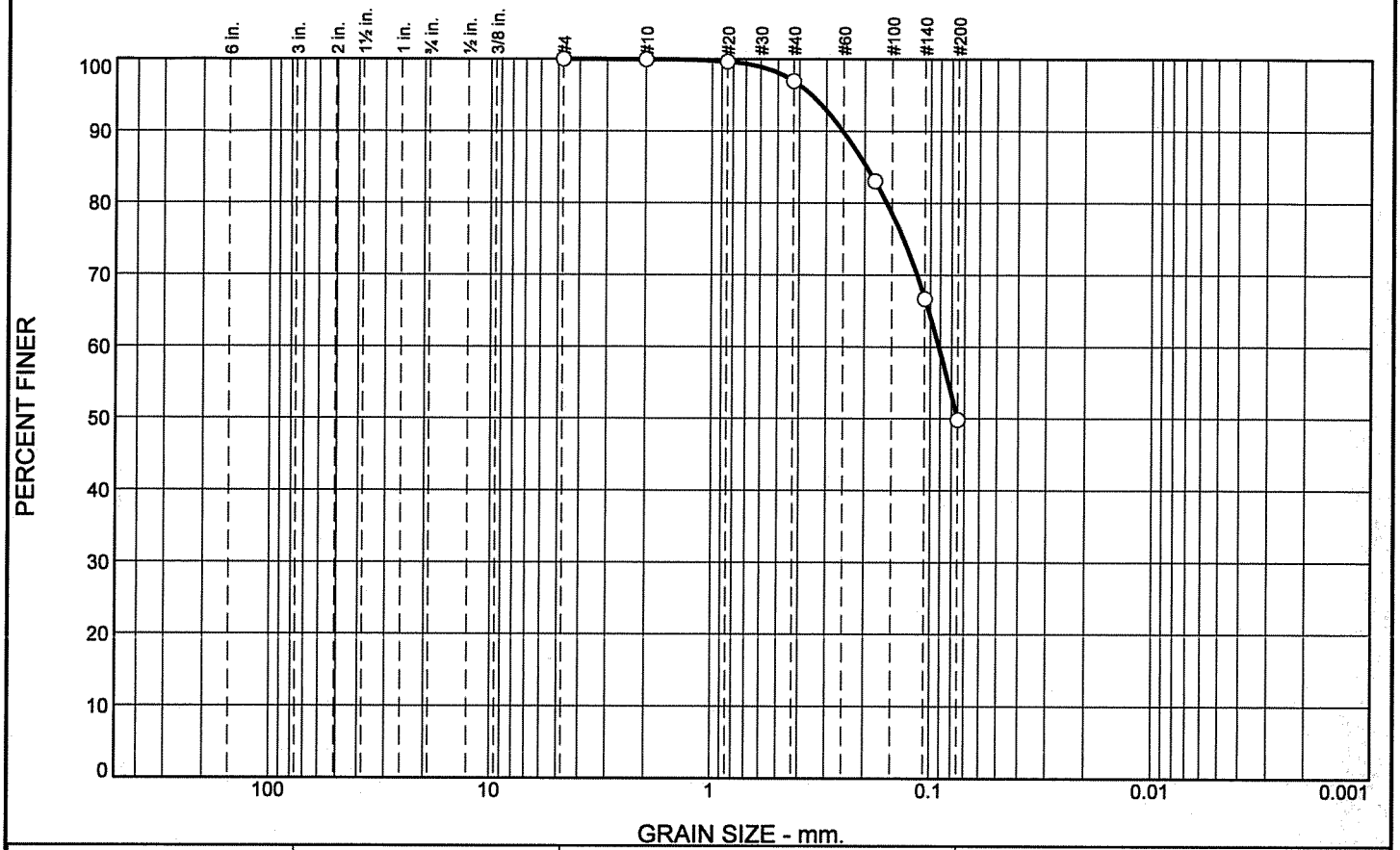
Date: 7/25/18

R.W. Gillespie & Associates, Inc.
Biddeford, Maine

Client: HNTB Corporation
Project: Scarborough Eastern Trail Project
Scarborough, ME
Project No: 0600-102 **Lab No.** 15112c

Tested By: JJB **Checked By:** SNP

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	3.1	47.0	49.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	100.0		
#20	99.6		
#40	96.9		
#80	83.1		
#140	66.7		
#200	49.9		

* (no specification provided)

Material Description

silty sand

Atterberg Limits

PL= LL= PI=

Classification

USCS= SM AASHTO= A-4(0)

Remarks

Moisture Content: 21.6%

Location: Boring: EB-SPC-205
Sample Number: S-12 **Depth:** 40'-42'

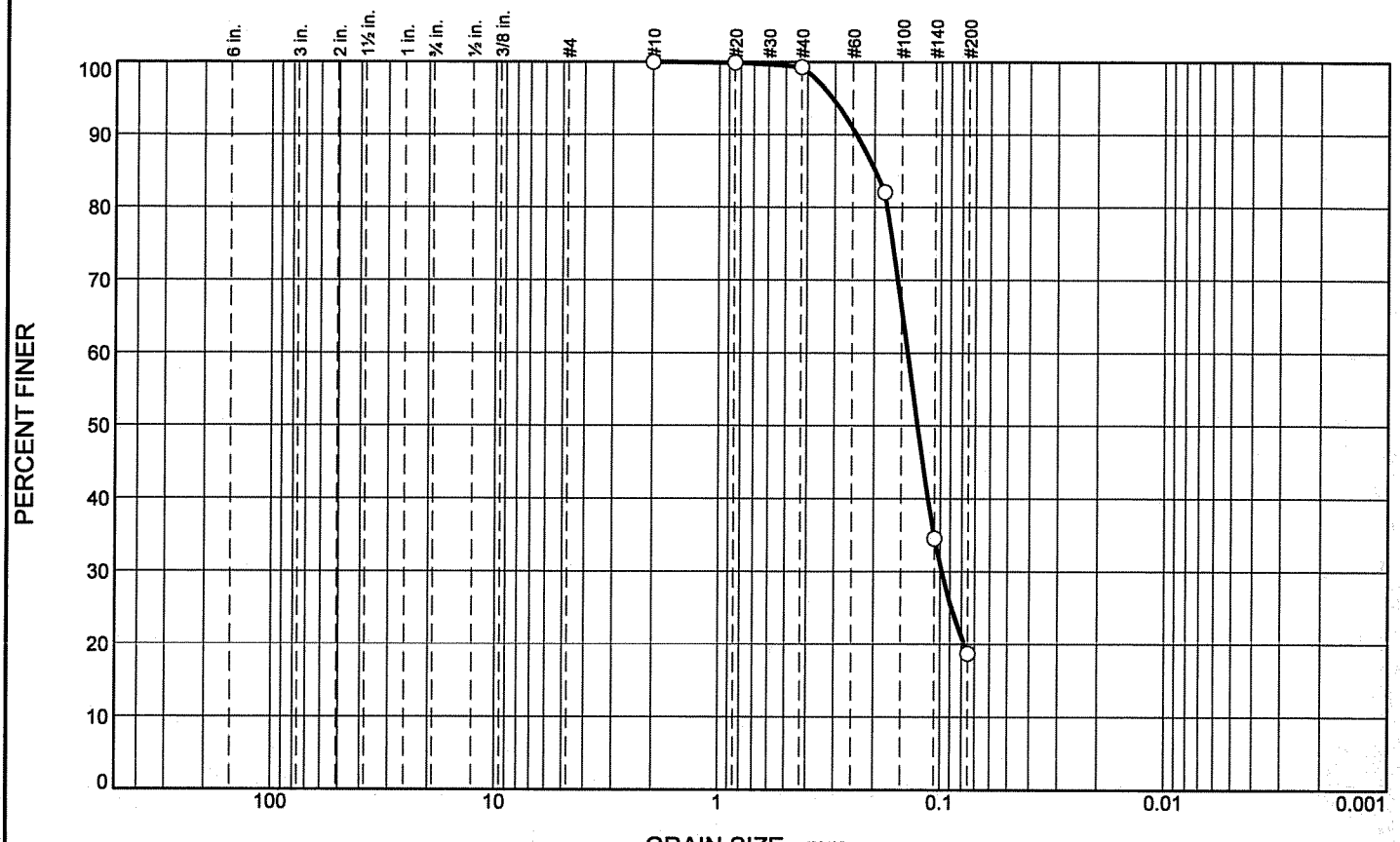
Date: 7/25/18

R.W. Gillespie & Associates, Inc.
Biddeford, Maine

Client: HNTB Corporation
Project: Scarborough Eastern Trail Project
Scarborough, ME
Project No: 0600-102 **Lab No.** 15112d

Tested By: JJB **Checked By:** SNP

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	0.7	80.6	18.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#20	99.9		
#40	99.3		
#80	82.1		
#140	34.6		
#200	18.7		

* (no specification provided)

Material Description

silty sand

Atterberg Limits

PL= LL= PI=

Classification

USCS= SM AASHTO= A-2-4(0)

Remarks

Moisture Content: 24.0%

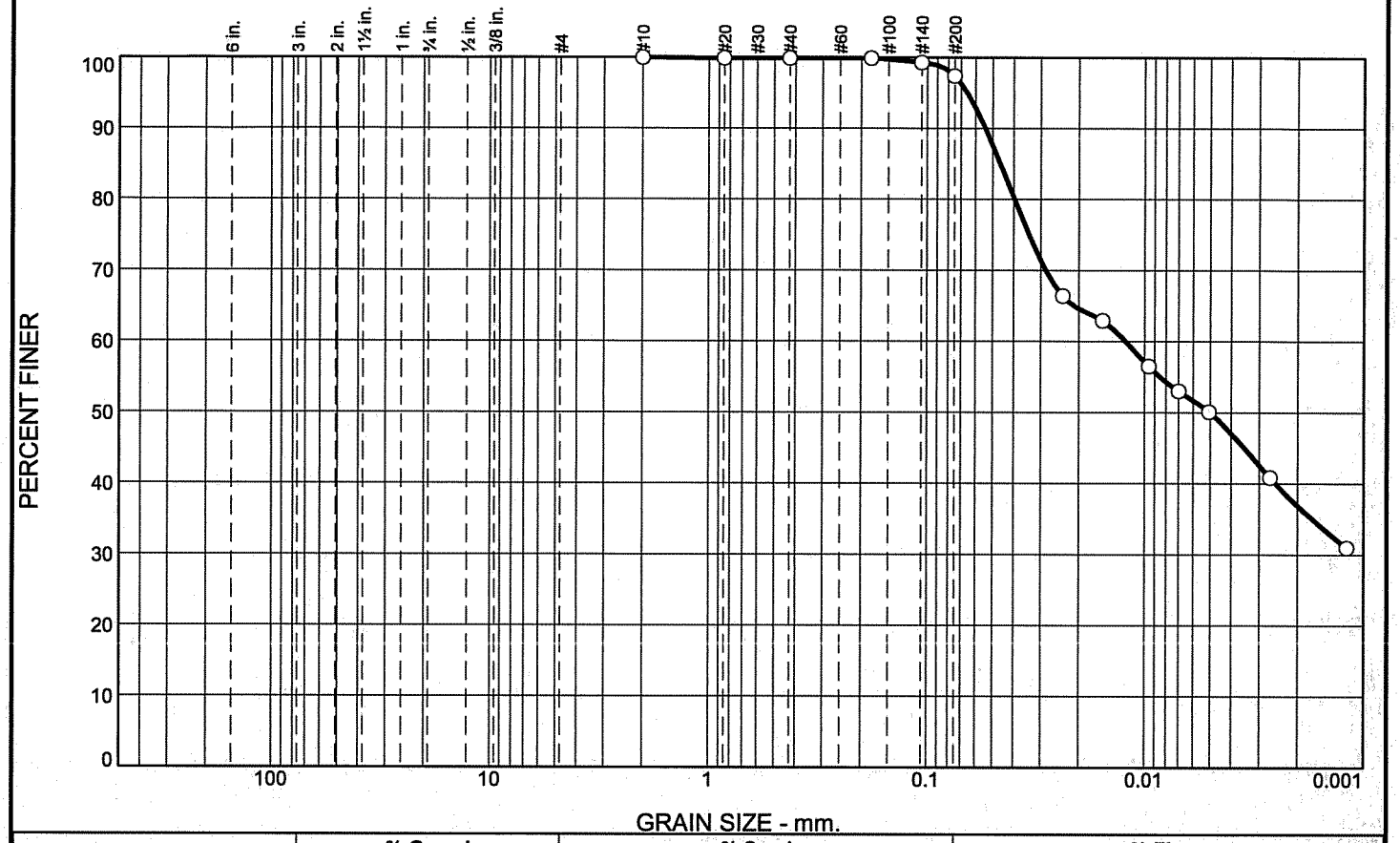
Location: Boring: EB-SPC-205
Sample Number: S-13 **Depth:** 45'-47' **Date:** 7/25/18

R.W. Gillespie & Associates, Inc.
Biddeford, Maine

Client: HNTB Corporation
Project: Scarborough Eastern Trail Project
Scarborough, ME
Project No: 0600-102 **Lab No.** 15112e

Tested By: JJB **Checked By:** SNP

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	0.1	2.5	47.4	50.0

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#20	99.9		
#40	99.9		
#80	99.9		
#140	99.3		
#200	97.4		

* (no specification provided)

Material Description

silty Clay

Atterberg Limits

PL= LL= NV PI=

Classification

USCS= AASHTO=

Remarks

Moisture Content: 29.9%

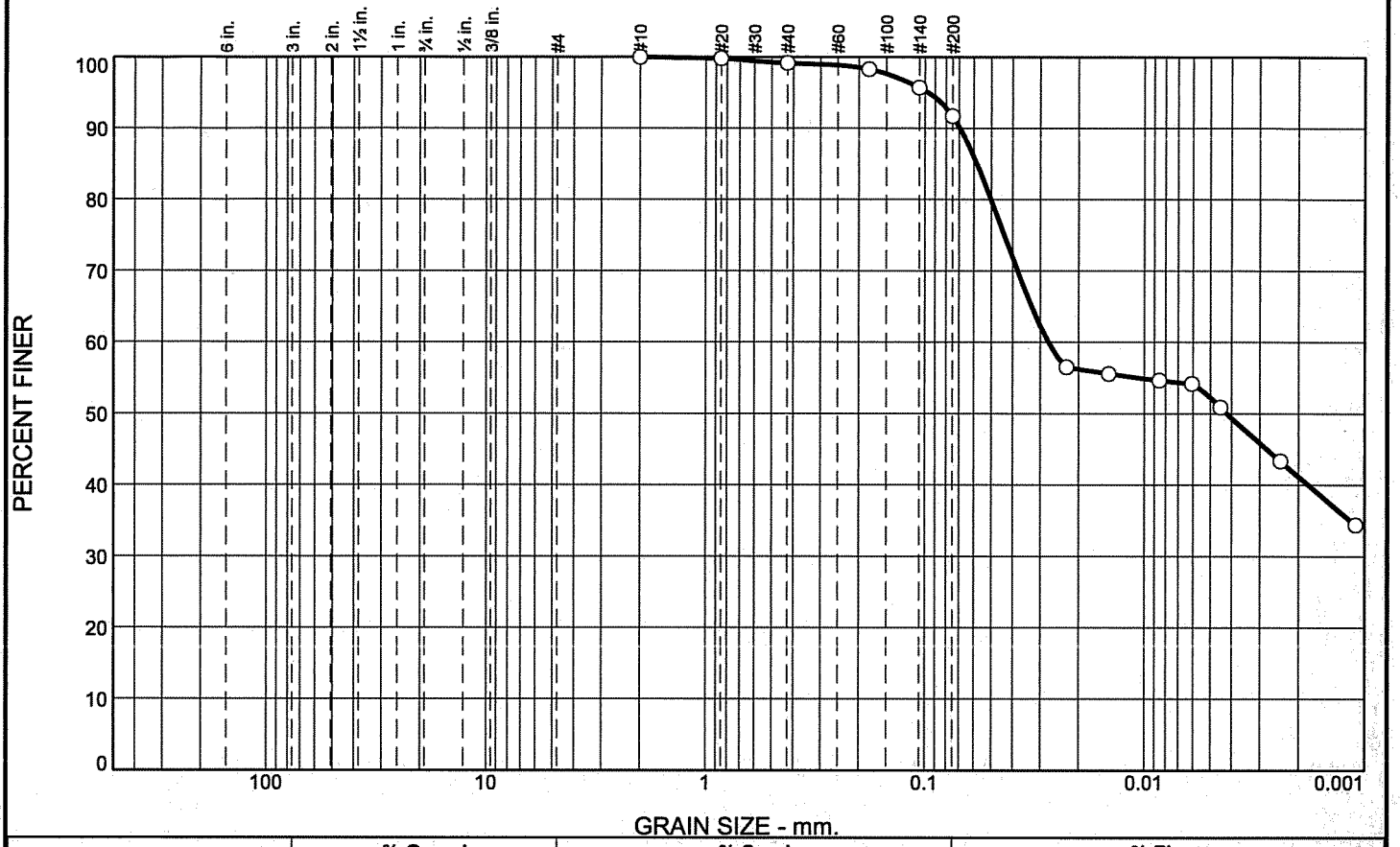
Location: EB-SNR-102
Sample Number: U-2 **Depth:** 35'-36' **Date:** 10/17/16

R.W. Gillespie & Associates, Inc.
Biddeford, Maine

Client: HNTB Corporation
Project: Scarborough Eastern Trail Project
Scarborough, ME
Project No: 0600-102 **Lab No.** 14285b

Tested By: JEL **Checked By:** MTG

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	0.9	7.4	39.4	52.3

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#20	99.8		
#40	99.1		
#80	98.3		
#140	95.7		
#200	91.7		

* (no specification provided)

Material Description

Silty Clay

Atterberg Limits

PL= LL= NV PI=

Classification

USCS= ML AASHTO= A-4(0)

Remarks

Moisture Content: 39.2%

Location: EB-SNR-102
Sample Number: U-1 **Depth:** 25'-27' **Date:** 10/17/16

R.W. Gillespie & Associates, Inc.
Biddeford, Maine

Client: HNTB Corporation
Project: Scarborough Eastern Trail Project
Scarborough, ME
Project No: 0600-102 **Lab No.** 14285a

Tested By: JEL **Checked By:** MTG

Laboratory Vane Shear Test Results

Project: Scarborough Eastern Trail **Client:** HNTB

Project No.: 0600-092 **Location:** Scarborough, ME

Boring No.	BB-SPC-102	**Lab No.**	12793a
Sample No.	U-1, 30'-32'		
Test No.	**S_u (Undisturbed)**	**S_u (Residual)**	**Moisture Content**
1	219 psf	0 psf	40.5%
2	209 psf	0 psf	31.2%
3	418 psf	0 psf	39.6%

Laboratory Vane Shear Test Results

Project: Scarborough Eastern Trail **Client:** HNTB

Project No.: 0600-092 **Location:** Scarborough, ME

Boring No.	BB-SPC-102	**Lab No.**	12793b
Sample No.	U-3, 50'-52'		
Test No.	**S_u (Undisturbed)**	**S_u (Residual)**	**Moisture Content**
1	480 psf	0 psf	43.2%
2	585 psf	0 psf	42.3%
3	345 psf	0 psf	39.4%
4	418 psf	0 psf	41.6%

GeoTesting EXPRESS

Client:	R.W. Gillespie & Associates				
Project:	Eastern Trail -Pan Am Crossing				
Location:	Scarborough, ME			Project No:	GTX-305568
Boring ID:	---	Sample Type:	---	Tested By:	jbr
Sample ID:	---	Test Date:	11/10/16	Checked By:	emm
Depth :	---	Test Id:	397501		

pH of Soil by ASTM D4972

Boring ID	Sample ID	Depth	Visual Description	pH of Soil in Distilled Water	pH of Soil in Calcium Chloride
BB-SPC-201	S-2 / S-3	5-12	Moist, brown sand with silt	6.7	5.9
EB-SPC-202	S-2 / S-3	5-12	Moist, brown sand with silt	5.1	4.4

Notes: Sample Preparation: screened through #10 sieve

Method B, pH paper used

GeoTesting EXPRESS

Client:	R.W. Gillespie & Associates
Project:	Eastern Trail -Pan Am Crossing
Location:	Scarborough, ME
GTX#:	305568
Test Date:	11/10/16
Tested By:	jbr
Checked By:	emm

Laboratory Measurement of Soil Resistivity Using the Wenner Four-Electrode Method by ASTM G57 (Laboratory Measurement)

Boring ID	Sample ID	Depth, ft.	Sample Description	Electrical Resistivity, ohm-cm	Electrical Conductivity, $(ohm\text{-}cm)^{-1}$
BB-SPC-201	S-2/S-3	5-12	Moist, brown sand with silt	8,264	1.21E-04
BB-SPC-202	S-2/S-3	5-12	Moist, brown sand with silt	2,583	3.87E-04

Notes: Test Equipment: Nilsson Model 400 Soil Resistance Meter, MC Miller Soil Box
Water added to sample to create a thick slurry prior to testing (saturated condition).
Electrical Conductivity is calculated as inverse of Electrical Resistivity (per ASTM G57)
Test conducted in standard laboratory atmosphere: 68-73 F

FUGRO CONSULTANTS, INC.

6100 HILLCROFT
PHONE (713) 369-5400

HOUSTON, TEXAS 77081
FAX (713) 369-5518

RESULTS OF TESTS

PROJECT: EASTERN TRAIL-PAN AM CROSSING (GTX 305568)
SAMPLE ID: EB-SPC-201, S-2//S-3, 5- 12'

FOR: GEOTESTING EXPRESS, INC.
125 NAGOG PARK ACTION, MA 01720

REPORTED TO: ETHAN MARRO

LAB NUMBER: 1104020

REPORT DATE: 11-08-16
CLIENT NUMBER:
JOB NUMBER: 04.1115-0003
REPORT NUMBER:
DATE SAMPLED:
TIME SAMPLED:
SAMPLED BY: CLIENT
DATE RECEIVED: 11-04-16
TIME RECEIVED: 1100
RECEIVED BY: SD

PARAMETER	RESULTS	UNITS	METHOD	TIME/DATE	ANALYST
Sulfate, Soluble	33 *	mg/kg	AASHTO T 290	1300/11-08-16	SD
Chloride, Soluble	118 *	mg/kg	AASHTO T 291	1400/11-08-16	SD

SO4CL 109-16

Respectfully submitted,

* Dry weight basis

Steve DeGregorio
Chemist

SD

THE RESULTS RELATE AS TO THE LOCATION TESTED AND NO OTHER REFERENCE SHALL BE MADE.
THIS REPORT SHALL NOT BE REPRODUCED EXCEPT IN FULL WITHOUT THE WRITTEN APPROVAL OF THE LABORATORY.

END OF REPORT

FUGRO CONSULTANTS, INC.

6100 HILLCROFT
PHONE (713) 369-5400

HOUSTON, TEXAS 77081
FAX (713) 369-5518

RESULTS OF TESTS

PROJECT:	EASTERN TRAIL-PAN AM CROSSING (GTX 305568) SAMPLE ID: EB-SPC-202, S-2//S-3, 5- 12'	REPORT DATE:	11-08-16
		CLIENT NUMBER:	
		JOB NUMBER:	04.1115-0003
FOR:	GEOTESTING EXPRESS, INC. 125 NAGOG PARK ACTION, MA 01720	REPORT NUMBER:	
		DATE SAMPLED:	
		TIME SAMPLED:	
REPORTED TO:	ETHAN MARRO	SAMPLED BY:	CLIENT
		DATE RECEIVED:	11-04-16
		TIME RECEIVED:	1100
LAB NUMBER:	1104019	RECEIVED BY:	SD

PARAMETER	RESULTS	UNITS	METHOD	TIME/DATE	ANALYST
Sulfate, Soluble	< 30 *	mg/kg	AASHTO T 290	1300/11-08-16	SD
Chloride, Soluble	< 30 *	mg/kg	AASHTO T 291	1400/11-08-16	SD

SO4CL 109-16

Respectfully submitted,

* Dry weight basis

Steve DeGregorio
Chemist

SD

THE RESULTS RELATE AS TO THE LOCATION TESTED AND NO OTHER REFERENCE SHALL BE MADE.
THIS REPORT SHALL NOT BE REPRODUCED EXCEPT IN FULL WITHOUT THE WRITTEN APPROVAL OF THE LABORATORY.

END OF REPORT

GeoTesting EXPRESS

Client:	R.W. Gillespie & Associates				
Project:	Eastern Trail -Pan Am Crossing				
Location:	Scarborough, ME			Project No:	GTX-305568
Boring ID:	BB-SPC-203	Sample Type:	cylinder	Tested By:	daa/rlc
Sample ID:	R-3	Test Date:	11/09/16	Checked By:	jsc
Depth :	149.5-149.9	Test Id:	397509		
Test Comment:	---				
Visual Description:	See photograph(s)				
Sample Comment:	---				

Bulk Density and Compressive Strength of Rock Core Specimens by ASTM D7012 Method C

Boring ID	Sample Number	Depth	Bulk Density, pcf	Compressive strength, psi	Failure Type	Meets ASTM D4543	Note(s)
BB-SPC-203	R-3	149.50-149.90 ft	167	505	2	No	1 , 2 , 3,*

Notes: Density determined on core samples by measuring dimensions and weight and then calculating.

All specimens tested at the approximate as-received moisture content and at standard laboratory temperature.

The axial load was applied continuously at a stress rate that produced failure in a test time between 2 and 15 minutes.

Failure Type: 1 = Intact Material Failure; 2 = Discontinuity Failure; 3 = Intact Material and Discontinuity Failure (See attached photographs)

1: Best effort end preparation. See Tolerance report for details.
2: The as-received core did not meet the ASTM side straightness tolerance due to irregularities in the sample as cored.
3: Specimen L/D < 2.
4: The as-received core did not meet the ASTM minimum diameter tolerance of 1.875 inches.
5: Specimen diameter is less than 10 times maximum particle size.
6: Specimen diameter is less than 6 times maximum particle size.

*Because the indicated tested specimens did not meet the ASTM D4543 standard tolerances, the results reported here may differ from those for a test specimen within tolerances.

GeoTesting EXPRESS

Client:	R.W. Gillespie & Associates	Test Date:	11/8/2016
Project Name:	Eastern Trail-Pan Am Crossing	Tested By:	daa/rlc
Project Location:	Scarborough, ME	Checked By:	jsc
GTX #:	305568		
Boring ID:	BB-SPC-203		
Sample ID:	R-3		
Depth:	149.5-149.9 ft		
Visual Description:	See photographs		

UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D4543

BULK DENSITY

	1	2	Average
Specimen Length, in:	2.88	2.88	2.88
Specimen Diameter, in:	1.99	2.00	2.00
Specimen Mass, g:	396.51		
Bulk Density, lb/ft³	167	**Minimum Diameter Tolerence Met?**	YES
Length to Diameter Ratio:	1.4	**Length to Diameter Ratio Tolerance Met?**	NO

DEVIATION FROM STRAIGHTNESS (Procedure S1)

Maximum gap between side of core and reference surface plate:
Is the maximum gap ≤ 0.02 in.? NO

Maximum difference must be < 0.020 in.
Straightness Tolerance Met? NO

END FLATNESS AND PARALLELISM (Procedure FP1)

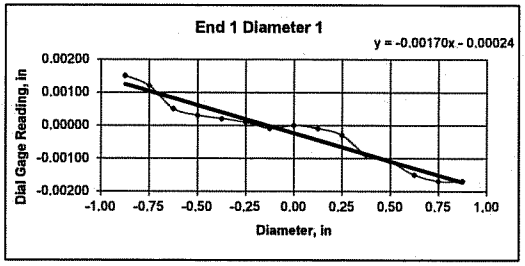
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00150	0.00120	0.00050	0.00030	0.00020	0.00010	-0.00010	0.00000	-0.00010	-0.00030	-0.00090	-0.00110	-0.00150	-0.00170	-0.00170
Diameter 2, in (rotated 90°)	-0.00130	-0.00130	-0.00110	-0.00100	-0.00070	-0.00040	0.00000	0.00000	0.00000	0.00000	0.00000	0.00010	0.00020	0.00020	0.00030

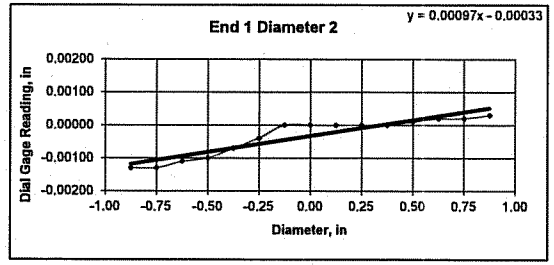
Difference between max and min readings, in:
0° = 0.00320 90° = 0.00160

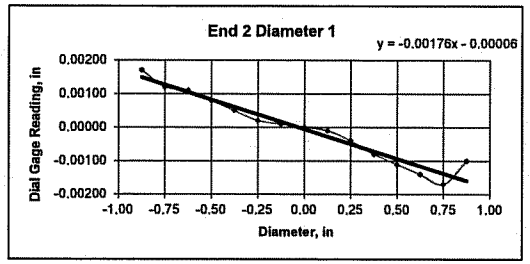
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00170	0.00120	0.00110	0.00080	0.00050	0.00020	0.00010	0.00000	-0.00010	-0.00040	-0.00080	-0.00110	-0.00140	-0.00170	-0.00100
Diameter 2, in (rotated 90°)	-0.00130	-0.00130	-0.00120	-0.00080	-0.00040	-0.00040	-0.00010	0.00000	0.00000	0.00000	0.00010	0.00050	0.00060	0.00050	-0.00030

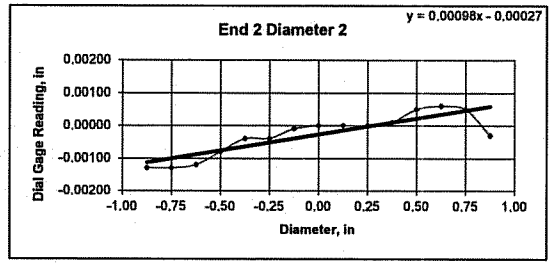
Difference between max and min readings, in:
0° = 0.0034 90° = 0.0019

Maximum difference must be < 0.0020 in. Difference = ± 0.00170
Flatness Tolerance Met? NO









DIAMETER 1

End 1:
Slope of Best Fit Line -0.00170
Angle of Best Fit Line: -0.09740

End 2:
Slope of Best Fit Line -0.00176
Angle of Best Fit Line: -0.10084

Maximum Angular Difference: 0.00344

Parallelism Tolerance Met? YES
Spherically Seated

DIAMETER 2

End 1:
Slope of Best Fit Line 0.00097
Angle of Best Fit Line: 0.05558

End 2:
Slope of Best Fit Line 0.00098
Angle of Best Fit Line: 0.05615

Maximum Angular Difference: 0.00057

Parallelism Tolerance Met? YES
Spherically Seated

PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00320	1.995	0.00160	0.092	YES
Diameter 2, in (rotated 90°)	0.00160	1.995	0.00080	0.046	YES
END 2					
Diameter 1, in	0.00340	1.995	0.00170	0.098	YES
Diameter 2, in (rotated 90°)	0.00190	1.995	0.00095	0.055	YES

Maximum angle of departure must be ≤ 0.25°

Perpendicularity Tolerance Met? YES

GeoTesting EXPRESS

Client:	R.W. Gillespie & Associates	Test Date:	11/9/2016
Project Name:	Eastern Trail-Pan Am Crossing	Tested By:	daa/rlc
Project Location:	Scarborough, ME	Checked By:	jsc
GTX #:	305568		
Boring ID:	BB-SPC-203	Tolerance measurements were performed using a machinist straightedge and feeler gauges to ASTM specifications.	
Sample ID:	R-3		
Depth:	149.5-149.9 ft		
Visual Description:	See photographs		

BEST EFFORT END FLATNESS TOLERANCES OF ROCK CORE SPECIMENS TO ASTM D4543

END FLATNESS

END 1

Diameter 1	Is the maximum gap $\leq \pm 0.001$ in.?	YES
Diameter 2 (rotated 90°)	Is the maximum gap $\leq \pm 0.001$ in.?	YES

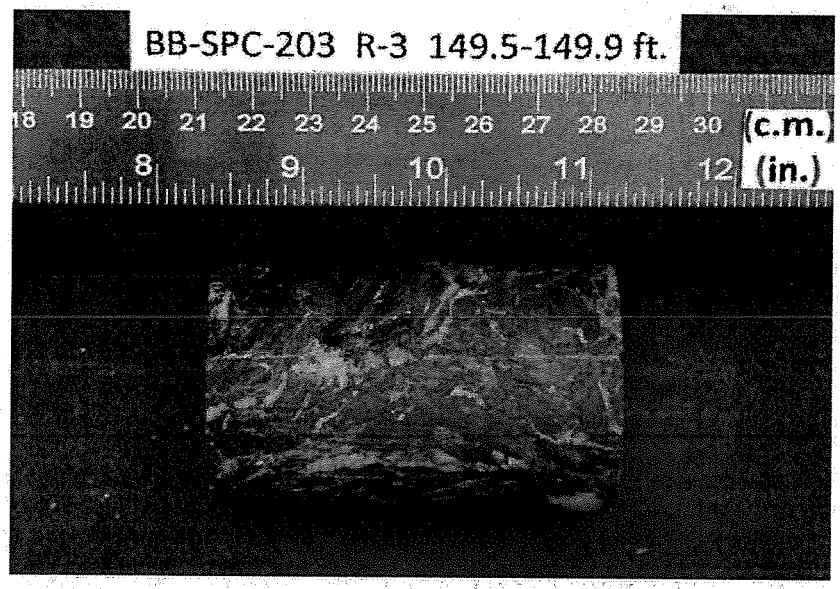
END 2

Diameter 1	Is the maximum gap $\leq \pm 0.001$ in.?	YES
Diameter 2 (rotated 90°)	Is the maximum gap $\leq \pm 0.001$ in.?	YES

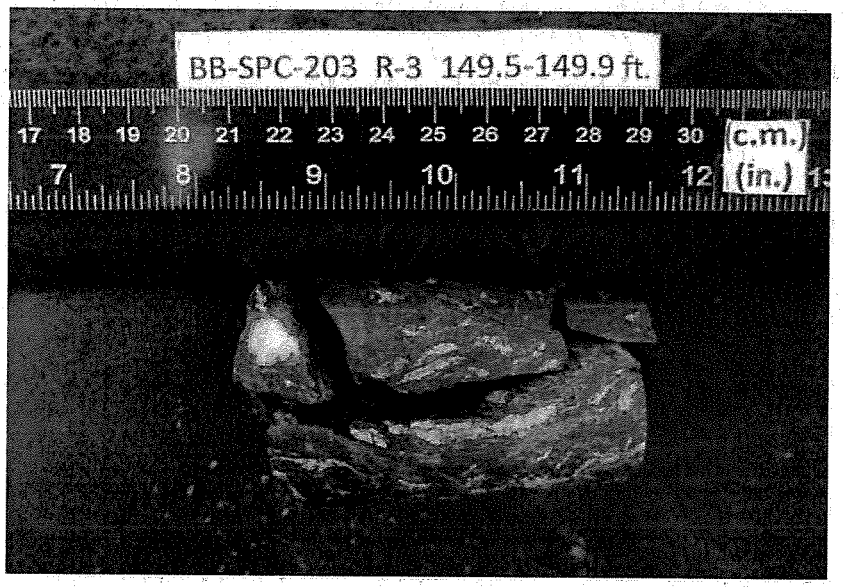
End Flatness Tolerance Met? YES

GeoTesting EXPRESS

Client:	R.W. Gillespie & Associates
Project Name:	Eastern Trail-Pan Am Crossing
Project Location:	Scarborough, ME
GTX #:	305568
Test Date:	11/9/2016
Tested By:	daa/rlc
Checked By:	jsc
Boring ID:	BB-SPC-203
Sample ID:	R-3
Depth, ft:	149.5-149.9



After cutting and grinding



After break

Unconfined Compressive Strength by ASTM 7012, Method C

Standard Test Methods for Compressive Strength of Elastic Moduli of Intact Rock Core Specimens under Varying States of Stress and Temperatures

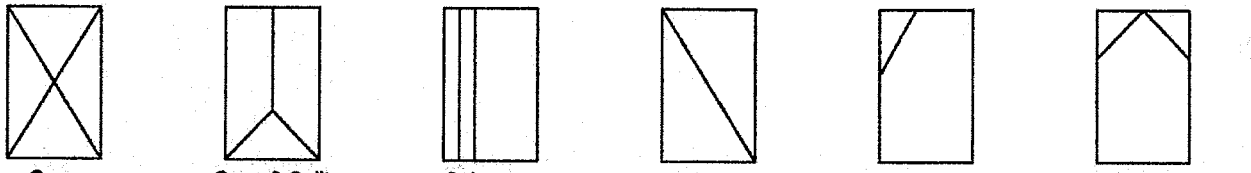
Project Name:	Scarborough Eastern Trail Project	Lab No.:	15212
Client:	HNTB Corporation	Test Date:	8/27/2018
Project No.:	0600-102		
Project Location:	Scarborough, ME		

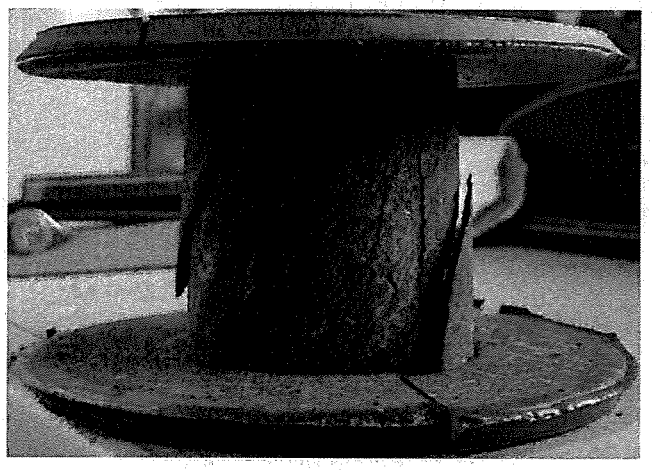
Boring ID	Sample ID	Depth (ft)	Compressive Strength (psi)	Failure Type	In conformance with ASTM D 4543
BB-SPC-201A	R-3	132.1	6,010	3	No

Note: Samples did not meet Dimensional and Shape Tolerances (ASTM D 4543) and were capped before testing.

Boring ID	Sample ID	Length (in.)	Diameter (in.)	L/D
BB-SPC-201A	R-3	2.3413	1.9804	1.18

Failure Types





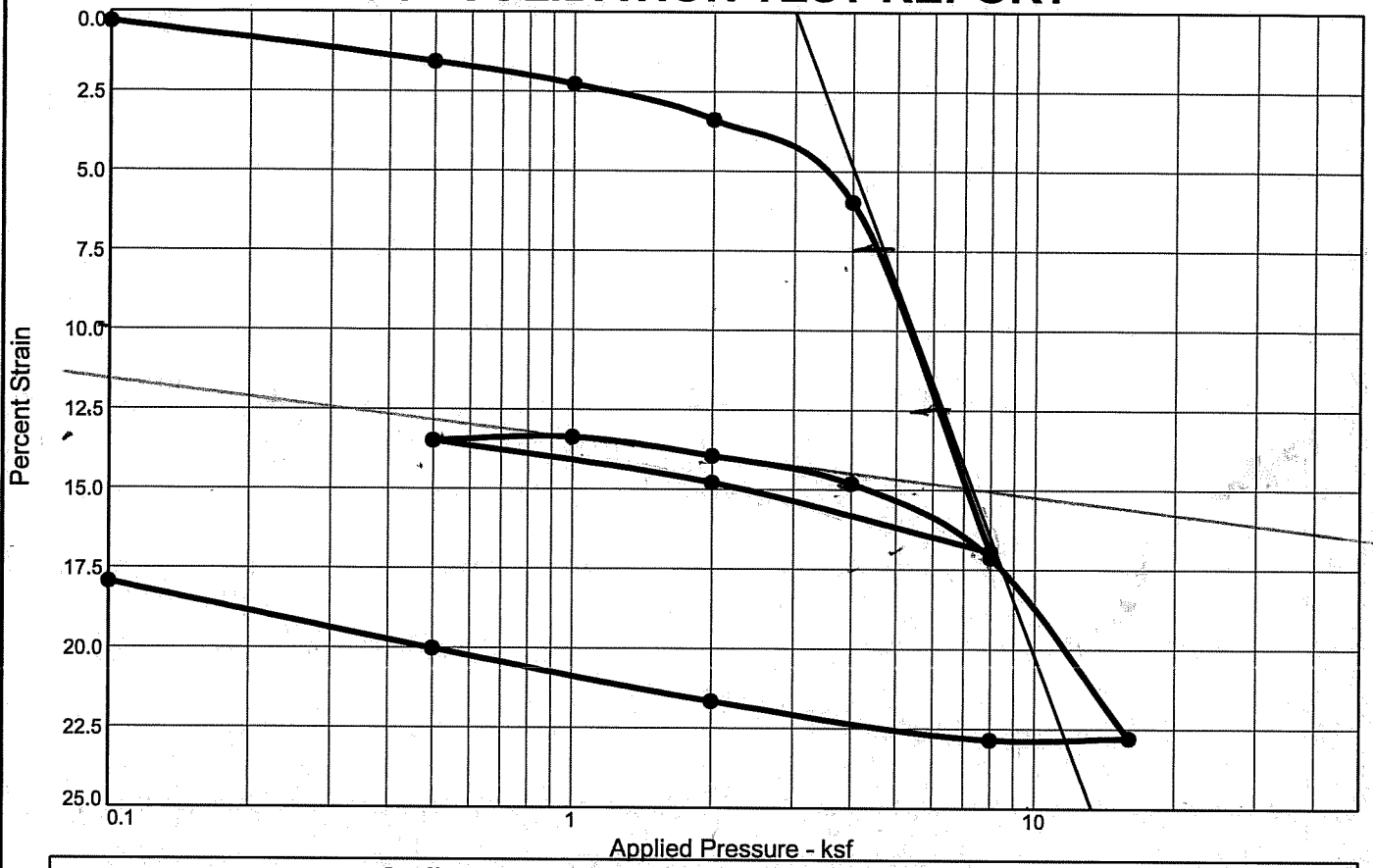
Tested By: JRF

Checked By: EJW

R.W. Gillespie & Associates

20 Pomerleau Street, Suite 100, Biddeford ME 04005, 207-286-8008 / 177 Shattuck Way, Suite 1 West, Newington NH 03801, 603-427-0244

CONSOLIDATION TEST REPORT



Coefficients of Consolidation and Secondary Consolidation

No.	Load (ksf)	C_v (ft.2/day)	C_α	No.	Load (ksf)	C_v (ft.2/day)	C_α	No.	Load (ksf)	C_v (ft.2/day)	C_α
1	0.10	0.482		8	0.50	0.117		15	2.00	0.239	
2	0.50	0.586		9	1.00	0.229		16	0.50	0.057	
3	1.00	0.585		10	2.00	0.206		17	0.10	0.020	
4	2.00	0.450		11	4.00	0.345					
5	4.00	0.206		12	8.00	0.119					
6	8.00	0.066		13	16.00	0.112					
7	2.00	0.431		14	8.00	0.899					

Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_r	Initial Void Ratio
Saturation	Moisture									
95.8 %	50.2 %	70.5	24.3	4.9	2.76		3.3	0.94	0.15	1.446

MATERIAL DESCRIPTION	USCS	AASHTO

Project No. 600-092 **Client:** HNTB

Project: Scarborough Eastern Trail Project

Source of Sample: BB-SPC-102 **Depth:** 30-32' **Sample Number:** U-1

R.W. Gillespie & Associates, Inc.

Biddeford, Maine

Remarks:

Tested By: JMB

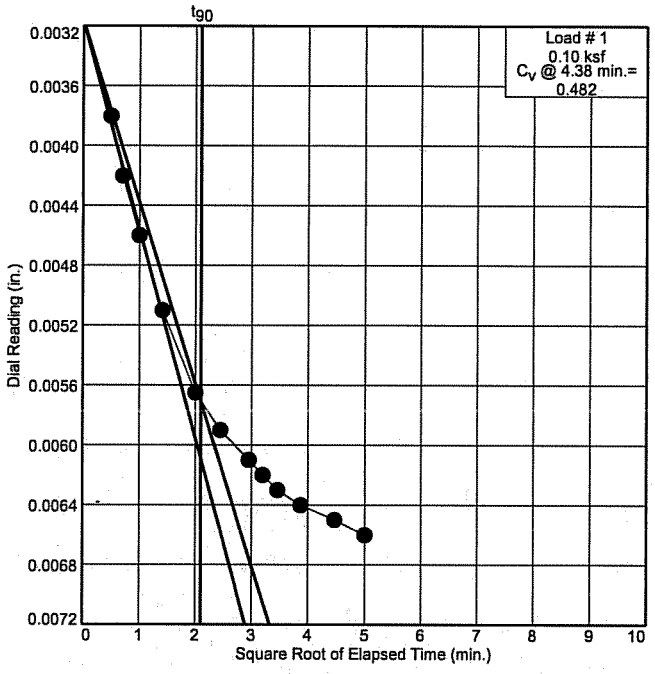
Lab No. 7 6

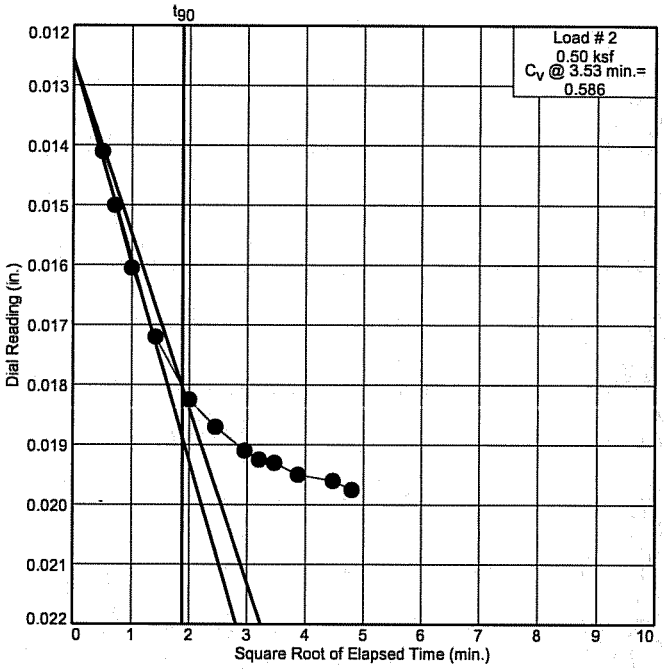
Tested By: _____ **Checked By:** _____

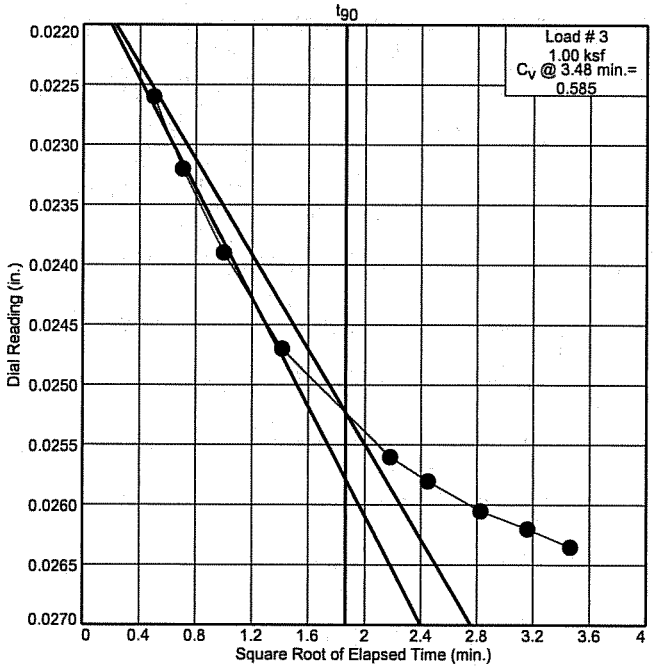
Dial Reading vs. Time

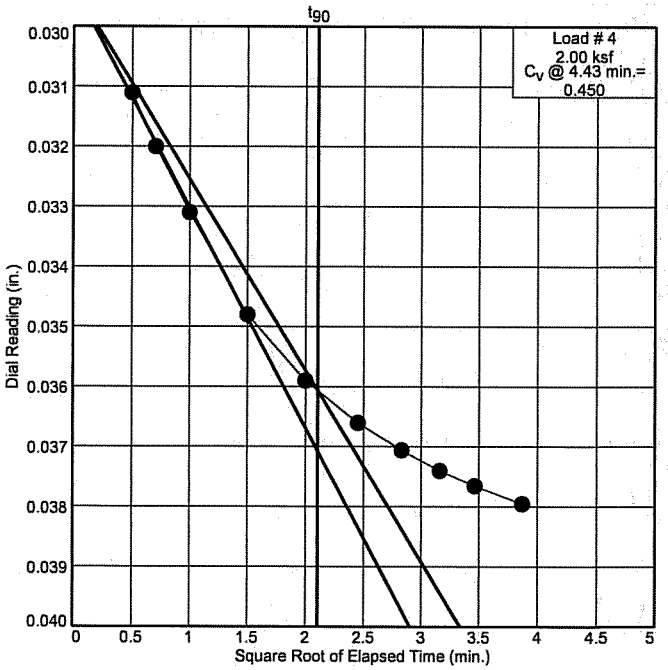
Project No.: 600-092
Project: Scarborough Eastern Trail Project

Source of Sample: BB-SPC-102 Depth: 30-32' Sample Number: U-1









R.W. Gillespie & Associates, Inc.

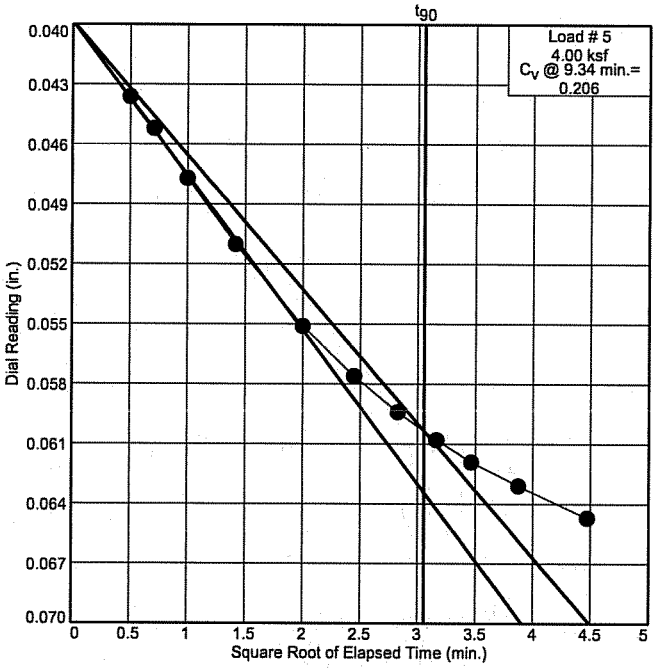
Biddeford, Maine

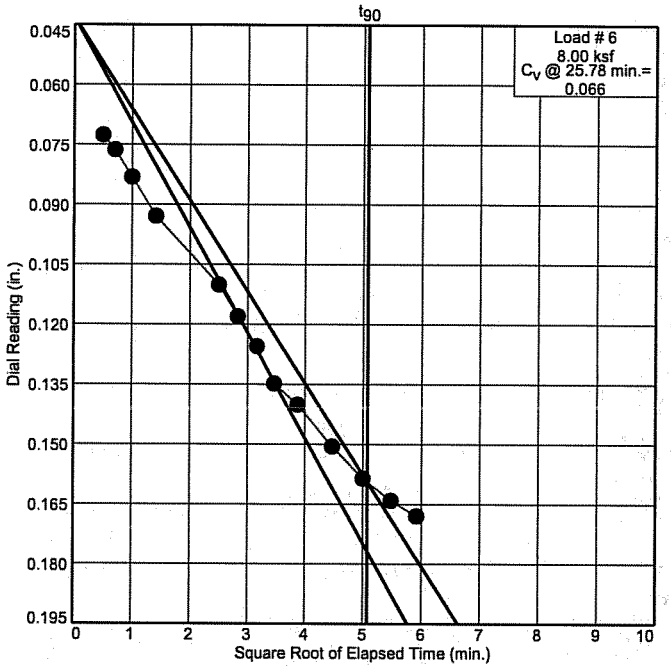
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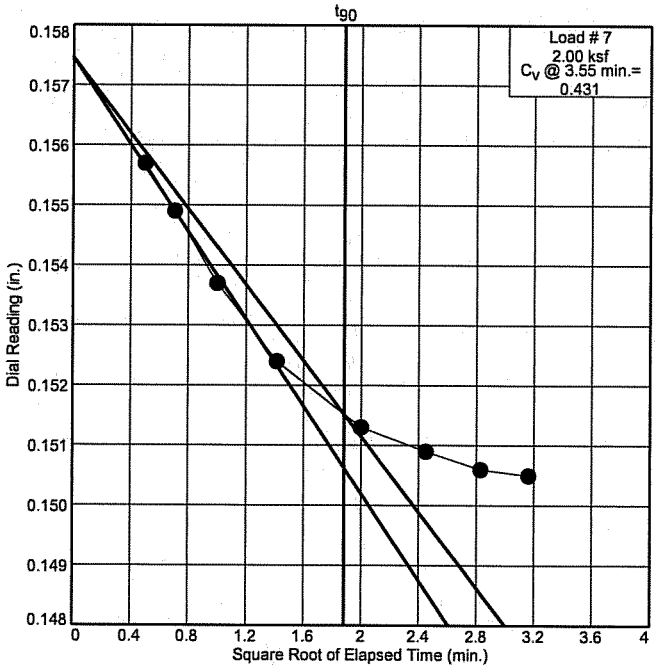
Dial Reading vs. Time

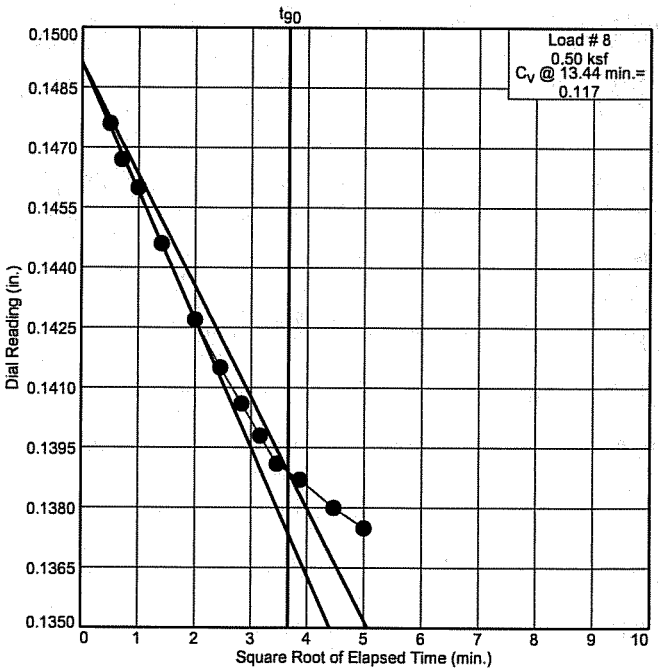
Project No.: 600-092
Project: Scarborough Eastern Trail Project

Source of Sample: BB-SPC-102 Depth: 30-32' Sample Number: U-1









R.W. Gillespie & Associates, Inc.

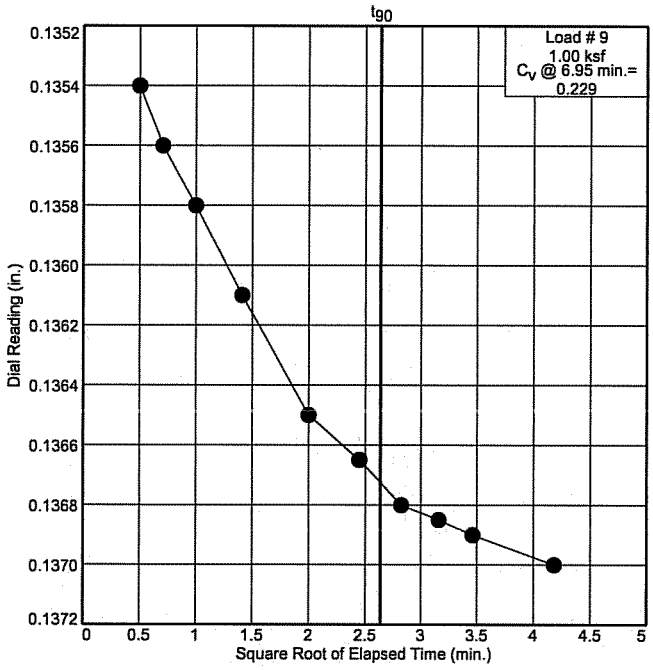
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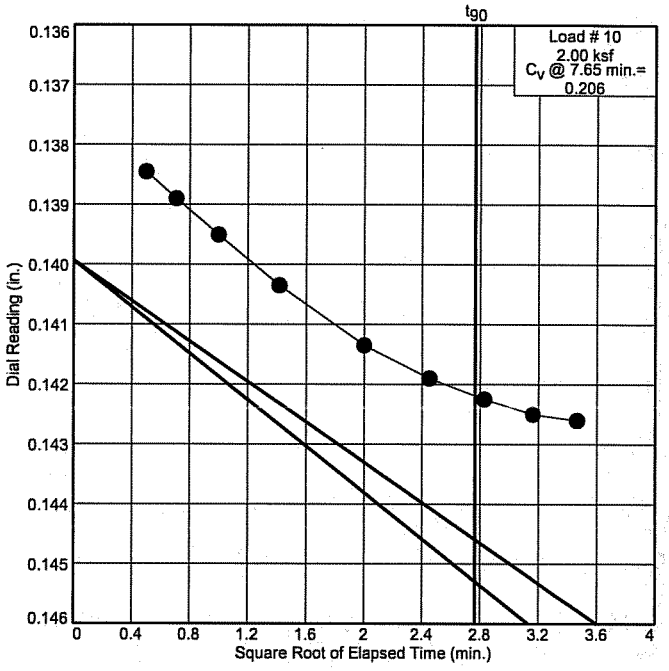
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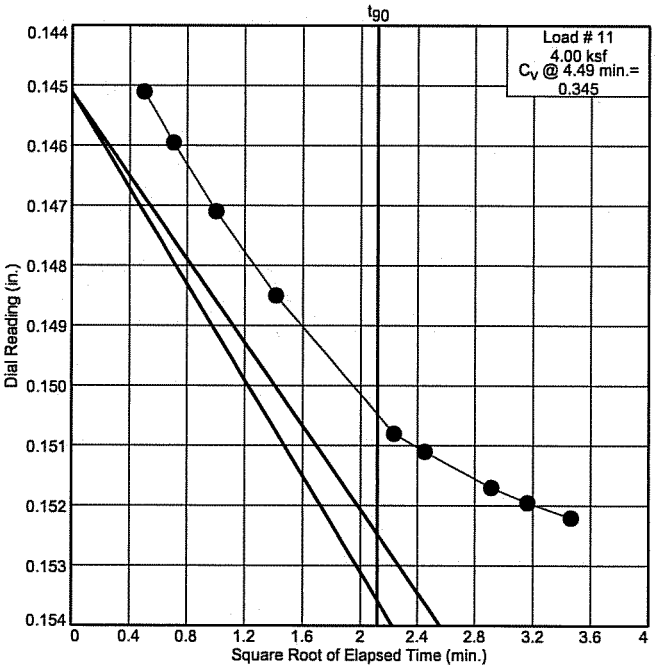
Dial Reading vs. Time

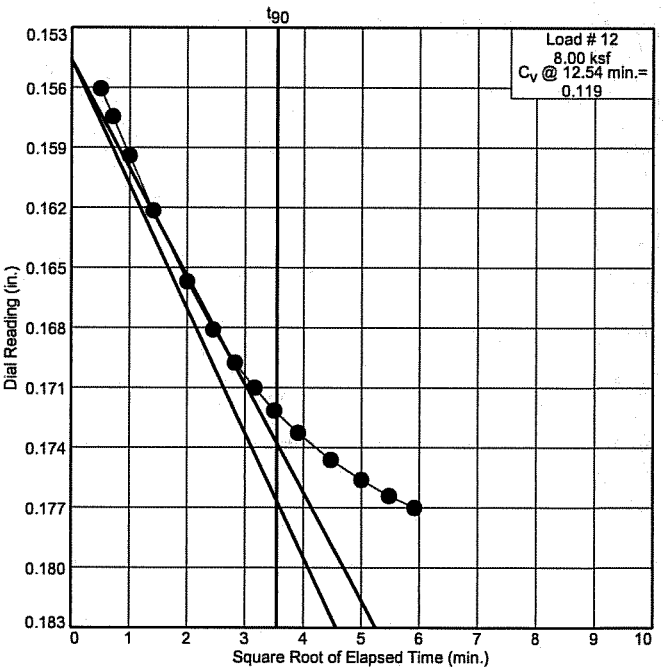
Project No.: 600-092
Project: Scarborough Eastern Trail Project

Source of Sample: BB-SPC-102 Depth: 30-32' Sample Number: U-1









R.W. Gillespie & Associates, Inc.

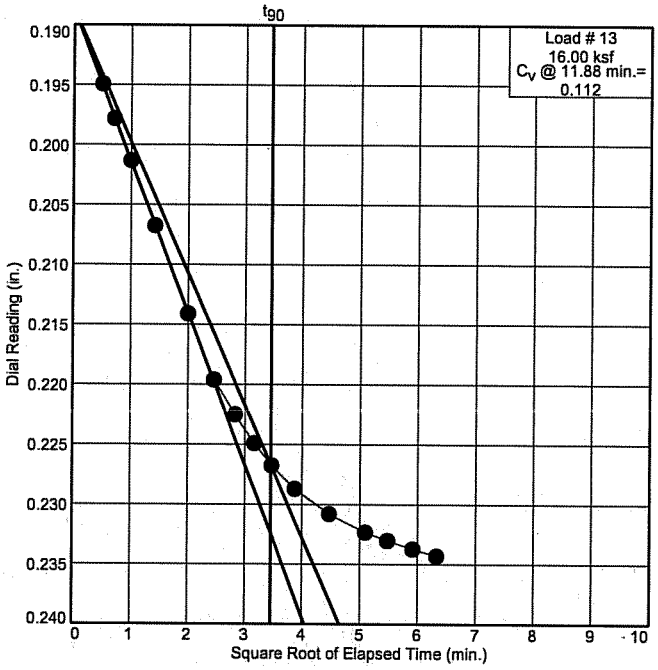
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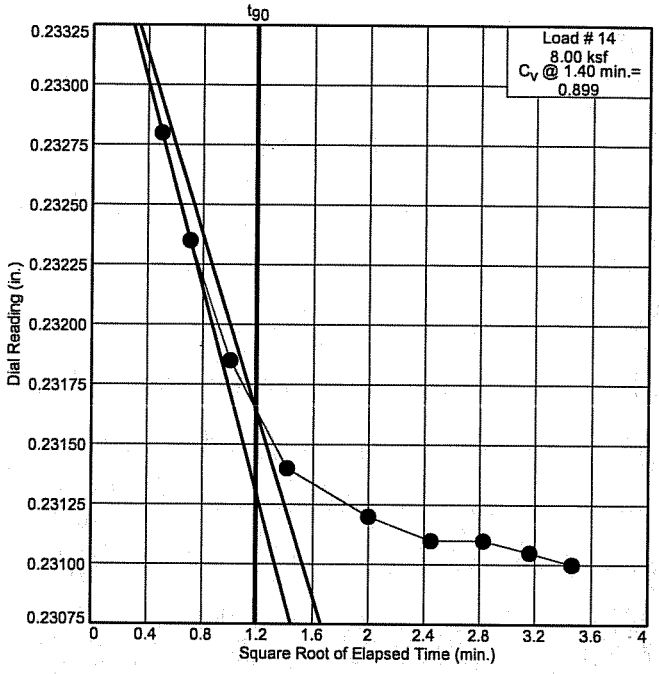
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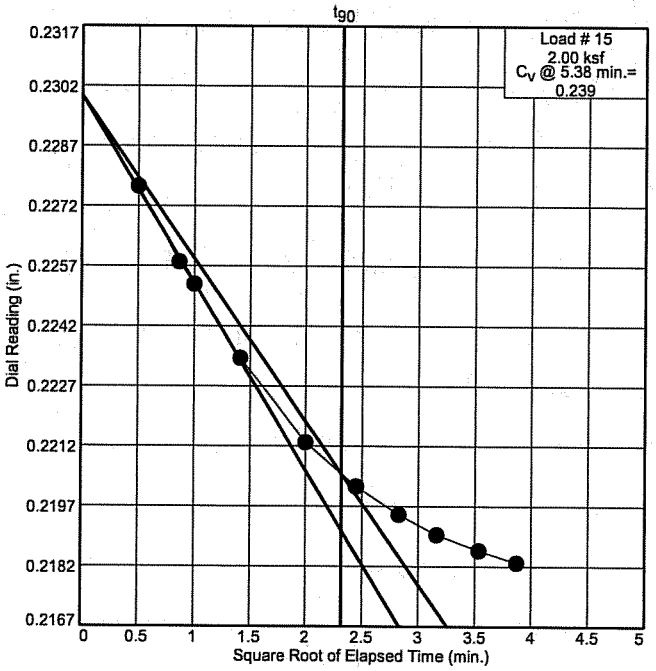
Dial Reading vs. Time

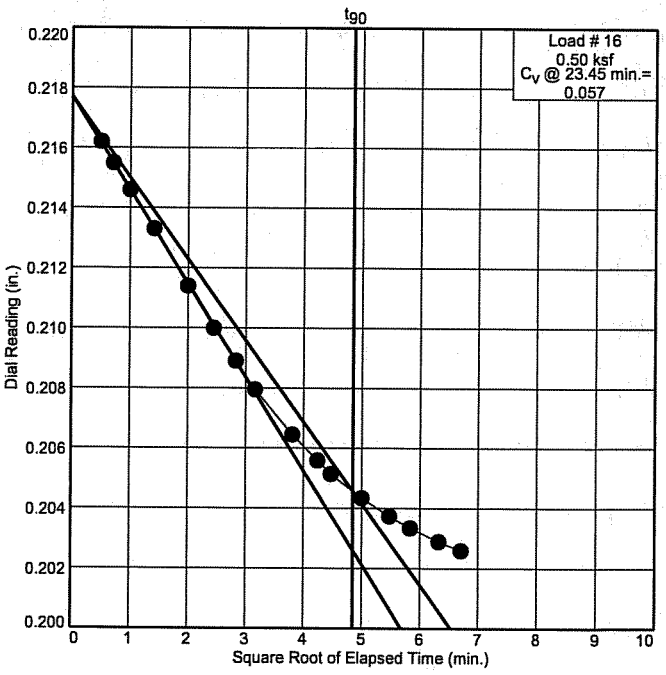
Project No.: 600-092
Project: Scarborough Eastern Trail Project

Source of Sample: BB-SPC-102 Depth: 30-32' Sample Number: U-1









R.W. Gillespie & Associates, Inc.

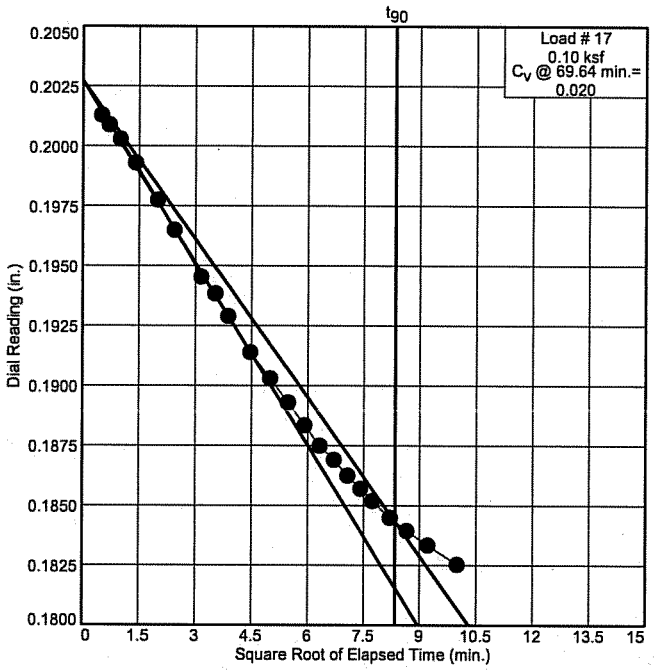
Biddeford, Maine

Lab No.

Dial Reading vs. Time

Project No.: 600-092
Project: Scarborough Eastern Trail Project

Source of Sample: BB-SPC-102 Depth: 30-32' Sample Number: U-1

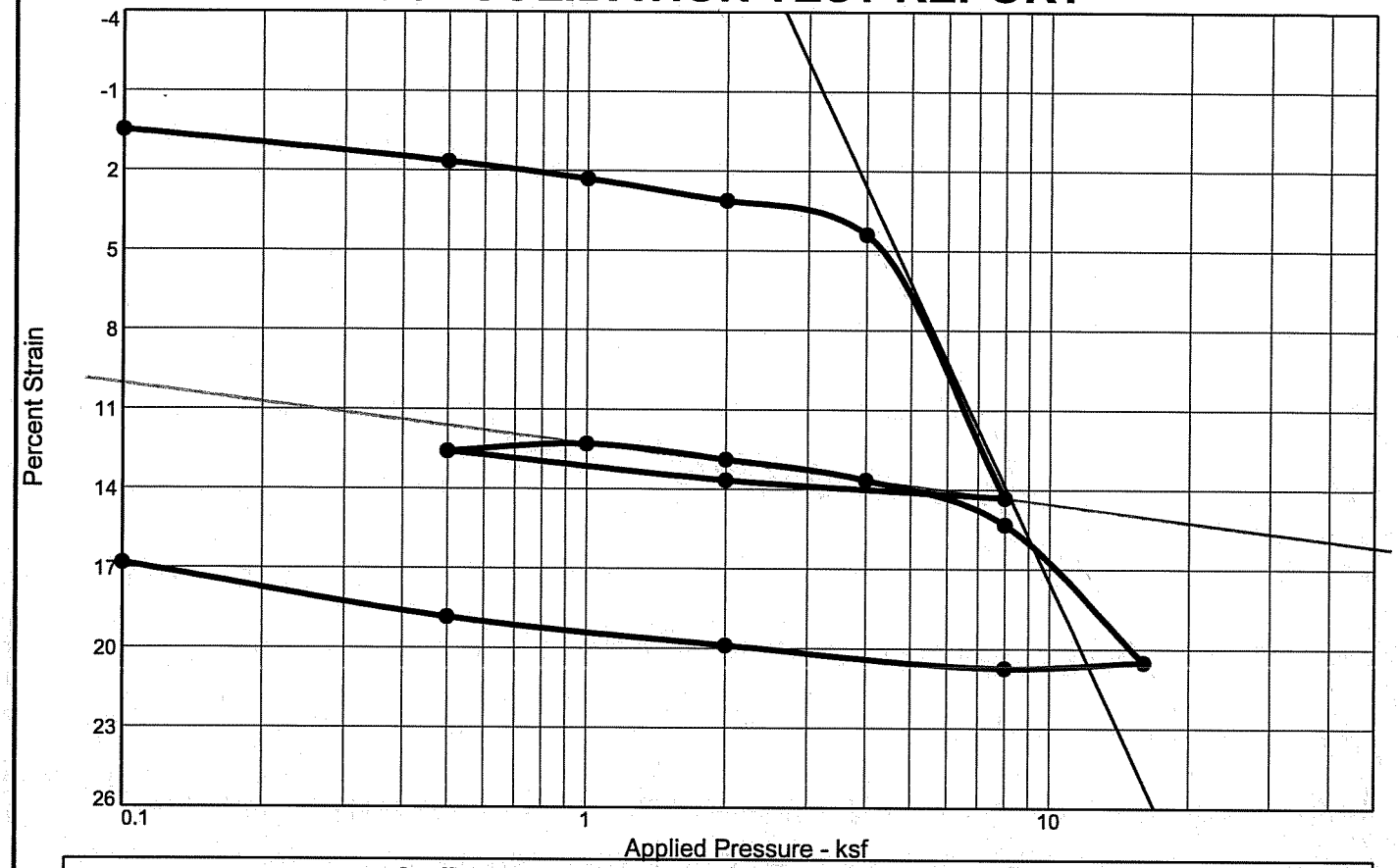


R.W. Gillespie & Associates, Inc.

Biddeford, Maine

Lab No.

CONSOLIDATION TEST REPORT



Coefficients of Consolidation and Secondary Consolidation

No.	Load (ksf)	C_v (ft.2/day)	C_α	No.	Load (ksf)	C_v (ft.2/day)	C_α	No.	Load (ksf)	C_v (ft.2/day)	C_α
1	0.10	1.103		8	0.50	0.069		15	2.00	0.131	
2	0.50	0.455		9	1.00	1.846		16	0.50	0.038	
3	1.00	0.268		10	2.00	0.140		17	0.10	0.005	
4	2.00	0.254		11	4.00	0.169					
5	4.00	0.251		12	8.00	0.113					
6	8.00	0.030		13	16.00	0.075					
7	2.00	0.229		14	8.00	0.409					

Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_r	Initial Void Ratio
Saturation	Moisture									
97.0 %	40.9 %	79.6			2.76		3.8	0.81	0.12	1.164

MATERIAL DESCRIPTION	USCS	AASHTO

Project No. 600-092 **Client:** HNTB

Project: Scarborough Eastern Trail Project

Source of Sample: BB-SPC-102 **Depth:** 50-52' **Sample Number:** U-3

R.W. Gillespie & Associates, Inc.

Biddeford, Maine

Remarks:

Tested By: JMB

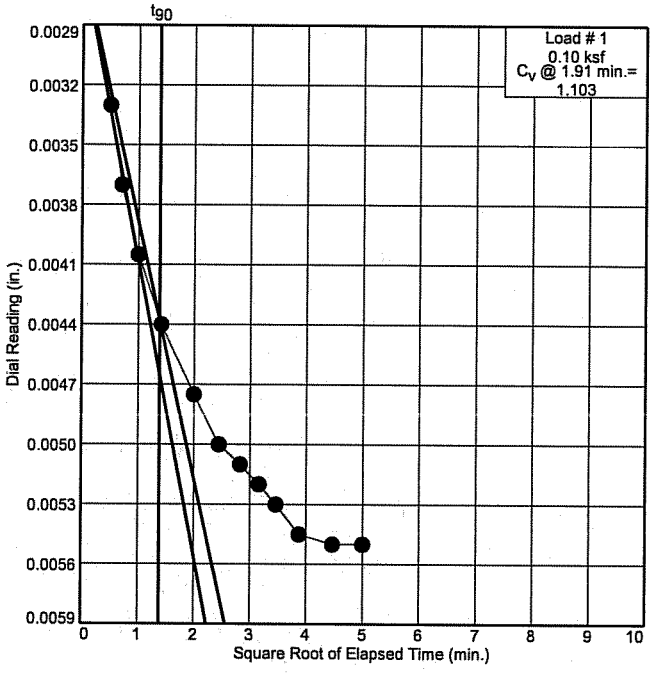
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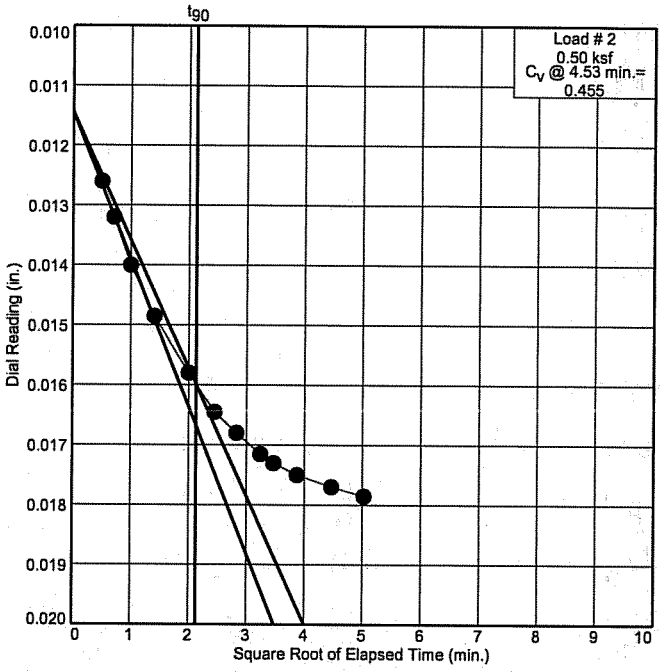
Tested By: _____ **Checked By:** _____

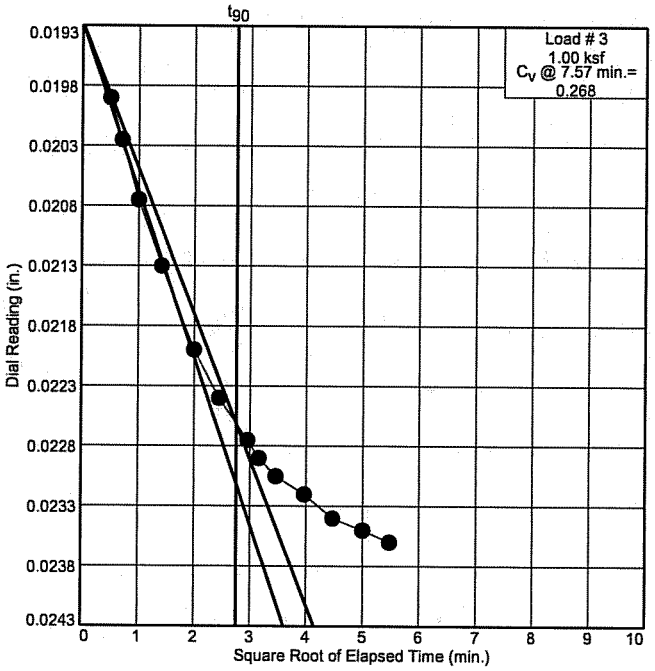
Dial Reading vs. Time

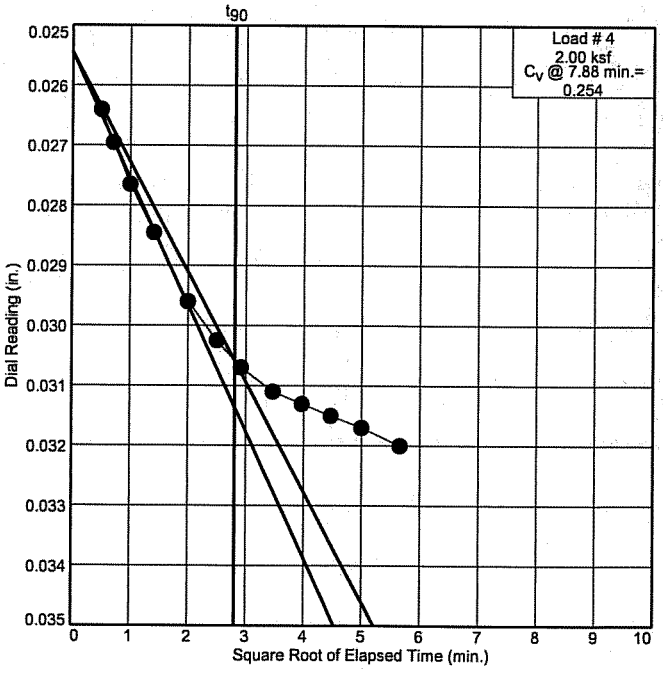
Project No.: 600-092
Project: Scarborough Eastern Trail Project

Source of Sample: BB-SPC-102 Depth: 50-52' Sample Number: U-3









R.W. Gillespie & Associates, Inc.

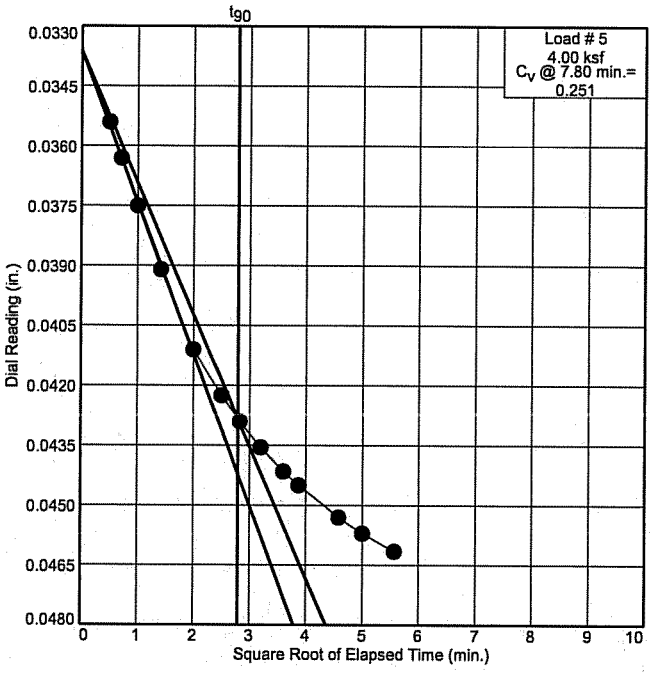
Biddeford, Maine

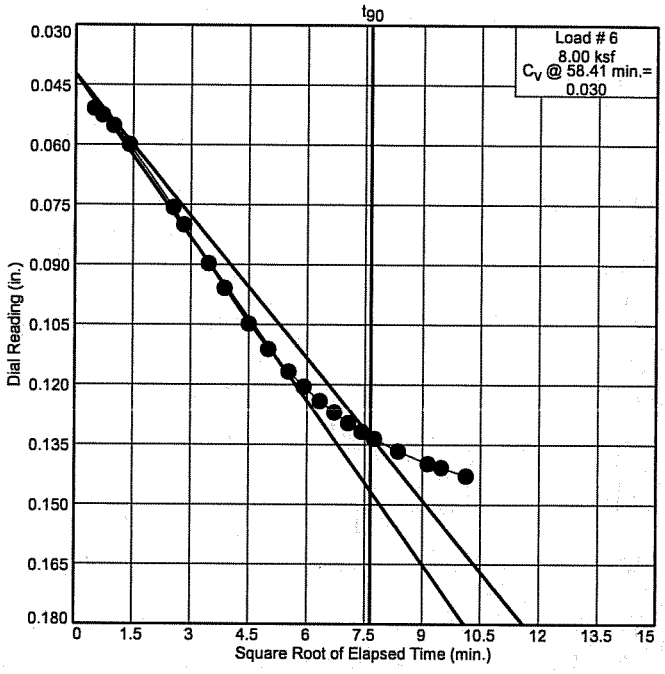
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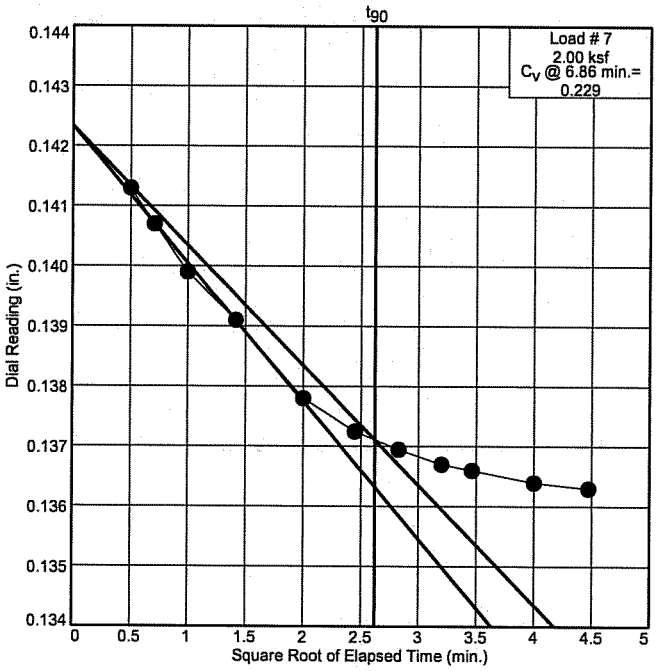
Dial Reading vs. Time

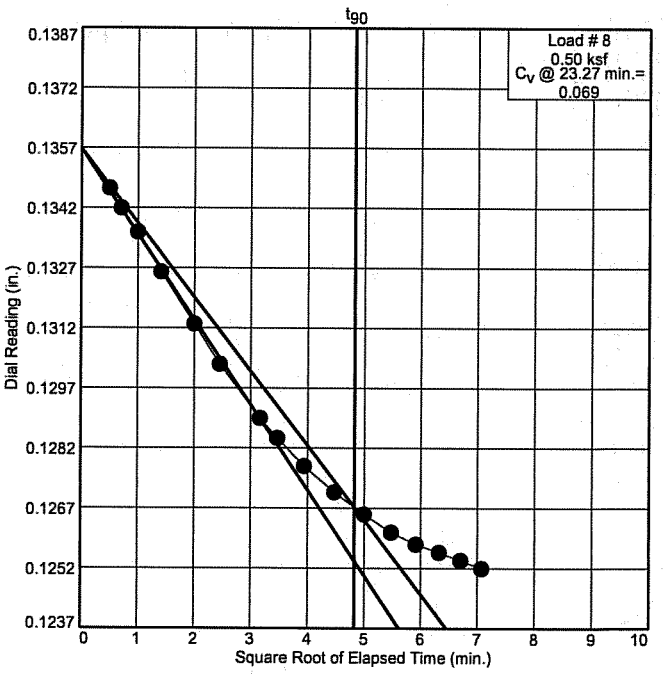
Project No.: 600-092
Project: Scarborough Eastern Trail Project

Source of Sample: BB-SPC-102 Depth: 50-52' Sample Number: U-3









R.W. Gillespie & Associates, Inc.

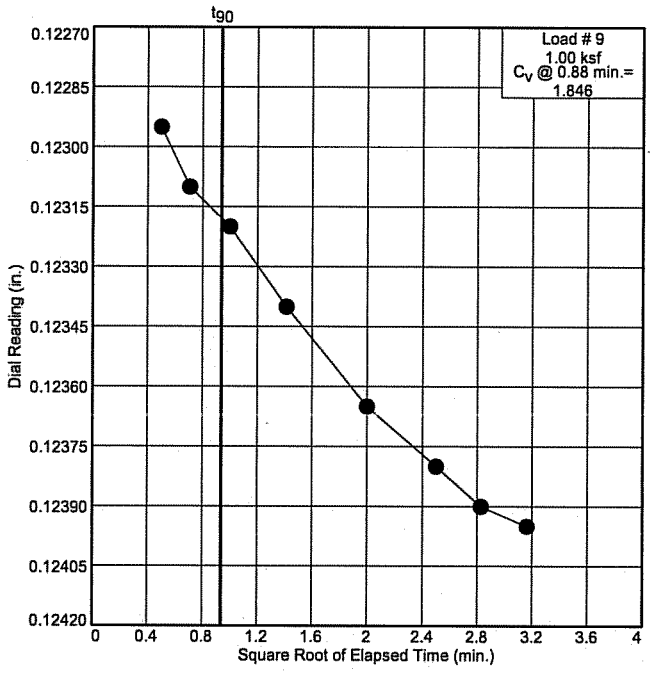
Biddeford, Maine

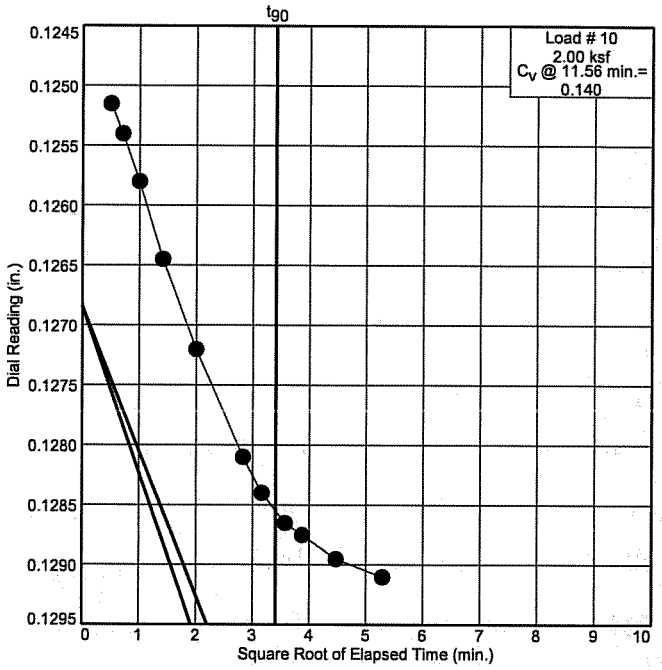
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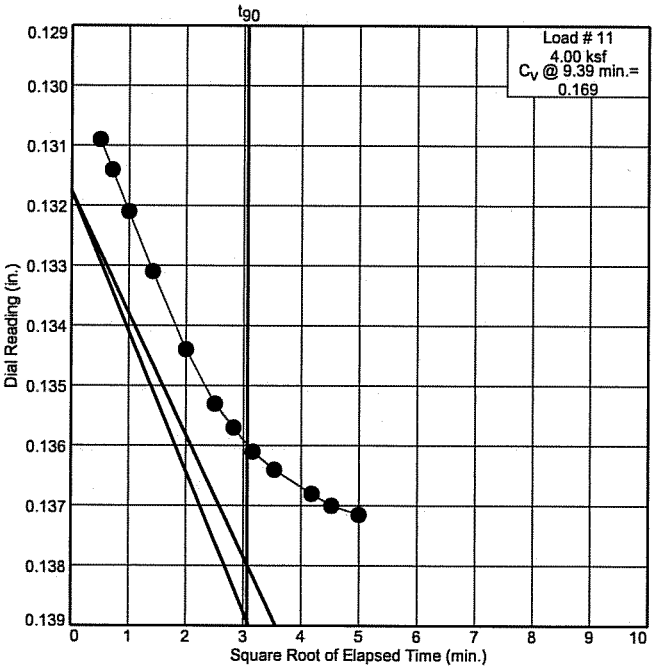
Dial Reading vs. Time

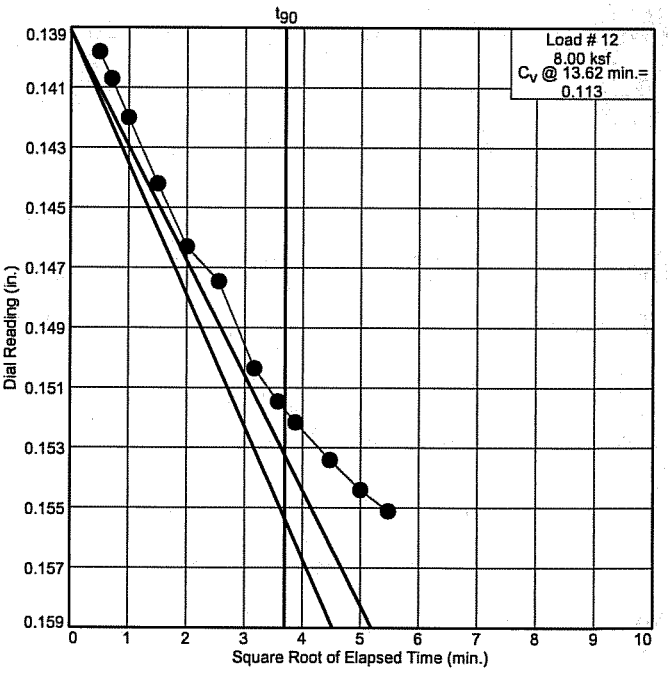
Project No.: 600-092
Project: Scarborough Eastern Trail Project

Source of Sample: BB-SPC-102 Depth: 50-52' Sample Number: U-3









R.W. Gillespie & Associates, Inc.

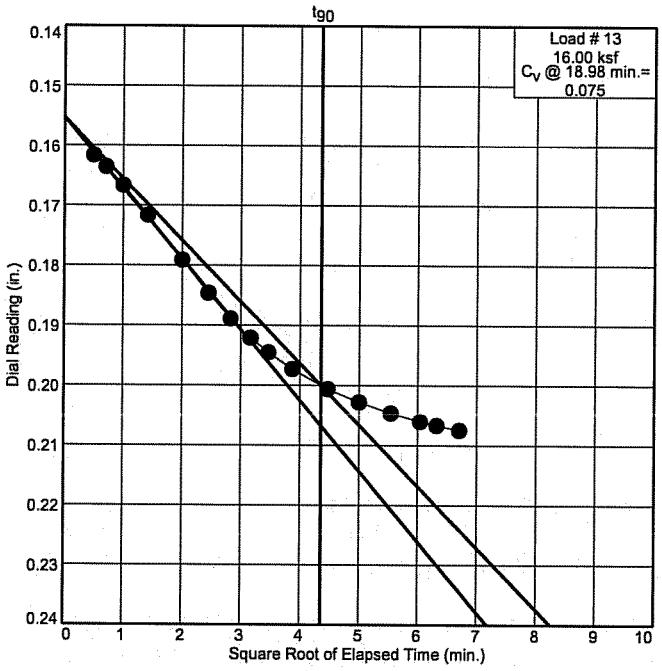
Biddeford, Maine

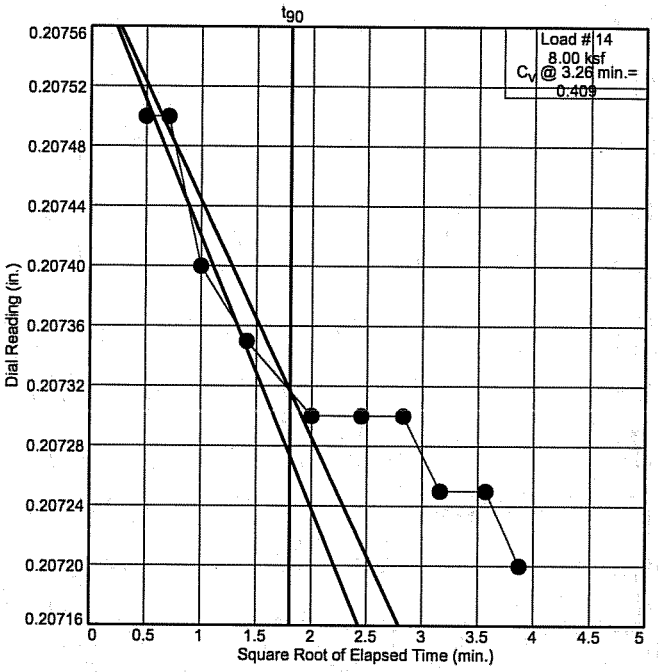
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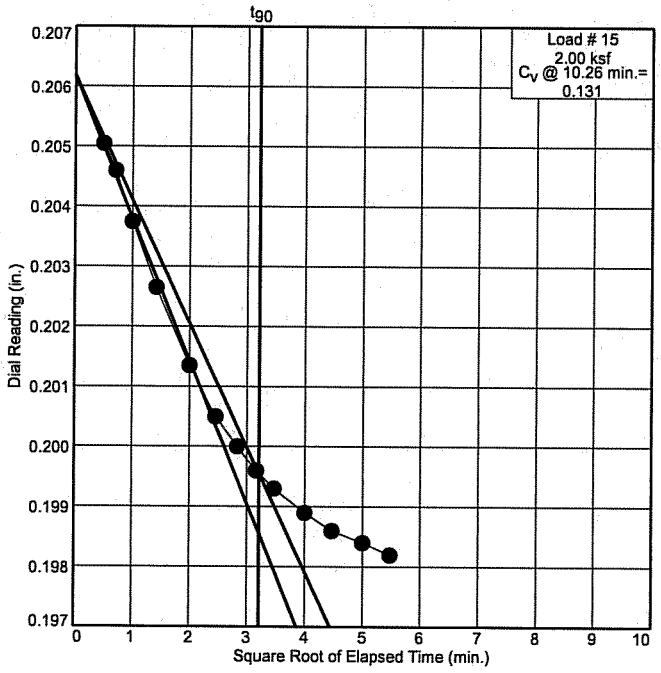
Dial Reading vs. Time

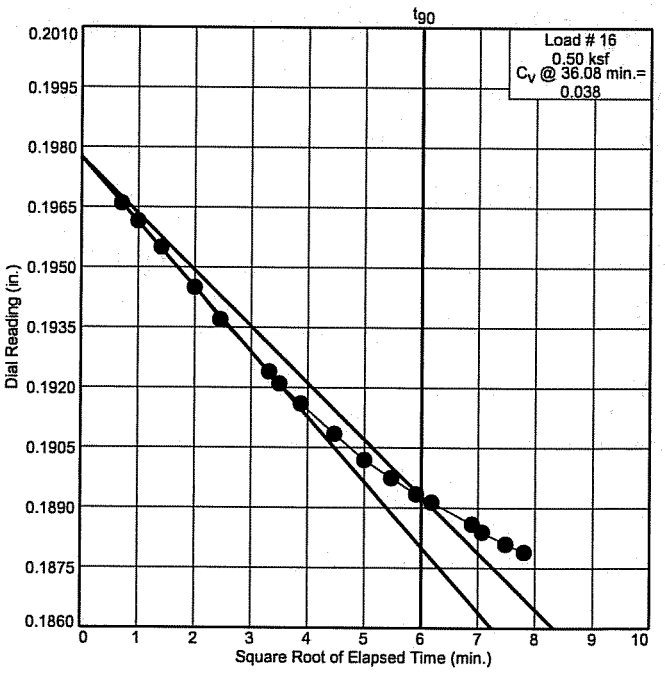
Project No.: 600-092
Project: Scarborough Eastern Trail Project

Source of Sample: BB-SPC-102 Depth: 50-52' Sample Number: U-3









R.W. Gillespie & Associates, Inc.

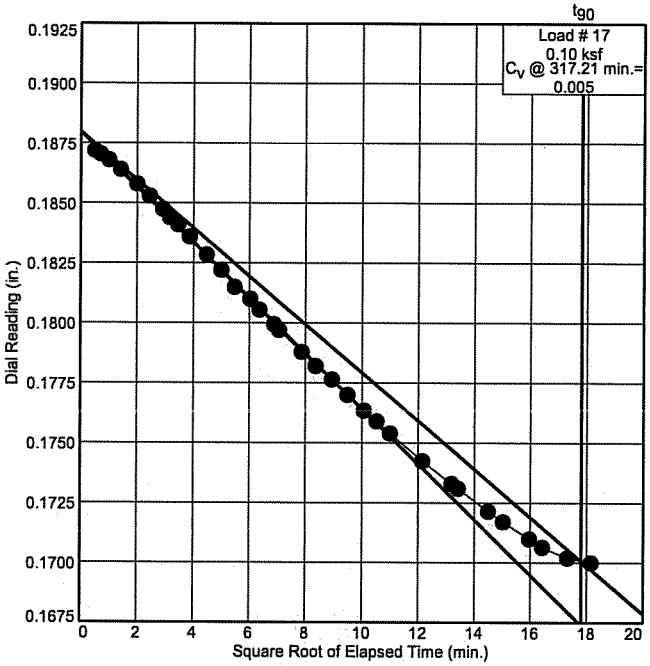
Biddeford, Maine

Lab No.

Dial Reading vs. Time

Project No.: 600-092
Project: Scarborough Eastern Trail Project

Source of Sample: BB-SPC-102 Depth: 50-52' Sample Number: U-3

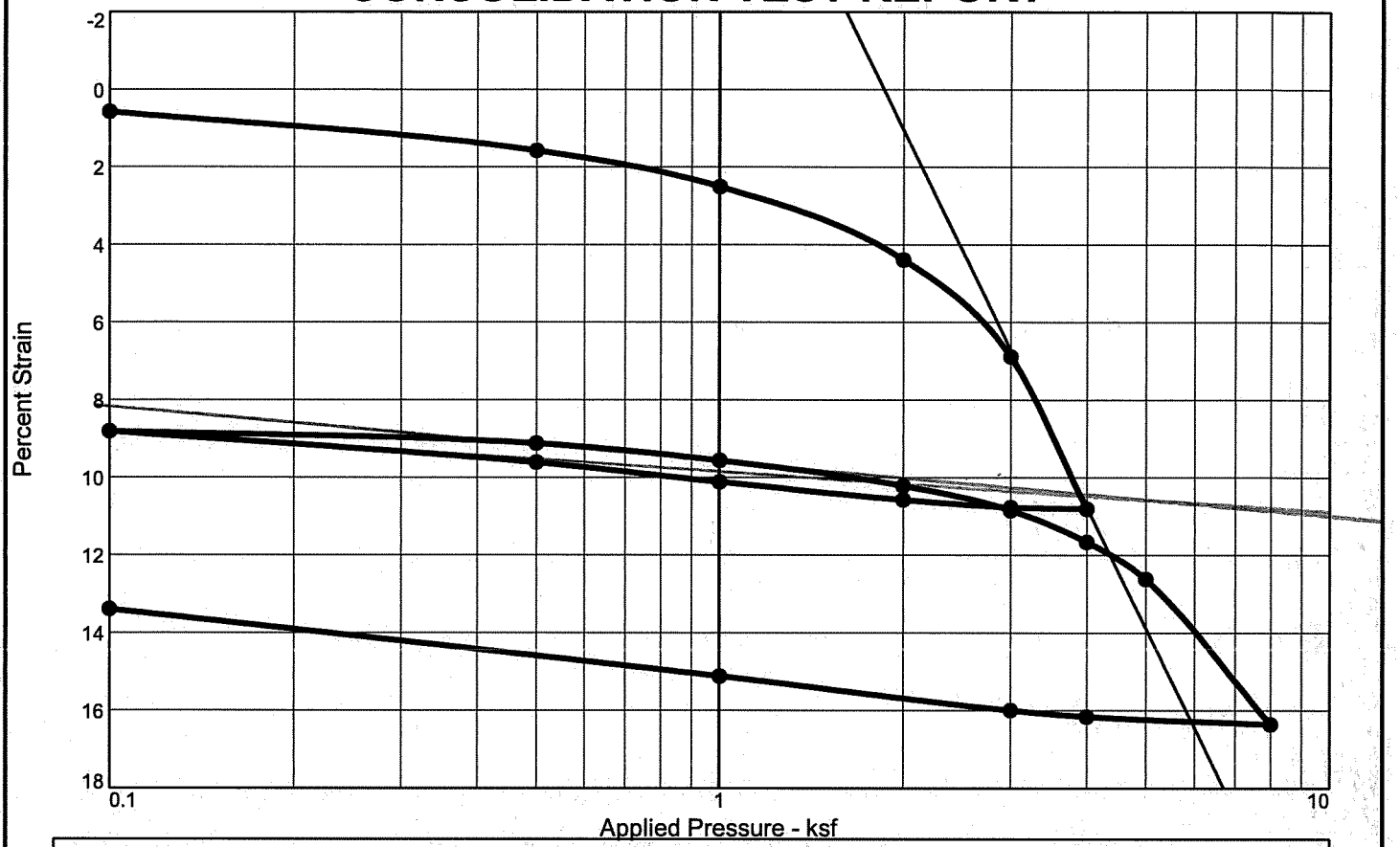


R.W. Gillespie & Associates, Inc.

Biddeford, Maine

Lab No.

CONSOLIDATION TEST REPORT



Coefficients of Consolidation and Secondary Consolidation

No.	Load (ksf)	C_v (ft.2/day)	C_α	No.	Load (ksf)	C_v (ft.2/day)	C_α	No.	Load (ksf)	C_v (ft.2/day)	C_α
4	2.00	0.274		13	1.00	1.431		22	0.10	0.071	
5	3.00	0.074		14	2.00	1.076					
6	4.00	0.012		15	3.00	0.621					
7	3.00	0.537		16	4.00	0.005					
10	0.50	0.173		17	5.00	0.002					
11	0.10	0.032		18	8.00	0.029					
12	0.50	1.143		21	1.00	0.239					

Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_r	Initial Void Ratio
Saturation	Moisture									
	43.2 %				2.76		2.4			

MATERIAL DESCRIPTION	USCS	AASHTO

Project No. 0600-102 **Client:** HNTB Corporation

Project: Scarborough Eastern Trail Project
Scarborough, ME

Location: BB-SPC-201 **Depth:** 30'-32' **Sample Number:** U-2

R.W. Gillespie & Associates, Inc.

Biddeford, Maine

Remarks:

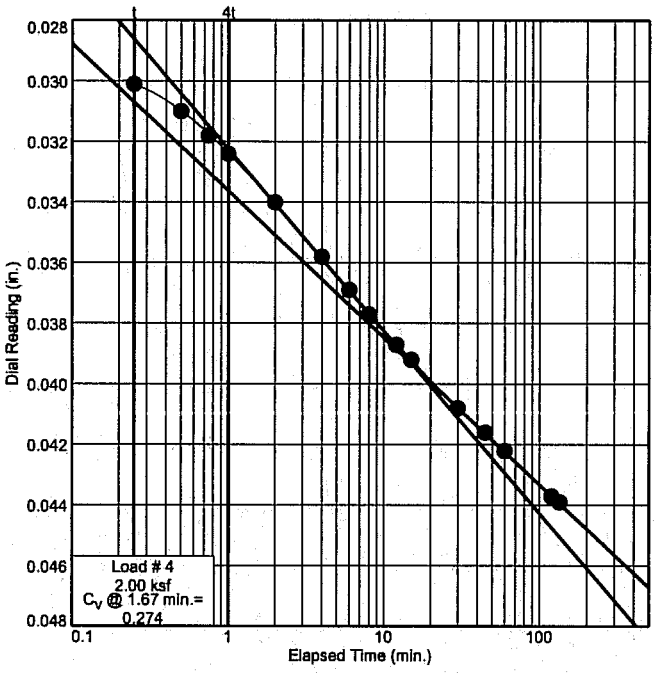
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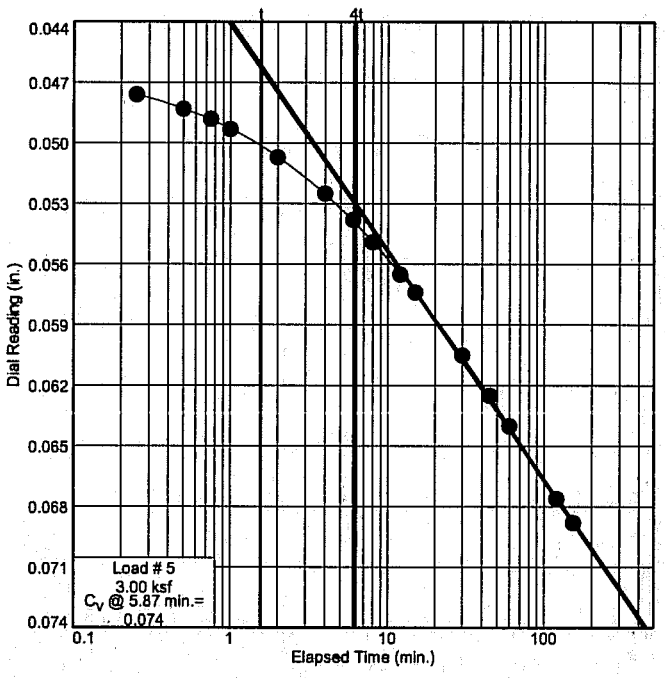
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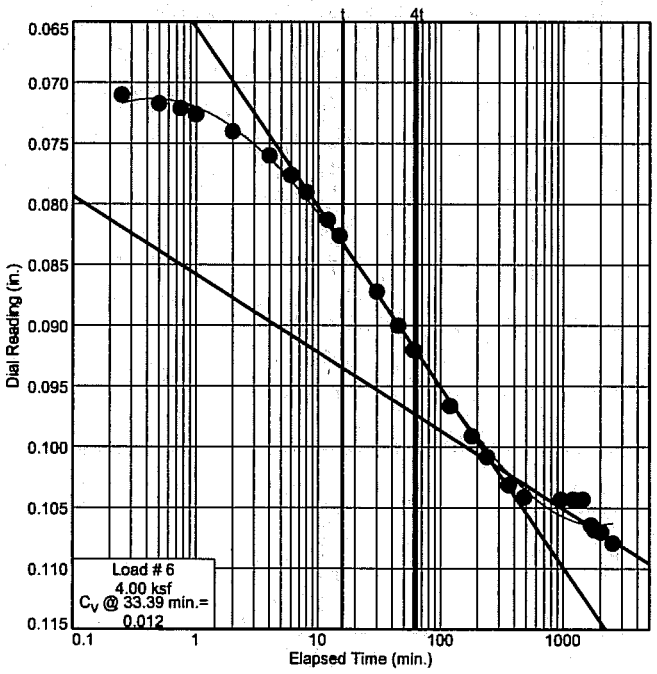
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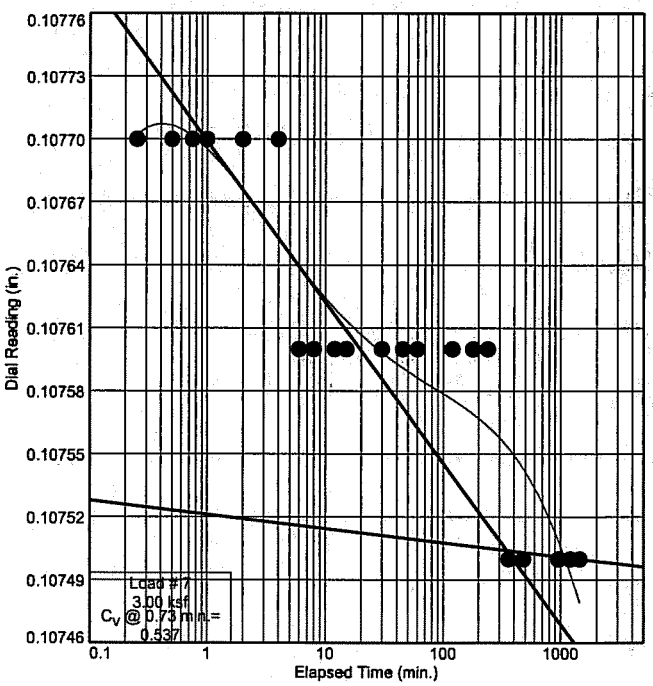
Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: BB-SPC-201 Depth: 30'-32' Sample Number: U-2









R.W. Gillespie & Associates, Inc.

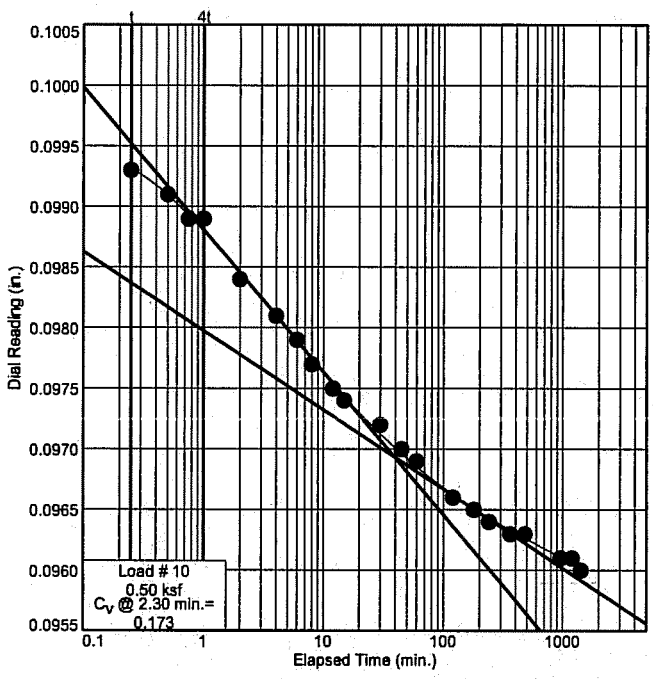
Biddeford, Maine

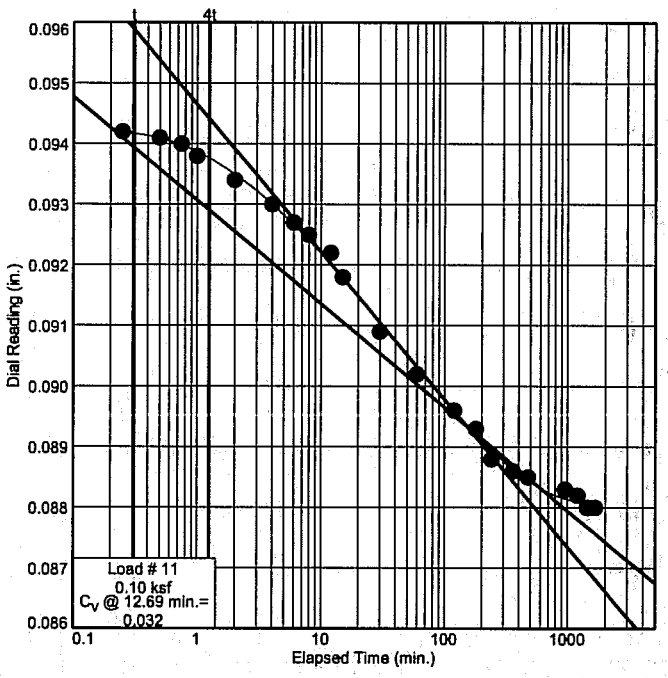
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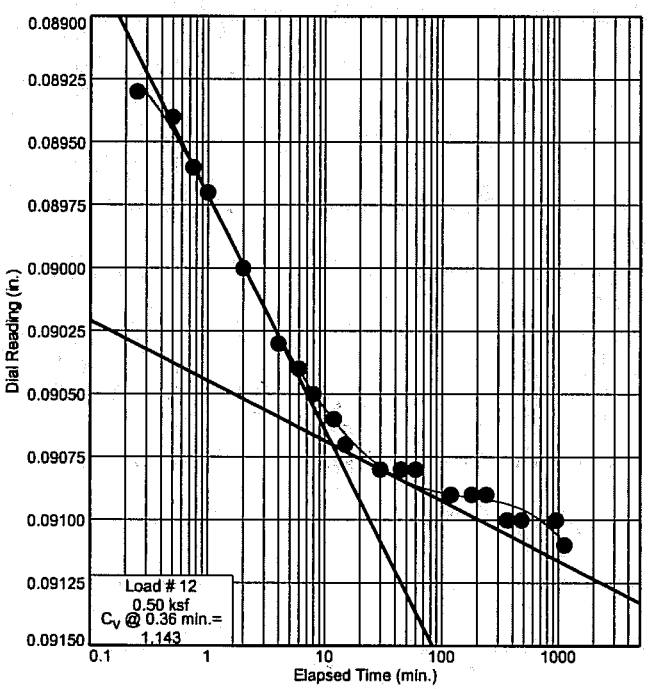
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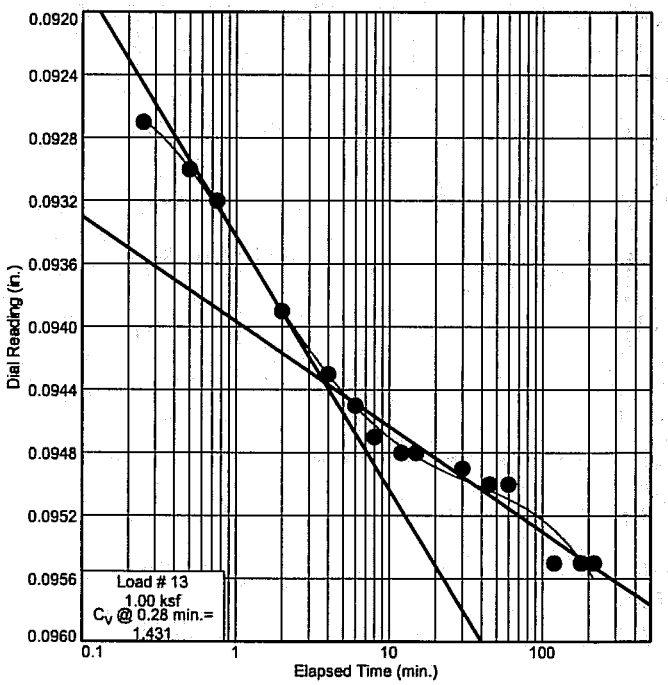
Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: BB-SPC-201 Depth: 30'-32' Sample Number: U-2









R.W. Gillespie & Associates, Inc.

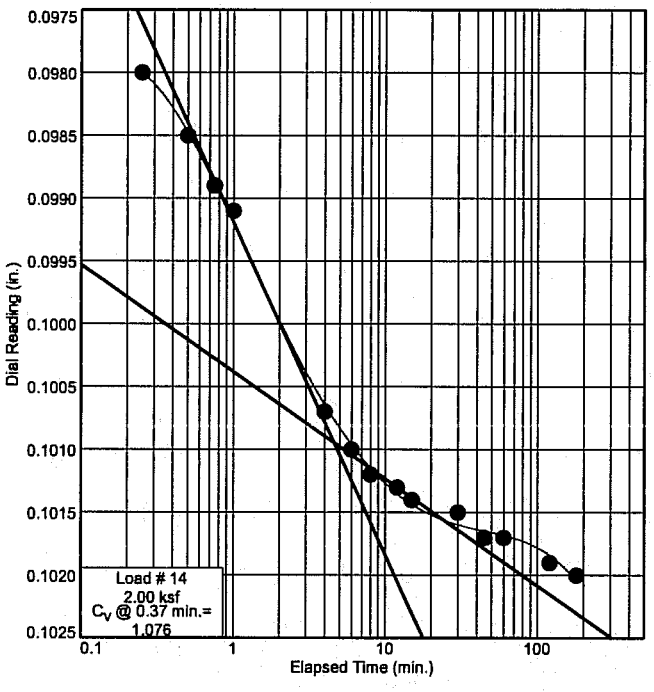
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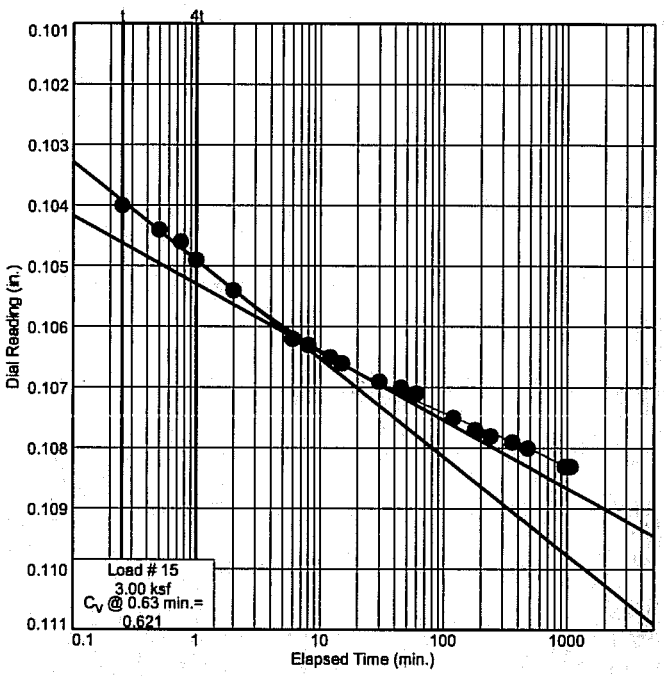
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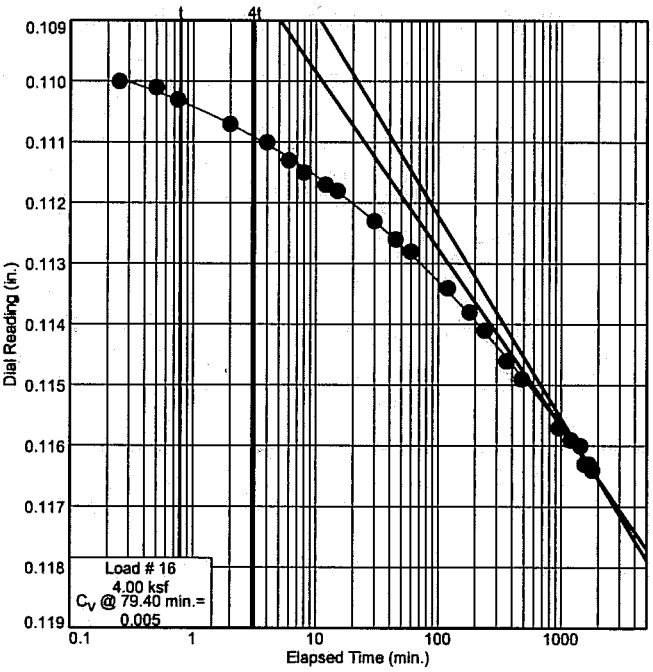
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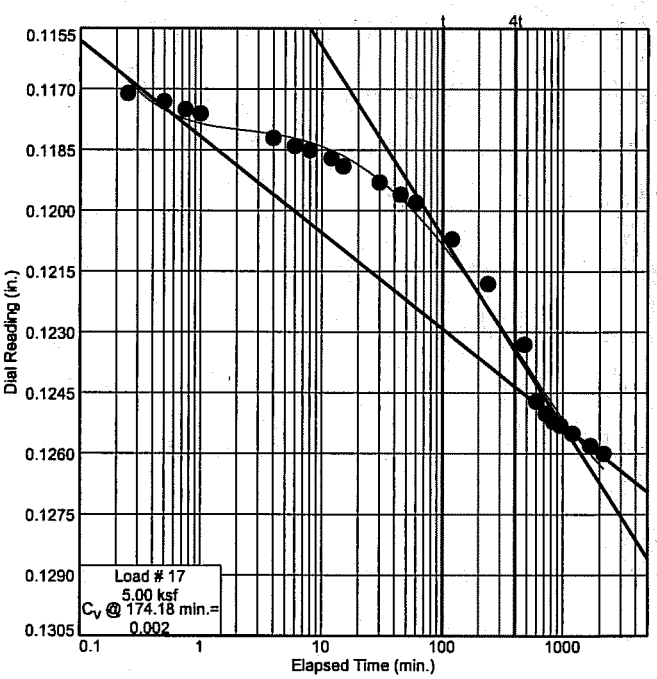
Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: BB-SPC-201 Depth: 30'-32' Sample Number: U-2









R.W. Gillespie & Associates, Inc.

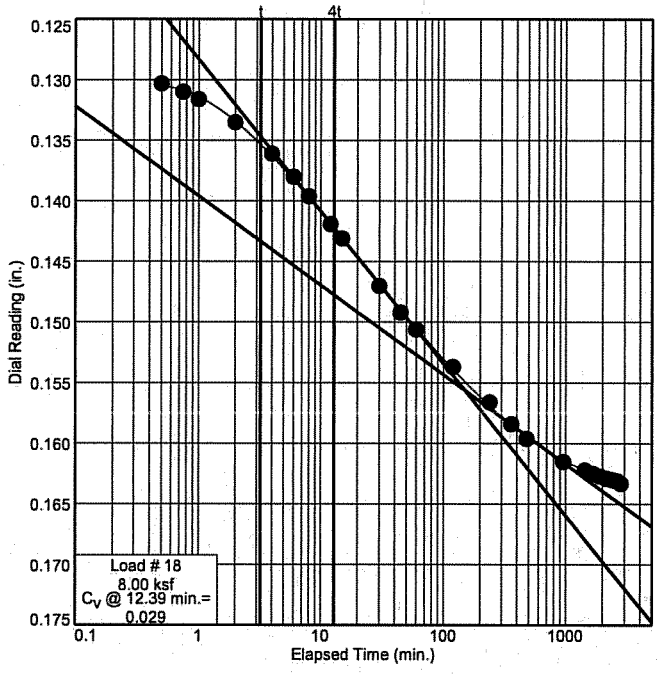
Biddeford, Maine

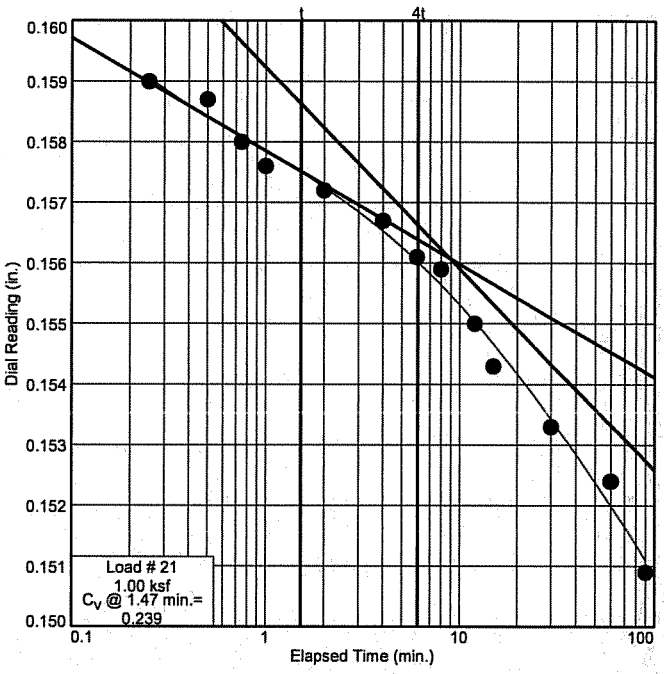
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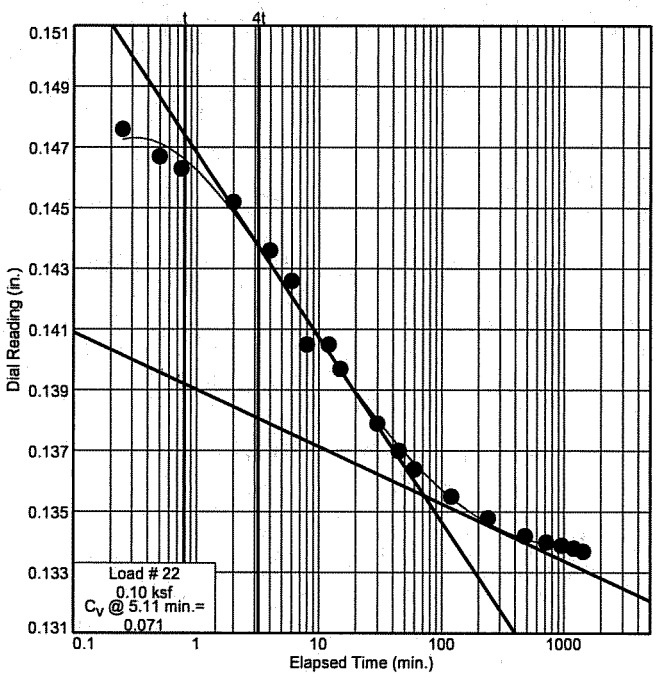
Dial Reading vs. Time

Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: BB-SPC-201 Depth: 30'-32' Sample Number: U-2





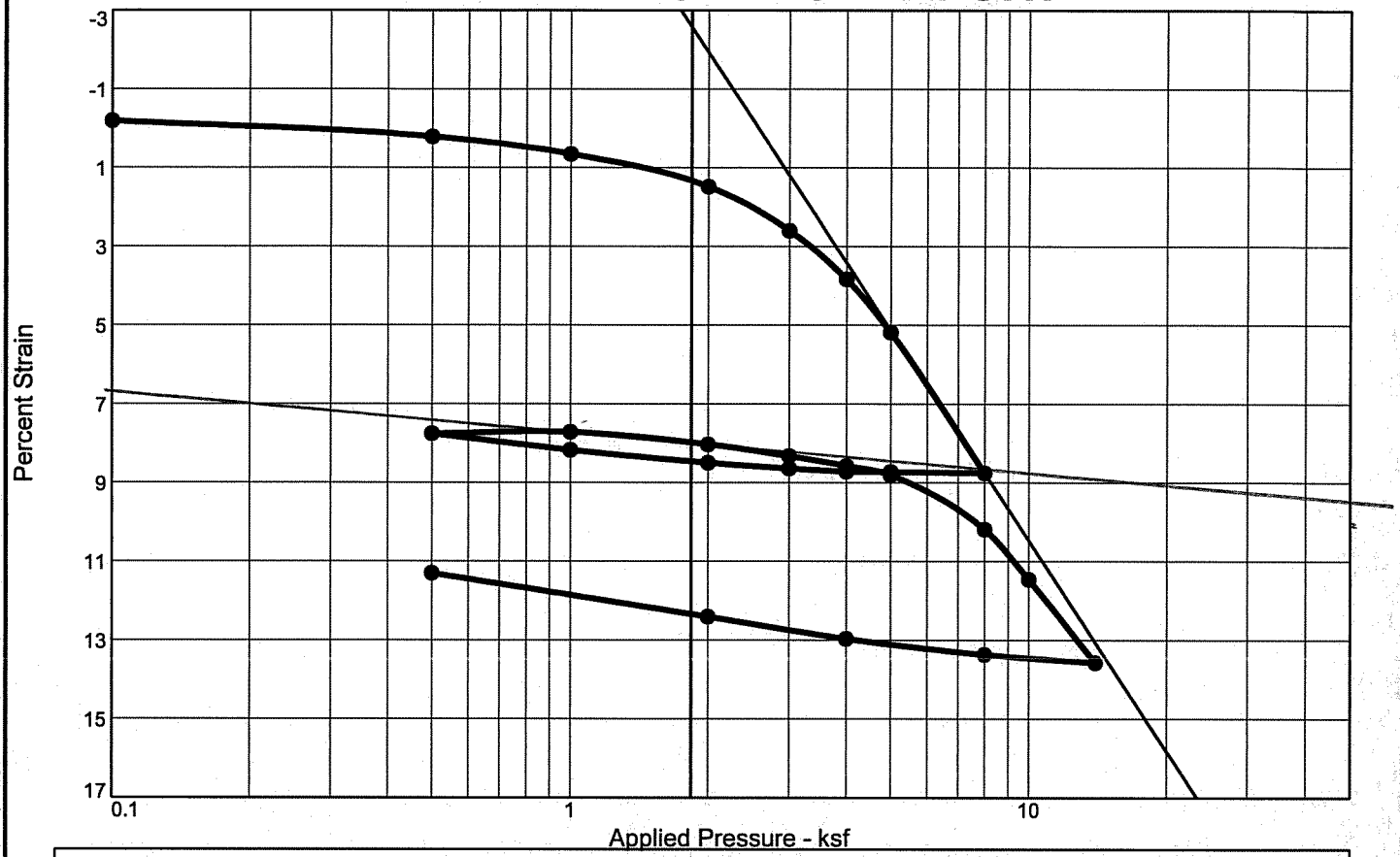


R.W. Gillespie & Associates, Inc.

Biddeford, Maine

Lab No.

CONSOLIDATION TEST REPORT



Coefficients of Consolidation and Secondary Consolidation

No.	Load (ksf)	C_v (ft.2/day)	C_α	No.	Load (ksf)	C_v (ft.2/day)	C_α	No.	Load (ksf)	C_v (ft.2/day)	C_α
2	0.50	0.379		11	3.00	0.097		18	4.00	0.261	
3	1.00	0.354		12	2.00	1.022		19	5.00	0.500	
4	2.00	0.378		13	1.00	0.314		20	8.00	0.293	
5	3.00	0.181		14	0.50	0.145		21	10.00	0.092	
6	4.00	0.095		15	1.00	1.627		22	14.00	0.209	
7	5.00	0.065		16	2.00	0.677		23	8.00	0.728	
8	8.00	0.092		17	3.00	0.561		24	4.00	0.686	

Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_r	Initial Void Ratio
Saturation	Moisture									
99.4 %	38.1 %	83.7			2.76		3.3	0.37	0.08	1.059

MATERIAL DESCRIPTION	USCS	AASHTO

Project No. 0600-102 **Client:** HNTB Corporation

Project: Scarborough Eastern Trail Project
Scarborough, ME

Location: BB-SPC-201 **Depth:** 30'-32' **Sample Number:** U-2

R.W. Gillespie & Associates, Inc.

Biddeford, Maine

Remarks:

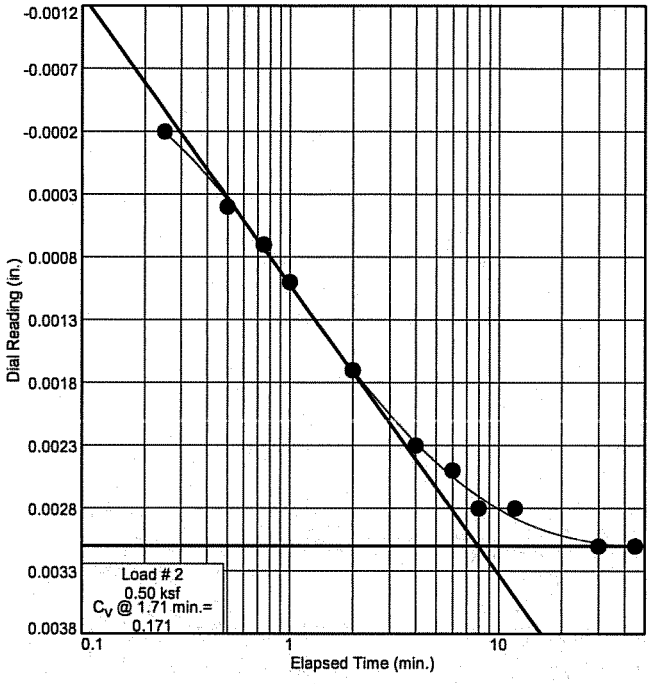
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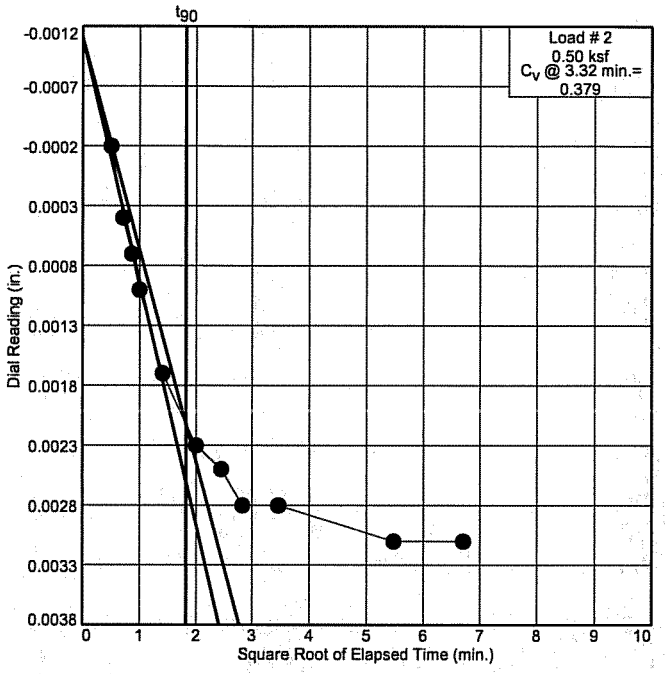
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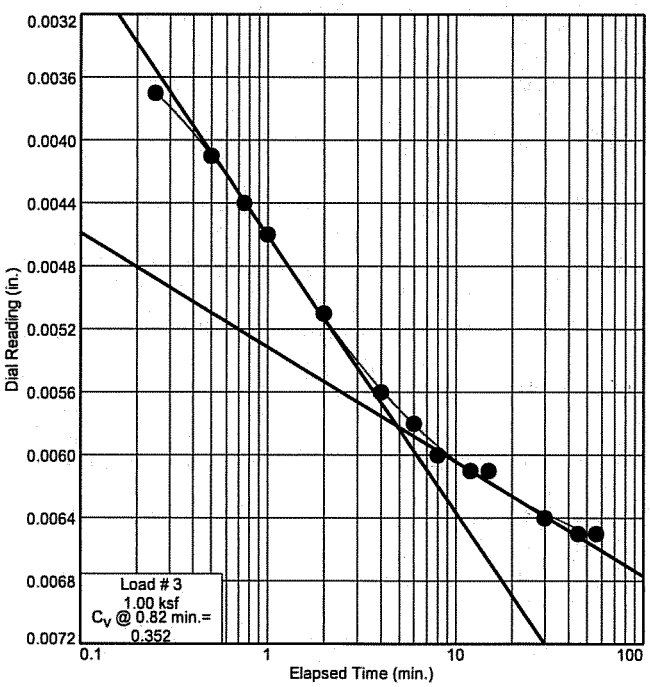
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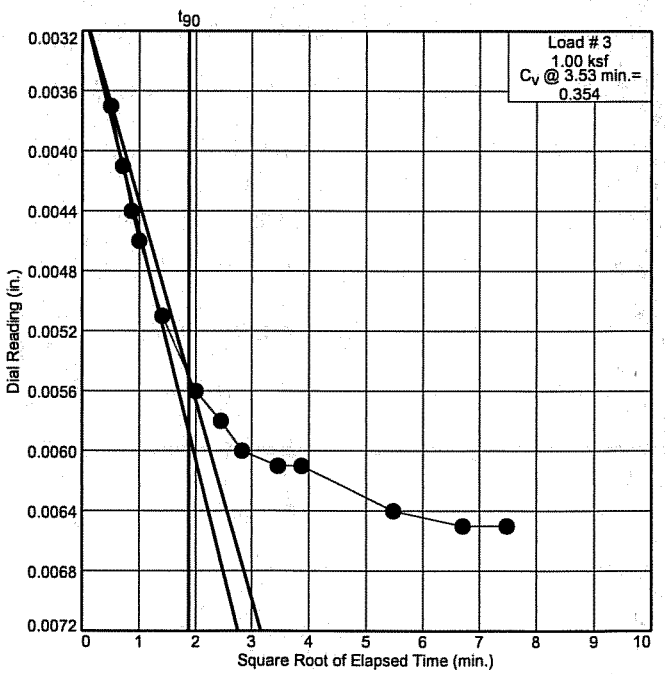
Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: BB-SPC-201 Depth: 30'-32' Sample Number: U-2









R.W. Gillespie & Associates, Inc.

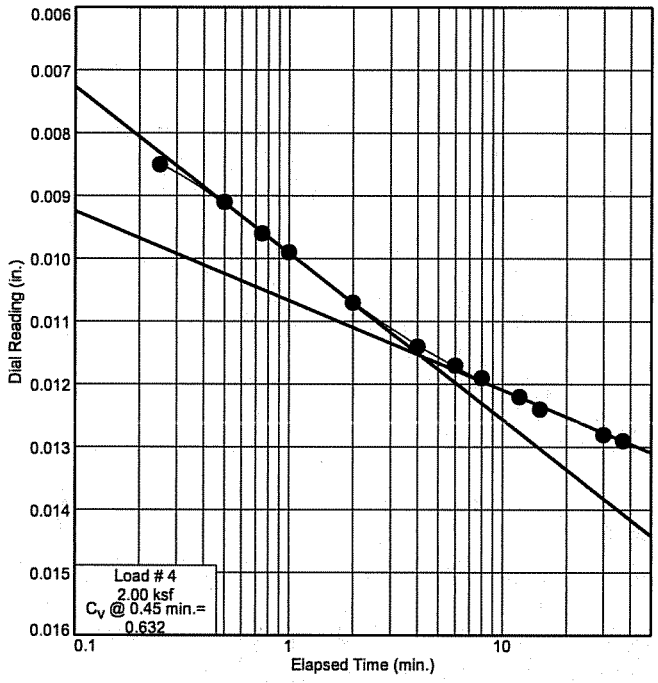
Biddeford, Maine

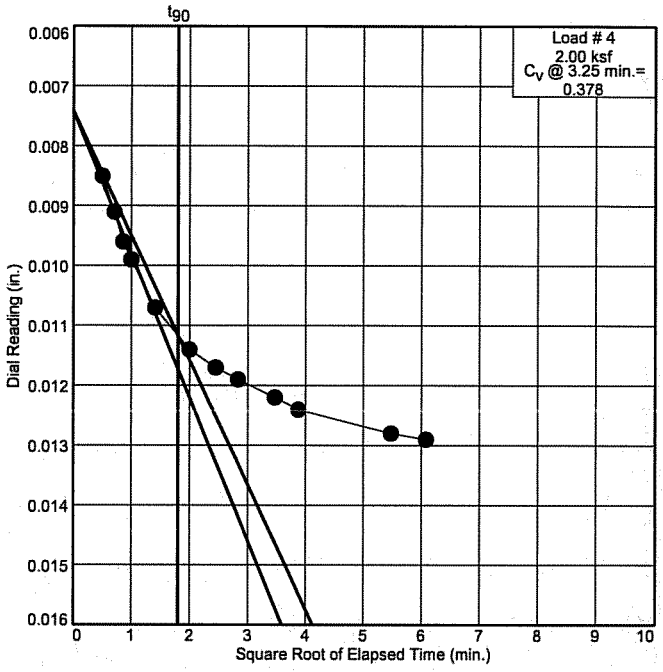
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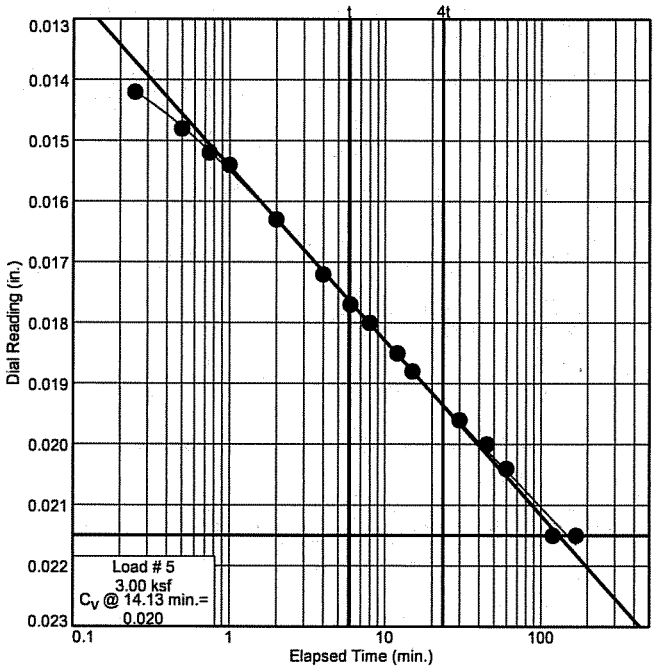
Dial Reading vs. Time

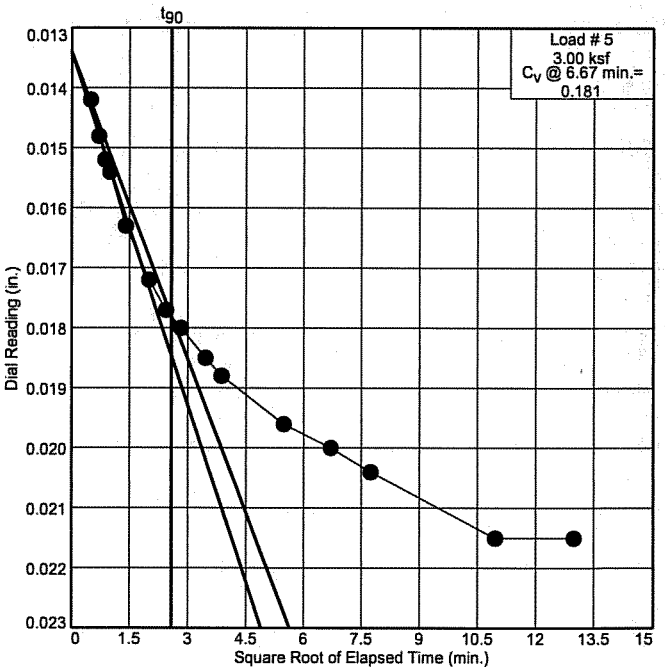
Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: BB-SPC-201 Depth: 30'-32' Sample Number: U-2









R.W. Gillespie & Associates, Inc.

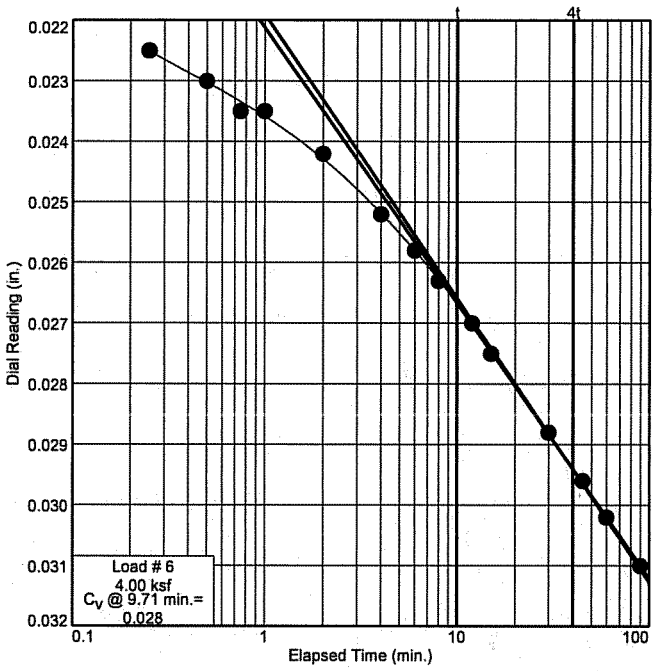
Biddeford, Maine

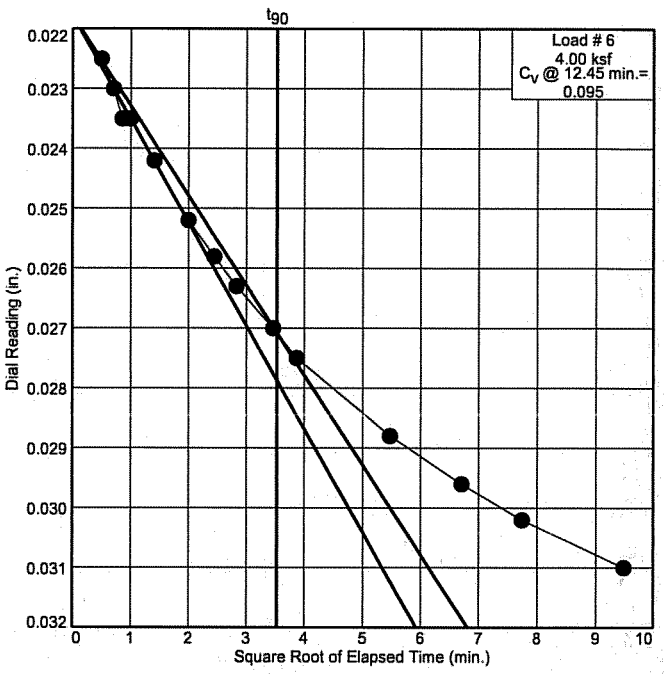
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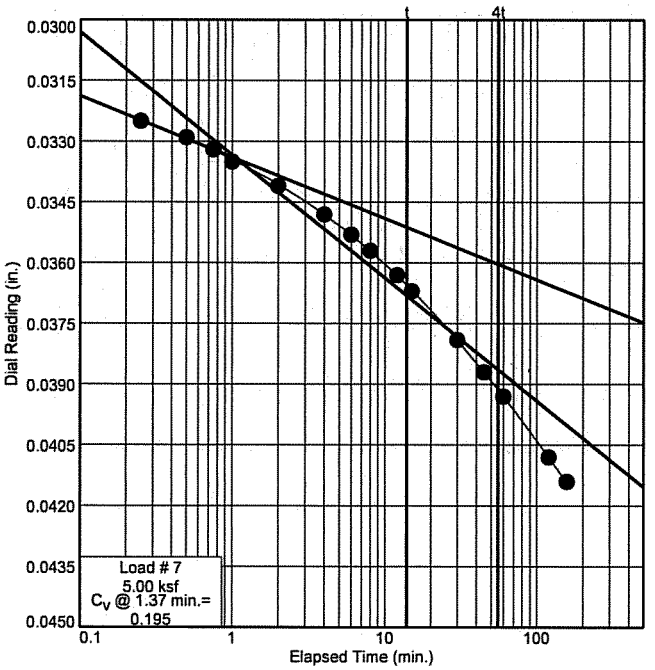
Dial Reading vs. Time

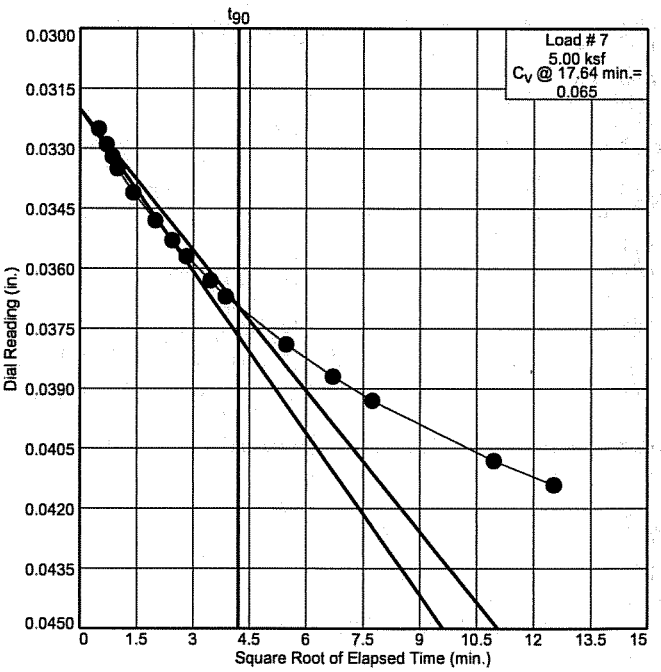
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Project: Scarborough Eastern Trail Project

Location: BB-SPC-201 Depth: 30'-32' Sample Number: U-2









R.W. Gillespie & Associates, Inc.

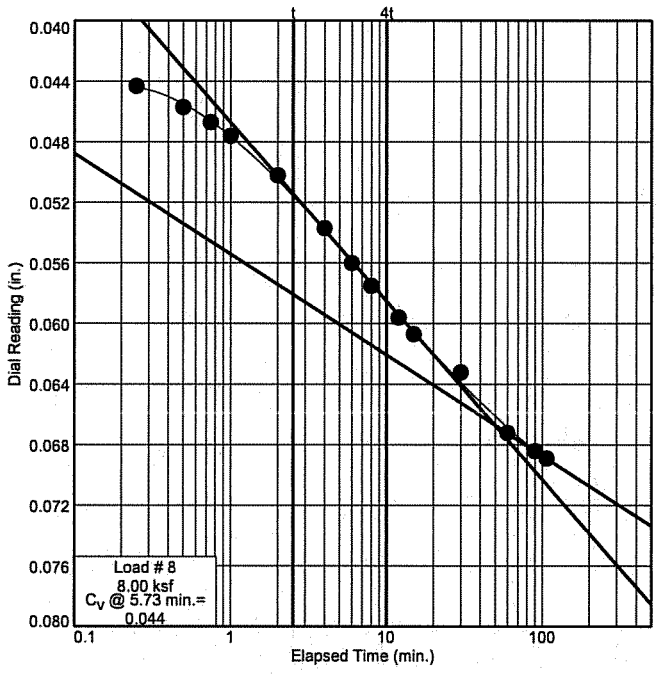
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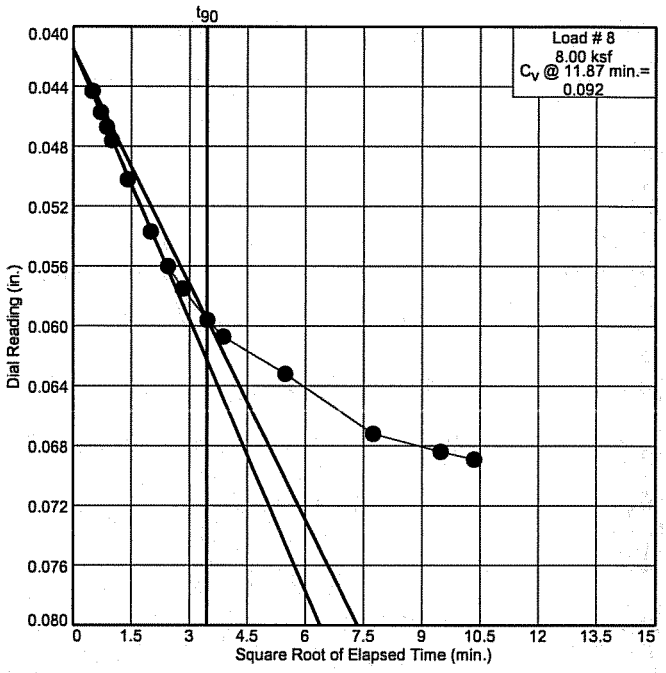
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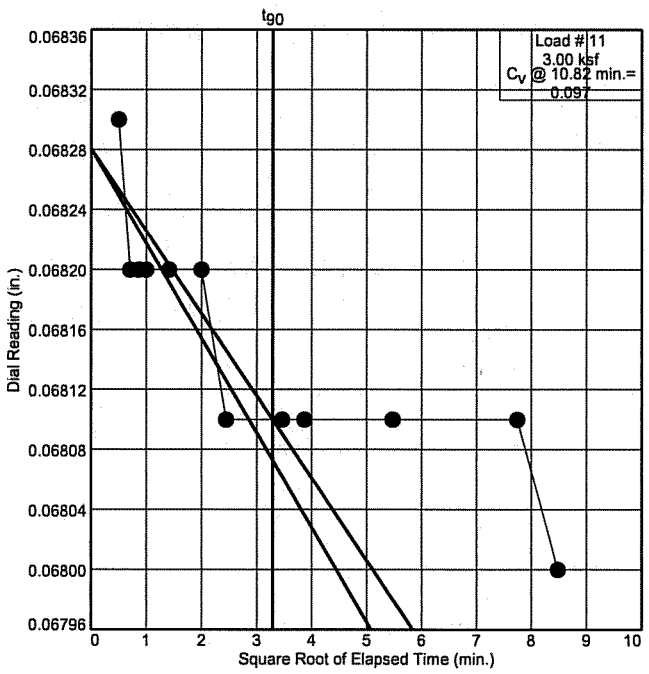
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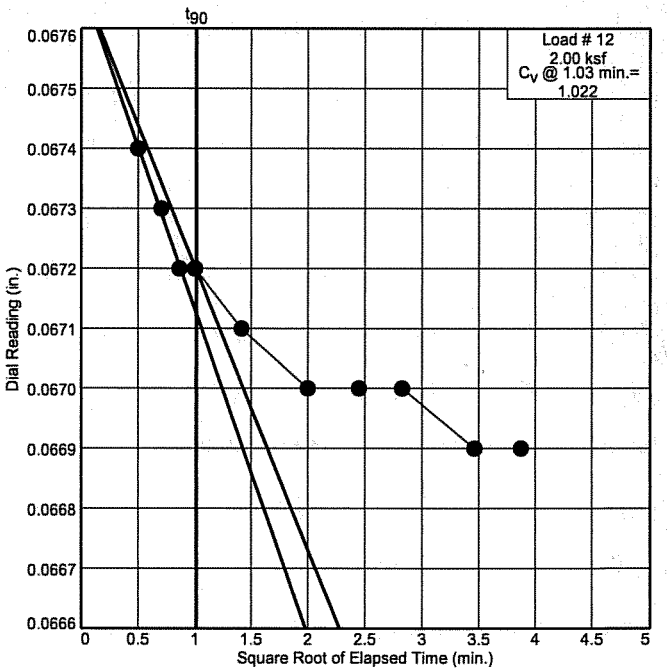
Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: BB-SPC-201 Depth: 30'-32' Sample Number: U-2









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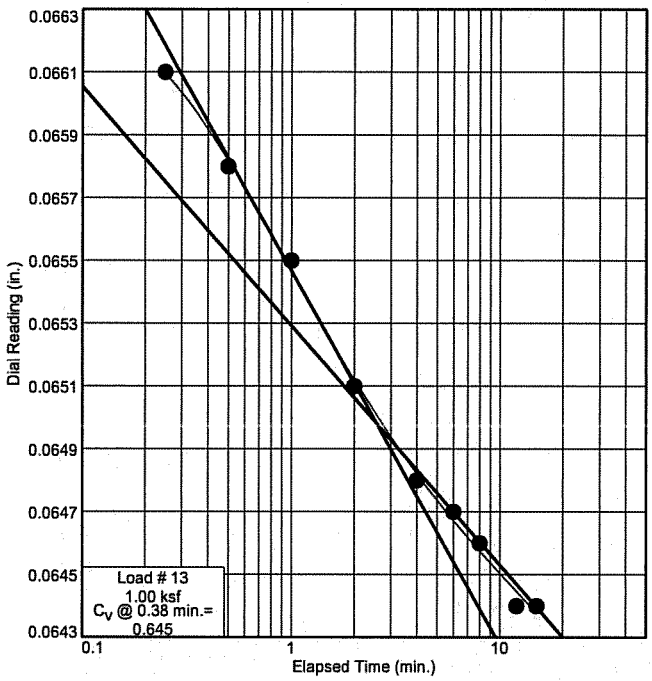
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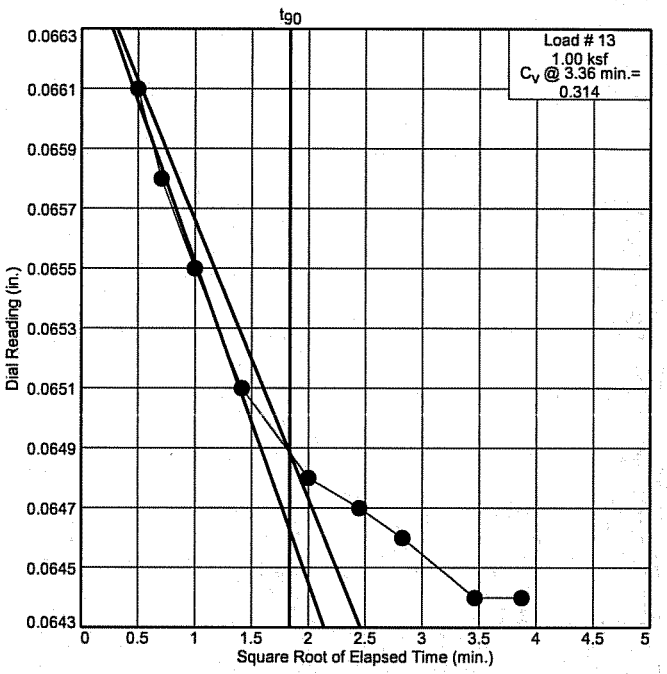
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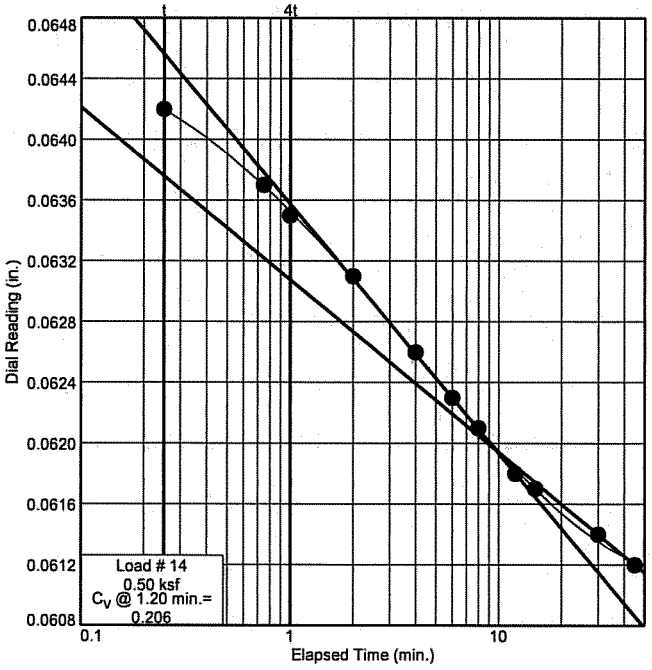
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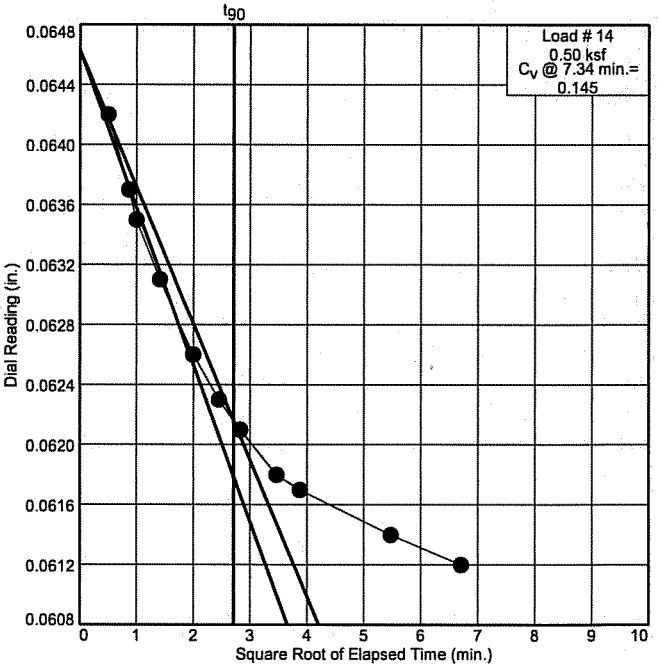
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Project: Scarborough Eastern Trail Project

Location: BB-SPC-201 Depth: 30'-32' Sample Number: U-2









R.W. Gillespie & Associates, Inc.

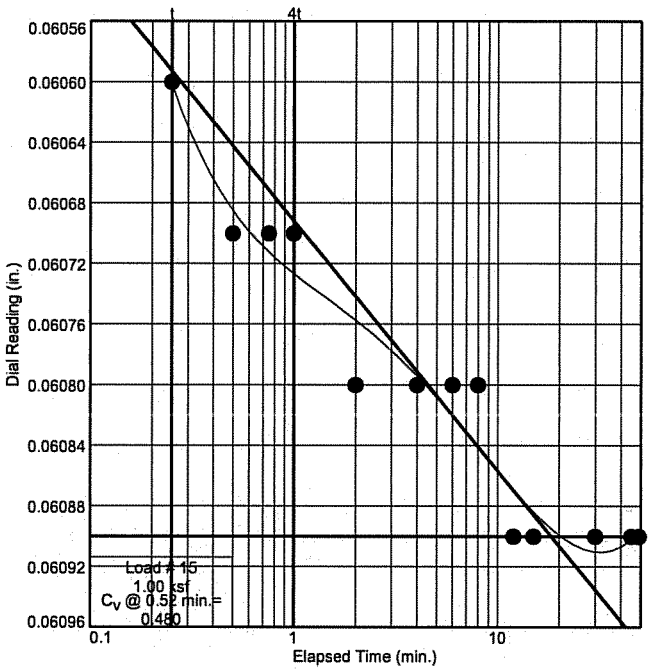
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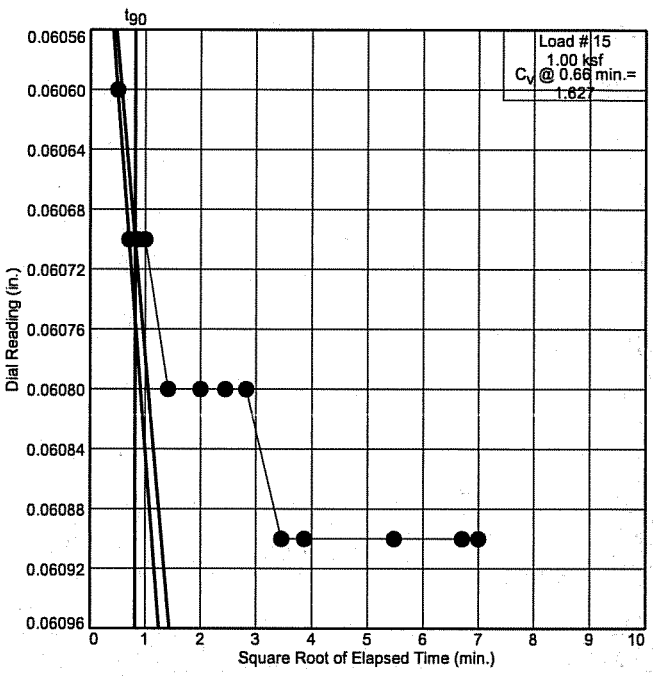
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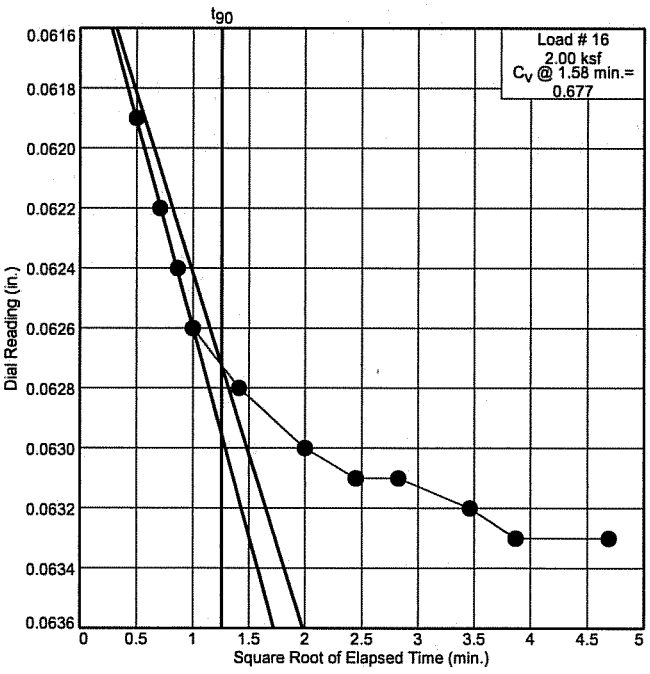
Dial Reading vs. Time

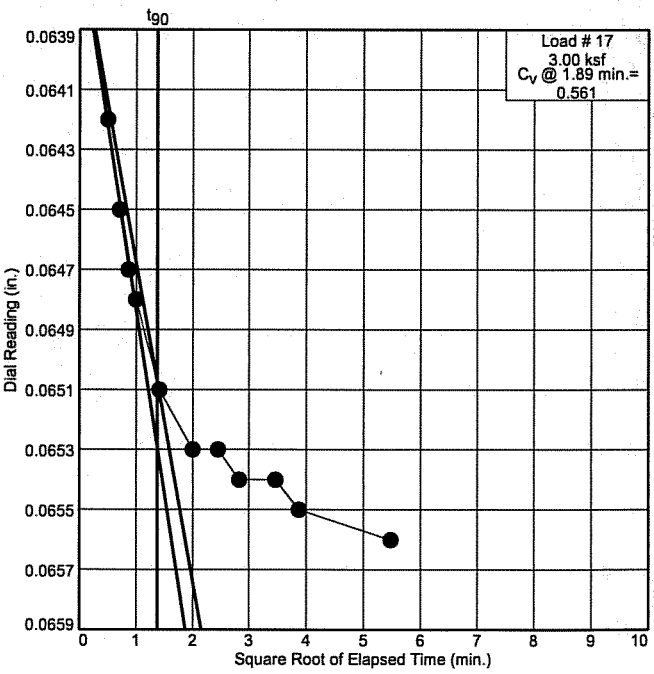
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Project: Scarborough Eastern Trail Project

Location: BB-SPC-201 Depth: 30'-32' Sample Number: U-2









R.W. Gillespie & Associates, Inc.

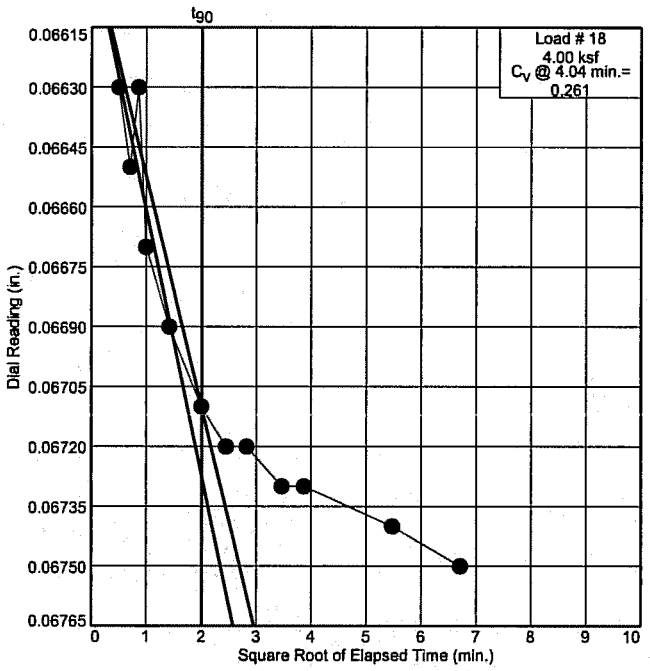
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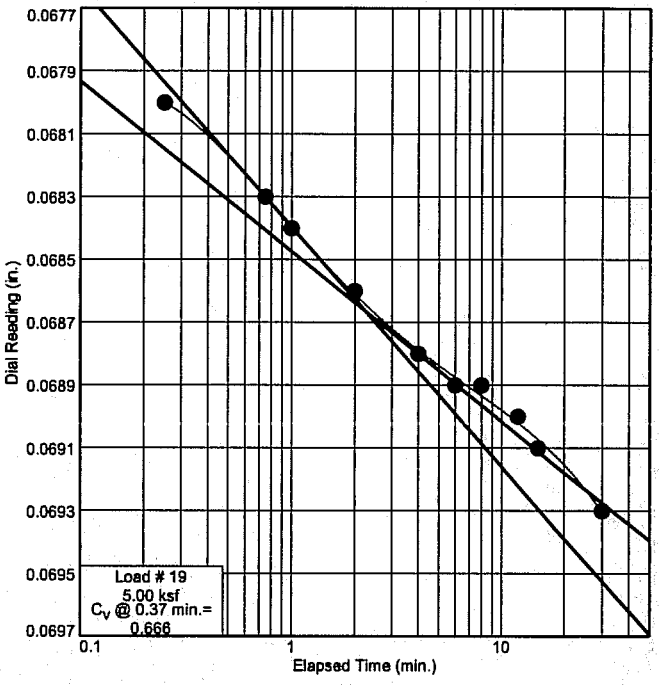
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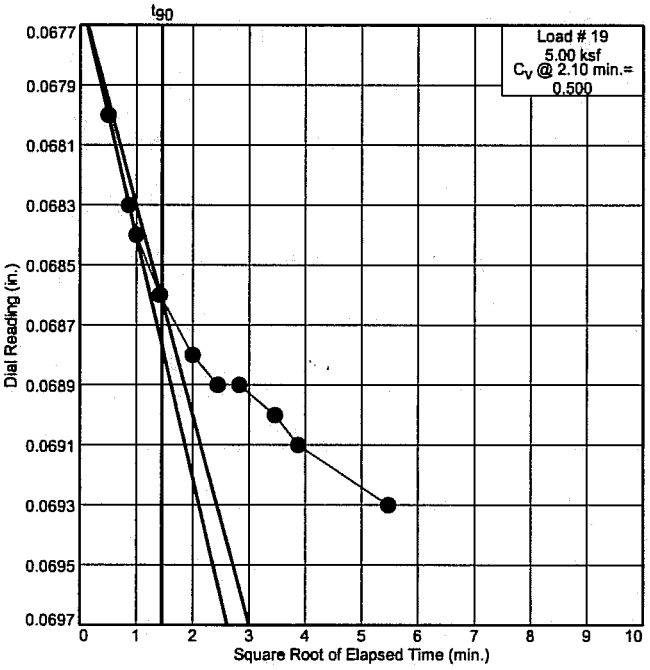
Dial Reading vs. Time

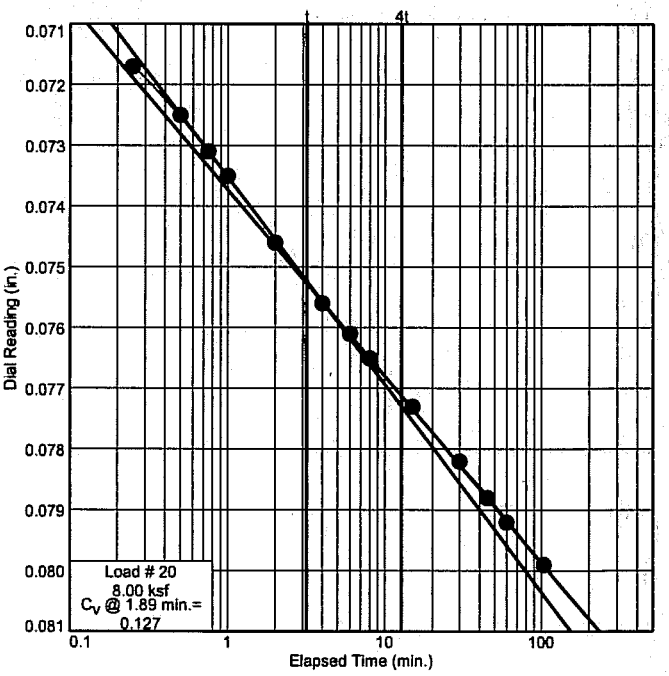
Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: BB-SPC-201 Depth: 30'-32' Sample Number: U-2









R.W. Gillespie & Associates, Inc.

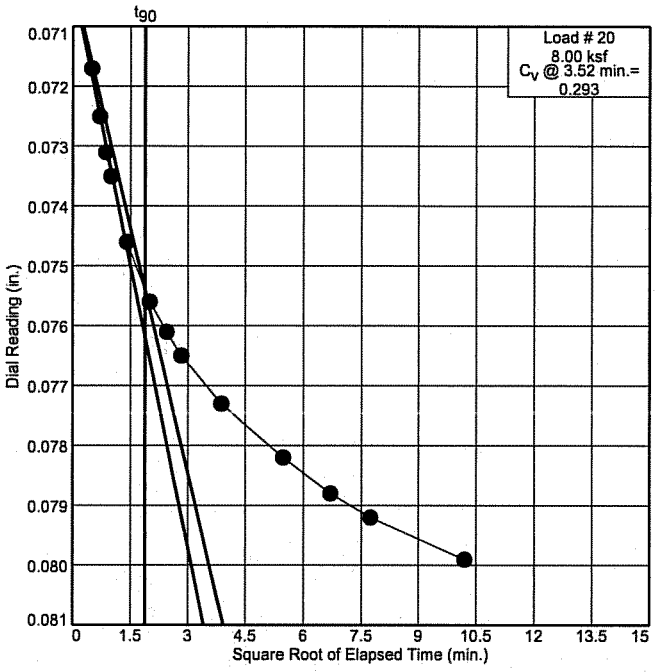
Biddeford, Maine

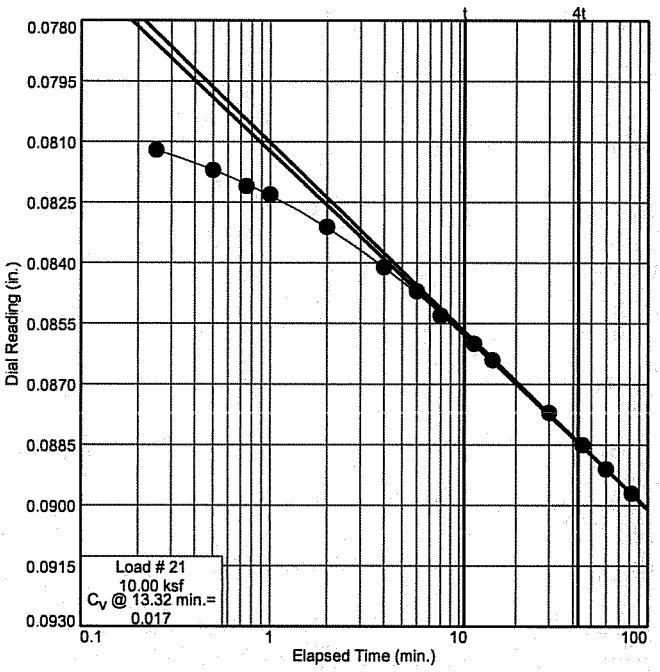
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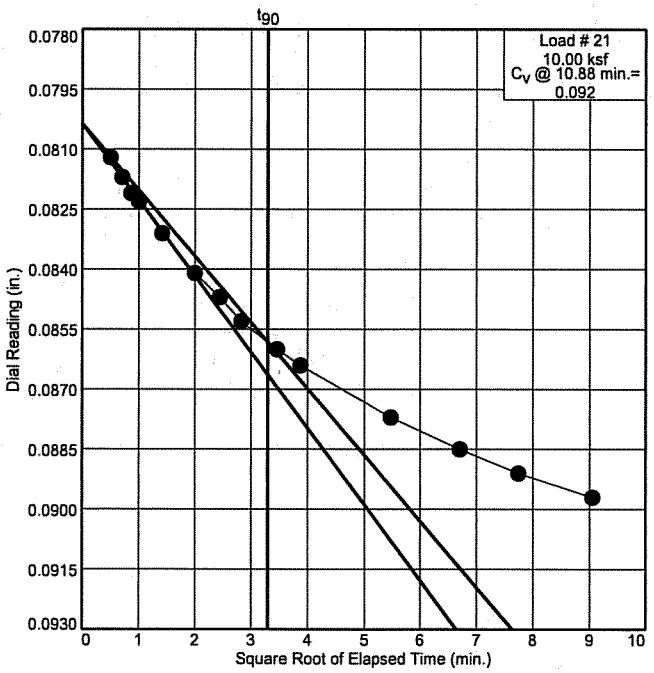
Dial Reading vs. Time

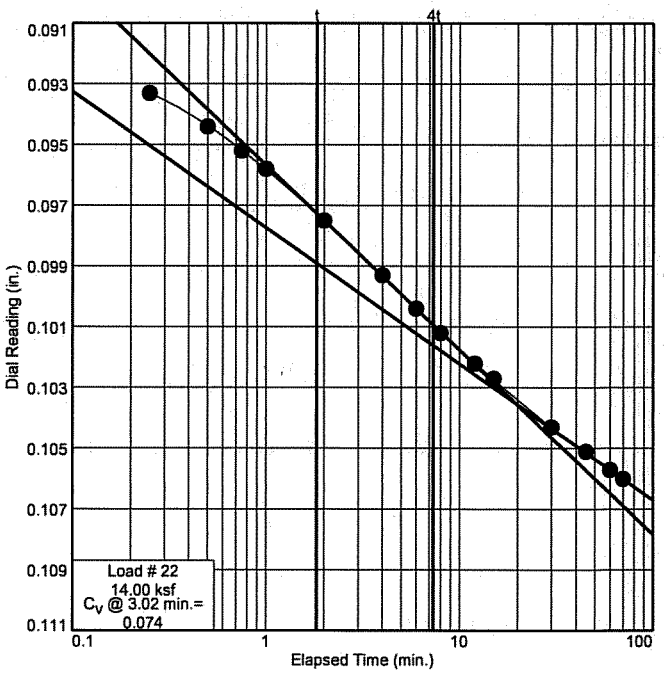
Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: BB-SPC-201 Depth: 30'-32' Sample Number: U-2









R.W. Gillespie & Associates, Inc.

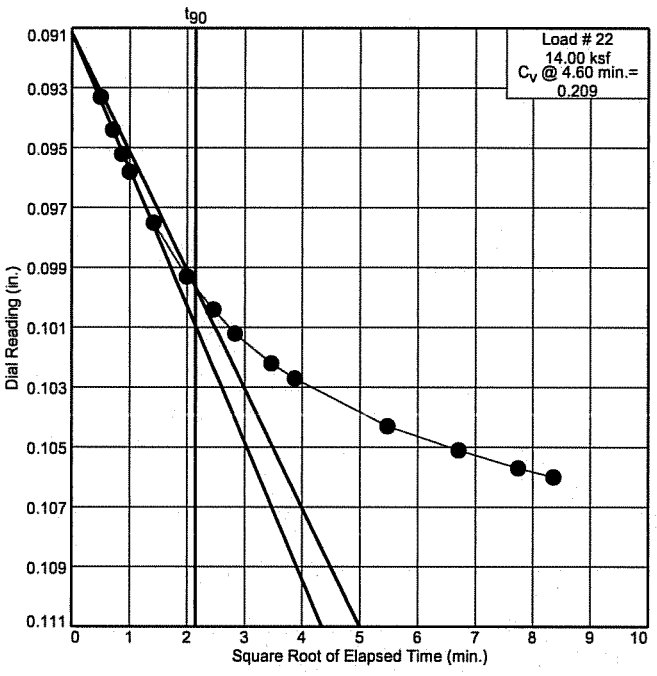
Biddeford, Maine

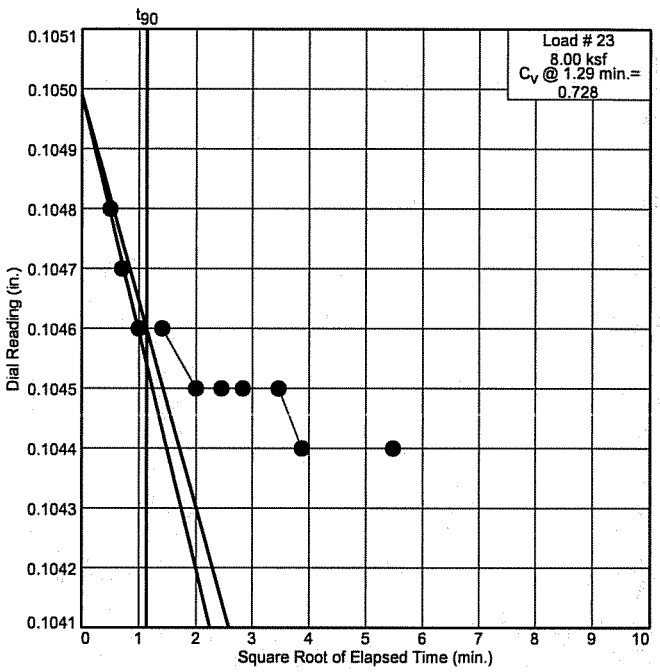
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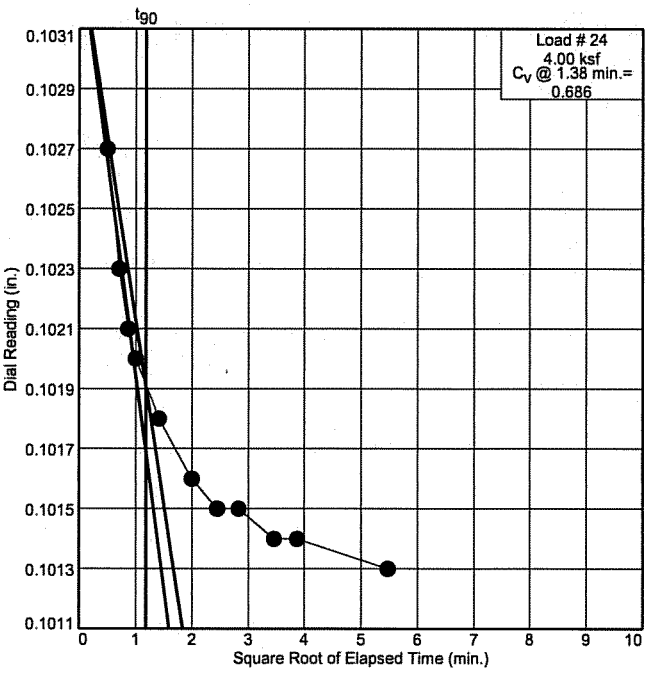
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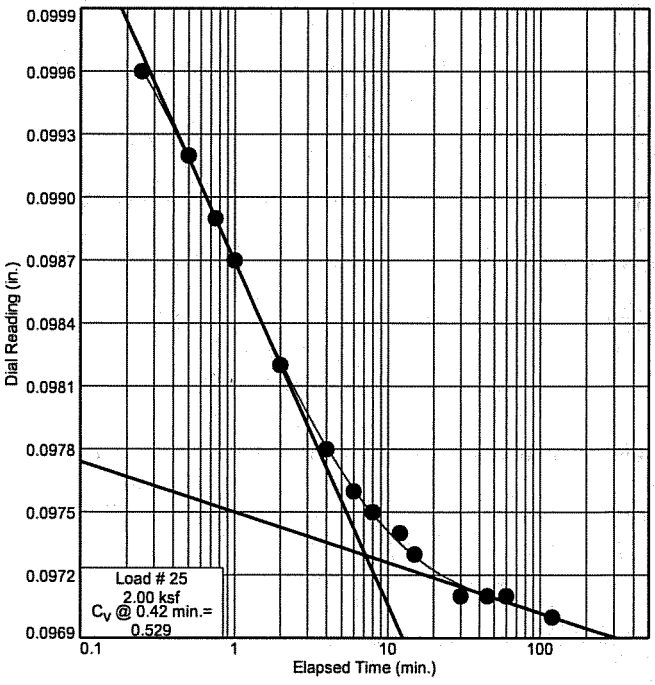
Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: BB-SPC-201 Depth: 30'-32' Sample Number: U-2









R.W. Gillespie & Associates, Inc.

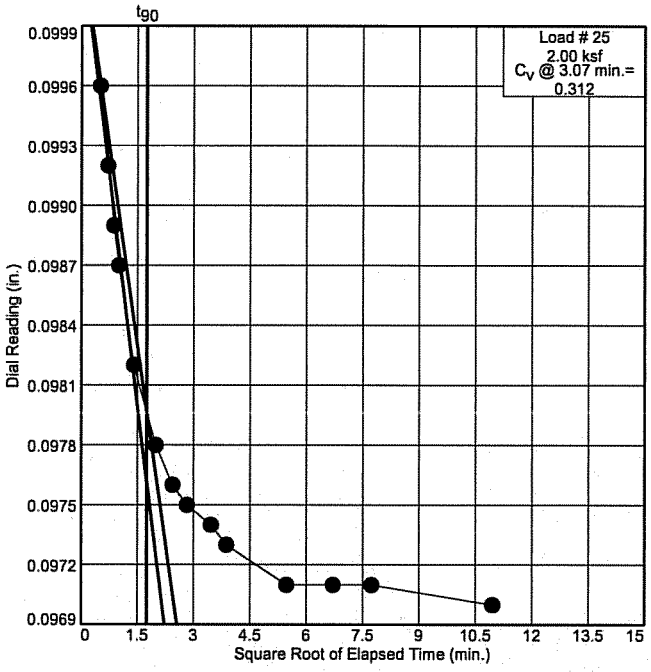
Biddeford, Maine

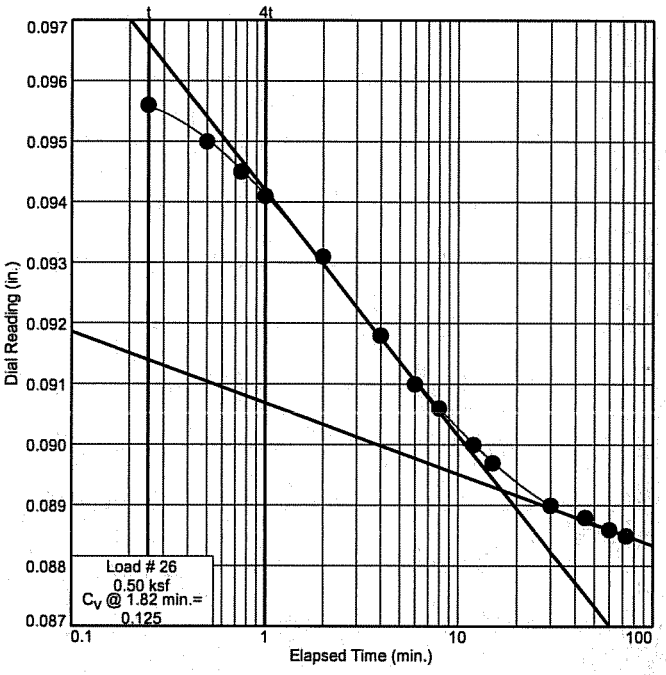
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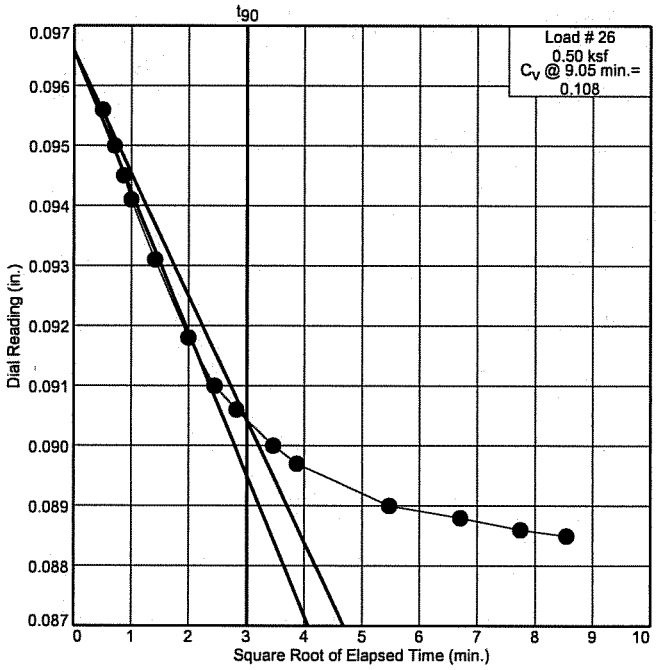
Dial Reading vs. Time

Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: BB-SPC-201 Depth: 30'-32' Sample Number: U-2





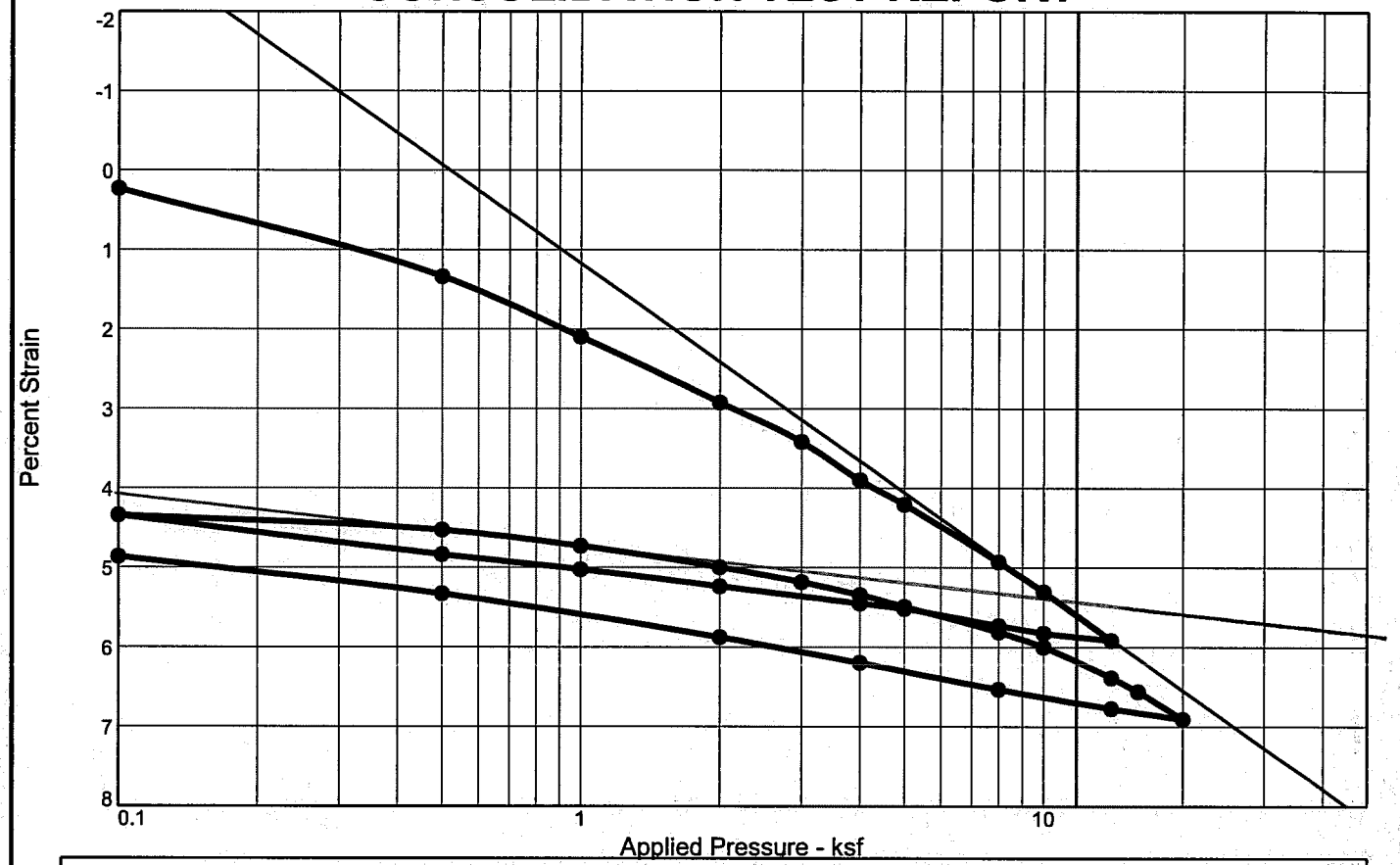


R.W. Gillespie & Associates, Inc.

Biddeford, Maine

Lab No. 14440

CONSOLIDATION TEST REPORT



Coefficients of Consolidation and Secondary Consolidation

No.	Load (ksf)	C_v (ft.2/day)	C_α	No.	Load (ksf)	C_v (ft.2/day)	C_α	No.	Load (ksf)	C_v (ft.2/day)	C_α
1	0.10	0.038									
6	4.00	0.426									
7	5.00	0.503									
9	10.00	0.302									
15	2.00	0.056									
17	0.50	0.018									

Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_r	Initial Void Ratio
Saturation	Moisture									
	30.0 %						10.6			

MATERIAL DESCRIPTION	USCS	AASHTO

Project No. 0600-102 **Client:** HNTB Corporation

Project: Scarborough Eastern Trail Project
Scarborough, ME

Location: BB-SPC-202 **Sample Number:** U-1 DEPTH: 20-21'

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Biddeford, Maine

Remarks:

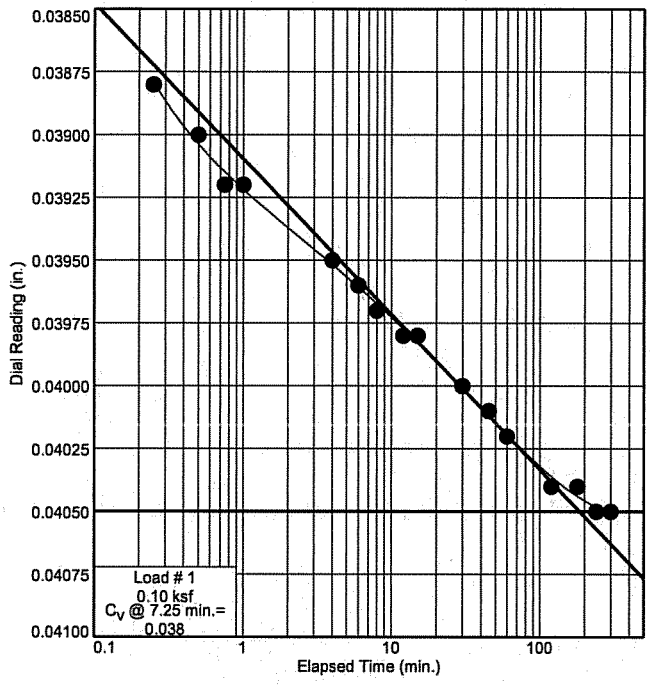
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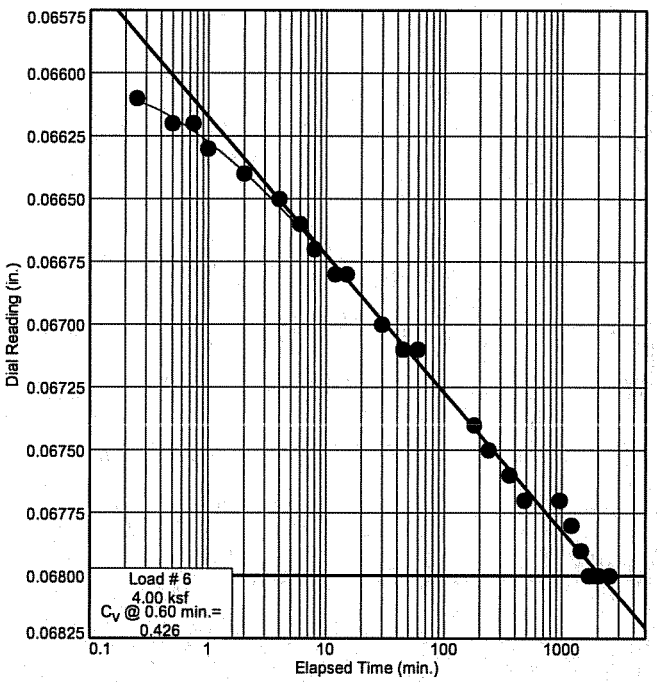
Tested By: _____ **Checked By:** _____

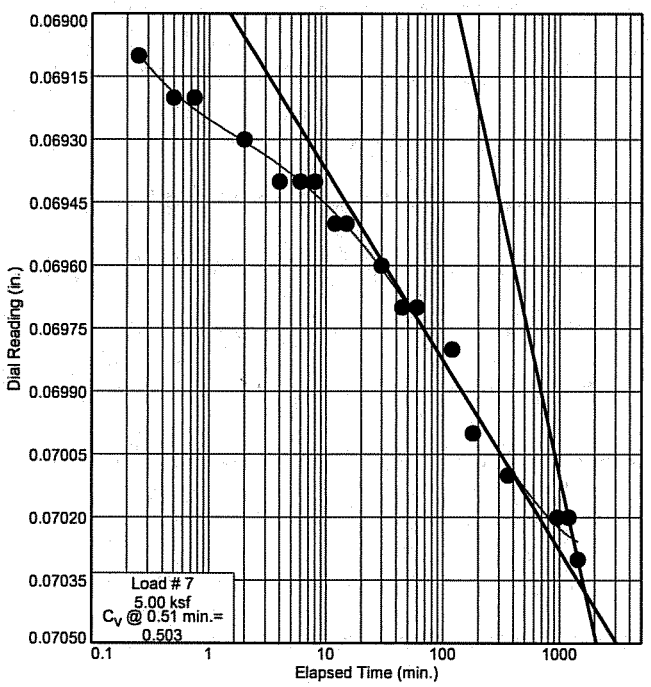
Dial Reading vs. Time

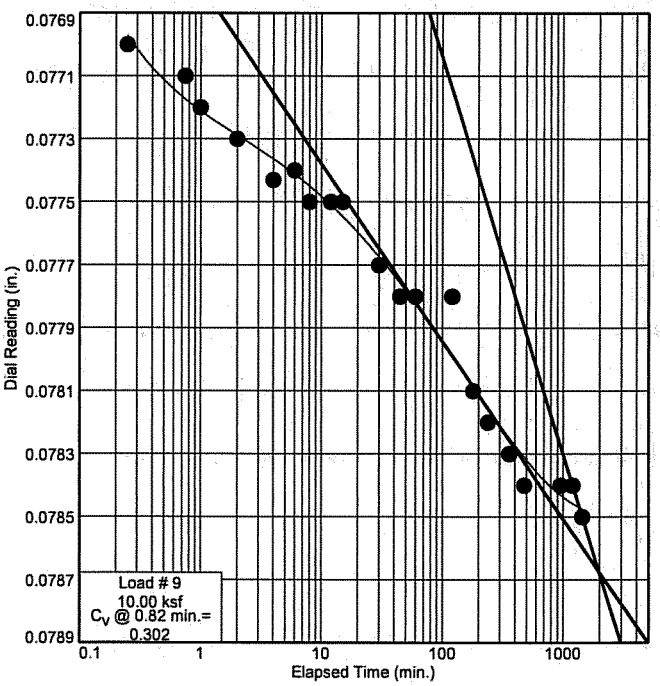
Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: BB-SPC-202 Sample Number: U-1









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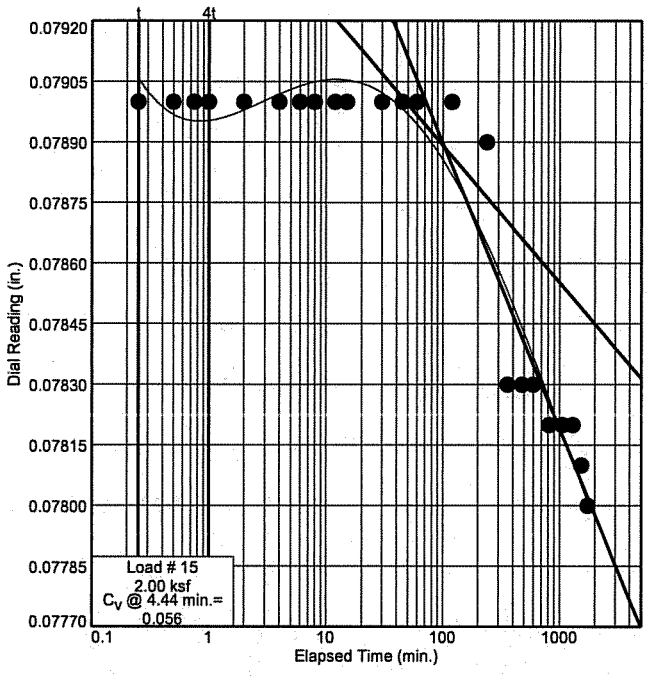
Biddeford, Maine

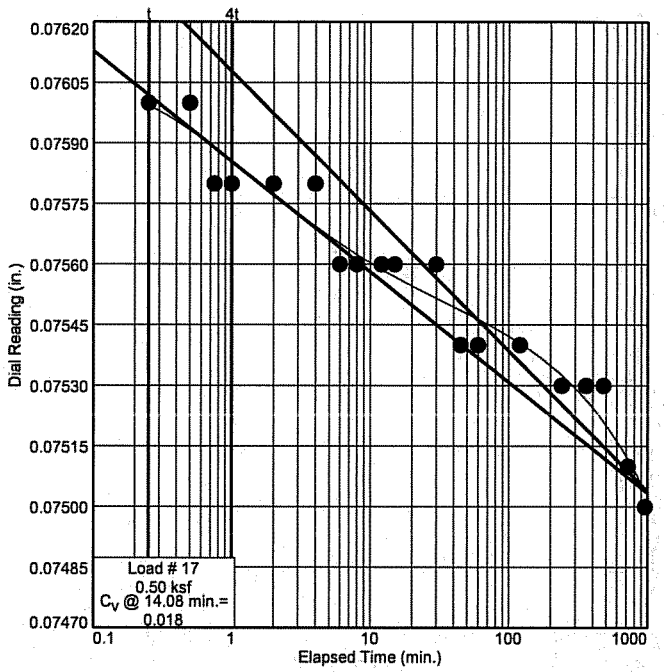
Lab No.

Dial Reading vs. Time

Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: BB-SPC-202 Sample Number: U-1



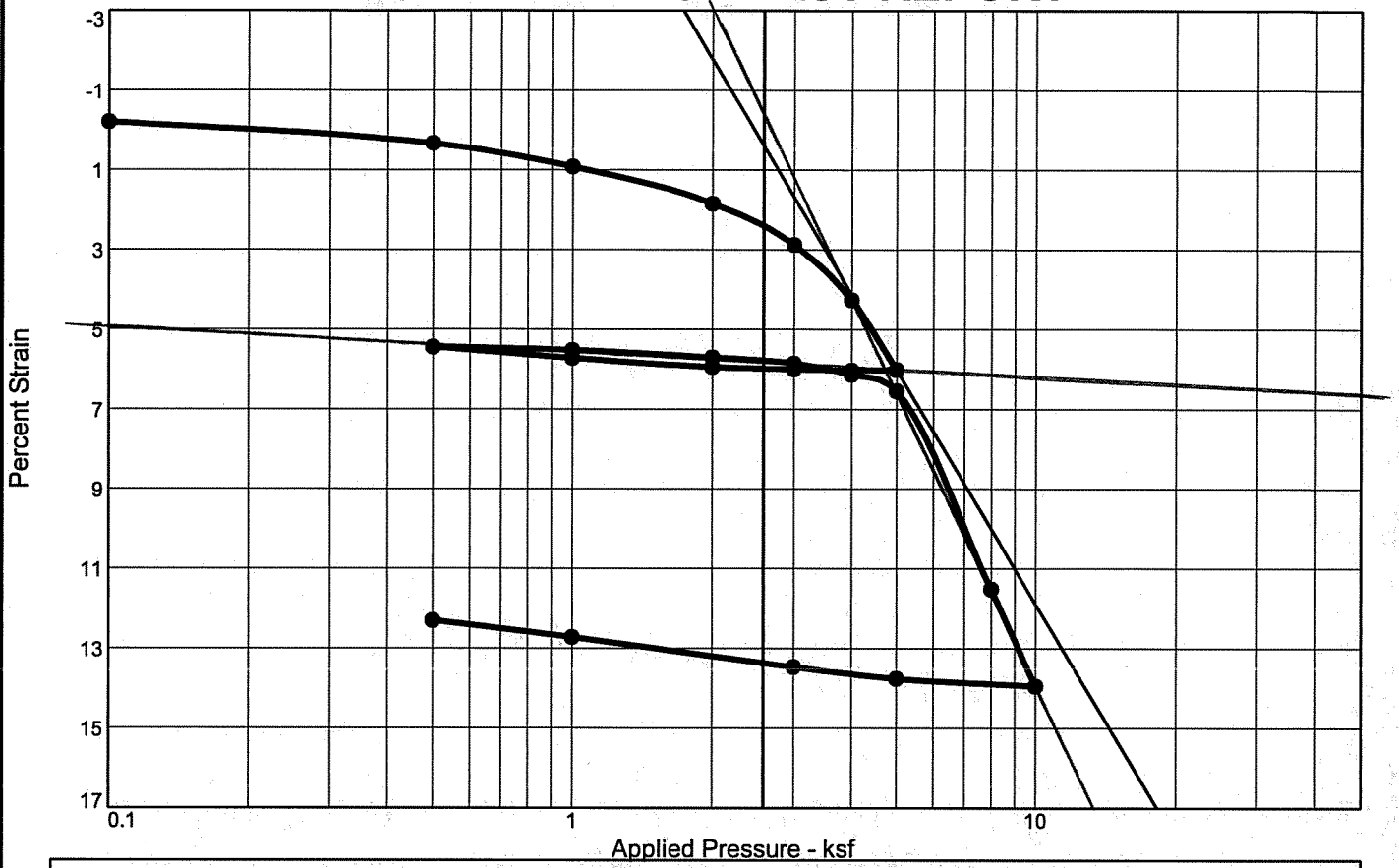


R.W. Gillespie & Associates, Inc.

Biddeford, Maine

Lab No.

CONSOLIDATION TEST REPORT



Coefficients of Consolidation and Secondary Consolidation

No.	Load (ksf)	C_v (ft.2/day)	C_α	No.	Load (ksf)	C_v (ft.2/day)	C_α	No.	Load (ksf)	C_v (ft.2/day)	C_α
2	0.50	0.406		10	2.00	1.053		17	5.00	0.510	
3	1.00	0.843		11	1.00	0.401		18	8.00	0.065	
4	2.00	0.389		12	0.50	0.289		19	10.00	0.059	
5	3.00	0.287		13	1.00	0.850		20	5.00	1.353	
6	4.00	0.087		14	2.00	0.728		21	3.00	0.558	
7	5.00	0.016		15	3.00	0.728		22	1.00	0.275	
9	3.00	0.111		16	4.00	0.512		23	0.50	0.080	

Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_r	Initial Void Ratio
Saturation	Moisture									
100.4 %	46.3 %	75.8			2.76		3.4	0.44	0.12	1.273

MATERIAL DESCRIPTION	USCS	AASHTO

Project No. 0600-102 **Client:** HNTB Corporation

Project: Scarborough Eastern Trail Project
Scarborough, ME

Location: BB-SPC-203 **Depth:** 25'-27' **Sample Number:** U-1B

R.W. Gillespie & Associates, Inc.

Biddeford, Maine

Remarks:

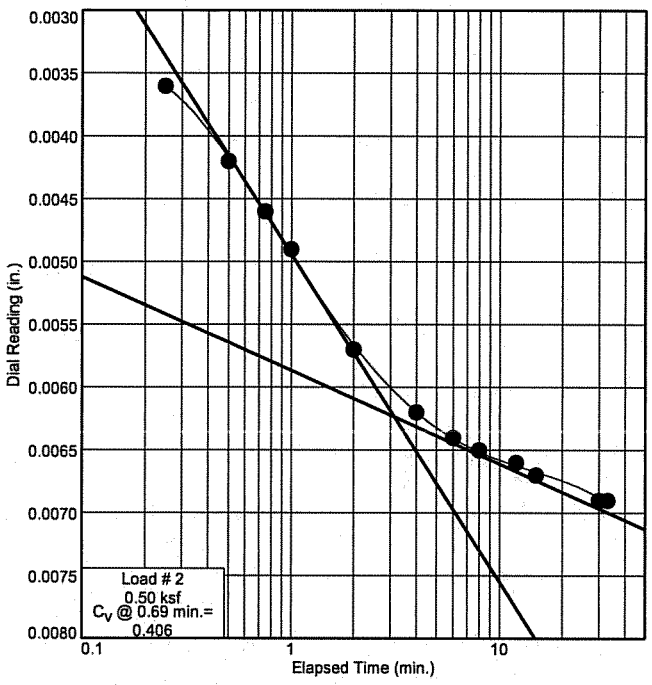
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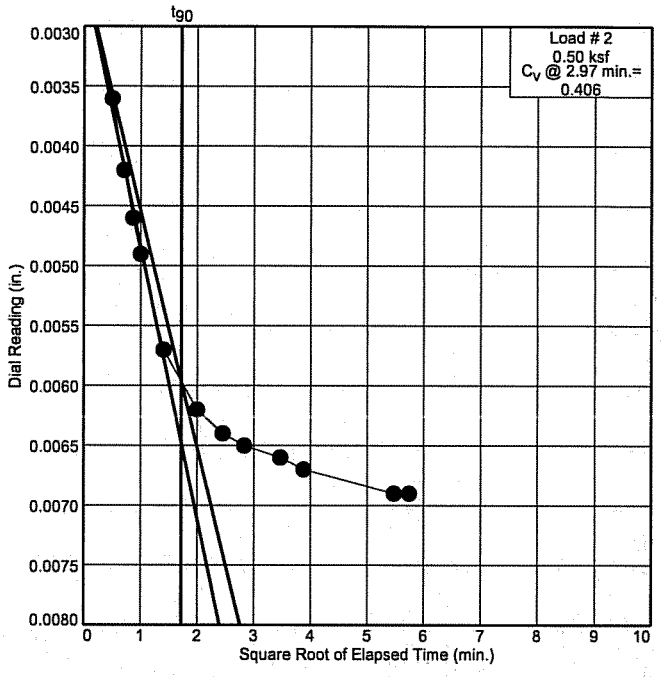
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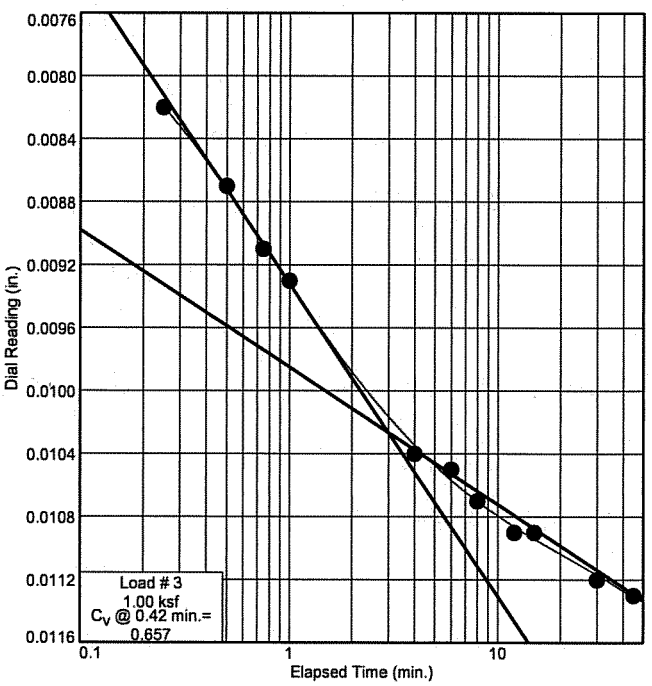
Dial Reading vs. Time

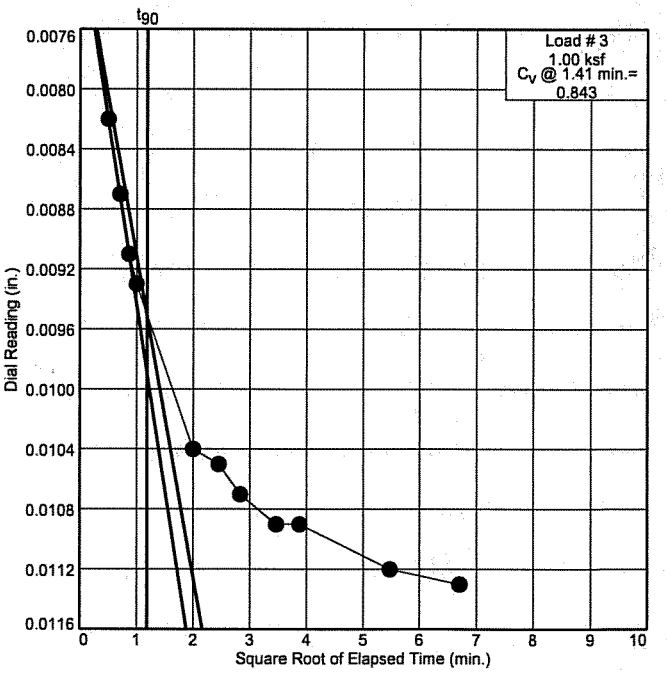
Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: BB-SPC-203 Depth: 25'-27' Sample Number: U-1









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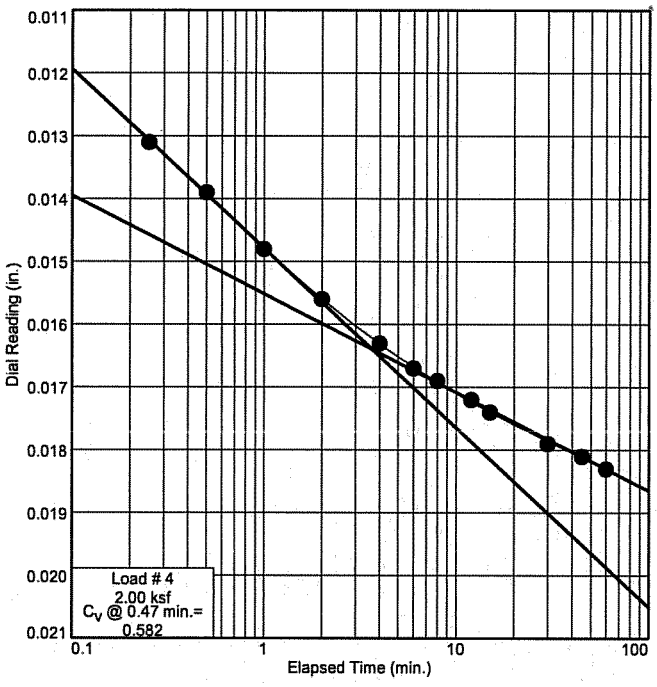
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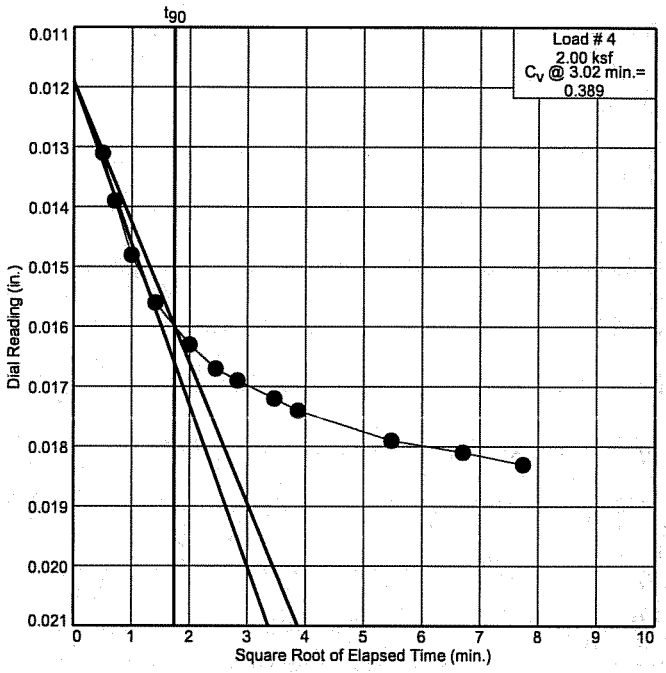
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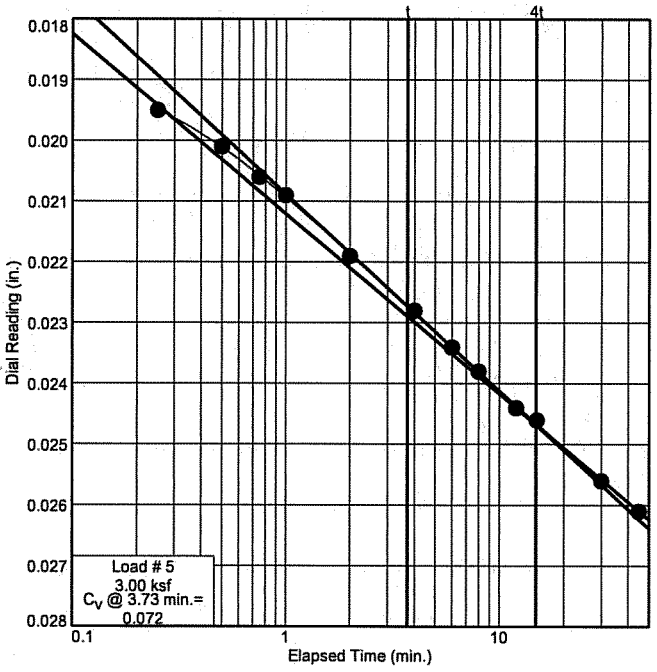
Dial Reading vs. Time

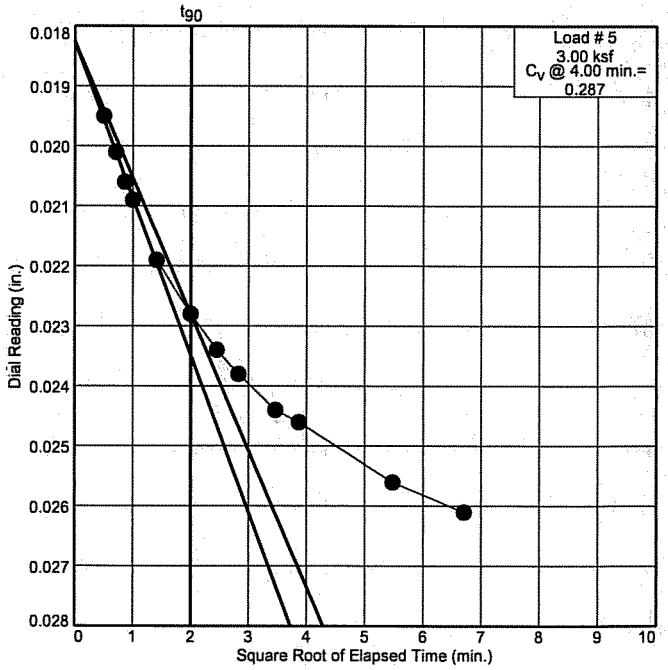
Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: BB-SPC-203 Depth: 25'-27' Sample Number: U-1









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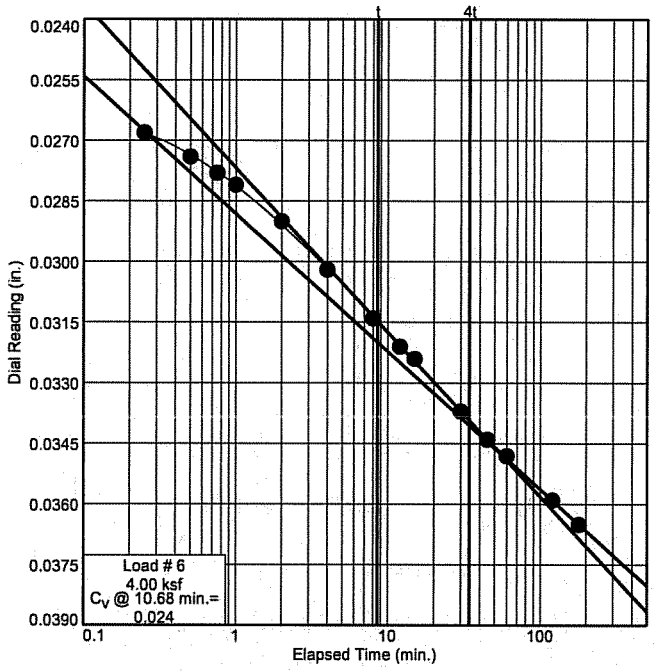
Biddeford, Maine

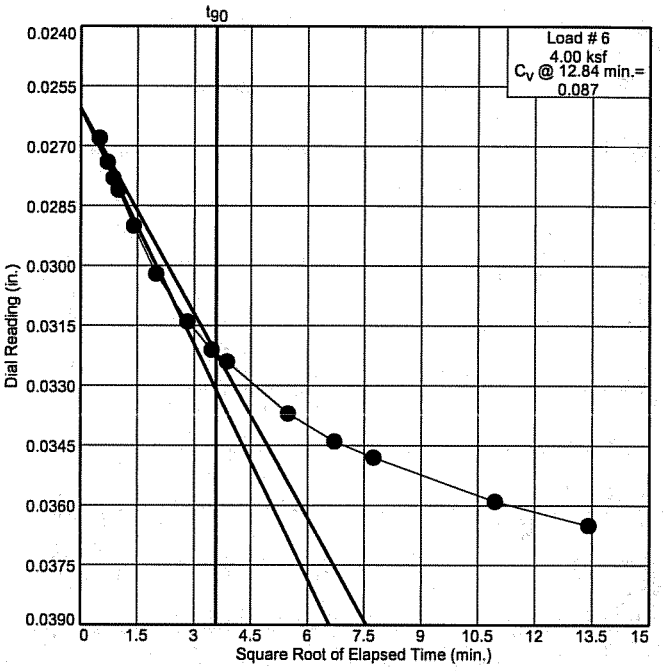
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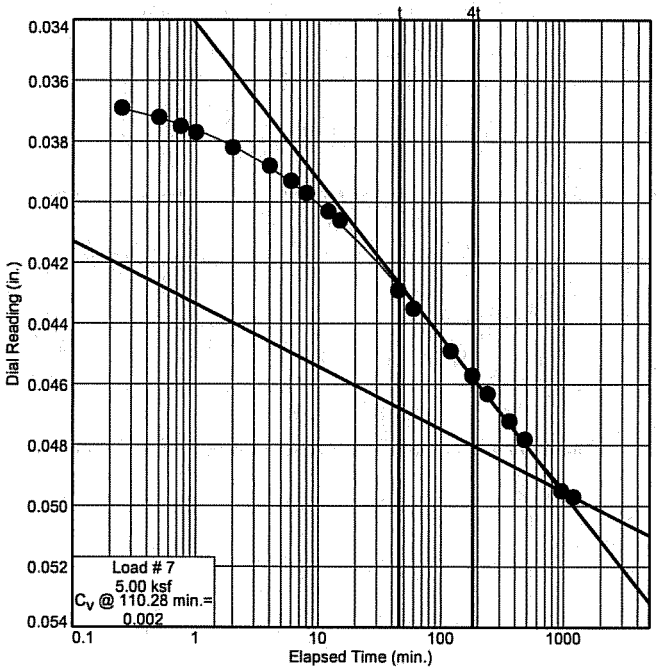
Dial Reading vs. Time

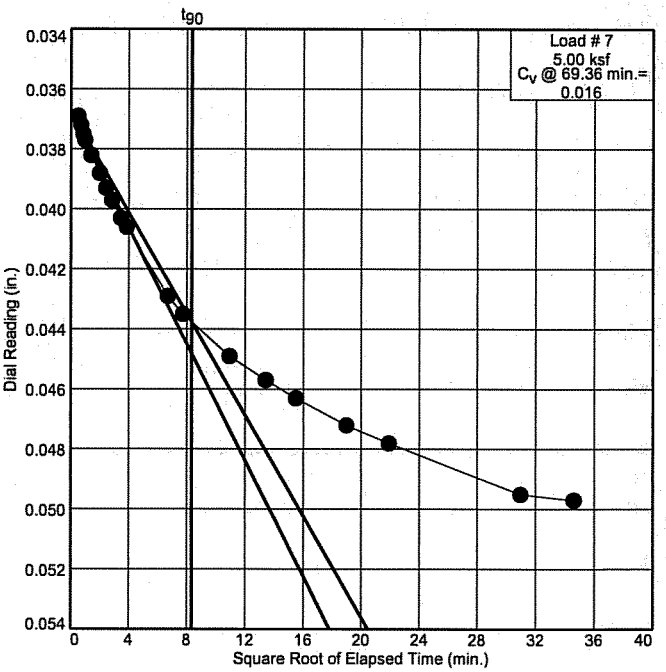
Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: BB-SPC-203 Depth: 25'-27' Sample Number: U-1









R.W. Gillespie & Associates, Inc.

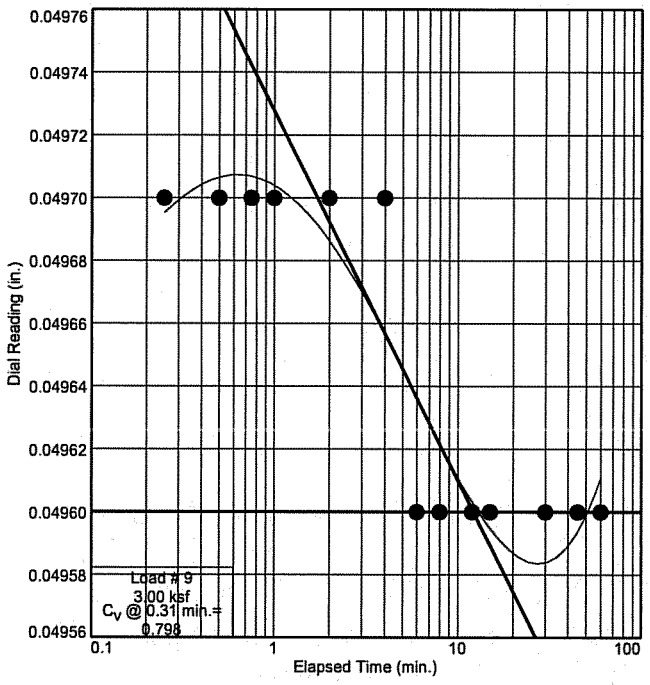
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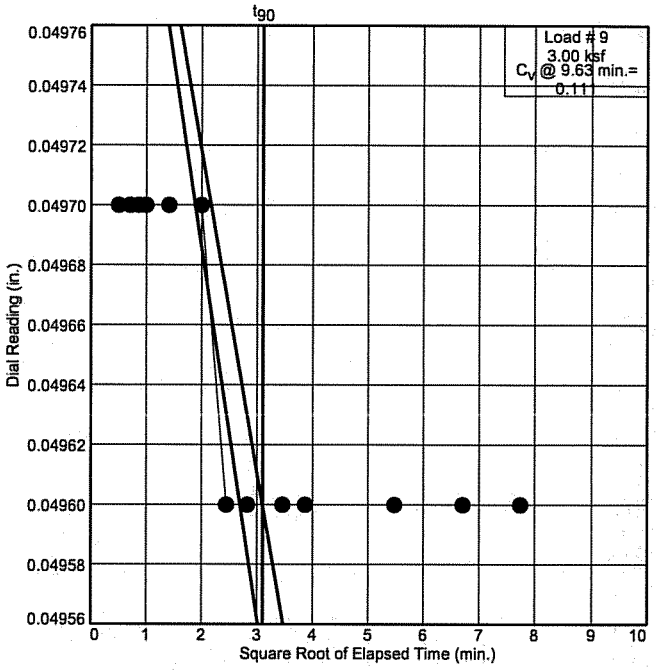
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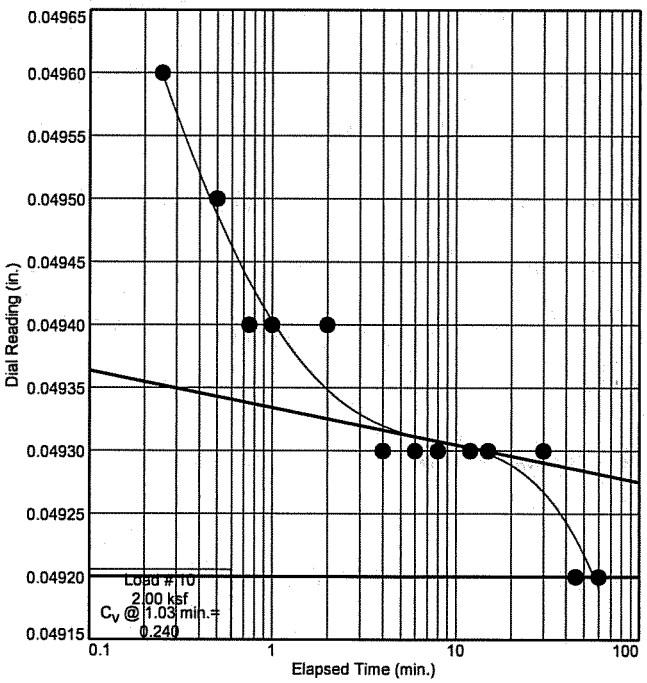
Dial Reading vs. Time

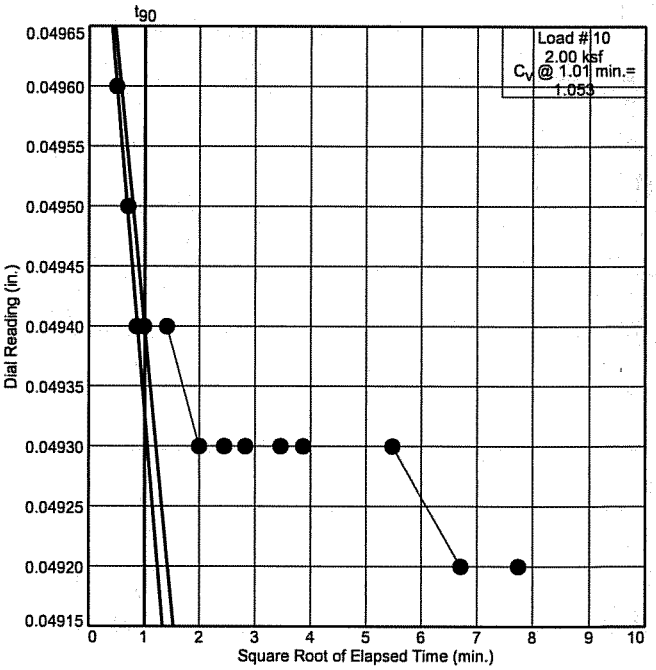
Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: BB-SPC-203 Depth: 25'-27' Sample Number: U-1









R.W. Gillespie & Associates, Inc.

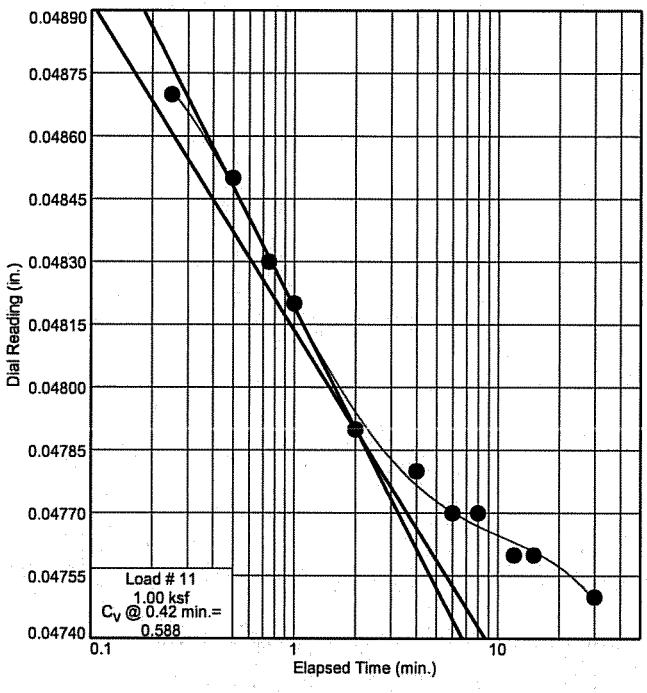
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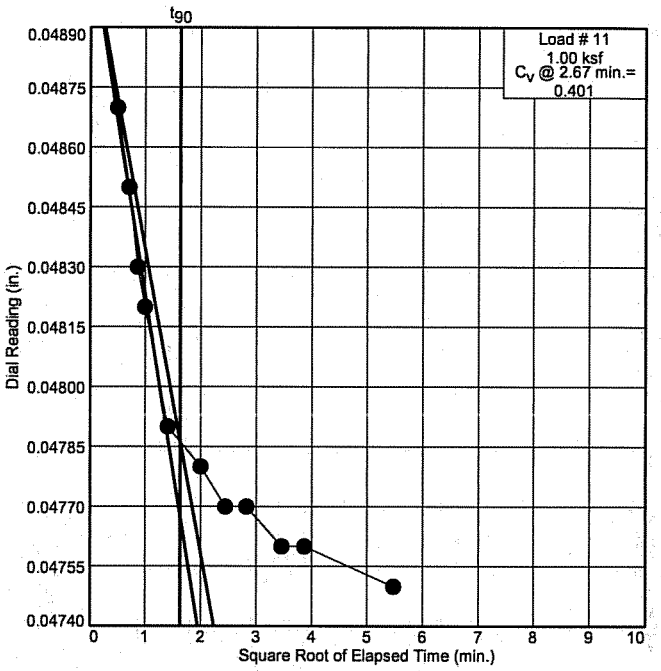
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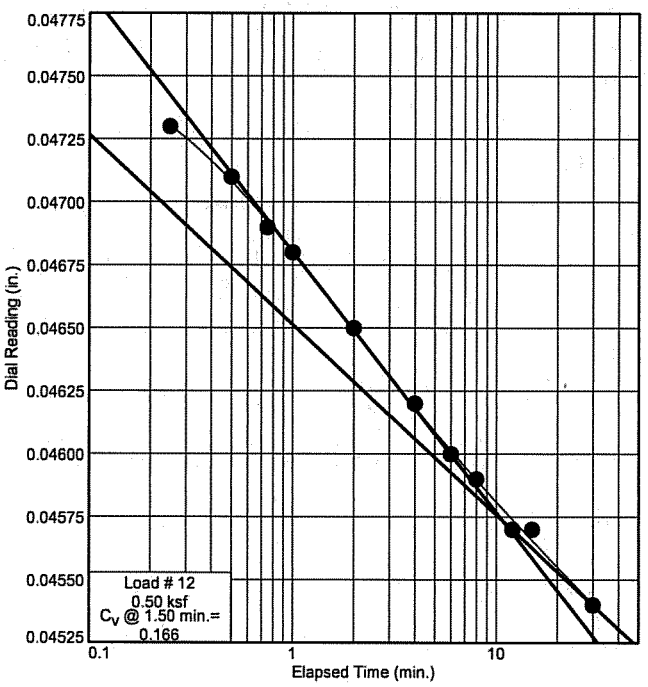
Dial Reading vs. Time

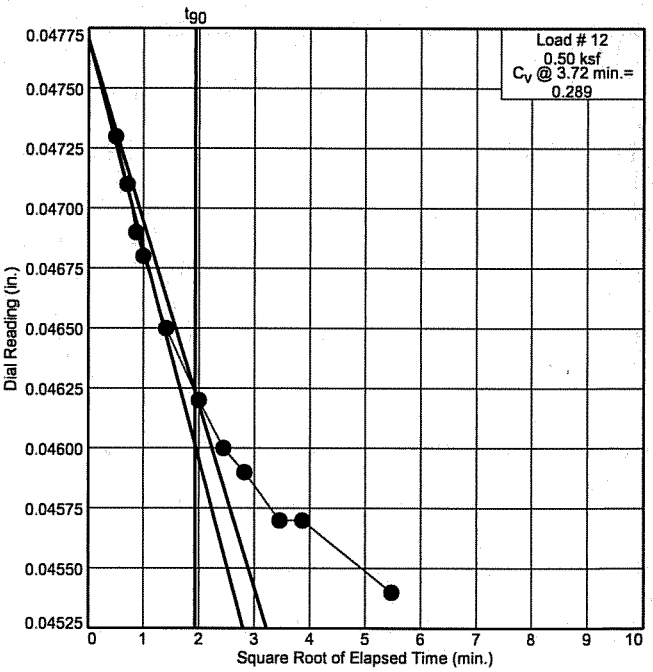
Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: BB-SPC-203 Depth: 25'-27' Sample Number: U-1









R.W. Gillespie & Associates, Inc.

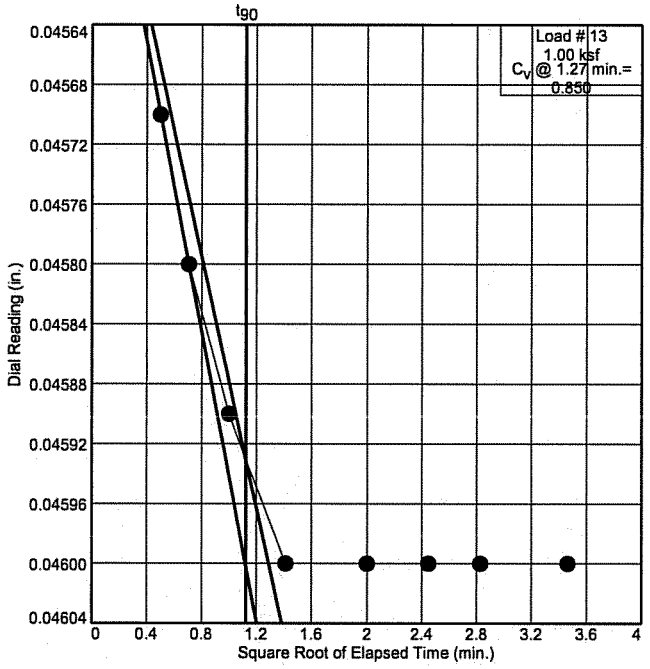
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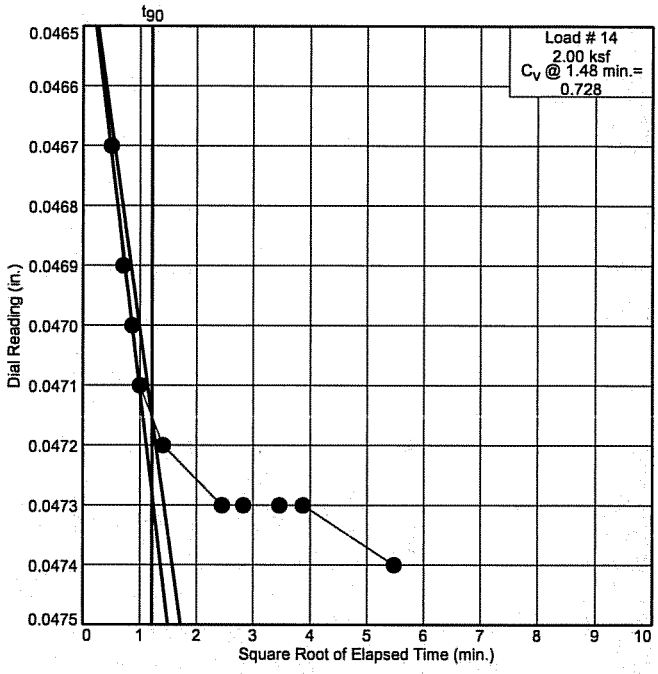
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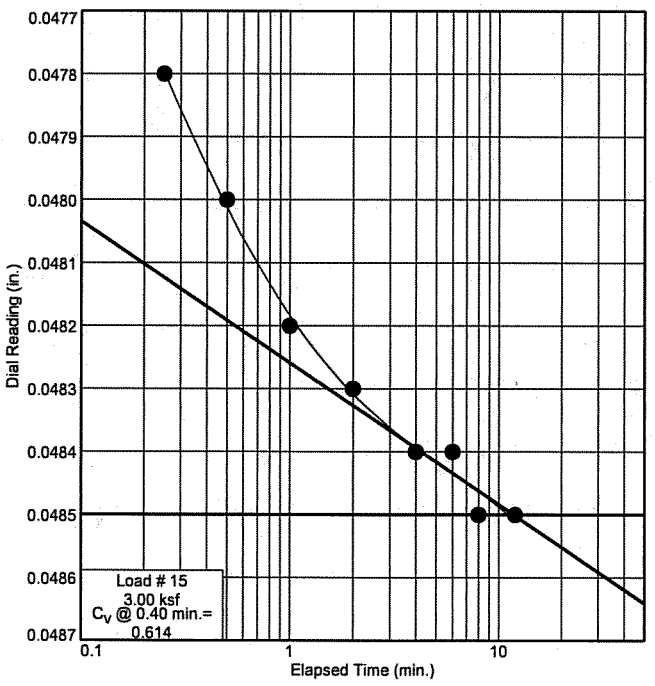
Dial Reading vs. Time

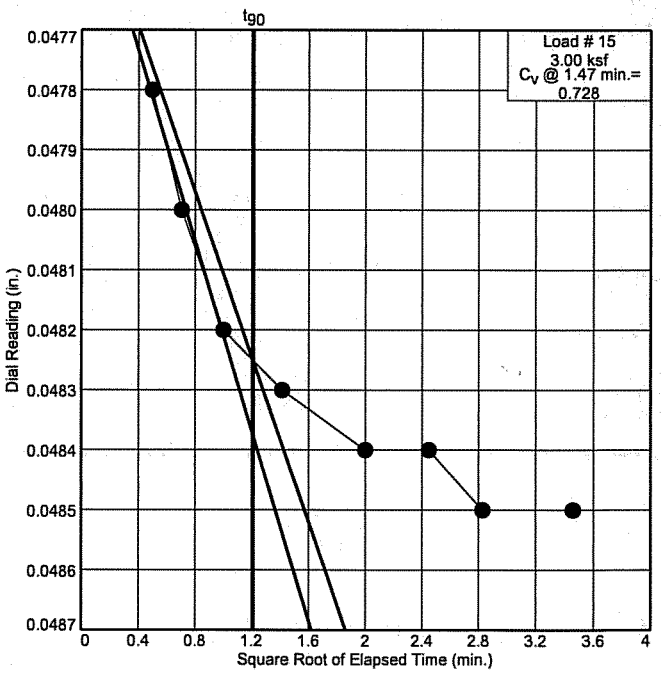
Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: BB-SPC-203 Depth: 25'-27' Sample Number: U-1









R.W. Gillespie & Associates, Inc.

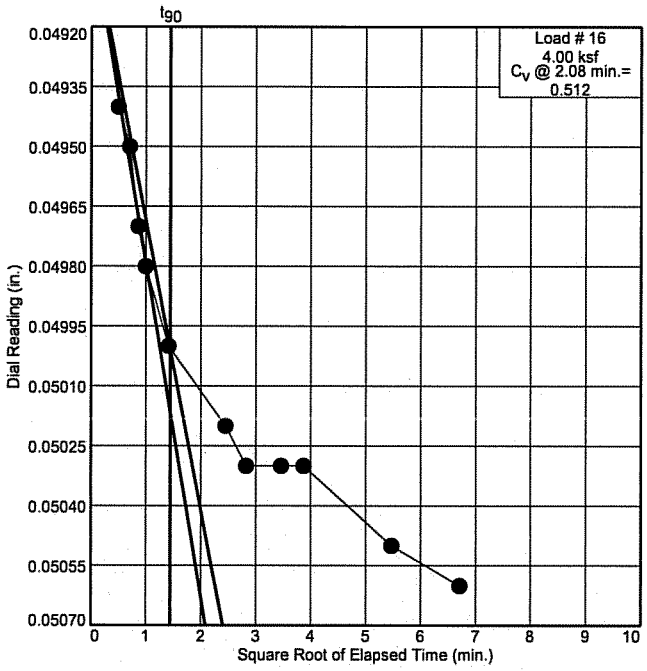
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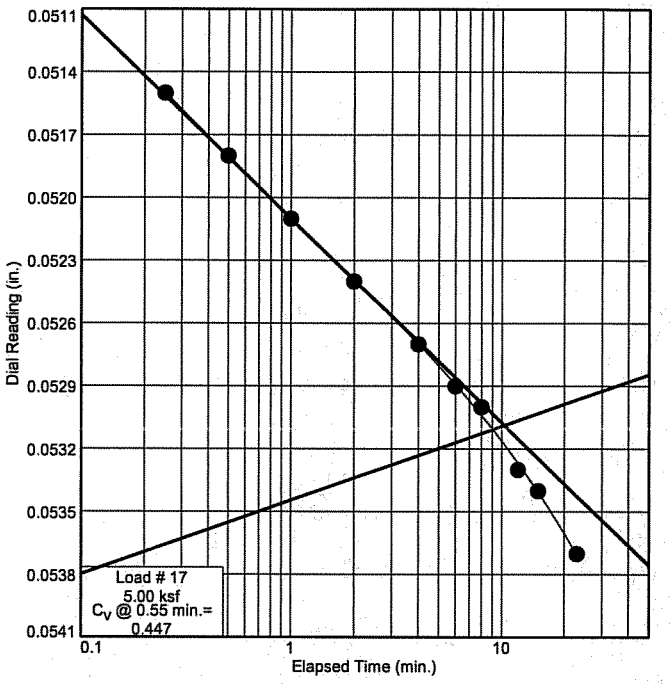
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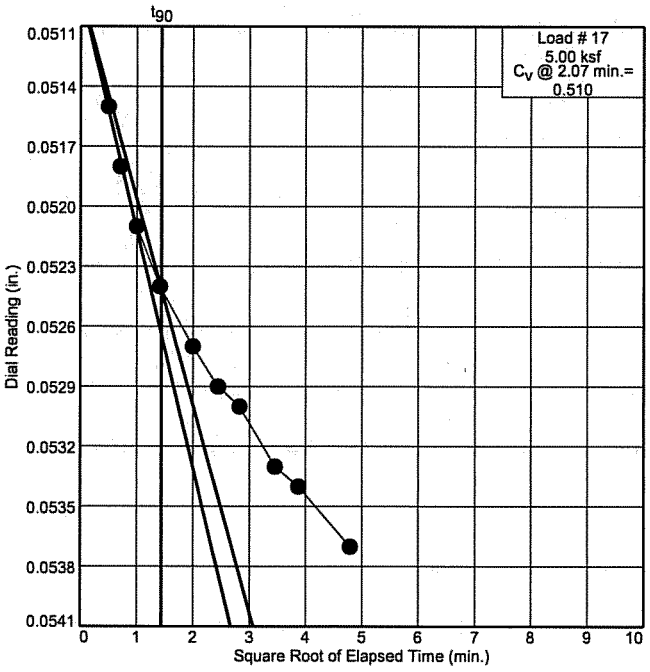
Dial Reading vs. Time

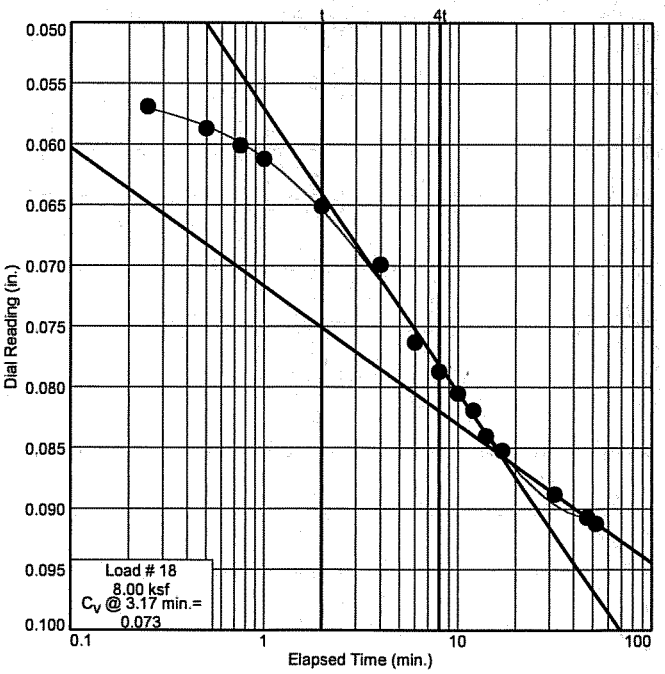
Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: BB-SPC-203 Depth: 25'-27' Sample Number: U-1









R.W. Gillespie & Associates, Inc.

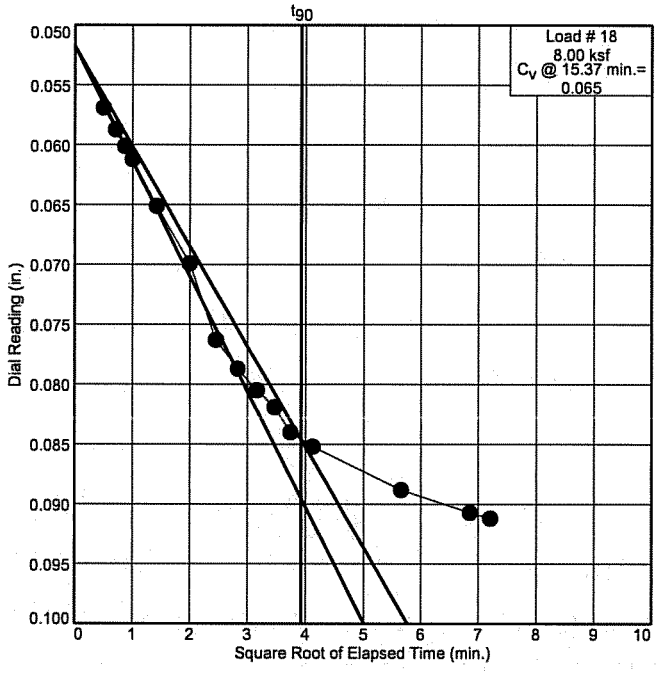
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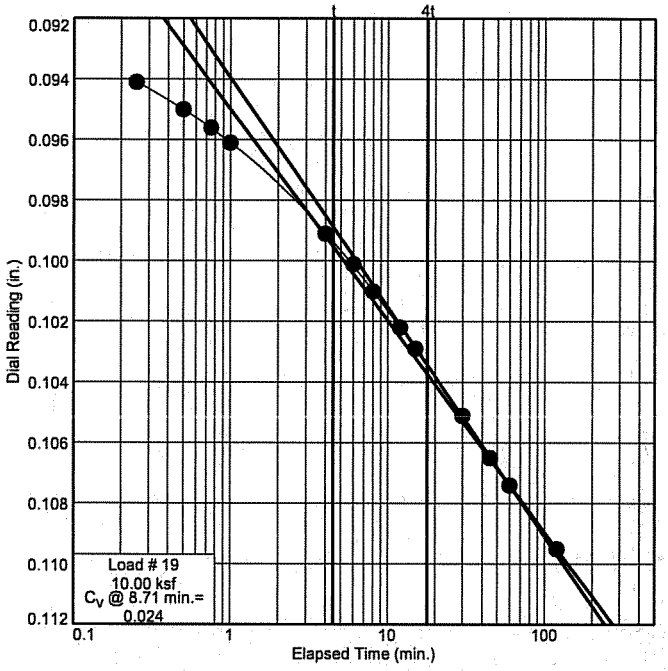
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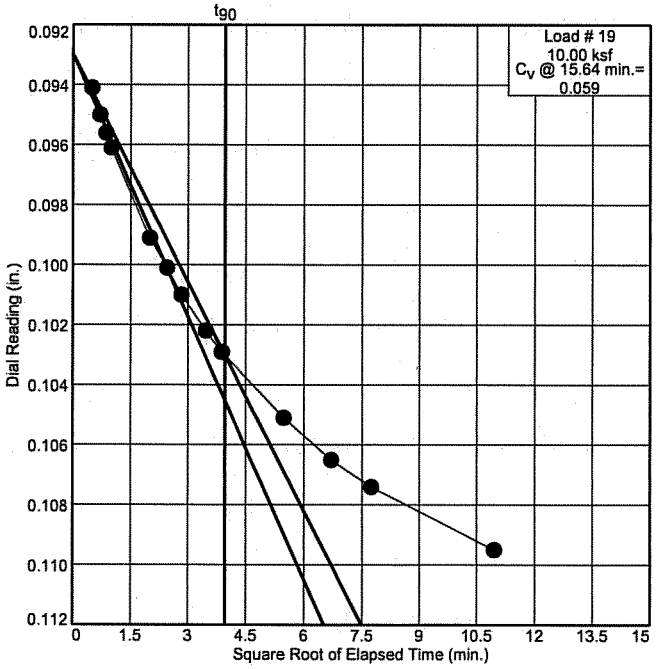
Dial Reading vs. Time

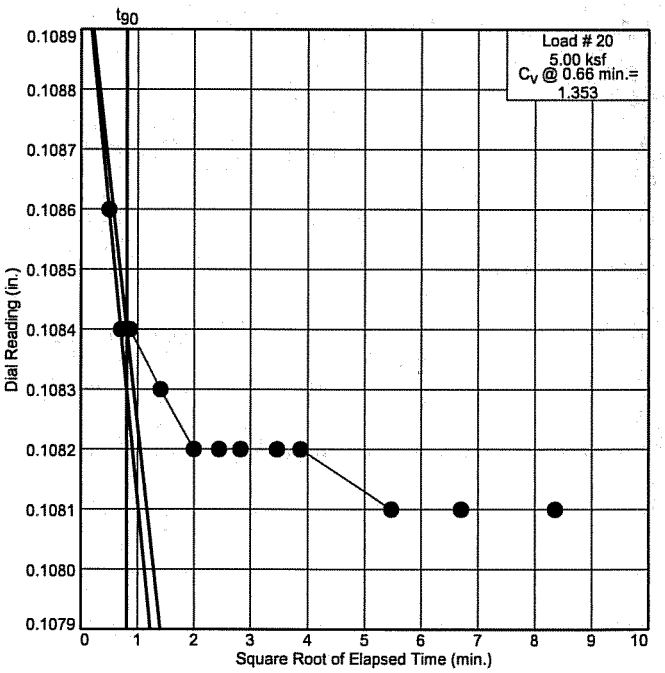
Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: BB-SPC-203 Depth: 25'-27' Sample Number: U-1









R.W. Gillespie & Associates, Inc.

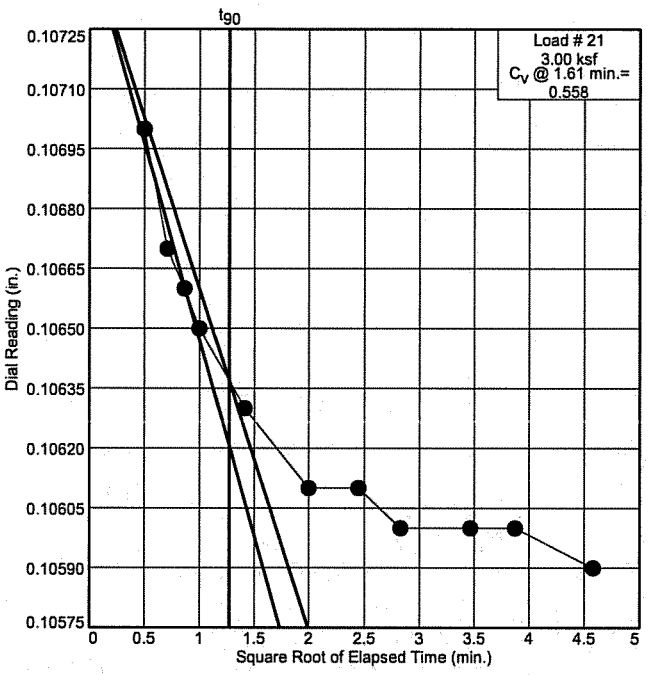
Biddeford, Maine

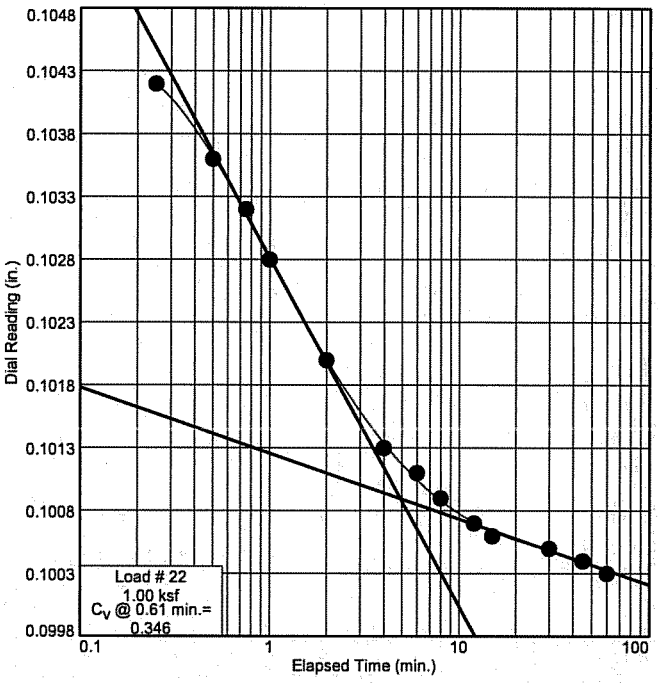
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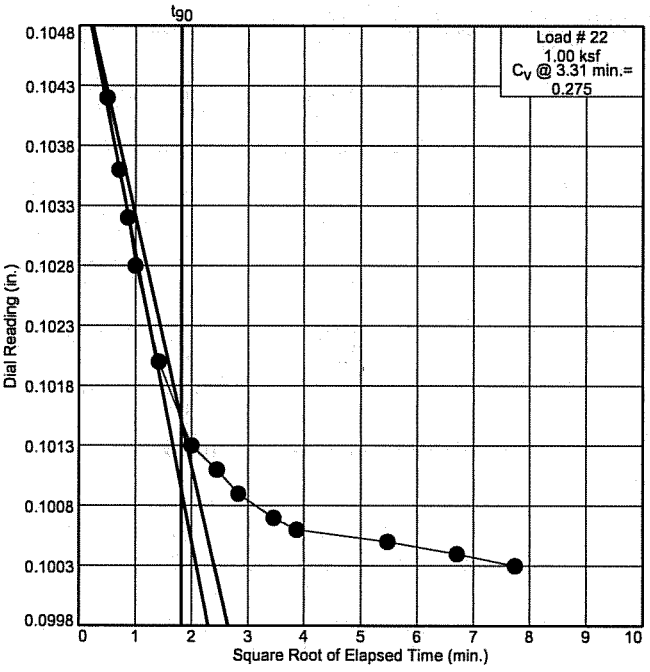
Dial Reading vs. Time

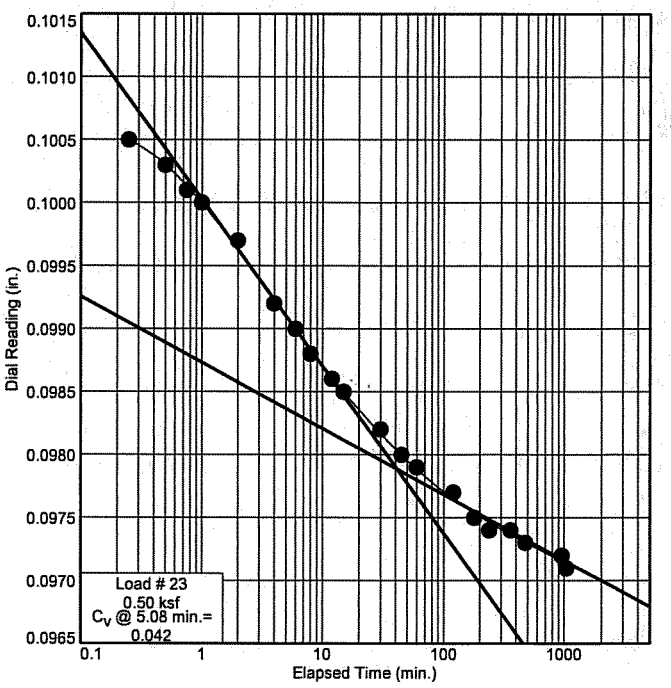
Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: BB-SPC-203 Depth: 25'-27' Sample Number: U-1









R.W. Gillespie & Associates, Inc.

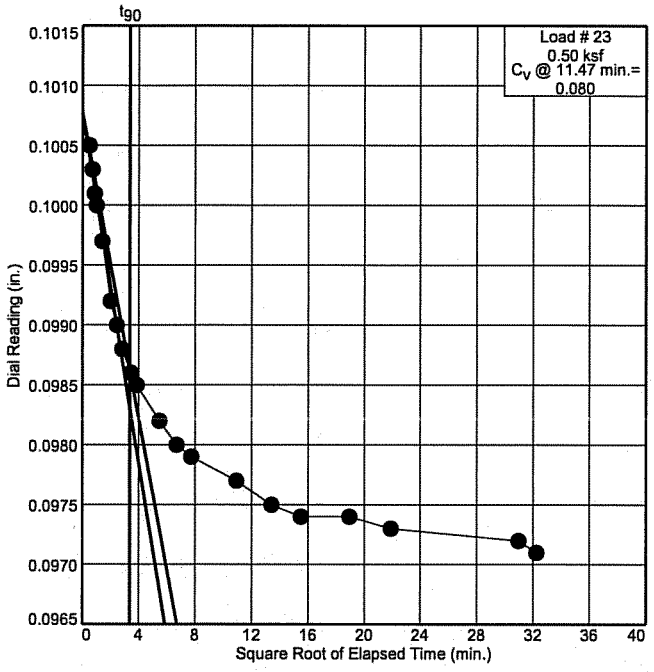
Biddeford, Maine

Lab No. 14438

Dial Reading vs. Time

Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: BB-SPC-203 Depth: 25'-27' Sample Number: U-1

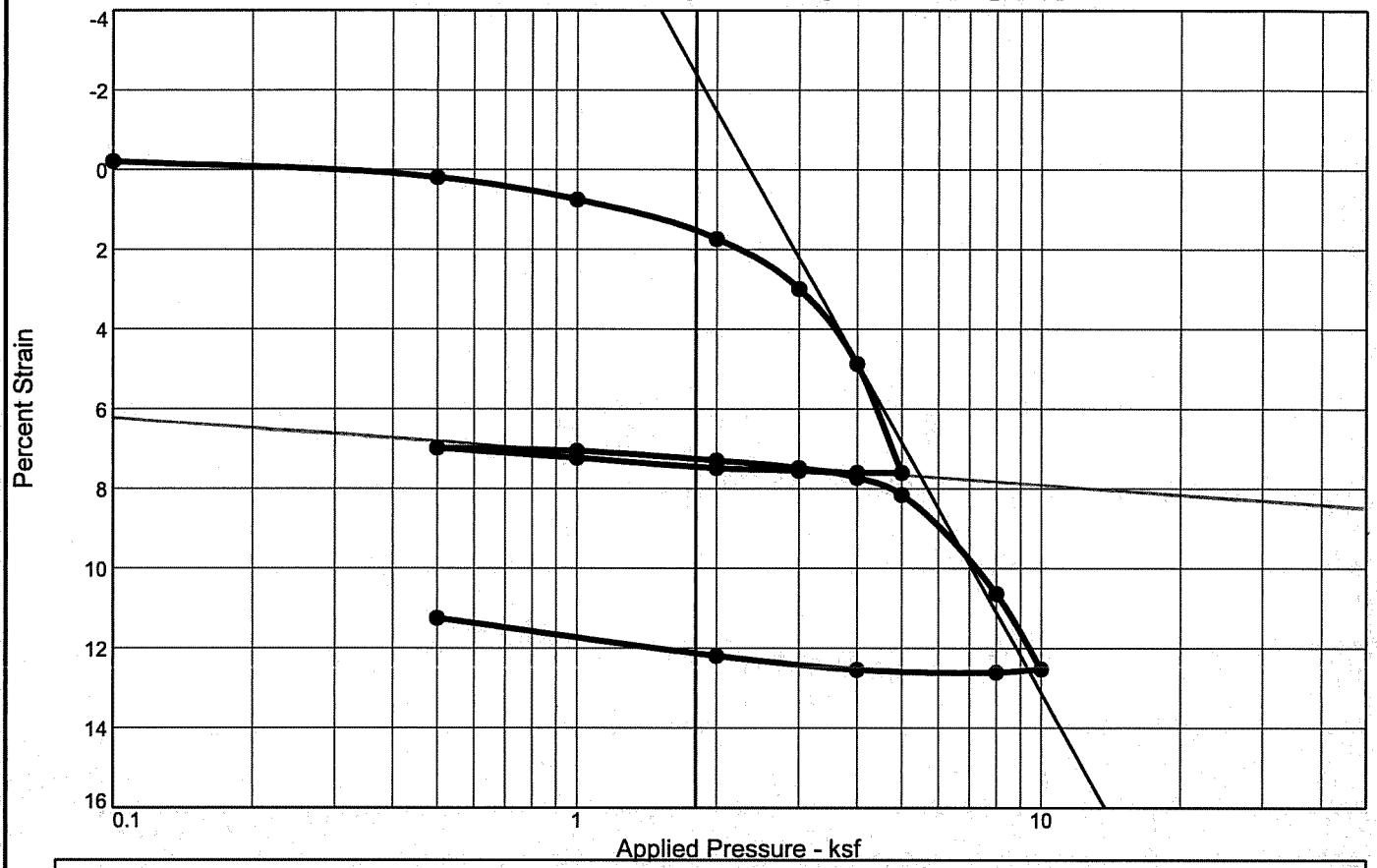


R.W. Gillespie & Associates, Inc.

Biddeford, Maine

Lab No. 14439

CONSOLIDATION TEST REPORT



Coefficients of Consolidation and Secondary Consolidation

No.	Load (ksf)	C_v (ft.2/day)	C_α	No.	Load (ksf)	C_v (ft.2/day)	C_α	No.	Load (ksf)	C_v (ft.2/day)	C_α
2	0.50	0.310		9	3.00	0.340		16	4.00	0.845	
3	1.00	0.274		10	2.00	0.393		17	5.00	0.181	
4	2.00	0.325		11	1.00	0.331		18	8.00	0.052	
5	3.00	0.147		12	0.50	0.140		19	10.00	0.064	
6	4.00	0.057	0.017	13	1.00	0.208		21	4.00	0.677	
7	5.00	0.008		14	2.00	0.376		22	2.00	0.374	
8	4.00	0.006		15	3.00	0.383		23	0.50	0.058	

Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_r	Initial Void Ratio
Saturation	Moisture									
93.8 %	39.7 %	79.4			2.76		2.9	0.45	0.07	1.169

MATERIAL DESCRIPTION	USCS	AASHTO

Project No. 0600-102 **Client:** HNTB Corporation

Project: Scarborough Eastern Trail Project
Scarborough, ME

Location: BB-SPC-203 **Depth:** 40'-42' **Sample Number:** ~~U-1~~ U-28

R.W. Gillespie & Associates, Inc.

Biddeford, Maine

Remarks:

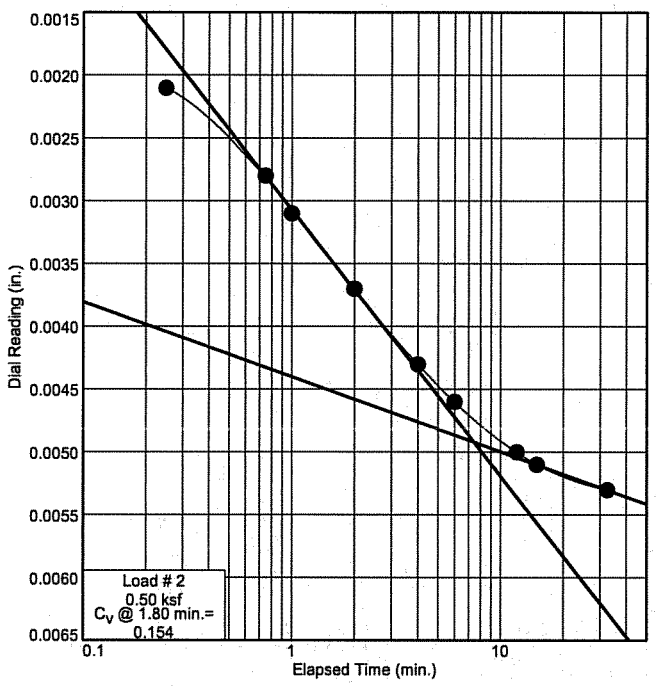
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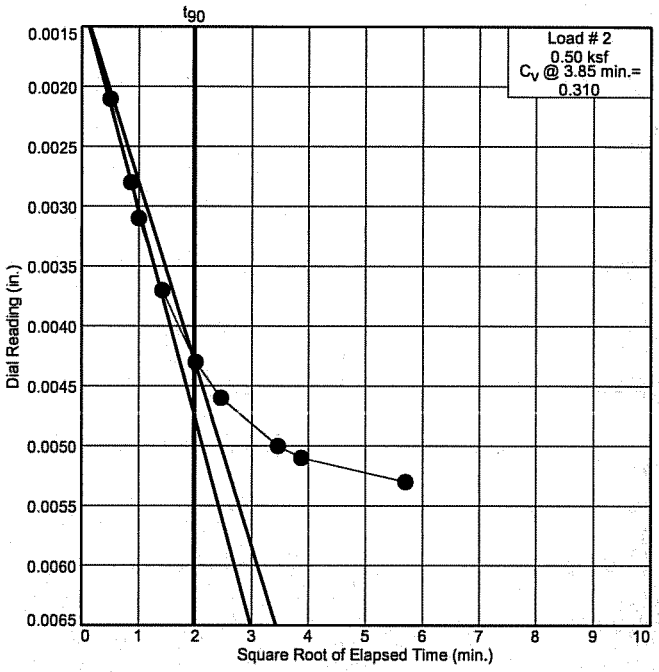
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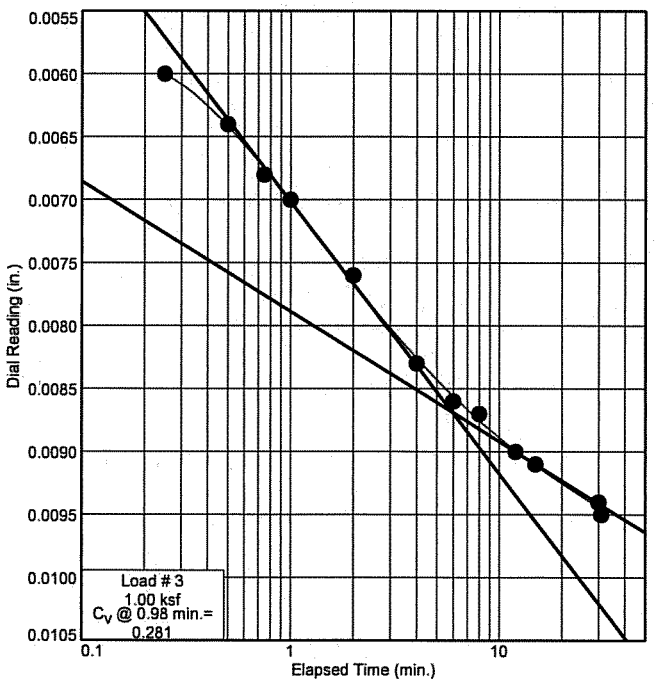
Dial Reading vs. Time

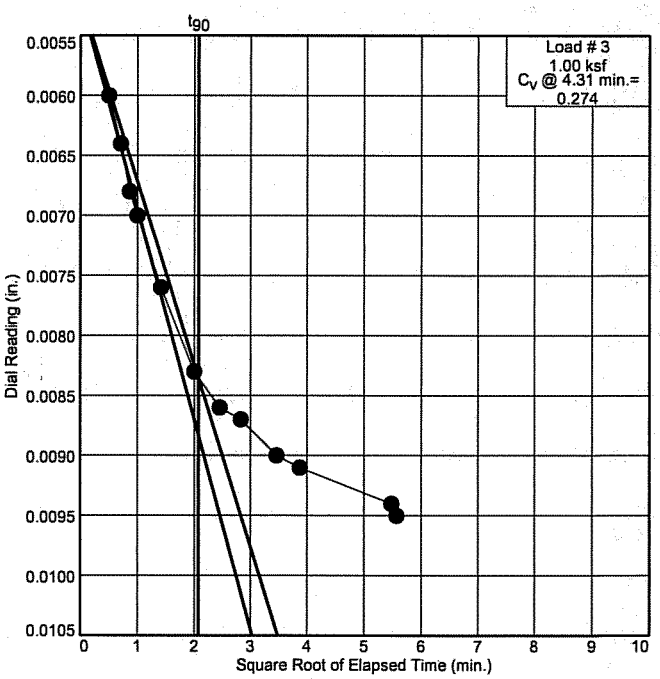
Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: BB-SPC-203 Depth: 40'-42' Sample Number: U-1









R.W. Gillespie & Associates, Inc.

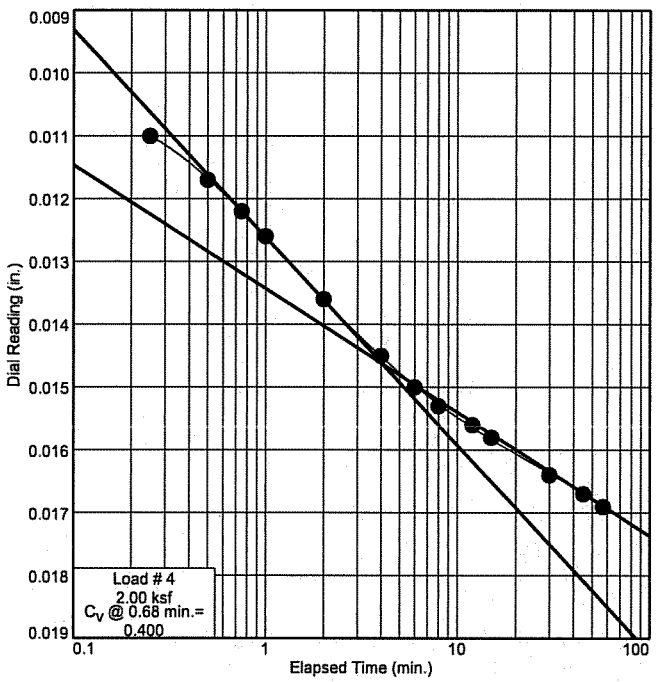
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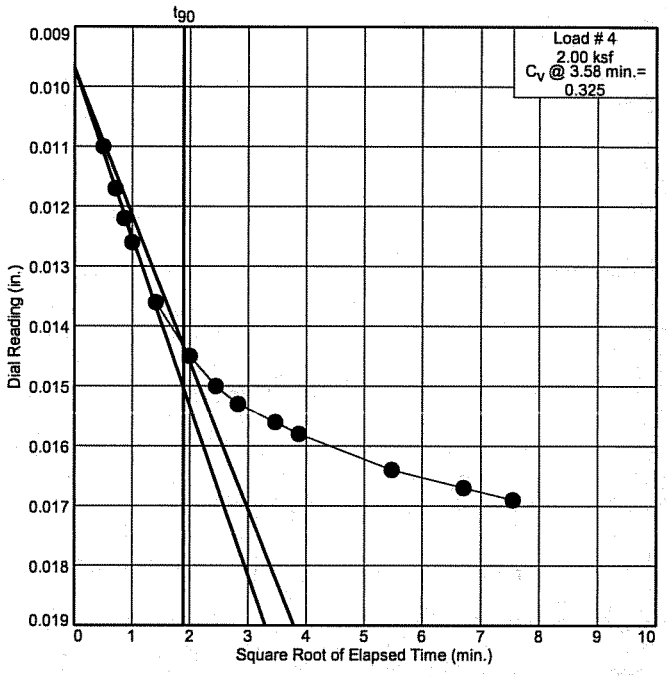
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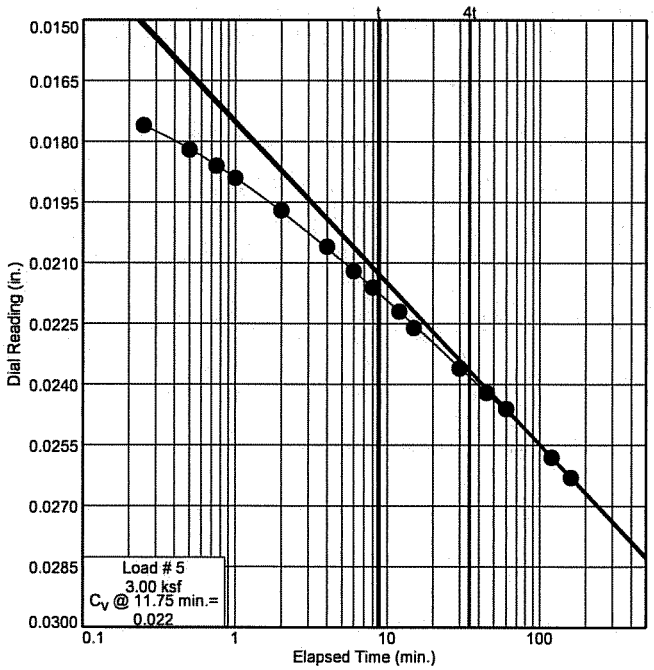
Dial Reading vs. Time

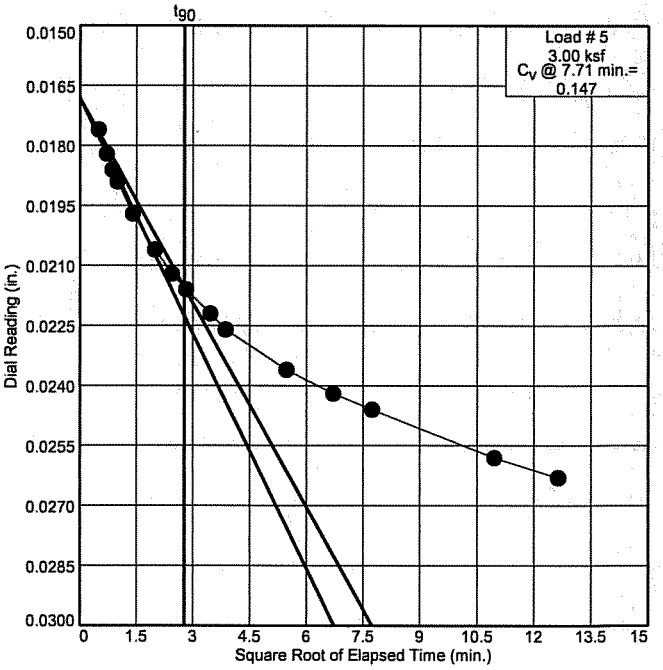
Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: BB-SPC-203 Depth: 40'-42' Sample Number: U-1









R.W. Gillespie & Associates, Inc.

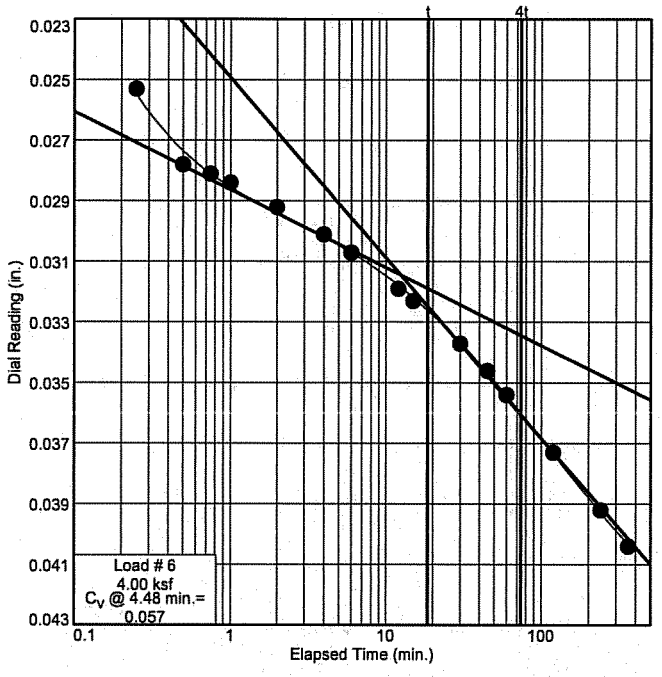
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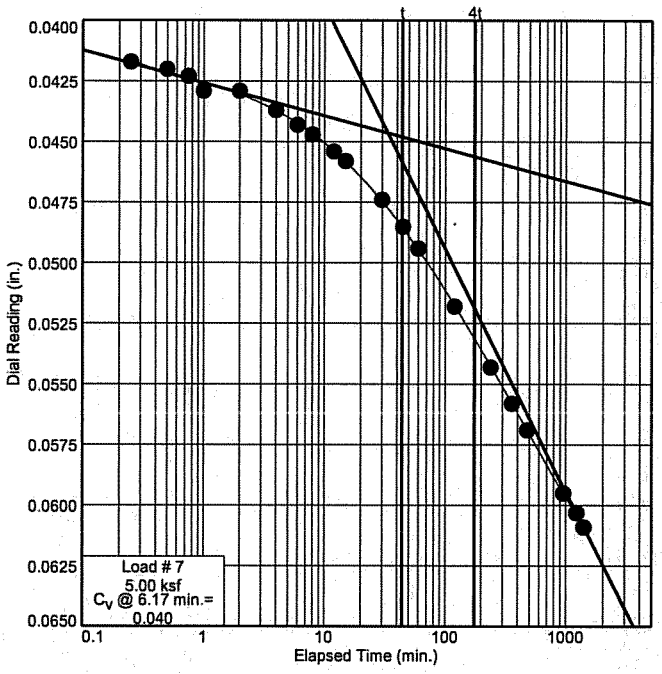
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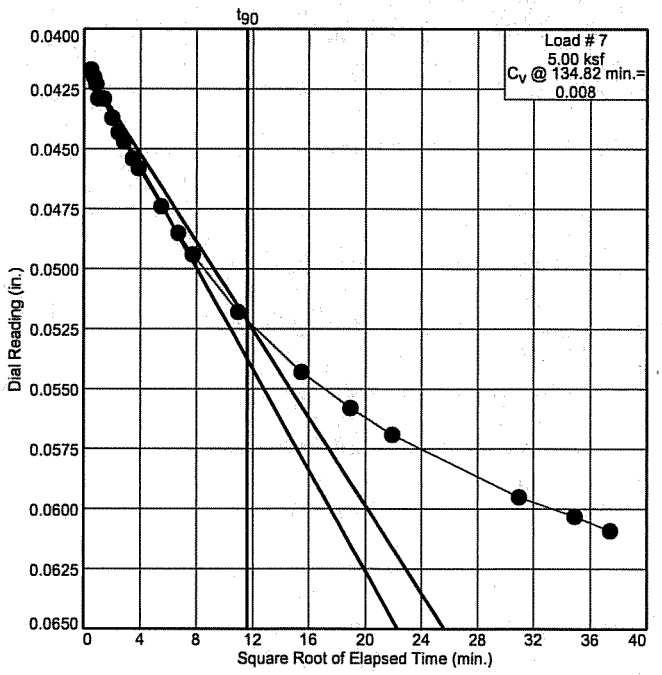
Dial Reading vs. Time

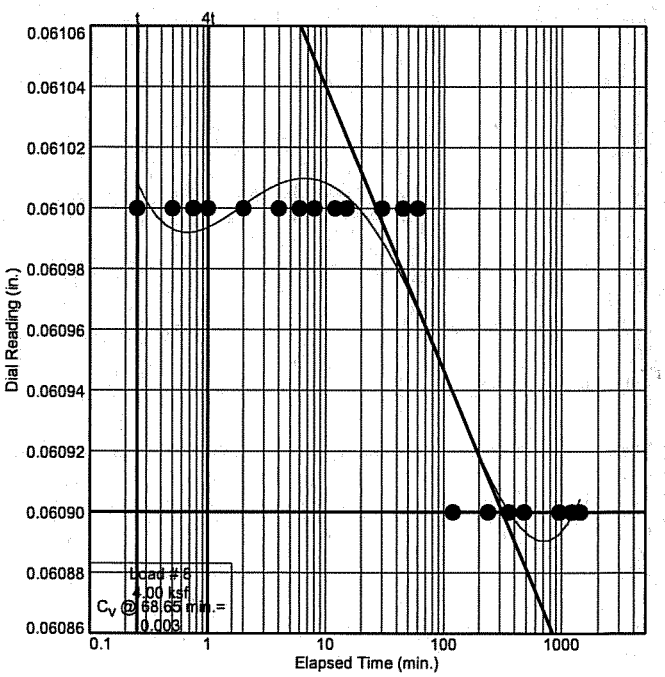
Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: BB-SPC-203 Depth: 40'-42' Sample Number: U-1









R.W. Gillespie & Associates, Inc.

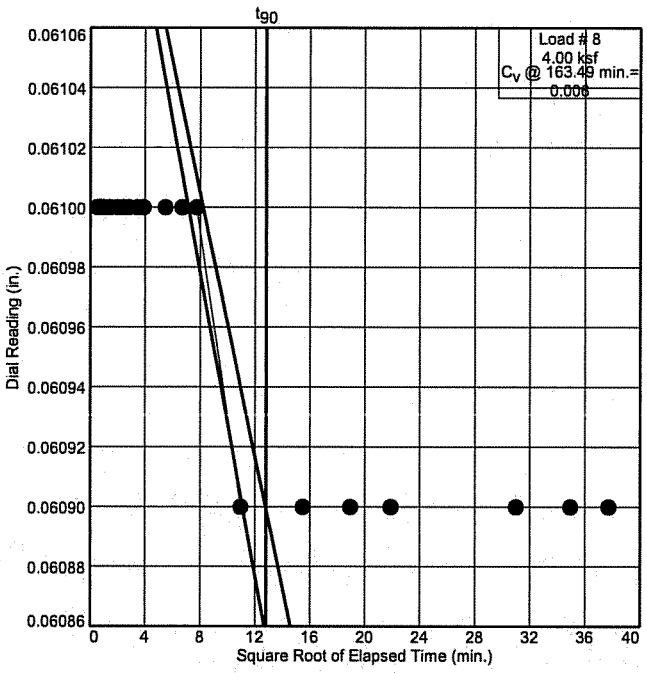
Biddeford, Maine

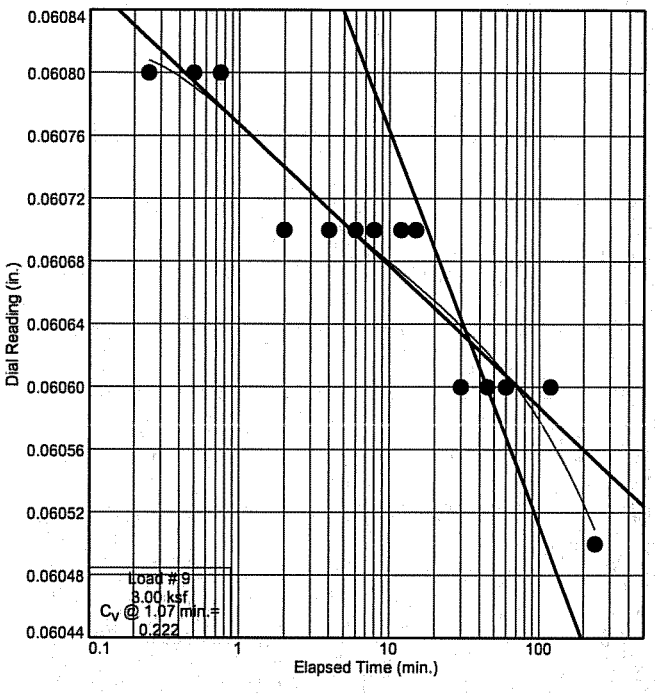
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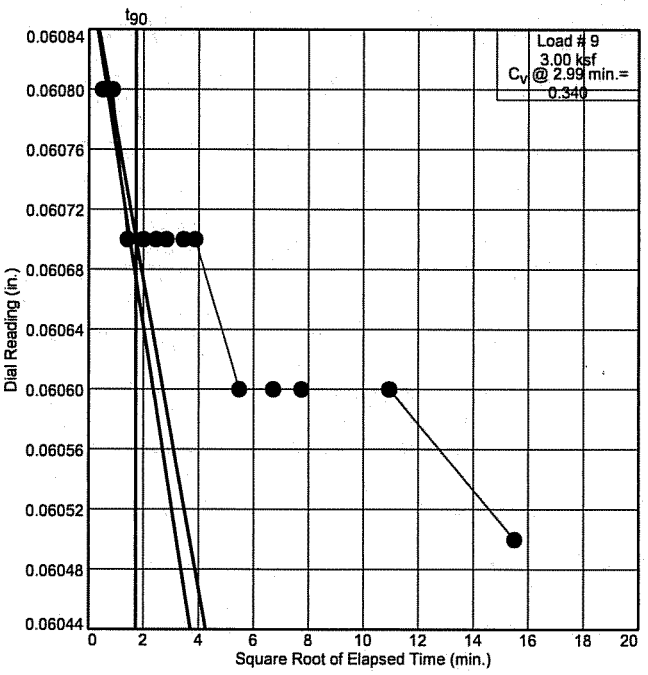
Dial Reading vs. Time

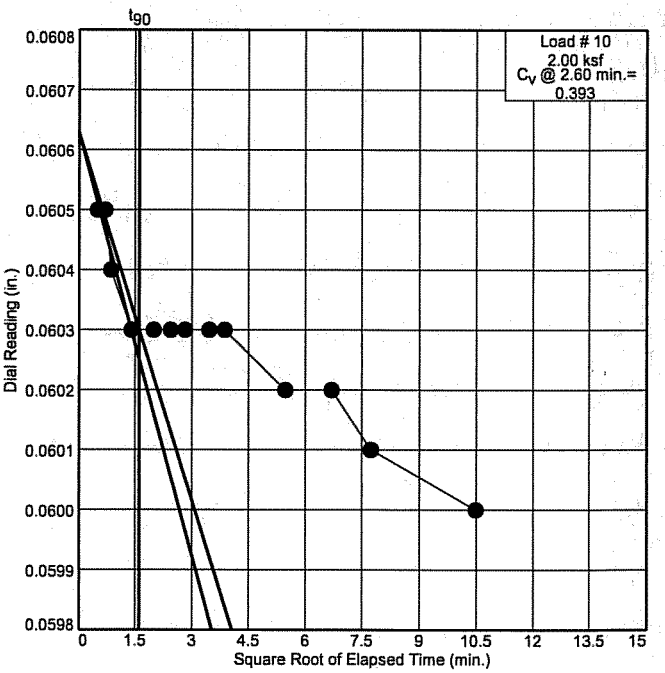
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Project: Scarborough Eastern Trail Project

Location: BB-SPC-203 Depth: 40'-42' Sample Number: U-1









R.W. Gillespie & Associates, Inc.

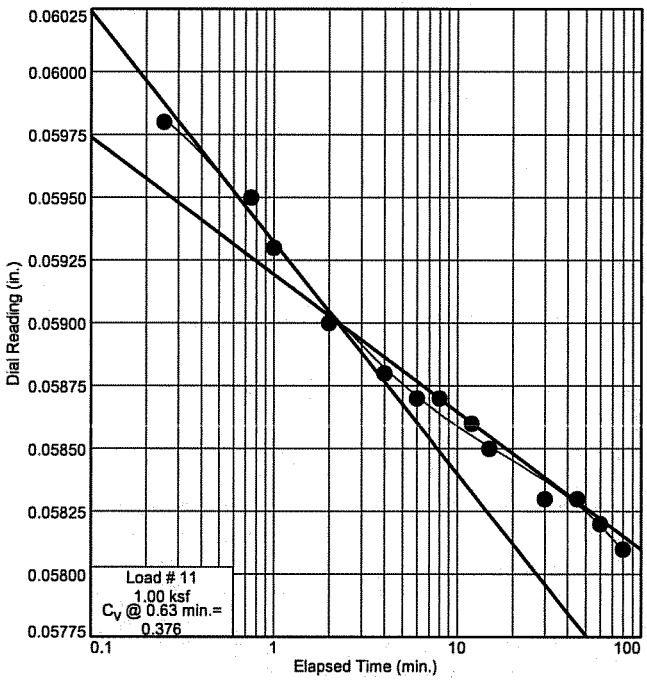
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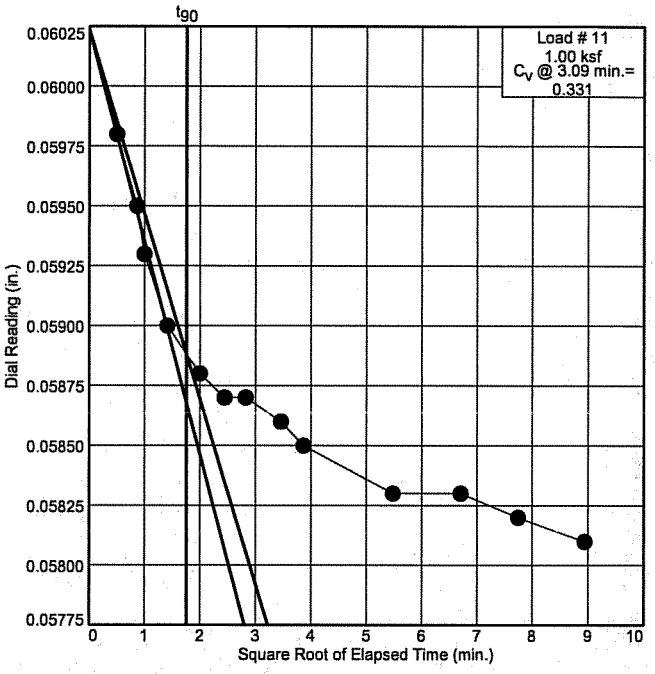
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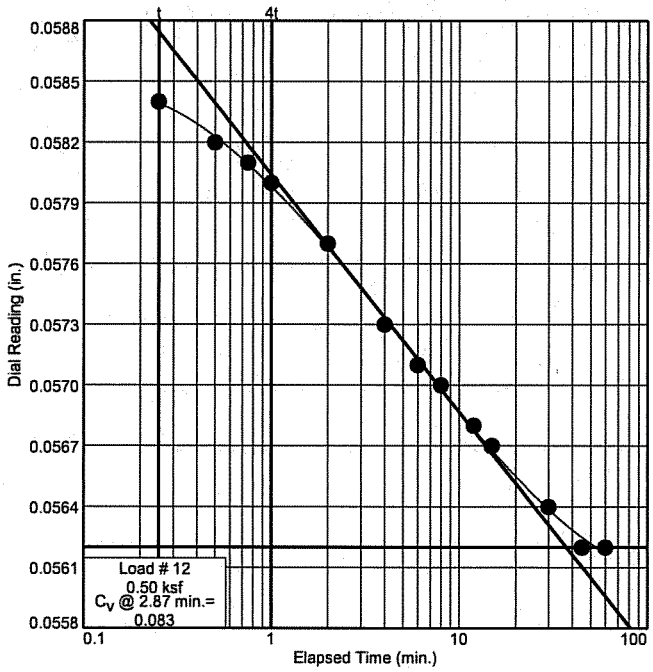
Dial Reading vs. Time

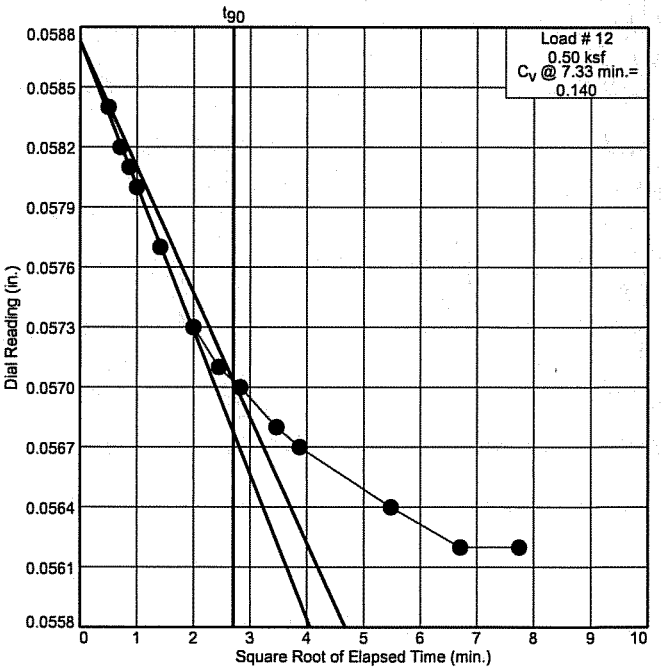
Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: BB-SPC-203 Depth: 40'-42' Sample Number: U-1









R.W. Gillespie & Associates, Inc.

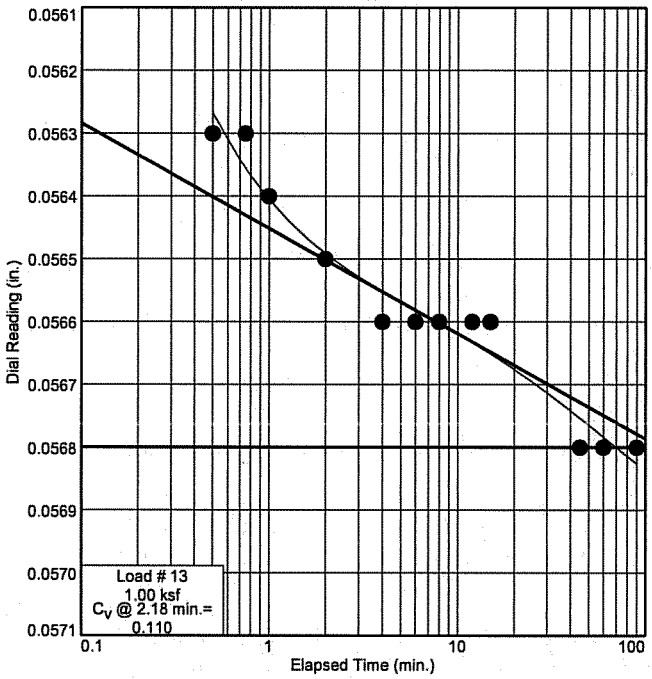
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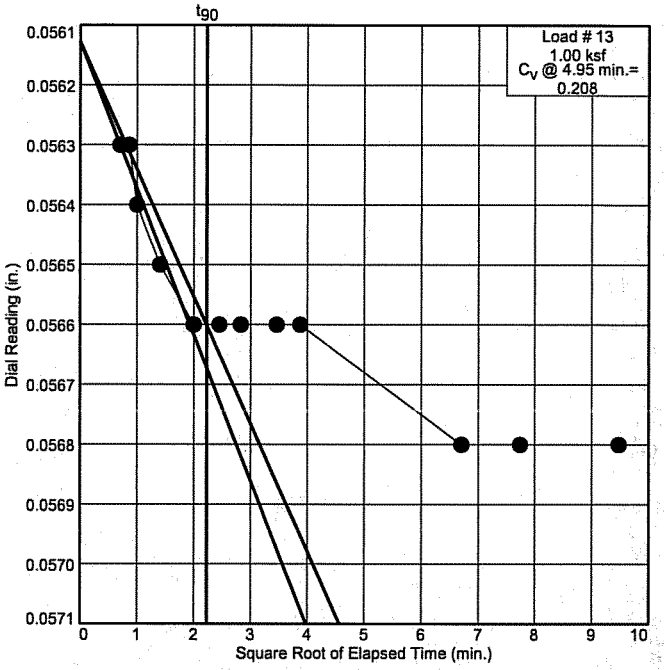
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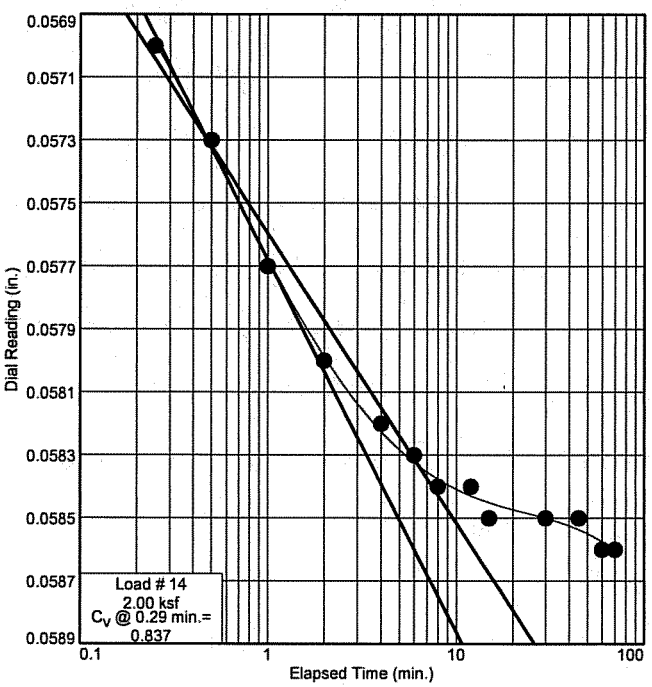
Dial Reading vs. Time

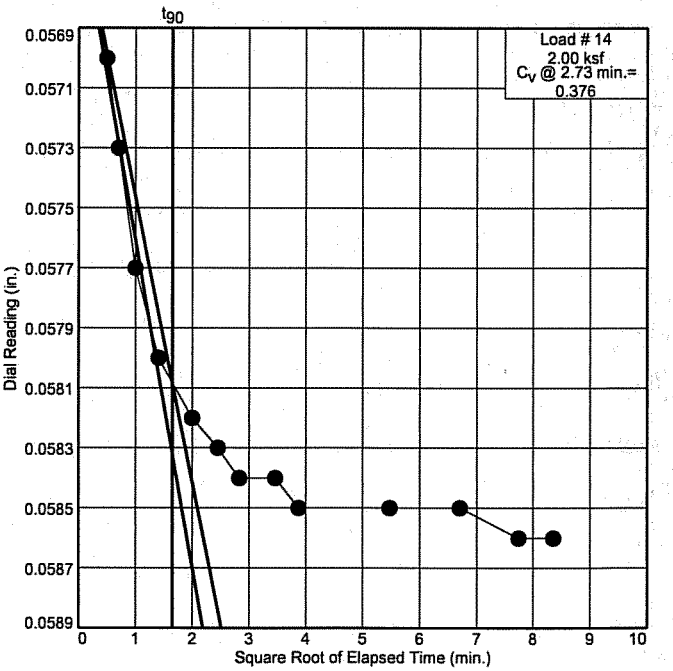
Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: BB-SPC-203 Depth: 40'-42' Sample Number: U-1









R.W. Gillespie & Associates, Inc.

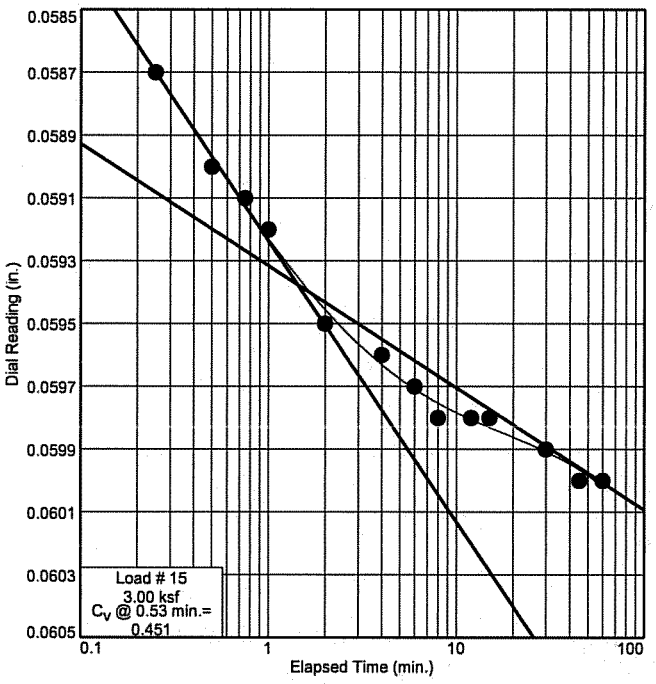
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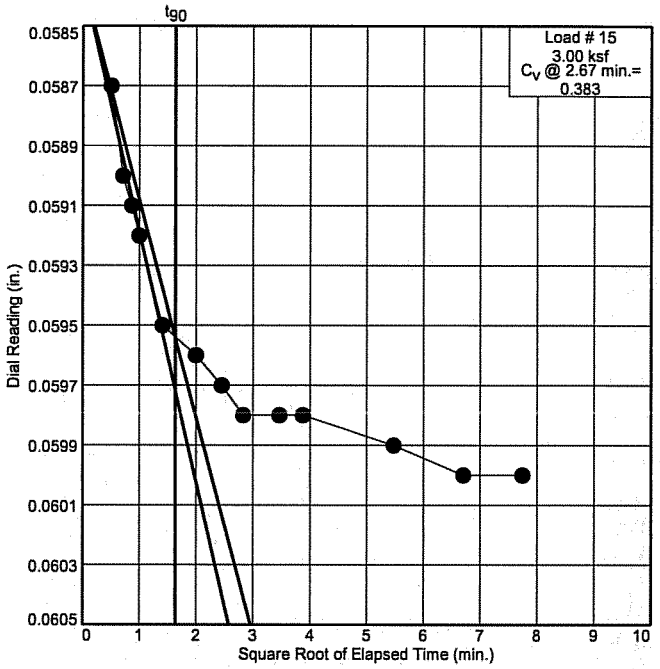
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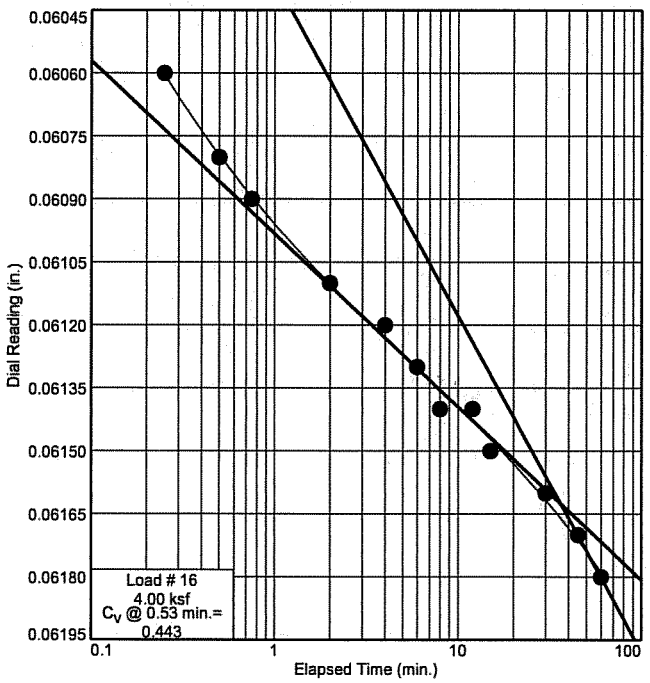
Dial Reading vs. Time

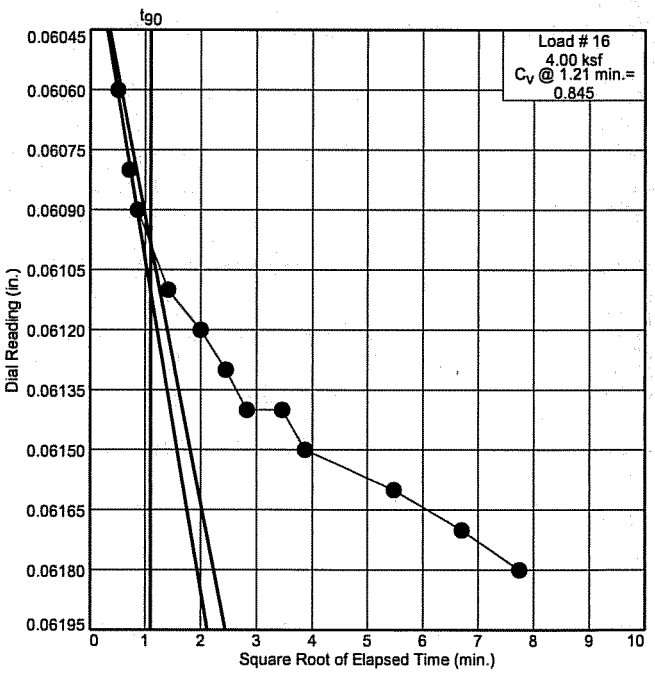
Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: BB-SPC-203 Depth: 40'-42' Sample Number: U-1









R.W. Gillespie & Associates, Inc.

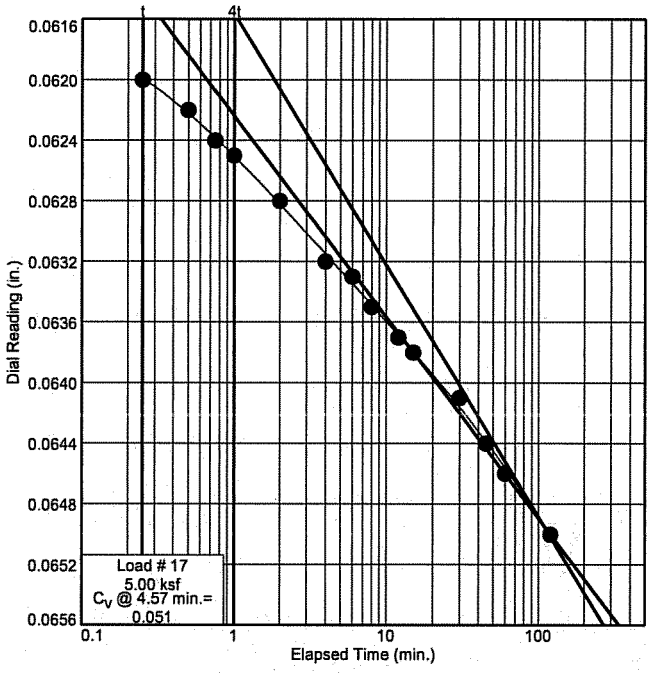
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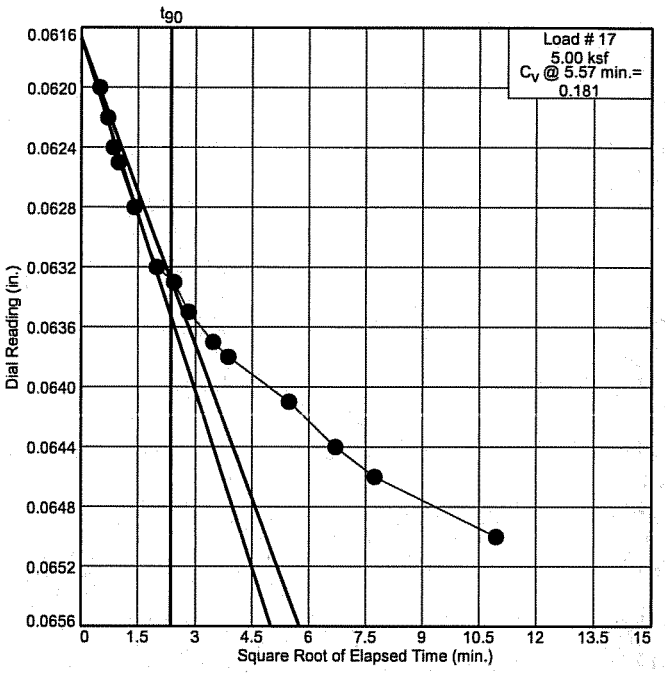
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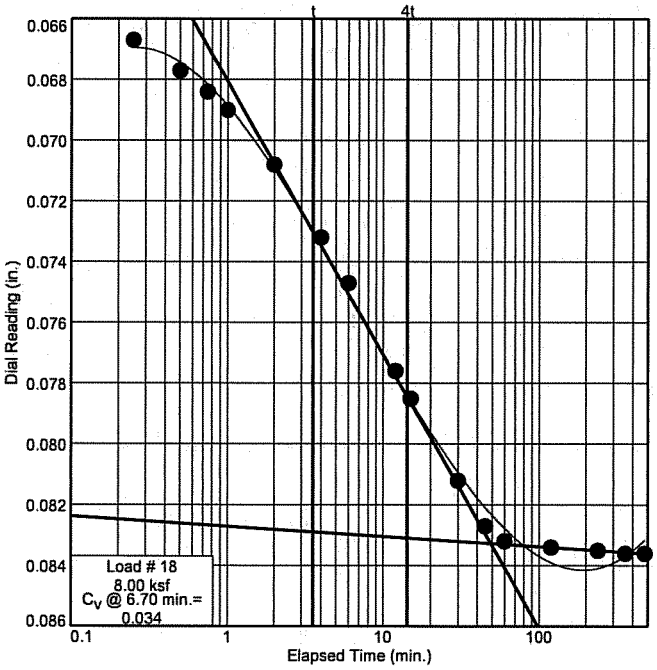
Dial Reading vs. Time

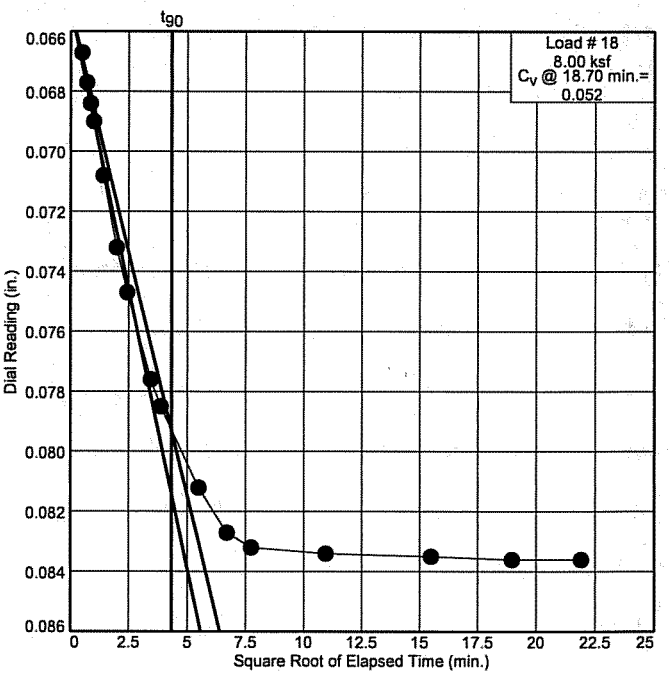
Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: BB-SPC-203 Depth: 40'-42' Sample Number: U-1









R.W. Gillespie & Associates, Inc.

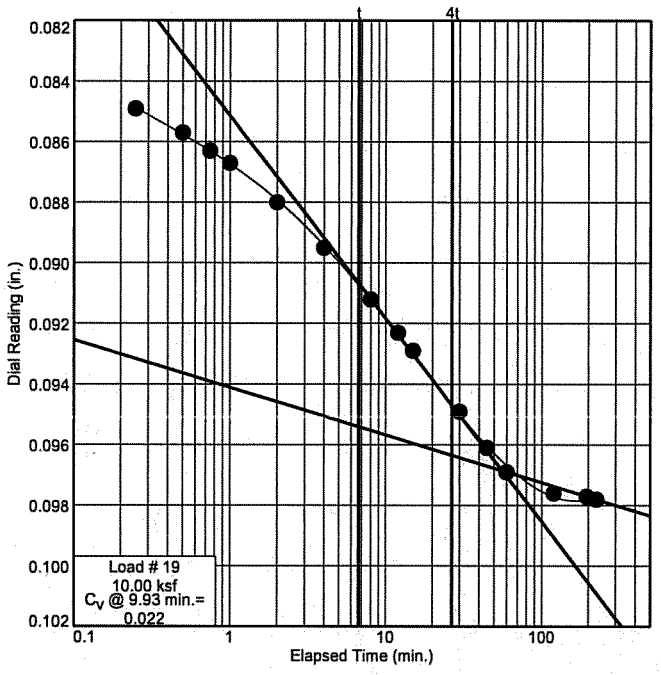
Biddeford, Maine

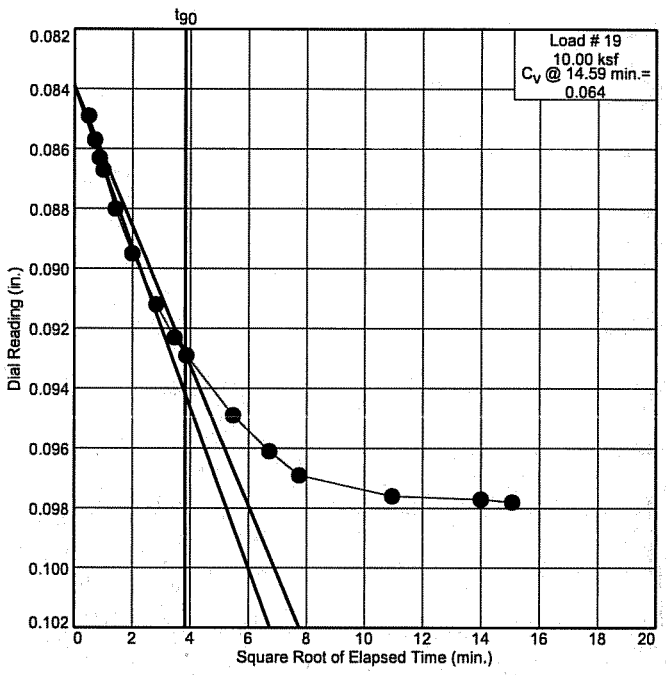
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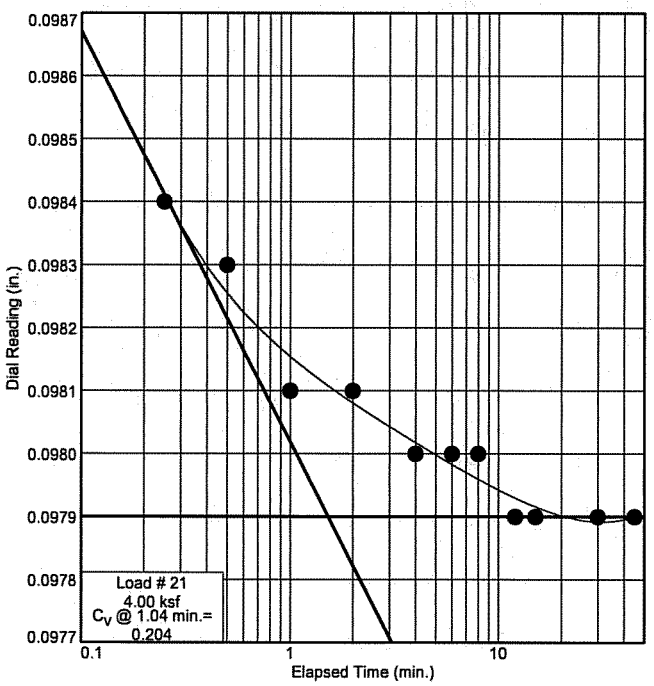
Dial Reading vs. Time

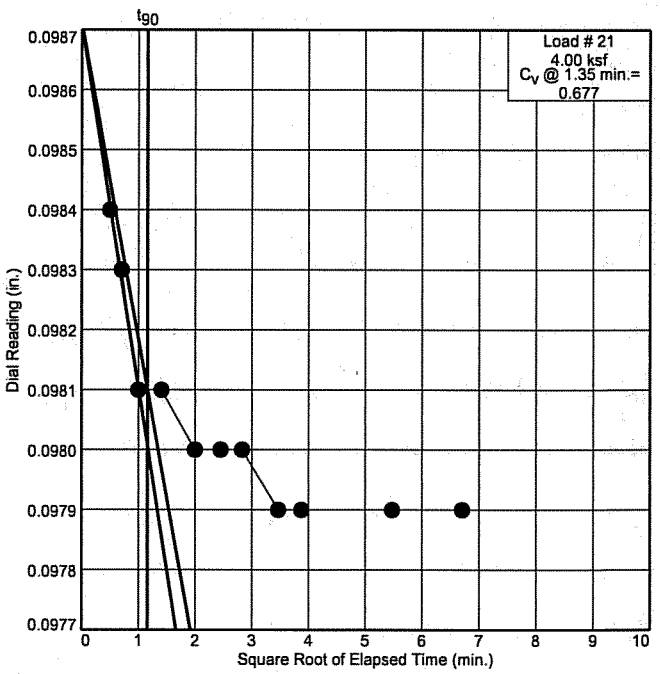
Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: BB-SPC-203 Depth: 40'-42' Sample Number: U-1









R.W. Gillespie & Associates, Inc.

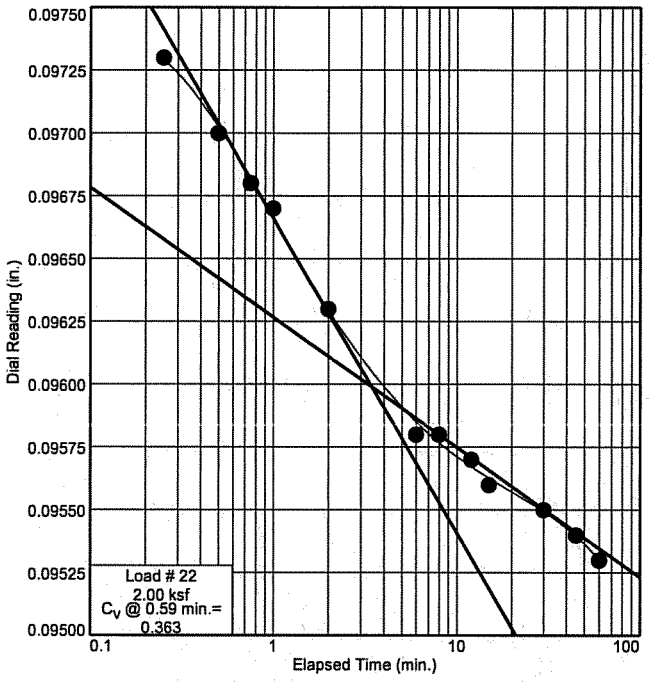
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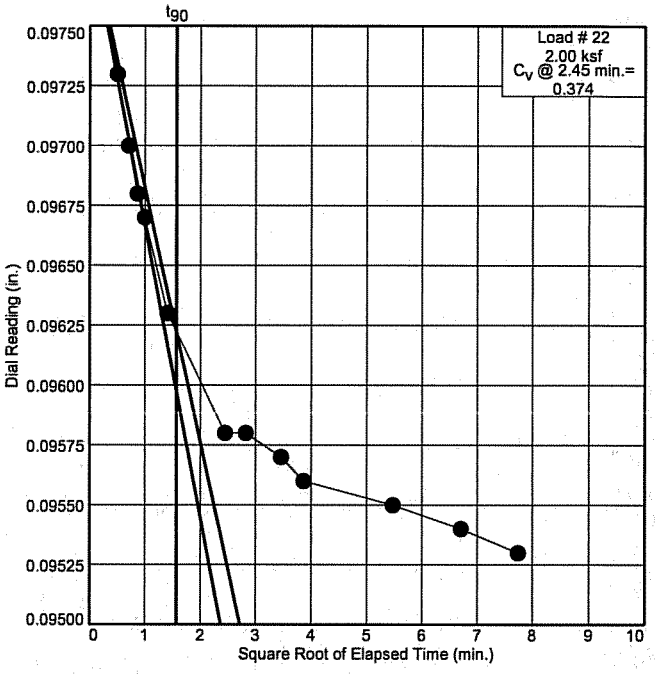
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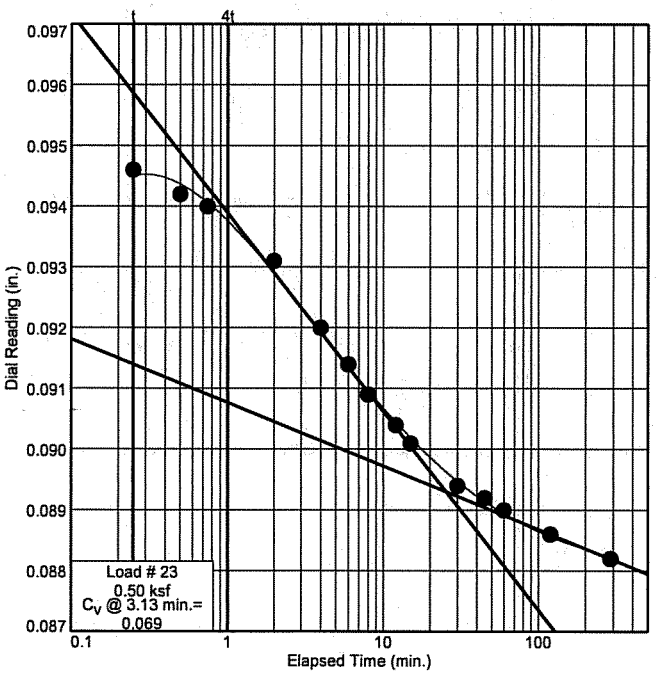
Dial Reading vs. Time

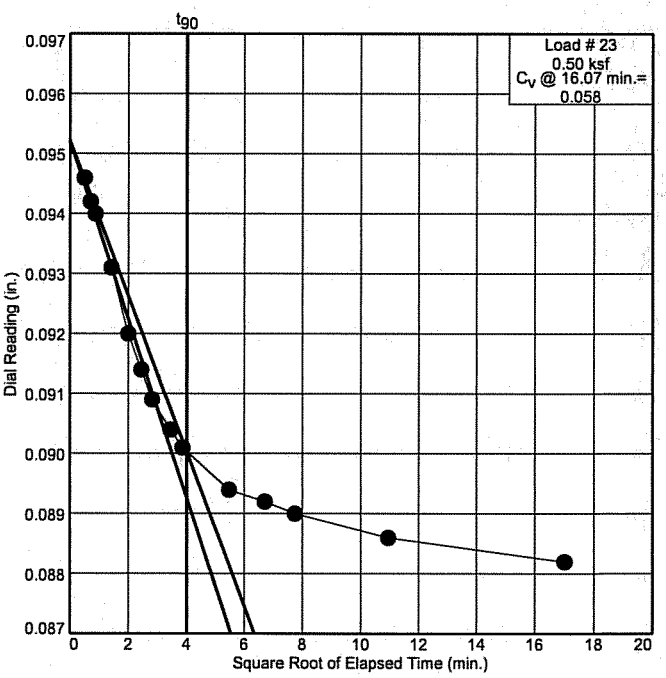
Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: BB-SPC-203 Depth: 40'-42' Sample Number: U-1









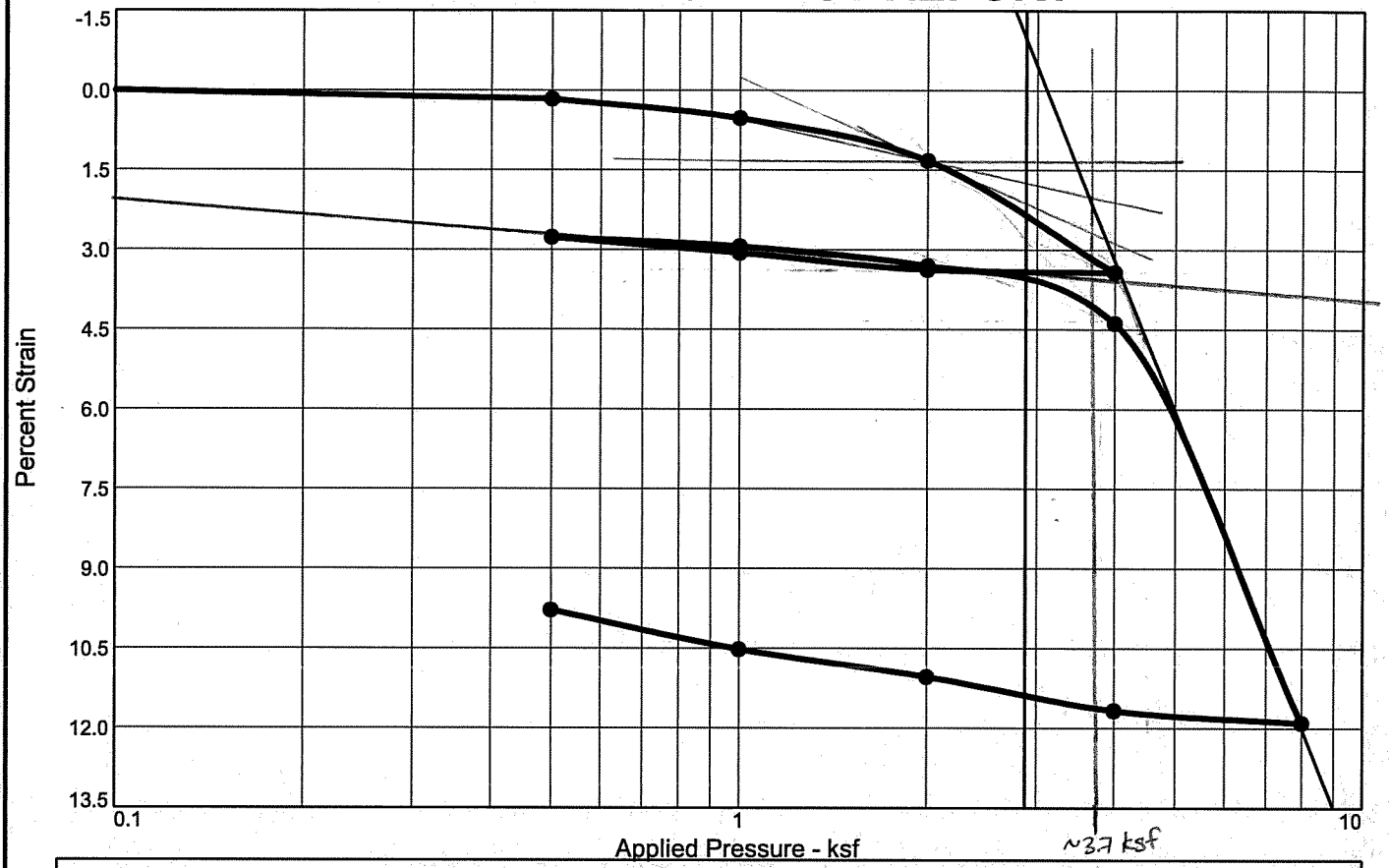
R.W. Gillespie & Associates, Inc.

Biddeford, Maine

Lab No. 14426

Reprint front sheet

CONSOLIDATION TEST REPORT



Coefficients of Consolidation and Secondary Consolidation

No.	Load (ksf)	C_v (ft.2/day)	C_α	No.	Load (ksf)	C_v (ft.2/day)	C_α	No.	Load (ksf)	C_v (ft.2/day)	C_α
1	0.50	5.118		8	1.00	6.527		15	0.50	0.448	
2	1.00	5.105		9	2.00	4.517					
3	2.00	2.744		10	4.00	2.409					
4	4.00	1.696		11	8.00	0.412					
5	2.00	5.753		12	4.00	4.928					
6	1.00	6.182		13	2.00	2.761					
7	0.50	2.117		14	1.00	4.983					

Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	Initial Void Ratio
Saturation	Moisture							
93.2 %	45.6 %	72.6	40.9	19.2	2.70	1.25	3.7 ok	1.321

MATERIAL DESCRIPTION	USCS	AASHTO
Lean Clay		

Project No. 0600-102 **Client:** HNTB Corporation

Project: Scarborough Eastern Trail Project
Scarborough, ME

Location: BB-SPC-203A **Depth:** 20'-22' **Sample Number:** U-1

R.W. Gillespie & Associates, Inc.

Biddeford, Maine

Remarks:

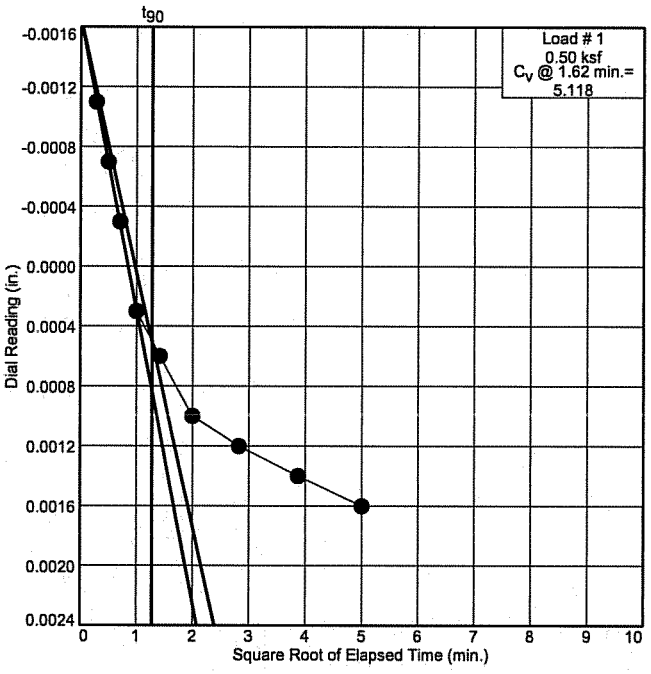
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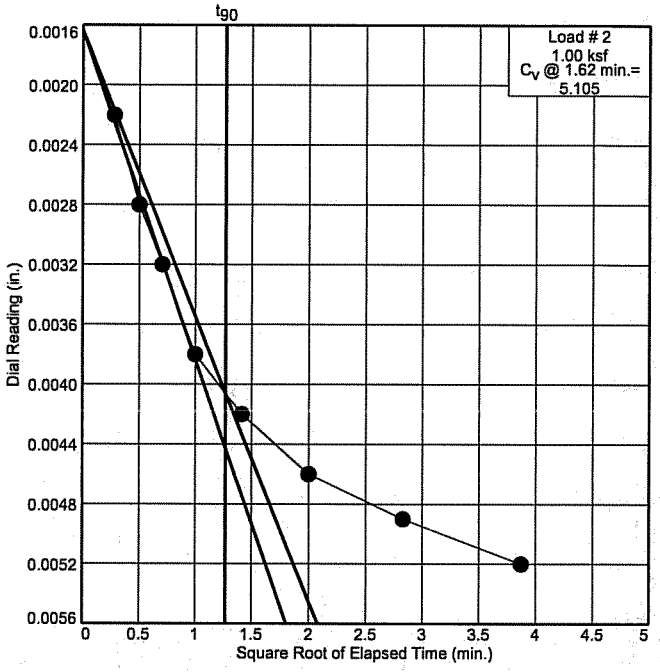
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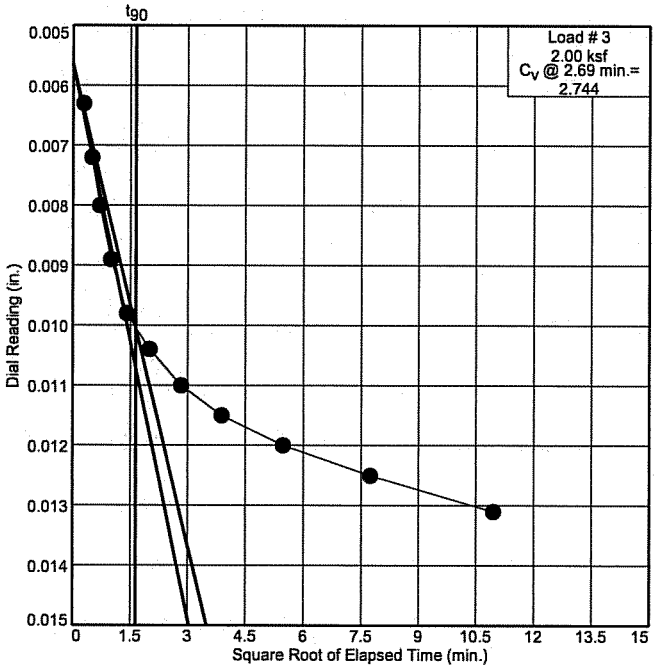
Dial Reading vs. Time

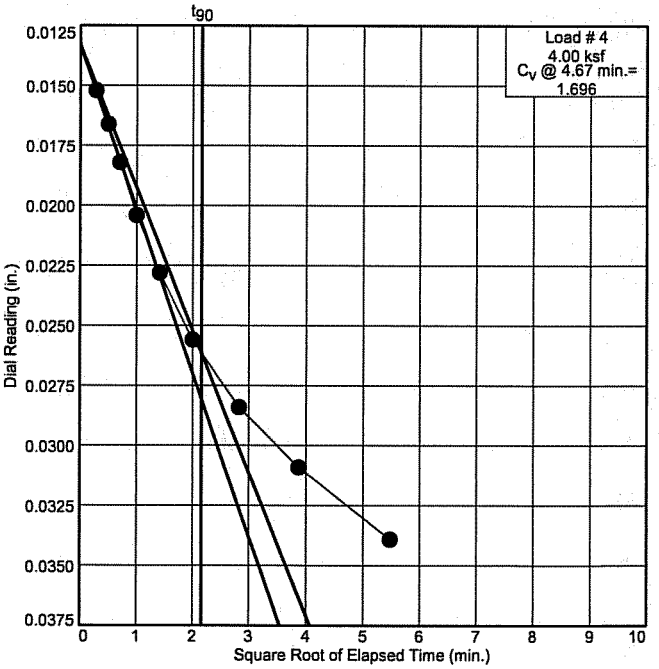
Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: BB-SPC-203A Depth: 20'-22' Sample Number: U-1









R.W. Gillespie & Associates, Inc.

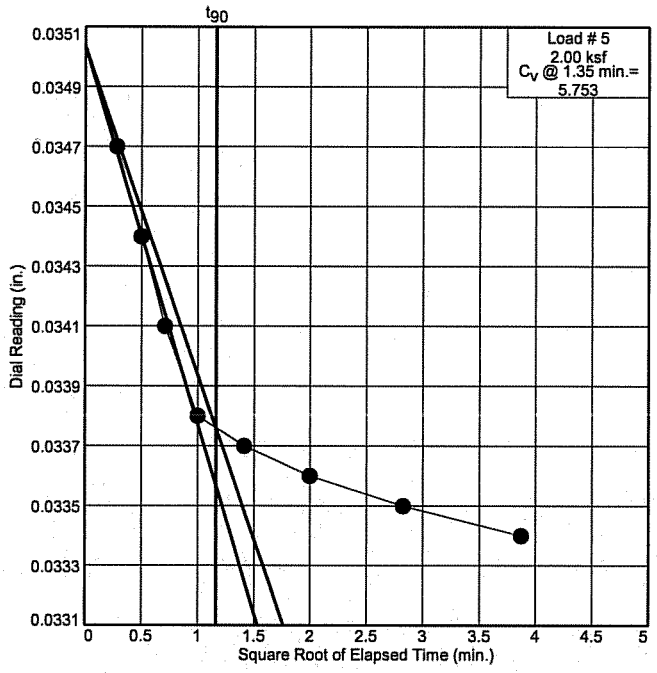
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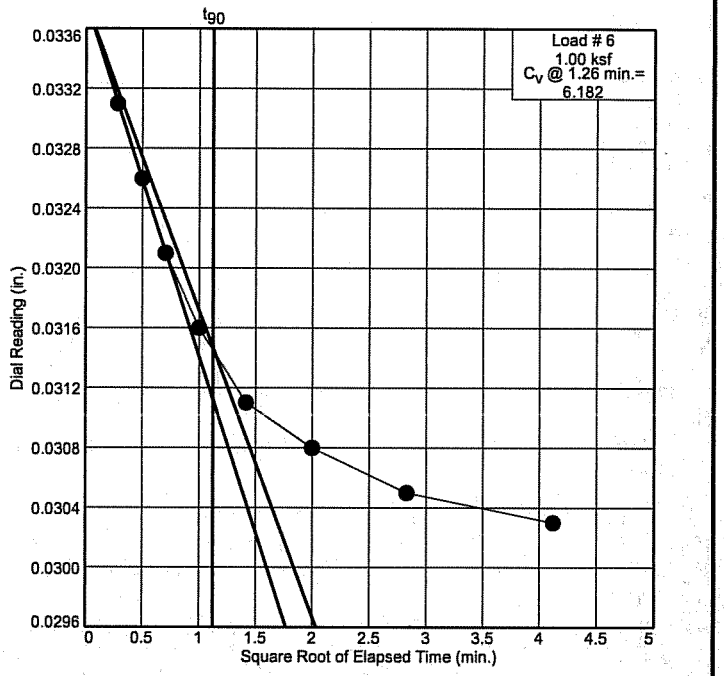
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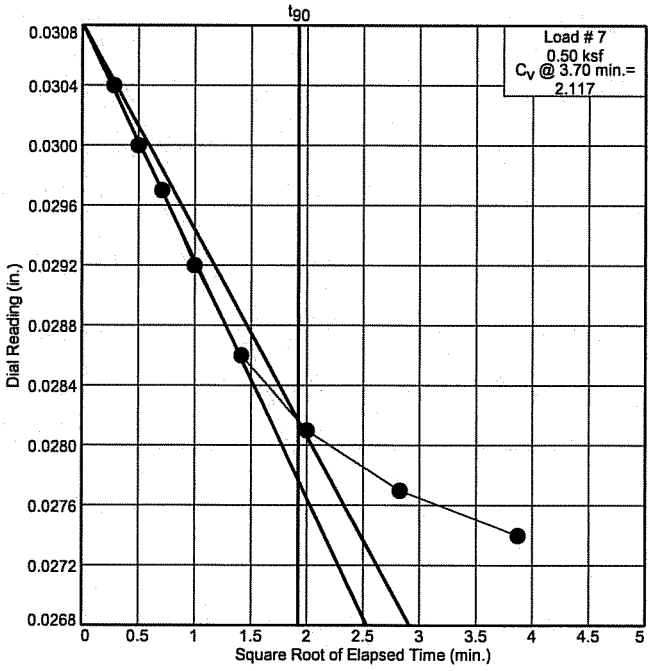
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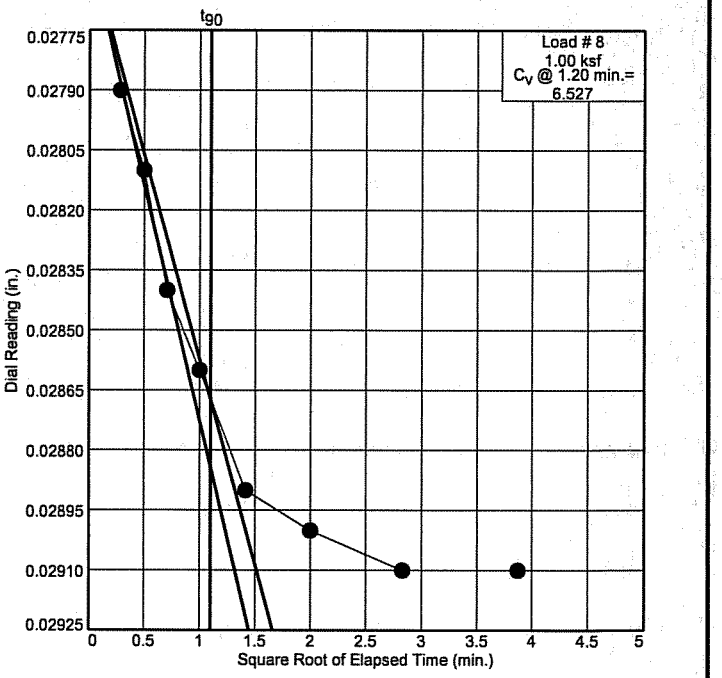
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Project: Scarborough Eastern Trail Project

Location: BB-SPC-203A Depth: 20'-22' Sample Number: U-1









R.W. Gillespie & Associates, Inc.

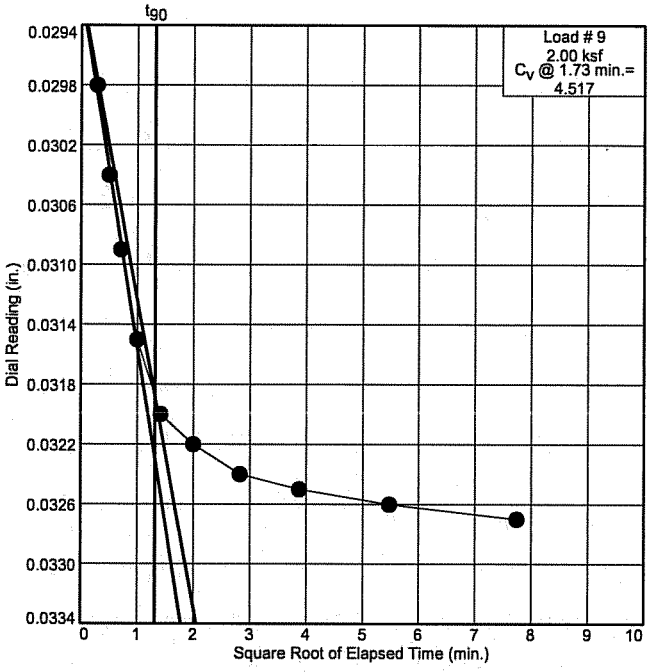
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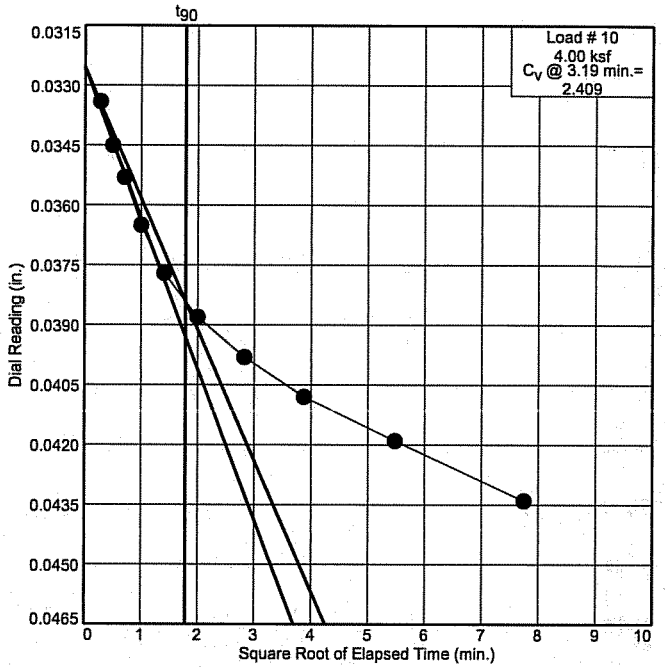
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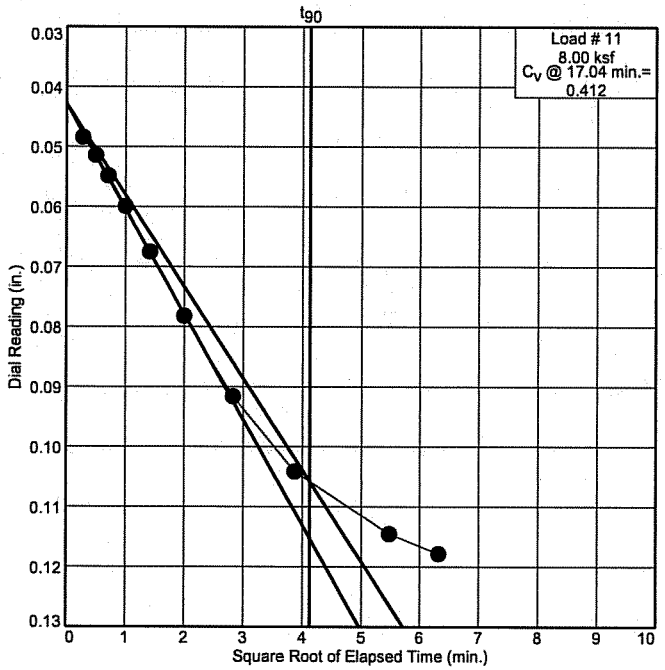
Dial Reading vs. Time

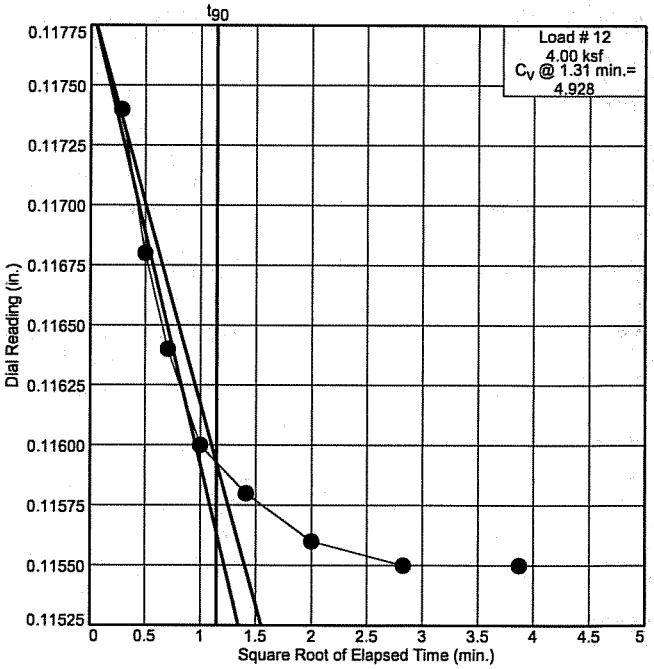
Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: BB-SPC-203A Depth: 20'-22' Sample Number: U-1









R.W. Gillespie & Associates, Inc.

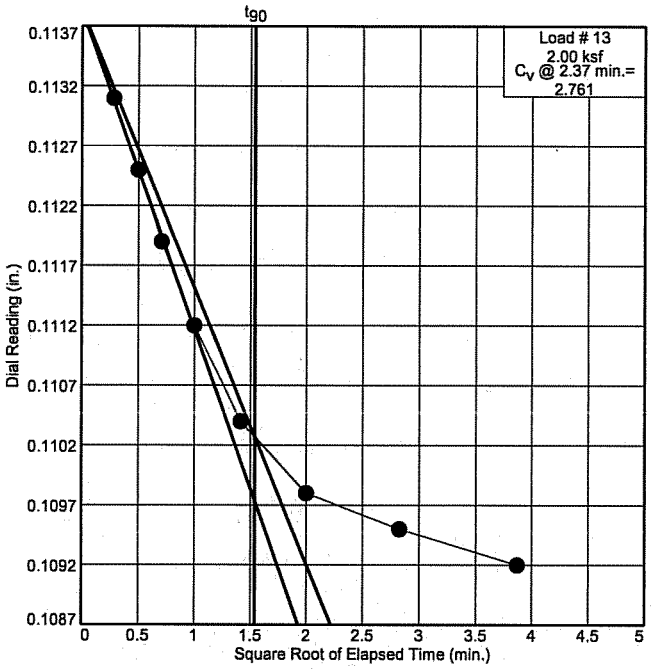
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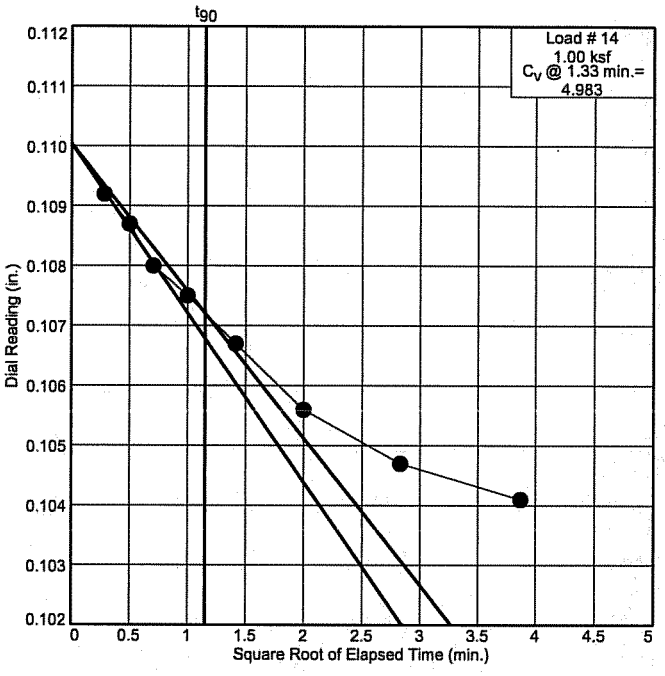
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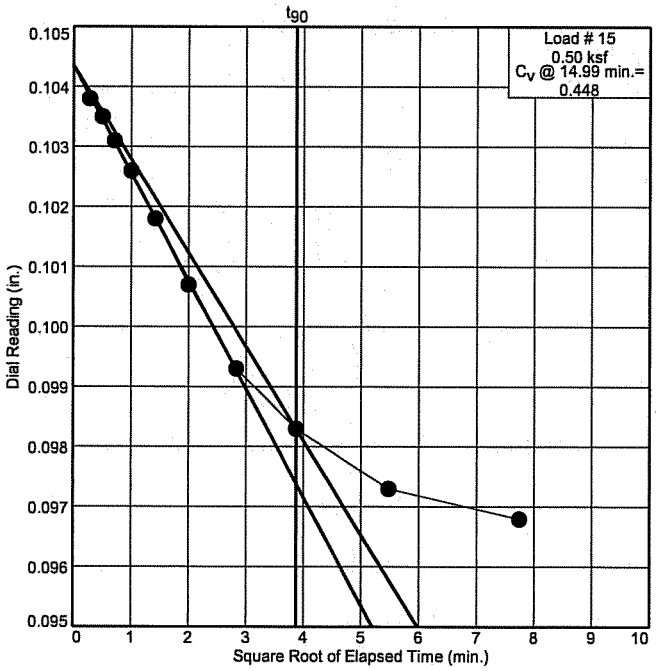
Dial Reading vs. Time

Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: BB-SPC-203A Depth: 20'-22' Sample Number: U-1





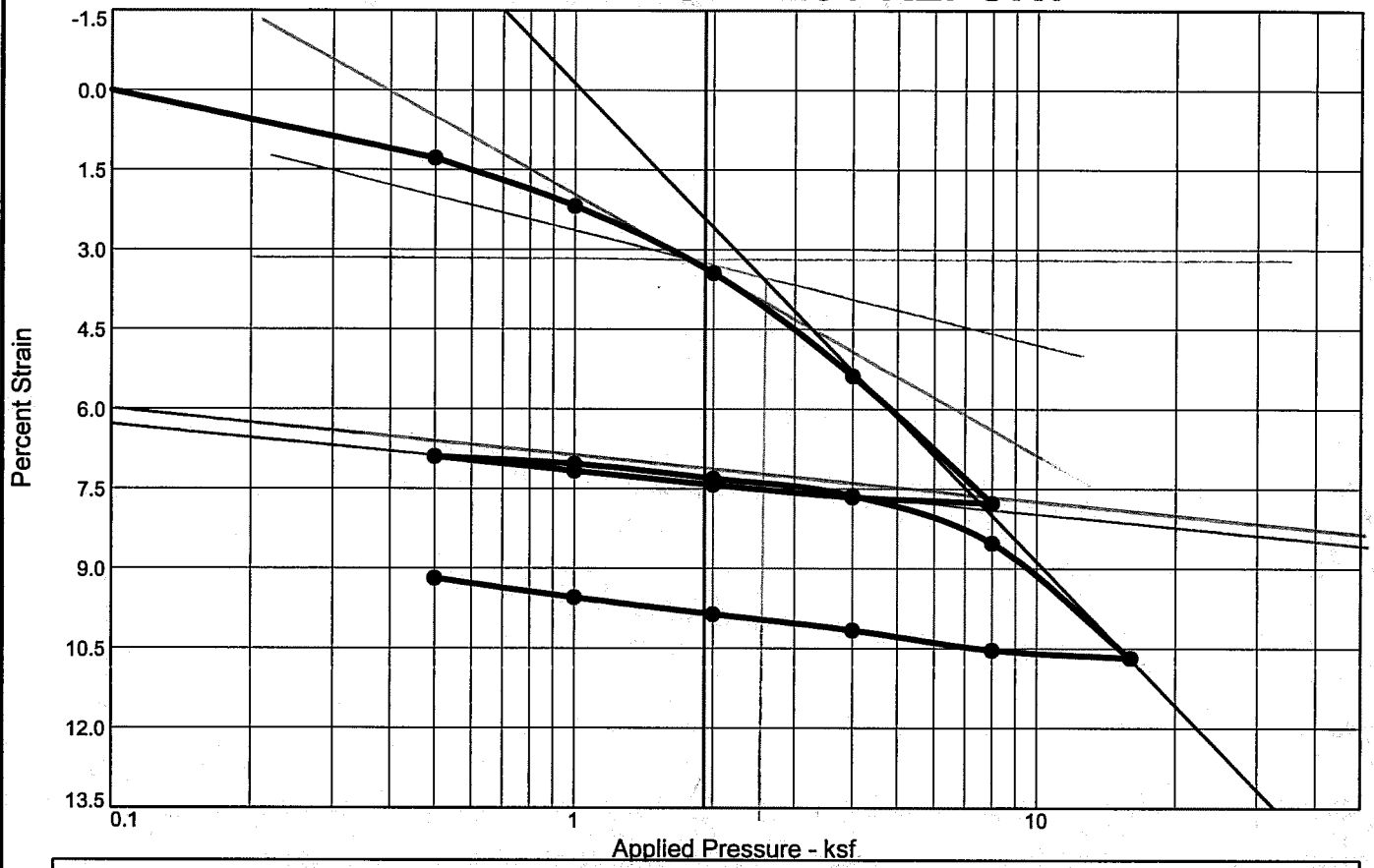


R.W. Gillespie & Associates, Inc.

Biddeford, Maine

Lab No. 15115a

CONSOLIDATION TEST REPORT



Coefficients of Consolidation and Secondary Consolidation

No.	Load (ksf)	C_v (ft.2/day)	C_α	No.	Load (ksf)	C_v (ft.2/day)	C_α	No.	Load (ksf)	C_v (ft.2/day)	C_α
1	0.50	2.707		8	1.00	8.388		15	8.00	20.396	
2	1.00	3.731		9	0.50	4.861		16	4.00	18.326	
3	2.00	3.040		10	1.00	10.217		17	2.00	8.605	
4	4.00	3.997		11	2.00	15.249		18	1.00	6.208	
5	8.00	4.415		12	4.00	16.800		19	0.50	2.892	
6	4.00	10.766		13	8.00	9.468					
7	2.00	10.704		14	16.00	8.306					

Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	Initial Void Ratio
Saturation	Moisture							
106.3 %	39.0 %	84.7			2.70	1.75	2.7 0V	0.990

MATERIAL DESCRIPTION	USCS	AASHTO
Lean Clay		

Project No. 0600-102 **Client:** HNTB Corporation

Project: Scarborough Eastern Trail Project
Scarborough, ME

Location: BB-SPC-203A **Depth:** 30'-32' **Sample Number:** U-2

R.W. Gillespie & Associates, Inc.

Biddeford, Maine

Remarks:

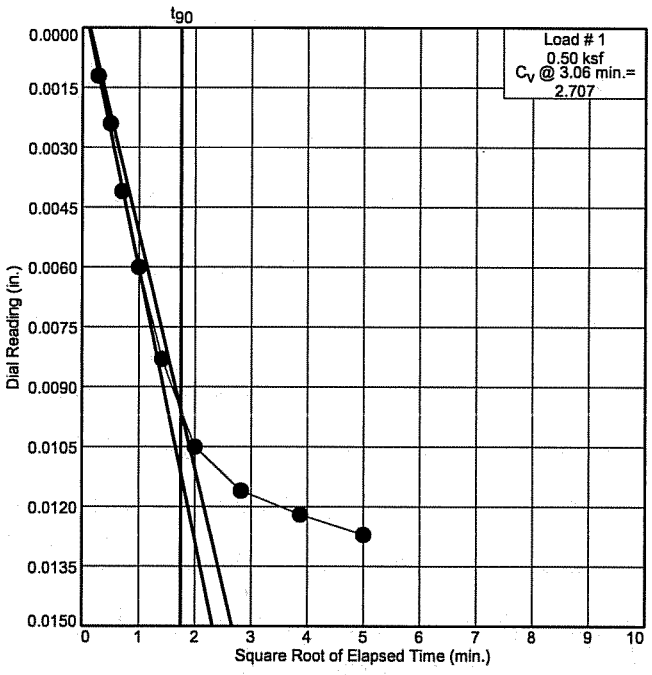
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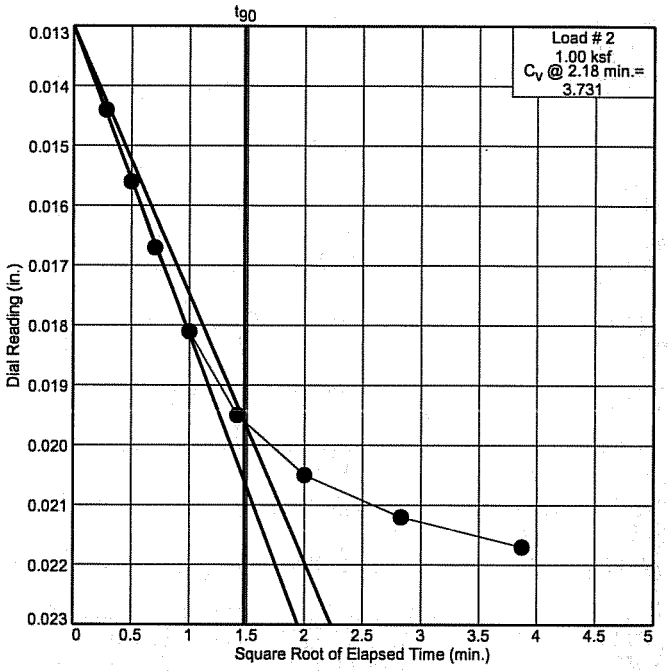
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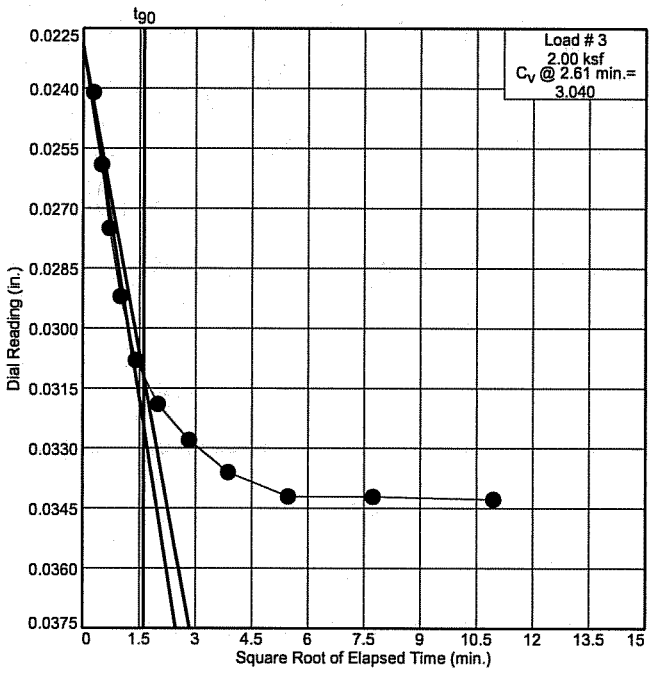
Dial Reading vs. Time

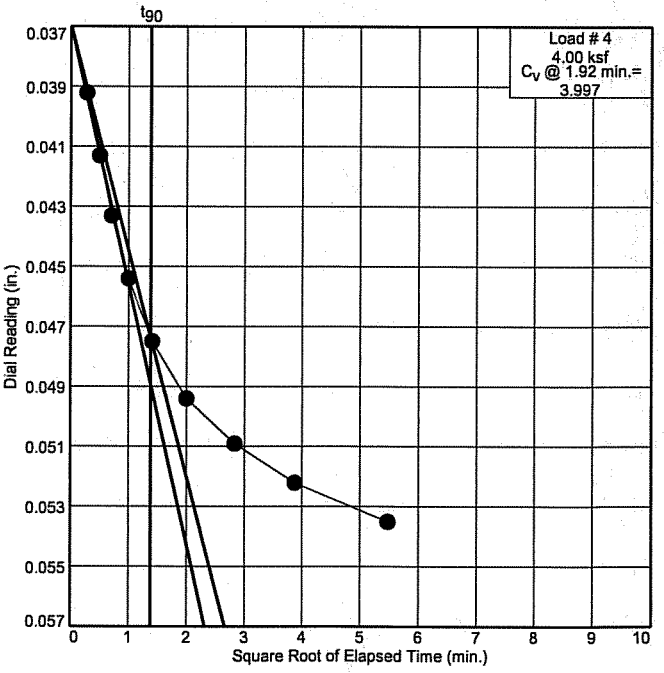
Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: BB-SPC-203A Depth: 30'-32' Sample Number: U-2









R.W. Gillespie & Associates, Inc.

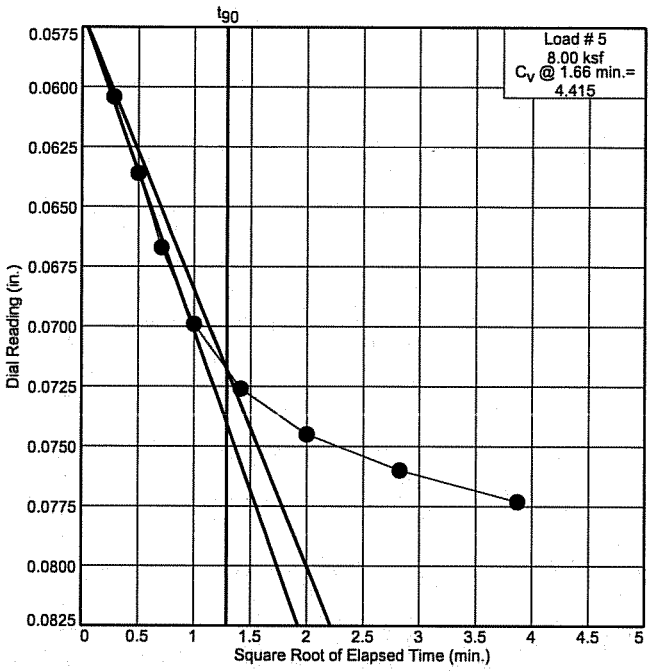
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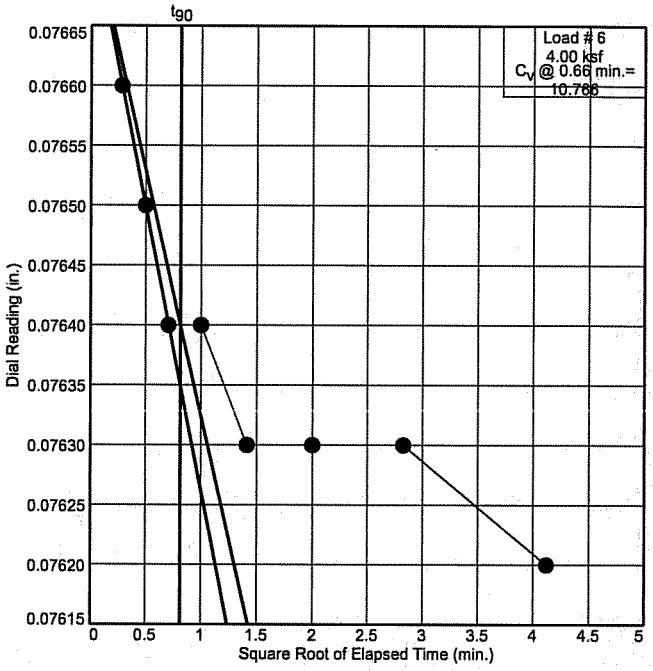
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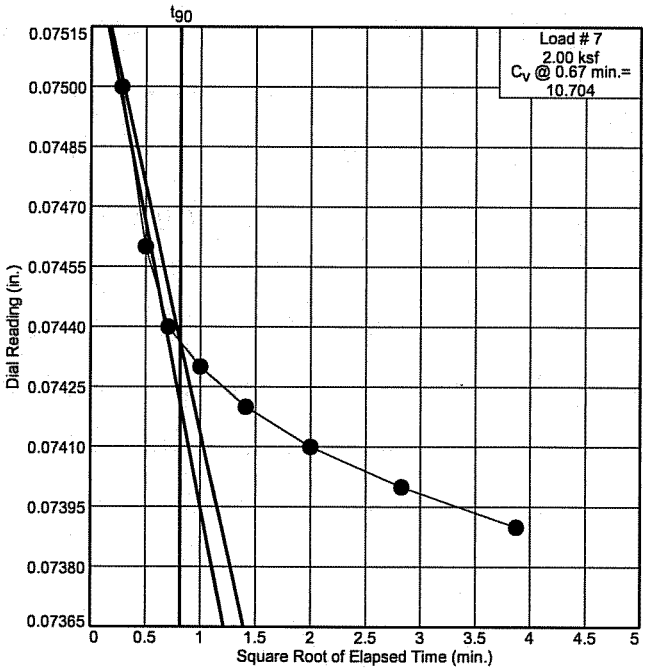
Dial Reading vs. Time

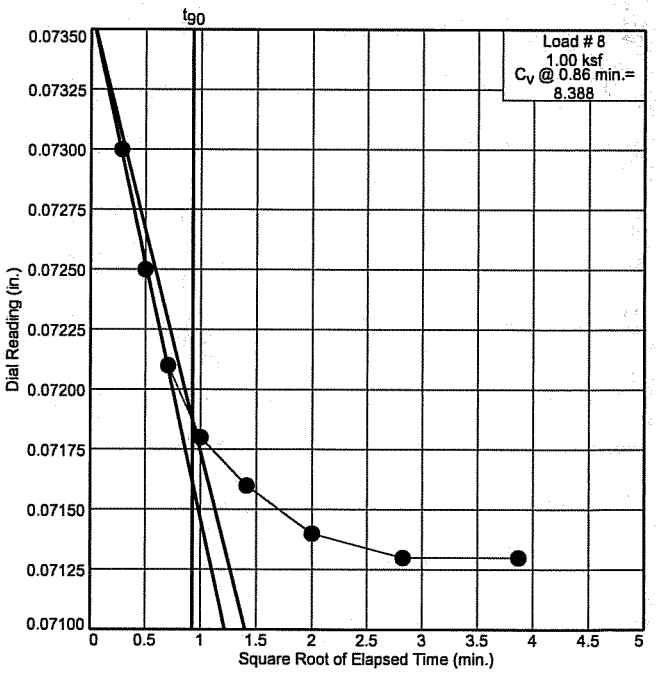
Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: BB-SPC-203A Depth: 30'-32' Sample Number: U-2









R.W. Gillespie & Associates, Inc.

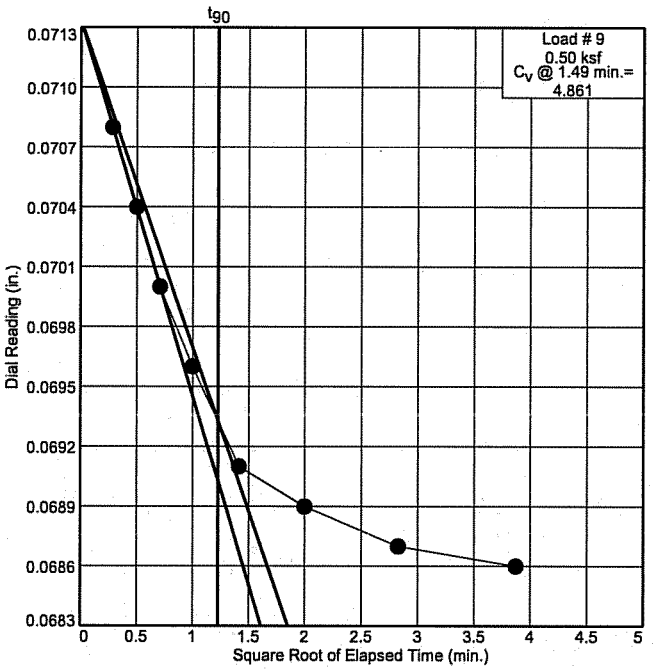
Biddeford, Maine

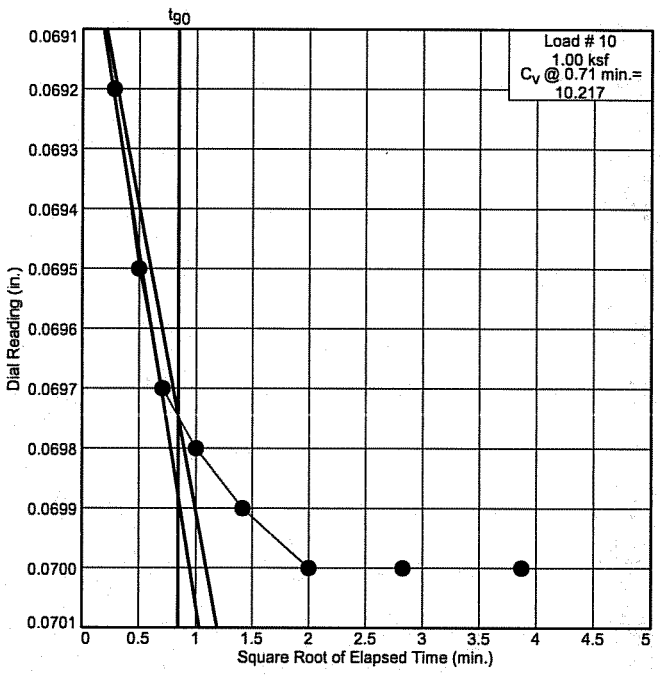
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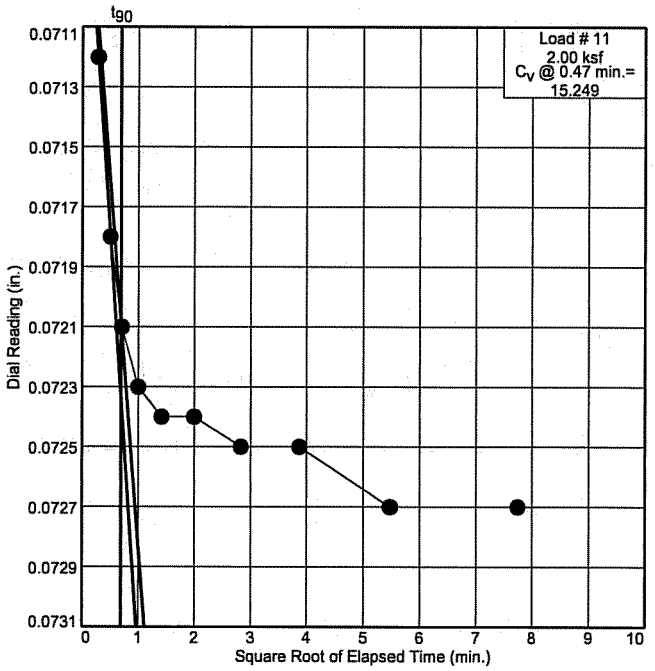
Dial Reading vs. Time

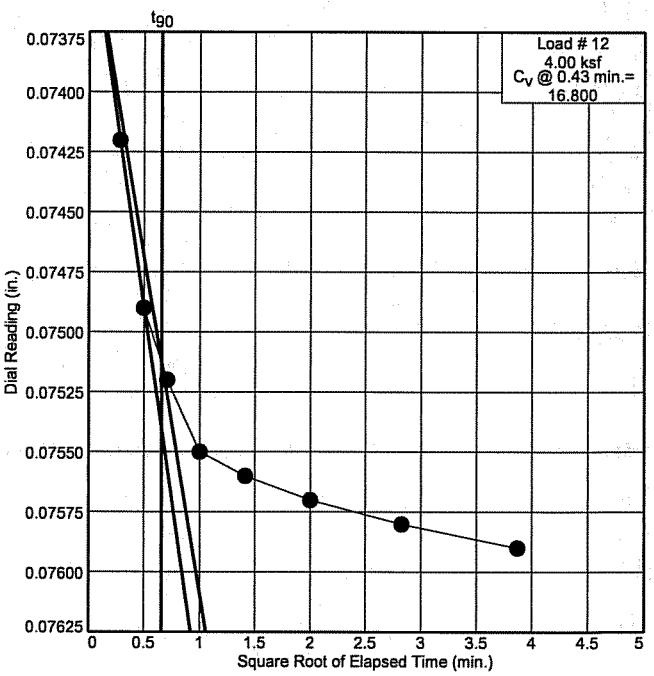
Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: BB-SPC-203A Depth: 30'-32' Sample Number: U-2









R.W. Gillespie & Associates, Inc.

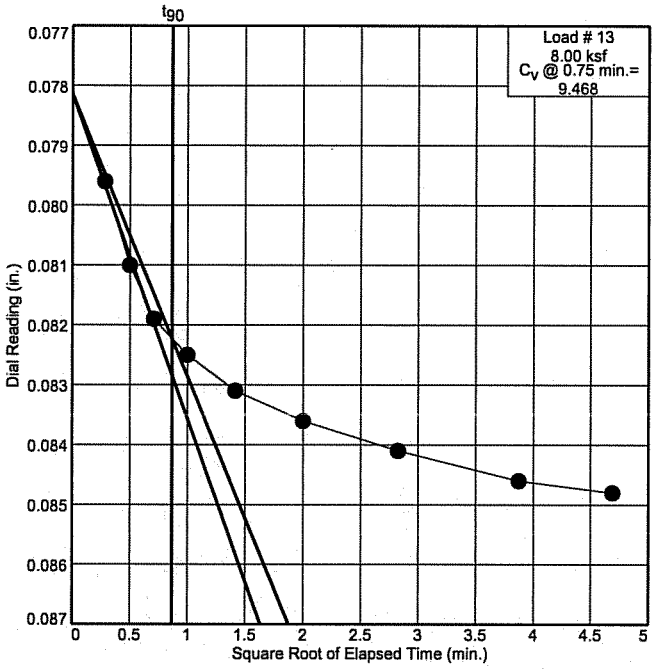
Biddeford, Maine

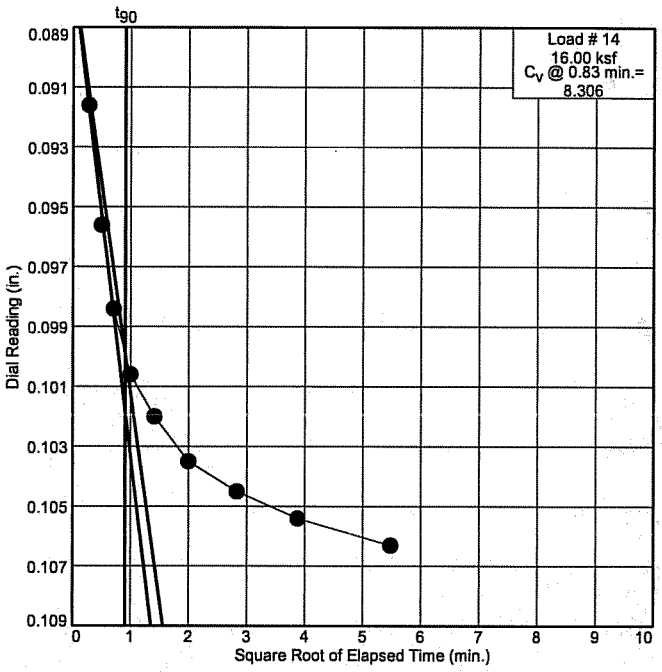
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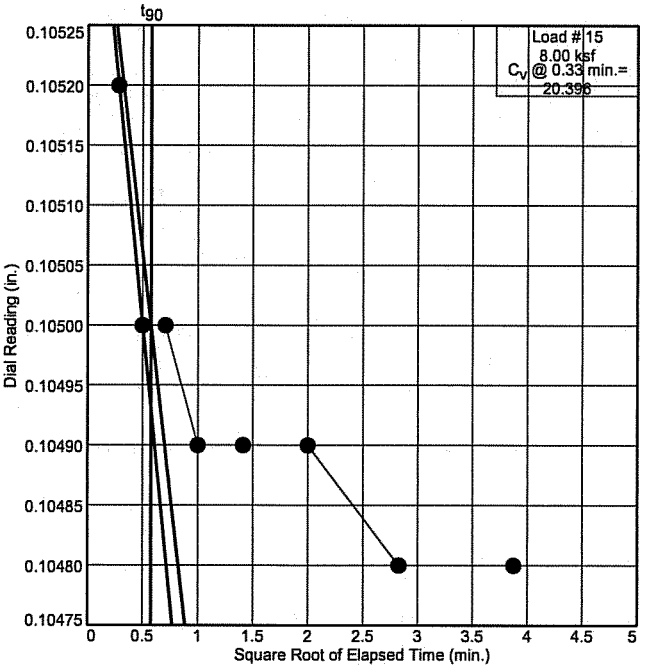
Dial Reading vs. Time

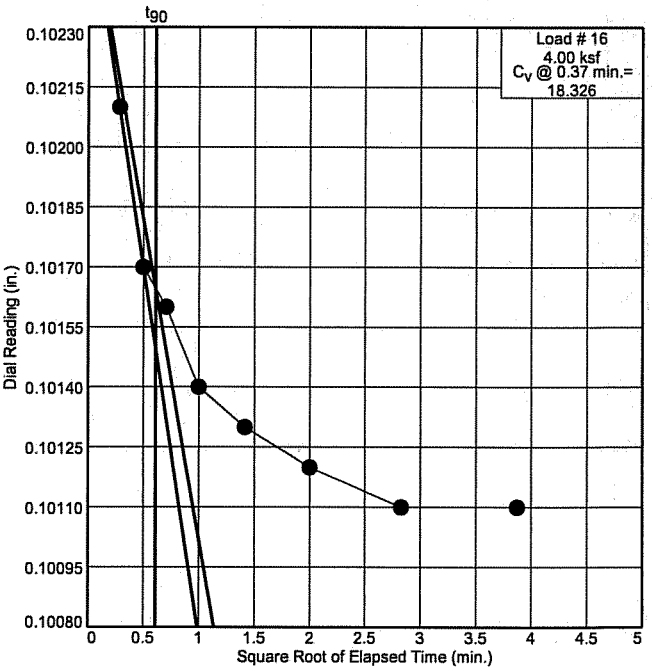
Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: BB-SPC-203A Depth: 30'-32' Sample Number: U-2









R.W. Gillespie & Associates, Inc.

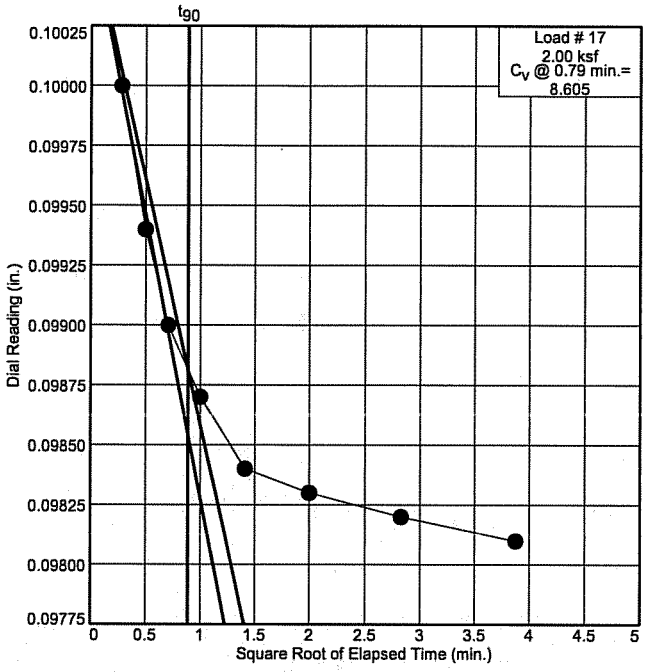
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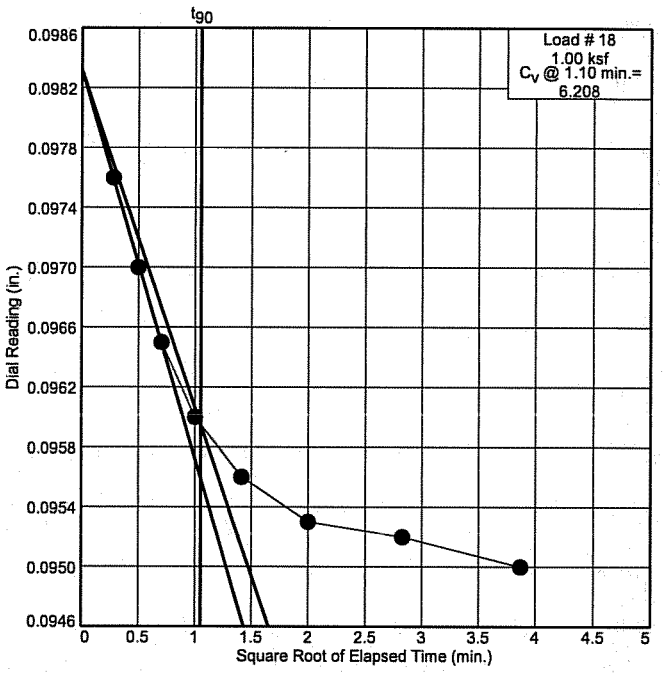
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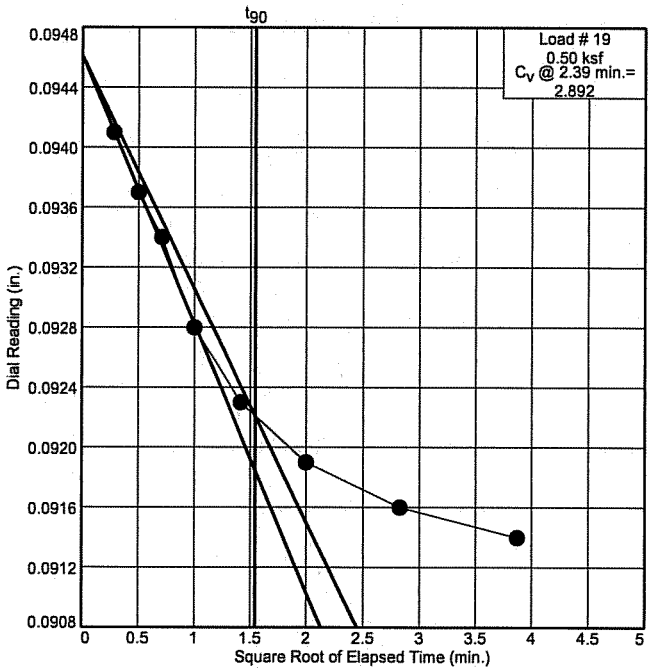
Dial Reading vs. Time

Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: BB-SPC-203A Depth: 30'-32' Sample Number: U-2





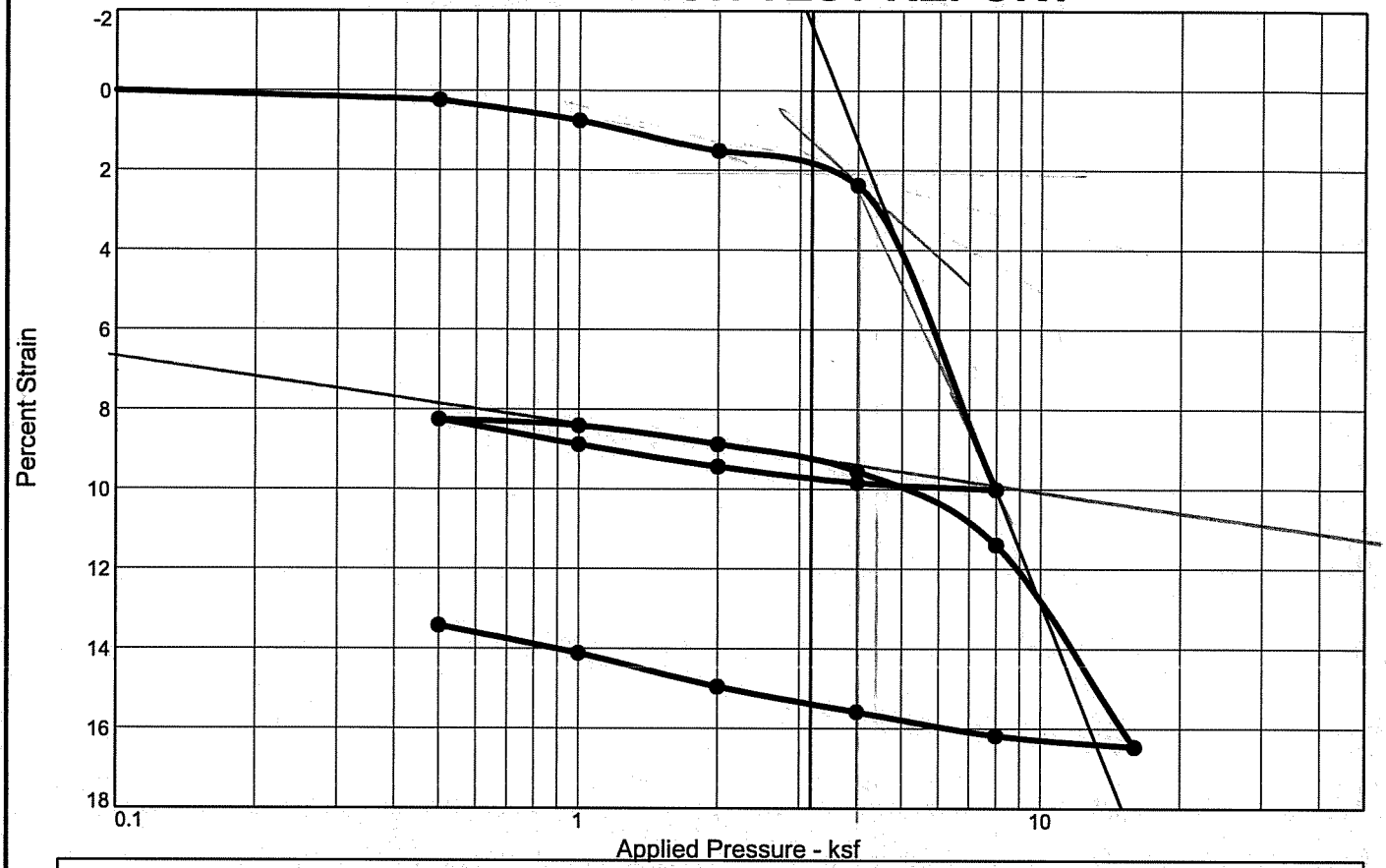


R.W. Gillespie & Associates, Inc.

Biddeford, Maine

Lab No. 15116b

CONSOLIDATION TEST REPORT



Coefficients of Consolidation and Secondary Consolidation

No.	Load (ksf)	C_v (ft.2/day)	C_α	No.	Load (ksf)	C_v (ft.2/day)	C_α	No.	Load (ksf)	C_v (ft.2/day)	C_α
1	0.50	1.110		8	1.00	0.523		15	8.00	3.785	
2	1.00	0.967		9	0.50	0.209		16	4.00	1.319	
3	2.00	1.245		10	1.00	1.374		17	2.00	0.600	
4	4.00	1.841		11	2.00	0.836		18	1.00	0.190	
5	8.00	0.161		12	4.00	0.982		19	0.50	0.093	
6	4.00	4.104		13	8.00	0.658					
7	2.00	1.633		14	16.00	0.328					

Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	Initial Void Ratio
Saturation	Moisture							
95.2 %	42.3 %	77.6	40.0	17.5	2.7	2.2	4.1	1.200

MATERIAL DESCRIPTION	USCS	AASHTO
Lean Clay		

Project No. 0600-102 **Client:** HNTB Corporation

Project: Scarborough Eastern Trail Project
Scarborough, ME

Location: BB-SPC-203A **Depth:** 40'-42' **Sample Number:** U-4

R.W. Gillespie & Associates, Inc.

Biddeford, Maine

Remarks:

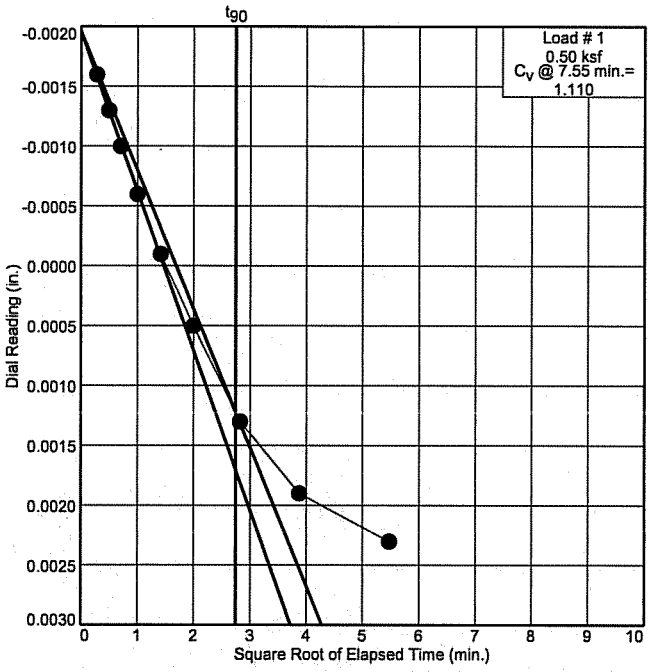
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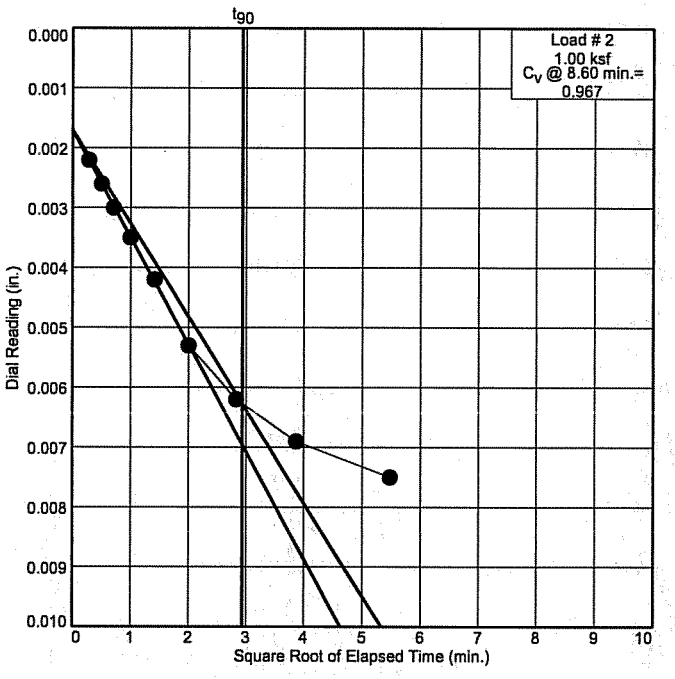
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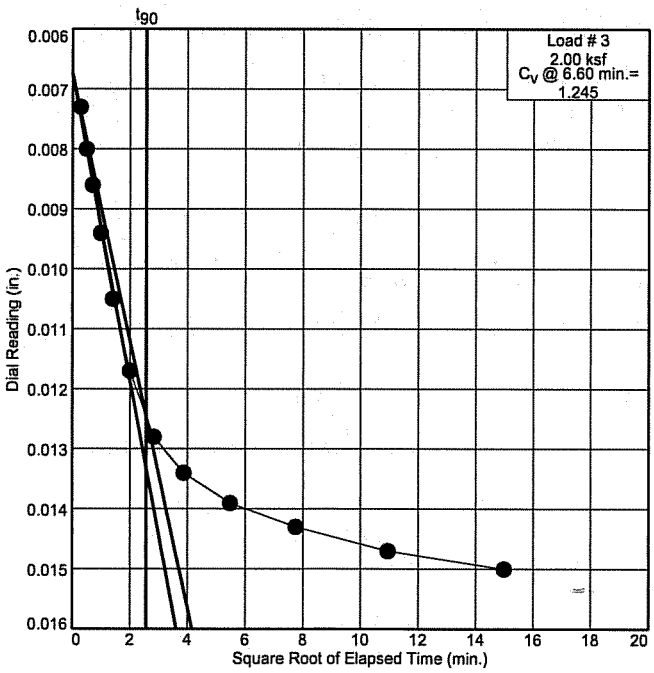
Dial Reading vs. Time

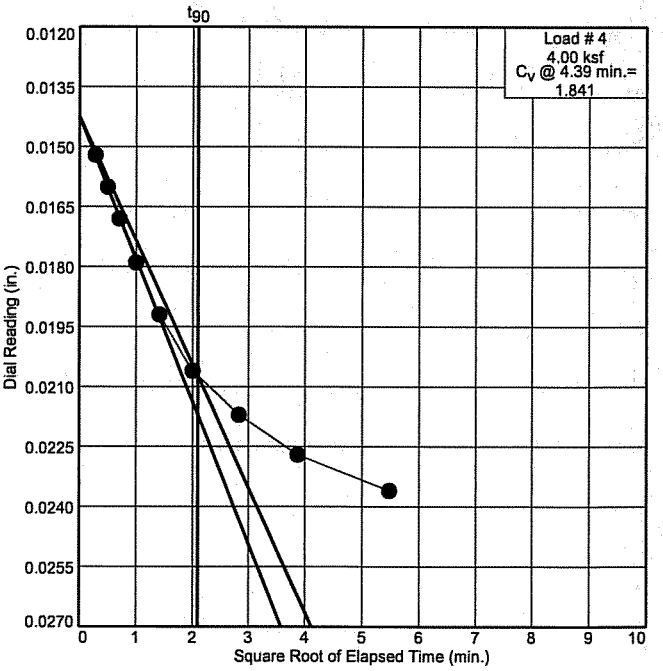
Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: BB-SPC-203A Depth: 40'-42' Sample Number: U-4









R.W. Gillespie & Associates, Inc.

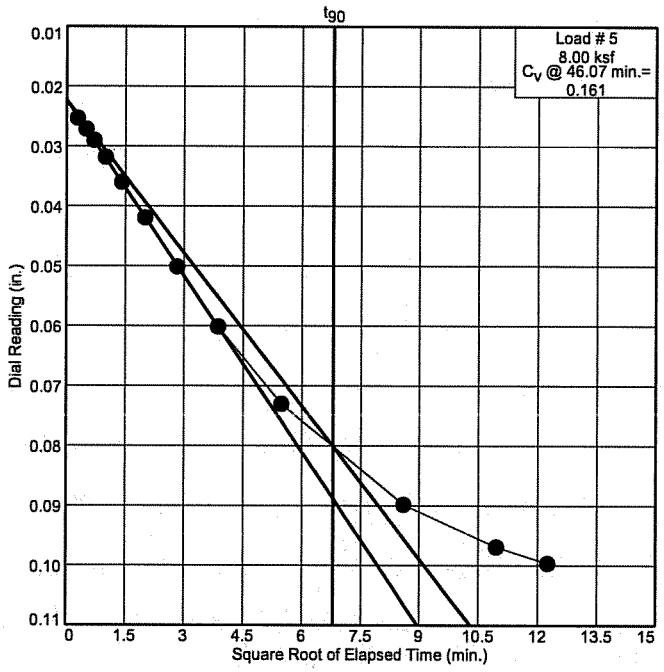
Biddeford, Maine

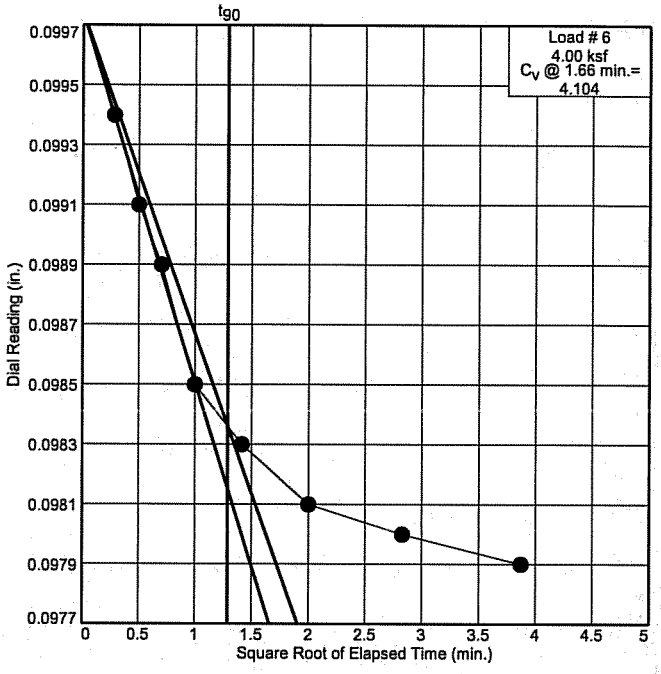
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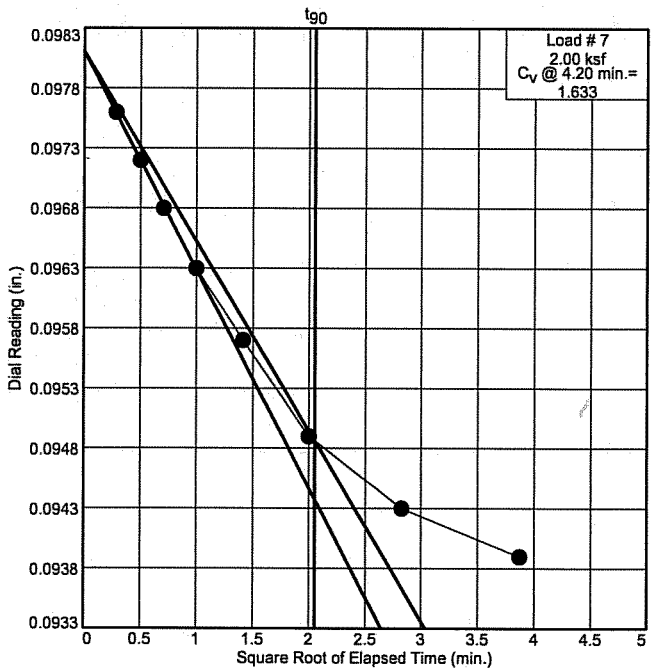
Dial Reading vs. Time

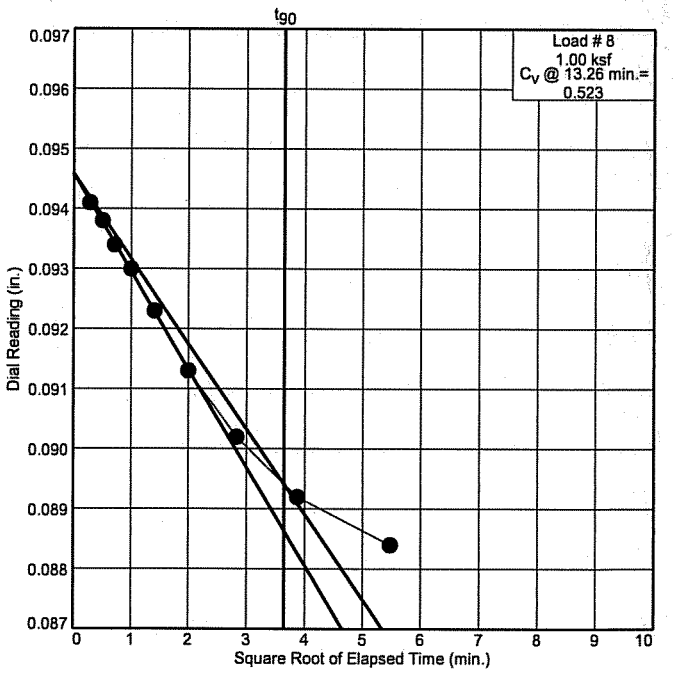
Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: BB-SPC-203A Depth: 40'-42' Sample Number: U-4









R.W. Gillespie & Associates, Inc.

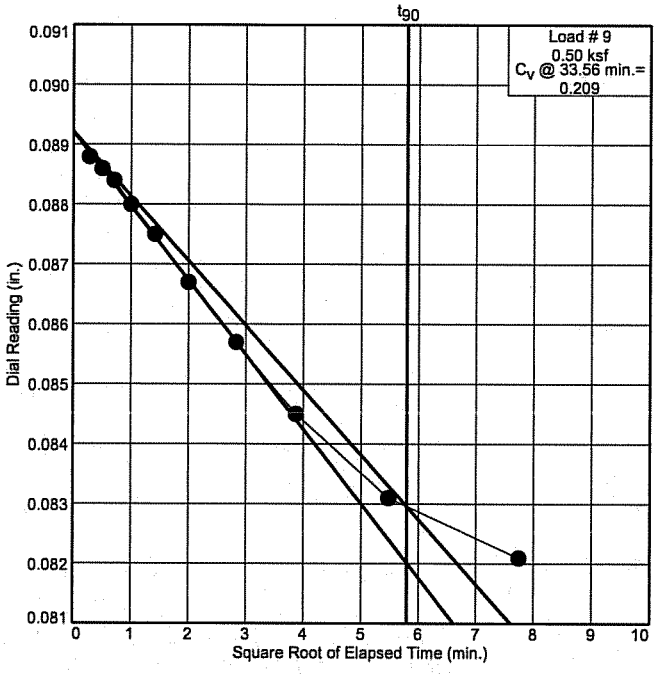
Biddeford, Maine

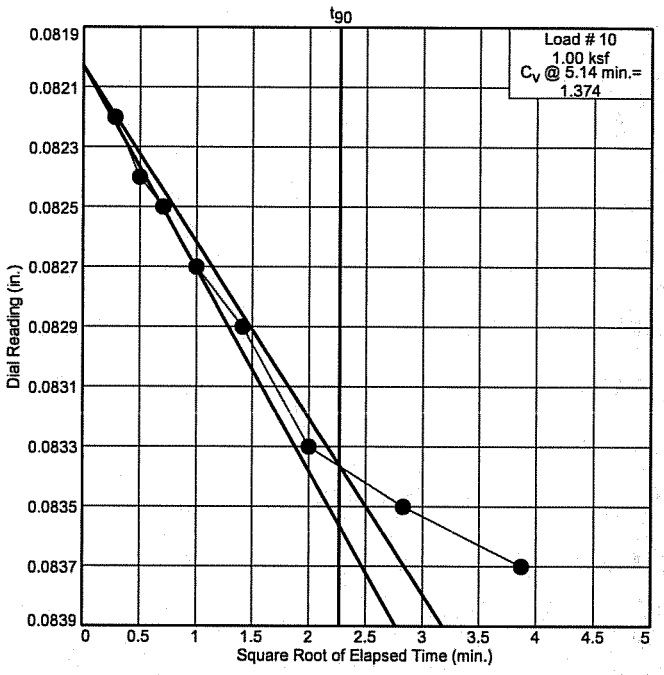
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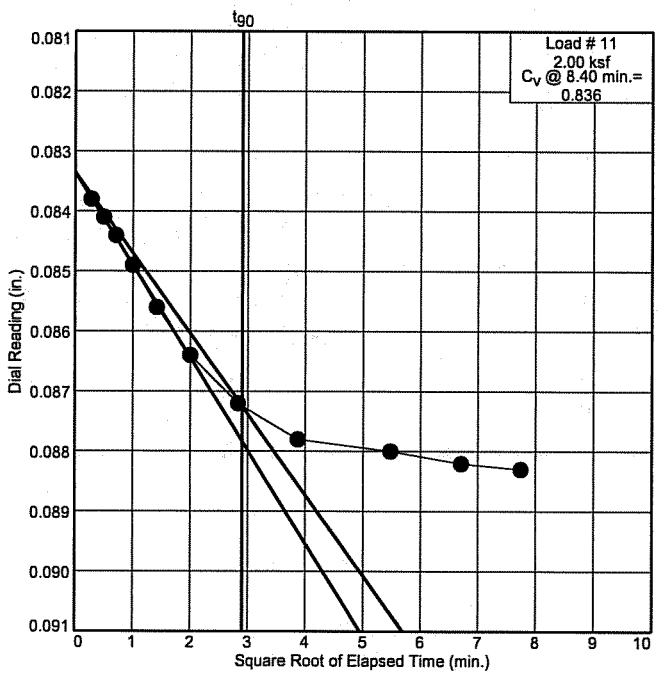
Dial Reading vs. Time

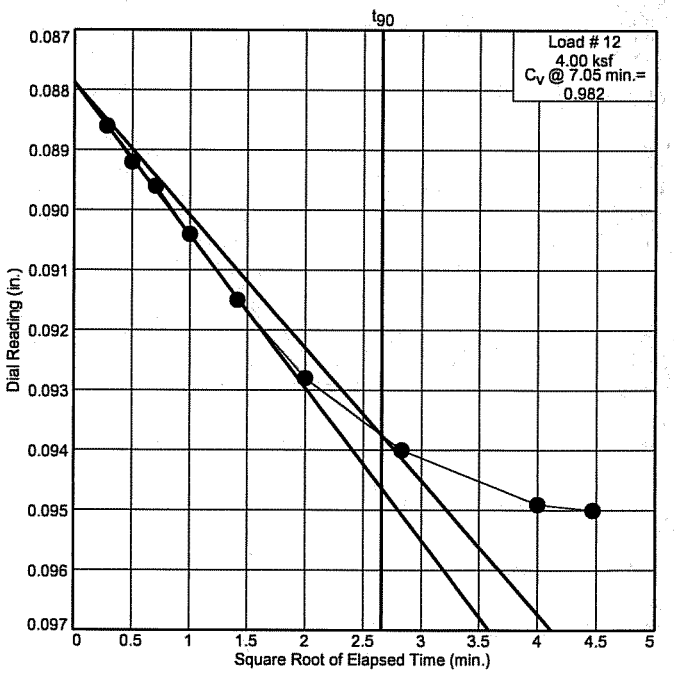
Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: BB-SPC-203A Depth: 40'-42' Sample Number: U-4









R.W. Gillespie & Associates, Inc.

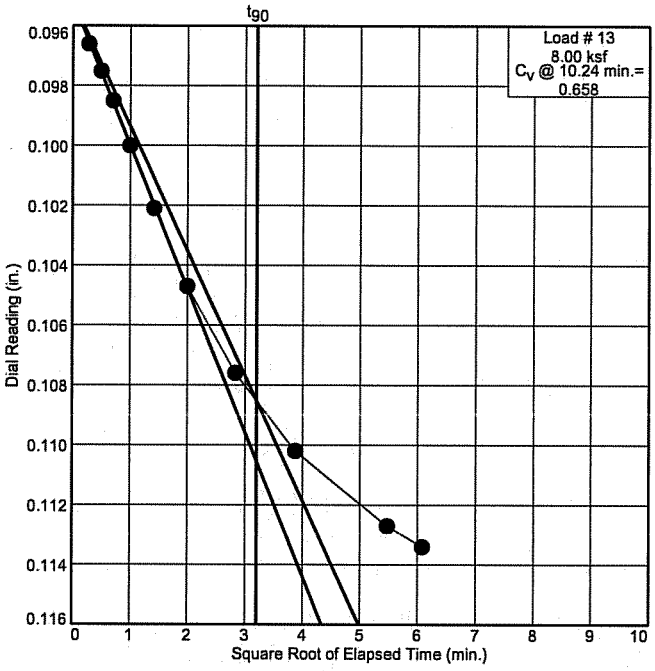
Biddeford, Maine

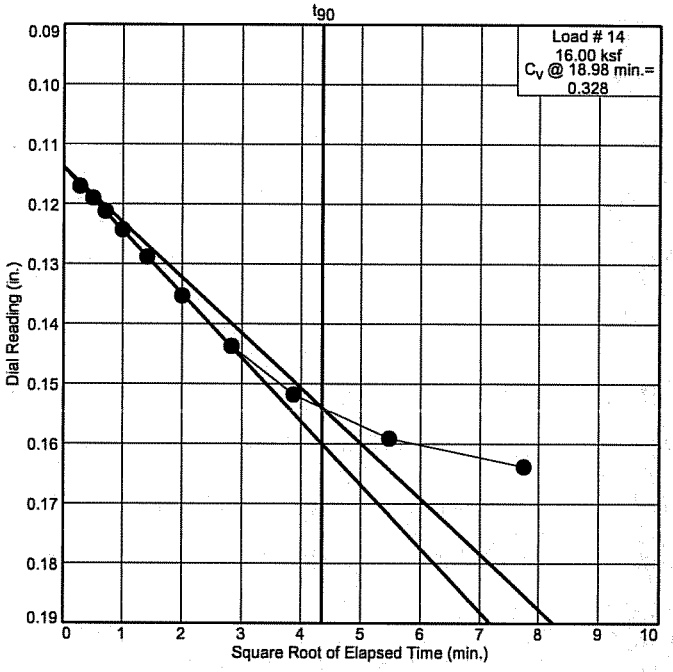
Lab No. 15114c

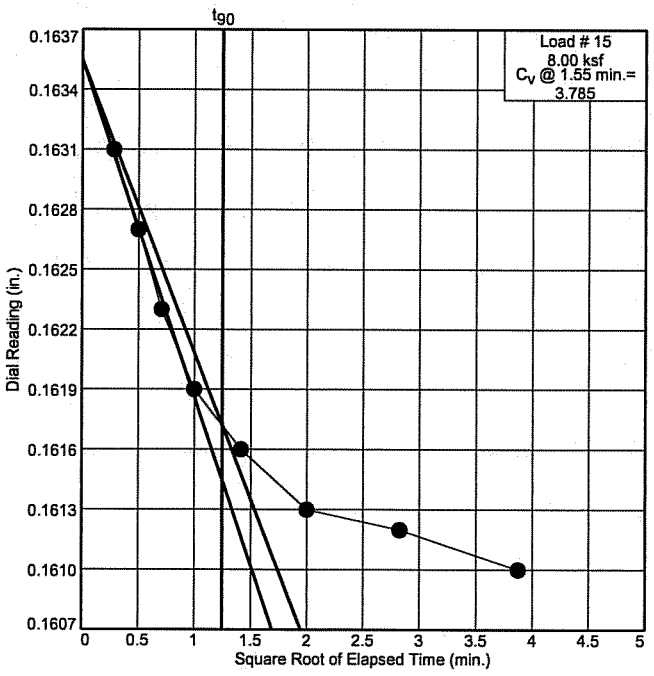
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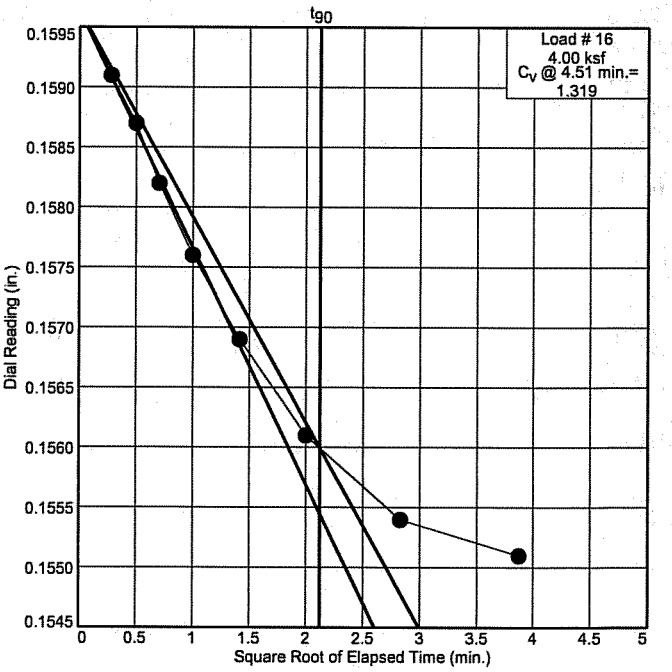
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Project: Scarborough Eastern Trail Project

Location: BB-SPC-203A Depth: 40'-42' Sample Number: U-4









R.W. Gillespie & Associates, Inc.

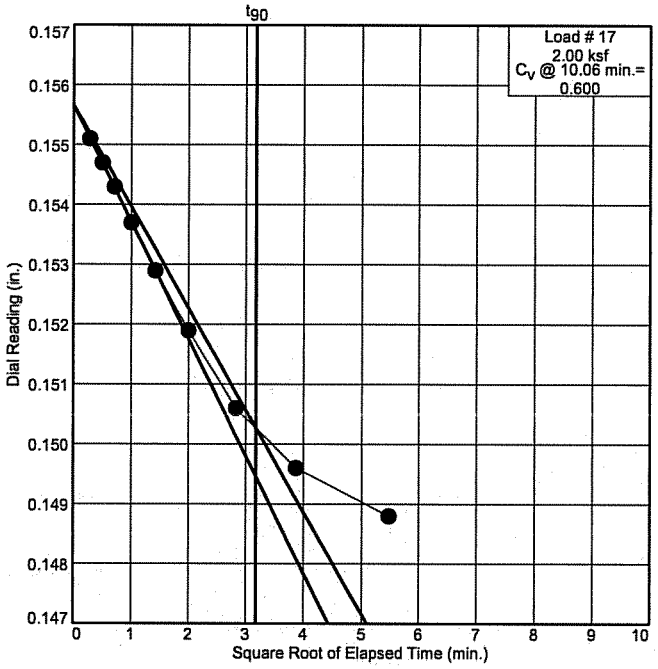
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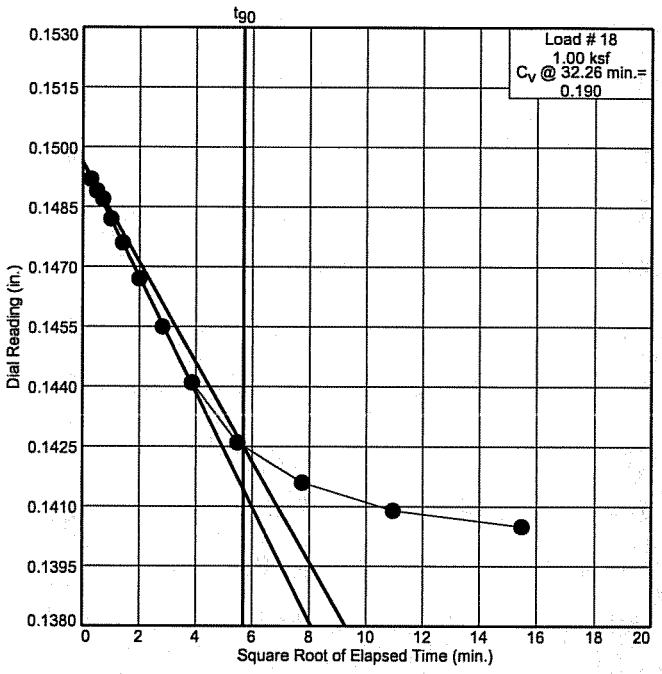
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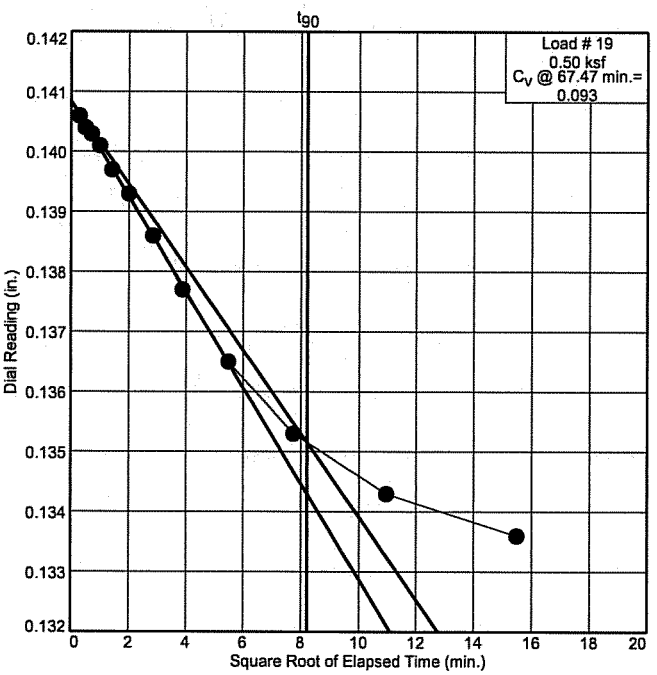
Dial Reading vs. Time

Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: BB-SPC-203A Depth: 40'-42' Sample Number: U-4





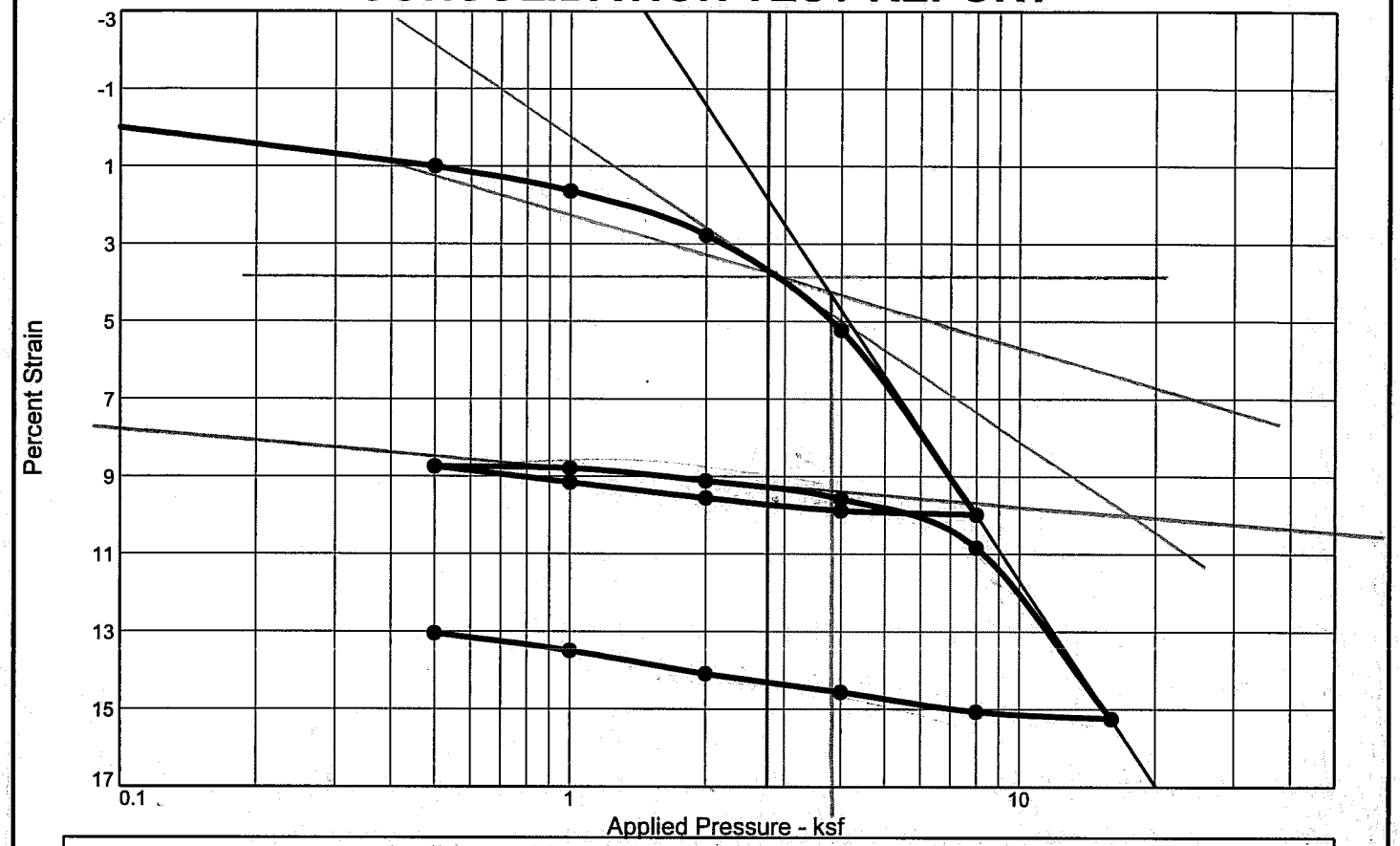


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Biddeford, Maine

Lab No. 15116c

CONSOLIDATION TEST REPORT



Coefficients of Consolidation and Secondary Consolidation

No.	Load (ksf)	C_v (ft.2/day)	C_α	No.	Load (ksf)	C_v (ft.2/day)	C_α	No.	Load (ksf)	C_v (ft.2/day)	C_α
1	0.50	0.524		8	1.00	0.901		15	8.00	3.946	
2	1.00	0.802		9	0.50	0.344		16	4.00	2.567	
3	2.00	0.654		10	1.00	20.986		17	2.00	0.723	
4	4.00	0.506		11	2.00	1.165		18	1.00	0.337	
5	8.00	0.317		12	4.00	1.418		19	0.50	0.141	
6	4.00	4.836		13	8.00	0.855					
7	2.00	2.412		14	16.00	0.343					

Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	Initial Void Ratio
Saturation	Moisture							
102.8 %	37.4 %	84.5	29.9	10.7	2.70	3.2	3.7	0.981

MATERIAL DESCRIPTION	USCS	AASHTO
Lean Clay		

Project No. 0600-102 **Client:** HNTB Corporation

Project: Scarborough Eastern Trail Project
Scarborough, ME

Location: BB-SPC-203A **Depth:** 60'-62' **Sample Number:** U-6

R.W. Gillespie & Associates, Inc.

Biddeford, Maine

Remarks:

Lab No. 15111d

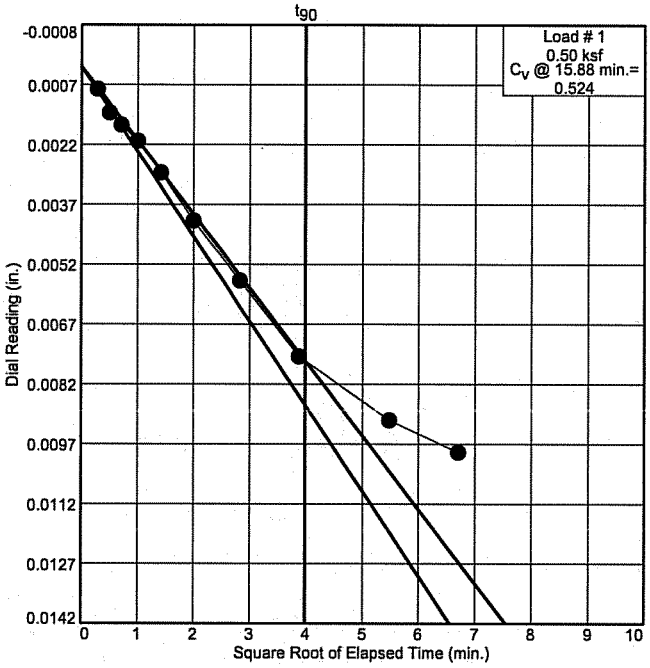
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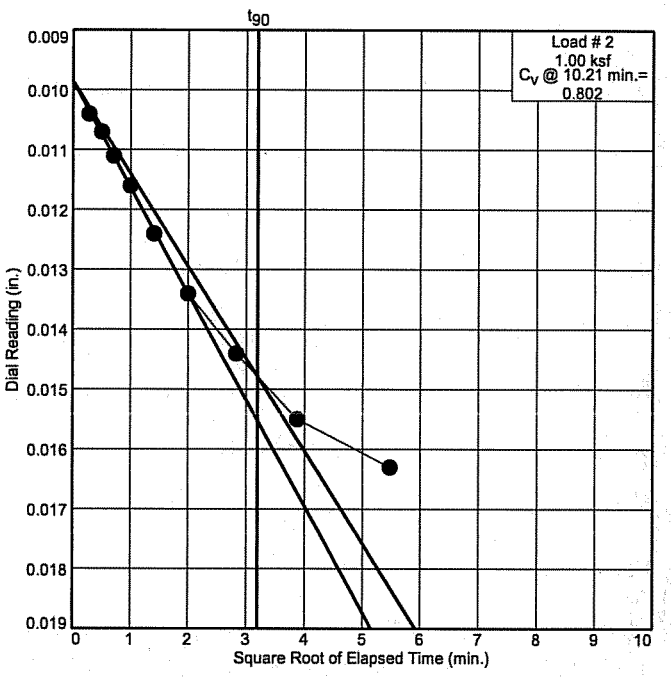
revise to match Dia. Cr

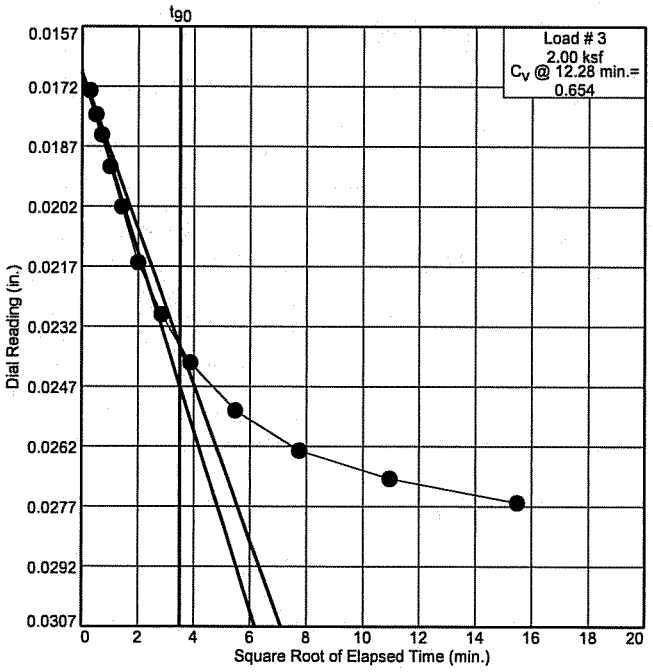
Dial Reading vs. Time

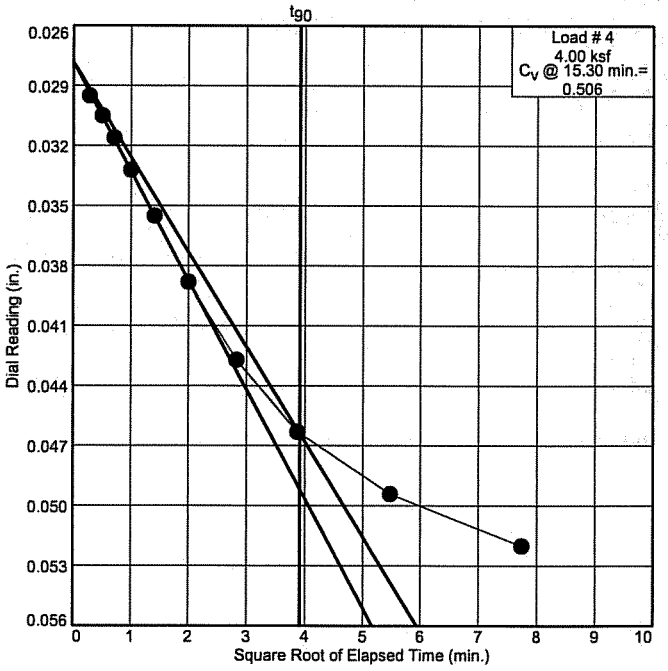
Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: BB-SPC-203A Depth: 60'-62' Sample Number: U-6









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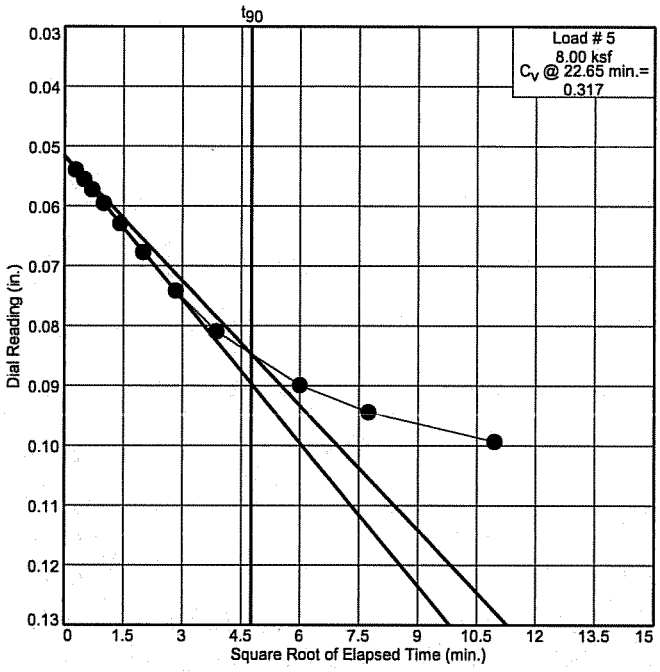
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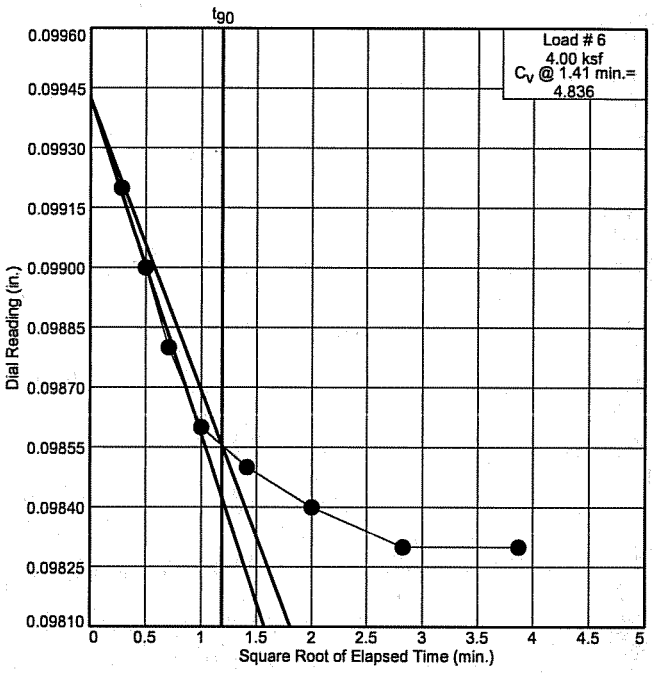
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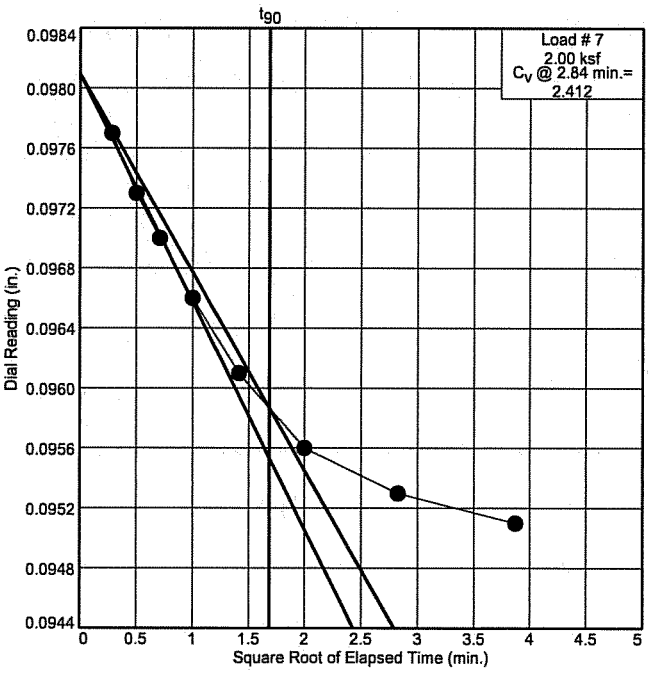
Dial Reading vs. Time

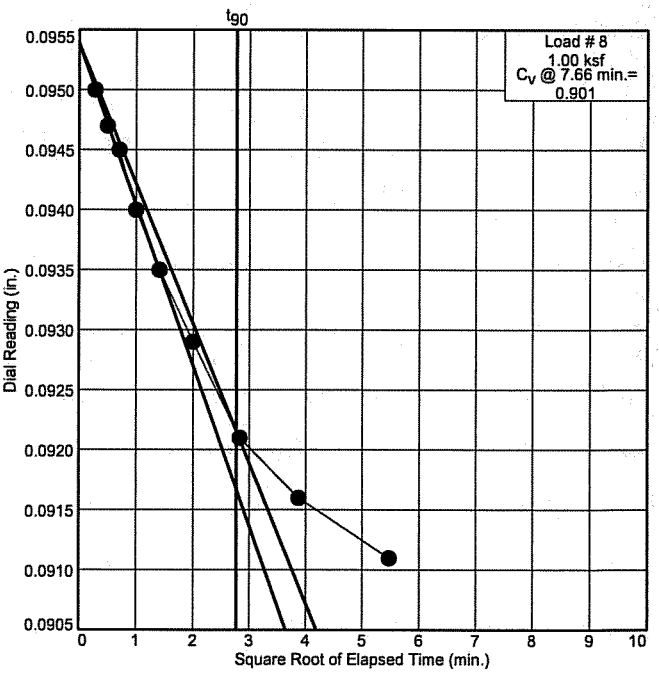
Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: BB-SPC-203A Depth: 60'-62' Sample Number: U-6









R.W. Gillespie & Associates, Inc.

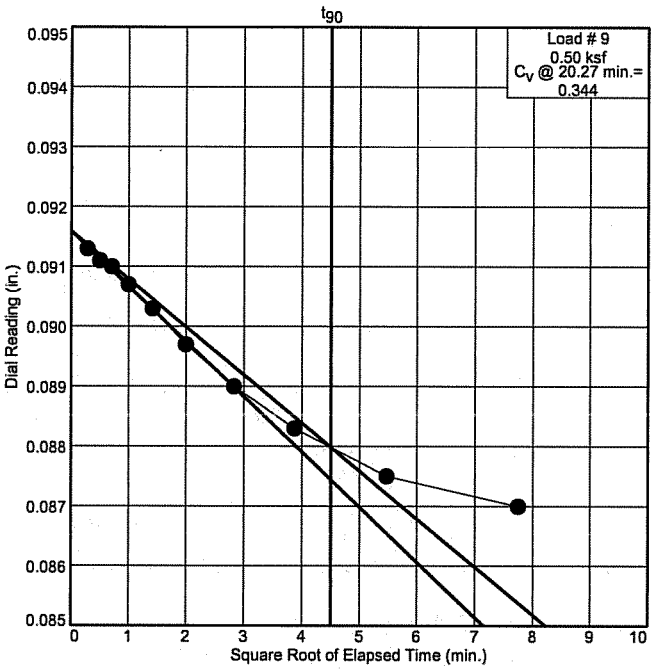
Biddeford, Maine

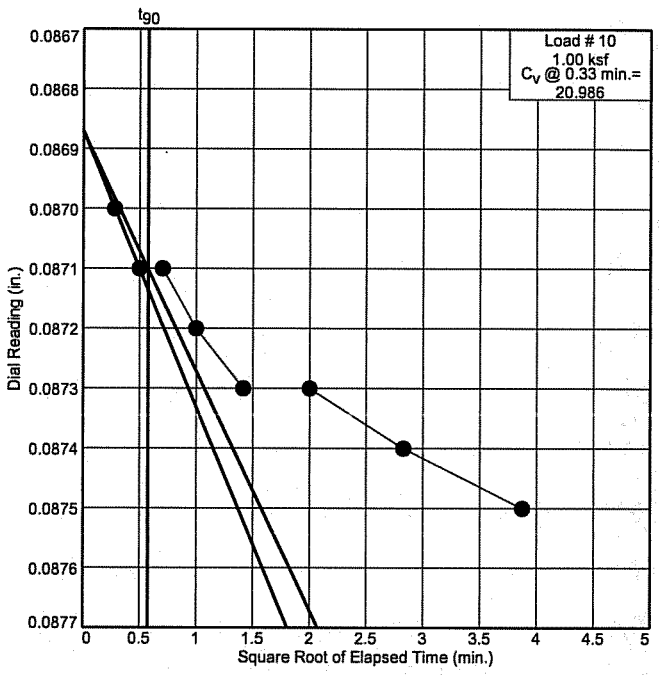
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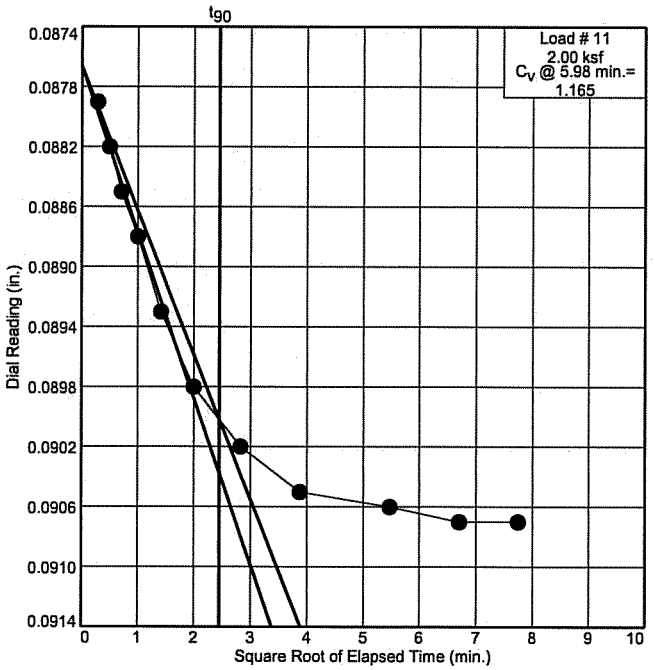
Dial Reading vs. Time

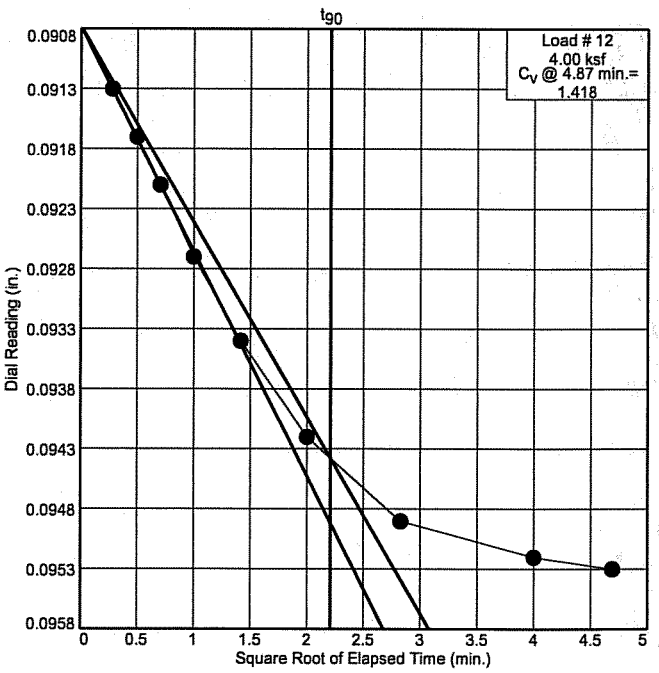
Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: BB-SPC-203A Depth: 60'-62' Sample Number: U-6









R.W. Gillespie & Associates, Inc.

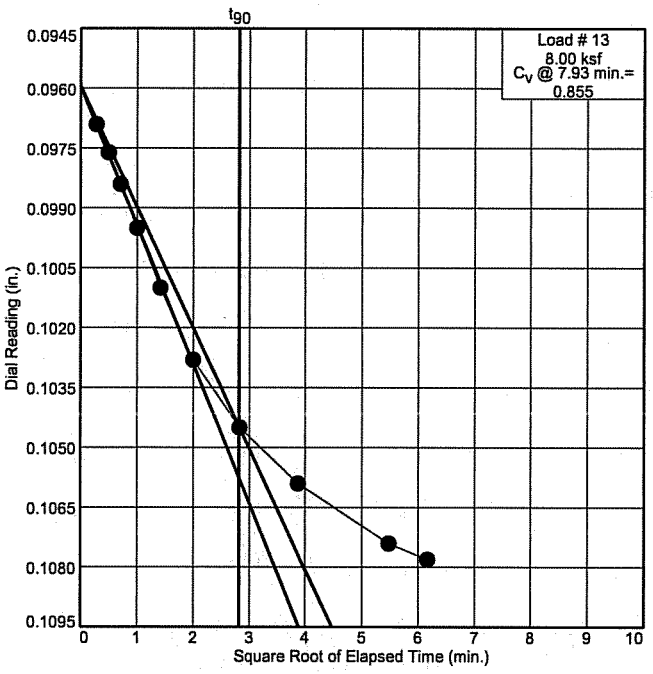
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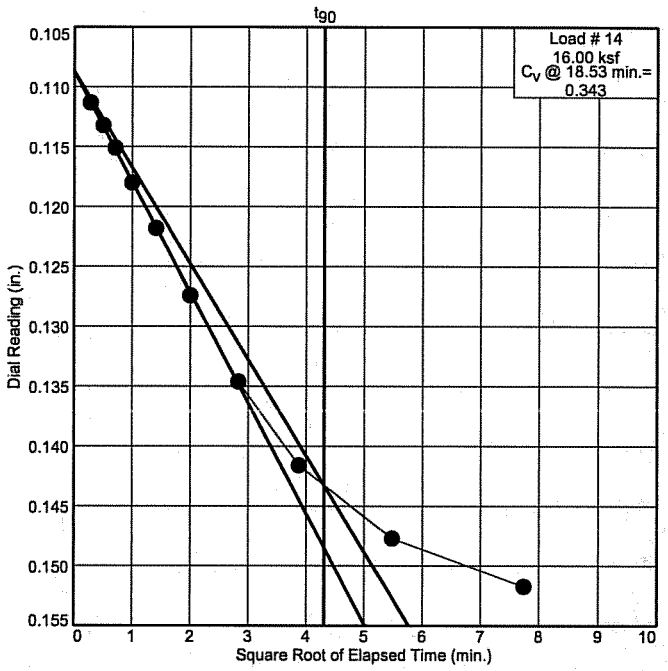
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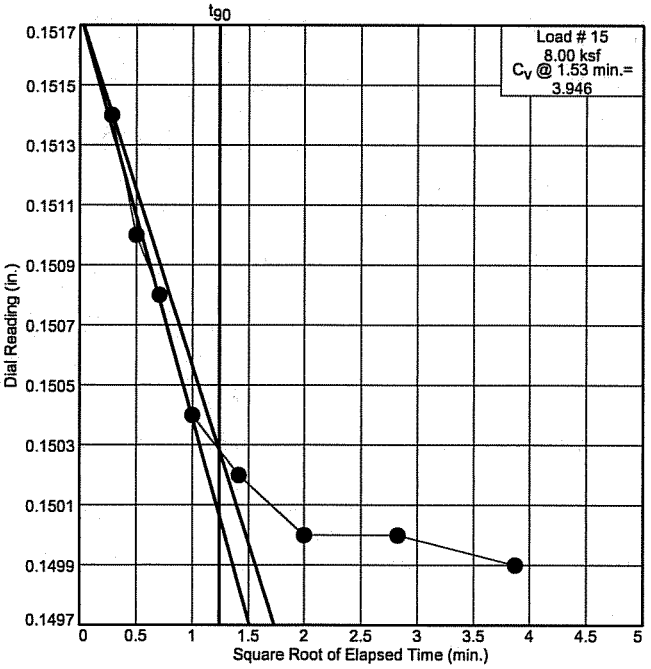
Dial Reading vs. Time

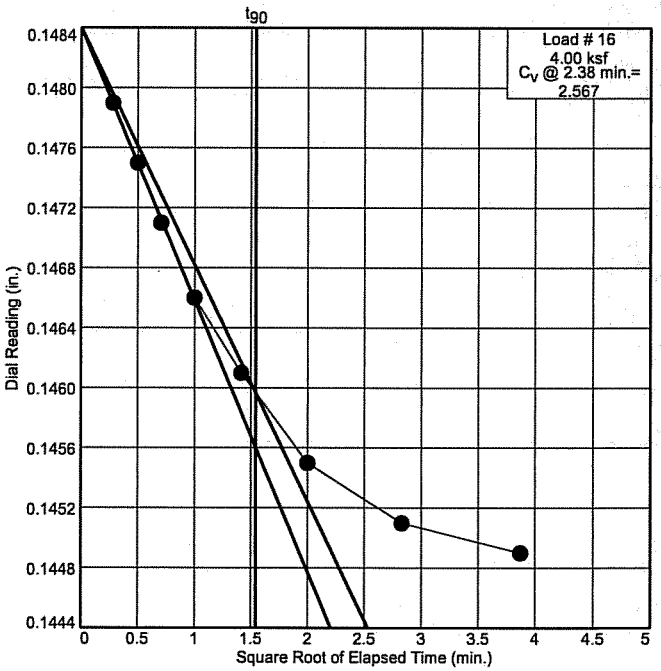
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Project: Scarborough Eastern Trail Project

Location: BB-SPC-203A Depth: 60'-62' Sample Number: U-6









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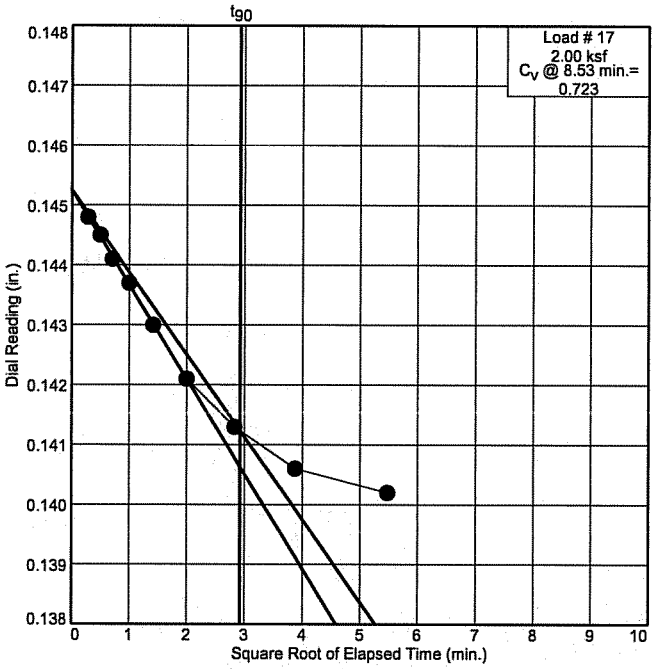
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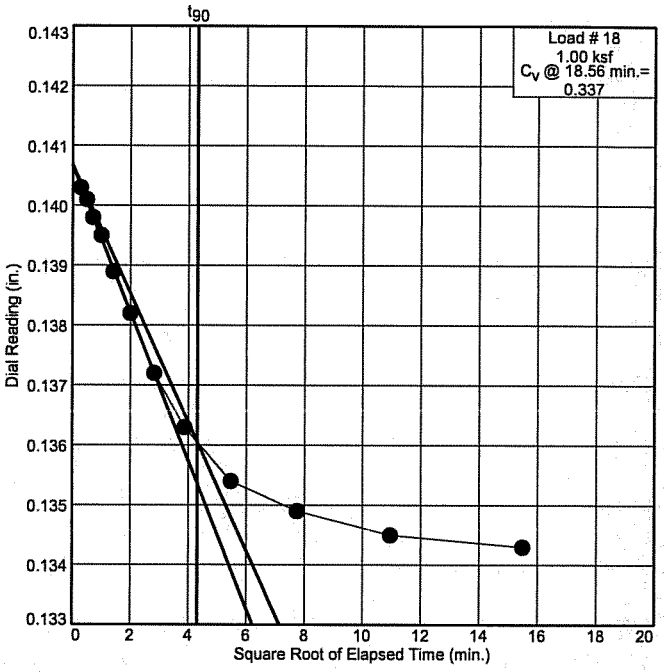
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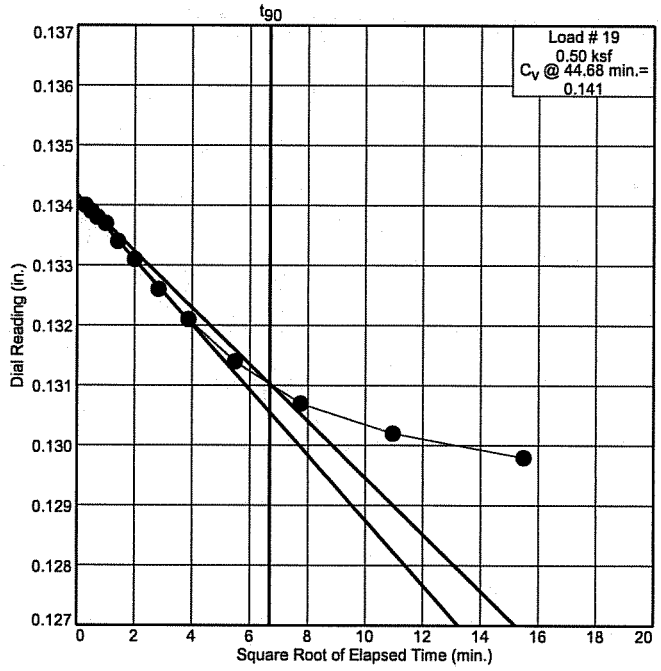
Dial Reading vs. Time

Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: BB-SPC-203A Depth: 60'-62' Sample Number: U-6





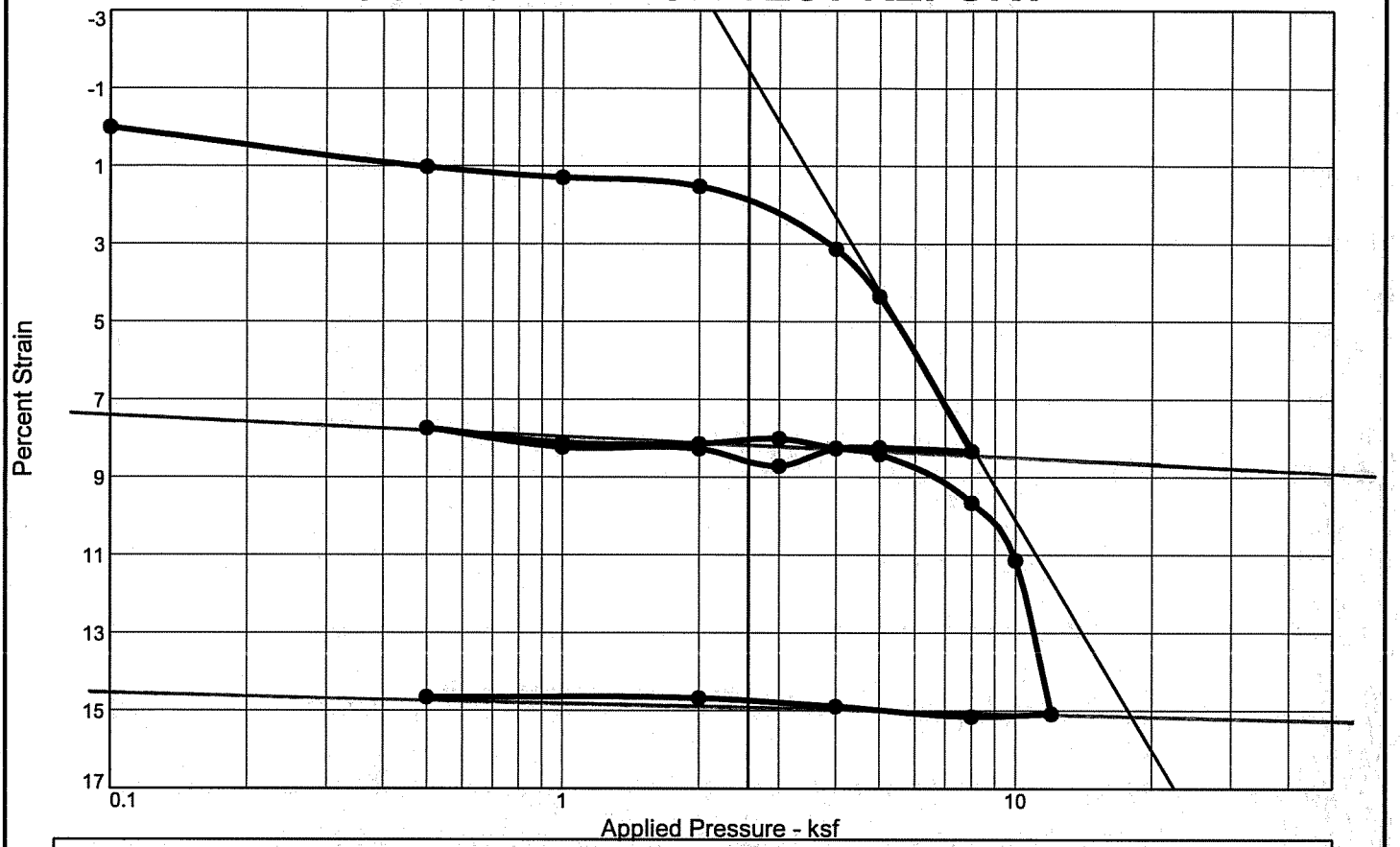


R.W. Gillespie & Associates, Inc.

Biddeford, Maine

Lab No. 15116d

CONSOLIDATION TEST REPORT



Coefficients of Consolidation and Secondary Consolidation

No.	Load (ksf)	C_v (ft.2/day)	C_α	No.	Load (ksf)	C_v (ft.2/day)	C_α	No.	Load (ksf)	C_v (ft.2/day)	C_α
2	0.50	0.106		9	4.00	0.077		16	3.00	2.684	
3	1.00	0.187		10	3.00	2.643		17	4.00	0.523	
4	2.00	0.089		11	2.00	0.230		18	5.00	0.543	
5	4.00	0.061		12	1.00	1.001		19	8.00	0.194	
6	5.00	0.033		13	0.50	0.139		20	10.00	0.087	
7	8.00	0.054		14	1.00	0.106		21	12.00	0.059	
8	5.00	2.660		15	2.00	1.220		22	8.00	1.259	

Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_r	Initial Void Ratio
Saturation	Moisture									
97.3 %	37.0 %	84.1	NV		2.76		3.9	0.40	0.08	1.049

MATERIAL DESCRIPTION	USCS	AASHTO
Silty Clay	ML	A-4(0)

Project No. 0600-102 **Client:** HNTB Corporation

Project: Scarborough Eastern Trail Project
Scarborough, ME

Location: EB-SNR-102 **Depth:** 25'-27' **Sample Number:** U-1

R.W. Gillespie & Associates, Inc.

Biddeford, Maine

Remarks:

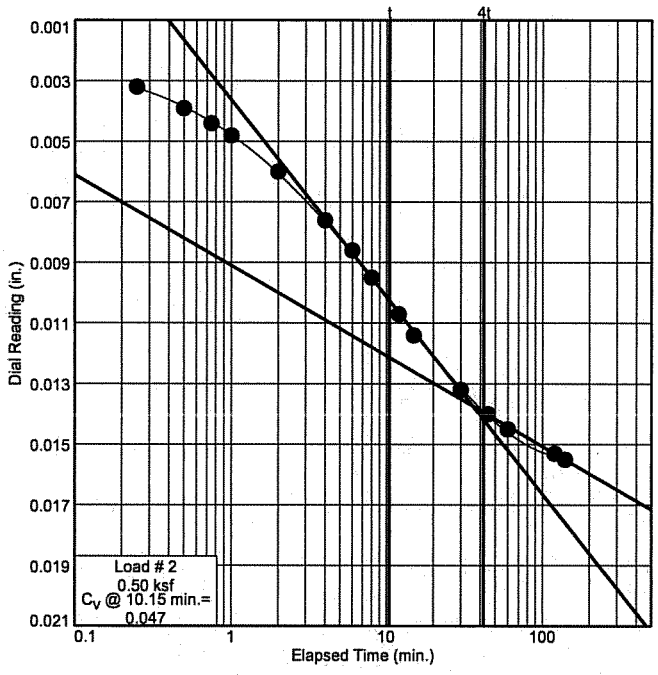
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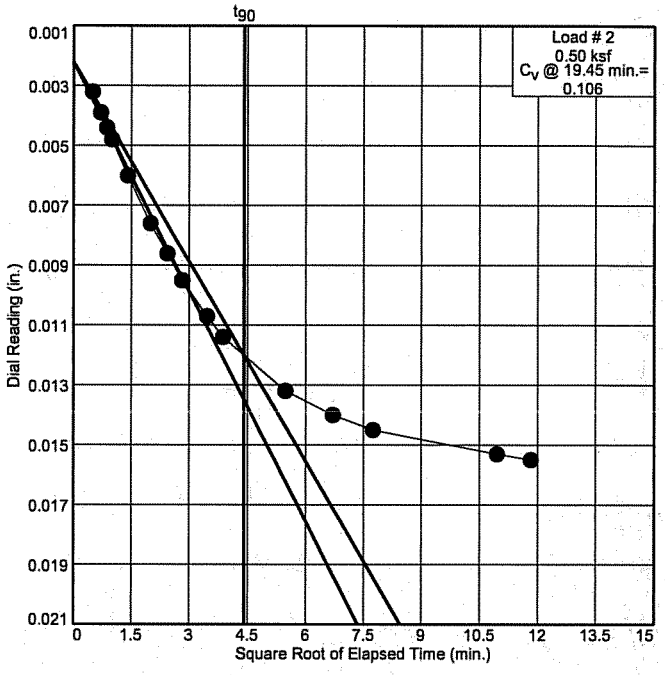
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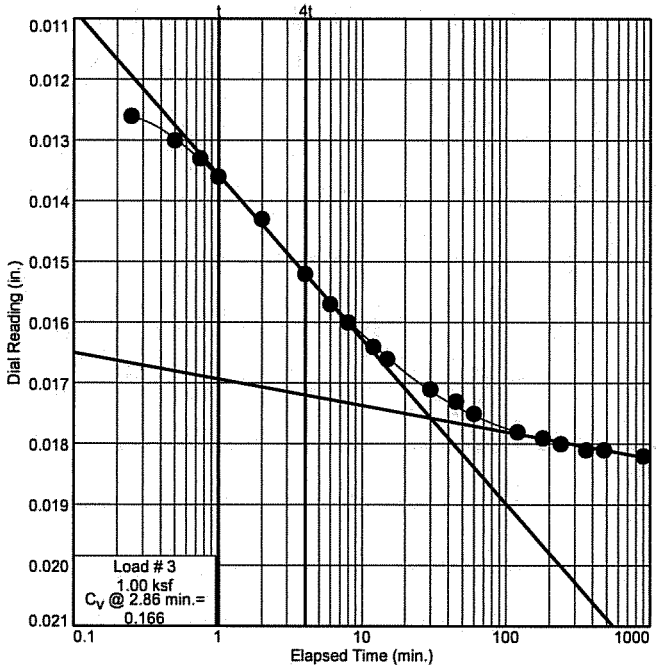
Dial Reading vs. Time

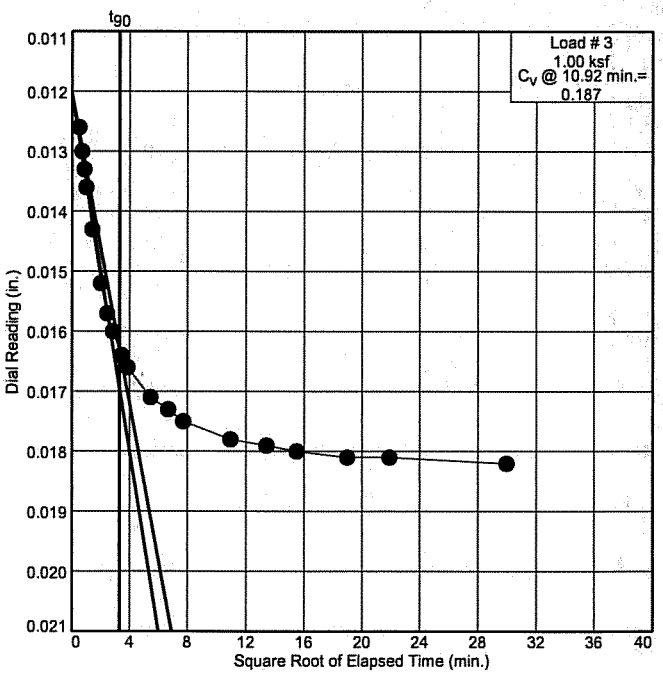
Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: EB-SNR-102 Depth: 25'-27' Sample Number: U-1









R.W. Gillespie & Associates, Inc.

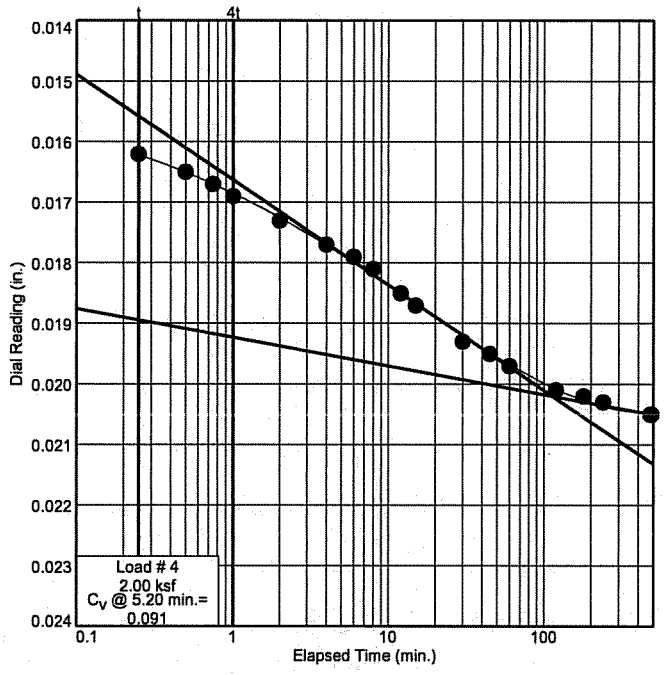
Biddeford, Maine

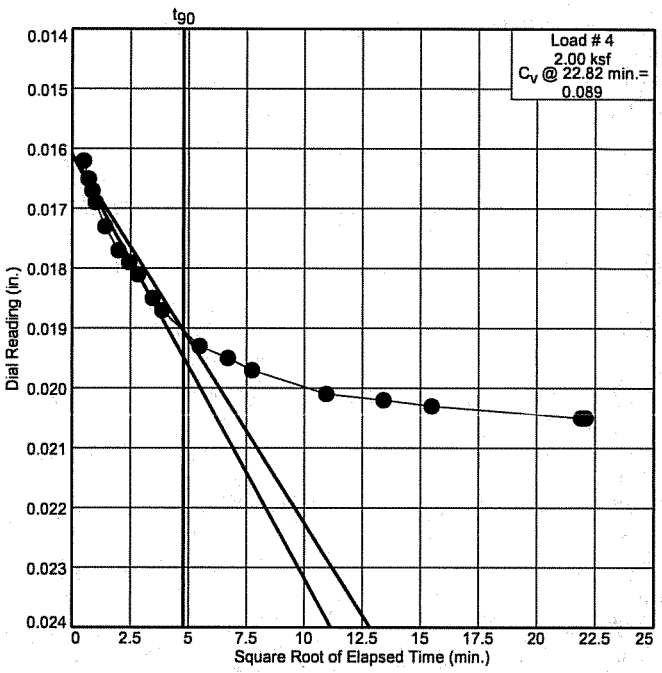
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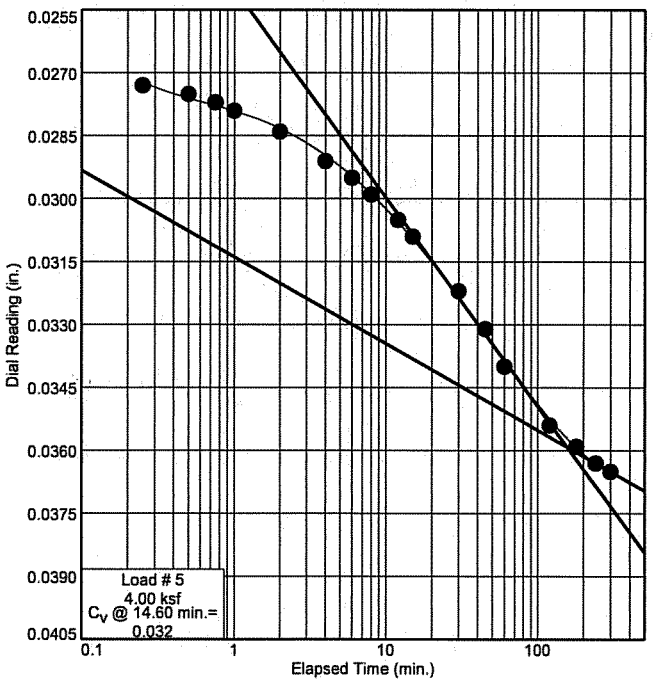
Dial Reading vs. Time

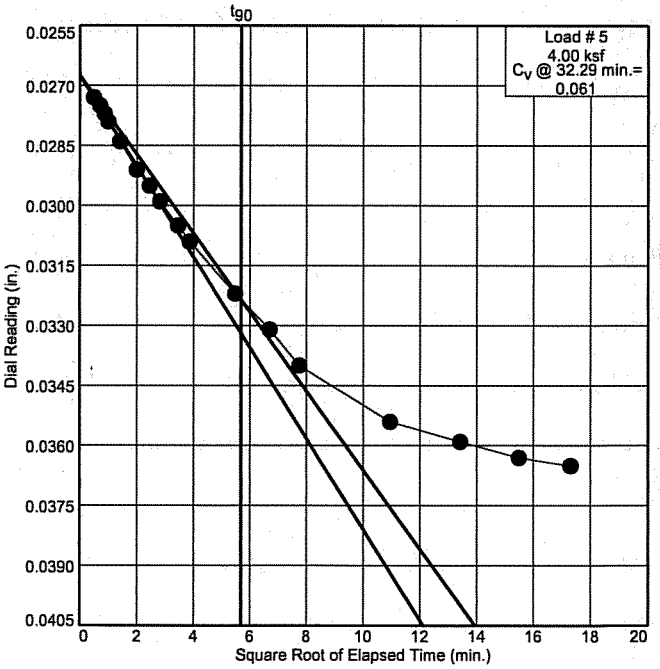
Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: EB-SNR-102 Depth: 25'-27' Sample Number: U-1









R.W. Gillespie & Associates, Inc.

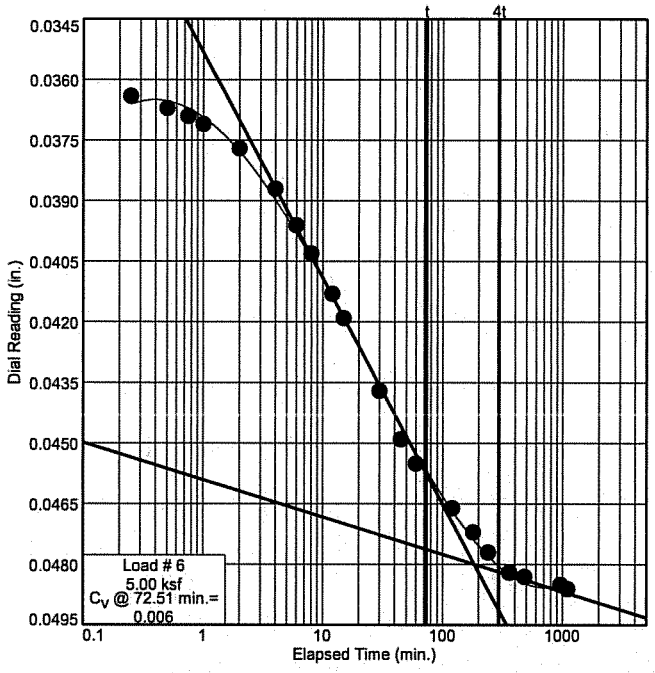
Biddeford, Maine

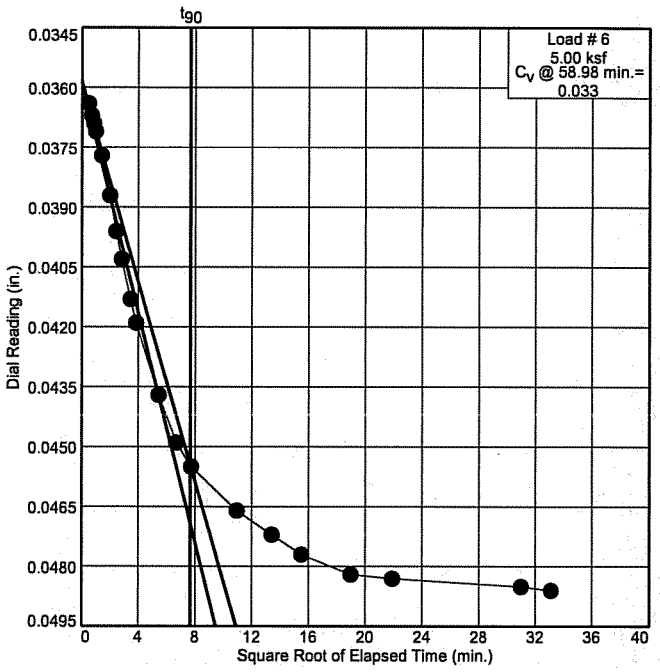
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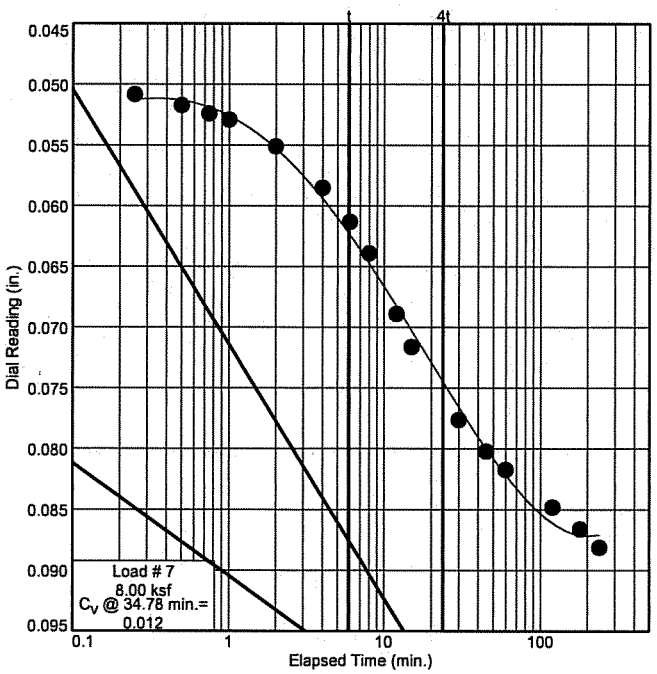
Dial Reading vs. Time

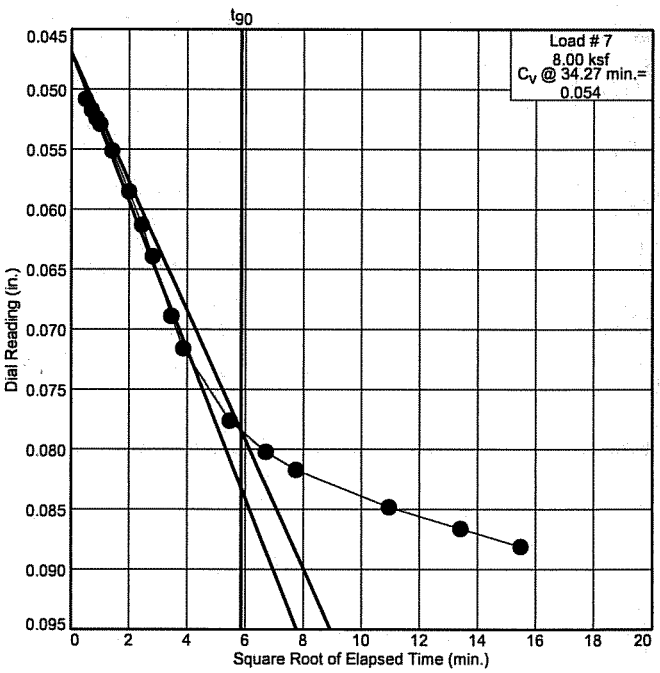
Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: EB-SNR-102 Depth: 25'-27' Sample Number: U-1









R.W. Gillespie & Associates, Inc.

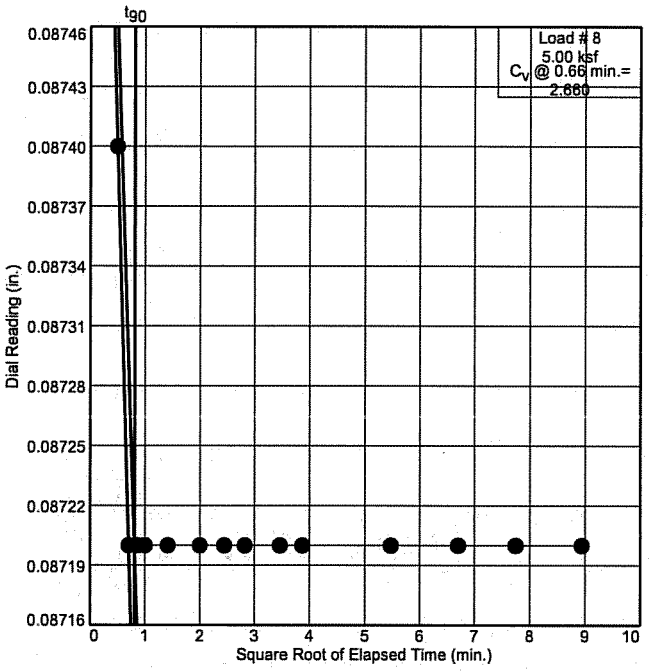
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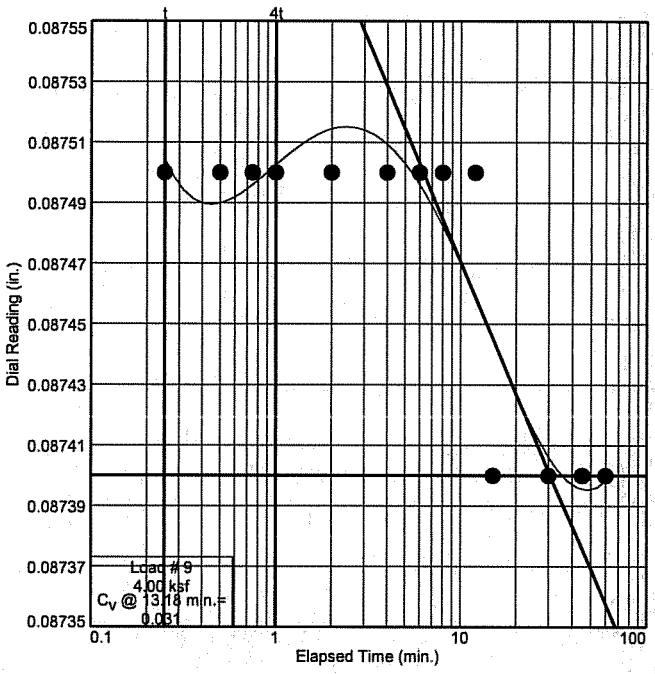
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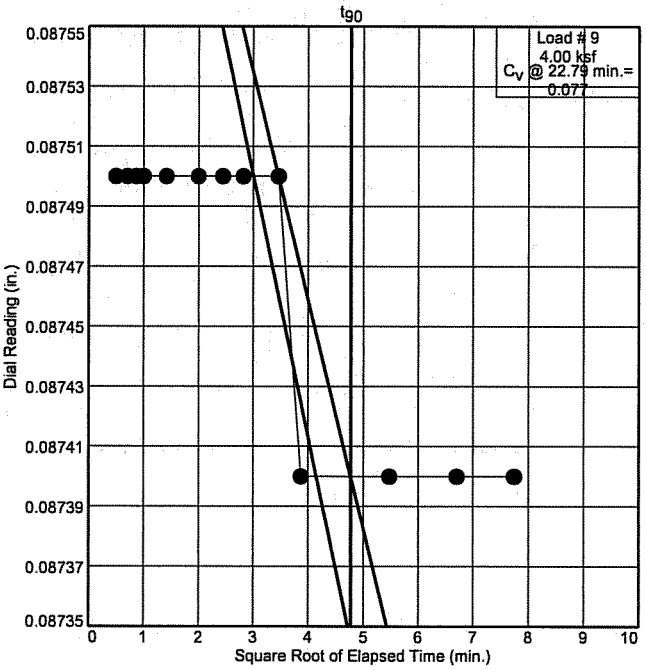
Dial Reading vs. Time

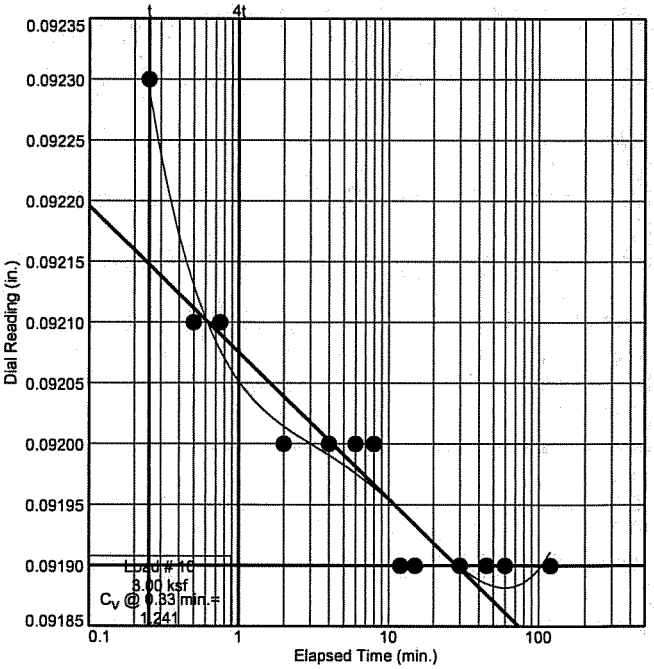
Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: EB-SNR-102 Depth: 25'-27' Sample Number: U-1









R.W. Gillespie & Associates, Inc.

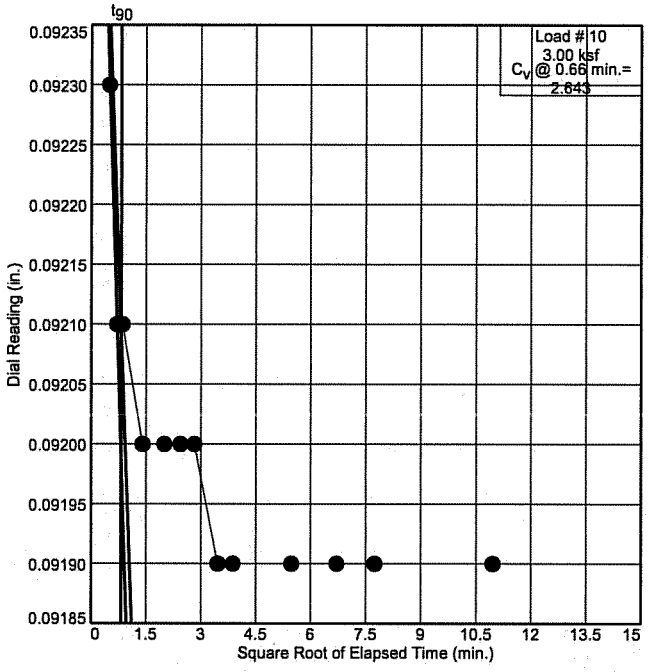
Biddeford, Maine

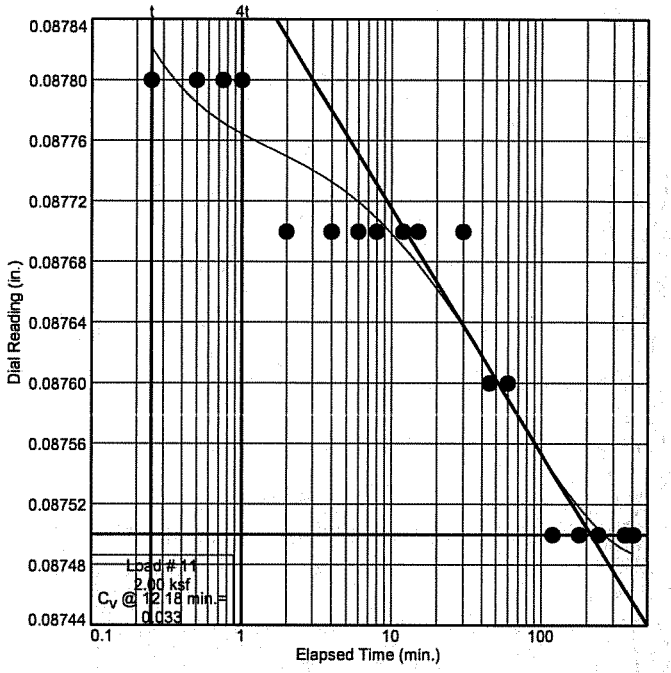
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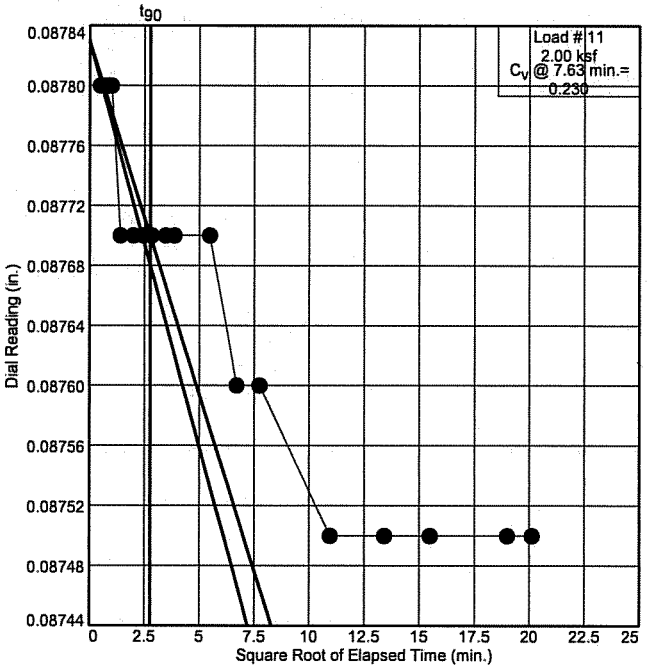
Dial Reading vs. Time

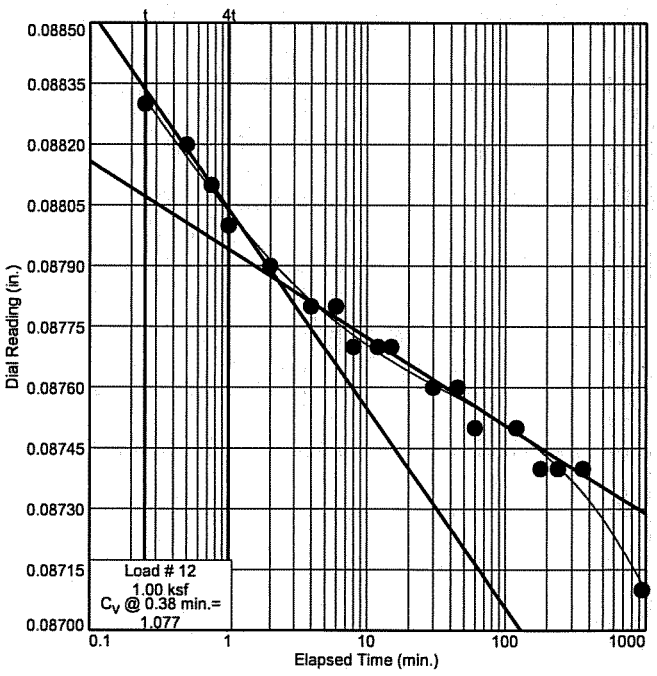
Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: EB-SNR-102 Depth: 25'-27' Sample Number: U-1









R.W. Gillespie & Associates, Inc.

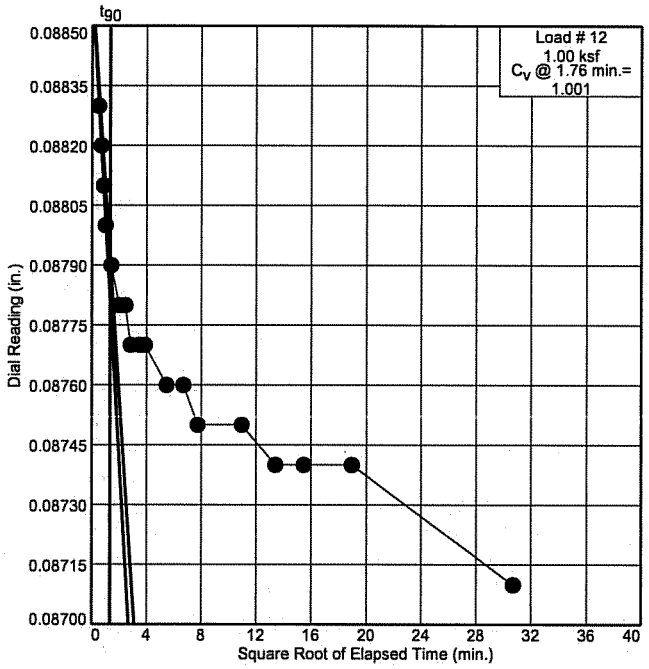
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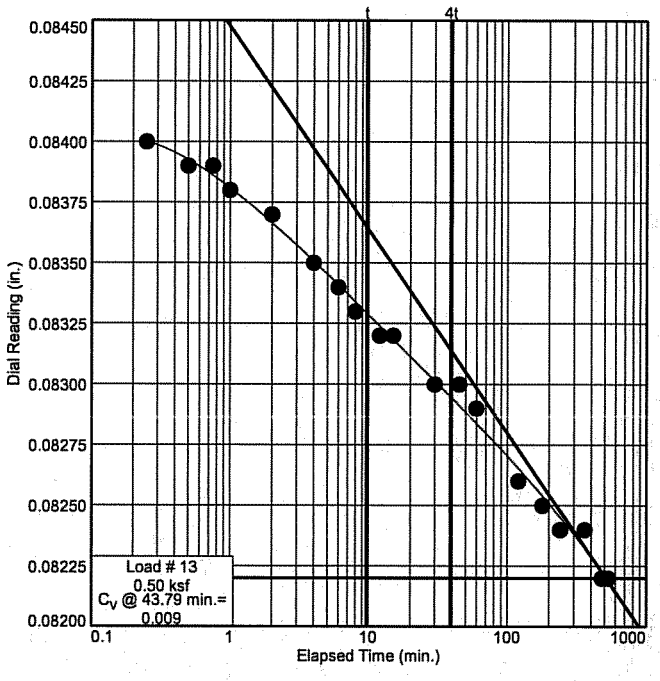
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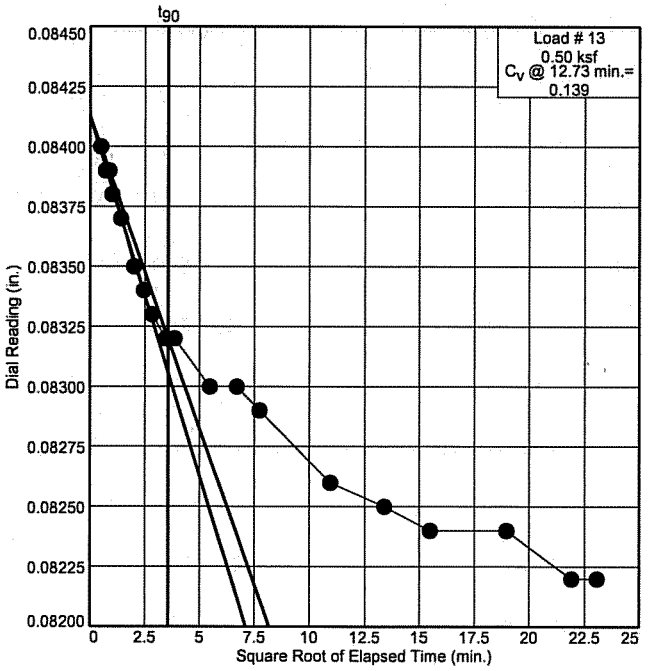
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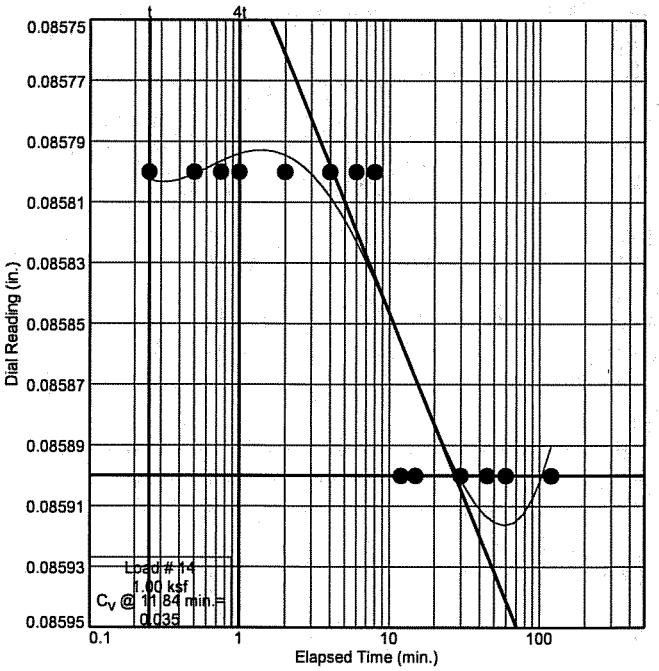
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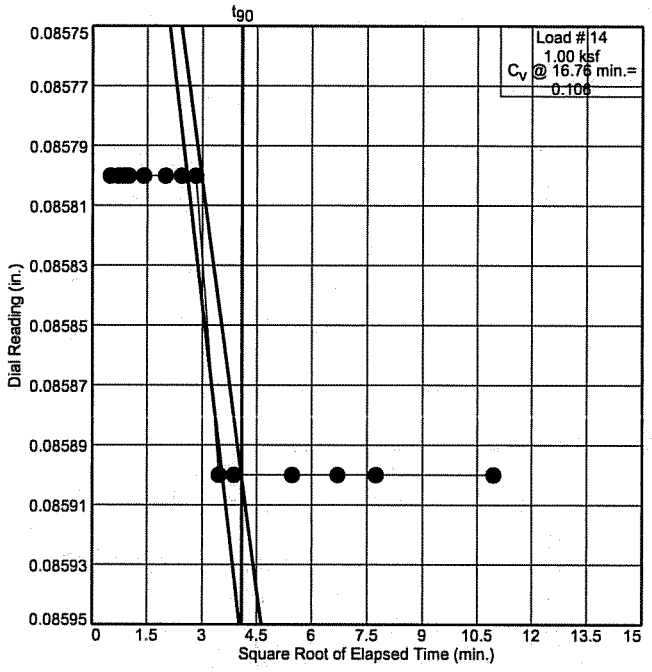
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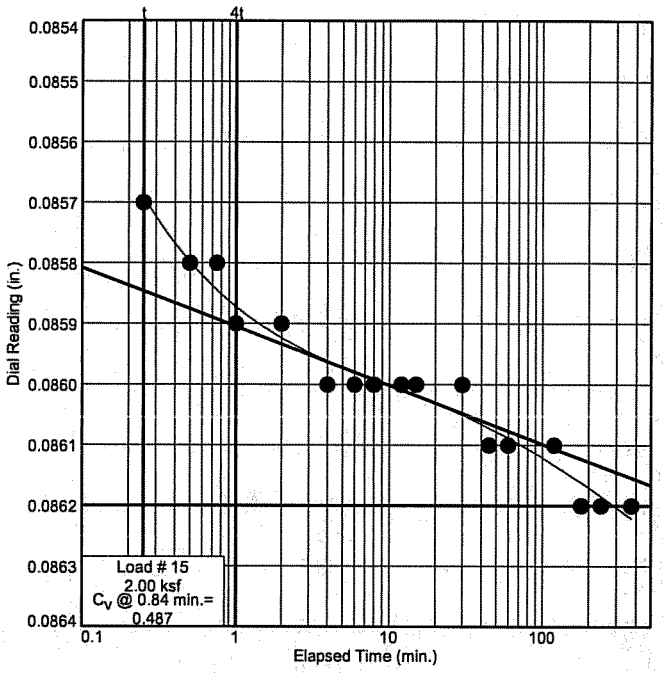
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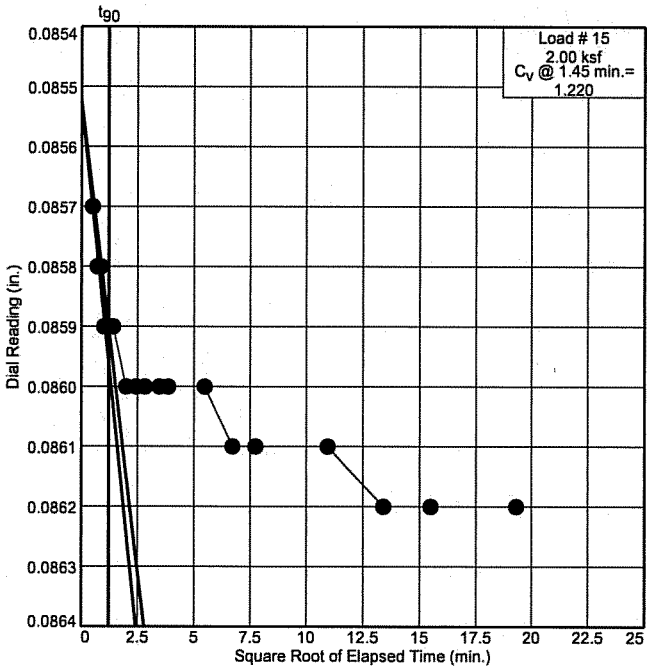
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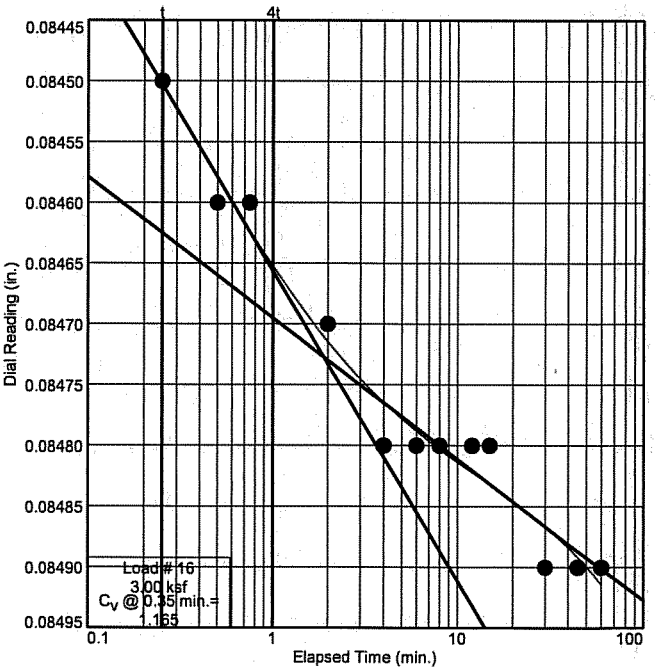
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Project: Scarborough Eastern Trail Project

Location: EB-SNR-102 Depth: 25'-27' Sample Number: U-1









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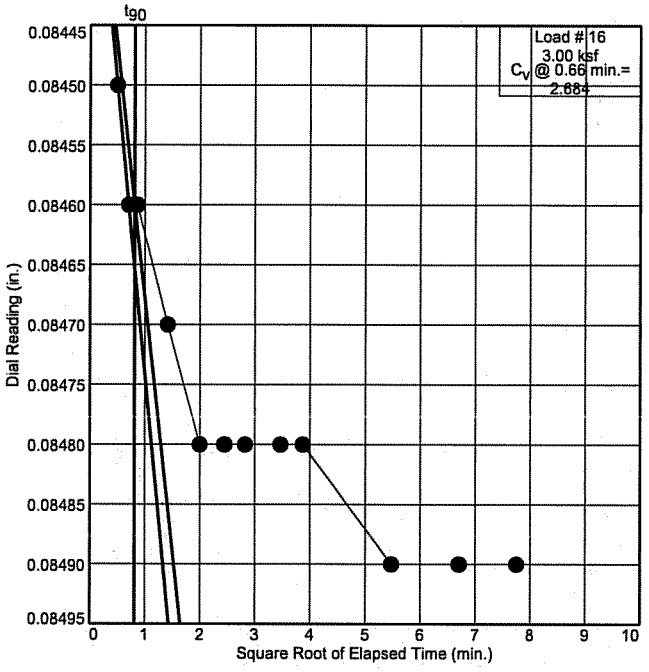
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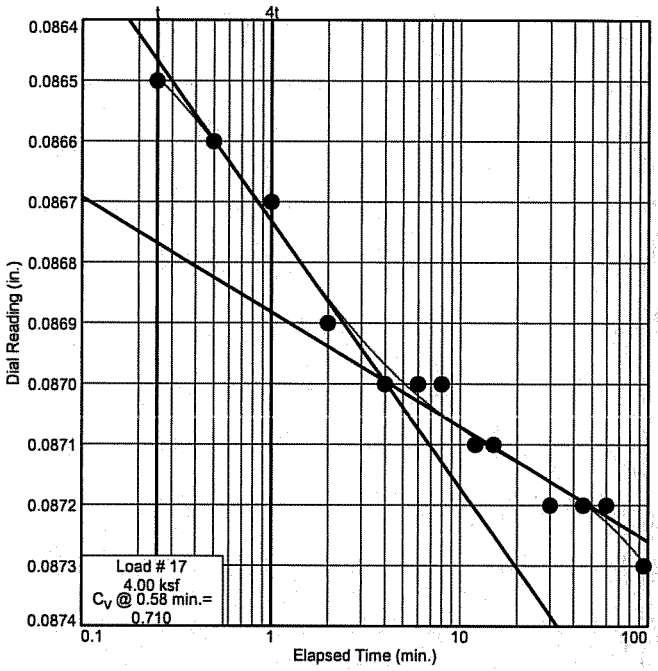
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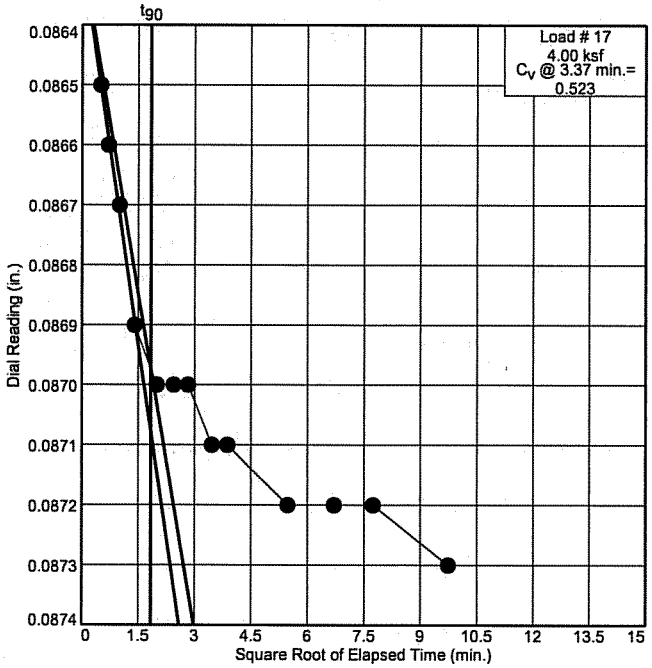
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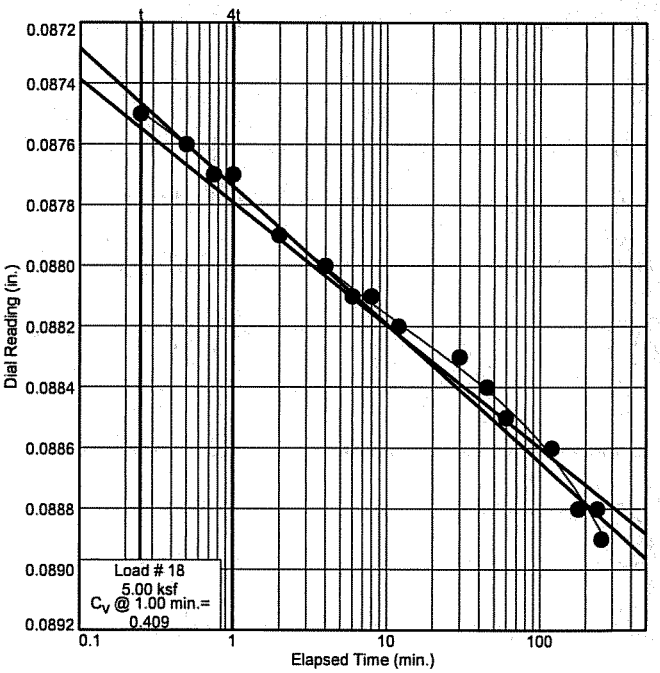
Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: EB-SNR-102 Depth: 25'-27' Sample Number: U-1









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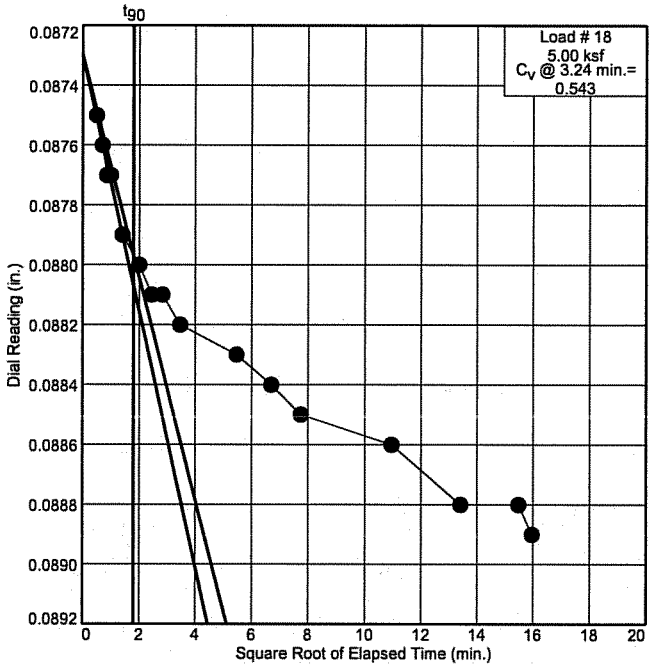
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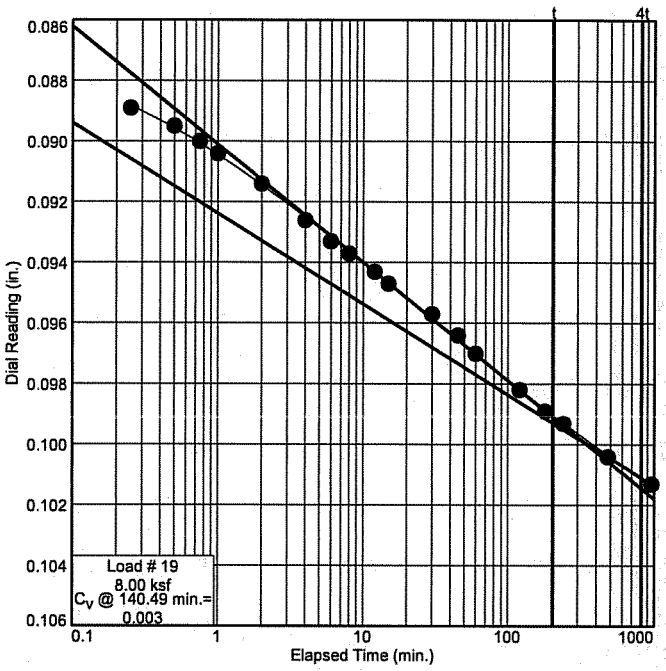
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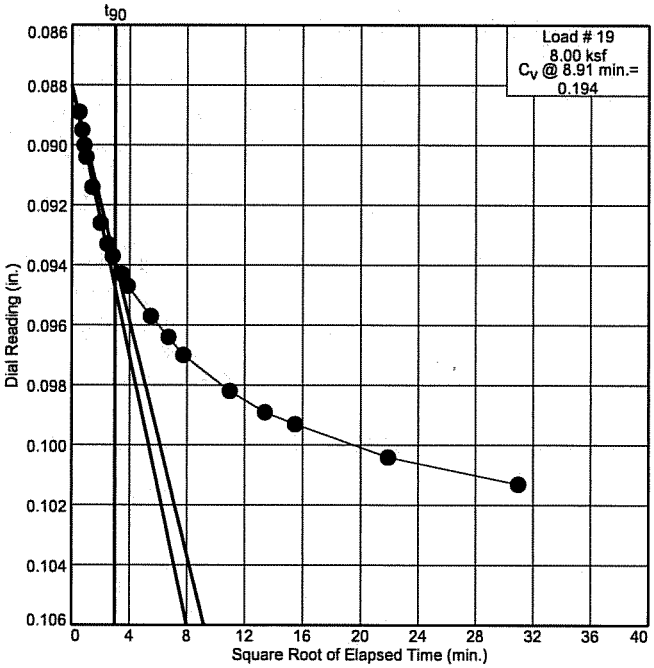
Dial Reading vs. Time

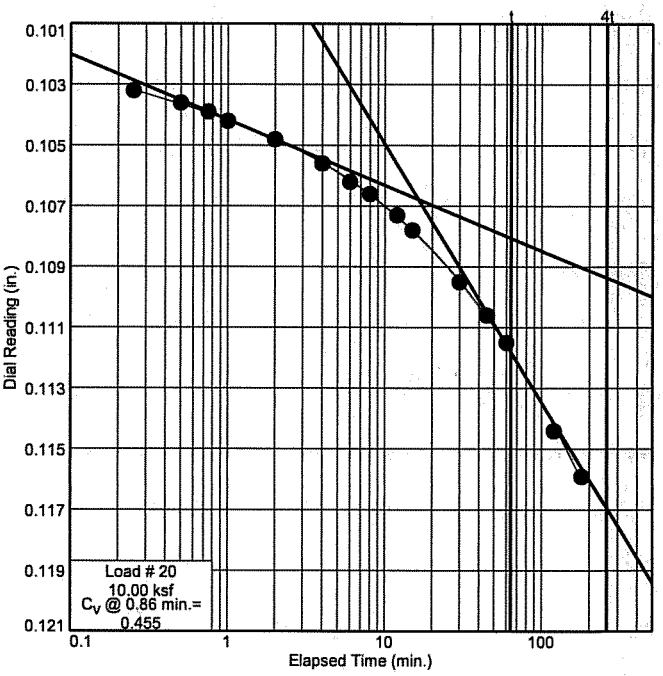
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Project: Scarborough Eastern Trail Project

Location: EB-SNR-102 Depth: 25'-27' Sample Number: U-1









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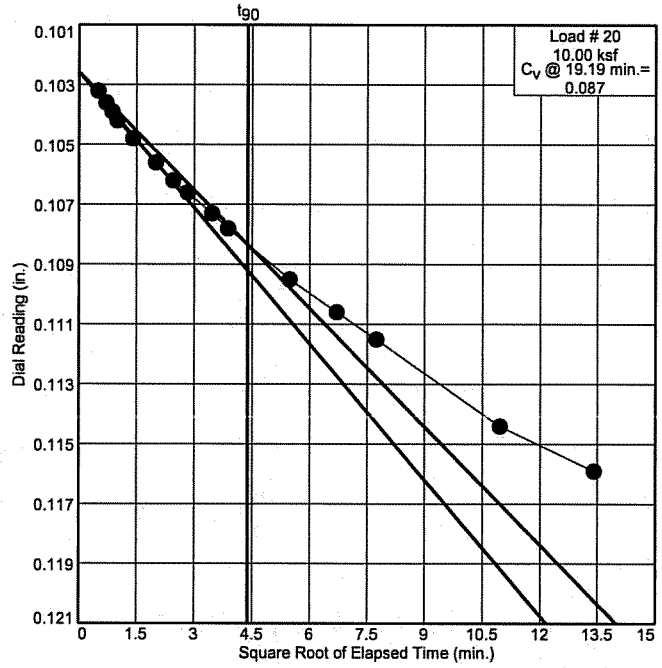
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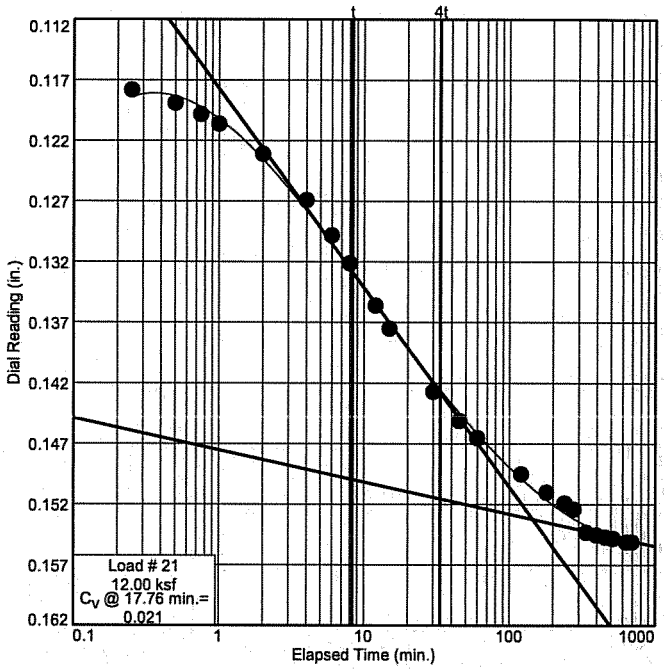
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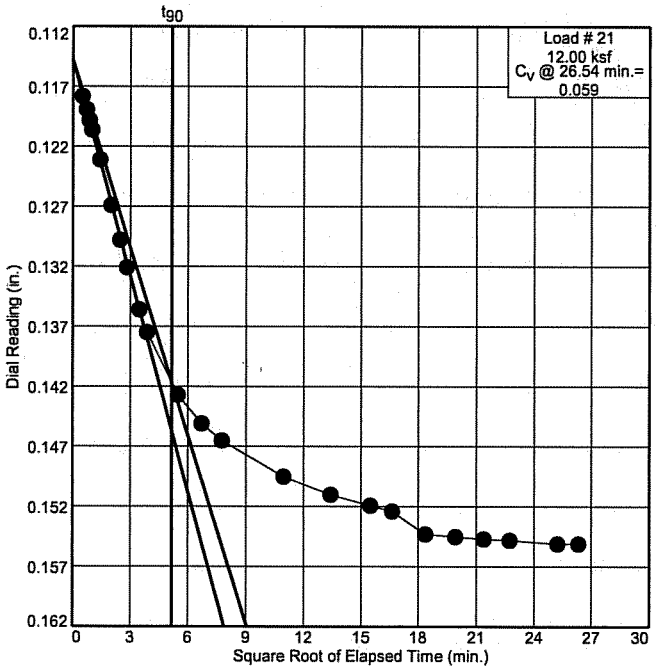
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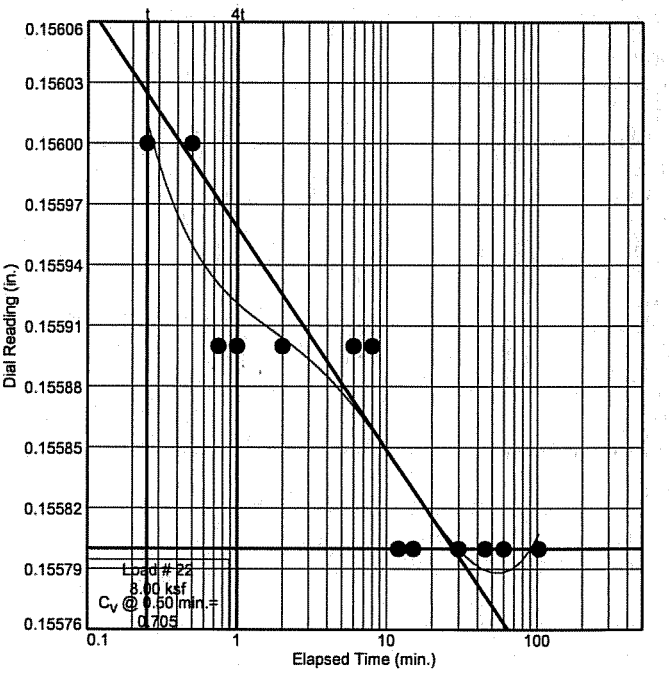
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Project: Scarborough Eastern Trail Project

Location: EB-SNR-102 Depth: 25'-27' Sample Number: U-1









R.W. Gillespie & Associates, Inc.

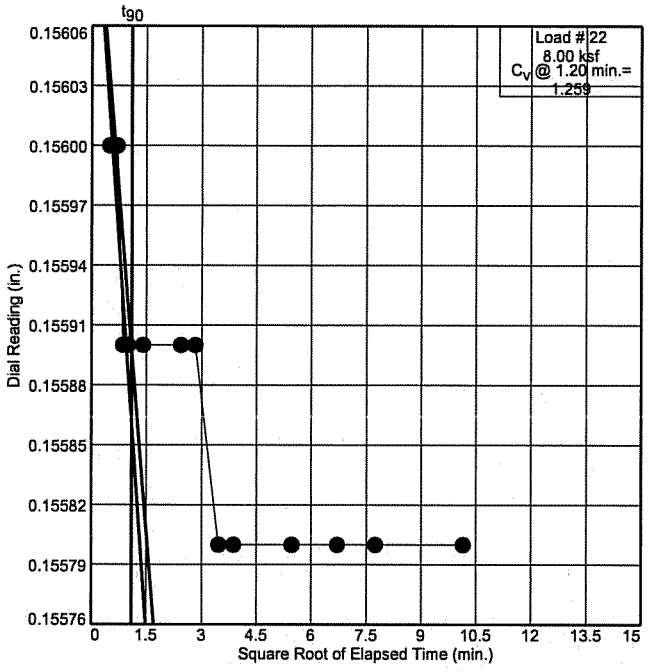
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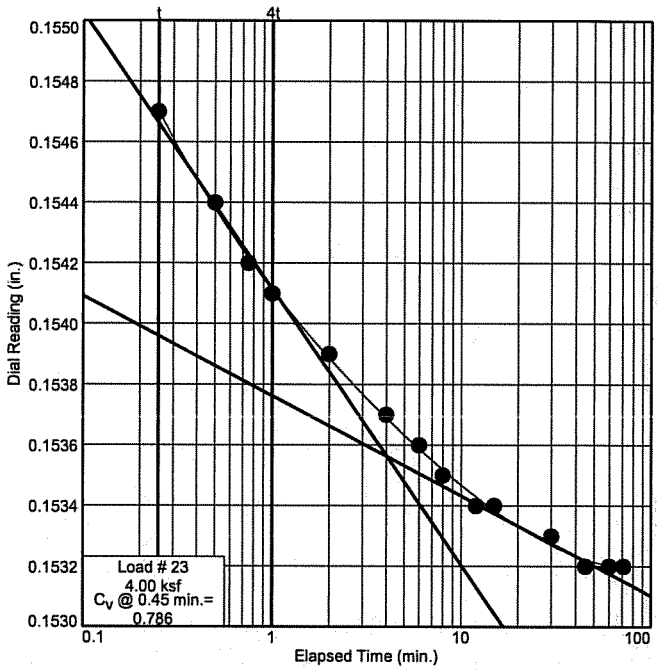
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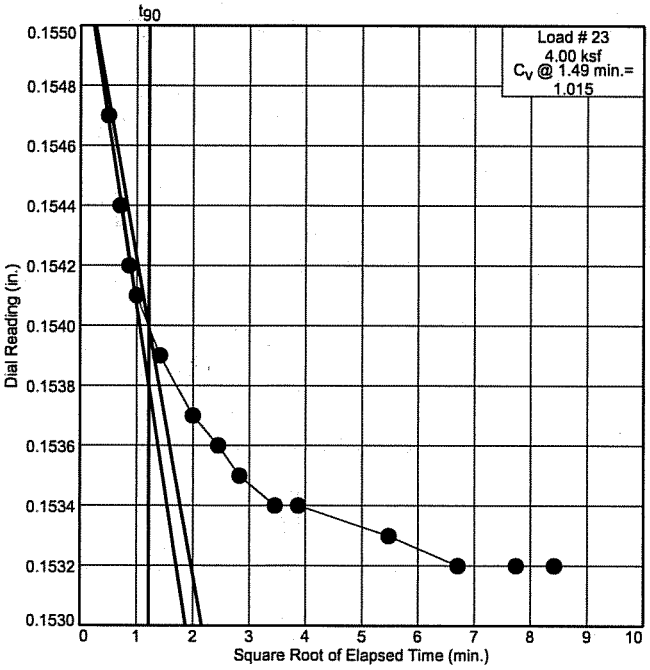
Dial Reading vs. Time

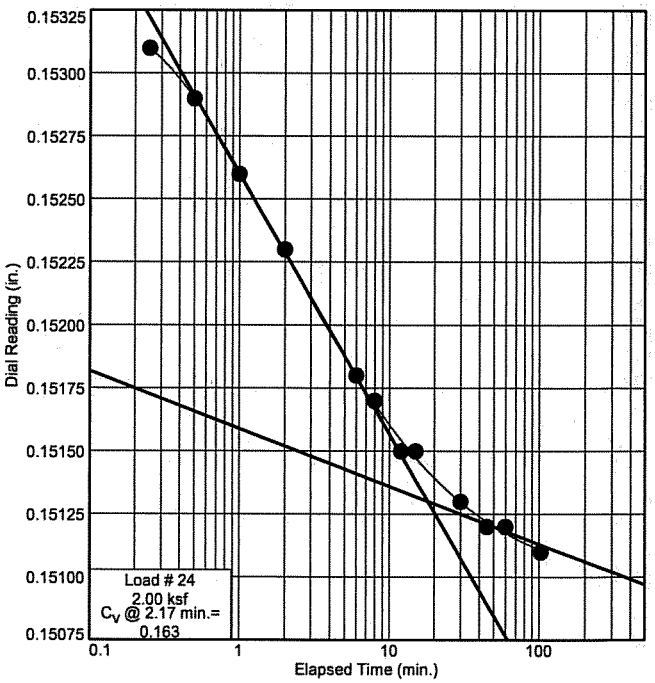
Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: EB-SNR-102 Depth: 25'-27' Sample Number: U-1









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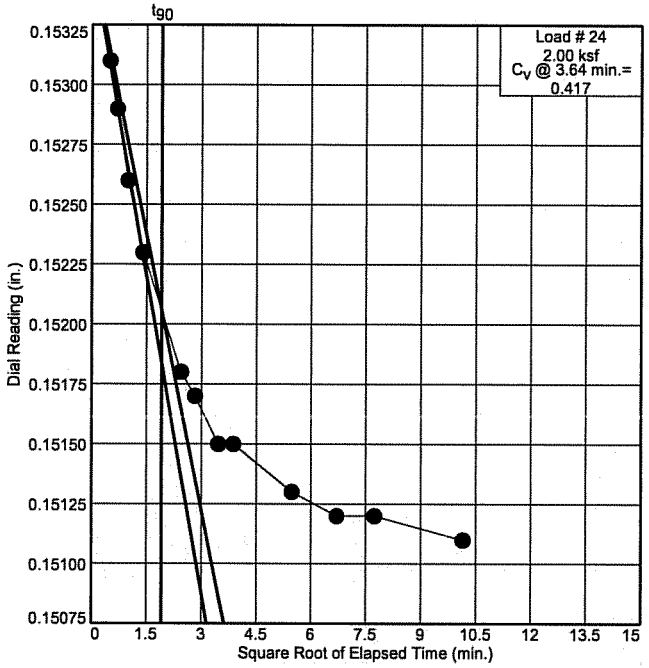
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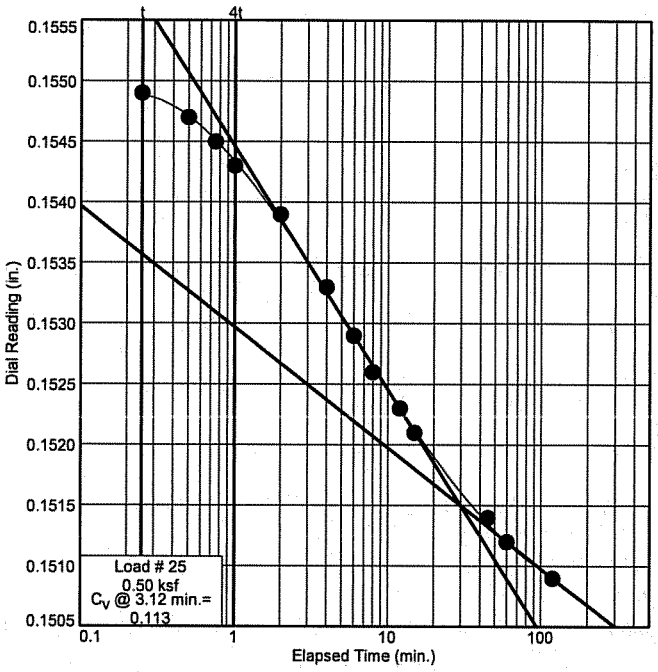
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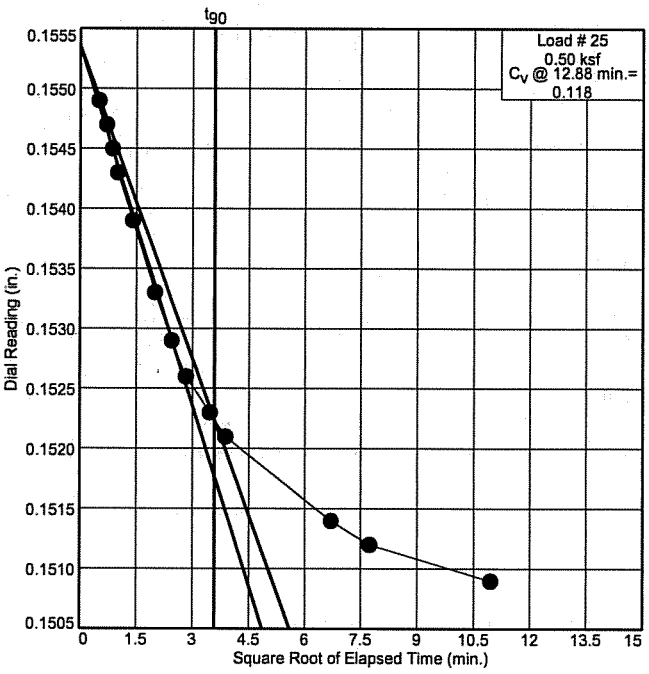
Dial Reading vs. Time

Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: EB-SNR-102 Depth: 25'-27' Sample Number: U-1





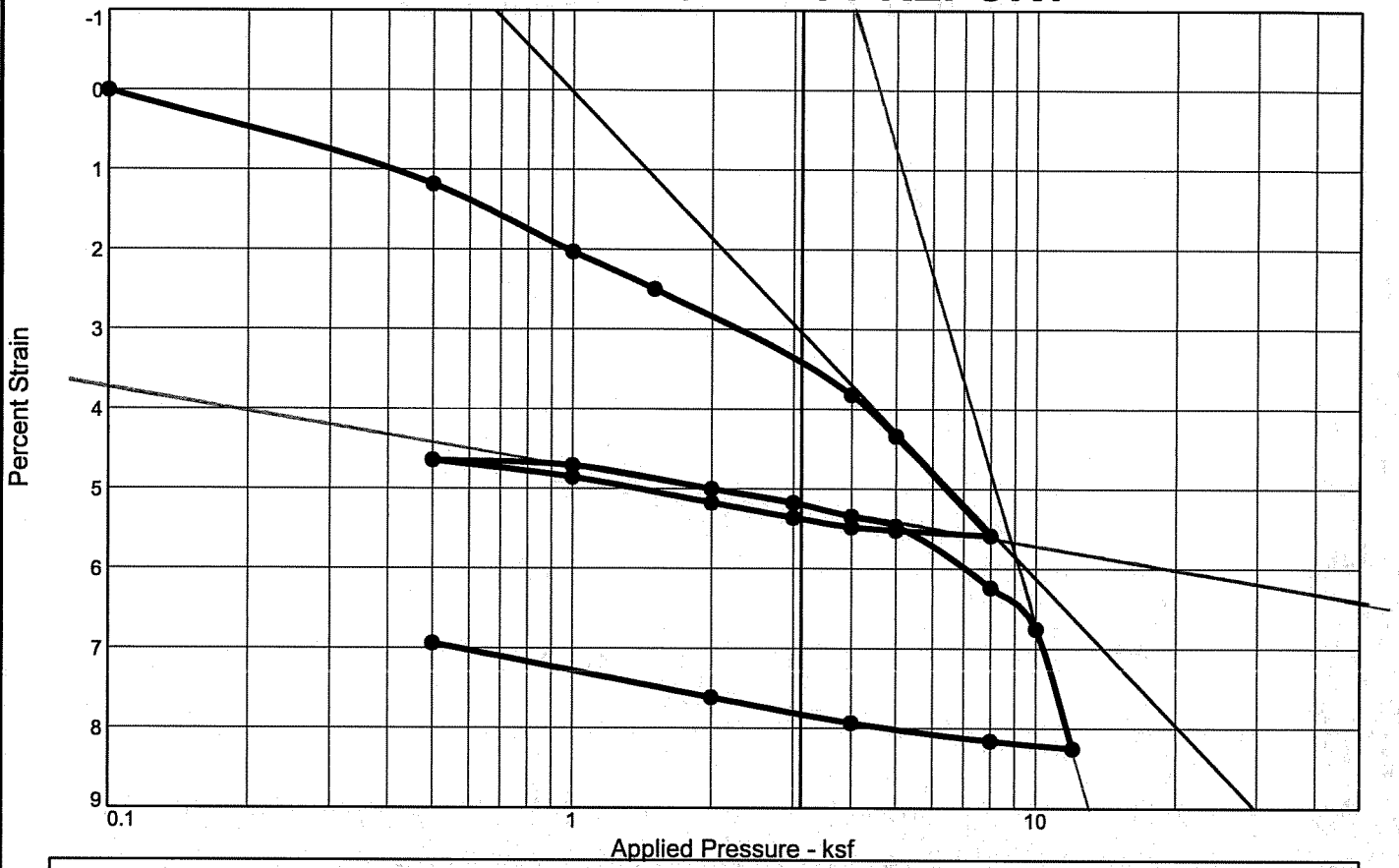


R.W. Gillespie & Associates, Inc.

Biddeford, Maine

Lab No. 14297a

CONSOLIDATION TEST REPORT



Coefficients of Consolidation and Secondary Consolidation

No.	Load (ksf)	C_v (ft.2/day)	C_α	No.	Load (ksf)	C_v (ft.2/day)	C_α	No.	Load (ksf)	C_v (ft.2/day)	C_α
2	0.50	0.363		12	1.00	1.643		19	8.00	0.411	
4	1.50	0.819		13	0.50	0.415		20	10.00	0.376	
5	4.00	0.400		14	1.00	1.664		21	12.00	0.204	
6	5.00	0.025		15	2.00	1.653		22	8.00	0.687	
7	8.00	0.364		16	3.00	1.649		23	4.00	0.571	
10	3.00	0.904		17	4.00	1.643		24	2.00	1.414	
11	2.00	0.025		18	5.00	0.405		25	0.50	0.406	

Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_r	Initial Void Ratio
Saturation	Moisture									
100.4 %	34.4 %	88.6	NV		2.76		3.7	0.12	0.04	0.945

MATERIAL DESCRIPTION	USCS	AASHTO
silty Clay		

Project No. 0600-102 **Client:** HNTB Corporation

Project: Scarborough Eastern Trail Project
Scarborough, ME

Location: EB-SNR-102 **Depth:** 35'-36' **Sample Number:** U-2

R.W. Gillespie & Associates, Inc.

Biddeford, Maine

Remarks:

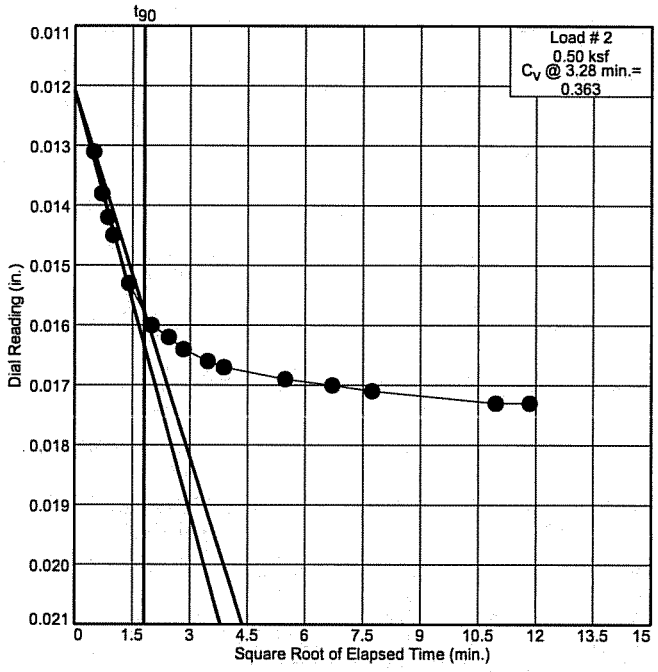
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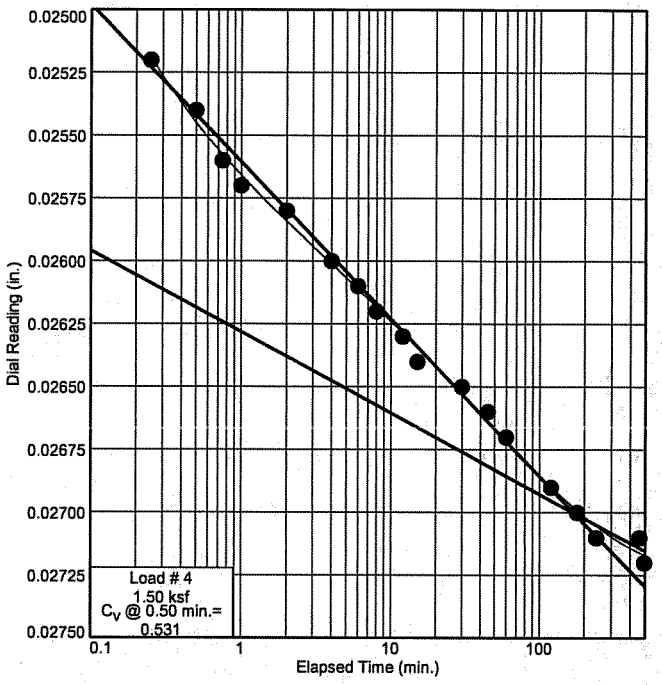
Tested By: _____ **Checked By:** _____

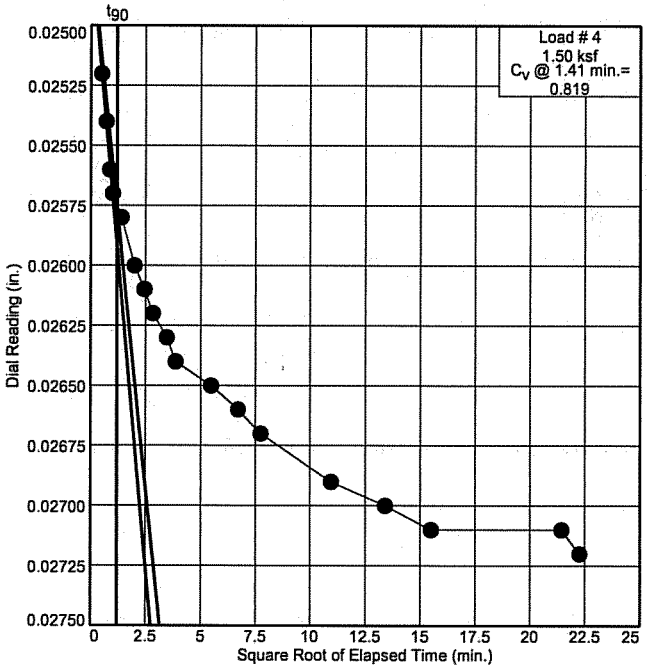
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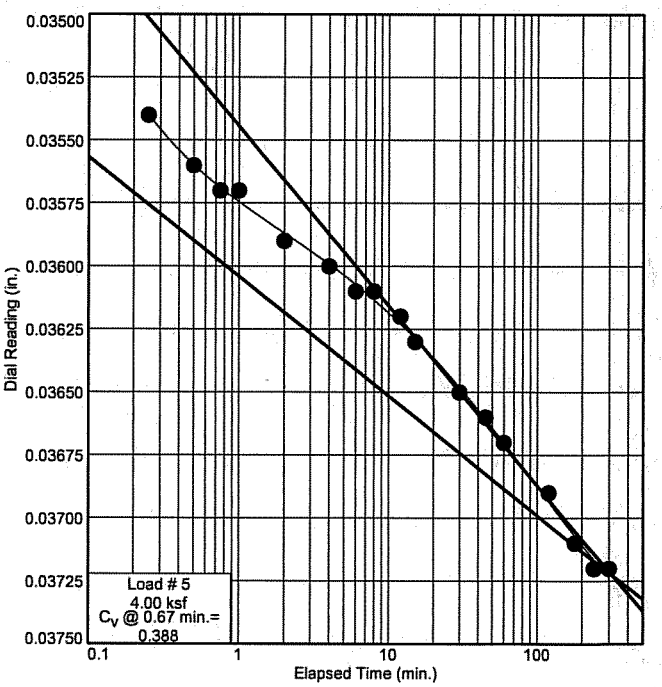
Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: EB-SNR-102 Depth: 35'-36' Sample Number: U-2









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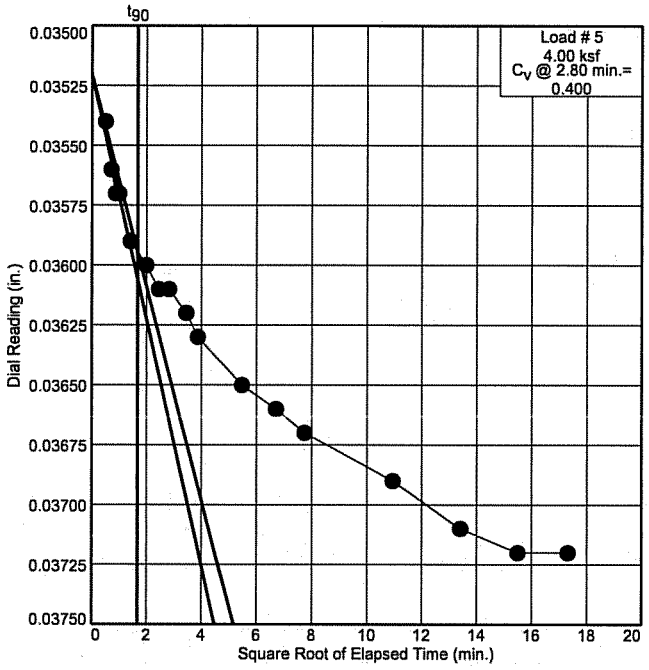
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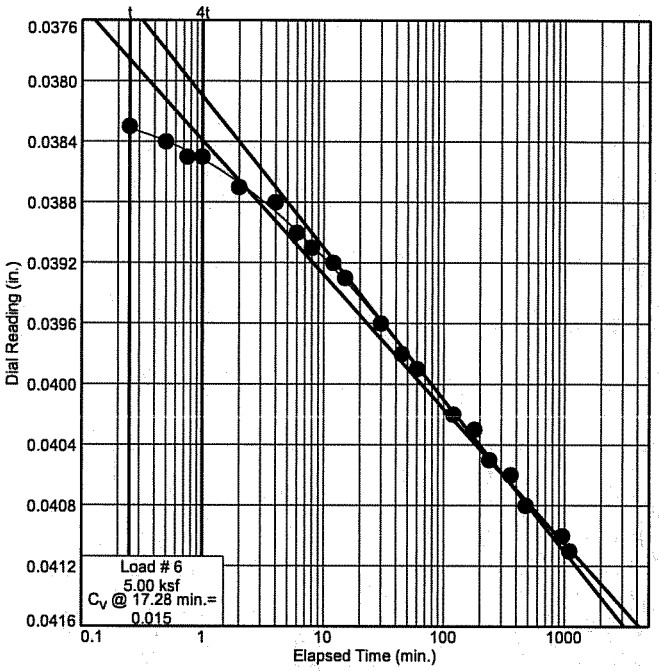
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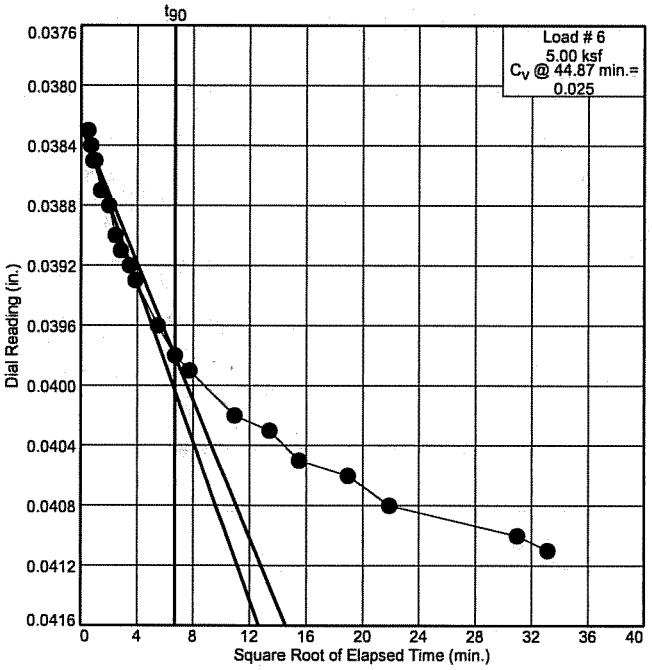
Dial Reading vs. Time

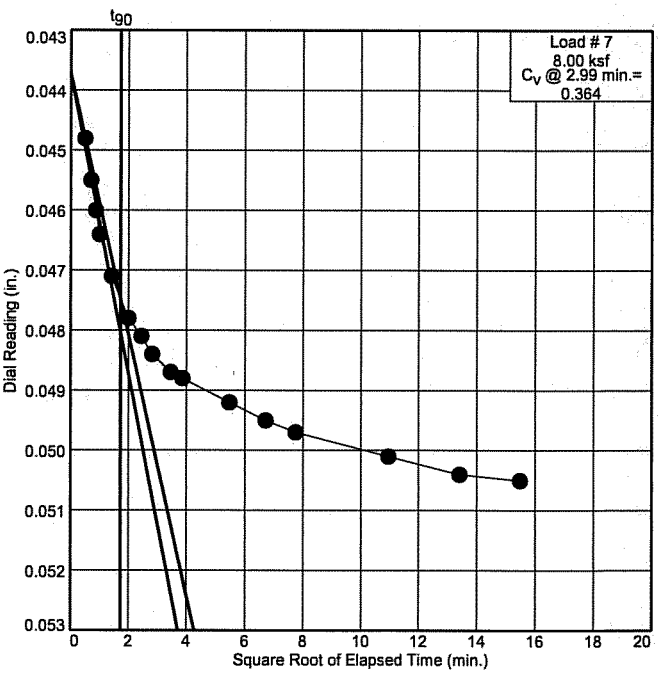
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Project: Scarborough Eastern Trail Project

Location: EB-SNR-102 Depth: 35'-36' Sample Number: U-2









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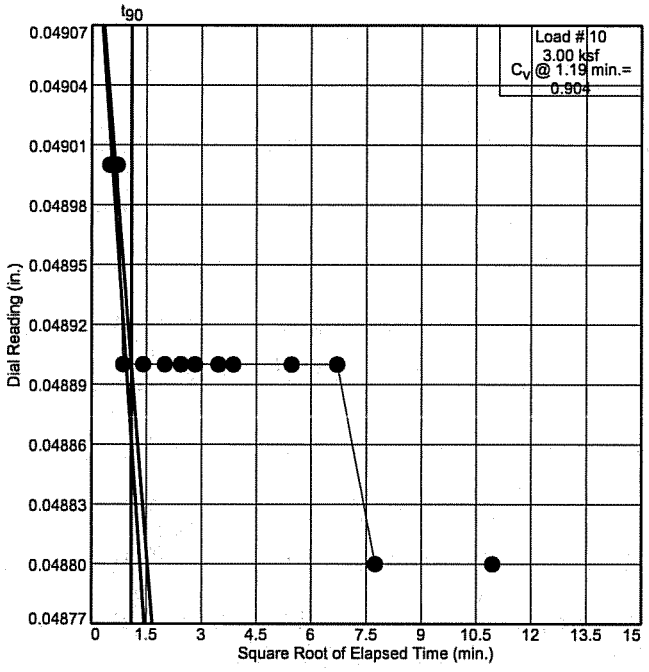
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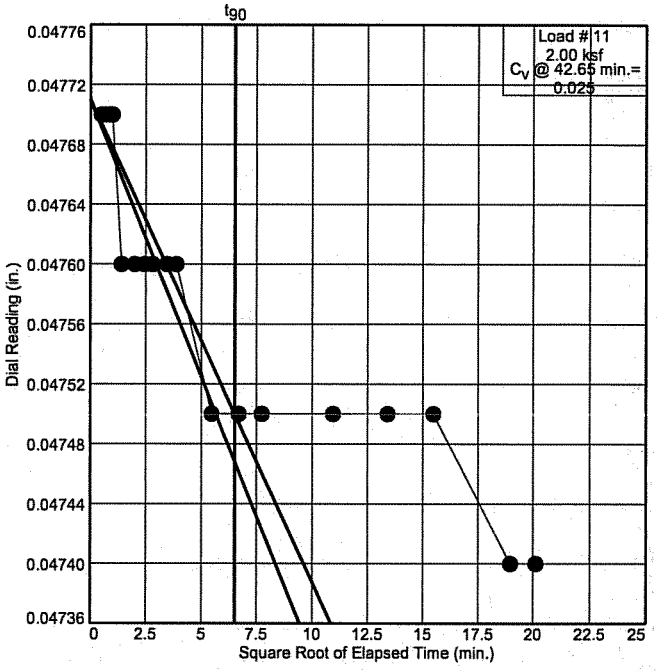
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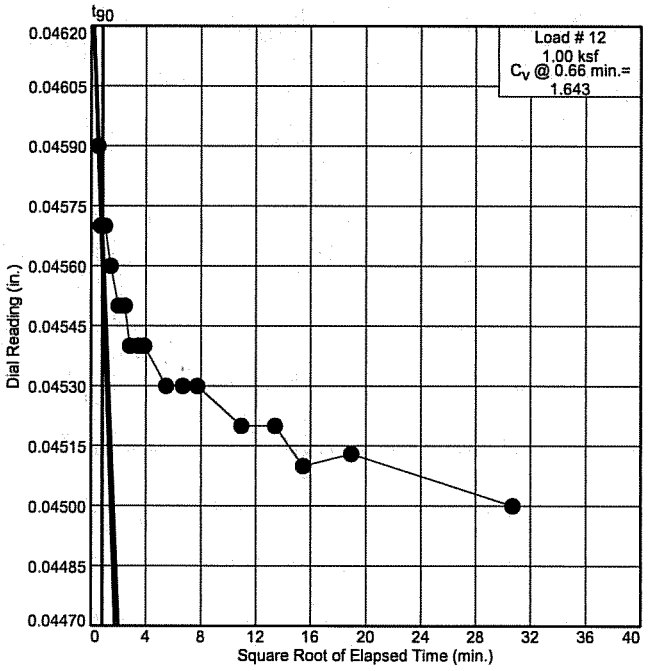
Dial Reading vs. Time

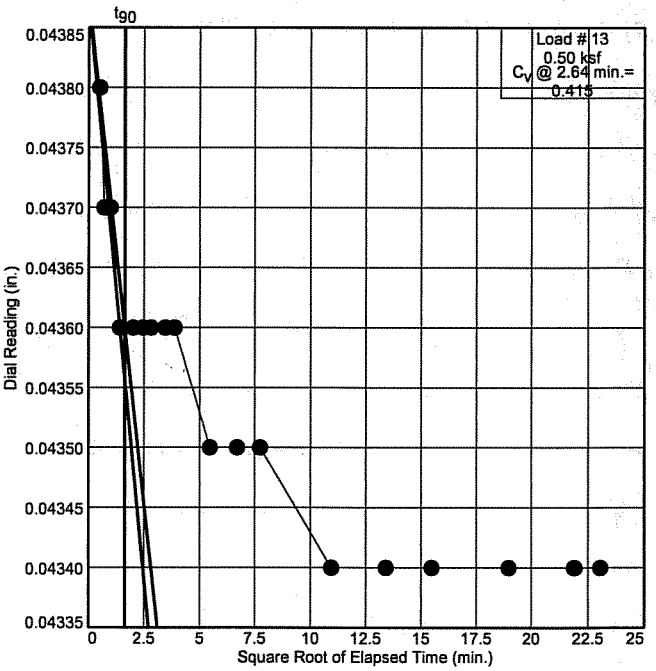
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Project: Scarborough Eastern Trail Project

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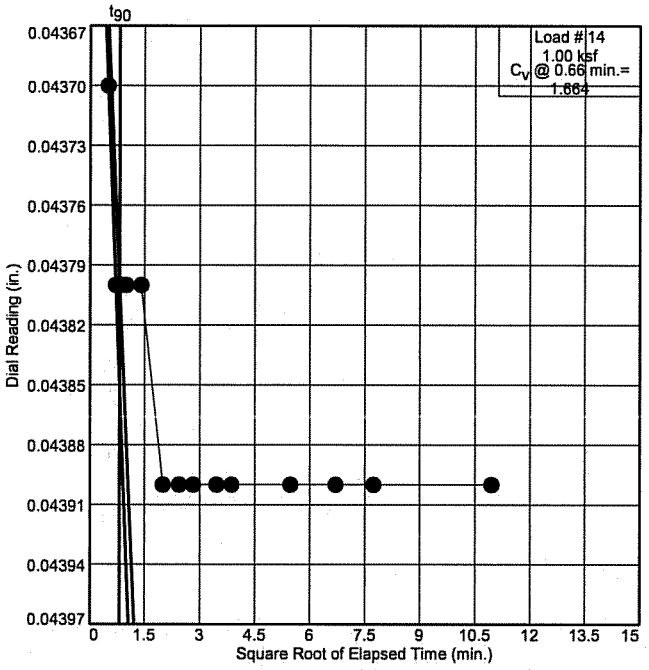
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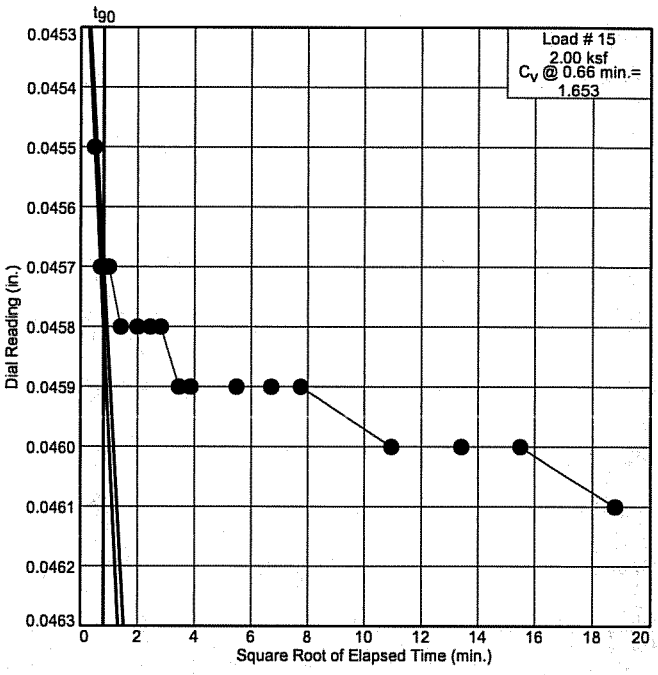
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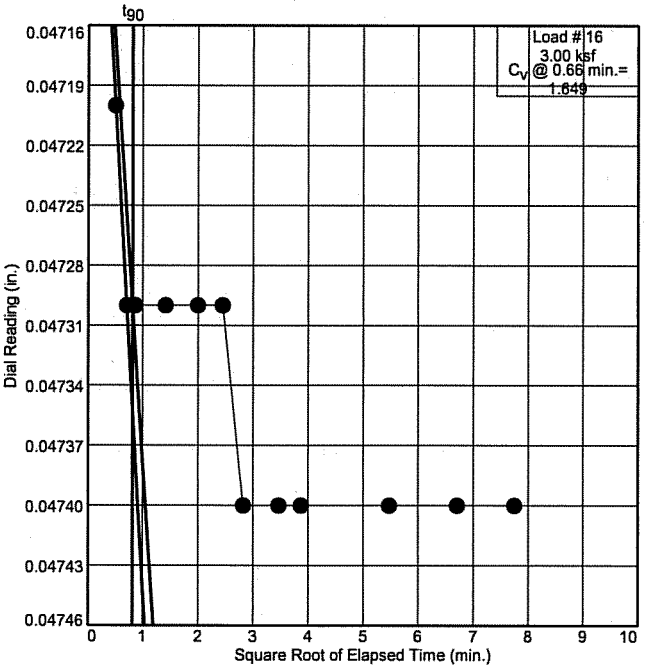
Dial Reading vs. Time

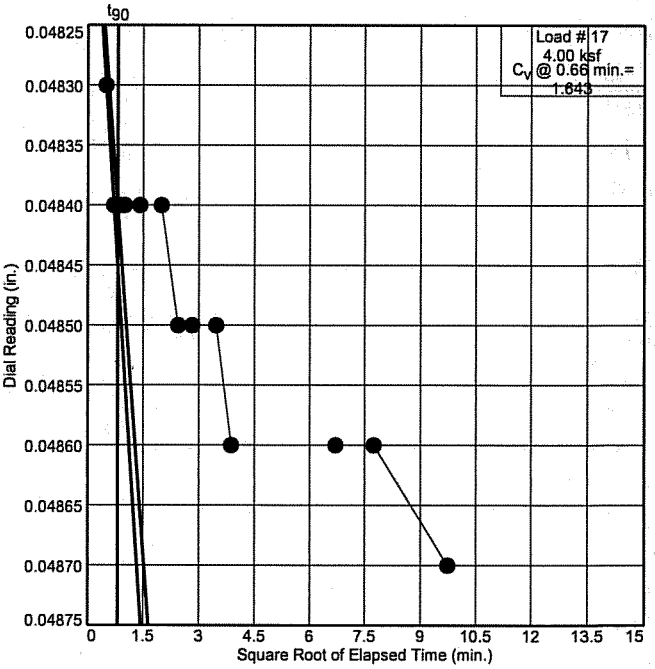
Project No.: 0600-102
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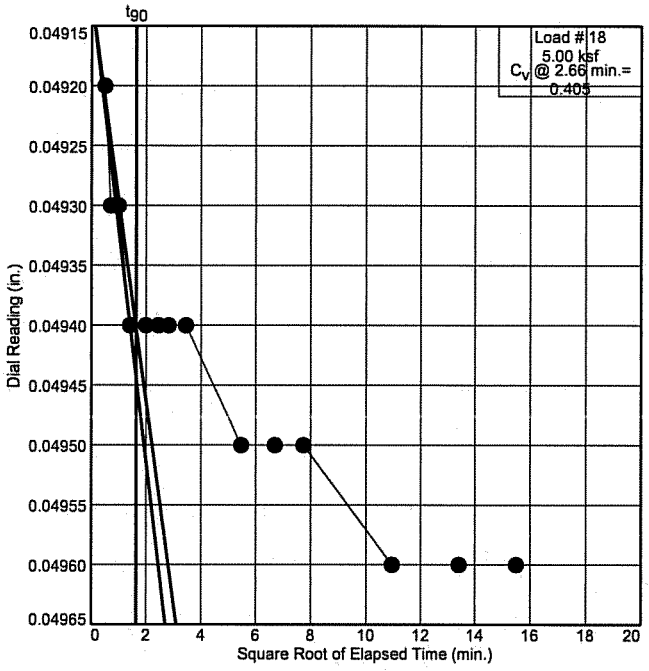
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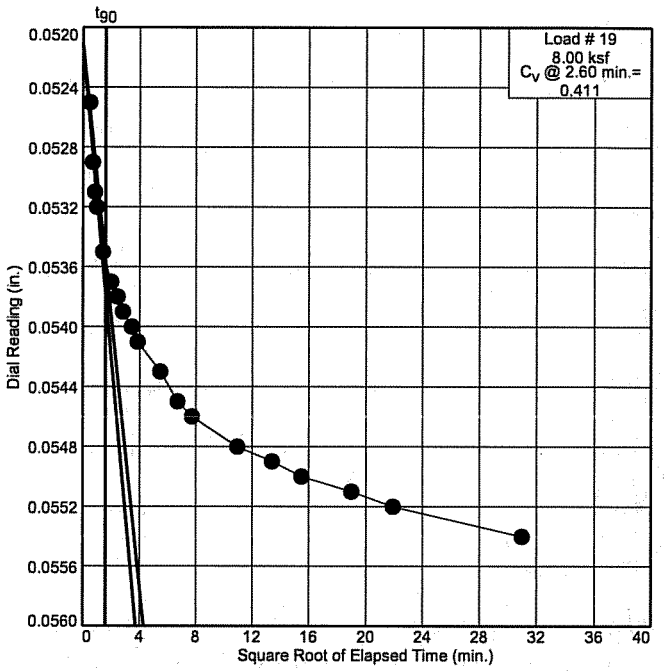
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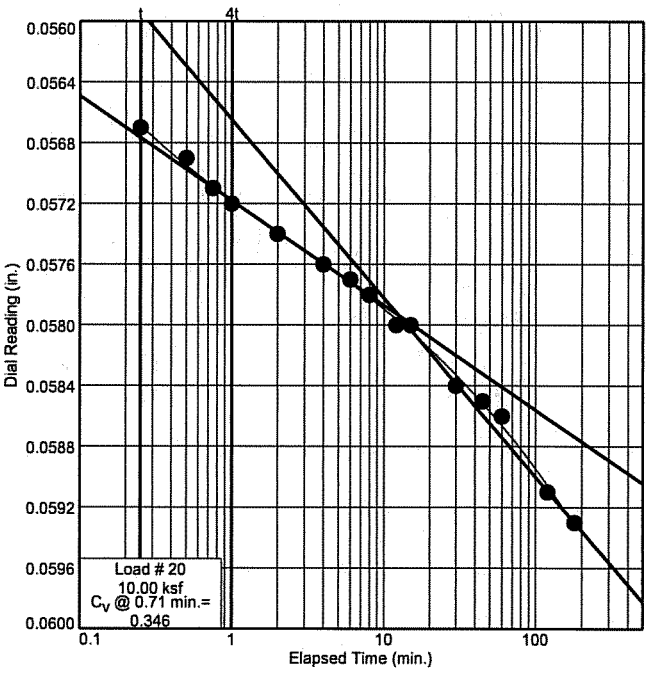
Dial Reading vs. Time

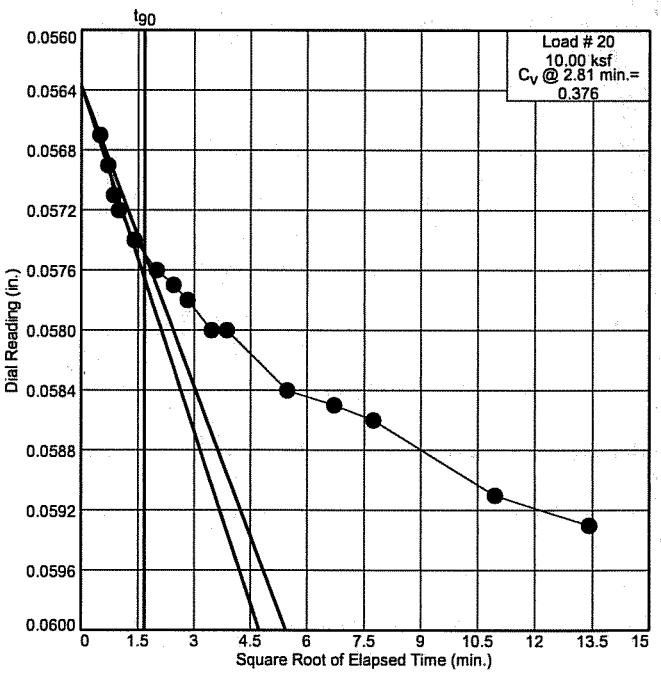
Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: EB-SNR-102 Depth: 35'-36' Sample Number: U-2









R.W. Gillespie & Associates, Inc.

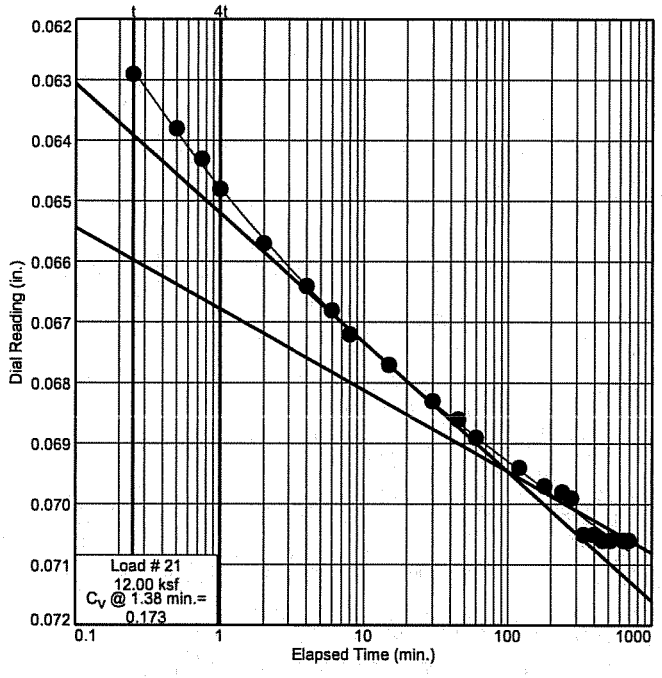
Biddeford, Maine

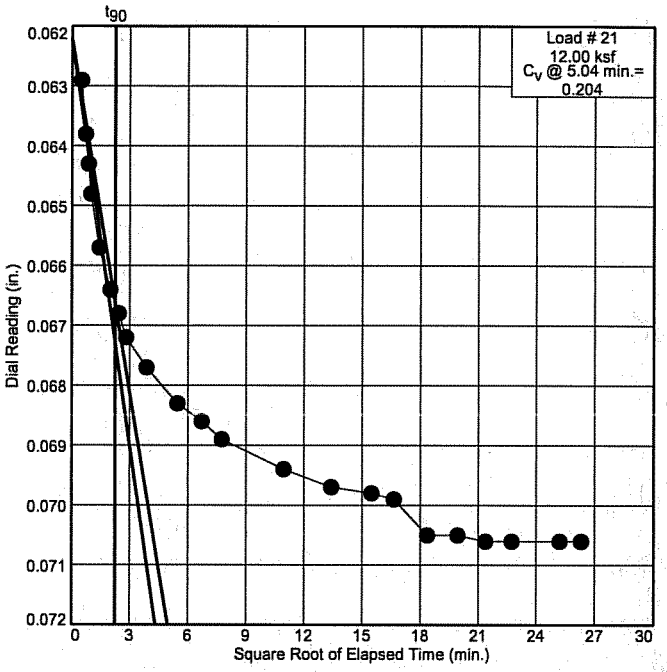
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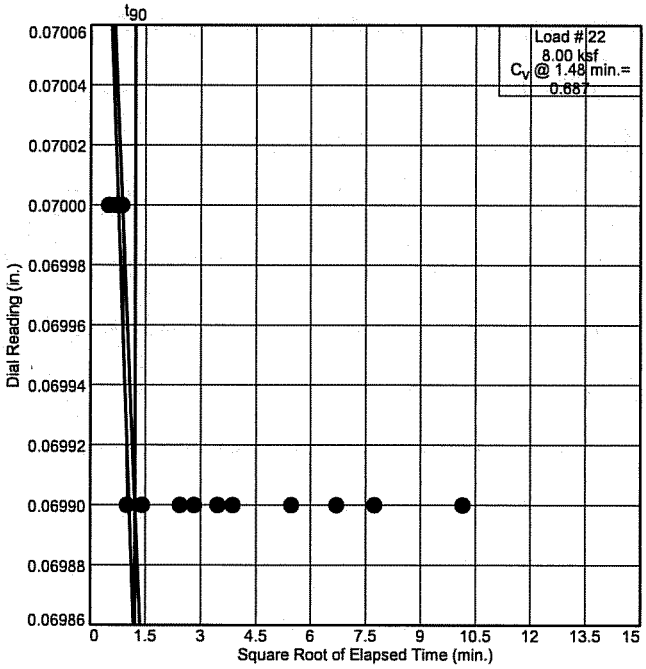
Dial Reading vs. Time

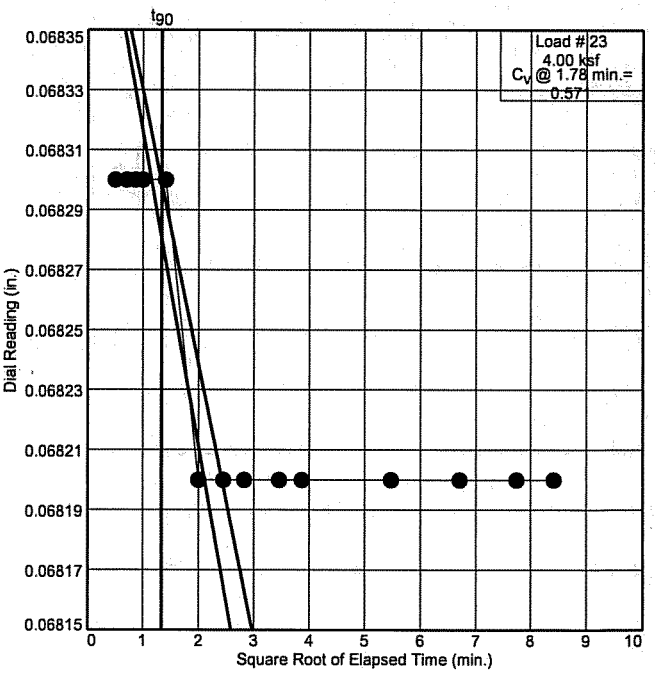
Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: EB-SNR-102 Depth: 35'-36' Sample Number: U-2









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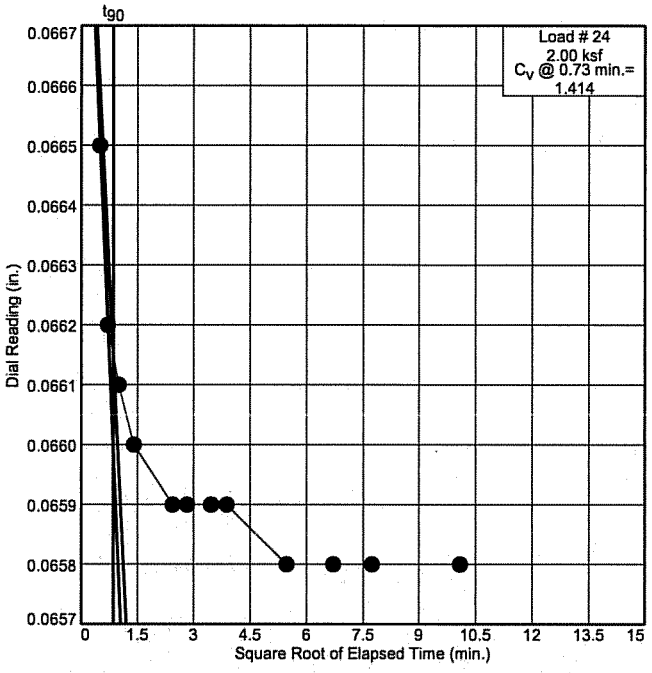
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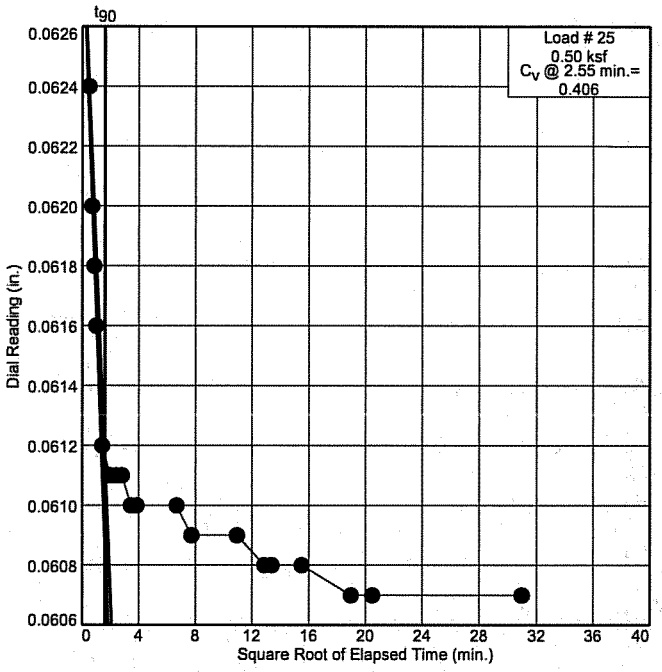
Lab No. 14291B

Dial Reading vs. Time

Project No.: 0600-102
Project: Scarborough Eastern Trail Project

Location: EB-SNR-102 Depth: 35'-36' Sample Number: U-2





APPENDIX D

GEOTECHNICAL ENGINEERING CALCULATIONS

Report of Geotechnical Evaluation
Scarborough Eastern Trail, MaineDOT Project No. STP-1942(610)
Scarborough, Maine

SITE CHARACTERISTICS – FROST AND SEISMIC

602-092
JMB

Reviewed EJW

--- ModBerg Results ---

Project Location: Sanford 2 NNW, Maine

Air Design Freezing Index = 1123 F-days
N-Factor = 0.80
Surface Design Freezing Index = 898 F-days
Mean Annual Temperature = 46.8 deg F
Design Length of Freezing Season = 116 days

Layer #:Type	t	w%	d	Cf	Cu	Kf	Ku	L
1-Coarse	60.2	10.0	125.0	28	34	2.0	1.6	1,800

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 = 5.01 ft = 60.2 in

Use 5 feet

600-092
JMB

Reviewed Esby

--- ModBerg Results ---

Project Location: Portland Wsfo Airport, Maine

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

Layer #:Type	t	w%	d	Cf	Cu	Kf	Ku	L
1-Coarse	63.7	10.0	125.0	28	34	2.0	1.6	1,800

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 = 5.31 ft = 63.7 in

Use 5 ft Esby

R.W. Gillespie & Associates
20 Pomerleau Street, Suite100 Biddeford, Maine 04005 (207) 286-8008
Fax: (207) 710-000 Email: rwg-a@rwg-a.com

NAME: ESW
DATE: AUG 2018
JOB NAME: Eastern Trail
JOB NUMBER: 0602-102
SHEET 1 OF 5

Seismic Site Class - West Abutment

Use RW Gillespie's seismic refraction shear-wave velocity test results on Presumpscot Formation deposits from Southern Maine Experience indicates Presumpscot Formation Deposits are Site Class D. See attached figures.

(A) Use boring BB-SPC-203/203A

Stratum	Thickness (ft)	$\bar{N}$ or $\bar{S}_u$	V_s (fps)
Sand w/ Silt	15	12	700
Silty Clay	10	820 psf	575
	10	500 psf	475
	8	415 psf	400
	7	535 psf	500
	15	600 psf	525
Silty Sand	35	24	850

$$\bar{V}_s = \frac{\sum d_i}{\sum \frac{d_i}{V_{s_i}}} = \frac{100}{0.16} = 625 \text{ fps} \longrightarrow \text{Site Class D}$$

R.W. Gillespie & Associates
20 Pomerleau Street, Suite100 Biddeford, Maine 04005 (207) 286-8008
Fax: (207) 710-000 Email: rwg-a@rwg-a.com

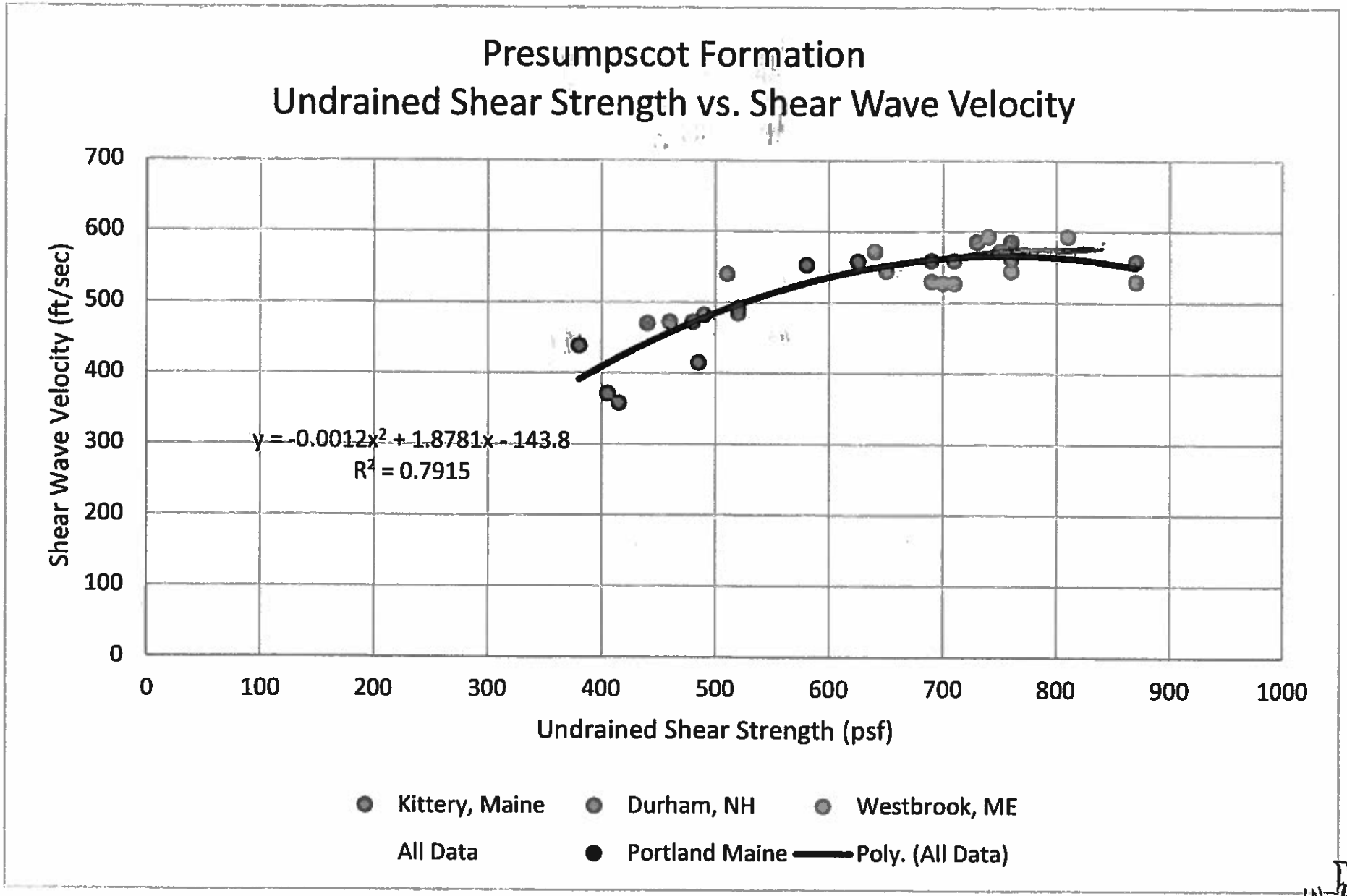
NAME: EJW
DATE: Aug 2018
JOB NAME: Eastern Trail
JOB NUMBER: 0600-102
SHEET 2 OF 5

(B) Use boring BB-SPC 201/201A

Stratum	Thickness	$\bar{N}$ or $\bar{S_u}$	Vs
Silty Sand	14	3	300
Silty Clay	10	650	550
	12	500	480
	16	490	480
Silty Sand	23	9	625
Silty Sand w Gravel	25	65	1000

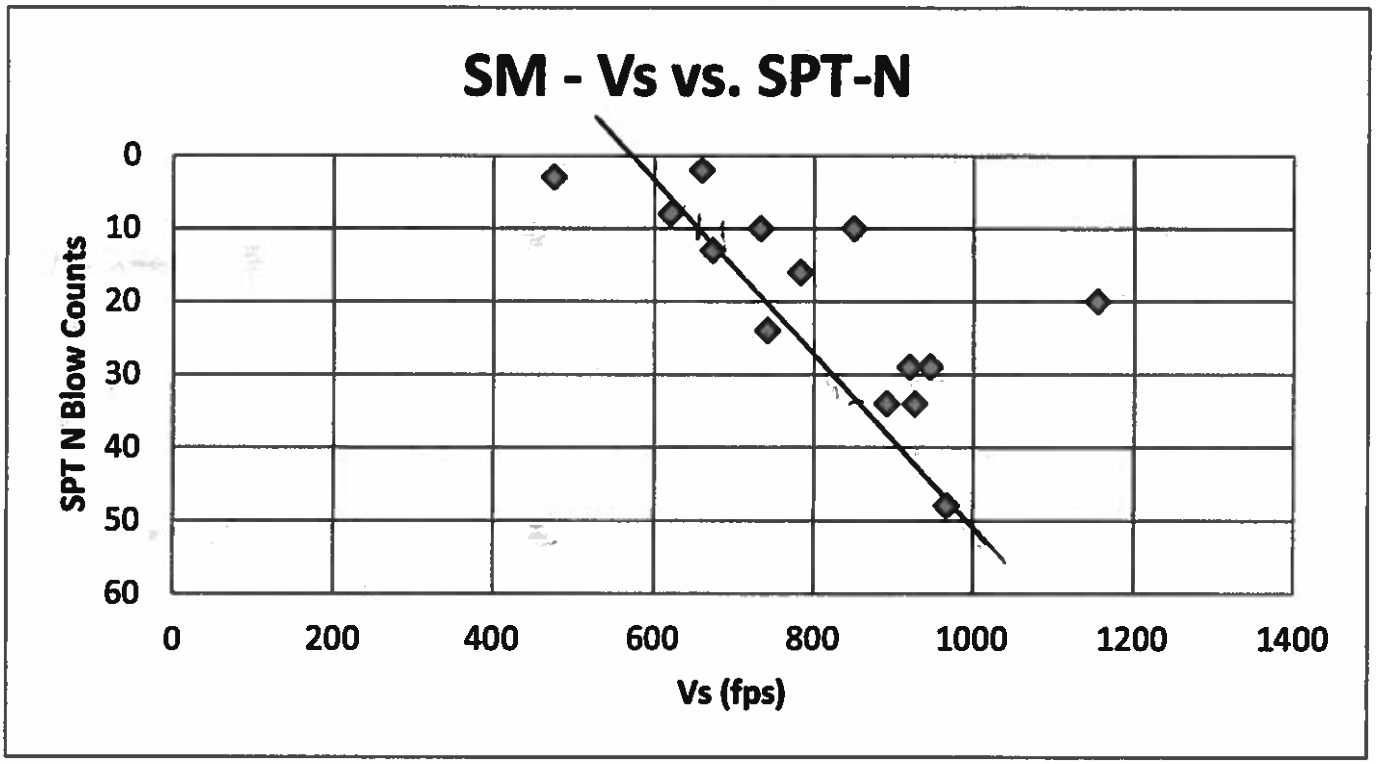
$$\bar{V_s} = \frac{\Sigma d_i}{\Sigma \frac{d_i}{V_s}} = \frac{100}{0.16} = 600 \text{ fps} \rightarrow \text{Site Class D}$$

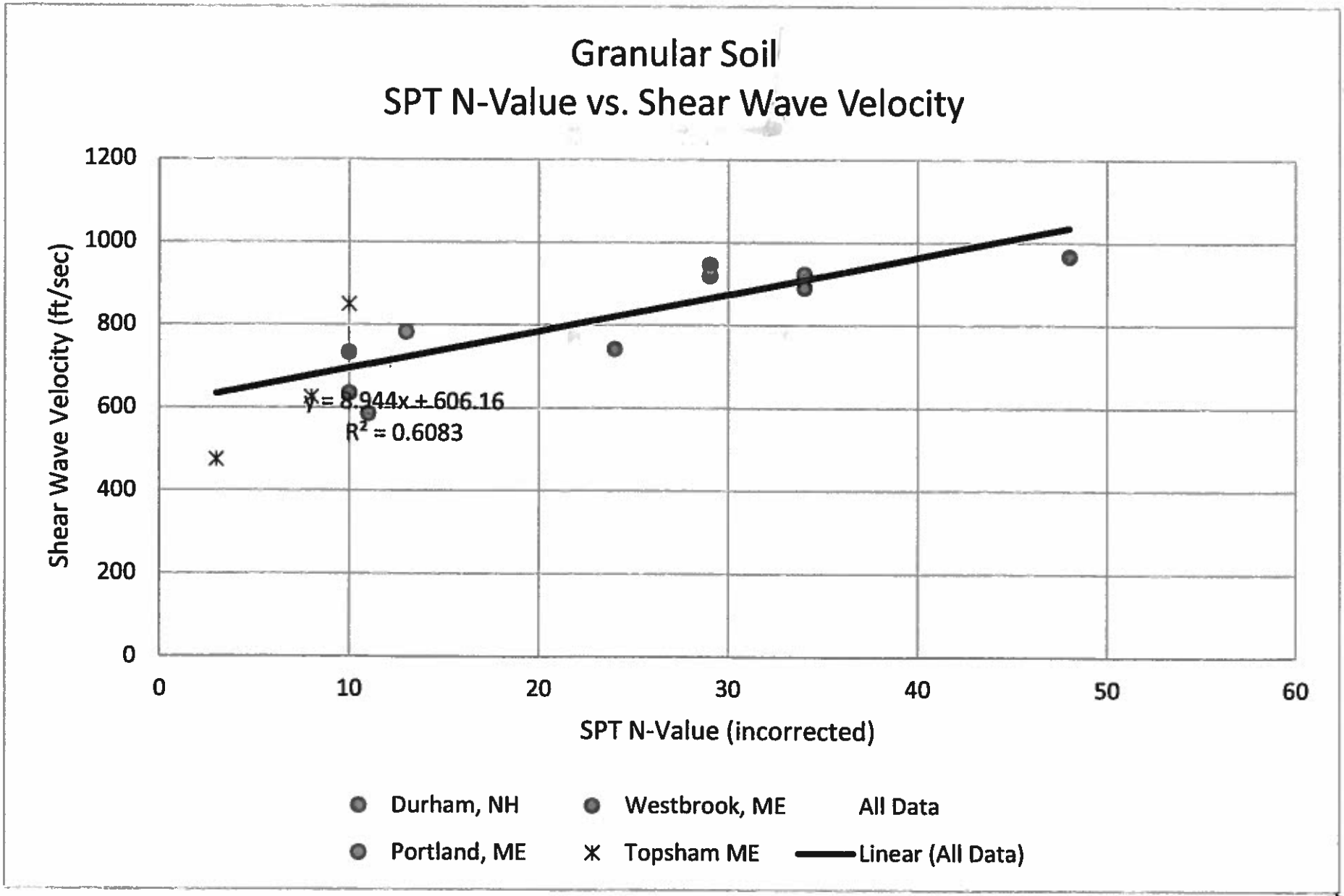
Site is borderline Site Class D and E



600-108
ESW
3/5

600-102
ESD
9/5





6000-02
E3-D
SK

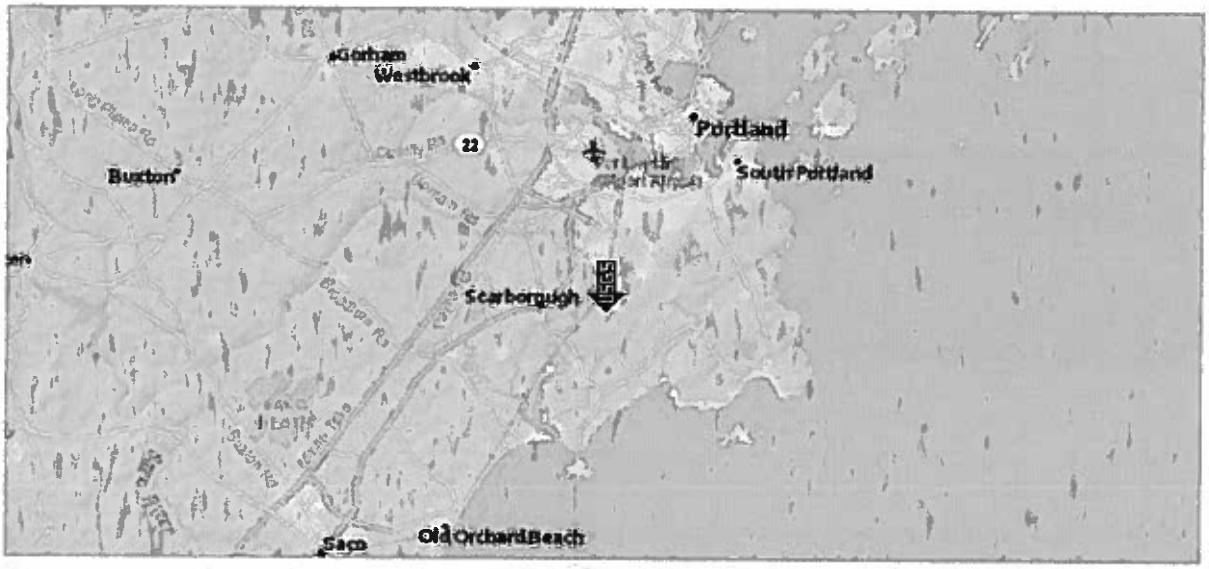
USGS Design Maps Summary Report

User-Specified Input

Building Code Reference Document 2009 AASHTO Guide Specifications for LRFD Seismic Bridge Design
(which utilizes USGS hazard data available in 2002)

Site Coordinates 43.598°N, 70.303°W

Site Soil Classification Site Class E – "Soft Clay Soil"

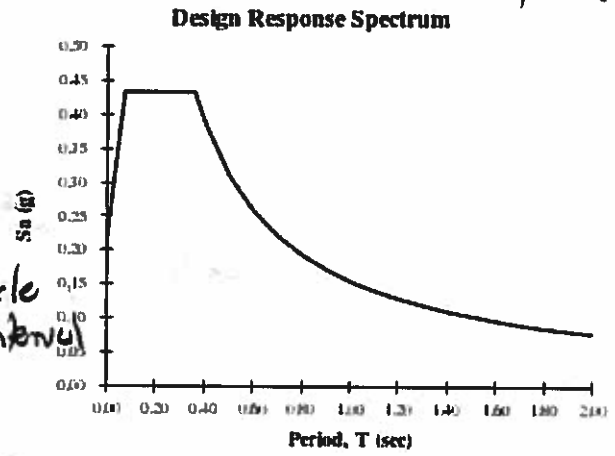


USGS-Provided Output

1600yr return interval. Concurs w/ mapping in AASHTO 2014

PGA =	0.087 g	A_s =	0.217 g
S_s =	0.173 g	S_{DS} =	0.433 g
S_1 =	0.044 g	S_{D1} =	0.155 g

vs. PGA = 0.05 from Maine Bridge Design Guide (500-yr return interval)



Although this information is a product of the U.S. Geological Survey, we provide no warranty, expressed or implied, as to the accuracy of the data contained therein. This tool is not a substitute for technical subject-matter knowledge.

0600-102

2018 UPDATE of MaineDOT Bridge Design Guide

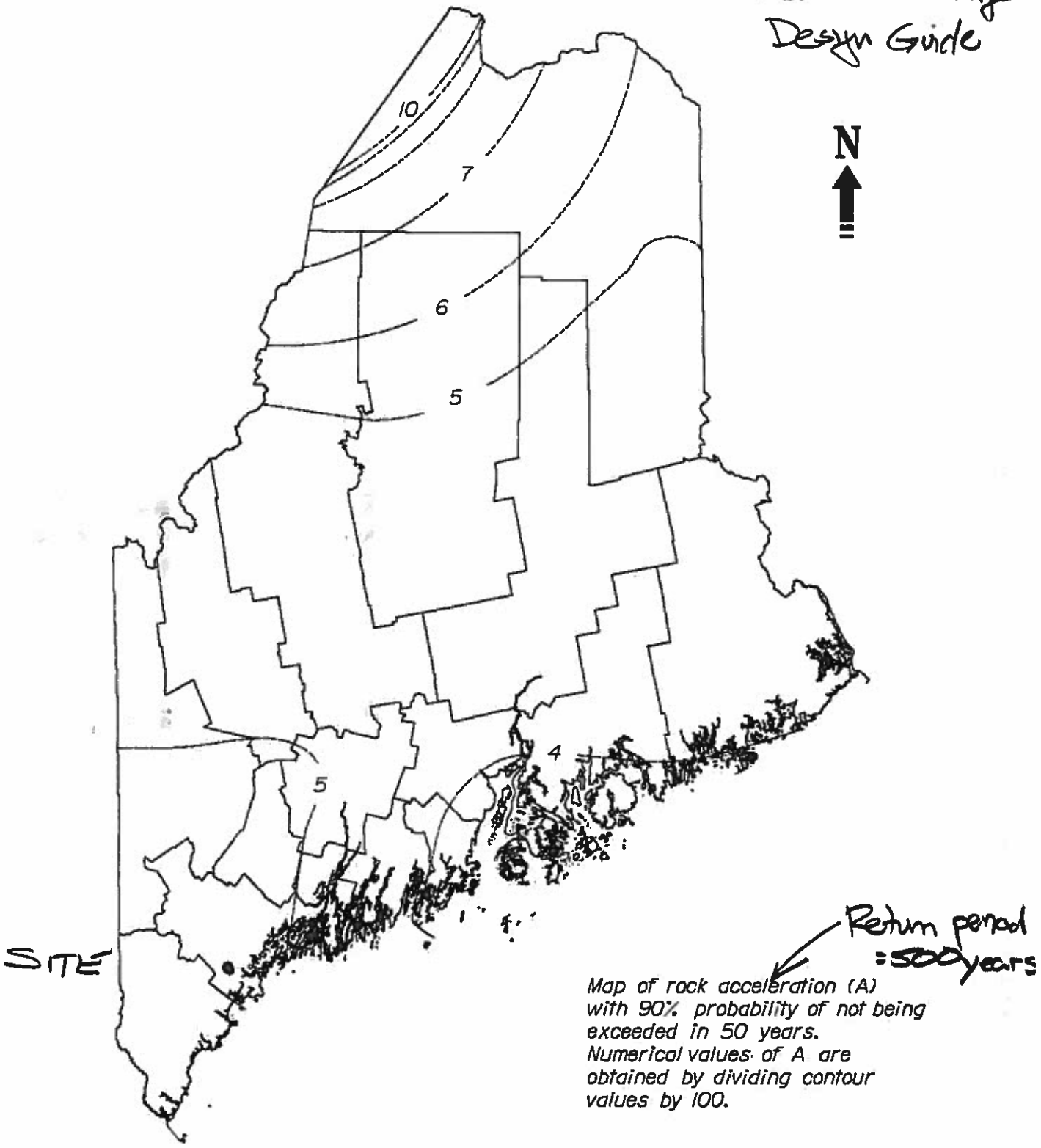


Figure 3-4 Seismic Performance Categories for Maine

Table 5-1. Values of F_a as a function of site class and mapped short-period spectral acceleration.

Site Class	Mapped Spectral Response Acceleration at Short Periods				
	$S_s \leq 0.25$ g	$S_s = 0.50$ g	$S_s = 0.75$ g	$S_s = 1.00$ g	$S_s \geq 1.25$ g
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	a	a	a	a	a

Table notes: Use straight line interpolation for intermediate values of S_s, where S_s is the spectral acceleration at 0.2 second obtained from the ground motion maps.

a Site-specific geotechnical investigation and dynamic site response analyses shall be performed (Article 3.4.3).

design approach also multiplies the resulting spectrum by a 2/3rd factor to account for the "reserve capacity" against collapse within most buildings.) The AASHTO procedure also involves anchoring the design spectrum at zero period (PGA) based on a 1,000-year return period hazard level. This approach compares to the IBC which assumes that the PGA is equal to 0.4 times the spectral acceleration at 0.2 seconds (that is, the short period spectral acceleration, S_s). The site coefficient used by AASHTO to adjust the PGA value (F_{pga}) for various soil classifications is identical to the coefficient used for the 0.2-second, short period site factor (F_a) recommended by the NCHRP 12-49 Project and used by IBC.

5. Similar to NCHRP 12-49 and IBC 2006, the NCHRP 20-07 document provided two tables for site modification factors to be applied to the two spectral ordinates for other site soil/rock categories. Table 5-1 tabulates site coefficients (F_a) at the short period range (that is, at 0.0-second and 0.2-second periods), and Table 5-2 tabulates site coefficients (F_v) at the 1-second period. (AASHTO subsequently adopted a separate table for F_{pga} to be applied to PGA. Values of F_{pga} are the same as F_a. Note also that AASHTO normalizes PGA to be dimensionless. The current version of AASHTO shows the same F_a and F_v values but without the units of gravitational acceleration (g).) The two site coefficient factors are applied to the three spectral ordinates from the new AASHTO 1,000-year maps and implementation CD for various site categories in relation to the reference USGS Site Class B condition.

Table 5-2. Values of F_v as a function of site class and mapped 1 second period spectral acceleration.

Site Class	Mapped Spectral Response Acceleration at 1 Second Periods				
	$S_1 \leq 0.1$ g	$S_1 = 0.2$ g	$S_1 = 0.3$ g	$S_1 = 0.4$ g	$S_1 \geq 0.5$ g
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	a	a	a	a	a

Table notes: Use straight line interpolation for intermediate values of S_1, where S_1 is the spectral acceleration at 1.0 second obtained from the ground motion maps.

a Site-specific geotechnical investigation and dynamic site response analyses shall be performed (Article 3.4.3).

NCHRP
REPORT 611

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Seismic Analysis and Design of Retaining Walls, Buried Structures, Slopes, and Embankments

TRANSPORTATION RESEARCH BOARD
OF THE NATIONAL ACADEMIES

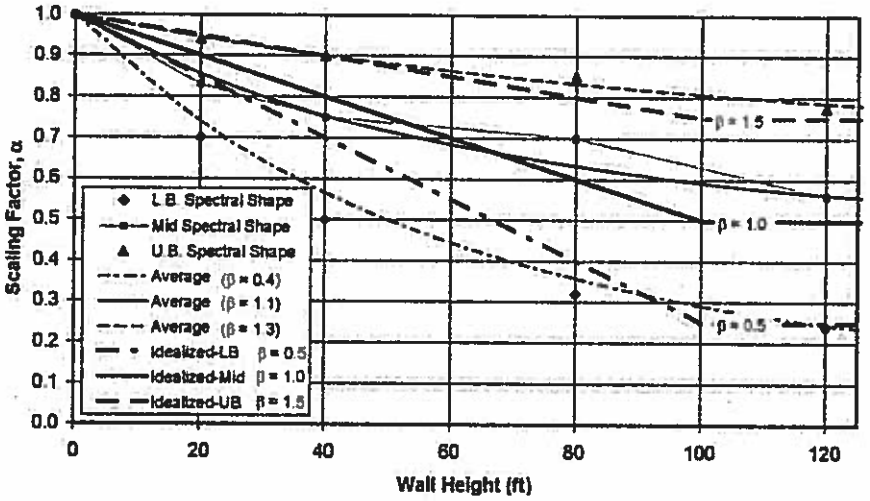


Figure 7-16. Simplified height-dependent scaling factor recommended for design.

Recommendations for seismic coefficients to be used for earth pressure evaluations based on the simplified straight line functions shown can be expressed by the following equations:

$$k_{av} = \alpha\, k_{max} \qquad (7\text{-}1)$$

where

k_{max} = peak seismic coefficient at the ground surface = F_{pga} PGA; and

α = fill height-dependent reduction factor.

For C, D, and E foundations soils

$$\alpha = 1 + 0.01H[(0.5\beta) - 1] \qquad (7\text{-}2)$$

where

H = fill height in feet; and

$\beta = F_v S_1 / k_{max}$.

For Site Class A and B foundation conditions (that is, hard and soft rock conditions) the above values of α should be increased by 20 percent. For wall heights greater than 100 feet, α coefficients may be assumed to be the 100-foot value. Note also for practical purposes, walls less than say 20 feet in height and on very firm ground conditions (B/C foundations), $k_{av} = k_{max}$ which has been the traditional assumption for design.

7.6 Displacement-Based Design for Gravity, Semi Gravity, and MSE Walls

The concept of allowing walls to slide during earthquake loading and displacement-based design (that is, assuming a Newmark sliding block analysis to compute displacements when accelerations exceed the horizontal limiting equilibrium yield acceleration) was introduced by Richards and Elms (1979). Based on this concept (as illustrated in Figure 7-17), Elms and Martin (1979) suggested that a design acceleration coefficient of 0.5A in M-O analyses would be adequate for limit equilibrium pseudo-static design, provided allowance be made for a horizontal wall displacement of 10A (in inches). The design acceleration coefficient (A) is the peak ground acceleration at the base of the sliding wedge behind the wall in gravitational units (that is, g). This concept was adopted by AASHTO in 1992, and is reflected in following paragraph taken from Article 11.6.5 of the 2007 AASHTO *LRFD Bridge Design Specifications*.

> Where all of the following conditions are met, seismic lateral loads may be reduced as provided in Article C11.6.5, as a result of lateral wall movement due to sliding, from values determined

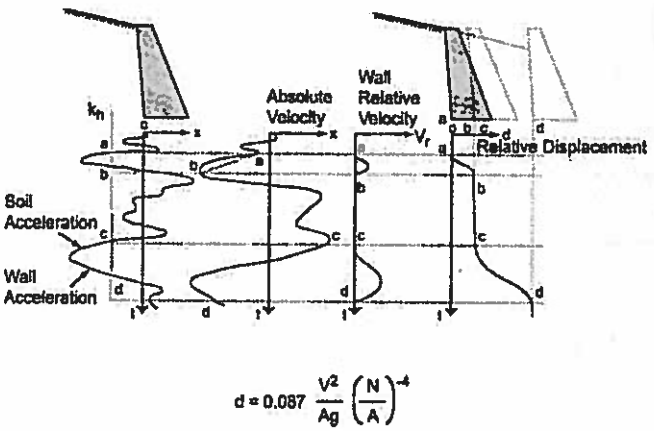


Figure 7-17. Concept of Newmark sliding block analysis (AASHTO, 2007).

based approach is that the amount of movement associated with the analysis is estimated, and sometimes this can be an important consideration. Note that both approaches assume that liquefaction or porewater pressure effects are not a consideration. Section 8.5.3 provides comments on the potential treatment of liquefaction.

8.3.1 Limit Equilibrium Approach

The limit equilibrium approach involves the following steps:

1. Conduct static slope stability analyses using appropriate resistance factors to confirm that performance meets static loading requirements. Typically these will be a C/D ratio of 1.3 to 1.5 for natural slopes and 1.5 for engineered slopes. A variety of factors should be considered when selecting the C/D ratio including the quality of the site characterization and the implications of failure. Both short-term, undrained stability, and long-term drained stability should normally be considered in this evaluation.
2. Establish the site peak ground acceleration coefficient k_{max} and spectral acceleration at one second, S_1 from the new AASHTO ground motions maps for a 1,000-year return period, including appropriate site soil modification factors.
3. Determine the corresponding PGV from correlation equations between S_1 and PGV (provided in Chapter 5).
4. Modify k_{max} to account for slope height effects for full slope or embankment height stability analyses (note that α factors described in Chapter 7 for retaining walls appear compatible with those for slopes based on comparison with analysis methods described above).
5. Reduce the resulting k_{max} by a factor of 0.5, as long as 1 to 2 inches of permanent displacement are permissible. If larger amounts of deformation are acceptable, further reductions in k_{max} are possible, but these would have to be determined by conducting separate calibration studies between displacement and the ratio of the yield acceleration (k_y) and k_{max}.
6. Conduct a conventional slope stability analysis using 0.5 k_{max}. If the factor of safety is at least 1.1, the slope meets seismic loading requirements.

8.3.2 Displacement-Based Approach

The following displacement-based methodology is recommended for slopes and embankments, where the static strength parameters can reasonably be assumed for seismic analyses:

1. Conduct static slope stability analyses using appropriate resistance factors to confirm that performance meets static loading requirements.
2. Establish the site peak ground acceleration coefficient k_{max} and spectral acceleration at one second, S_1 from the new AASHTO ground motion maps for a 1,000-year return period, including appropriate site soil modification factors.
3. Determine the corresponding PGV from correlation equations between S_1 and PGV (provided in Chapter 5).
4. Modify k_{max} to account for slope height effects for full slope or embankment height stability analyses (note that α factors described in Chapter 7 for retaining walls appear compatible with those for slopes based on comparison with analysis methods described above).
5. Determine the yield acceleration (k_y) using a pseudo-static stability analysis for the slope (that is, the seismic coefficient corresponding to a factor of safety equal to 1.0). Note that these stability analyses should normally be conducted using the undrained strength of the soil because of the short-term loading from the earthquake.
6. Establish the earthquake slope displacement potential corresponding to the value of k_y/k_{max} using the Newmark displacement chart recommendations given in Chapter 5.
7. Evaluate the acceptability of the displacement based on performance criteria established by the owner for the specific project site.

8.4 Example Application

The proposed displacement-based methodology is illustrated by considering an existing slope located in the State of Washington. This slope is next to a heavily traveled roadway. The roadway is being widened to accommodate projected increases in traffic. Stability analyses were required to determine the potential effects of seismic loading to the slopes located above and below the roadway.

8.4.1 Problem Description

Seismic stability of the natural slopes was evaluated for the following conditions:

- Slope angles ranging from 2H:1V up to 1H:1V.
- Soils comprised of glacial till and fill. Till is a dense silty sand with gravel. Standard penetration test (SPT) blowcounts range from 30 blows per foot to refusal. Soil strength values were interpreted from SPT blowcounts. (See Appendix J for sections and assigned properties).
- Groundwater located at the base of the slope.
- The firm-ground values of PGA, S_s, and S_1 for site are estimated to be 0.41g, 0.92g, and 0.30g, respectively, for the 1,000-year earthquake based on the USGS deaggregation website. (Note that at the time the example was developed, the new AASHTO ground motion hazard maps and implementation CD were not available to the NCHRP 12-70

SEISMIC COEFFICIENTS FOR GLOBAL STABILITY ANALYSIS

$PGA = 0.087_g$ (2013 AASHTO) PGA=0.05 MDBG

$S_1 = 0.044_g$ (2013 AASHTO), $S_s = 0.173$ (2013 AASHTO)

SITE CLASS E "SOFT CLAY SOIL"

$F_v = 3.5$ (Table 5-2) — Site Class D: 2.4

$F_{pga} = 2.5$ (Table 5-1) — 1.6

As - $k_{max} = F_{pga} PGA$

$k_{max} = 2.5 \times 0.087_g = 0.218_g$ — 0.14

$SD_1 = 3.5(0.044) = 0.154$ (Zone 1 per Table 3.10.6-1) — $S_{D1} = 2.4(0.044) = 0.11$

$\beta = F_v S_1 / k_{max}$

$\beta = 3.5 \times 0.044_g / 0.218_g = 0.708$ → Site Class D: $\beta = 0.75$

$\alpha = 1 + 0.01H[(6.5\beta)-1]$

$\alpha = 1 + 0.01(16')[0.5(0.708)-1]$ — FOR BIN WALL

$\alpha = 0.897$

Site Class D: $\alpha = 0.9$

$k_s = 0.5 \alpha k_{max}$ — 0.5(0.9)0.14

$k_s = 0.5 \times 0.897 \times 0.218$

$k_s = 0.0975 \approx 0.1$ — FOR Wall — $K_s = 0.063$

$\alpha = 1 + 0.01(22.5')[(0.5(0.708)-1]$

$\alpha = 0.855$ — FOR EMBANKMENT — $\alpha = 0.86$

$K_s = 0.5 \times 0.855 \times 0.218 = 0.093$ — $k_s = 0.06$

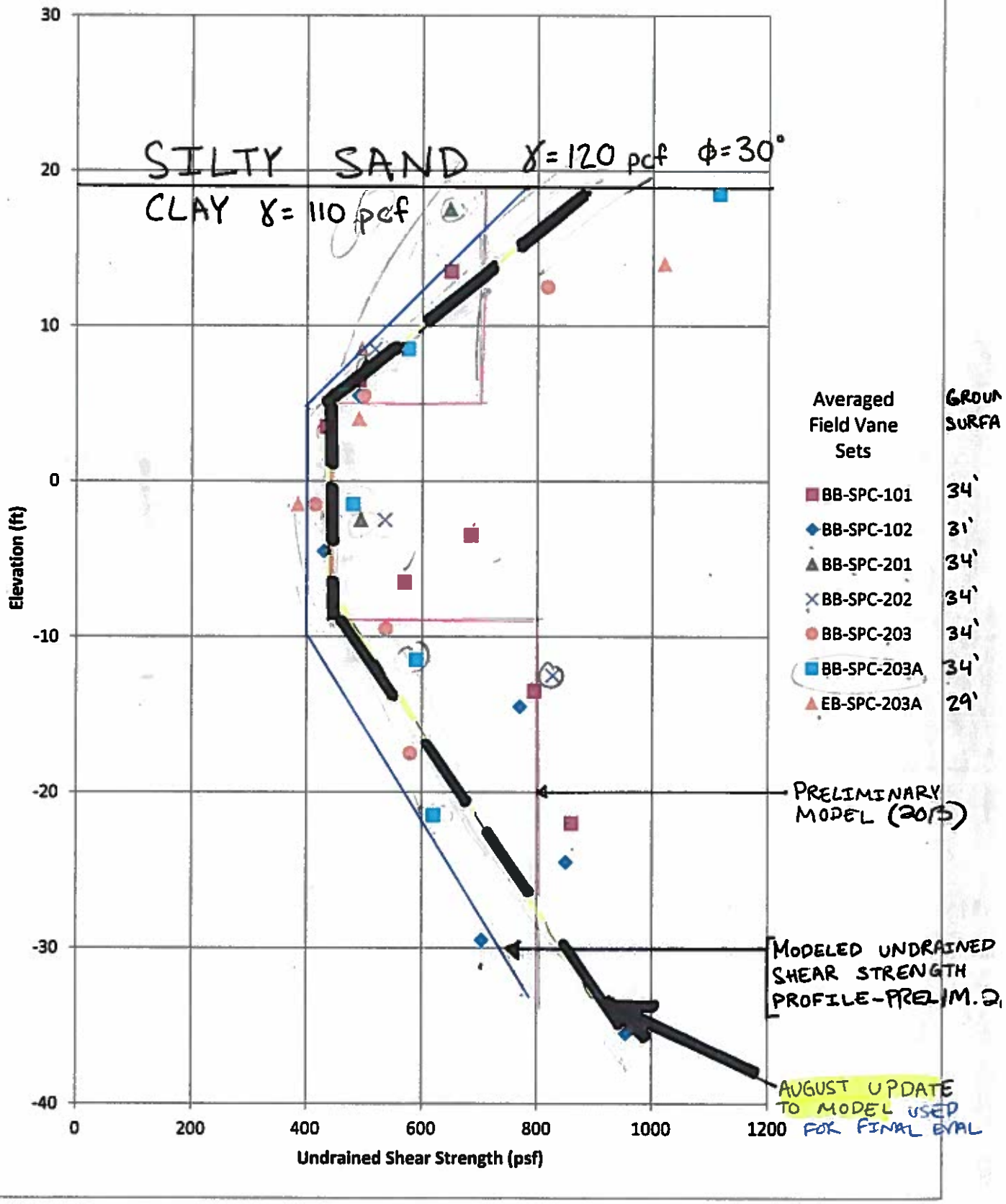
$0.5\ k_{max} = 0.5(.218) = 0.109$ — = 0.07

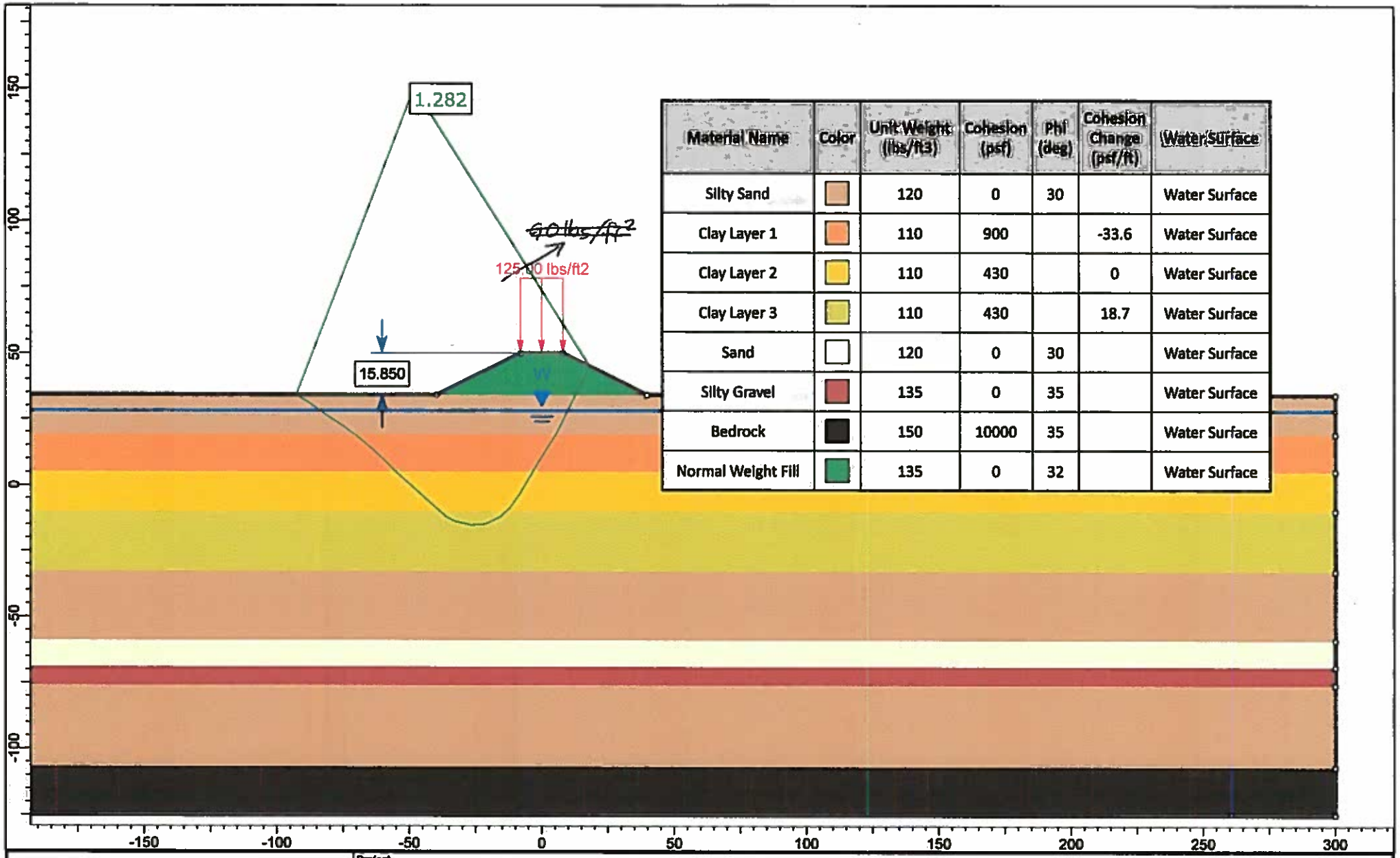
EMBANKMENTS – STABILITY EVALUATION

600-102
25 JULY 201
SNF

Pan Am Crossing Field Vanes - Eastern Trail, Scarborough, Maine

UPDATED AUGUST 2



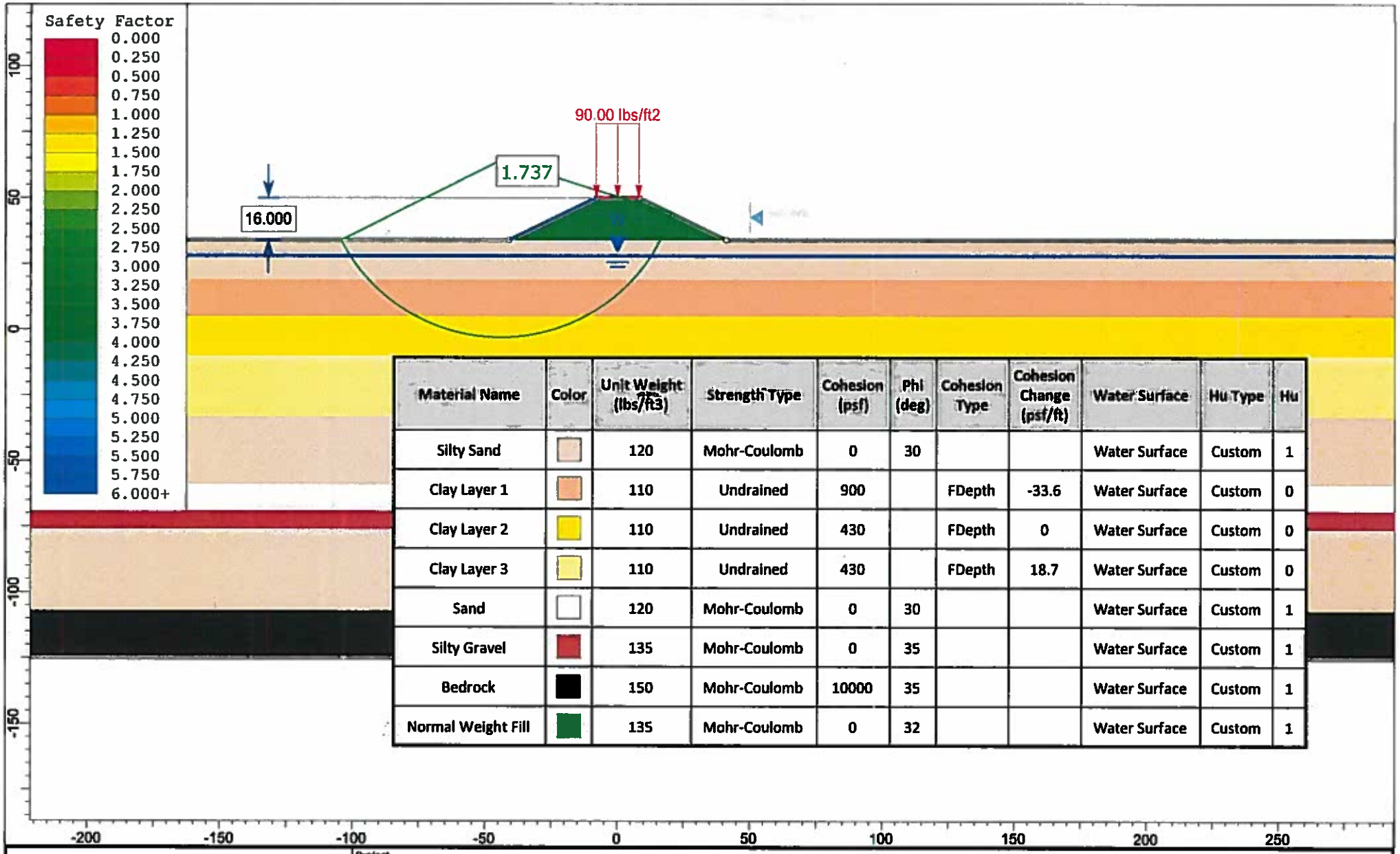


Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)	Cohesion Change (psf/ft)	Water Surface
Silty Sand		120	0	30		Water Surface
Clay Layer 1		110	900		-33.6	Water Surface
Clay Layer 2		110	430		0	Water Surface
Clay Layer 3		110	430		18.7	Water Surface
Sand		120	0	30		Water Surface
Silty Gravel		135	0	35		Water Surface
Bedrock		150	10000	35		Water Surface
Normal Weight Fill		135	0	32		Water Surface

rocscience

SLIDEINTERPRET 8.016

Project	Scarborough Eastern Trail		
Analysis Description	West Embankment Station 121+42		
Drawn By	SNP	Scale	1:600
Company	R. W. Gillespie & Associates Inc.		
Date	22 January 2019	File Name	1-21-19.slmd

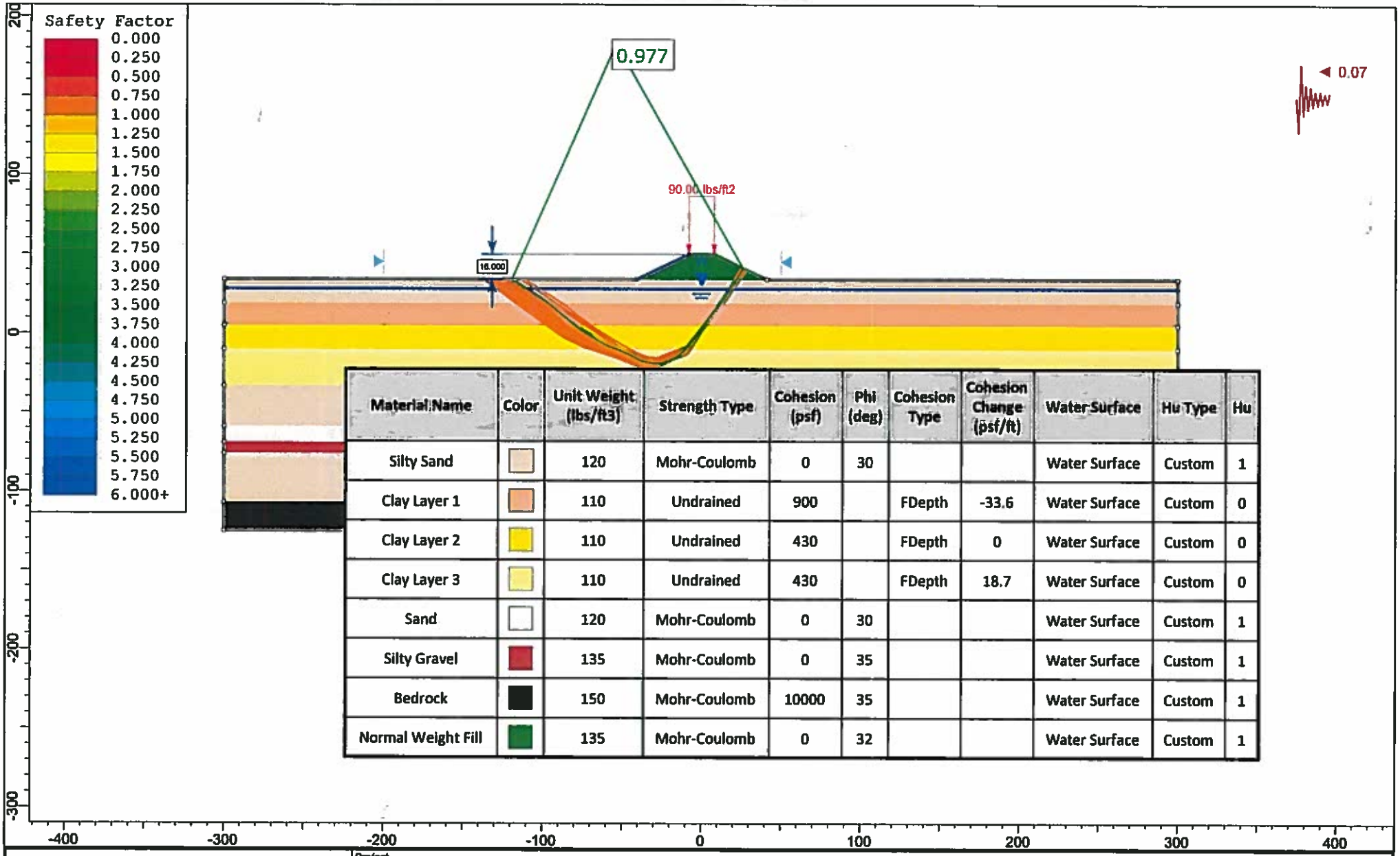


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Cohesion Change (psf/ft)	Water Surface	Hu Type	Hu
Silty Sand		120	Mohr-Coulomb	0	30			Water Surface	Custom	1
Clay Layer 1		110	Undrained	900		FDepth	-33.6	Water Surface	Custom	0
Clay Layer 2		110	Undrained	430		FDepth	0	Water Surface	Custom	0
Clay Layer 3		110	Undrained	430		FDepth	18.7	Water Surface	Custom	0
Sand		120	Mohr-Coulomb	0	30			Water Surface	Custom	1
Silty Gravel		135	Mohr-Coulomb	0	35			Water Surface	Custom	1
Bedrock		150	Mohr-Coulomb	10000	35			Water Surface	Custom	1
Normal Weight Fill		135	Mohr-Coulomb	0	32			Water Surface	Custom	1

rocscience

SLIDEINTERPRET 8.016

Project	Scarborough Eastern Trail		
Analysis Description	West Embankment Station 121+42		
Drawn By: SNP	Scale: 1:600	Company: R. W. Gillespie & Associates Inc.	
Date: 22 January 2019		File Name: 1-21-19.slmd	

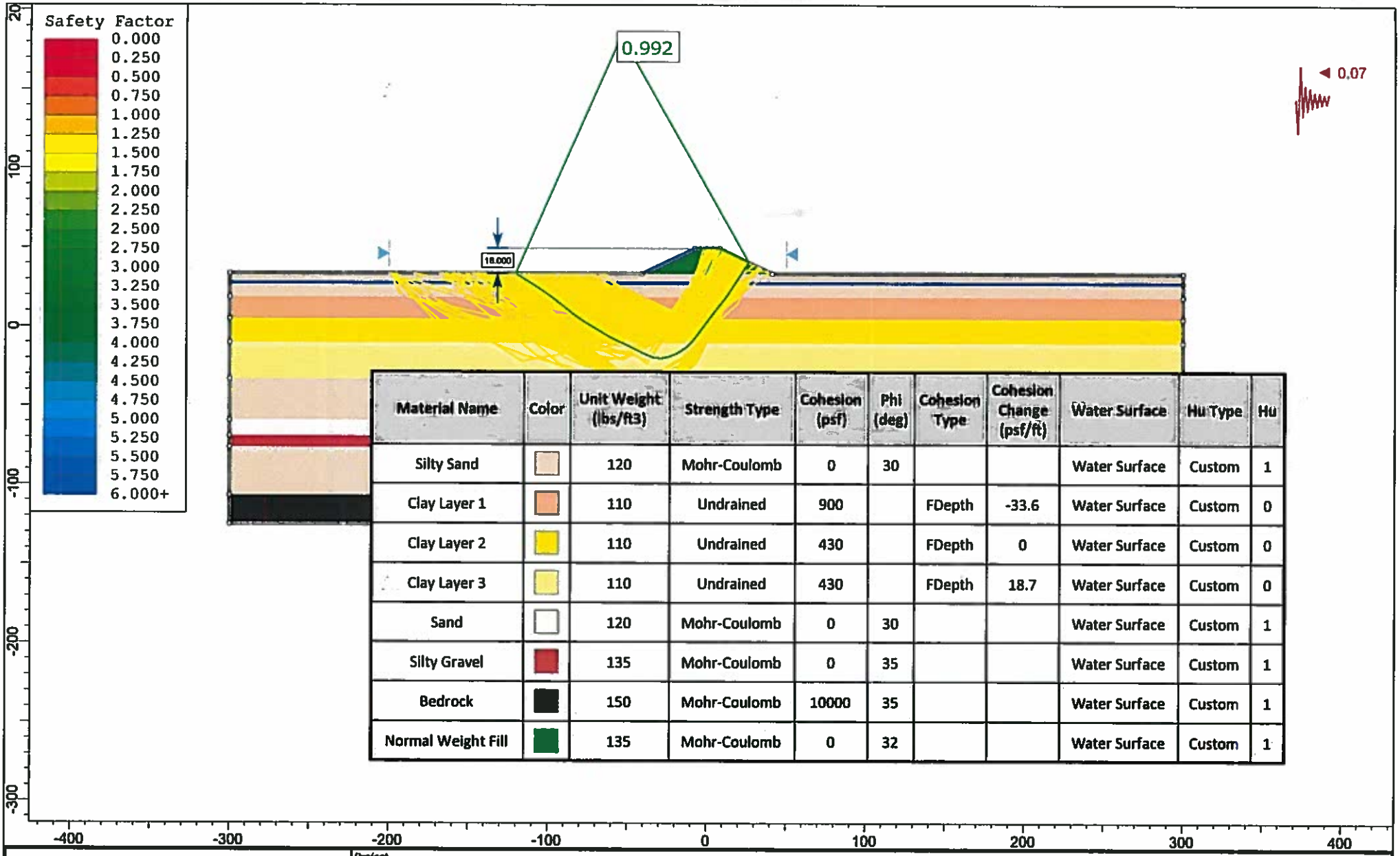


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Cohesion Change (psf/ft)	Water Surface	Hu Type	Hu
Silty Sand		120	Mohr-Coulomb	0	30			Water Surface	Custom	1
Clay Layer 1		110	Undrained	900		FDepth	-33.6	Water Surface	Custom	0
Clay Layer 2		110	Undrained	430		FDepth	0	Water Surface	Custom	0
Clay Layer 3		110	Undrained	430		FDepth	18.7	Water Surface	Custom	0
Sand		120	Mohr-Coulomb	0	30			Water Surface	Custom	1
Silty Gravel		135	Mohr-Coulomb	0	35			Water Surface	Custom	1
Bedrock		150	Mohr-Coulomb	10000	35			Water Surface	Custom	1
Normal Weight Fill		135	Mohr-Coulomb	0	32			Water Surface	Custom	1

rocscience

SLIDEINTERPRET 8.016

Project	Scarborough Eastern Trail		
Analysis Description	West Embankment Station ~~122+42~~ 121+42		
Drawn By: SNP	Scale: 1:1000	Company: R. W. Gillespie & Associates Inc.	
Date: 22 January 2019		File Name: 1-21-19.slmd	

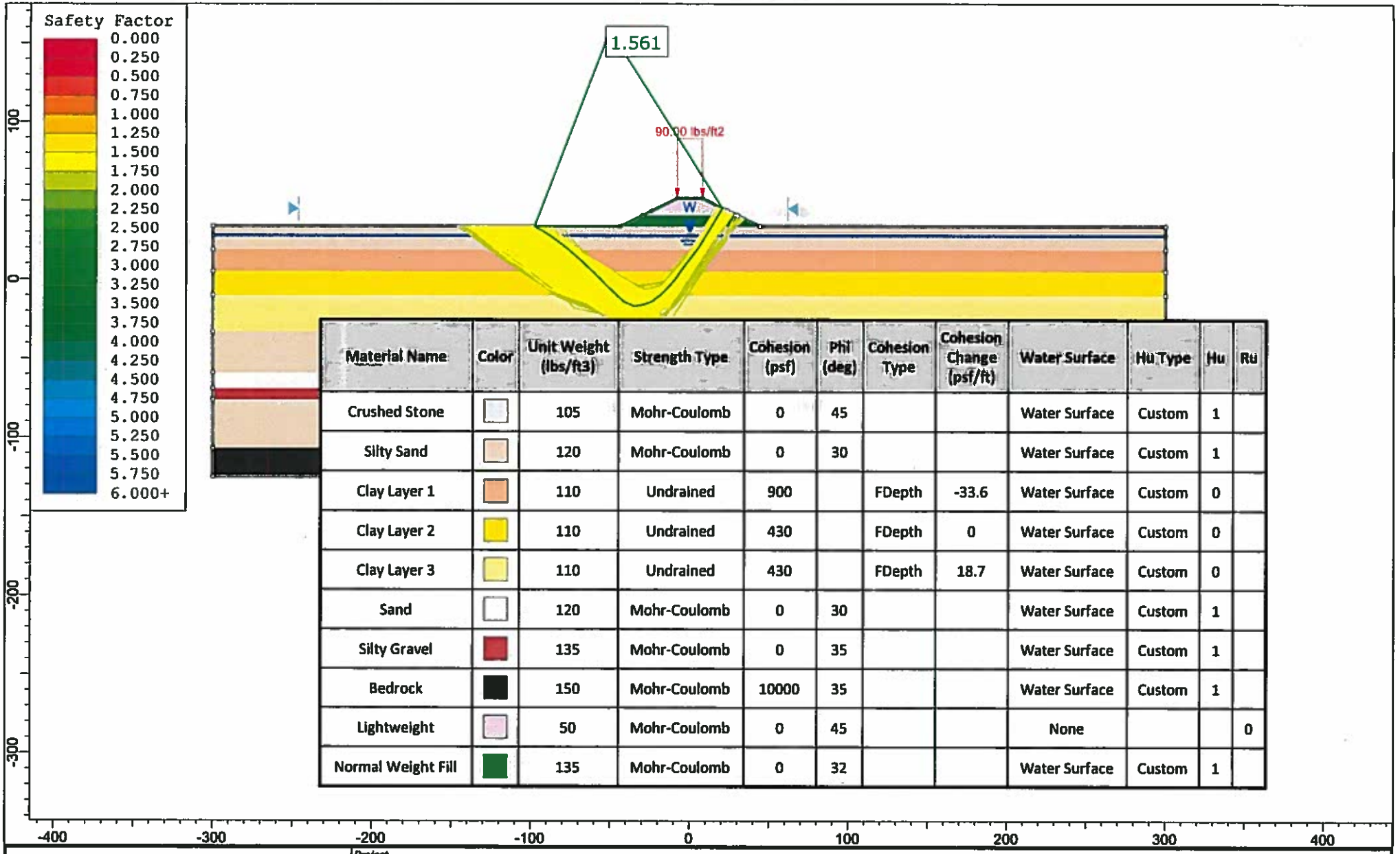


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Cohesion Change (psf/ft)	Water Surface	Hu Type	Hu
Silty Sand		120	Mohr-Coulomb	0	30			Water Surface	Custom	1
Clay Layer 1		110	Undrained	900		FDepth	-33.6	Water Surface	Custom	0
Clay Layer 2		110	Undrained	430		FDepth	0	Water Surface	Custom	0
Clay Layer 3		110	Undrained	430		FDepth	18.7	Water Surface	Custom	0
Sand		120	Mohr-Coulomb	0	30			Water Surface	Custom	1
Silty Gravel		135	Mohr-Coulomb	0	35			Water Surface	Custom	1
Bedrock		150	Mohr-Coulomb	10000	35			Water Surface	Custom	1
Normal Weight Fill		135	Mohr-Coulomb	0	32			Water Surface	Custom	1

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SLIDEINTERPRET 8.016

Project	Scarborough Eastern Trail		
Analysis Description	West Embankment Station 121+42		
Drawn By	SNP	Scale	1:1000
Company	R. W. Gillespie & Associates Inc.		
Date	22 January 2019	File Name	1-21-19.slmd



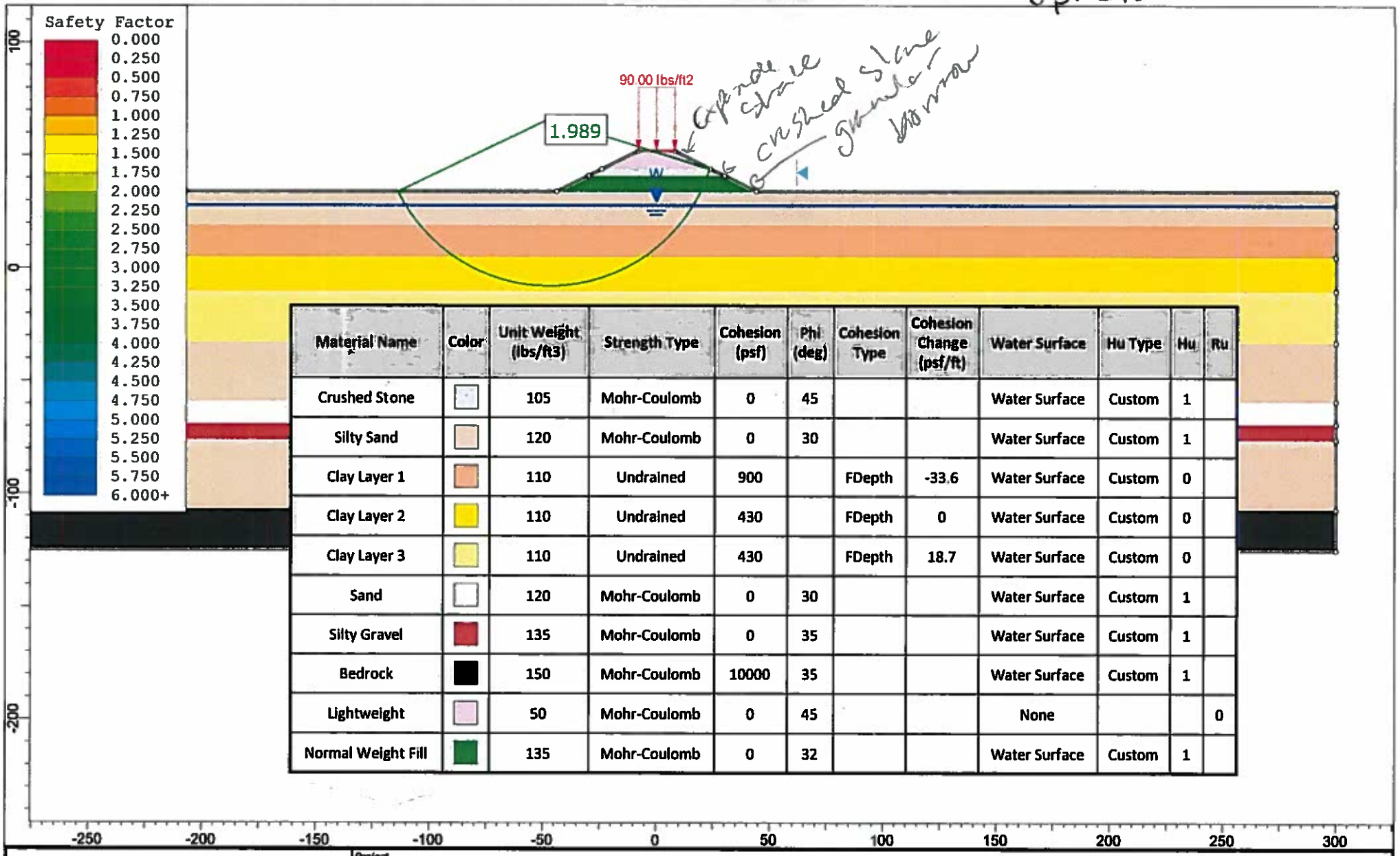
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Cohesion Change (psf/ft)	Water Surface	Hu Type	Hu	Ru
Crushed Stone		105	Mohr-Coulomb	0	45			Water Surface	Custom	1	
Silty Sand		120	Mohr-Coulomb	0	30			Water Surface	Custom	1	
Clay Layer 1		110	Undrained	900		FDepth	-33.6	Water Surface	Custom	0	
Clay Layer 2		110	Undrained	430		FDepth	0	Water Surface	Custom	0	
Clay Layer 3		110	Undrained	430		FDepth	18.7	Water Surface	Custom	0	
Sand		120	Mohr-Coulomb	0	30			Water Surface	Custom	1	
Silty Gravel		135	Mohr-Coulomb	0	35			Water Surface	Custom	1	
Bedrock		150	Mohr-Coulomb	10000	35			Water Surface	Custom	1	
Lightweight		50	Mohr-Coulomb	0	45			None			0
Normal Weight Fill		135	Mohr-Coulomb	0	32			Water Surface	Custom	1	

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SLIDEINTERPRET 8.016

Project	Scarborough Eastern Trail		
Analysis Description	West Embankment Station 121+92		
Drawn By: SNP	Scale: 1:1000	Company: R. W. Gillespie & Associates Inc.	
Date: 22 January 2019		File Name: 1-21-19.slmd	

Option 1

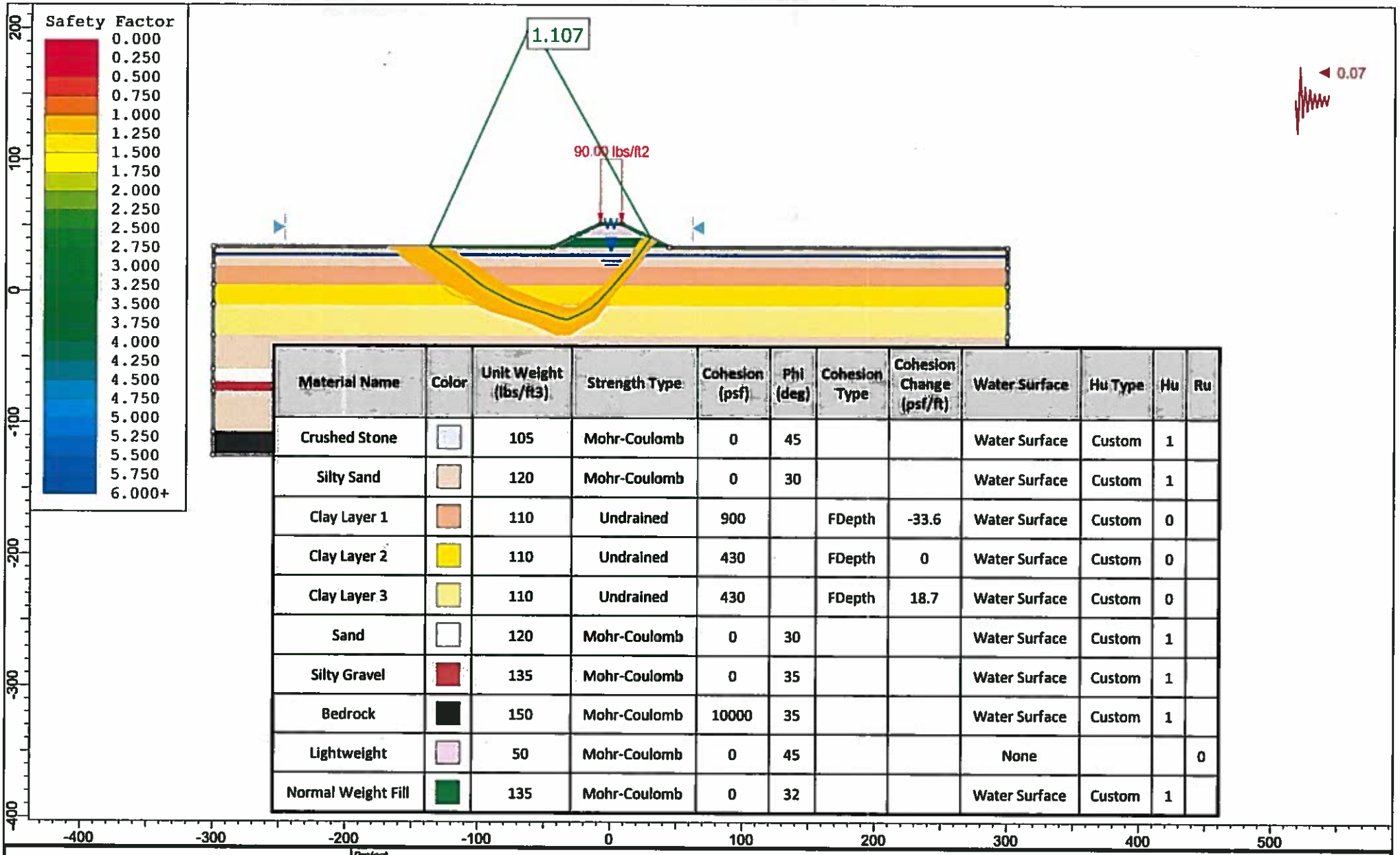


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Cohesion Change (psf/ft)	Water Surface	Hu Type	Hu	Ru
Crushed Stone		105	Mohr-Coulomb	0	45			Water Surface	Custom	1	
Silty Sand		120	Mohr-Coulomb	0	30			Water Surface	Custom	1	
Clay Layer 1		110	Undrained	900		FDepth	-33.6	Water Surface	Custom	0	
Clay Layer 2		110	Undrained	430		FDepth	0	Water Surface	Custom	0	
Clay Layer 3		110	Undrained	430		FDepth	18.7	Water Surface	Custom	0	
Sand		120	Mohr-Coulomb	0	30			Water Surface	Custom	1	
Silty Gravel		135	Mohr-Coulomb	0	35			Water Surface	Custom	1	
Bedrock		150	Mohr-Coulomb	10000	35			Water Surface	Custom	1	
Lightweight		50	Mohr-Coulomb	0	45			None			0
Normal Weight Fill		135	Mohr-Coulomb	0	32			Water Surface	Custom	1	

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SLIDEINTERPRET 8.016

Project	Scarborough Eastern Trail		
Analysis Description	West Embankment Station 121+92		
Drawn By: SNP	Scale: 1:700	Company: R. W. Gillespie & Associates Inc.	
Date: 22 January 2019		File Name: 1-21-19.slmd	

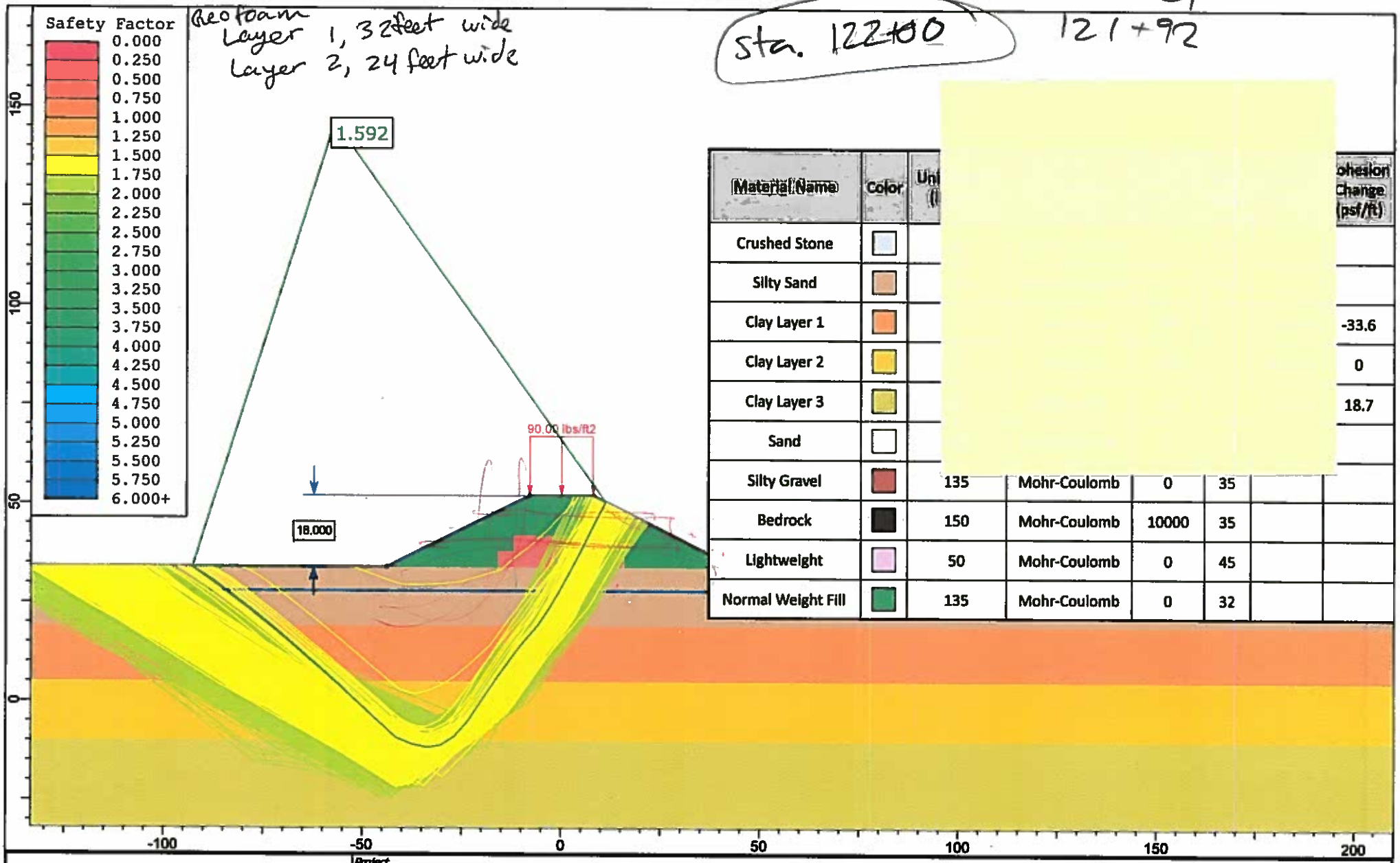


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Cohesion Change (psf/ft)	Water Surface	Hu Type	Hu	Ru
Crushed Stone		105	Mohr-Coulomb	0	45			Water Surface	Custom	1	
Silty Sand		120	Mohr-Coulomb	0	30			Water Surface	Custom	1	
Clay Layer 1		110	Undrained	900		FDepth	-33.6	Water Surface	Custom	0	
Clay Layer 2		110	Undrained	430		FDepth	0	Water Surface	Custom	0	
Clay Layer 3		110	Undrained	430		FDepth	18.7	Water Surface	Custom	0	
Sand		120	Mohr-Coulomb	0	30			Water Surface	Custom	1	
Silty Gravel		135	Mohr-Coulomb	0	35			Water Surface	Custom	1	
Bedrock		150	Mohr-Coulomb	10000	35			Water Surface	Custom	1	
Lightweight		50	Mohr-Coulomb	0	45			None			0
Normal Weight Fill		135	Mohr-Coulomb	0	32			Water Surface	Custom	1	

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SLIDEINTERPRET 8.016

Project	Scarborough Eastern Trail		
Analysis Description	West Embankment Station 121+92		
Drawn By	SNP	Scale	1:1200
Company	R. W. Gillespie & Associates Inc.		
Date	22 January 2019	File Name	1-21-19.slmd

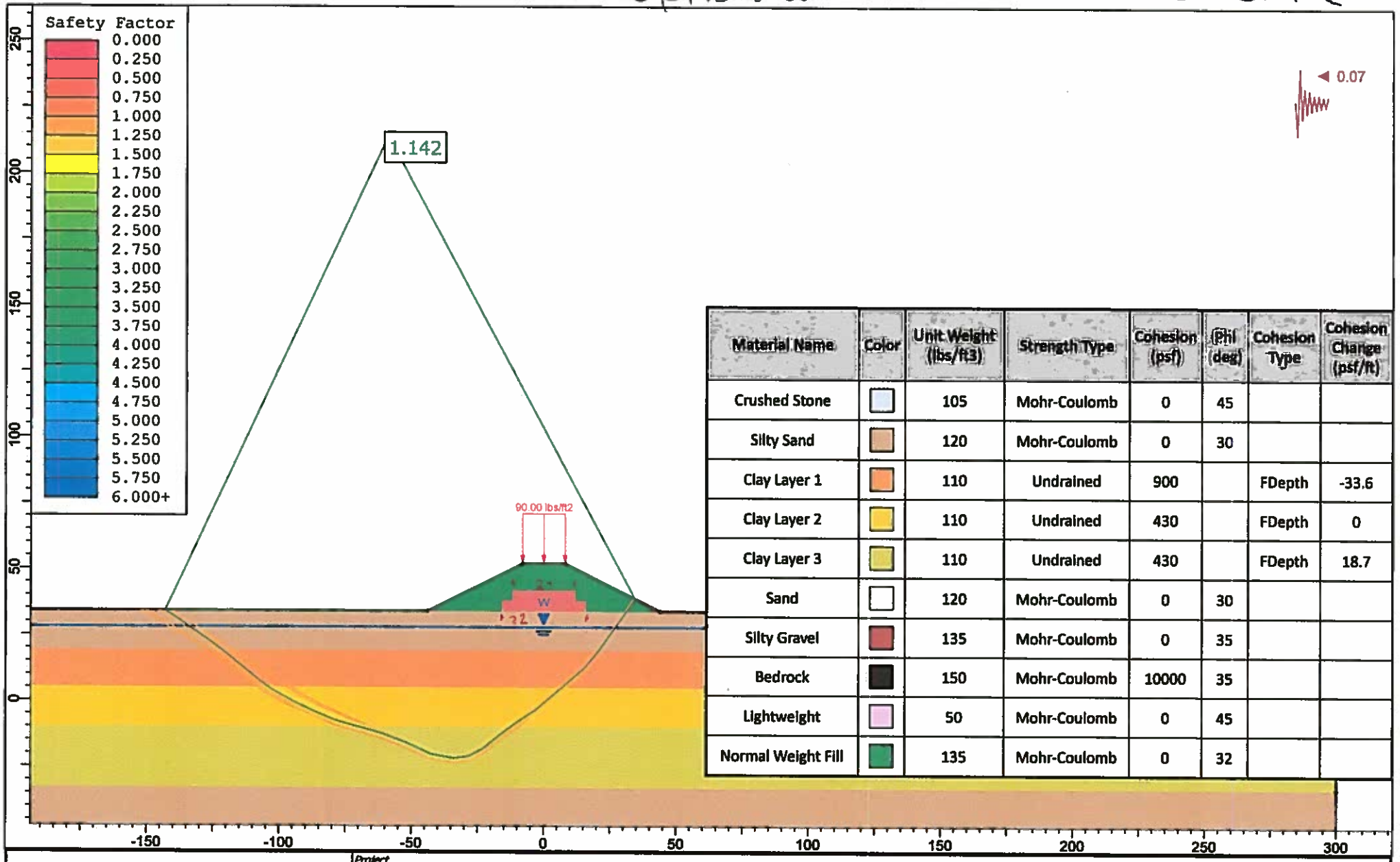


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SLIDEINTERPRET 8.016

Project	Scarborough Eastern Trail		
Analysis Description	West Embankment Station 121+~~92~~92		
Drawn By	SNP	Scale 1:400	Company R. W. Gillespie & Associates Inc.
Date	22 January 2019		File Name 2-1-2019.slmd

Option 2 SEISMIC

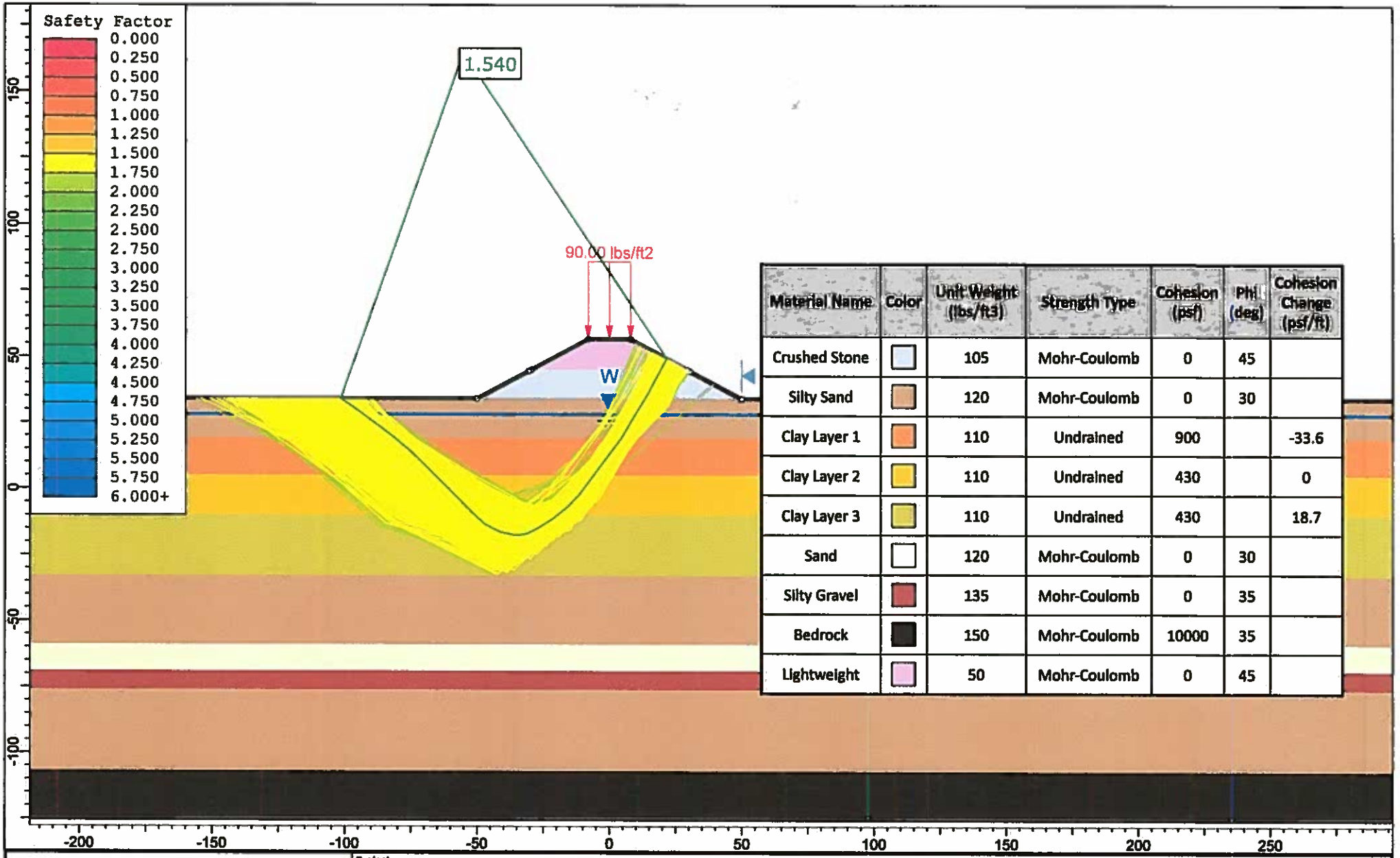


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Cohesion Change (psf/ft)
Crushed Stone		105	Mohr-Coulomb	0	45		
Silty Sand		120	Mohr-Coulomb	0	30		
Clay Layer 1		110	Undrained	900		FDepth	-33.6
Clay Layer 2		110	Undrained	430		FDepth	0
Clay Layer 3		110	Undrained	430		FDepth	18.7
Sand		120	Mohr-Coulomb	0	30		
Silty Gravel		135	Mohr-Coulomb	0	35		
Bedrock		150	Mohr-Coulomb	10000	35		
Lightweight		50	Mohr-Coulomb	0	45		
Normal Weight Fill		135	Mohr-Coulomb	0	32		

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SLIDEINTERPRET 8.016

Project	Scarborough Eastern Trail		
Analysis Description	West Embankment Station 121+~~12~~92		
Drawn By	SNP	Scale 1:600	Company R. W. Gillespie & Associates Inc.
Date	22 January 2019		File Name 2-1-2019.slmd

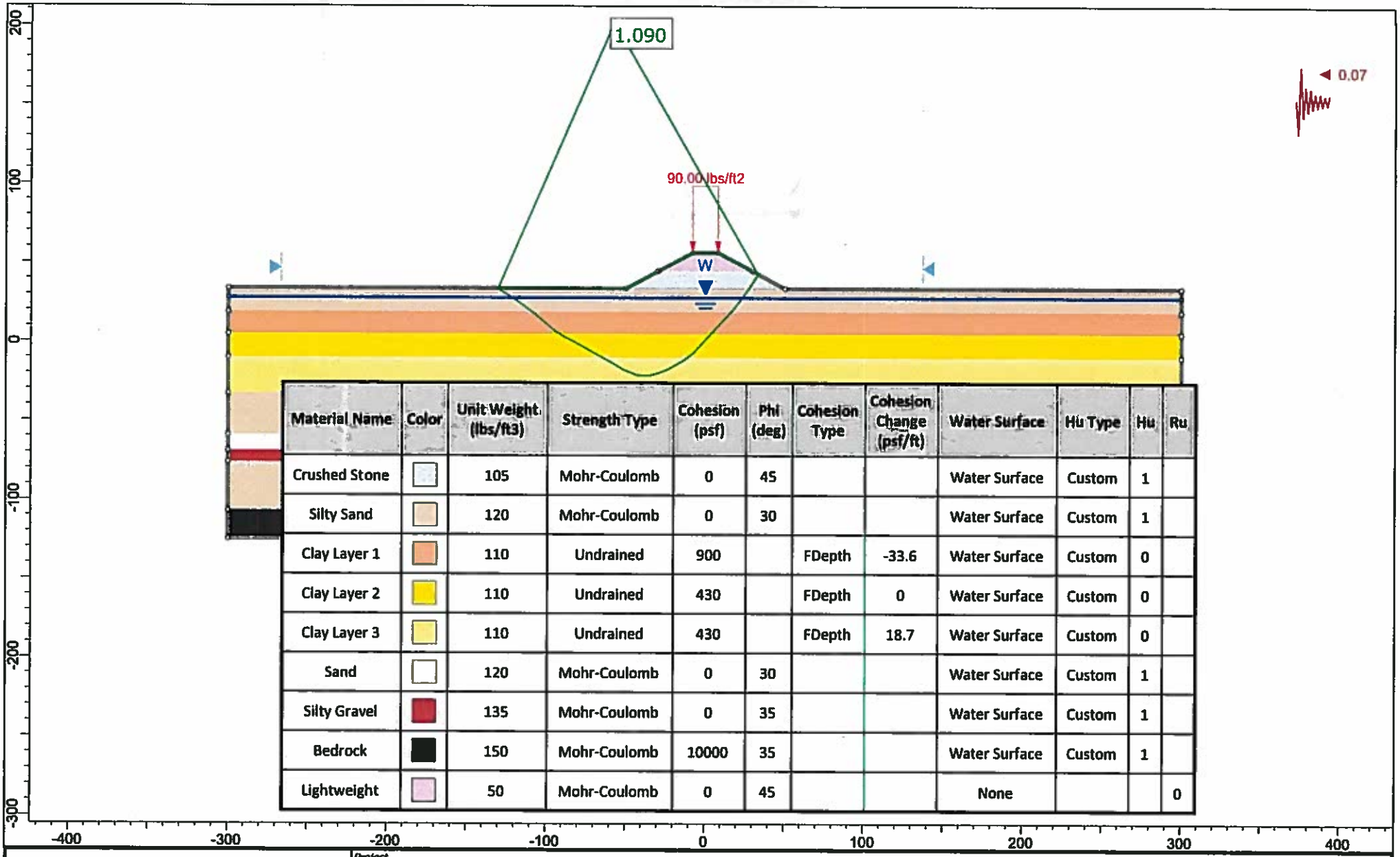


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Change (psf/ft)
Crushed Stone		105	Mohr-Coulomb	0	45	
Silty Sand		120	Mohr-Coulomb	0	30	
Clay Layer 1		110	Undrained	900		-33.6
Clay Layer 2		110	Undrained	430		0
Clay Layer 3		110	Undrained	430		18.7
Sand		120	Mohr-Coulomb	0	30	
Silty Gravel		135	Mohr-Coulomb	0	35	
Bedrock		150	Mohr-Coulomb	10000	35	
Lightweight		50	Mohr-Coulomb	0	45	

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SLIDEINTERPRET 8.016

Project	Scarborough Eastern Trail		
Analysis Description	West Embankment Station ~~121+42~~ 122+42		
Drawn By: SNP	Scale: 1:600	Company: R. W. Gillespie & Associates Inc.	
Date: 22 January 2019		File Name: 1-21-19.slmd	

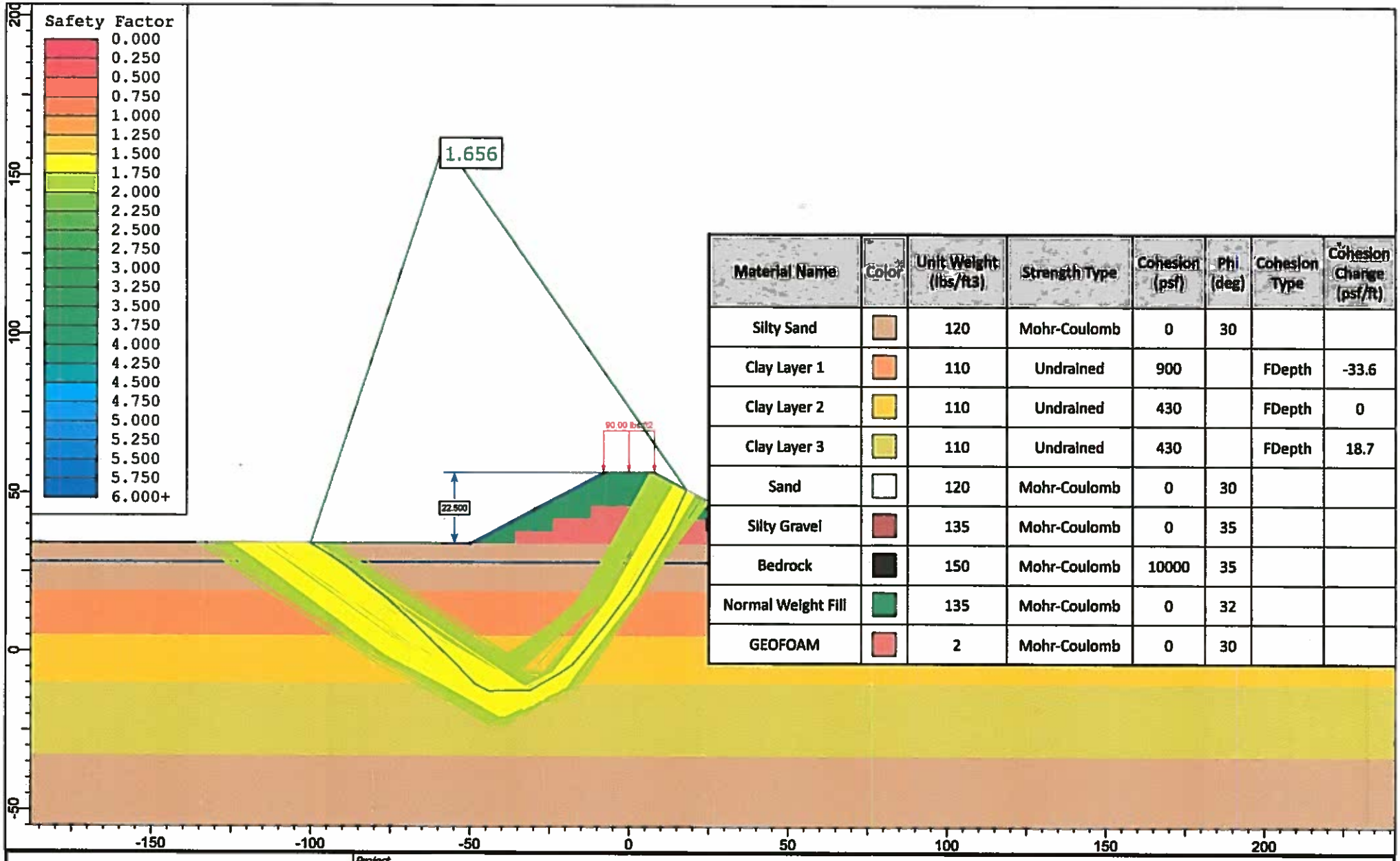


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Cohesion Change (psf/ft)	Water Surface	Hu Type	Hu	Ru
Crushed Stone		105	Mohr-Coulomb	0	45			Water Surface	Custom	1	
Silty Sand		120	Mohr-Coulomb	0	30			Water Surface	Custom	1	
Clay Layer 1		110	Undrained	900		FDepth	-33.6	Water Surface	Custom	0	
Clay Layer 2		110	Undrained	430		FDepth	0	Water Surface	Custom	0	
Clay Layer 3		110	Undrained	430		FDepth	18.7	Water Surface	Custom	0	
Sand		120	Mohr-Coulomb	0	30			Water Surface	Custom	1	
Silty Gravel		135	Mohr-Coulomb	0	35			Water Surface	Custom	1	
Bedrock		150	Mohr-Coulomb	10000	35			Water Surface	Custom	1	
Lightweight		50	Mohr-Coulomb	0	45			None			0

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SLIDEINTERPRET 8.016

Project	Scarborough Eastern Trail		
Analysis Description	West Embankment Station 122+42		
Drawn By	SNP	Scale	1:1000
Company	R. W. Gillespie & Associates Inc.		
Date	22 January 2019	File Name	1-21-19.slmd

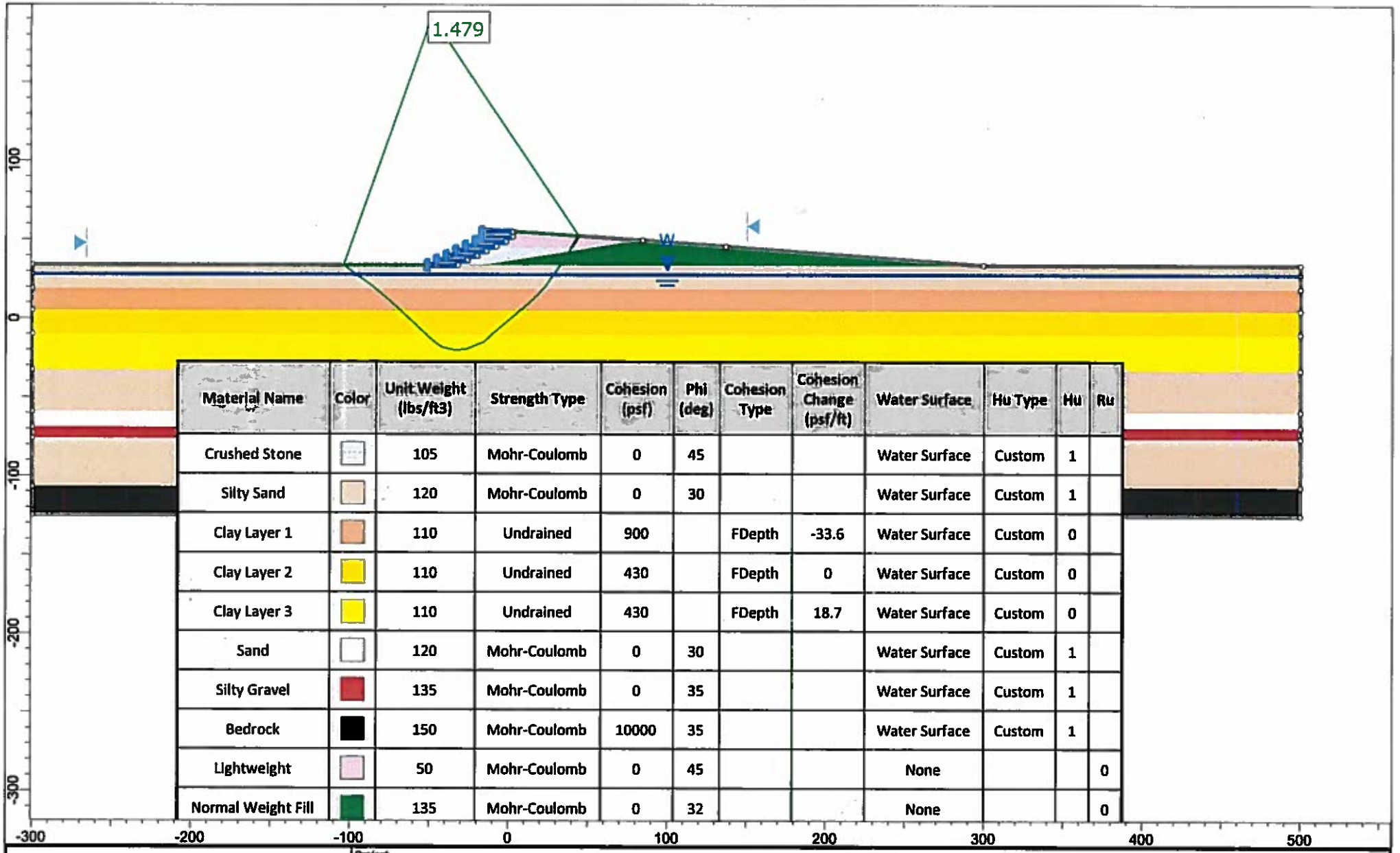


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Cohesion Change (psf/ft)
Silty Sand		120	Mohr-Coulomb	0	30		
Clay Layer 1		110	Undrained	900		FDepth	-33.6
Clay Layer 2		110	Undrained	430		FDepth	0
Clay Layer 3		110	Undrained	430		FDepth	18.7
Sand		120	Mohr-Coulomb	0	30		
Silty Gravel		135	Mohr-Coulomb	0	35		
Bedrock		150	Mohr-Coulomb	10000	35		
Normal Weight Fill		135	Mohr-Coulomb	0	32		
GEOFOAM		2	Mohr-Coulomb	0	30		

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SLIDEINTERPRET 8.016

Project	Scarborough Eastern Trail		
Analysis Description	West Embankment Station ~~121+42~~ 122+42		
Drawn By	SNP	Scale 1:500	Company R. W. Gillespie & Associates Inc.
Date	22 January 2019		File Name 1-24-2019.slmd

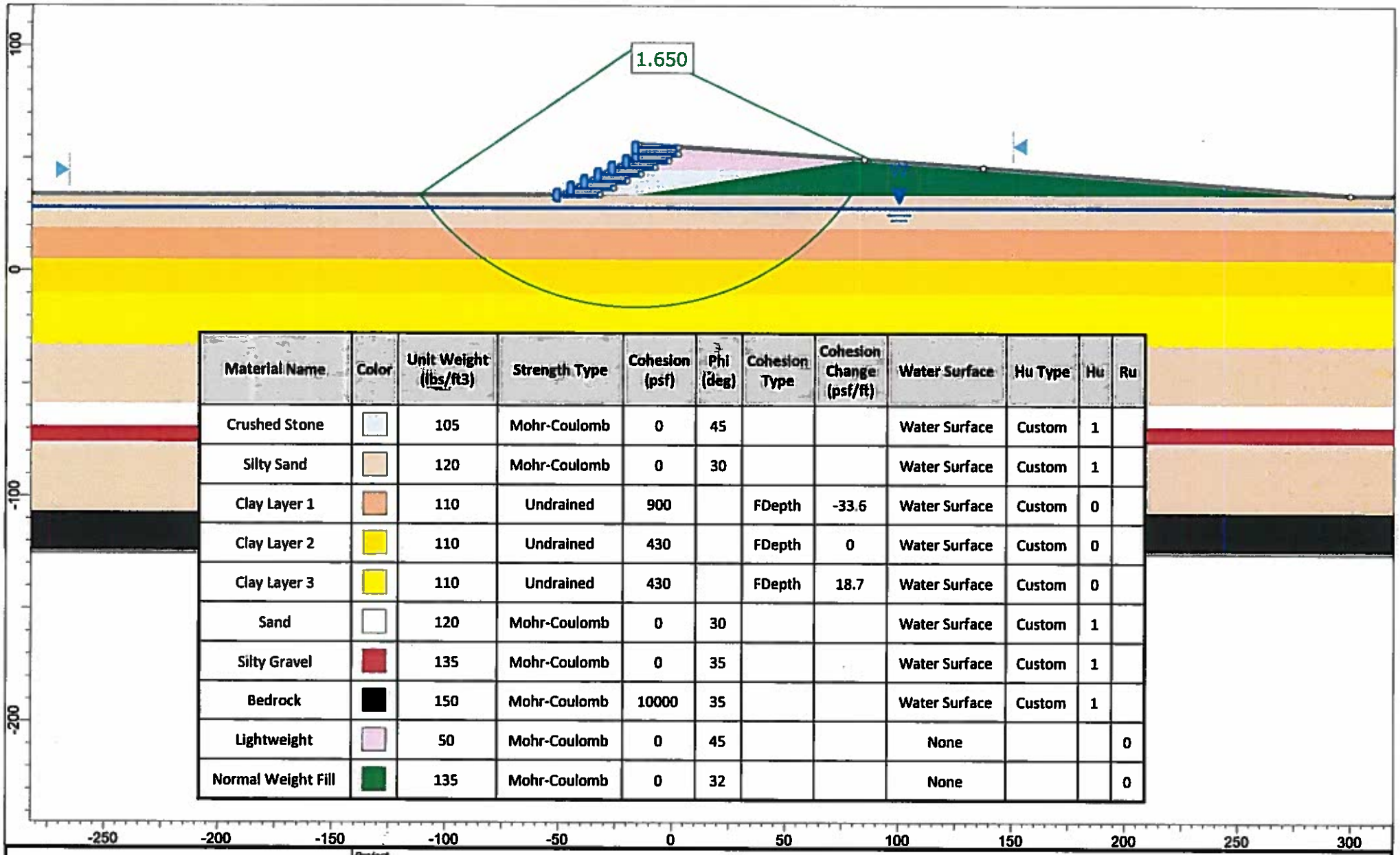


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Cohesion Change (psf/ft)	Water Surface	Hu Type	Hu	Ru
Crushed Stone		105	Mohr-Coulomb	0	45			Water Surface	Custom	1	
Silty Sand		120	Mohr-Coulomb	0	30			Water Surface	Custom	1	
Clay Layer 1		110	Undrained	900		FDepth	-33.6	Water Surface	Custom	0	
Clay Layer 2		110	Undrained	430		FDepth	0	Water Surface	Custom	0	
Clay Layer 3		110	Undrained	430		FDepth	18.7	Water Surface	Custom	0	
Sand		120	Mohr-Coulomb	0	30			Water Surface	Custom	1	
Silty Gravel		135	Mohr-Coulomb	0	35			Water Surface	Custom	1	
Bedrock		150	Mohr-Coulomb	10000	35			Water Surface	Custom	1	
Lightweight		50	Mohr-Coulomb	0	45			None			0
Normal Weight Fill		135	Mohr-Coulomb	0	32			None			0

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SLIDEINTERPRET 8.016

Project	Scarborough Eastern Trail		
Analysis Description	West Abutment		
Drawn By	SNP	Scale	1:1000
Company	R. W. Gillespie & Associates Inc.		
Date	22 January 2019	File Name	11-30-18.slmd

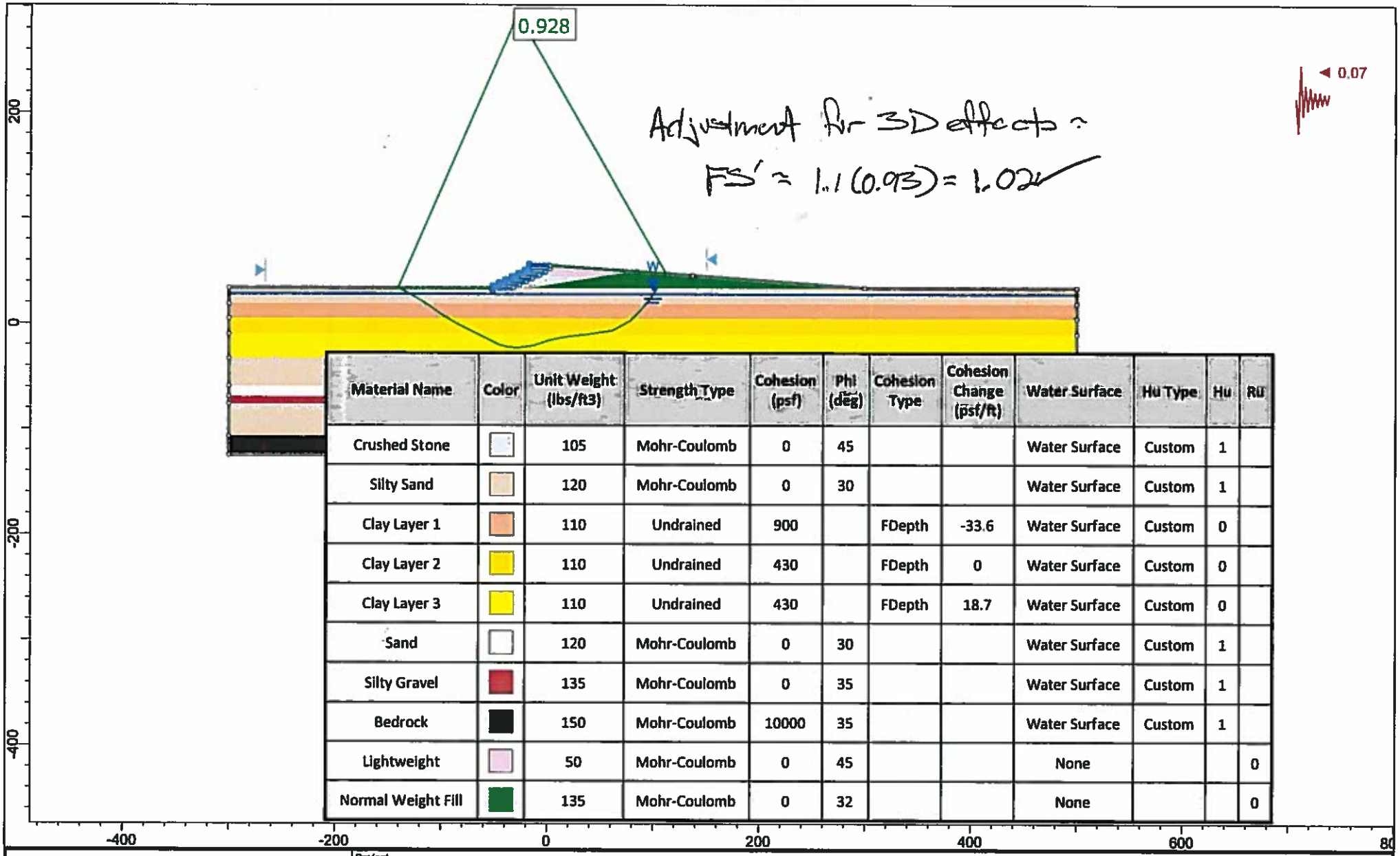


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Cohesion Change (psf/ft)	Water Surface	Hu Type	Hu	Ru
Crushed Stone		105	Mohr-Coulomb	0	45			Water Surface	Custom	1	
Silty Sand		120	Mohr-Coulomb	0	30			Water Surface	Custom	1	
Clay Layer 1		110	Undrained	900		FDepth	-33.6	Water Surface	Custom	0	
Clay Layer 2		110	Undrained	430		FDepth	0	Water Surface	Custom	0	
Clay Layer 3		110	Undrained	430		FDepth	18.7	Water Surface	Custom	0	
Sand		120	Mohr-Coulomb	0	30			Water Surface	Custom	1	
Silty Gravel		135	Mohr-Coulomb	0	35			Water Surface	Custom	1	
Bedrock		150	Mohr-Coulomb	10000	35			Water Surface	Custom	1	
Lightweight		50	Mohr-Coulomb	0	45			None			0
Normal Weight Fill		135	Mohr-Coulomb	0	32			None			0

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SLIDEINTERPRET 8.016

Project	Scarborough Eastern Trail		
Analysis Description	West Abutment		
Drawn By	SNP	Scale	1:700
Company	R. W. Gillespie & Associates Inc.		
Date	22 January 2019	File Name	11-30-18.slmd



Adjustment for 3D effects:

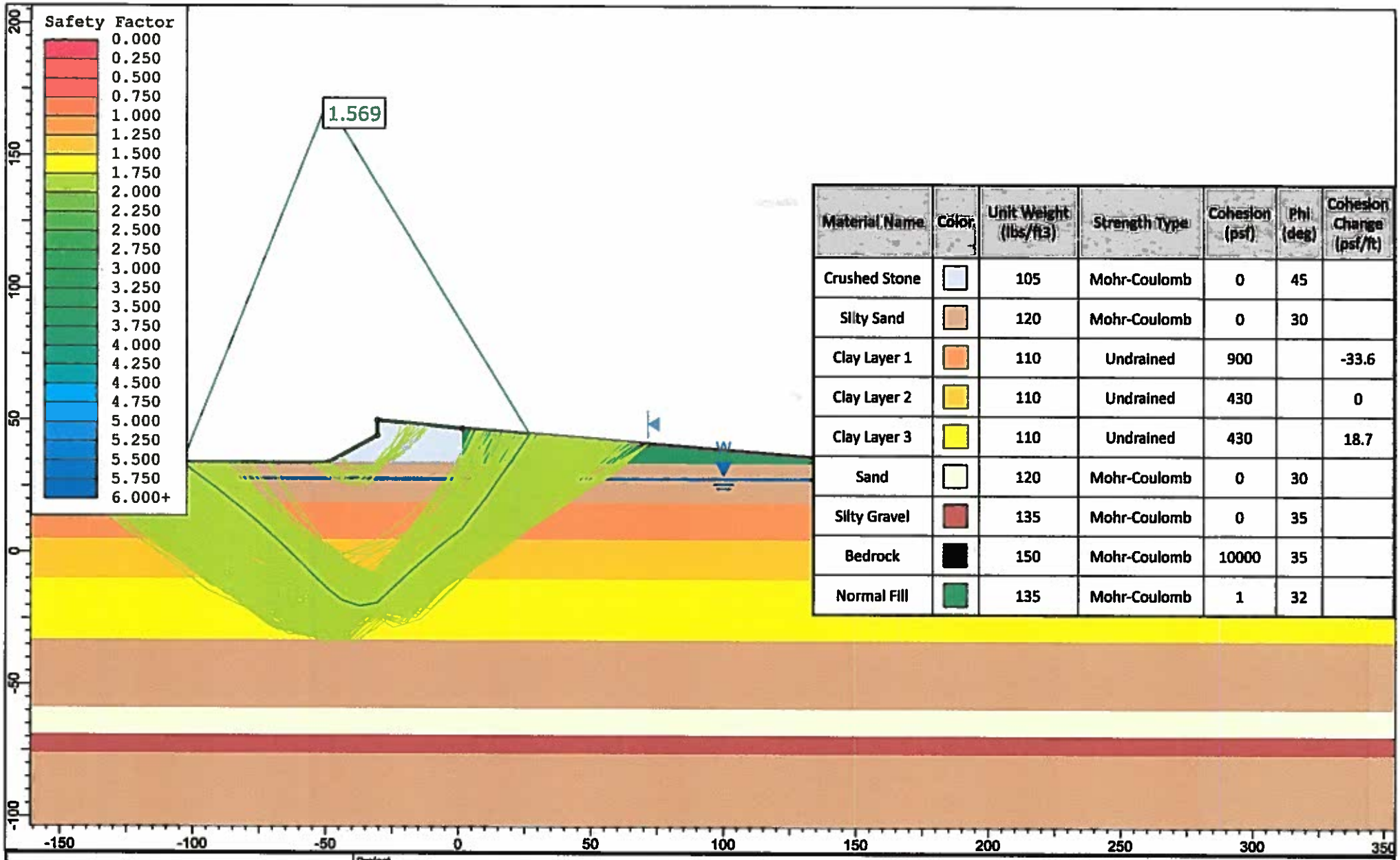
$FS' = 1.1(0.93) = 1.02$ ✓

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Cohesion Change (psf/ft)	Water Surface	Hu Type	Hu	Ru
Crushed Stone		105	Mohr-Coulomb	0	45			Water Surface	Custom	1	
Silty Sand		120	Mohr-Coulomb	0	30			Water Surface	Custom	1	
Clay Layer 1		110	Undrained	900		FDepth	-33.6	Water Surface	Custom	0	
Clay Layer 2		110	Undrained	430		FDepth	0	Water Surface	Custom	0	
Clay Layer 3		110	Undrained	430		FDepth	18.7	Water Surface	Custom	0	
Sand		120	Mohr-Coulomb	0	30			Water Surface	Custom	1	
Silty Gravel		135	Mohr-Coulomb	0	35			Water Surface	Custom	1	
Bedrock		150	Mohr-Coulomb	10000	35			Water Surface	Custom	1	
Lightweight		50	Mohr-Coulomb	0	45			None			0
Normal Weight Fill		135	Mohr-Coulomb	0	32			None			0

rocscience

SLIDEINTERPRET 8.016

Project	Scarborough Eastern Trail		
Analysis Description	West Abutment		
Drawn By: SNP	Scale: 1:1500	Company: R. W. Gillespie & Associates Inc.	
Date: 22 January 2019		File Name: 11-30-18.slmd	

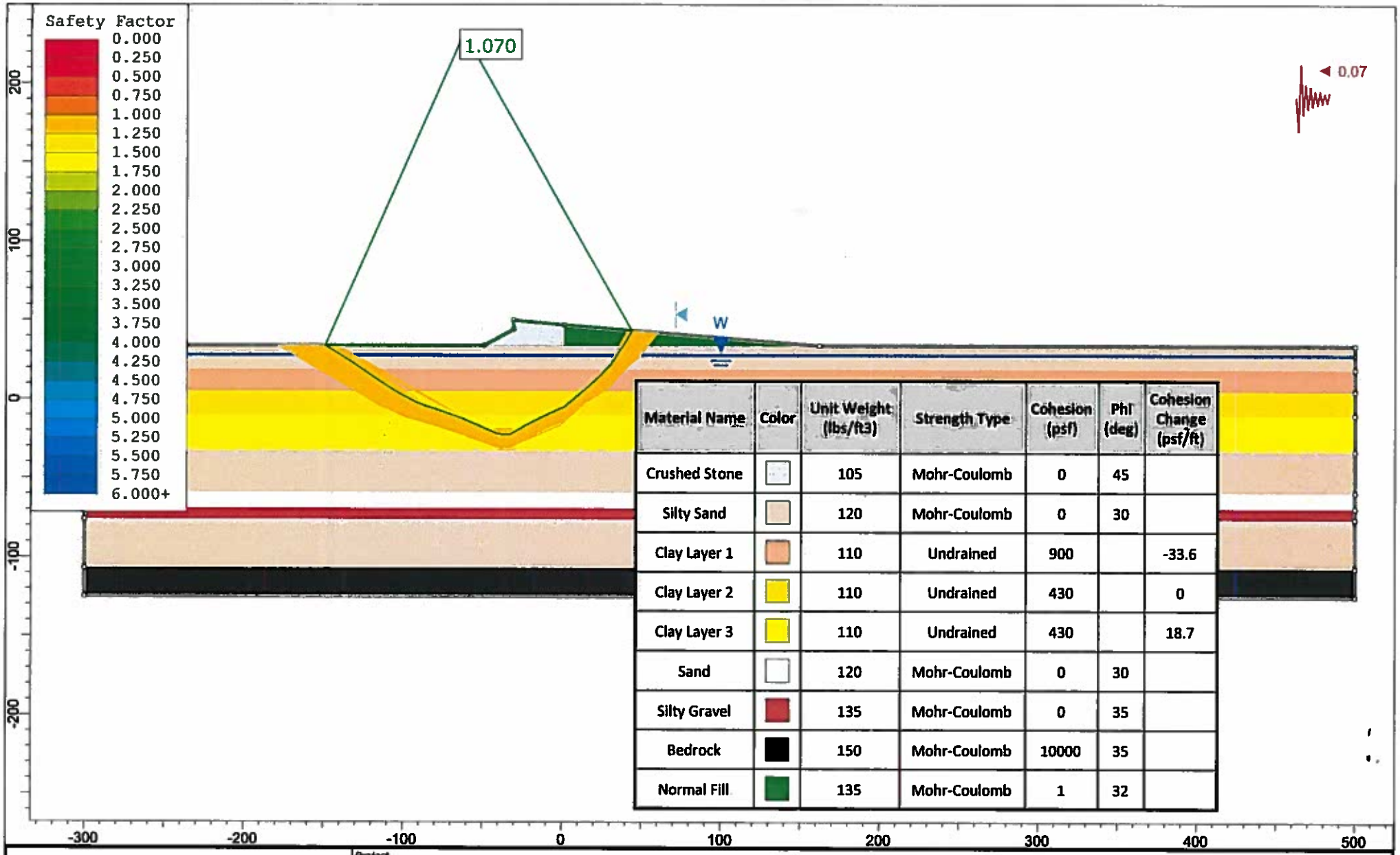


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Change (psf/ft)
Crushed Stone		105	Mohr-Coulomb	0	45	
Silty Sand		120	Mohr-Coulomb	0	30	
Clay Layer 1		110	Undrained	900		-33.6
Clay Layer 2		110	Undrained	430		0
Clay Layer 3		110	Undrained	430		18.7
Sand		120	Mohr-Coulomb	0	30	
Silty Gravel		135	Mohr-Coulomb	0	35	
Bedrock		150	Mohr-Coulomb	10000	35	
Normal Fill		135	Mohr-Coulomb	1	32	

rocscience

SLIDEINTERPRET 8.016

Project	Scarborough Eastern Trail STARK		
Analysis Description	~~13' Embankment Cross Section with 105 pcf Fill~~ EAST ABUTMENT		
Drawn By	SNP	Scale 1:600	Company R. W. Gillespie & Associates Inc.
Date	30 JULY 2018		File Name 10.5 Feet Lightweight Greater Shear Strength.slmd

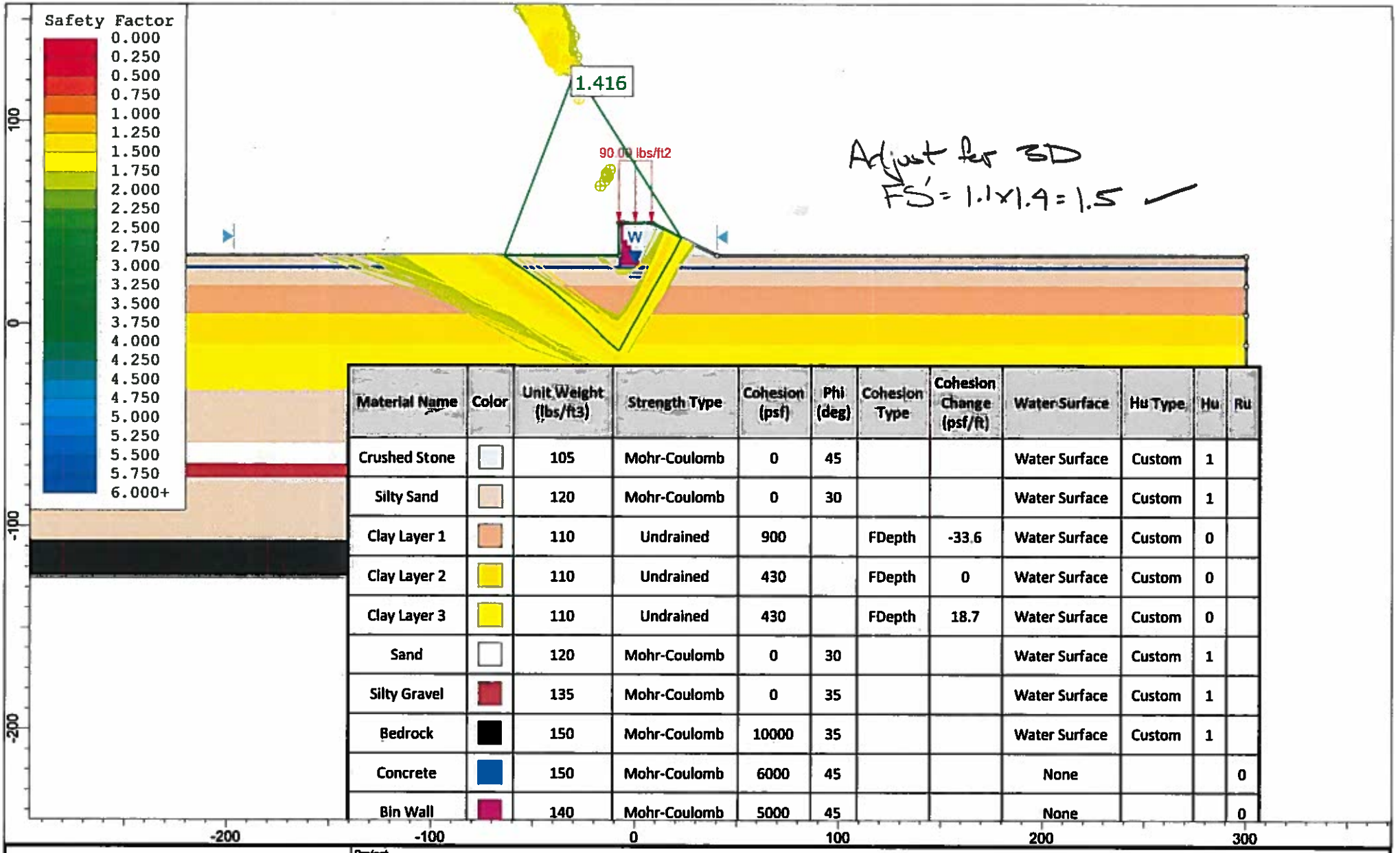


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Change (psf/ft)
Crushed Stone		105	Mohr-Coulomb	0	45	
Silty Sand		120	Mohr-Coulomb	0	30	
Clay Layer 1		110	Undrained	900		-33.6
Clay Layer 2		110	Undrained	430		0
Clay Layer 3		110	Undrained	430		18.7
Sand		120	Mohr-Coulomb	0	30	
Silty Gravel		135	Mohr-Coulomb	0	35	
Bedrock		150	Mohr-Coulomb	10000	35	
Normal Fill		135	Mohr-Coulomb	1	32	

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SLIDEINTERPRET 8.016

Project	Scarborough Eastern Trail		
Analysis Description	East Abutment -SEISMIC		
Drawn By: SNP	Scale: 1:1000	Company: R. W. Gillespie & Associates Inc.	
Date: 22 January 2019		File Name: 10.5 Feet Lightweight Greater Shear Strength.slmd	



Adjust for 3D

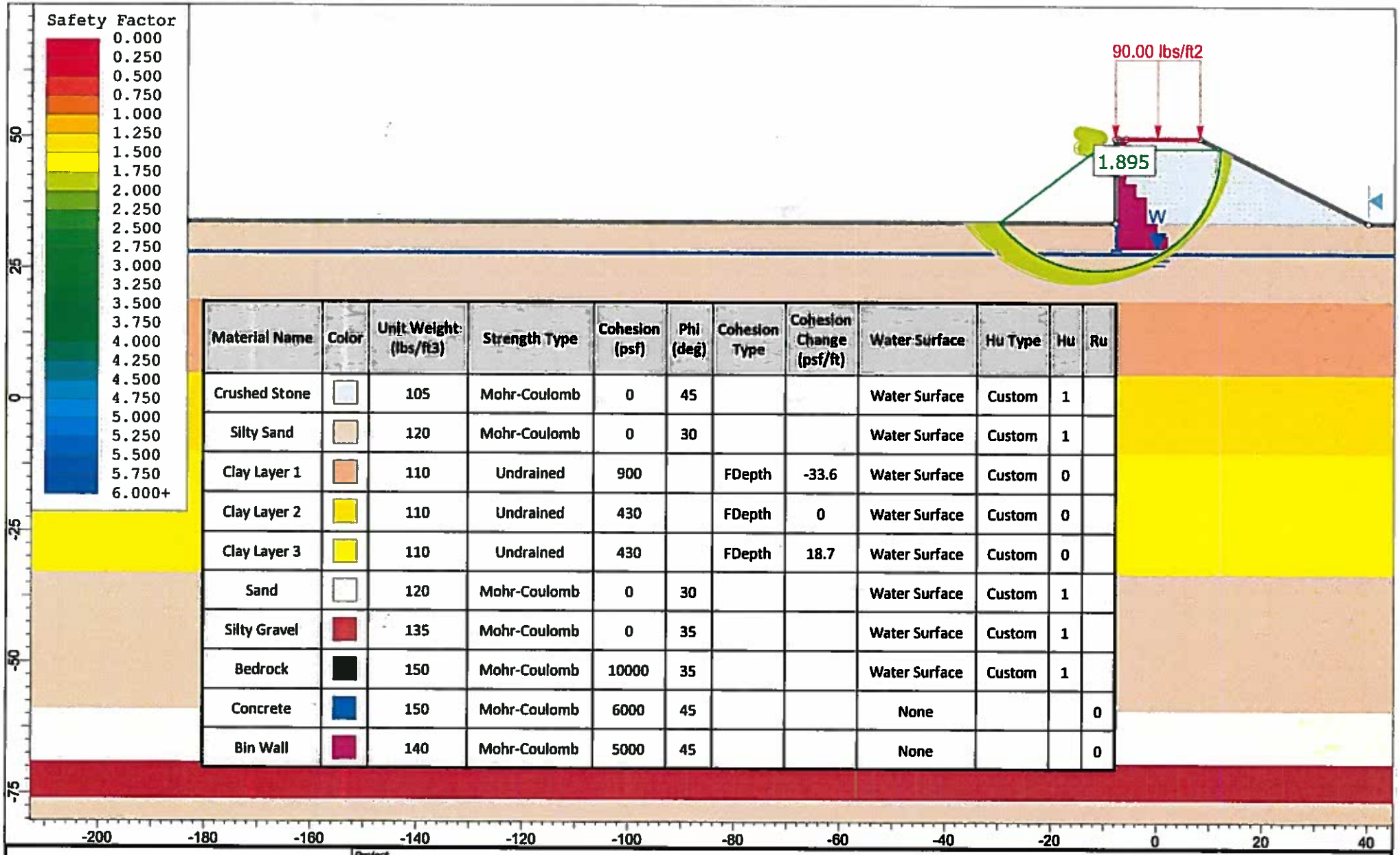
FS = 1.1 x 1.9 = 1.5 ✓

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Cohesion Change (psf/ft)	Water Surface	Hu Type	Hu	Ru
Crushed Stone		105	Mohr-Coulomb	0	45			Water Surface	Custom	1	
Silty Sand		120	Mohr-Coulomb	0	30			Water Surface	Custom	1	
Clay Layer 1		110	Undrained	900		FDepth	-33.6	Water Surface	Custom	0	
Clay Layer 2		110	Undrained	430		FDepth	0	Water Surface	Custom	0	
Clay Layer 3		110	Undrained	430		FDepth	18.7	Water Surface	Custom	0	
Sand		120	Mohr-Coulomb	0	30			Water Surface	Custom	1	
Silty Gravel		135	Mohr-Coulomb	0	35			Water Surface	Custom	1	
Bedrock		150	Mohr-Coulomb	10000	35			Water Surface	Custom	1	
Concrete		150	Mohr-Coulomb	6000	45			None			0
Bin Wall		140	Mohr-Coulomb	5000	45			None			0

rocscience

SLIDEINTERPRET 8.016

Project	Scarborough Eastern Trail		
Analysis Description	East Embankment Station 124+62		
Drawn By	SNP	Scale	1:778
Company	R. W. Gillespie & Associates Inc.		
Date	22 January 2019	File Name	East Embankment Greater Shear Strengths.slmd

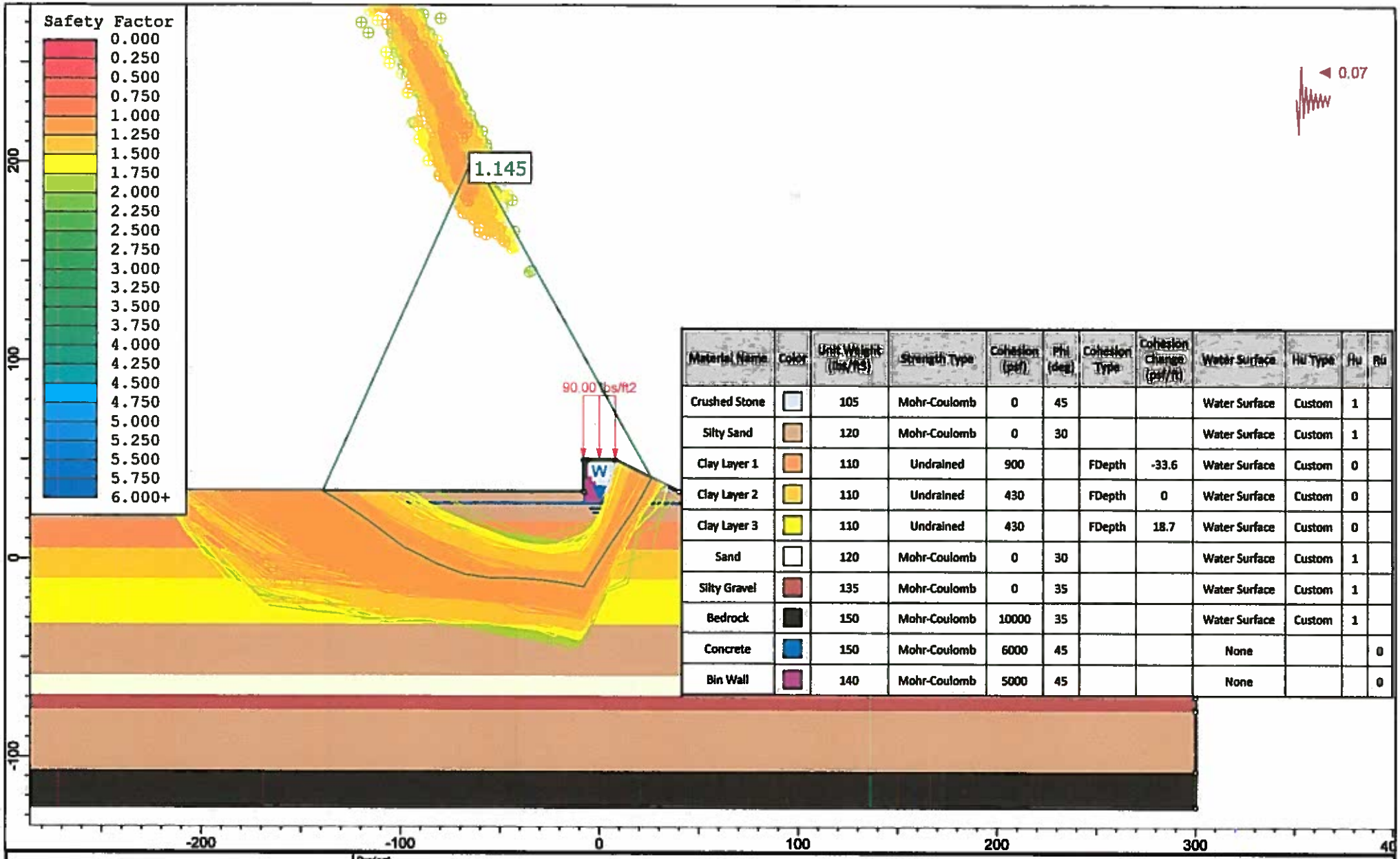


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Cohesion Change (psf/ft)	Water Surface	Hu Type	Hu	Ru
Crushed Stone		105	Mohr-Coulomb	0	45			Water Surface	Custom	1	
Silty Sand		120	Mohr-Coulomb	0	30			Water Surface	Custom	1	
Clay Layer 1		110	Undrained	900		FDepth	-33.6	Water Surface	Custom	0	
Clay Layer 2		110	Undrained	430		FDepth	0	Water Surface	Custom	0	
Clay Layer 3		110	Undrained	430		FDepth	18.7	Water Surface	Custom	0	
Sand		120	Mohr-Coulomb	0	30			Water Surface	Custom	1	
Silty Gravel		135	Mohr-Coulomb	0	35			Water Surface	Custom	1	
Bedrock		150	Mohr-Coulomb	10000	35			Water Surface	Custom	1	
Concrete		150	Mohr-Coulomb	6000	45			None			0
Bin Wall		140	Mohr-Coulomb	5000	45			None			0

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SLIDEINTERPRET 8.016

Project	Scarborough Eastern Trail		
Analysis Description	East Embankment Station 124+62		
Drawn By	SNP	Scale	1:300
Company	R. W. Gillespie & Associates Inc.		
Date	22 January 2019	File Name	East Embankment Greater Shear Strengths.slmd

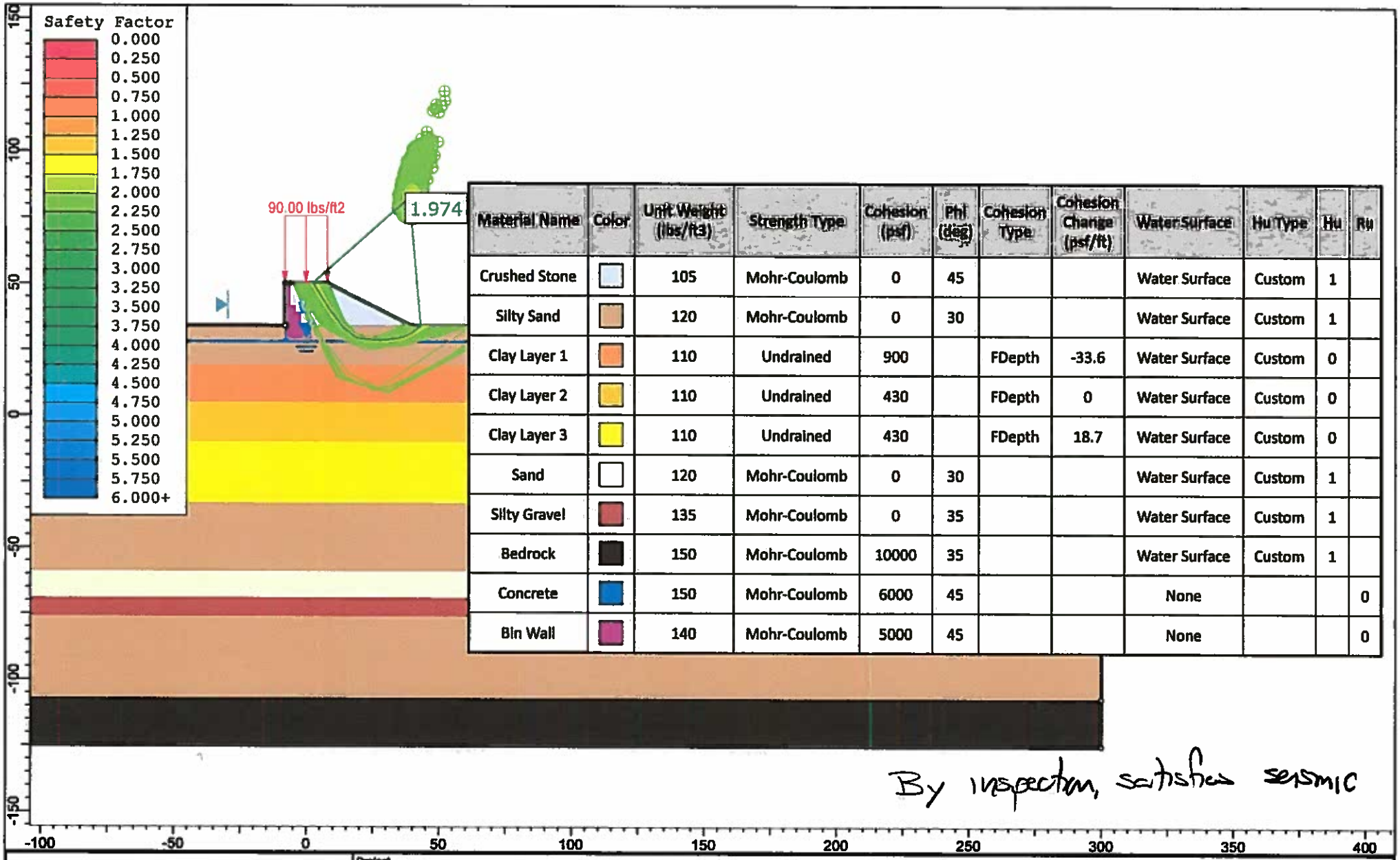


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Cohesion Change (psf/ft)	Water Surface	Ru Type	Ru	Ru
Crushed Stone		105	Mohr-Coulomb	0	45			Water Surface	Custom	1	
Silty Sand		120	Mohr-Coulomb	0	30			Water Surface	Custom	1	
Clay Layer 1		110	Undrained	900		FDepth	-33.6	Water Surface	Custom	0	
Clay Layer 2		110	Undrained	430		FDepth	0	Water Surface	Custom	0	
Clay Layer 3		110	Undrained	430		FDepth	18.7	Water Surface	Custom	0	
Sand		120	Mohr-Coulomb	0	30			Water Surface	Custom	1	
Silty Gravel		135	Mohr-Coulomb	0	35			Water Surface	Custom	1	
Bedrock		150	Mohr-Coulomb	10000	35			Water Surface	Custom	1	
Concrete		150	Mohr-Coulomb	6000	45			None			0
Bin Wall		140	Mohr-Coulomb	5000	45			None			0

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SLIDEINTERPRET 8.016

Project	Scarborough Eastern Trail -SEISMIC		
Analysis Description	East Embankment Station ~~125+25~~ 12+62		
Drawn By: SNP	Scale: 1:800	Company: R. W. Gillespie & Associates Inc.	
Date: 22 January 2019		File Name: East Embankment Greater Shear Strengths.slmd	



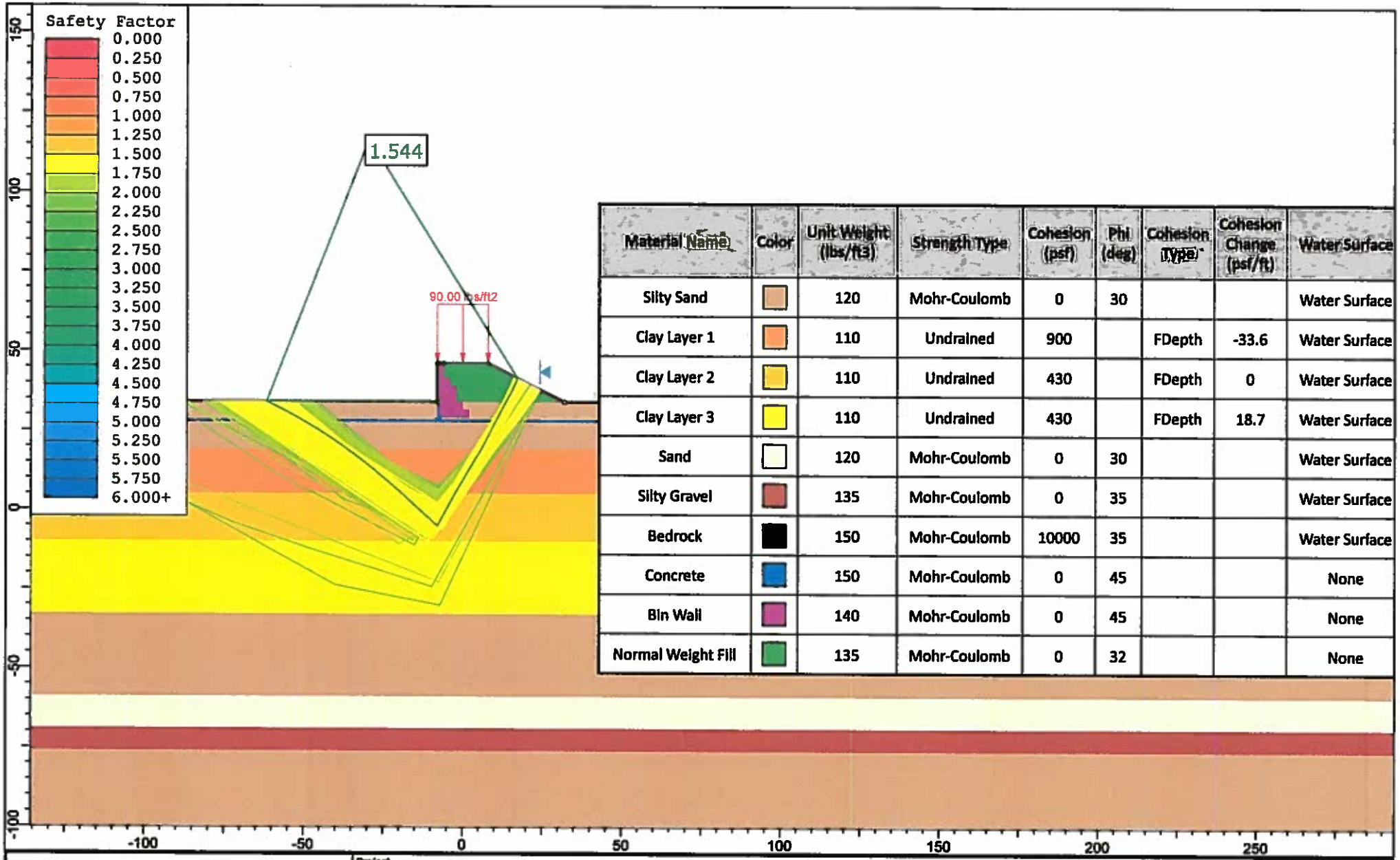
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Cohesion Change (psf/ft)	Water Surface	Hu Type	Hu	Ru
Crushed Stone		105	Mohr-Coulomb	0	45			Water Surface	Custom	1	
Silty Sand		120	Mohr-Coulomb	0	30			Water Surface	Custom	1	
Clay Layer 1		110	Undrained	900		FDepth	-33.6	Water Surface	Custom	0	
Clay Layer 2		110	Undrained	430		FDepth	0	Water Surface	Custom	0	
Clay Layer 3		110	Undrained	430		FDepth	18.7	Water Surface	Custom	0	
Sand		120	Mohr-Coulomb	0	30			Water Surface	Custom	1	
Silty Gravel		135	Mohr-Coulomb	0	35			Water Surface	Custom	1	
Bedrock		150	Mohr-Coulomb	10000	35			Water Surface	Custom	1	
Concrete		150	Mohr-Coulomb	6000	45			None			0
Bin Wall		140	Mohr-Coulomb	5000	45			None			0

By inspection, satisfies seismic

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SLIDEINTERPRET 8.016

Project	Scarborough Eastern Trail		
Analysis Description	East Embankment Station ~~125+25~~ 124+62		
Drawn By: SNP	Scale: 1:600	Company: R. W. Gillespie & Associates Inc.	
Date: 22 January 2019		File Name: East Embankment Greater Shear Strengths.slmd	

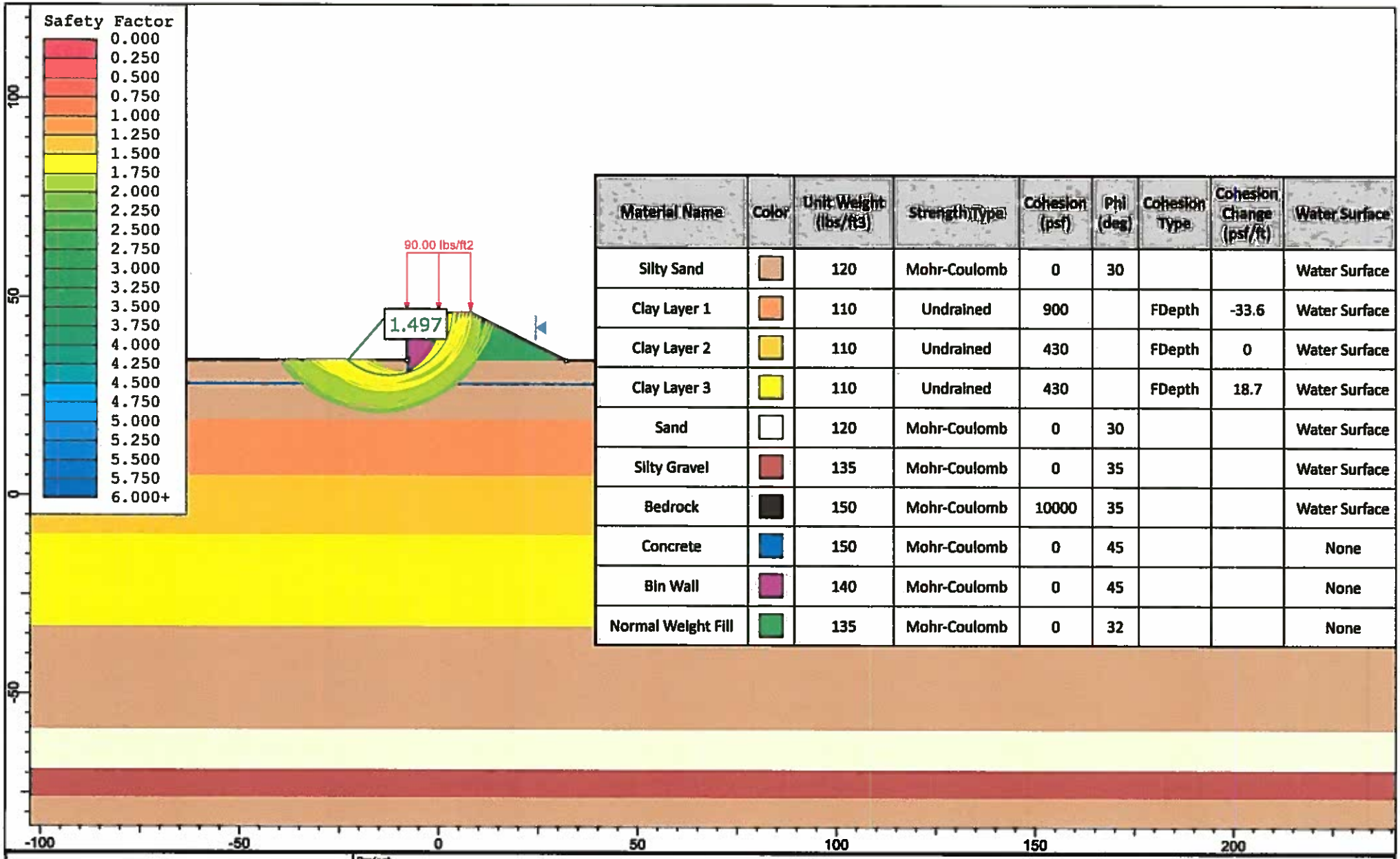


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Cohesion Change (psf/ft)	Water Surface
Silty Sand		120	Mohr-Coulomb	0	30			Water Surface
Clay Layer 1		110	Undrained	900		FDepth	-33.6	Water Surface
Clay Layer 2		110	Undrained	430		FDepth	0	Water Surface
Clay Layer 3		110	Undrained	430		FDepth	18.7	Water Surface
Sand		120	Mohr-Coulomb	0	30			Water Surface
Silty Gravel		135	Mohr-Coulomb	0	35			Water Surface
Bedrock		150	Mohr-Coulomb	10000	35			Water Surface
Concrete		150	Mohr-Coulomb	0	45			None
Bin Wall		140	Mohr-Coulomb	0	45			None
Normal Weight Fill		135	Mohr-Coulomb	0	32			None

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SLIDEINTERPRET 8.016

Project	Scarborough Eastern Trail		
Analysis Description	East Embankment Station 125+25		
Drawn By	SNP	Scale	1:500
Company	R. W. Gillespie & Associates Inc.		
Date	22 January 2019	File Name	East Embankment Greater Shear Strengths.slmd

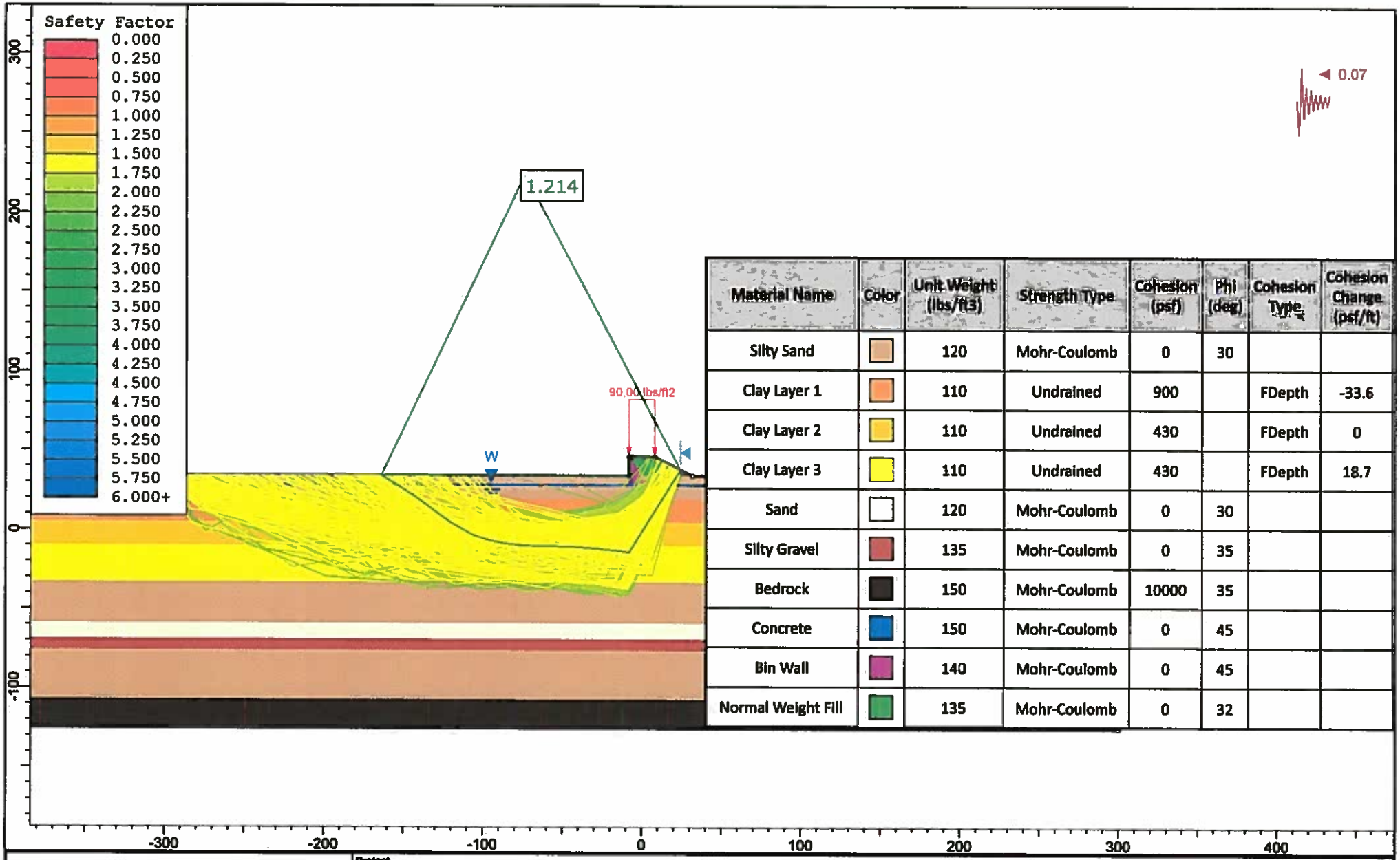


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Cohesion Change (psf/ft)	Water Surface
Silty Sand		120	Mohr-Coulomb	0	30			Water Surface
Clay Layer 1		110	Undrained	900		FDepth	-33.6	Water Surface
Clay Layer 2		110	Undrained	430		FDepth	0	Water Surface
Clay Layer 3		110	Undrained	430		FDepth	18.7	Water Surface
Sand		120	Mohr-Coulomb	0	30			Water Surface
Silty Gravel		135	Mohr-Coulomb	0	35			Water Surface
Bedrock		150	Mohr-Coulomb	10000	35			Water Surface
Concrete		150	Mohr-Coulomb	0	45			None
Bin Wall		140	Mohr-Coulomb	0	45			None
Normal Weight Fill		135	Mohr-Coulomb	0	32			None

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SLIDEINTERPRET 8.015

Project	Scarborough Eastern Trail		
Analysis Description	East Embankment Station 125+25		
Drawn By	SNP	Scale	1:400
Company	R. W. Gillespie & Associates Inc.		
Date	22 January 2019	File Name	East Embankment Greater Shear Strengths.slmd

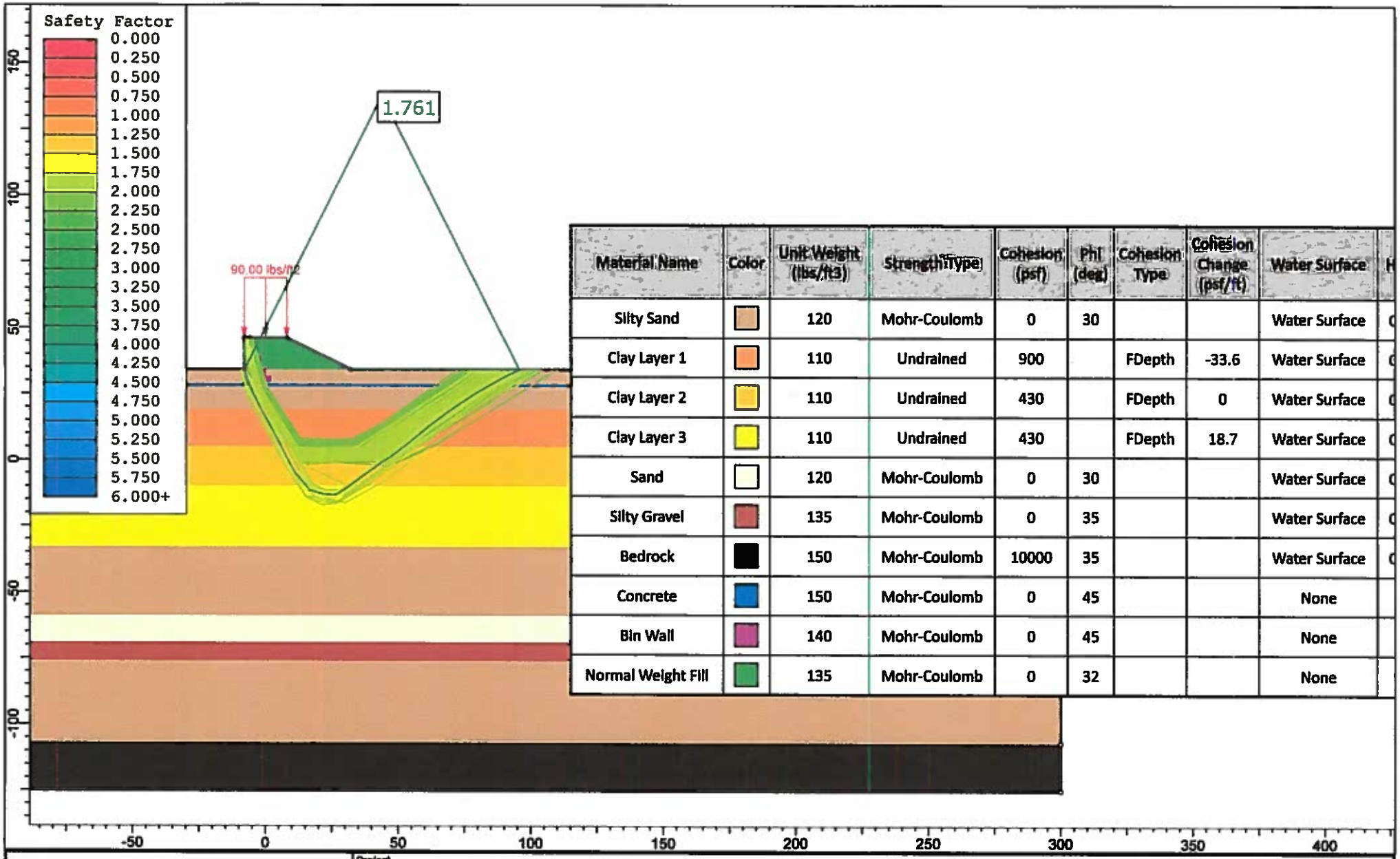


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Cohesion Change (psf/ft)
Silty Sand		120	Mohr-Coulomb	0	30		
Clay Layer 1		110	Undrained	900		FDepth	-33.6
Clay Layer 2		110	Undrained	430		FDepth	0
Clay Layer 3		110	Undrained	430		FDepth	18.7
Sand		120	Mohr-Coulomb	0	30		
Silty Gravel		135	Mohr-Coulomb	0	35		
Bedrock		150	Mohr-Coulomb	10000	35		
Concrete		150	Mohr-Coulomb	0	45		
Bin Wall		140	Mohr-Coulomb	0	45		
Normal Weight Fill		135	Mohr-Coulomb	0	32		

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SLIDEINTERPRET 8.016

Project	Scarborough Eastern Trail		
Analysis Description	East Embankment Station 125+25 SEISMIC		
Drawn By	SNP	Scale	1:1000
Company	R. W. Gillespie & Associates Inc.		
Date	22 January 2019	File Name	East Embankment Greater Shear Strengths.slmd



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Cohesion Change (psf/ft)	Water Surface	H
Silty Sand		120	Mohr-Coulomb	0	30			Water Surface	C
Clay Layer 1		110	Undrained	900		FDepth	-33.6	Water Surface	C
Clay Layer 2		110	Undrained	430		FDepth	0	Water Surface	C
Clay Layer 3		110	Undrained	430		FDepth	18.7	Water Surface	C
Sand		120	Mohr-Coulomb	0	30			Water Surface	C
Silty Gravel		135	Mohr-Coulomb	0	35			Water Surface	C
Bedrock		150	Mohr-Coulomb	10000	35			Water Surface	C
Concrete		150	Mohr-Coulomb	0	45			None	
Bin Wall		140	Mohr-Coulomb	0	45			None	
Normal Weight Fill		135	Mohr-Coulomb	0	32			None	

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SLIDEINTERPRET 8.016

Project	Scarborough Eastern Trail		
Analysis Description	East Embankment Station 125+25		
Drawn By	SNP	Scale	1:600
Company	R. W. Gillespie & Associates Inc.		
Date	22 January 2019	File Name	East Embankment Greater Shear Strengths.slmd

R.W. Gillespie & Associates

Liquefaction Analysis Results

R.W. Gillespie & Associates

Job Title : Eastern Trail
Job No. : 0600-102
Client : HNTB
Address : Scarborough, Maine
Calculated By : EJW Reviewed By :

Borehole : BB-SPC-203
Coordinates:
X = 0
Y = 0
Z = 0

Input Assumption	Setting
Field Test Type :	Standard Penetration Test (SPT)
Apply All Corrections to SPT?	True
Groundwater Level (ft) =	8
Earthquake Magnitude M =	7
Magnitude Scaling Factor (MSF) :	1.19 (Idriss, 1997 -NCEER)
Fines Content Correction :	(according to user settings)
Depth Reduction Factor (Rd) :	NCEER, 1997
Relative Density (Dr) Estimation :	Idriss & Boulanger, 2003
Site Topography :	Gently Sloped : 0.5 %
Ground Improvement Feature :	None
Peak Ground Acceleration PGA (g) =	0.14

CRR Formula	Selected?
NCEER Workshop (1997)	True
Boulanger & Idriss (2014)	True
Vancouver Task Force (2007)	False
Cetin et al. (2004)	False
Chinese Code	False
Seed et al. (1983)	True
Japanese Highway Bridge Code	False
Tokimatsu & Yoshimi (1983)	False
Shibata (1981)	False
Kokusho et al. (1983)	False

Depth (ft)	SPT Blow Counts (N)
1	4
3	26
6	15
8	13
11	5
13	7
16	0
65	0
66	10
71	24
76	28
81	33
86	26
91	20
96	24
100	47

Layer Thickness (ft)	Soil Type	Unit Weight (lb/ft3)	Fines Content (%)	D50 (mm)	Check Liquefaction/ S	Su (ksf)
8	Sand	120	7	2	True	0
7	Sand	120	40	2	True	0
51.5	Clay	110	85	0.002	False	0
37	Sand	125	25	2	True	0

LIQUEFACTION ASS
BASEMENT
W/BT ABUTMENT
600-102

NOT liquefaction susceptible for design event
(FS > 1)

Depth (ft)	Rd	Rd_J&B	Overburden Stress (ksf)		Fines Content (%)	SPT Test				Relative Density Dr (%)	Simp. CSR	CSR_J&B	CRR7.5		
			Total	Effective		N	Co	Cn	N1(60)				NCEER Worksho	Boulange r & Idriss	Seed et al. (1985)
1	0.998	1.004	0.12	0.12	7	4	1.15	1.7	8	41.7	0.091	0.091	-	-	-
3	0.993	0.998	0.37	0.37	7	26	1.15	1.7	51	100	0.09	0.091	-	-	-
6	0.986	0.989	0.73	0.73	7	15	1.15	1.62	28	78.4	0.09	0.09	-	-	-
8	0.981	0.981	0.98	0.98	7	13	1.15	1.35	20	66.8	0.089	0.089	0.22	0.22	0.32
11	0.974	0.97	1.35	1.16	40	5	1.22	1.21	7	54.8	0.103	0.103	0.15	0.15	0.22
13	0.97	0.962	1.59	1.28	40	7	1.32	1.12	10	61.6	0.11	0.109	0.19	0.18	0.27
16	0.963	0.948	1.95	1.45	85	0	1.38	1.03	0	33	0.118	0.116	-	-	-
65	0.645	0.691	7.45	3.89	85	0	1.53	0.77	0	33	0.112	0.12	-	-	-
66	0.637	0.686	7.56	3.94	85	10	1.53	0.77	12	54.4	0.111	0.12	-	-	-
71	0.596	0.663	8.19	4.26	25	24	1.54	0.74	27	87.1	0.104	0.116	0.69	0.77	0.73
76	0.559	0.642	8.83	4.58	25	28	1.54	0.72	31	92	0.098	0.113	0.68	0.77	0.8
81	0.546	0.623	9.46	4.91	25	33	1.54	0.7	36	97.8	0.096	0.109	0.67	0.76	0.8
86	0.534	0.606	10.1	5.23	25	26	1.54	0.68	27	86.9	0.094	0.107	0.66	0.76	0.72
91	0.522	0.592	10.74	5.56	25	20	1.54	0.66	20	76.7	0.092	0.104	0.26	0.33	0.45
96	0.51	0.579	11.38	5.88	25	24	1.54	0.65	24	82	0.09	0.102	0.64	0.52	0.56
100	0.5	0.57	11.89	6.14	25	47	1.54	0.63	46	100	0.088	0.1	0.63	0.75	0.8

Depth (ft)	CRR7.5 (ave)	Safety Factor			Safety Factor	Probability of Liquefaction PL(%)	
		NCEER Worksho	Boulanger &	Seed et al.		Youd & Noble	Cetin et al. 2004
1	-	-	-	-	-	-	-
3	-	-	-	-	-	-	-
6	-	-	-	-	-	-	-
8	0.25	2.97	2.95	4.22	**3.38**	20	0
11	0.17	1.73	1.76	2.54	**2.01**	30.4	0
13	0.21	2.05	2	2.93	**2.33**	24.5	0
16	-	-	-	-	-	-	-
65	-	-	-	-	-	-	-
66	-	-	-	-	-	-	-
71	0.73	7.9	7.91	8.3	**8.04**	14	0
76	0.75	8.27	8.12	9.75	**8.71**	5.8	0
81	0.74	8.3	8.33	9.95	**8.86**	1.6	0
86	0.71	8.35	8.52	9.14	**8.67**	13.6	0
91	0.35	3.38	3.8	5.79	**4.32**	10.7	0
96	0.57	8.48	6.09	7.43	**7.33**	17.5	0
100	0.73	8.55	8.96	10.84	**9.45**	0.1	0

R.W. Gillespie & Associates

Liquefaction Analysis Results

R.W. Gillespie & Associates

Job Title : Eastern Trail
Job No. : 0600-102
Client : HNTB
Address : Scarborough, Maine
Calculated By : EJW Reviewed By :

Borehole : BB-SPC-201
Coordinates:
X = 0
Y = 0
Z = 0

Input Assumption	Setting
Field Test Type :	Standard Penetration Test (SPT)
Apply All Corrections to SPT?	True
Groundwater Level (ft) =	7
Earthquake Magnitude M =	7
Magnitude Scaling Factor (MSF) :	1.19 (Idriss, 1997 -NCEER)
Fines Content Correction :	(according to user settings)
Depth Reduction Factor (Rd) :	NCEER, 1997
Relative Density (Dr) Estimation :	Idriss & Boulanger, 2003
Site Topography :	Gently Sloped : 0.5 %
Ground Improvement Feature :	None
Peak Ground Acceleration PGA (g) =	0.14

CRR Formula	Selected?
NCEER Workshop (1997)	True
Boulanger & Idriss (2014)	False
Vancouver Task Force (2007)	False
Cetin et al. (2004)	False
Chinese Code	False
Seed et al. (1983)	True
Japanese Highway Bridge Code	False
Tokimatsu & Yoshimi (1983)	False
Shibata (1981)	False
Kokusho et al. (1983)	False

Depth (ft)	SPT Blow Counts (N)
1	22
6	2
11	3
16	0
56	8
61	12
66	12
71	5
76	51
81	60
86	43
91	40
96	100
100	95

Layer Thickness (ft)	Soil Type	Unit Weight (lb/ft3)	Fines Content (%)	D50 (mm)	Check Liquefaction/ S	Su (ksf)
6	Sand	120	35	2	True	0
8	Sand	120	35	2	True	0
41	Clay	110	85	0.002	False	550
10	Sand	125	25	2	True	0
5	Sand	125	5	2	True	0
30	Sand	125	25	2	True	0

0600-102

LIQUEFACTION ASSESSMENT

EAST ABUTMENT

Not liquefaction susceptible for design event.

(FS > 1)

Depth (ft)	Rd	Rd_I&B	Overburden Stress (ksf)		Fines Content (%)	SPT Test				Relative Density Dr (%)	Simpl CSR	CSR_I&B	CRR7.5		CRR7.5 (ave)
			Total	Effective		N	Co	Cn	N1(60)				NCEER Worksho	Seed et al. (1983)	
1	0.998	1.004	0.12	0.12	35	22	1.15	1.7	43	100	0.091	0.091	-	-	-
6	0.986	0.989	0.73	0.73	35	2	1.15	1.62	4	45.4	0.09	0.09	-	-	-
11	0.974	0.97	1.35	1.1	35	3	1.22	1.25	5	47.8	0.109	0.108	0.11	0.18	0.15
16	0.963	0.948	1.94	1.38	85	0	1.38	1.07	0	33	0.123	0.122	-	-	-
56	0.718	0.736	6.44	3.38	25	8	1.53	0.81	10	57.7	0.124	0.127	0.15	0.24	0.2
61	0.678	0.71	7.08	3.71	25	12	1.53	0.78	14	66.5	0.118	0.123	0.2	0.31	0.26
66	0.637	0.686	7.72	4.03	5	12	1.53	0.76	14	55.1	0.111	0.119	0.13	0.22	0.18
71	0.596	0.663	8.36	4.36	25	5	1.54	0.74	6	48	0.104	0.116	0.1	0.18	0.14
76	0.559	0.642	8.99	4.69	25	51	1.54	0.72	56	100	0.098	0.112	0.68	0.8	0.74
81	0.546	0.623	9.63	5.01	25	60	1.54	0.7	64	100	0.096	0.109	0.66	0.8	0.73
86	0.534	0.606	10.27	5.34	25	43	1.54	0.68	45	100	0.094	0.106	0.65	0.8	0.73
91	0.522	0.592	10.91	5.66	25	40	1.54	0.66	41	100	0.092	0.104	0.64	0.8	0.72
96	0.51	0.579	11.54	5.99	25	100	1.54	0.64	99	100	0.089	0.102	0.63	0.8	0.72
100	0.5	0.57	12.05	6.25	25	95	1.54	0.63	92	100	0.088	0.1	0.63	0.8	0.71

Depth (ft)	Safety Factor		Safety Factor	Probability of Liquefaction PL(%)	
	NCEER Workshop	Seed et al.		Youd & Noble	Calif et al. 2004
1	-	-	-	-	-
6	-	-	-	-	-
11	1.25	1.99	1.62	38.4	0.4
16	-	-	-	-	-
56	1.45	2.29	1.87	29.9	3.5
61	1.99	3.18	2.59	21.1	0
66	1.42	2.38	1.9	30.9	34.6
71	1.13	2.09	1.61	34.3	55.7
76	8.26	9.78	9.02	0	0
81	8.29	9.99	9.14	0	0
86	8.34	10.2	9.27	0.1	0
91	8.4	10.43	9.41	0.4	0
96	8.47	10.67	9.57	0	0
100	8.53	10.87	9.7	0	0

MECHANICALLY STABILIZED EARTH – ALLOWABLE BEARING PRESSURE

R.W. Gillespie & Associates
20 Pomerleau Street, Suite100 Biddeford, Maine 04005 (207) 286-8008
Fax: (207) 710-000 Email: rwg-a@rwg-a.com

NAME: SNP
DATE: 5 DECEMBER 2018
JOB NAME: EASTERN SCARBOROUGH TR
JOB NUMBER: 0600-102
SHEET 1 OF 2

BEARING RESISTANCE $\phi_f = 32°$ $C = 500$ psf AASHTO LRFD

$q_1 = \frac{1}{2} N_{cm} + \gamma D_f N_{qm} C_{wq} + 0.5 \gamma B N_{\gamma m} C_{w\gamma}$ [Eq. 10.6.3.1.2a-1, 0-15' BELOW GROUND SAND]

($\frac{1}{2}$ → 0; D_f → 0)

$N_{\gamma m} = N_\gamma S_\gamma d_\gamma$ (d_γ → 1)

$N_\gamma = 30.2$ [FOR $\phi_f = 32°$] ✓ Table 10.6.3.1.2a-1

$S_\gamma = 1 - 0.4\left(\frac{B}{L}\right) = 1 - 0.4\left(\frac{15}{85}\right)$

$S_\gamma = 0.93$ ✓

$N_{\gamma m} = 30.2 \times 0.93$

$N_{\gamma m} = 28.1$ ✓

$q_1 = 0.5 \times 120 \text{ pcf} \times 15' \times 28.1 \times 0.5$

$q_1 = 12{,}640$ psf ✓

$q_1 = 6{,}100$ psf for B=7

$q_2 = \frac{1}{2} N_{cm} + \gamma D_f N_{qm} C_{wq} + 0.5 \gamma B N_{\gamma m} C_{w\gamma}$ [SILTY CLAY]

($N_{\gamma m}$ → 0)

$N_{cm} = N_c S_c d_c$ (d_c → 1)

$N_c = 5.14$ [FOR $\phi_f = 0°$] ✓

$S_c = 1 + \left(\frac{B}{5L}\right) = 1 + \left(\frac{15}{5 \cdot 85}\right)$

$S_c = 1.035$ ✓

$N_{cm} = 5.14 \times 1.035$

$N_{cm} = 5.32$ ✓

$N_{qm} = N_q S_q d_q$; $N_q = 1.0$ $S_q = 1.0$ $d_q = 1.0$

$N_{qm} = 1.0$

EJK

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R.W. Gillespie & Associates
20 Pomerleau Street, Suite100 Biddeford, Maine 04005 (207) 286-8008
Fax: (207) 710-000 Email: rwg-a@rwg-a.com

NAME: SNP
DATE: 5 DECEMBER 2018
JOB NAME: EASTERN SCARBOROUGH TR.
JOB NUMBER: 06000-102
SHEET 2 OF 2

BEARING PRESSURE CONT.

$q_2 = 500\ psf \times 5.32 + 110\ pcf \times 15' \times 1.0 \times 0.5 = 2,660 + 825$

$q_2 = 3,485\ psf$ ✓

$H_{crit} = \frac{(3B)\ln\left(\frac{q_1}{q_2}\right)}{2(1+B/L)} = \frac{(3\times 15')\ln\left(\frac{12,640\ psf}{3,485\ psf}\right)}{2(1+15/85)} = \frac{58}{2.35} = 24.6\ ft$ — Eq. 10.6.3.1.2d

For B=7': $H_{crit} = 13'$, $\frac{27}{2.1}$ (Shear in Sand)

$q_n = \left[q_2 + \left(\frac{1}{K}\right)\left(c_1' N_{co} + \Phi_1'\right)\right] e^{2\left[1+\left(\frac{B}{L}\right)\right] K \tan\Phi_1' \left(\frac{H}{B}\right)} - \left(\frac{1}{K}\right) c_1' N_{co} + \Phi_1'$ (with $c_1' = 0$)

— Eq. 10.6.3.1.2f-1

$K = \frac{1-\sin^2\Phi_1'}{1+\sin^2\Phi_1'} = \frac{1-\sin^2(32)}{1+\sin^2(32)} = \frac{0.72}{1.28}$ ✓ $= 0.5625$ ✓

$q_n = \left[3,485\ psf + 0\right] e^{2\left[1+\frac{15}{85}\right] 0.5625 \tan 32 \left(\frac{14'}{15}\right)^{\frac{H}{B}}} - 0$

$3,485\ (2.16)$

$q_n = 7,540$ ✓

For B=7': $q_n = 16,100\ psf$

$q_R = \Phi_b\, q_n$ — Per Table 11.5.7.1 for MSE Walls

$q_R = 0.65 \times 7,540\ psf$

$q_R = 4,900\ psf$ ✓

$q_r = 0.65(6,100) = 3,700\ psf$

Chkd: EJW ✓

Bearing Resistance	Reinforcement Width, 15' to 7' (H_N = 2')	7' (H_N = 10')	
Strength: $q_{nominal}$	7,500	6,100	Φ=1
	6,800	5,500	Φ=0.9
Service: q_r	3,400	2,700	Φ=0.65

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LATERAL MSE WALL DISPLACEMENT

R.W. Gillespie & Associates
20 Pomerleau Street, Suite100 Biddeford, Maine 04005 (207) 286-8008
Fax: (207) 710-000 Email: rwg-a@rwg-a.com

NAME: BSK
DATE: 22 JAN 2018
JOB NAME: BADGER'S TRAIL
JOB NUMBER: 0602-102
SHEET 1 OF 1

Lateral MSE Wall Displacement

USE AASHTO Figure C11.10.4.2-1

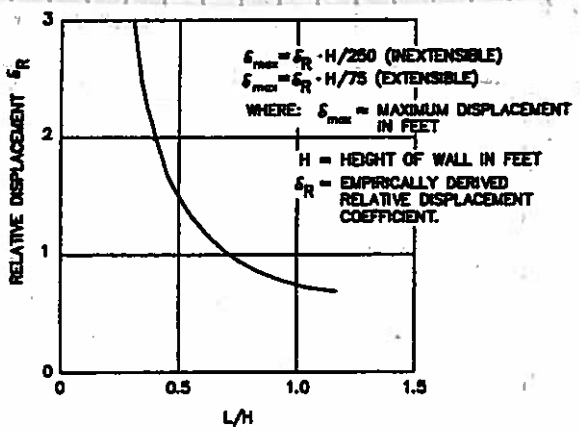
Minimum required reinforcement length = 70% of wall height.

$\therefore \frac{L}{H} = 0.7 \longrightarrow \delta_R = 1.0$

$\delta_{max} = \frac{H}{250} \delta_R$ — H = Wall height in feet for inextensible reinforcement (req'd by MaineDOT)

H = 15' ± at max. height

$\delta_{max} = \frac{15}{250} = 0.06$ feet ≈ 0.7 inches

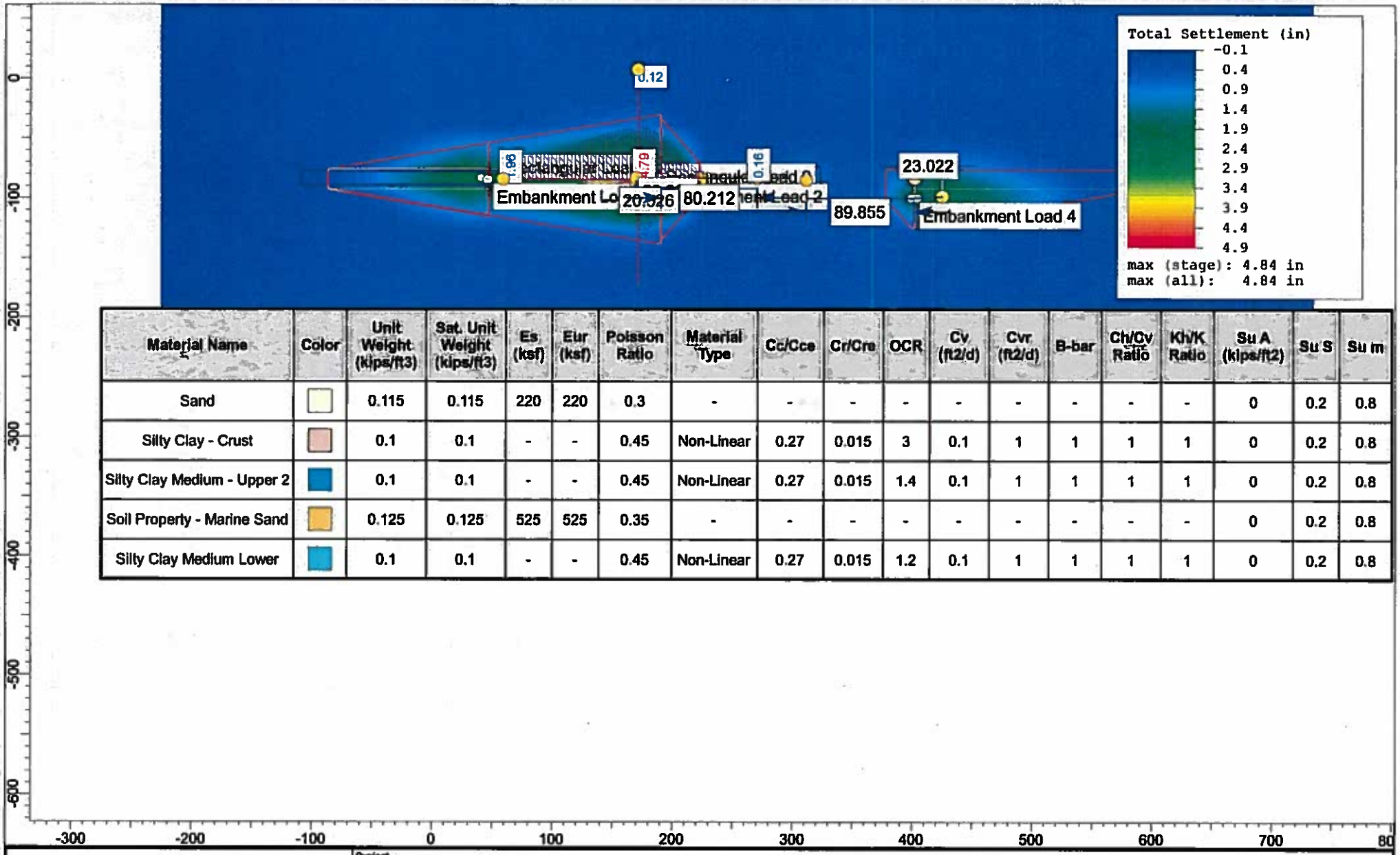


Based on 20 ft. high walls, relative displacement increases approximately 25% for every 400 psf of surcharge. Experience indicates that for higher walls, the surcharge effect may be greater.

Note: This figure is only a guide. Actual displacement will depend, in addition to the parameters addressed in the figure, on soil characteristics, compaction effort, and contractor workmanship.

Figure C11.10.4.2-1—Empirical Curve for Estimating Anticipated Lateral Displacement during Construction for MSE Walls

SETTLEMENT ANALYSIS OF EMBANKMENTS

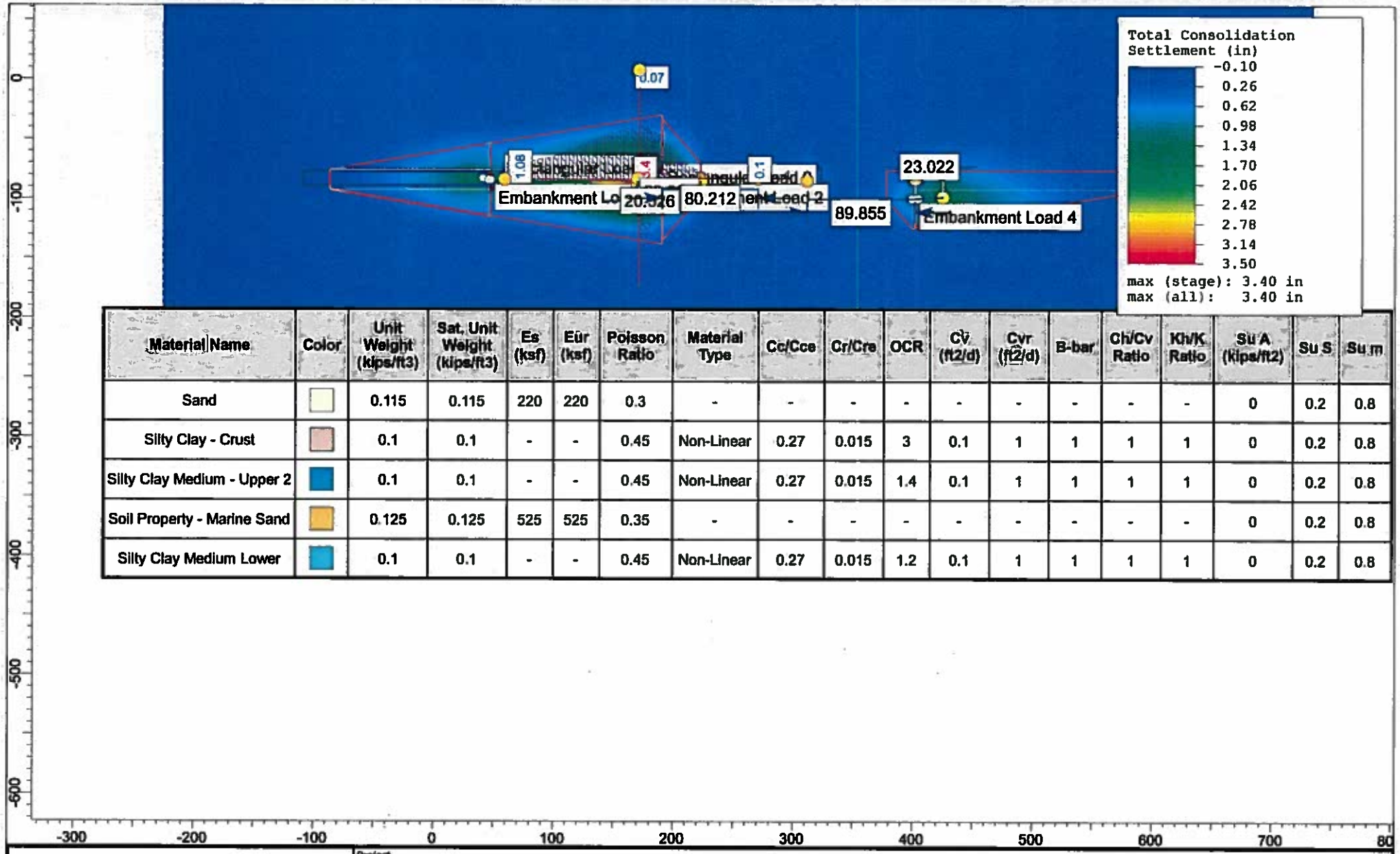


Material Name	Color	Unit Weight (kips/ft3)	Sat. Unit Weight (kips/ft3)	Es (ksf)	Eur (ksf)	Poisson Ratio	Material Type	Cc/Cce	Cr/Cre	OCR	Cv (ft2/d)	Cvr (ft2/d)	B-bar	Ch/Cv Ratio	Kh/K Ratio	Su A (kips/ft2)	Su S	Su m
Sand		0.115	0.115	220	220	0.3	-	-	-	-	-	-	-	-	-	0	0.2	0.8
Silty Clay - Crust		0.1	0.1	-	-	0.45	Non-Linear	0.27	0.015	3	0.1	1	1	1	1	0	0.2	0.8
Silty Clay Medium - Upper 2		0.1	0.1	-	-	0.45	Non-Linear	0.27	0.015	1.4	0.1	1	1	1	1	0	0.2	0.8
Soil Property - Marine Sand		0.125	0.125	525	525	0.35	-	-	-	-	-	-	-	-	-	0	0.2	0.8
Silty Clay Medium Lower		0.1	0.1	-	-	0.45	Non-Linear	0.27	0.015	1.2	0.1	1	1	1	1	0	0.2	0.8

rocscience

SETTLE3D 4.016

Project	Eastern Trail Pan-Am Crossing
Analysis Description	Bridge Approach Embankments
Drawn By	EJW
Company	R.W. Gillespie & Associates, Inc.
Date	3/10/2018, 6:27:54 AM
File Name	30 Nov 2018.s3z



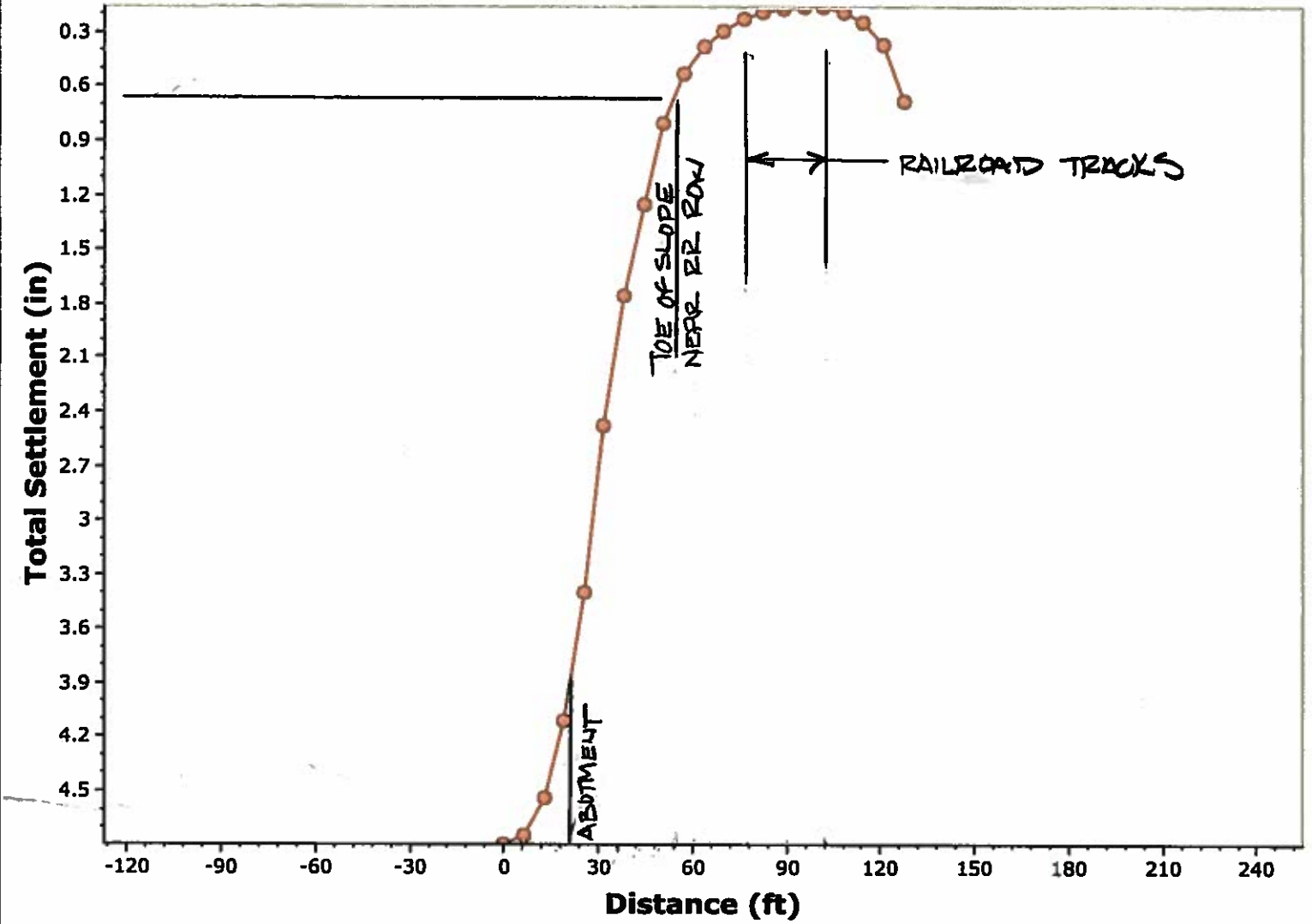
Material Name	Color	Unit Weight (kips/ft3)	Sat. Unit Weight (kips/ft3)	Es (ksf)	Eur (ksf)	Poisson Ratio	Material Type	Cc/Cce	Cr/Cre	OCR	Cv (ft2/d)	Cvr (ft2/d)	B-bar	Ch/Cv Ratio	Kh/K Ratio	Su A (kips/ft2)	Su S	Su m
Sand		0.115	0.115	220	220	0.3	-	-	-	-	-	-	-	-	-	0	0.2	0.8
Silty Clay - Crust		0.1	0.1	-	-	0.45	Non-Linear	0.27	0.015	3	0.1	1	1	1	1	0	0.2	0.8
Silty Clay Medium - Upper 2		0.1	0.1	-	-	0.45	Non-Linear	0.27	0.015	1.4	0.1	1	1	1	1	0	0.2	0.8
Soil Property - Marine Sand		0.125	0.125	525	525	0.35	-	-	-	-	-	-	-	-	-	0	0.2	0.8
Silty Clay Medium Lower		0.1	0.1	-	-	0.45	Non-Linear	0.27	0.015	1.2	0.1	1	1	1	1	0	0.2	0.8

rocscience

SETTLE3D 4.016

Project	Eastern Trail Pan-Am Crossing
Analysis Description	Bridge Approach Embankments
Drawn By	EJW
Company	R.W. Gillespie & Associates, Inc.
Date	3/10/2018, 6:27:54 AM
File Name	30 Nov 2018.s3z

Distance vs. Total Settlement



Settlement Profile Through Centerline of West Embankment and Pier

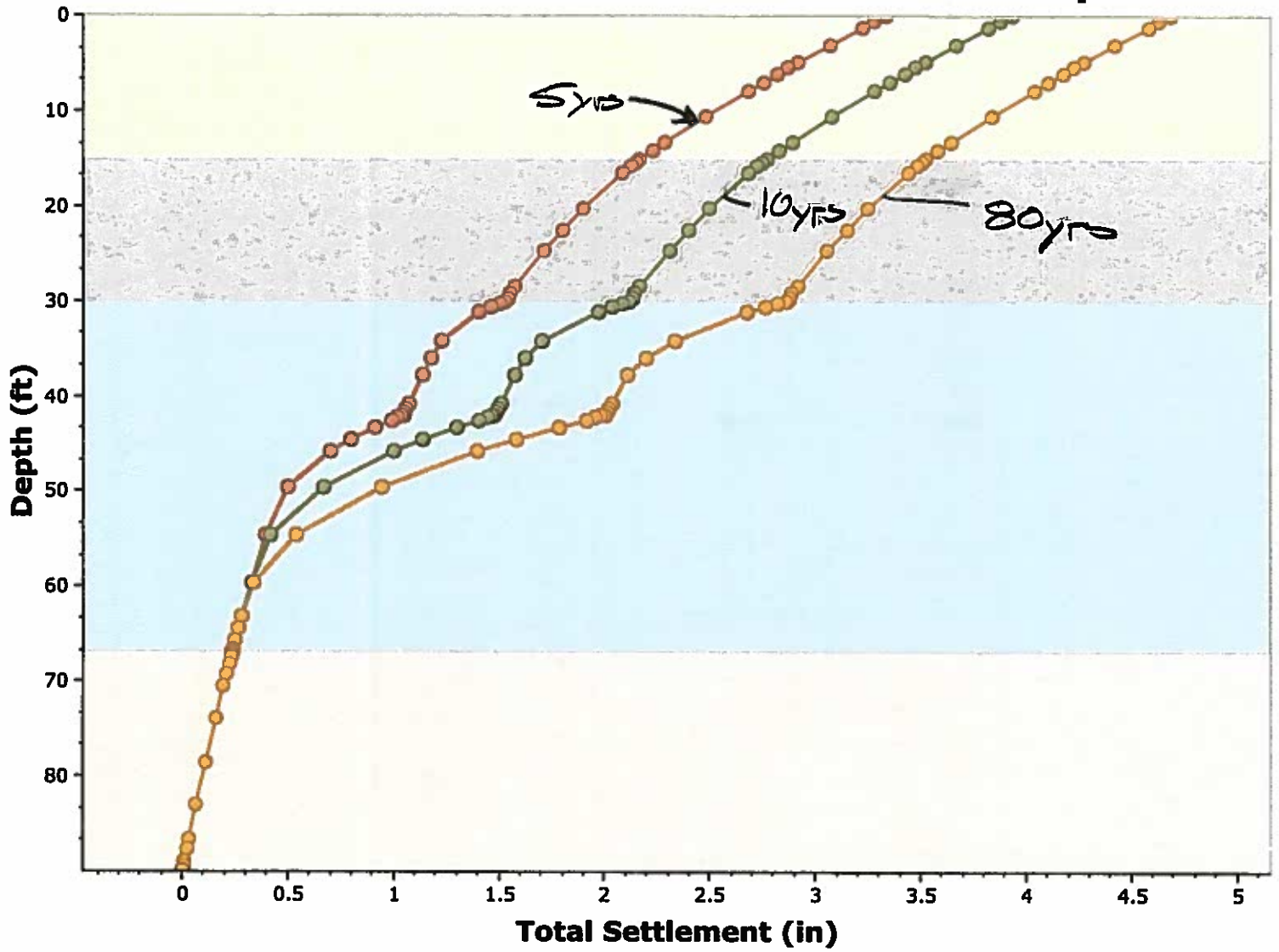
Query Line 5 (Stage 8 = 29200 d)

Reference Stage: None
Total Settlement at Depth = 0.5 ft

Project	Eastern Trail Pan-Am Crossing		
Analysis Description	Bridge Approach Embankments		
Drawn By	EJW	Company	R.W. Gillespie & Associates, Inc.
Date	3/10/2018, 6:27:54 AM	File Name	30 Nov 2018.s3z

SETTLE3D 4.016

Total Settlement vs. Depth



Point of Maximum Settlement at West Embankment

Query Point 3 (Stage 4 = 1825 d)
Query Point 3 (Stage 5 = 3650 d)
Query Point 3 (Stage 8 = 29200 d)

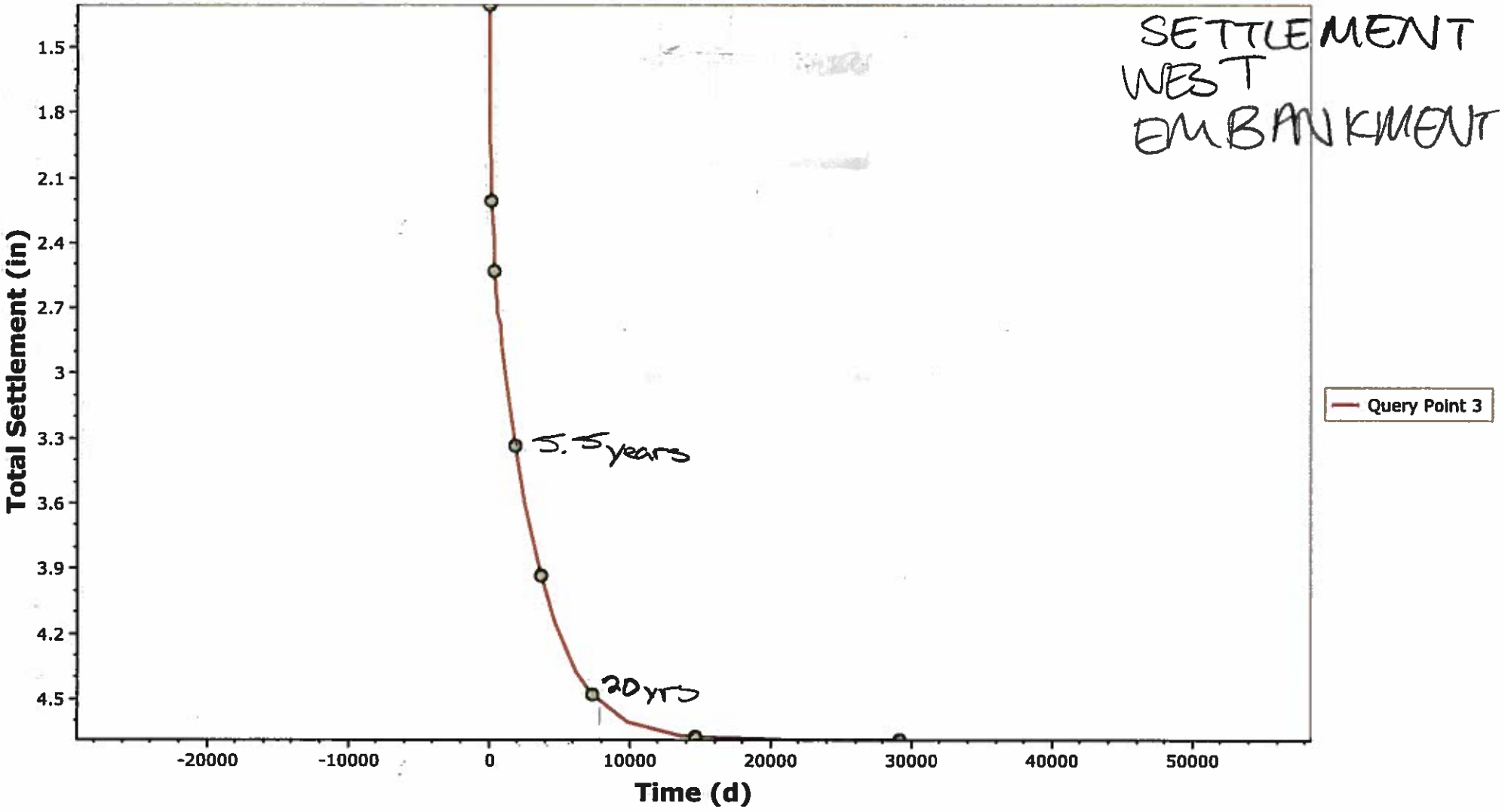
Reference Stage: None

rocscience	Project	Eastern Trail Pan-Am Crossing	
	Analysis Description	Bridge Approach Embankments	
	Drawn By	EJW	Company: R.W. Gillespie & Associates, Inc.
	Date	3/10/2018, 6:27:54 AM	File Name: 30 Nov 2018.s3z

SETTLE3D 4.016

Time vs. Total Settlement

POINT OF MAX SETTLEMENT WEST EMBANKMENT

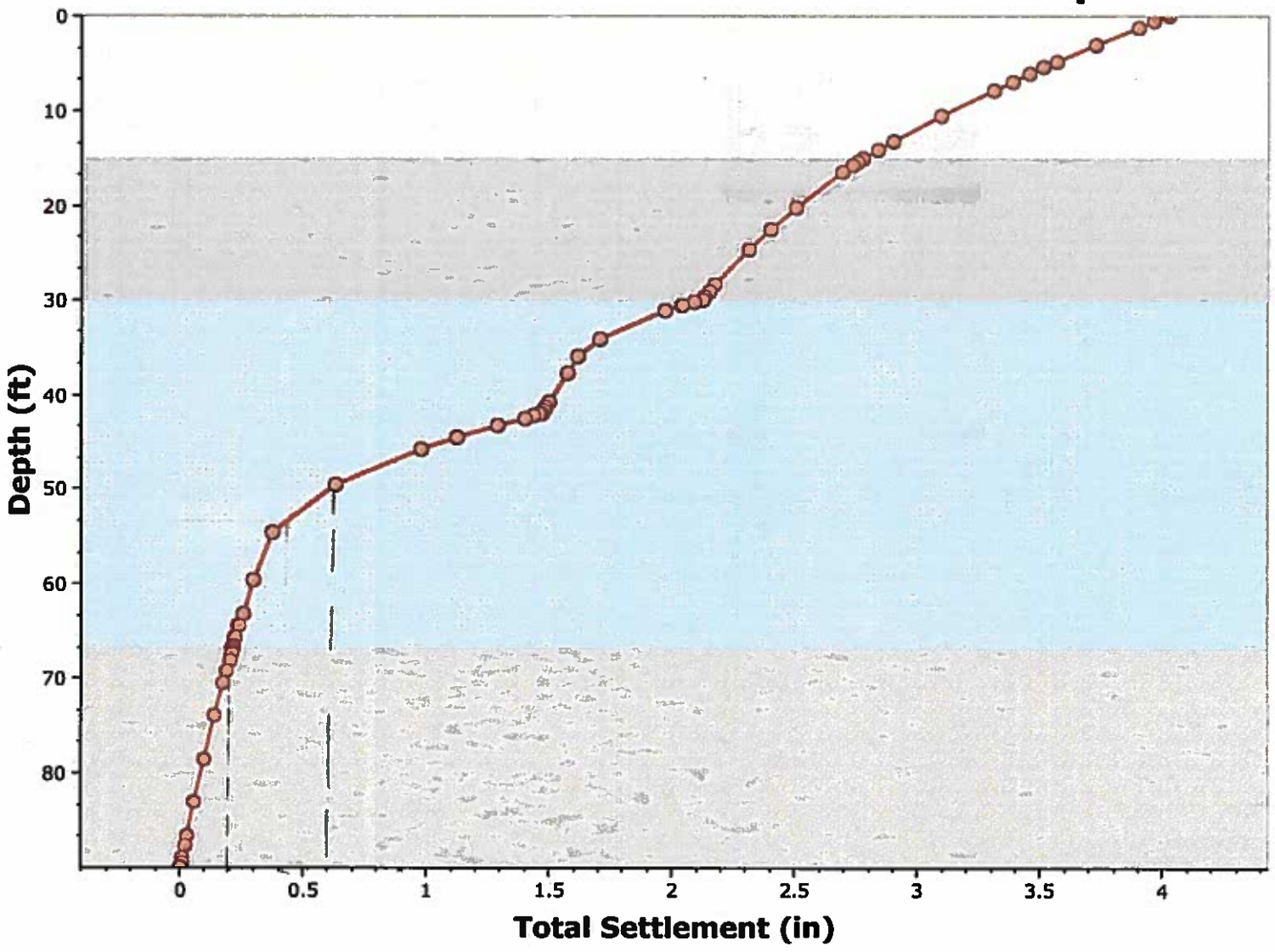


Reference Stage: None
Total Settlement at Depth = 0 ft

Project	Eastern Trail Pan-Am Crossing		
Analysis Description	Bridge Approach Embankments		
Drawn By	EJW	Company	R.W. Gillespie & Associates, Inc.
Date	3/10/2018, 6:27:54 AM	File Name	30 Nov 2018.s3z

SETTLE3D 4.016

Total Settlement vs. Depth



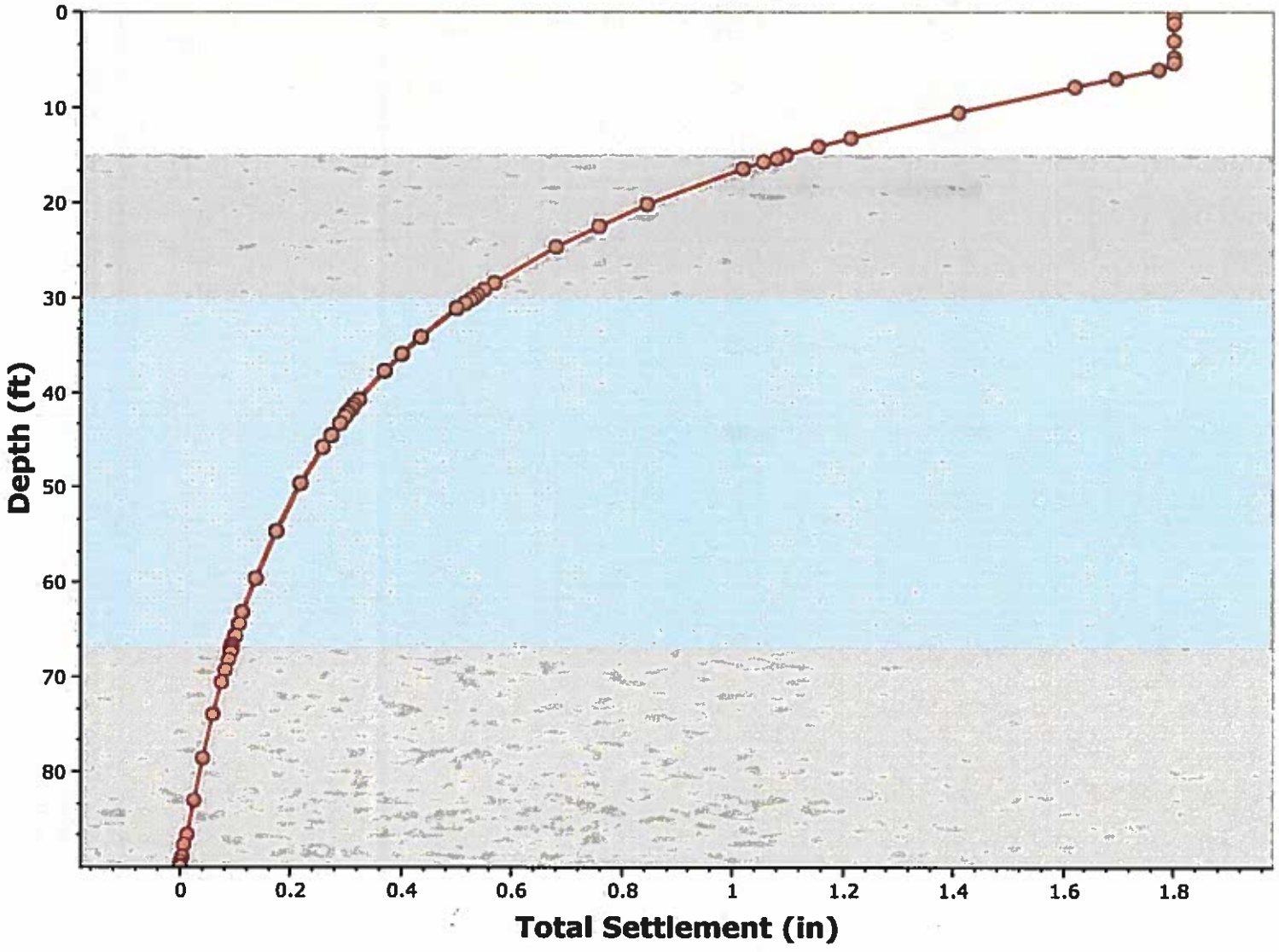
WEST ABUTMENT

Reference Stage: None

rocscience	Project: Eastern Trail Pan-Am Crossing		
	Analysis Description: Bridge Approach Embankments		
	Drawn By: EJW	Company: R.W. Gillespie & Associates, Inc.	
	Date: 3/10/2018, 6:27:54 AM	File Name: 30 Nov 2018.s3z	

SETTLE3D 4.016

Total Settlement vs. Depth



PIER FOOTING (If soil supported)

Query Point 7 (Stage 8 = 29200 d)

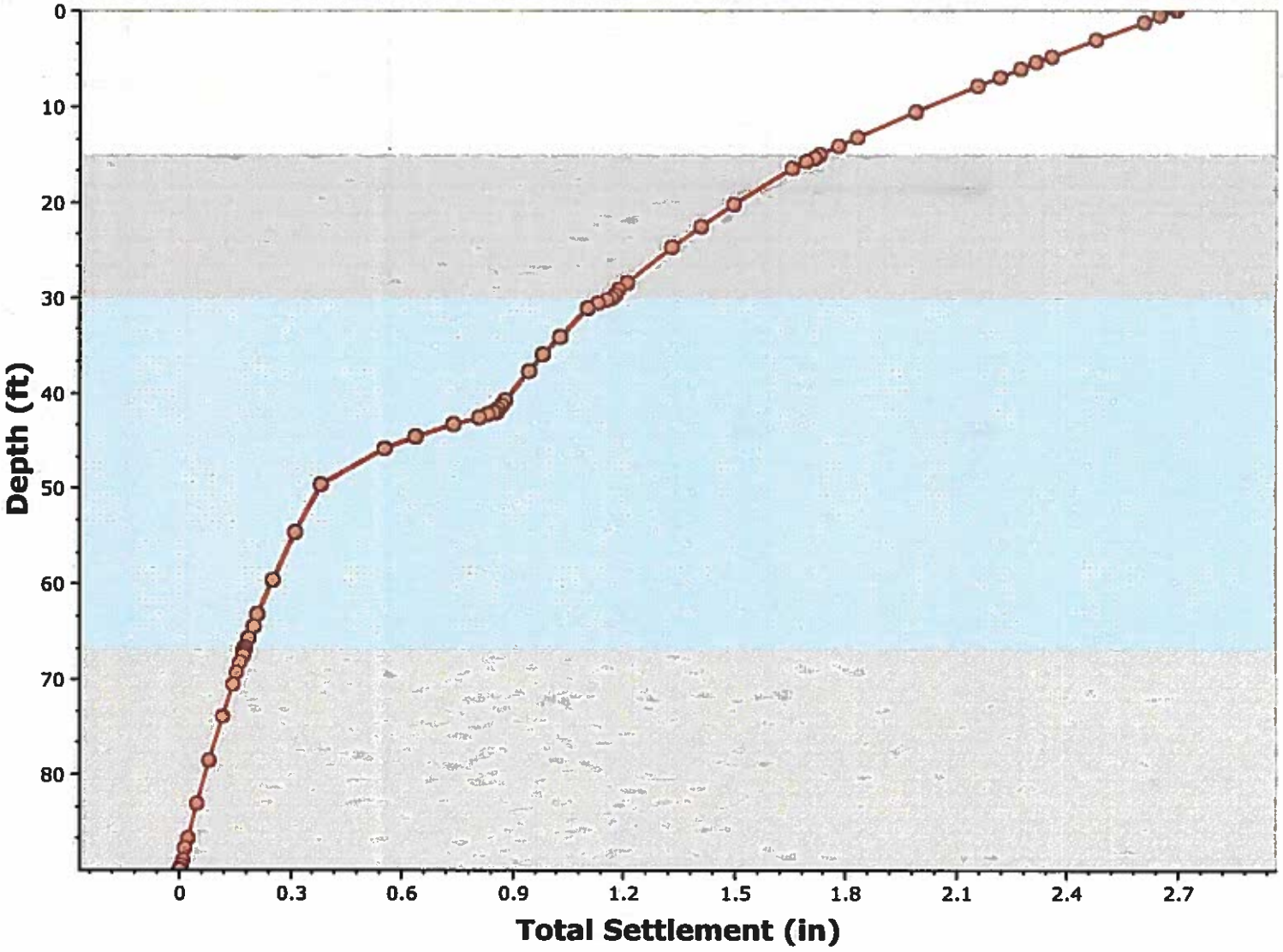
80 yrs

Reference Stage: None

Project	Eastern Trail Pan-Am Crossing		
Analysis Description	Bridge Approach Embankments		
Drawn By	EJW	Company	R.W. Gillespie & Associates, Inc.
Date	3/10/2018, 6:27:54 AM	File Name	30 Nov 2018.s3z

SETTLE3D 4.016

Total Settlement vs. Depth



POINT OF MAX SETTLEMENT EAST EMBANKMENT

Query Point 5 (Stage 8 = 29200 d)

80yrs

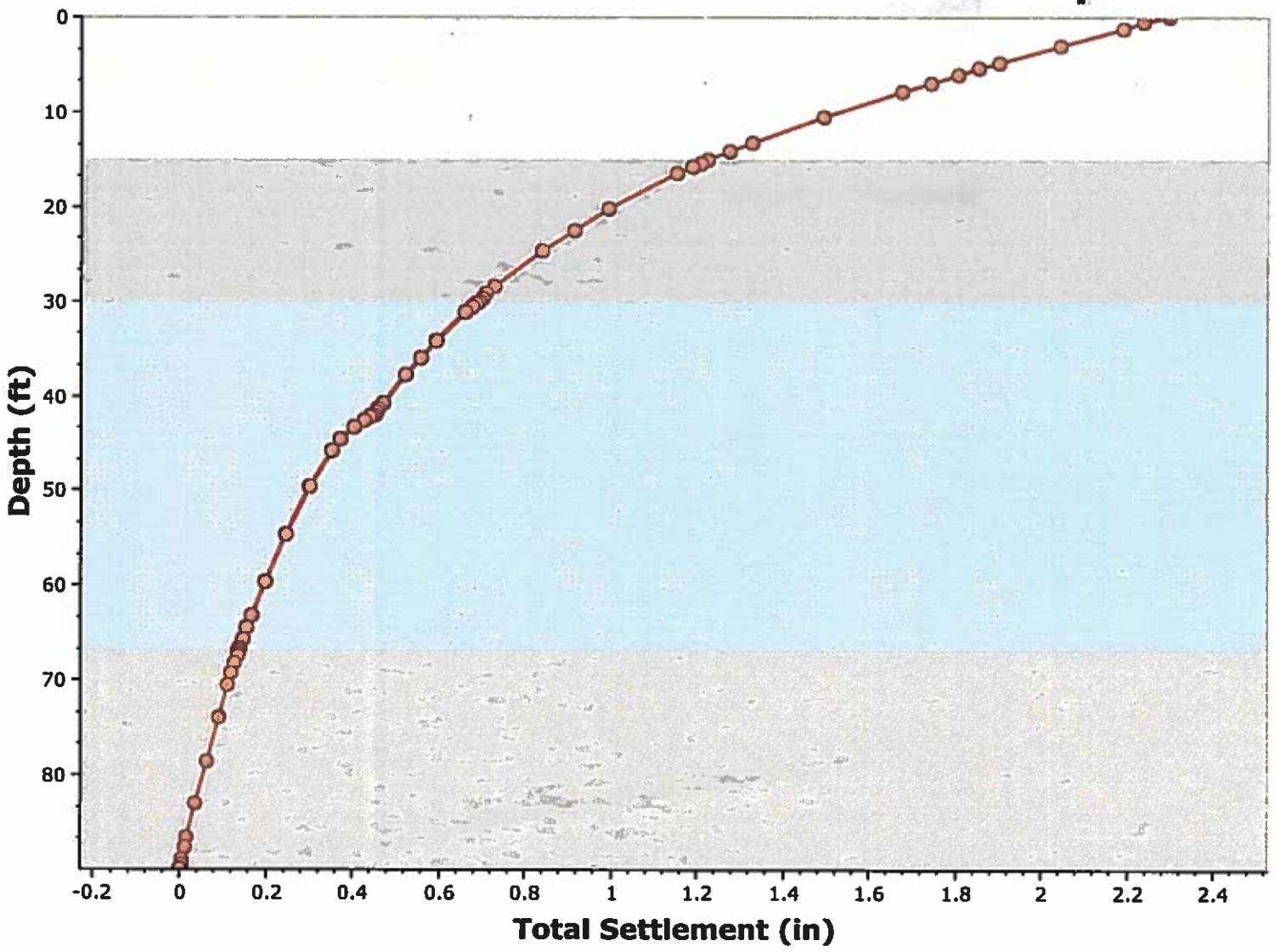
Reference Stage: None

Project	Eastern Trail Pan-Am Crossing		
Analysis Description	Bridge Approach Embankments		
Drawn By	EJW	Company	R.W. Gillespie & Associates, Inc.
Date	3/10/2018, 6:27:54 AM	File Name	30 Nov 2018.s3z

SETTLE3D 4.016

Total Settlement vs. Depth

EAST ABUTMENT



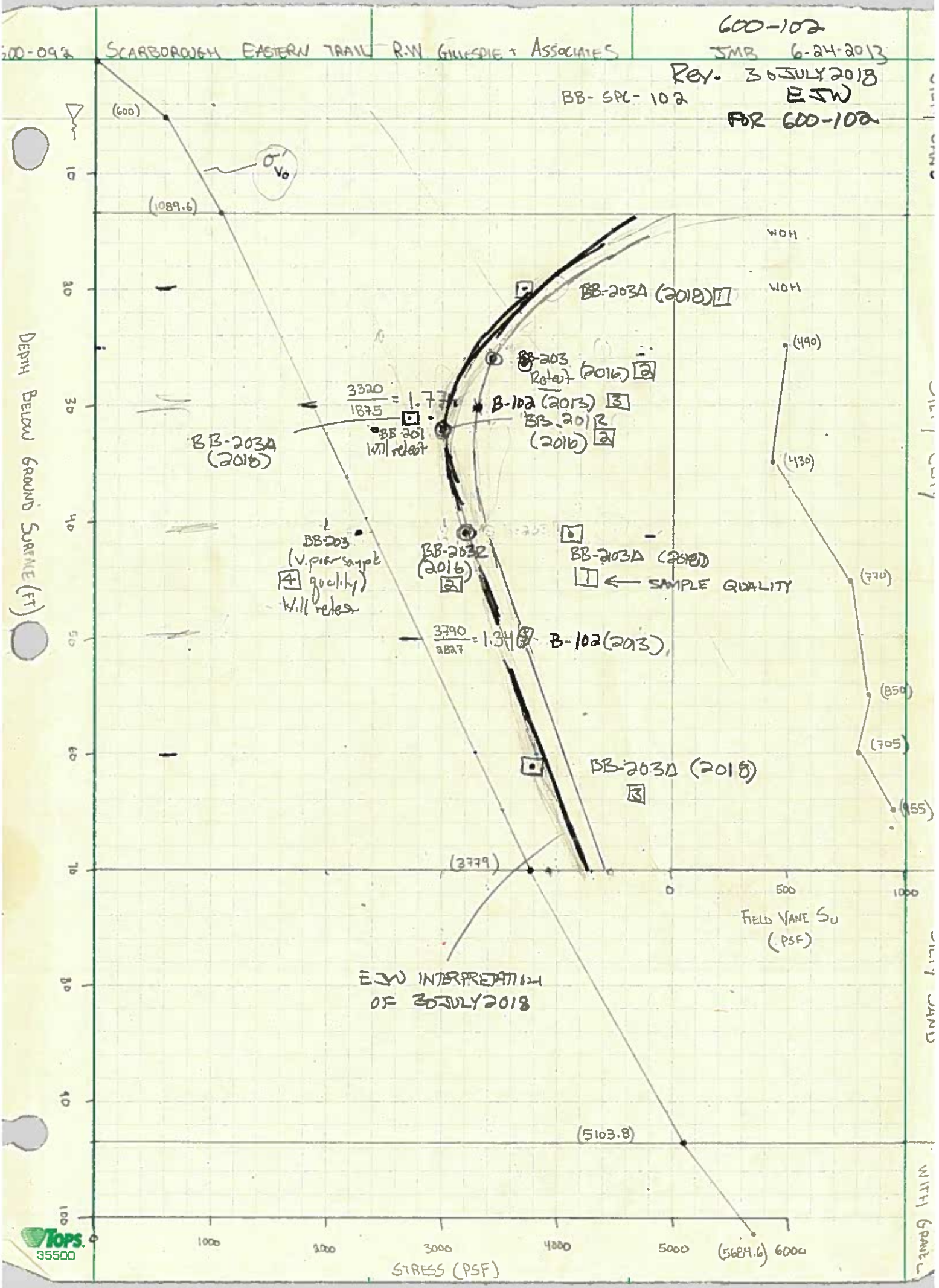
Query Point 4 (Stage 8 = 29200 d)

80 yrs

Reference Stage: None

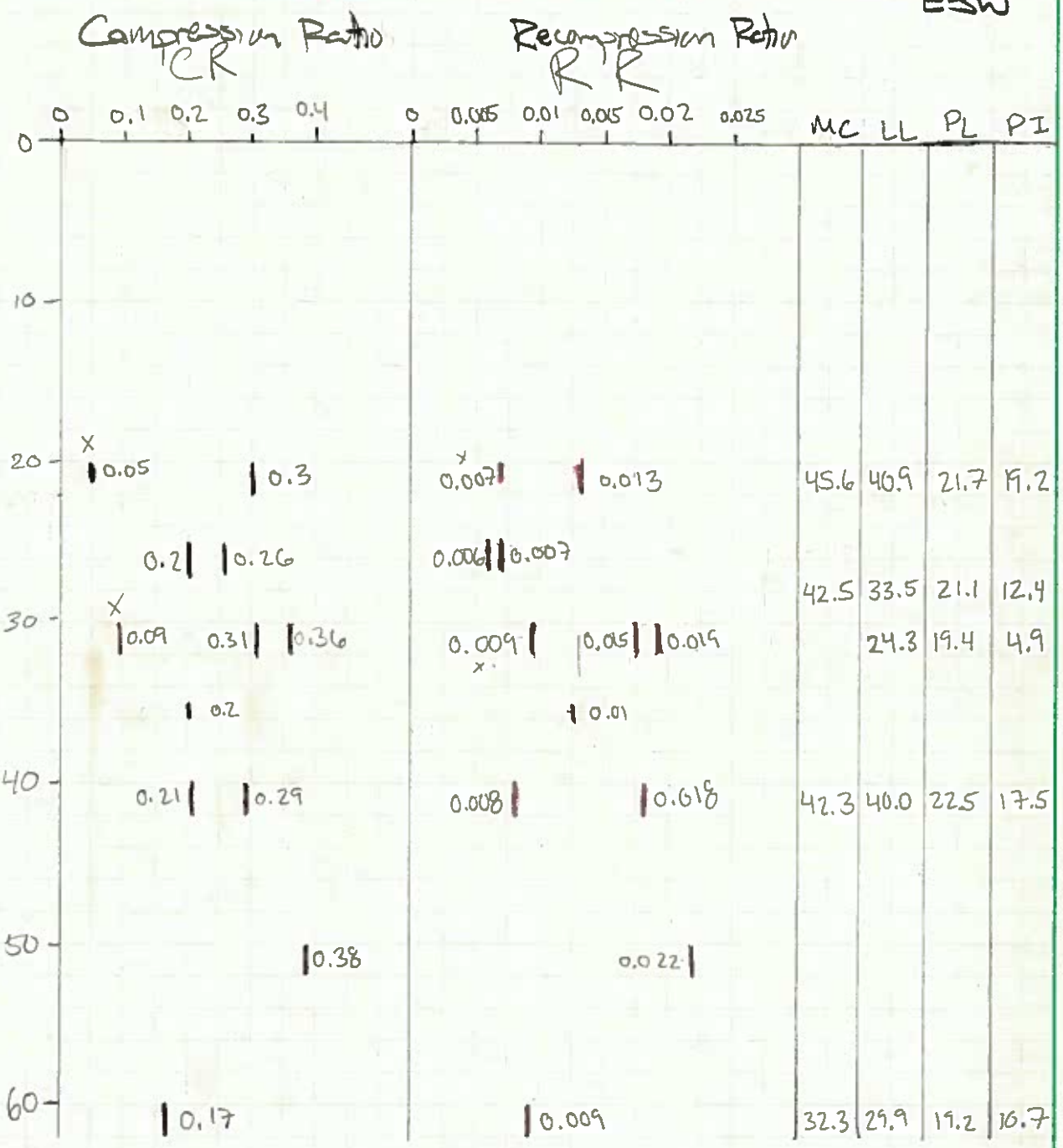
rocscience	Project	Eastern Trail Pan-Am Crossing	
	Analysis Description	Bridge Approach Embankments	
	Drawn By	EJW	Company: R.W. Gillespie & Associates, Inc.
	Date	3/10/2018, 6:27:54 AM	File Name: 30 Nov 2018.s3z

SETTLE3D 4.016


600-102
SCARBOROUGH EASTERN TRAIL R.W GILLESPIE + ASSOCIATES
JMB 6-24-2013
Rev. 30 JULY 2018
EJW
FOR 600-102
BB-SPC-102
(600)
σ'_{vo}
(1089.6)
WOH
BB-203A (2018) 1
WOH
BB-203 Relet (2016) 2
3320/1875 = 1.77
B-102 (2013) 3
BB-203 Will relet
BB-203A (2018)
BB-203R (2016) 2
(490)
(430)
BB-203 (V. poor sample quality) 4 Will relet
BB-203R (2016) 2
BB-203A (2018) 1 ← SAMPLE QUALITY
(770)
3790/3837 = 1.34
B-102 (2013)
(850)
(705)
BB-203A (2018) 3
(955)
(3779)
0
500
1000
FIELD VANE S_U (PSF)
EJW INTERPRETATION OF 30 JULY 2018
(5103.8)
1000
2000
3000
4000
5000
(5684.6)
6000
STRESS (PSF)
DEPTH BELOW GROUND SURFACE (FT)
10
20
30
40
50
60
70
80
90
100
SILTY SAND
SILTY SAND WITH GRAVEL
TOPS 35500

600-102 | RWG&A | SNP 27 NOV 2018

ESW



$\bar{x}_{10} = 0.27$

Use CR = 0.27

$\bar{x}_{12} = 0.012$

$\bar{x}_{10} = 0.013$

Use RR = 0.015

$C_{v_r} \approx 2 \times 10^{-2}$ cm²/sec ≈ 2 ft²/day

$C_{v_p} \approx 0.2$ ft²/day

NGI Sample Quality Category (Lunne et al. 2006)

OCR	Excellent to Very Good (1)	Good to Fair (2)	Poor (3)	Very Poor (4)
1-2	<0.04	0.04-0.07	0.07-0.14	>0.14
2-4	<0.03	0.03-0.05	0.05-0.10	>0.10
	$\Delta e/e_0$ at σ'_{v0}			

QUALITY OF SAMPLES BB-SPC-203A

U-1 20-22' $e_o = 1.321$ ε at σ_{vo}' → 0.008.

$$\frac{\Delta e}{e_o} = \frac{\varepsilon(1+e_o)}{e_o} = \frac{0.008(1+1.321)}{1.321} = 0.014 \quad (1)$$

U-2 30-32' $e_o = 0.990$ ε at σ_{vo}' → 0.032

$$\frac{\Delta e}{e_o} = \frac{\varepsilon(1+e_o)}{e_o} = \frac{0.032(1+0.990)}{0.990} = 0.064 \quad (2)$$

U-3 40-42' $e_o = 1.200$ ε at σ_{vo}' → 0.015

$$\frac{\Delta e}{e_o} = \frac{\varepsilon(1+e_o)}{e_o} = \frac{0.015(1+1.200)}{1.200} = 0.028 \quad (1)$$

U-5 60-62' $e_o = 0.981$ ε at σ_{vo}' → 0.042

$$\frac{\Delta e}{e_o} = \frac{\varepsilon(1+e_o)}{e_o} = \frac{0.042(1+0.981)}{0.981} = 0.085 \quad (3)$$

QUALITY OF SAMPLES

$$\frac{\Delta e}{e_o} = \frac{\varepsilon(1+e_o)}{e_o}$$

BB-SPC-203 U-1A 25-27'

$\frac{0.038(1+1.314)}{1.314} = 0.067$ Quality (2)

BB-SPC-203 U-1B 25-27'

$\frac{0.039(1+1.273)}{1.273} = 0.070$ Quality (2)

BB-SPC-201 U-2 30-32'

$\frac{0.034(1+1.059)}{1.059} = 0.0666$ Quality (2)

BB-SPC-102 U-1 30-32'

$\frac{0.062(1+1.446)}{1.446} = 0.1049$ Quality (3)

U-3

$\frac{0.043(1+1.446)}{1.446} = 0.073$ Quality (3)

U-3 50-52

BB-SPC-203 U-2A 40-42'

$\frac{0.11(1+1.136)}{1.136} = 0.207$ Quality (4)

BB-SPC-203 U-2B 40-42'

$\frac{0.024(1+1.169)}{1.169} = 0.0445$ Quality (2)

Consolidation Test Data Summary

RWG&A Project: 0600-102

Project: Eastern Trail, Scarborough Client: HNTB Date: 26 Nov 2018

Boring	Sample	Depth	Quality	NM	eo	Overburden	Past Pressure	Cc	Cr	CR	RR
BB-SPC-102	U-1	30-32	3	50.2	1.446		3.3	0.94	0.15	0.36	0.019
BB-SPC-102	U-3	50-52	3	43.2	1.164		3.8	0.81	0.12	0.38	0.022
BB-SPC-201	U-2	30-32	2	38.1	1.059		2.4			0.31	0.015
BB-SPC-202	U-1	20-21		30			10.6			0.05	0.007
BB-SPC-203	U-1B	25-27	2	46.3	1.273		3.4	0.44	0.012	0.26	0.006
BB-SPC-203	U-2B	40-42	2	39.7	1.169		2.9	0.45	0.07	0.21	0.008
BB-SPC-203A	U-1	20-22	1	45.6	1.321	1.25	3.6	0.7	0.06	0.3	0.013
BB-SPC-203A	U-2	30-32	2	39	0.99	1.75	3.3	0.18	0.03	0.091	0.009
BB-SPC-203A	U-3	40-42	1	42.3	1.2	2.2	4.1	0.65	0.06	0.29	0.018
BB-SPC-203A	U-5	60-62	3	37.4	0.981	3.2	3.6	0.36	0.05	0.17	0.009
EB-SNR-102	U-1	25-27		37	1.049		3.9	0.4	0.08	0.2	0.007
EB-SNR-102	U-2	35-36		34.4	0.945		3.7	0.12	0.04	0.2	0.01
									AVG	0.268	0.0127
									STD	0.071926	0.005579
									UPPER L	0.339926	0.018279

DRIVEN PILE EVALUATIONS

Project: Scarborough Eastern Trail
Project No.: 0600-102
Calculations by: EJW Checked by MRG
Date: 8-Feb-2019

Abutment 1 (West Abutment) - Pedestrian Bridge Over PanAm Railroad Right of Way (Boring BB-SPC-203)

1. Structural Axial Resistance of Individual H Piles (Article 10.7.3.13)

A. Strength Limit State

Steel Yield Stress (psi)	50,000
Elastic Modulus, E (psi)	29,000,000
Corrosion Loss (in)	0

			Manufactured					With Corrosion					
			Flange						Flange				
Pile Size	Weight Per Foot	Depth of Section (in)	Width (in)	Thickness (in)	Web Thickness (in)	Area of Steel, Ag (si)	"Box" Perimeter Area (si/inch)	Depth of Section	Width (in)	Thickness (in)	Web Thickness (in)	Area of Steel, Ag (si)	"Box" Area (si)
HP-8X	36	8.02	8.155	0.445	0.445	10.63	32.4	8.02	8.16	0.445	0.445	10.63	65.4
HP-10X	42	9.7	10.075	0.420	0.415	12.31	39.6	9.70	10.08	0.420	0.415	12.31	97.7
	57	9.99	10.225	0.565	0.565	16.88	40.4	9.99	10.23	0.565	0.565	16.88	102.1
HP-12X	53	11.78	12.045	0.435	0.435	15.41	47.7	11.780	12.045	0.435	0.435	15.41	141.9
	63	11.94	12.125	0.515	0.515	18.37	48.1	11.940	12.125	0.515	0.515	18.37	144.8
	74	12.13	12.215	0.610	0.605	21.87	48.7	12.130	12.215	0.610	0.605	21.87	148.2
	84	12.28	12.295	0.685	0.685	24.79	49.2	12.280	12.295	0.685	0.685	24.79	151.0

Nominal Compressive Resistance: Based on AASHTO LRFD 2014 with 2015 and 2016 Revisions Article 6.9.4.1.

Nominal Compressive Resistance, $Pn = (0.658^{(Po/Pe)})Po$ where Po = Q Fy Ag; Q =1 for non-slender pile

$$Pe = \frac{\pi^2 E\, Ag}{\left(\frac{Kl}{r_s}\right)^2}$$

where l = unbraced length in plane of bucking = 0 for embedded pile and Pe = infinity

Therefore Pn = Fy Ag

Pile Size	Weight Per Foot	Area of Steel, Ag (si)	Pn (kips)
HP-8X	36	10.63	531
HP-10X	42	12.31	616
	57	16.88	844
HP-12X	53	15.41	771
	63	18.37	919
	74	21.87	1,094
	84	24.79	1,239

Project: Scarborough Eastern Trail
Project No.: 0600-102
Calculations by: EJW Checked by MRG
Date: 8-Feb-2019

Abutment 1 (West Abutment) - Pedestrian Bridge Over PanAm Railroad Right of Way (Boring BB-SPC-203)

Factored Compressive Resistance: Based on AASHTO LRFD Article 6.5.4.2

Factored Compressive Resistance, $\Pr = \emptyset c\, Pn,$ where $\emptyset\, c = resistance\ factor\ from\ Article\ 6.5.4.2$

Øc = 0.50 for H-piles where subject to damage due to severe driving conditions where a pile tip is necessary.

Øc = 0.60 for H-piles where driven in good conditions where a pile tip is unnecessary.

Øc = 0.5

Pile Size	Weight Per Foot	Area of Steel, Ag (si)	Pn (Article 6.9.4.1) (kips)	Pc (kips)
HP-8X	36	10.63	531	266
HP-10X	42	12.31	616	308
	57	16.88	844	422
HP-12X	53	15.41	771	385
	63	18.37	919	459
	74	21.87	1,094	547
	84	24.79	1,239	620

B. Service/Extreme Limit State

Nominal Compressive Strength is the same as calculated for Strength Limit State

Per Article 10.5.5.1, resistance factors for the service limit states = 1.0

Per Article 10.5.5.3.3. Extreme Limit compressive resistance factor, Ø, = 1.0 except for uplift where the Ø = 0.8

Per Article 6.5.5, Extreme Event Limit Stat compressive resistance factor, Ø, = 1.0

Øc = 1.0

Pile Size	Weight Per Foot	Area of Steel, Ag (si)	Pn (Article 6.9.4.1) (kips)	Pc (kips)
HP-8X	36	10.63	531	531
HP-10X	42	12.31	616	616
	57	16.88	844	844
HP-12X	53	15.41	771	771
	63	18.37	919	919
	74	21.87	1,094	1,094
	84	24.79	1,239	1,239

Project: Scarborough Eastern Trail
Project No.: 0600-102
Calculations by: EJW Checked by MRG
Date: 8-Feb-2019

Abutment 1 (West Abutment) - Pedestrian Bridge Over PanAm Railroad Right of Way (Boring BB-SPC-203)

2. Geotechnical Axial Resistance of Individual H Piles

Downdrag Forces: Consolidation of silty clay deposits due to embankment filling will induce downdrag forces on piles. Per Article 3.11.8 full downdrag can be assumed to develop of if the settlement in the soil layer is 0.4 inches or greater. Article 10.7.1.6.2 Nominal pile resistance available should be adequate to resist to structural loads plus downdrag loads.

Thickest soil deposits at substructure locations were encountered at west abutment boring (BB-SPC- 203). Service limit state evaluation of settlement indicates settlement exceeds 0.4 inches to a depth of 30 to 35 feet below current ground surface. Consider soil deposits above 40-foot depth plus embankment fill in downdrag

See attached calculations of downdragshear stress, DD'

DD' = 33,600 total pounds per square foot/ft of pile box area over upper 40 feet

Downdrag Load Factors (Article 10.7.3.7)

Strength Limit State: γ_p	1.4	Article C3.11.8, Table 3.4.1-1
Extreme Limit State: γ_p	1.4	
Service Limit State: γ_p	1.0	

Pile Size	Weight Per Foot	"Box" Perimeter Area (si/inch)	"Box" Perimeter Area (sf/ft)	Nominal DD (kips)	Strength Factored DD (kips)	Extreme Factored DD (kips)	Service Factored DD (kips)
HP-8X	36	32.35	2.70	91	127	127	91
HP-10X	42	39.55	3.30	111	155	155	111
	57	40.43	3.37	113	158	158	113
HP-12X	53	47.65	3.97	133	187	187	133
	63	48.13	4.01	135	189	189	135
	74	48.69	4.06	136	191	191	136
	84	49.15	4.10	138	193	193	138

Project: Scarborough Eastern Trail
Project No.: 0600-102
Calculations by: EJW Checked by MRG
Date: 8-Feb-2019

Abutment 1 (West Abutment) - Pedestrian Bridge Over PanAm Railroad Right of Way (Boring BB-SPC-203)

A1. Strength Limit State - Axial Loading

Approximately 74 feet of granular soil deposits overlies bedrock. Underlying soils consist of 37 feet of marine silty sand over 37 feet of glacial till deposits (N>50). Settlement evaluations indicate total thickness of silty clay deposits compress. Ignore contribitution of adhesion in silty clay deposits to geotechnical resistance.

Evalute geotechnical resistance using Nordlund (Article 10.7.3.8.6f) for piles bearing in glacial till deposits/

Soil Properties:

	Average N-Value	N60	Cn	N1 60
Marine Sand	26	39	0.70	27
Glacial Till	70	105	0.55	58

Stratum	Total Thickness (ft)	Mid Layer Depth Below Preconstruction Grades (ft)	Preconstruction Effective Stress (psf)	Embankment Vertical Stress (psf)	Total Unit Weight (pcf)	Internal Angle of Friction (Table 10.4.6.2.4.1)	Side Resistance			Tip Resistance	
							Ks (Figure 10.7.3.8.6f-4)	δ/Ø (Figure 10.7.3.8.6f-6	Correction Factor, Cf (Figure 10.3.8.6f-5)	α (Figure 10.7.3.8.6f-7)	N'q (Figure 10.7.3.8.6f-8)
Marine Sand	36.5	84.75	4964.85	1575	135	36	1.3	0.8	0.9	0.7	100
Glacial Till	37.5	122	7669.2	1575	135	43	1.7	0.8	0.85	0.8	300

Unit side resistance: $q_s = K_\delta C_f \sigma' \frac{\sin(\delta)}{\cos(w)}$ (Eq. 10.7.3.8.6f-1)

Marine Sand Unit Side Resistance, qs (psf)	3686
Glacial Till Unit Side Resistance, qs (psf)	7547

Tip resistance: $q_p = \alpha N'_q \sigma'$ (Eq. 10.7.3.8.6.f-2)

	From Eq. 10.7.3.8.6.f-2	Limit Unit Tip Resistance (Figure 10.7.3.8.6f-9)
Marine Sand Unit Tip Resistance, qp (psf)	457,790	250,000
Glacial Till Unit Tip Resistance, qp (psf)	2,218,608	650,000

Pile Size	Weight Per Foot	Steel Area (si)	"Box" Perimeter Area (si/inch)	"Box" Perimeter Area (sf/ft)	Marine Sand		Glacial Till		Total Nominal Resistance for Pile Driven Through Glacial Till (kips)
					Full Penetration Nominal Side Resistance, Rs (kips)	Nominal Tip Resistance, Rp (kips)	Full Penetration Nominal Side Resistance, Rs (kips)	Nominal Tip Resistance, Rp (kips)	
HP-8X	36	10.63	32.35	2.70	363	18	763	48	1,174
HP-10X	42	12.31	39.55	3.30	443	21	933	56	1,432
	57	16.88	40.43	3.37	453	29	953	76	1,483
HP-12X	53	15.41	47.65	3.97	534	27	1,124	70	1,728
	63	18.37	48.13	4.01	540	32	1,135	83	1,758
	74	21.87	48.69	4.06	546	38	1,148	99	1,793
	84	24.79	49.15	4.10	551	43	1,159	112	1,822

Project: Scarborough Eastern Trail
Project No.: 0600-102
Calculations by: EJW Checked by MRG
Date: 8-Feb-2019

Abutment 1 (West Abutment) - Pedestrian Bridge Over PanAm Railroad Right of Way (Boring BB-SPC-203)

Calculate Factored Geotechnical Resistance: $Rr = \emptyset_{stat} R_p + \emptyset_{stat} R_s \quad (Eq. 10.7.3.8.6a - 2)$

$\emptyset_{stat}$ = 0.45 (Table 10.5.5.2.3-1)

Pile Size	Weight Per Foot	Marine Sand		Glacial Till		Total Factored Resistance for Pile Driven Through Glacial Till (kips)
		Full Penetration Factored Side Resistance, Rsr (kips)	Factored Tip Resistance, Rpr (kips)	Full Penetration Factored Side Resistance, Rsr (kips)	Factored Tip Resistance, Rpr (kips)	
HP-8X	36	163	8	343	22	528
HP-10X	42	200	10	420	25	644
	57	204	13	429	34	667
HP-12X	53	240	12	506	31	777
	63	243	14	511	37	791
	74	246	17	517	44	807
	84	248	19	522	50	820

Compare Factored Resistance to Factored Loads

Strenth III Limit State Pile Load at Abutment (kips): 161 (HNTB email 19 Nov 2018; 2:23 PM)

Pile Size	Weight Per Foot	Factored Structural Load (kips)	Factored Downdrag, DD (kips)	Total Factored Load (kips)
HP-8X	36	161	127	288
HP-10X	42	161	155	316
	57	161	158	319
HP-12X	53	161	187	348
	63	161	189	350
	74	161	191	352
	84	161	193	354

Total service state factored load is greater than full penetration resistance available in marine sand. Piles will need to bear in glacial till deposits. Estimate pile embedment length needed in glacial till deposits.

$$Lr = \frac{Factored\ Load - Factored\ Skin\ Resistance\ in\ Marine\ Sand - Factored\ Glacial\ Till\ Tip\ Resistance}{Factored\ Unit\ Skin\ Resistance\ in\ Glacial\ Till\ x\ Pile\ Perimeter\ Box\ Area}$$

Factored Glacial Till Unit Side Resistance, qs (psf) 3,396

Pile Size	Weight Per Foot	Factored Structural Load (kips)	Factored Downdrag, DD (kips)	Total Factored Load (kips)	Minimum Factored Side Resistance Required in Glacial Till (kips)	Calculated Minimum Pile Embedment Length in Glacial Till (ft)
HP-8X	36	161	127	288	103	11
HP-10X	42	161	155	316	91	8
	57	161	158	319	81	7
HP-12X	53	161	187	348	76	6
	63	161	189	350	70	5
	74	161	191	352	62	4
	84	161	193	354	55	4

Project: Scarborough Eastern Trail
Project No.: 0600-102
Calculations by: EJW Checked by MRG
Date: 8-Feb-2019

Abutment 1 (West Abutment) - Pedestrian Bridge Over PanAm Railroad Right of Way (Boring BB-SPC-203)

A2. Strength Limit State - Uplift Resistance (Article 10.7.3.10)

$R_R = \emptyset_{up} R_s$ Eq. 10.1.3.10-1

Use downdrag force as Rn via Tomlinson for silty clay $\emptyset_{up}$ = 0.25

For silty sand and glacial till via Nordlund $\emptyset_{up}$ = 0.35

From above and using minimum glacial till embedment lengths calculated in Section A1 above.

Pile Size	Weight Per Foot	Nominal Downdrag Load, Rs (kips)	Nominal Skin Resistance, Rs, in Marine Sand (kips)	Nominal Skin Resistance, Rs, in Glacial Till (kips)	Nominal Uplift Resistance (kips)	Factored Uplift Resistance (kips)
HP-8X	36	91	363	229	682	230
HP-10X	42	111	443	203	757	254
	57	113	453	180	747	250
HP-12X	53	133	534	169	837	280
	63	135	540	154	829	277
	74	136	546	137	820	273
	84	138	551	123	812	270

B. Service and Extreme Limit State

Axial Resistance Factor, Ø 1.0 per Articles 6.5.5, 10.5.5.1, and 10.5.5.5.

Uplift Resistance Factor, $\emptyset_{up}$ 0.8

B1. Axial Resistance:

Pile Size	Weight Per Foot	Nominal Skin Resistance, Rs, in Marine Sand (kips)	Nominal Skin Resistance, Rs, in Glacial Till (kips) Using Estimated Pile Embedment Length	Nominal Tip Resistance (kips)	Nominal Axial Resistance, Rn (kips)	Factored Axial Resistance (kips)
HP-8X	36	363	229	48	640	640
HP-10X	42	443	203	56	702	702
	57	453	180	76	710	710
HP-12X	53	534	169	70	773	773
	63	540	154	83	777	777
	74	546	137	99	782	782
	84	551	123	112	786	786

Project: Scarborough Eastern Trail
Project No.: 0600-102
Calculations by: EJW Checked by MRG
Date: 8-Feb-2019

Abutment 1 (West Abutment) - Pedestrian Bridge Over PanAm Railroad Right of Way (Boring BB-SPC-203)

B2. Uplift Resistance:

Pile Size	Weight Per Foot	Nominal Downdrag Load, Rs (kips)	Nominal Skin Resistance, Rs, in Marine Sand (kips)	Nominal Skin Resistance, Rs, in Glacial Till (kips)	Nominal Uplift Resistance (kips)	Factored Uplift Resistance (kips)
HP-8X	36	91	363	229	682	546
HP-10X	42	111	443	203	757	606
	57	113	453	180	747	598
HP-12X	53	133	534	169	837	669
	63	135	540	154	829	663
	74	136	546	137	820	656
	84	138	551	123	812	649

C. Drivability Analysis for Axial Resistance of Individual H-Piles

For steel piles in compression or tension:

Ødr = 0.9 Øda x fy (Eq. 10.7.8-1)
Øda = 1.0 Table 10.5.5.2.3-1 which references Article 6.5.4.2
fy = 50,000 psi

Ødr = 45,000 psi maximum allowable driving stress

Project: Scarborough Eastern Trail
Project No.: 0600-102
Calculations by: EJW Checked by MRG
Date: 8-Feb-2019

Pier - Pedestrian Bridge Over PanAm Railroad Right of Way (BB-SPC-202)

1. Structural Axial Resistance of Individual H Piles (Article 10.7.3.13)

A. Strength Limit State

Steel Yield Stress (psi)	50,000
Elastic Modulus, E (psi)	29,000,000
Corrosion Loss (in)	0

			Manufactured					With Corrosion					
			Flange						Flange				
Pile Size	Weight Per Foot	Depth of Section (in)	Width (in)	Thickness (in)	Web Thickness (in)	Area of Steel, Ag (si)	"Box" Perimeter Area (si/inch)	Depth of Section	Width (in)	Thickness (in)	Web Thickness (in)	Area of Steel, Ag (si)	"Box" Area (si)
HP-8X	36	8.02	8.155	0.445	0.445	10.63	32.4	8.02	8.16	0.445	0.445	10.63	65.4
HP-10X	42	9.7	10.075	0.420	0.415	12.31	39.6	9.70	10.08	0.420	0.415	12.31	97.7
	57	9.99	10.225	0.565	0.565	16.88	40.4	9.99	10.23	0.565	0.565	16.88	102.1
HP-12X	53	11.78	12.045	0.435	0.435	15.41	47.7	11.780	12.045	0.435	0.435	15.41	141.9
	63	11.94	12.125	0.515	0.515	18.37	48.1	11.940	12.125	0.515	0.515	18.37	144.8
	74	12.13	12.215	0.610	0.605	21.87	48.7	12.130	12.215	0.610	0.605	21.87	148.2
	84	12.28	12.295	0.685	0.685	24.79	49.2	12.280	12.295	0.685	0.685	24.79	151.0

Nominal Compressive Resistance: Based on AASHTO LRFD 2014 with 2015 and 2016 Revisions Article 6.9.4.1.

Nominal Compressive Resistance, $Pn = (0.658^{(Po/Pe)})Po$ where Po = Q Fy Ag; Q =1 for non-slender pile

$$Pe = \frac{\pi^2 E\ Ag}{\left(\frac{Kl}{r_s}\right)^2}$$

where l = unbraced length in plane of bucking = 0 for embedded pile and Pe = infinity

Therefore Pn = Fy Ag

Pile Size	Weight Per Foot	Area of Steel, Ag (si)	Pn (kips)
HP-8X	36	10.63	531
HP-10X	42	12.31	616
	57	16.88	844
HP-12X	53	15.41	771
	63	18.37	919
	74	21.87	1,094
	84	24.79	1,239

Project: Scarborough Eastern Trail
Project No.: 0600-102
Calculations by: EJW Checked by MRG
Date: ##########

Abutment 2 (East Abutment) - Pedestrian Bridge Over PanAm Railroad Right of Way (BB-SPC-201)

1. Structural Axial Resistance of Individual H Piles (Article 10.7.3.13)

A. Strength Limit State

Steel Yield Stress (psi) 50,000
Elastic Modulus, E (psi) 29,000,000
Corrosion Loss (in) 0

			Manufactured					With Corrosion					
			Flange						Flange				
Pile Size	Weight Per Foot	Depth of Section (in)	Width (in)	Thickness (in)	Web Thickness (in)	Area of Steel, Ag (si)	"Box" Perimeter Area (si/inch)	Depth of Section	Width (in)	Thickness (in)	Web Thickness (in)	Area of Steel, Ag (si)	"Box" Area (si)
HP-8X	36	8.02	8.155	0.445	0.445	10.63	32.4	8.02	8.16	0.445	0.445	10.63	65.4
HP-10X	42	9.7	10.075	0.420	0.415	12.31	39.6	9.70	10.08	0.420	0.415	12.31	97.7
	57	9.99	10.225	0.565	0.565	16.88	40.4	9.99	10.23	0.565	0.565	16.88	102.1
HP-12X	53	11.78	12.045	0.435	0.435	15.41	47.7	11.780	12.045	0.435	0.435	15.41	141.9
	63	11.94	12.125	0.515	0.515	18.37	48.1	11.940	12.125	0.515	0.515	18.37	144.8
	74	12.13	12.215	0.610	0.605	21.87	48.7	12.130	12.215	0.610	0.605	21.87	148.2
	84	12.28	12.295	0.685	0.685	24.79	49.2	12.280	12.295	0.685	0.685	24.79	151.0

Nominal Compressive Resistance: Based on AASHTO LRFD 2014 with 2015 and 2016 Revisions Article 6.9.4.1.

Nominal Compressive Resistance, $Pn = (0.658^{(Po/Pe)})Po$ where Po = Q Fy Ag; Q =1 for non-slender pile

$$Pe = \frac{\pi^2 E\, Ag}{\left(\frac{Kl}{r_s}\right)^2}$$

where l = unbraced length in plane of bucking = 0 for embedded pile and Pe = infinity

Therefore Pn = Fy Ag

Pile Size	Weight Per Foot	Area of Steel, Ag (si)	Pn (kips)
HP-8X	36	10.63	531
HP-10X	42	12.31	616
	57	16.88	844
HP-12X	53	15.41	771
	63	18.37	919
	74	21.87	1,094
	84	24.79	1,239

Project: Scarborough Eastern Trail
Project No.: 0600-102
Calculations by: EJW Checked by MRG
Date: ##########

Abutment 2 (East Abutment) - Pedestrian Bridge Over PanAm Railroad Right of Way (BB-SPC-201)

Factored Compressive Resistance: Based on AASHTO LRFD Article 6.5.4.2

Factored Compressive Resistance, $\mathrm{Pr} = \ \emptyset c\ Pn,$ where $\emptyset\ c = resistance\ factor\ from\ Article\ 6.5.4.2$

Øc = 0.50 for H-piles where subject to damage due to severe driving conditions where a pile tip is necessary.

Øc = 0.60 for H-piles where driven in good conditions where a pile tip is unnecessary.

Øc = 0.5

Pile Size	Weight Per Foot	Area of Steel, Ag (si)	Pn (Article 6.9.4.1) (kips)	Pc (kips)
HP-8X	36	10.63	531	266
HP-10X	42	12.31	616	308
	57	16.88	844	422
HP-12X	53	15.41	771	385
	63	18.37	919	459
	74	21.87	1,094	547
	84	24.79	1,239	620

B. Service/Extreme Limit State

Nominal Compressive Strength is the same as calculated for Strength Limit State

Per Article 10.5.5.1, resistance factors for the service limit states = 1.0

Per Article 10.5.5.3.3. Extreme Limit compressive resistance factor, Ø, = 1.0 except for uplift where the Ø = 0.8

Per Article 6.5.5, Extreme Event Limit Stat compressive resistance factor, Ø, = 1.0

Øc = 1.0

Pile Size	Weight Per Foot	Area of Steel, Ag (si)	Pn (Article 6.9.4.1) (kips)	Pc (kips)
HP-8X	36	10.63	531	531
HP-10X	42	12.31	616	616
	57	16.88	844	844
HP-12X	53	15.41	771	771
	63	18.37	919	919
	74	21.87	1,094	1,094
	84	24.79	1,239	1,239

Project: Scarborough Eastern Trail
Project No.: 0600-102
Calculations by: EJW Checked by MRG
Date: ##########

Abutment 2 (East Abutment) - Pedestrian Bridge Over PanAm Railroad Right of Way (BB-SPC-201)

2. Geotechnical Axial Resistance of Individual H Piles

Downdrag Forces: Consolidation of silty clay deposits due to embankment filling will induce downdrag forces on piles. Per Article 3.11.8 full downdrag can be assumed to develop of if the settlement in the soil layer is 0.4 inches or greater. Article 10.7.1.6.2 Nominal pile resistance available should be adequate to resist to structural loads plus downdrag loads.

Thickest soil deposits at substructure locations were encountered at west abutment boring (BB-SPC- 203). Service limit state evaluation of settlement indicates settlement exceeds 0.4 inches to a depth of 30 to 35 feet below current ground surface. Consider soil deposits above 40-foot depth plus embankment fill in downdrag

See attached calculations of downdragshear stress, DD'

DD' = 33,600 total pounds per square foot/ft of pile box area over upper 40 feet

Downdrag Load Factors (Article 10.7.3.7)

Strength Limit State: γ_p 1.4 Article C3.11.8, Table 3.4.1-1

Extreme Limit State: γ_p 1.4

Service Limit State: γ_p 1.0

Pile Size	Weight Per Foot	"Box" Perimeter Area (si/inch)	"Box" Perimeter Area (sf/ft)	Nominal DD (kips)	Strength Factored DD (kips)	Extreme Factored DD (kips)	Service Factored DD (kips)
HP-8X	36	32.35	2.70	91	127	127	91
HP-10X	42	39.55	3.30	111	155	155	111
	57	40.43	3.37	113	158	158	113
HP-12X	53	47.65	3.97	133	187	187	133
	63	48.13	4.01	135	189	189	135
	74	48.69	4.06	136	191	191	136
	84	49.15	4.10	138	193	193	138

Project: Scarborough Eastern Trail
Project No.: 0600-102
Calculations by: EJW Checked by MRG
Date: ##########

Abutment 2 (East Abutment) - Pedestrian Bridge Over PanAm Railroad Right of Way (BB-SPC-201)

A1. Strength Limit State - Axial Loading

Approximately 69 feet of granular soil deposits overlies bedrock. Underlying soils consist of 17 feet of marine silty sand over 52 feet of glacial till deposits (N>50). Settlement evaluations indicate total thickness of silty clay deposits compress. Ignore contribitution of adhesion in silty clay deposits to geotechnical resistance.

Evalute geotechnical resistance using Nordlund (Article 10.7.3.8.6f) for piles bearing in glacial till deposits/

Soil Properties:

	Average N-Value	N60	Cn	N1 60
Marine Sand	9	13.5	0.81	11
Glacial Till	48.5	72.75	0.64	46

Stratum	Total Thickness (ft)	Mid Layer Depth Below Preconstruction Grades (ft)	Preconstruction Effective Stress (psf)	Embankment Vertical Stress (psf)	Total Unit Weight (pcf)	Internal Angle of Friction (Table 10.4.6.2.4.1)	Side Resistance			Tip Resistance	
							Ks (Figure 10.7.3.8.6f-4)	δ/Ø (Figure 10.7.3.8.6f-6	Correction Factor, Cf (Figure 10.3.8.6f-5)	α (Figure 10.7.3.8.6f-7)	N'q (Figure 10.7.3.8.6f-8)
Marine Sand	17	63.5	3569.6	1575	135	32	1	0.8	0.9	0.55	40
Glacial Till	52	98	5946.8	1575	135	39	1.7	0.8	0.9	0.75	140

Unit side resistance: $q_s = K_\delta C_f \sigma' \frac{\sin(\delta)}{\cos(w)}$ (Eq. 10.7.3.8.6f-1)

Marine Sand Unit Side Resistance, qs (psf)	2001
Glacial Till Unit Side Resistance, qs (psf)	5962

Tip resistance: $q_p = \alpha N'_q \sigma'$ (Eq. 10.7.3.8.6.f-2)

	From Eq. 10.7.3.8.6.f-2	Limit Unit Tip Resistance (Figure 10.7.3.8.6f-9)
Marine Sand Unit Tip Resistance, qp (psf)	113,181	35,000
Glacial Till Unit Tip Resistance, qp (psf)	789,789	325,000

Pile Size	Weight Per Foot	Steel Area (si)	"Box" Perimeter Area (si/inch)	"Box" Perimeter Area (sf/ft)	Marine Sand		Glacial Till		Total Nominal Resistance for Pile Driven Through Glacial Till (kips)
					Full Penetration Nominal Side Resistance, Rs (kips)	Nominal Tip Resistance, Rp (kips)	Full Penetration Nominal Side Resistance, Rs (kips)	Nominal Tip Resistance, Rp (kips)	
HP-8X	36	10.63	32.35	2.70	92	3	836	24	951
HP-10X	42	12.31	39.55	3.30	112	3	1,022	28	1,162
	57	16.88	40.43	3.37	115	4	1,044	38	1,197
HP-12X	53	15.41	47.65	3.97	135	4	1,231	35	1,401
	63	18.37	48.13	4.01	136	4	1,243	41	1,421
	74	21.87	48.69	4.06	138	5	1,258	49	1,445
	84	24.79	49.15	4.10	139	6	1,270	56	1,465

Project: Scarborough Eastern Trail
Project No.: 0600-102
Calculations by: EJW Checked by MRG
Date: ##########

Abutment 2 (East Abutment) - Pedestrian Bridge Over PanAm Railroad Right of Way (BB-SPC-201)

Calculate Factored Geotechnical Resistance: $Rr = \emptyset_{stat} R_p + \emptyset_{stat} R_s \quad (Eq. 10.7.3.8.6a - 2)$

$\emptyset_{stat}$ = 0.45 (Table 10.5.5.2.3-1)

Pile Size	Weight Per Foot	Marine Sand		Glacial Till		Total Factored Resistance for Pile Driven Through Glacial Till (kips)
		Full Penetration Factored Side Resistance, Rsr (kips)	Factored Tip Resistance, Rpr (kips)	Full Penetration Factored Side Resistance, Rsr (kips)	Factored Tip Resistance, Rpr (kips)	
HP-8X	36	41	1	376	11	428
HP-10X	42	50	1	460	13	523
	57	52	2	470	17	539
HP-12X	53	61	2	554	16	630
	63	61	2	560	19	640
	74	62	2	566	22	650
	84	63	3	571	25	659

Compare Factored Resistance to Factored Loads

Strenth III Limit State Pile Load at Abutment (kips): 171 (HNTB email 19 Nov 2018; 2:23 PM)

Pile Size	Weight Per Foot	Factored Structural Load (kips)	Factored Downdrag, DD (kips)	Total Factored Load (kips)
HP-8X	36	171	127	298
HP-10X	42	171	155	326
	57	171	158	329
HP-12X	53	171	187	358
	63	171	189	360
	74	171	191	362
	84	171	193	364

Total service state factored load is greater than full penetration resistance available in marine sand. Piles will need to bear in glacial till deposits. Estimate pile embedment length needed in glacial till deposits.

$$Lr = \frac{Factored\ Load - Factored\ Skin\ Resistance\ in\ Marine\ Sand - Factored\ Glacial\ Till\ Tip\ Resistance}{Factored\ Unit\ Skin\ Resistance\ in\ Glacial\ Till\ x\ Pile\ Perimeter\ Box\ Area}$$

Factored Glacial Till Unit Side Resistance, qs (psf) 2,683

Pile Size	Weight Per Foot	Factored Structural Load (kips)	Factored Downdrag, DD (kips)	Total Factored Load (kips)	Minimum Factored Side Resistance Required in Glacial Till (kips)	Calculated Minimum Pile Embedment Length in Glacial Till (ft)
HP-8X	36	171	127	298	246	34
HP-10X	42	171	155	326	263	30
	57	171	158	329	261	29
HP-12X	53	171	187	358	281	26
	63	171	189	360	280	26
	74	171	191	362	278	25
	84	171	193	364	276	25

Project: Scarborough Eastern Trail
Project No.: 0600-102
Calculations by: EJW Checked by MRG
Date: ##########

Abutment 2 (East Abutment) - Pedestrian Bridge Over PanAm Railroad Right of Way (BB-SPC-201)

A2. Strength Limit State - Uplift Resistance (Article 10.7.3.10)

$R_R = \emptyset_{up} R_s$ Eq. 10.1.3.10-1

Use downdrag force as Rn via Tomlinson for silty clay $\emptyset$up = 0.25

For silty sand and glacial till via Nordlund $\emptyset_{up}$ = 0.35

From above and using minimum glacial till embedment lengths calculated in Section A1 above.

Pile Size	Weight Per Foot	Nominal Downdrag Load, Rs (kips)	Nominal Skin Resistance, Rs, in Marine Sand (kips)	Nominal Skin Resistance, Rs, in Glacial Till (kips)	Nominal Uplift Resistance (kips)	Factored Uplift Resistance (kips)
HP-8X	36	91	92	546	728	246
HP-10X	42	111	112	585	807	272
	57	113	115	580	807	271
HP-12X	53	133	135	625	894	299
	63	135	136	621	893	299
	74	136	138	617	891	298
	84	138	139	613	890	298

B. Service and Extreme Limit State

Axial Resistance Factor, $\emptyset$ 1.0 per Articles 6.5.5, 10.5.5.1, and 10.5.5.5.

Uplift Resistance Factor, $\emptyset_{up}$ 0.8

B1. Axial Resistance:

Pile Size	Weight Per Foot	Nominal Skin Resistance, Rs, in Marine Sand (kips)	Nominal Skin Resistance, Rs, in Glacial Till (kips) Using Estimated Pile Embedment Length	Nominal Tip Resistance (kips)	Nominal Axial Resistance, Rn (kips)	Factored Axial Resistance (kips)
HP-8X	36	92	546	24	662	662
HP-10X	42	112	585	28	725	725
	57	115	580	38	732	732
HP-12X	53	135	625	35	795	795
	63	136	621	41	799	799
	74	138	617	49	804	804
	84	139	613	56	808	808

Project: Scarborough Eastern Trail
Project No.: 0600-102
Calculations by: EJW Checked by MRG
Date: ##########

Abutment 2 (East Abutment) - Pedestrian Bridge Over PanAm Railroad Right of Way (BB-SPC-201)

B2. Uplift Resistance:

Pile Size	Weight Per Foot	Nominal Downdrag Load, Rs (kips)	Nominal Skin Resistance, Rs, in Marine Sand (kips)	Nominal Skin Resistance, Rs, in Glacial Till (kips)	Nominal Uplift Resistance (kips)	Factored Uplift Resistance (kips)
HP-8X	36	91	92	546	728	583
HP-10X	42	111	112	585	807	646
	57	113	115	580	807	646
HP-12X	53	133	135	625	894	715
	63	135	136	621	893	714
	74	136	138	617	891	713
	84	138	139	613	890	712

C. Drivability Analysis for Axial Resistance of Individual H-Piles

For steel piles in compression or tension:

Ødr = 0.9 Øda x fy (Eq. 10.7.8-1)
Øda = 1.0 Table 10.5.5.2.3-1 which references Article 6.5.4.2
fy = 50,000 psi

Ødr = 45,000 psi maximum allowable driving stress

Project: Scarborough Eastern Trail
Project No.: 0600-102
Calculations by: EJW Checked by MRG
Date: 8-Feb-2019

Pier - Pedestrian Bridge Over PanAm Railroad Right of Way (BB-SPC-202)

Factored Compressive Resistance: Based on AASHTO LRFD Article 6.5.4.2

Factored Compressive Resistance, $\mathrm{Pr} = \emptyset c\, Pn,$ where $\emptyset\, c = resistance\ factor\ from\ Article\ 6.5.4.2$

Øc = 0.50 for H-piles where subject to damage due to severe driving conditions where a pile tip is necessary.
Øc = 0.60 for H-piles where driven in good conditions where a pile tip is unnecessary.
Øc = 0.5

Pile Size	Weight Per Foot	Area of Steel, Ag (si)	Pn (Article 6.9.4.1) (kips)	Pc (kips)
HP-8X	36	10.63	531	266
HP-10X	42	12.31	616	308
	57	16.88	844	422
HP-12X	53	15.41	771	385
	63	18.37	919	459
	74	21.87	1,094	547
	84	24.79	1,239	620

B. Service/Extreme Limit State

Nominal Compressive Strength is the same as calculated for Strength Limit State
Per Article 10.5.5.1, resistance factors for the service limit states = 1.0
Per Article 10.5.5.3.3. Extreme Limit compressive resistance factor, Ø, = 1.0 except for uplift where the Ø = 0.8
Per Article 6.5.5, Extreme Event Limit Stat compressive resistance factor, Ø, = 1.0
Øc = 1.0

Pile Size	Weight Per Foot	Area of Steel, Ag (si)	Pn (Article 6.9.4.1) (kips)	Pc (kips)
HP-8X	36	10.63	531	531
HP-10X	42	12.31	616	616
	57	16.88	844	844
HP-12X	53	15.41	771	771
	63	18.37	919	919
	74	21.87	1,094	1,094
	84	24.79	1,239	1,239

Project: Scarborough Eastern Trail
Project No.: 0600-102
Calculations by: EJW Checked by MRG
Date: 8-Feb-2019

Pier - Pedestrian Bridge Over PanAm Railroad Right of Way (BB-SPC-202)

2. Geotechnical Axial Resistance of Individual H Piles

Downdrag Forces: Consolidation of silty clay deposits due to embankment filling will induce downdrag forces on piles. Per Article 3.11.8 full downdrag can be assumed to develop of if the settlement in the soil layer is 0.4 inches or greater. Article 10.7.1.6.2 Nominal pile resistance available should be adequate to resist to structural loads plus downdrag loads.

Thickest soil deposits at substructure locations were encountered at west abutment boring (BB-SPC- 203). Service limit state evaluation of settlement indicates settlement exceeds 0.4 inches to a depth of 30 to 35 feet below current ground surface. Consider soil deposits above 40-foot depth plus embankment fill in downdrag

See attached calculations of downdragshear stress, DD'

DD' = - total pounds per square foot/ft of pile box area over upper 40 feet

Downdrag Load Factors (Article 10.7.3.7)

Strength Limit State: γ_p	1.4	Article C3.11.8, Table 3.4.1-1
Extreme Limit State: γ_p	1.4	
Service Limit State: γ_p	1.0	

Pile Size	Weight Per Foot	"Box" Perimeter Area (si/inch)	"Box" Perimeter Area (sf/ft)	Nominal DD (kips)	Strength Factored DD (kips)	Extreme Factored DD (kips)	Service Factored DD (kips)
HP-8X	36	32.35	2.70	-	0	0	0
HP-10X	42	39.55	3.30	-	0	0	0
	57	40.43	3.37	-	0	0	0
HP-12X	53	47.65	3.97	-	0	0	0
	63	48.13	4.01	-	0	0	0
	74	48.69	4.06	-	0	0	0
	84	49.15	4.10	-	0	0	0

Project: Scarborough Eastern Trail
Project No.: 0600-102
Calculations by: EJW Checked by MRG
Date: 8-Feb-2019

Pier - Pedestrian Bridge Over PanAm Railroad Right of Way (BB-SPC-202)

A1. Strength Limit State - Axial Loading

Approximately 61 feet of granular soil deposits overlies bedrock. Underlying soils consist of about 20 feet of marine silty sand over about 41 feet of glacial till deposits (N>50). Settlement evaluations indicate total thickness of silty clay deposits compress. Ignore contribitution of adhesion in silty clay deposits to geotechnical resistance.

Evalute geotechnical resistance using Nordlund (Article 10.7.3.8.6f) for piles bearing in glacial till deposits/

Soil Properties:

	Average N-Value	N60	Cn	N1 60
Marine Sand	20	30	0.76	23
Glacial Till	70	105	0.60	62

Stratum	Total Thickness (ft)	Mid Layer Depth Below Preconstruction Grades (ft)	Preconstruction Effective Stress (psf)	Embankment Vertical Stress (psf)	Total Unit Weight (pcf)	Internal Angle of Friction (Table 10.4.6.2.4.1)	Side Resistance			Tip Resistance	
							Ks (Figure 10.7.3.8.6f-4)	δ/Ø (Figure 10.7.3.8.6f-6	Correction Factor, Cf (Figure 10.3.8.6f-5)	α (Figure 10.7.3.8.6f-7)	N'q (Figure 10.7.3.8.6f-8)
Marine Sand	20	70	4169	1575	135	35	1.2	0.8	0.9	0.65	60
Glacial Till	41	105.5	6746.3	1575	135	43	1.7	0.8	0.85	0.8	300

Unit side resistance: $q_s = K_\delta C_f \sigma' \frac{\sin(\delta)}{\cos(w)}$ (Eq. 10.7.3.8.6f-1)

Marine Sand Unit Side Resistance, qs (psf)	2912
Glacial Till Unit Side Resistance, qs (psf)	6793

Tip resistance: $q_p = \alpha N'_q \sigma'$ (Eq. 10.7.3.8.6.f-2)

	From Eq. 10.7.3.8.6.f-2	Limit Unit Tip Resistance (Figure 10.7.3.8.6f-9)
Marine Sand Unit Tip Resistance, qp (psf)	224,016	250,000
Glacial Till Unit Tip Resistance, qp (psf)	1,997,112	650,000

Pile Size	Weight Per Foot	Steel Area (si)	"Box" Perimeter Area (si/inch)	"Box" Perimeter Area (sf/ft)	Marine Sand		Glacial Till		Total Factored Nominal for Pile Driven Through Glacial Till (kips)
					Full Penetration Nominal Side Resistance, Rs (kips)	Nominal Tip Resistance, Rp (kips)	Full Penetration Nominal Side Resistance, Rs (kips)	Nominal Tip Resistance, Rp (kips)	
HP-8X	36	10.63	32.35	2.70	157	17	751	48	956
HP-10X	42	12.31	39.55	3.30	192	19	918	56	1,166
	57	16.88	40.43	3.37	196	26	938	76	1,211
HP-12X	53	15.41	47.65	3.97	231	24	1,106	70	1,407
	63	18.37	48.13	4.01	234	29	1,117	83	1,434
	74	21.87	48.69	4.06	236	34	1,130	99	1,465
	84	24.79	49.15	4.10	239	39	1,141	112	1,491

Project: Scarborough Eastern Trail
Project No.: 0600-102
Calculations by: EJW Checked by MRG
Date: 8-Feb-2019

Pier - Pedestrian Bridge Over PanAm Railroad Right of Way (BB-SPC-202)

Calculate Factored Geotechnical Resistance: $Rr = \emptyset_{stat} R_p + \emptyset_{stat} R_s \quad (Eq.\ 10.7.3.8.6a - 2)$

$\emptyset_{stat}$ = 0.45 (Table 10.5.5.2.3-1)

Pile Size	Weight Per Foot	Marine Sand		Glacial Till		Total Factored Resistance for Pile Driven Through Glacial Till (kips)
		Full Penetration Factored Side Resistance, Rsr (kips)	Factored Tip Resistance, Rpr (kips)	Full Penetration Factored Side Resistance, Rsr (kips)	Factored Tip Resistance, Rpr (kips)	
HP-8X	36	71	7	338	22	430
HP-10X	42	86	9	413	25	524
	57	88	12	422	34	545
HP-12X	53	104	11	498	31	633
	63	105	13	503	37	645
	74	106	15	509	44	659
	84	107	17	513	50	671

Compare Factored Resistance to Factored Loads

Strenth III Limit State Pile Load at Pier (kips): 138 (HNTB email 19 Nov 2018; 2:23 PM)

Pile Size	Weight Per Foot	Factored Structural Load (kips)	Factored Downdrag, DD (kips)	Total Factored Load (kips)
HP-8X	36	138	0	138
HP-10X	42	138	0	138
	57	138	0	138
HP-12X	53	138	0	138
	63	138	0	138
	74	138	0	138
	84	138	0	138

Total service state factored load is greater than full penetration resistance available in marine sand. Piles will need to bear in glacial till deposits. Estimate pile embedment length needed in glacial till deposits.

$$Lr = \frac{Factored\ Load - Factored\ Skin\ Resistance\ in\ Marine\ Sand - Factored\ Glacial\ Till\ Tip\ Resistance}{Factored\ Unit\ Skin\ Resistance\ in\ Glacial\ Till\ x\ Pile\ Perimeter\ Box\ Area}$$

Factored Glacial Till Unit Side Resistance, qs (psf) 3,057

Pile Size	Weight Per Foot	Factored Structural Load (kips)	Factored Downdrag, DD (kips)	Total Factored Load (kips)	Minimum Factored Side Resistance Required in Glacial Till (kips)	Calculated Minimum Pile Embedment Length in Glacial Till (ft)
HP-8X	36	138	0	138	46	6
HP-10X	42	138	0	138	27	3
	57	138	0	138	15	1
HP-12X	53	138	0	138	3	0
	63	138	0	138	0	-
	74	138	0	138	0	-
	84	138	0	138	0	-

Project: Scarborough Eastern Trail
Project No.: 0600-102
Calculations by: EJW Checked by MRG
Date: 8-Feb-2019

Pier - Pedestrian Bridge Over PanAm Railroad Right of Way (BB-SPC-202)

A2. Strength Limit State - Uplift Resistance (Article 10.7.3.10)

$R_R = \emptyset_{up} R_s$ Eq. 10.1.3.10-1

Use downdrag force as Rn via Tomlinson for silty clay Øup = 0.25

For silty sand and glacial till via Nordlund $\emptyset_{up}$ = 0.35

From above and using minimum glacial till embedment lengths calculated in Section A1 above.

Pile Size	Weight Per Foot	Nominal Downdrag Load, Rs (kips)	Nominal Skin Resistance, Rs, in Marine Sand (kips)	Nominal Skin Resistance, Rs, in Glacial Till (kips)	Nominal Uplift Resistance (kips)	Factored Uplift Resistance (kips)
HP-8X	36	0	157	102	259	91
HP-10X	42	0	192	59	251	88
	57	0	196	34	230	81
HP-12X	53	0	231	6	237	83
	63	0	234	0	234	82
	74	0	236	0	236	83
	84	0	239	0	239	84

B. Service and Extreme Limit State

Axial Resistance Factor, Ø 1.0 per Articles 6.5.5, 10.5.5.1, and 10.5.5.5.

Uplift Resistance Factor, $\emptyset_{up}$ 0.8

B1. Axial Resistance:

Pile Size	Weight Per Foot	Nominal Skin Resistance, Rs, in Marine Sand (kips)	Nominal Skin Resistance, Rs, in Glacial Till (kips) Using Estimated Pile Embedment Length	Nominal Tip Resistance (kips)	Nominal Axial Resistance, Rn (kips)	Factored Axial Resistance (kips)
HP-8X	36	157	102	48	307	307
HP-10X	42	192	59	56	307	307
	57	196	34	76	307	307
HP-12X	53	231	6	70	307	307
	63	234	0	83	317	317
	74	236	0	99	335	335
	84	239	0	112	350	350

Project: Scarborough Eastern Trail
Project No.: 0600-102
Calculations by: EJW Checked by MRG
Date: 8-Feb-2019

Pier - Pedestrian Bridge Over PanAm Railroad Right of Way (BB-SPC-202)

B2. Uplift Resistance:

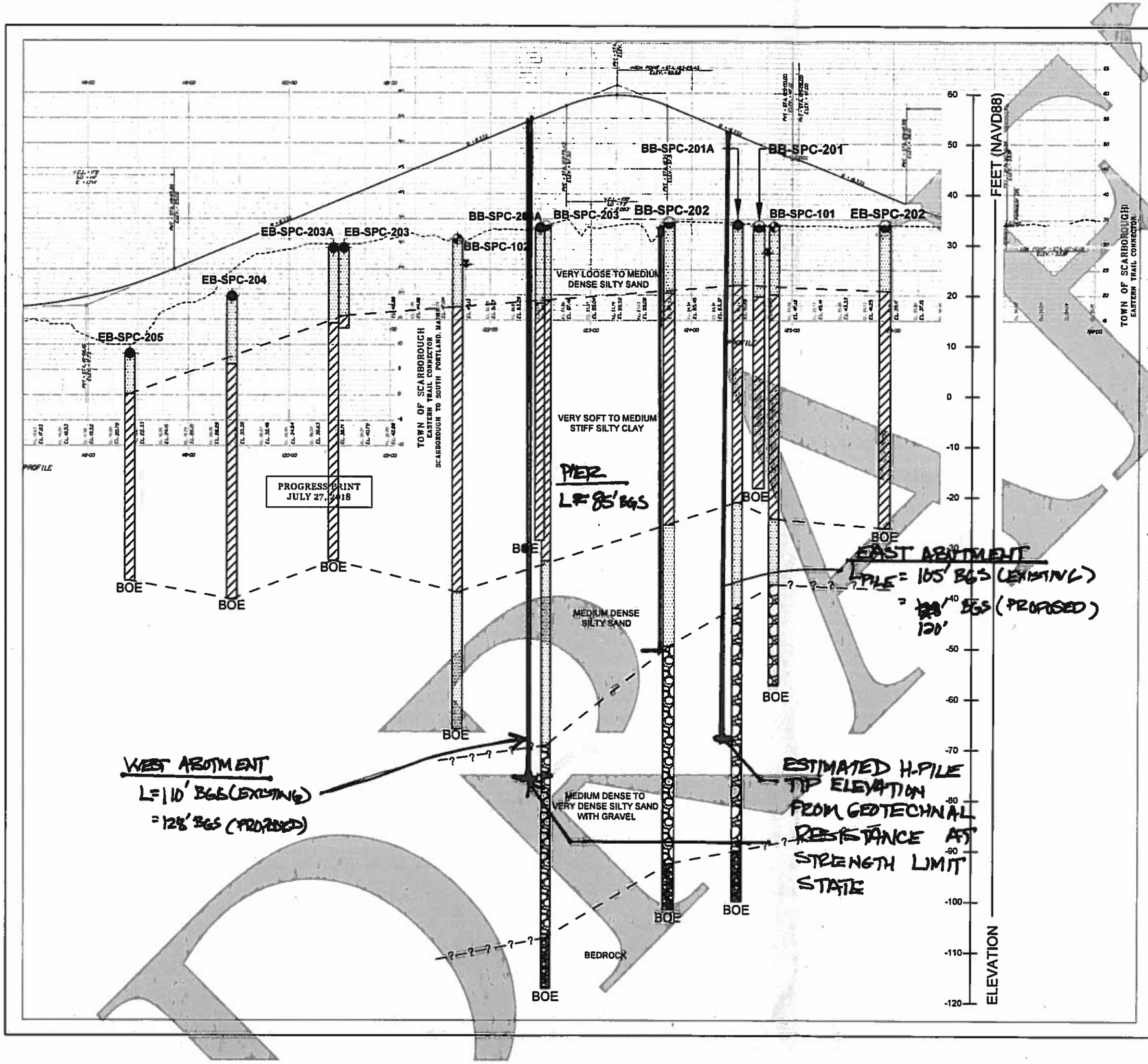
Pile Size	Weight Per Foot	Nominal Downdrag Load, Rs (kips)	Nominal Skin Resistance, Rs, in Marine Sand (kips)	Nominal Skin Resistance, Rs, in Glacial Till (kips)	Nominal Uplift Resistance (kips)	Factored Uplift Resistance (kips)
HP-8X	36	0	157	102	259	207
HP-10X	42	0	192	59	251	201
	57	0	196	34	230	184
HP-12X	53	0	231	6	237	190
	63	0	234	0	234	187
	74	0	236	0	236	189
	84	0	239	0	239	191

C. Drivability Analysis for Axial Resistance of Individual H-Piles

For steel piles in compression or tension:

Ødr = 0.9 Øda x fy (Eq. 10.7.8-1)
Øda = 1.0 Table 10.5.5.2.3-1 which references Article 6.5.4.2
fy = 50,000 psi

Ødr = 45,000 psi maximum allowable driving stress



LEGEND:

- BB-SPC-101 — APPROXIMATE LOCATION OF SOIL BORING DRILLED OCTOBER 2013
- BB-SPC-201 — APPROXIMATE LOCATION OF SOIL BORING DRILLED SEPTEMBER & OCTOBER 2016
- BB-SPC-203A — APPROXIMATE LOCATION OF SOIL BORING DRILLED JUNE 2018

NOTES:

1. REFER TO FIGURE 2, "EXPLORATION LOCATION PLAN" FOR BORING LOCATIONS.
2. LINES REPRESENTING CHANGES BETWEEN STRATA ARE BASED UPON INTERPOLATION BETWEEN SUBSURFACE EXPLORATIONS AND MAY NOT REPRESENT ACTUAL FIELD CONDITIONS AT OTHER THAN SPECIFIC EXPLORATION LOCATIONS.
3. REFER TO APPENDIX B FOR LOGS OF TEST BORINGS.

SOURCE:
DRAWING TITLED "PANAM CROSSING PROFILE - TOWN OF SCARBOROUGH, EASTERN TRAIL CONNECTOR, SCARBOROUGH T SOUTH PORTLAND" DATED 27 JULY 2018.

HORIZONTAL 0 100 150 200
VERTICAL 0 20 30 40
SCALE, FEET

FIGURE 4
INTERPRETIVE SUBSURFACE PROFILE
PAN AM CROSSING
GEOTECHNICAL EVALUATION
SCARBOROUGH EASTERN TRAIL
SCARBOROUGH, MAINE

NOVEMBER 2018 PROJECT NO. 0600-102

R.W. Gillespie & Associates
Geotechnical Engineering • Materials Testing Servies
Environmental Consulting • www.rwgillespie.com

R.W. Gillespie & Associates
20 Pomerleau Street, Suite100 Biddeford, Maine 04005 (207) 286-8008
Fax: (207) 710-000 Email: rwg-a@rwg-a.com

NAME: EJW
DATE: NOV/2018 (19 AUG 2019)
JOB NAME: SCARBOROUGH EASTERN TRAIL
JOB NUMBER: 0630-102
SHEET 1 OF

Pile Downdrag Forces

Consider pile downdrag based on settlement evaluations per Maine DOT Bridge Design Guide Section 5.7.6, "as little as ½" of differential settlement may induced down drag forces.

∴ Consider downdrag where differential consolidation settlement between soil and pile, assuming no pile settlement occurs is less than 0.4 inches.

Estimated settlement amounts

	Immediate (in)	Consolidation (in)	Total (in)	Clay Thickness where ρ > 0.4"* (ft)
West Abutment	1.3	2.5	4±	35
East Abutment	1.2	1.1	2.2	20
Pier	–	–	–	–

* See attached ρ vs. depth

Downdrag Material	Pile Embedment Thickness (ft)	γt (pcf)	c (psf)	Φ (deg)	α** Method	Meyerhof SPT*** Method (N60/50)
Embankment Fill	16	105	0	40	–	0.4 ksf
Near Surface Sand	12	125	0	35	–	0.6 ksf
Silty Clay Upper	15	110	750	0	1.0	–
Silty Clay Middle	20	110	450	0	1.0	–

** Article 10.7.3.8.6b
*** Article 10.7.3.8.6g

Load Factor for Downdrag $\gamma_p = 1.4$ per Table 3.4.1-2 (α method) — Strength & Extreme
$= 1.0$ Service Limit State

Resistance Factor for Downdrag $\Phi_{dyn} = 0.65$ per Table 10.5.5.2.3.1 (Dynamic Testing)

R.W. Gillespie & Associates
20 Pomerleau Street, Suite100 Biddeford, Maine 04005 (207) 286-8008
Fax: (207) 710-000 Email: rwg-a@rwg-a.com

NAME: ESW
DATE: NOV 2018 (19 AUG 2018)
JOB NAME: Scarborough Gaetler Trail
JOB NUMBER: 0600-102
SHEET 2 OF

Dowdrag Forces, DD

$$DD = \underset{0}{q_s} A_p \cdot L_E + \underset{0}{q_s} A_p L_S + \underset{cb}{\alpha} S_u A_p \cdot L_{cb} + \underset{cv}{\alpha} S_u A_p \cdot L_{cM}$$

(Embankment + Sfhccal Sand + Upper Silty Clay + Middle Silty Clay)

$$= A_p \cdot [\underset{0}{q_s} L_E + \underset{0}{q_s} L_S + \alpha S_u L_{cL} + \alpha S_u L_c]$$

$$= A_p \cdot [40\text{psf}(16') + 600\text{psf}(12') + 1.0(750\text{psf})15' + 1.0(450\text{psf})20]$$

$$= A_p \cdot [6{,}400\ \text{lb/ft} + 7{,}200\ \text{lb/ft} + 11{,}250\ \text{lb/ft} + 6{,}750\ \text{lb/ft}]$$

$$= A_p \cdot 33{,}850\ \text{lb/ft}$$ where A_p = box area of H-pile per AASHTO C16.7.3.8.6b.

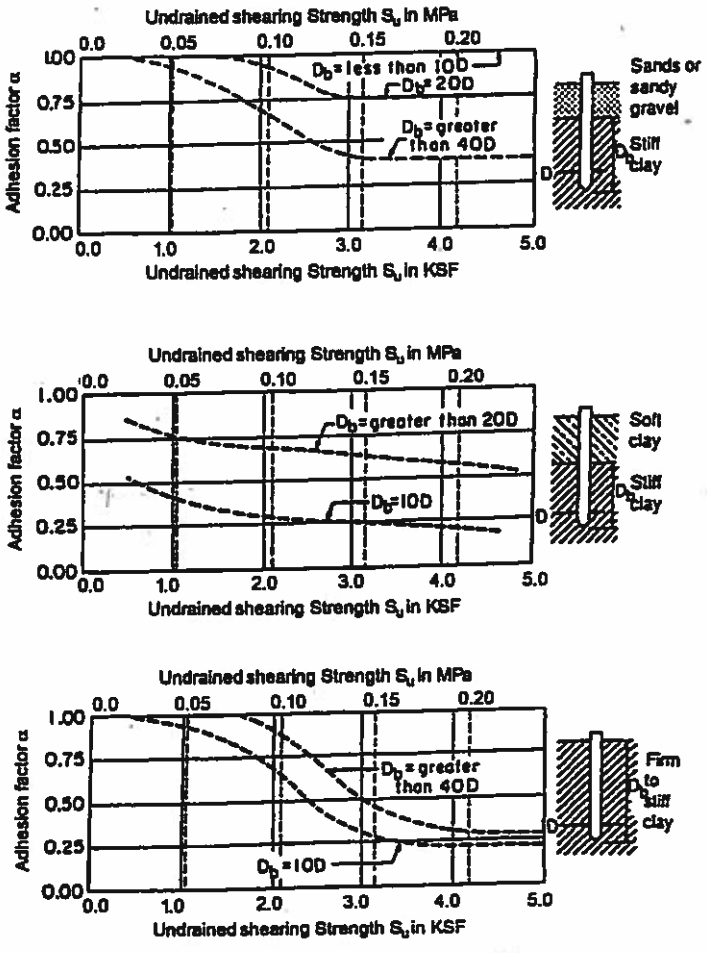


Figure 10.7.3.8.6b-1—Design Curves for Adhesion Factors for Piles Driven into Clay Soils after Tomlinson (1980)

10.7.3.8.6c—β-Method

The β-method, based on effective stress, may be used for predicting side resistance of prismatic piles. The nominal unit skin friction for this method, in ksf, shall be related to the effective stresses in the ground as:

$$q_s = \beta \sigma'_v \qquad (10.7.3.8.6c\text{-}1)$$

where:

σ'_v = vertical effective stress (ksf)

β = a factor taken from Figure 10.7.3.8.6c-1

C10.7.3.8.6c

The β-method has been found to work best for piles in normally consolidated and lightly overconsolidated clays. The method tends to overestimate side resistance of piles in heavily overconsolidated soils. Esrig and Kirby (1979) suggested that for heavily overconsolidated clays, the value of β should not exceed two.

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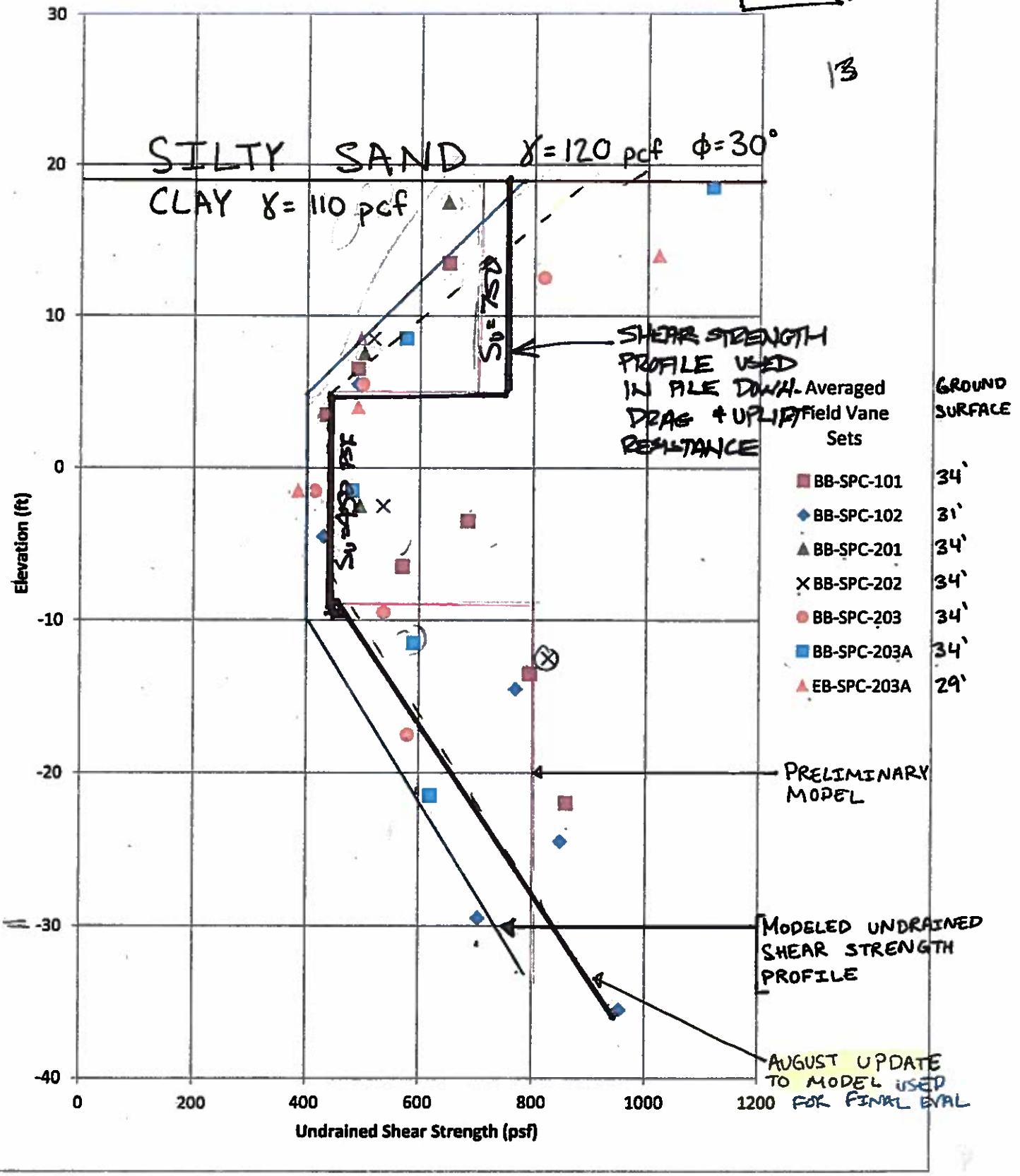
600-102
25 JULY 2018
SNP

Pan Am Crossing Field Vanes - Eastern Trail, Scarborough, Maine

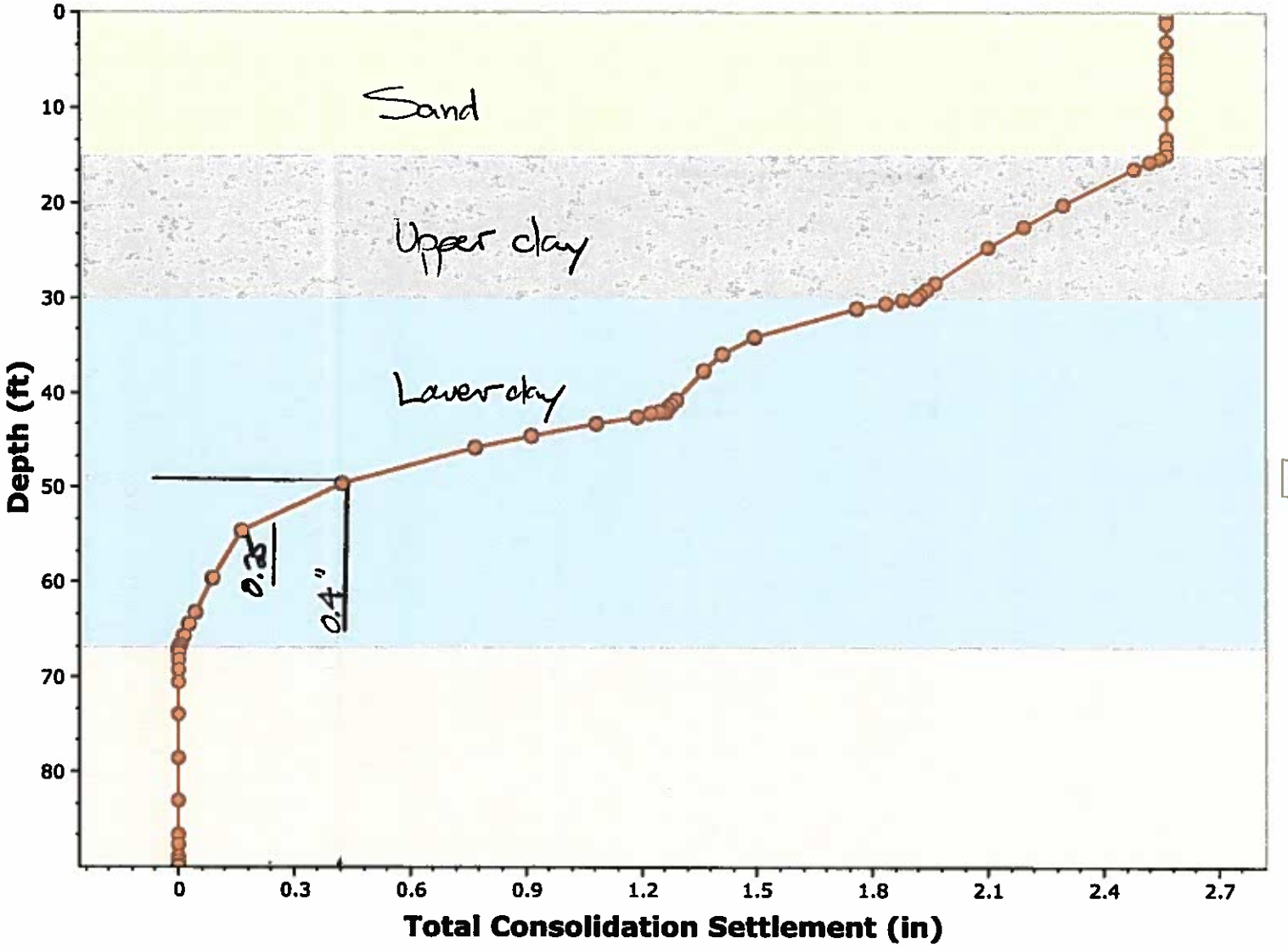
UPDATED AUGUST 2018

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13



Total Consolidation Settlement vs. Depth



West Abutment

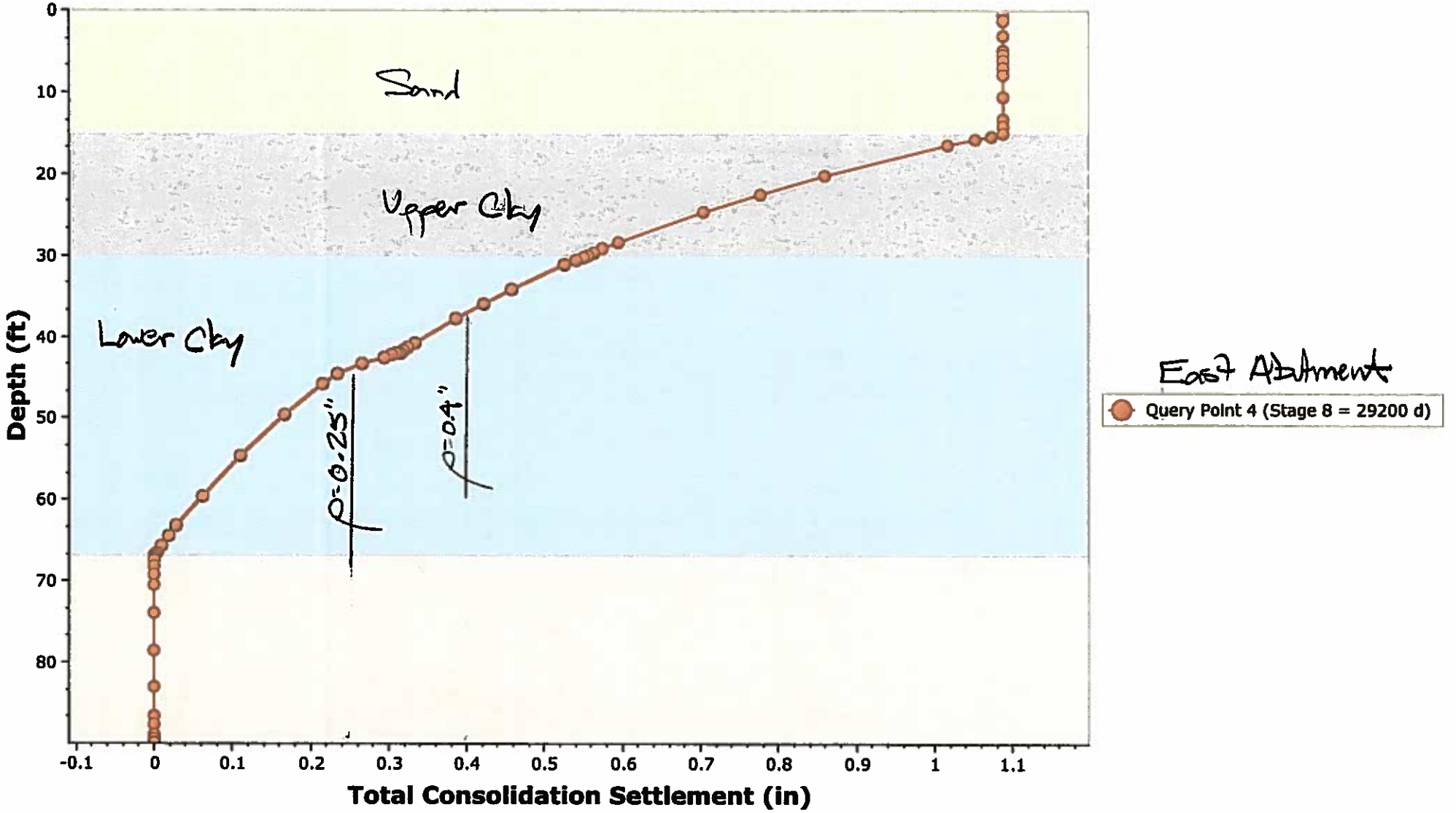
Query Point 8 (Stage 8 = 29200 d)

Reference Stage: None

Project	Eastern Trail Pan-Am Crossing		
Analysis Description	Bridge Approach Embankments		
Drawn By	EJW	Company	R.W. Gillespie & Associates, Inc.
Date	3/10/2018, 6:27:54 AM	File Name	30 Nov 2018.s3z

SETTLE3D 4.016

Total Consolidation Settlement vs. Depth



Reference Stage: None

Project	Eastern Trail Pan-Am Crossing		
Analysis Description	Bridge Approach Embankments		
Drawn By	EJW	Company	R.W. Gillespie & Associates, Inc.
Date	3/10/2018, 6:27:54 AM	File Name	30 Nov 2018.s3z

SETTLE3D 4.016

R.W. Gillespie & Associates
20 Pomerleau Street, Suite100 Biddeford, Maine 04005 (207) 286-8008
Fax: (207) 710-000 Email: rwg-a@rwg-a.com

NAME: EDW
DATE: 21 JAN 2019
JOB NAME: EASTERN TRAIL
JOB NUMBER: 0600-102
SHEET 1 OF 1

Estimated Pile Settlement

- Say abutment group for axial loads is 4x1 with 3-foot separation between piles.
  Use
  Use Q_p = 450 kips per pile (Service Limit State)

Per AASHTO 10.7.2.3.2, use equivalent footing at a depth = 2/3 the pile embedment in strata resisting pile loads

Marine sands are less dense than glacial till. Marine sands are thicker (and therefore would settle more) at the west abutment.

At west abutment, sand thickness = 37 feet ± (N1,60=30)

$$\frac{2D_p}{3} = \frac{2(37)}{3} = 25 \text{ feet} = D'$$

Use B = 12 inches = 1-foot →

$$q = \frac{Q}{A} \quad \frac{450 \text{ kips} \times 4 \text{ piles}}{1 \text{ ft} \times (3' \times 9')} = 67 \text{ ksf}$$

$$\rho \approx \frac{qBI}{N_{1,60}} \qquad I = 1 - 0.125(D'/B) \geq 0.5 \quad \text{(Eq. 10.7.2.3.2)}$$
$$= 1 - 0.125(25/1) = -2 \rightarrow \text{Use } 0.5$$

$$\rho = \frac{67(1 \text{ ft})0.5}{30} = 1.1 \text{ inches (overestimate per C10.7.2.3.2)}$$

Piles will bear in glacial till at End of Drive

∴ $N_{1,60} > 30$

∴ Settlement will be less than 1 inch

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SHEET 1 OF 1

+ Boring Logs

Drivability Evaluation - WEAP Evaluation ∴ Evaluate for 10 blows/in

Substructure	Strength Factored Design Resistance (kips)		Prelim Pile Length Below Current Grade	Below Dredge Elev	Reference Soil Boring Log	Depth Below Bottom of Clay
East Abutment	350±	780±	97 ft	105 ft	BB-SPC-201	42
Pier	160	360±	85 ft	80± ft	BB-SPC-202	12
10%-6% West Abutment	350	780	108 ft	116 ft	BB-SPC-203	42±

↳ Service + Extreme Resistance

Estimated Hammer Energy

$R_{ndr} = 1.75\sqrt{E_d}\log(10N_b) - 100$ R_{ndr} in kips

→ N_b = 15 blows per inch (Use 10-15)
10 blows per inch

$\sqrt{E_d} = \left(\frac{R_{ndr}+100}{1.75\log 10N_b}\right)^2$

Abutments:

$E_d = \left(\frac{350+100}{1.75\log(150)}\right)^2 =$ Strength: 13,900 ft-lb (N_b = 15 bl/in); 16,500 ft-lb (N_b = 10 bl/in)

Service + Ex → 53,400 ft-lb (N_b = 15 bl/in); 63,200 ft-lb (N_b = 10 bl/in)

Pier

$E_d = \left(\frac{160+100}{1.75\log(150)}\right)^2 =$ Strength: 4,700 ft-lb (N_b = 15 blows/in); 5,500 ft-lb (N_b = 0 blows/in)

14,600 ft-lb (N_b = 5 bl/in); 17,300 ft-lb (N_b = 10 bl/in)

↳ Service + Extrem

Preliminary Pile Types HP 10x42
HP 12x53
HP 12x74

HP10x42

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Eastern Trail - Scarborough 0600-102

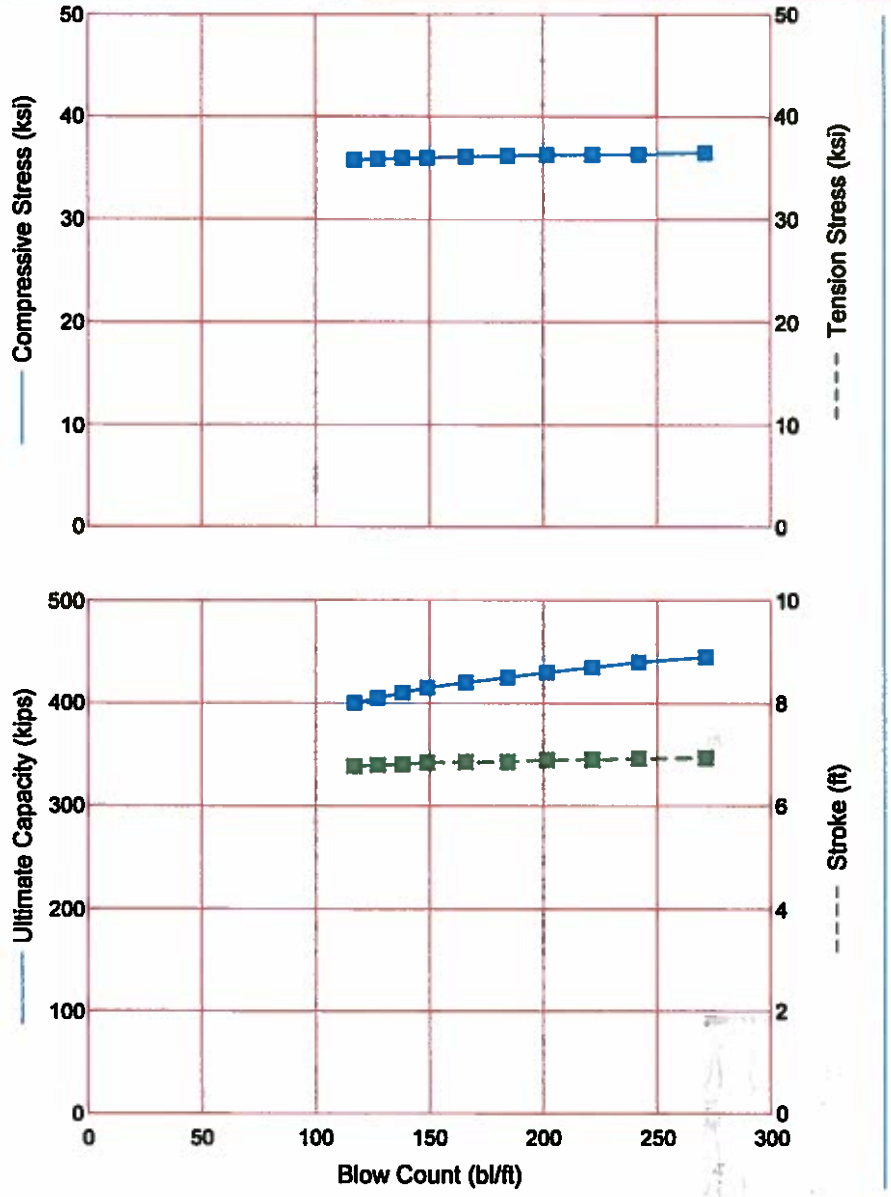
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GRLWEAP (TM) Version 2005

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count bl/ft		Stroke ft	Energy kips-ft
400.0	35.83	0.00	117.0	10 bpi	6.77	28.47
405.0	35.90	0.00	127.3		6.79	28.49
410.0	35.98	0.00	138.1		6.81	28.55
415.0	36.02	0.00	149.0		6.84	28.67
420.0	36.14	0.00	165.9		6.85	28.63
425.0	36.22	0.00	184.4	15 bpi	6.86	28.61
430.0	36.29	0.00	201.7		6.89	28.65
435.0	36.33	0.00	221.6		6.90	28.65
440.0	36.36	0.00	241.9		6.92	28.71
445.0	36.49	0.00	271.3		6.93	28.68

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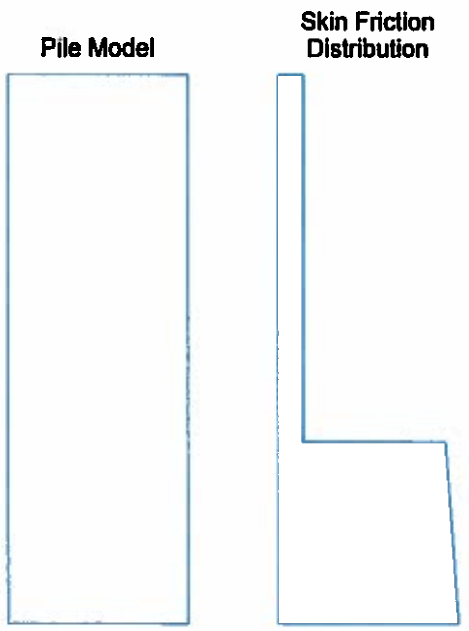
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HP10x42



DELMAG D 30

Efficiency	0.800	
Helmet	1.24	kips
Skin Quake	0.100	in
Toe Quake	0.167	in
Skin Damping	0.088	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	105.00	ft
Pile Penetration	105.00	ft
Pile Top Area	12.40	in2



Res. Shaft = 86 %
(Proportional)

HP 12x53

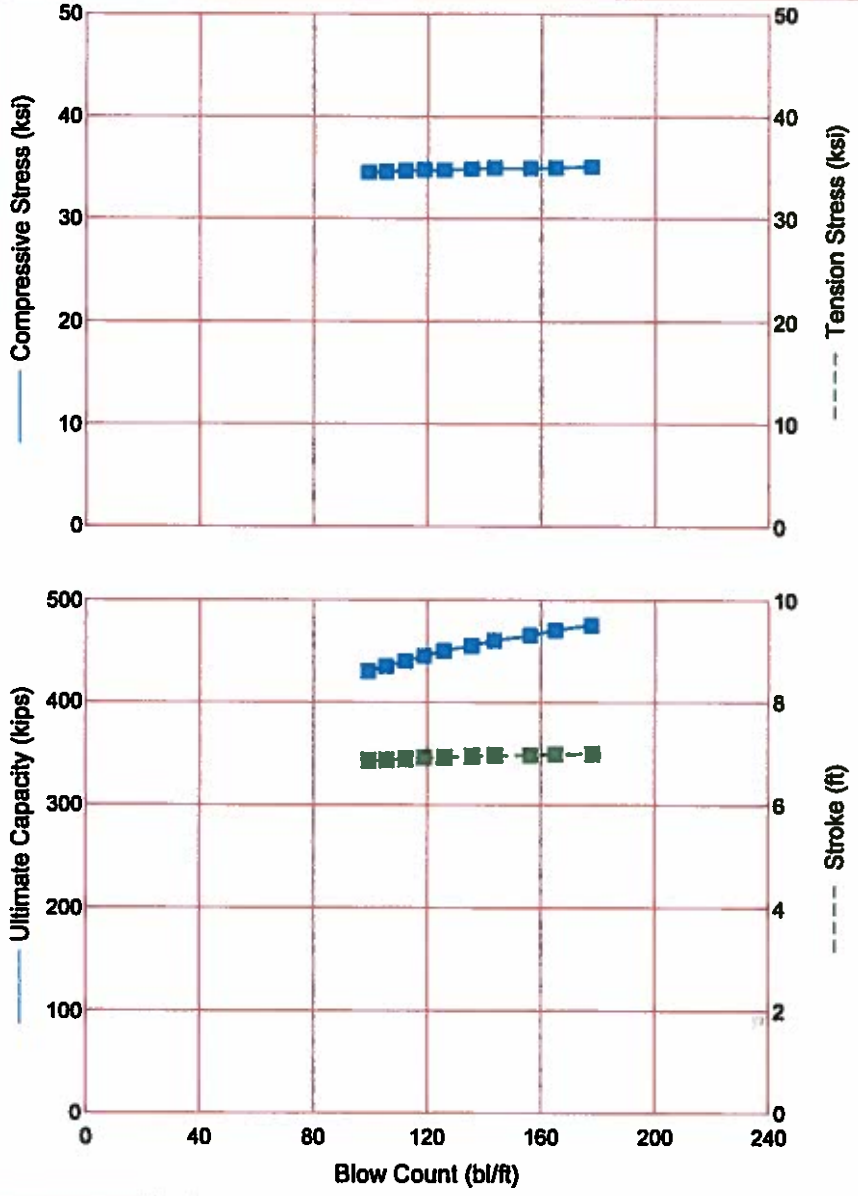
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Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count bl/ft		Stroke ft	Energy kips-ft
430.0	34.49	0.00	99.4		6.87	27.19
435.0	34.55	0.00	105.6		6.88	27.18
440.0	34.64	0.00	112.3		6.90	27.16
445.0	34.73	0.00	118.9		6.92	27.20
450.0	34.70	0.00	125.9	10bpi	6.93	27.22
455.0	34.84	0.00	135.6		6.95	27.18
460.0	34.91	0.00	143.7		6.97	27.24
465.0	34.89	0.00	156.3		6.97	27.16
470.0	34.95	0.00	165.1		6.99	27.26
475.0	35.06	0.00	177.8	15bpi	7.00	27.23

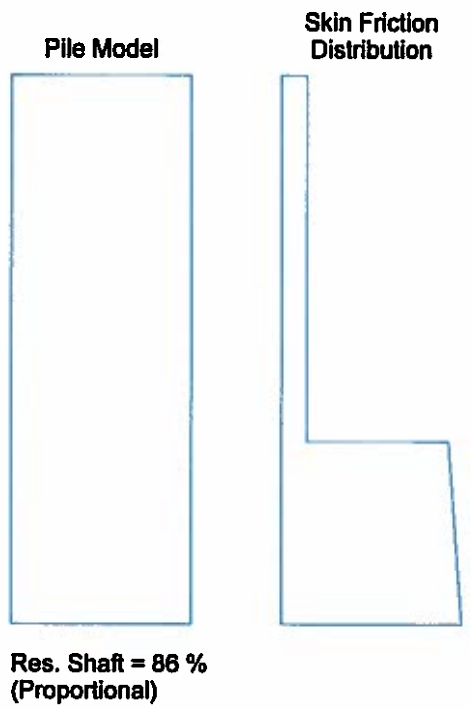
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Toe Damping	0.150	sec/ft
Pile Length	105.00	ft
Pile Penetration	105.00	ft
Pile Top Area	15.50	in2



Res. Shaft = 86 %
(Proportional)

HP 12x74

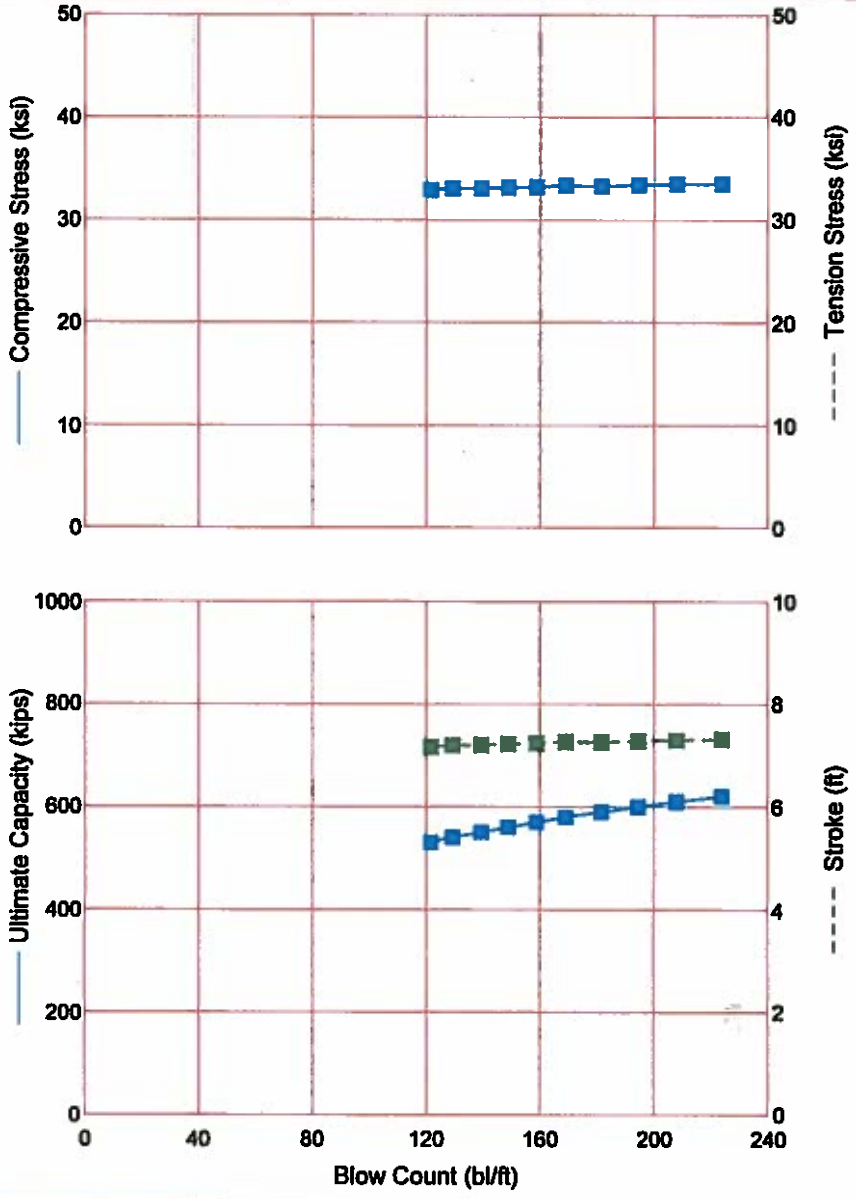
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Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count bl/ft		Stroke ft	Energy kips-ft
530.0	32.95	0.00	121.7		7.15	25.32
540.0	33.10	0.00	129.3		7.18	25.42
550.0	33.11	0.00	139.5	10 bpi	7.20	25.38
560.0	33.16	0.00	148.9		7.22	25.40
570.0	33.23	0.00	158.8		7.23	25.39
580.0	33.37	0.00	169.1		7.26	25.44
590.0	33.31	0.00	181.6	15 bpi	7.26	25.35
600.0	33.45	0.00	194.6		7.28	25.32
610.0	33.52	0.00	208.0		7.29	25.40
620.0	33.56	0.00	223.8		7.31	25.39

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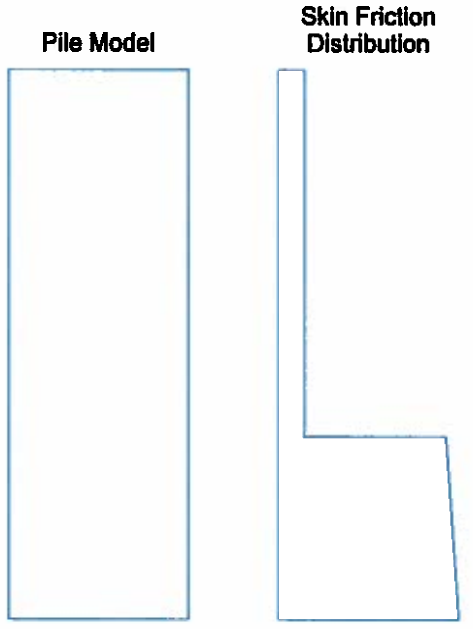
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