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FOUNDATION INVESTIGATION & DESIGN REPORT

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THE CORPORATION OF THE CITY OF MISSISSAUGA

PROCUREMENT No. FA. 49.304-05

WO 2008-11036

Terraprobe

GEOCREs No:
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Terraprobe

Consulting Geotechnical & Environmental Engineering
Construction Materials Engineering, Inspection & Testing

DRAFT

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WO 2008-11036

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February 08, 2008

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FOUNDATION INVESTIGATION REPORT
HIGHWAY 401 UNDERPASS STRUCTURE AT CREEKBANK ROAD
THE CORPORATION OF THE CITY OF MISSISSAUGA, ONTARIO
PROCUREMENT No. FA. 49.304-05

PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted at the site where a new underpass structure will be constructed to carry Creekbank Road over the existing express and collector lanes of Highway 401 in the City of Mississauga, Ontario.

The purpose of this investigation was to explore the subsurface conditions at the site and based on the data obtained, to provide a borehole location plan, records of boreholes, stratigraphic profile and cross-sections, laboratory test results and a description of the subsurface conditions. A model of the subsurface conditions was developed from the data obtained.

Terraprobe conducted the investigation as a sub-consultant to Giffels Associates Limited (Giffels), under The Corporation of The City of Mississauga Procurement Number FA. 49.304-05.

2 SITE DESCRIPTION

The site is located in an urban setting about 755± m east of the Dixie Road underpass and about 565± m west of Etobicoke Creek which flows southerly under Highway 401. There is commercial development on the south side of Highway 401. On the north side of Highway 401 are open lands under the jurisdiction of the Greater Toronto Airports Authority.

Creekbank Road is a north-south oriented roadway that ends about 330± m north of its intersection with Matheson Boulevard. Highway 401 is a 20-lane divided freeway (with express and collectors) conforming to a rural cross-section.

The site is located in the physiographic region of Southern Ontario referred to as the Peel Plain whose topography slopes gradually and gently towards Lake Ontario. Etobicoke Creek and other rivers have cut deep valleys across the Peel Plain.

The Peel Plain is known to consist of generally clayey and silty soils that cover the central portion of the regions of York, Peel and Halton[1]. There are exceptions to be noted in these major soil groups. Trains of sandy alluvium can be found at various places in the stream valleys. These overburden soils are underlain by the Georgian Bay Formation.

[1] Chapman and Putnam, "The Physiography of South Ontario", 3rd Edition, 1984.

The Georgian Bay Formation is of Middle Ordovician Age. This unit consists essentially of blue-grey and green-grey shales with interbeds of calcareous sandstone, siltstone and grey argillaceous limestone. The limestone and siltstones are very fossiliferous.

Stress relief features such as folds and faults are common in the Georgian Bay Formation. In these features the rock is heavily fractured and sheared, and contains layers of shale rubble and clay. Weathering is much deeper than the surrounding rock in these features and often there can be lateral displacement of the stress relief features resulting in sound unweathered bedrock overlying fractured and weathered bedrock. Weathering near the soil contact is generally accompanied by softening and increased fissility.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project were carried out during the period December 09, 2007 to January 08, 2008 and consisted of drilling and sampling fourteen boreholes to depths ranging from 13.5 m to 17.7 m. The boreholes were numbered CR-1, CR-1A, CR-2, CR-3, CR-4, CR-4A, CR-5, CR-5A, CR-6, CR-7, CR-8, CR-8A, CR-9 and CR-10. Their approximate locations are shown on the attached Borehole Locations and Soil Strata Drawings in Appendix C.

The borehole locations, their coordinates and geodetic elevations, were established in the field by surveyors from Strada Survey Inc. based on drawings provided by Giffels. Access to some specific borehole locations was difficult due to the locally steep slopes near the ditch line of the highway or conflicting utilities. These boreholes were relocated to be as close as feasible to their staked out locations while allowing safe operation of the drill rig. Terraprobe obtained utility clearances prior to drilling.

The drilling, sampling and in-situ testing operations were conducted with truck and track mounted drill rigs owned and operated by DBW Drilling Limited of Ajax, Ontario, Groundworks Drilling Limited of Toronto, Ontario and Drill Tech Drilling of Newmarket, Ontario. Solid stem auger drilling techniques were used to advance the boreholes and samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT) in the overburden soils. One borehole at each abutment and pier location was also advanced 3.0 m to 3.1 m into bedrock by NQ size diamond coring techniques.

Groundwater conditions in the open boreholes were observed throughout the drilling operations. A standpipe piezometer consisting of 19 mm PVC pipe with a slotted screen enclosed in sand was installed in selected boreholes to permit longer term groundwater level monitoring. The locations and completion details of the piezometers are presented in Table 3.1.

Table 3.1 – Piezometer Installation Details

Piezometer Location	Piezometer Details	
	Tip Depth/ Elevation (m)	Completion Details
CR2	15.4/142.4	Piezometer with 3 m slotted screen installed with filter sand to 11.5 m and bentonite seal from 11.5 m to ground surface.
CR7	15.3/142.0	Piezometer with 3 m slotted screen installed with filter sand to 11.5 m and bentonite seal from 11.5 m to ground surface.
CR8A	16.8/140.3	Piezometer with 3 m slotted screen installed with filter sand to 13.1 m and bentonite seal from 13.1 m to ground surface.
CR9	15.3/142.1	Piezometer with 3 m slotted screen installed with filter sand to 11.6 m, bentonite seal from 11.6 m to 10.4 m, drill cuttings from 10.4 m to 1.2 m, and bentonite seal from 1.2 m to ground surface.
CR10	13.3/146.5	Piezometer with 3 m slotted screen installed with filter sand to 9.7 m, bentonite seal from 9.7 m to 8.5 m, drill cuttings from 8.5 m to 1.2 m, and bentonite seal from 1.2 m to ground surface.

Members of Terraprobe technical staff supervised the drilling and sampling operations on a full time basis. The supervisors logged the boreholes and processed the recovered soil samples for transport to Terraprobe's Brampton laboratory for further examination and testing.

4 LABORATORY TESTING

The recovered soil samples were subjected to Visual Identification (VI) and natural moisture content determination. Selected samples were subjected to gradation analysis and Atterberg Limits tests were also conducted on selected samples retrieved from the cohesive deposits. The results of this testing program are shown on the Record of Borehole sheets in Appendix A. The grain size distribution curves and a plasticity charts are illustrated in Appendix B.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets in Appendix A. Details of the encountered soil stratigraphy are presented in this appendix and on the "Borehole Locations and Soil Strata" drawings in Appendix C. An overall description of the stratigraphy is given in the following paragraphs. However, the factual data presented in the Record of Borehole Sheets governs any interpretation of the site conditions.

In general, the site is underlain by a flexible pavement structure, topsoil, fill and native overburden deposits of very stiff to hard silty clay till, very dense sand and gravel and very dense silt and sand.

5.1 Topsoil

Topsoil approximately 100 mm to 200 mm thick was encountered at the site. Topsoil thickness may vary between and beyond the boreholes.

5.2 Pavement Structure

Four boreholes were extended through the existing paved shoulders of Highway 401. These boreholes encountered a pavement structure consisting of 130 mm to 150 mm thick asphalt concrete underlain by 450 mm to 460 mm thick granular material consisting of sand and gravel with trace to some silt. Sand and gravel and gravelly sand fill were also encountered in the boreholes drilled near the outer shoulders of the highway collector lanes. The granular material extends to depths ranging from 0.5 m to 0.9 m below ground surface or to elevations ranging from 157.3 m to 156.4 m.

The grain size distribution curves of four samples of the granular fill are illustrated in Figure B1. The results show a grain size distribution consisting of 26-45% gravel, 43-51% sand, 9-18% silt and 2-6% clay size particles.

Standard Penetration tests in this fill material yielded 'N' values ranging from 10 to more than 100 blows for 0.3 m penetration indicating a compact to very dense relative density.

The moisture content of samples of the granular fill ranged from 2% to 18% by weight.

5.3 Silty Clay Fill

A layer of silty clay fill was encountered in some of the boreholes extending to depths ranging from 0.7 m (Elev. 159.1 m) to 1.4 m (Elev. 156.3 m) below ground surface.

The grain size distribution curves of samples of this fill material are presented in Figure B2. These results show a grain size distribution consisting of 0-10% gravel, 18-28% sand, 45-51% silt and 17-31% clay size particles.

Standard Penetration tests in the fill material yielded 'N' values ranging from 6 to 26 blows for 0.3 m penetration indicating a firm to very stiff consistency.

The moisture content of samples from this deposit ranged from 11% to 22% by weight.

5.4 Silty Clay Till

Across the site a major deposit of silty clay till containing occasional sand seams and partings was encountered. This deposit was fully penetrated in some of the boreholes where it extends to depths ranging from 11.7 m (Elev. 145.7 m) to 14.6 m (Elev. 143.4 m) below ground surface. One shallow borehole (CR10) was terminated in this deposit at a depth of 13.5 m (Elev. 146.3 m) below ground surface.

The grain size distribution curves of samples of this silty clay till are presented in Figures B3 to B6. These results show a grain size distribution consisting of 0-11% gravel,

1-37% sand, 37-69% silt and 13-30% clay size particles. Till soils are also known to contain cobbles and boulders due to their mode of deposition.

Samples of the silty clay till were also subjected to Atterberg Limits tests and the results are illustrated in Figure B7 to B9. The summarized index values from these tests are presented herein.

Liquid Limit:	19-31%
Plastic Limit:	13-20%
Plasticity Index:	6-11%
Natural Moisture Content:	7-18%

These values are characteristic of clayey soils of low plasticity.

Standard Penetration tests in this silty clay till deposit yielded 'N' values ranging from 17 to more than 100 blows for 0.3 m penetration indicating a very stiff to hard consistency.

The moisture content of samples from this deposit ranged from 4% to 21% by weight.

5.5 Sand and Gravel

An 800 mm thick layer of sand and gravel was encountered in Borehole CR5 extending to a depth of 9.9 m (Elev. 147.7 m) below ground surface.

A sample from this granular deposit was subjected to a grain size analysis and the results are presented in Figure B10. These results show a grain size distribution consisting of 38% gravel, 38% sand, 19% silt and 5% clay size particles.

A Standard Penetration test conducted in this sand and gravel layer gave an 'N' value more than 100 blows for 0.3 m penetration. Based on this result the sand and gravel is considered to have a very dense relative density.

The moisture content of a sample retrieved from this stratum was 9% by weight.

5.6 Silt and Sand (Possible Till)

A deposit of silt and sand was encountered in Borehole CR9 where it extends to a depth of 13.7 m (Elev. 143.7 m).

Refer to Figure B11 where the grain size distribution curve of a sample of the silt and sand is illustrated. The results show a grain size distribution consisting of 6% gravel, 41% sand, 45% silt and 8% clay size particles. Cobbles and boulders can also be expected within the matrix of till soils.

A blow count of more than 100 blows for 0.3 m penetration was obtained from a Standard Penetration test in this stratum indicating a very dense relative density.

The moisture content of a sample from this deposit was 13% by weight.

5.7 Bedrock (Georgian Bay Formation)

The overburden soils described above are underlain by the Georgian Bay Formation.

The bedrock was penetrated in some of the boreholes by the solid stem augering method. However, the penetrability of the bedrock by the auger does not necessarily indicate poor rock quality.

Bedrock was also proved by coring one borehole at each abutment and pier location. Table 5.1 summarizes the bedrock depth and the elevations to the top of bedrock. The data indicates a generally flat bedrock surface across the site.

Table 5.1 – Depth to Bedrock

Location	BH Number	Depth to Bedrock (m)	Top of Bedrock Elevation (m)
North Abutment	CR1	14.6	143.4
	CR1A	14.6	143.4
	CR2	14.3	143.5
South Abutment	CR7	13.7	143.6
	CR8	13.7	143.4
	CR8A	13.7	143.4
North Pier	CR3	13.7	144.0
	CR4	13.7	143.9
	CR4A	13.7	143.9
South Pier	CR5	13.7	143.9
	CR5A	13.7	143.9
	CR6	13.7	143.8
North Approach	CR9	13.7	143.7

The bedrock is described as moderately to highly weathered and its colour is grey. It is thickly bedded and weak to medium strong with moderately strong to strong limestone interbeds. Total core recovery in the bedrock ranged from 42% to 100%. The RQD values generally ranged from 0% to 69% indicating a rock mass of very poor to fair rock quality. Clay seams ranging from 25 mm to 150 mm in thickness were also encountered in the rock.

5.8 Water Levels

Stand pipe piezometers were installed in Boreholes CR2, CR7, CR8A, CR9 and CR10. The water level readings measured on a separate visit made after the completion of drilling are presented in Table 5.2.

Table 5.2 – Water Level Measurements

Borehole	Date	Water Levels	
		Depth (m)	Elevation (m)
CR2	January 14, 2008	5.4	152.4
CR7	January 14, 2008	4.0	153.3
CR8A	January 14, 2008	4.7	152.4
CR9	January 14, 2008	4.6	152.8
CR10	January 14, 2008	6.6	153.2

These observations suggest that the local groundwater level at the site is likely to exist at elevations ranging between 152.4 m and 153.3 m.

It should also be noted that perched water tables can also be expected to occur where wet sand seams are encountered within the silty clay till. All groundwater observations at this site are short term and the levels are expected to fluctuate seasonally and after severe weather events.

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FOUNDATION DESIGN REPORT
HIGHWAY 401 UNDERPASS STRUCTURE AT CREEKBANK ROAD
THE CORPORATION OF THE CITY OF MISSISSAUGA, ONTARIO
PROCUREMENT No. FA. 49.304-05

PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

6 GENERAL

This report presents interpretation of the geotechnical data in the factual report and provides geotechnical design recommendations to assist the design team to select and design a suitable foundation system and approaches for the proposed structure.

A three span 20.7 m wide underpass structure is required to carry Creekbank Road over the east bound and west bound express and collectors of Highway 401. The span length between the north abutment and north pier will be 45 m and a span length of 35 m is proposed between the south abutment and south pier. A span length of 52.85 m is proposed between piers.

The proposed finished grades at the structure will be about Elev. 165.2± m at the north abutment and Elev. 165.1± m at the south abutment. Finished grades of Elev. 165.8± m and Elev. 165.7± m are proposed at the north and south piers respectively.

The discussion and recommendations presented in this report are based on our understanding of the project and the factual data obtained in the course of the investigations.

7 STRUCTURE FOUNDATIONS

The proposed bridge is a three span structure with two abutments and two piers as foundation elements.

The stratigraphy encountered across the site consists of a flexible pavement structure and/or topsoil and firm to very stiff silty clay fill underlain by very stiff to hard silty clay till, very dense sand and gravel and very dense silt and sand (possible till). The groundwater level exists at elevations ranging between 152.4 m and 153.3 m and perched water can be expected within the more permeable sand seams embedded within the silty clay till.

Consideration was given to the following foundation types:

- Spread footings
- Augered Caissons (drilled shafts)
- Driven piles

A comparison of the foundation alternatives based on advantages and disadvantages of each option is included in Appendix D.

7.1 Spread Footings

Based on the subsurface stratigraphy encountered at this site and the vertical alignment of Creekbank Road, spread footings are feasible for supporting the piers provided the design loads are not excessive.

Consideration was also given to supporting the abutments on spread footings but the proposed geometry will require abutment stems greater than 8 m in height which would likely be impractical. Furthermore it is recognized that an integral abutment is being proposed which would rule out this foundation option from further consideration.

The recommended founding depths and geotechnical resistances for footings (minimum footing width of 2 m) founded on undisturbed competent natural soils are tabulated below.

Table 7.1 – Geotechnical Resistances at Abutment Locations

Borehole Location	Existing Ground Surface Elev. (m)	Recommended Bottom of Footing Level Below Existing Ground Surface (m)	Founding Elevation (m)	Factored Geotech. Resistance at ULS (kPa)	Geotech. Resistance at SLS (kPa)	Subgrade Material
North Pier CR3 & CR4	157.7± (CR3) 157.6± (CR4)	Below 1.4± (CR3) Below 1.0±(CR4)	Below 156.3 Below 156.6	525	350	Silty Clay Till
South Pier CR5 & CR6	157.6± (CR5) 157.5± (CR6)	Below 0.6± (CR5) Below 0.6 ±(CR6)	Below 157.0 Below 157.5	525	350	Silty Clay Till

The values given in Table 7.1 are for vertical, concentric loads only. Effects of load inclination and eccentricity should be taken into account as illustrated in CHBDC 2006, Clause 6.7.3 and Clause 6.7.4.

The SLS values quoted above corresponds to a settlement of up to 25 mm, a significant portion of which will be complete by the end of construction.

The silty clay till is susceptible to disturbance when wet. It is therefore recommended that allowance be made to pour a 150 mm thick layer of lean concrete (mud mat) on the foundation bearing surfaces as soon as possible after excavation and approval.

Resistance to lateral forces/sliding resistance between the concrete footing and the subgrade soils should be evaluated in accordance with the CHBDC 20006. Assume an ultimate coefficient of friction of 0.6 for the very stiff to hard silty clay till.

7.2 Augered Caissons (Drilled Shafts)

Consideration can be given to supporting the abutments and piers on augered end bearing caissons. The recommended axial resistances of end bearing caissons of various diameters are given in Table 7.2.

Table 7.2 - Axial Resistance of End Bearing Caissons

Caisson Diameter (m)	Factored Axial Resistance U.L.S (kN)	SLS (25 mm Settlement) (kN)
1.0	5,400	2,700
1.5	10,400	5,200

The corresponding base elevations and estimated depths for end bearing caissons outlined in the table above are given in Table 7.3.

Table 7.3 - Base Elevations of End Bearing Caissons

Support Location	Borehole No.	Estimated Base Elevation (m)	*Estimated Depth (m)	Founding Stratum
North Abutment	CR1 & CR2	148.0	13.5	Silty Clay Till
South Abutment	CR7 & CR8	148.0	13.0	Silty Clay Till
North Pier	CR3 & CR4	147.0	10.0	Silty Clay Till
South Pier	CR5 & CR6	147.0	10.0	Silty Clay Till

* Estimated depths based on underside abutment elevations of Elev. 161.5 ±m (north abutment) and Elev. 161.0 ±m (south abutment) and caisson cut off elevation of Elev. 157 ±m at the piers.

Higher geotechnical resistances are available but this will require extending the caissons to bedrock. However, this will require excavating the hard silty clay till an additional 3.0 m to 4.6 m below the base elevations given in Table 7.3. Moreover, the caissons will have to be socketted at least 1.5 m into the shale bedrock.

End bearing caissons socketted approximately 1.5 m into the shale bedrock can be designed using a factored axial resistance at ULS of 6.5 MPa. For 1 m and 1.5 m diameter caissons this equates to factored axial resistances at ULS of 5,000 kN and 11,000 kN respectively. Serviceability Limit State resistances will not apply to caissons founded on the shale bedrock.

7.2.1 Caisson Installation

Caisson installation should be in accordance with Special Provision No. 903S01.

Caisson installation at the abutment and pier locations would be carried out through silty clay fill, and silty clay till containing occasional water bearing sand seams and partings. It is recommended that the caissons be constructed by using temporary steel liners to support the sidewalls and to provide seepage cut-off as and where required. This construction scheme will also allow for hand cleaning and inspection of the bearing surface.

The hard glacial till may contain random boulder inclusions. In this regard caisson augering equipment should be capable of removing and penetrating boulders and any other obstructions.

Caissons designed as end bearing on the shale bedrock will require excavations that will have a higher probability of encountering boulders compared to caissons designed for end bearing. Boulders encountered during construction can present costly construction

challenges, especially at the pier locations. This risk (and cost) can be reduced if end bearing caissons are founded on the hard silty clay till. We would therefore recommend that this aspect be taken into consideration in the design.

The concrete should be placed using good tremie techniques. The liner should be withdrawn as concrete is placed. During liner withdrawal, the level of concrete in the caisson hole must always be at least 0.5 m above the bottom of the temporary liner.

7.3 Driven Piles

It is recognised that an integral abutment bridge (which requires pile foundations) offers significant long term advantages. The subsurface conditions at the site are considered suitable for the design of foundations supported on steel H-piles.

7.3.1 Axial Resistance

Two steel pile sections believed to be currently available have been considered for use in the proposed foundations.

Piles driven at the abutment and pier locations and encountering effective refusal in the hard silty clay till should be designed on the basis of the concentric, axial geotechnical resistances given in Table 7.4. The actual pile tip elevations will be controlled as described in Section 7.3.3 Pile Installation.

Table 7.4 – Axial Resistance of Various Pile Sections

Location	PILE TYPE - HP 310x110				
	Reference Boreholes	Estimated Pile Tip Elevation (m)	Founding Stratum	Factored Axial Resistance U.L.S (kN)	SLS (25 mm Settlement) (kN)
North Abutment	CR1	148.5±	Silty Clay Till	1600	1200
	CR2	149.5±	Silty Clay Till		
South Abutment	CR7	147.5±	Silty Clay Till		
	CR8	147.5±	Silty Clay Till		
North Pier	CR3	148.5±	Silty Clay Till		
	CR4	148.0±	Silty Clay Till		
South Pier	CR5	147.5±	Silty Clay Till		
	CR6	148.0±	Silty Clay Till		

Table 7.4 – Axial Resistance of Various Pile Sections (Cont'd)

Location	PILE TYPE – HP 360X132				
	Reference Boreholes	Estimated Pile Tip Elevation (m)	Founding Stratum	Factored Axial Resistance U.L.S (kN)	SLS (25 mm Settlement) (kN)
North Abutment	CR1	147.5±	Silty Clay Till	2100	1600
	CR2	148.5±	Silty Clay Till		
South Abutment	CR7	146.5±	Silty Clay Till		
	CR8	146.5±	Silty Clay Till		
North Pier	CR3	147.5±	Silty Clay Till		
	CR4	147.0±	Silty Clay Till		
South Pier	CR5	146.5±	Silty Clay Till		
	CR6	147.0±	Silty Clay Till		

The structural resistance of the pile should be checked by the structural designer.

The H-piles for the recommended foundation scheme will be driven to effective refusal. Piles will penetrate through till layers that are likely to contain cobbles and boulders. It is therefore recommended that the pile tips be fitted with driving shoes to provide reinforcement to the pile section.

The contract documents should contain a NSSP alerting the contractor to the fact that cobbles and boulders may be encountered in the soil. The NSSP should require the QVE to terminate driving before the pile is damaged by overdriving. If the required resistance according to the Hiley Formula is not achieved and further driving is likely to cause damage to the pile, pile driving should cease and the contract administrator should be notified. Suggested wording for the NSSP is included in Appendix F.

7.3.2 Pile Tips

Due to the possible presence of cobbles and boulders in the till layers; the tips of all piles should be fitted with driving shoes to protect the pile tip from damage.

7.3.3 Pile Installation

Pile installation should be in accordance with Special Provision No. 903S01.

The Contract Documents should contain a NSSP alerting the Contractor to the presence of cobbles and boulders in the till soils.

7.3.4 Pile Driving

Pile driving should be controlled by the Hiley Formula and an ultimate pile resistance to be specified by the designer in accordance with Clause 3.3.2 (b) Construction Stage of the Structural Manual. The appropriate pile driving note is "Piles to be driven in accordance with Standard SS 103-11 using an ultimate resistance of "R" kN per pile". Piles should be driven with a suitable hammer capable of delivering a rated energy of at least 55 kJ/blow, but not more than 70 kJ/blow. "R" must have the minimum values shown in Table 7.5.

Table 7.5 – Ultimate geotechnical Resistance of Piles

Pile	Ultimate Resistance (R) (kN)
HP 310X110	3,200 kN
HP 360X132	4,200 kN

7.3.5 Integral Abutment Considerations

The integral abutment design requires that the piles possess flexibility in the upper 3 m of the pile length. At both abutment locations the upper 3 m to 3.3 m of pile will lie in the RSS core. In order to provide the required flexibility in the piles, and to ensure that superstructure movement does not damage the RSS wall (false abutment) a 2-CSP system is recommended as per MTO SO-96-01. An outer CSP is placed around an inner sand filled CSP (about 600 mm in diameter).

After the pile is driven, the space between the pile and the inner CSP should be filled with sand. An NSSP should be included in the contract drawings specifying the gradation of the sand according to Table 7.6.

Table 7.6 – Integral Abutment Sand Grading

MTO Sieve Designation		Percentage Passing
2 mm	#10	100%
600 µm	#30	80%-100%
425 µm	#40	40%-80%
250 µm	#60	5%-25%
150 µm	#100	0%-6%

7.4 Downdrag

Downdrag on the caissons and/or piles is not considered to be an issue at this site.

7.5 Lateral Resistance

The lateral resistance of the caissons and piles may be calculated using a value for the coefficient of horizontal subgrade reaction (k_s) and ultimate lateral resistance (p_{ult}) as follows:

k_s = n_h z / D [cohesionless soils] (kN/m^3)

k_s = 67 S_u/D [cohesive soils] (kN/m^3)

p_{ult} = 3 γ z K_p [cohesionless soils] (kPa)

p_{ult} = 9 S_u [cohesive soils] (kPa)

where	z	=	depth of embedment of caisson or pile	(m)
	D	=	caisson diameter or pile width	(m)
	S_u	=	undrained shear strength (Table 7.7)	(kPa)
	n_h	=	coefficient of horizontal subgrade reaction (Table 7.7)	(kN/m^3)
	γ	=	unit weight (Table 7.7)	(kN/m^3)
	K_p	=	passive earth pressure coefficient $(1+\sin\phi)/(1-\sin\phi)$	
	ϕ	=	angle of internal friction (Table 7.7)	(degrees)

The above equations and recommended parameters may be used to analyze the interaction between a caisson or pile and the surrounding soil. The lateral pressures obtained from the analysis must not exceed the ultimate lateral resistance or the factored structural flexural resistance of the caisson or pile. For design purposes a maximum horizontal passive resistance of 120 kN (ULS) is recommended.

The spring constant, K, for analysis may be obtained by the expression, $K = k_s \times L \times D$ (kN/m), where k_s is the coefficient of horizontal subgrade reaction (kN/m^3), D is the pile width or caisson diameter (m) and L is the length (m) of the pile segment or element used in the analysis. The ultimate lateral resistance, P_{ult}, may be obtained from the expression, $P_{ult} = p_{ult} \times L \times D$.

Table 7.7 – Recommended Soil Parameters

Area Reference Borehole No.	* Applicable Elevation (m)	Soil Type	Bulk Unit Weight (kN/m^3)	Angle of Internal Friction (ϕ) Degrees	Undrained Shear Strength (S_u) (kPa)	Recommended n_h Value (kN/m^3)**
North Abutment	161.4 – 156.4	Granular Fill	21.0	32	-	6000
CR1	156.4 – 143.4	Silty Clay Till	20.0	0	225	-
North Abutment	161.4 – 156.4	Granular Fill	21.0	32	-	6000
CR2	156.4 – 143.5	Silty Clay Till	20.0	0	225	-
South Abutment	161.2 – 156.2	Granular Fill	21.0	32	-	6000
CR7	156.2 – 143.6	Silty Clay Till	20.0	0	225	-
South Abutment	161.2 – 156.2	Granular Fill	21.0	32	-	6000
CR8	156.2 – 143.4	Silty Clay Till	20.0	0	225	-

* Assumes that the piles for the north and south abutments will lie within granular fill comprising the RSS Core and new granular fill below the RSS Walls.

** Values estimated based on Table 20.3 data, Canadian Foundation Engineering Manual, 3rd edition, 1992.

Since the H-piles are end bearing, the vertical resistance will not be significantly affected by the pile spacing. Pile or caisson interaction should be considered with reference to CHBDC Clause 6.8.9.2.

For lateral soil/pile and soil/caisson group interaction analysis, the equation for k_s quoted above may be used in conjunction with appropriate reduction factors.

Where a pile group is oriented ***perpendicular*** to the direction of loading, group action may be considered by reducing values for k_s by a reduction factor R as follows:

Caisson/Pile Spacing Perpendicular to Direction of Loading	Horizontal Subgrade Reaction Reduction Factor, R
4 D*	1.00
1 D*	0.50

* D is the width of the caisson or pile, and spacing is measured centre to centre

Where a caisson/pile group is oriented ***parallel*** to the direction of loading, group action may be considered by reducing values for k_s by a reduction factor R as follows:

Caisson/Pile Spacing Parallel to Direction of Loading	Horizontal Subgrade Reaction Reduction Factor, R
8 D	1.00
6 D	0.70
4 D	0.40
3 D	0.25

Intermediate values may be obtained by interpolation.

At the pier locations, the lateral resistance may be provided by battered piles. From a constructability perspective it is recommended that the batter be limited to not more than 4H:1V due to the difficulty in installing piles at greater inclinations.

7.6 Recommended Foundation

It is recognised that an integral abutment bridge (which requires pile foundations) offers significant long-term advantages. The use of H-piles at the abutments allows for the design of an integral abutment structure. From a geotechnical point of view, it is recommended that the bridge abutments be supported on steel H-piles.

The piers can be supported on either augered caissons or H-piles. From a constructability perspective caisson construction will likely have a lesser impact on traffic users compared to pile foundations. Moreover, caisson installation would be relatively safer to highway users due to the lack of noise and distraction and the exclusion of elevated hammers and piles that are normal with pile driving operations. Based on the foregoing, augered caissons are therefore recommended for supporting the piers.

Refer to Appendix D for a detailed comparison of the advantages and disadvantages of the various foundation options.

7.7 Frost Cover

Pile caps and footings should be provided with a minimum of 1.2 m of earth cover or equivalent protection.

8 EXCAVATION AND BACKFILL

8.1 General

All excavations must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the native soils at this site may be classified as Type 2 soils above the water table and Type 3 soils below the water table. Excavations above the water table may be sloped at 1.5H:1V

8.2 Foundations

The excavation and backfilling for structures must be carried out in accordance with SP 902S01.

9 GROUNDWATER CONTROL

The groundwater level at this site is estimated to range between Elev. 152.4 m and Elev. 153.3 m and groundwater is unlikely to be encountered in shallow excavations.

It is anticipated that unwatering would be limited to controlling any surface runoff or seepage from relatively permeable soil seams. A system of interceptor trenches and pumping from strategically located filtered sumps should be sufficient for this purpose. The design of the unwatering system should be the responsibility of the Contractor.

Any accumulation of water from the base of excavations should be removed prior to placing concrete or compacting granular fill. Placement of concrete or compacting engineered fill must be done in the dry.

10 APPROACH FILL

Approach embankment construction using either non-cohesive earth fill or SSM is feasible provided the embankment is constructed on the underlying hard silty clay glacial till encountered at this site. It is envisaged that the maximum embankment heights will be about 8.5 ± m and 6 ± m at the north and south approaches respectively.

The total post construction settlement of the subgrade soils due to loads imposed by up to 8.5± m high approach embankments is estimated to be in the order of 15 mm. A significant portion of this settlement will be essentially complete by the end of construction.

The embankment fill will also experience settlement. This settlement is expected to be about 85 mm and 60 mm for 8.5± m and 6± m high embankments respectively. The settlement within the non-cohesive fill should be immediate in nature and essentially be complete shortly after construction has been completed.

The global, internal and surficial stability of the approach embankment fill will depend on the slope geometry and also to a large degree on the material used to construct the embankment.

Embankments constructed using, select subgrade material or non-cohesive earth fill will have stable side slopes at inclinations of up to 2H:1V.

For the purpose of embankment stability analyses, the commercially available slope stability program Slide 5.0 developed by Rocscience Inc. was used. The Bishop's simplified method for stability analysis was employed.

Global stability analyses were conducted for 2H:1V earth fill or SSM embankments. For embankments constructed on the native hard silty clay till, factors of safety against global failure of 1.3 and greater were obtained for both long term and short term conditions.

Embankment construction should be in accordance with OPSS 206, as amended by Special Provision "Amendment to OPSS 206, December 1993", dated November 2002.

Where earth fill embankments are higher than 8 m, mid-height berms should be incorporated in the design. The berms should:

- extend for the length through which the embankment height exceeds 8 m
- be at least 2 m wide
- have 2% positive drainage to shed run-off water.

Earth fill embankment slopes and cut slopes must be provided with erosion protection in accordance with OPSS 571 and OPSS 572.

Proper erosion control measures should also be implemented during construction. Temporary erosion and sediment control must be provided in accordance with OPSS Special Provision SP 577S01, SP 577F02 and SP 577S03 as appropriate.

11 RETAINED SOIL SYSTEMS

Retained soil system (RSS) walls may be used subject to the requirements presented in this section. It is understood that an RSS false abutment is proposed. RSS could also be used for wing walls and other retaining structures.

RSS walls should be specified to be "High Performance" and "High Appearance". The contract drawings should include information on the longitudinal alignment of the wall in plan, the top and base elevations of the wall in profile, cross-sectional space constraints and an NSSP for the RSS wall.

11.1 Foundation

The performance of an RSS is dependent, among other factors, on the characteristics of its foundation. Failure to provide an adequate foundation may lead to settlement and distortion of the RSS and, in severe cases, to possible failure of the system.

At this site, it is recommended that the levelling pad for the RSS wall be centred on top of a pad of engineered fill consisting of OPSS Granular A compacted in accordance with the

OPSS 501, Method A. The engineered fill pad should be at least twice as wide as the levelling pad.

In addition to the requirements for the levelling pad, the RSS can be founded on well compacted approach fill or on the native very stiff to hard silty clay till. All founding subgrades should be inspected and evaluated by the Quality Verification Engineer (QVE), at the time of construction. Details of the RSS arrangement on engineered fill are illustrated in Figure E1.

The recommended geotechnical resistances and construction details for the proposed RSS walls are tabulated below.

RSS Location	RSS Wall Base Elev. (m)	Max. Wall Height (m)	Required SLS Bearing Resistance (kPa)	Recommended Geotechnical Resistances (kPa)		Relevant Borehole & Subgrade Soil	Additional Requirements
NE Wall	158.4 for 8.5 m	6.8	177	340	225	CR2 Silty Clay Till	Subexcavate to Elev. 156.5 ±m, replace with 1.9 ±m of compacted Gran. A to base of wall.
NW Wall	158.4 for 8.5 m	6.8	177	340	225	CR1 Silty Clay Till	Subexcavate to Elev. 156.5 ±m, replace with 1.9 ±m of compacted Gran. A to base of wall.
North Abutment Face	158.4	6.8	177	340	225	CR1 & CR2 Silty Clay Till	Subexcavate to Elev. 156.5 ±m, replace with 1.9 ±m of compacted Gran. A to base of wall.

RSS Location	RSS Wall Base Elev. (m)	Max. Wall Height (m)	Required SLS Bearing Resistance (kPa)	Recommended Geotechnical Resistances (kPa)		Relevant Borehole & Subgrade Soil	Additional Requirements
SE Wall	157.9 for 10.0 m	7.2	187	340	225	CR8 Silty Clay Till	Subexcavate to Elev. 156 ±m, replace with 1.9 ±m of compacted Gran. A to base of wall.
SW Wall	157.9 for 10.0 m	7.2	187	340	225	CR7 Silty Clay Till	Subexcavate to Elev. 156 ±m, replace with 1.9 m of compacted Gran. A to base of wall.
South Abutment Face	157.9	7.2	187	340	225	CR7 & CR8 Silty Clay Till	Subexcavate to Elev. 156 ±m, replace with 1.9 ±m of compacted Gran. A to base of wall.

The settlement of RSS walls founded on engineered fill or native soils will depend on the thickness of the engineered fill, the material used, the foundation soils and the quality of construction. However, settlements are expected to be less than 25 mm and to occur essentially as the RSS is constructed.

The following parameters may be used for the design of the RSS:

- Ultimate coefficient of sliding resistance of cast in-situ concrete levelling pad on Granular A = 0.70
- Ultimate coefficient of sliding resistance of RSS mass on compact earth fill = 0.55
- Ultimate coefficient of sliding resistance of RSS mass on native silty clay till = 0.6

All topsoil, organics and soft soils should be removed from the subgrade of the RSS wall prior to placement of the fill.

The entire block of reinforced earth must also be designed against various modes of failure including sliding and overturning.

11.2 Global Stability

The global stability of the RSS wall is dependent on the characteristics of the embankment fill and the foundation soils, the geometry of the embankment and location of the RSS within the embankment. The RSS wall will be in the form of a rectangular block from the levelling pad extending to the full height.

RSS walls will be used for a false abutment and also as wing walls at the abutments. It is envisaged that the RSS will be founded on an engineered fill core resting on Granular "A" fill used to raise the grade at the site to the design grade of the levelling pad and RSS wall.

Stability analyses of RSS walls were carried out considering the following variables:

- RSS founded on Granular 'A' fill at Elev. 158.4 m and Elev. 157.9 m at the north and south abutments respectively.
- Granular 'A' fill – outer slope of 2H:1V (angle of internal friction, ϕ, of 35°, cohesion of 0, and unit weight, γ, of 22.8 kN/m^3).
- Fill behind the RSS is horizontal.

Analysis carried out on RSS walls at the abutment locations yielded a factor of safety 1.3 using a conventional anchor length of 50% of the height of the wall.

Consequently, it may be assumed that RSS walls at the abutment locations founded on a Granular 'A' pad, will be stable against global failure. The actual design configuration must be checked for global stability prior to finalization.

The internal stability of the RSS wall should be analyzed by the supplier/designer of the proprietary product selected for this site.

12 BACKFILL TO ABUTMENTS

In the case of integral or semi-integral abutments, backfill to the abutment should be granular material. In the case of a conventional abutment, granular backfill is recommended but rock backfill can be permitted. Granular material is recommended as backfill.

The backfill to the abutment walls should be in accordance with OPSS 902 as amended by Special Provision 902S01. Granular backfill should be placed to the extents shown in OPSD 3101.150.

All granular material should meet the specifications of Special Provision 110F13 "Amendment to OPSS 1010, March 1993".

Compaction equipment to be used adjacent to retaining structures should be restricted in accordance with SSP 105S10.

The design of the abutment should incorporate a subdrain as shown in OPSD 3101.150.

13 EARTH PRESSURE

For cases where backfill to the abutment is placed in accordance with OPSD 3101.150, as recommended, the lateral earth pressure will be governed by the properties of the material within the backfill limits shown in the respective OPSD, i.e. a line projected up at 1.5H:1V for granular backfill.

If the support system allows yielding of the wall (unrestrained system), active horizontal earth pressure may be used in the geotechnical design of the structure. If the support system does not allow yielding (restrained system), at-rest horizontal earth pressures should be used. The amount of wall movement required for the development of active, passive and at-rest earth pressures may be interpreted using Figure C6.9.1(a) in the Commentary to the CHBDC.

Earth pressures acting on the structure should be computed in accordance with Clause 6.9 of the CHBDC but generally are given by the expression:

$P_h = K(\gamma h + q)$
P_h = horizontal pressure on the wall (kPa)
K = earth pressure coefficient (see table 13.1)
γ = unit weight of retained soil (see table 13.1)
h = depth below top of fill where pressure is computed (m)
q = value of any surcharge (kPa)

The lateral earth pressure equation provided above assumes that the surcharge value is constant with depth which is a conservative assumption. In accordance with Clause 6.9.3 of the CHBDC, a compaction surcharge should be added. The magnitude should be 12 kPa at the top of fill and decreasing to 0 kPa at a depth of 2.0 m for Granular B Type I or at a depth of 1.7 m for Granular A or Granular B Type II.

Earth pressure coefficients for backfill to the abutment wall are dependent on the material used as backfill. Typical values for a horizontal surface and a sloping surface (2H:1V) are given in Table 13.1.

Table 13.1 – Earth Pressure Coefficients

Wall Condition	Earth Pressure Coefficient (K)			
	OPSS Granular A or OPSS Granular B Type II $\phi = 35°$; $\gamma = 22.8$ kN/m^3		OPSS Granular B Type I $\phi = 32°$; $\gamma = 21.2$ kN/m^3	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (Unrestrained Wall)	0.27	0.40*	0.31	0.48*
At rest (Restrained Wall)	0.43	-	0.47	-
Passive (Movement Towards Soil Mass)	3.70	-	3.30	-

* For wing walls.

In conventional design, the use of a material with a high friction angle and low active pressure coefficient (e.g. Granular A, Granular B Type II) might be preferred as it results in lower earth pressures acting on the wall. In the case of integral or semi-integral abutments, material with a lower passive pressure coefficient (e.g. Granular B Type I) might be preferred as it results in lower forces acting on the ballast wall as the wall moves toward the soil mass.

The factors in the table above are "ultimate" values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.9.1 (a) in the Commentary to the CHBDC, 20006.

14 SEISMIC CONSIDERATIONS

14.1 Seismic Design Parameters

The site is treated as lying in Seismic Zone 0. The following seismic parameters should be used for design:

- Velocity Related Seismic Zone 0
- Zonal Velocity Ratio 0.05
- Acceleration Related Seismic Zone 1
- Zonal Acceleration Ratio 0.05
- Peak Horizontal Acceleration 0.08

The soil profile type at this site has been classified as Type I. Therefore, according to Clause 4.4.6, Table 4.4 of the CHBDC, a Site Coefficient "S" (ground motion amplification factor) of 1.0 should be used in seismic design.

14.2 Liquefaction Potential

There is no potential for liquefaction of the foundation soils below the abutments and pier.

The immediate approach fills will bear on stiff to hard clayey silt till and therefore there is negligible potential for soil liquefaction below the approaches.

14.3 Retaining Wall Dynamic Earth Pressures

In accordance with Clause 4.6.4 of the CHBDC, retaining structures should be designed using active (K_{AE}) and passive (K_{PE}) earth pressure coefficients that incorporate the effects of earthquake loading.

In calculating the active, passive and at rest earth pressure coefficients the angle of friction between the wall and backfill material is assumed to be 0.5 ϕ. For the design of retaining walls, the coefficients of horizontal earth pressure in Table 14.1 may be used:

Table 14.1 – Earth Pressure Coefficient for Earthquake Loading

Earth Pressure Coefficient (K) for Earthquake Loading				
	OPSS Granular A or OPSS Granular B Type II $\phi = 35°$; $\delta = 17.5°$ $\gamma = 22.8$ kN/m^3		OPSS Granular B Type I $\phi = 32°$; $\delta = 16°$ $\gamma = 21.2$ kN/m^3	
Wall Condition	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (K_{AE})*	0.27	0.46	0.31	0.58
Passive (K_{PE})	3.70	-	3.30	-
At Rest (K_{OE})**	0.53	-	0.58	-

* After Mononobe and Okabe, passive case assumes a horizontal surface in front of the wall.
** After Woods

15 CONSTRUCTION CONCERNS

During construction, the Contract Administrator should employ experienced geotechnical staff to observe construction activities related to foundation construction.

Potential construction concerns include, but are not necessarily limited to:

- the possibility of boulders and hard driving conditions being encountered while driving the pile through the till soils.
- The possibility of boulders being encountered while undertaking caisson excavations.
- the nature and geotechnical properties of non-cohesive earth fill used in the approach fills.

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APPENDICES

TERRAPROBE LIMITED

LIMITATIONS AND RISK

Procedures

The soil conditions were confirmed at the borehole locations only and conditions may vary between and beyond the boreholes. The boundaries between the various strata as shown on the logs are based on non-continuous sampling. These boundaries represent an inferred transition between the various strata, rather than a precise plane of stratigraphic change.

This investigation has been carried out using investigation techniques and engineering analysis methods consistent with those ordinarily exercised by Terraprobe and other engineering practitioners, working under similar conditions and subject to the time, financial and physical constraints applicable to this project. The discussions and recommendations that have been presented are based on the factual data obtained.

It must be recognized that there are special risks whenever engineering or related disciplines are applied to identify subsurface conditions. Even a comprehensive sampling and testing programme implemented in accordance with the most stringent level of care may fail to detect certain conditions. Terraprobe has assumed for the purposes of providing design parameters and advice, that the conditions that exist between sampling points are similar to those found at the sample locations. The conditions that Terraprobe has interpreted to exist between sampling points can differ from those that actually exist.

It may not be possible to drill a sufficient number of boreholes or sample and report them in a way that would provide all the subsurface information that could affect construction costs, techniques, equipment and scheduling. Contractors bidding on or undertaking work on the project should be directed to draw their own conclusions as to how the subsurface conditions may affect them, based on their own investigations and their own interpretations of the factual investigation results, cognizant of the risks implicit in the subsurface investigation activities.

Changes In Site And Scope

It must be recognized that the passage of time, natural occurrences, and direct or indirect human intervention at or near the site have the potential to alter subsurface conditions. Groundwater levels are particularly susceptible to seasonal fluctuations.

The design advice is based on the factual data obtained from this investigation made at the site by Terraprobe and are intended for use by the owner and its retained designers in the design phase of the project. If there are changes to the project scope and development features, or there is any additional information relevant to the interpretations made of the subsurface information, the geotechnical design parameters and comments relating to constructibility issues and quality control may not be relevant or complete for the revised project. Terraprobe should be retained to review the implications of such changes with respect to the contents of this report

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EXPLANATION OF TERMS USED IN REPORT

N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS $\bar{N}$.

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

c_u (kPa)	0 – 12	12 – 25	25 – 50	50 – 100	100 – 200	>200
	VERY SOFT	*SOFT*	*FIRM*	*STIFF*	*VERY STIFF*	*HARD*

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 – 5	5 – 10	10 – 30	30 – 50	>50
	VERY LOOSE	*LOOSE*	*COMPACT*	*DENSE*	*VERY DENSE*

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND/OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY IS:

RQD (%)	0 – 25	25 – 50	50 – 75	75 – 90	90 – 100
	VERY POOR	*POOR*	*FAIR*	*GOOD*	*EXCELLENT*

JOINTING AND BEDDING:

SPACING	50mm	50 – 300mm	0.3m – 1m	1m – 3m	>3m
JOINTING	*VERY CLOSE*	*CLOSE*	*MOD. CLOSE*	*WIDE*	*VERY WIDE*
BEDDING	*VERY THIN*	*THIN*	*MEDIUM*	*THICK*	*VERY THICK*

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

SS	SPLIT SPOON	TP	THINWALL PISTON
WS	WASH SAMPLE	OS	OSTERBERG SAMPLE
ST	SLOTTED TUBE SAMPLE	RC	ROCK CORE
BS	BLOCK SAMPLE	PH	TW ADVANCED HYDRAULICALLY
CS	CHUNK SAMPLE	PM	TW ADVANCED MANUALLY
TW	THINWALL OPEN	FS	FOIL SAMPLE

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ε	%	LINEAR STRAIN
$\varepsilon_1, \varepsilon_2, \varepsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
C_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	- °	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	- °	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY = c_u / τ_r

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES
ρ_w	kg/m^3	DENSITY OF WATER
γ_w	kN/m^3	UNIT WEIGHT OF WATER
ρ	kg/m^3	DENSITY OF SOIL
γ	kN/m^3	UNIT WEIGHT OF SOIL
ρ_d	kg/m^3	DENSITY OF DRY SOIL
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL

e	1,%	VOID RATIO
n	1,%	POROSITY
w	1,%	WATER CONTENT
S_r	%	DEGREE OF SATURATION
w_L	%	LIQUID LIMIT
w_P	%	PLASTIC LIMIT
w_S	%	SHRINKAGE LIMIT
I_P	%	PLASTICITY INDEX = $(w_L - w_P)$
I_L	1	LIQUIDITY INDEX = $(w - w_P)/I_P$
I_C	1	CONSISTENCY INDEX = $(w_L - w)/I_P$
e_{max}	1,%	VOID RATIO IN LOOSEST STATE

e_{min}	1,%	VOID RATIO IN DENSEST STATE
I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
D	mm	GRAIN DIAMETER
D_n	mm	n PERCENT - DIAMETER
C_u	1	UNIFORMITY COEFFICIENT
h	m	HYDRAULIC HEAD OR POTENTIAL
q	m^3/s	RATE OF DISCHARGE
v	m/s	DISCHARGE VELOCITY
i	1	HYDRAULIC GRADIENT
k	m/s	HYDRAULIC CONDUCTIVITY
j	kN/m^3	SEEPAGE FORCE

APPENDIX A

Record of Borehole Sheets

Terraprobe Limited

RECORD OF BOREHOLE No CR1

1 OF 2

METRIC

W.P. _____ LOCATION Coords: N:4834116.3 E:610483.5 ORIGINATED BY HA

DIST _____ HWY 401 BOREHOLE TYPE Solid Stem Augers COMPILED BY DB

DATUM Geodetic DATE 11.12.07 CHECKED BY RA

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
158.0	Ground Surface							
0.0	150mm TOPSOIL							
0.2	FILL - Sand and Gravel, some silt, trace clay, compact, brown, dry	1	SS	18	35	51	12	2
157.3 / 0.7	FILL - Silty Clay, some sand, trace gravel, stiff, brown, moist	2	SS	14	1	18	50	31
156.6 / 1.4	SILTY CLAY sandy, trace gravel, occasional sand seams, very stiff to hard, brown, damp (GLACIAL TILL)	3	SS	35				
		4	SS	29	2	20	48	30
		5	SS	47				
	grey	6	SS	40				
		7	SS	30				
		8	SS	100/ 2.5cm				
		9	SS	100/ 15cm				
		10	SS	100/ 13cm				
		11	SS	100/ 13cm				
		12	SS	140/ 28cm				
143.4 / 14.6								

Elevation scale: 157, 156, 155, 154, 153, 152, 151, 150, 149, 148, 147, 146, 145, 144

Dynamic cone penetration resistance plot: 20 40 60 80 100

Shear strength kPa: ○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE (20 40 60 80 100)

Plastic limit w_P — Natural moisture content w — Liquid limit w_L; Water content (%): 10 20 30

Unit weight γ kN/m³

Remarks & grain size distribution (%): GR SA SI CL

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No CR1

2 OF 2

METRIC

W.P. _____ LOCATION Coords: N 4834116.3 E 610483.5 ORIGINATED BY HA

DIST _____ HWY 401 BOREHOLE TYPE Solid Stem Augers COMPILED BY DB

DATUM Geodetic DATE 11.12.07 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	SHALE BEDROCK grey (Georgian Bay Formation)		13	SS	100/ 8cm	▽					
141.9 / 16.1	End of Borehole		14	SS	100/ 8cm		142				
	Wet cave at 15.8m (Elev. 142.2m) upon completion of drilling.										

+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No CR1A

1 OF 2

METRIC

W.P. _____ LOCATION Coords: N 4834116.3 E 610483.5 ORIGINATED BY HA

DIST _____ HWY 401 BOREHOLE TYPE Solid Stem Augers & NQ Rock Coring COMPILED BY DB

DATUM Geodetic DATE 06.01.08 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB VANE) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
158.0	Ground Surface										
0.0	Augered to 14.6m without sampling. Refer to BH CR1 for inferred soil stratigraphy.										
							157				
							156				
							155				
							154				
							153				
							152				
							151				
							150				
							149				
							148				
							147				
							146				
							145				
							144				
143.4	Commence NQ Rock Coring										RUN#1 TCR=75% SCR=50% RQD=21%
14.6			1	RUN	NQ						

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

ONTARIO MOT 1-06-1162 CREEKBANK.GPJ ONTARIO MOT.GDT 29/01/08

RECORD OF BOREHOLE No CR1A

2 OF 2

METRIC

W.P. _____ LOCATION Coords: N 4834116.3 E 610483.5 ORIGINATED BY HA

DIST _____ HWY 401 BOREHOLE TYPE Solid Stem Augers & NQ Rock Coring COMPILED BY DB

DATUM Geodetic DATE 06.01.08 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB VANE) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	SHALE BEDROCK Moderately to highly weathered, thickly bedded, weak to medium strong with strong limestone interbeds. 32mm thick clay seam at 15.0m. (Georgian Bay Formation) *(continued)*		2	RUN	NQ		142				RUN#2 TCR=92% SCR=83% RQD=14%
			3	RUN	NQ		141				RUN#3 TCR=100% SCR=57% RQD=15%
140.3 17.7	End of Borehole										

+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

ONTARIO MOT 1-06-1162 CREEKBANK.GPJ ONTARIO MOT.GDT 29/01/08

RECORD OF BOREHOLE No CR2

1 OF 2

METRIC

W.P. _____ LOCATION Coords: N 4834128.2 E 610501.3 ORIGINATED BY AS

DIST _____ HWY 401 BOREHOLE TYPE Solid Stem Augers COMPILED BY DB

DATUM Geodetic DATE 10.12.07 CHECKED BY RA

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
157.8	Ground Surface							
0.0	130mm TOPSOIL							
0.1	FILL - Sand and Gravel, some silt, compact, brown, dry	1	SS	22				
157.1 / 0.7	FILL - Silty Clay, some sand, trace gravel,inferred very stiff,brown,moist							
156.8 / 1.0	SILTY CLAY trace sandy to sandy, trace gravel, occasional sand seams, very stiff to hard, brown, damp to moist (GLACIAL TILL)	2	SS	32				
		3	SS	29				
		4	SS	53	4	24	47	25
		5	SS	106				
	grey	6	SS	66	8	34	41	17
		7	SS	127				
		8	SS	100/13cm				
		9	SS	127	0	1	69	30
		10	SS	100/13cm				
		11	SS	100/13cm				
		12	SS	100/5cm				
143.5 / 14.3	SHALE BEDROCK grey (Georgian Bay Formation)							

Elevation scale: 157, 156, 155, 154, 153, 152, 151, 150, 149, 148, 147, 146, 145, 144, 143

Dynamic cone penetration resistance plot: 20 40 60 80 100

Shear strength kPa: ○ Unconfined, ● Quick triaxial, + Field vane, × Lab vane (20 40 60 80 100)

Plastic limit w_p, natural moisture content w, liquid limit w_L; Water content (%): 10 20 30

Unit weight γ kN/m³

Ground water conditions

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No CR2

2 OF 2

METRIC

W.P. _____ LOCATION Coords: N 4834128.2 E 610501.3 ORIGINATED BY AS

DIST _____ HWY 401 BOREHOLE TYPE Solid Stem Augers COMPILED BY DB

DATUM Geodetic DATE 10.12.07 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	PLASTIC LIMIT / NATURAL MOISTURE CONTENT / LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	w_P — w — w_L	γ	GR SA SI CL
								○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE	WATER CONTENT (%) 10 20 30	kN/m³	
142.5			13	SS	100/ 8cm				○		
15.3	End of Borehole Auger Refusal at 15.3m Piezometer Installation consists of a 19mm diameter, schedule 40 PVC pipe with a 3.0m slotted screen. Water Level Readings: Date Depth(m) Elevation(m) Jan 14 08 5.4 152.4										

+ 3, × 3 : Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No CR3

1 OF 2

METRIC

W.P. _____ LOCATION Coords: N 4834096.0 E 610504.1 ORIGINATED BY HA

DIST _____ HWY 401 BOREHOLE TYPE Solid Stem Augers COMPILED BY DB

DATUM Geodetic DATE 13.12.07 CHECKED BY RA

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	PLASTIC LIMIT w_P	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w_L	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
157.7	Ground Surface												
0.0	130mm ASPHALT												
0.1 / 157.1	460mm FILL - Gravel and Sand, some silt, very dense, brown, dry	1	SS	66						45	43	9	3
0.6 / 156.3	FILL - Silty Clay, sandy, trace gravel, very stiff, brown, damp	2	SS	26	157					10	28	45	17
1.4	SILTY CLAY sandy, trace gravel, occasional sand seams, very stiff to hard, brown, damp (GLACIAL TILL)	3	SS	27	156								
		4	SS	48	155					5	24	45	26
		5	SS	37	154								
	grey	6	SS	17	153								
		7	SS	23	152								
		8	SS	100/ 10cm	151								
		9	SS	80	150								
		10	SS	100/ 13cm	149								
		11	SS	100/ 10cm	148								
144.0 / 13.7	SHALE BEDROCK grey (Georgian Bay Formation)	12	SS	100/ 15cm	147								
142.7					146								
					145								
					144								
					143								

GROUND WATER CONDITIONS

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB VANE
20 40 60 80 100

WATER CONTENT (%) 10 20 30

STRAT PLOT

REMARKS & GRAIN SIZE DISTRIBUTION (%)

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

ONTARIO MOT 1-06-1162 CREEKBANK.GPJ ONTARIO MOT.GDT 29/01/08

RECORD OF BOREHOLE No CR3

2 OF 2

METRIC

W.P. _____ LOCATION Coords N 4834096.0 E 610504.1 ORIGINATED BY HA

DIST _____ HWY 401 BOREHOLE TYPE Solid Stem Augers COMPILED BY DB

DATUM Geodetic DATE 13.12.07 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB VANE; 20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L (WATER CONTENT (%) 10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						GR SA SI CL
15.0	End of Borehole Auger Refusal at 15.0m Wet cave at 9.8m (Elev. 147.9m) upon completion of drilling		13	SS	100/ 5cm						

+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No CR4

1 OF 2

METRIC

W.P. _____ LOCATION Coords: N:4834107.1 E:610519.9 ORIGINATED BY AS

DIST _____ HWY 401 BOREHOLE TYPE Solid Stem Augers COMPILED BY DB

DATUM Geodetic DATE 12.12.07 CHECKED BY RA

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
157.6	Ground Surface							
0.0	150mm ASPHALT							
0.2	460mm FILL - Sand and Gravel, trace silt, very dense, brown, dry	1	SS	79				
157.0								
0.6	FILL - Silty Clay, sandy, trace gravel, brown, moist							
156.6								
1.0	SILTY CLAY sandy, trace gravel, occasional sand seams, hard, brown, damp (GLACIAL TILL)	2	SS	35				
		3	SS	31				
	grey	4	SS	43				
		5	SS	32	2	37	42	19
		6	SS	33				
		7	SS	43	6	34	44	16
		8	SS	187/ 28cm	8	30	49	13
		9	SS	100/ 5cm				
		10	SS	100/ 8cm				
		11	SS	100/ 10cm				
143.9								
13.7	SHALE BEDROCK grey (Georgian Bay Formation)	12	SS	100/ 5cm				
		13	SS	100/ 5cm				
142.7								

Elevation scale: 157, 156, 155, 154, 153, 152, 151, 150, 149, 148, 147, 146, 145, 144, 143

Dynamic cone penetration resistance plot: 20 40 60 80 100

Shear strength kPa: ○ Unconfined ● Quick triaxial + Field vane × Lab vane (20 40 60 80 100)

Plastic limit w_P — Natural moisture content w — Liquid limit w_L; Water content (%): 10 20 30

Unit weight γ kN/m³

Remarks & grain size distribution (%): GR SA SI CL

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No CR4

METRIC

W.P. _____ LOCATION Coords: N 4834107.1 E 610519.9 ORIGINATED BY AS

DIST _____ HWY 401 BOREHOLE TYPE Solid Stem Augers COMPILED BY DB

DATUM Geodetic DATE 12.12.07 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
14.9	End of Borehole Auger Refusal at 14.9m Borehole was caving at 13.7m (Elev. 143.9m) and unstabilized water level at 10.7m (Elev. 146.9m) upon completion of drilling.										

+ 3, × 3 Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No CR4A

1 OF 2

METRIC

W.P. _____ LOCATION Coords: N 4834107.1 E 610519.9 ORIGINATED BY AS

DIST _____ HWY 401 BOREHOLE TYPE Solid Stem Augers & NQ Rock Coring COMPILED BY DB

DATUM Geodetic DATE 19.12.07 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
157.6 / 0.0	Ground Surface										
	Augered to 13.7m without sampling. Refer to BH CR4 for inferred soil stratigraphy.						157				
							156				
							155				
							154				
							153				
							152				
							151				
							150				
							149				
							148				
						▽	147				
							146				
							145				
143.9 / 13.7	Commence NQ Rock Coring						144				
			1	RUN	NQ		143				RUN#1 TCR=92% SCR=57% RQD=15%

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

ONTARIO MOT 1-06-1162 CREEKBANK.GPJ ONTARIO MOT.GDT 29/01/08

RECORD OF BOREHOLE No CR4A

2 OF 2

METRIC

W.P. _____ LOCATION Coords: N 4834107.1 E 610519.9 ORIGINATED BY AS

DIST _____ HWY 401 BOREHOLE TYPE Solid Stem Augers & NQ Rock Coring COMPILED BY DB

DATUM Geodetic DATE 19.12.07 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	SHALE BEDROCK Moderately to highly weathered to 13.9m, slightly weathered below, thickly bedded, grey, weak to medium strong. 75mm thick clay seam at 13.4m, 50mm thick clay seam at 14.0m and 25mm thick clay seam at 15.4m. (Georgian Bay Formation) *(continued)*		2	RUN	NQ		142 141				RUN#2 TCR=98% SCR=83% RQD=18%
140.9 16.7	End of Borehole										

+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

ONTARIO MOT 1-06-1162 CREEKBANK.GPJ ONTARIO MOT.GDT 29/01/08

RECORD OF BOREHOLE No CR5

1 OF 2

METRIC

W.P. _____ LOCATION Coords: N 4834055.3 E 610545.5 ORIGINATED BY AS

DIST _____ HWY 401 BOREHOLE TYPE Solid Stem Augers COMPILED BY DB

DATUM Geodetic DATE 21.12.07 CHECKED BY RA

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION	GR	SA	SI	CL	REMARKS & GRAIN SIZE DISTRIBUTION (%)
157.6	Ground Surface									
0.0	150mm ASPHALT									
0.2 / 157.0	450mm FILL - Sand and Gravel, some silt, very dense, brown, dry	1	SS	102	157	33	49	15	3	
0.6	SILTY CLAY sandy, trace to some gravel, very stiff to hard, brown, damp (GLACIAL TILL)	2	SS	45						
		3	SS	37	156					
		4	SS	42	155					
	grey	5	SS	36						
					154					
		6	SS	28	153	11	33	39	17	
					152					
		7	AS	25						no recovery in sampler
					151					
		8	SS	100/13cm	150					
					149					
148.5 / 9.1	SAND AND GRAVEL some silt, trace clay, very dense, grey, wet	9	SS	100/13cm		38	38	19	5	
147.7 / 9.9	SILTY CLAY sandy, trace to some gravel, hard, grey, damp (GLACIAL TILL)				148					
		10	SS	100/13cm	147					
					146					
		11	SS	100/5cm	145					
143.9 / 13.7	SHALE BEDROCK grey (Georgian Bay Formation)	12	SS	100/10cm	144					
					143					

STRAT PLOT

GROUND WATER CONDITIONS

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB VANE
20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L
WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No CR5

2 OF 2

METRIC

W.P. _____ LOCATION Coords: N 4834055.3 E 610545.5 ORIGINATED BY AS

DIST _____ HWY 401 BOREHOLE TYPE Solid Stem Augers COMPILED BY DB

DATUM Geodetic DATE 21.12.07 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						GR SA SI CL
142.3											
15.3	End of Borehole		13	SS	100/ 5cm				○		
	Borehole was open and unstabilized water level at 9.1m (Elev. 148.5m) upon completion of drilling										

+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No CR5A

1 OF 2

METRIC

W.P. _____ LOCATION Coords: N:4834055.3 E:610545.5 ORIGINATED BY HA

DIST _____ HWY 401 BOREHOLE TYPE Solid Stem Augers & NQ Rock Coring COMPILED BY DB

DATUM Geodetic DATE 08.01.08 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB VANE) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
157.6 / 0.0	Ground Surface										
	Augered to 13.7m without sampling. Refer to BH CR5 for inferred soil stratigraphy.						157				
							156				
							155				
							154				
							153				
							152				
							151				
							150				
							149				
							148				
							147				
							146				
							145				
143.9	Commence NQ Rock Coring						144				RUN#1 TCR=42% SCR=10% RQD=0%
13.7			1	RUN	NQ						
			2	RUN	NQ		143				RUN#2 TCR=100% SCR=81% RQD=60%

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

ONTARIO MOT 1-06-1162 CREEKBANK.GPJ ONTARIO MOT.GDT 29/01/08

RECORD OF BOREHOLE No CR5A

2 OF 2

METRIC

W.P. _____ LOCATION Coords: N 4834055.3 E 610545.5 ORIGINATED BY HA

DIST _____ HWY 401 BOREHOLE TYPE Solid Stem Augers & NQ Rock Coring COMPILED BY DB

DATUM Geodetic DATE 08.01.08 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	SHALE BEDROCK Moderately weathered to 14.5m, slightly weathered below, thickly bedded, grey, weak to medium strong with medium strong limestone interbeds. 40mm thick clay seam at 16.4m. (Georgian Bay Formation) *(continued)*		3	RUN	NQ		142 141				RUN#3 TCR=100% SCR=92% RQD=31%
140.8 / 16.8	End of Borehole										

+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

ONTARIO MOT 1-06-1162 CREEKBANK.GPJ ONTARIO MOT.GDT 29/01/08

RECORD OF BOREHOLE No CR6

1 OF 1 **METRIC**

W.P. _____ LOCATION Coords: N 4834064.4 E 610559.0 ORIGINATED BY JS

DIST _____ HWY 401 BOREHOLE TYPE Solid Stem Augers COMPILED BY DB

DATUM Geodetic DATE 21.12.07 CHECKED BY RA

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
157.5	Ground Surface							
0.0	150mm ASPHALT							
0.2 / 156.9	450mm FILL - Sand and Gravel, trace silt, very dense, brown, dry	1	SS	59				
0.6	SILTY CLAY sandy, trace gravel, occasional sand seams, very stiff to hard, brown, damp (GLACIAL TILL)	2	SS	33				
		3	SS	68	1	31	42	26
	--- grey	4	SS	45				
		5	SS	31	10	36	37	17
		6	SS	25				
		7	SS	17	6	36	39	19
		8	SS	100/18cm				sampler wet at 7.6m
		9	SS	100/17cm				
		10	SS	100/8cm				
		11	SS	100/5cm				
143.8 / 13.7	SHALE BEDROCK - grey (Georgian Bay Formation)	12	SS	100/5cm				
13.8	End of Borehole							

Wet cave at 9.9m (Elev. 147.6m) and unstabilized water level at 7.0m (Elev. 150.5m) upon completion of drilling.

GROUND WATER CONDITIONS: ▽

ELEVATION SCALE: 157, 156, 155, 154, 153, 152, 151, 150, 149, 148, 147, 146, 145, 144

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE (20 40 60 80 100)

PLASTIC LIMIT w_p — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%): 10 20 30

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No CR7

1 OF 2

METRIC

W.P. _____ LOCATION Coords: N:4834036.1 E:610564.9 ORIGINATED BY AS

DIST _____ HWY 401 BOREHOLE TYPE Solid Stem Augers COMPILED BY DB

DATUM Geodetic DATE 09.12.07 CHECKED BY RA

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
157.3	Ground Surface							
0.0	100mm TOPSOIL							
0.1	FILL - Gravelly Sand, some silt, trace clay, compact, brown, moist	1	SS	10	26	50	18	6
156.4								
0.9	SILTY CLAY sandy, trace gravel, hard, brown, damp (GLACIAL TILL)	2	SS	86				
		3	SS	75				
		4	SS	72				
	grey	5	SS	56	10	36	37	17
		6	SS	72				
		7	SS	44	8	32	42	18
		8	SS	162/ 28cm				
		9	SS	110				
		10	SS	144				
		11	SS	100/ 10cm				
143.6								
13.7	SHALE BEDROCK grey (Georgian Bay Formation)	12	SS	100/ 10cm				

Elevation scale: 157, 156, 155, 154, 153, 152, 151, 150, 149, 148, 147, 146, 145, 144, 143

Dynamic cone penetration resistance plot: 20 40 60 80 100

Shear strength kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE (20 40 60 80 100)

Plastic limit w_P — Natural moisture content w — Liquid limit w_L; Water content (%): 10 20 30

Unit weight γ kN/m³

Ground water conditions

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

ONTARIO MOT 1-06-1162 CREEKBANK.GPJ ONTARIO MOT.GDT 29/01/08

RECORD OF BOREHOLE No CR7

2 OF 2

METRIC

W.P. _____ LOCATION Coords: N 4834036.1 E 610564.9 ORIGINATED BY AS

DIST _____ HWY 401 BOREHOLE TYPE Solid Stem Augers COMPILED BY DB

DATUM Geodetic DATE 09.12.07 CHECKED BY RA

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT / NATURAL MOISTURE CONTENT / LIQUID LIMIT			REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa (20 40 60 80 100) ○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE	w_P — w — w_L WATER CONTENT (%) (10 20 30)		γ kN/m³	GR SA SI CL
142.0												
15.3	End of Borehole		13	SS	100/10cm		142		○			
	Piezometer Installation consists of a 19mm diameter, schedule 40 PVC pipe with a 3.0m slotted screen.											
	Water Level Readings:											
	Date Depth(m) Elevation(m)											
	Jan.14.08 4.0 153.3											

$+^{3}$, $\times^{3}$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No CR8

1 OF 1

METRIC

W.P. _____ LOCATION Coords: N 4834044.9 E 610578.8 ORIGINATED BY HA

DIST _____ HWY 401 BOREHOLE TYPE Solid Stem Augers COMPILED BY DB

DATUM Geodetic DATE 10.12.07 CHECKED BY RA

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
157.1	Ground Surface							
0.0	100mm TOPSOIL							
0.1	FILL - Sand and Gravel, trace to some silt, compact, brown, damp	1	SS	11				
156.6 / 0.5	FILL - Silty Clay, trace sand, trace gravel, stiff, brown, damp							
0.7	brown	2	SS	45	3	27	45	25
	grey	3	SS	27				
	SILTY CLAY sandy, trace gravel, occasional sand seams and partings, very stiff to hard, damp (GLACIAL TILL)	4	SS	28				
		5	SS	37	7	34	42	17
		6	SS	43				
		7	SS	28	5	37	40	18
		8	SS	100/13cm				
		9	SS	100/10cm				
		10	SS	94				
		11	SS	46				
143.4 / 13.7	SHALE BEDROCK - grey (Georgian Bay Formation)	12	SS	100/13cm				
143.1 / 14.0	End of Borehole							

Wet cave at 5.5m (Elev. 151.6m) upon completion of drilling

GROUND WATER CONDITIONS

ELEVATION SCALE: 157, 156, 155, 154, 153, 152, 151, 150, 149, 148, 147, 146, 145, 144

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE (20 40 60 80 100)

PLASTIC LIMIT w_p — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_l

WATER CONTENT (%): 10 20 30

UNIT WEIGHT γ kN/m³

$+^{3}$, $\times^{3}$ Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No CR8A

1 OF 2

METRIC

W.P. _____ LOCATION Coords: N:4834044.9 E:610578.8 ORIGINATED BY AS

DIST _____ HWY 401 BOREHOLE TYPE Solid Stem Augers & NQ Rock Coring COMPILED BY DB

DATUM Geodetic DATE 10.12.07 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
157.1 / 0.0	Ground Surface						157				
	Augered to 13.7m without sampling. Refer to BH CR8 for inferred soil stratigraphy.						156				
							155				
							154				
							153				
							152				
							151				
							150				
							149				
							148				
							147				
							146				
							145				
							144				
143.4	Commence NQ Rock Coring										
13.7			1	RUN	NQ		143				RUN#1 TCR=97% SCR=90% RQD=40%

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

ONTARIO MOT 1-06-1162 CREEKBANK.GPJ ONTARIO MOT GDT 29/01/08

RECORD OF BOREHOLE No CR8A

2 OF 2

METRIC

W.P. _____ LOCATION Coords: N 4834044.9 E 610578.8 ORIGINATED BY AS

DIST _____ HWY 401 BOREHOLE TYPE Solid Stem Augers & NQ Rock Coring COMPILED BY DB

DATUM Geodetic DATE 10.12.07 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	SHALE BEDROCK Moderately to highly weathered to 14.2m, slightly weathered below, thickly bedded, grey, weak to medium strong. 150mm thick clay seam at 13.8m, 50mm thick clay seam at 14.1m and 50mm thick clay seam at 16.3m. (Georgian Bay Formation) *(continued)*		2	RUN	NQ		142 141				RUN#2 TCR=99% SCR=87% RQD=35%
140.3 16.8	End of Borehole Piezometer Installation consists of a 19mm diameter, schedule 40 PVC pipe with a 3.0m slotted screen. Water Level Readings: Date Depth(m) Elevation(m) Jan.14.08 4.7 152.4										

+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No CR9

1 OF 2

METRIC

W.P. _____ LOCATION Coords: N 4834143.8 E 610466.9 ORIGINATED BY HA

DIST _____ HWY 401 BOREHOLE TYPE Solid Stem Augers COMPILED BY DB

DATUM Geodetic DATE 07.01.08 CHECKED BY RA

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa (○ UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB VANE)	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
157.4	Ground Surface								
0.0	150mm TOPSOIL								
0.2	FILL - Silty Clay, trace sand, trace gravel, firm, brown, moist	1	SS	6	157				
156.7 / 0.7	SILTY CLAY sandy, trace gravel, occasional sand seams, hard, brown, damp (GLACIAL TILL)	2	SS	44	156				
		3	SS	47					
		4	SS	78	155				
	grey	5	SS	121	154				
		6	SS	40	153				
		7	SS	108	152				
		8	SS	100/15cm	151				
		9	SS	93	150				
		10	SS	100/15cm	149				
145.7 / 11.7	SILT AND SAND trace clay, trace gravel, very dense, grey, moist (Possible Glacial Till)	11	SS	100/15cm	148				6 41 45 8
143.7 / 13.7	SHALE BEDROCK grey (Georgian Bay Formation)	12	SS	100/13cm	147				
					146				
					145				
					144				
					143				

Continued Next Page

+ 3, × 3. Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No CR9

2 OF 2

METRIC

W.P. _____ LOCATION Coords: N 4834143.8 E 610466.9 ORIGINATED BY HA

DIST _____ HWY 401 BOREHOLE TYPE Solid Stem Augers COMPILED BY DB

DATUM Geodetic DATE 07.01.08 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						GR SA SI CL
142.1											
15.3	End of Borehole		13	SS	100/ 13cm				○		

Wet cave at 13.4m (Elev. 144.0m) upon completion of drilling.

Piezometer Installation consists of a 19mm diameter, schedule 40 PVC pipe with a 3.0m slotted screen.

Water Level Readings:

Date	Depth(m)	Elevation(m)
Jan.14.08	4.6	152.8

+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No CR10

1 OF 2

METRIC

W.P. _____ LOCATION Coords: N:4834023.4 E:610589.2 ORIGINATED BY HA

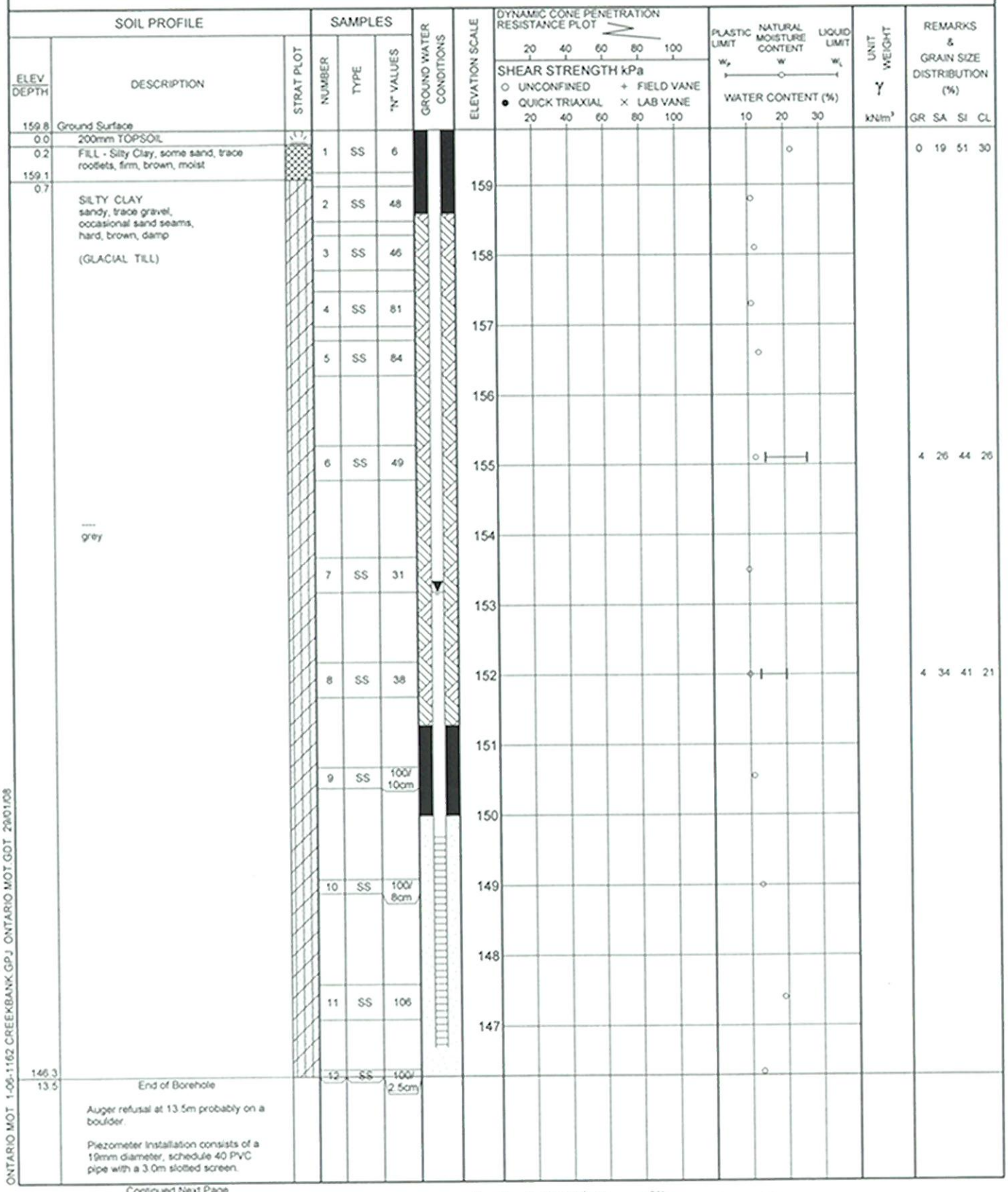
DIST _____ HWY 401 BOREHOLE TYPE Solid Stem Augers COMPILED BY DB

DATUM Geodetic DATE 20.12.07 CHECKED BY RA

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GR	SA	SI	CL
159.8	Ground Surface							
0.0	200mm TOPSOIL							
0.2	FILL - Silty Clay, some sand, trace rootlets, firm, brown, moist	1	SS	6	0	19	51	30
159.1 / 0.7	SILTY CLAY sandy, trace gravel, occasional sand seams, hard, brown, damp (GLACIAL TILL)	2	SS	48				
		3	SS	46				
		4	SS	81				
		5	SS	84				
		6	SS	49	4	26	44	26
	grey	7	SS	31				
		8	SS	38	4	34	41	21
		9	SS	100/10cm				
		10	SS	100/8cm				
		11	SS	106				
146.3 / 13.5	End of Borehole	12	SS	100/2.5cm				

Auger refusal at 13.5m probably on a boulder.

Piezometer installation consists of a 19mm diameter, schedule 40 PVC pipe with a 3.0m slotted screen.



Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No CR10

2 OF 2

METRIC

W.P. _____ LOCATION Coords: N 4834023.4 E 610589.2 ORIGINATED BY HA

DIST _____ HWY 401 BOREHOLE TYPE Solid Stem Augers COMPILED BY DB

DATUM Geodetic DATE 20.12.07 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	Water Level Readings:										
	Date Depth(m) Elevation(m)										
	Jan.14.08 6.6 153.2										

+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

APPENDIX B

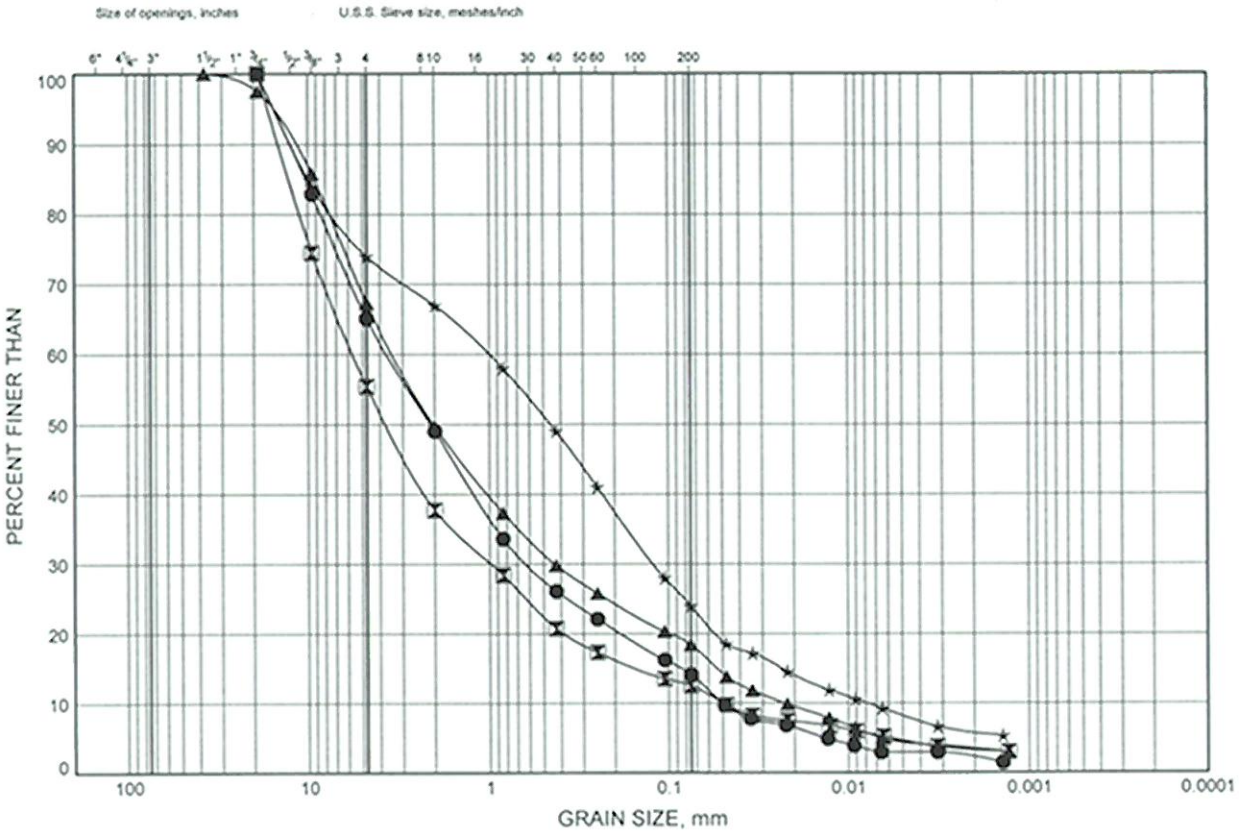
Laboratory Test Results

Terraprobe Limited

GRAIN SIZE DISTRIBUTION

FIGURE B1

FILL - Sand and Gravel



COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	CR1	0.3	157.7
⧗	CR3	0.3	157.4
▲	CR5	0.4	157.2
★	CR7	0.3	157.0

Date January 2008

Project 1-06-1162

Prep'd DB

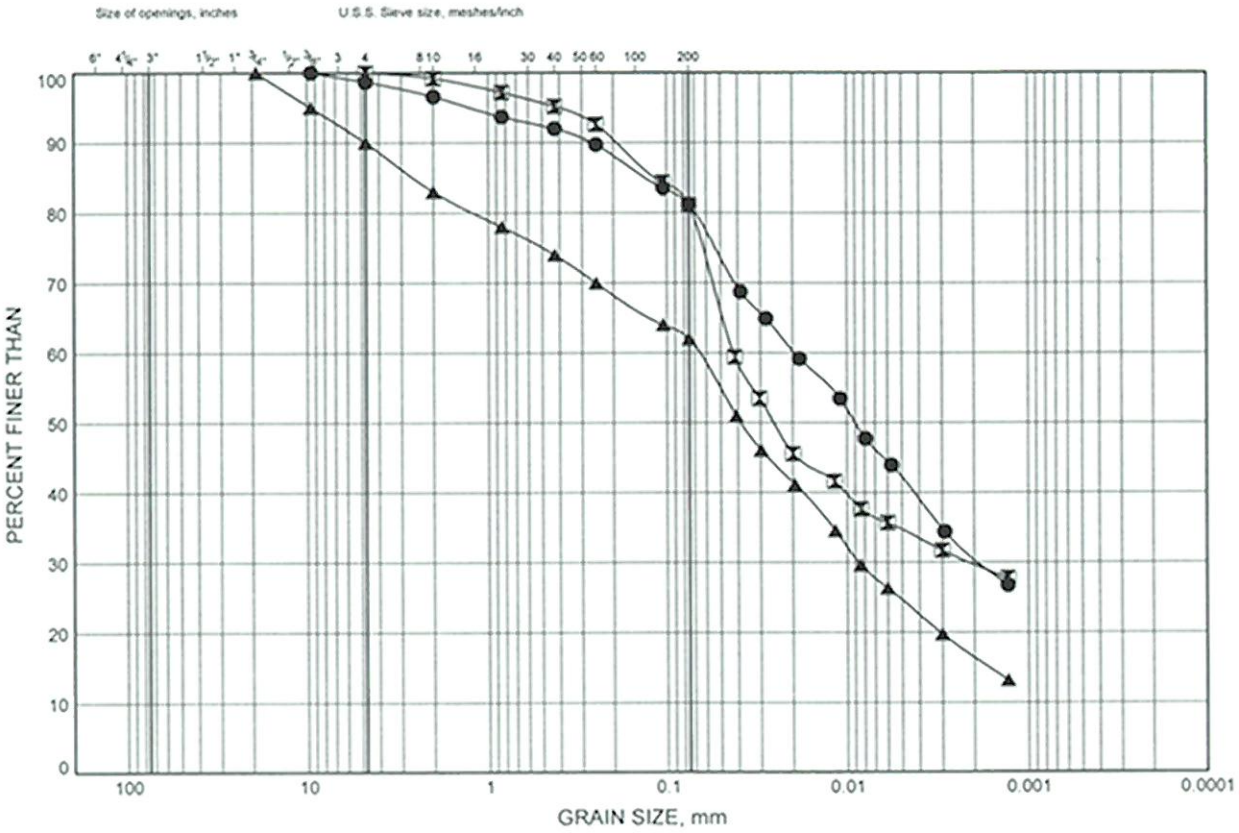
Chkd. RA

GSD 1-06-1162 CREEKBANK.GPJ 29/01/08

GRAIN SIZE DISTRIBUTION

FIGURE B2

FILL - Silty Clay



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	CR1	1.0	157.0
▲	CR3	1.0	156.7
⧗	CR10	0.3	159.5

Date January 2008

Project 1-06-1162

Prep'd DB

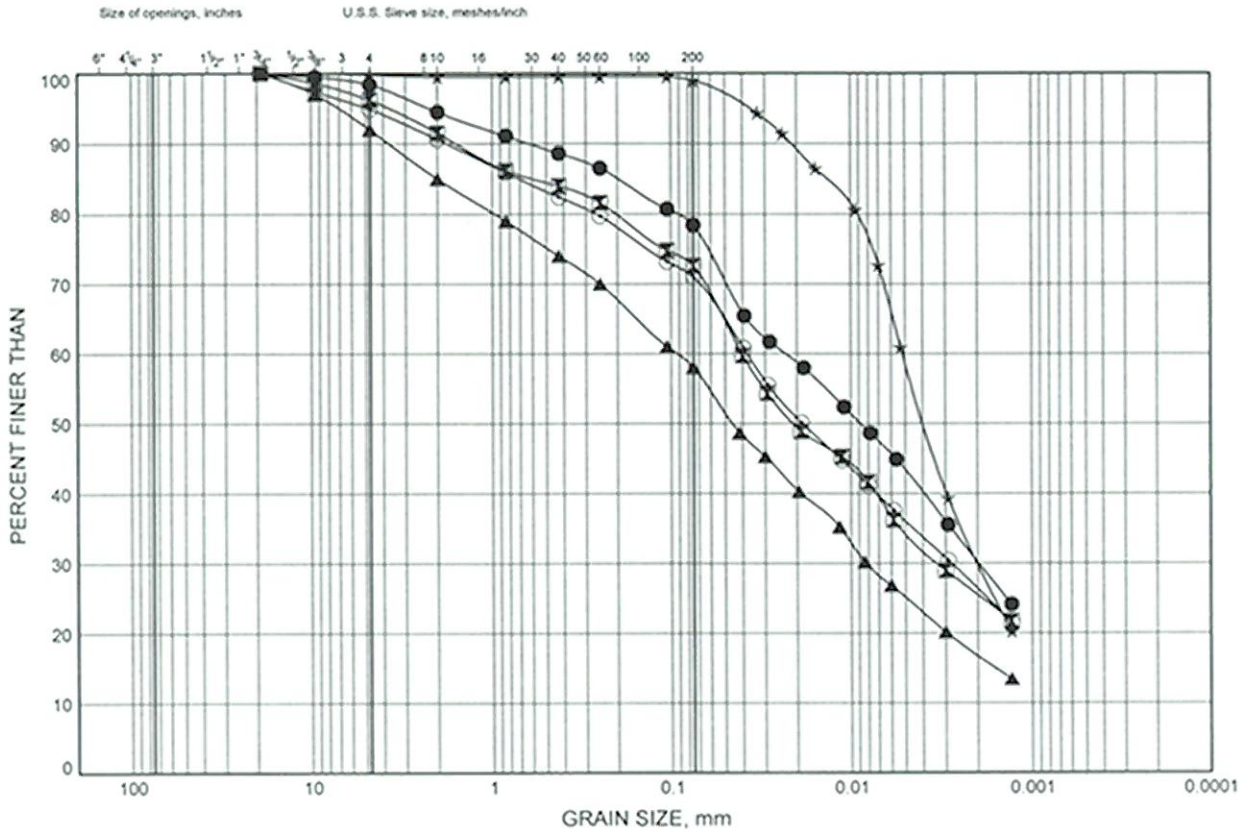
Chkd. RA

GSD 1-06-1162 CREEKBANK.GPJ 23/01/08

GRAIN SIZE DISTRIBUTION

FIGURE B3

SILTY CLAY - TILL



COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	CR1	2.5	155.5
⧗	CR2	2.5	155.3
▲	CR2	4.7	153.1
★	CR2	9.3	148.5
⊙	CR3	2.5	155.2

Date January 2008

Project 1-06-1162

Prep'd DB

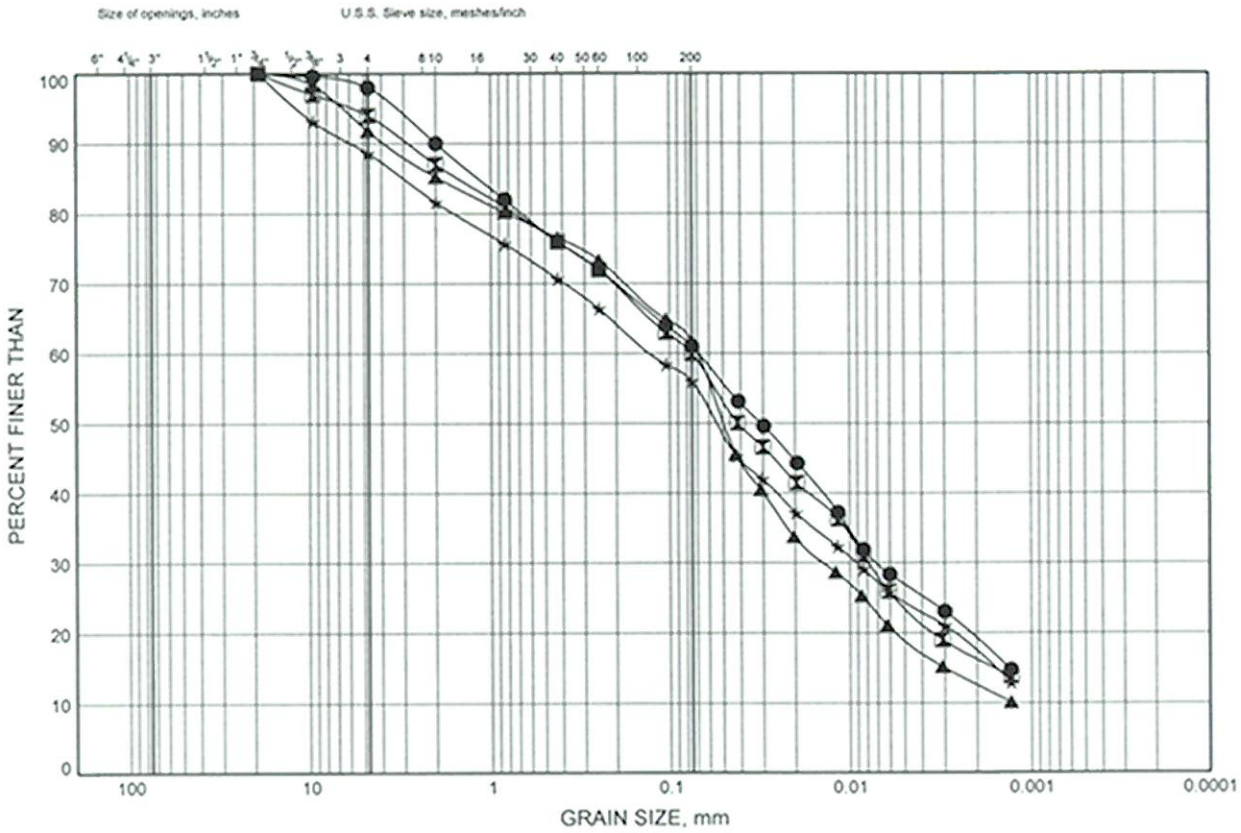
Chkd. RA

GSD 1-06-1162 CREEKBANK.GPJ 29/01/08

GRAIN SIZE DISTRIBUTION

FIGURE B4

SILTY CLAY - TILL



COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	CR4	3.2	154.4
⧗	CR4	6.3	151.3
▲	CR4	7.8	149.8
★	CR5	4.7	152.9

Date January 2008

Project 1-06-1162

Prep'd DB

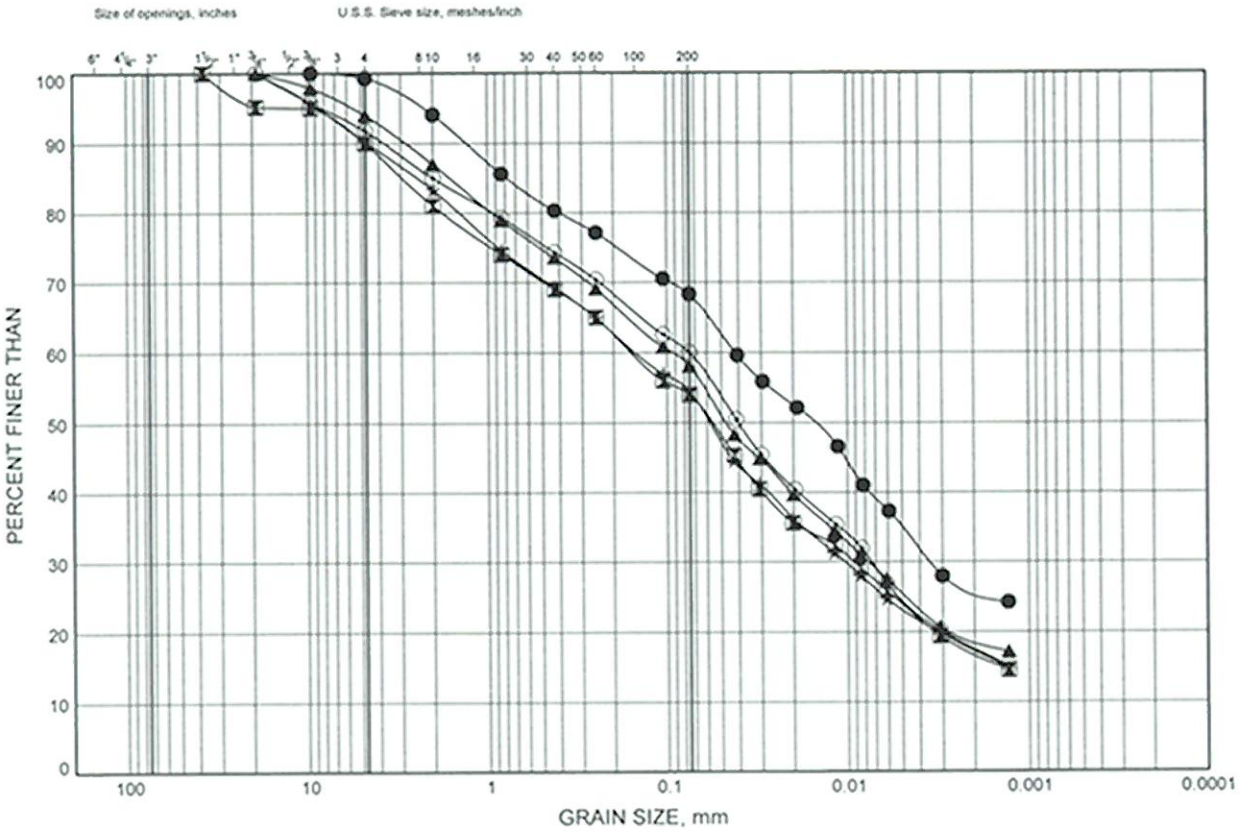
Chkd. RA

GSD 1-06-1162 CREEKBANK.GPJ 29/01/08

GRAIN SIZE DISTRIBUTION

FIGURE B5

SILTY CLAY - TILL



COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	CR6	1.7	155.8
⧗	CR6	3.2	154.3
▲	CR6	6.3	151.2
★	CR7	3.2	154.1
⊙	CR7	6.3	151.0

Date January 2008

Project 1-06-1162

Prep'd DB

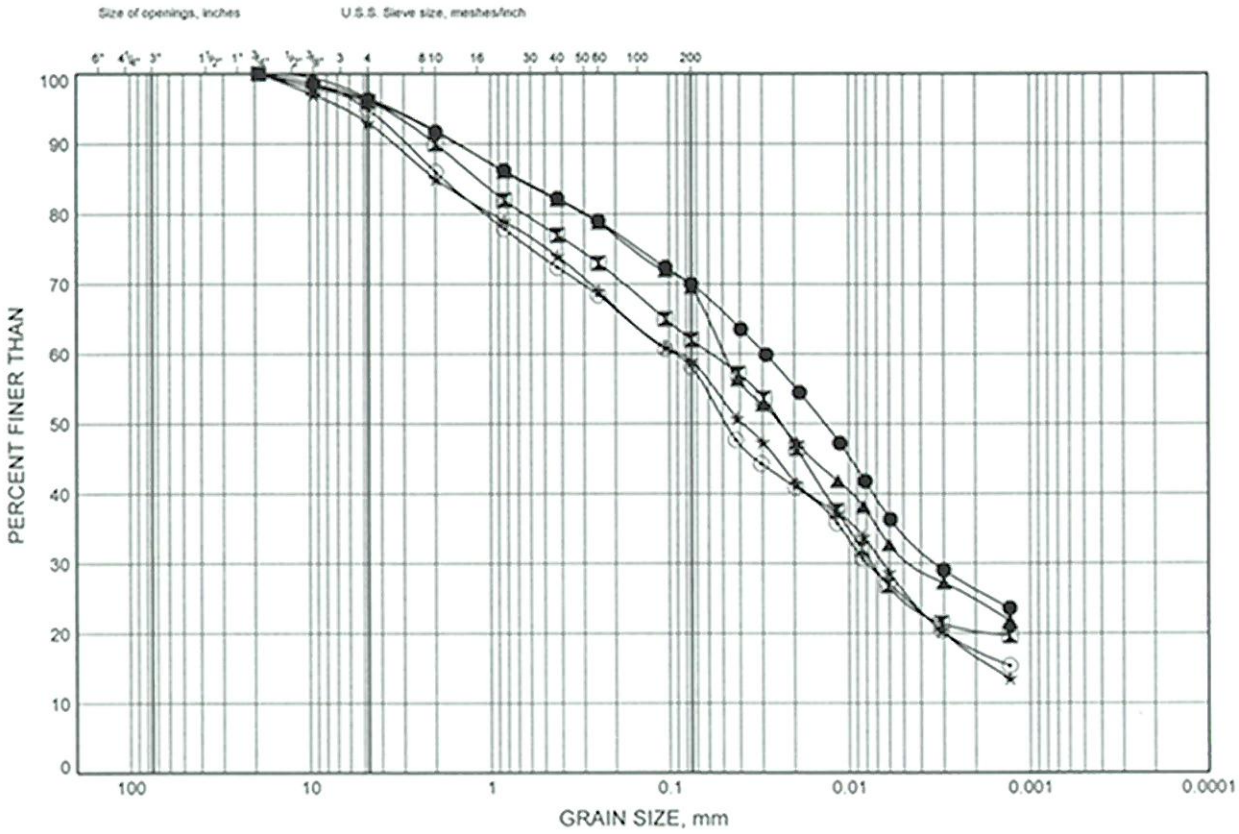
Chkd. RA

GSD 1-06-1162 CREEKBANK.GPJ 29/01/08

GRAIN SIZE DISTRIBUTION

FIGURE B6

SILTY CLAY - TILL



COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
▲	CR8	1.0	156.1
★	CR8	3.2	153.9
⊙	CR8	6.3	150.8
●	CR10	4.7	155.1
⧗	CR10	7.8	152.0

Date January 2008

Project 1-06-1162

Prep'd DB

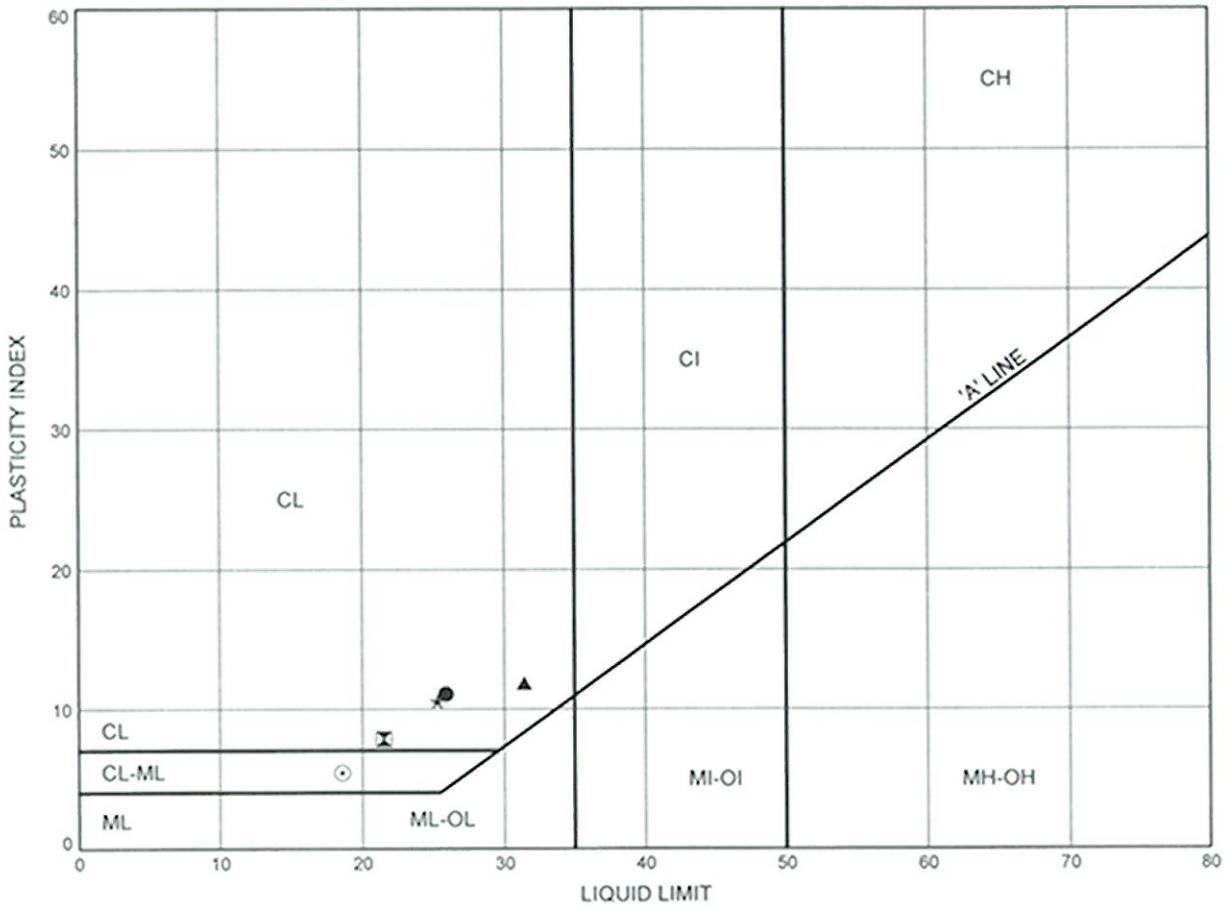
Chkd. RA

GSD 1-06-1162 CREEKBANK.GPJ 29/01/08

ATTERBERG LIMITS TEST RESULTS

FIGURE B7

SILTY CLAY - TILL



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	CR1	2.5	155.8
⧗	CR2	4.7	153.2
▲	CR2	9.3	148.6
★	CR3	2.5	155.2
⊙	CR4	6.3	151.3

Date January 2008

Project 1-06-1162

Prep'd DB

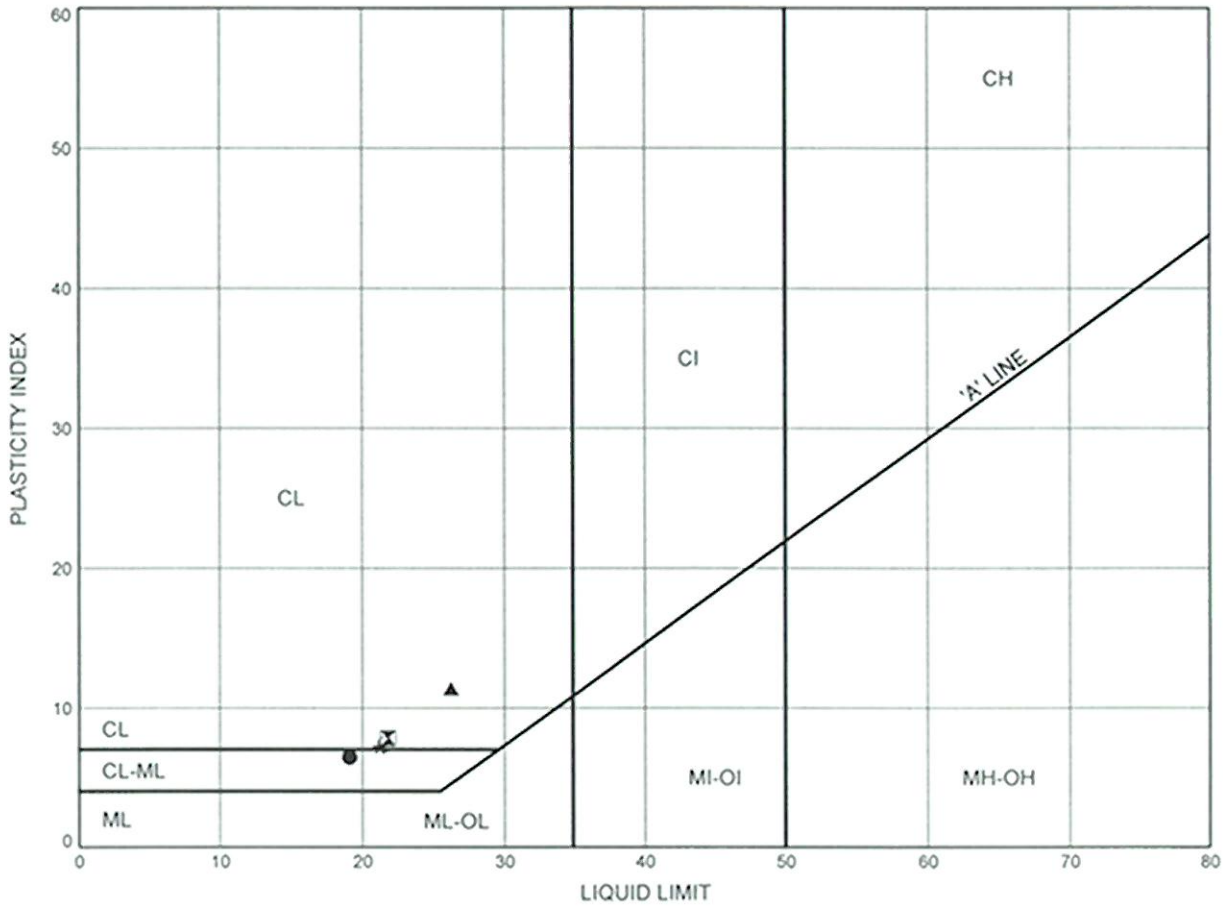
Chkd. RA

ALTR 1-06-1162 CREEKBANK.GPJ 29/01/08

ATTERBERG LIMITS TEST RESULTS

FIGURE B8

SILTY CLAY - TILL



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	CR4	7.8	149.8
⧗	CR5	4.7	152.9
▲	CR6	1.7	155.8
★	CR6	3.2	154.3
⊙	CR7	3.2	154.1

Date January 2008

Project 1-06-1162

Prep'd DB

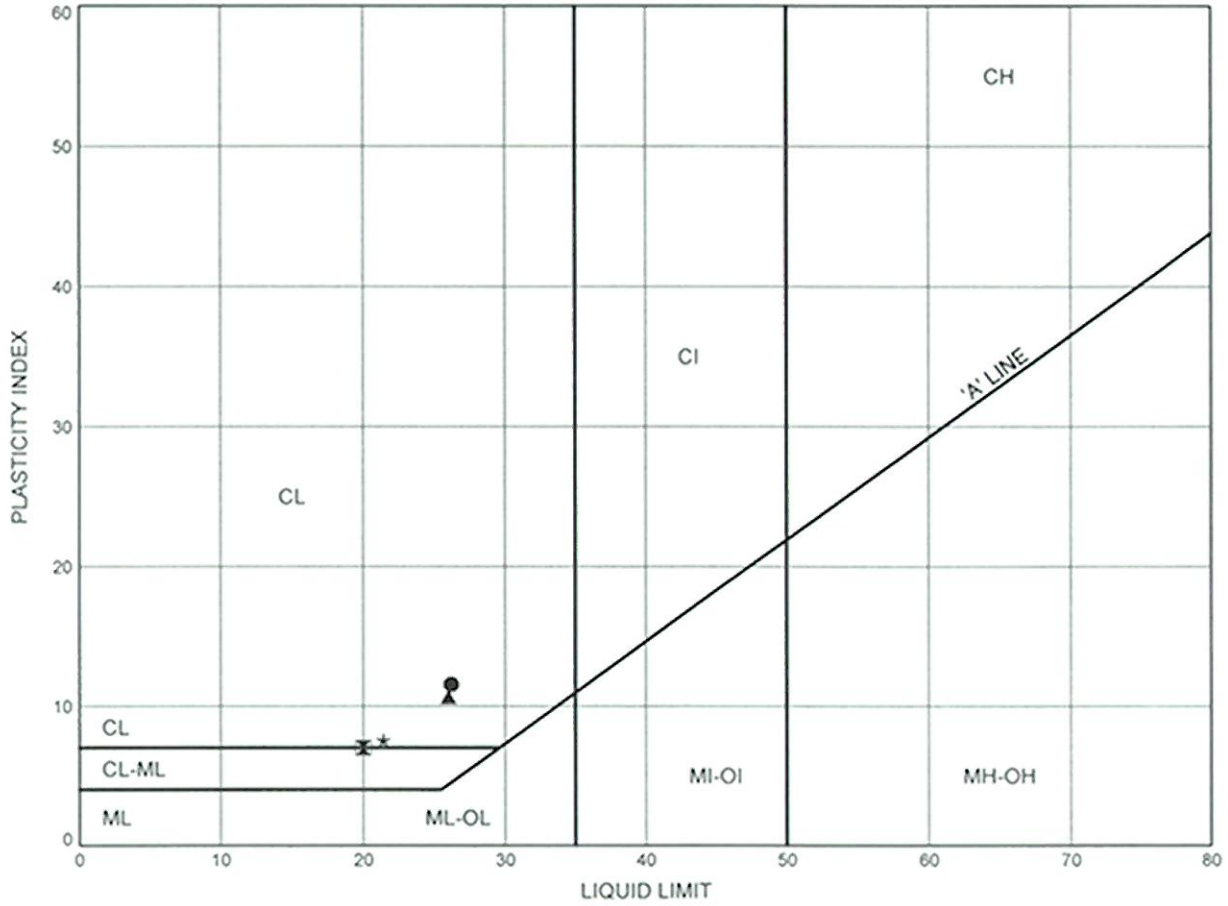
Chkd. RA

ALTR 1-06-1162 CREEKBANK.GPJ 29/01/08

ATTERBERG LIMITS TEST RESULTS

FIGURE B9

SILTY CLAY - TILL



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
▲	CR8	1.0	156.1
✶	CR8	3.2	153.9
●	CR10	4.7	155.1
⧗	CR10	7.8	152.0

Date January 2008

Project 1-06-1162

Prep'd DB

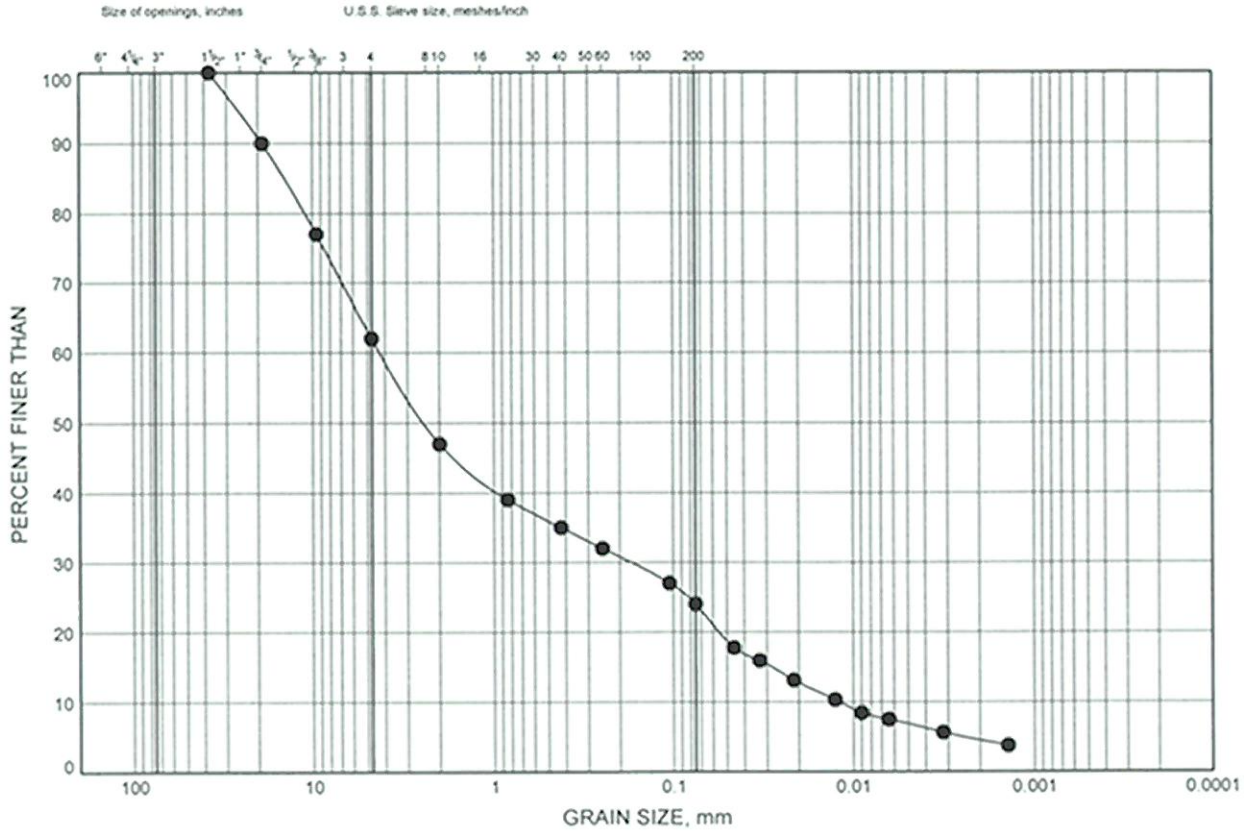
Chkd. RA

ALTR 1-06-1162 CREEKBANK.GPJ 29/01/08

GRAIN SIZE DISTRIBUTION

FIGURE B10

SAND AND GRAVEL



COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	CR5	9.3	148.4

Date January 2008

Project 1-06-1162

Prep'd DB

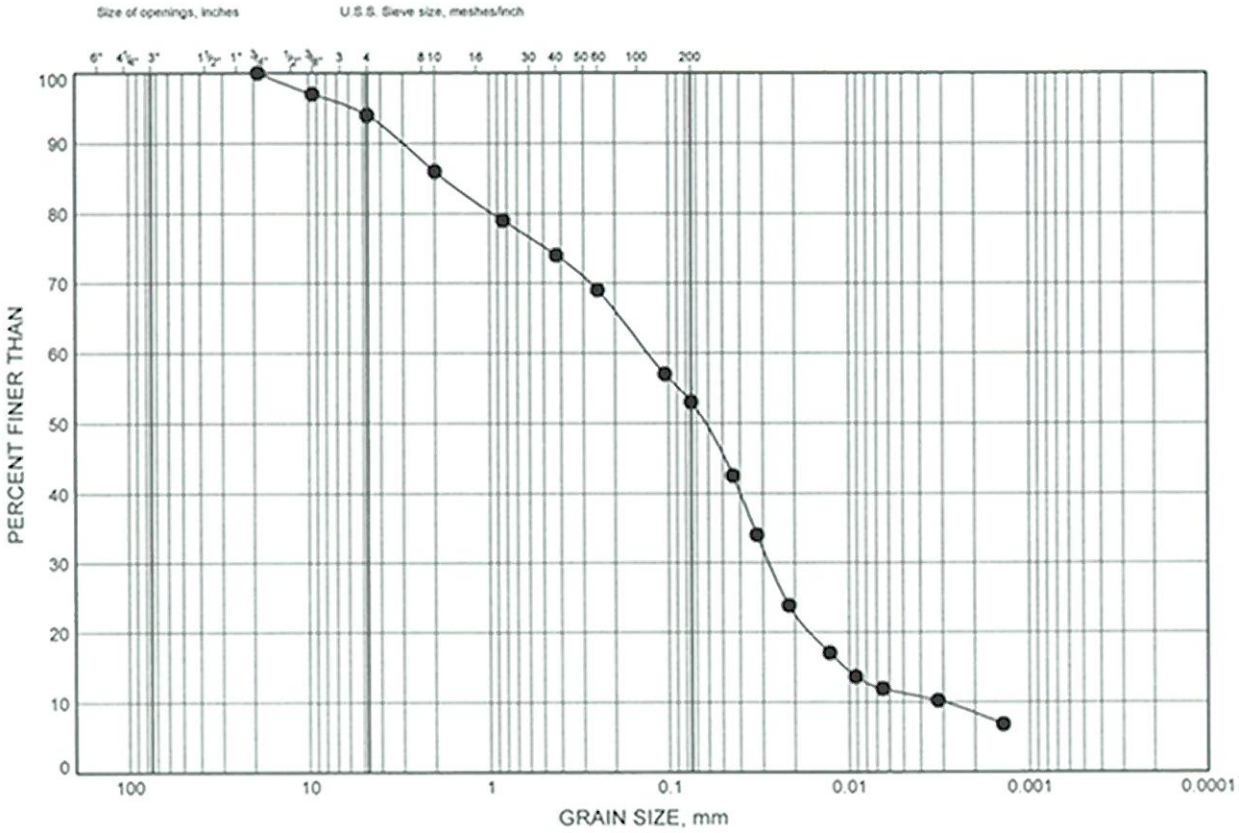
Chkd. RA

GSD 1-06-1162 CREEKBANK.GPJ 29/01/08

GRAIN SIZE DISTRIBUTION

FIGURE B11

SILT AND SAND



COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	CR9	12.3	145.1

Date January 2008

Project 1-06-1162

Prep'd DB

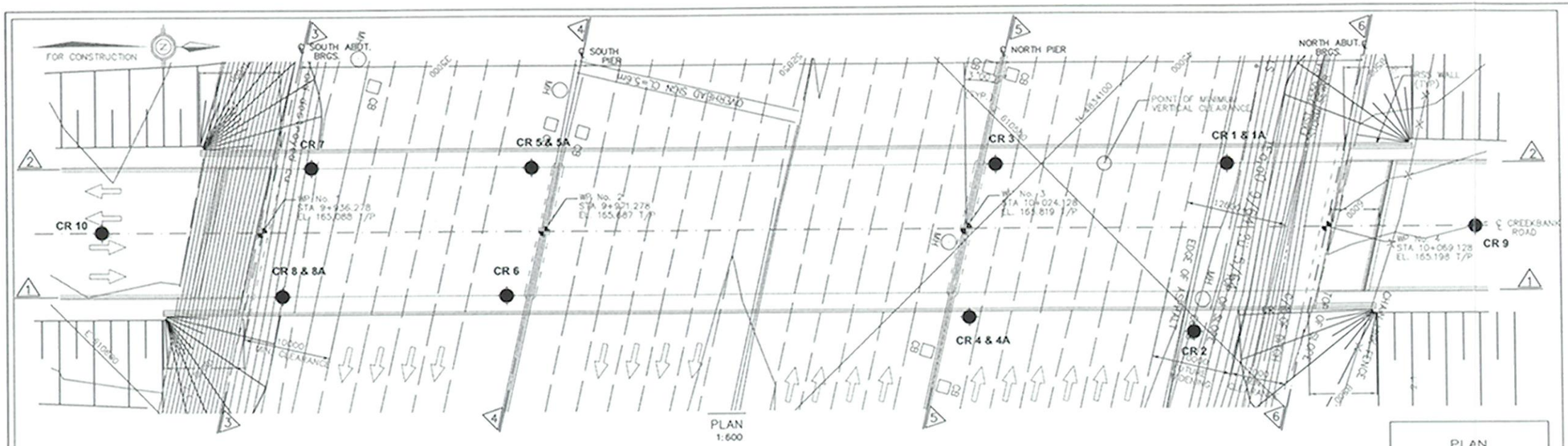
Chkd. RA

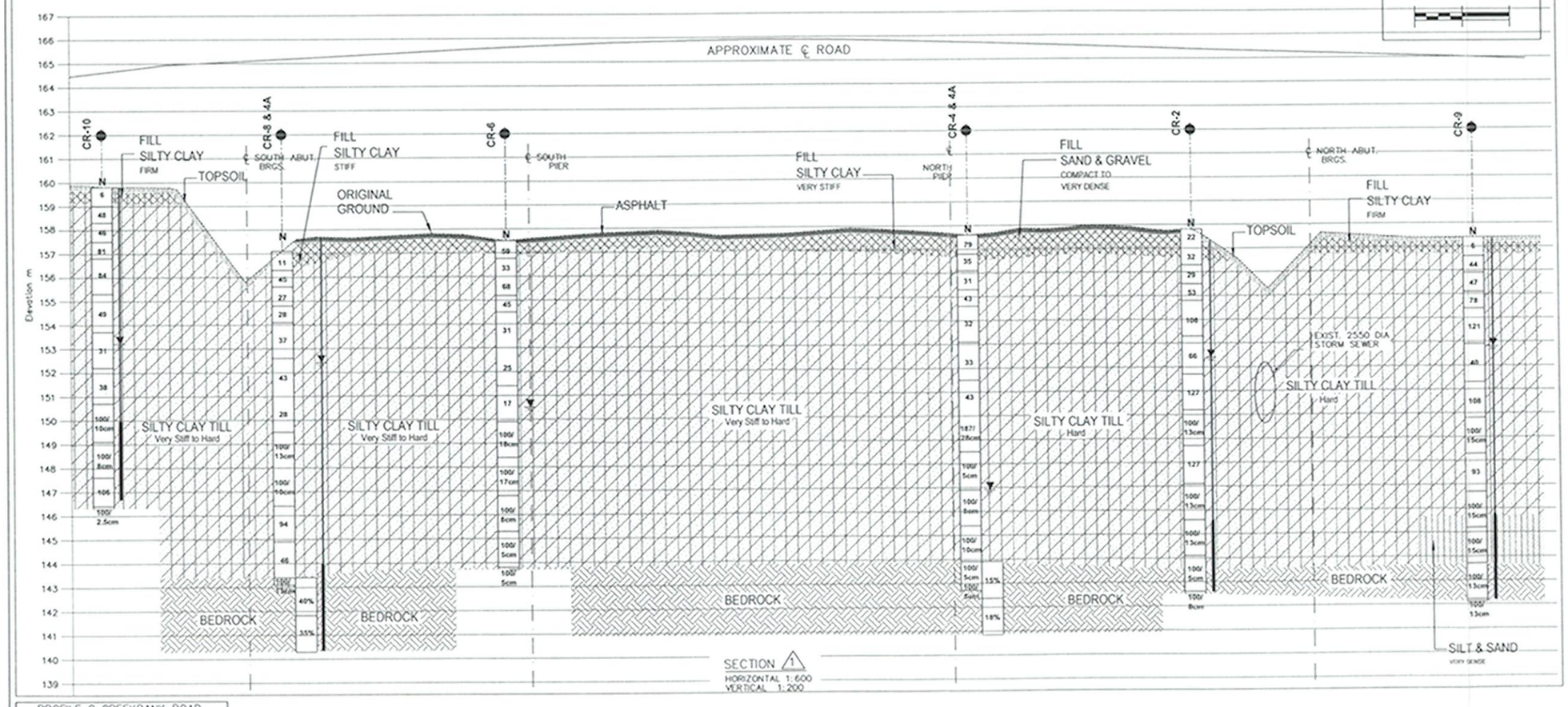
GSD 1-06-1162 CREEKBANK.GPJ 29/01/08

APPENDIX C

Drawings titled "Borehole Locations and Soil Strata"

Terraprobe Limited





PROFILE ℄ CREEKBANK ROAD
SECTION 1
6m 0 6m HOR
2m 0 2m VERT

DRAWING NOT TO BE SCALED
50mm ON ORIGINAL DRAWING

SERVICE DATA

SERVICE	DATE	INIT.	SERVICE	DATE	INIT.
SAN. SEWERS			GAS MAINS		
STM. SEWERS			BELL U/G CABLE		
WATERMAINS			HYDRO U/G CABLE		
M.O.E.					

REVISIONS

DATE	DETAILS	INIT.

- Bore Hole
- Dynamic Cone Penetration Test (Cone)
- Bore Hole & Cone
- 'N' — Blows/0.3m (Std Pen Test, 475 J/blow)
- CONE — Blows/0.3m (60° Cone, 475 J/blow)
- WL at Time of Investigation
- WL in Piezometer Jan. 14, 2008
- Piezometer
- 90% — Rock Quality Designation
- A/R — Auger Refusal

No	ELEVATION	COORDINATES	
		NORTHING	EASTING
CR-1	158.0	4834116.3	610483.5
CR-1A	158.0	4834116.3	610483.5
CR-2	157.8	4834128.2	610501.3
CR-3	157.7	4834096.0	610504.1
CR-4	157.6	4834107.1	610519.9
CR-4A	157.6	4834107.1	610519.9
CR-5	157.6	4834055.3	610545.5
CR-5A	157.6	4834055.3	610545.5
CR-6	157.5	4834064.4	610559.0
CR-7	157.3	4834036.1	610564.9
CR-8	157.1	4834044.9	610578.8
CR-8A	157.1	4834044.9	610578.8
CR-9	157.4	4834143.8	610466.9
CR-10	159.8	4834023.4	610589.2

Giffels
An Ingenium Ground Company

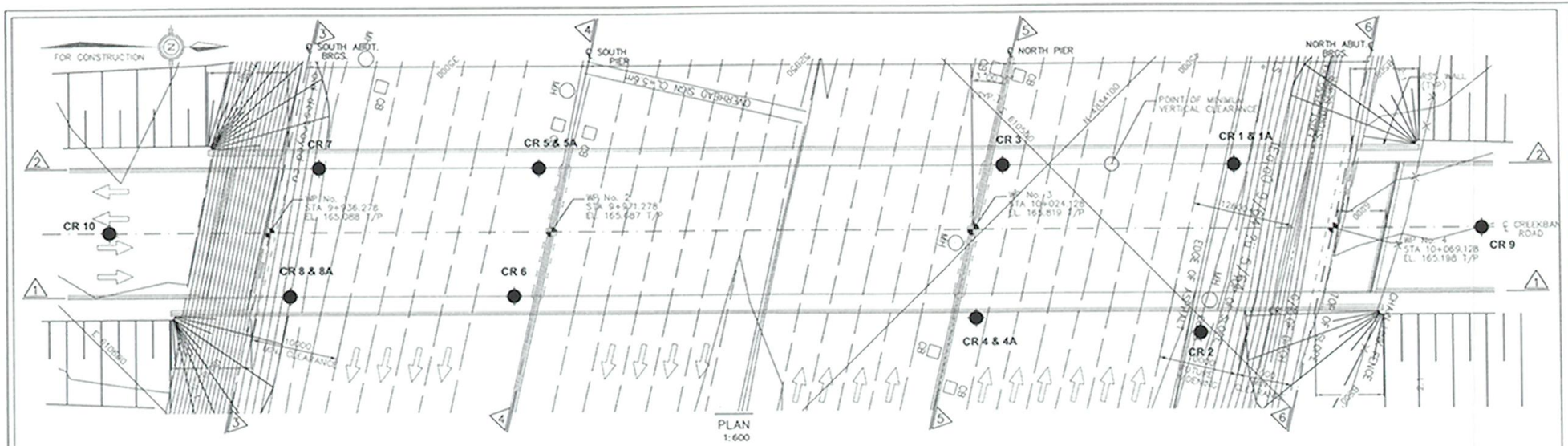
Terraprobe
Consulting Geotechnical & Environmental Engineering
Construction Materials Engineering, Inspection & Testing

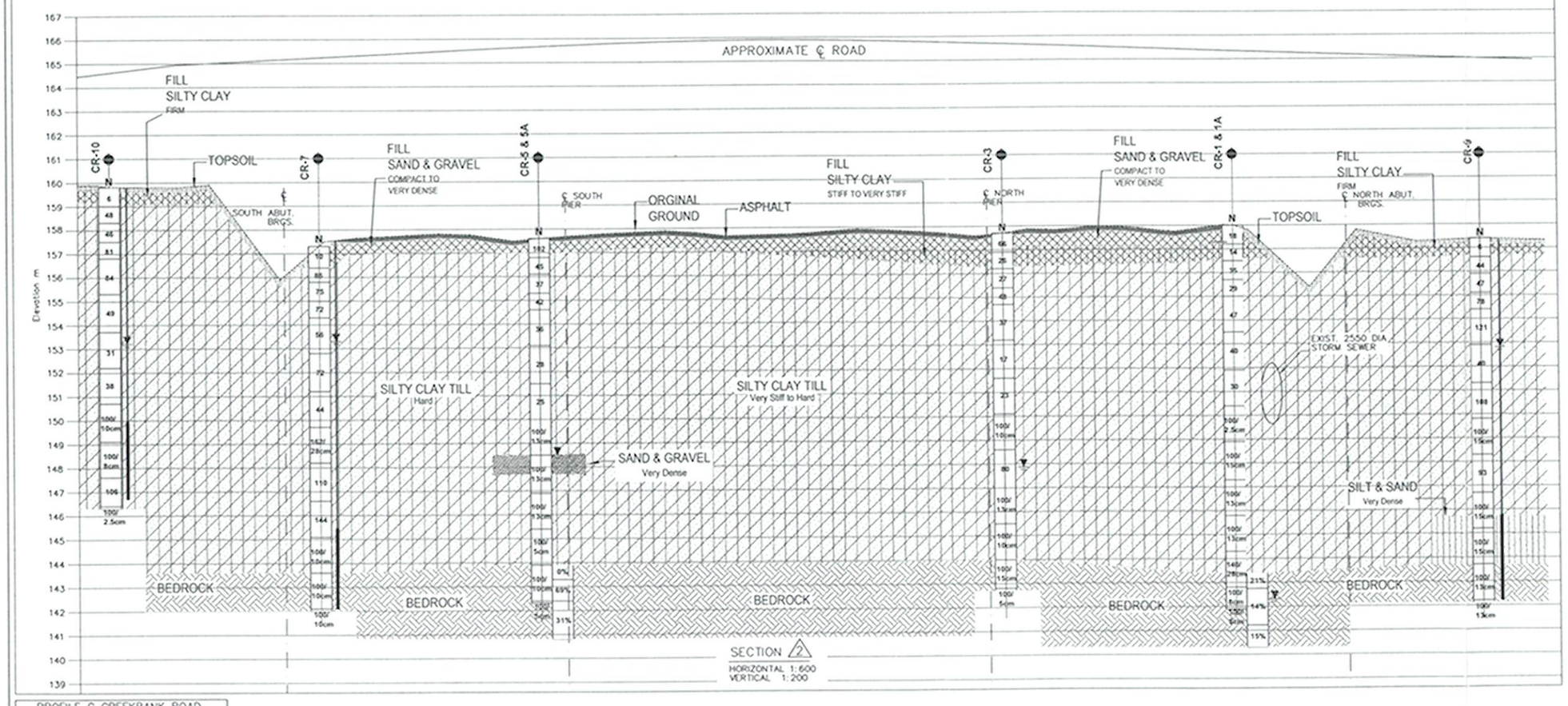
DESIGN BY | APPROVED BY

MISSISSAUGA
Leadingtodayfortomorrow
PRODUCED FOR – T&W, ENGINEERING AND WORKS

HIGHWAY 401 UNDERPASS at CREEKBANK ROAD

SCALE HOR. 1:600 VERT. 1:200	AREA	PROJECT No.
C.A.D.D. BY S.F.	CHECKED BY R.A.	PLAN No. C–
DATE FEB 2008	SHEET __ OF __	





PROFILE ℄ CREEKBANK ROAD
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6m 0 6m HOR
2m 0 2m VERT

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50mm ON ORIGINAL DRAWING

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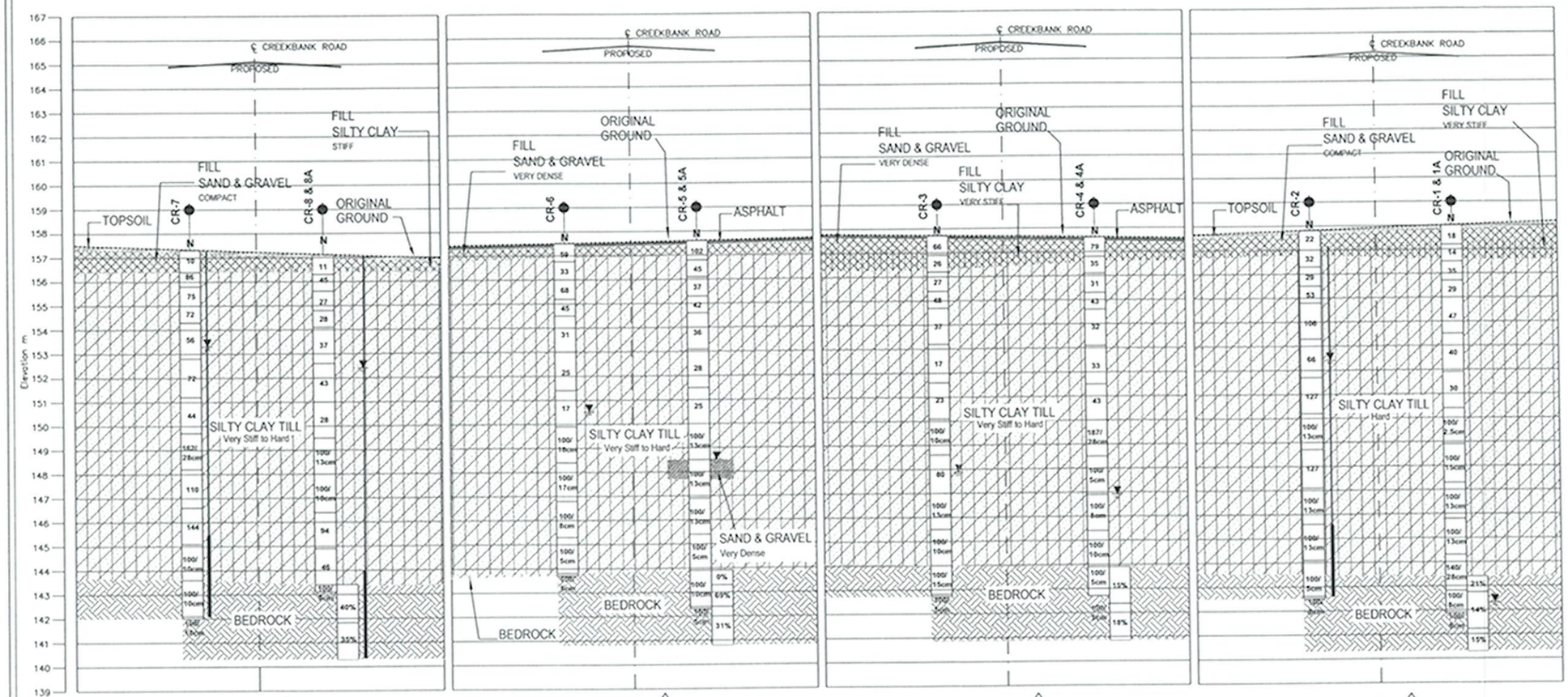
HIGHWAY 401 UNDERPASS at CREEKBANK ROAD

SCALE HOR. 1:600 VERT. 1:200	AREA	PROJECT No.
C.A.D.D. BY S.F.	CHECKED BY R.A.	PLAN No. C–
DATE FEB 2008	SHEET __ OF __	



SECTION 3
HORIZONTAL 1:600
VERTICAL 1:200

SECTION 4
HORIZONTAL 1:600
VERTICAL 1:200

SECTION 5
HORIZONTAL 1:600
VERTICAL 1:200

SECTION 6
HORIZONTAL 1:600
VERTICAL 1:200

PROFILE ℄ CREEKBANK ROAD
SECTION 1

6m 0 6m HOR
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APPENDIX D

Foundation Comparison

Terraprobe Limited

COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

Foundation Element	Driven Piles	Augered Caissons	Footing on Native Soil
North & South Abutments	*Advantages:* i. High geotechnical resistances available. ii. Allows choice of integral or semi-integral abutment design. iii. Readily installed. *Disadvantages:* i. Higher unit cost compared to footings. ii. Construction concerns related to the possibility of piles being obstructed by a boulder during driving.	*Advantages:* i. High geotechnical resistances available. *Disadvantages:* i. Precludes consideration of an integral abutment structure. ii. Higher unit cost compared to other options. iii. Construction concerns related to caisson excavation encountering boulders during construction.	*Advantages:* i. High geotechnical resistances available. ii. Less costly compared to other options. *Disadvantages:* i. Precludes consideration of an integral abutment structure. ii. Conventional abutments may be impractical due to height.
Piers	*Advantages:* i. High geotechnical resistances available. ii. Readily installed. iii. Only pile driving equipment required on site thereby resulting in lower equipment mobilization costs. *Disadvantages:* i. Higher unit cost compared to spread footings. ii. Construction concerns related to the possibility of piles being obstructed by a boulder during driving. iii. Safety concerns related to pile driving operations distracting highway users.	*Advantages:* i. High geotechnical resistances available. ii. From a safety perspective, less distracting to highway users compared to pile installation. *Disadvantages:* i. Higher unit cost compared to other options. ii. Higher equipment mobilization costs since both pile driving and caisson construction equipment required on site if an integral abutment bridge is constructed. iii. Construction concerns related to caisson excavation encountering boulders during construction.	*Advantages:* i. Less costly compared to other options. *Disadvantages:* i. Design loads may require relatively large footing sizes. ii. Will require relatively large open cut excavations in an area where limited space is available.

APPENDIX E

Figures

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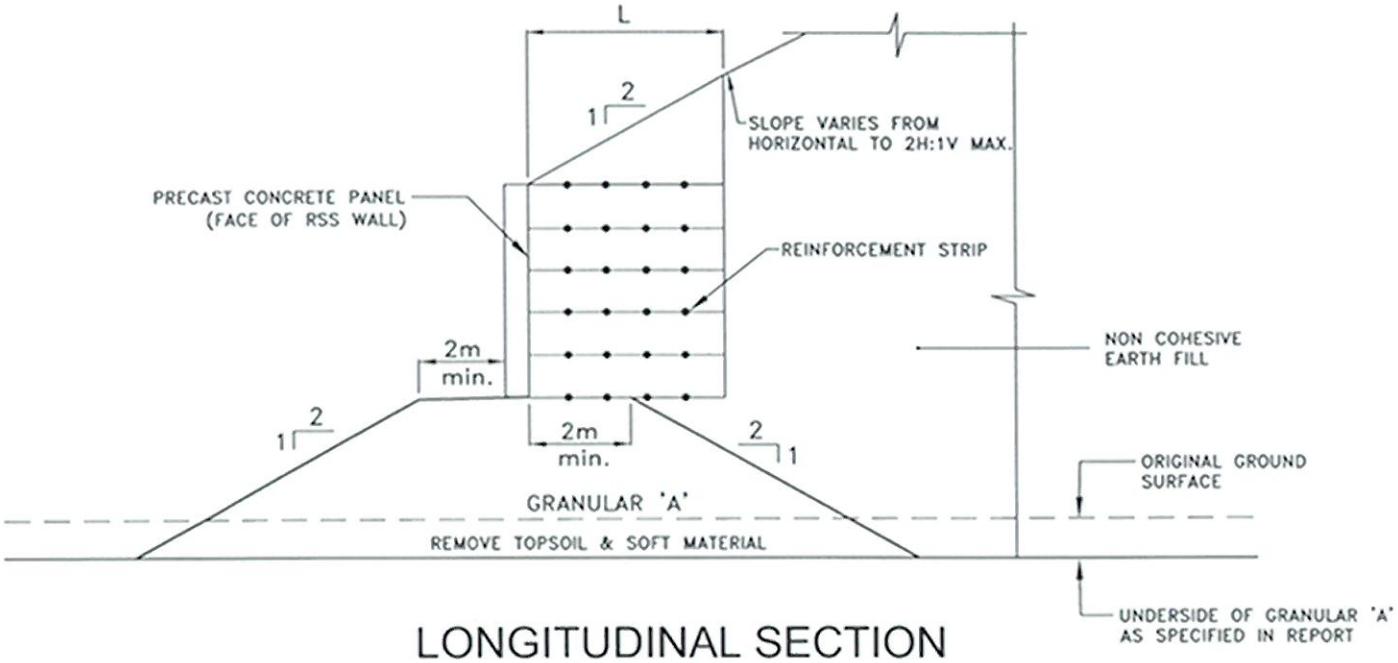

LONGITUDINAL SECTION

N.T.S.

NOTES:

1. REMOVE TOPSOIL AND OR SOFT SUBSOIL UNDER AREA OF COMPACTED GRANULAR 'A' AND EARTH FILL.
2. PLACE GRANULAR 'A' AND EARTH FILL TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO OPSS 501.
3. CONSTRUCT LEVELING PAD AND RSS MASS.
4. PLACE REMAINDER OF GRANULAR "A" AND EARTH FILL AS REQUIRED.

RSS MASS ON COMPACTED FILL SHOWING GRANULAR 'A' CORE

APPENDIX F

Suggested NSSP Wording

Terraprobe Limited

In this report reference is made to the following Provincial Standard:

- SP 903S01

The contract documents should contain a NSSP containing the following wording:

Cobbles and Boulders

"The Contractor is informed that the soils at this site may contain cobbles and boulders that could impede the progress of pile driving and caisson construction operations. The soil conditions are described in the Foundation Investigation Report prepared for this site".

If a pile encounters refusal on cobbles and/or boulders the QVE should terminate driving before the pile is damaged by overdriving. If the required resistance according to the Hiley Formula is not achieved and further driving is likely to cause damage to the pile, pile driving should cease and the contract administrator should be notified.

Boulder excavations during caisson construction may require the use of specialized equipment. Solidly embedded boulders may have to be cored. The Contractor should be fully prepared to undertake boulder excavations as and where required.

Boulders encountered at the recommended caisson base elevations will require removal and the excavation must be extended deeper to competent strata to be approved by the QVE.