APPENDIX 'A'

GEOTECHNICAL REPORTS

Quality Engineering | Valued Relationships

Morrison Hershfield

Empress Street Rehabilitation
Sub-Surface Investigation

Prepared for:

Morrison Hershfield
25 Scurfield Blvd, Unit 1
Winnipeg, MB R3Y 1G4
Attention: Brad Sacher

Distribution:

Project Number:
0035-037-00

Date:
January 24, 2017
Final Report

Quality Engineering | Valued Relationships

January 24, 2017 Our File No. 0035-037-00

Brad Sacher, P.Eng.
Morrison Hershfield
59 Scurfield Blvd, Unit 1
Winnipeg, MB R3Y 1V2

RE: Empress Street Rehabilitation
Sub-Surface Investigation Report

TREK Geotechnical Inc. is pleased to submit our report for the road sub-surface investigations for the Empress Street Rehabilitation project.

Please contact the undersigned if you have any questions. Thank you for the opportunity to serve you on this assignment.

Sincerely,

TREK Geotechnical Inc.
Per:

Nelson John Ferreira, Ph.D., P. Eng.
Geotechnical Engineer, Principal
Tel: 204.975.9433 ext. 103

cc: Paul Bevel, B.Sc., (TREK Geotechnical)

Revision History

Revision No.	Author	Issue Date	Description
0	PB	January 24, 2017	Final Report

Authorization Signatures

Prepared By:

Paul Bevel, B.Sc.

PROVINCE OF MANITOBA
N.J. FERREIRA
Member
22892
REGISTERED PROFESSIONAL ENGINEER
Jan. 24, 2017

Reviewed By:

Nelson John Ferreira, Ph.D., P.Eng.
Geotechnical Engineer

APEGM
Certificate of Authorization
Trek Geotechnical Inc.
No. 4877 Date: Jan 24 2017

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1.0 Introduction

This report summarizes the results of the road sub-surface investigation completed for the Empress Street Rehabilitation project. Information collected describes the pavement structure of the existing road as well as the soil stratigraphy beneath the pavement structure. A riverbank sub-surface investigation has also been carried out for this project. This information will be included in a separate report.

2.0 Sub-Surface Investigation and Laboratory Program

A total of 17 test holes were drilled approximately every 100 m at the locations shown on Figure 01. The sub-surface investigation was conducted between November 01, 2016 and November 03, 2016. The road test holes were drilled to a depth of 3.1 m below road surface with the exception of RH16-03 to RH16-07 which were drilled deeper than 3.1 m, to power auger refusal (PAR). Test holes TH16-01 and TH16-02 were drilled as part of the riverbank sub-surface investigation. Roadway test holes RH16-03 to RH16-07 were drilled deeper to gain additional information for the riverbank assessment. The drilling was performed by Paddock Drilling Ltd. using their Acker RM5 truck mounted drill rig equipped with 125 mm diameter solid stem augers. The pavement structure (asphalt/concrete) was cored by Paul Bevel, B.Sc. of TREK Geotechnical Inc. (TREK) using a portable coring press equipped with a hollow 150 mm diameter diamond core drill bit. The sub-surface conditions observed during drilling were visually classified by Junhui Wu of TREK. Other pertinent information such as sloughing, seepage, groundwater and drilling conditions were also recorded. Disturbed (auger cuttings) samples retrieved during the sub-surface investigation were transported to TREK's material testing laboratory for further testing. Core samples were also retrieved and logged at TREK's material testing laboratory.

The laboratory testing program consisted of moisture content determination, Atterberg limits, and grain size analysis (mechanical sieve and hydrometer methods) on select samples. Laboratory testing results are included on the test hole logs in Appendix A, while the individual test results are included in Appendix B with a summary table. Photos of the asphalt and concrete pavement cores are included in Appendix C. Test hole locations noted on the test hole logs were determined using a handheld GPS.

3.0 Closure

The information provided in this report is in accordance with current engineering principles and practices (Standard of Practice). The findings of this report were based on information provided (field investigation, laboratory testing, geometries). Soil conditions are natural deposits that can be highly variable across a site. If sub-surface conditions are different than the conditions previously encountered on-site or those presented here, we should be notified to adjust our findings if necessary.

All information provided in this report is subject to our standard terms and conditions for engineering services, a copy of which is provided to each of our clients with the original scope of work, or a mutually executed standard engineering services agreement. If these conditions are not attached, and you are not already in possession of such terms and conditions, contact our office and you will be promptly provided with a copy.

This report has been prepared by TREK Geotechnical Inc. (the Consultant) for the exclusive use of Morrison Hershfield (the Client) and their agents for the work product presented in the report. Any findings or recommendations provided in this report are not to be used or relied upon by any third parties, except as agreed to in writing by the Client and Consultant prior to use.

Figure 1

Test Hole Location Plan

0 25 50 75 100 m
SCALE = 1 : 2 000 (279 mm x 432 mm)

LEGEND: TEST HOLE (TREK, 2016)

NOTES: 1. AERIAL IMAGE FROM THE CITY OF WINNIPEG 2016.

Figure 01
Test Hole Location Plan

Appendix A

Test Hole Logs

EXPLANATION OF FIELD AND LABORATORY TESTING

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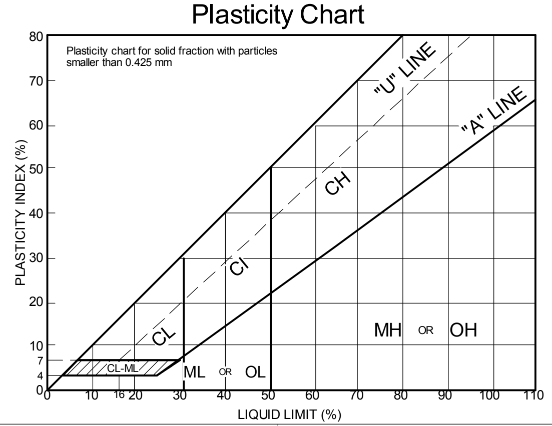
GENERAL NOTES

1. Classifications are based on the United Soil Classification System and include consistency, moisture, and color. Field descriptions have been modified to reflect results of laboratory tests where deemed appropriate.
2. Descriptions on these test hole logs apply only at the specific test hole locations and at the time the test holes were drilled. Variability of soil and groundwater conditions may exist between test hole locations.
3. When the following classification terms are used in this report or test hole logs, the primary and secondary soil fractions may be visually estimated.

Major Divisions			USCS Classi-fication	Symbols	Typical Names	Laboratory Classification Criteria		
Coarse-Grained soils (More than half the material is larger than No. 200 sieve size)	Gravels (More than half of coarse fraction is larger than 4.75 mm)	Clean gravel (Little or no fines)	GW		Well-graded gravels, gravel-sand mixtures, little or no fines	Determine percentages of sand and gravel from grain size curve, depending on percentage of fines (fraction smaller than No. 200 sieve) coarse-grained soils are classified as follows: Less than 5 percent...... GW, GP, SW, SP; More than 12 percent....... GM, GC, SM, SC; 6 to 12 percent...... Borderline case4s requiring dual symbols*	$C_U = \frac{D_{60}}{D_{10}}$ greater than 4; $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			GP		Poorly-graded gravels, gravel-sand mixtures, little or no fines		Not meeting all gradation requirements for GW	
		Gravel with fines (Appreciable amount of fines)	GM		Silty gravels, gravel-sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4	Above "A" line with P.I. between 4 and 7 are border-line cases requiring use of dual symbols
			GC		Clayey gravels, gravel-sand-silt mixtures		Atterberg limits above "A" line or P.I. greater than 7	
	Sands (More than half of coarse fraction is smaller than 4.75 mm)	Clean sands (Little or no fines)	SW		Well-graded sands, gravelly sands, little or no fines		$C_U = \frac{D_{60}}{D_{10}}$ greater than 6; $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			SP		Poorly-graded sands, gravelly sands, little or no fines		Not meeting all gradation requirements for SW	
		Sands with fines (Appreciable amount of fines)	SM		Silty sands, sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4	Above "A" line with P.I. between 4 and 7 are border-line cases requiring use of dual symbols
			SC		Clayey sands, sand-clay mixtures		Atterberg limits above "A" line or P.I. greater than 7	
Fine-Grained soils (More than half the material is smaller than No. 200 sieve size)	Silts and Clays (Liquid limit less than 50)		ML		Inorganic silts and very fine sands, rock floor, silty or clayey fine sands or clayey silts with slight plasticity	Plasticity Chart		
			CL		Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays			
			OL		Organic silts and organic silty clays of low plasticity			
	Silts and Clays (Liquid limit greater than 50)		MH		Inorganic silts, micaceous or distomaceous fine sandy or silty soils, organic silts			
			CH		Inorganic clays of high plasticity, fat clays			
			OH		Organic clays of medium to high plasticity, organic silts			
	Highly Organic Soils		Pt		Peat and other highly organic soils	Von Post Classification Limit	Strong colour or odour, and often fibrous texture	

Plasticity Chart



Material		Particle Size: mm	Particle Size: ASTM Sieve sizes
Sand	Coarse	2.00 to 4.75	#10 to #4
	Medium	0.425 to 2.00	#40 to #10
	Fine	0.075 to 0.425	#200 to #40
Silt or Clay		< 0.075	< #200

Material		Particle Size: mm	Particle Size: ASTM Sieve Sizes
Boulders		> 300	> 12 in.
Cobbles		75 to 300	3 in. to 12 in.
Gravel	Coarse	19 to 75	3/4 in. to 3 in.
	Fine	4.75 to 19	#4 to 3/4 in.

* Borderline classifications used for soils possessing characteristics of two groups are designated by combinations of groups symbols. For example; GW-GC, well-graded gravel-sand mixture with clay binder.

Other Symbol Types

	Asphalt		Bedrock (undifferentiated)		Cobbles
	Concrete		Limestone Bedrock		Boulders and Cobbles
	Fill		Cemented Shale		Silt Till
			Non-Cemented Shale		Clay Till

EXPLANATION OF FIELD AND LABORATORY TESTING

LEGEND OF ABBREVIATIONS AND SYMBOLS

LL - Liquid Limit (%)
PL - Plastic Limit (%)
PI - Plasticity Index (%)
MC - Moisture Content (%)
SPT - Standard Penetration Test
RQD - Rock Quality Designation
Qu - Unconfined Compression
Su - Undrained Shear Strength
VW - Vibrating Wire Piezometer
SI - Slope Inclinometer

▽ Water Level at Time of Drilling

▼ Water Level at End of Drilling

▼ Water Level After Drilling as Indicated on Test Hole Logs

FRACTION OF SECONDARY SOIL CONSTITUENTS ARE BASED ON THE FOLLOWING TERMINOLOGY

TERM	EXAMPLES	PERCENTAGE
and	and CLAY	35 to 50 percent
"y" or "ey"	clayey, silty	20 to 35 percent
some	some silt	10 to 20 percent
trace	trace gravel	1 to 10 percent

TERMS DESCRIBING CONSISTENCY OR COMPACTION CONDITION

The Standard Penetration Test blow count (N) of a non-cohesive soil can be related to compactness condition as follows:

Descriptive Terms	SPT (N) (Blows/300 mm)
Very loose	< 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	> 50

The Standard Penetration Test blow count (N) of a cohesive soil can be related to its consistency as follows:

Descriptive Terms	SPT (N) (Blows/300 mm)
Very soft	< 2
Soft	2 to 4
Firm	4 to 8
Stiff	8 to 15
Very stiff	15 to 30
Hard	> 30

The undrained shear strength (Su) of a cohesive soil can be related to its consistency as follows:

Descriptive Terms	Undrained Shear Strength (kPa)
Very soft	< 12
Soft	12 to 25
Firm	25 to 50
Stiff	50 to 100
Very stiff	100 to 200
Hard	> 200

Sub-Surface Log

TREK GEOTECHNICAL

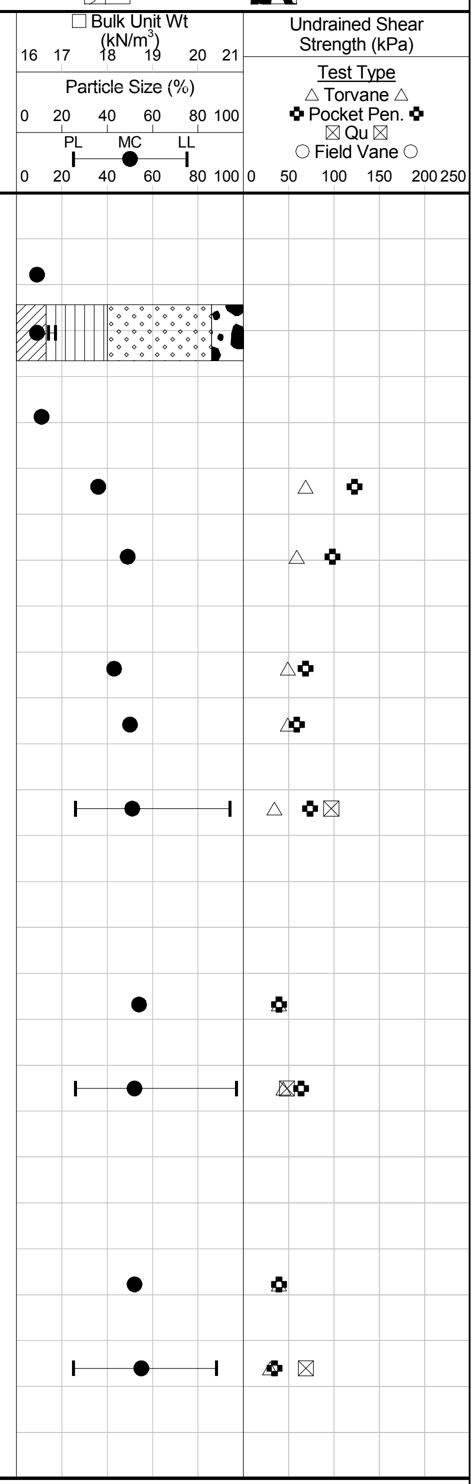
Client:	Morrison Hershfield	**Project Number:**	0035-037-00
Project Name:	Empress Widening	**Location:**	Empress St. From Portage to St. Matthews
Contractor:	Paddock Drilling Ltd.	**Ground Elevation:**	233.10 m
Method:	125 mm Solid Stem Auger, Acker RM5	**Date Drilled:**	2 November 2016

Sample Type: Grab (G) | Shelby Tube (T) | Split Spoon (SS) | Split Barrel (SB) | Core (C)

Particle Size Legend: Fines | Clay | Silt | Sand | Gravel | Cobbles | Boulders

Elevation (m)	Depth (m)	MATERIAL DESCRIPTION	Sample Type	Sample Number	SPT (N)
233.1	0.0	ASPHALT (40 mm thick)			
232.8		CONCRETE (240 mm thick)			
		SAND (FILL) - silty, some clay, some gravel (<40 mm dia.) - brown - moist, compact - poorly graded, sub-angular to angular	Grab	G74	
			Grab	G75	
			Grab	G76	
231.6	1.5	CLAY - silty, trace silt inclusions (<10 mm dia.) - brown - moist, stiff - high plasticity	Grab	G77	
			Grab	G78	
			Grab	G79	
			Grab	G80	
		- firm below 3.1 m	Shelby Tube	T81	
			Grab	G82	
			Shelby Tube	T83	
			Grab	G84	
			Shelby Tube	T85	

Depth scale (m): 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0, 5.5, 6.0, 6.5



Logged By: Junhui Wu **Reviewed By:** Nelson Ferreira **Project Engineer:** Michael Van Helden

TREK GEOTECHNICAL

Sub-Surface Log

Elevation (m)	Depth (m)	Soil Symbol	MATERIAL DESCRIPTION	Sample Type	Sample Number	SPT (N)
			- grey, soft below 7.3 m		G86	
					T87	
					G88	
223.0	10.0		SILT (TILL) - trace clay, trace sand, trace gravel - brown - moist, compact - no to low plasticity		G89	
					G90	
220.8					SS91	

Chart headings: Bulk Unit Wt (kN/m^3) 16 17 18 19 20 21; Particle Size (%) 0 20 40 60 80 100; PL MC LL 0 20 40 60 80 100; Undrained Shear Strength (kPa) — Test Type: △ Torvane △, ✜ Pocket Pen. ✜, ☒ Qu ☒, ○ Field Vane ○; 0 50 100 150 200 250

Depth scale: 7.5, 8.0, 8.5, 9.0, 9.5, 10.0, 10.5, 11.0, 11.5, 12.0

END OF TEST HOLE AT 12.3 m DEPTH IN SILT TILL
Notes:
1. Power auger refusal at 12.3 m depth.
2. Seepage observed below 12.2 m depth
3. Test hole open to 12.2 m depth after completion of test hole.
4. Water level observed at 5.2 m depth after completion of test hole.
5. Test hole backfilled with auger cuttings, bentonite, sand, and cold patch asphalt to surface.
6. Test hole location: North bound curb lane of Empress St., 1.7 m west from east curb, 450 m south of Eastway, 14U 5527650N, 629535E.

Logged By: Junhui Wu **Reviewed By:** Nelson Ferreira **Project Engineer:** Michael Van Helden

Sub-Surface Log

Client: Morrison Hershfield
Project Number: 0035-037-00
Project Name: Empress Widening
Location: Empress St. From Portage to St. Matthews
Contractor: Paddock Drilling Ltd.
Ground Elevation: 233.39 m
Method: 125 mm Solid Stem Auger, Acker RM5
Date Drilled: 2 November 2016

Sample Type: Grab (G) Shelby Tube (T) Split Spoon (SS) Split Barrel (SB) Core (C)

Particle Size Legend: Fines Clay Silt Sand Gravel Cobbles Boulders

Elevation (m)	Depth (m)	MATERIAL DESCRIPTION	Sample Number
233.3		ASPHALT (37 mm thick)	
233.1		CONCRETE (237 mm thick)	
233.0		SAND (FILL) - some silt, some gravel (<20 mm dia.), trace to some clay - brown, moist, compact	G104
	0.5		G105
232.6		ORGANIC CLAY - silty - black, moist, soft, high plasticity	
232.3	1.0	SILT - clayey, trace to some sand - light brown, moist to wet, soft, intermediate plasticity	G106
		CLAY - silty, trace silt inclusions (<10 mm dia.) - brown - moist, stiff - high plasticity	G107
	1.5		G108
	2.0	- firm below 2.1 m	G109
	2.5		
	3.0		G110
	3.5		
	4.0		
	4.5		G111
	5.0		T112
	5.5		
	6.0	- grey below 5.8 m	G113
	6.5		

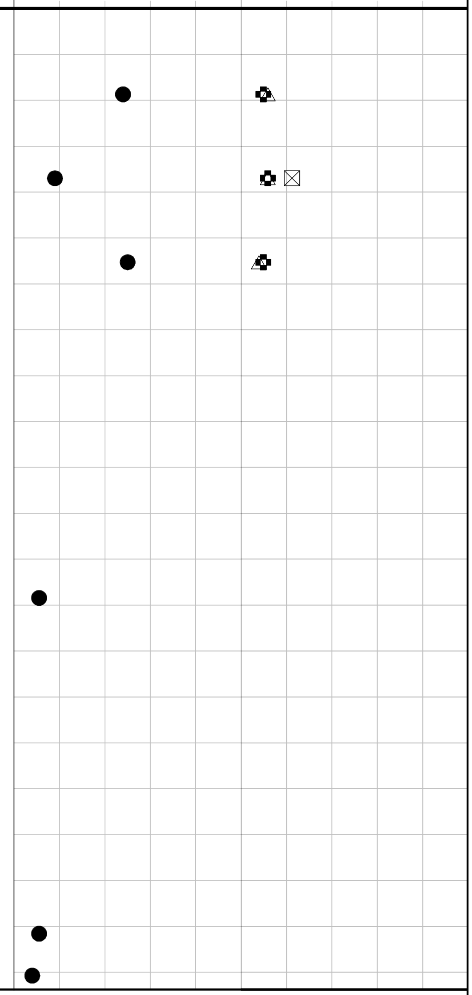
Column headings: Elevation (m); Depth (m); Soil Symbol; MATERIAL DESCRIPTION; Sample Type; Sample Number; SPT (N); Bulk Unit Wt (kN/m³) 16 17 18 19 20 21; Particle Size (%) 0 20 40 60 80 100; PL MC LL 0 20 40 60 80 100; Undrained Shear Strength (kPa) Test Type △ Torvane △ ✿ Pocket Pen. ✿ ☒ Qu ☒ ○ Field Vane ○ 0 50 100 150 200 250

TREK GEOTECHNICAL

Sub-Surface Log

Elevation (m)	Depth (m)	Soil Symbol	MATERIAL DESCRIPTION	Sample Type	Sample Number	SPT (N)
	7.5 – 10.0				G114, T115, G116	
223.3	10.0		SILT (TILL) - some sand, trace clay, trace gravel (<10 mm dia.) - grey - moist, compact - no to low plasticity		G117	
221.0	12.3				G118, G119	

Bulk Unit Wt (kN/m^3): 16 17 18 19 20 21
Particle Size (%): 0 20 40 60 80 100
PL MC LL: 0 20 40 60 80 100
Undrained Shear Strength (kPa)
Test Type
△ Torvane △
✥ Pocket Pen. ✥
☒ Qu ☒
○ Field Vane ○
0 50 100 150 200 250



END OF TEST HOLE AT 12.3 m DEPTH IN SILT TILL
Notes:
1. Power auger refusal at 12.3 m depth.
2. Seepage observed below 12.2 m depth
3. Test hole open to 12.0 m depth after completion of test hole.
4. Water level observed at 12.0 m depth after completion of test hole.
5. Test hole backfilled with auger cuttings, bentonite, sand, and cold patch asphalt to surface.
6. Test hole location: North bound curb lane of Empress St., 1.7 m west from east curb, 75 m south of Maroons Rd., (14U 5527640N, 629563E).

Logged By: Junhui Wu **Reviewed By:** Nelson Ferreira **Project Engineer:** Michael Van Helden

Sub-Surface Log

TREK GEOTECHNICAL

Client:	Morrison Hershfield	**Project Number:**	0035-037-00
Project Name:	Empress Widening	**Location:**	Empress St. From Portage to St. Matthews
Contractor:	Paddock Drilling Ltd.	**Ground Elevation:**	233.58 m
Method:	125 mm Solid Stem Auger, Acker RM5	**Date Drilled:**	3 November 2016

Sample Type: Grab (G) | Shelby Tube (T) | Split Spoon (SS) | Split Barrel (SB) | Core (C)

Particle Size Legend: Fines | Clay | Silt | Sand | Gravel | Cobbles | Boulders

Elevation (m)	Depth (m)	MATERIAL DESCRIPTION	Sample Type	Sample Number
233.5		ASPHALT (30 mm thick)		
233.3		CONCRETE (243 mm thick)		
233.2		SAND (FILL) - some silt, some gravel (<20 mm dia.), trace to some clay - brown, moist, compact	Grab	G120, G121
233.0	0.5	CLAY - silty, brown, moist, firm, high plasticity		
		SILT - some clay, some sand - light brown - moist to wet, soft, non plastic	Grab	G122
232.5	1.0			
232.4		CLAY - silty, trace silt inclusions (<10 mm dia.) - brown, moist, stiff, high plasticity	Grab	G123
		SILT - clayey - light brown - moist, soft - low plasticity	Grab	G124, G125
231.4	2.0	CLAY - silty, trace silt inclusions (<10 mm dia.) - brown - moist, stiff - high plasticity	Grab	G126
	3.7	- firm below 3.7 m	Grab	G127
			Grab	G128
			Grab	G129
			Shelby Tube	T130

Depth scale: 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0, 5.5, 6.0, 6.5

Columns: SPT (N); Bulk Unit Wt (kN/m³) 16 17 18 19 20 21; Particle Size (%) 0 20 40 60 80 100; PL MC LL 0 20 40 60 80 100; Undrained Shear Strength (kPa) Test Type △ Torvane △, ✪ Pocket Pen. ✪, ⊠ Qu ⊠, ○ Field Vane ○ 0 50 100 150 200 250

Logged By: Junhui Wu **Reviewed By:** Nelson Ferreira **Project Engineer:** Michael Van Helden

TREK GEOTECHNICAL

Sub-Surface Log

Elevation (m)	Depth (m)	Soil Symbol	MATERIAL DESCRIPTION	Sample Type	Sample Number	SPT (N)
226.3			CLAY - silty, trace sand, trace gravel (<15 mm dia.) - grey - moist, soft - high plasticity			
	7.5				G131	
	8.0					
	8.5					
	9.0				G132	
					T133	
	9.5					
223.8			SILT (TILL) - some sand, trace clay, trace gravel (<10 mm dia.) - light brown - moist, compact - no to low plasticity			
	10.0					
	10.5				G134	
	11.0					
	11.5					
	12.0				G135	
			- moist, dense below 12.2 m			
	12.5					
	13.0					
					G136	
220.1						

Bulk Unit Wt (kN/m^3): 16 17 18 19 20 21

Particle Size (%): 0 20 40 60 80 100

PL MC LL: 0 20 40 60 80 100

Undrained Shear Strength (kPa)

Test Type
△ Torvane △
✚ Pocket Pen. ✚
☒ Qu ☒
○ Field Vane ○

0 50 100 150 200 250

END OF TEST HOLE AT 13.5 m DEPTH IN SILT TILL
Notes:
1. Power auger refusal at 13.5 m depth.
2. Sloughing observed at 2.0 m, Seepage observed below 13.4 m depth
3. Test hole open to 2.1 m depth after completion of test hole.
4. Test hole backfilled with auger cuttings, bentonite, sand and cold patch asphalt to groud surface.
5. Test hole location: North bound curb lane of Empress St., 1.7 m west from east curb, 35 m north of Maroons Rd., (14U 5527751N, 629564E).

Logged By: Junhui Wu **Reviewed By:** Nelson Ferreira **Project Engineer:** Michael Van Helden

Sub-Surface Log

Client:	Morrison Hershfield	**Project Number:**	0035-037-00
Project Name:	Empress Widening	**Location:**	Empress St. From Portage to St. Matthews
Contractor:	Paddock Drilling Ltd.	**Ground Elevation:**	233.59 m
Method:	125 mm Solid Stem Auger, Acker RM5	**Date Drilled:**	3 November 2016

Sample Type: Grab (G) | Shelby Tube (T) | Split Spoon (SS) | Split Barrel (SB) | Core (C)

Particle Size Legend: Fines | Clay | Silt | Sand | Gravel | Cobbles | Boulders

Columns: Elevation (m) | Depth (m) | Soil Symbol | MATERIAL DESCRIPTION | Sample Type | Sample Number | SPT (N) | Bulk Unit Wt (kN/m^3) 16 17 18 19 20 21; Particle Size (%) 0 20 40 60 80 100; PL MC LL 0 20 40 60 80 100 | Undrained Shear Strength (kPa) — Test Type: △ Torvane △, ✤ Pocket Pen. ✤, ⊠ Qu ⊠, ○ Field Vane ○; 0 50 100 150 200 250

Elevation (m)	Material Description
233.6	ASPHALT (25 mm thick)
233.3	CONCRETE (235 mm thick)
233.3	SAND (FILL) - gravelly, trace clay, trace silt, grey, moist, compact, well graded fine sand to fine gravel, sub-angular to angular, carbonate (limestone)
233.0	CLAY - silty, brown, moist, stiff, high plasticity
	SILT AND CLAY - some sand - brown - moist, soft - intermediate plasticity
232.4	CLAY - silty, trace oxidation - brown - moist, stiff - high plasticity - 0.1 m thick of silt layer at 2.0 m depth
231.2	SILT - some clay, some fine sand - brown - moist, soft - no plasticity
230.5	CLAY - silty, trace silt inclusions (<10 mm dia.), trace oxidation - brown - moist, firm - high plasticity - grey below 4.6 m - soft below 6.1 m

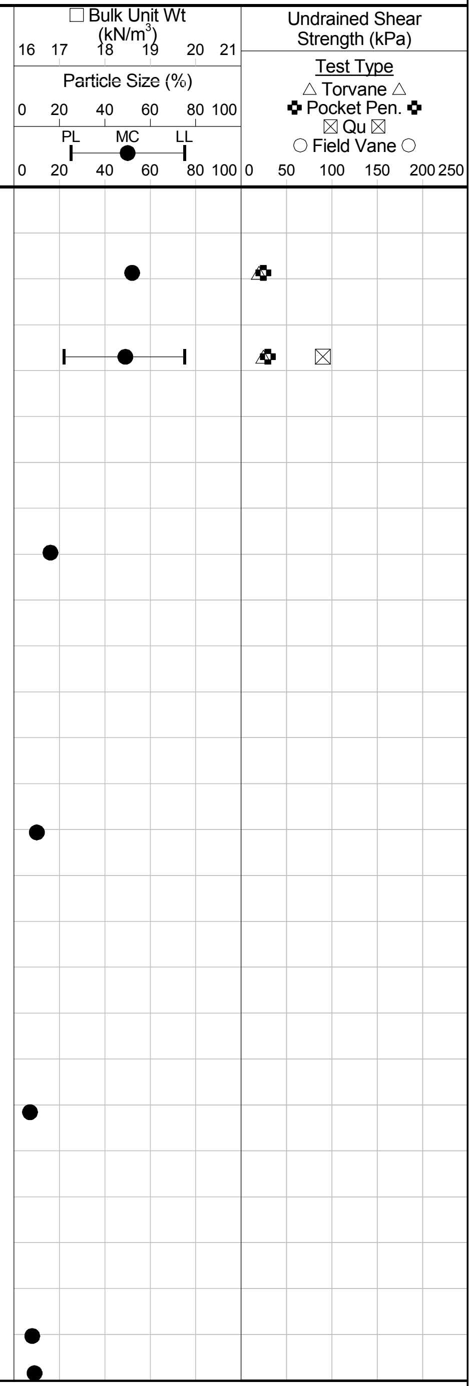
Samples: G137, G138, G139, G140, G141, G142, G143, G144, G145, G146, T147, G148

Depth scale (m): 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0, 5.5, 6.0, 6.5

Logged By: Junhui Wu **Reviewed By:** Nelson Ferreira **Project Engineer:** Michael Van Helden

Sub-Surface Log

Elevation (m)	Depth (m)	MATERIAL DESCRIPTION	Sample Type	Sample Number
	7.5			G149
	8.0			T150
224.8	8.8	SILT (TILL) - some sand, trace to some clay, trace gravel (<10 mm dia.) - light brown - moist, compact - no to low plasticity		G151
	10.5	- moist, trace gravel (<20 mm dia.) below 10.7 m		G152
	12.0			G153
	13.3			G154
220.1	13.5			SS155



END OF TEST HOLE AT 13.5 m DEPTH IN SILT TILL
Notes:
1. Power auger refusal at 13.5 m depth
2. Sloughing observed at 2.4 m depth.
3. Test hole open to 2.4 m depth after completion of test hole.
4. Test hole backfilled with auger cuttings, bentonite, sand, and cold patch asphalt to ground surface.
5. Test hole location: North bound curb lane of Empress St., 1.7 m west from east curb, 125 m north of Maroons Rd.14U 5527843N, 629567E.

Logged By: Junhui Wu **Reviewed By:** Nelson Ferreira **Project Engineer:** Michael Van Helden

TREK GEOTECHNICAL

Sub-Surface Log

Client:	Morrison Hershfield	**Project Number:**	0035-037-00
Project Name:	Empress Widening	**Location:**	Empress St. From Portage to St. Matthews
Contractor:	Paddock Drilling Ltd.	**Ground Elevation:**	233.60 m
Method:	125 mm Solid Stem Auger, Acker RM5	**Date Drilled:**	3 November 2016

Sample Type: Grab (G) Shelby Tube (T) Split Spoon (SS) Split Barrel (SB) Core (C)

Particle Size Legend: Fines Clay Silt Sand Gravel Cobbles Boulders

Bulk Unit Wt (kN/m³): 16 17 18 19 20 21; Particle Size (%): 0 20 40 60 80 100; PL MC LL: 0 20 40 60 80 100; Undrained Shear Strength (kPa), Test Type: △ Torvane △, ✜ Pocket Pen. ✜, ☒ Qu ☒, ○ Field Vane ○: 0 50 100 150 200 250

Elevation (m)	Depth (m)	MATERIAL DESCRIPTION	Sample Type	Sample Number
233.6		ASPHALT (30 mm thick)		
233.3		CONCRETE (245 mm thick)		
233.3	0.5	SAND AND GRAVEL (FILL) - (<20 mm dia. gravel), trace clay, trace silt, grey, moist, well graded fine sand to fine gravel, sub-angular to angular	Grab	G156, G157
232.8		ORGANIC CLAY - silty - black, moist, firm, high plasticity		
232.1	1.0	CLAY, silty, trace silt inclusions (<10 mm dia.) - brown - moist, stiff - high plasticity	Grab	G158, G159
230.9	1.5 – 2.5	SILT - clayey - brown - moist, soft - low to intermediate plasticity	Grab	G160, G161
	3.0 – 6.5	CLAY - silty, trace silt inclusions (<10 mm dia.), trace oxidation - brown - moist, stiff - high plasticity - firm below 3.1 m - grey below 5.8 m	Grab / Shelby Tube	G162, G163, G164, G165, T166

Depth scale (m): 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0, 5.5, 6.0, 6.5

Logged By: Junhui Wu **Reviewed By:** Nelson Ferreira **Project Engineer:** Michael Van Helden

Elevation (m)	Depth (m)	Soil Symbol	MATERIAL DESCRIPTION	Sample Type	Sample Number	SPT (N)	Bulk Unit Wt (kN/m³) 16 17 18 19 20 21 / Particle Size (%) 0 20 40 60 80 100 / PL MC LL 0 20 40 60 80 100	Undrained Shear Strength (kPa) Test Type: △ Torvane △, ✤ Pocket Pen. ✤, ⊠ Qu ⊠, ○ Field Vane ○ 0 50 100 150 200 250
	7.5				G167			
	8.0							
	8.5							
	9.0		- soft below 8.8 m		G168			
			- trace sand, trace gravel, grey, soft below 9.3 m		T169			
223.8	9.5							
	10.0		SILT (TILL) - some clay, some sand, trace gravel (<20 mm dia.) - light brown - moist, compact - low plasticity					
	10.5				G170			
	11.0							
	11.5							
	12.0				G171			
	12.5							
	13.0							
					G172			
220.1	13.5				G173			

END OF TEST HOLE AT 13.5 m DEPTH IN SILT TILL

Notes:

1. Power auger refusal at 13.5 m depth.
2. Sloughage observed at 2.5 m, seepage observed below 12.2 m depth
3. Test hole open to 3.1 m depth after removal of augers.
4. Test hole backfilled with auger cuttings, bentonite, sand, and cold patch asphalt to ground surface.
5. Test hole location: North bound curb lane of Empress St., 1.7 m west from east curb, 80 m south of St. Matthews Ave. (14U 5527966N, 629572E).

Logged By: Junhui Wu **Reviewed By:** Nelson Ferreira **Project Engineer:** Michael Van Helden

Sub-Surface Log

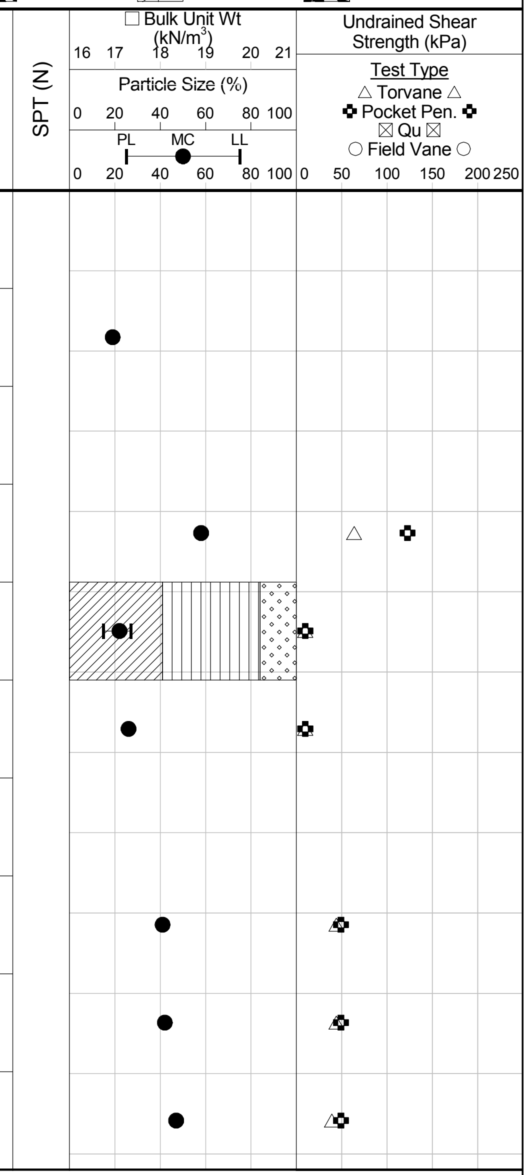
Client:	Morrison Hershfield	**Project Number:**	0035-037-00
Project Name:	Empress Widening	**Location:**	Empress St. From Portage to St. Matthews
Contractor:	Paddock Drilling Ltd.	**Ground Elevation:**	233.54 m
Method:	125 mm Solid Stem Auger, Acker RM5	**Date Drilled:**	1 November 2016

Sample Type: Grab (G) Shelby Tube (T) Split Spoon (SS) Split Barrel (SB) Core (C)

Particle Size Legend: Fines Clay Silt Sand Gravel Cobbles Boulders

Elevation (m)	MATERIAL DESCRIPTION	Sample Type	Sample Number
233.5	ASPHALT (56 mm thick)		
	CONCRETE (214 mm thick)		
233.3	CLAY AND SAND (FILL) - some silt, trace gravel (<20 mm dia.) - brown - moist, soft - low plasticity	Grab	G51
232.7	CLAY - silty, trace oxidation - black - moist, stiff - high plasticity - brown below 1.0 m	Grab	G52
232.3	SILT - clayey, some sand - brown - moist, soft - low to intermediate plasticity	Grab	G53
		Grab	G54
231.4	CLAY - silty, trace silt inclusions (<10 mm dia.) - brown - moist, firm - high plasticity	Grab	G55
		Grab	G56
230.5		Grab	G57

Depth (m): 0.5, 1.0, 1.5, 2.0, 2.5, 3.0



END OF TEST HOLE AT 3.1 m DEPTH IN CLAY

Notes:

1. No seepage or sloughing observed.
2. Test hole backfilled with auger cuttings, bentonite, sand, and cold patch asphalt to ground surface.
3. Test hole location: North bound curb lane of Empress St., 1.7 m west from east curb, 345 m south of Eastway, 14U 5526790N, 629631E.

Logged By: Junhui Wu **Reviewed By:** Nelson Ferreira **Project Engineer:** Michael Van Helden

TREK GEOTECHNICAL

Sub-Surface Log

Client:	Morrison Hershfield	**Project Number:**	0035-037-00
Project Name:	Empress Widening	**Location:**	Empress St. From Portage to St. Matthews
Contractor:	Paddock Drilling Ltd.	**Ground Elevation:**	234.28 m
Method:	125 mm Solid Stem Auger, Acker RM5	**Date Drilled:**	1 November 2016

Sample Type: Grab (G) Shelby Tube (T) Split Spoon (SS) Split Barrel (SB) Core (C)

Particle Size Legend: Fines Clay Silt Sand Gravel Cobbles Boulders

Elevation (m)	Depth (m)	MATERIAL DESCRIPTION	Sample Type	Sample Number	SPT (N)	Bulk Unit Wt (kN/m³) / Particle Size (%) / PL MC LL	Undrained Shear Strength (kPa) Test Type: △ Torvane △, ✿ Pocket Pen. ✿, ☒ Qu ☒, ○ Field Vane ○
234.2		ASPHALT (43 mm thick)					
234.0		CONCRETE (225 mm thick)					
	0.5	SAND (FILL) - some clay, trace silt, trace gravel (<20 mm dia.) - brown, moist - moist, compact - well graded fine sand to fine gravel, sub-angular to angular	G	G58			
			G	G59			
	1.0		G	G60			
			G	G61			
	1.5		G	G62			
232.4		CLAY - silty, trace sand - black - moist, stiff - high plasticity					
232.1	2.0	SILT - trace clay - light brown - damp - low plasticity - clayey, moist, soft, low to intermediate plasticity below 2.3	G	G63			
			G	G64			
231.5	2.5	CLAY - silty, trace silt inclusions (<10 mm dia.) - brown - moist, stiff - high plasticity	G	G65			
231.2	3.0						

END OF TEST HOLE AT 3.1 m DEPTH IN CLAY
Notes:
1. No seepage or sloughing observed.
2. Test hole backfilled with auger cuttings, bentonite, sand, and cold patch asphalt to ground surface.
3. Test hole location: North bound curb lane of Empress St., 1.7 m west from east curb, 255 m south of Eastway, (14U 5526878N, 629704E).

Logged By: Junhui Wu **Reviewed By:** Nelson Ferreira **Project Engineer:** Michael Van Helden

TREK GEOTECHNICAL

Sub-Surface Log

Client:	Morrison Hershfield	**Project Number:**	0035-037-00
Project Name:	Empress Widening	**Location:**	Empress St. From Portage to St. Matthews
Contractor:	Paddock Drilling Ltd.	**Ground Elevation:**	236.16 m
Method:	125 mm Solid Stem Auger, Acker RM5	**Date Drilled:**	2 November 2016

Sample Type: Grab (G) | Shelby Tube (T) | Split Spoon (SS) | Split Barrel (SB) | Core (C)

Particle Size Legend: Fines | Clay | Silt | Sand | Gravel | Cobbles | Boulders

Elevation (m)	Depth (m)	MATERIAL DESCRIPTION	Sample Type	Sample Number	SPT (N)
236.1	0.0	ASPHALT (75 mm)			
		CONCRETE (225 mm)			
235.9	0.3	SAND (FILL) - some gravel (<20 mm dia.), trace clay, trace silt - brown - moist, compact - well graded fine sand to fine gravel, sub-angular to angular	G	G92	
			G	G93	
		- some clay below 1.2m	G	G94	
			G	G95	
			G	G96	
233.1	3.1		G	G97	

Bulk Unit Wt (kN/m³): 16 17 18 19 20 21
Particle Size (%): 0 20 40 60 80 100
PL MC LL: 0 20 40 60 80 100
Undrained Shear Strength (kPa): 0 50 100 150 200 250
Test Type: △ Torvane △, Pocket Pen., ☒ Qu ☒, ○ Field Vane ○

END OF TEST HOLE AT 3.1 m DEPTH IN SAND (FILL)
Notes:
1. No Seepage or sloughing observed.
2. Test hole backfilled with auger cuttings, bentonite, sand, and cold patch asphalt to ground surface.
3. Test hole location: North bound curb lane of Empress St., 1.7 m west from east curb, 153 m south of Eastway, (14U 5526985N, 629717E).

Logged By: Junhui Wu **Reviewed By:** Nelson Ferreira **Project Engineer:** Michael Van Helden

Sub-Surface Log

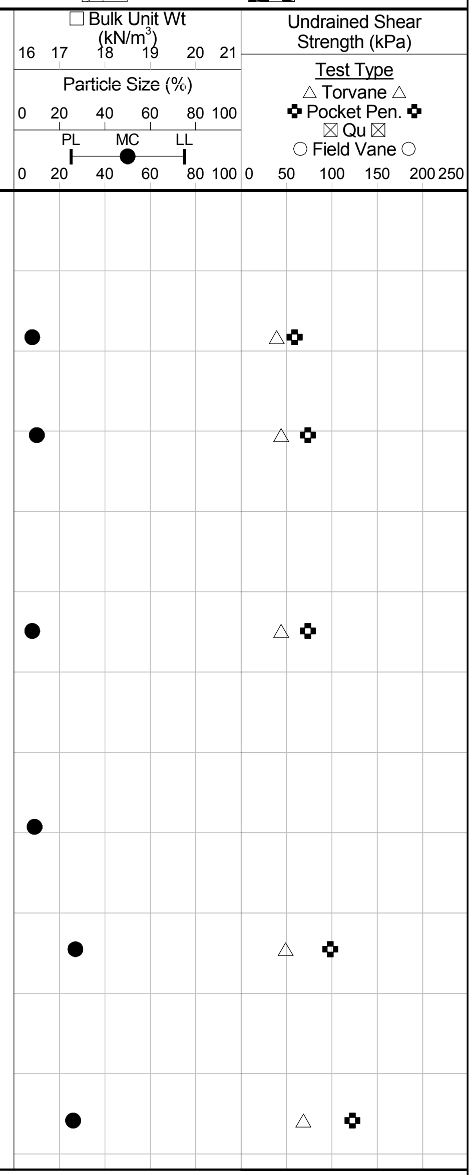
TREK GEOTECHNICAL

Client:	Morrison Hershfield	**Project Number:**	0035-037-00
Project Name:	Empress Widening	**Location:**	Empress St. From Portage to St. Matthews
Contractor:	Paddock Drilling Ltd.	**Ground Elevation:**	236.03 m
Method:	125 mm Solid Stem Auger, Acker RM5	**Date Drilled:**	2 November 2016

Sample Type: Grab (G) Shelby Tube (T) Split Spoon (SS) Split Barrel (SB) Core (C)

Particle Size Legend: Fines Clay Silt Sand Gravel Cobbles Boulders

Elevation (m)	Depth (m)	MATERIAL DESCRIPTION	Sample Type	Sample Number	SPT (N)
236.0	0.0	ASPHALT (75 mm thick)			
		CONCRETE (237 mm thick)			
235.7		CLAY (FILL) - sandy, some silt, some gravel (<50 mm dia.) - brown - moist, firm - intermediate plasticity	Grab	G98	
	0.5		Grab	G99	
	1.0				
			Grab	G100	
	1.5				
234.2		SAND (FILL) - some gravel (<40 mm dia.), trace clay, trace silt - brown - moist, compact - well graded fine sand to fine gravel, sub-angular to angular	Grab	G101	
233.9	2.0	CLAY - silty, trace rootlets - black - moist, stiff - high plasticity	Grab	G102	
	2.5	- light brown below 2.6 m			
233.0	3.0		Grab	G103	



END OF TEST HOLE AT 3.1 m DEPTH IN CLAY

Notes:

1. No seepage or sloughing observed.
2. Test hole backfilled with auger cuttings, bentonite, sand, and cold patch asphalt to ground surface.
3. Test hole location: North bound curb lane Empress St, 1.8 m west from east curb, 91 m south of Eastway, (14U 5527053N 629719E).

Logged By: Junhui Wu **Reviewed By:** Nelson Ferreira **Project Engineer:** Michael Van Helden

Sub-Surface Log

Test Hole RH16-12

TREK GEOTECHNICAL

Client:	Morrison Hershfield	**Project Number:**	0035-037-00
Project Name:	Empress Widening	**Location:**	Empress St. From Portage to St. Matthews
Contractor:	Paddock Drilling Ltd.	**Ground Elevation:**	233.97 m
Method:	125 mm Solid Stem Auger, Acker RM5	**Date Drilled:**	1 November 2016

Sample Type: Grab (G) | Shelby Tube (T) | Split Spoon (SS) | Split Barrel (SB) | Core (C)

Particle Size Legend: Fines | Clay | Silt | Sand | Gravel | Cobbles | Boulders

Elevation (m)	Depth (m)	MATERIAL DESCRIPTION	Sample Type	Sample Number	SPT (N)	MC (%)	Undrained Shear Strength (kPa): Torvane △ / Pocket Pen. ✿
233.9		ASPHALT (65 mm thick)					
		CONCRETE (205 mm thick)					
233.7		SAND (FILL) - some gravel (<20 mm dia.), trace clay, trace silt -brown, - moist, compact - well graded fine sand to fine gravel, sub-angular to angular	G	G15		●	
			G	G16		●	
233.1		CLAY - silty - mottled black and brown - moist, stiff - high plasticity	G	G17		●	△ ✿
			G	G18		●	△ ✿
			G	G19		●	△ ✿
		- trace silt inclusions (<10 mm dia.), brown below 2.1 m					
			G	G20		●	△ ✿
230.9	3.0		G	G21		●	△ ✿

Chart axes: Bulk Unit Wt (kN/m^3): 16 17 18 19 20 21; Particle Size (%): 0 20 40 60 80 100; PL MC LL: 0 20 40 60 80 100; Undrained Shear Strength (kPa): 0 50 100 150 200 250. Test Type: △ Torvane △, ✿ Pocket Pen. ✿, ⊠ Qu ⊠, ○ Field Vane ○. Depth scale: 0.5, 1.0, 1.5, 2.0, 2.5, 3.0.

END OF TEST HOLE AT 3.1 m DEPTH IN CLAY

Notes:
1. No Seepage or sloughing observed.
2. Test hole backfilled with auger cuttings, bentonite, sand, and cold patch asphalt to ground surface.
3. Test hole location: South bound curb lane Empress St., 1.8 m east from west curb, 16 m north of Eastway, (14U 5527141N, 629707E).

Logged By: Junhui Wu **Reviewed By:** Nelson Ferreira **Project Engineer:** Michael Van Helden

TREK GEOTECHNICAL

Sub-Surface Log

Client:	Morrison Hershfield	**Project Number:**	0035-037-00
Project Name:	Empress Widening	**Location:**	Empress St. From Portage to St. Matthews
Contractor:	Paddock Drilling Ltd.	**Ground Elevation:**	233.59 m
Method:	125 mm Solid Stem Auger, Acker RM5	**Date Drilled:**	2 November 2016

Sample Type: Grab (G) | Shelby Tube (T) | Split Spoon (SS) | Split Barrel (SB) | Core (C)

Particle Size Legend: Fines | Clay | Silt | Sand | Gravel | Cobbles | Boulders

Column headings: Elevation (m) | Depth (m) | Soil Symbol | MATERIAL DESCRIPTION | Sample Type | Sample Number | SPT (N) | Bulk Unit Wt (kN/m^3) 16 17 18 19 20 21; Particle Size (%) 0 20 40 60 80 100; PL MC LL 0 20 40 60 80 100 | Undrained Shear Strength (kPa), Test Type: △ Torvane △, ✪ Pocket Pen. ✪, ☒ Qu ☒, ○ Field Vane ○; 0 50 100 150 200 250

Elevation (m)	MATERIAL DESCRIPTION	Sample Number
233.5	ASPHALT (59 mm thick)	
233.3	CONCRETE (206 mm thick)	
233.2	SAND (FILL) - some clay, trace gravel (<25 mm dia.) - brown, moist	G66
232.7	CLAY - silty, trace silt inclusions (<10 mm dia.) - black - moist, firm - high plasticity	G67
231.9	SILT - clayey - grey - moist to wet moist, very soft to soft - low plasticity	G68, G69
230.5	CLAY - silty, trace silt inclusions (<10 mm dia.) - dark brown - moist, stiff - high plasticity - 50 mm thick layer of silt at 2.3 m depth, soft	G70, G71, G72, G73

Depth (m): 0.5, 1.0, 1.5, 2.0, 2.5, 3.0

END OF TEST HOLE AT 3.1 m DEPTH IN CLAY
Notes:
1. No seepage or sloughing observed.
2. Test hole backfilled with auger cuttings, bentonite, sand, and cold patch asphalt to ground surface.
3. Test hole location: South bound median lane, 4.5 m east from west curb, 35 m south of Westway, (14U 5527244N, 629683E).

Logged By:	Junhui Wu	**Reviewed By:**	Nelson Ferreira	**Project Engineer:**	Michael Van Helden

Sub-Surface Log

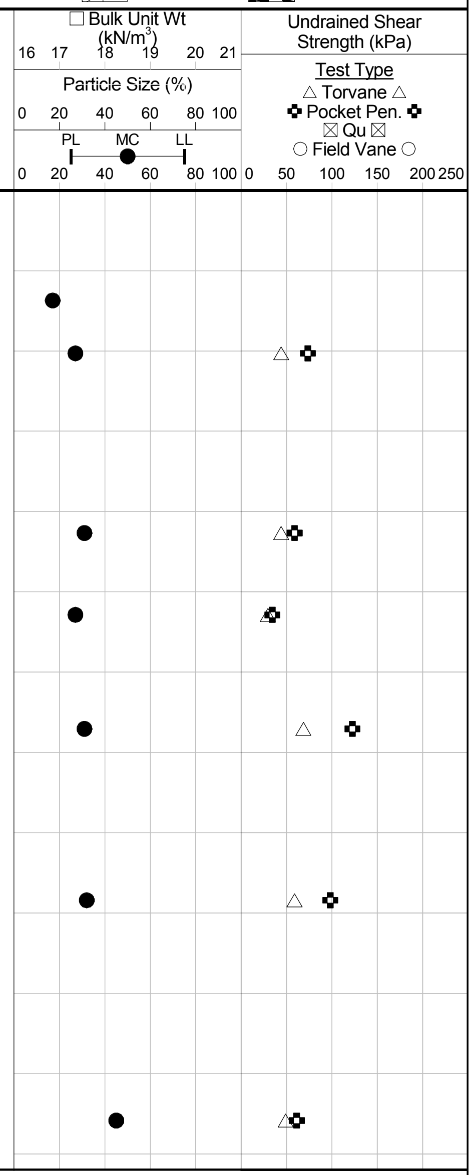
TREK GEOTECHNICAL

Client:	Morrison Hershfield	**Project Number:**	0035-037-00
Project Name:	Empress Widening	**Location:**	Empress St. From Portage to St. Matthews
Contractor:	Paddock Drilling Ltd.	**Ground Elevation:**	233.69 m
Method:	125 mm Solid Stem Auger, Acker RM5	**Date Drilled:**	1 November 2016

Sample Type: Grab (G) | Shelby Tube (T) | Split Spoon (SS) | Split Barrel (SB) | Core (C)

Particle Size Legend: Fines | Clay | Silt | Sand | Gravel | Cobbles | Boulders

Elevation (m)	Depth (m)	MATERIAL DESCRIPTION	Sample Type	Sample Number	SPT (N)
233.6		ASPHALT (62 mm thick)			
233.4		CONCRETE (212 mm thick)			
233.3		CLAY and SAND (FILL) - trace silt, trace gravel (<20 mm dia.) - brown, moist, soft, low plasticity	Grab	G01	
	0.5	CLAY - silty - mottled brown and black - moist, firm to stiff - high plasticity	Grab	G02	
	1.0		Grab	G03	
			Grab	G04	
	1.5		Grab	G05	
	2.0		Grab	G06	
		- brown below 2.3 m			
	2.5				
230.6	3.0		Grab	G07	



END OF TEST HOLE AT 3.1 m DEPTH IN CLAY
Notes:
1. No seepage or sloughing observed.
2. Test hole backfilled with auger cuttings, bentonite, sand, and cold patch asphalt to ground surface.
3. Test hole location: South bound curb lane Empress St., 1.9 m east from west curb, 113 m north from Westway, (14U 5527369 N, 629634E).

Logged By: Junhui Wu **Reviewed By:** Nelson Ferreira **Project Engineer:** Michael Van Helden

TREK GEOTECHNICAL

Sub-Surface Log

Client:	Morrison Hershfield	**Project Number:**	0035-037-00
Project Name:	Empress Widening	**Location:**	Empress St. From Portage to St. Matthews
Contractor:	Paddock Drilling Ltd.	**Ground Elevation:**	233.18 m
Method:	125 mm Solid Stem Auger, Acker RM5	**Date Drilled:**	1 November 2016

Sample Type: Grab (G) Shelby Tube (T) Split Spoon (SS) Split Barrel (SB) Core (C)

Particle Size Legend: Fines Clay Silt Sand Gravel Cobbles Boulders

Elevation (m)	Depth (m)	MATERIAL DESCRIPTION	Sample Type	Sample Number
233.1		ASPHALT (43 mm thick)		
		CONCRETE (218 mm thick)		
232.9				
232.9		SAND (FILL) - trace silt, trace clay, trace gravel (<10 mm dia.), brown, moist		
		CLAY - silty, trace silt inclusions (<10 mm dia.) - black - moist, soft - high plasticity	G	G08
	0.5			
		- stiff below 0.76 m	G	G09
	1.0		G	G10
			G	G11
	1.5	- trace inclusions (<20 mm dia.), brown, stiff	G	G12
	2.0			
		- light brown below 2.1 m	G	G13
	2.5			
230.1	3.0		G	G14

SPT (N)

Bulk Unit Wt (kN/m³): 16 17 18 19 20 21

Particle Size (%): 0 20 40 60 80 100

PL MC LL: 0 20 40 60 80 100

Undrained Shear Strength (kPa)

Test Type: △ Torvane △, ✿ Pocket Pen. ✿, ⊠ Qu ⊠, ○ Field Vane ○

0 50 100 150 200 250

END OF TEST HOLE AT 3.1 m DEPTH IN CLAY

Notes:

1. No Seepage or sloughing observed.
2. Test hole backfilled with auger cuttings, bentonite, sand, and cold patch asphalt to ground surface.
3. Test hole location: South bound curb lane Empress St., 1.7 m east from west curb, 190 m north of Westway, (14U 5527449N 629603E).

Logged By: Junhui Wu **Reviewed By:** Nelson Ferreira **Project Engineer:** Michael Van Helden

Sub-Surface Log

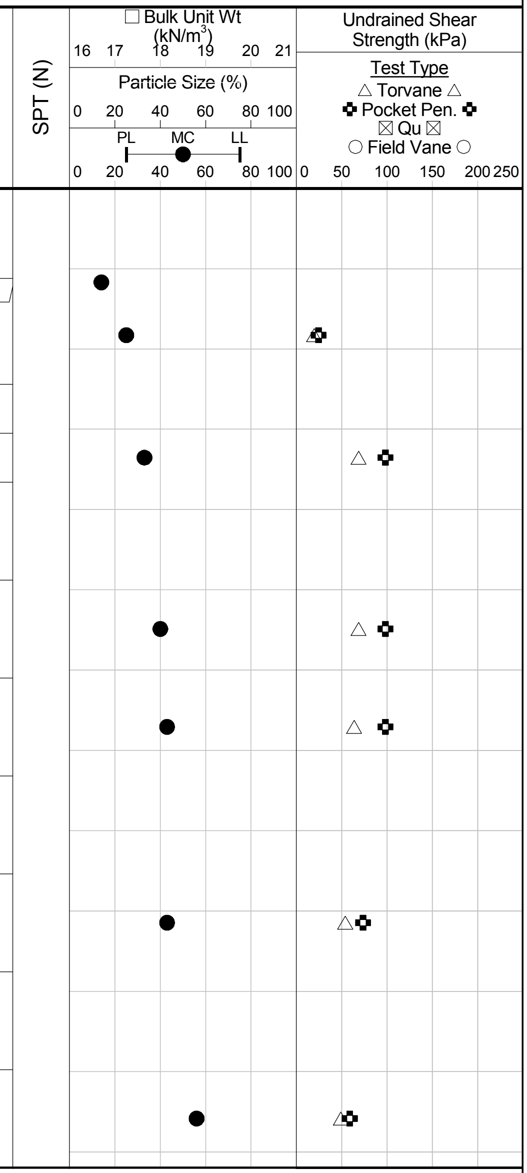
TREK GEOTECHNICAL

Client:	Morrison Hershfield	**Project Number:**	0035-037-00
Project Name:	Empress Widening	**Location:**	Empress St. From Portage to St. Matthews
Contractor:	Paddock Drilling Ltd.	**Ground Elevation:**	233.29 m
Method:	125 mm Solid Stem Auger, Acker RM5	**Date Drilled:**	1 November 2016

Sample Type: Grab (G) Shelby Tube (T) Split Spoon (SS) Split Barrel (SB) Core (C)

Particle Size Legend: Fines Clay Silt Sand Gravel Cobbles Boulders

Elevation (m)	Depth (m)	MATERIAL DESCRIPTION	Sample Type	Sample Number
233.2		ASPHALT (60 mm thick)		
233.0		CONCRETE (215 mm thick)		
233.0		SAND (FILL) - some gravel (<20 mm dia.), trace clay, trace silt, brown, moist	Grab	G29
	0.5	CLAY (FILL)- silty, trace sand, trace gravel (<20 mm dia.) - mottled brown and black - moist, soft - high plasticity	Grab	G30
232.5	1.0	CLAY - silty, trace silt inclusions (<10 mm dia.) - light brown - moist, stiff - high plasticity - dark brown below 1.1 m	Grab	G31
	1.5		Grab	G32
			Grab	G33
	2.0			
	2.5		Grab	G34
230.2	3.0	- firm to stiff below 2.7 m	Grab	G35



END OF TEST HOLE AT 3.1 m DEPTH IN CLAY

Notes:

1. No seepage or sloughing observed.
2. Test hole backfilled with auger cuttings, bentonite, sand, and cold patch asphalt to ground surface.
3. Test hole location: South bound curb lane Empress St., 1.8 m east from west curb, 180 m north of Westway, (14U 5527447 N, 629562 E).

Logged By: Junhui Wu **Reviewed By:** Nelson Ferreira **Project Engineer:** Michael Van Helden

TREK GEOTECHNICAL

Sub-Surface Log

Client:	Morrison Hershfield	**Project Number:**	0035-037-00
Project Name:	Empress Widening	**Location:**	Empress St. From Portage to St. Matthews
Contractor:	Paddock Drilling Ltd.	**Ground Elevation:**	233.36 m
Method:	125 mm Solid Stem Auger, Acker RM5	**Date Drilled:**	1 November 2016

Sample Type: Grab (G) | Shelby Tube (T) | Split Spoon (SS) | Split Barrel (SB) | Core (C)

Particle Size Legend: Fines | Clay | Silt | Sand | Gravel | Cobbles | Boulders

Bulk Unit Wt (kN/m³): 16 17 18 19 20 21
Particle Size (%): 0 20 40 60 80 100
PL MC LL: 0 20 40 60 80 100
Undrained Shear Strength (kPa): 0 50 100 150 200 250
Test Type: △ Torvane △, ✤ Pocket Pen. ✤, ⊠ Qu ⊠, ○ Field Vane ○

Elevation (m)	Depth (m)	MATERIAL DESCRIPTION	Sample Type	Sample Number	SPT (N)
233.3	0.0	ASPHALT (75 mm thick)			
233.1		CONCRETE (203 mm thick)			
233.0		SAND (FILL) - trace clay, trace silt, trace gravel (<20 mm dia.), brown, moist			
232.7	0.5	CLAY - silty, trace sand - black - moist, stiff, high plasticity	G	G36	
232.6		SILT - trace to some clay, light brown, moist, soft, low plasticity	G	G37	
230.3	1.0–3.0	CLAY - silty, trace silt inclusions (<10 mm dia.) - brown - moist, stiff - high plasticity - firm to stiff below 1.5 m	G	G38, G39, G40, G41, G42	

END OF TEST HOLE AT 3.1 m DEPTH IN SILT TILL
Notes:
1. No seepage or sloughing observed.
2. Test hole backfilled with auger cuttings, bentonite, sand, and cold patch asphalt to ground surface.
3. Test hole location: South bound curb lane Empress St., 1.8 m east from west curb, 56 m north from Westway, (14U 5527320 N, 629557 E).

Logged By: Junhui Wu **Reviewed By:** Nelson Ferreira **Project Engineer:** Michael Van Helden

Sub-Surface Log

TREK GEOTECHNICAL

Client:	Morrison Hershfield	**Project Number:**	0035-037-00
Project Name:	Empress Widening	**Location:**	Empress St. From Portage to St. Matthews
Contractor:	Paddock Drilling Ltd.	**Ground Elevation:**	233.16 m
Method:	125 mm Solid Stem Auger, Acker RM5	**Date Drilled:**	1 November 2016

Sample Type: Grab (G), Shelby Tube (T), Split Spoon (SS), Split Barrel (SB), Core (C)

Particle Size Legend: Fines, Clay, Silt, Sand, Gravel, Cobbles, Boulders

Elevation (m)	Depth (m)	MATERIAL DESCRIPTION	Sample Type	Sample Number
233.1		ASPHALT (31 mm thick)		
232.9		CONCRETE (206 mm thick)		
232.9		CLAY AND SAND (FILL) - trace gravel (<20 mm dia.) - brown, moist, soft, low plasticity	G	G43
	0.5	CLAY - silty, trace oxidation - brown - moist, soft - high plasticity - firm below 0.45 m	G	G44, G45, G46
231.9	1.0			
	1.5	SILT - clayey - brown - moist, soft - intermediate plasticity	G	G47, G48
230.7	2.0		G	G49
	2.5	CLAY - silty, trace silt inclusions (<10 mm dia.) - brown - moist, firm - high plasticity	G	G50
230.1	3.0			

Column headings: SPT (N); Bulk Unit Wt (kN/m³): 16 17 18 19 20 21; Particle Size (%): 0 20 40 60 80 100; PL MC LL: 0 20 40 60 80 100; Undrained Shear Strength (kPa), Test Type: △ Torvane △, Pocket Pen., ⊠ Qu ⊠, ○ Field Vane ○: 0 50 100 150 200 250

END OF TEST HOLE AT 3.1 m DEPTH IN CLAY
Notes:
1. No seepage or sloughing observed.
2. Test hole backfilled with auger cuttings, bentonite, sand, and cold patch asphalt to ground surface.
3. Test hole location: South bound curb lane Empress St., 1.8 m east from west curb, 93 m south of Westway, (14U 5527179N, 629555E).

Logged By: Junhui Wu **Reviewed By:** Nelson Ferreira **Project Engineer:** Michael Van Helden

Sub-Surface Log

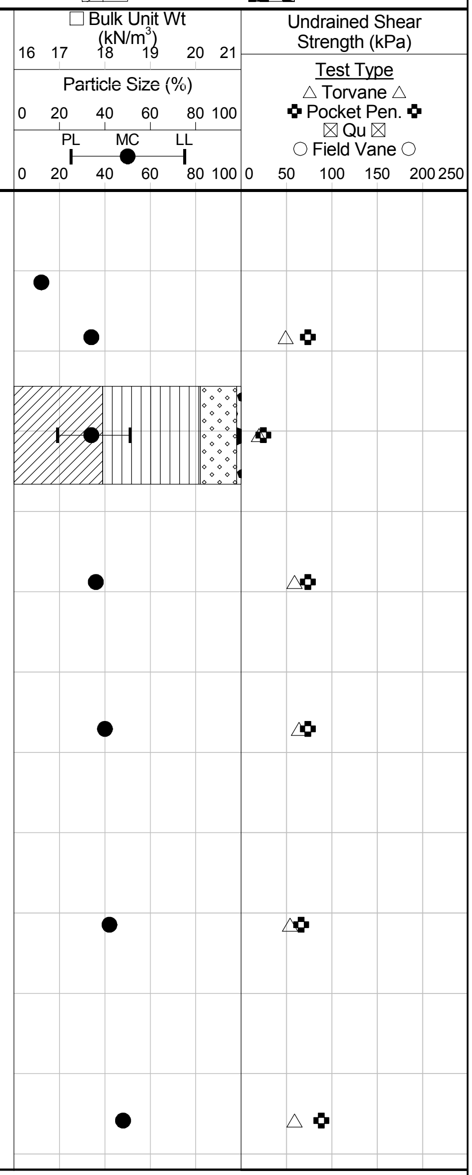
TREK GEOTECHNICAL

Client:	Morrison Hershfield	**Project Number:**	0035-037-00
Project Name:	Empress Widening	**Location:**	Empress St. From Portage to St. Matthews
Contractor:	Paddock Drilling Ltd.	**Ground Elevation:**	233.49 m
Method:	125 mm Solid Stem Auger, Acker RM5	**Date Drilled:**	1 November 2016

Sample Type: Grab (G) Shelby Tube (T) Split Spoon (SS) Split Barrel (SB) Core (C)

Particle Size Legend: Fines Clay Silt Sand Gravel Cobbles Boulders

Elevation (m)	Depth (m)	MATERIAL DESCRIPTION	Sample Type	Sample Number	SPT (N)
233.5		ASPHALT (38 mm thick)			
233.2		CONCRETE (225 mm thick)			
233.2		SAND (FILL) - some gravel (<20 mm dia.), trace clay, trace silt, brown, moist	Grab	G22	
232.9	0.5	CLAY - silty - black - moist, stiff - high plasticity	Grab	G23	
232.4	1.0	CLAY and SILT - light brown - moist, soft - high plasticity	Grab	G24	
	1.5	CLAY - silty, trace silt inclusions (<10 mm dia.) - brown - moist, stiff - high plasticity	Grab	G25	
	2.0		Grab	G26	
	2.5		Grab	G27	
230.4	3.0		Grab	G28	



END OF TEST HOLE AT 3.1 m DEPTH IN CLAY

Notes:

1. Seepage and sloughing observed below 0.76 m depth.
2. Test hole open to 0.76 m depth after removal of augers.
3. Test hole backfilled with auger cuttings, bentonite, sand, and cold patch asphalt to ground surface.
4. Test hole location: South bound curb lane Empress St., 1.8 m east from west curb, 275 m north of Westway, (14 U 5527539 N, 629552E).

Logged By: Junhui Wu **Reviewed By:** Nelson Ferreira **Project Engineer:** Michael Van Helden

Appendix B

Lab Testing Summary Table & Lab Testing Results

Empress Widening
Road Sub-Surface Investigation
Summary Table

Test Hole No.	Test Hole Location	Pavement Surface		Pavement Structure Material		Subgrade Description	Sample Depth (m)		Moisture Content (%)	Grain Size Analysis				Atterberg Limits		
		Type	Thickness (mm)	Type	Thickness (mm)		Top (m)	Bottom (m)		Gravel (%)	Sand (%)	Silt (%)	Clay (%)	Liquid	Plastic	Plasticity Index
RH16-03	North bound curb lane of Empress St., 1.7 m west from east curb, 450 m south of Eastway, (14U 5527650m N, 629535m E).	Asphalt	40	Concrete	240				-							
						SAND (FILL)	0.3	0.6	8.5							
						SILT & SAND	0.6	0.9	8.8	14	46	27	13	17	14	3
						SILT & SAND	1.1	1.4	10.8							
						CLAY	1.5	1.7	36.2							
						CLAY	1.8	2.1	48.5							
						CLAY	2.4	2.7	42.9							
						CLAY	2.7	3.0	49.7							
RH16-04	North bound curb lane of Empress St., 1.7 m west from east curb, 75 m south of Maroons Rd. (14U 5527640m N, 629563m E).	Asphalt	37	Concrete	237											
						SAND (FILL)	0.2	0.3	16.7							
						ORGANIC CLAY	0.3	0.8	39.7							
						SILT	0.8	1.1	28.5	2	10	70	19	28	19	9
						CLAY	1.1	1.5	35.1							
						CLAY	1.5	1.8	42.9							
						CLAY	2.1	2.4	44.1							
						CLAY	2.7	3.0	47.4							
RH16-05	North bound curb lane of Empress St., 1.7 m west from east curb, 35 m north of Maroons Rd. (14U 5527751m N, 629564m E).	Asphalt	30	Concrete	243											
						SAND and CLAY	0.2	0.4	16.3							
						CLAY	0.4	0.6	18.6							
						SILT	0.6	1.1	21.2	0	15	67	19	NP	NP	NP
						CLAY	1.1	1.2	25.2							
						SILT	1.2	1.5	20.3							
						SILT	1.5	1.8	22.1							
						CLAY	2.1	2.4	49.3							
RH16-06	North bound curb lane of Empress St., 1.7 m west from east curb, 125 m north of Maroons Rd. (14U 5527843m N, 629567m E).	Asphalt	25	Concrete	235											
						SAND & GRAVEL (FILL)	0.2	0.3	9.0							
						CLAY	0.3	0.6	28.9							
						SILT	0.6	0.9	24.9							
						SILT	0.9	1.2	27.5	0	16	43	40	33	11	23
						CLAY	1.2	1.5	35.5							
						CLAY	1.9	2.0	22.0							
						CLAY	2.1	2.4	38.0							
						SILT	2.4	2.7	23.0	0	12	72	16	NP	NP	NP

Empress Widening
Road Sub-Surface Investigation
Summary Table

Test Hole No.	Test Hole Location	Pavement Surface		Pavement Structure Material		Subgrade Description	Sample Depth (m)		Moisture Content (%)	Grain Size Analysis				Atterberg Limits		
		Type	Thickness (mm)	Type	Thickness (mm)		Top (m)	Bottom (m)		Gravel (%)	Sand (%)	Silt (%)	Clay (%)	Liquid	Plastic	Plasticity Index
RH16-07	North bound curb lane of Empress St.,1.7 m west from east curb, 80 m south of St. Matthews Ave. (14U 5527966m N, 629572m E).	Asphalt	30	Concrete	245											
						SAND & GRAVEL (FILL)	0.2	0.3	6.9							
						ORGANIC CLAY	0.3	0.6	28.8	2	18	47	33	54	20	33
						CLAY	0.8	1.1	28.9							
						CLAY	1.2	1.5	25.5							
						SILT	1.5	1.8	23.4							
						SILT	2.1	2.7	25.2							
						CLAY	2.7	3.0	38.8							
RH16-08	North bound curb lane of Empress St., 1.7 m west from east curb, 345 m south of Eastway, (14U 5526790m N, 629631m E).	Asphalt	56	Concrete	214											
						CLAY & SAND (FILL)	0.3	0.6	18.5							
						CLAY	0.9	1.2	58.0							
						SILT	1.2	1.5	22.4	0	16	43	40	27	15	12
						SILT	1.5	1.8	25.7							
						CLAY	2.1	2.4	41.2							
						CLAY	2.4	2.7	42.1							
						CLAY	2.7	3.0	46.6							
RH16-09	North bound curb lane of Empress St., 1.7 m west from east curb, 255 m south of Eastway, (14U 5526878m N, 629704m E).	Asphalt	43	Concrete	225											
						SAND (FILL)	0.3	0.6	7.6							
						SAND (FILL)	0.6	0.9	6.1							
						SAND (FILL)	0.9	1.2	7.3							
						SAND (FILL)	1.2	1.5	10.5							
						CLAY (FILL)	1.8	2.1	25.1							
						SILT	2.1	2.3	16.3							
						SILT	2.3	2.4	23.2							
						CLAY	2.7	3.0	31.5							
RH16-10	North bound curb lane of Empress St., 1.7 m west from east curb, 153 m south of Eastway, (14U 5526985m N, 629717m E).	Asphalt	75	Concrete	225											
						SAND (FILL)	0.3	0.6	6.5							
						SAND (FILL)	0.6	0.9	5.9							
						SAND (FILL)	1.2	1.5	6.9							
						SAND (FILL)	1.5	1.8	8.4							
						SAND (FILL)	2.1	2.4	8.2							
						SAND (FILL)	2.7	3.0	7.5							

Empress Widening
Road Sub-Surface Investigation
Summary Table

Test Hole No.	Test Hole Location	Pavement Surface		Pavement Structure Material		Subgrade Description	Sample Depth (m)		Moisture Content (%)	Grain Size Analysis				Atterberg Limits		
		Type	Thickness (mm)	Type	Thickness (mm)		Top (m)	Bottom (m)		Gravel (%)	Sand (%)	Silt (%)	Clay (%)	Liquid	Plastic	Plasticity Index
RH16-11	North bound curb lane of Empress St., 1.8 m west from east curb, 91 m south of Eastway, (14U 5527053m N, 629719m E).	Asphalt	75	Concrete	237				-							
						CLAY (FILL)	0.3	0.6	8.3							
						CLAY (FILL)	0.6	0.9	10.0							
						CLAY (FILL)	1.2	1.5	8.1							
						SAND (FILL)	1.8	2.1	8.5							
						CLAY	2.1	2.6	27.1							
						CLAY	2.7	3.0	25.7							
RH16-12	South bound curb lane of Empress St., 1.8 m east from west curb, 16 m north of Eastway, (14U 5527141m N, 629707m E).	Asphalt	65	Concrete	205											
						SAND (FILL)	0.3	0.5	15.5							
						SAND (FILL)	0.5	1.1	8.2							
						CLAY	1.1	1.2	27.1							
						CLAY	1.2	1.5	31.1							
						CLAY	1.5	1.8	33.7							
						CLAY	2.1	2.4	33.2							
						CLAY	2.7	3.0	27.5							
RH16-13	South bound curb lane of Empress St., 4.5 m east of west curb, 35 m south of Westway, (14U 5527244m N, 629683m E).	Asphalt	59	Concrete	206											
						SAND (FILL)	0.3	0.4	5.9							
						CLAY	0.4	0.6	28.9							
						SILT	0.9	1.2	25.7							
						SILT	1.2	1.5	27.3							
						CLAY	1.7	1.8	40.0							
						CLAY	2.2	2.3	35.8							
						CLAY	2.4	2.7	47.1							
						CLAY	2.7	3.0	47.0							
RH16-14	South bound curb lane of Empress St., 1.9 m east from west curb, 113 m north of Westway, (14U 5527369m N, 629634m E).	Asphalt	62	Concrete	212											
						SAND & CLAY (FILL)	0.3	0.4	16.7							
						CLAY	0.4	0.6	27.3							
						CLAY	0.9	1.2	30.9							
						CLAY	1.2	1.4	26.6							
						CLAY	1.5	1.8	30.8							
						CLAY	2.1	2.3	31.6							
						CLAY	2.7	3.0	45.2							
RH16-15	South bound curb lane of Empress St., 1.7 m east from west curb, 190 m north of Westway, (14U 5527449m N, 629603m E).	Asphalt	43	Concrete	218											
						SAND (FILL)	0.3	0.3	-							
						CLAY	0.3	0.6	34.7							
						CLAY	0.8	0.9	43.3							
						CLAY	0.9	1.2	41.6							
						CLAY	1.2	1.5	37.4							
						CLAY	1.5	1.8	36.0							
						CLAY	2.1	2.3	39.4							
						CLAY	2.7	3.0	44.7							

Empress Widening
Road Sub-Surface Investigation
Summary Table

Test Hole No.	Test Hole Location	Pavement Surface		Pavement Structure Material		Subgrade Description	Sample Depth (m)		Moisture Content (%)	Grain Size Analysis				Atterberg Limits		
		Type	Thickness (mm)	Type	Thickness (mm)		Top (m)	Bottom (m)		Gravel (%)	Sand (%)	Silt (%)	Clay (%)	Liquid	Plastic	Plasticity Index
RH16-16	South bound curb lane of Empress St., 1.8 m east from west curb, 180 m north of Westway, (14U 5527447m N, 629562m E).	Asphalt	60	Concrete	215											
						SAND (FILL)	0.2	0.3	13.9							
						CLAY (FILL)	0.3	0.6	25.2							
						CLAY	0.8	0.9	33.0							
						CLAY	1.2	1.5	39.6							
						CLAY	1.5	1.8	43.2							
						CLAY	2.1	2.4	43.4							
						CLAY	2.7	3.0	56.0							
RH16-17	South bound curb lane of Empress St., 1.8 m east from west curb, 56 m north of Westway, (14U 5527320m N, 629557m E).	Asphalt	75	Concrete	203											
						SAND (FILL)	0.4	0.6	26.1							
						CLAY	0.6	0.8	17.9							
						CLAY	0.8	0.9	30.3							
						CLAY	1.2	1.5	37.8							
						CLAY	1.5	1.8	40.4							
						CLAY	2.1	2.4	47.8							
						CLAY	2.7	3.0	52.4							
RH16-18	South bound curb lane of Empress St., 1.8 m east from west curb, 93 m south of Westway, U14 (5527179m N, 629555m E).	Asphalt	31	Concrete	206											
						CLAY & SAND (FILL)	0.2	0.3	22.5							
						CLAY	0.3	0.5	29.7							
						CLAY	0.5	0.6	29.4							
						CLAY	0.6	0.9	31.3							
						SILT	1.2	1.5	23.2							
						SILT	1.5	1.8	24.3							
						SILT	2.1	2.4	22.6							
						CLAY	2.4	2.7	50.5							
RH16-19	South bound curb lane of Empress St., 1.8 m east from west curb, 275 m north of Westway, (14U 5527539m N, 629552m E).	Asphalt	38	Concrete	225											
						SAND (FILL)	0.2	0.3	12.2							
						CLAY	0.3	0.6	33.7							
						CLAY & SILT	0.6	0.9	33.9	2	16	43	40	51	19	31
						CLAY	1.1	1.5	35.6							
						CLAY	1.5	1.8	39.8							
						CLAY	2.1	2.4	42.0							
						CLAY	2.7	3.0	47.5							

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Moisture Content Report
ASTM D2216-98

Project No.	0035-037-00
Client	Morrison Hershfield
Project	Empress Widening
Sample Date	01-Nov-16
Test Date	18-Nov-16
Technician	JW

Test Pit	RH16-14	RH16-14	RH16-14	RH16-14	RH16-14	RH16-14
Depth (m)	0.3 - 0.4	0.4 - 0.6	0.9 - 1.2	1.2 - 1.4	1.5 - 1.8	2.1 - 2.3
Sample #	G01	G02	G03	G04	G05	G06
Tare ID	W103	Z18	W77	Z115	F122	F90
Mass of tare	8.4	8.8	8.5	8.6	8.5	8.5
Mass wet + tare	408.3	388.9	345.8	434.5	372.0	375.0
Mass dry + tare	351.2	307.5	266.2	345.1	286.4	287.0
Mass water	57.1	81.4	79.6	89.4	85.6	88.0
Mass dry soil	342.8	298.7	257.7	336.5	277.9	278.5
Moisture %	16.7%	27.3%	30.9%	26.6%	30.8%	31.6%

Test Pit	RH16-14	RH16-15	RH16-15	RH16-15	RH16-15	RH16-15
Depth (m)	2.7 - 3.0	0.3 - 0.6	0.8 - 0.9	0.9 - 1.2	1.2 - 1.5	1.5 - 1.8
Sample #	G07	G08	G09	G10	G11	G12
Tare ID	H30	Z05	A21	AB57	E114	N106
Mass of tare	8.6	8.5	8.5	6.8	8.6	8.6
Mass wet + tare	371.2	357.9	395.2	431.7	355.1	369.8
Mass dry + tare	258.4	267.9	278.3	306.8	260.7	274.2
Mass water	112.8	90.0	116.9	124.9	94.4	95.6
Mass dry soil	249.8	259.4	269.8	300.0	252.1	265.6
Moisture %	45.2%	34.7%	43.3%	41.6%	37.4%	36.0%

Test Pit	RH16-15	RH16-15	RH16-12	RH16-12	RH16-12	RH16-12
Depth (m)	2.1 - 2.3	2.7 - 3.0	0.3 - 0.5	0.5 - 1.1	1.1 - 1.2	1.2 - 1.5
Sample #	G13	G14	G15	G16	G17	G18
Tare ID	H31	W01	Z121	F110	K14	AB29
Mass of tare	8.8	8.5	8.5	8.4	8.5	8.9
Mass wet + tare	426.9	428.8	399.0	469.5	362.9	366.2
Mass dry + tare	308.7	299.0	346.7	434.5	287.4	281.4
Mass water	118.2	129.8	52.3	35.0	75.5	84.8
Mass dry soil	299.9	290.5	338.2	426.1	278.9	272.5
Moisture %	39.4%	44.7%	15.5%	8.2%	27.1%	31.1%

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Moisture Content Report
ASTM D2216-98

Project No. 0035-037-00
Client Morrison Hershfield
Project Empress Widening

Sample Date 01-Nov-16
Test Date 18-Nov-16
Technician JW

Test Pit	RH16-12	RH16-12	RH16-12	RH16-19	RH16-19	RH16-19
Depth (m)	1.5 - 1.8	2.1 - 2.4	2.7 - 3.0	0.27-0.3	0.3 - 0.6	0.6 - 1.1
Sample #	G19	G20	G21	G22	G23	G24
Tare ID	H55	N99	D11	E44	Z54	W69
Mass of tare	8.7	8.6	8.5	8.4	8.1	8.3
Mass wet + tare	387.4	396.3	489.2	347.0	365.6	544.1
Mass dry + tare	292.0	299.7	385.4	310.3	275.4	408.4
Mass water	95.4	96.6	103.8	36.7	90.2	135.7
Mass dry soil	283.3	291.1	376.9	301.9	267.3	400.1
Moisture %	33.7%	33.2%	27.5%	12.2%	33.7%	33.9%

Test Pit	RH16-19	RH16-19	RH16-19	RH16-19	RH16-16	RH16-16
Depth (m)	1.1 - 1.5	1.5 - 1.8	2.1 - 2.4	2.7 - 3.0	0.28-0.3	0.3 - 0.6
Sample #	G25	G26	G27	G28	G29	G30
Tare ID	P01	AC32	AA01	Z101	H19	H16
Mass of tare	8.7	6.6	6.7	7.9	8.5	8.4
Mass wet + tare	362.8	453.9	357.6	349.0	275.1	370.8
Mass dry + tare	269.9	326.5	253.8	239.1	242.6	297.9
Mass water	92.9	127.4	103.8	109.9	32.5	72.9
Mass dry soil	261.2	319.9	247.1	231.2	234.1	289.5
Moisture %	35.6%	39.8%	42.0%	47.5%	13.9%	25.2%

Test Pit	RH16-16	RH16-16	RH16-16	RH16-16	RH16-16	RH16-17
Depth (m)	0.8 - 0.9	1.2 - 1.5	1.5 - 1.8	2.1 - 2.4	2.7 - 3.0	0.4 - 0.6
Sample #	G31	G32	G33	G34	G35	G36
Tare ID	W25	K20	AB66	AB83	H9	N72
Mass of tare	8.6	8.5	6.4	7.0	8.9	8.5
Mass wet + tare	350.0	379.4	353.2	425.6	366.0	384.0
Mass dry + tare	265.3	274.2	248.6	299.0	237.8	306.2
Mass water	84.7	105.2	104.6	126.6	128.2	77.8
Mass dry soil	256.7	265.7	242.2	292.0	228.9	297.7
Moisture %	33.0%	39.6%	43.2%	43.4%	56.0%	26.1%

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Moisture Content Report
ASTM D2216-98

Project No.	0035-037-00
Client	Morrison Hershfield
Project	Empress Widening
Sample Date	01-Nov-16
Test Date	18-Nov-16
Technician	JW

Test Pit	RH16-17	RH16-17	RH16-17	RH16-17	RH16-17	RH16-17
Depth (m)	0.6 - 0.8	0.8 - 0.9	1.2 - 1.5	1.5 - 1.8	2.1 - 2.4	2.7 - 3.0
Sample #	G37	G38	G39	G40	G41	G42
Tare ID	Z70	F21	E82	AA12	N69	H72
Mass of tare	8.4	8.4	8.9	7.0	8.7	8.5
Mass wet + tare	358.3	361.7	389.7	366.9	405.0	361.9
Mass dry + tare	305.1	279.5	285.2	263.3	276.9	240.4
Mass water	53.2	82.2	104.5	103.6	128.1	121.5
Mass dry soil	296.7	271.1	276.3	256.3	268.2	231.9
Moisture %	17.9%	30.3%	37.8%	40.4%	47.8%	52.4%

Test Pit	RH16-18	RH16-18	RH16-18	RH16-18	RH16-18	RH16-18
Depth (m)	0.2 - 0.3	0.3 - 0.5	0.5 - 0.6	0.6 - 0.9	1.2 - 1.5	1.5 - 1.8
Sample #	G43	G44	G45	G46	G47	G48
Tare ID	P03	W85	E52	Z52	E51	Z103
Mass of tare	8.6	8.7	8.5	8.7	8.6	8.5
Mass wet + tare	361.4	368.4	384.3	421.0	356.3	386.2
Mass dry + tare	296.5	286.1	299.0	322.6	290.9	312.3
Mass water	64.9	82.3	85.3	98.4	65.4	73.9
Mass dry soil	287.9	277.4	290.5	313.9	282.3	303.8
Moisture %	22.5%	29.7%	29.4%	31.3%	23.2%	24.3%

Test Pit	RH16-18	RH16-18	RH16-08	RH16-08	RH16-08	RH16-08
Depth (m)	2.1 - 2.4	2.4 - 2.7	0.3 - 0.6	0.9 - 1.2	1.2 - 1.5	1.5 - 1.8
Sample #	G49	G50	G51	G52	G53	G54
Tare ID	AB25	E111	A6	N11	AB69	C25
Mass of tare	6.8	8.6	9	8.7	6.7	8.4
Mass wet + tare	344.6	362.5	424.8	378.0	398.1	404.8
Mass dry + tare	282.4	243.8	359.9	242.5	326.4	323.8
Mass water	62.2	118.7	64.9	135.5	71.7	81.0
Mass dry soil	275.6	235.2	350.9	233.8	319.7	315.4
Moisture %	22.6%	50.5%	18.5%	58.0%	22.4%	25.7%

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Moisture Content Report
ASTM D2216-98

Project No. 0035-037-00
Client Morrison Hershfield
Project Empress Widening

Sample Date 01-Nov-16
Test Date 18-Nov-16
Technician JW

Test Pit	RH16-08	RH16-08	RH16-08	RH16-09	RH16-09	RH16-09
Depth (m)	2.1 - 2.4	2.4 - 2.7	2.7 - 3.0	0.3 - 0.6	0.6 - 0.9	0.9 - 1.2
Sample #	G55	G56	G57	G58	G59	G60
Tare ID	Z01	W95	W92	D49	K29	F93
Mass of tare	8.5	8.7	8.8	8.5	8.4	8.4
Mass wet + tare	354.1	407.8	352.5	437.1	506.5	411.3
Mass dry + tare	253.3	289.5	243.3	407.0	477.7	384.0
Mass water	100.8	118.3	109.2	30.1	28.8	27.3
Mass dry soil	244.8	280.8	234.5	398.5	469.3	375.6
Moisture %	41.2%	42.1%	46.6%	7.6%	6.1%	7.3%

Test Pit	RH16-09	RH16-09	RH16-09	RH16-09	RH16-09	RH16-13
Depth (m)	1.2 - 1.5	1.5 - 1.8	2.1 - 2.3	2.3 - 2.4	2.7 - 3.0	0.3 - 0.4
Sample #	G61	G62	G63	G64	G65	G66
Tare ID	AB97	AC31	N10	N48	P15	Z114
Mass of tare	6.8	6.7	8.5	8.5	8.6	8.5
Mass wet + tare	414.3	420.4	389.9	384.0	432.8	362.0
Mass dry + tare	375.2	353.3	336.4	313.2	331.3	342.3
Mass water	39.1	67.1	53.5	70.8	101.5	19.7
Mass dry soil	368.4	346.6	327.9	304.7	322.7	333.8
Moisture %	10.6%	19.4%	16.3%	23.2%	31.5%	5.9%

Test Pit	RH16-13	RH16-13	RH16-13	RH16-13	RH16-13	RH16-13
Depth (m)	0.4 - 0.6	0.9 - 1.2	1.2 - 1.5	1.7 - 1.8	2.3 - 2.3	2.4 - 2.7
Sample #	G67	G68	G69	G70	G71	G72
Tare ID	E38	H33	Z63	N03	E88	W75
Mass of tare	8.8	8.7	8.4	8.6	8.8	8.4
Mass wet + tare	420.0	454.4	433.4	358.0	261.3	370.5
Mass dry + tare	327.7	363.2	342.3	258.2	194.8	254.5
Mass water	92.3	91.2	91.1	99.8	66.5	116.0
Mass dry soil	318.9	354.5	333.9	249.6	186.0	246.1
Moisture %	28.9%	25.7%	27.3%	40.0%	35.8%	47.1%

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Moisture Content Report
ASTM D2216-98

Project No. 0035-037-00
Client Morrison Hershfield
Project Empress Widening

Sample Date 01-Nov-16
Test Date 18-Nov-16
Technician JW

Test Pit	RH16-13	RH16-03	RH16-03	RH16-03	RH16-03	RH16-03
Depth (m)	2.7 - 3.0	0.3 - 0.6	0.6 - 0.9	1.1 - 1.4	1.5 - 1.7	1.8 - 2.1
Sample #	G73	G74	G75	G76	G77	G78
Tare ID	H77	N19	N10	N04	C23	F56
Mass of tare	8.5	8.5	8.3	8.6	8.5	8.5
Mass wet + tare	411.0	333.5	515.7	487.3	379.9	367.5
Mass dry + tare	282.3	308.0	474.8	440.7	281.2	250.3
Mass water	128.7	25.5	40.9	46.6	98.7	117.2
Mass dry soil	273.8	299.5	466.5	432.1	272.7	241.8
Moisture %	47.0%	8.5%	8.8%	10.8%	36.2%	48.5%

Test Pit	RH16-03	RH16-03	RH16-03	RH16-03	RH16-03	RH16-03
Depth (m)	2.4 - 2.7	2.7 - 3.0	3.7 - 4.0	5.8 - 6.1	7.3 - 7.6	8.2 - 8.5
Sample #	G79	G80	G82	G84	G86	G88
Tare ID	AC39	W94	P05	D36	K5	AB98
Mass of tare	6.6	8.5	8.5	8.5	8.6	6.5
Mass wet + tare	390.0	395.7	407.9	348.2	405.0	402.8
Mass dry + tare	274.9	267.1	267.2	231.7	268.9	270.1
Mass water	115.1	128.6	140.7	116.5	136.1	132.7
Mass dry soil	268.3	258.6	258.7	223.2	260.3	263.6
Moisture %	42.9%	49.7%	54.4%	52.2%	52.3%	50.3%

Test Pit	RH16-03	RH16-03	RH16-03	RH16-10	RH16-10	RH16-10
Depth (m)	10.1 - 10.4	11.9 - 12.2	12.2 - 12.3	0.3 - 0.6	0.6 - 0.9	1.2 - 1.5
Sample #	G89	G90	SS91	G92	G93	G94
Tare ID	E22	E44	E87	D48	Z59	Z122
Mass of tare	8.6	8.5	8.5	8.5	8.6	8.4
Mass wet + tare	186.7	445.8	75.3	461.4	398.6	406.4
Mass dry + tare	164.2	405.3	69.5	433.8	376.9	380.8
Mass water	22.5	40.5	5.8	27.6	21.7	25.6
Mass dry soil	155.6	396.8	61.0	425.3	368.3	372.4
Moisture %	14.5%	10.2%	9.5%	6.5%	5.9%	6.9%

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Moisture Content Report
ASTM D2216-98

Project No.	0035-037-00
Client	Morrison Hershfield
Project	Empress Widening
Sample Date	01-Nov-16
Test Date	18-Nov-16
Technician	JW

Test Pit	RH16-10	RH16-10	RH16-10	RH16-11	RH16-11	RH16-11
Depth (m)	1.5 - 1.8	2.1 - 2.4	2.7 - 3.0	0.3 - 0.6	0.6 - 0.9	1.2 - 1.5
Sample #	G95	G96	G97	G98	G99	G100
Tare ID	Z23	N59	K34	Z114	AB18	N73
Mass of tare	8	8.5	8.8	8.3	6.7	8.4
Mass wet + tare	465.7	399.9	435.3	391.6	464.6	357.1
Mass dry + tare	430.4	370.4	405.4	362.1	422.9	331
Mass water	35.3	29.5	29.9	29.5	41.7	26.1
Mass dry soil	422.4	361.9	396.6	353.8	416.2	322.6
Moisture %	8.4%	8.2%	7.5%	8.3%	10.0%	8.1%

Test Pit	RH16-11	RH16-11	RH16-11	RH16-04	RH16-04	RH16-04
Depth (m)	1.8 - 2.1	2.1 - 2.6	2.7 - 3.0	0.28-0.3	0.3 - 0.8	0.8 - 1.1
Sample #	G101	G102	G103	G104	G105	G106
Tare ID	AB09	A7	A107	F17	H12	Z02
Mass of tare	6.7	8.2	8.6	8.5	8.7	8.5
Mass wet + tare	360.9	351.1	360.3	228.9	395.1	551.3
Mass dry + tare	333.2	277.9	288.5	197.4	285.2	431
Mass water	27.7	73.2	71.8	31.5	109.9	120.3
Mass dry soil	326.5	269.7	279.9	188.9	276.5	422.5
Moisture %	8.5%	27.1%	25.7%	16.7%	39.7%	28.5%

Test Pit	RH16-04	RH16-04	RH16-04	RH16-04	RH16-04	RH16-04
Depth (m)	1.1 - 1.5	1.5 - 1.8	2.1 - 2.4	2.7 - 3.0	4.3 - 4.6	5.8 - 6.1
Sample #	G107	G108	G109	G110	G111	G113
Tare ID	W88	W27	Z85	H50	P11	F04
Mass of tare	8.5	8.5	8.5	8.5	8.6	8.5
Mass wet + tare	420.2	354.9	455.4	420.7	401.7	356.3
Mass dry + tare	313.2	250.9	318.6	288.2	259.4	242.5
Mass water	107.0	104.0	136.8	132.5	142.3	113.8
Mass dry soil	304.7	242.4	310.1	279.7	250.8	234.0
Moisture %	35.1%	42.9%	44.1%	47.4%	56.7%	48.6%

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Moisture Content Report
ASTM D2216-98

Project No. 0035-037-00
Client Morrison Hershfield
Project Empress Widening

Sample Date 01-Nov-16
Test Date 18-Nov-16
Technician JW

Test Pit	RH16-04	RH16-04	RH16-04	RH16-04	RH16-04	RH16-05
Depth (m)	7.3 - 7.6	8.2 - 8.5	10.1 - 10.4	11.9 - 12.2	12.2 - 12.3	0.2 - 0.4
Sample #	G114	G116	G117	G118	G119	G120
Tare ID	F88	W73	H31	K31	E9	D47
Mass of tare	8.6	8.7	8.6	8.5	8.6	8.5
Mass wet + tare	404.8	452.5	460.5	385.3	217.2	367
Mass dry + tare	276.8	304.3	414.2	347.7	201.9	316.7
Mass water	128.0	148.2	46.3	37.6	15.3	50.3
Mass dry soil	268.2	295.6	405.6	339.2	193.3	308.2
Moisture %	47.7%	50.1%	11.4%	11.1%	7.9%	16.3%

Test Pit	RH16-05	RH16-05	RH16-05	RH16-05	RH16-05	RH16-05
Depth (m)	0.4 - 0.6	0.6 - 1.1	1.1 - 1.2	1.2 - 1.5	1.5 - 1.8	2.1 - 2.4
Sample #	G121	G122	G123	G124	G125	G126
Tare ID	H5	H15	Z108	E48	F124	W72
Mass of tare	8.6	8.5	8.5	8.5	8.5	8.5
Mass wet + tare	372	363.9	379.9	456.2	369.7	397.8
Mass dry + tare	314.9	301.7	305.2	380.5	304.3	269.3
Mass water	57.1	62.2	74.7	75.7	65.4	128.5
Mass dry soil	306.3	293.2	296.7	372.0	295.8	260.8
Moisture %	18.6%	21.2%	25.2%	20.3%	22.1%	49.3%

Test Pit	RH16-05	RH16-05	RH16-05	RH16-05	RH16-05	RH16-05
Depth (m)	3.7 - 4.0	4.1 - 4.4	5.8 - 6.1	7.3 - 7.6	8.8 - 9.1	10.4 - 10.7
Sample #	G127	G128	G129	G131	G132	G134
Tare ID	N50	Z109	G74	N27	D6	K11
Mass of tare	8.5	8.5	8.6	8.6	8.5	8.5
Mass wet + tare	352.1	384.6	359	382.2	507.5	486.5
Mass dry + tare	229.6	241.9	237.7	252.7	438	433.8
Mass water	122.5	142.7	121.3	129.5	69.5	52.7
Mass dry soil	221.1	233.4	229.1	244.1	429.5	425.3
Moisture %	55.4%	61.1%	52.9%	53.1%	16.2%	12.4%

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Moisture Content Report
ASTM D2216-98

Project No. 0035-037-00
Client Morrison Hershfield
Project Empress Widening

Sample Date 01-Nov-16
Test Date 18-Nov-16
Technician JW

Test Pit	RH16-05	RH16-05	RH16-06	RH16-06	RH16-06	RH16-06
Depth (m)	11.9 - 12.2	13.1 - 13.4	0.3 - 0.3	0.3 - 0.6	0.6 - 0.9	0.9 - 1.2
Sample #	G135	G136	G137	G138	G139	G140
Tare ID	F24	N68	N59	C22	E40	H52
Mass of tare	8.5	8.6	8.5	8.5	8.5	8.4
Mass wet + tare	229.7	247	210.3	340.7	417.3	347.8
Mass dry + tare	205.2	227.9	193.7	266.3	335.7	274.6
Mass water	24.5	19.1	16.6	74.4	81.6	73.2
Mass dry soil	196.7	219.3	185.2	257.8	327.2	266.2
Moisture %	12.5%	8.7%	9.0%	28.9%	24.9%	27.5%

Test Pit	RH16-06	RH16-06	RH16-06	RH16-06	RH16-06	RH16-06
Depth (m)	1.2 - 1.5	1.9 - 2.0	2.1 - 2.4	2.4 - 2.7	3.0 - 3.4	4.3 - 4.6
Sample #	G141	G142	G143	G144	G145	G146
Tare ID	F84	Z43	Z771	F56	N66	Z08
Mass of tare	8.5	8.5	8.5	8.4	8.3	8.3
Mass wet + tare	357.3	371.2	403.1	450.6	375.8	364.9
Mass dry + tare	266	305.7	294.5	367.8	246	241.4
Mass water	91.3	65.5	108.6	82.8	129.8	123.5
Mass dry soil	257.5	297.2	286.0	359.4	237.7	233.1
Moisture %	35.5%	22.0%	38.0%	23.0%	54.6%	53.0%

Test Pit	RH16-06	RH16-06	RH16-06	RH16-06	RH16-06	RH16-06
Depth (m)	5.8 - 6.1	7.3 - 7.6	8.8 - 9.1	10.4 - 10.7	11.9 - 12.2	13.1 - 13.4
Sample #	G148	G149	G151	G152	G153	G154
Tare ID	E13	P20	H57	W90	N75	A26
Mass of tare	8.6	8.5	8.5	8.5	8.6	8.5
Mass wet + tare	397.6	369.4	354.3	494.4	366.7	413
Mass dry + tare	258.7	246.5	305.8	449.9	344.5	384.8
Mass water	138.9	122.9	48.5	44.5	22.2	28.2
Mass dry soil	250.1	238.0	297.3	441.4	335.9	376.3
Moisture %	55.5%	51.6%	16.3%	10.1%	6.6%	7.5%

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Moisture Content Report
ASTM D2216-98

Project No. 0035-037-00
Client Morrison Hershfield
Project Empress Widening

Sample Date 01-Nov-16
Test Date 18-Nov-16
Technician JW

Test Pit	RH16-06	RH16-07	RH16-07	RH16-07	RH16-07	RH16-07
Depth (m)	13.4 - 13.5	0.28-0.33	0.3 - 0.6	0.8 - 1.1	1.2 - 1.5	1.5 - 1.8
Sample #	SS155	G156	G157	G158	G159	G160
Tare ID	AC20	Z84	AB01	E28	F37	N14
Mass of tare	6.4	8.5	6.5	8.3	8.2	8.3
Mass wet + tare	274	326	377.4	364.3	411.8	347.2
Mass dry + tare	253	305.5	294.5	284.4	329.7	283
Mass water	21.0	20.5	82.9	79.9	82.1	64.2
Mass dry soil	246.6	297.0	288.0	276.1	321.5	274.7
Moisture %	8.5%	6.9%	28.8%	28.9%	25.5%	23.4%

Test Pit	RH16-07	RH16-07	RH16-07	RH16-07	RH16-07	RH16-07
Depth (m)	2.1 - 2.7	2.7 - 3.0	3.0 - 3.4	4.3 - 4.6	5.8 - 6.1	7.3 - 7.6
Sample #	G161	G162	G163	G164	G165	G167
Tare ID	AA05	AC12	Z134	D45	E41	AB90
Mass of tare	6.5	6.5	8.3	8.3	8.3	6.7
Mass wet + tare	451.1	361.8	367.9	446.6	432.1	370.5
Mass dry + tare	361.7	262.5	242.5	302.5	283.5	248.7
Mass water	89.4	99.3	125.4	144.1	148.6	121.8
Mass dry soil	355.2	256.0	234.2	294.2	275.2	242.0
Moisture %	25.2%	38.8%	53.5%	49.0%	54.0%	50.3%

Test Pit	RH16-07	RH16-07	RH16-07	RH16-07	RH16-07	
Depth (m)	8.8 - 9.1	10.4 - 10.7	11.9 - 12.2	13.1 - 13.4	13.4 - 13.5	
Sample #	G168	G170	G171	G172	G173	
Tare ID	N83	AB93	F19	H26	E34	
Mass of tare	8.6	6.5	8.5	8.4	8.5	
Mass wet + tare	364	459.5	422.7	400.7	147.6	
Mass dry + tare	244.2	418.5	385.8	364.5	134.8	
Mass water	119.8	41.0	36.9	36.2	12.8	
Mass dry soil	235.6	412.0	377.3	356.1	126.3	
Moisture %	50.8%	10.0%	9.8%	10.2%	10.1%	

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Atterberg Limits
ASTM D4318-10e1

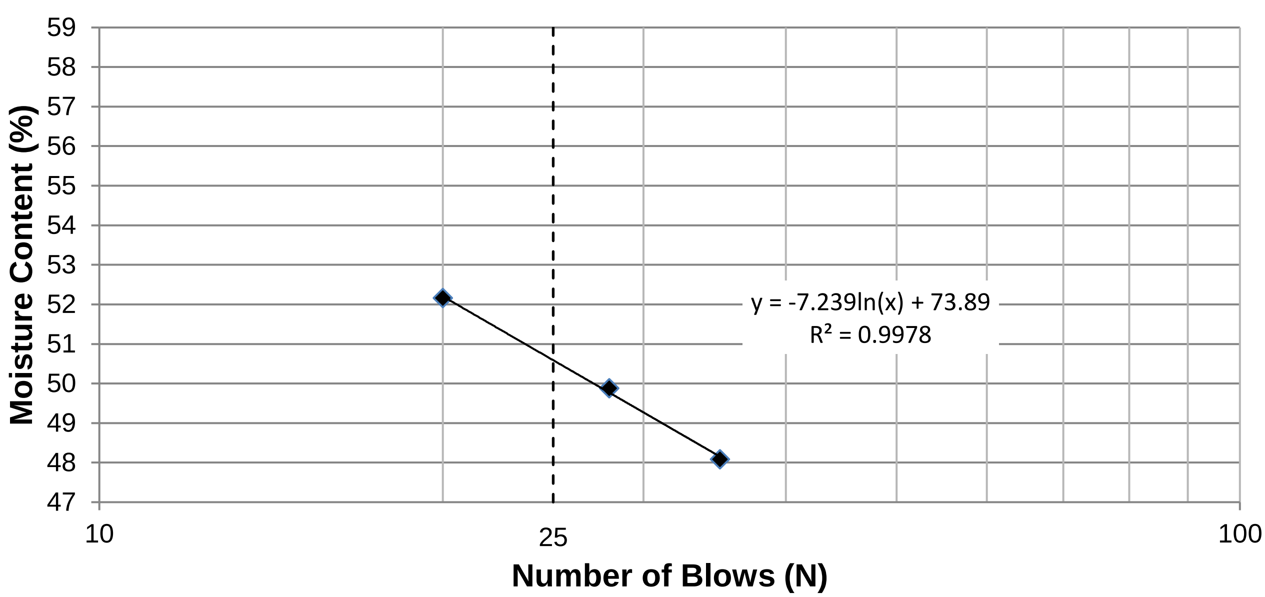
Project No.	0035-037-00
Client	Morrison Hershfield
Project	Empress Widening

Test Hole	RH16-19
Sample #	G24
Depth (m)	0.6-1.1
Sample Date	01-Nov-16
Test Date	28-Nov-16
Technician	JW

Liquid Limit	51
Plastic Limit	19
Plasticity Index	31

Liquid Limit

Trial #	**1**	**2**	**3**	**4**	**5**
Number of Blows (N)	35	28	20		
Mass Wet Soil + Tare (g)	20.182	21.009	19.082		
Mass Dry Soil + Tare (g)	18.095	18.754	17.368		
Mass Tare (g)	13.755	14.233	14.082		
Mass Water (g)	2.087	2.255	1.714		
Mass Dry Soil (g)	4.340	4.521	3.286		
Moisture Content (%)	48.088	49.878	52.161		



Plastic Limit

Trial #	**1**	**2**	**3**	**4**	**5**
Mass Wet Soil + Tare (g)	20.621	21.966			
Mass Dry Soil + Tare (g)	19.565	20.673			
Mass Tare (g)	14.186	13.950			
Mass Water (g)	1.056	1.293			
Mass Dry Soil (g)	5.379	6.723			
Moisture Content (%)	19.632	19.232			

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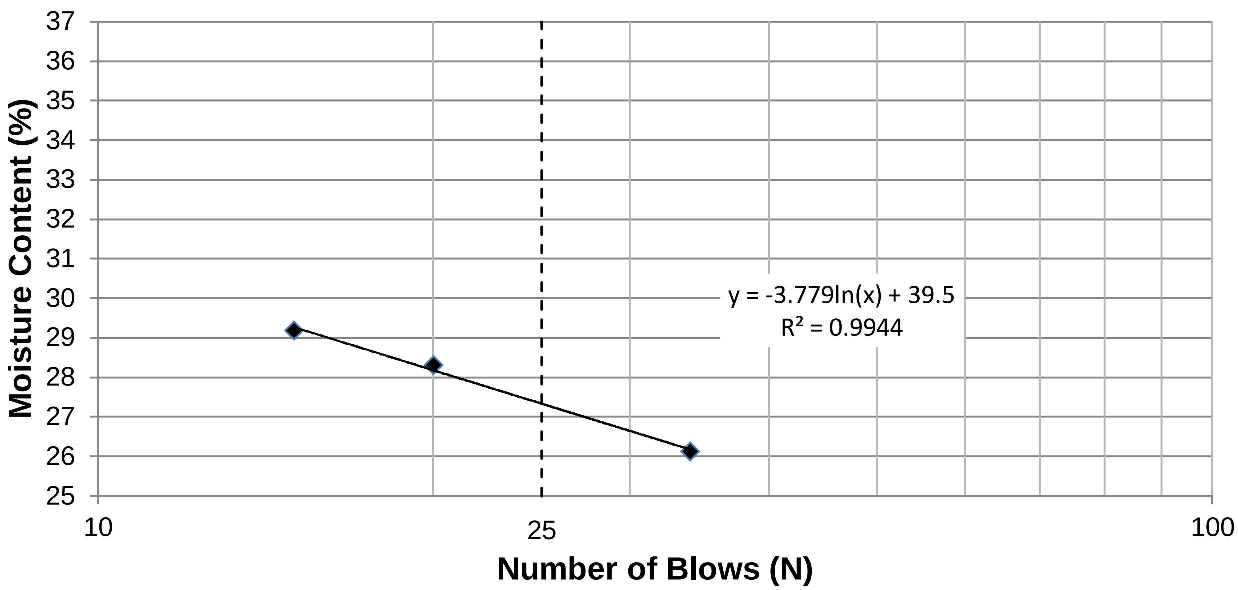
Atterberg Limits
ASTM D4318-10e1

Project No.	0035-037-00
Client	Morrison Hershfield
Project	Empress Widening
Test Hole	RH16-08
Sample #	G53
Depth (m)	1.2-1.5
Sample Date	01-Nov-16
Test Date	20-Dec-16
Technician	SGBR

Liquid Limit	27
Plastic Limit	15
Plasticity Index	12

Liquid Limit

Trial #	**1**	**2**	**3**	**4**	**5**
Number of Blows (N)	15	34	20		
Mass Wet Soil + Tare (g)	35.390	31.951	37.724		
Mass Dry Soil + Tare (g)	30.568	28.243	32.520		
Mass Tare (g)	14.043	14.051	14.140		
Mass Water (g)	4.822	3.708	5.204		
Mass Dry Soil (g)	16.525	14.192	18.380		
Moisture Content (%)	29.180	26.127	28.313		



Plastic Limit

Trial #	**1**	**2**	**3**	**4**	**5**
Mass Wet Soil + Tare (g)	21.109	20.233			
Mass Dry Soil + Tare (g)	20.099	19.427			
Mass Tare (g)	13.744	14.056			
Mass Water (g)	1.010	0.806			
Mass Dry Soil (g)	6.355	5.371			
Moisture Content (%)	15.893	15.007			

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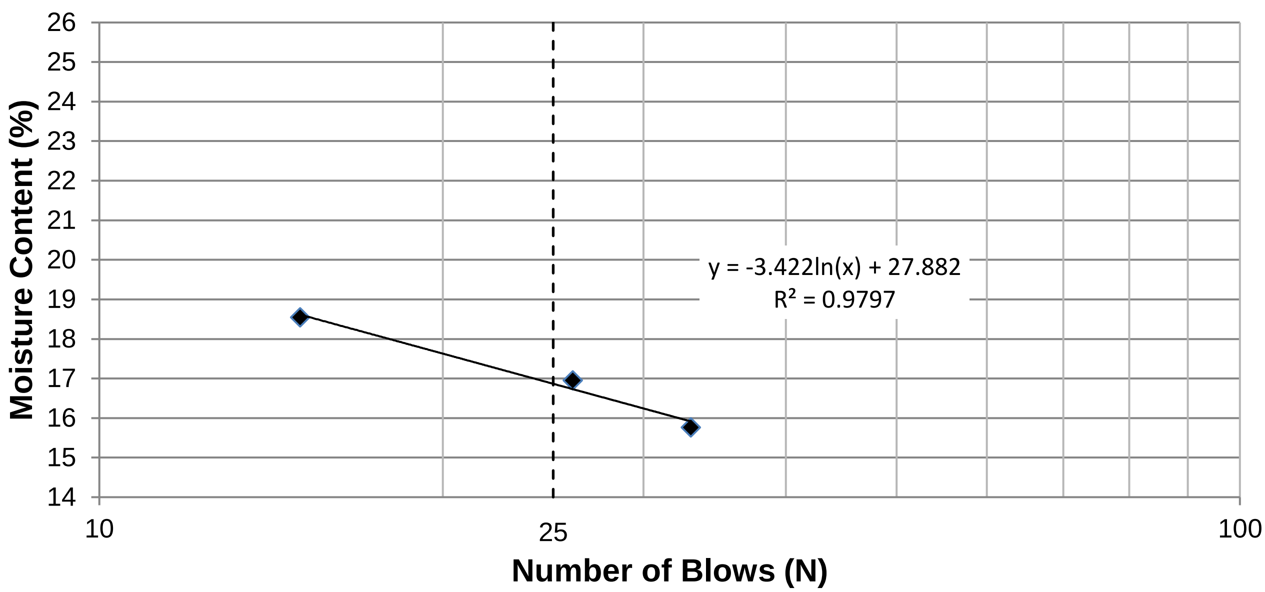
Atterberg Limits
ASTM D4318-10e1

Project No.	0035-037-00
Client	Morrison Hershfield
Project	Empress Widening
Test Hole	RH16-03
Sample #	G75
Depth (m)	0.6-0.9
Sample Date	01-Nov-16
Test Date	28-Nov-16
Technician	JW

Liquid Limit	17
Plastic Limit	14
Plasticity Index	3

Liquid Limit

Trial #	**1**	**2**	**3**	**4**	**5**
Number of Blows (N)	15	26	33		
Mass Wet Soil + Tare (g)	23.624	22.561	20.834		
Mass Dry Soil + Tare (g)	22.174	21.304	19.933		
Mass Tare (g)	14.356	13.891	14.216		
Mass Water (g)	1.450	1.257	0.901		
Mass Dry Soil (g)	7.818	7.413	5.717		
Moisture Content (%)	18.547	16.957	15.760		



Plastic Limit

Trial #	**1**	**2**	**3**	**4**	**5**
Mass Wet Soil + Tare (g)	30.753	26.599			
Mass Dry Soil + Tare (g)	28.771	25.107			
Mass Tare (g)	14.139	14.155			
Mass Water (g)	1.982	1.492			
Mass Dry Soil (g)	14.632	10.952			
Moisture Content (%)	13.546	13.623			

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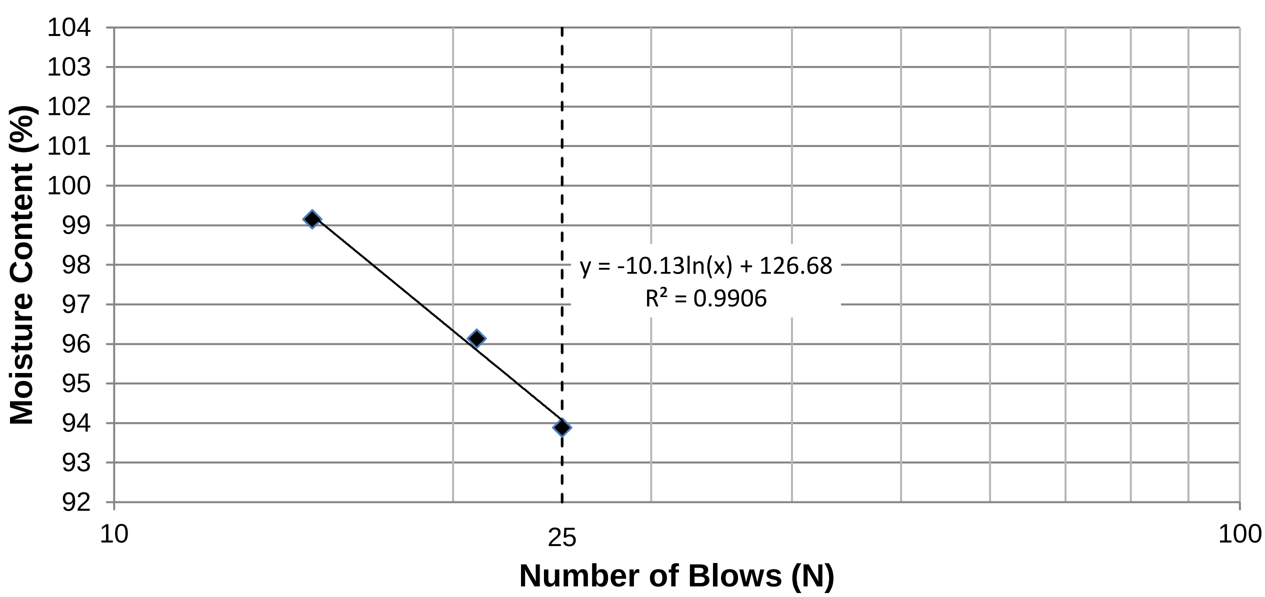
Atterberg Limits
ASTM D4318-10e1

Project No.	0035-037-00
Client	Morrison Hershfield
Project	Empress Widening
Test Hole	RH16-03
Sample #	T81
Depth (m)	3.0-3.7
Sample Date	02-Nov-16
Test Date	06-Dec-16
Technician	MM

Liquid Limit	94
Plastic Limit	26
Plasticity Index	68

Liquid Limit

Trial #	**1**	**2**	**3**	**4**	**5**
Number of Blows (N)	25	21	15		
Mass Wet Soil + Tare (g)	22.874	22.967	22.143		
Mass Dry Soil + Tare (g)	18.762	18.719	18.184		
Mass Tare (g)	14.382	14.300	14.191		
Mass Water (g)	4.112	4.248	3.959		
Mass Dry Soil (g)	4.380	4.419	3.993		
Moisture Content (%)	93.881	96.130	99.149		



Plastic Limit

Trial #	**1**	**2**	**3**	**4**	**5**
Mass Wet Soil + Tare (g)	20.060	20.333			
Mass Dry Soil + Tare (g)	18.837	19.082			
Mass Tare (g)	14.202	14.321			
Mass Water (g)	1.223	1.251			
Mass Dry Soil (g)	4.635	4.761			
Moisture Content (%)	26.386	26.276			

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Atterberg Limits
ASTM D4318-10e1

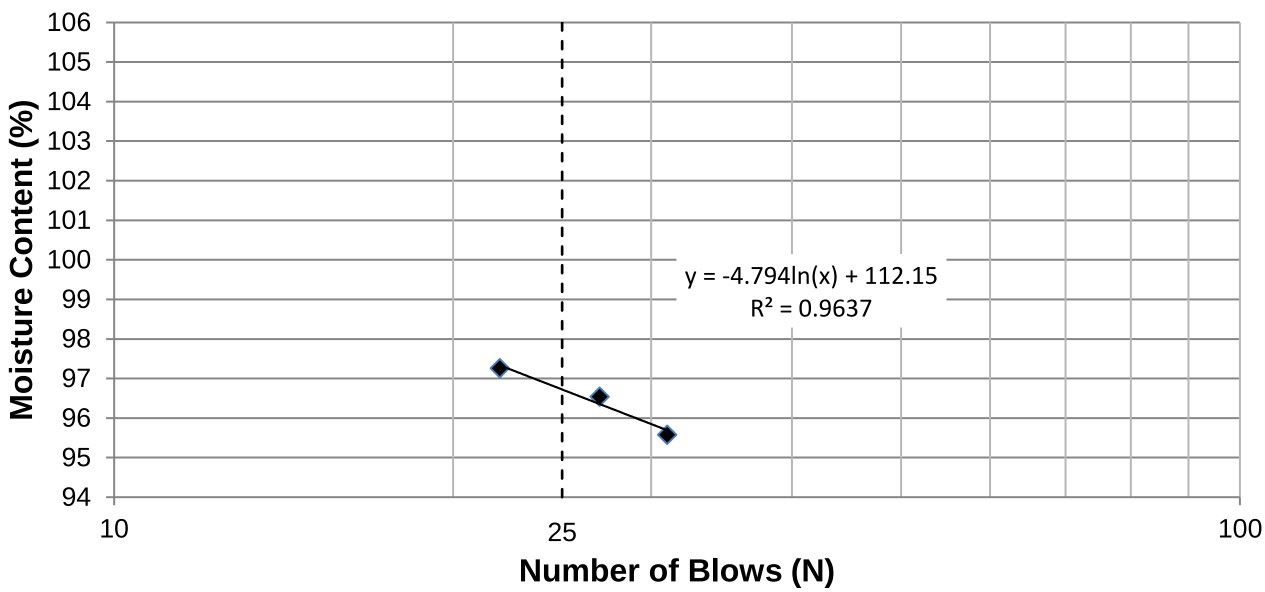
Project No.	0035-037-00
Client	Morrison Hershfield
Project	Empress Widening

Test Hole	RH16-03
Sample #	T83
Depth (m)	4.6-5.2
Sample Date	02-Nov-16
Test Date	06-Dec-16
Technician	MM

Liquid Limit	97
Plastic Limit	26
Plasticity Index	70

Liquid Limit

Trial #	**1**	**2**	**3**	**4**	**5**
Number of Blows (N)	31	22	27		
Mass Wet Soil + Tare (g)	21.503	19.314	18.421		
Mass Dry Soil + Tare (g)	17.718	16.793	16.302		
Mass Tare (g)	13.758	14.201	14.107		
Mass Water (g)	3.785	2.521	2.119		
Mass Dry Soil (g)	3.960	2.592	2.195		
Moisture Content (%)	95.581	97.261	96.538		



Plastic Limit

Trial #	**1**	**2**	**3**	**4**	**5**
Mass Wet Soil + Tare (g)	20.136	19.689			
Mass Dry Soil + Tare (g)	18.896	18.554			
Mass Tare (g)	14.153	14.287			
Mass Water (g)	1.240	1.135			
Mass Dry Soil (g)	4.743	4.267			
Moisture Content (%)	26.144	26.599			

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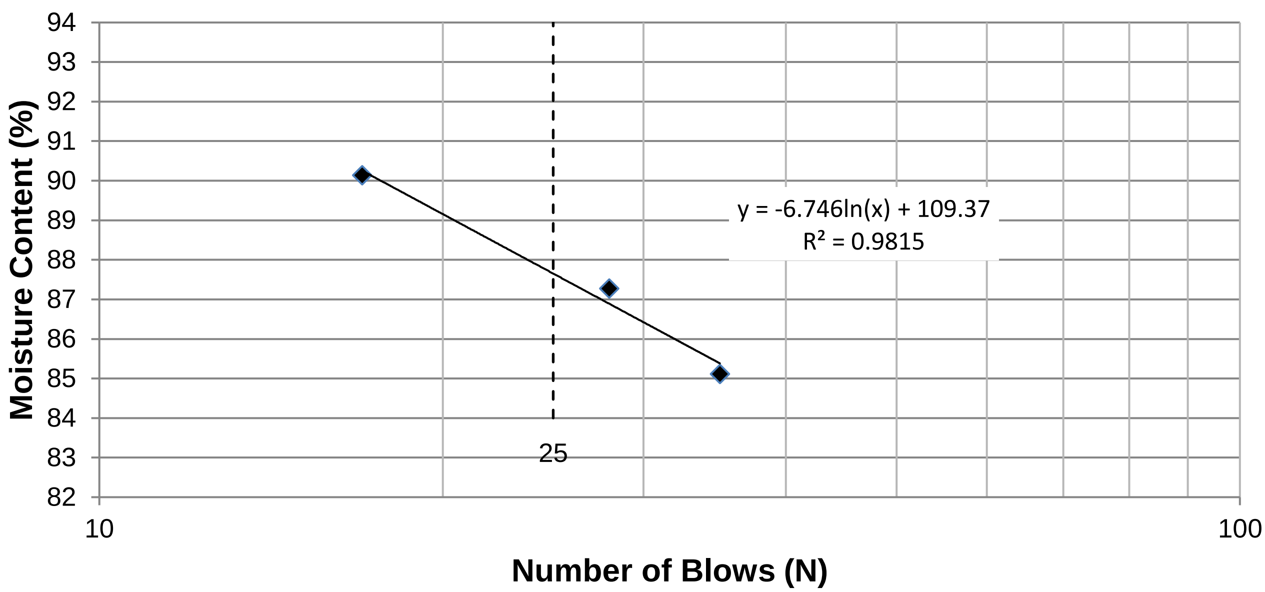
Atterberg Limits
ASTM D4318-10e1

Project No.	0035-037-00
Client	Morrison Hershfield
Project	Empress Widening
Test Hole	RH16-03
Sample #	T85
Depth (m)	6.1-6.7
Sample Date	03-Nov-16
Test Date	06-Dec-16
Technician	MM

Liquid Limit	88
Plastic Limit	25
Plasticity Index	62

Liquid Limit

Trial #	**1**	**2**	**3**	**4**	**5**
Number of Blows (N)	35	28	17		
Mass Wet Soil + Tare (g)	19.594	18.694	20.038		
Mass Dry Soil + Tare (g)	16.992	16.513	17.169		
Mass Tare (g)	13.935	14.014	13.986		
Mass Water (g)	2.602	2.181	2.869		
Mass Dry Soil (g)	3.057	2.499	3.183		
Moisture Content (%)	85.116	87.275	90.135		



Plastic Limit

Trial #	**1**	**2**	**3**	**4**	**5**
Mass Wet Soil + Tare (g)	19.996	20.064			
Mass Dry Soil + Tare (g)	18.824	18.889			
Mass Tare (g)	14.205	14.209			
Mass Water (g)	1.172	1.175			
Mass Dry Soil (g)	4.619	4.680			
Moisture Content (%)	25.373	25.107			

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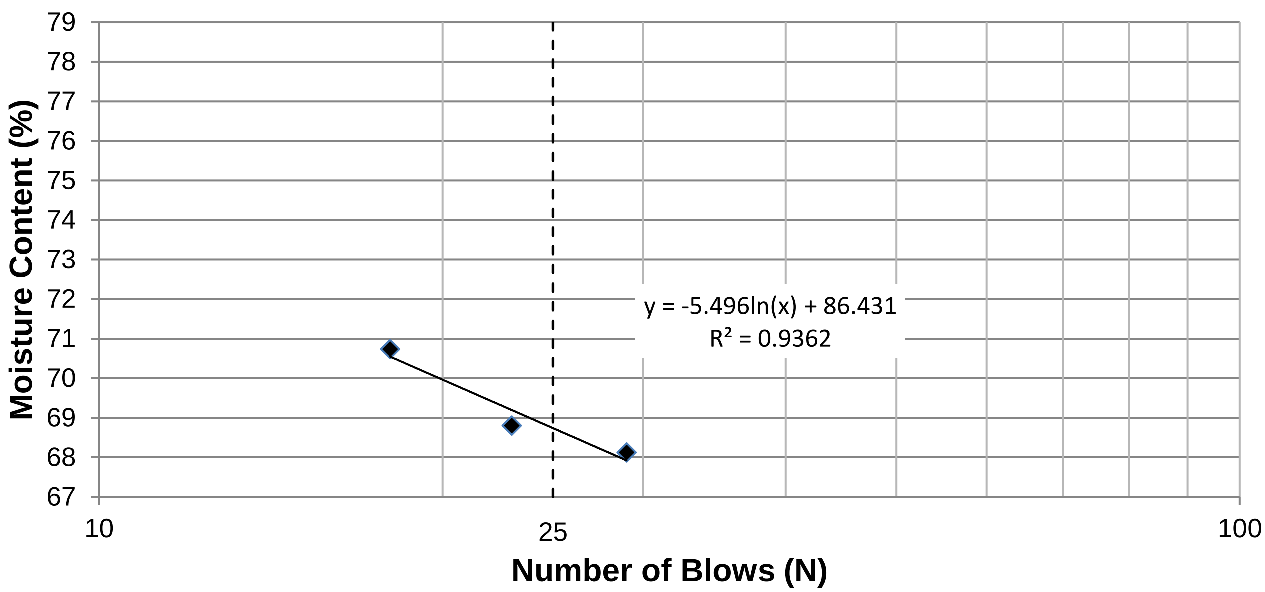
Atterberg Limits
ASTM D4318-10e1

Project No.	0035-037-00
Client	Morrison Hershfield
Project	Empress Widening
Test Hole	RH16-03
Sample #	T87
Depth (m)	7.6-8.2
Sample Date	03-Nov-16
Test Date	06-Dec-16
Technician	MM

Liquid Limit	69
Plastic Limit	25
Plasticity Index	44

Liquid Limit

Trial #	**1**	**2**	**3**	**4**	**5**
Number of Blows (N)	29	23	18		
Mass Wet Soil + Tare (g)	20.284	22.327	19.728		
Mass Dry Soil + Tare (g)	17.811	18.988	17.405		
Mass Tare (g)	14.181	14.135	14.121		
Mass Water (g)	2.473	3.339	2.323		
Mass Dry Soil (g)	3.630	4.853	3.284		
Moisture Content (%)	68.127	68.803	70.737		



Plastic Limit

Trial #	**1**	**2**	**3**	**4**	**5**
Mass Wet Soil + Tare (g)	19.957	20.284			
Mass Dry Soil + Tare (g)	18.784	19.036			
Mass Tare (g)	14.047	13.952			
Mass Water (g)	1.173	1.248			
Mass Dry Soil (g)	4.737	5.084			
Moisture Content (%)	24.763	24.548			

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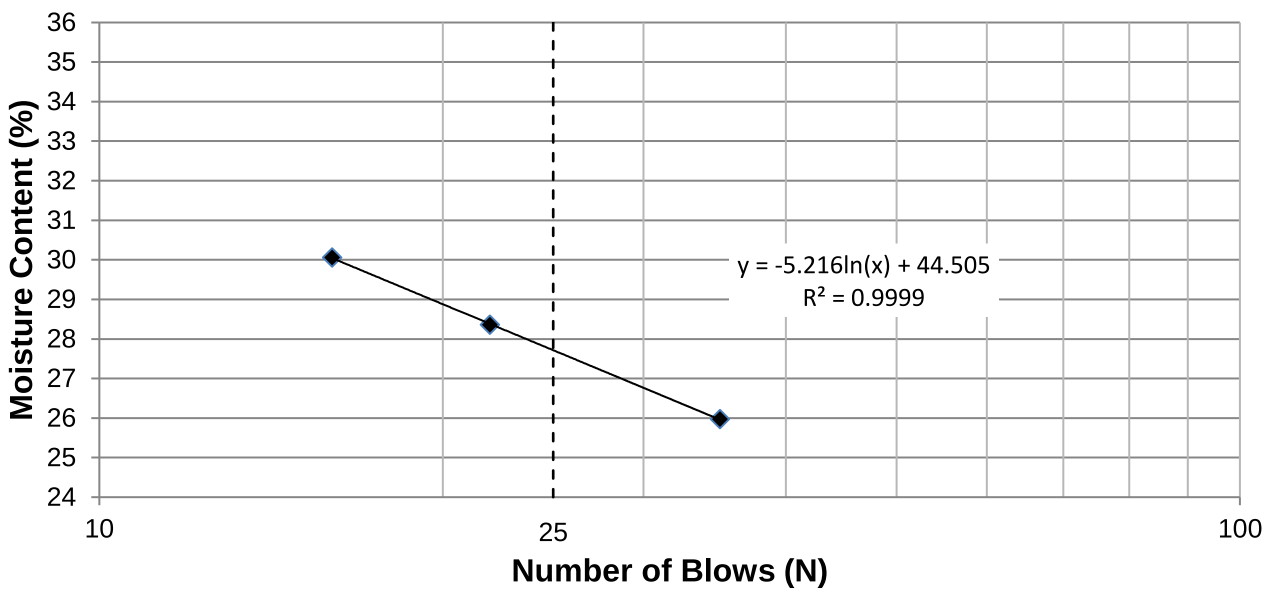
Atterberg Limits
ASTM D4318-10e1

Project No.	0035-037-00
Client	Morrison Hershfield
Project	Empress Widening
Test Hole	RH16-04
Sample #	G106
Depth (m)	0.8-1.1
Sample Date	01-Nov-16
Test Date	28-Nov-16
Technician	JW

Liquid Limit	28
Plastic Limit	19
Plasticity Index	9

Liquid Limit

Trial #	**1**	**2**	**3**	**4**	**5**
Number of Blows (N)	22	35	16		
Mass Wet Soil + Tare (g)	22.490	22.521	23.000		
Mass Dry Soil + Tare (g)	20.609	20.801	20.960		
Mass Tare (g)	13.976	14.178	14.173		
Mass Water (g)	1.881	1.720	2.040		
Mass Dry Soil (g)	6.633	6.623	6.787		
Moisture Content (%)	28.358	25.970	30.057		



Plastic Limit

Trial #	**1**	**2**	**3**	**4**	**5**
Mass Wet Soil + Tare (g)	28.061	30.811			
Mass Dry Soil + Tare (g)	25.904	28.186			
Mass Tare (g)	14.221	14.084			
Mass Water (g)	2.157	2.625			
Mass Dry Soil (g)	11.683	14.102			
Moisture Content (%)	18.463	18.614			

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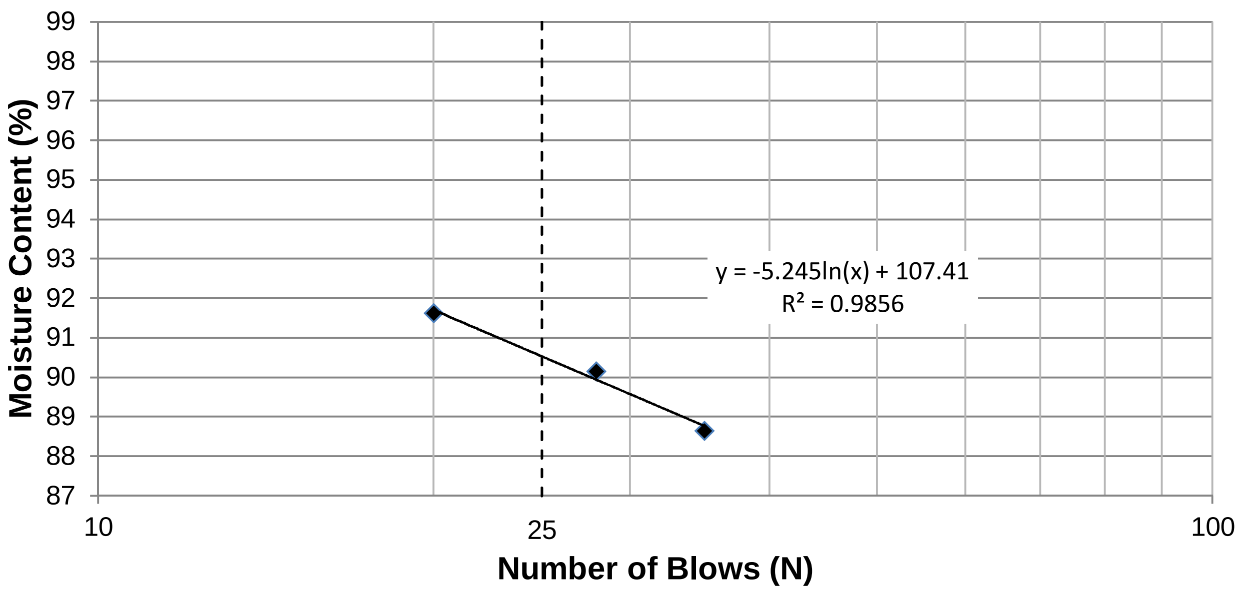
Atterberg Limits
ASTM D4318-10e1

Project No.	0035-037-00
Client	Morrison Hershfield
Project	Empress Widening
Test Hole	RH16-04
Sample #	T112
Depth (m)	4.6-5.2
Sample Date	27-Oct-16
Test Date	06-Dec-16
Technician	MM

Liquid Limit	91
Plastic Limit	25
Plasticity Index	65

Liquid Limit

Trial #	**1**	**2**	**3**	**4**	**5**
Number of Blows (N)	35	28	20		
Mass Wet Soil + Tare (g)	19.712	18.724	21.340		
Mass Dry Soil + Tare (g)	17.105	16.502	17.896		
Mass Tare (g)	14.164	14.037	14.137		
Mass Water (g)	2.607	2.222	3.444		
Mass Dry Soil (g)	2.941	2.465	3.759		
Moisture Content (%)	88.643	90.142	91.620		



Plastic Limit

Trial #	**1**	**2**	**3**	**4**	**5**
Mass Wet Soil + Tare (g)	20.009	20.036			
Mass Dry Soil + Tare (g)	18.819	18.855			
Mass Tare (g)	14.078	14.133			
Mass Water (g)	1.190	1.181			
Mass Dry Soil (g)	4.741	4.722			
Moisture Content (%)	25.100	25.011			

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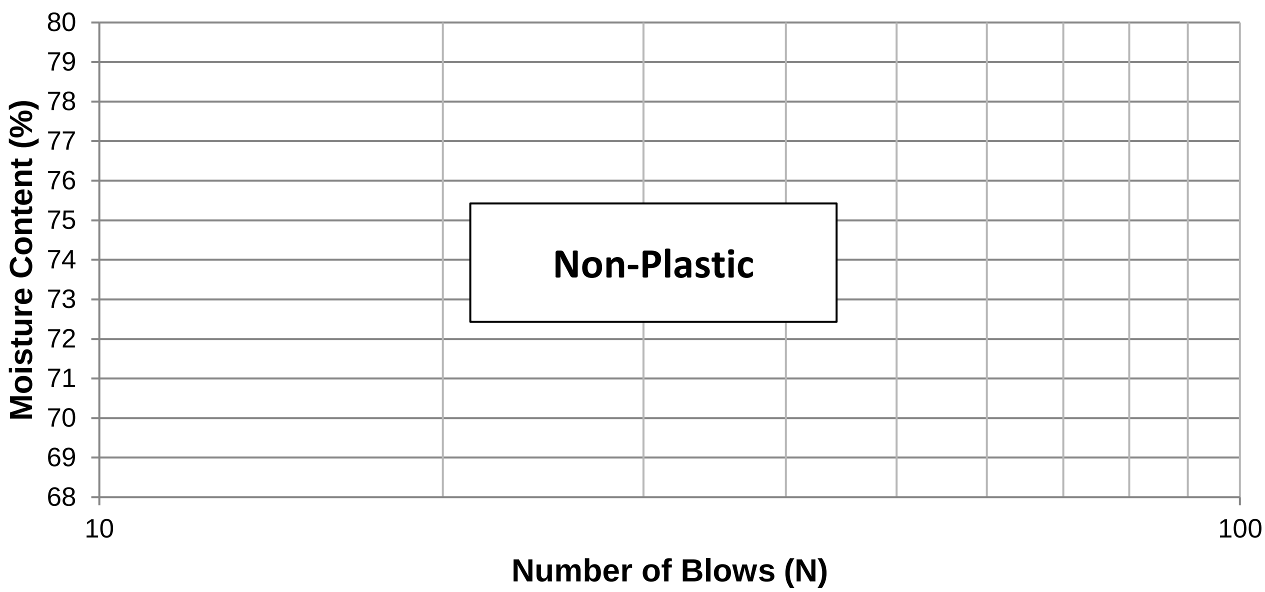
Atterberg Limits
ASTM D4318

Project No.	0035-037-00
Client	Morrison Hershfield
Project	Empress
Test Hole	RH16-05
Sample #	G122
Depth (m)	0.6 - 1.1
Sample Date	01-Nov-16
Test Date	06-Dec-16
Technician	MM

Liquid Limit	
Plastic Limit	
Plasticity Index	

Liquid Limit

Trial #	**1**	**2**	**3**	**4**	**5**
Number of Blows (N)					
Mass Wet Soil + Tare (g)					
Mass Dry Soil + Tare (g)					
Mass Tare (g)					
Mass Water (g)					
Mass Dry Soil (g)					
Moisture Content (%)					



Plastic Limit

Trial #	**1**	**2**	**3**	**4**	**5**
Mass Wet Soil + Tare (g)					
Mass Dry Soil + Tare (g)					
Mass Tare (g)					
Mass Water (g)					
Mass Dry Soil (g)					
Moisture Content (%)					

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Atterberg Limits
ASTM D4318-10e1

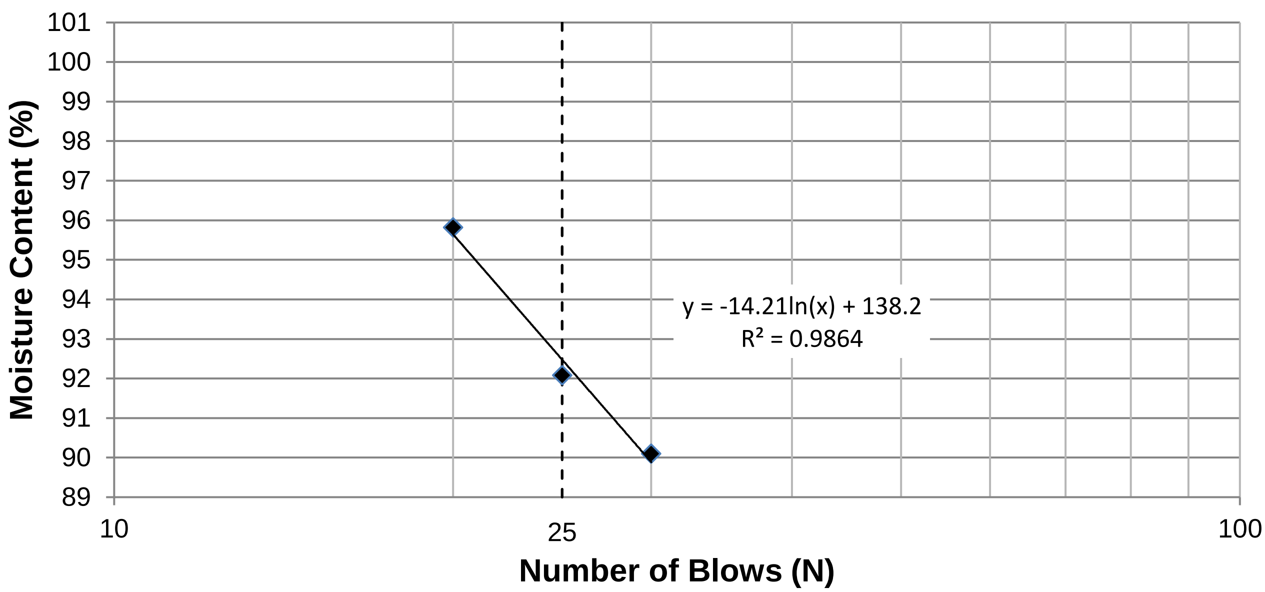
Project No.	0035-037-00
Client	Morrison Hershfield
Project	Empress Widening

Test Hole	RH16-05
Sample #	G127
Depth (m)	3.7-4.0
Sample Date	01-Nov-16
Test Date	28-Nov-16
Technician	JW

Liquid Limit	92
Plastic Limit	28
Plasticity Index	64

Liquid Limit

Trial #	**1**	**2**	**3**	**4**	**5**
Number of Blows (N)	25	30	20		
Mass Wet Soil + Tare (g)	21.990	19.015	21.607		
Mass Dry Soil + Tare (g)	18.220	16.521	17.823		
Mass Tare (g)	14.126	13.753	13.874		
Mass Water (g)	3.770	2.494	3.784		
Mass Dry Soil (g)	4.094	2.768	3.949		
Moisture Content (%)	92.086	90.101	95.822		



Plastic Limit

Trial #	**1**	**2**	**3**	**4**	**5**
Mass Wet Soil + Tare (g)	21.064	22.651			
Mass Dry Soil + Tare (g)	19.542	20.778			
Mass Tare (g)	14.187	14.140			
Mass Water (g)	1.522	1.873			
Mass Dry Soil (g)	5.355	6.638			
Moisture Content (%)	28.422	28.216			

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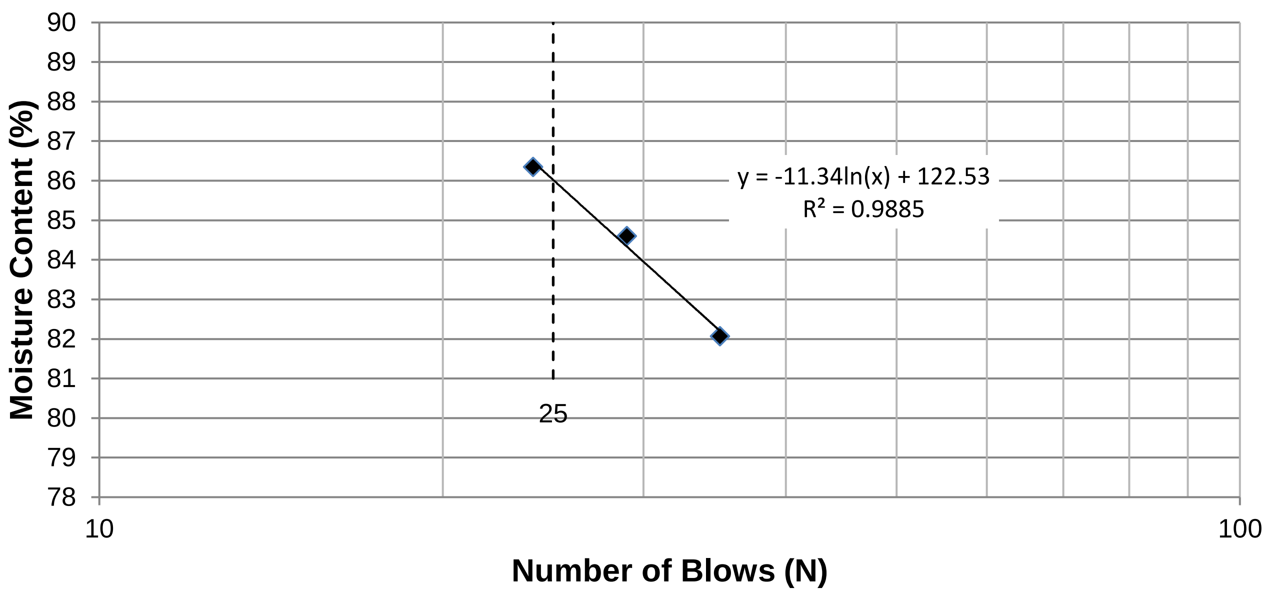
Atterberg Limits
ASTM D4318-10e1

Project No.	0035 - 037 - 00
Client	Morrison Hershfield
Project	Empress Overpass
Test Hole	RH16 - 05
Sample #	T130
Depth (m)	6.1-6.7
Sample Date	03-Nov-16
Test Date	08-Dec-16
Technician	MK / MM

Liquid Limit	86
Plastic Limit	24
Plasticity Index	62

Liquid Limit

Trial #	**1**	**2**	**3**	**4**	**5**
Number of Blows (N)	35	29	24		
Mass Wet Soil + Tare (g)	20.670	21.065	22.346		
Mass Dry Soil + Tare (g)	17.722	17.834	18.569		
Mass Tare (g)	14.130	14.015	14.195		
Mass Water (g)	2.948	3.231	3.777		
Mass Dry Soil (g)	3.592	3.819	4.374		
Moisture Content (%)	82.071	84.603	86.351		



Plastic Limit

Trial #	**1**	**2**	**3**	**4**	**5**
Mass Wet Soil + Tare (g)	20.125	20.213			
Mass Dry Soil + Tare (g)	18.988	18.930			
Mass Tare (g)	14.160	13.886			
Mass Water (g)	1.137	1.283			
Mass Dry Soil (g)	4.828	5.044			
Moisture Content (%)	23.550	25.436			

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Atterberg Limits
ASTM D4318-10e1

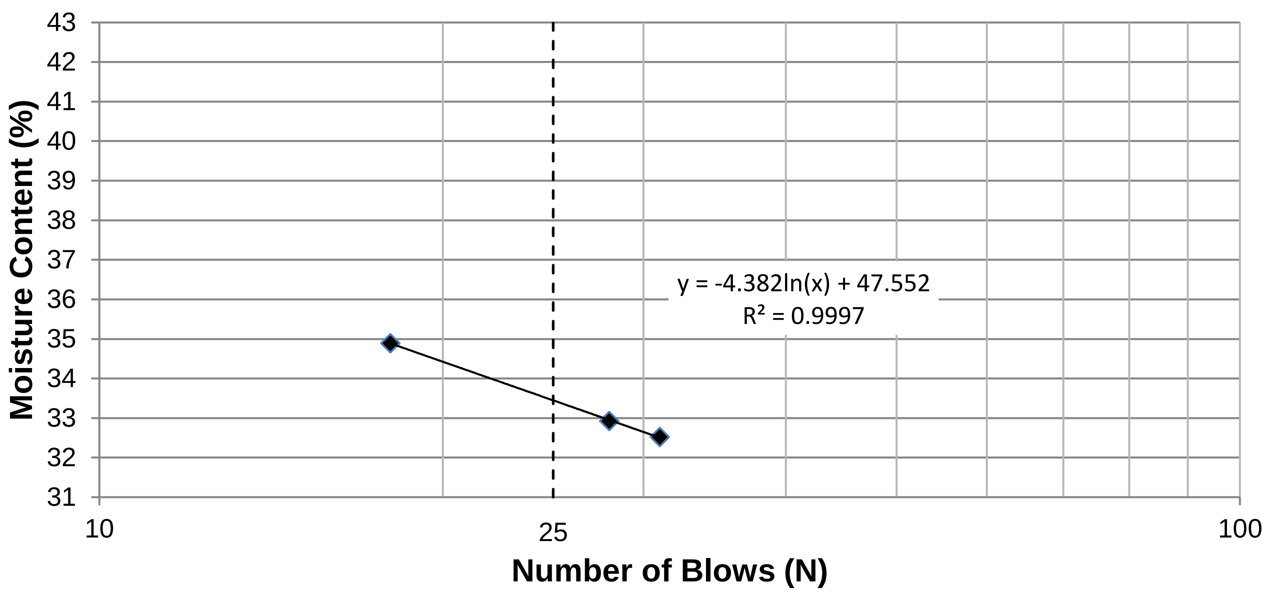
Project No.	0035-037-00
Client	Morrison Hershfield
Project	Empress Widening

Test Hole	RH16-06
Sample #	G140
Depth (m)	0.9-1.2
Sample Date	01-Nov-16
Test Date	20-Dec-16
Technician	SGBR

Liquid Limit	33
Plastic Limit	11
Plasticity Index	23

Liquid Limit

Trial #	**1**	**2**	**3**	**4**	**5**
Number of Blows (N)	28	31	18		
Mass Wet Soil + Tare (g)	34.312	30.673	28.887		
Mass Dry Soil + Tare (g)	29.300	26.611	25.050		
Mass Tare (g)	14.078	14.122	14.053		
Mass Water (g)	5.012	4.062	3.837		
Mass Dry Soil (g)	15.222	12.489	10.997		
Moisture Content (%)	32.926	32.525	34.891		



Plastic Limit

Trial #	**1**	**2**	**3**	**4**	**5**
Mass Wet Soil + Tare (g)	22.205	23.081			
Mass Dry Soil + Tare (g)	21.361	22.279			
Mass Tare (g)	14.123	14.149			
Mass Water (g)	0.844	0.802			
Mass Dry Soil (g)	7.238	8.130			
Moisture Content (%)	11.661	9.865			

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Atterberg Limits
ASTM D4318

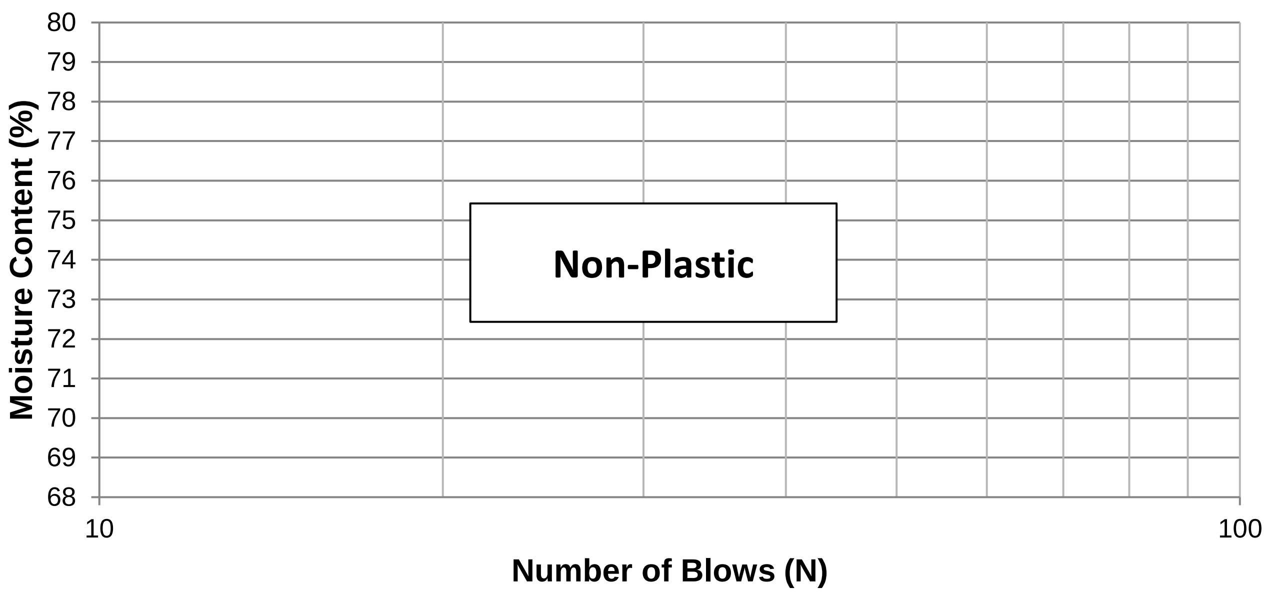
Project No.	0035-037-00
Client	Morrison Hershfield
Project	Empress

Test Hole	RH16-06
Sample #	G144
Depth (m)	2.4 - 2.7
Sample Date	01-Nov-16
Test Date	06-Dec-16
Technician	MM

Liquid Limit	
Plastic Limit	
Plasticity Index	

Liquid Limit

Trial #	**1**	**2**	**3**	**4**	**5**
Number of Blows (N)					
Mass Wet Soil + Tare (g)					
Mass Dry Soil + Tare (g)					
Mass Tare (g)					
Mass Water (g)					
Mass Dry Soil (g)					
Moisture Content (%)					



Plastic Limit

Trial #	**1**	**2**	**3**	**4**	**5**
Mass Wet Soil + Tare (g)					
Mass Dry Soil + Tare (g)					
Mass Tare (g)					
Mass Water (g)					
Mass Dry Soil (g)					
Moisture Content (%)					

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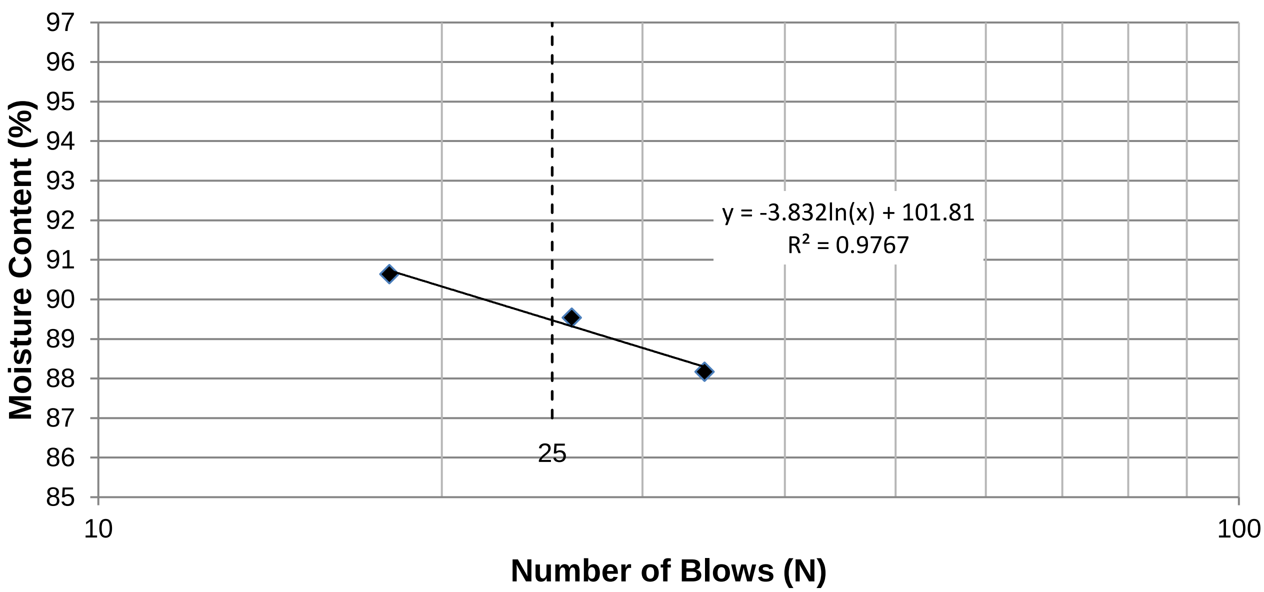
Atterberg Limits
ASTM D4318-10e1

Project No.	0035 - 037 - 00
Client	Morrison Hershfield
Project	Empress Overpass
Test Hole	RH16 - 06
Sample #	T147
Depth (m)	4.6-5.2
Sample Date	03-Nov-16
Test Date	08-Dec-16
Technician	MK / MM

Liquid Limit	89
Plastic Limit	25
Plasticity Index	65

Liquid Limit

Trial #	**1**	**2**	**3**	**4**	**5**
Number of Blows (N)	26	34	18		
Mass Wet Soil + Tare (g)	20.036	20.816	20.741		
Mass Dry Soil + Tare (g)	17.314	17.701	17.623		
Mass Tare (g)	14.274	14.168	14.183		
Mass Water (g)	2.722	3.115	3.118		
Mass Dry Soil (g)	3.040	3.533	3.440		
Moisture Content (%)	89.539	88.169	90.640		



Plastic Limit

Trial #	**1**	**2**	**3**	**4**	**5**
Mass Wet Soil + Tare (g)	20.080	20.201			
Mass Dry Soil + Tare (g)	18.889	18.989			
Mass Tare (g)	14.122	14.099			
Mass Water (g)	1.191	1.212			
Mass Dry Soil (g)	4.767	4.890			
Moisture Content (%)	24.984	24.785			

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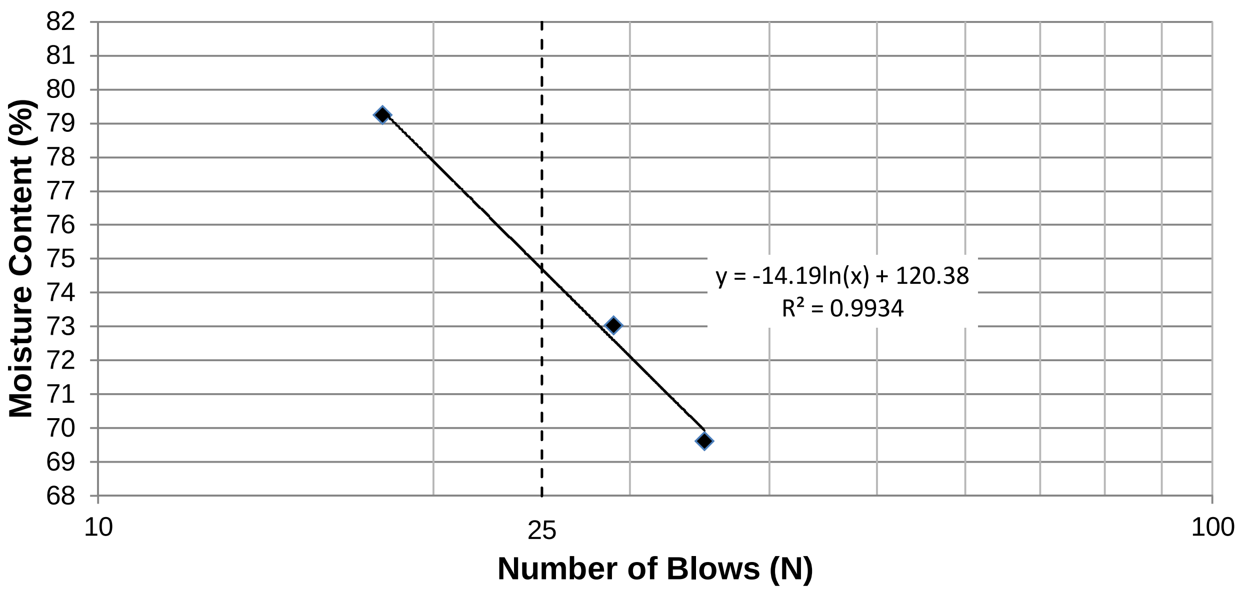
Atterberg Limits
ASTM D4318-10e1

Project No.	0035-037-00
Client	Morrison Hershfield
Project	Empress Widening
Test Hole	RH16-06
Sample #	T150
Depth (m)	7.6-8.2
Sample Date	01-Nov-16
Test Date	22-Dec-16
Technician	JW

Liquid Limit	75
Plastic Limit	22
Plasticity Index	53

Liquid Limit

Trial #	**1**	**2**	**3**	**4**	**5**
Number of Blows (N)	18	29	35		
Mass Wet Soil + Tare (g)	30.128	29.884	33.298		
Mass Dry Soil + Tare (g)	23.128	23.252	25.425		
Mass Tare (g)	14.294	14.172	14.115		
Mass Water (g)	7.000	6.632	7.873		
Mass Dry Soil (g)	8.834	9.080	11.310		
Moisture Content (%)	79.239	73.040	69.611		



Plastic Limit

Trial #	**1**	**2**	**3**	**4**	**5**
Mass Wet Soil + Tare (g)	21.216	22.272			
Mass Dry Soil + Tare (g)	19.957	20.827			
Mass Tare (g)	14.262	14.192			
Mass Water (g)	1.259	1.445			
Mass Dry Soil (g)	5.695	6.635			
Moisture Content (%)	22.107	21.778			

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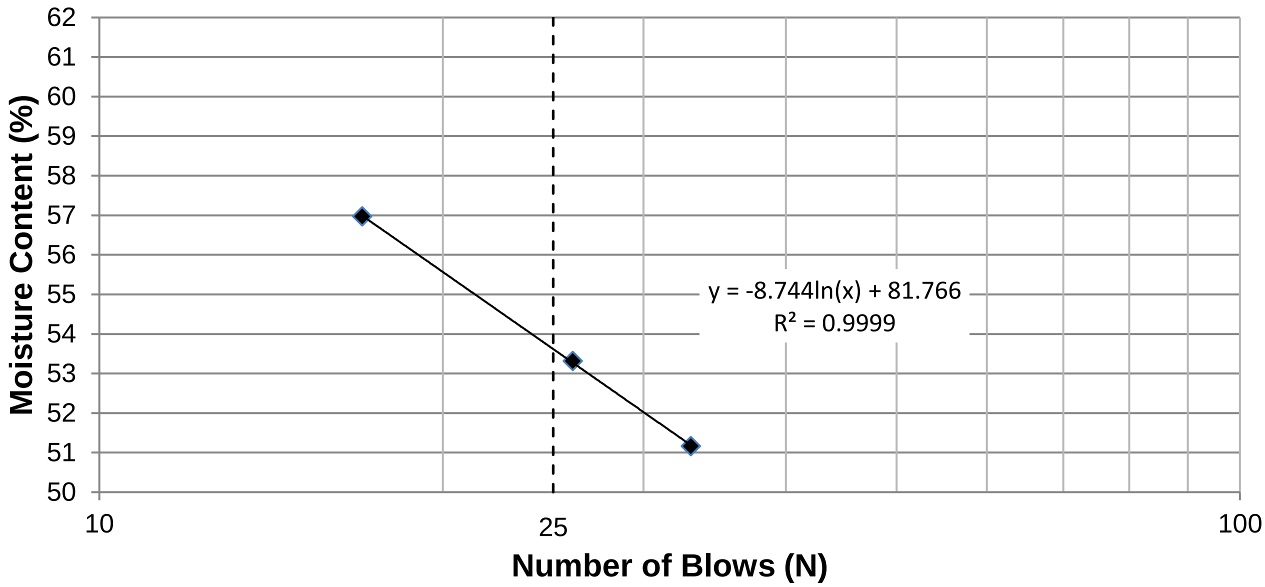
Atterberg Limits
ASTM D4318-10e1

Project No.	0035-037-00
Client	Morrison Hershfield
Project	Empress Widening
Test Hole	RH16-07
Sample #	G157
Depth (m)	0.3-0.6
Sample Date	01-Nov-16
Test Date	28-Nov-16
Technician	JW

Liquid Limit	54
Plastic Limit	20
Plasticity Index	33

Liquid Limit

Trial #	**1**	**2**	**3**	**4**	**5**
Number of Blows (N)	33	17	26		
Mass Wet Soil + Tare (g)	19.110	18.740	17.673		
Mass Dry Soil + Tare (g)	17.397	17.078	16.435		
Mass Tare (g)	14.049	14.161	14.113		
Mass Water (g)	1.713	1.662	1.238		
Mass Dry Soil (g)	3.348	2.917	2.322		
Moisture Content (%)	51.165	56.976	53.316		



Plastic Limit

Trial #	**1**	**2**	**3**	**4**	**5**
Mass Wet Soil + Tare (g)	22.216	22.025			
Mass Dry Soil + Tare (g)	20.797	20.718			
Mass Tare (g)	13.884	14.290			
Mass Water (g)	1.419	1.307			
Mass Dry Soil (g)	6.913	6.428			
Moisture Content (%)	20.527	20.333			

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Atterberg Limits
ASTM D4318-10e1

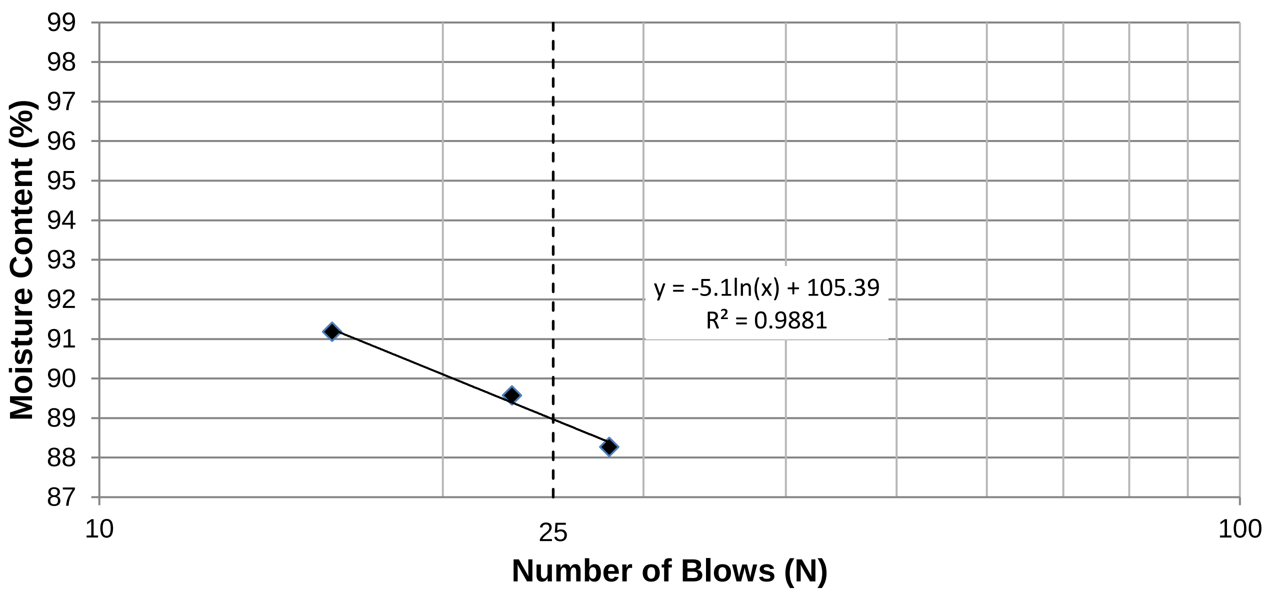
Project No.	0035-037-00
Client	Morrison Hershfield
Project	Empress Widening

Test Hole	RH16-07
Sample #	T166
Depth (m)	6.1-6.7
Sample Date	03-Nov-16
Test Date	06-Dec-16
Technician	MM

Liquid Limit	89
Plastic Limit	27
Plasticity Index	62

Liquid Limit

Trial #	**1**	**2**	**3**	**4**	**5**
Number of Blows (N)	28	23	16		
Mass Wet Soil + Tare (g)	20.124	20.296	21.615		
Mass Dry Soil + Tare (g)	17.369	17.332	17.924		
Mass Tare (g)	14.248	14.023	13.876		
Mass Water (g)	2.755	2.964	3.691		
Mass Dry Soil (g)	3.121	3.309	4.048		
Moisture Content (%)	88.273	89.574	91.181		



Plastic Limit

Trial #	**1**	**2**	**3**	**4**	**5**
Mass Wet Soil + Tare (g)	20.062	20.200			
Mass Dry Soil + Tare (g)	18.823	18.890			
Mass Tare (g)	14.274	14.116			
Mass Water (g)	1.239	1.310			
Mass Dry Soil (g)	4.549	4.774			
Moisture Content (%)	27.237	27.440			

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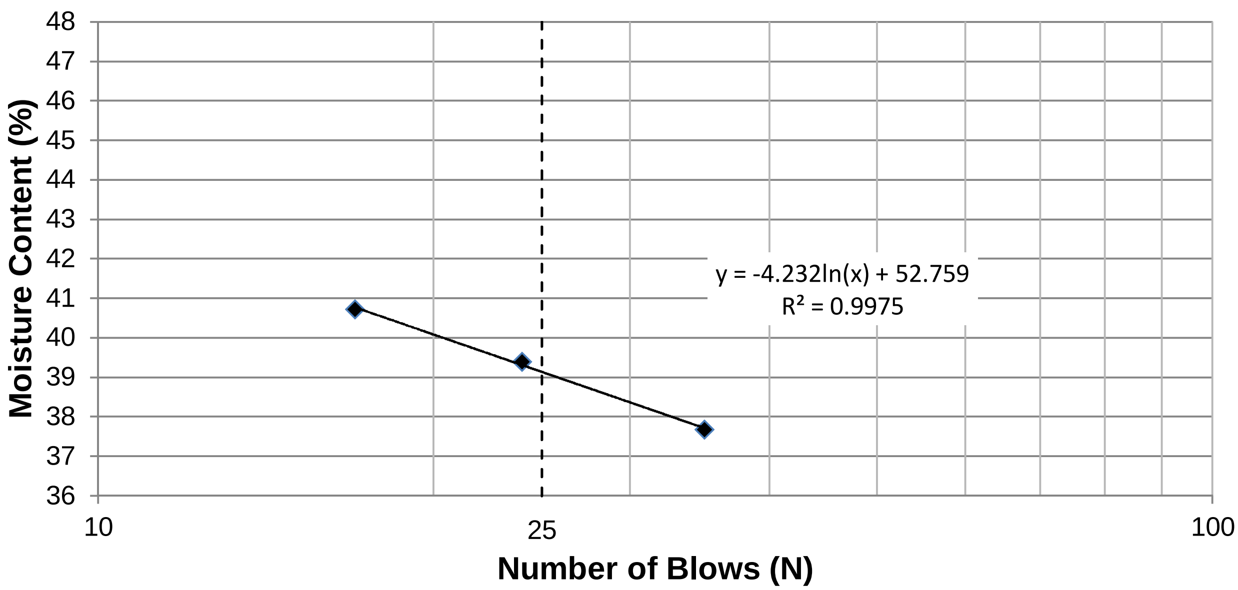
Atterberg Limits
ASTM D4318-10e1

Project No.	0035-037-00
Client	Morrison Hershfield
Project	Empress Widening
Test Hole	RH16-07
Sample #	T169
Depth (m)	9.1-9.8
Sample Date	01-Nov-16
Test Date	22-Dec-16
Technician	JW

Liquid Limit	39
Plastic Limit	13
Plasticity Index	27

Liquid Limit

Trial #	**1**	**2**	**3**	**4**	**5**
Number of Blows (N)	17	35	24		
Mass Wet Soil + Tare (g)	32.383	30.654	33.228		
Mass Dry Soil + Tare (g)	27.146	26.121	27.872		
Mass Tare (g)	14.286	14.088	14.278		
Mass Water (g)	5.237	4.533	5.356		
Mass Dry Soil (g)	12.860	12.033	13.594		
Moisture Content (%)	40.723	37.671	39.400		



Plastic Limit

Trial #	**1**	**2**	**3**	**4**	**5**
Mass Wet Soil + Tare (g)	24.681	24.685			
Mass Dry Soil + Tare (g)	23.441	23.530			
Mass Tare (g)	13.939	13.908			
Mass Water (g)	1.240	1.155			
Mass Dry Soil (g)	9.502	9.622			
Moisture Content (%)	13.050	12.004			

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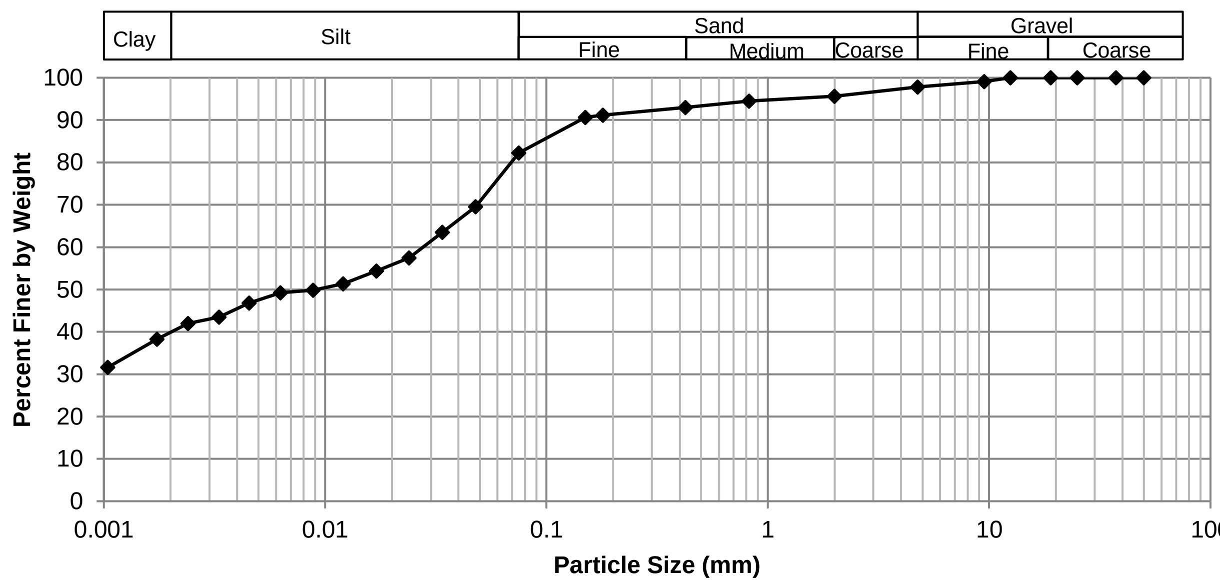
Grain Size Analysis (Hydrometer Method)
ASTM D422

Project No.	0035-037
Client	Morrison Hershfield
Project	Empress Widening
Test Hole	RH16-19
Sample #	G24
Depth (m)	0.6 - 1.1
Sample Date	2-Nov-16
Test Date	28-Nov-16
Technician	JW

Gravel	2.2%
Sand	15.6%
Silt	42.5%
Clay	39.7%

Particle Size Distribution Curve



Gravel		Sand		Silt and Clay	
Particle Size (mm)	**Percent Passing**	**Particle Size (mm)**	**Percent Passing**	**Particle Size (mm)**	**Percent Passing**
50.0	100.00	4.75	97.82	0.0750	82.24
37.5	100.00	2.00	95.64	0.0479	69.59
25.0	100.00	0.825	94.53	0.0338	63.52
19.0	100.00	0.425	92.99	0.0239	57.44
12.5	100.00	0.180	91.13	0.0171	54.41
9.50	99.12	0.150	90.66	0.0121	51.37
4.75	97.82	0.075	82.24	0.0088	49.85
				0.0063	49.24
				0.0045	46.81
				0.0033	43.47
				0.0024	41.95
				0.0017	38.31
				0.0010	31.62

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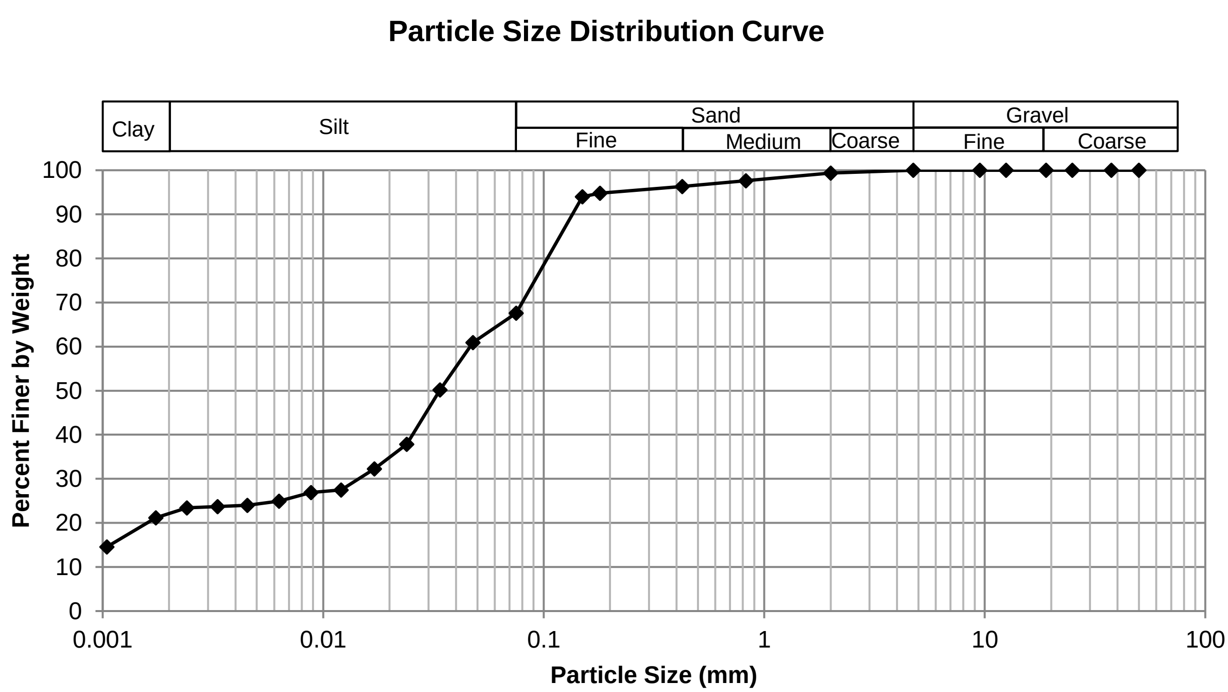
Grain Size Analysis (Hydrometer Method)
ASTM D422

Project No. 0035-037
Client Morrison Hershfield
Project Empress Widening

Test Hole RH16-08
Sample # G53
Depth (m) 1.2 - 1.5
Sample Date 1-Nov-16
Test Date 21-Dec-16
Technician MM

Gravel	0.0%
Sand	15.6%
Silt	42.5%
Clay	39.7%



Gravel		Sand		Silt and Clay	
Particle Size (mm)	**Percent Passing**	**Particle Size (mm)**	**Percent Passing**	**Particle Size (mm)**	**Percent Passing**
50.0	100.00	4.75	100.00	0.0750	67.62
37.5	100.00	2.00	99.40	0.0479	60.89
25.0	100.00	0.825	97.59	0.0338	50.16
19.0	100.00	0.425	96.34	0.0239	37.85
12.5	100.00	0.180	94.77	0.0171	32.23
9.50	100.00	0.150	94.03	0.0121	27.50
4.75	100.00	0.075	67.62	0.0088	26.87
				0.0063	24.97
				0.0045	24.03
				0.0033	23.71
				0.0024	23.40
				0.0017	21.19
				0.0010	14.56

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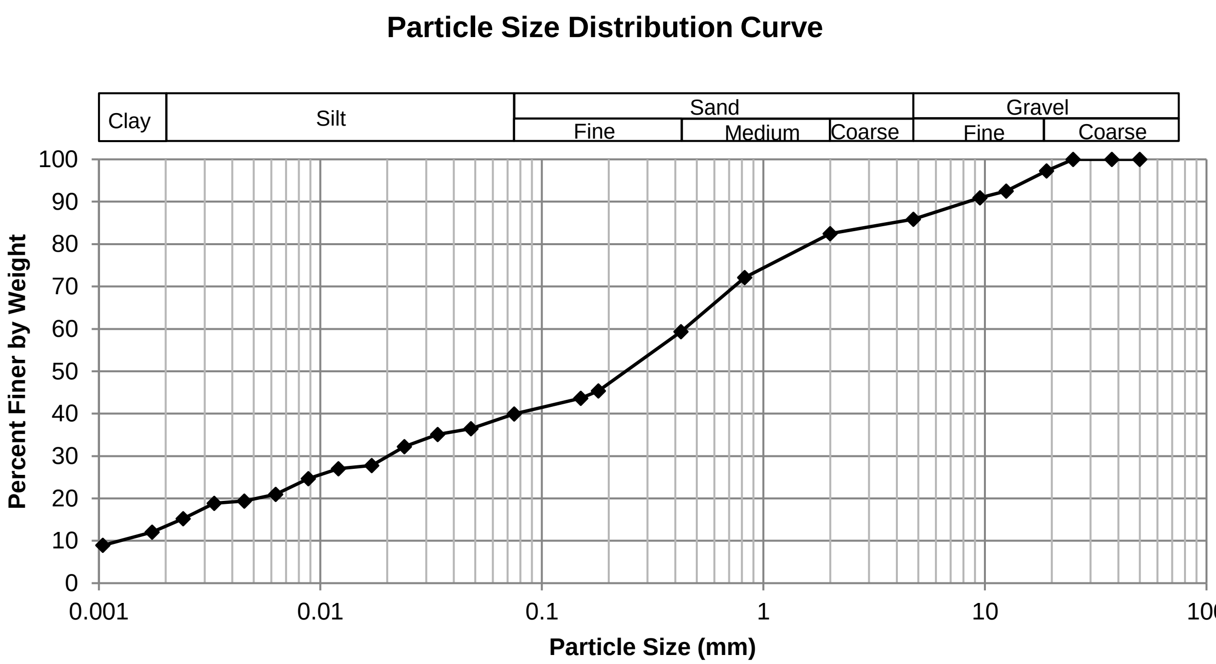
Grain Size Analysis (Hydrometer Method)
ASTM D422

Project No.	0035-037
Client	Morrison Hershfield
Project	Empress Widening
Test Hole	RH16-003
Sample #	G75
Depth (m)	0.6 - 0.9
Sample Date	2-Nov-16
Test Date	28-Nov-16
Technician	JW

Gravel	14.1%
Sand	45.9%
Silt	26.6%
Clay	13.3%



Gravel		Sand		Silt and Clay	
Particle Size (mm)	**Percent Passing**	**Particle Size (mm)**	**Percent Passing**	**Particle Size (mm)**	**Percent Passing**
50.0	100.00	4.75	85.86	0.0750	39.94
37.5	100.00	2.00	82.46	0.0479	36.43
25.0	100.00	0.825	72.10	0.0338	35.12
19.0	97.30	0.425	59.33	0.0239	32.24
12.5	92.53	0.180	45.36	0.0171	27.79
9.50	90.93	0.150	43.64	0.0121	27.00
4.75	85.86	0.075	39.94	0.0088	24.65
				0.0063	20.98
				0.0045	19.41
				0.0033	18.89
				0.0024	15.22
				0.0017	12.08
				0.0010	8.93

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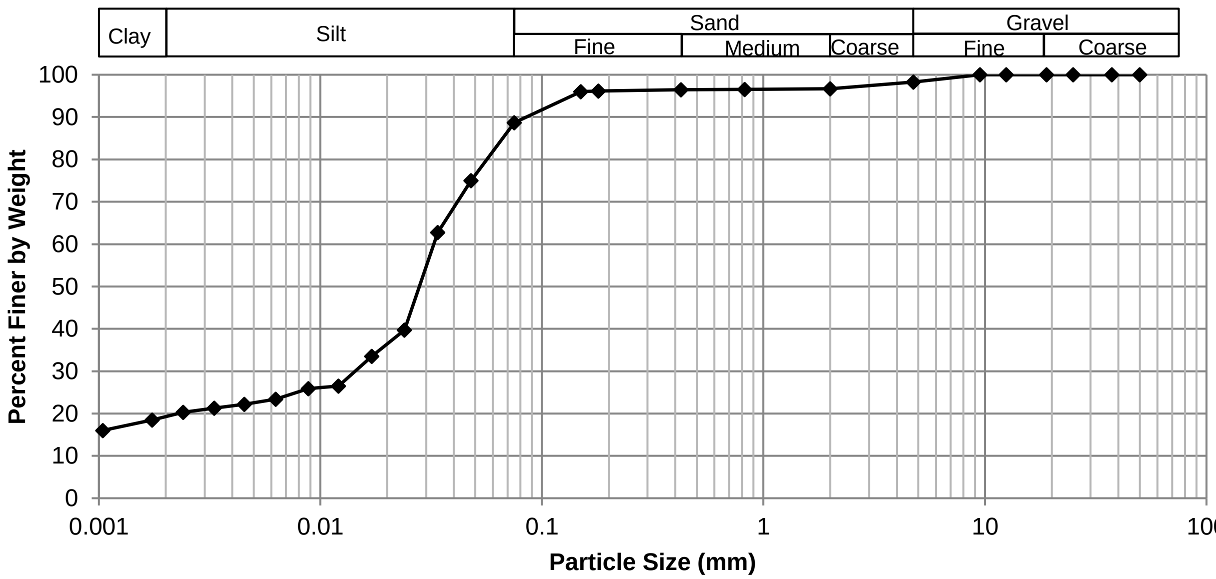
Grain Size Analysis (Hydrometer Method)
ASTM D422

Project No.	0035-037-00
Client	Morrison Hershfield
Project	Empress Widening
Test Hole	RH16-04
Sample #	G106
Depth (m)	0.8 - 1.1
Sample Date	2-Nov-16
Test Date	28-Nov-16
Technician	JW

Gravel	1.7%
Sand	9.6%
Silt	69.5%
Clay	19.2%

Particle Size Distribution Curve



Gravel		Sand		Silt and Clay	
Particle Size (mm)	**Percent Passing**	**Particle Size (mm)**	**Percent Passing**	**Particle Size (mm)**	**Percent Passing**
50.0	100.00	4.75	98.26	0.0750	88.65
37.5	100.00	2.00	96.66	0.0479	75.01
25.0	100.00	0.825	96.57	0.0338	62.72
19.0	100.00	0.425	96.45	0.0239	39.68
12.5	100.00	0.180	96.17	0.0171	33.53
9.50	100.00	0.150	96.03	0.0121	26.46
4.75	98.26	0.075	88.65	0.0088	25.85
				0.0063	23.39
				0.0045	22.16
				0.0033	21.24
				0.0024	20.32
				0.0017	18.47
				0.0010	16.01

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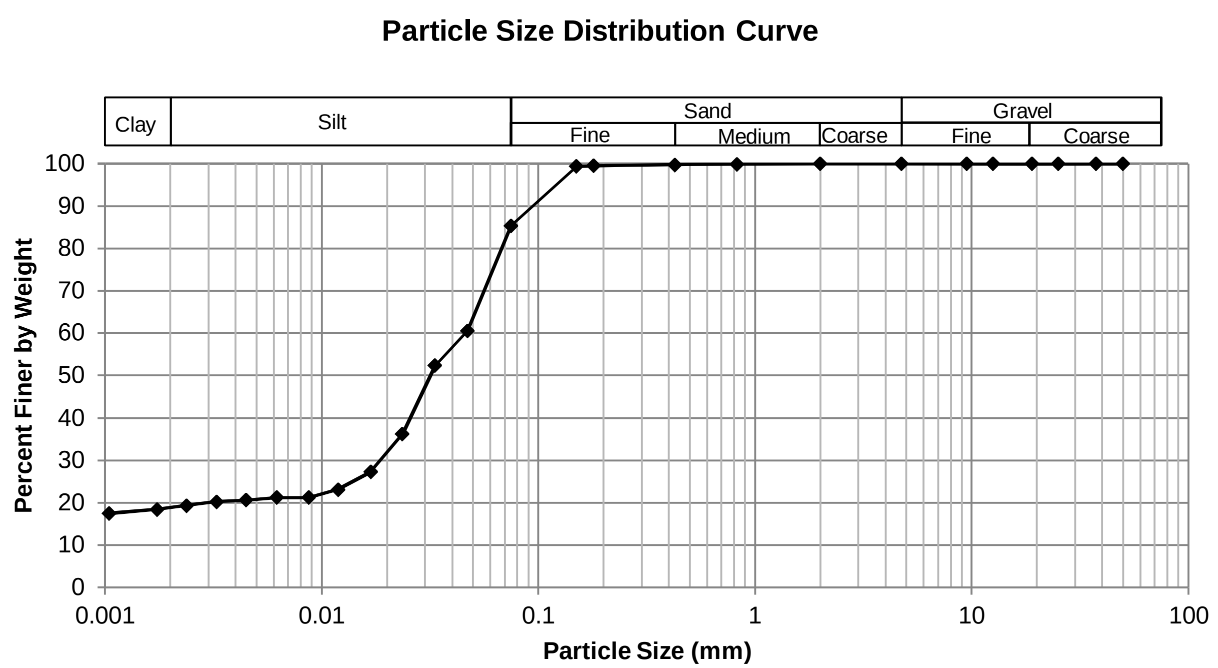
Grain Size Analysis (Hydrometer Method)
ASTM D422

Project No.	0035-037-00
Client	Morrison Hershfield
Project	Empress Widening
Test Hole	RH16-05
Sample #	G122
Depth (m)	0.6 - 1.1
Sample Date	1-Nov-16
Test Date	8-Dec-16
Technician	MM

Gravel	0.0%
Sand	14.6%
Silt	66.5%
Clay	18.8%



Gravel		Sand		Silt and Clay	
Particle Size (mm)	**Percent Passing**	**Particle Size (mm)**	**Percent Passing**	**Particle Size (mm)**	**Percent Passing**
50.0	100.00	4.75	100.00	0.0750	85.36
37.5	100.00	2.00	100.00	0.0471	60.62
25.0	100.00	0.825	99.87	0.0333	52.37
19.0	100.00	0.425	99.75	0.0236	36.17
12.5	100.00	0.180	99.51	0.0168	27.28
9.50	100.00	0.150	99.39	0.0119	23.15
4.75	100.00	0.075	85.36	0.0087	21.24
				0.0062	21.24
				0.0045	20.61
				0.0033	20.29
				0.0024	19.34
				0.0017	18.46
				0.0010	17.50

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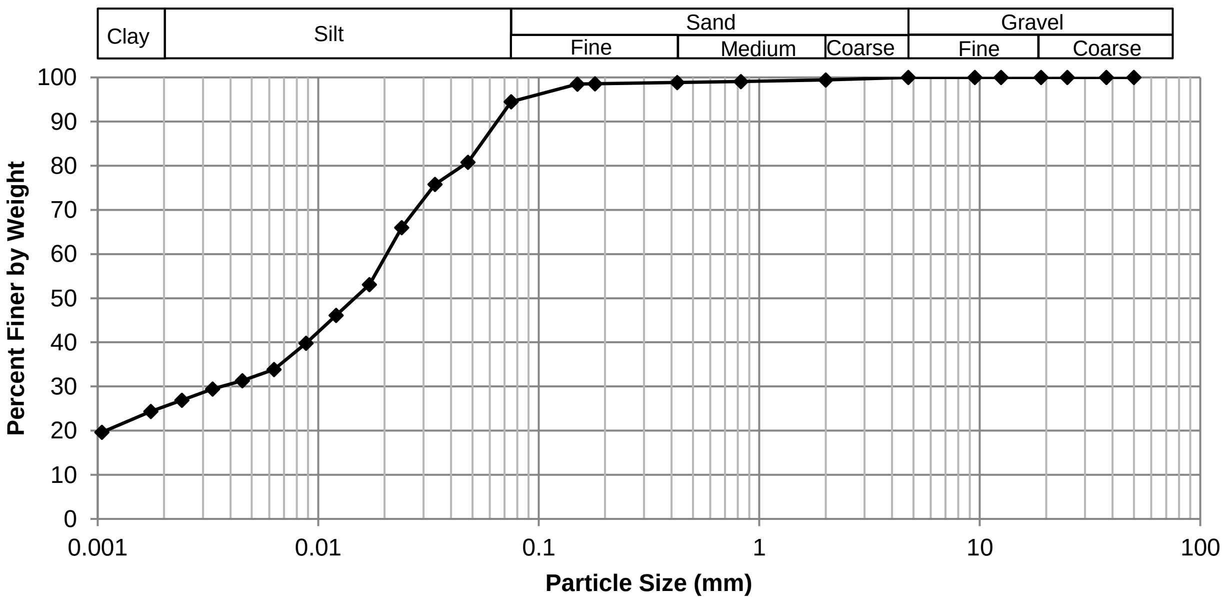
Grain Size Analysis (Hydrometer Method)
ASTM D422

Project No.	0035-037
Client	Morrison Hershfield
Project	Empress Widening

Test Hole	TH16-06
Sample #	G140
Depth (m)	0.9 - 1.2
Sample Date	1-Nov-16
Test Date	21-Dec-16
Technician	MM

Gravel	0.0%
Sand	15.6%
Silt	42.5%
Clay	39.7%

Particle Size Distribution Curve



Gravel		Sand		Silt and Clay	
Particle Size (mm)	**Percent Passing**	**Particle Size (mm)**	**Percent Passing**	**Particle Size (mm)**	**Percent Passing**
50.0	100.00	4.75	100.00	0.0750	94.49
37.5	100.00	2.00	99.46	0.0479	80.83
25.0	100.00	0.825	99.11	0.0338	75.77
19.0	100.00	0.425	98.86	0.0239	65.98
12.5	100.00	0.180	98.57	0.0171	53.10
9.50	100.00	0.150	98.49	0.0121	46.15
4.75	100.00	0.075	94.49	0.0088	39.83
				0.0063	33.83
				0.0045	31.31
				0.0033	29.41
				0.0024	26.88
				0.0017	24.36
				0.0010	19.62

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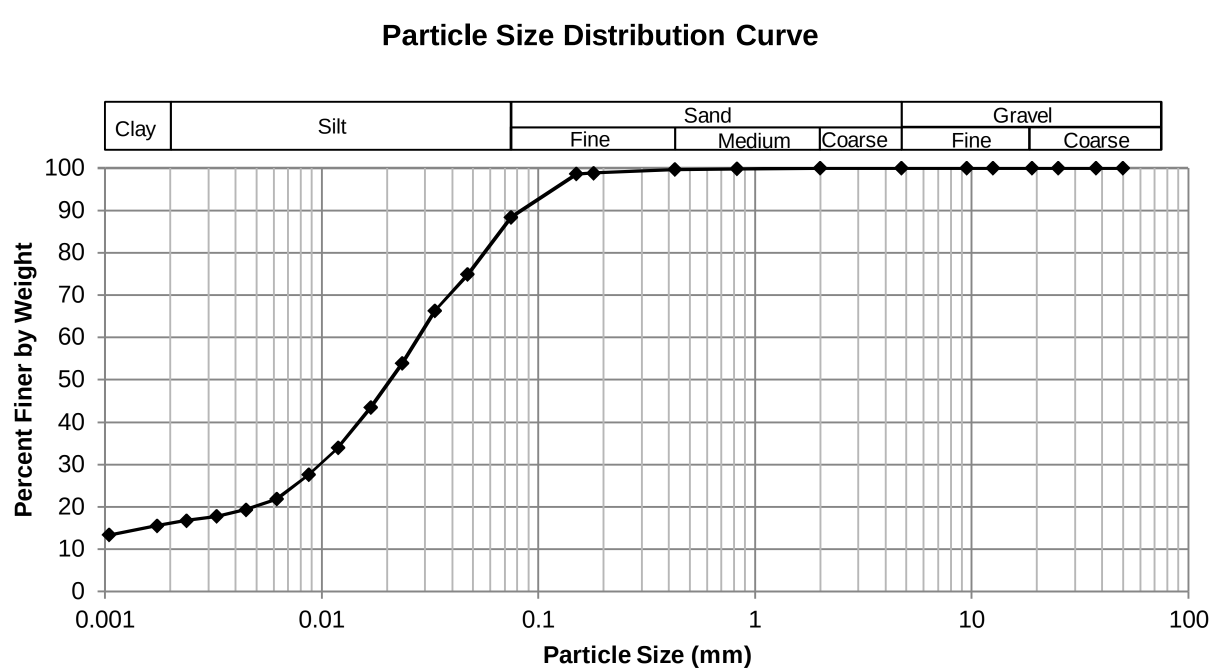
Grain Size Analysis (Hydrometer Method)
ASTM D422

Project No.	0035-037-00
Client	Morrison Hershfield
Project	Empress Widening
Test Hole	RH16-06
Sample #	G144
Depth (m)	2.4 - 2.7
Sample Date	1-Nov-16
Test Date	8-Dec-16
Technician	MM

Gravel	0.0%
Sand	11.6%
Silt	72.3%
Clay	16.1%



Gravel		Sand		Silt and Clay	
Particle Size (mm)	**Percent Passing**	**Particle Size (mm)**	**Percent Passing**	**Particle Size (mm)**	**Percent Passing**
50.0	100.00	4.75	100.00	0.0750	88.40
37.5	100.00	2.00	100.00	0.0471	74.91
25.0	100.00	0.825	99.79	0.0333	66.34
19.0	100.00	0.425	99.64	0.0236	53.95
12.5	100.00	0.180	98.87	0.0168	43.47
9.50	100.00	0.150	98.59	0.0119	33.95
4.75	100.00	0.075	88.40	0.0087	27.60
				0.0062	21.88
				0.0045	19.34
				0.0033	17.75
				0.0024	16.80
				0.0017	15.60
				0.0010	13.38

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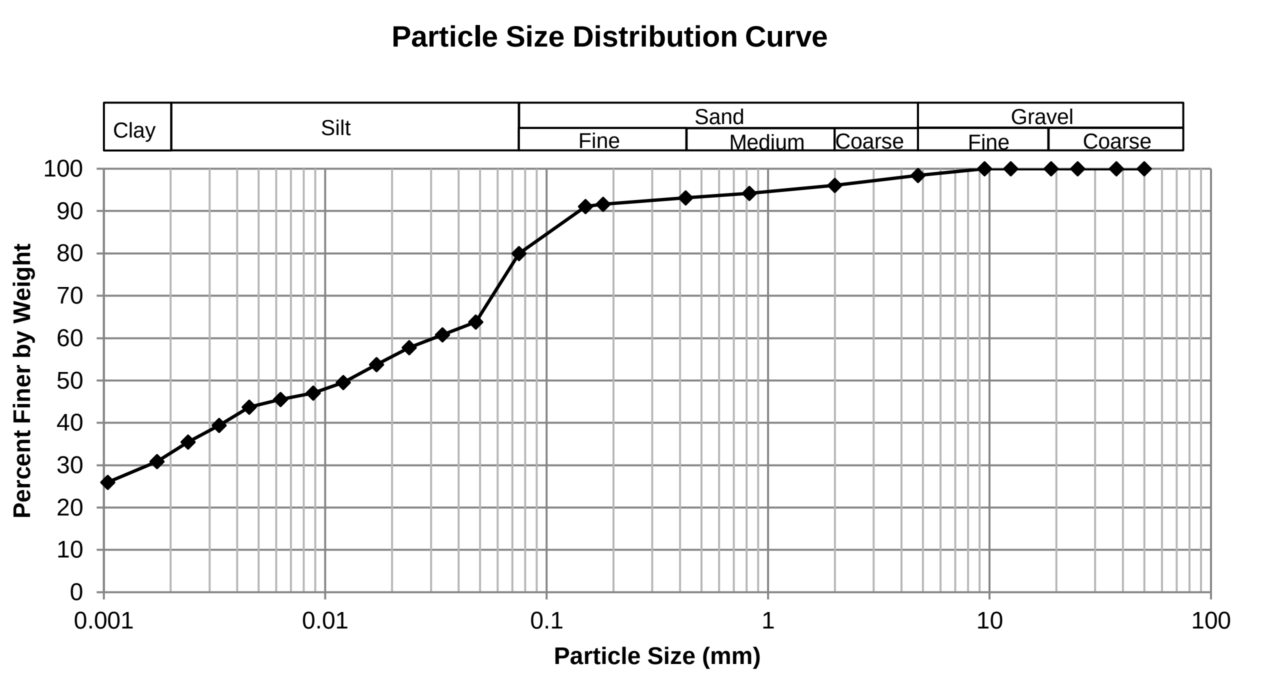
Grain Size Analysis (Hydrometer Method)
ASTM D422

Project No.	0035-037
Client	Morrison Hershfield
Project	Empress Widening

Test Hole	RH16-07
Sample #	G157
Depth (m)	0.3 - 0.6
Sample Date	2-Nov-16
Test Date	28-Nov-16
Technician	JW

Gravel	1.6%
Sand	18.4%
Silt	47.4%
Clay	32.6%



Gravel		Sand		Silt and Clay	
Particle Size (mm)	**Percent Passing**	**Particle Size (mm)**	**Percent Passing**	**Particle Size (mm)**	**Percent Passing**
50.0	100.00	4.75	98.45	0.0750	80.00
37.5	100.00	2.00	96.09	0.0479	63.85
25.0	100.00	0.825	94.20	0.0338	60.80
19.0	100.00	0.425	93.13	0.0239	57.74
12.5	100.00	0.180	91.59	0.0171	53.77
9.50	100.00	0.150	91.08	0.0121	49.50
4.75	98.45	0.075	80.00	0.0088	47.06
				0.0063	45.53
				0.0045	43.70
				0.0033	39.42
				0.0024	35.45
				0.0017	30.87
				0.0010	25.99

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Shelby Tube Visual

Project No.	0035-037-00
Client	Morrison Hershfield
Project	Empress Widening
Test Hole	RH16-03
Sample #	T81
Depth (m)	3.0 - 3.5
Sample Date	02-Nov-16
Test Date	01-Dec-16
Technician	MM

Tube Extraction

Recovery (mm) 465

Bottom - 3.5 m | **Top - 3 m**

PP Tv	Qu Bulk	Keep	Moisture Content Visual
70 mm	160 mm	160 mm	75 mm

Visual Classification

Material	CLAY
Composition	silty
trace silt inclusions (<10mmø)	
trace organics	
Color	motled grey/brown
Moisture	moist
Consistency	firm
Plasticity	high plasticity
Structure	homogeneous
Gradation	

Torvane

Reading	0.35
Vane Size (s,m,l)	m
Undrained Shear Strength (kPa)	34.3

Pocket Penetrometer

Reading	**1**	1.50
	2	1.50
	3	1.55
	Average	1.52
Undrained Shear Strength (kPa)		74.4

Moisture Content

Tare ID	AB61
Mass tare (g)	6.5
Mass wet + tare (g)	419.2
Mass dry + tare (g)	280.7
Moisture %	50.5%

Unit Weight

Bulk Weight (g)		1010.1
Length (mm)	**1**	143.76
	2	143.50
	3	143.20
	4	143.84
Average Length (m)		0.144
Diam. (mm)	**1**	72.74
	2	72.61
	3	72.26
	4	72.46
Average Diameter (m)		0.073
Volume (m^3)		5.93E-04
Bulk Unit Weight (kN/m^3)		16.7
Bulk Unit Weight (pcf)		106.3
Dry Unit Weight (kN/m^3)		11.1
Dry Unit Weight (pcf)		70.7

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Unconfined Compressive Strength
ASTM D2166

Project No.	0035-037-00
Client	Morrison Hershfield
Project	Empress Widening
Test Hole	RH16-03
Sample #	T81
Depth (m)	3.0 - 3.5
Sample Date	2-Nov-16
Test Date	1-Dec-16
Technician	MM

Unconfined Strength

	kPa	ksf
Max q_u	96.8	2.0
Max S_u	48.4	1.0

Specimen Data

Description CLAY - silty, trace silt inclusions (<10mmø), trace organics, motled grey/brown, moist, firm, high plasticity, homogeneous,

Length	143.6	(mm)	**Moisture %**	51%	
Diameter	72.5	(mm)	**Bulk Unit Wt.**	16.7	(kN/m^3)
L/D Ratio	2.0		**Dry Unit Wt.**	11.1	(kN/m^3)
Initial Area	0.00413	(m^2)	**Liquid Limit**	-	
Load Rate	1.00	(%/min)	**Plastic Limit**	-	
			Plasticity Index	-	

Undrained Shear Strength Tests

Torvane

Reading	**Undrained Shear Strength**	
tsf	**kPa**	**ksf**
0.35	34.3	0.72
Vane Size		
m		

Pocket Penetrometer

	Reading	**Undrained Shear Strength**	
	tsf	**kPa**	**ksf**
	1.50	73.6	1.54
	1.50	73.6	1.54
	1.55	76.0	1.59
Average	**1.52**	**74.4**	**1.55**

Failure Geometry

Sketch:

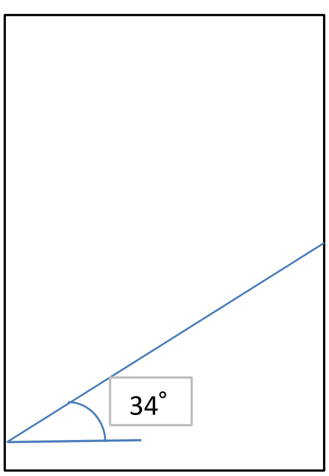

Photo:

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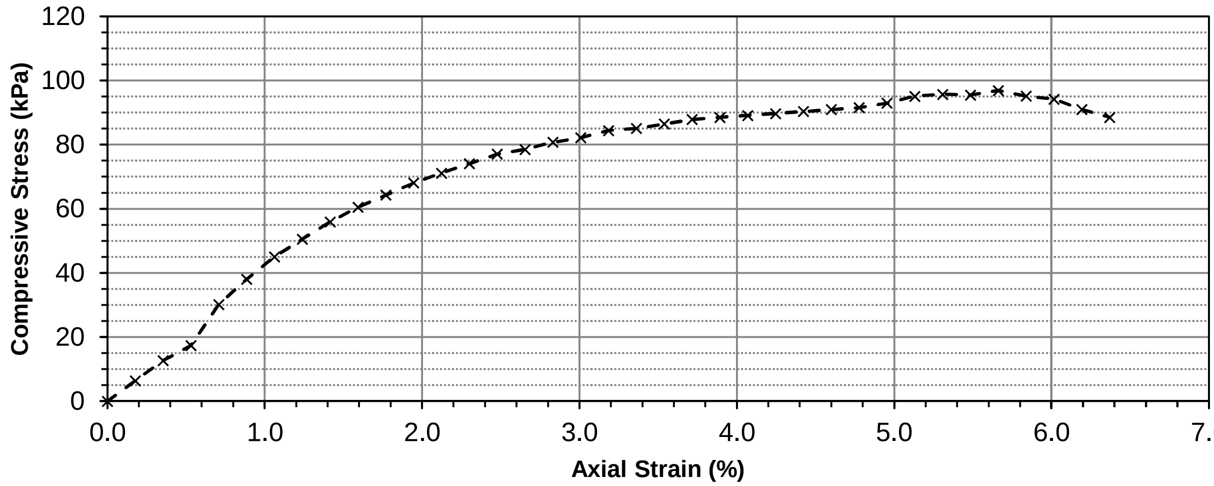
Unconfined Compressive Strength
ASTM D2166

Project No. 0035-037-00
Client Morrison Hershfield
Project Empress Widening

Unconfined Compression Test Graph



Unconfined Compression Test Data

Deformation Dial Reading	Load Ring Dial Reading	Deflection (mm)	Axial Strain (%)	Corrected Area (m^2)	Axial Load (N)	Compressive Stress, q_u (kPa)	Shear Stress, S_u (kPa)
0	0	0.0000	0.00	0.004130	0.0	0.00	0.00
10	8	0.2540	0.18	0.004138	26.2	6.32	3.16
20	16	0.5080	0.35	0.004145	52.4	12.63	6.32
30	22	0.7620	0.53	0.004152	72.1	17.35	8.68
40	38	1.0160	0.71	0.004160	125.3	30.12	15.06
50	48	1.2700	0.88	0.004167	158.3	37.98	18.99
60	57	1.5240	1.06	0.004175	187.9	45.02	22.51
70	64	1.7780	1.24	0.004182	211.0	50.46	25.23
80	71	2.0320	1.42	0.004190	234.1	55.88	27.94
90	77	2.2860	1.59	0.004197	253.9	60.49	30.24
100	82	2.5400	1.77	0.004205	270.4	64.30	32.15
110	87	2.7940	1.95	0.004212	286.8	68.09	34.05
120	91	3.0480	2.12	0.004220	300.0	71.10	35.55
130	95	3.3020	2.30	0.004227	313.2	74.09	37.04
140	99	3.5560	2.48	0.004235	326.4	77.07	38.54
150	101	3.8100	2.65	0.004243	333.1	78.50	39.25
160	104	4.0640	2.83	0.004251	343.2	80.74	40.37
170	106	4.3180	3.01	0.004258	349.9	82.17	41.08
180	109	4.5720	3.18	0.004266	360.0	84.39	42.19
190	110	4.8260	3.36	0.004274	363.4	85.02	42.51
200	112	5.0800	3.54	0.004282	370.1	86.45	43.22
210	114	5.3340	3.72	0.004290	376.9	87.85	43.93
220	115	5.5880	3.89	0.004298	380.2	88.48	44.24
230	116	5.8420	4.07	0.004305	383.6	89.09	44.54

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Unconfined Compressive Strength
ASTM D2166

Project No. 0035-037-00
Client Morrison Hershfield
Project Empress Widening

Unconfined Compression Test Data (cont'd)

Deformation Dial Reading	Load Ring Dial Reading	Deflection (mm)	Axial Strain (%)	Corrected Area (m^2)	Axial Load (N)	Compressive Stress, q_u (kPa)	Shear Stress, S_u (kPa)
240	117	6.0960	4.2459	0.004313	387.0	89.71	44.85
250	118	6.3500	4.42	0.004321	390.3	90.33	45.16
260	119	6.6040	4.60	0.004329	393.7	90.93	45.46
270	120	6.8580	4.78	0.004337	397.0	91.54	45.77
280	122	7.1120	4.95	0.004345	403.8	92.93	46.46
290	125	7.3660	5.13	0.004354	413.9	95.07	47.54
300	126	7.6200	5.31	0.004362	417.2	95.66	47.83
310	126	7.8740	5.48	0.004370	417.2	95.48	47.74
320	128	8.1280	5.66	0.004378	424.0	96.85	48.42
330	126	8.3820	5.84	0.004386	417.2	95.12	47.56
340	125	8.6360	6.01	0.004395	413.9	94.19	47.09
350	121	8.8900	6.19	0.004403	400.4	90.95	45.47
360	118	9.1440	6.37	0.004411	390.3	88.49	44.24

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Shelby Tube Visual

Project No.	0035-037-00
Client	Morrison Hershfield
Project	Empress Widening

Test Hole	RH16-03
Sample #	T83
Depth (m)	4.6 - 5.1
Sample Date	02-Nov-16
Test Date	01-Dec-16
Technician	MM

Tube Extraction

Recovery (mm) 575

Bottom - 5.1 m / **Top - 4.6 m**

Moisture Content Visual	Qu Bulk	Keep	PP Tv
125 mm	160 mm	160 mm	130 mm

Visual Classification

Material	CLAY
Composition	silty
trace silt inclusions (<25mmø)	
trace gravel (<15mmø)	
Color	brown
Moisture	moist
Consistency	soft to firm
Plasticity	high plasticity
Structure	homogeneous
Gradation	

Torvane

Reading	0.43
Vane Size (s,m,l)	m
Undrained Shear Strength (kPa)	41.7

Pocket Penetrometer

Reading	**1**	1.25
	2	1.40
	3	1.25
	Average	1.30
Undrained Shear Strength (kPa)		63.7

Moisture Content

Tare ID	W07
Mass tare (g)	8.7
Mass wet + tare (g)	410.4
Mass dry + tare (g)	273.6
Moisture %	51.6%

Unit Weight

Bulk Weight (g)		996.1
Length (mm)	**1**	143.49
	2	143.84
	3	143.45
	4	143.43
Average Length (m)		0.144
Diam. (mm)	**1**	71.63
	2	71.56
	3	72.08
	4	72.02
Average Diameter (m)		0.072
Volume (m^3)		5.82E-04
Bulk Unit Weight (kN/m^3)		16.8
Bulk Unit Weight (pcf)		106.9
Dry Unit Weight (kN/m^3)		11.1
Dry Unit Weight (pcf)		70.5

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Unconfined Compressive Strength

ASTM D2166

Project No.	0035-037-00
Client	Morrison Hershfield
Project	Empress Widening

Test Hole	RH16-03
Sample #	T83
Depth (m)	4.6 - 5.1
Sample Date	2-Nov-16
Test Date	1-Dec-16
Technician	MM

Unconfined Strength

	kPa	ksf
Max q_u	46.0	1.0
Max S_u	23.0	0.5

Specimen Data

Description CLAY - silty, trace silt inclusions (<25mmø), trace gravel (<15mmø), brown, moist, soft to firm, high plasticity, homogeneous,

Length	143.6	(mm)	**Moisture %**	52%	
Diameter	71.8	(mm)	**Bulk Unit Wt.**	16.8	(kN/m^3)
L/D Ratio	2.0		**Dry Unit Wt.**	11.1	(kN/m^3)
Initial Area	0.00405	(m^2)	**Liquid Limit**	-	
Load Rate	1.00	(%/min)	**Plastic Limit**	-	
			Plasticity Index	-	

Undrained Shear Strength Tests

Torvane

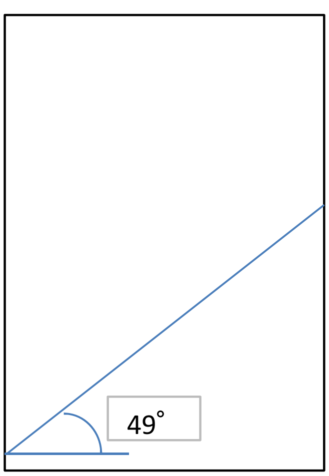
Reading	**Undrained Shear Strength**	
tsf	**kPa**	**ksf**
0.43	41.7	0.87
Vane Size		
m		

Pocket Penetrometer

	Reading	**Undrained Shear Strength**	
	tsf	**kPa**	**ksf**
	1.25	61.3	1.28
	1.40	68.7	1.43
	1.25	61.3	1.28
Average	**1.30**	**63.8**	**1.33**

Failure Geometry

Sketch:



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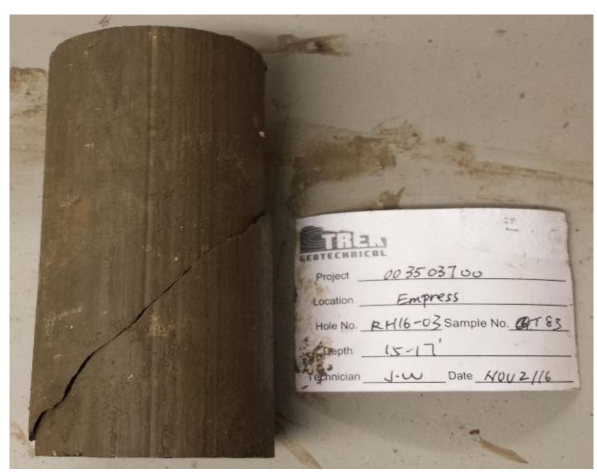

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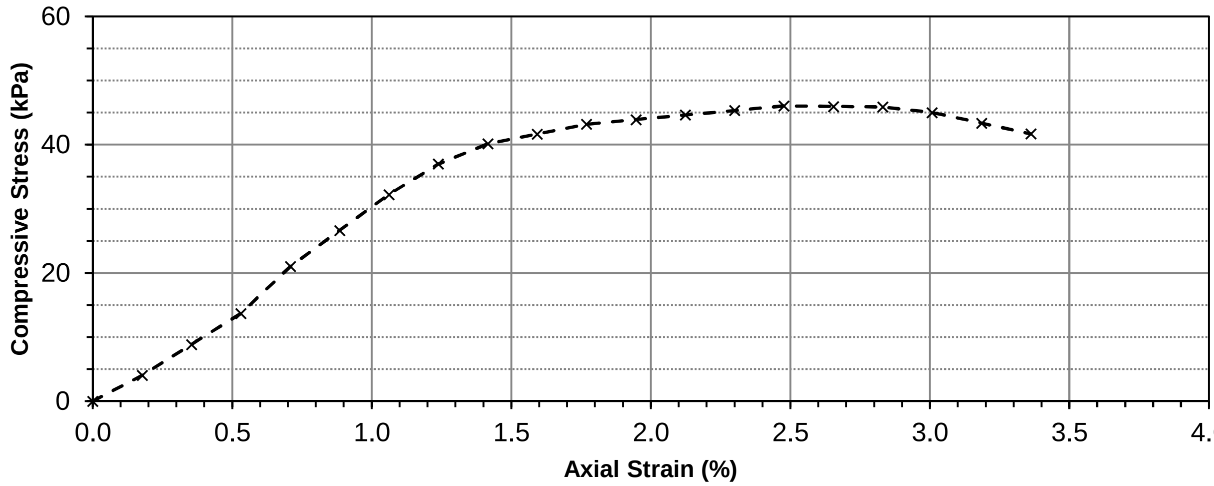
Unconfined Compressive Strength
ASTM D2166

Project No. 0035-037-00
Client Morrison Hershfield
Project Empress Widening

Unconfined Compression Test Graph



Unconfined Compression Test Data

Deformation Dial Reading	Load Ring Dial Reading	Deflection (mm)	Axial Strain (%)	Corrected Area (m^2)	Axial Load (N)	Compressive Stress, q_u (kPa)	Shear Stress, S_u (kPa)
0	0	0.0000	0.00	0.004051	0.0	0.00	0.00
10	5	0.2540	0.18	0.004059	16.3	4.03	2.01
20	11	0.5080	0.35	0.004066	36.0	8.85	4.43
30	17	0.7620	0.53	0.004073	55.7	13.66	6.83
40	26	1.0160	0.71	0.004080	85.7	21.01	10.50
50	33	1.2700	0.88	0.004088	108.8	26.62	13.31
60	40	1.5240	1.06	0.004095	131.9	32.21	16.10
70	46	1.7780	1.24	0.004102	151.7	36.98	18.49
80	50	2.0320	1.42	0.004110	164.9	40.11	20.06
90	52	2.2860	1.59	0.004117	171.4	41.64	20.82
100	54	2.5400	1.77	0.004124	178.0	43.16	21.58
110	55	2.7940	1.95	0.004132	181.4	43.89	21.95
120	56	3.0480	2.12	0.004139	184.6	44.61	22.30
130	57	3.3020	2.30	0.004147	187.9	45.32	22.66
140	58	3.5560	2.48	0.004154	191.2	46.03	23.02
150	58	3.8100	2.65	0.004162	191.2	45.95	22.97
160	58	4.0640	2.83	0.004169	191.2	45.86	22.93
170	57	4.3180	3.01	0.004177	187.9	44.99	22.50
180	55	4.5720	3.18	0.004185	181.4	43.34	21.67
190	53	4.8260	3.36	0.004192	174.7	41.68	20.84

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Shelby Tube Visual

Project No.	0035-037-00
Client	Morrison Hershfield
Project	Empress Widening
Test Hole	RH16-03
Sample #	T85
Depth (m)	6.1 - 6.7
Sample Date	02-Nov-16
Test Date	01-Dec-16
Technician	MM

Tube Extraction

Recovery (mm) 555

Bottom - 6.7 m | **Top - 6.1 m**

PP Tv	Qu Bulk	Keep	Moisture Content Visual
115 mm	160 mm	160 mm	120 mm

Visual Classification

Material	CLAY
Composition	silty
trace silt inclusions (<25mmø)	
trace sand	
trace organics	
Color	brown
Moisture	moist
Consistency	soft to firm
Plasticity	high plasticity
Structure	homogeneous
Gradation	

Torvane

Reading	0.30
Vane Size (s,m,l)	m
Undrained Shear Strength (kPa)	29.4

Pocket Penetrometer

Reading	**1**	0.70
	2	0.70
	3	0.70
	Average	0.70
Undrained Shear Strength (kPa)		34.3

Moisture Content

Tare ID	AB31
Mass tare (g)	7.2
Mass wet + tare (g)	394.8
Mass dry + tare (g)	258.1
Moisture %	54.5%

Unit Weight

Bulk Weight (g)		1025.4
Length (mm)	**1**	145.40
	2	145.44
	3	145.67
	4	145.47
Average Length (m)		0.145
Diam. (mm)	**1**	72.01
	2	71.74
	3	71.63
	4	72.40
Average Diameter (m)		0.072
Volume (m^3)		5.91E-04
Bulk Unit Weight (kN/m^3)		17.0
Bulk Unit Weight (pcf)		108.2
Dry Unit Weight (kN/m^3)		11.0
Dry Unit Weight (pcf)		70.1

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Unconfined Compressive Strength
ASTM D2166

Project No.	0035-037-00
Client	Morrison Hershfield
Project	Empress Widening
Test Hole	RH16-03
Sample #	T85
Depth (m)	6.1 - 6.7
Sample Date	2-Nov-16
Test Date	1-Dec-16
Technician	MM

Unconfined Strength

	kPa	ksf
Max q_u	68.5	1.4
Max S_u	34.2	0.7

Specimen Data

Description CLAY - silty, trace silt inclusions (<25mmø), trace sand, trace organics, brown, moist, soft to firm, high plasticity, homogeneous,

Length	145.5	(mm)	**Moisture %**	54%	
Diameter	71.9	(mm)	**Bulk Unit Wt.**	17.0	(kN/m^3)
L/D Ratio	2.0		**Dry Unit Wt.**	11.0	(kN/m^3)
Initial Area	0.00407	(m^2)	**Liquid Limit**	-	
Load Rate	1.00	(%/min)	**Plastic Limit**	-	
			Plasticity Index	-	

Undrained Shear Strength Tests

Torvane

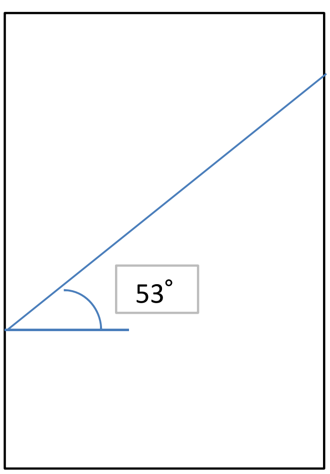
Reading	Undrained Shear Strength	
tsf	kPa	ksf
0.30	29.4	0.61
Vane Size		
m		

Pocket Penetrometer

	Reading	Undrained Shear Strength	
	tsf	kPa	ksf
	0.70	34.3	0.72
	0.70	34.3	0.72
	0.70	34.3	0.72
Average	**0.70**	**34.3**	**0.72**

Failure Geometry

Sketch:



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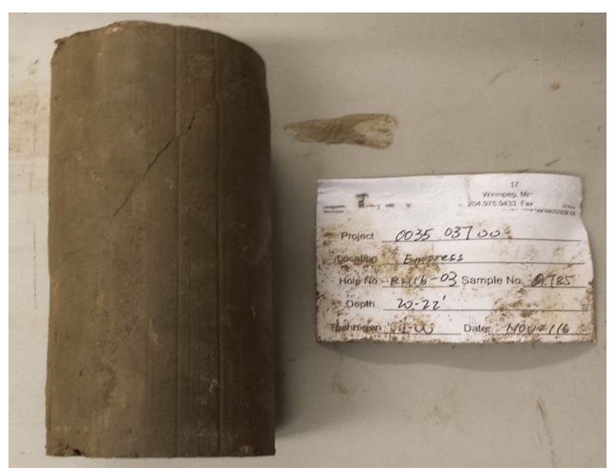

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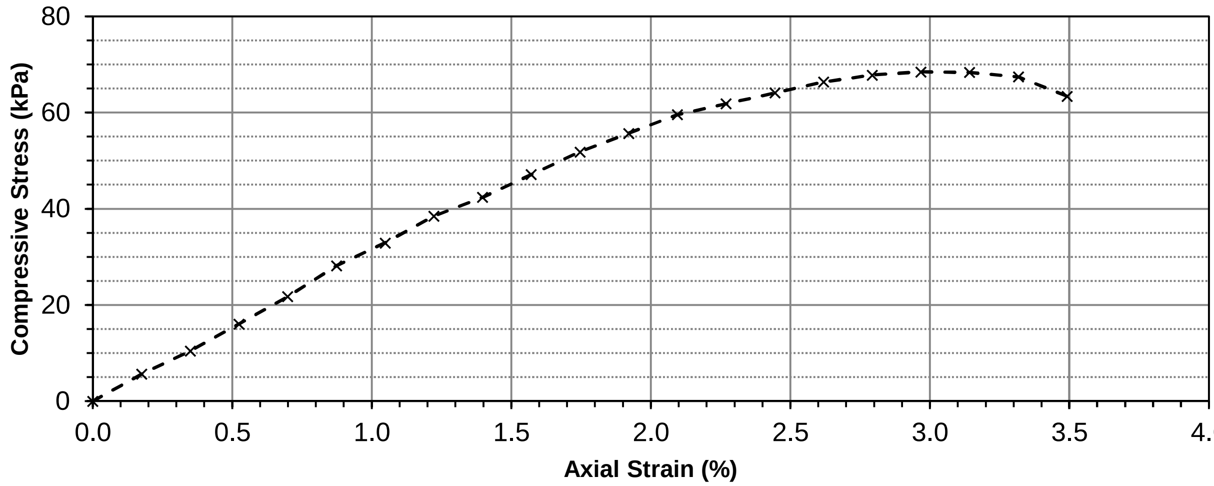
Unconfined Compressive Strength
ASTM D2166

Project No. 0035-037-00
Client Morrison Hershfield
Project Empress Widening

Unconfined Compression Test Graph



Unconfined Compression Test Data

Deformation Dial Reading	Load Ring Dial Reading	Deflection (mm)	Axial Strain (%)	Corrected Area (m^2)	Axial Load (N)	Compressive Stress, q_u (kPa)	Shear Stress, S_u (kPa)
0	0	0.0000	0.00	0.004065	0.0	0.00	0.00
10	7	0.2540	0.17	0.004072	22.9	5.62	2.81
20	13	0.5080	0.35	0.004080	42.5	10.43	5.21
30	20	0.7620	0.52	0.004087	65.5	16.03	8.01
40	27	1.0160	0.70	0.004094	89.0	21.74	10.87
50	35	1.2700	0.87	0.004101	115.4	28.14	14.07
60	41	1.5240	1.05	0.004108	135.2	32.90	16.45
70	48	1.7780	1.22	0.004116	158.3	38.46	19.23
80	53	2.0320	1.40	0.004123	174.7	42.38	21.19
90	59	2.2860	1.57	0.004130	194.5	47.10	23.55
100	65	2.5400	1.75	0.004138	214.3	51.80	25.90
110	70	2.7940	1.92	0.004145	230.8	55.68	27.84
120	75	3.0480	2.09	0.004152	247.3	59.55	29.78
130	78	3.3020	2.27	0.004160	257.2	61.82	30.91
140	81	3.5560	2.44	0.004167	267.1	64.09	32.04
150	84	3.8100	2.62	0.004175	276.9	66.34	33.17
160	86	4.0640	2.79	0.004182	283.5	67.80	33.90
170	87	4.3180	2.97	0.004190	286.8	68.46	34.23
180	87	4.5720	3.14	0.004197	286.8	68.34	34.17
190	86	4.8260	3.32	0.004205	283.5	67.43	33.72
200	81	5.0800	3.49	0.004212	267.1	63.40	31.70

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Shelby Tube Visual

Project No.	0035-037-00
Client	Morrison Hershfield
Project	Empress Widening
Test Hole	RH16-03
Sample #	T87
Depth (m)	7.6 - 8.1
Sample Date	02-Nov-16
Test Date	01-Dec-16
Technician	MM

Tube Extraction

Recovery (mm) 515

Bottom - 8.1 m | **Top - 7.6 m**

PP Tv	Qu Bulk	Keep	Moisture Content Visual
70 mm	160 mm	160 mm	125 mm

Visual Classification

Material	CLAY
Composition	silty
trace silt inclusions (<20mmø)	
trace gravel (<25mmø)	
Color	motled grey/brown
Moisture	moist
Consistency	soft to firm
Plasticity	high plasticity
Structure	homogeneous
Gradation	

Torvane

Reading	0.25
Vane Size (s,m,l)	m
Undrained Shear Strength (kPa)	24.5

Pocket Penetrometer

Reading	**1**	0.60
	2	0.60
	3	0.65
	Average	0.62
Undrained Shear Strength (kPa)		30.2

Moisture Content

Tare ID	AB73
Mass tare (g)	6.6
Mass wet + tare (g)	561.9
Mass dry + tare (g)	386.9
Moisture %	46.0%

Unit Weight

Bulk Weight (g)		1079.8
Length (mm)	**1**	144.36
	2	144.69
	3	144.75
	4	144.51
Average Length (m)		0.145
Diam. (mm)	**1**	72.89
	2	72.94
	3	72.82
	4	72.66
Average Diameter (m)		0.073
Volume (m^3)		6.02E-04
Bulk Unit Weight (kN/m^3)		17.6
Bulk Unit Weight (pcf)		111.9
Dry Unit Weight (kN/m^3)		12.0
Dry Unit Weight (pcf)		76.7

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Unconfined Compressive Strength
ASTM D2166

Project No.	0035-037-00
Client	Morrison Hershfield
Project	Empress Widening

Test Hole	RH16-03
Sample #	T87
Depth (m)	7.6 - 8.1
Sample Date	2-Nov-16
Test Date	1-Dec-16
Technician	MM

Unconfined Strength

	kPa	**ksf**
Max q_u	73.8	1.5
Max S_u	36.9	0.8

Specimen Data

Description CLAY - silty, trace silt inclusions (<20mmø), trace gravel (<25mmø), motled grey/brown, moist, soft to firm, high plasticity, homogeneous,

Length	144.6	(mm)	**Moisture %**	46%	
Diameter	72.8	(mm)	**Bulk Unit Wt.**	17.6	(kN/m^3)
L/D Ratio	2.0		**Dry Unit Wt.**	12.0	(kN/m^3)
Initial Area	0.00417	(m^2)	**Liquid Limit**	-	
Load Rate	1.00	(%/min)	**Plastic Limit**	-	
			Plasticity Index	-	

Undrained Shear Strength Tests

Torvane

Reading	**Undrained Shear Strength**	
tsf	**kPa**	**ksf**
0.25	24.5	0.51
Vane Size		
m		

Pocket Penetrometer

	Reading	**Undrained Shear Strength**	
	tsf	**kPa**	**ksf**
	0.60	29.4	0.61
	0.60	29.4	0.61
	0.65	31.9	0.67
Average	**0.62**	**30.2**	**0.63**

Failure Geometry

Sketch:

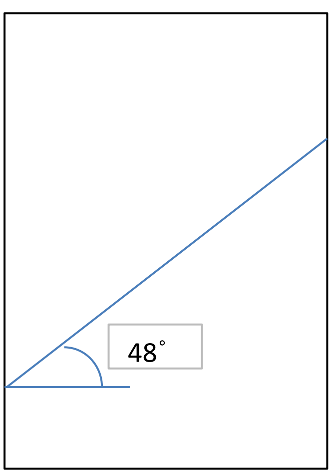

Photo:

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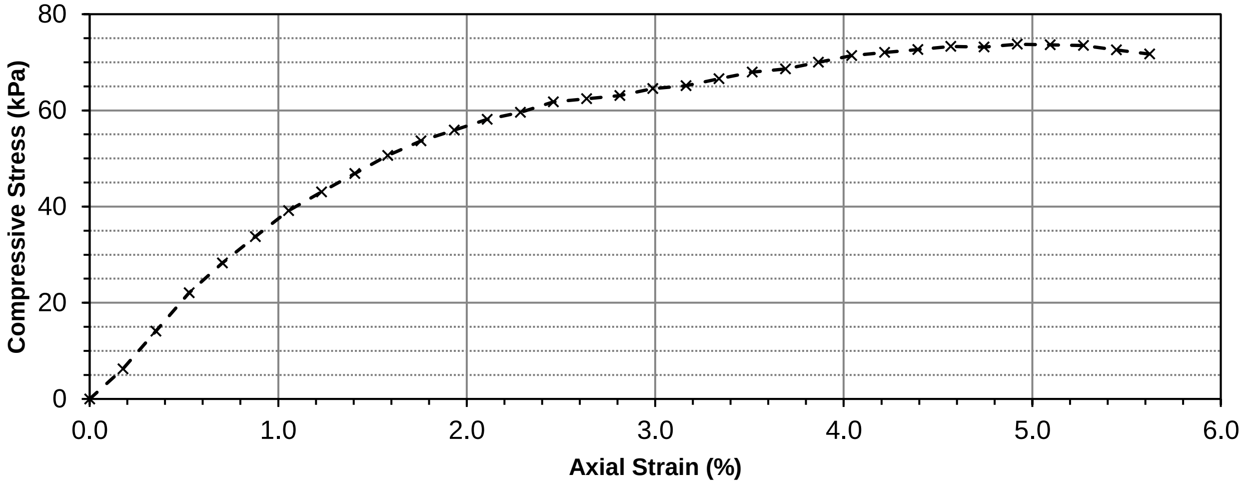
Unconfined Compressive Strength
ASTM D2166

Project No. 0035-037-00
Client Morrison Hershfield
Project Empress Widening

Unconfined Compression Test Graph



Unconfined Compression Test Data

Deformation Dial Reading	Load Ring Dial Reading	Deflection (mm)	Axial Strain (%)	Corrected Area (m^2)	Axial Load (N)	Compressive Stress, q_u (kPa)	Shear Stress, S_u (kPa)
0	0	0.0000	0.00	0.004166	0.0	0.00	0.00
10	8	0.2540	0.18	0.004173	26.2	6.27	3.13
20	18	0.5080	0.35	0.004180	58.9	14.10	7.05
30	28	0.7620	0.53	0.004188	92.3	22.04	11.02
40	36	1.0160	0.70	0.004195	118.7	28.29	14.14
50	43	1.2700	0.88	0.004203	141.8	33.73	16.87
60	50	1.5240	1.05	0.004210	164.9	39.16	19.58
70	55	1.7780	1.23	0.004217	181.4	43.00	21.50
80	60	2.0320	1.41	0.004225	197.8	46.82	23.41
90	65	2.2860	1.58	0.004233	214.3	50.63	25.32
100	69	2.5400	1.76	0.004240	227.5	53.65	26.82
110	72	2.7940	1.93	0.004248	237.4	55.89	27.94
120	75	3.0480	2.11	0.004255	247.3	58.11	29.05
130	77	3.3020	2.28	0.004263	253.9	59.55	29.77
140	80	3.5560	2.46	0.004271	263.8	61.77	30.88
150	81	3.8100	2.64	0.004278	267.1	62.42	31.21
160	82	4.0640	2.81	0.004286	270.4	63.08	31.54
170	84	4.3180	2.99	0.004294	276.9	64.50	32.25
180	85	4.5720	3.16	0.004302	280.2	65.15	32.57
190	87	4.8260	3.34	0.004309	286.8	66.56	33.28
200	89	5.0800	3.51	0.004317	293.4	67.97	33.99
210	90	5.3340	3.69	0.004325	296.7	68.61	34.30
220	92	5.5880	3.87	0.004333	303.3	70.00	35.00
230	94	5.8420	4.04	0.004341	309.9	71.39	35.70

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Unconfined Compressive Strength
ASTM D2166

Project No. 0035-037-00
Client Morrison Hershfield
Project Empress Widening

Unconfined Compression Test Data (cont'd)

Deformation Dial Reading	Load Ring Dial Reading	Deflection (mm)	Axial Strain (%)	Corrected Area (m^2)	Axial Load (N)	Compressive Stress, q_u (kPa)	Shear Stress, S_u (kPa)
240	95	6.0960	4.2164	0.004349	313.2	72.02	36.01
250	96	6.3500	4.39	0.004357	316.5	72.65	36.32
260	97	6.6040	4.57	0.004365	319.8	73.27	36.64
270	97	6.8580	4.74	0.004373	319.8	73.14	36.57
280	98	7.1120	4.92	0.004381	323.1	73.75	36.88
290	98	7.3660	5.09	0.004389	323.1	73.62	36.81
300	98	7.6200	5.27	0.004397	323.1	73.48	36.74
310	97	7.8740	5.45	0.004406	319.8	72.60	36.30
320	96	8.1280	5.62	0.004414	316.5	71.72	35.86

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Shelby Tube Visual

Project No.	0035-037-00
Client	Morrison Hershfield
Project	Empress Widening
Test Hole	RH16-04
Sample #	T112
Depth (m)	4.6 - 5.2
Sample Date	27-Oct-16
Test Date	29-Nov-16
Technician	MM

Tube Extraction

Recovery (mm) 610

Bottom - 5.2 m			Top - 4.6 m
Keep	Qu Bulk	Moisture Content Visual PP Tv	Keep
160 mm	160 mm	160 mm	130 mm

Visual Classification

Material	CLAY
Composition	silty
trace silt inclusions (<15mmø)	
trace organics	
Color	brown
Moisture	moist
Consistency	firm
Plasticity	high plasticity
Structure	homogeneous
Gradation	

Torvane

Reading	0.50
Vane Size (s,m,l)	m
Undrained Shear Strength (kPa)	49.0

Pocket Penetrometer

Reading	**1**	1.50
	2	1.40
	3	1.40
	Average	1.43
Undrained Shear Strength (kPa)		70.3

Moisture Content

Tare ID	F18
Mass tare (g)	8.4
Mass wet + tare (g)	421.5
Mass dry + tare (g)	293.4
Moisture %	44.9%

Unit Weight

Bulk Weight (g)		997.0
Length (mm)	**1**	143.18
	2	143.30
	3	143.43
	4	143.50
Average Length (m)		0.143
Diam. (mm)	**1**	71.97
	2	72.17
	3	72.56
	4	72.45
Average Diameter (m)		0.072
Volume (m^3)		5.88E-04
Bulk Unit Weight (kN/m^3)		16.6
Bulk Unit Weight (pcf)		105.8
Dry Unit Weight (kN/m^3)		11.5
Dry Unit Weight (pcf)		73.0

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Unconfined Compressive Strength
ASTM D2166

Project No.	0035-037-00
Client	Morrison Hershfield
Project	Empress Widening
Test Hole	RH16-04
Sample #	T112
Depth (m)	4.6 - 5.2
Sample Date	27-Oct-16
Test Date	29-Nov-16
Technician	MM

Unconfined Strength

	kPa	ksf
Max q_u	88.0	1.8
Max S_u	44.0	0.9

Specimen Data

Description CLAY - silty, trace silt inclusions (<15mmø), trace organics, brown, moist, firm, high plasticity, homogeneous,

Length	143.4	(mm)	**Moisture %**	45%	
Diameter	72.3	(mm)	**Bulk Unit Wt.**	16.6	(kN/m^3)
L/D Ratio	2.0		**Dry Unit Wt.**	11.5	(kN/m^3)
Initial Area	0.00410	(m^2)	**Liquid Limit**	-	
Load Rate	1.00	(%/min)	**Plastic Limit**	-	
			Plasticity Index	-	

Undrained Shear Strength Tests

Torvane

Reading	Undrained Shear Strength	
tsf	**kPa**	**ksf**
0.50	49.0	1.02
Vane Size		
m		

Pocket Penetrometer

	Reading	Undrained Shear Strength	
	tsf	**kPa**	**ksf**
	1.50	73.6	1.54
	1.40	68.7	1.43
	1.40	68.7	1.43
Average	**1.43**	**70.3**	**1.47**

Failure Geometry

Sketch:

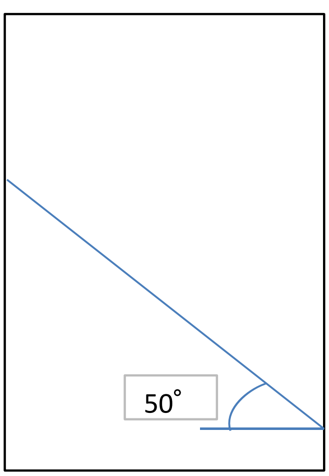

Photo:

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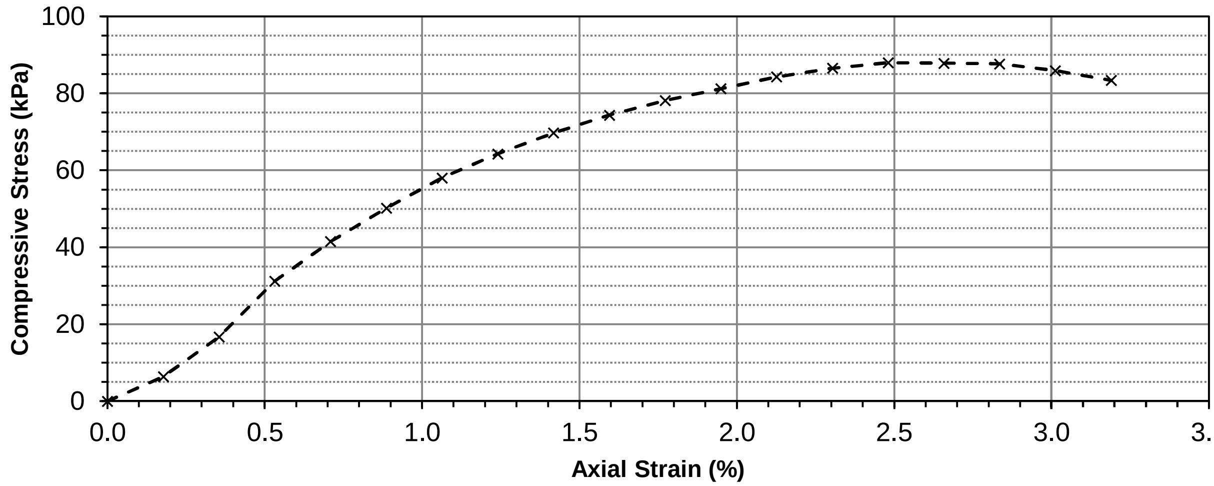
Unconfined Compressive Strength
ASTM D2166

Project No. 0035-037-00
Client Morrison Hershfield
Project Empress Widening

Unconfined Compression Test Graph



Unconfined Compression Test Data

Deformation Dial Reading	Load Ring Dial Reading	Deflection (mm)	Axial Strain (%)	Corrected Area (m^2)	Axial Load (N)	Compressive Stress, q_u (kPa)	Shear Stress, S_u (kPa)
0	0	0.0000	0.00	0.004104	0.0	0.00	0.00
10	8	0.2540	0.18	0.004111	26.2	6.36	3.18
20	21	0.5080	0.35	0.004119	68.8	16.70	8.35
30	39	0.7620	0.53	0.004126	128.6	31.17	15.58
40	52	1.0160	0.71	0.004133	171.4	41.48	20.74
50	63	1.2700	0.89	0.004141	207.7	50.17	25.08
60	73	1.5240	1.06	0.004148	240.7	58.02	29.01
70	81	1.7780	1.24	0.004156	267.1	64.27	32.13
80	88	2.0320	1.42	0.004163	290.2	69.70	34.85
90	94	2.2860	1.59	0.004171	309.9	74.31	37.15
100	99	2.5400	1.77	0.004178	326.4	78.12	39.06
110	103	2.7940	1.95	0.004186	339.8	81.18	40.59
120	107	3.0480	2.13	0.004193	353.3	84.25	42.12
130	110	3.3020	2.30	0.004201	363.4	86.50	43.25
140	112	3.5560	2.48	0.004208	370.1	87.95	43.98
150	112	3.8100	2.66	0.004216	370.1	87.79	43.90
160	112	4.0640	2.83	0.004224	370.1	87.63	43.82
170	110	4.3180	3.01	0.004232	363.4	85.87	42.94
180	107	4.5720	3.19	0.004239	353.3	83.33	41.67

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Shelby Tube Visual

Project No.	0035-037-00
Client	Morrison Hershifield
Project	Empress Widening
Test Hole	RH16-04
Sample #	T115
Depth (m)	7.6 - 8.3
Sample Date	3-Nov-16
Test Date	15-Dec-16
Technician	SGBR

Tube Extraction

Recovery (mm) 660

Bottom - 8.3 m **Top - 7.6 m**

Clay till, with grey clay silt inclusions	Keep	QU Bulk	Visual PP Tv	MC
100 mm	290 mm	170 mm	160 mm	40 mm

Visual Classification

Material	Clay
Composition	silty, silt inclusions
trace fine gravel	
bottom 100mm (Clay with till inclusion/trace sand)	
Color	mottled greenish brown
Moisture	moist
Consistency	soft
Plasticity	high plasticity
Structure	
Gradation	

Torvane

Reading	0.30
Vane Size (s,m,l)	m
Undrained Shear Strength (kPa)	29.4

Pocket Penetrometer

Reading	**1**	0.50
	2	0.50
	3	0.75
	Average	0.58
Undrained Shear Strength (kPa)		28.6

Moisture Content

Tare ID	43
Mass tare (g)	371.7
Mass wet + tare (g)	1866.4
Mass dry + tare (g)	1637.5
Moisture %	18.1%

Unit Weight

Bulk Weight (g)		1000.3
Length (mm)	**1**	143.12
	2	142.47
	3	142.26
	4	142.76
Average Length (m)		0.143
Diam. (mm)	**1**	71.71
	2	72.88
	3	72.23
	4	71.48
Average Diameter (m)		0.072
Volume (m^3)		5.82E-04
Bulk Unit Weight (kN/m^3)		16.9
Bulk Unit Weight (pcf)		107.3
Dry Unit Weight (kN/m^3)		14.3
Dry Unit Weight (pcf)		90.9

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Unconfined Compressive Strength
ASTM D2166

Project No.	0035-037-00
Client	Morrison Hershifield
Project	Empress Widening

Test Hole	RH16-04
Sample #	T115
Depth (m)	7.6 - 8.2
Sample Date	3-Nov-16
Test Date	15-Dec-16
Technician	SGBR

Unconfined Strength

	kPa	**ksf**
Max q_u	56.4	1.2
Max S_u	28.2	0.6

Specimen Data

Description Clay - silty, silt inclusions, trace fine gravel, bottom 100mm (Clay with till inclusion/trace sand), mottled greenish brown, moist, soft, high plasticity, ,

Length	142.7	(mm)	**Moisture %**	18%	
Diameter	72.1	(mm)	**Bulk Unit Wt.**	16.9	(kN/m^3)
L/D Ratio	2.0		**Dry Unit Wt.**	14.3	(kN/m^3)
Initial Area	0.00408	(m^2)	**Liquid Limit**	-	
Load Rate	1.00	(%/min)	**Plastic Limit**	-	
			Plasticity Index	-	

Undrained Shear Strength Tests

Torvane

Reading	**Undrained Shear Strength**	
tsf	**kPa**	**ksf**
0.36	35.3	0.74
Vane Size		
m		

Pocket Penetrometer

	Reading	**Undrained Shear Strength**	
	tsf	**kPa**	**ksf**
	0.50	24.5	0.51
	0.50	24.5	0.51
	0.75	36.8	0.77
Average	**0.58**	**28.6**	**0.60**

Failure Geometry

Sketch:

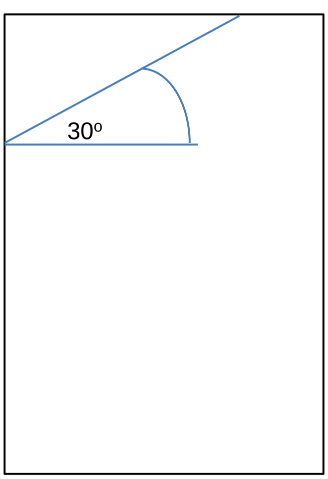

Photo:

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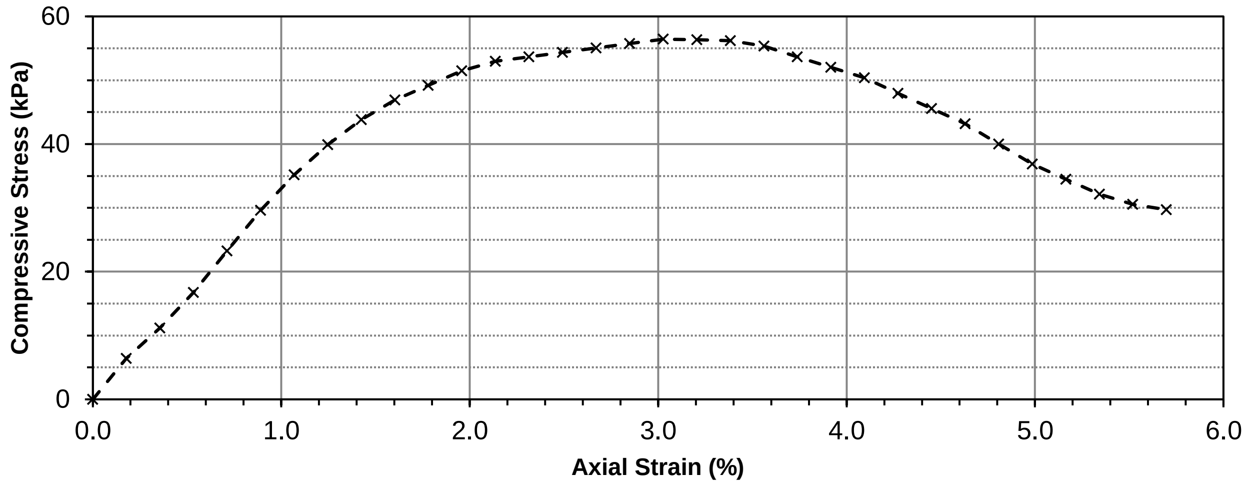
Unconfined Compressive Strength
ASTM D2166

Project No. 0035-037-00
Client Morrison Hershifield
Project Empress Widening

Unconfined Compression Test Graph



Unconfined Compression Test Data

Deformation Dial Reading	Load Ring Dial Reading	Deflection (mm)	Axial Strain (%)	Corrected Area (m^2)	Axial Load (N)	Compressive Stress, q_u (kPa)	Shear Stress, S_u (kPa)
0	0	0.0000	0.00	0.004080	0.0	0.00	0.00
10	8	0.2540	0.18	0.004087	26.2	6.40	3.20
20	14	0.5080	0.36	0.004095	45.8	11.19	5.59
30	21	0.7620	0.53	0.004102	68.8	16.77	8.38
40	29	1.0160	0.71	0.004109	95.6	23.26	11.63
50	37	1.2700	0.89	0.004117	122.0	29.63	14.81
60	44	1.5240	1.07	0.004124	145.1	35.17	17.59
70	50	1.7780	1.25	0.004131	164.9	39.90	19.95
80	55	2.0320	1.42	0.004139	181.4	43.82	21.91
90	59	2.2860	1.60	0.004146	194.5	46.91	23.46
100	62	2.5400	1.78	0.004154	204.4	49.21	24.60
110	65	2.7940	1.96	0.004161	214.3	51.50	25.75
120	67	3.0480	2.14	0.004169	220.9	52.99	26.49
130	68	3.3020	2.31	0.004177	224.2	53.68	26.84
140	69	3.5560	2.49	0.004184	227.5	54.37	27.18
150	70	3.8100	2.67	0.004192	230.8	55.05	27.53
160	71	4.0640	2.85	0.004200	234.1	55.75	27.87
170	72	4.3180	3.03	0.004207	237.4	56.43	28.21
180	72	4.5720	3.20	0.004215	237.4	56.32	28.16
190	72	4.8260	3.38	0.004223	237.4	56.22	28.11
200	71	5.0800	3.56	0.004231	234.1	55.34	27.67
210	69	5.3340	3.74	0.004238	227.5	53.67	26.84
220	67	5.5880	3.92	0.004246	220.9	52.02	26.01
230	65	5.8420	4.10	0.004254	214.3	50.38	25.19

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Unconfined Compressive Strength
ASTM D2166

Project No. 0035-037-00
Client Morrison Hershifield
Project Empress Widening

Unconfined Compression Test Data (cont'd)

Deformation Dial Reading	Load Ring Dial Reading	Deflection (mm)	Axial Strain (%)	Corrected Area (m^2)	Axial Load (N)	Compressive Stress, q_u (kPa)	Shear Stress, S_u (kPa)
240	62	6.0960	4.2733	0.004262	204.4	47.96	23.98
250	59	6.3500	4.45	0.004270	194.5	45.55	22.78
260	56	6.6040	4.63	0.004278	184.6	43.16	21.58
270	52	6.8580	4.81	0.004286	171.4	40.00	20.00
280	48	7.1120	4.99	0.004294	158.3	36.86	18.43
290	45	7.3660	5.16	0.004302	148.3	34.48	17.24
300	42	7.6200	5.34	0.004310	138.5	32.13	16.06
310	40	7.8740	5.52	0.004318	131.9	30.54	15.27
320	39	8.1280	5.70	0.004327	128.6	29.72	14.86

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Shelby Tube Visual

Project No.	0035-037-00
Client	Morrison Hershfield
Project	Empress Widening
Test Hole	RH16-05
Sample #	T130
Depth (m)	6.1 - 6.8
Sample Date	03-Nov-16
Test Date	02-Dec-16
Technician	MM

Tube Extraction

Recovery (mm) 670

Bottom - 6.8 m **Top - 6.1 m**

Moisture Content Visual	Keep	Bulk	PP Tv
180 mm	160 mm	160 mm	170 mm

Visual Classification

Material	CLAY
Composition	silty
trace silt inclusions (<30mmø)	
Color	grey
Moisture	moist
Consistency	soft to firm
Plasticity	high plasticity
Structure	homogeneous
Gradation	

Torvane

Reading	0.18
Vane Size (s,m,l)	m
Undrained Shear Strength (kPa)	17.2

Pocket Penetrometer

Reading	**1**	0.45
	2	0.45
	3	0.65
	Average	0.52
Undrained Shear Strength (kPa)		25.3

Moisture Content

Tare ID	Z57
Mass tare (g)	8.4
Mass wet + tare (g)	412.1
Mass dry + tare (g)	265.9
Moisture %	56.8%

Unit Weight

Bulk Weight (g)		996.3
Length (mm)	**1**	140.21
	2	139.96
	3	140.37
	4	139.96
Average Length (m)		0.140
Diam. (mm)	**1**	72.59
	2	72.37
	3	72.14
	4	72.29
Average Diameter (m)		0.072
Volume (m^3)		5.76E-04
Bulk Unit Weight (kN/m^3)		17.0
Bulk Unit Weight (pcf)		108.0
Dry Unit Weight (kN/m^3)		10.8
Dry Unit Weight (pcf)		68.9

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Shelby Tube Visual

Project No.	0035-037-00
Client	Morrison Hershfield
Project	Empress Widening
Test Hole	RH16-05
Sample #	T133
Depth (m)	9.1 - 9.7
Sample Date	02-Nov-16
Test Date	29-Nov-16
Technician	MM

Tube Extraction

Recovery (mm) 525

Bottom - 9.7 m | **Top - 9.1 m**

Kept	Kept	Kept	Moisture Content
145 mm	160 mm	160 mm	60 mm

Visual Classification

Material	CLAY
Composition	silty
trace silt inclusions (<20mmø)	
Color	grey
Moisture	moist
Consistency	firm
Plasticity	high plasticity
Structure	homogeneous
Gradation	

Torvane

Reading	0.35
Vane Size (s,m,l)	m
Undrained Shear Strength (kPa)	34.3

Pocket Penetrometer

Reading	**1**	0.75
	2	1.00
	3	0.75
	Average	0.83
Undrained Shear Strength (kPa)		40.9

Moisture Content

Tare ID	E123
Mass tare (g)	8.9
Mass wet + tare (g)	350.3
Mass dry + tare (g)	253
Moisture %	39.9%

Unit Weight

Bulk Weight (g)		876.9
Length (mm)	**1**	125.40
	2	125.58
	3	125.44
	4	125.62
Average Length (m)		0.126
Diam. (mm)	**1**	72.45
	2	72.21
	3	72.11
	4	72.59
Average Diameter (m)		0.072
Volume (m^3)		5.16E-04
Bulk Unit Weight (kN/m^3)		16.7
Bulk Unit Weight (pcf)		106.1
Dry Unit Weight (kN/m^3)		11.9
Dry Unit Weight (pcf)		75.9

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Shelby Tube Visual

Project No.	0035-037-00
Client	Morrison Hershfield
Project	Empress Widening
Test Hole	RH16-06
Sample #	T147
Depth (m)	4.6 - 5.2
Sample Date	03-Nov-16
Test Date	02-Dec-16
Technician	MM

Tube Extraction

Recovery (mm) 690

Bottom - 5.3 m | **Top - 4.6 m**

Moisture Content / Visual / PP / Tv	Qu / Bulk	Keep	Slough
170 mm	160 mm	160 mm	200 mm

Visual Classification

Material	CLAY
Composition	silty
trace silt inclusions (<25mmø)	
Color	motled grey/brown
Moisture	moist
Consistency	firm to stiff
Plasticity	high plasticity
Structure	homogeneous
Gradation	

Torvane

Reading	0.55
Vane Size (s,m,l)	m
Undrained Shear Strength (kPa)	53.9

Pocket Penetrometer

Reading	**1**	1.60
	2	1.50
	3	1.50
	Average	1.53
Undrained Shear Strength (kPa)		75.2

Moisture Content

Tare ID	E92
Mass tare (g)	8.4
Mass wet + tare (g)	389.4
Mass dry + tare (g)	253.5
Moisture %	55.4%

Unit Weight

Bulk Weight (g)		1032.2
Length (mm)	**1**	140.24
	2	140.33
	3	140.17
	4	140.49
Average Length (m)		0.140
Diam. (mm)	**1**	73.48
	2	73.12
	3	73.61
	4	73.54
Average Diameter (m)		0.073
Volume (m^3)		5.94E-04
Bulk Unit Weight (kN/m^3)		17.0
Bulk Unit Weight (pcf)		108.4
Dry Unit Weight (kN/m^3)		11.0
Dry Unit Weight (pcf)		69.8

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Unconfined Compressive Strength
ASTM D2166

Project No.	0035-037-00
Client	Morrison Hershfield
Project	Empress Widening
Test Hole	RH16-06
Sample #	T147
Depth (m)	4.6 - 5.2
Sample Date	3-Nov-16
Test Date	2-Dec-16
Technician	MM

Unconfined Strength

	kPa	**ksf**
Max q_u	107.9	2.3
Max S_u	53.9	1.1

Specimen Data

Description CLAY - silty, trace silt inclusions (<25mmø), motled grey/brown, moist, firm to stiff, high plasticity, homogeneous,

Length	140.3	(mm)	**Moisture %**	55%	
Diameter	73.4	(mm)	**Bulk Unit Wt.**	17.0	(kN/m^3)
L/D Ratio	1.9		**Dry Unit Wt.**	11.0	(kN/m^3)
Initial Area	0.00424	(m^2)	**Liquid Limit**	-	
Load Rate	1.00	(%/min)	**Plastic Limit**	-	
			Plasticity Index	-	

Undrained Shear Strength Tests

Torvane

Reading	**Undrained Shear Strength**	
tsf	**kPa**	**ksf**
0.55	53.9	1.13
Vane Size		
m		

Pocket Penetrometer

	Reading	**Undrained Shear Strength**	
	tsf	**kPa**	**ksf**
	1.60	78.5	1.64
	1.50	73.6	1.54
	1.50	73.6	1.54
Average	**1.53**	**75.2**	**1.57**

Failure Geometry

Sketch:

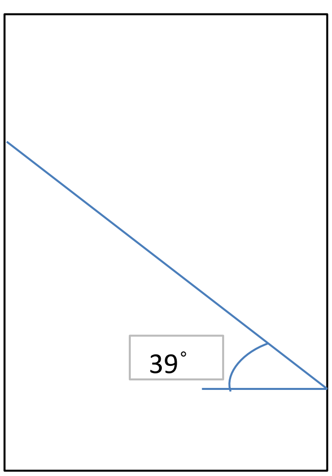

Photo:

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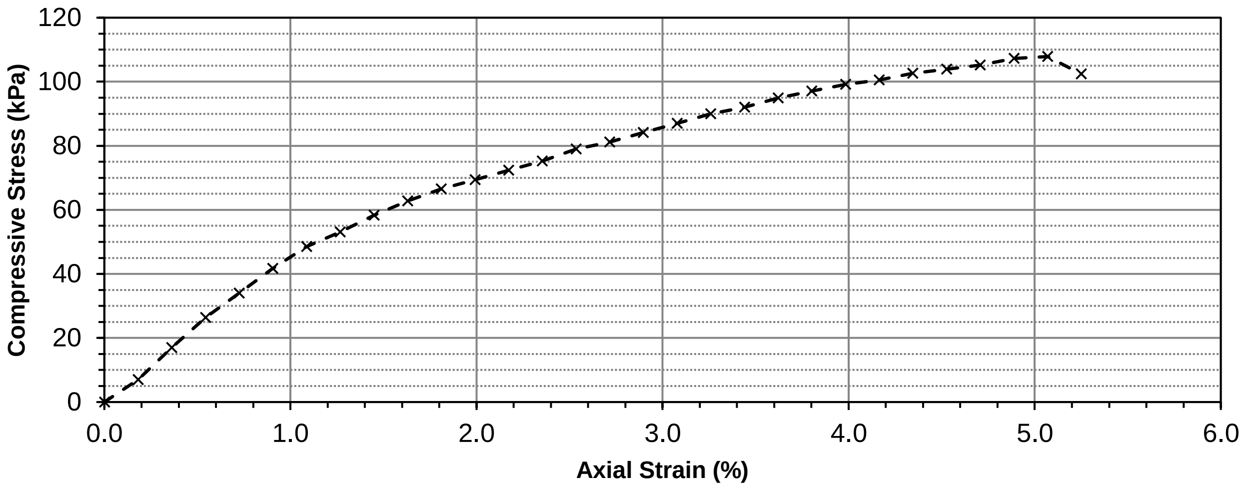
Tel: 204.975.9433 Fax: 204.975.9435

Unconfined Compressive Strength
ASTM D2166

Project No. 0035-037-00
Client Morrison Hershfield
Project Empress Widening

Unconfined Compression Test Graph



Unconfined Compression Test Data

Deformation Dial Reading	Load Ring Dial Reading	Deflection (mm)	Axial Strain (%)	Corrected Area (m^2)	Axial Load (N)	Compressive Stress, q_u (kPa)	Shear Stress, S_u (kPa)
0	0	0.0000	0.00	0.004236	0.0	0.00	0.00
10	9	0.2540	0.18	0.004243	29.4	6.94	3.47
20	22	0.5080	0.36	0.004251	72.1	16.95	8.48
30	34	0.7620	0.54	0.004259	112.1	26.32	13.16
40	44	1.0160	0.72	0.004267	145.1	34.00	17.00
50	54	1.2700	0.91	0.004274	178.0	41.65	20.82
60	63	1.5240	1.09	0.004282	207.7	48.51	24.26
70	69	1.7780	1.27	0.004290	227.5	53.03	26.51
80	76	2.0320	1.45	0.004298	250.6	58.30	29.15
90	82	2.2860	1.63	0.004306	270.4	62.79	31.39
100	87	2.5400	1.81	0.004314	286.8	66.49	33.24
110	91	2.7940	1.99	0.004322	300.0	69.42	34.71
120	95	3.0480	2.17	0.004330	313.2	72.34	36.17
130	99	3.3020	2.35	0.004338	326.4	75.25	37.62
140	104	3.5560	2.53	0.004346	343.2	78.97	39.48
150	107	3.8100	2.72	0.004354	353.3	81.14	40.57
160	111	4.0640	2.90	0.004362	366.8	84.08	42.04
170	115	4.3180	3.08	0.004370	380.2	87.01	43.50
180	119	4.5720	3.26	0.004378	393.7	89.91	44.96
190	122	4.8260	3.44	0.004387	403.8	92.06	46.03
200	126	5.0800	3.62	0.004395	417.2	94.94	47.47
210	129	5.3340	3.80	0.004403	427.4	97.06	48.53
220	132	5.5880	3.98	0.004411	437.5	99.17	49.59
230	134	5.8420	4.16	0.004420	444.2	100.50	50.25

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Unconfined Compressive Strength
ASTM D2166

Project No. 0035-037-00
Client Morrison Hershfield
Project Empress Widening

Unconfined Compression Test Data (cont'd)

Deformation Dial Reading	Load Ring Dial Reading	Deflection (mm)	Axial Strain (%)	Corrected Area (m^2)	Axial Load (N)	Compressive Stress, q_u (kPa)	Shear Stress, S_u (kPa)
240	137	6.0960	4.3447	0.004428	454.3	102.59	51.30
250	139	6.3500	4.53	0.004436	461.1	103.92	51.96
260	141	6.6040	4.71	0.004445	467.8	105.24	52.62
270	144	6.8580	4.89	0.004453	477.9	107.31	53.65
280	145	7.1120	5.07	0.004462	481.3	107.86	53.93
290	138	7.3660	5.25	0.004470	457.7	102.38	51.19

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Shelby Tube Visual

Project No.	0035-037-00
Client	Morrison Hershifield
Project	Empress Widening
Test Hole	TH16-06
Sample #	T150
Depth (m)	7.6 - 8.2
Sample Date	01-Nov-16
Test Date	15-Dec-16
Technician	SGBR

Tube Extraction

Recovery (mm) 620

Bottom - 8.2 m **Top - 7.6 m**

MC	QU	PP TV Visual	KEEP
45 mm	150 mm	150 mm	275 mm

Visual Classification

Material	Clay
Composition	silty, trace silt inclusions
trace medium sand, trace fine gravel	
Color	mottled dark grey
Moisture	moist
Consistency	soft
Plasticity	high plasticity
Structure	inclusion
Gradation	

Torvane

Reading	0.24
Vane Size (s,m,l)	m
Undrained Shear Strength (kPa)	23.5

Pocket Penetrometer

Reading	**1**	0.50
	2	0.50
	3	0.70
	Average	0.57
Undrained Shear Strength (kPa)		27.8

Moisture Content

Tare ID	K8
Mass tare (g)	532.3
Mass wet + tare (g)	1966.6
Mass dry + tare (g)	1496.2
Moisture %	48.8%

Unit Weight

Bulk Weight (g)		1049.2
Length (mm)	**1**	146.42
	2	146.24
	3	146.61
	4	146.23
Average Length (m)		0.146
Diam. (mm)	**1**	73.15
	2	73.13
	3	72.47
	4	74.13
Average Diameter (m)		0.073
Volume (m^3)		6.16E-04
Bulk Unit Weight (kN/m^3)		16.7
Bulk Unit Weight (pcf)		106.3
Dry Unit Weight (kN/m^3)		11.2
Dry Unit Weight (pcf)		71.4

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Unconfined Compressive Strength
ASTM D2166

Project No.	0035-037-00
Client	Morrison Hershifield
Project	Empress Widening

Test Hole	TH16-06
Sample #	T150
Depth (m)	7.6 - 8.2
Sample Date	1-Nov-16
Test Date	15-Dec-16
Technician	SGBR

Unconfined Strength

	kPa	ksf
Max q_u	90.2	1.9
Max S_u	45.1	0.9

Specimen Data

Description Clay - silty, trace silt inclusions, trace medium sand, trace fine gravel, mottled dark grey, moist, soft, high plasticity, inclusion,

Length	146.4	(mm)	**Moisture %**	49%	
Diameter	73.2	(mm)	**Bulk Unit Wt.**	16.7	(kN/m^3)
L/D Ratio	2.0		**Dry Unit Wt.**	11.2	(kN/m^3)
Initial Area	0.00421	(m^2)	**Liquid Limit**	-	
Load Rate	1.00	(%/min)	**Plastic Limit**	-	
			Plasticity Index	-	

Undrained Shear Strength Tests

Torvane

Reading	Undrained Shear Strength	
tsf	**kPa**	**ksf**
0.24	23.5	0.49
Vane Size		
m		

Pocket Penetrometer

	Reading	Undrained Shear Strength	
	tsf	**kPa**	**ksf**
	0.50	24.5	0.51
	0.50	24.5	0.51
	0.70	34.3	0.72
Average	**0.57**	**27.8**	**0.58**

Failure Geometry

Sketch:

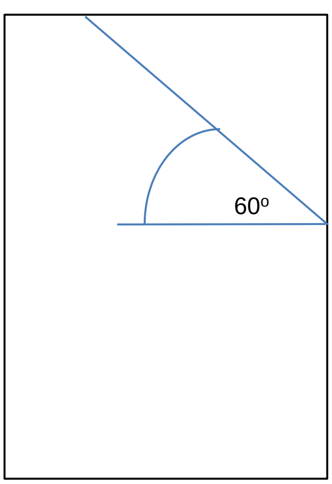

Photo:

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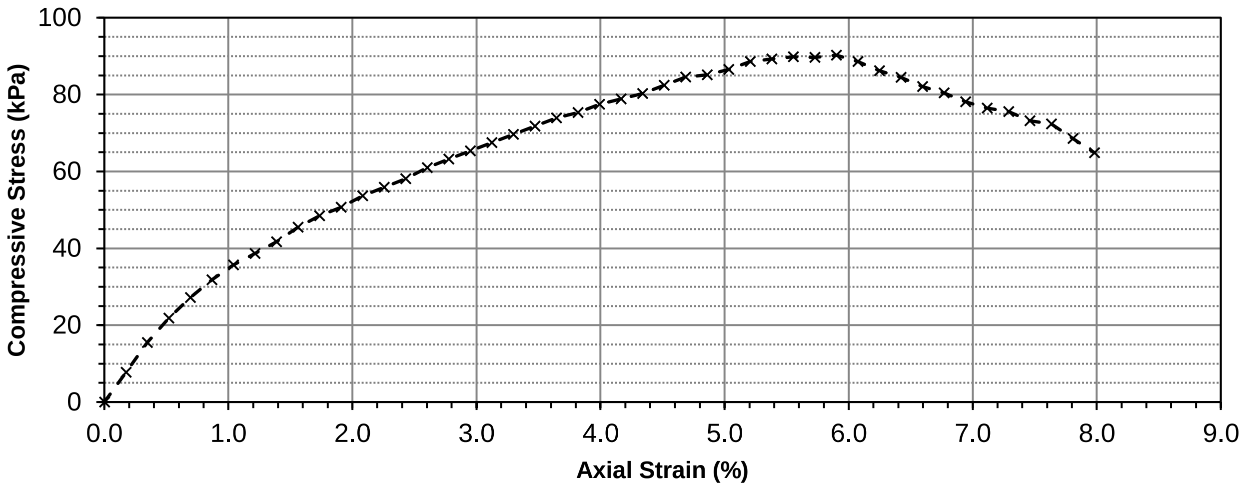
Unconfined Compressive Strength
ASTM D2166

Project No. 0035-037-00
Client Morrison Hershifield
Project Empress Widening

Unconfined Compression Test Graph



Unconfined Compression Test Data

Deformation Dial Reading	Load Ring Dial Reading	Deflection (mm)	Axial Strain (%)	Corrected Area (m^2)	Axial Load (N)	Compressive Stress, q_u (kPa)	Shear Stress, S_u (kPa)
0	0	0.0000	0.00	0.004211	0.0	0.00	0.00
10	10	0.2540	0.17	0.004218	32.7	7.75	3.88
20	20	0.5080	0.35	0.004225	65.5	15.50	7.75
30	28	0.7620	0.52	0.004233	92.3	21.81	10.90
40	35	1.0160	0.69	0.004240	115.4	27.21	13.61
50	41	1.2700	0.87	0.004248	135.2	31.83	15.91
60	46	1.5240	1.04	0.004255	151.7	35.65	17.82
70	50	1.7780	1.21	0.004262	164.9	38.68	19.34
80	54	2.0320	1.39	0.004270	178.0	41.69	20.85
90	59	2.2860	1.56	0.004277	194.5	45.48	22.74
100	63	2.5400	1.74	0.004285	207.7	48.48	24.24
110	66	2.7940	1.91	0.004293	217.6	50.69	25.35
120	70	3.0480	2.08	0.004300	230.8	53.67	26.83
130	73	3.3020	2.26	0.004308	240.7	55.87	27.94
140	76	3.5560	2.43	0.004315	250.6	58.06	29.03
150	80	3.8100	2.60	0.004323	263.8	61.02	30.51
160	83	4.0640	2.78	0.004331	273.7	63.19	31.59
170	86	4.3180	2.95	0.004339	283.5	65.35	32.67
180	89	4.5720	3.12	0.004346	293.4	67.52	33.76
190	92	4.8260	3.30	0.004354	303.3	69.66	34.83
200	95	5.0800	3.47	0.004362	313.2	71.80	35.90
210	98	5.3340	3.64	0.004370	323.1	73.94	36.97
220	100	5.5880	3.82	0.004378	329.7	75.31	37.66
230	103	5.8420	3.99	0.004386	339.8	77.48	38.74

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Unconfined Compressive Strength
ASTM D2166

Project No. 0035-037-00
Client Morrison Hershifield
Project Empress Widening

Unconfined Compression Test Data (cont'd)

Deformation Dial Reading	Load Ring Dial Reading	Deflection (mm)	Axial Strain (%)	Corrected Area (m^2)	Axial Load (N)	Compressive Stress, q_u (kPa)	Shear Stress, S_u (kPa)
240	105	6.0960	4.1646	0.004394	346.6	78.88	39.44
250	107	6.3500	4.34	0.004402	353.3	80.26	40.13
260	110	6.6040	4.51	0.004410	363.4	82.41	41.20
270	113	6.8580	4.69	0.004418	373.5	84.54	42.27
280	114	7.1120	4.86	0.004426	376.9	85.15	42.58
290	116	7.3660	5.03	0.004434	383.6	86.51	43.26
300	119	7.6200	5.21	0.004442	393.7	88.63	44.31
310	120	7.8740	5.38	0.004450	397.0	89.22	44.61
320	121	8.1280	5.55	0.004458	400.4	89.82	44.91
330	121	8.3820	5.73	0.004466	400.4	89.65	44.83
340	122	8.6360	5.90	0.004475	403.8	90.24	45.12
350	120	8.8900	6.07	0.004483	397.0	88.57	44.28
360	117	9.1440	6.25	0.004491	387.0	86.16	43.08
370	115	9.3980	6.42	0.004500	380.2	84.50	42.25
380	112	9.6520	6.59	0.004508	370.1	82.11	41.05
390	110	9.9060	6.77	0.004516	363.4	80.46	40.23
400	107	10.1600	6.94	0.004525	353.3	78.08	39.04
410	105	10.4140	7.11	0.004533	346.6	76.45	38.23
420	104	10.6680	7.29	0.004542	343.2	75.56	37.78
430	101	10.9220	7.46	0.004550	333.1	73.20	36.60
440	100	11.1760	7.64	0.004559	329.7	72.32	36.16
450	95	11.4300	7.81	0.004567	313.2	68.57	34.29
460	90	11.6840	7.98	0.004576	296.7	64.85	32.42

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Shelby Tube Visual

Project No.	0035-037-00
Client	Morrison Hershfield
Project	Empress Widening
Test Hole	RH16-07
Sample #	T166
Depth (m)	6.1 - 6.7
Sample Date	03-Nov-16
Test Date	02-Dec-16
Technician	MM

Tube Extraction

Recovery (mm) 610

Bottom - 6.7 m **Top - 6.1 m**

Moisture Content Visual	PPT v	Keep	Qu Bulk	Keep
100 mm		160 mm	160 mm	190 mm

Visual Classification

Material	CLAY
Composition	silty
trace silt inclusions (<15mmø)	
Color	grey
Moisture	moist
Consistency	soft - firm
Plasticity	high plasticity
Structure	homogeneous
Gradation	

Torvane

Reading	0.38
Vane Size (s,m,l)	m
Undrained Shear Strength (kPa)	36.8

Pocket Penetrometer

Reading	**1**	1.30
	2	1.35
	3	1.50
	Average	1.38
Undrained Shear Strength (kPa)		67.8

Moisture Content

Tare ID	E33
Mass tare (g)	8.5
Mass wet + tare (g)	407.7
Mass dry + tare (g)	262.3
Moisture %	57.3%

Unit Weight

Bulk Weight (g)		963.3
Length (mm)	**1**	142.41
	2	142.50
	3	142.27
	4	142.53
Average Length (m)		0.142
Diam. (mm)	**1**	71.71
	2	71.50
	3	71.25
	4	71.64
Average Diameter (m)		0.072
Volume (m^3)		5.72E-04
Bulk Unit Weight (kN/m^3)		16.5
Bulk Unit Weight (pcf)		105.1
Dry Unit Weight (kN/m^3)		10.5
Dry Unit Weight (pcf)		66.8

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Unconfined Compressive Strength
ASTM D2166

Project No.	0035-037-00
Client	Morrison Hershfield
Project	Empress Widening

Test Hole	RH16-07
Sample #	T166
Depth (m)	6.1 - 6.7
Sample Date	3-Nov-16
Test Date	2-Dec-16
Technician	MM

Unconfined Strength

	kPa	ksf
Max q_u	76.3	1.6
Max S_u	38.2	0.8

Specimen Data

Description CLAY - silty, trace silt inclusions (<15mmø), grey, moist, soft - firm, high plasticity, homogeneous,

Length	142.4	(mm)	**Moisture %**	57%	
Diameter	71.5	(mm)	**Bulk Unit Wt.**	16.5	(kN/m^3)
L/D Ratio	2.0		**Dry Unit Wt.**	10.5	(kN/m^3)
Initial Area	0.00402	(m^2)	**Liquid Limit**	-	
Load Rate	1.00	(%/min)	**Plastic Limit**	-	
			Plasticity Index	-	

Undrained Shear Strength Tests

Torvane

Reading	**Undrained Shear Strength**	
tsf	**kPa**	**ksf**
0.38	36.8	0.77
Vane Size		
m		

Pocket Penetrometer

	Reading	**Undrained Shear Strength**	
	tsf	**kPa**	**ksf**
	1.30	63.8	1.33
	1.35	66.2	1.38
	1.50	73.6	1.54
Average	**1.38**	**67.9**	**1.42**

Failure Geometry

Sketch:

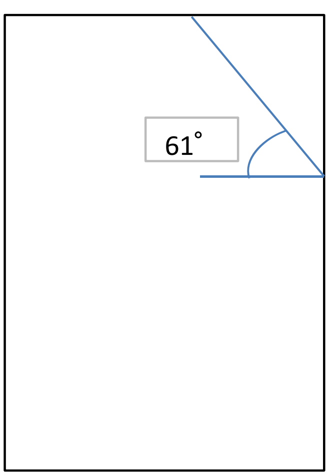

Photo:

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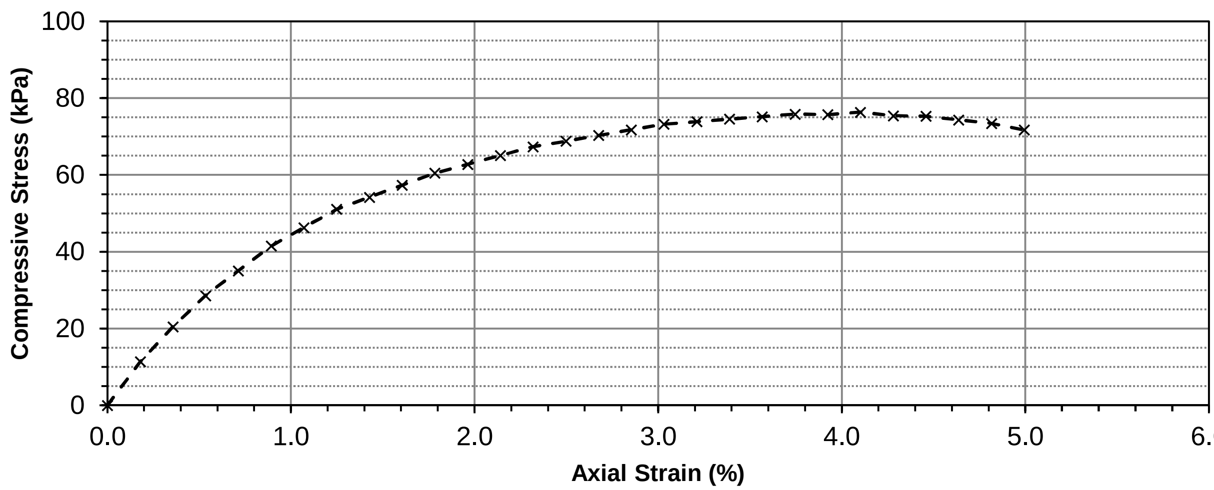
Unconfined Compressive Strength
ASTM D2166

Project No. 0035-037-00
Client Morrison Hershfield
Project Empress Widening

Unconfined Compression Test Graph



Unconfined Compression Test Data

Deformation Dial Reading	Load Ring Dial Reading	Deflection (mm)	Axial Strain (%)	Corrected Area (m^2)	Axial Load (N)	Compressive Stress, q_u (kPa)	Shear Stress, S_u (kPa)
0	0	0.0000	0.00	0.004018	0.0	0.00	0.00
10	14	0.2540	0.18	0.004025	45.8	11.38	5.69
20	25	0.5080	0.36	0.004032	82.4	20.44	10.22
30	35	0.7620	0.54	0.004040	115.4	28.56	14.28
40	43	1.0160	0.71	0.004047	141.8	35.03	17.52
50	51	1.2700	0.89	0.004054	168.1	41.47	20.74
60	57	1.5240	1.07	0.004061	187.9	46.27	23.14
70	63	1.7780	1.25	0.004069	207.7	51.06	25.53
80	67	2.0320	1.43	0.004076	220.9	54.19	27.10
90	71	2.2860	1.61	0.004084	234.1	57.33	28.67
100	75	2.5400	1.78	0.004091	247.3	60.45	30.22
110	78	2.7940	1.96	0.004098	257.2	62.75	31.37
120	81	3.0480	2.14	0.004106	267.1	65.05	32.52
130	84	3.3020	2.32	0.004113	276.9	67.33	33.66
140	86	3.5560	2.50	0.004121	283.5	68.80	34.40
150	88	3.8100	2.68	0.004128	290.2	70.28	35.14
160	90	4.0640	2.85	0.004136	296.7	71.75	35.87
170	92	4.3180	3.03	0.004144	303.3	73.20	36.60
180	93	4.5720	3.21	0.004151	306.6	73.86	36.93
190	94	4.8260	3.39	0.004159	309.9	74.52	37.26
200	95	5.0800	3.57	0.004167	313.2	75.17	37.58
210	96	5.3340	3.75	0.004174	316.5	75.83	37.91
220	96	5.5880	3.92	0.004182	316.5	75.69	37.84
230	97	5.8420	4.10	0.004190	319.8	76.33	38.17

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Unconfined Compressive Strength

ASTM D2166

Project No. 0035-037-00
Client Morrison Hershfield
Project Empress Widening

Unconfined Compression Test Data (cont'd)

Deformation Dial Reading	Load Ring Dial Reading	Deflection (mm)	Axial Strain (%)	Corrected Area (m^2)	Axial Load (N)	Compressive Stress, q_u (kPa)	Shear Stress, S_u (kPa)
240	96	6.0960	4.2801	0.004198	316.5	75.41	37.70
250	96	6.3500	4.46	0.004205	316.5	75.27	37.63
260	95	6.6040	4.64	0.004213	313.2	74.34	37.17
270	94	6.8580	4.82	0.004221	309.9	73.42	36.71
280	92	7.1120	4.99	0.004229	303.3	71.72	35.86

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Shelby Tube Visual

Project No.	0035-037-00
Client	Morrison Hershifield
Project	Empress Widening

Test Hole	TH16-07
Sample #	T169
Depth (m)	9.1 - 9.8
Sample Date	03-Nov-16
Test Date	15-Dec-16
Technician	SGBR

Tube Extraction

Recovery (mm) 600

Bottom - 9.7 m		Top - 9.1 m
Visual PP Tv Moisture Content	Qu γ_{Bulk}	Keep
155 mm	155 mm	290 mm

Visual Classification

Material	Clay till
Composition	silty, silt inclusion
trace medium sand, trace fine gravel	
Color	mottled light brown
Moisture	moist
Consistency	clay till
Plasticity	high
Structure	
Gradation	

Torvane

Reading	0.35
Vane Size (s,m,l)	m
Undrained Shear Strength (kPa)	34.3

Pocket Penetrometer

Reading	**1**	0.70
	2	0.50
	3	0.50
	Average	0.57
Undrained Shear Strength (kPa)		27.8

Moisture Content

Tare ID	HA
Mass tare (g)	377.6
Mass wet + tare (g)	1818.6
Mass dry + tare (g)	1346.5
Moisture %	48.7%

Unit Weight

Bulk Weight (g)		1249.5
Length (mm)	**1**	154.61
	2	155.02
	3	155.50
	4	154.92
Average Length (m)		0.155
Diam. (mm)	**1**	71.37
	2	73.27
	3	72.88
	4	70.31
Average Diameter (m)		0.072
Volume (m^3)		6.30E-04
Bulk Unit Weight (kN/m^3)		19.4
Bulk Unit Weight (pcf)		123.7
Dry Unit Weight (kN/m^3)		13.1
Dry Unit Weight (pcf)		83.2

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Unconfined Compressive Strength
ASTM D2166

Project No.	0035-037-00
Client	Morrison Hershifield
Project	Empress Widening

Test Hole	TH16-07
Sample #	T169
Depth (m)	9.1 - 9.8
Sample Date	3-Nov-16
Test Date	15-Dec-16
Technician	SGBR

Unconfined Strength

	kPa	ksf
Max q_u	56.9	1.2
Max S_u	28.5	0.6

Specimen Data

Description CLAY - silty, silt inclusion, trace medium sand, trace fine gravel, mottled brown, moist, firm, high, ,

Length	155.0	(mm)	**Moisture %**	49%	
Diameter	72.0	(mm)	**Bulk Unit Wt.**	19.4	(kN/m^3)
L/D Ratio	2.2		**Dry Unit Wt.**	13.1	(kN/m^3)
Initial Area	0.00407	(m^2)	**Liquid Limit**	-	
Load Rate	1.00	(%/min)	**Plastic Limit**	-	
			Plasticity Index	-	

Undrained Shear Strength Tests

Torvane

Reading	Undrained Shear Strength	
tsf	kPa	ksf
0.48	47.1	0.98
Vane Size		
m		

Pocket Penetrometer

	Reading	Undrained Shear Strength	
	tsf	kPa	ksf
	1.20	58.9	1.23
	0.50	24.5	0.51
	0.50	24.5	0.51
Average	**0.73**	**36.0**	**0.75**

Failure Geometry

Sketch:

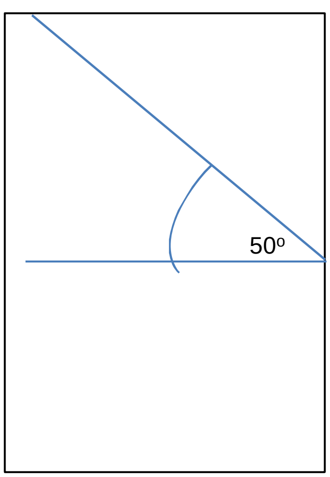

Photo:

TREK GEOTECHNICAL
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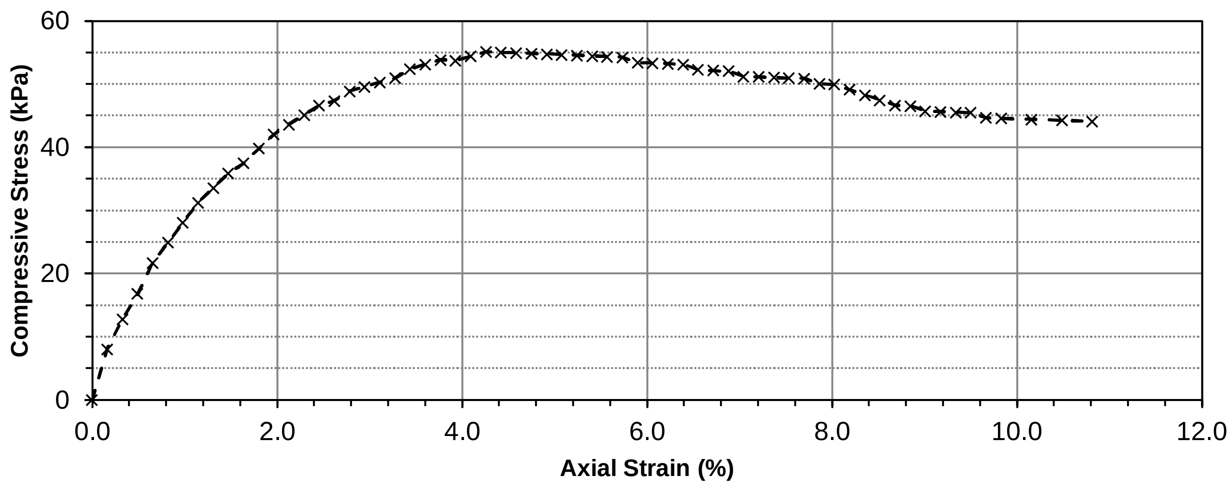
Unconfined Compressive Strength
ASTM D2166

Project No. 0035-037-00
Client Morrison Hershifield
Project Empress Widening

Unconfined Compression Test Graph



Unconfined Compression Test Data

Deformation Dial Reading	Load Ring Dial Reading	Deflection (mm)	Axial Strain (%)	Corrected Area (m^2)	Axial Load (N)	Compressive Stress, q_u (kPa)	Shear Stress, S_u (kPa)
0	0	0.0000	0.00	0.004067	0.0	0.00	0.00
10	10	0.2540	0.16	0.004073	32.7	8.03	4.01
20	16	0.5080	0.33	0.004080	52.4	12.84	6.42
30	21	0.7620	0.49	0.004087	68.8	16.83	8.41
40	27	1.0160	0.66	0.004094	89.0	21.74	10.87
50	31	1.2700	0.82	0.004100	102.2	24.93	12.46
60	35	1.5240	0.98	0.004107	115.4	28.09	14.05
70	39	1.7780	1.15	0.004114	128.6	31.26	15.63
80	42	2.0320	1.31	0.004121	138.5	33.60	16.80
90	45	2.2860	1.47	0.004128	148.3	35.94	17.97
100	47	2.5400	1.64	0.004134	155.0	37.48	18.74
110	50	2.7940	1.80	0.004141	164.9	39.81	19.90
120	53	3.0480	1.97	0.004148	174.7	42.12	21.06
130	55	3.3020	2.13	0.004155	181.4	43.64	21.82
140	57	3.5560	2.29	0.004162	187.9	45.15	22.58
150	59	3.8100	2.46	0.004169	194.5	46.66	23.33
160	60	4.0640	2.62	0.004176	197.8	47.37	23.68
170	62	4.3180	2.79	0.004183	204.4	48.86	24.43
180	63	4.5720	2.95	0.004190	207.7	49.57	24.79
190	64	4.8260	3.11	0.004197	211.0	50.28	25.14
200	65	5.0800	3.28	0.004204	214.3	50.97	25.49
210	67	5.3340	3.44	0.004212	220.9	52.45	26.22
220	68	5.5880	3.60	0.004219	224.2	53.14	26.57
230	69	5.8420	3.77	0.004226	227.5	53.83	26.91

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Unconfined Compressive Strength
ASTM D2166

Project No. 0035-037-00
Client Morrison Hershifield
Project Empress Widening

Unconfined Compression Test Data (cont'd)

Deformation Dial Reading	Load Ring Dial Reading	Deflection (mm)	Axial Strain (%)	Corrected Area (m^2)	Axial Load (N)	Compressive Stress, q_u (kPa)	Shear Stress, S_u (kPa)
240	69	6.0960	3.9326	0.004233	227.5	53.74	26.87
250	70	6.3500	4.10	0.004240	230.8	54.42	27.21
260	71	6.6040	4.26	0.004248	234.1	55.12	27.56
270	71	6.8580	4.42	0.004255	234.1	55.02	27.51
280	71	7.1120	4.59	0.004262	234.1	54.93	27.46
290	71	7.3660	4.75	0.004270	234.1	54.83	27.42
300	71	7.6200	4.92	0.004277	234.1	54.74	27.37
310	71	7.8740	5.08	0.004284	234.1	54.64	27.32
320	71	8.1280	5.24	0.004292	234.1	54.55	27.27
330	71	8.3820	5.41	0.004299	234.1	54.45	27.23
340	71	8.6360	5.57	0.004307	234.1	54.36	27.18
350	71	8.8900	5.74	0.004314	234.1	54.27	27.13
360	70	9.1440	5.90	0.004322	230.8	53.40	26.70
370	70	9.3980	6.06	0.004329	230.8	53.31	26.65
380	70	9.6520	6.23	0.004337	230.8	53.21	26.61
390	70	9.9060	6.39	0.004344	230.8	53.12	26.56
400	69	10.1600	6.55	0.004352	227.5	52.27	26.14
410	69	10.4140	6.72	0.004360	227.5	52.18	26.09
420	69	10.6680	6.88	0.004367	227.5	52.09	26.04
430	68	10.9220	7.05	0.004375	224.2	51.24	25.62
440	68	11.1760	7.21	0.004383	224.2	51.15	25.58
450	68	11.4300	7.37	0.004390	224.2	51.06	25.53
460	68	11.6840	7.54	0.004398	224.2	50.97	25.49
470	68	11.9380	7.70	0.004406	224.2	50.88	25.44
480	67	12.1920	7.87	0.004414	220.9	50.05	25.02
490	67	12.4460	8.03	0.004422	220.9	49.96	24.98
500	66	12.7000	8.19	0.004430	217.6	49.13	24.56
510	65	12.9540	8.36	0.004438	214.3	48.30	24.15
520	64	13.2080	8.52	0.004445	211.0	47.47	23.73
530	63	13.4620	8.68	0.004453	207.7	46.65	23.32
540	63	13.7160	8.85	0.004461	207.7	46.56	23.28
550	62	13.9700	9.01	0.004469	204.4	45.73	22.87
560	62	14.2240	9.18	0.004478	204.4	45.65	22.82
570	62	14.4780	9.34	0.004486	204.4	45.57	22.78
580	62	14.7320	9.50	0.004494	204.4	45.48	22.74
590	61	14.9860	9.67	0.004502	201.1	44.67	22.34
600	61	15.2400	9.83	0.004510	201.1	44.59	22.29
620	61	15.7480	10.16	0.004527	201.1	44.43	22.21
640	61	16.2560	10.49	0.004543	201.1	44.27	22.13
660	61	16.7640	10.81	0.004560	201.1	44.10	22.05

Appendix C

Photographs of Pavement Core Samples

Photo 1: Pavement Core Sample at Test Hole RH16-03

Photo 2: Pavement Core Sample at Test Hole RH16-04

Photo 3: Pavement Core Sample at Test Hole RH16-05

Photo 4: Pavement Core Sample at Test Hole RH16-06

Photo 5: Pavement Core Sample at Test Hole RH16-07

Photo 6: Pavement Core Sample at Test Hole RH16-08

Photo 7: Pavement Core Sample at Test Hole RH16-09

Photo 8: Pavement Core Sample at Test Hole RH16-10

Photo 9: Pavement Core Sample at Test Hole RH16-11

Photo 10: Pavement Core Sample at Test Hole RH16-12

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Photo 16: Pavement Core Sample at Test Hole RH16-18

Photo 17: Pavement Core Sample at Test Hole RH16-19

GEOTECHNICAL Quality Engineering | Valued Relationships

Morrison Hershfield

Empress Pedestrian Ramp

Geotechnical Investigation
Final Report

Prepared for:
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Project Number:
0035-037-00

Date:
March 13, 2018

March 13, 2018 Our File No. 0035-037-00

Beth Phillips, P. Eng., C.I.M.
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RE: Empress Pedestrian Ramps Geotechnical Investigation Final Report

TREK Geotechnical Inc. is pleased to submit our Geotechnical Investigation Report for the above noted project.

Please contact the undersigned if you have any questions. Thank you for the opportunity to provide our services on this project.

Sincerely,

TREK Geotechnical Inc.
Per:

Michael Van Helden, Ph.D., P.Eng
Senior Geotechnical Engineer
Tel: 204.975.9433

Encl.

Revision History

Revision No.	Project Engineer	Issue Date	Description
0	MVH	March 13, 2018	Final Report

Authorization Signatures

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No. 4877 Date: March 13, 2018

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Senior Technical Reviewer

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1.0 Introduction

This report summarizes the results of a geotechnical investigation and the assessment of foundation alternatives carried out by TREK Geotechnical Inc. (TREK) for the preliminary design of the Empress Overpass pedestrian ramps in Winnipeg, Manitoba. The terms of reference for this assignment are included in our proposal to Morrison Hershfield (MH), dated January 3, 2018. The scope of work includes geotechnical investigation and preliminary design parameters for the design of ramp foundations and associated works.

2.0 Background

TREK understands that two ramps are being proposed extending west from the Empress overpass down to the north and south sides of Portage Avenue. Currently the overpass is only accessible by stairs off of Portage Avenue and requires universal accessibility as part of overall active transportation improvements to the area. The north ramp is anticipated to be approximately 60 m long running parallel to Portage Avenue, while the south ramp is expected to have three segments totalling approximately 45 m in length running both parallel and perpendicular to Portage Avenue. Morrison Hershfield has provided preliminary drawings for anticipated geometry of the ramps. We understand proposed ramps will consist of a combination of structural walkways (primarily the north ramp) connected to Mechanically Stabilized Earth (MSE) wall embankments. The structures are anticipated to supported by single-column piers founded on deep foundations with anticipated factored point-loads of up to 445 kN.

3.0 Sub-Surface Conditions

3.1 Subsurface Investigations

Three test holes (THs) were drilled between February 9 and February 23, 2018 in the vicinity of the ramp footprints as shown on Figure 01. Two shallow holes (TH18-02 and -03) were drilled near the south ramp to design for shallow foundations and embankments. One deep test hole (TH18-01) was drilled to power auger refusal near the north ramp to evaluate deep foundation alternatives. Upon review of the results the drill rig was re-mobilized on a separate date to continue TH18-01 by coring into bedrock.

The test holes were completed under supervision of TREK personnel and were visually classified based on the Unified Soil Classification System (USCS). Disturbed (auger cutting and split spoon) and relatively undisturbed (Shelby tube) samples were recovered during drilling. Standard Penetration testing (SPT) was carried out in the till to measure compactness (consistency) and obtain disturbed (split spoon) samples. Continuous core samples of the underlying bedrock were also recovered in TH18-01. Soil and rock samples were transported to TREK's soils laboratory in Winnipeg, Manitoba for further classification and testing. The test hole logs are attached include a description of the soils units encountered, sample type and depth, the results of field and laboratory testing and other pertinent information such as sloughing and groundwater seepage.

Laboratory testing consisted of the determination of field moisture content, bulk unit weight measurements, unconfined compression tests and uniaxial compressive strength test were performed on select samples. Results of the laboratory testing are summarized on the detailed test hole logs, and are included separately in Appendix D

3.2 Subsurface Conditions

A brief description of the soil units encountered at the test hole locations is provided below. All interpretations of soil stratigraphy for the purposes of design should refer to the detailed information provided on the attached test hole logs.

The soil stratigraphy generally consists of fill overlying silty clay, silt till, sand and dolomitic limestone. In TH18-02, a silt layer was encountered underlying the fill. The fill consisted of varying layers of silt, sand or clay and extends up to 2.8 m at the test hole locations. Silt encountered below the fill in TH18-02 is soft and of low to intermediate plasticity, and extends from 2.2 to 2.8 m depth. High plastic, silty clay underlies the fill or silt layers. The clay is brown, moist and stiff to very stiff becoming softer with depth and contains trace precipitates and trace silt inclusions. Silt till underlies the clay at 10.7 m depth (in TH18-01) and is generally compact becoming very dense with depth, light brown, moist, and of low plasticity, and contains some sand, trace to some gravel, trace clay and trace cobbles. A layer of poorly graded sand is contained within the till from 12.1 m to 13.4 m depth and is brown, wet and compact to dense. Dolomitic limestone bedrock extends below 14.8 m depth is light brown to cream colour, has rock quality designation (RQD) between 78-100%, is classified as grade R3-R4 (strong) confirmed by a uniaxial compressive strength of 53.4 MPa at 14.8 m depth.

3.4 Groundwater Conditions

Seepage and sloughing conditions at the time of drilling are noted on the attached test hole logs. Seepage and sloughing were observed in TH18-01 within the sand layer at approximately 12.2 – 13.1 m depth. Seepage and sloughing were not observed in TH18-02 and TH18-03.

These observations are short-term and should not be considered reflective of (static) groundwater levels at the site which would require monitoring over an extended period of time to determine. It is important to recognize that groundwater conditions may vary seasonally, annually, or as a result of construction activities.

4.0 Foundation Recommendations

Recommendations for design and construction of foundations are provided below. Based on observed conditions and anticipated loads for the structural walkways, we consider cast-in-place concrete (CIPC) friction piles will be the most cost-effective alternative provided sufficient capacity can be achieved within the geometric constraints of the site. Other feasible foundation alternatives include driven precast prestressed concrete hexagonal (PPCH) piles and driven steel H-piles. Recommendations for shallow foundations are also provided for structural and MSE walls.

4.1 Limit States Design (CHBDC)

Limit states design requires consideration of distinct loading scenarios comparing the structural loads to the foundation bearing capacity using resistance and load factors that are based on probabilistic reliability criteria. Two general design scenarios are evaluated corresponding to the serviceability and ultimate capacity requirements.

The Ultimate Limit State (ULS) is concerned with ensuring that the maximum structural loads do not exceed the nominal (ultimate) capacity of the foundation units. The ULS foundation bearing capacity is obtained by multiplying the nominal (ultimate) bearing capacity by a resistance factor (reduction factor), which is then compared to the factored (increased) structural loads. The ULS bearing capacity must be greater or equal to the maximum factored load. Table 1summarizes the resistance factors that can be used for the design of foundations as per the CHBDC depending upon the method of analysis and verification testing completed during construction. The CHBDC also requires that the degree of understanding of soil conditions (which can be classified as either low, typical or high) be assessed in the selection of the resistance factors. Based on the local exploration performed for this project and TREK's extensive experience with the proposed pile types in similar geological conditions in Winnipeg, we consider the current level of understanding at the site to be high for the design of deep foundations. For shallow foundations, some uncertainty exists regarding the presence of fill soils or silt at the subgrade level; provided that bearing surface inspection is conducted by TREK during construction, we consider the degree of understanding of soil conditions to be high for shallow foundations as well. CHBDC also requires that the resistance factor be modified by a consequence factor which ranges from 0.9 for high consequence structures to 1.15 for low consequence structures. The structures for this project are interpreted to be of typical consequence based on the CHBDC guidelines and as such a consequence factor 1.0 is applied in our recommendations.

The Service Limit State (SLS) is concerned with limiting deformation or settlement of the foundation under service loading conditions such that the integrity of the structure will not be impacted. The SLS should generally be analysed by calculating the settlement resulting from applied service loads and comparing this to the settlement tolerance of the structure. However, the settlement tolerance of the structure is typically not defined at the preliminary design stage. As such, SLS bearing capacities (or unit resistances) provided are developed on the basis of limiting settlement to approximately 25 mm or less, unless a methodology to estimate foundation settlement is provided. A more detailed settlement analysis should be conducted to refine the estimated settlement and/or adjust the SLS vertical bearing resistance if a more stringent settlement tolerance is required.

Table 1. ULS Resistance Factors for Foundations (CHBDC, 2014)

Description	Resistance Factor for Typical Degree of Understanding of Soil Conditions	Resistance Factor for High Degree of Understanding of Soil Conditions
Shallow foundations with a typical degree of understanding of soil conditions and using empirical analysis	0.50	0.60
Shallow foundations for analysis of sliding on cohesive material	0.60	0.65
Deep foundations in compression based on static analysis	0.40	0.45
Deep foundations in compression based on dynamic testing	0.50	0.55
Deep foundations in tension based on static analysis	0.30	0.40

4.2 Deep Foundations

4.2.1 *Cast-In-Place Concrete Friction Piles (CIPC)*

CIPC friction piles will derive a majority of their resistance in shaft friction (adhesion) with a relatively small contribution from end bearing. Table 2 provides the recommended axial (compressive and uplift) unit resistances for shaft adhesion and end bearing. Cast in place friction piles in Winnipeg typically exhibit less than 25 mm of pile head displacement under loading approaching the nominal capacity. As such, piles designed on the basis of ULS resistances provided are expected to exhibit no greater than 25 mm of settlement at unfactored service loads.

Table 2. Recommended ULS and SLS Resistances for CIPC Friction Piles

<table>
<tr><th rowspan="3">Pile Depth Below Ground Surface at Test Hole Location (m)</th><th colspan="3">ULS Axial Unit Resistance (kPa)</th></tr>
<tr><th colspan="2">Compression
$\phi = 0.45$</th><th>Uplift
$\phi = 0.4$</th></tr>
<tr><th>Shaft Adhesion</th><th>End Bearing</th><th>Shaft Adhesion</th></tr>
<tr><td>0 to X (Note 1)</td><td>-</td><td>-</td><td>-</td></tr>
<tr><td>X to 5 (Note 1)</td><td>15</td><td>155</td><td>13</td></tr>
<tr><td>5 to 9</td><td>11</td><td>78</td><td>9</td></tr>
</table>

Notes: Skin friction should be neglected within the depth “X” of frost penetration, disturbance or fill soils. For piles subjected to freezing conditions, the top 2.5 m of the pile should be neglected (as shown in the table).

<u>CIPC Friction Pile Design Recommendations:</u>

1. The weight of the embedded portion of the pile may be neglected.
2. The piles should have a minimum shaft diameter of 406 mm.
3. Pile lengths should be limited to a depth of 9 m below the existing ground surface to avoid penetrating the till and to protect against heaving at the base of the pile shaft.
4. Piles should have a minimum spacing of 3 pile diameters measured centre to centre. If a closer spacing is required, TREK should be contacted to provide an efficiency (reduction) factor to account for potential group effects.
5. Piles require steel reinforcement to be designed by a qualified structural engineer for the anticipated axial (compression and uplift), lateral, and bending loads induced from the structure, as well as additional forces developed in the piles induced by seasonal movements of surrounding bearing soils.

CIPC Friction Pile Installation Recommendations:

1. Seepage and sloughing (caving) conditions were not observed during test hole drilling within the clay and are considered unlikely to occur during drilling of the pile shafts. However, if seepage and sloughing conditions occur, temporary steel casings (*i.e.* sleeves) should be used to control groundwater and maintain stability of the drilled shaft. Care should be taken in removing sleeves to prevent sloughing (necking) of the shaft walls and a reduction in the cross-sectional area of the pile.
2. Concrete should be placed in one continuous operation immediately after the completion of drilling the pile hole to avoid construction problems such as sloughing or caving of the pile hole and groundwater seepage. Concrete should be poured under dry conditions. If groundwater is encountered, it should be controlled and removed. If water cannot be controlled and removed, the concrete should be placed using tremie methods.
3. Concrete placed by free-fall methods should be directed through the middle of the pile shaft and steel reinforcing cage to prevent striking of the drilled shaft walls to protect against soil contamination of the concrete

4.2.2 Driven Precast Prestressed Concrete Hexagonal Piles

Driven precast prestressed concrete hexagonal (PPCH) piles driven to practical refusal will derive a majority of their resistance in end bearing with a smaller contribution from shaft adhesion. It is likely that practical refusal will occur in dense till or bedrock. The recommended factored ULS capacities for PPCH piles driven to practical refusal are provided in Table 3. Pile head settlement at the Service Limit State (SLS) can be evaluated by adding up to 10 mm of pile tip displacement to the elastic shortening of the pile section under unfactored service loads.

Pile Size (mm)	Refusal Criteria (Blows/ 25mm)	Factored ULS Axial Resistance		
		Compression Capacity (kN)		Uplift Shaft Adhesion (kPa)
		$\phi = 0.45$	$\phi = 0.55$	$\phi = 0.4$
305	5	620	760	9
356	8	865	1,060	
406	12	1,110	1,365	

Note: Resistance factor of $\phi = 0.55$ requires dynamic pile testing (PDA testing) of production piles.

The piles should be driven to at least three consecutive sets of the refusal criteria outlined in Table 3, using a diesel hammer having a minimum rated energy of 40 kJ or a hydraulic drop hammer having a minimum rated energy of 20 kJ.

Driven PPCH Pile Design Recommendations

The following recommendations apply to the design of driven PPCH piles:

1. The weight of the embedded portion of the pile may be neglected.
2. The piles must be designed to withstand design loads, handling stresses, and driving stresses induced during installation.
3. The piles should be cured for at least 7 days prior to driving.
4. Pile spacing should not be less than 2.5 pile diameters, measured centre to centre. If a closer spacing is required, TREK should be contacted to provide an efficiency (reduction) factor to account for potential group effects.
5. Uplift resistance should be neglected along the portion of the pile embedded within fill and/or silt layers. If pre-boring is completed (see pile design recommendation number 6) and the length of the pre-bore hole extends below the bottom of the soil layers described above, the pre-bore length should be neglected from uplift resistance.
6. To aid in pile alignment and reduce pile heave during driving, pre-boring should be undertaken. A typical pre-bore length is about 3 m; however, once the pile design is complete, TREK can assist in developing an appropriate pre-boring plan for the piles prior to construction. The pre-bore diameter should be no more than 50 mm larger than the pile diameter.

Driven PPCH Pile Installation Recommendations

The following recommendations apply to the installation of driven PPCH piles:

1. The pile-driving hammer should have the capability of adjusting the delivered energy to operate at higher settings during driving if the delivered energy is not sufficient to mobilize the ultimate pile capacity. The driving system should also have the capability of adjusting the delivered energy to operate at lower settings to prevent high tensile stresses during easy driving and to prevent pile damage upon sudden pile refusal.
2. The pile-driving hammer should be equipped with a pile cushion to protect the pile head from damage during driving from direct impact with the steel driving helmet. The pile cushion should consist of a minimum of 100 mm of compressible material such as plywood or hardwood (*e.g.* oak). The pile cushion should fit tightly inside the pile helmet.
3. The tops of each pile should be checked such that no reinforcing strands protrude from the asphalt topping, in order to prevent spalling during driving, and that the pile head is square.
4. Piles should be driven continuously once driving is initiated to the required refusal criteria.
5. Where a steel follower is required to install piles below the ground surface, the refusal criteria should be increased by 50% in order to account for additional energy losses through the use of the follower.
6. Re-driving of all piles in groups should be specified along with the requirement to monitor for pile heave. All piles exhibiting heave of 6 mm or more should be re-driven to a minimum of one set of the practical refusal criteria.

7. Pile verticality (plumbness) should be measured on all piles after practical refusal has been achieved to check if verticality is within the limits of the structural design. It is common local practice to specify a maximum acceptable percentage that the pile can be out of vertical plumbness (e.g. 2% out of plumb).
8. Inspection of all driven piles should be performed by TREK personnel to confirm that the refusal criteria have been met and to record that pile installation has been completed according to the design.
9. Any piles damaged, out of plumb an excessive amount or reaching premature refusal may need to be replaced. The structural designer will have to assess non-conforming piles to determine if they are acceptable. PDA testing with CAPWAP analysis is recommended for any piles that are suspected to not meet the design capacity or to be damaged if a structural solution is not possible.
10. PDA testing of precast concrete piles is considered good practice to verify end-bearing capacity, that piles have been installed without exceeding the permissible driving stresses such that no pile damage occurs and to verify the relationship between driving resistance and capacity. PDA testing is therefore recommended.

4.2.3 Driven Steel H-Piles

Steel H-piles driven to refusal on bedrock are considered suitable to support the proposed ramp structures. This pile type will derive a majority of its resistance in end bearing with a relatively small contribution from shaft adhesion. Care should be taken when reaching refusal to prevent pile damage.

The axial compressive capacity of steel piles will be controlled by the structural capacity of the pile (based on the strength of steel pile used) due to the high rock strength, rock mass quality and ultimate tip resistance of the pile, provided the piles are driven to refusal on bedrock. The pile head settlement under unfactored service loads can be calculated based on 5 mm or less of pile tip displacement plus elastic shortening of the pile.

Steel H-piles driven to refusal will derive their uplift resistance in skin friction within overburden deposits. An average ULS skin friction of 10 kPa should be used for soils above bedrock for the purposes of uplift resistance calculations.

Design Recommendations

1. The weight of the embedded portion of the pile should be neglected in design.
2. Pile spacing should not be less than 2.5 pile diameters, measured centre to centre. If a closer spacing is required, TREK should be contacted to review the pile layout.
3. The piles must be structurally designed to withstand the design loads, handling stresses, and driving stresses.
4. All piles should be fitted with driving tips to help protect the pile tip during installation. The driving tip must be designed to withstand driving stresses and long-term design load cases.

Installation Recommendations

1. A pile driving system (*i.e.* pile-driving hammer) capable of developing at least 350 J/cm^2 (open-ended diesel hammers) or 250 J/cm^2 (hydraulic hammers) should be specified for driving steel piles. The minimum developed energy for the hammer can be calculated by multiplying this value by the cross-sectional area of the pile in cross-section. For example, an HP310x110 steel H-pile has a cross-sectional area of 141 cm^2 and therefore should be driven with at least 49 kJ of developed energy for a diesel hammer. Developed energy is the potential energy of the ram and can be estimated by measuring the blow rate of the hammer (single-acting diesel hammers), ram velocity or ram drop height. The pile-driving hammer should have the capability of adjusting the fuel setting or stroke to deliver higher energy to the pile during driving if the energy is not sufficient to drive the pile to the required tip elevation. The driving system should also have the capability of adjusting the fuel setting or stroke to deliver lower energy to prevent pile damage upon sudden pile refusal.
2. Piles should be driven to refusal on bedrock. Pile installation should be completed carefully near refusal to avoid overdriving of the piles, which could lead to pile damage or misalignment. Refusal is generally considered to be reached when three consecutive sets of 12 blows of the hammer produce 25 mm (1”) or less of pile penetration (per set), provided that a driving system capable of producing the required delivered energy to the pile per blow is used.
3. Driving stresses in the pile should not exceed 90% of the yield stress of the pile material.
4. The Contractor should be required to submit a proposed driving system for approval a minimum of 7 days prior to the start of pile driving. The pile driving system should be capable of installing the piles to the required tip elevation within specified allowable driving stresses.
5. All piles driven within 5 pile diameters of one another should be monitored for pile heave and where heave is observed, all piles should be checked and piles exhibiting heave should be re-driven to one set of the specified refusal criteria.
6. Pile verticality (plumbness) should be measured on all piles after practical refusal has been achieved to check if verticality is within the limits of the structural design. It is common local practice to specify a maximum acceptable percentage that the pile can be out of vertical plumbness (e.g. 2% out of plumb) or out of the specified batter.
7. Inspection of all driven H-piles should be performed by TREK personnel to confirm that the refusal criteria have been met and to record that pile installation has been completed according to the design.
8. Any piles damaged, out of plumb an excessive amount or reaching premature refusal may need to be replaced. The structural designer will have to assess non-conforming piles to determine if they are acceptable. PDA testing with CAPWAP analysis is recommended for any piles that are suspected to not meet the design capacity or to be damaged if a structural solution is not possible.
9. PDA testing of driven steel piles is considered good practice to verify end-bearing capacity, that piles have been installed without exceeding the permissible driving stresses such that no pile damage occurs and to verify the relationship between driving resistance and capacity. PDA testing is therefore recommended.

4.3 Shallow Foundations

Embankments for the pedestrian ramps will likely be constrained by using either consist of pre-cast or cast-in-place concrete walls bearing on strip footings or MSE walls. Strip footings or MSE walls bearing on undisturbed firm to stiff clay can be designed based on a ULS and SLS bearing resistances of 130 kPa and 80 kPa respectively.

For shallow footings, the SLS bearing resistance is based on a settlement of 25 mm or less and the ULS bearing resistance was calculated using a resistance factor of 0.6. Shallow footings can be expected to be subject to vertical movements associated with seasonal shrinkage and swelling of the clay bearing soils. If a footing is founded above 2.5 m depth they will also be subject to seasonal movements related to freeze/thaw. In this case, rigid polystyrene insulation should be included to provide an equivalent frost penetration depth of 2.5 m.

For MSE walls, settlement will be dependent on the height and footprint of the embankments and should be reviewed by TREK. Based on past experience with MSE walls on Winnipeg clays the ULS bearing capacity will not be exceeded provided the wall height is less than 2 m. However, applied bearing pressures calculated by the MSE wall supplier should be compared to the ULS bearing capacity of the clay once the MSE wall design is complete. Additionally, the global embankment stability of MSE walls will need to be reviewed by TREK once the MSE wall geometry has been determined.

Additional recommendations for the design and construction of shallow foundations are provided below.

<u>Shallow Footing Design Recommendations:</u>

1. Footings should have a minimum base width of 0.6 m.
2. Footings should be designed by a qualified structural engineer to resist axial, lateral, and bending loads from the structure.

<u>Shallow Footing Installation Recommendations:</u>

1. All fill, silts, organics and/or any other deleterious material should be completely stripped such that the bearing surface consists of undisturbed native stiff to very stiff silty clay. A soft silt layer approximately 0.5 m thick at 2.1 m depth was encountered in one test hole however could be present across the site. Where silt is encountered at the design bearing surface, it should be removed and replaced with 20 mm down crushed limestone base material overlying non-woven geotextile. The crushed limestone should be placed in lifts no greater than 150 mm and compacted to a minimum of 100% SPMDD. Depending on the design subgrade elevation for the footings or walls, up to 3.1 m of fill may need to be removed.
2. Excavations for footings should be completed by an excavator equipped with a smooth bladed bucket operating from the edge of the excavation. The contractor should work carefully to prevent disturbance to the bearing surface at all times.
3. Over-excavation of the bearing surface should be avoided. If a levelling course is required below the footing it may be constructed using 20 mm down crushed rock compacted to 100% of Standard Proctor Maximum Dry Density (SPMDD).
4. The bearing surface should be protected from freezing, drying, inundation with water or disturbance

at all times. If groundwater seepage is encountered, it should be controlled and removed from the excavation, such that concrete is placed under dry conditions.

5. The final bearing surface should be inspected and documented by TREK personnel prior to concrete placement to verify the adequacy of the bearing surface and proper installation of footings.

Resistance to Overturning, Uplift and Sliding:

If exterior footings are subjected to lateral loads, they must be designed to resist overturning, uplift and sliding forces. Lateral loading will result in the development of overturning and uplift forces and consequently a non-uniform applied pressure distribution under the footing base. In this regard, the maximum applied pressure should not exceed the ULS bearing resistance and the minimum applied pressure should not be less than 0 kPa (*i.e.* the eccentric resultant vertical force shall not be more than B/6 away from the footing centreline). Resistance to overturning and uplift forces due to lateral loading will be provided from the weight of the material used to backfill the footing excavation and the structural dead loads. A unit weight of 17 kN/m³ can be used for clay fill provided it is compacted to a minimum of 95% of the SPMDD. For the evaluation of sliding of the footing bearing directly on native clay, a friction angle of 15 degrees may be used along the concrete/clay interface. A geotechnical resistance factor of 0.6 should be used when assessing sliding resistance on clay in accordance with Table 6.2 of CHBDC. However, it is our understanding that footings may be cast on a low-strength concrete "mud-slab" underlain by a well-compacted layer of granular base course. In this case, sliding resistance between the mud-slab and granular base course may be calculated based on a sliding friction angle of 30 degrees with a resistance factor of 0.8 applied (CHBDC Table 6.2 for non-cohesive soils).

4.4 Ad-freezing Effects

Buried concrete sections or steel piles subjected to freezing conditions should be designed to resist ad-freeze and uplift forces related to frost action acting along the vertical face of the member within the maximum depth of frost penetration (2.4 m). In this regard concrete members may be subject to an ad-freeze bond stress of 65 kPa and steel members to 100 kPa within the depth of frost penetration.

Ad-freeze forces will be resisted by structural dead loads and uplift resistance provided by the length of the wall below the depth of frost penetration. The following design recommendations apply to ad-freeze forces:

1. A load factor (α) of 1.2 may be used in the calculation of ad-freezing forces.
2. A reduction factor of 0.8 may be used in calculation of the geotechnical resistance for the ULS condition with an ultimate (nominal) uplift resistance of 28 kPa. Structural dead loads should be added to the resistance.
3. The calculated geotechnical resistance plus the structural dead loads must be greater than the factored ad-freezing forces.
4. Measures such as flat lying rigid polystyrene insulation could be considered to reduce frost penetration depths and thereby ad-freezing and uplift forces.
5. Use of non-frost susceptible soils such sand and gravel with minimal fines as backfill around piles and buried structures could be considered to minimize ad-freeze forces.

4.5 Lateral Pile Analysis

The soil response (subgrade reaction) to lateral loads can be modeled in a simplified manner that assumes the soil around a pile can be simulated by a series of horizontal springs for preliminary design of pile foundations. The soil behaviour can be estimated using an equivalent spring constant referred to as the lateral subgrade reaction modulus (K_s) as provided in Table 4. The majority of lateral resistance will typically be offered by the upper 5 to 10 m of soil, depending on the relative stiffness of the pile and soil units. Void spaces surrounding piles due to pre-boring activities should be in-filled with lean-mix concrete to ensure compliance with the surrounding soil. If in-filling is not completed, the depth of the pre-bore should be neglected from lateral pile resistance calculations effectively leaving the pre-bore portion of the pile as a free cantilever beam condition.

Table 4. Recommended Values for Lateral Subgrade Reaction Modulus (K_s)

Depth Below Final Grade (m)	Ks (kN/m3)
0 to X (or depth of pre-bore)	0
X (or depth of pre-bore) to 5 m	3,100/d
5 m to 10.5 m (bottom of clay)	1600/d
Till or bedrock	4,400z/d

Notes: Skin friction should be neglected within the depth "X" of frost penetration, disturbance, fill soils or depth of pre-bore
1) d = pile diameter (m)
2) z = pile depth (m)

It should be understood that using the lateral subgrade reaction modulus assumes a linear response to lateral loading and therefore is only appropriate under the following conditions:

- maximum pile deflections are small (less than 1% of the pile diameter),
- loading is static (non-cyclical), and
- pile material behaves linear elastically (does not reach yield conditions).

If one or more of these conditions are not met, a more rigorous analysis that includes non-linear behavior of the piles and surrounding soil is required. In this regard, as part of detailed design, a lateral pile analysis that incorporates the material and section properties of the piles, final lateral deflection criteria and a more realistic elastic-plastic model of the soil response to loading should be carried out by TREK once the final design grades are determined to confirm the lateral load capacity of the piles.

4.6 Lateral Earth Pressures

The magnitude of lateral earth pressures from retained soil against buried structures will depend on the backfill material type, method of placing and compacting the backfill and the magnitude of horizontal deflection of the retaining wall after the backfill is placed. Cohesive soils should not be used as backfill against buried walls as these soils could generate excessive lateral earth pressures from swelling.

An active pressure coefficient (K_a) of 0.3 should be used to calculate lateral loads from free draining granular soils against retaining structures which are free to translate horizontally by at least 0.2 percent of the retaining wall height. For retaining structures which are not free to translate, an at-rest earth pressure coefficient (K_o) of 0.5 should be used. Surcharge loading should also be included in the earth pressure distribution to account for surface loads, based on the appropriate earth pressure coefficient.

Over-compaction of the backfill soils adjacent to buried walls may result in earth pressures that are considerably higher than those predicted in design. Compaction of the granular fills within about 1.5 m of the vertical walls should be conducted with a light hand operated vibrating plate compactor and the number of compaction passes should be limited to achieve a maximum of 92% of Standard Proctor Maximum Dry Density (SPMDD). Backfilling procedures should be reviewed during construction to verify that they are consistent with the design assumptions.

4.7 Foundation Inspection

CHBDC (2014) does not provide commentary on field review for construction of foundations. Section *4.2.2.3 Field Review* of the NBCC (2015) states that the designer or other suitably qualified person shall carry out a field review on:

a) continuous basis during:

   i. the construction of all deep foundation units with all pertinent information recorded for each *foundation unit*,

   ii. during the installation and removal of retaining structures and related backfilling operations,

   iii. during the placement of engineered fills that are to be used to support the *foundation units*, and

b) as-required, unless otherwise directed by the *authority having jurisdiction*,

   i. in the construction of all *shallow foundation units,* and

   ii. in excavating, dewatering and other related works

In consideration of the above and relative to this particular project, TREK is familiar with the geotechnical conditions and the basis for the foundation recommendations and can provide any design modifications deemed to be necessary should altered subsurface conditions be encountered. We recommend that TREK, as the geotechnical engineer of record, be retained to observe the installation of any foundation elements as noted in the NBCC.

4.8 Foundation Concrete

Based on TREK's experience with soils in the Winnipeg area the degree of exposure for concrete subjected to sulphate attack is classified as severe according to Table 3, CSA A23.1-14 (Concrete Materials and Methods of Concrete Construction). Accordingly, all concrete in contact with the native soil should be made with high sulphate-resistant cement (HS or HSb). Furthermore, the concrete should have a minimum specified 56 day compressive strength of 32 MPa and have a maximum water to cement ratio of 0.45 in accordance with Table 2, CSA A23.1-14 for concrete with severe sulphate exposure (S2). Concrete which may be exposed to freezing and thawing should be adequately air entrained to improve freeze-thaw durability in accordance with Table 4, CSA A23.1-14.

5.0 Temporary Excavations

Excavations must be carried out in compliance with the appropriate regulations under the Manitoba Workplace Safety and Health Act. If existing (adjacent) structures or infrastructure prevent an open excavation, TREK can provide recommendations and design parameters for shoring systems upon request.

Any open-cut excavation greater than 3 m deep (although not anticipated) must be designed and sealed by a professional engineer and should be reviewed by the geotechnical engineer of record (TREK) prior to commencement of installation. Furthermore, maintaining the stability of the excavation slopes for the duration of construction should be the responsibility of the Contractor. Stockpiles of excavated material and heavy equipment should be kept away from the edge of the excavation by a distance equal to or greater than the depth of excavation.

Dewatering measures should be completed as necessary to maintain a dry excavation and permit proper completion of the work. If seepage is encountered, it should be directed to a sump pit and pumped out of the excavation. If saturated silts and sands are encountered, shoring or slope flattening may be required. Gravel buttressing could be used in conjunction with sump pits for dewatering to prevent wet silts and sands from entering the excavation. Surface water should be diverted away from the excavation and the excavation should be backfilled as soon as possible following construction.

6.0 Closure

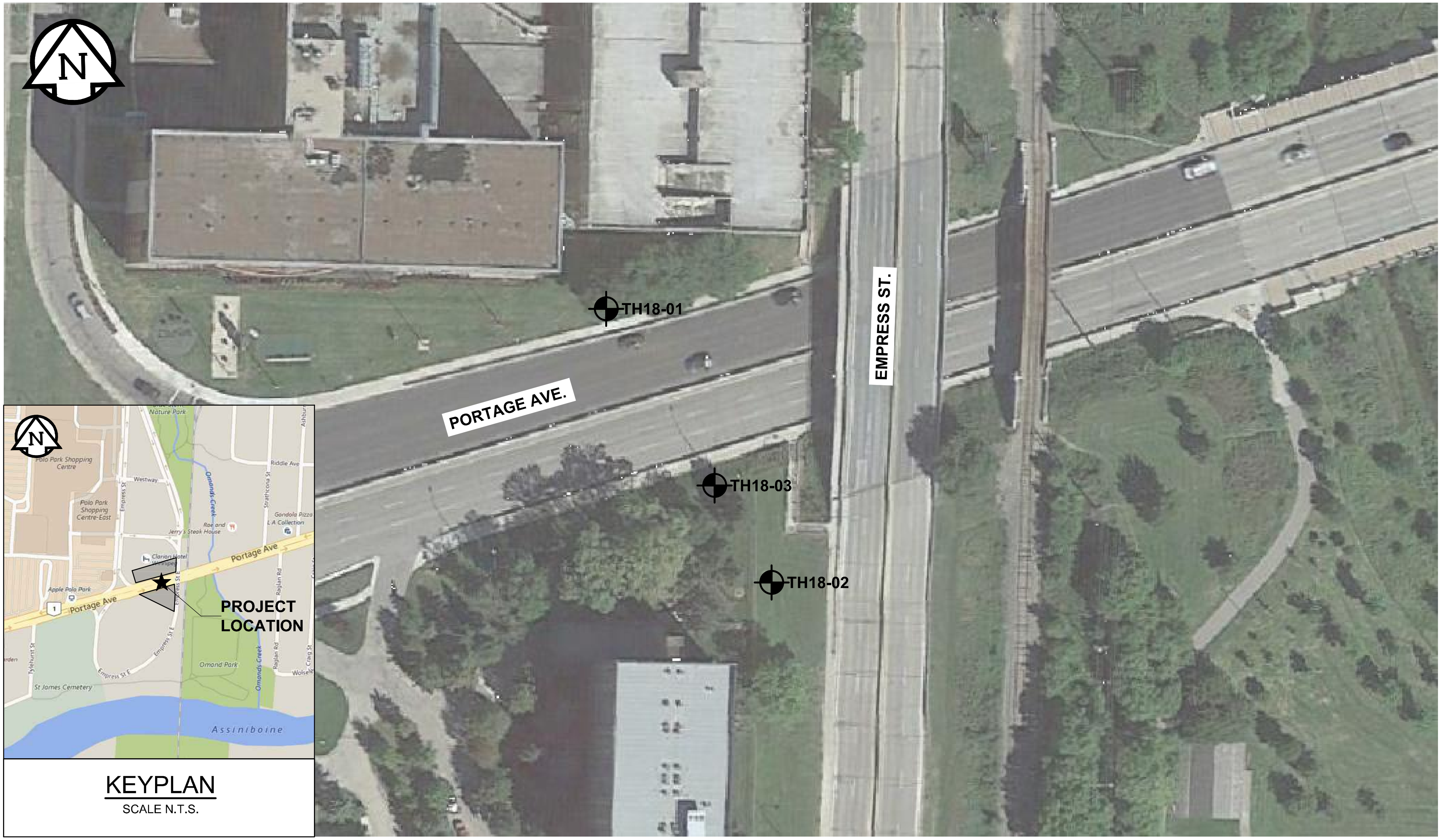
The geotechnical information provided in this report is in accordance with current engineering principles and practices (Standard of Practice). The findings of this report were based on information provided (field investigation, laboratory testing, geometries). Soil conditions are natural deposits that can be highly variable across a site. If sub-surface conditions are different than the conditions previously encountered on-site or those presented here, we should be notified to adjust our findings if necessary.

All information provided in this report is subject to our standard terms and conditions for engineering services, a copy of which is provided to each of our clients with the original scope of work, or a mutually executed standard engineering services agreement. If these conditions are not attached, and you are not already in possession of such terms and conditions, contact our office and you will be promptly provided with a copy.

This report has been prepared by TREK Geotechnical Inc. (the Consultant) for the exclusive use of Morrison Hershfield (the Client) and their agents for the work product presented in the report. Any findings or recommendations provided in this report are not to be used or relied upon by any third parties, except as agreed to in writing by the Client and Consultant prior to use.

Figures

0035 037 00
Morrison Hershfield Ltd.
Empress Overpass Pedestrian Ramp



ANSI full bleed A (11.00 x 8.50 Inches)

Fig 001 2018-03-05 Test Hole Location Plan(PED RAMP)0 A 035 037 00 SL.dwg, 3/12/2018 9:39:56 AM

0 10 20 30 40 50 m
SCALE = 1 : 1 000 (216 mm x 279 mm)

LEGEND: TEST HOLE (TREK, 2018)

NOTES: 1. IMAGE FROM GOOGLE EARTH AUGUST 24, 2015

Figure 01
Test Hole Location Plan

Test Hole Logs

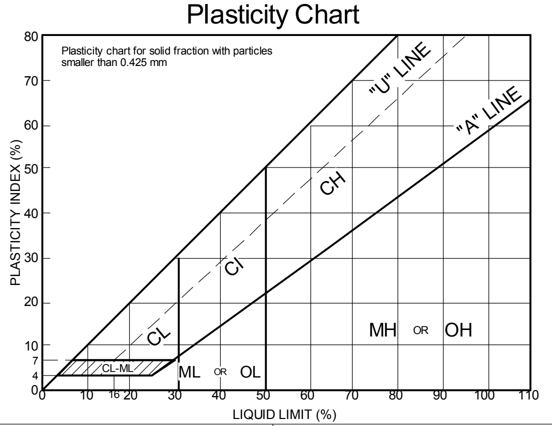
EXPLANATION OF FIELD AND LABORATORY TESTING

TREK GEOTECHNICAL

GENERAL NOTES

1. Classifications are based on the United Soil Classification System and include consistency, moisture, and color. Field descriptions have been modified to reflect results of laboratory tests where deemed appropriate.
2. Descriptions on these test hole logs apply only at the specific test hole locations and at the time the test holes were drilled. Variability of soil and groundwater conditions may exist between test hole locations.
3. When the following classification terms are used in this report or test hole logs, the primary and secondary soil fractions may be visually estimated.

Major Divisions			USCS Classification	Symbols	Typical Names	Laboratory Classification Criteria		
Coarse-Grained soils (More than half the material is larger than No. 200 sieve size)	Gravels (More than half of coarse fraction is larger than 4.75 mm)	Clean gravel (Little or no fines)	GW		Well-graded gravels, gravel-sand mixtures, little or no fines	Determine percentages of sand and gravel from grain size curve, depending on percentage of fines (fraction smaller than No. 200 sieve) coarse-grained soils are classified as follows: Less than 5 percent...... GW, GP, SW, SP; More than 12 percent...... GM, GC, SM, SC; 6 to 12 percent...... Borderline case4s requiring dual symbols*	$C_U = \frac{D_{60}}{D_{10}}$ greater than 4; $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			GP		Poorly-graded gravels, gravel-sand mixtures, little or no fines		Not meeting all gradation requirements for GW	
		Gravel with fines (Appreciable amount of fines)	GM		Silty gravels, gravel-sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4	Above "A" line with P.I. between 4 and 7 are border-line cases requiring use of dual symbols
			GC		Clayey gravels, gravel-sand-silt mixtures		Atterberg limits above "A" line or P.I. greater than 7	
	Sands (More than half of coarse fraction is smaller than 4.75 mm)	Clean sands (Little or no fines)	SW		Well-graded sands, gravelly sands, little or no fines		$C_U = \frac{D_{60}}{D_{10}}$ greater than 6; $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			SP		Poorly-graded sands, gravelly sands, little or no fines		Not meeting all gradation requirements for SW	
		Sands with fines (Appreciable amount of fines)	SM		Silty sands, sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4	Above "A" line with P.I. between 4 and 7 are border-line cases requiring use of dual symbols
			SC		Clayey sands, sand-clay mixtures		Atterberg limits above "A" line or P.I. greater than 7	
Fine-Grained soils (More than half the material is smaller than No. 200 sieve size)	Silts and Clays (Liquid limit less than 50)		ML		Inorganic silts and very fine sands, rock floor, silty or clayey fine sands or clayey silts with slight plasticity	Plasticity Chart		
			CL		Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays			
			OL		Organic silts and organic silty clays of low plasticity			
	Silts and Clays (Liquid limit greater than 50)		MH		Inorganic silts, micaceous or distomaceous fine sandy or silty soils, organic silts			
			CH		Inorganic clays of high plasticity, fat clays			
			OH		Organic clays of medium to high plasticity, organic silts			
	Highly Organic Soils		Pt		Peat and other highly organic soils	Von Post Classification Limit	Strong colour or odour, and often fibrous texture	



Material		Particle Size mm	Particle Size ASTM Sieve sizes
Sand	Coarse	2.00 to 4.75	#10 to #4
	Medium	0.425 to 2.00	#40 to #10
	Fine	0.075 to 0.425	#200 to #40
Silt or Clay		< 0.075	< #200
Boulders		> 300	> 12 in.
Cobbles		75 to 300	3 in. to 12 in.
Gravel	Coarse	19 to 75	3/4 in. to 3 in.
	Fine	4.75 to 19	#4 to 3/4 in.

* Borderline classifications used for soils possessing characteristics of two groups are designated by combinations of groups symbols. For example; GW-GC, well-graded gravel-sand mixture with clay binder.

Other Symbol Types

	Asphalt		Bedrock (undifferentiated)		Cobbles
	Concrete		Limestone Bedrock		Boulders and Cobbles
	Fill		Cemented Shale		Silt Till
			Non-Cemented Shale		Clay Till

EXPLANATION OF FIELD AND LABORATORY TESTING

LEGEND OF ABBREVIATIONS AND SYMBOLS

LL - Liquid Limit (%)
PL - Plastic Limit (%)
PI - Plasticity Index (%)
MC - Moisture Content (%)
SPT - Standard Penetration Test
RQD - Rock Quality Designation
Qu - Unconfined Compression
Su - Undrained Shear Strength
VW - Vibrating Wire Piezometer
SI - Slope Inclinometer

▽ Water Level at Time of Drilling

▼ Water Level at End of Drilling

▼ Water Level After Drilling as Indicated on Test Hole Logs

FRACTION OF SECONDARY SOIL CONSTITUENTS ARE BASED ON THE FOLLOWING TERMINOLOGY

TERM	EXAMPLES	PERCENTAGE
and	and CLAY	35 to 50 percent
"y" or "ey"	clayey, silty	20 to 35 percent
some	some silt	10 to 20 percent
trace	trace gravel	1 to 10 percent

TERMS DESCRIBING CONSISTENCY OR COMPACTION CONDITION

The Standard Penetration Test blow count (N) of a non-cohesive soil can be related to compactness condition as follows:

Descriptive Terms	SPT (N) (Blows/300 mm)
Very loose	< 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	> 50

The Standard Penetration Test blow count (N) of a cohesive soil can be related to its consistency as follows:

Descriptive Terms	SPT (N) (Blows/300 mm)
Very soft	< 2
Soft	2 to 4
Firm	4 to 8
Stiff	8 to 15
Very stiff	15 to 30
Hard	> 30

The undrained shear strength (Su) of a cohesive soil can be related to its consistency as follows:

Descriptive Terms	Undrained Shear Strength (kPa)
Very soft	< 12
Soft	12 to 25
Firm	25 to 50
Stiff	50 to 100
Very stiff	100 to 200
Hard	> 200

EXPLANATION OF ROCK CLASSIFICATION

(Canadian Foundation Engineering Manual, 4th Edition, 2006)

Grade*	Term	Uniaxial Comp. Strength (MPa)	Point Load Index (MPa)	Field Estimate of Strength	Examples
R6	Extremely strong	>250	>10	Specimen can only be chipped with a geological hammer	Fresh basalt, chert, diabase, gneiss, granite, quartzite
R5	Very strong	100-250	4-10	Specimen requires many blows of a geological hammer to fracture it	Amphibolite, sandstone, basalt, gabbro, gneiss, granodiorite, peridotite, rhyolite, tuff
R4	Strong	50-100	2-4	Specimen requires more than one blow of a geological hammer to fracture it	Limestone, marble, sandstone, schist
R3	Medium Strong	25-50	1-2	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with a single blow from a geological hammer	Concrete, phyllite, schist, siltstone
R2	Weak	5-25	***	Can be peeled with a pocket knife with difficulty, shallow indentation made by a firm blow with the point of a geological hammer	Chalk, claystone, potash, marl, siltstone, shale, rocksalt
R1	Very weak	1-5	***	Crumbles under firm blows with point of a geological hammer, can be peeled with a pocket knife	Highly weathered or altered rock, shale
R0	Extremely weak	0.25-1	***	Indented by thumbnail	Stiff fault gouge

* Grade according to ISRM (1981).

** All rock types exhibit a broad range of uniaxial comprehensive strengths reflecting heterogeneity in composition and anisotropy in structure. Strong rocks are characterized by well-interlocked crystal fabric and few voids.

*** Rocks with a uniaxial compressive strength below 25 MPa are likely to yield highly ambiguous results under point load testing.

TREK GEOTECHNICAL

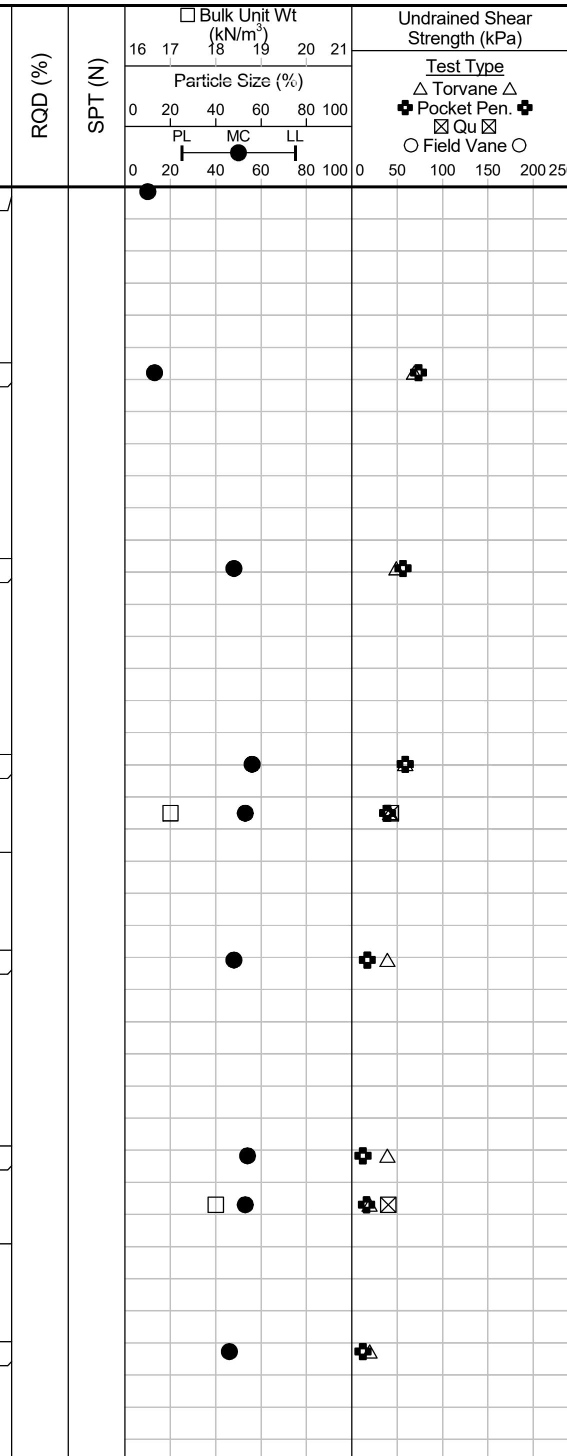
Sub-Surface Log

Client:	Morrison Hershfield	**Project Number:**	0035 037 00
Project Name:	Empress Pedestrian Ramp	**Location:**	UTM N-5527023, E-621661
Contractor:	Maple Leaf Drilling Ltd.	**Ground Elevation:**	Not Measured
Method:	125 mm Solid Stem Auger / HQ Coring, Acker MP5-T Track Mount	**Date Drilled:**	2018 February 9 - 2018 February 23

Sample Type: Grab (G) Shelby Tube (T) Split Spoon (SS) Split Barrel (SB) Core (C)

Particle Size Legend: Fines Clay Silt Sand Gravel Cobbles Boulders

Depth (m)	MATERIAL DESCRIPTION	Sample Type	Sample Number
0.0–1.2	SILT (Fill) - sandy, some gravel (<10 mm diameter), trace clay - light brown - dry, loose - no plasticity	Grab	G01
1.2–10.0	CLAY - silty, trace precipitates, trace silt inclusions(<20 mm diameter) - brown - moist, stiff - high plasticity	Grab	G02
		Grab	G03
	- grey below 4.6 m	Grab	G04
		Shelby Tube	T05
	- firm below 5.8 m	Grab	G06
		Grab	G07
		Shelby Tube	T08
	- soft to firm below 8.5 m	Grab	G09



Logged By: Jenna Roadley	**Reviewed By:** Kent Bannister	**Project Engineer:** Michael Van Helden

Sub-Surface Log

Depth (m)	Material Description	Sample Number	RQD (%)	SPT (N)
10.1	- some till inclusions below 10.1 m	G10		
10.7	SILT (Till) - some sand, trace clay, trace gravel - light brown - moist, compact to dense - low plasticity	G11		
		S12		29
11.6	- compact to dense below 11.6 m	S34		68 / 229mm
12.0	SAND - some silt, some gravel - brown - wet, compact to dense - poorly graded, coarse sand to fine gravel	G14		
		S15		50 / 149mm
		S35		50 / 88mm
13.5	SILT (Till) - some sand, some gravel, trace clay, trace cobbles - light brown - moist, very dense - low plasticity	S17		50 / 76mm
		G18		
14.6	- power auger refusal at 14.6 m, switch to HQ coring			
14.8	DOLOMITIC LIMESTONE - Red River Formation, Upper Fort Garry Member - light brown to cream - vuggy throughout - weakly calcareous, R3-R4 - weak horizontal layering, very few fractures - uniaxial compressive strength of 53.4 MPa at 14.8 m	C38	100	
		C39	78	
		C40	90	

END OF TEST HOLE AT 18.3 m IN BEDROCK
Notes:
1) Seepage observed below 12.2 m.
2) Sloughing observed below 13.1 m.
3) Test hole open to 13.1 m and water level at 7.3 m below surface before switching to HQ coring.
4) Test hole backfilled with auger cuttings and bentonite to surface.

Logged By: Jenna Roadley **Reviewed By:** Kent Bannister **Project Engineer:** Michael Van Helden

GEOTECHNICAL

Sub-Surface Log

Client:	Morrison Hershfield	**Project Number:**	0035 037 00
Project Name:	Empress Pedestrian Ramp	**Location:**	UTM N-5526975, E-629690
Contractor:	Maple Leaf Drilling Ltd.	**Ground Elevation:**	Not Measured
Method:	125 mm Solid Stem Auger / HQ Coring, Acker MP5-T Track Mount	**Date Drilled:**	2018 February 9

Sample Type: Grab (G) | Shelby Tube (T) | Split Spoon (SS) | Split Barrel (SB) | Core (C)

Particle Size Legend: Fines | Clay | Silt | Sand | Gravel | Cobbles | Boulders

Column headings: Depth (m) | Soil Symbol | MATERIAL DESCRIPTION | Sample Type | Sample Number | RQD (%) | SPT (N) | ☐ Bulk Unit Wt (kN/m^3) 16 17 18 19 20 21; Particle Size (%) 0 20 40 60 80 100; PL MC LL 0 20 40 60 80 100 | Undrained Shear Strength (kPa): Test Type △ Torvane △, ✤ Pocket Pen. ✤, ⊠ Qu ⊠, ○ Field Vane ○; 0 50 100 150 200 250

Depth (m)	MATERIAL DESCRIPTION	Samples
0.0 – 1.5	SAND (Fill) - silty, trace gravel - light brown - moist, loose to compact - poorly graded, fine sand, trace coarse sand	G19, G20
1.5 – 2.1	CLAY (Fill) - silty, trace sand, trace gravel - blackish grey - moist, very stiff, high plasticity	G21
2.1 – 2.7	SILT - some clay - light brown - moist, soft, low to intermediate plasticity	G22
2.7 – 6.7	CLAY - silty - mottled brown and grey - moist, stiff to very stiff - high plasticity	G23, G24, T25, G26, T27

At G23 (about 3.0 m) the chart marks ">>" beside the Torvane symbol.

END OF TEST HOLE AT 6.7 m DEPTH IN CLAY
Notes:
1) No seepage or sloughing observed.
2) Test hole open to 6.7 m and dry upon completion of drilling.
3) Test hole backfilled with auger cuttings and bentonite to surface.

Logged By: Jenna Roadley **Reviewed By:** Kent Bannister **Project Engineer:** Michael Van Helden

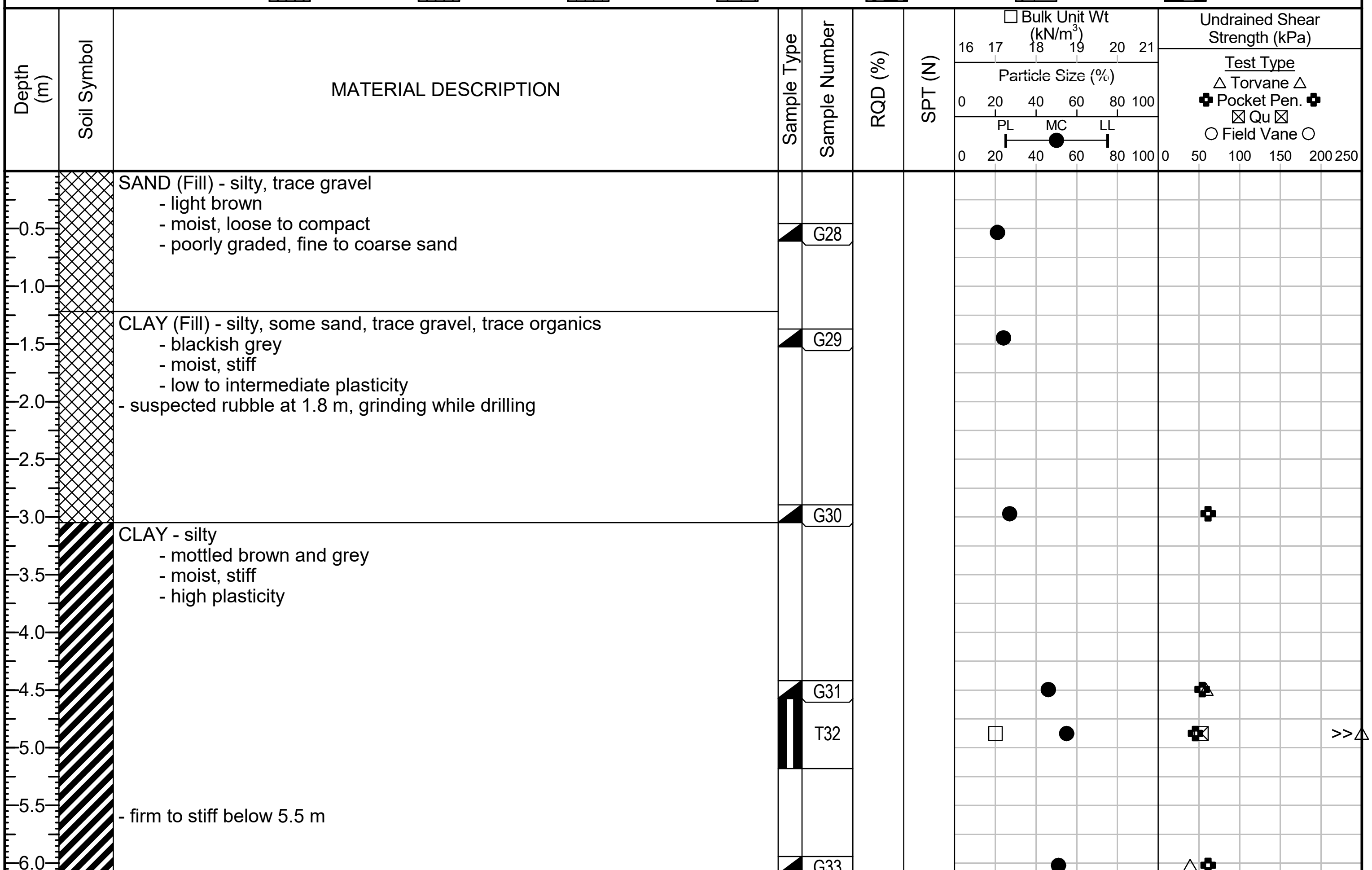
Sub-Surface Log

TREK GEOTECHNICAL

Client:	Morrison Hershfield	**Project Number:**	0035 037 00
Project Name:	Empress Pedestrian Ramp	**Location:**	UTM N-5526992, E-629680
Contractor:	Maple Leaf Drilling Ltd.	**Ground Elevation:**	Not Measured
Method:	125 mm Solid Stem Auger / HQ Coring, Acker MP5-T Track Mount	**Date Drilled:**	2018 February 9

Sample Type: Grab (G) Shelby Tube (T) Split Spoon (SS) Split Barrel (SB) Core (C)

Particle Size Legend: Fines Clay Silt Sand Gravel Cobbles Boulders



END OF TEST HOLE AT 6.1 m DEPTH IN CLAY

Notes:

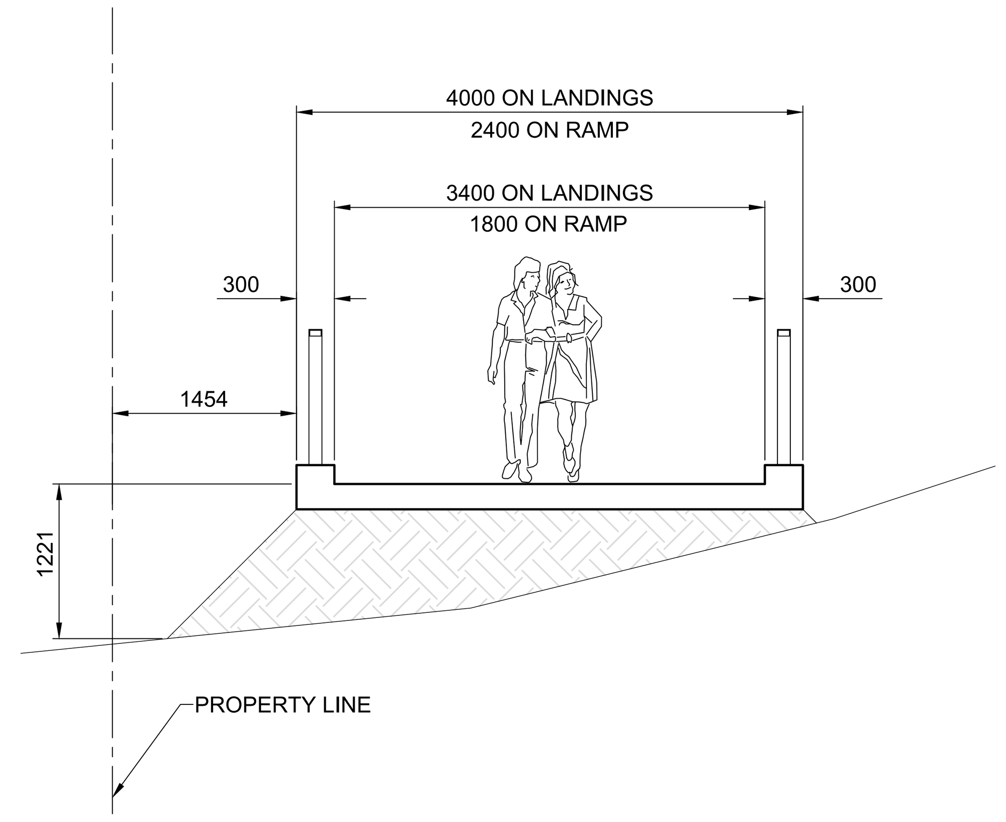
1) No seepage or sloughing observed.
2) Test hole open to 6.1 m and dry upon completion of drilling.
3) Test hole backfilled with auger cuttings and bentonite to surface.

Logged By: Jenna Roadley **Reviewed By:** Kent Bannister **Project Engineer:** Michael Van Helden

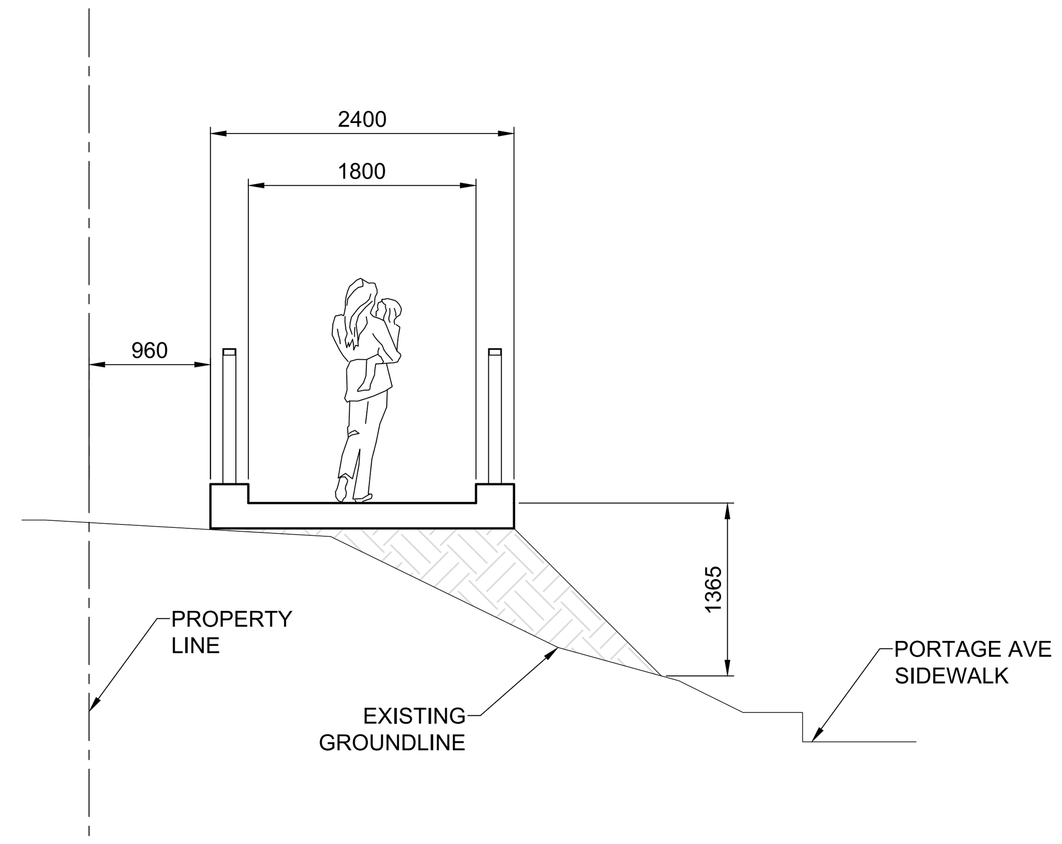
Appendix A

Existing Information

ISO A1 - 594mm x 841mm



A SECTION - SLAB ON GRADE
SCALE: 1:50



B SECTION - SLAB ON GRADE
SCALE: 1:50

PLAN VIEW ALTERNATIVE #4
SCALE 1:200

LEGEND

PHONE

POWER

WATER

FENCE

GUARDRAIL

MAINTENANCE VEHICLE PATH

NOTE:
1. FOR SECTIONS OF ELEVATED PORTIONS OF RAMPS SEE SHEETS No. 7 & 8.

PRELIMINARY
NOT FOR CONSTRUCTION

ENGINEERS GEOSCIENTISTS MANITOBA
Certificate of Authorization
MORRISON HERSHFIELD
No. 1736

METRIC
WHOLE NUMBERS INDICATE MILLIMETRES
DECIMALIZED NUMBERS INDICATE METRES

LOCATION APPROVED
UNDERGROUND STRUCTURES

SUPR. U/G STRUCTURES COMMITTEE | DATE

NOTE:
LOCATION OF UNDERGROUND STRUCTURES AS SHOWN ARE BASED ON THE BEST INFORMATION AVAILABLE BUT NO GUARANTEE IS GIVEN THAT ALL EXISTING UTILITIES ARE SHOWN OR THAT THE GIVEN LOCATIONS ARE EXACT. CONFIRMATION OF EXISTENCE AND EXACT LOCATION OF ALL SERVICES MUST BE OBTAINED FROM THE INDIVIDUAL UTILITIES BEFORE PROCEEDING WITH CONSTRUCTION.

BM
ELEV

No.	REVISIONS	YY/MM/DD	BY
A	ISSUED FOR CLIENT REVIEW	18/01/23	ALP

MORRISON HERSHFIELD

DESIGNED BY	ALP	CHECKED BY	BAP
DRAWN BY	ALP	APPROVED BY	AF
HOR SCALE	AS SHOWN	RELEASED FOR CONSTRUCTION	
VERT SCALE	AS SHOWN		
DATE		DATE	

PROFESSIONAL'S SEAL

CONSULTANT FILE NAME
W160034 - GEOMETRY.DWG

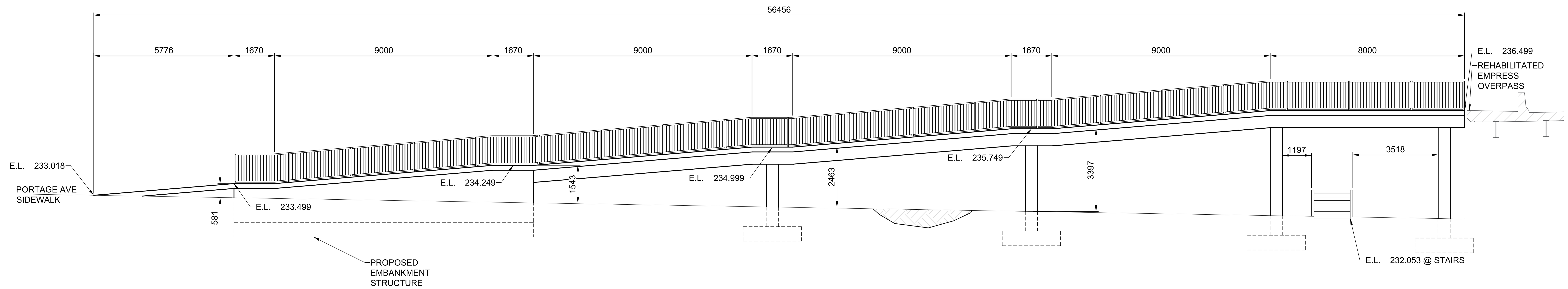
Winnipeg
THE CITY OF WINNIPEG
PUBLIC WORKS DEPARTMENT
ENGINEERING DIVISION

EMPRESS OVERPASS BRIDGE
PEDESTRIAN RAMPS

PLAN VIEW OF ALTERNATIVES

CITY DRAWING NUMBER: N/A
SHEET 1 OF 2
DRAWING No. 01
REV A

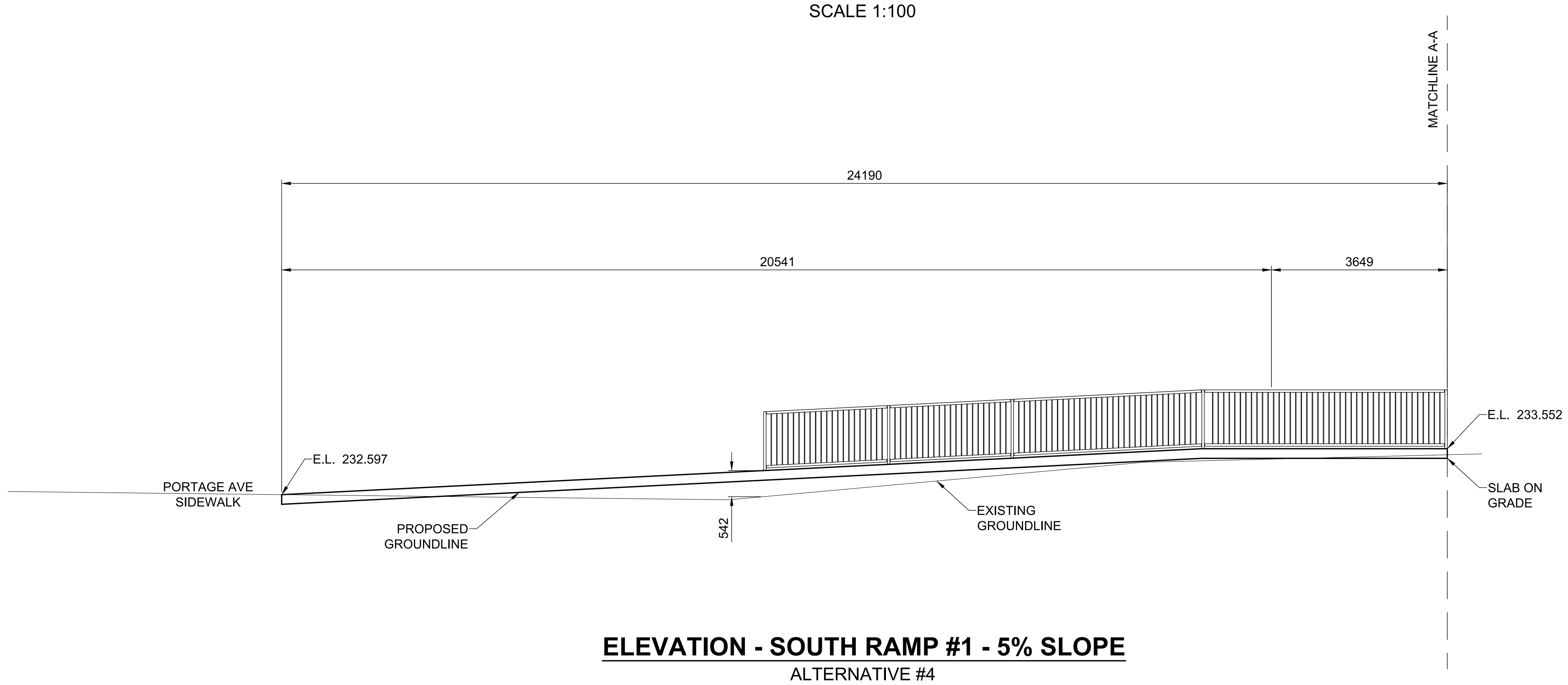
O:\Winnipeg\Proj\W160034\4 Design\Structural\Ramps\W160034 - Geometry.dwg Last Saved: 1/22/2018 4:36 PM by ALazcanoPerez Plotted: 1/23/2018 9:48 AM by Andrei Lazcano Perez



ELEVATION - NORTH RAMP 8.3% WITH INTERMEDIATE LANDINGS

ALTERNATIVE #4

SCALE 1:100



ELEVATION - SOUTH RAMP #1 - 5% SLOPE

ALTERNATIVE #4

SCALE 1:75

ELEVATION - SOUTH RAMP #2 - 5% SLOPE

ALTERNATIVE #4

SCALE 1:75

PRELIMINARY
NOT FOR CONSTRUCTION

ENGINEERS GEOSCIENTISTS MANITOBA
Certificate of Authorization
MORRISON HERSHFIELD
No. 1736

METRIC
WHOLE NUMBERS INDICATE MILLIMETRES
DECIMALIZED NUMBERS INDICATE METRES

LOCATION APPROVED
UNDERGROUND STRUCTURES

SUPR. U/G STRUCTURES COMMITTEE | DATE

NOTE:
LOCATION OF UNDERGROUND STRUCTURES AS SHOWN ARE BASED ON THE BEST INFORMATION AVAILABLE BUT NO GUARANTEE IS GIVEN THAT ALL EXISTING UTILITIES ARE SHOWN OR THAT THE GIVEN LOCATIONS ARE EXACT. CONFIRMATION OF EXISTENCE AND EXACT LOCATION OF ALL SERVICES MUST BE OBTAINED FROM THE INDIVIDUAL UTILITIES BEFORE PROCEEDING WITH CONSTRUCTION.

BM
ELEV

No.	REVISIONS	YY/MM/DD	BY
A	ISSUED FOR CLIENT REVIEW	18/01/23	

MORRISON HERSHFIELD

DESIGNED BY	ALP	CHECKED BY	BAP
DRAWN BY	ALP	APPROVED BY	AF
HOR SCALE	AS SHOWN	RELEASED FOR CONSTRUCTION	
VERT SCALE	AS SHOWN		
DATE		DATE	

PROFESSIONAL'S SEAL

CONSULTANT FILE NAME
W160034 - GEOMETRY.DWG

Winnipeg

THE CITY OF WINNIPEG
PUBLIC WORKS DEPARTMENT
ENGINEERING DIVISION

EMPRESS OVERPASS BRIDGE PEDESTRIAN RAMPS

NORTH AND SOUTH RAMP ELEVATIONS ALTERNATIVE #4

CITY DRAWING NUMBER: N/A
SHEET 2 OF 2
DRAWING No. 02
REV A

ISO A1 - 594mm x 841mm

O:\Winnipeg\Proj\W160034\04 Design\Structural\Ramps\W160034 - Geometry.dwg Last Saved: 1/22/2018 4:36 PM by ALazcanoPerez Plotted: 1/23/2018 9:48 AM by Andrei Lazcano Perez

Appendix B

Laboratory Testing

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1712 St. James Street
Winnipeg, MB R3H 0L3
Tel: 204.975.9433 Fax: 204.975.9435

Moisture Content Report
ASTM D2216-10

Project No. 0035-037-00
Client Morrison Hershfield
Project Empress Pedestrain Ramp

Sample Date 9-Feb-18
Test Date 2-Mar-18
Technician DS

Test Pit	TH18-01	TH18-01	TH18-01	TH18-01	TH18-01	TH18-01
Depth (m)	0.0 - 0.1	1.4 - 1.5	2.9 - 3.0	4.4 - 4.6	5.9 - 6.1	7.5 - 7.6
Sample #	G01	G02	G03	G04	G06	G07
Tare ID	AC27	AB06	E85	F451	F86	H68
Mass of tare	6.6	6.8	8.6	8.4	8.6	8.4
Mass wet + tare	174.6	275.4	326.4	322.4	341.6	310.2
Mass dry + tare	159.0	243.8	223.8	209.8	233.8	204.6
Mass water	15.6	31.6	102.6	112.6	107.8	105.6
Mass dry soil	152.4	237.0	215.2	201.4	225.2	196.2
Moisture %	10.2%	13.3%	47.7%	55.9%	47.9%	53.8%

Test Pit	TH18-01	TH18-01	TH18-01	TH18-01	TH18-01	TH18-01
Depth (m)	9.0 - 9.1	10.1 - 10.2	10.5 - 10.7	10.7 - 11.1	11.4 - 11.9	11.6 - 11.7
Sample #	G09	G10	G11	S12	S34	G13
Tare ID	Z36	P12	F109	Z40	W42	N47
Mass of tare	8.6	8.4	8.6	8.6	8.4	8.4
Mass wet + tare	344.2	315	318	301.6	308.4	331.8
Mass dry + tare	238.6	227.0	282.4	274.6	282.8	300.6
Mass water	105.6	88.0	35.6	27.0	25.6	31.2
Mass dry soil	230.0	218.6	273.8	266.0	274.4	292.2
Moisture %	45.9%	40.3%	13.0%	10.2%	9.3%	10.7%

Test Pit	TH18-01	TH18-01	TH18-01	TH18-01	TH18-01	TH18-02
Depth (m)	12.0 - 12.2	12.2 - 12.6	13.0 - 12.6	13.7 - 13.8	13.8 - 13.9	0.5 - 0.6
Sample #	G14	S15	G16	S17	G18	G19
Tare ID	Z47	E22	N24	F32	E81	P33
Mass of tare	8.6	8.6	8.6	8.2	8.8	8.4
Mass wet + tare	241.6	348.2	343.4	317.6	303.4	345.4
Mass dry + tare	214.4	295.4	309.8	283.6	267.6	327.6
Mass water	27.2	52.8	33.6	34.0	35.8	17.8
Mass dry soil	205.8	286.8	301.2	275.4	258.8	319.2
Moisture %	13.2%	18.4%	11.2%	12.3%	13.8%	5.6%

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Moisture Content Report
ASTM D2216-10

Project No. 0035-037-00
Client Morrison Hershfield
Project Empress Pedestrain Ramp

Sample Date 9-Feb-18
Test Date 2-Mar-18
Technician DS

Test Pit	TH18-02	TH18-02	TH18-02	TH18-02	TH18-02	TH18-02
Depth (m)	1.4 - 1.5	2.0 - 2.1	2.3 - 2.4	2.9 - 3.0	4.1 - 4.3	5.5 - 5.6
Sample #	G20	G21	G22	G23	G24	G26
Tare ID	P31	E109	K33	N28	A5	E128
Mass of tare	8.6	8.6	8.6	8.2	8.0	8.4
Mass wet + tare	289.6	317.8	327.0	323.8	293.2	303.0
Mass dry + tare	249.0	240.4	269.4	250.0	196.0	197.0
Mass water	40.6	77.4	57.6	73.8	97.2	106.0
Mass dry soil	240.4	231.8	260.8	241.8	188.0	188.6
Moisture %	16.9%	33.4%	22.1%	30.5%	51.7%	56.2%

Test Pit	TH18-03	TH18-03	TH18-03	TH18-03	TH18-03	
Depth (m)	0.5 - 0.6	1.4 - 1.5	2.9 - 3.0	4.4 - 4.6	5.9 - 6.1	
Sample #	G28	G29	G30	G31	G33	
Tare ID	D18	K30	C11	W69	AB45	
Mass of tare	8.6	8.6	8.2	8.4	6.8	
Mass wet + tare	283.2	302.8	315.8	302.6	295.2	
Mass dry + tare	235.4	246.6	250.4	210.4	198.4	
Mass water	47.8	56.2	65.4	92.2	96.8	
Mass dry soil	226.8	238.0	242.2	202.0	191.6	
Moisture %	21.1%	23.6%	27.0%	45.6%	50.5%	

Test Pit						
Depth (m)						
Sample #						
Tare ID						
Mass of tare						
Mass wet + tare						
Mass dry + tare						
Mass water						
Mass dry soil						
Moisture %						

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Shelby Tube Visual

Project No.	0035-037-00
Client	Morrison Hershfield
Project	Empress Pedestrain Ramp

Test Hole	TH18-01
Sample #	T05
Depth (m)	4.6 - 5.3
Sample Date	9-Feb-18
Test Date	1-Mar-17
Technician	DS

Tube Extraction

Recovery (mm) 710 Over Push

Bottom - 5.3 m	5.17 m	5.09 m	4.93 m	4.80m	4.65 m	**Top - 4.6 m**
PP Tv	Moisture Content Visual	Qu Bulk	Moisture Content Visual	Keep	Slough	
140 mm	80 mm	160 mm	130 mm	150 mm	50 mm	

Visual Classification

Material	Clay
Composition	silty
trace silt inclusions (<10mm diam.)	
Color	mottled grey and brown
Moisture	moist
Consistency	firm
Plasticity	high plasticity
Structure	
Gradation	

Torvane

Reading	0.43
Vane Size (s,m,l)	m
Undrained Shear Strength (kPa)	42.2

Pocket Penetrometer

Reading	**1**	0.80
	2	0.80
	3	0.80
	Average	0.80
Undrained Shear Strength (kPa)		39.2

Moisture Content

Tare ID	N37
Mass tare (g)	8.4
Mass wet + tare (g)	386.9
Mass dry + tare (g)	256
Moisture %	52.9%

Unit Weight

Bulk Weight (g)		1005.0
Length (mm)	**1**	147.67
	2	148.06
	3	147.58
	4	147.66
Average Length (m)		0.148
Diam. (mm)	**1**	70.82
	2	70.61
	3	71.19
	4	71.07
Average Diameter (m)		0.071
Volume (m^3)		5.84E-04
Bulk Unit Weight (kN/m^3)		16.9
Bulk Unit Weight (pcf)		107.5
Dry Unit Weight (kN/m^3)		11.0
Dry Unit Weight (pcf)		70.3

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Unconfined Compressive Strength
ASTM D2166

Project No.	0035-037-00
Client	Morrison Hershfield
Project	Empress Pedestrain Ramp
Test Hole	TH18-01
Sample #	T05
Depth (m)	4.6 - 5.3
Sample Date	9-Feb-18
Test Date	1-Mar-17
Technician	DS

Unconfined Strength

	kPa	ksf
Max q_u	86.2	1.8
Max S_u	43.1	0.9

Specimen Data

Description Clay - silty, trace silt inclusions (<10mm diam.), mottled grey and brown, moist, firm, high plasticity,

Length	147.7	(mm)	**Moisture %**	53%	
Diameter	70.9	(mm)	**Bulk Unit Wt.**	16.9	(kN/m^3)
L/D Ratio	2.1		**Dry Unit Wt.**	11.0	(kN/m^3)
Initial Area	0.00395	(m^2)	**Liquid Limit**	-	
Load Rate	1.00	(%/min)	**Plastic Limit**	-	
			Plasticity Index	-	

Undrained Shear Strength Tests

Torvane

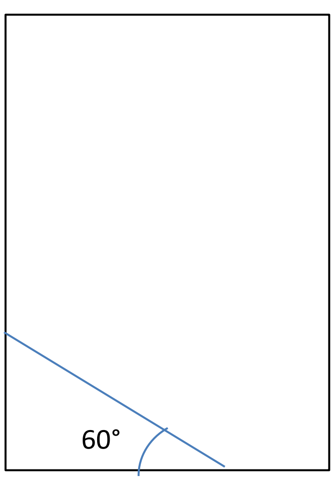
Reading	Undrained Shear Strength	
tsf	**kPa**	**ksf**
0.43	42.2	0.88
Vane Size		
m		

Pocket Penetrometer

	Reading	Undrained Shear Strength	
	tsf	**kPa**	**ksf**
	0.80	39.2	0.82
	0.80	39.2	0.82
	0.80	39.2	0.82
Average	**0.80**	**39.2**	**0.82**

Failure Geometry

Sketch:



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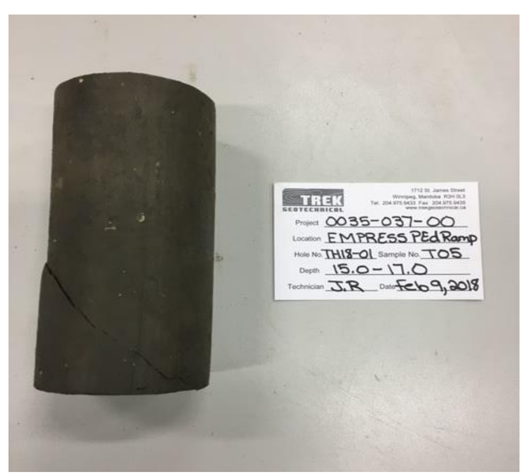

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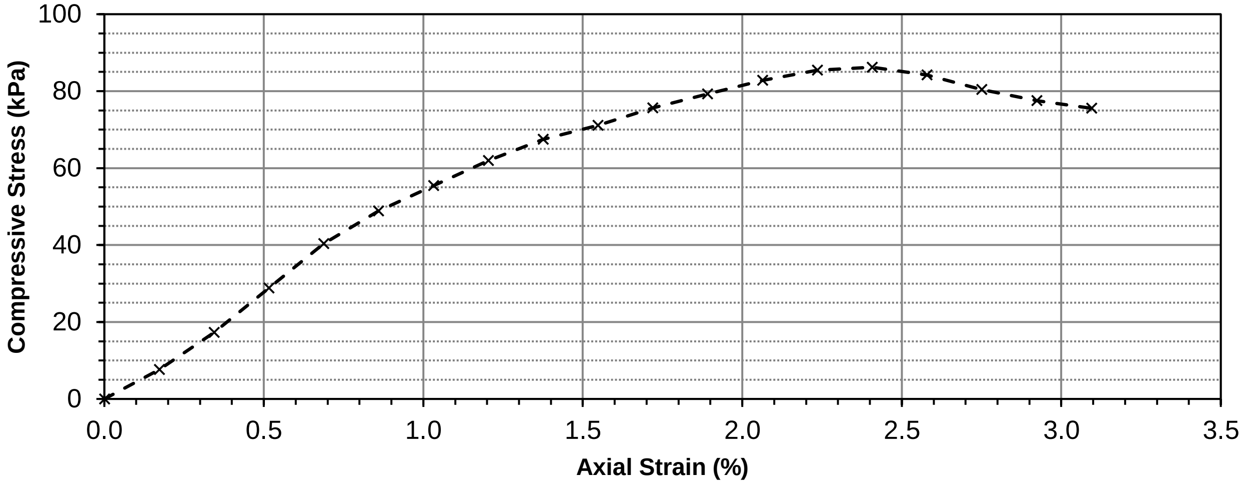
Unconfined Compressive Strength
ASTM D2166

Project No. 0035-037-00
Client Morrison Hershfield
Project Empress Pedestrain Ramp

Unconfined Compression Test Graph



Unconfined Compression Test Data

Deformation Dial Reading	Load Ring Dial Reading	Deflection (mm)	Axial Strain (%)	Corrected Area (m^2)	Axial Load (N)	Compressive Stress, q_u (kPa)	Shear Stress, S_u (kPa)
0	0	0.0000	0.00	0.003951	0.0	0.00	0.00
10	8	0.2540	0.17	0.003957	30.2	7.64	3.82
20	18	0.5080	0.34	0.003964	68.5	17.28	8.64
30	30	0.7620	0.52	0.003971	114.4	28.82	14.41
40	42	1.0160	0.69	0.003978	160.4	40.32	20.16
50	51	1.2700	0.86	0.003985	194.8	48.88	24.44
60	58	1.5240	1.03	0.003992	221.2	55.42	27.71
70	65	1.7780	1.20	0.003999	247.6	61.93	30.97
80	71	2.0320	1.38	0.004006	270.3	67.48	33.74
90	75	2.2860	1.55	0.004013	285.4	71.13	35.56
100	80	2.5400	1.72	0.004020	304.1	75.65	37.83
110	84	2.7940	1.89	0.004027	319.1	79.24	39.62
120	88	3.0480	2.06	0.004034	334.0	82.81	41.40
130	91	3.3020	2.23	0.004041	345.2	85.44	42.72
140	92	3.5560	2.41	0.004048	349.0	86.21	43.11
150	90	3.8100	2.58	0.004055	341.5	84.21	42.11
160	86	4.0640	2.75	0.004062	326.5	80.38	40.19
170	83	4.3180	2.92	0.004069	315.3	77.48	38.74
180	81	4.5720	3.09	0.004077	307.8	75.51	37.76

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Shelby Tube Visual

Project No.	0035-037-00
Client	Morrison Hershfield
Project	Empress Pedestrain Ramp
Test Hole	TH18-01
Sample #	T08
Depth (m)	7.6 - 8.3
Sample Date	09-Feb-18
Test Date	01-Mar-17
Technician	DS

Tube Extraction

Recovery (mm)	670	Over Push

Bottom - 8.3 m	8.11 m	7.95 m	7.82m	7.65 m	**Top - 7.6 m**
Keep	Qu Bulk	PP Tv	Moisture Content Visual	Slough	
160 mm	160 mm	130 mm	170 mm	50 mm	

Visual Classification

Material	Clay
Composition	silty
trace coarse sand	
trace gravel (<15 mm diam.)	
trace silt inclusions (<5 mm diam.)	
Color	grey
Moisture	moist
Consistency	soft
Plasticity	high plasticity
Structure	
Gradation	

Torvane

Reading	0.20
Vane Size (s,m,l)	m
Undrained Shear Strength (kPa)	19.6

Pocket Penetrometer

Reading	**1**	0.30
	2	0.30
	3	0.40
	Average	0.33
Undrained Shear Strength (kPa)		16.3

Moisture Content

Tare ID	P20
Mass tare (g)	8.7
Mass wet + tare (g)	413.4
Mass dry + tare (g)	272.6
Moisture %	53.4%

Unit Weight

Bulk Weight (g)		1073.7
Length (mm)	**1**	146.75
	2	146.13
	3	145.58
	4	145.83
Average Length (m)		0.146
Diam. (mm)	**1**	71.61
	2	71.75
	3	71.37
	4	71.29
Average Diameter (m)		0.072
Volume (m^3)		5.87E-04
Bulk Unit Weight (kN/m^3)		18.0
Bulk Unit Weight (pcf)		114.3
Dry Unit Weight (kN/m^3)		11.7
Dry Unit Weight (pcf)		74.5

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Unconfined Compressive Strength

ASTM D2166

Project No.	0035-037-00
Client	Morrison Hershfield
Project	Empress Pedestrain Ramp
Test Hole	TH18-01
Sample #	T08
Depth (m)	7.6 - 8.3
Sample Date	9-Feb-18
Test Date	1-Mar-17
Technician	DS

Unconfined Strength

	kPa	ksf
Max q_u	80.5	1.7
Max S_u	40.2	0.8

Specimen Data

Description Clay - silty, trace coarse sand, trace gravel (<15 mm diam.), trace silt inclusions (<5 mm diam.), grey, moist, soft, high plasticity,

Length	146.1	(mm)	**Moisture %**	53%	
Diameter	71.5	(mm)	**Bulk Unit Wt.**	18.0	(kN/m^3)
L/D Ratio	2.0		**Dry Unit Wt.**	11.7	(kN/m^3)
Initial Area	0.00402	(m^2)	**Liquid Limit**	-	
Load Rate	1.00	(%/min)	**Plastic Limit**	-	
			Plasticity Index	-	

Undrained Shear Strength Tests

Torvane

Reading	Undrained Shear Strength	
tsf	**kPa**	**ksf**
0.20	19.6	0.41
Vane Size		
m		

Pocket Penetrometer

	Reading	Undrained Shear Strength	
	tsf	**kPa**	**ksf**
	0.30	14.7	0.31
	0.30	14.7	0.31
	0.40	19.6	0.41
Average	**0.33**	**16.4**	**0.34**

Failure Geometry

Sketch:

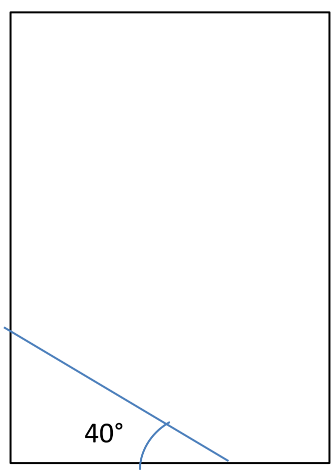

Photo:

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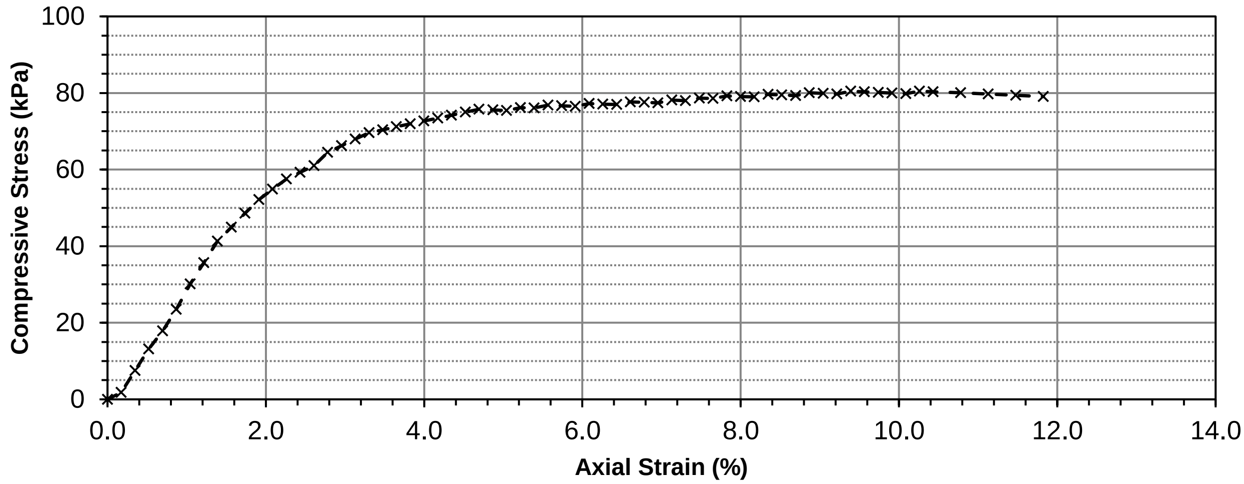
Unconfined Compressive Strength

ASTM D2166

Project No. 0035-037-00
Client Morrison Hershfield
Project Empress Pedestrain Ramp

Unconfined Compression Test Graph



Unconfined Compression Test Data

Deformation Dial Reading	Load Ring Dial Reading	Deflection (mm)	Axial Strain (%)	Corrected Area (m^2)	Axial Load (N)	Compressive Stress, q_u (kPa)	Shear Stress, S_u (kPa)
0	0	0.0000	0.00	0.004016	0.0	0.00	0.00
10	2	0.2540	0.17	0.004023	7.3	1.80	0.90
20	8	0.5080	0.35	0.004030	30.2	7.50	3.75
30	14	0.7620	0.52	0.004037	53.2	13.18	6.59
40	19	1.0160	0.70	0.004044	72.3	17.89	8.94
50	25	1.2700	0.87	0.004051	95.3	23.53	11.76
60	32	1.5240	1.04	0.004058	122.1	30.09	15.04
70	38	1.7780	1.22	0.004065	145.1	35.68	17.84
80	44	2.0320	1.39	0.004072	168.0	41.26	20.63
90	48	2.2860	1.56	0.004080	183.3	44.94	22.47
100	52	2.5400	1.74	0.004087	198.6	48.58	24.29
110	56	2.7940	1.91	0.004094	213.7	52.19	26.09
120	59	3.0480	2.09	0.004101	225.0	54.86	27.43
130	62	3.3020	2.26	0.004109	236.3	57.52	28.76
140	64	3.5560	2.43	0.004116	243.9	59.25	29.62
150	66	3.8100	2.61	0.004123	251.4	60.98	30.49
160	70	4.0640	2.78	0.004131	266.5	64.52	32.26
170	72	4.3180	2.96	0.004138	274.1	66.23	33.12
180	74	4.5720	3.13	0.004145	281.6	67.94	33.97
190	76	4.8260	3.30	0.004153	289.1	69.62	34.81
200	77	5.0800	3.48	0.004160	292.9	70.40	35.20
210	78	5.3340	3.65	0.004168	296.6	71.17	35.58
220	79	5.5880	3.83	0.004175	300.4	71.93	35.97
230	80	5.8420	4.00	0.004183	304.1	72.70	36.35

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Unconfined Compressive Strength
ASTM D2166

Project No. 0035-037-00
Client Morrison Hershfield
Project Empress Pedestrain Ramp

Unconfined Compression Test Data (cont'd)

Deformation Dial Reading	Load Ring Dial Reading	Deflection (mm)	Axial Strain (%)	Corrected Area (m^2)	Axial Load (N)	Compressive Stress, q_u (kPa)	Shear Stress, S_u (kPa)
240	81	6.0960	4.17	0.004191	307.8	73.46	36.73
250	82	6.3500	4.35	0.004198	311.6	74.22	37.11
260	83	6.6040	4.52	0.004206	315.3	74.97	37.49
270	84	6.8580	4.69	0.004214	319.1	75.72	37.86
280	84	7.1120	4.87	0.004221	319.1	75.58	37.79
290	84	7.3660	5.04	0.004229	319.1	75.45	37.72
300	85	7.6200	5.22	0.004237	322.8	76.19	38.10
310	85	7.8740	5.39	0.004245	322.8	76.05	38.03
320	86	8.1280	5.56	0.004252	326.5	76.79	38.40
330	86	8.3820	5.74	0.004260	326.5	76.65	38.32
340	86	8.6360	5.91	0.004268	326.5	76.51	38.25
350	87	8.8900	6.09	0.004276	330.3	77.24	38.62
360	87	9.1440	6.26	0.004284	330.3	77.10	38.55
370	87	9.3980	6.43	0.004292	330.3	76.96	38.48
380	88	9.6520	6.61	0.004300	334.0	77.68	38.84
390	88	9.9060	6.78	0.004308	334.0	77.54	38.77
400	88	10.1600	6.96	0.004316	334.0	77.39	38.70
410	89	10.4140	7.13	0.004324	337.8	78.11	39.06
420	89	10.6680	7.30	0.004332	337.8	77.97	38.98
430	90	10.9220	7.48	0.004340	341.5	78.68	39.34
440	90	11.1760	7.65	0.004348	341.5	78.53	39.27
450	91	11.4300	7.82	0.004357	345.2	79.25	39.62
460	91	11.6840	8.00	0.004365	345.2	79.10	39.55
470	91	11.9380	8.17	0.004373	345.2	78.95	39.47
480	92	12.1920	8.35	0.004381	349.0	79.65	39.83
490	92	12.4460	8.52	0.004390	349.0	79.50	39.75
500	92	12.7000	8.69	0.004398	349.0	79.35	39.67
510	93	12.9540	8.87	0.004406	352.7	80.05	40.02
520	93	13.2080	9.04	0.004415	352.7	79.89	39.95
530	93	13.4620	9.22	0.004423	352.7	79.74	39.87
540	94	13.7160	9.39	0.004432	356.5	80.43	40.22
550	94	13.9700	9.56	0.004440	356.5	80.28	40.14
560	94	14.2240	9.74	0.004449	356.5	80.12	40.06
570	94	14.4780	9.91	0.004458	356.5	79.97	39.98
580	94	14.7320	10.09	0.004466	356.5	79.81	39.91
590	95	14.9860	10.26	0.004475	360.2	80.50	40.25
600	95	15.2400	10.43	0.004483	360.2	80.34	40.17
620	95	15.7480	10.78	0.004501	360.2	80.03	40.01
640	95	16.2560	11.13	0.004519	360.2	79.72	39.86
660	95	16.7640	11.48	0.004536	360.2	79.40	39.70
680	95	17.2720	11.82	0.004554	360.2	79.09	39.55

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Shelby Tube Visual

Project No.	0035-037-00
Client	Morrison Hershfield
Project	Empress Pedestrain Ramp
Test Hole	TH18-02
Sample #	T27
Depth (m)	6.1 - 6.7
Sample Date	09-Feb-18
Test Date	02-Mar-17
Technician	DS

Tube Extraction

Recovery (mm) 640 Over Push

Bottom - 6.7 m	6.58 m	6.42 m	6.29m	6.12 m **Top - 6.1 m**
Moisture Content Visual	Qu Bulk	PP Tv	Keep	Slough
160 mm	160 mm	130 mm	170 mm	20 mm

Visual Classification

Material	Clay
Composition	silty
trace gravel (<15 mm diam.)	
trace silt inclusions (<5 mm diam.)	
Color	mottled brown and light brown
Moisture	moist
Consistency	firm
Plasticity	high plasticity
Structure	
Gradation	

Torvane

Reading	0.38
Vane Size (s,m,l)	m
Undrained Shear Strength (kPa)	37.3

Pocket Penetrometer

Reading	**1**	0.80
	2	0.75
	3	0.75
	Average	0.77
Undrained Shear Strength (kPa)		37.6

Moisture Content

Tare ID	W71
Mass tare (g)	8.5
Mass wet + tare (g)	400.3
Mass dry + tare (g)	261
Moisture %	55.2%

Unit Weight

Bulk Weight (g)		987.6
Length (mm)	**1**	143.94
	2	144.81
	3	144.31
	4	143.66
Average Length (m)		0.144
Diam. (mm)	**1**	71.51
	2	71.52
	3	70.25
	4	71.09
Average Diameter (m)		0.071
Volume (m^3)		5.72E-04
Bulk Unit Weight (kN/m^3)		16.9
Bulk Unit Weight (pcf)		107.7
Dry Unit Weight (kN/m^3)		10.9
Dry Unit Weight (pcf)		69.4

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Unconfined Compressive Strength
ASTM D2166

Project No.	0035-037-00
Client	Morrison Hershfield
Project	Empress Pedestrain Ramp

Test Hole	TH18-02
Sample #	T27
Depth (m)	6.1 - 6.7
Sample Date	9-Feb-18
Test Date	2-Mar-17
Technician	DS

Unconfined Strength

	kPa	ksf
Max q_u	140.4	2.9
Max S_u	70.2	1.5

Specimen Data

Description Clay - silty, trace gravel (<15 mm diam.), trace silt inclusions (<5 mm diam.), mottled brown and light brown, moist, firm, high plasticity,

Length	144.2	(mm)	**Moisture %**	55%	
Diameter	71.1	(mm)	**Bulk Unit Wt.**	16.9	(kN/m^3)
L/D Ratio	2.0		**Dry Unit Wt.**	10.9	(kN/m^3)
Initial Area	0.00397	(m^2)	**Liquid Limit**	-	
Load Rate	1.00	(%/min)	**Plastic Limit**	-	
			Plasticity Index	-	

Undrained Shear Strength Tests

Torvane

Reading	Undrained Shear Strength	
tsf	**kPa**	**ksf**
0.38	37.3	0.78
Vane Size		
m		

Pocket Penetrometer

	Reading	Undrained Shear Strength	
	tsf	**kPa**	**ksf**
	0.80	39.2	0.82
	0.75	36.8	0.77
	0.75	36.8	0.77
Average	**0.77**	**37.6**	**0.79**

Failure Geometry

Sketch:

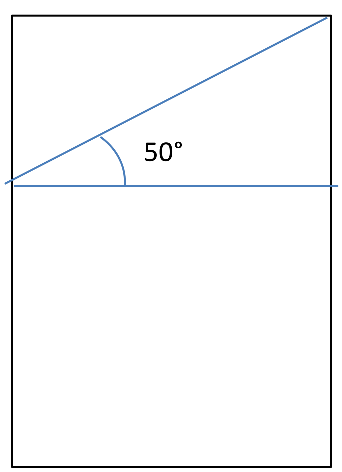

Photo:

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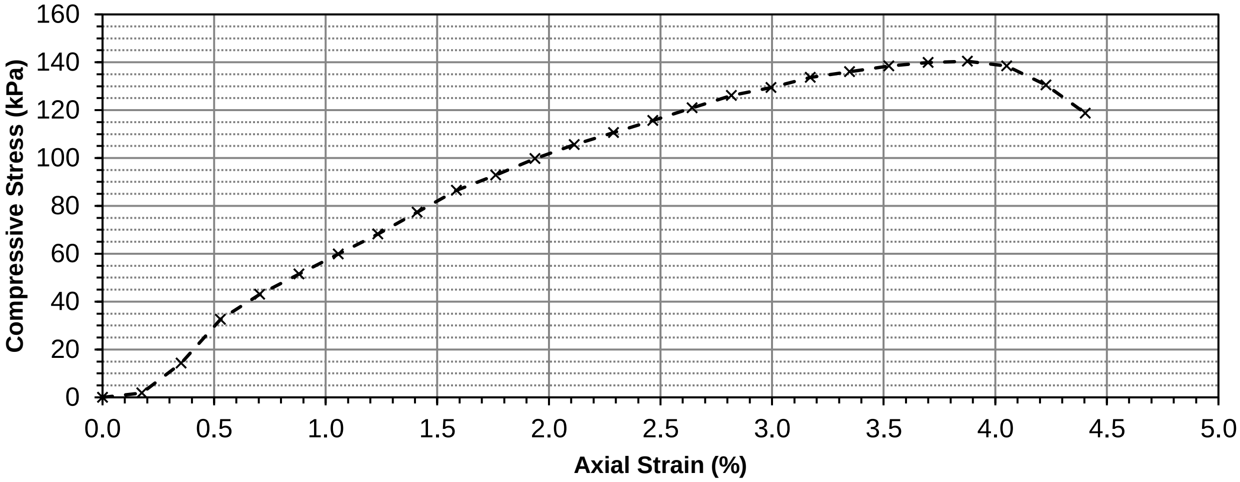
Unconfined Compressive Strength
ASTM D2166

Project No. 0035-037-00
Client Morrison Hershfield
Project Empress Pedestrain Ramp

Unconfined Compression Test Graph



Unconfined Compression Test Data

Deformation Dial Reading	Load Ring Dial Reading	Deflection (mm)	Axial Strain (%)	Corrected Area (m^2)	Axial Load (N)	Compressive Stress, q_u (kPa)	Shear Stress, S_u (kPa)
0	0	0.0000	0.00	0.003970	0.0	0.00	0.00
10	2	0.2540	0.18	0.003977	7.3	1.82	0.91
20	15	0.5080	0.35	0.003984	57.0	14.31	7.16
30	34	0.7620	0.53	0.003991	129.8	32.51	16.26
40	45	1.0160	0.70	0.003998	171.9	42.99	21.49
50	54	1.2700	0.88	0.004005	206.1	51.46	25.73
60	63	1.5240	1.06	0.004012	240.1	59.84	29.92
70	72	1.7780	1.23	0.004019	274.1	68.19	34.10
80	82	2.0320	1.41	0.004026	311.6	77.39	38.69
90	92	2.2860	1.59	0.004033	349.0	86.52	43.26
100	99	2.5400	1.76	0.004041	375.2	92.85	46.42
110	107	2.7940	1.94	0.004048	403.6	99.71	49.86
120	114	3.0480	2.11	0.004055	428.3	105.63	52.81
130	120	3.3020	2.29	0.004063	449.5	110.65	55.33
140	126	3.5560	2.47	0.004070	470.9	115.70	57.85
150	132	3.8100	2.64	0.004077	492.9	120.88	60.44
160	138	4.0640	2.82	0.004085	514.9	126.05	63.03
170	142	4.3180	2.99	0.004092	529.6	129.41	64.71
180	147	4.5720	3.17	0.004100	547.9	133.65	66.82
190	150	4.8260	3.35	0.004107	558.9	136.09	68.04
200	153	5.0800	3.52	0.004114	569.4	138.39	69.19
210	155	5.3340	3.70	0.004122	576.4	139.83	69.91
220	156	5.5880	3.88	0.004130	579.9	140.42	70.21
230	154	5.8420	4.05	0.004137	572.9	138.47	69.24

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Unconfined Compressive Strength
ASTM D2166

Project No. 0035-037-00
Client Morrison Hershfield
Project Empress Pedestrain Ramp

Unconfined Compression Test Data (cont'd)

Deformation Dial Reading	Load Ring Dial Reading	Deflection (mm)	Axial Strain (%)	Corrected Area (m^2)	Axial Load (N)	Compressive Stress, q_u (kPa)	Shear Stress, S_u (kPa)
240	145	6.0960	4.23	0.004145	540.6	130.42	65.21
250	132	6.3500	4.40	0.004152	492.9	118.70	59.35

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Shelby Tube Visual

Project No.	0035-037-00
Client	Morrison Hershfield
Project	Empress Pedestrain Ramp
Test Hole	TH18-03
Sample #	T32
Depth (m)	4.6 - 5.3
Sample Date	9-Feb-18
Test Date	2-Mar-17
Technician	DS

Tube Extraction

Recovery (mm) 700 Over Push

Bottom - 5.3 m	5.14 m	4.98 m	4.81 m	4.65 m Top - 4.6 m
PP Tv	Qu Bulk	Moisture Content Visual	Keep	Slough
170 mm	160 mm	165 mm	160 mm	45 mm

Visual Classification

Material	Clay
Composition	silty
trace silt inclusions (<20 mm diam.)	
Color	mottled grey and brown
Moisture	moist
Consistency	stiff
Plasticity	high plasticity
Structure	
Gradation	

Torvane

Reading	0.52
Vane Size (s,m,l)	m
Undrained Shear Strength (kPa)	51.0

Pocket Penetrometer

Reading	**1**	1.00
	2	0.80
	3	1.00
	Average	0.93
Undrained Shear Strength (kPa)		45.8

Moisture Content

Tare ID	W104
Mass tare (g)	8.6
Mass wet + tare (g)	396.6
Mass dry + tare (g)	258.8
Moisture %	55.1%

Unit Weight

Bulk Weight (g)		993.4
Length (mm)	**1**	146.46
	2	146.47
	3	146.71
	4	146.92
Average Length (m)		0.147
Diam. (mm)	**1**	71.55
	2	71.54
	3	71.37
	4	71.81
Average Diameter (m)		0.072
Volume (m^3)		5.90E-04
Bulk Unit Weight (kN/m^3)		16.5
Bulk Unit Weight (pcf)		105.1
Dry Unit Weight (kN/m^3)		10.6
Dry Unit Weight (pcf)		67.8

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Unconfined Compressive Strength
ASTM D2166

Project No.	0035-037-00
Client	Morrison Hershfield
Project	Empress Pedestrain Ramp

Test Hole	TH18-03
Sample #	T32
Depth (m)	4.6 - 5.3
Sample Date	9-Feb-18
Test Date	2-Mar-17
Technician	DS

Unconfined Strength

	kPa	ksf
Max q_u	105.2	2.2
Max S_u	52.6	1.1

Specimen Data

Description Clay - silty, trace silt inclusions (<20 mm diam.), mottled grey and brown, moist, stiff, high plasticity,

Length	146.6	(mm)	**Moisture %**	55%	
Diameter	71.6	(mm)	**Bulk Unit Wt.**	16.5	(kN/m^3)
L/D Ratio	2.0		**Dry Unit Wt.**	10.6	(kN/m^3)
Initial Area	0.00402	(m^2)	**Liquid Limit**	-	
Load Rate	1.00	(%/min)	**Plastic Limit**	-	
			Plasticity Index	-	

Undrained Shear Strength Tests

Torvane

Reading	**Undrained Shear Strength**	
tsf	**kPa**	**ksf**
0.52	51.0	1.07
Vane Size		
m		

Pocket Penetrometer

	Reading	**Undrained Shear Strength**	
	tsf	**kPa**	**ksf**
	1.00	49.1	1.02
	0.80	39.2	0.82
	1.00	49.1	1.02
Average	**0.93**	**45.8**	**0.96**

Failure Geometry

Sketch:

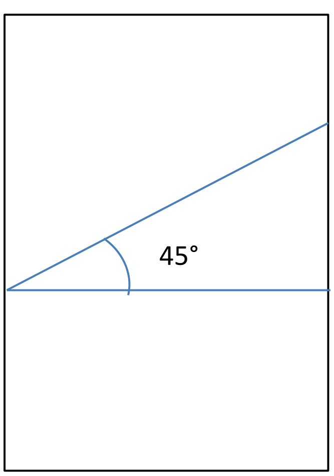
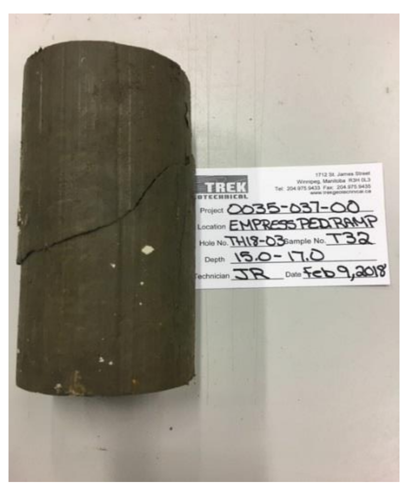


Photo:



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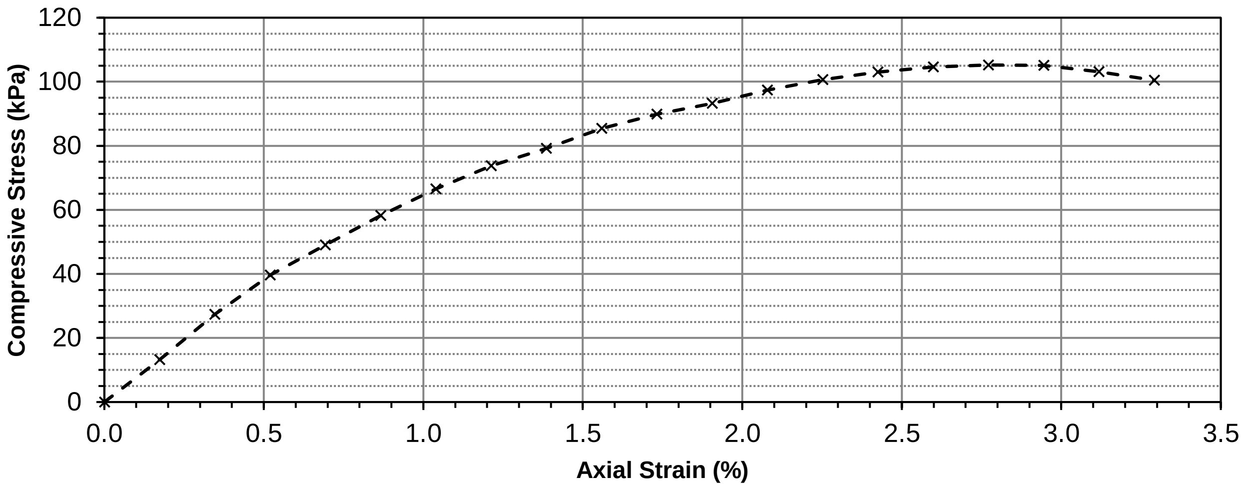
Unconfined Compressive Strength
ASTM D2166

Project No. 0035-037-00
Client Morrison Hershfield
Project Empress Pedestrain Ramp

Unconfined Compression Test Graph



Unconfined Compression Test Data

Deformation Dial Reading	Load Ring Dial Reading	Deflection (mm)	Axial Strain (%)	Corrected Area (m^2)	Axial Load (N)	Compressive Stress, q_u (kPa)	Shear Stress, S_u (kPa)
0	0	0.0000	0.00	0.004023	0.0	0.00	0.00
10	14	0.2540	0.17	0.004030	53.2	13.20	6.60
20	29	0.5080	0.35	0.004037	110.6	27.40	13.70
30	42	0.7620	0.52	0.004044	160.4	39.66	19.83
40	52	1.0160	0.69	0.004051	198.6	49.02	24.51
50	62	1.2700	0.87	0.004058	236.3	58.24	29.12
60	71	1.5240	1.04	0.004065	270.3	66.49	33.25
70	79	1.7780	1.21	0.004072	300.4	73.76	36.88
80	85	2.0320	1.39	0.004079	322.8	79.13	39.57
90	92	2.2860	1.56	0.004086	349.0	85.40	42.70
100	97	2.5400	1.73	0.004094	367.7	89.82	44.91
110	101	2.7940	1.91	0.004101	382.4	93.26	46.63
120	106	3.0480	2.08	0.004108	400.1	97.39	48.70
130	110	3.3020	2.25	0.004115	414.2	100.65	50.33
140	113	3.5560	2.42	0.004123	424.8	103.04	51.52
150	115	3.8100	2.60	0.004130	431.9	104.57	52.29
160	116	4.0640	2.77	0.004137	435.4	105.24	52.62
170	116	4.3180	2.94	0.004145	435.4	105.05	52.53
180	114	4.5720	3.12	0.004152	428.3	103.16	51.58
190	111	4.8260	3.29	0.004160	417.8	100.43	50.22

GEOTECHNICAL Quality Engineering | Valued Relationships

MEMORANDUM

Date	March 12, 2018
To	Jenna Roadley, TREK Geotechnical
From	Angela Fidler-Kliewer, TREK Geotechnical
Project No.	0035-037-00
Project	Empress Pedestrian Ramp
Subject	Additional Laboratory Testing Results – Rock Core Sample
Distribution	Michael Van Helden

Attached are the additional laboratory testing results for the above noted project. The testing included compressive strength determinations on one rock core sample with the results shown below.

Test Hole	TH18-01
Sample Number	C38
Top Depth (m)	14.8
Bottom Depth (m)	14.9
Compressive Strength (Mpa)	53.4
Sample Unit Wt. (kg/m^3)	2447

Regards,

Angela Fidler-Kliewer, C.Tech.

Attach.

Review Control:

Prepared By: AFK	*Reviewed By:* AFK	*Checked By:* AFK

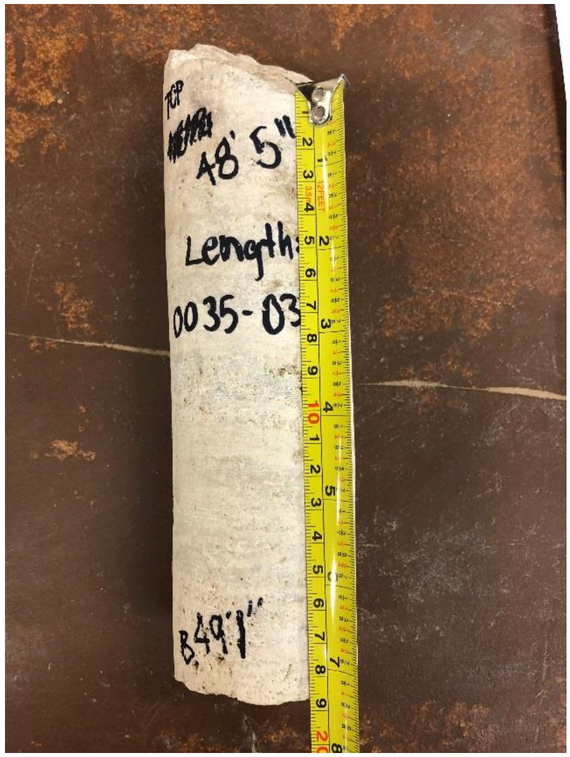


Photo 1: Original Core Sample

Photo 2: Top of Core Sample before Break

Photo 3: Bottom of Core Sample before break.

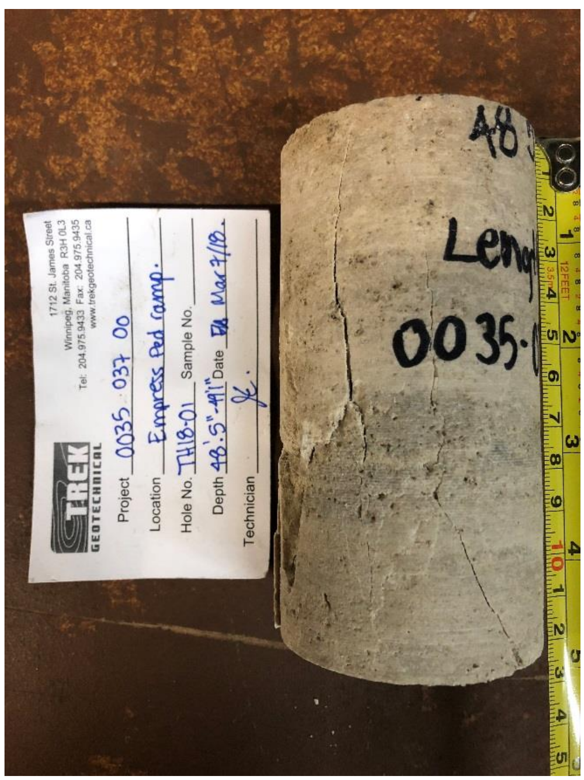


Photo 3: Core Sample after break

Quality Engineering | Valued Relationships

June 11, 2018 File No. 0035-037-00

Beth Phillips, P. Eng., C.I.M.
Morrison Hershfield
59 Scurfield Blvd
Winnipeg, Manitoba
R3Y 1G4

RE Empress Pedestrian Ramp– Addendum Letter (1st Revision)
Cast-in-Place End Bearing Caisson and Rock-Socketed Caisson Recommendations

This letter is an addendum to TREK's original geotechnical report issued on March 13, 2018 to Morrison Hershfield (MH). This addendum provides additional foundation recommendations for the proposed pedestrian ramps for the Empress overpass in Winnipeg, MB. Refer to the original report for information regarding the geotechnical investigation and preliminary design parameters for the design of ramp foundations (cast-in-place concrete friction piles, driven precast concrete hexagonal piles and driven steel H-piles) and other associated works.

We understand due to concerns about vibrations during construction and proximity to existing structures, drilled piles are the preferred pile type for the site, however cast-in-place concrete friction piles within the clay will not provide sufficient capacity for the proposed structures. This addendum provides recommendations for cast-in-place concrete end bearing caissons and rock-socketed caissons.

Limit States Design

For completeness and further to Section 4.1 of our previous report, Table 1 summarizes the resistance factors that can be used for the design of cast-in-place concrete deep foundations as per the CHBDC depending upon the method of analysis and verification testing completed during construction, which may include static load testing.

Table 1. ULS Resistance Factors for Foundations (CHBDC, 2014)

Description	Resistance Factor for Typical Degree of Understanding of Soil Conditions	Resistance Factor for High Degree of Understanding of Soil Conditions
Deep foundations in compression based on static analysis	0.40	0.45
Deep foundations in compression based on static load testing	0.60	0.70
Deep foundations in tension based on static analysis	0.30	0.40
Deep foundations in tension based on static load testing	0.50	0.60

Cast-in-Place Concrete End Bearing Caissons

Cast-in-place concrete (CIPC) end bearing caissons installed in very dense silt till are a suitable foundation alternative to support the proposed structure. The caissons should be constructed with straight shafts and will derive a majority of their axial-compressive resistance in end bearing with a relatively small contribution from shaft friction. Enlarged-base (belled) caissons are not recommended given the till conditions encountered. Caissons subjected to frost jacking and tension loads will derive a majority of their axial-uplift resistance in shaft friction. Table 2 provides the recommended ULS end bearing and shaft friction (adhesion) resistance values for axial-compressive and axial-tensile (uplift) loading conditions for mechanically-cleaned and hand-cleaned caissons bearing in very dense silt till, based on static analysis (no load testing). An increased resistance factor of up to 0.70 may be used for design, if static load testing is undertaken. The pile head displacement under unfactored service loads for evaluation of the Service Limit State can be calculated based on settlement of the caisson base of 0.5% to 1.0% of the pile diameter, plus elastic shortening of the pile shaft.

Table 2. Recommended Unit Resistances for CIPC End-Bearing Caissons on Till

Construction Method	Factored ULS Axial Resistance (kPa)		
	Compression $\phi = 0.45$		Uplift $\phi = 0.40$
	Shaft Adhesion	Unit End Bearing	Shaft Adhesion
Hand-Cleaned	13.5	900	12
Mechanically-Cleaned		450	

<u>End-Bearing Caisson Design Recommendations</u>

The following recommendations apply to the design of CIPC end bearing caissons:

1. The weight of the embedded portion of the pile should be included in the calculation of pile dead loads.
2. Shaft adhesion should be neglected within the upper 2.5 m below final grade, or to the depth of fill soils below the ground surface.
3. Caissons must be founded in the very dense silt till. Given the relative small thickness of very dense till encountered, caissons may need to be advanced to bedrock.
4. Caisson bases that are to be hand-cleaned must have a minimum shaft diameter of 760 mm to permit down-hole entry of personnel to clean the caisson base and to perform inspection. Due to current safety regulations, the maximum allowable depth that a person can enter a confined space below the ground surface is 22.9 m (75 feet), restricting down-hole inspection in many areas of the Winnipeg area.
5. A minimum pile length of 8 m below ground surface is recommended to protect against frost jacking.

6. Caissons should have a minimum spacing of 2.5 caisson diameters measured centre to centre. If a closer spacing is required, TREK should be contacted to provide an efficiency (reduction) factor to account for potential group effects.
7. All caissons require steel reinforcement design by a qualified structural engineer for the anticipated axial (compression and uplift), lateral, and bending loads from the structure.
8. Grade beams and caisson caps should be constructed with a minimum 150 mm void space between soils and the underside of the concrete to minimize the effects of soil heave due to swelling or frost action. Void forms should be selected such that they can deform 150 mm without exceeding the tolerable uplift resistance of the structure or pile.
9. Lateral pile resistance should be calculated as per Section 4.5 of our previous report. The pile tip should be assumed to have a fixed displacement boundary with free rotation (i.e. a pinned connection).

End-Bearing Caisson Installation Recommendations

The following recommendations apply to the installation of CIPC friction piles:

1. Temporary steel casings (i.e. sleeves) should be on site and used if sloughing of the caisson hole occurs, to control groundwater seepage if encountered, and/or if down-hole entry is required. Care should be taken in removing sleeves to prevent sloughing (necking) of the shaft walls and a reduction in the cross-sectional area of the pile. In this regard, sloughing and seepage conditions were encountered within the sand unit at approximately 12 to 13 m depth, and full-length sleeving of pile holes is expected to be required.
2. Cobbles were encountered in the test hole, which may also indicate the presence of boulders. The foundation contractor must expect to encounter boulders during installation of the caissons. Chopping and removal of boulders may be necessary to advance the caisson shaft to the very dense silt till.
3. Caisson bases must be free of loose and/or disturbed soil.
4. Concrete should be placed immediately after the completion of drilling the caisson hole and under dry conditions to avoid construction problems such as sloughing or caving of the caisson hole and groundwater seepage. If groundwater is encountered it should be controlled and removed. If water cannot be controlled and removed, the concrete should be placed using tremie methods.
5. Concrete placed by free-fall methods should be directed through the middle of the caisson shaft and steel reinforcing cage to prevent striking of the caisson walls to protect against soil contamination of the concrete.
6. Concrete should be placed in one continuous operation.
7. The recommended resistances are based on a high degree of understanding of soil conditions, and therefore the drilling of all caisson shafts should be observed and documented by TREK Geotechnical to verify the soil conditions and proper installation of the caissons. Reduced resistance factors associated with a low or typical degree of understanding should be used if TREK is not retained to observe installation.

Cast-in-Place Concrete Rock-Socketed Caissons

Cast-in-place concrete (CIPC) rock-socketed caissons installed in dolomitic limestone bedrock are a suitable foundation alternative to support the proposed structure. The mobilized axial-compressive capacity of the caissons is expected to consist of a combination of side shear (shaft friction) and end-bearing resistance from within the socketed portion of the caisson (*i.e.* the rock socket). Rock-socketed caissons subjected to axial-uplift forces (due to frost-jacking and/or tension loads) derive a majority of their resistance in side shear from within the rock socket. The resistance developed along the caisson shaft above the rock socket is considered negligible.

Table 3 provides the recommended ULS side shear and end bearing resistance values for axial-compressive and axial-uplift loading conditions for rock-socketed caissons bearing in dolomitic limestone bedrock. These values are based on conventional design practice and are generally based on empirical data on rock-socketed pile performance. Additional recommendations are provided below on the potential for static load testing to be used to increase the factored pile capacity. The pile head displacement under unfactored service loads for evaluation of the Service Limit State can be calculated based on up to 5 mm of settlement within the rock socket, plus elastic shortening of the pile shaft above the socket.

Experience has shown that the quality of limestone bedrock can vary significantly between pile locations and with depth. Fractured rock would be considered as poor quality weathered limestone containing open or infilled fractures; massive rock would be relatively high quality rock and generally free of such discontinuities. Although all core samples obtained in TH18-01 were of good to excellent rock quality and contained very few fractures, rock mass properties may vary across the site. In general, Winnipeg area bedrock can be highly fractured and of poor quality within the upper 2 to 3 m of the unit becoming more competent with depth.

Table 3. Recommended ULS Resistances for Rock-Socketed Caissons

Method of Confirmation of Resistance Values	Factored ULS Axial Unit Resistance (MPa)		
	Compression $\phi = 0.45$		Uplift $\phi = 0.4$
	Side Shear	End Bearing	Side Shear
Proof Holes Below Base of Rock Socket (Massive Rock)	1.2	3.5	1.0

It must be recognized that the quality of the bedrock and thickness of the upper fractured zone can change over short distances and may differ from that observed in the core samples. Constructability issues including inadequate supply of sleeves or reinforcement, design changes, delays and cost overruns have been observed more frequently on projects where proof-coring is not conducted at each socket location to verify the rock mass conditions prior to construction. As such, we consider proof-coring to be required in order to use resistance factors associated with a high degree of understanding for design and the unit resistances in Table 3, and nonetheless recommended in any case as proper practice for the design and construction of rock-socketed caissons. Proof-coring generally provides the added benefit of providing detailed stratigraphic information for each caisson that a piling contractor can use to develop a drilling and sleeving plan. The socket length should be confirmed based on the observed rock quality and an allowance should be carried at the design stage for the possibility of socket lengthening by 1.5 to 2 times the design socket length.

Due to current safety regulations, the maximum allowable depth that a person can enter a confined space below the ground surface is 22.9 m (75 feet), restricting down-hole inspection in many areas of the Winnipeg area. In the event that down-hole inspection and proof-coring cannot be performed, and inspection of the rock socket is based on video monitoring and evaluation of retrieved rock cores, the design of the caissons is limited to side shear resistance only, the end bearing resistance is to be ignored, and resistance factors of 0.40 and 0.30 should be used for axial compressive and tensile resistance associated with a typical degree of understanding of sub-surface conditions.

Proof-coring was traditionally accomplished by means of manual drilling with personnel entering the completed rock-socketed. Due to current safety restrictions on down-hole entry (as noted previously), proof-coring may be completed from the ground surface prior to construction after the location of the caissons have been determined.

Rock-Socketed Caisson Design Recommendations

The following recommendations apply to the design of CIPC rock-socketed caissons:

1. The weight of the embedded portion of the pile should be included in the calculation of caisson dead loads.
2. Shaft adhesion should only apply within the rock-socket, and should be neglected within overburden soils.
3. Rock sockets must have a minimum shaft diameter of 762 mm to permit down-hole entry of personnel to clean the caisson base and to perform inspection, if depth restrictions are not applicable.
4. A minimum rock socket diameter of 0.5 m and minimum rock socket length of 2 m should be used. The base of the rock socket should be at least 3 pile diameters below the surface of the rock.
5. Caissons should have a minimum spacing of 2.5 caisson diameters measured centre to centre. If a closer spacing is required, TREK should be contacted to provide an efficiency (reduction) factor to account for potential group effects.
6. The rock socket must be installed in competent rock as determined by TREK on the basis of proof-core samples and rock cores retrieved during construction.
7. All caissons require steel reinforcement design by a qualified structural engineer for the anticipated axial (compression and uplift), lateral, and bending loads from the structure.
8. Grade beams and caisson caps should be constructed with a minimum 150 mm void space between soils and the underside of the concrete to minimize the effects of soil heave due to swelling or frost action.
9. Lateral pile resistance offered by overburden soils is considered negligible in relation to the strength and stiffness of the structural pile section. As such, lateral pile resistance can be evaluated assuming

a fixed condition within the rock-socket with no resistance offered by the overburden soils (i.e. governed by structural pile capacity).

Rock-Socketed Caisson Installation Recommendations

The following recommendations apply to the installation of CIPC rock-socketed caissons:

1. Temporary steel casings (i.e. sleeves) must be used to advance the caisson shaft to the bedrock surface occur, to control groundwater seepage if encountered, and/or if down-hole entry is required. Care should be taken in removing sleeves to prevent sloughing (necking) of the shaft walls and a reduction in the cross-sectional area of the caisson.
2. Cobbles were encountered in the test hole, which may indicate the presence of boulders as well. The foundation contractor must expect to encounter boulders during installation of the caissons. Chopping and removal of boulders may be necessary to advance the caisson shaft past silt till and into bedrock.
3. Caisson bases must be free of loose and/or disturbed soil.
4. Concrete should be placed immediately after the completion of drilling the caisson shaft and under dry conditions to avoid construction problems such as sloughing or caving of the shaft and groundwater seepage. If groundwater is encountered it should be controlled and removed. If water cannot be controlled and removed, the concrete should be placed using tremie methods.
5. Concrete placed by free-fall methods should be directed through the middle of the caisson shaft and steel reinforcing cage to prevent striking of the shaft walls to protect against soil contamination of the concrete.
6. Concrete should be placed in one continuous operation.
7. The recommended resistances are based on a high degree of understanding of soil conditions, and therefore that the drilling of all caisson shafts is observed and documented by TREK Geotechnical to verify the soil conditions and proper installation of the caissons. Reduced resistance factors associated with a low degree of understanding should be used if TREK is not retained to observe installation.

Design of Rock-Socketed Caissons with Static Load Testing

We recommend that if rock-socketed caissons are to be considered for this project that a load testing program utilizing Osterberg Cell technology be considered to provide measured load–settlement characteristics from a test rock-socket. This type of testing has been used rarely in Winnipeg however it has been used extensively abroad to prove out load capacities for this foundation type that are 5 to 10 times greater than conventional foundation designs. The load test provides additional advantages for the project including the use of higher resistance factors (0.70 for static load tests instead of 0.45 for static analysis) when evaluating the ULS condition. To undertake this program, sacrificial test rock sockets would be required for the load tests which could be installed during the design phase. Typically, the cost of a static load test would be in the order of $50,000 to $100,000. Non-destructive Osterberg Cell tests can also be considered for which a production pile can be used, which is typically less costly but is limited to proving a load capacity that is less than the ultimate capacity.

For the purpose of preliminary design and cost comparison to evaluate the cost-benefits of load testing, this foundation type can be designed with the unit resistances provided in Table 3 but an increased ULS end-bearing resistance of 15 MPa. These values correspond to nominal (unfactored) values of 2.7 MPa in side shear and 35 MPa in end-bearing, and will need to be substantiated from the results of the proposed Osterberg Cell load tests. If the load tests are undertaken in conjunction with proof-coring and down-hole inspection for production caissons (as described previously), resistance factors of 0.70 and 0.60 can be applied to the nominal values obtained from the testing for bearing resistance and uplift resistance, respectively. As with any load testing program there is always a risk that the measured resistance will be lower than the anticipated nominal values used in design. This risk needs to be understood when a load testing program is initiated.

Closure

The geotechnical information provided in this report is in accordance with current engineering principles and practice (Standard of Practice). The findings of this report were based on information provided (field investigation and laboratory testing). Soil conditions are natural deposits that can be highly variable across a site. If subsurface conditions are different than the conditions previously encountered on-site or those presented here, we should be notified to adjust our findings if necessary.

All information provided in this report is subject to our standard terms and conditions for engineering services, a copy of which is provided to each of our clients with the original scope of work or standard engineering services agreement. If these conditions are not attached, and you are not already in possession of such terms and conditions, contact our office and you will be promptly provided with a copy.

This report has been prepared by TREK Geotechnical Inc. (the Consultant) for the exclusive use of Morrison Hershfield (the Client) and their agents for the work product presented in the report. Any findings or recommendations provided in this report are not to be used or relied upon by any third parties, except as agreed to in writing by the Client and Consultant prior to use.

If you have any questions, please contact the undersigned.

Kind Regards,

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RE Empress Avenue Reconstruction
Detailed Design Geotechnical Report – Addendum No. 1
Recommendations for Embankment Widening near St. Matthews Ave. Culvert

This letter is an addendum to TREK's original detailed design geotechnical report issued on May 17, 2018 to Morrison Hershfield (MH). This addendum provides additional recommendations related to shallow foundations and retaining walls for embankment widening near St. Matthews Avenue. Our previous analysis did not capture the proposed widening at this location, where existing rockfill ribs (installed in 2015) are present.

The creek bank in this area was previous stabilized using rockfill ribs (designed by TREK in 2014) and the design at that time did not consider further embankment widening. The rockfill ribs were designed to achieve a factor of safety of 1.30 with the existing slope geometry under critical groundwater conditions, and therefore additional measures will be necessary to offset the impact of any proposed fill placement for embankment widening. Due to the constrained geometry at this location, retaining walls will be required to limit the embankment footprint. Deep flexible wall systems such as a driven steel sheet piles could be implemented without the requirement further slope stabilization, however gravity retaining walls (e.g. cast-in-place gravity walls, MSE walls) will result in additional loading on the creek bank and reduce the factor of safety below the design criteria. As such, additional slope stabilization works will be required to offset the net loading on the creek bank associated with gravity walls. Through preliminary discussions with MH, lightweight fill was identified as the most cost-effective alternative to improve stability for this section.

Limit States Design

Limit states design requires consideration of distinct loading scenarios comparing the structural loads to the foundation bearing capacity using resistance and load factors that are based on probabilistic reliability criteria. Two general design scenarios are evaluated corresponding to the serviceability and ultimate capacity requirements.

The Ultimate Limit State (ULS) is concerned with ensuring that the maximum structural loads do not exceed the nominal (ultimate) capacity of the foundation units. The ULS foundation bearing capacity is obtained by multiplying the nominal (ultimate) bearing capacity by a resistance factor (reduction factor), which is then compared to the factored (increased) structural loads. The ULS bearing capacity must be greater or equal to the maximum factored load. Table 1summarizes the resistance factors that can be used for the design of foundations as per the Canadian Highway Bridge Design Code (CHBDC, 2014) depending upon the method of analysis and verification testing completed during construction. The CHBDC also requires that the degree of understanding of soil conditions (which can be classified as either low, typical or high) be assessed in the selection of the resistance factors. For shallow foundations, some uncertainty exists regarding the presence of fill soils or silt at the subgrade level; provided that bearing surface inspection is conducted by TREK during construction, we consider the degree of understanding of soil conditions to be high for shallow foundations as well. CHBDC also requires that the resistance factor be modified by a consequence factor which ranges from 0.9 for high consequence structures to 1.15 for low consequence structures. The structures for this project are interpreted to be of typical consequence based on the CHBDC guidelines and as such a consequence factor 1.0 is applied in our recommendations.

The Service Limit State (SLS) is concerned with limiting deformation or settlement of the foundation under service loading conditions such that the integrity of the structure will not be impacted. The SLS should generally be analysed by calculating the settlement resulting from applied service loads and comparing this to the settlement tolerance of the structure. However, the settlement tolerance of the structure is typically not defined at the preliminary design stage. As such, SLS bearing capacities (or unit resistances) provided are developed on the basis of limiting settlement to approximately 25 mm or less, unless a methodology to estimate foundation settlement is provided. A more detailed settlement analysis should be conducted to refine the estimated settlement and/or adjust the SLS vertical bearing resistance if a more stringent settlement tolerance is required.

Table 1. ULS Resistance Factors for Foundations (CHBDC, 2014)

Description	Resistance Factor for Typical Degree of Understanding of Soil Conditions	Resistance Factor for High Degree of Understanding of Soil Conditions
Shallow foundations with a typical degree of understanding of soil conditions and using empirical analysis	0.50	0.60
Shallow foundations for analysis of sliding on cohesive material	0.60	0.65

Shallow Foundations

Embankment widening fill near St. Matthews Avenue may be retained using cast-in-place concrete gravity walls bearing on strip footings. Strip footings bearing on undisturbed firm clay can be designed based on a ULS and SLS bearing resistances of 120 kPa and 65 kPa respectively. However, due to slope stability considerations, gravity walls must be designed such that the total unfactored loads due to the structure and backfill at the SLS are less than or equal to the weight of existing soil removed above the bearing surface. A bulk unit weight of 17.5 kN/m^3 for existing soils can be used to estimate the weight of soil removed. It is anticipated that lightweight fill will be required to offset structural loads and satisfy this requirement. Resistance against uplift due to buoyancy of lightweight fill should be evaluated if the bearing surface extends below the Q1% creek level at this location (231.2 m ±). Based on recent experiences, the unit weights of cellular concrete and expanded polystyrene (EPS) foam are about 4.5 kN/m^3 and 0.3 kN/m^3, respectively. These values may differ than those provided in the manufacturers specifications which should be used for final design.

Given that the applied bearing stress will be less than or equal to the existing vertical effective soil stress at the bearing surface, footing settlement will occur elastically due to recompression of foundation soils following offloading and is expected to be less than 25 mm. The ULS bearing resistance was calculated using a resistance factor of 0.6. Shallow footings can be expected to be subject to vertical movements associated with seasonal shrinkage and swelling of the clay bearing soils. If a footing is founded above 2.5 m depth they will also be subject to seasonal movements related to freeze/thaw. In this case, rigid polystyrene insulation should be included to provide an equivalent frost penetration depth of 2.5 m.

Additional recommendations for the design and construction of shallow foundations are provided below.

<u>Shallow Footing Design Recommendations:</u>

1. Footings should have a minimum base width of 0.6 m.
2. Footings should be designed by a qualified structural engineer to resist axial, lateral, and bending loads from the structure.

<u>Shallow Footing Installation Recommendations:</u>

1. All fill, silts, organics and/or any other deleterious material should be completely stripped such that the bearing surface consists of undisturbed native firm to stiff silty clay. Silt layers were encountered in various nearby test holes, at depths of up to 3.0 m. Where silt is encountered at the design bearing surface, it should be removed and replaced with 20 mm down crushed limestone base material overlying non-woven geotextile. The crushed limestone should be placed in lifts no greater than 150 mm and compacted to a minimum of 100% Standard Proctor Maximum Dry Density (SPMDD). Depending on the design subgrade elevation for the footings or walls, up to 3.1 m of fill may need to be removed.
2. Excavations for footings should be completed by an excavator equipped with a smooth bladed bucket operating from the edge of the excavation. The contractor should work carefully to prevent disturbance to the bearing surface at all times. In the event the surface is disturbed, the clay sub-grade should be either recompacted to 95% of SPMDD or excavated further to undisturbed clay sub-grade.

3. Over-excavation of the bearing surface should be avoided. If a levelling course is required below the footing it may be constructed using 20 mm down crushed rock compacted to 100% of SPMDD.
4. The bearing surface should be protected from freezing, drying, inundation with water or disturbance at all times. If groundwater seepage is encountered, it should be controlled and removed from the excavation, such that concrete is placed under dry conditions.
5. The final bearing surface should be inspected and documented by TREK personnel prior to concrete placement to verify the adequacy of the bearing surface and proper installation of footings.

Resistance to Overturning, Uplift and Sliding:

If exterior footings are subjected to lateral loads, they must be designed to resist overturning, uplift and sliding forces. Lateral loading will result in the development of overturning and uplift forces and consequently a non-uniform applied pressure distribution under the footing base. In this regard, the maximum applied pressure should not exceed the ULS bearing resistance and the minimum applied pressure should not be less than 0 kPa (*i.e.* the eccentric resultant vertical force shall not be more than B/6 away from the footing centreline). Resistance to overturning and uplift forces due to lateral loading will be provided from the weight of the material used to backfill the footing excavation and the structural dead loads. A unit weight of 17 kN/m³ can be used for clay fill provided it is compacted to a minimum of 95% of the SPMDD.

For the evaluation of sliding of the footing bearing directly on native clay, a friction angle of 15 degrees may be used along the concrete/clay interface. A geotechnical resistance factor of 0.6 should be used when assessing sliding resistance on clay in accordance with Table 6.2 of CHBDC. However, it is our understanding that footings may be cast on a low-strength concrete "mud-slab" underlain by a well-compacted layer of granular base course. In this case, sliding resistance between the mud-slab and granular base course may be calculated based on a sliding friction angle of 30 degrees with a resistance factor of 0.8 applied (CHBDC Table 6.2 for non-cohesive soils). If a geotextile is used between the clay subgrade and the granular base course, it should be a non-woven geotextile that is on the City of Winnipeg's approved products list.

Lateral Earth Pressures

The magnitude of lateral earth pressures from retained soil against buried structures will depend on the backfill material type, method of placing and compacting the backfill and the magnitude of horizontal deflection of the retaining wall after the backfill is placed. Cohesive soils should not be used as backfill against buried walls as these soils could generate excessive lateral earth pressures from swelling.

For gravity walls backfilled with free-draining granular soils, an active pressure coefficient (K_a) of 0.3 should be used to calculate lateral loads against retaining structures which are free to translate horizontally by at least 0.2 percent of the retaining wall height. For retaining structures which are not free to translate, an at-rest earth pressure coefficient (K_o) of 0.5 should be used. Where the retaining structure is free to translate at least 2 percent of the retaining wall height towards the backfill soil, a passive earth pressure coefficient (K_p) of 3.3 should be used. Surcharge loading should also be included in the earth pressure distribution to account for surface loads, based on the appropriate earth pressure coefficient.

Over-compaction of the backfill soils adjacent to buried walls may result in earth pressures that are considerably higher than those predicted in design. Compaction of the granular fills within about 1.5 m of the vertical walls should be conducted with a light hand operated vibrating plate compactor and the number of compaction passes should be limited to achieve a maximum of 92% of Standard Proctor Maximum Dry Density (SPMDD). Backfilling procedures should be reviewed during construction to verify that they are consistent with the design assumptions.

Table 2 provides the recommended earth pressure coefficients and bulk unit weights of the silty clay layer for calculation of lateral earth pressures on cantilevered retaining walls (e.g. sheet pile walls). Surcharge loads and hydrostatic water pressure should be incorporated into the design of cantilevered walls, as well as an adequate factor of safety against instability. In consideration of the sloping ground surface downslope of the wall, some passive resistance will need to be ignored. Once the wall geometry and location has been determined, TREK should be contacted to provide further guidance and review earth pressure calculations. Due to the complex soil-structure interaction of flexible unbraced cantilever (e.g. sheet pile) walls, design based on traditional Rankine earth pressure theory may result in excessively deep embedment required in some cases. If this arises, TREK can provide finite-element analysis for design of retaining structures to optimize sheet pile embedment.

Table 8. Recommended Lateral Earth Pressure Parameters for Retaining Walls in Silty Clay

Design Parameter	Earth Pressure Coefficients and Bulk Unit Weights
At-rest (K_o)	0.65
Active (K_a)	0.5
Passive (K_p)	2.0
Bulk Unit Weight, Y (kN/m^3)	17.5

A certain amount of ground movement behind the wall will occur, and is largely unavoidable. The amount of movement that will occur cannot be accurately predicted, mainly because the movement is as much a function of installation procedures and workmanship as it is a function of theoretical considerations.

Closure

The geotechnical information provided in this report is in accordance with current engineering principles and practice (Standard of Practice). The findings of this report were based on information provided (field investigation and laboratory testing). Soil conditions are natural deposits that can be highly variable across a site. If subsurface conditions are different than the conditions previously encountered on-site or those presented here, we should be notified to adjust our findings if necessary.

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If you have any questions, please contact the undersigned.

Kind Regards,

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RE Empress Pedestrian Ramp– Addendum # 2
Concrete Slabs and Pavement Recommendations

This letter is an addendum to TREK's original geotechnical report issued on March 13, 2018 to Morrison Hershfield (MH). This addendum provides additional recommendations for proposed concrete pavements that are required at the entrances to the pedestrian ramps. Refer to the original report for information regarding the geotechnical investigation.

Pavement Recommendations

Recommended pavement sections for concrete pavements are provided in Table 4. The structure provided is for lighter and heavier vehicle use areas and is comparable to typical sections used for City of Winnipeg road works. Crushed limestone base and sub-base materials that meet with the City of Winnipeg Specifications CW 3110 (latest revision) are recommended. Some seasonal movements of concrete pavements should be anticipated.

Table 4. Recommended Pavement Sections for Asphalt Roads and Parking Areas

Material	Layer Thickness		Compaction Requirements
	Car Parking Areas	Heavy Vehicular Loads	
Concrete	150 mm	150 mm	Mix design by others
20 mm down crushed limestone (Base)	75 mm	100 mm	100% of the SPMDD
50 down crushed limestone (Sub-Base)	250 mm	350 mm	98% of the SPMDD
Non-Woven Geotextile (Geotex 801 or equivalent)	Required	Required	Install as per manufacturer's recommendations

Additional Pavement Recommendations:

1. Organics, fill soils and silts should be completely removed such that the sub-grade consists of undisturbed silty clay. Based on the depths of silt observed in the test holes this would require removal of up to 3.0 m of soil, which is likely not feasible. If some risk of additional movement is tolerable fill and silt soils may be left in place.

2. Excavation should be completed with an excavator equipped with a smooth bucket and operating from the edge of the excavation in order to minimize disturbance to the exposed sub-grade. If silt is present at the subgrade level, it should be expected to be highly sensitive to disturbance. If fill is encountered at the subgrade, it should be scarified, moisture conditioned and recompacted to a minimum of 95% of Standard Proctor Maximum Dry Density (SPMDD).
3. After excavation, the sub-grade should be inspected by TREK. The sub-grade should be proof-rolled with a fully loaded tandem axle truck (if possible) to detect soft areas or silt. Soft and /or silt areas should be repaired as per directions provided by TREK. This will likely consist of excavating an additional 300 mm and placing a non-woven geotextile on the sub-grade and backfilling with a suitable granular sub-base material to raise grades. If proof-rolling cannot be conducted, TREK may recommend the above repair in absence of the ability to detect weak or soft areas.
4. The sub-grade should be protected from mechanical disturbance, freezing, drying, or inundation with water. If any of these conditions occur the sub-grade should be scarified, moisture conditioned as appropriate, and re-compacted to a minimum of 95% of the SPMDD. Alternatively, unsuitable / disturbed material can be sub-excavated and replaced with compacted sub-base materials.
5. The granular sub-base and base materials should be placed in lift thicknesses no greater than 150 mm and compacted as per recommendations in Table 04.
6. Fill required to raise grades should consist of well-graded 100 or 50 mm down crushed rock or recycled concrete compacted to 98% SPMDD.

Closure

The geotechnical information provided in this report is in accordance with current engineering principles and practice (Standard of Practice). The findings of this report were based on information provided (field investigation and laboratory testing). Soil conditions are natural deposits that can be highly variable across a site. If subsurface conditions are different than the conditions previously encountered on-site or those presented here, we should be notified to adjust our findings if necessary.

All information provided in this report is subject to our standard terms and conditions for engineering services, a copy of which is provided to each of our clients with the original scope of work or standard engineering services agreement. If these conditions are not attached, and you are not already in possession of such terms and conditions, contact our office and you will be promptly provided with a copy.

This report has been prepared by TREK Geotechnical Inc. (the Consultant) for the exclusive use of Morrison Hershfield (the Client) and their agents for the work product presented in the report. Any findings or recommendations provided in this report are not to be used or relied upon by any third parties, except as agreed to in writing by the Client and Consultant prior to use.

If you have any questions, please contact the undersigned.

Kind Regards,

TREK Geotechnical
Per:

Reviewed By:

PROVINCE OF MANITOBA
2018/06/20
M. J. VAN HELDEN
Member
24729
REGISTERED PROFESSIONAL ENGINEER

Michael Van Helden, Ph.D, P.Eng.
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Certificate of Authorization
Trek Geotechnical Inc.
No. 4877 Date: June 20, 2018

May 17, 2018 File No. 0035-037-00

Mr. Brad Sacher, P.Eng.
Morrison Hershfield Ltd.
Suite 1, 59 Scurfield Blvd.
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RE Empress Street Rehabilitation - Portage Avenue to St. Matthews Avenue
Updated Slope Stability Analysis for Detailed Design

This letter report provides a summary of riverbank stability analysis of permanent works conducted as part of the Detailed Design (DD) phase of the Empress Street Rehabilitation project. TREK Geotechnical (TREK) was retained by Morrison Hershfield Ltd. (MHL) for Detailed Design of slope stability improvements along Empress Street, including riverbank stabilization, erosion protection, and sidewalk widening with fill.

TREK previously submitted a Preliminary Design (PD) report, which included riverbank stability analysis of existing conditions and proposed works. Additional information on general site conditions and an overview of proposed works is provided in our PD report.

The current report summarizes slope stability analyses conducted to refine limits and optimize slope geometries and stabilization works in support of a City of Winnipeg Waterways Permit and Provincial regulatory approvals for the project. The proposed works are shown on the attached 90% review drawings prepared by MHL (Appendix A).

Background

Along Omand's creek, the design of new roadways, sidewalks and cycle tracks is now complete and as such a detailed design of stabilization works is required to determine the layout, extent and geometry of stabilization works. The proposed slope geometries shown on the 90% drawings, including the location of the sidewalk /cycle track, fill placement or regrading works, riprap and rockfill ribs, are generally consistent with our PD recommendations. However, the southern limit of rockfill ribs (south of Maroons Road) as it relates to proposed slope geometries was not yet determined at the PD stage and requires further slope stability analysis. This additional analysis and recommendations for the extent of rockfill ribs in this area is provided herein.

Along the Assiniboine riverbank, an historical instability is evident, with the head scarp approaching within a few metres of the edge of the existing roadway. However, no evidence of recent movement was identified at the PD stage and as such the area was deemed marginally stable during the PD phase. Recent slope inclinometer monitoring during DD has identified approximately 50 mm of differential shear movement has occurred since PD along the pre-existing slide. As a result, the pre-existing slide is no longer considered "marginally stable" but is rather actively unstable, therefore further slope stability analysis was undertaken to re-design slope stabilization works and achieve a higher degree of relative improvement. In this regard, our previous slope stability analysis calculated a factor of safety of unity (FS=1.00) for an assumed pre-failure geometry (back analysis), and a factor of safety above unity (about FS=1.15) for the existing slope geometry using the results of the back-analysis (e.g. strengths and groundwater levels). Given the slope has undergone significant movement recently, the factor of safety is essentially at unity (FS=1.0) for the existing slope geometry. In the regard, a revised back analysis was undertaken using the existing slope geometry.

Unless otherwise specified herein, all soil layers, parameters, groundwater conditions and design criteria used in the current analysis are consistent with our PD report. Stabilization measures were designed to achieve a target factory of safety of 1.3 at the roadway along both the Omand's Creek and Assiniboine River banks, and at minimum no net change over existing conditions.

Slope Stability Analysis Results - Assiniboine Riverbank

Tables 2 compares the results of existing stability and final design works along the Assiniboine Riverbank. Slope stability results figures are included in Appendix B, as referenced in the tables. A detailed summary of the analysis cases and results are provided below.

Revised Back Analysis and Existing Stability

The revised back-analysis using existing slope geometry and the previously reported groundwater conditions required modification to the strength of the lacustrine clay layer. Beyond the observed instability (in the upper bank area) the clay was assumed to be relatively intact and assigned a conservative strength of c=5kPa and ϕ'=14°, which is reflective of fully-softened strength and consistent with the strengths assumed in the PD. Downslope of this zone, where slope movements have recently occurred, reduced (residual) strength parameters within the lacustrine clay of c'= 3.5 kPa and ϕ'=12.5° were assumed. With these revised parameters, a factor of safety of 1.00 is calculated for a near-critical slip surface that closely matches the observed zone of inclinometer movement and tension crack location visible at ground surface. The factor of safety at the existing roadway is 1.04, while the factor of safety at the location of the proposed roadway is 1.19.

Road Realignment, Cycle Track, Sidewalk and Stabilization Alternatives

The proposed roadway realignment, cycle track and sidewalk design has remained unchanged from the PD and involves shifting the road further from the river along Cross-section B by approximately 3.6 m, and adding a cycle track and sidewalk along the edge of roadway (overall width 6.0 m). The proposed design results in minor offloading downslope of the sidewalk. Without further stabilization alternatives, the observed instability remains critical (FS = 1.04) and the factors of safety at the proposed sidewalk and roadway are 1.06 and 1.16, respectively. In order to achieve the target factor of safety of 1.30 at the roadway, a clay toe berm is proposed extending down from Elev. 230.0 m ± at a 7H:1V slope with a cross-sectional area of about 10 m^2 (Figure 01). The addition of the toe berm results in a factor of safety of 1.38 for the observed instability, 1.24 for the proposed sidewalk and 1.32 for the proposed roadway, which satisfies the design criteria. Table 2 below summarizes and compares the results of the stability analysis.

Table 2. Summary of Stability Analysis for the Assiniboine Riverbank

Case: Description	Cross-Section	Factors of Safety			Figure No. (Appendix B)
		Slip Surface	FS	% Improvement	
Back Analysis of Existing Geometry	B	Observed Movement	1.00	N/A	B-1
		Existing Road	1.04	N/A	
		Proposed Sidewalk	1.02	N/A	
		Proposed Road	1.19	N/A	
Road Realignment, Cycle Track, Sidewalk and Regrading	B	Observed Movement	1.04	4%	B-2
		Proposed Sidewalk	1.06	4%	
		Proposed Road	1.16	5%	
Stabilization Works: Clay toe berm	B	Observed Movement	1.38	38%	B-3
		Proposed Sidewalk / New Critical	1.24	22%	
		Proposed Road	1.32	18%	

Stability Modeling Results - Omand's Creek

Table 3 summarizes and compares the results from the stability assessment of existing conditions and stability improvement alternatives considered along Omand's Creek. Slope stability results figures are included in Appendix B, as referenced in the tables.

Existing slope geometry was obtained from MHL at the indicated cross-sections. The critical slip surface daylights behind the proposed infrastructure, beyond the sidewalk and within the roadway, in all cases.

Rockfill ribs are required upstream (north) of Sta. 1+302. South of Sta. 1+302, the existing slope angle flattens and the proposed works include offloading, therefore there is no net reduction in stability results from the proposed works. Additional cross-sections north of Sta. 1+302 were also analysed, and the results were similar where rockfill ribs are required to offset the impact of fill placement and achieve no net reduction to existing stability.

In summary, new rockfill ribs should extend from the existing ribs south of St. Matthews Avenue (Sta 1+030 ±) to approximately 145 m south of the centreline of Maroons Road (Sta. 1+300).

Table 3. Summary of Stability Analysis along Omand's Creek

Cross-Section (North to South)	Case: Description	Slope Height and Angle	Factors of Safety		Figure No. (Appendix B)
			FS at Infrastructure (Critical)	% Improvement	
STA 1+302	Existing Conditions	4.4 m @ 5.0H:1V	1.43	N/A	B-4
	Sidewalk widening and associated regrading	4.6 m @ 4.6H:1V	1.39	-2.7 %	B-5
	Sidewalk widening, associated regrading and rockfill ribs		1.48	4.6 %	B-6
STA 1+234	Existing Conditions	4.5 m @ 4.2H:1V	1.33	N/A	B-7
	Sidewalk widening and associated regrading		1.34	0.2 %	B-8
STA 1+203	Existing Ground	4.6 m @ 4.0H:1V	1.30	N/A	B-9
	Sidewalk widening and associated regrading	5.1 m @ 4.3H:1V	1.33	2.4 %	B-10

Closure

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All information provided in this report is subject to our standard terms and conditions for engineering services, a copy of which is provided to each of our clients with the original scope of work or standard engineering services agreement. If these conditions are not attached, and you are not already in possession of such terms and conditions, contact our office and you will be promptly provided with a copy.

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We thank you for the opportunity to provide engineering services on this assignment. If you have any questions regarding the findings or recommendations presented, please contact the undersigned at your earliest convenience.

Kind Regards,

TREK Geotechnical
Per:

PROVINCE OF MANITOBA
2018/05/17
M. J. VAN HELDEN
Member
24729
REGISTERED PROFESSIONAL ENGINEER

Michael Van Helden, Ph.D., P.Eng.
Senior Geotechnical Engineer

Reviewed By:

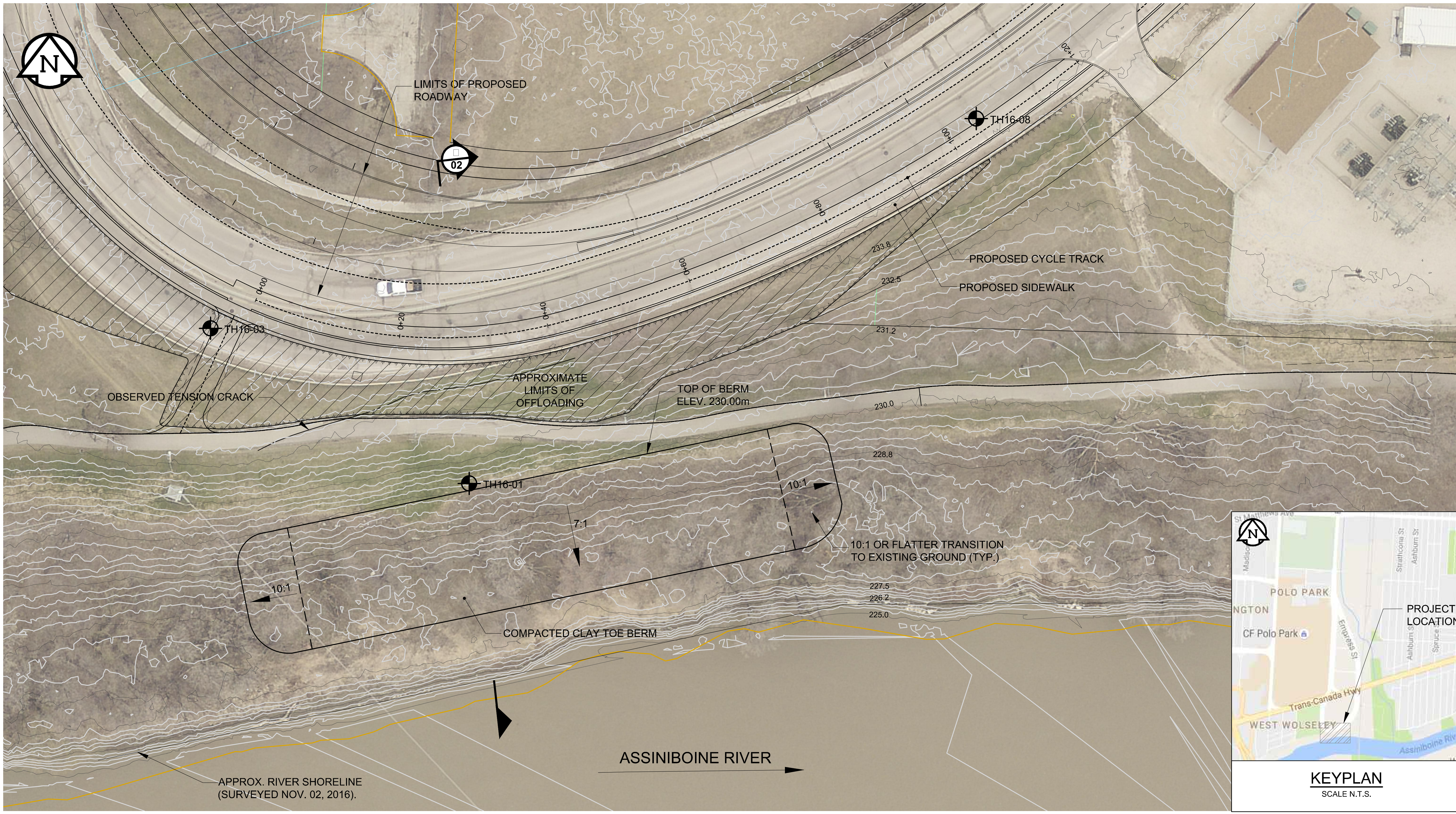
Nelson John Ferreira, M.Sc., Ph.D., P.Eng.,
Senior Geotechnical Engineer

ENGINEERS
GEOSCIENTISTS
MANITOBA
Certificate of Authorization
TREK GEOTECHNICAL INC.
No. 4877 Date: May 17, 2018

Attach.

Figures

0035 037 00
Morrison Hershfield Ltd.
Regional Street Reconstruction - Empress Street & Portage Avenue to St. Matthews Avenue



0 5 10 15 20 25 m
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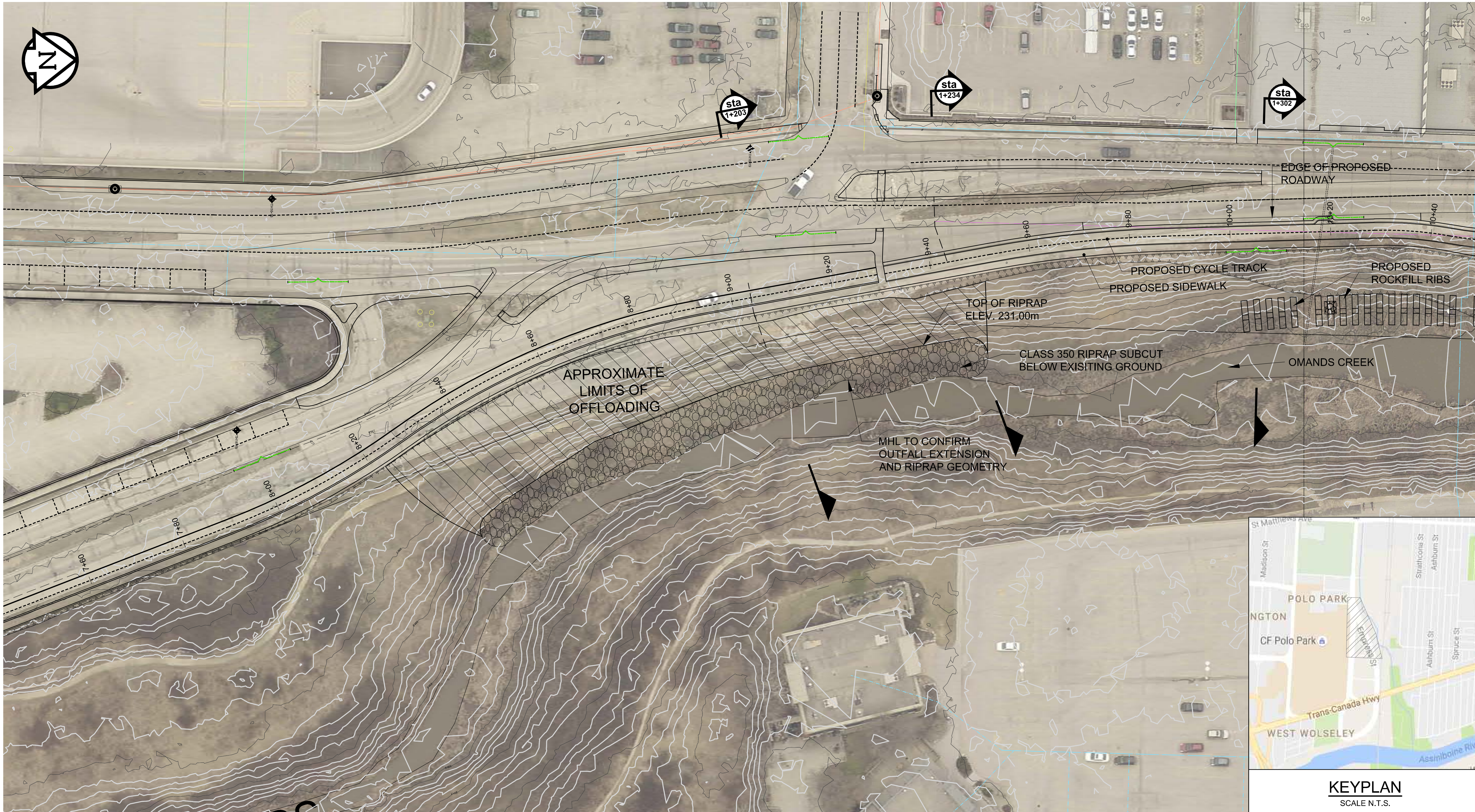
LEGEND: TEST HOLE (TREK, 2016)

NOTES:
1. AERIAL IMAGE FROM THE CITY OF WINNIPEG 2016.
2. CONTOURS FROM LIDAR SURVEY PROVIDED BY MORRISON HERSHFIELD LTD.
3. ROADWAY AND PATHWAY RECONFIGURATION PROVIDED BY MORRISON HERSHFIELD LTD.

Figure 01
Assiniboine Riverbank Plan

0035 037 00
Morrison Hershfield Ltd.
Regional Street Reconstruction - Empress Street & Portage Avenue to St. Matthews Avenue



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LEGEND:
TEST HOLE (TREK, 2016)

NOTES:
1. AERIAL IMAGE FROM THE CITY OF WINNIPEG 2016.
2. CONTOURS FROM LIDAR SURVEY PROVIDED BY MORRISON HERSHFIELD LTD.
3. ROADWAY AND PATHWAY RECONFIGURATION PROVIDED BY MORRISON HERSHFIELD LTD.

Figure 02
Omands Creek Plan - Natural Creek Section

Fig 003 2017-03-15 Plan and Sections 0 A SL.dwg 4/26/2018 1:52:14 PM

ANSI full Bleed B (11.00 x 17.00 Inches)

0 10 20 30 40 50 m
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LEGEND:
TEST HOLE (TREK, 2016)

NOTES:
1. AERIAL IMAGE FROM THE CITY OF WINNIPEG 2016.
2. CONTOURS FROM LIDAR SURVEY PROVIDED BY MORRISON HERSHFIELD LTD.
3. ROADWAY AND PATHWAY RECONFIGURATION PROVIDED BY MORRISON HERSHFIELD LTD.

Figure 03
Omands Creek Plan
Straightened Creek Section

Appendix B

Slope Stability Analysis Results

0035 037 00
Empress Overpass

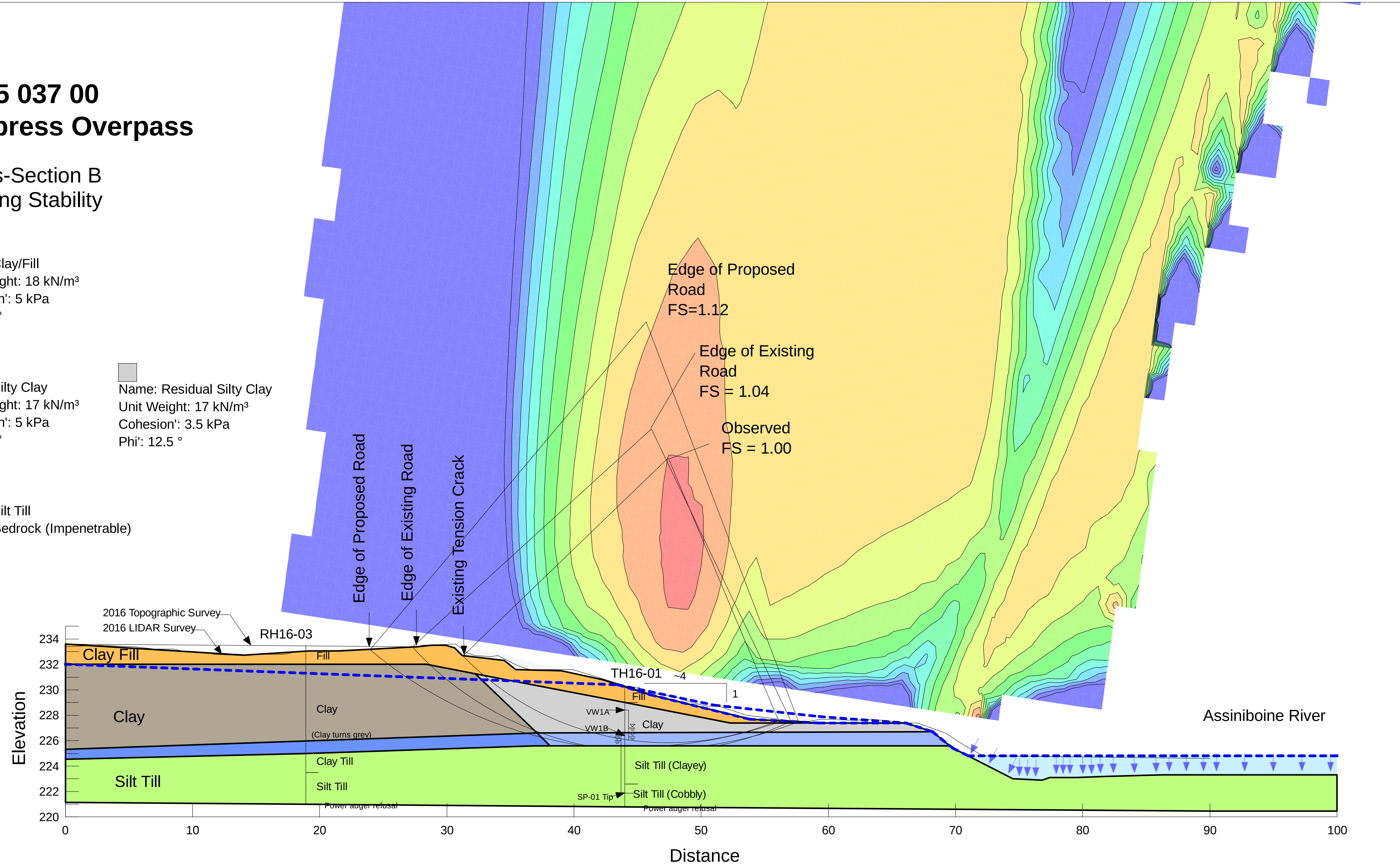
Cross-Section B
Existing Stability

Name: Clay/Fill
Unit Weight: 18 kN/m³
Cohesion': 5 kPa
Phi': 17 °

Name: Silty Clay
Unit Weight: 17 kN/m³
Cohesion': 5 kPa
Phi': 14 °

Name: Residual Silty Clay
Unit Weight: 17 kN/m³
Cohesion': 3.5 kPa
Phi': 12.5 °

Name: Silt Till
Model: Bedrock (Impenetrable)



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Model Output B-1
Cross-Section B

0035 037 00
Morrison Hershfield Ltd.
Empress Str. Stability Asessment

0035 037 00
Empress Overpass

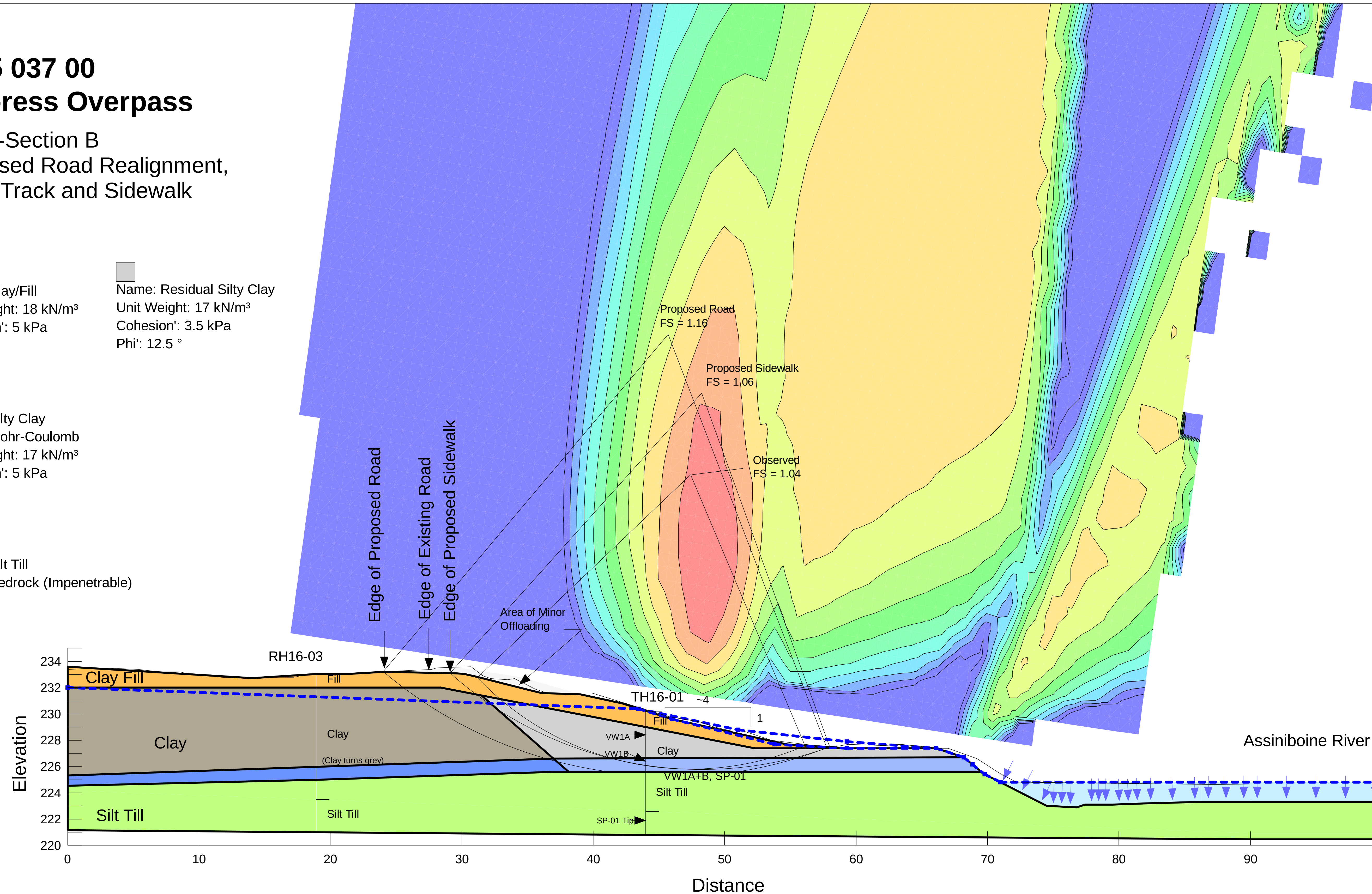
Cross-Section B
Proposed Road Realignment, Cycle Track and Sidewalk

Name: Clay/Fill
Unit Weight: 18 kN/m³
Cohesion': 5 kPa
Phi': 17 °

Name: Residual Silty Clay
Unit Weight: 17 kN/m³
Cohesion': 3.5 kPa
Phi': 12.5 °

Name: Silty Clay
Model: Mohr-Coulomb
Unit Weight: 17 kN/m³
Cohesion': 5 kPa
Phi': 14 °

Name: Silt Till
Model: Bedrock (Impenetrable)



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Model Output B-2
Cross-Section B

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0035 037 00
Empress Overpass

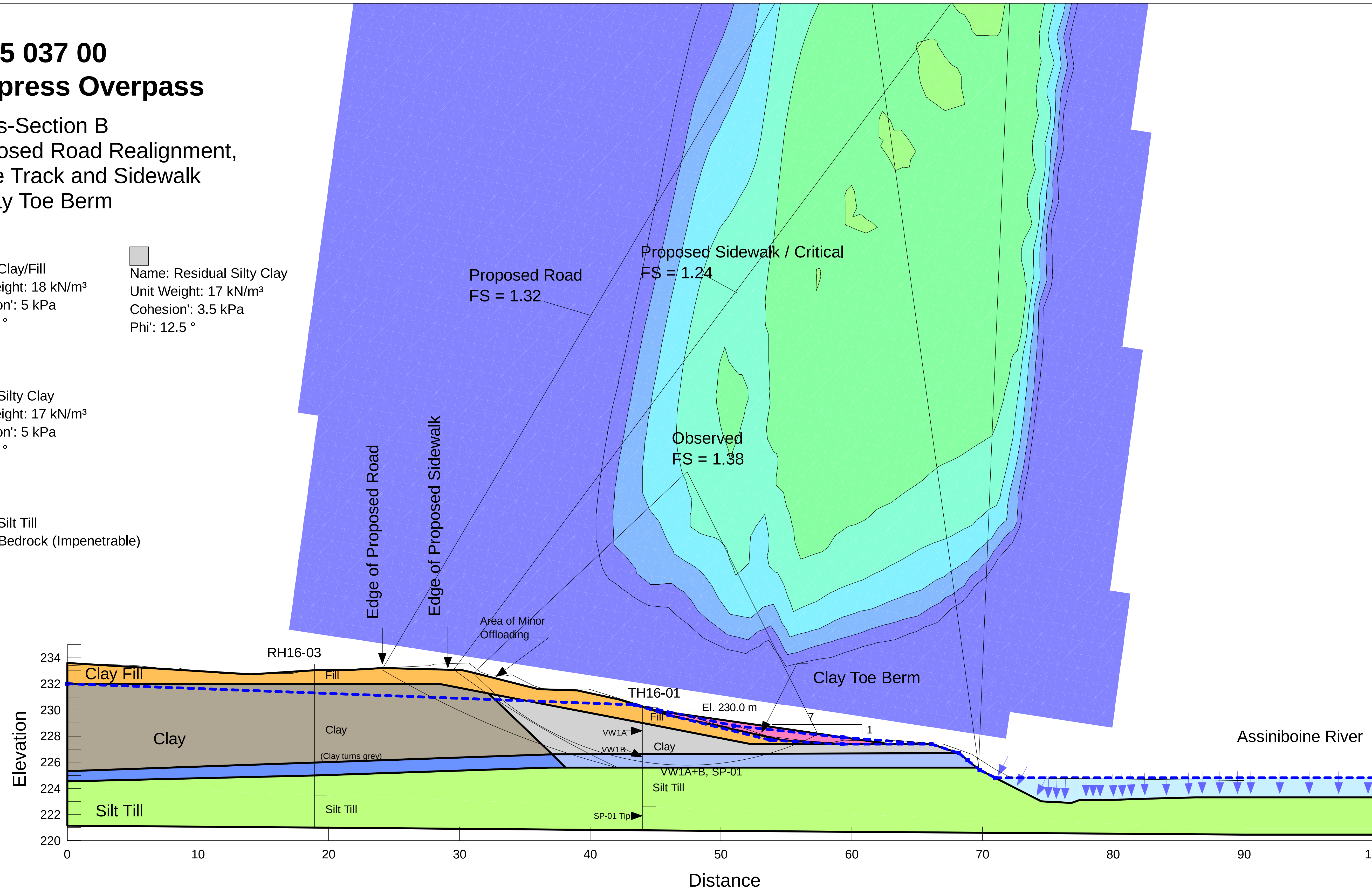
Cross-Section B
Proposed Road Realignment,
Cycle Track and Sidewalk
+ Clay Toe Berm

Name: Clay/Fill
Unit Weight: 18 kN/m³
Cohesion': 5 kPa
Phi': 17 °

Name: Residual Silty Clay
Unit Weight: 17 kN/m³
Cohesion': 3.5 kPa
Phi': 12.5 °

Name: Silty Clay
Unit Weight: 17 kN/m³
Cohesion': 5 kPa
Phi': 14 °

Name: Silt Till
Model: Bedrock (Impenetrable)



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Model Output B-3
Cross-Section B

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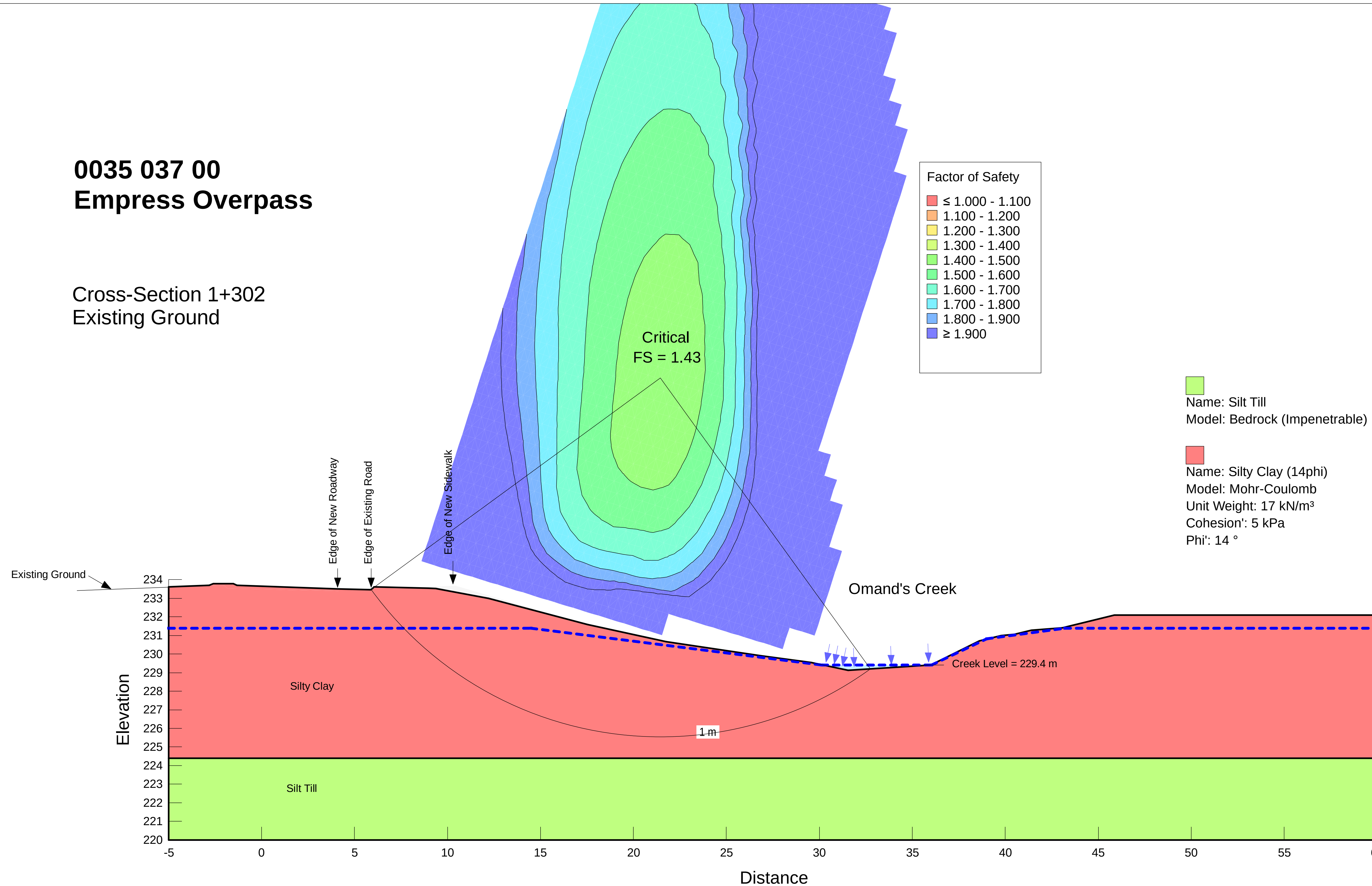
0035 037 00
Empress Overpass

Cross-Section 1+302
Existing Ground



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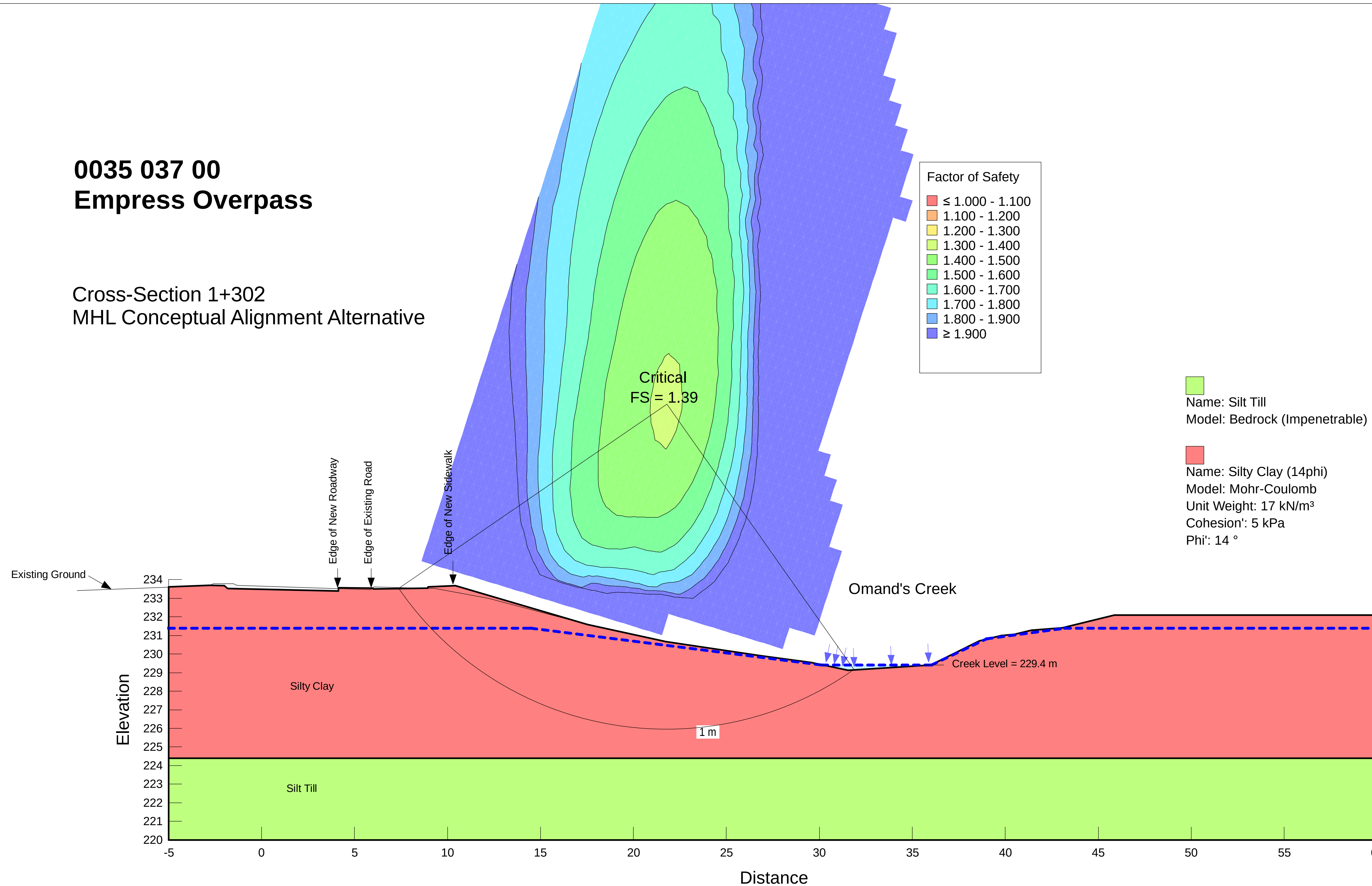
Empress Overpass

Cross-Section 1+302
MHL Conceptual Alignment Alternative



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Model Output B-5
Cross-Section STA 1+302

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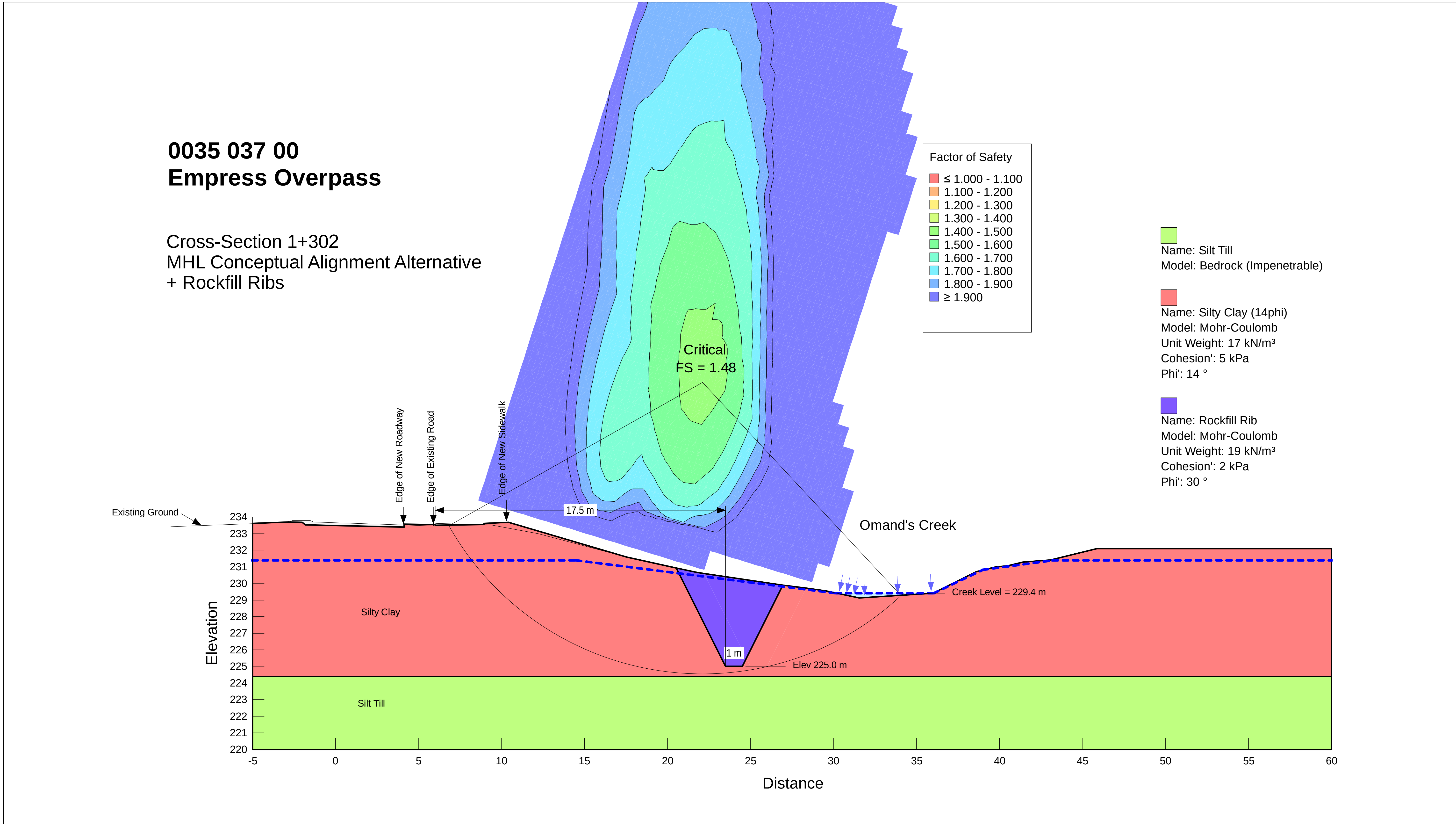

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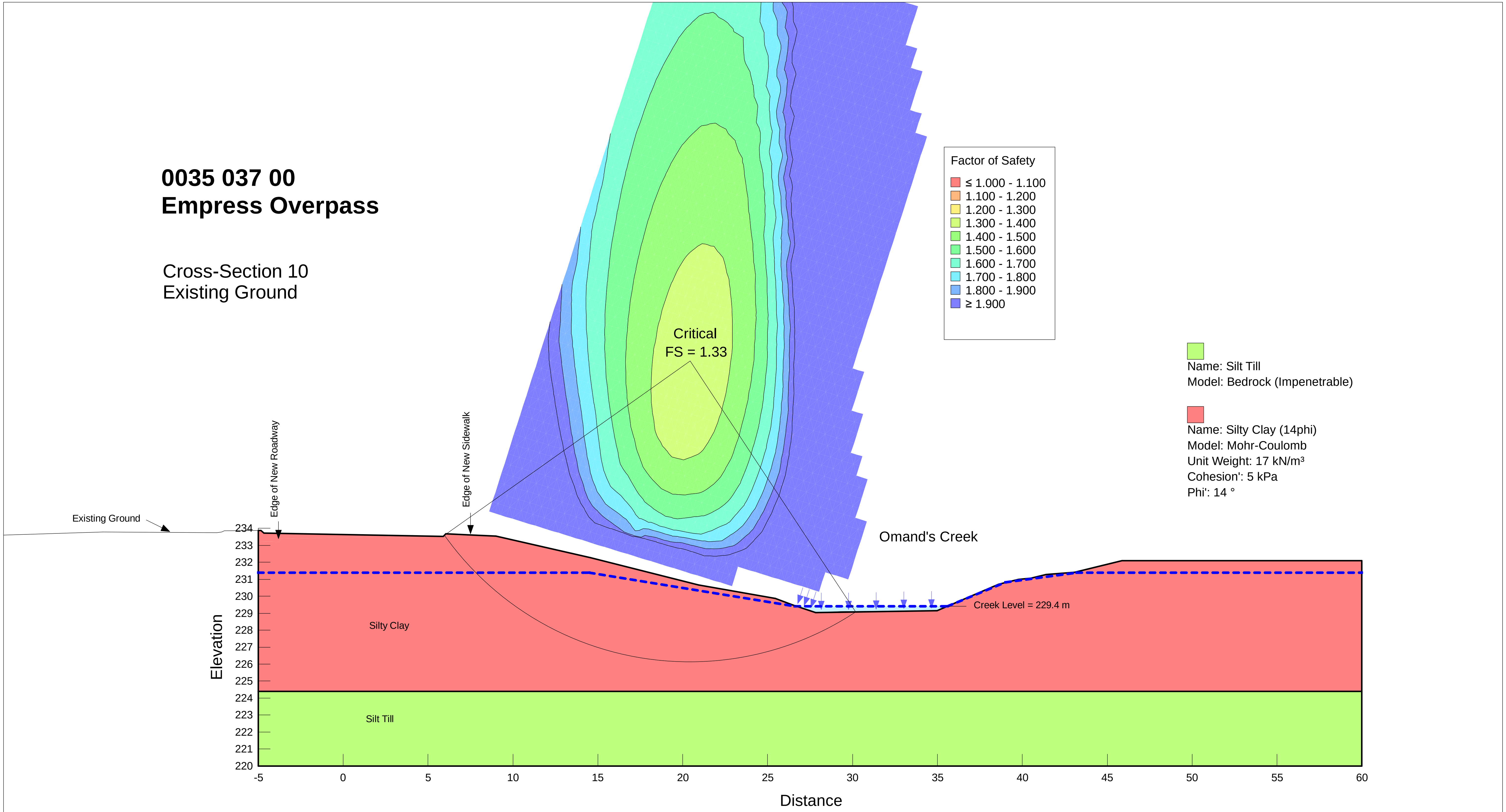
0035 037 00
Empress Overpass

Cross-Section 10
Existing Ground



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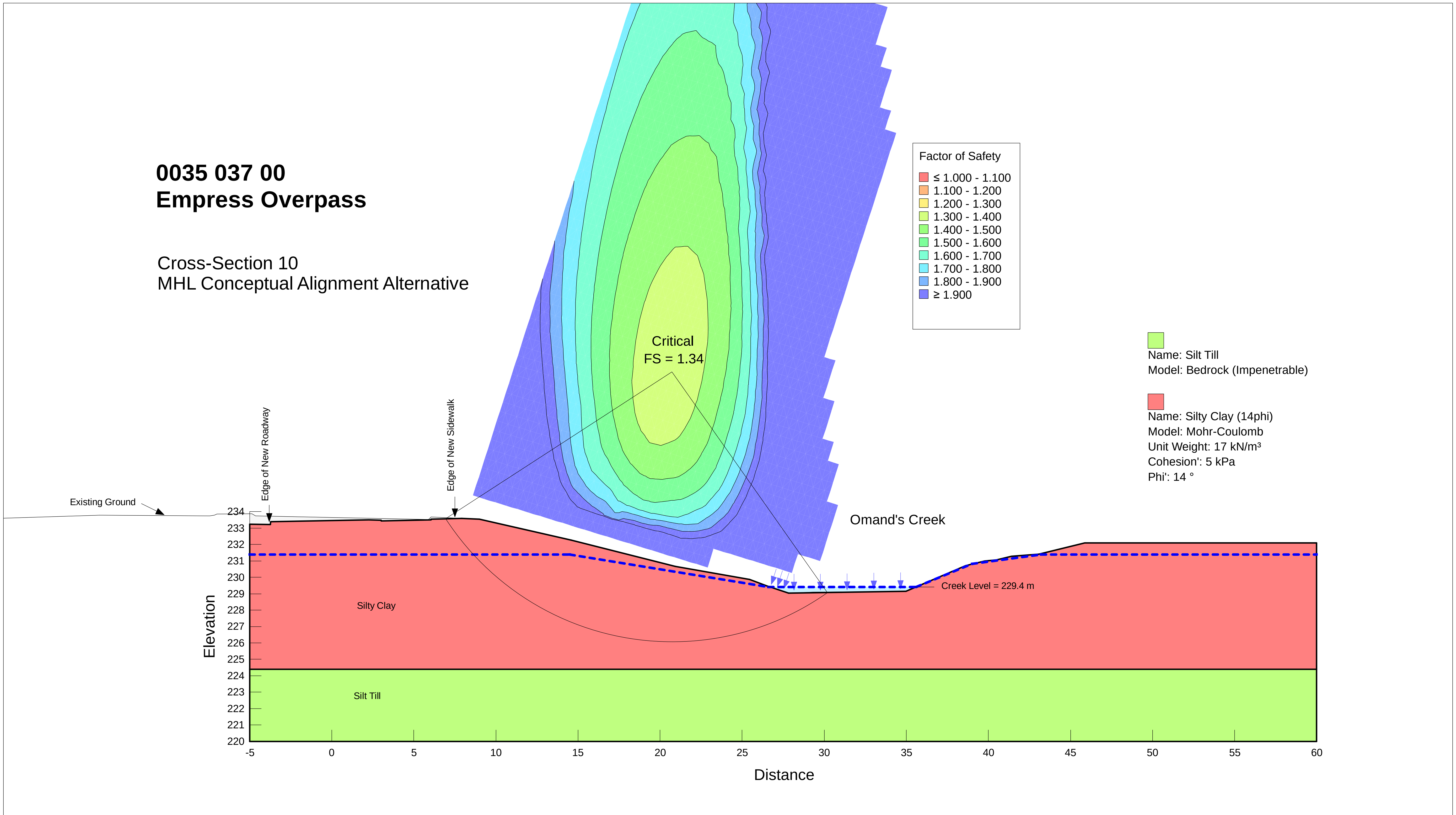

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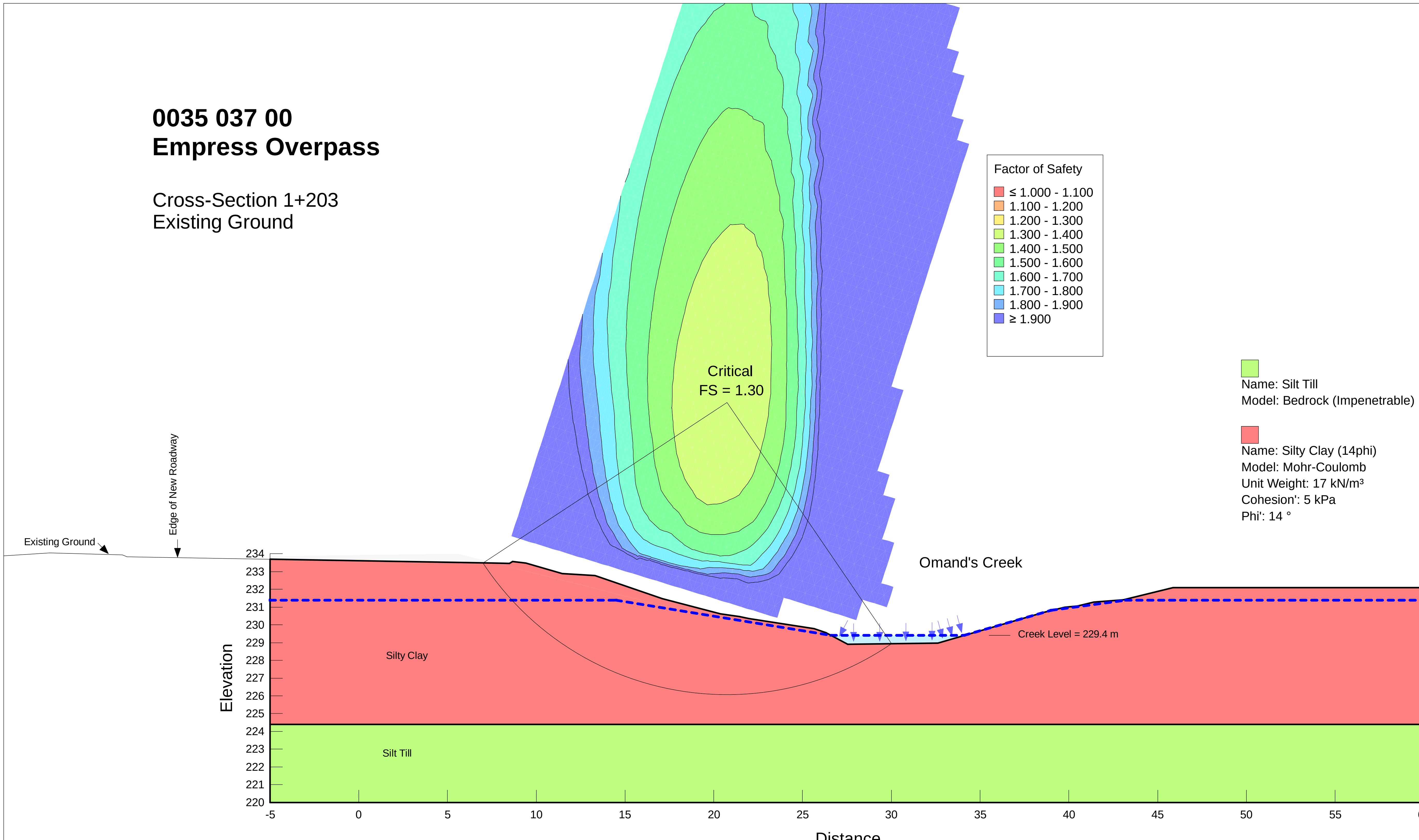
M-003.gsz

Model Output B-8
Cross-Section 10

0035 037 00
Empress Overpass

Cross-Section 1+203
Existing Ground



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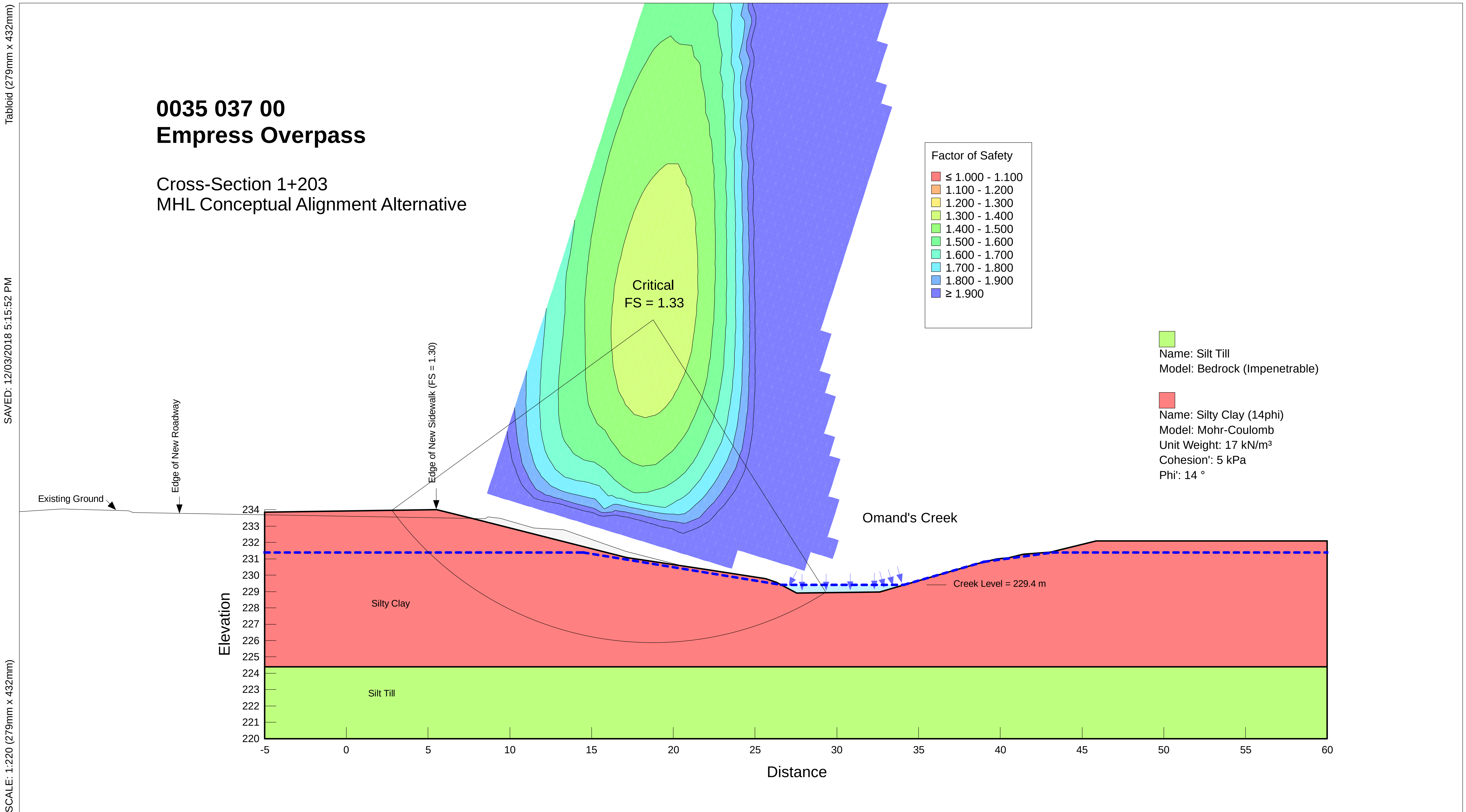

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Model Output B-10
Cross-Section STA 1+203