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Construction Materials Engineering, Inspection & Testing

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FOUNDATION INVESTIGATION & DESIGN REPORT
VALLEYWOOD BOULEVARD UNDERPASS STRUCTURE
HIGHWAY 410 EXTENSION – PHASE III
FROM 300 m EAST OF HEART LAKE ROAD TO HIGHWAY 10
AGREEMENT No. 2005-A-000230, W.P. 10~~8~~5-00-01, SITE #: 24-745

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted at the site of the Valleywood Boulevard underpass structure on the proposed four-lane of Highway 410 in the Town of Caledon, Ontario. Previous, preliminary investigations were carried out by Golder Associates Ltd. (Golder) and the Ministry of Transportation (MTO), and the factual data from these investigations have been used as general reference for the preparation of this report.

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 Ltd., under the Ministry of Transportation Ontario (MTO) Agreement Number 2005-A-000230.

The following documents are referenced in the preparation of this report:

- Golder Associates Ltd., "Supplementary Foundation Feasibility Investigation, Proposed Highway 410 Extension, Bovaird Drive to Highway 10", W.P. 22-79-00, MTO District 6, Toronto, GEOCRES No. 30M12-208, dated April 1999.
- Ministry of Transportation, "Highway 410 Route Planning Study, Bovaird Drive Northerly to Highway 10", W.P. 22-79-00, MTO District 6, GEOCRES 30M12-208, dated January 24, 1989.

2 SITE DESCRIPTION

The site is located about 50 m east of Valleywood Boulevard and 175 m northeast of Highway 10 in the Town of Caledon. There are residential subdivisions to the north and east of the site. The topography across the site is flat with light vegetation consisting of grass and small shrubs.

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 Queenston Formation.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project were carried out during the period February 15 to 28, 2005 and consisted of drilling and sampling six boreholes to depths ranging from 12.4 m to 26.4 m. The boreholes were numbered VB1, VB2, VB3, VB4, VB5 and VB6 and their approximate locations are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix C.

The borehole locations were established in the field by surveyors from Shiu Geomatics Limited who also provided Terraprobe with their coordinates and geodetic elevations. Terraprobe obtained utility clearances prior to drilling.

The drilling, sampling and in-situ testing operations were conducted with a track mounted CME 75 drill rig owned and operated by Groundworks Drilling Limited of Toronto, Ontario. Solid and hollow-stem auger drilling techniques were used to advance the boreholes. Samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT) in the overburden soils.

Groundwater conditions in the open boreholes were observed throughout the drilling operations. Standpipes piezometers consisting of 19 mm PVC pipe with a slotted screen (enclosed either in sand or geotextile fabric) were installed in selected boreholes to permit longer term groundwater level monitoring. The locations and completion details of the piezometers are shown in Table 3.1.

Table 3.1 – Piezometer Installation Details

Piezometer Location	Piezometer Details	
	Tip Depth/ Elevation (m)	Completion Details
VB1	12.4/245.9	Piezometer with 1.5 m slotted screen installed with filter sand to 10.8 m, bentonite seal from 10.8 m to 9.1 m, drill cuttings from 9.1 m to 0.6 m and bentonite seal from 0.6 m to ground surface.
VB2	24.3/234.9	Piezometer with 1.5 m slotted screen wrapped in filter cloth, drill cuttings from 24.3 m to 0.9 m and bentonite seal from 0.9 m to ground surface.

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

Table 3.1 – Piezometer Installation Details (Cont'd)

VB3	26.2/233.0	Piezometer with 1.5 m slotted screen wrapped in filter cloth, drill cuttings from 26.2 m to 0.6 m and bentonite seal from 0.6 m to ground surface.
VB5	25.9/232.2	Piezometer with 1.5 m slotted screen wrapped in filter cloth, drill cuttings from 25.9 m to 1.2 m and bentonite seal from 1.2 m to ground surface.
VB6	13.7/244.1	Piezometer with 1.5 m slotted screen installed with filter sand to 12.2 m, bentonite seal from 12.2 m to 11.8 m, drill cuttings from 11.8 m to 0.9 m and bentonite seal from 0.9 m to ground surface.

Members of Terraprobe's 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 also subjected to gradation analysis. Atterberg Limits tests and unit weight 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 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" drawing 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 topsoil and overburden deposits of clayey silt till, sand and silt till, silty sand, clayey silt and silt till.

5.1 Topsoil

Topsoil approximately 100 mm to 500 mm thick was encountered across the site. Topsoil thickness may vary between and beyond the boreholes.

5.2 Clayey Silt Till

Across the site a clayey silt till deposit was encountered below the topsoil layer. This clayey silt till layer extends to depths ranging from 10.1 m (Elev. 248.2 m) to 13.2 m (Elev. 245.6 m) below ground surface.

The grain size distribution curves of tested samples of this clayey silt till are presented in Figure B1. These results show a grain size distribution consisting of 4-9% gravel, 18-33% sand, 40-46% silt and 16-38% clay size particles. Till soils are also known to contain cobbles and boulders due to their mode of deposition.

Two samples were also subjected to Atterberg Limits tests and the results are presented in Figure B2. The index values from these tests are summarized below:

Liquid Limit:	23-28%
Plastic Limit:	14-15%
Plasticity Index:	9-13%
Natural Moisture Content:	9-16%

These values are characteristic of clayey soils of low plasticity.

Standard Penetration tests in this clayey silt till layer yielded 'N' values ranging from 7 to 52 blows for 0.3 m penetration indicating a firm to hard consistency. The moisture content of samples from this deposit ranged from 7% to 21% by weight.

5.3 Sand and Silt Till

The site is underlain by layers of sand and silt till. These layers were fully penetrated in the deeper boreholes (Boreholes VB2, VB3, VB4 and VB5) at depths of 20.8 m to 23.9 m below ground surface or to elevations ranging from 238 m to 234.3 m. At some of the boreholes (Boreholes VB2, VB3 and VB5) the sand and silt till is divided by a layer of silty sand.

The results of grain size distribution tests conducted on samples obtained from these deposits are illustrated in Figure B3. These results show grain size distributions consisting of 1-35% gravel, 28-59% sand, 29-49% silt and 2-9% clay size particles. Cobbles and boulders can also be expected in till soils.

Standard Penetration tests in the sand and silt till gave 'N' values that ranged from 14 to more than 100 blows for 0.3 m penetration. Based on these results the sand and silt till is considered to have a generally compact to very dense relative density. The moisture content of samples from this stratum ranged from 4% to 16% by weight.

5.4 Silty Sand

In some of the deep boreholes a layer of silty sand was encountered. This deposit was not encountered at Borehole VB4. This silty sand layer extends to depths ranging from 15.4 m to 20.2 m below ground surface or to elevations ranging from 242.7 m to 239 m.

The grain size distribution curve of a sample from this deposit is illustrated in Figure B4.

The silty sand deposit is considered to have a very dense relative density based on SPT 'N' values that ranged from 51 to 57 blows for 0.3 m penetration. The moisture content of samples from the deposit ranges from 17% to 22% by weight.

5.5 Clayey Silt

A layer of grey clayey silt was encountered in some boreholes. In Borehole VB5 this layer is 1.6m thick and extends to a depth of 25.4 m (Elev.232.7 m) below ground surface. In Boreholes VB3 and VB4 the clayey silt layer extends to borehole termination depths of 26.2 m below ground surface.

Two samples from this clayey silt deposit were subjected to grain size distribution tests and the results are illustrated in Figure B5. These results show a grain size distribution consisting of 0 % gravel, 1-7 % sand, 65-71 % silt and 22-34 % clay size particles.

A sample of the clayey silt was also subjected to an Atterberg Limits test and the results are presented in Figure B6. The index values from this test are summarized below:

Liquid Limit:	24%
Plastic Limit:	19%
Plasticity Index:	5%
Natural Moisture Content:	20%

These values are characteristic of clayey soils of low plasticity

SPT 'N' values ranged from 100 to more than 100 blows for 0.3 m penetration in the deposit indicating a hard consistency. The moisture content by weight of samples from this deposit varies from 11% to 20%.

5.6 Silt Till

Discontinuous layers of silt till were encountered across the site. This deposit was encountered in Boreholes VB2 and VB5 where it extends to the termination depths of the boreholes and possibly beyond.

The grain size distribution curve of a sample from this deposit is illustrated in Figure B7. These results show a grain size distribution consisting of 0 % gravel, 4 % sand, 90 % silt and 6 % clay size particles.

SPT 'N' values more than 100 blows for 0.3 m penetration were obtained in this deposit indicating a very dense relative density. The moisture content of samples from this stratum ranged from 16% to 19% by weight.

5.7 Water Levels

A standpipe piezometer was installed in Boreholes VB1, VB2, VB3, VB5 and VB6. The water level readings measured on separate visits made after the completion of drilling are presented in Table 5.1.

Table 5.1 – Water Level Measurements

Borehole	Date	Water Levels	
		Depth (m)	Elevation (m)
VB1	April 18, 2005	7.5	250.8
	Sept. 09, 2005	8.2	250.1
VB2	April 18, 2005	8.7	250.5
	Sept. 09, 2005	8.8	250.4
VB3	April 18, 2005	8.5	250.7
	Sept. 09, 2005	9.2	250.0
VB5	April 18, 2005	8.2	249.9
	Sept. 09, 2005	9.0	249.1
VB6	April 18, 2005	7.0	250.8
	Sept. 09, 2005	*	*

* Damaged piezometer

These observations suggest that the local groundwater level at the site varies from about 249± m to 251± m. 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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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 two span underpass structure is required to carry the realigned Valleywood Boulevard over the proposed four lanes of Highway 410. Between the north abutment and pier the span will be 27.8 m. A span length of 25 m is proposed between the south abutment and pier. The structure width varies from 27.1± m to 21.8± m at the north and south abutments respectively.

The proposed finished grades at the structure will be about Elev. 266.7± m at the north abutment and Elev. 267.0± m at the south abutment. The approach fill heights at the north and south abutments will be about 9.0± m and 8.5± m respectively. RSS walls are also proposed at both abutments.

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

7 STRUCTURE FOUNDATIONS

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

The stratigraphy encountered at the abutment and pier locations consists of a surficial layer of topsoil underlain by firm to hard clayey silt till and compact to very dense sand and silt till. There are also discontinuous layers of very dense silty sand, hard clayey silt and very dense silt till. The groundwater level ranges between Elev. 249± m and Elev. 251± m.

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 is included in Appendix D.

7.1 Spread Footings (Abutments)

Based on the subsurface stratigraphy encountered at this site and the vertical alignment of Valleywood Boulevard, both bridge abutments can be supported on spread footings.

The recommended founding depths and geotechnical resistances for abutment 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 U.L.S (kPa)	Geotech. Resistance at S.L.S (kPa)	Subgrade Material
VB2 South Abutment	259.2	1.2	258.0	375	250	Clayey Silt Till
VB5 North Abutment	258.1	1.1	257.0	375	250	Clayey Silt Till

These values are for vertical, concentric loads only. Effects of load inclination and eccentricity should be taken into account as illustrated in CHBDC, 2000 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.

Resistance to lateral forces/sliding resistance between the concrete footing and very stiff clayey silt till should be evaluated in accordance with the CHBDC, 2000. Assume an ultimate coefficient of friction of 0.6.

7.2 Spread Footings (Pier)

A spread footing is not considered to be a feasible option for supporting the pier. Stiff to very stiff clayey silt till was encountered in Borehole VB4. The geotechnical resistance of the soils encountered at Borehole VB4 is relatively low and settlements under the footing load will be relatively large.

Consideration was also given to placing the footings on an engineered fill pad. This scheme will require excavations extending to about Elev. 253± m for a 2 m wide footing based on a proposed top of footing elevation of 257.9 m. This is likely to be impractical since excavations up to 6± m below ground surface will be required.

7.3 Augered Caissons (Drilled Shafts)

Augered caisson foundations were also considered for supporting the structure. However, the overburden at the abutment locations is not considered to be suitable for this scheme. The caissons will be founded in the very dense sand and silt till at depths in the order of 21 m to 22 m below original ground surface. The base of the caissons would be about 12± m to 14± m below the groundwater level, resulting in high hydrostatic heads at the base.

The permeable nature of the sand and silt till at the abutment locations would make it difficult to seal the bottom of the liner into the founding stratum to exclude groundwater. Unwatering of the caisson would also be impractical.

Installation of deep caissons is also expected to be a more expensive option than driven piles.

For these reasons, a caisson foundation is not recommended.

7.4 Driven Piles

The subsurface conditions at the site are considered suitable for the design of foundations supported on steel H-piles.

At the abutment locations steel H-piles are likely to encounter effective refusal in the very dense sand and silt till. At the pier location steel H-piles are expected to encounter effective refusal in the hard clayey silt.

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

7.4.1 Axial Resistance

Piles driven at the abutment and pier locations and encountering effective refusal in the very dense sand and silt till or the hard clayey silt till should be designed on the basis of the concentric, axial geotechnical resistances given in Table 7.2. The actual pile tip elevations will be controlled as described in Section 7.4.7 Pile Installation.

Table 7.2 – Axial Resistance of Various Pile Sections

Location	PILE TYPE - HP 310x110				
	Reference Borehole	Estimated Pile Tip Elevation (m)	Founding Stratum	Factored Axial Resistance U.L.S (kN)	SLS (25 mm Settlement) (kN)
North Abutment	VB5	235.5±	Sand & Silt Till	1600	1200
South Abutment	VB2	238.0±	Sand & Silt Till		
Pier	VB3 & VB4	236.0±	Clayey Silt		

Table 7.2 – Axial Resistance of Various Pile Sections (Cont'd)

Location	PILE TYPE – HP 360X132				
	Reference Borehole	**Estimated Pile Tip Elevation (m)**	**Founding Stratum**	**Factored Axial Resistance U.L.S (kN)**	**SLS (25 mm Settlement) (kN)**
North Abutment	VB5	234.5±	Sand & Silt Till	2100	1600
South Abutment	VB2	237.0±	Sand & Silt Till		
Pier	VB3 & VB4	235.0±	Clayey Silt		

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.

Oversize materials (e.g. greater than 75 mm nominal diameter) should not be used in the embankment fills through which the piles will be driven.

7.4.2 Downdrag

Downdrag on the piles is not considered to be an issue at this site.

7.4.3 Integral Abutment Considerations

The ground conditions at this site are considered suitable for an integral abutment design.

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 of pile will lie in the compacted fill of the approach embankment. Therefore, to provide the required flexibility in the piles, the upper 3 m of the piles should be surrounded by a 600 mm diameter CSP as specified by the integral abutment design procedures.

After the pile is driven, the space between the pile and the CSP should be filled with sand. An NSSP should be included in the contract drawings specifying the gradation of the sand according to Table 7.3.

Table 7.3 – 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%

7.4.4 Lateral Resistance

The lateral resistance of the 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} = $3 \,.\, \gamma \,.\, z \,.\, K_p$ [cohesionless soils] (kPa)
p_{ult} = $9\ S_u$ [cohesive soils] (kPa)

where z = depth of embedment of pile (m)
D = pile width (m)
S_u = undrained shear strength (Table 7.4) (kPa)
n_h = coefficient of horizontal subgrade reaction (Table 7.4) (kN/m^3)
γ = unit weight (Table 7.4) (kN/m^3)
K_p = passive earth pressure coefficient $(1+\sin\phi)/(1-\sin\phi)$
ϕ = angle of internal friction (Table 7.4) (degrees)

The above equations and recommended parameters may be used to analyze the interaction between a 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 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 (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 7.4 – Recommended Soil Parameters

Area Reference Borehole No	Applicable Elevation	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 VB5	258.0 – 257.5	Clayey Silt Till	20	0	50	-
	257.5 – 249.5	Clayey Silt Till	21	0	200	-
	249.5 – 245.7	Clayey Silt Till	21	0	100	-
	245.7 – 243.4	Sand & Silt Till	21	35	-	8500
	243.4 – 242.7	Silty Sand	21	35	-	8500
	242.7 – 234.3	Sand & Silt Till	21	35	-	8500
	234.3 – 232.7	Clayey Silt	21	0	200	-
	232.7 – 231.7	Silt Till	21	35	-	8500
South Abutment VB2	259.1 – 258.4	Clayey Silt Till	20	0	50	-
	258.4 – 247.5	Clayey Silt Till	21	0	200	-
	247.5 – 246.0	Sand & Silt Till	19	30	-	4000
	246.0 – 241.5	Sand & Silt Till	21	35	-	8500
	241.5 – 239.9	Silty Sand	21	35	-	8500
	239.9 – 235.3	Sand & Silt Till	21	35	-	8500
	235.3 – 234.5	Silt Till	21	35	-	8500

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

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

For lateral soil/pile 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:

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 pile, and spacing is measured centre to centre

Where a 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:

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.

For conventional abutments and at the pier, the lateral resistance may be provided by battered piles.

7.4.5 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.

7.4.6 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.

7.4.7 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	3,600 kN
HP 360X132	4,800 kN

Hiley formula calculations need not be carried out until the pile has been driven below Elev. 239 m at the south abutment, Elev. 236.5 m at the north abutment, and Elev. 237 m at the pier location.

7.5 Recommended Foundation

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 all foundations for the main bridge structure be supported on steel H-piles.

7.6 Frost Cover

Pile caps and footings should be provided with a minimum of 1.2 m of earth cover over the footing base (founding elevation).

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 4 soils below the water table. Excavation below the groundwater level is not recommended without prior dewatering. Provided dewatering is carried out as described below, excavations may be sloped at 2H:1V

8.2 Foundations

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

9 GROUNDWATER CONTROL

The groundwater level is estimated to range between 7.0 m and 9.0 m below ground surface and it is unlikely that excavations will extend below the groundwater level.

If dewatering is required the design of the unwatering system should be the responsibility of the Contractor. A suitable system that might be employed can include gravity drainage from dug ditches and pumping from strategically placed filtered sumps.

Any accumulation of water from the base of the excavation 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 EMBANKMENTS

Approach embankment construction using either non-cohesive earth fill or rock fill is feasible provided the embankment is constructed on the underlying stiff to hard clayey silt glacial till encountered at this site. At the north approach this will require stripping of the surficial soils below the footprint of the embankment to elevations ranging from about 257 m (Borehole VB6) to 257.5 m (Borehole VB5). At the south approach the surficial soils below the footprint of the embankment should be stripped to elevations ranging from about 257.5 m (Borehole VB1) to 258.5 m (Borehole VB2).

Oversize materials (e.g. greater than 75 mm nominal diameter) should not be used in the embankment fills through which piles will be driven.

The total post construction settlement due to loads imposed by 8.5± m to 9± m high approach embankments is estimated to be in the order of 75 mm. A significant portion of this settlement will be essentially complete by the end of construction.

The embankments will also experience settlement resulting from consolidation of the fill. This settlement is expected to range from about 85 mm to 90 mm for 8.5± m and 9± 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 granular material, select subgrade material or non-cohesive earth fill will have stable side slopes at inclinations of up to 2H:1V. If the embankment is constructed of rock fill, it may be assumed that the side slopes will be stable at inclinations up to 1.25H:1V.

For the purpose of embankment stability analyses, the commercially available slope stability program Slope W developed by Geo-Slope International Ltd. 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 and for 1.25H:1V rock fill embankments.

For earth or rock fill embankments constructed on the native very stiff clayey silt till, factors of safety against global failure of 1.4 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.

The approach fills should be constructed in advance of pile driving operations.

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 572.

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 clayey silt 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
				ULS	SLS		
NW Wall	258.5 for 3.2 m	8.2	213	340	225	VB5 Clayey Silt Till	Subexcavate to Elev. 257 m, replace with 1.5 m of compacted Gran. A to base of wall.
	262.0 for 6.0 m	4.7	122	225	150	VB5 New Fill	Compacted 1.5 m of Gran. A to underside of RSS along step.
NE Wall	258.5 for 6.5 m	8.2	213	340	225	VB5 Clayey Silt Till	Subexcavate to Elev. 257 m, replace with 1.5 m of compacted Gran. A to base of wall.
	262.0 for 6.0 m	4.7	122	225	150	VB5 New Fill	Compacted 1.5 m of Gran. A to underside of RSS along step.
North Abutment Face	258.5	8.2	213	340	225	VB5 Clayey Silt Till	Subexcavate to Elev. 257 m, replace with 1.5 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
				ULS	SLS		
SW Wall	259.0 for 3.5 m	8.0	208	340	225	VB2 Clayey Silt Till	Subexcavate to Elev. 258 m, replace with 1 m of compacted Gran. A to base of wall.
	262.0 for 6.0 m	5.0	130	225	150	VB2 New Fill	Compacted 1.5 m of Gran. A to underside of RSS along step.
SE Wall	259.0 for 5.1 m	8.0	208	340	225	VB2 Clayey Silt Till	Subexcavate to Elev. 258 m, replace with 1 m of compacted Gran. A to base of wall.
	262.0 for 6.0 m	5.0	130	225	150	VB2 New Fill	Compacted 1.5 m of Gran. A to underside of RSS along step.
South Abutment Face	259.0	8.0	208	340	225	VB2 Clayey Silt Till	Subexcavate to Elev. 258 m, replace with 1 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 clayey silt 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 ground surface extending to the full height, or in the form of a triangular wedge resting on a lower embankment slope.

RSS walls are likely to 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 native clayey silt till.

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

- RSS founded at the base of the embankment.
- RSS founded on earth fill – outer slope of 2H:1V (angle of internal friction, ϕ, of 30°, cohesion of 0, and unit weight, γ, of 19 kN/m^3).
- Fill behind the RSS is horizontal.

Analysis carried out on RSS walls located at the base of the embankment yielded a factor of safety 1.5 using a conventional anchor length of 60% of the height of the wall. However, for analyses carried out on RSS walls located up a slope in the embankment earth fill, anchor lengths of up to 175% of the wall height are required to achieve an acceptable factor of safety of 1.4.

Consequently, it may be assumed that RSS walls founded at the base of the embankment will be stable against global failure. For an RSS wall founded in the embankment slope, the specific geometry and soil conditions must be analyzed to determine the requirements for global stability. 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. A NSSP is required to specify grading limits for the rock fill. The rock fill used as backfill to the abutment should be limited to fragments no greater than 300 mm and should include adequate spalls to fill voids in the rock fill.

In all cases where the approach embankment consists of rock fill and granular backfill to the abutment wall is used, the granular backfill must consist of OPSS Granular B Type II.

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 3501.000, and rock backfill should be placed to the extents shown in OPSD 3505.000.

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 3501.000 or OPSD 3505.000, as applicable.

13 EARTH PRESSURE

For cases where backfill to the abutment is placed in accordance with OPSD 3501.000 or OPSD 3505.000, 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 and 1.25H:1V for rock 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)

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 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		Rock Fill $\phi = 42°$; $\gamma = 19.0$ kN/m^3	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	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*	0.20	0.28*
At rest (Restrained Wall)	0.43	-	0.47	-	0.33	-
Passive (Movement Towards Soil Mass)	3.70	-	3.30	-	5.0	-

* 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. However, the use of Granular B Type I may be restricted if the approach embankment consists of rock fill.

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, 2000.

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 Table 4.4.6.1 of the CHBDC, a Site Coefficient "S" (ground motion amplification factor) of 1.0 should be used in seismic design.

14.2 Liquefaction Potential

The potential for liquefaction of the foundations soils was assessed using the Seed and Idriss (1971) method[1]. There is no potential for liquefaction of the foundation soils below the abutments and pier.

Furthermore, since the foundation loads will be transferred by steel piles to the very dense sand and silt till or the hard clayey silt, the vertical geotechnical resistance of the piles will not be compromised.

The immediate approach embankments will bear on very stiff to hard clayey silt till above the groundwater level and therefore there is negligible potential for soil liquefaction below the embankments. Some toe failure may occur but is expected to be limited and readily repairable.

[1] Seed, H.B. and Idriss, I.M. 1971, "Simplified Procedure for Evaluating Soil Liquefaction Potential" *Journal of Soil Mechanics and Foundations Division,* ASCE, Vol. 101, No. SM9, September, pp. 1249-1273.

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						
Wall Condition	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		Rock Fill $\phi = 42°$; $\delta = 21°$ $\gamma = 19.0$ kN/m^3	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (K_{AE})*	0.28	0.45	0.31	0.55	0.21	0.30
Passive (K_{PE})	3.69	-	3.26	-	5.05	-
At Rest (K_{OE})**	0.53	-	0.58	-	0.44	-

* 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 piles encountering boulders.
- the nature and geotechnical properties of non-cohesive earth fill used in the approach fills.

LICENSED PROFESSIONAL ENGINEER
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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

This report was prepared for the express use of the Ministry of Transportation, its retained design consultants and Giffels Associates Ltd. It is not for use by others. The Ministry of Transportation, its retained design consultants and Giffels Associates Ltd., are authorized users.

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 VB1

METRIC

W.P. 108-00-01 | LOCATION N4844600.9;E278073.8 | ORIGINATED BY MS

DIST | HWY 410 Phase III | BOREHOLE TYPE Solid Stem Augers | COMPILED BY DB

DATUM Geodetic | DATE 15.02.05 | CHECKED BY RA

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
258.3										
0.0	300mm TOPSOIL, dark brown	1	SS	8						
258.0 / 0.3	CLAYEY SILT - Sandy, weathered, trace rootlets; trace gravel, damp to moist, firm to 0.7m, very stiff to hard below (GLACIAL TILL)	2	SS	34						
		3	SS	26			5	29	46	20
		4	SS	45		22.5				
		5	SS	38						
		6	SS	38						
		7	SS	21		23.2				
	brown / grey	8	SS	29						
		9	SS	36						
248.2 / 10.1	SAND AND SILT trace to some gravel, trace clay, damp to moist, very dense, grey (GLACIAL TILL)	10	SS	100						
		11	SS	100/ 25cm						
245.9 / 12.4	End of Borehole									

Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.

WATER LEVEL READINGS

Date	Depth(m)	Elevation(m)
Ap.18.05	7.5	250.8
Sept.09.05	8.2	250.1

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

ELEVATION SCALE: 258, 257, 256, 255, 254, 253, 252, 251, 250, 249, 248, 247, 246

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

RECORD OF BOREHOLE No VB2

METRIC

W.P. 108-00-01 | LOCATION N4844620.9;E278074.5 | ORIGINATED BY MS

DIST ___ HWY 410 Phase III | BOREHOLE TYPE Hollow Stem Augers | COMPILED BY DB

DATUM Geodetic | DATE 25.02.05 - 28.02.05 | CHECKED BY RA

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
259.2							
0.1	100mm TOPSOIL, dark brown / weathered, trace rootlets	1	SS	7	259		
	CLAYEY SILT - Sandy, trace gravel, damp to moist, firm to 0.7m, very stiff to hard below (GLACIAL TILL)	2	SS	22	258		
		3	SS	29			
		4	SS	34	257		
		5	SS	22	256		
	brown / grey	6	SS	32	255	22.5	
		7	SS	27			8 31 42 19
		8	SS	16	254	21.2	
		9	SS	25	253		
					252		
		10	SS	37	251		
		11	SS	29	250		Feb. 25, 2005 / Feb. 28, 2005
					249		
		12	SS	28	248		
247.5 / 11.7	SAND AND SILT some gravel, trace clay, moist, grey (GLACIAL TILL)	13	SS	14	247		15 40 39 6
	compact / very dense				246		
		14	SS	100/13cm	245		

Column headings: SOIL PROFILE; SAMPLES; 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

Continued Next Page

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

RECORD OF BOREHOLE No VB2

METRIC

W.P. 108-00-01 LOCATION N4844620.9;E278074.5 ORIGINATED BY MS

DIST ______ HWY 410 Phase III BOREHOLE TYPE Hollow Stem Augers COMPILED BY DB

DATUM Geodetic DATE 25.02.05 - 28.02.05 CHECKED BY RA

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SAND AND SILT some gravel, trace clay, moist, very dense, grey (GLACIAL TILL) *(continued)*	15	SS	100/ 15cm	
		16	SS	100/ 10cm	
241.5 / 17.7	**SILTY FINE SAND** wet, very dense, grey	17	SS	57	0 82 (18)
239.9 / 19.3	**SAND AND SILT** trace to some gravel, trace clay, damp to moist, very dense, grey (GLACIAL TILL)	18	SS	100/ 13cm	
		19	SS	100/ 13cm	
		20	SS	102/ 25cm	
235.3 / 23.9	**SILT** trace sand, trace clay, damp, very dense, grey (GLACIAL TILL)	21	SS	103	
234.5 / 24.7	End of Borehole				

Elevation scale: 244, 243, 242, 241, 240, 239, 238, 237, 236, 235

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

End of Borehole

Wet cave at 20.1m on completion.

Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen wrapped in filter cloth.

WATER LEVEL READINGS

Date	Depth(m)	Elevation(m)
Ap.18.05	8.7	250.5
Sept.09.05	8.8	250.4

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

RECORD OF BOREHOLE No VB3

METRIC

W.P. 108-00-01 | LOCATION N4844645.2;E278064.4 | ORIGINATED BY MS

DIST | HWY 410 Phase III | BOREHOLE TYPE Solid Stem Augers & Hollow Stem Augers | COMPILED BY DB

DATUM Geodetic | DATE 18.02.05 - 22.02.05 | CHECKED BY RA

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	UNIT WEIGHT γ (kN/m³)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
259.2							
0.0	500mm TOPSOIL moist, compact, dark brown	1	SS	12	259		
258.7 / 0.5	weathered, trace rootlets						
		2	SS	12	258		
	stiff ---- very stiff to hard	3	SS	17		21.3	
		4	SS	18	257		9 33 42 16
		5	SS	31	256	21.6	
	brown ---- grey				255		
	CLAYEY SILT - Sandy, trace gravel, damp to moist (GLACIAL TILL)	6	SS	16	254		
		7	SS	37	253		
					252		
		8	SS	26	251		
		9	SS	20	250		4 18 40 38
					249		Feb. 17, 2005 / Feb. 18, 2005
		10	SS	16	248		
	---- stiff	11	SS	14	247		
246.0 / 13.2	SAND AND SILT trace to some gravel, trace clay, moist to wet, very dense, grey (GLACIAL TILL)				246		
		12	SS	65	245		

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

Continued Next Page

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

RECORD OF BOREHOLE No VB3

METRIC

W.P. 108-00-01 LOCATION N4844645.2;E278064.4 ORIGINATED BY MS

DIST ______ HWY 410 Phase III BOREHOLE TYPE Solid Stem Augers & Hollow Stem Augers COMPILED BY DB

DATUM Geodetic DATE 18.02.05 - 22.02.05 CHECKED BY RA

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SAND AND SILT trace to some gravel, trace clay, moist to wet, dense to very dense, grey (GLACIAL TILL) *(continued)*	13	SS	80	244	
					243	
		14	SS	118	242	
					241	
		15	SS	34		3 39 49 9
239.9 19.3	**SILTY FINE SAND** wet, dense, grey				240	
239.0 20.2	**SAND AND SILT** trace to some gravel, trace clay, moist, very dense inferred, grey (Possible Till)	16	SS	51	239	
					238	
237.7 21.5	**CLAYEY SILT** trace sand, occasional silty fine sand seams and partings, moist, hard, grey	17	SS	119	237	
		18	SS	100	236	Feb. 18, 2005 / Feb. 21, 2005
					235	
		19	SS	103		0 1 65 34
					234	Feb. 21, 2005 / Feb. 22, 2005
233.0 26.2	End of Borehole	20	SS	100/ 14cm	233	

Column headings: 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

Wet cave at 10.7m on completion.

Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen wrapped with filter cloth.

WATER LEVEL READINGS

Date	Depth(m)	Elevation(m)
Ap.18.05	8.5	250.7
Sept.09.05	9.2	250.0

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

RECORD OF BOREHOLE No VB4

METRIC

W.P. 108-00-01 LOCATION N4844646.5;E278081.6 ORIGINATED BY MS

DIST ____ HWY 410 Phase III BOREHOLE TYPE Solid Stem Augers COMPILED BY DB

DATUM Geodetic DATE 17.02.05 CHECKED BY RA

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
258.8 / 0.0	**500mm TOPSOIL** moist, very loose, dark brown	1	SS	3						
258.3 / 0.5	weathered, trace rootlets				258					
		2	SS	14						
		3	SS	24	257	22.2				
		4	SS	20						
	brown ---- grey	5	SS	12	256					
					255					
		6	SS	15	254		5	31	46	18
	stiff to very stiff ---- hard				253					
		7	SS	34	252					
	---- stiff to very stiff									
	CLAYEY SILT - Sandy, trace gravel, damp to moist (GLACIAL TILL)	8	SS	13	251					
					250					
		9	SS	13	249	21.0				
		10	SS	13	248					
					247					
		11	SS	16	246					
245.6 / 13.2	**SAND AND SILT** - Gravelly, trace clay, damp to moist, very dense, grey (GLACIAL TILL)	12	SS	100/ 18cm	245		35	28	29	8
					244					

Continued Next Page

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

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No VB4

 METRIC

W.P. 108-00-01 LOCATION N4844646.5;E278081.6 ORIGINATED BY MS

DIST ____ HWY 410 Phase III BOREHOLE TYPE Solid Stem Augers COMPILED BY DB

DATUM Geodetic DATE 17.02.05 CHECKED BY RA

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SAND AND SILT - Gravelly, trace clay, damp to moist, grey (GLACIAL TILL) *(continued)*	13	SS	95	243	
		14	SS	90	242	
	very dense / compact				241	
		15	SS	25	240	
	very dense				239	
		16	SS	61	238	
238.0 20.8	**CLAYEY SILT** trace sand, occasional silty fine sand seams and partings, damp, hard, grey	17	SS	123	237	
					236	
		18	SS	118	235	0 7 70 23
		19	SS	126	234	
					233	
		20	SS	100/ 15cm		
232.6 26.2	End of Borehole					

Water level at 10.7m (not stabilized) and hole open to 18.9 on completion.

Attempted piezometer installation. Pipe dropped into hole during backfilling.

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O 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; STRAT PLOT

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

RECORD OF BOREHOLE No VB5

METRIC

W.P. 108-00-01	LOCATION N4844673.3;E278067.9		ORIGINATED BY MS
DIST	HWY 410 Phase III	BOREHOLE TYPE Solid Stem Augers & Hollow Stem Augers	COMPILED BY DB
DATUM Geodetic	DATE 23.02.05 - 24.02.05		CHECKED BY RA

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
258.1	
0.1	100mm TOPSOIL, dark brown weathered, trace rootlets, firm ---- **CLAYEY SILT** - Sandy, trace gravel, damp to moist (GLACIAL TILL) brown ---- grey very stiff to hard ---- stiff
245.7 / 12.4	**SAND AND SILT** trace to some gravel, trace clay, damp to moist, very dense, grey (GLACIAL TILL)
243.4 / 14.7	**SILTY SAND** - trace gravel

SAMPLES

NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ (kN/m³)
1	SS	7	
2	SS	52	
3	SS	23	
4	SS	25	
5	SS	43	
6	SS	25	
7	SS	33	21.7
8	SS	34	
9	SS	32	
10	SS	23	
11	SS	13	
12	SS	14	
13	SS	28	20.8
14	SS	83	

Column headings: STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (258 – 244); 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

Continued Next Page

+ ³, × ³: Numbers refer to Sensitivity

○ ³% STRAIN AT FAILURE

RECORD OF BOREHOLE No VB5

METRIC

W.P. 108-00-01 | LOCATION N4844673.3;E278067.9 | ORIGINATED BY MS

DIST ______ HWY 410 Phase III | BOREHOLE TYPE Solid Stem Augers & Hollow Stem Augers | COMPILED BY DB

DATUM Geodetic | DATE 23.02.05 - 24.02.05 | CHECKED BY RA

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILTY SAND - trace gravel, wet, grey *(continued)*				243	
242.7 / 15.4	**SAND AND SILT** trace to some gravel, trace clay, moist, dense to very dense, grey (GLACIAL TILL)	15	SS	54	242	
		16	SS	107	241	Feb. 23, 2005 / Feb. 24, 2005
		17	SS	34	240	
		18	SS	45	239 / 238	
		19	SS	100/ 13cm	237 / 236	
		20	SS	100/ 13cm	235	
234.3 / 23.8	**CLAYEY SILT** trace sand, trace gravel, damp, hard, grey	21	SS	159	234 / 233	
232.7 / 25.4	**SILT** trace sand, trace clay, damp, very dense, grey (GLACIAL TILL)	22	SS	110	232	0 4 90 6
231.7 / 26.4	End of Borehole					

Water level at 9.1m (not stabilized) and hole open to 18.3m on completion.

Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen wrapped in filter cloth.

WATER LEVEL READINGS

Date	Depth(m)	Elevation(m)
Ap.18.05	8.2	249.9
Sept.09.05	9.0	249.1

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; 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

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

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No VB6

METRIC

W.P. 108-00-01	LOCATION N4844693.5;E278068.7	ORIGINATED BY MS
DIST ____ HWY 410 Phase III	BOREHOLE TYPE Solid Stem Augers	COMPILED BY DB
DATUM Geodetic	DATE 15.02.05	CHECKED BY RA

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
257.8	
0.0	**400mm TOPSOIL** moist, loose, dark brown
257.4 / 0.4	weathered, trace rootlets
	CLAYEY SILT - Sandy, trace gravel, damp to moist, firm to stiff to 1.4m, very stiff to hard below (GLACIAL TILL)
	brown ---- grey
246.1 / 11.7	**SAND AND SILT** trace gravel, trace clay, damp to moist, very dense, grey (GLACIAL TILL)
243.6 / 14.2	End of Borehole. Wet cave at 13.7m on completion.

SAMPLES

NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ (kN/m³)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	7		
2	SS	11		
3	SS	20		
4	SS	42		
5	SS	45		
6	SS	45	21.7	
7	SS	22		
8	SS	17		
9	SS	15		
10	SS	16		
11	SS	100/ 23cm		
12	SS	73		1 59 38 2

Elevation scale: 257, 256, 255, 254, 253, 252, 251, 250, 249, 248, 247, 246, 245, 244

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

Continued Next Page

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

ONTARIO MOT 1-00-0350 HWY 410 VALLEYWOOD BLVD.GPJ ONTARIO MOT.GDT 20/09/05

Ministry of Transportation
Ontario

Foundation Design

RECORD OF BOREHOLE No VB6

METRIC

W.P. 108-00-01 LOCATION N4844693.5;E278068.7 ORIGINATED BY MS

DIST ____ HWY 410 Phase III BOREHOLE TYPE Solid Stem Augers COMPILED BY DB

DATUM Geodetic DATE 15.02.05 CHECKED BY RA

SOIL PROFILE		SAMPLES						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	GROUND WATER CONDITIONS	ELEVATION SCALE				
	Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS Date Depth(m) Elevation(m) Ap.18.05 7.0 250.8 Sept.09.05 - Damaged Piezometer										

ONTARIO MOT 1-00-0350 HWY 410 VALLEYWOOD BLVD.GPJ ONTARIO MOT.GDT 20/09/05

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

APPENDIX B

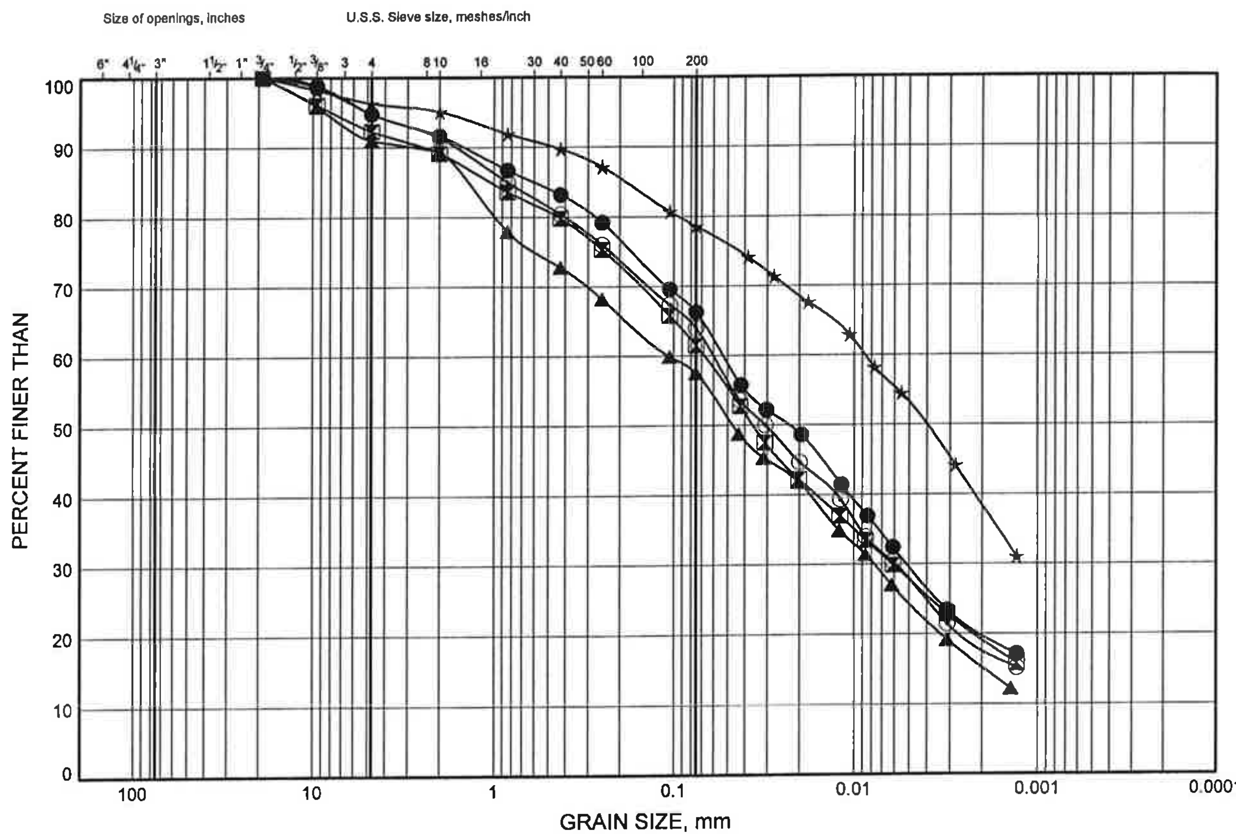
Laboratory Test Results

Terraprobe Limited

GRAIN SIZE DISTRIBUTION

FIGURE B1

Clayey Silt Till



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

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	VB1	1.7	256.6
⊠	VB2	4.7	254.5
▲	VB3	2.5	256.7
★	VB3	9.3	249.9
⊙	VB4	4.7	254.1

Date September 2005

Project 108-00-01

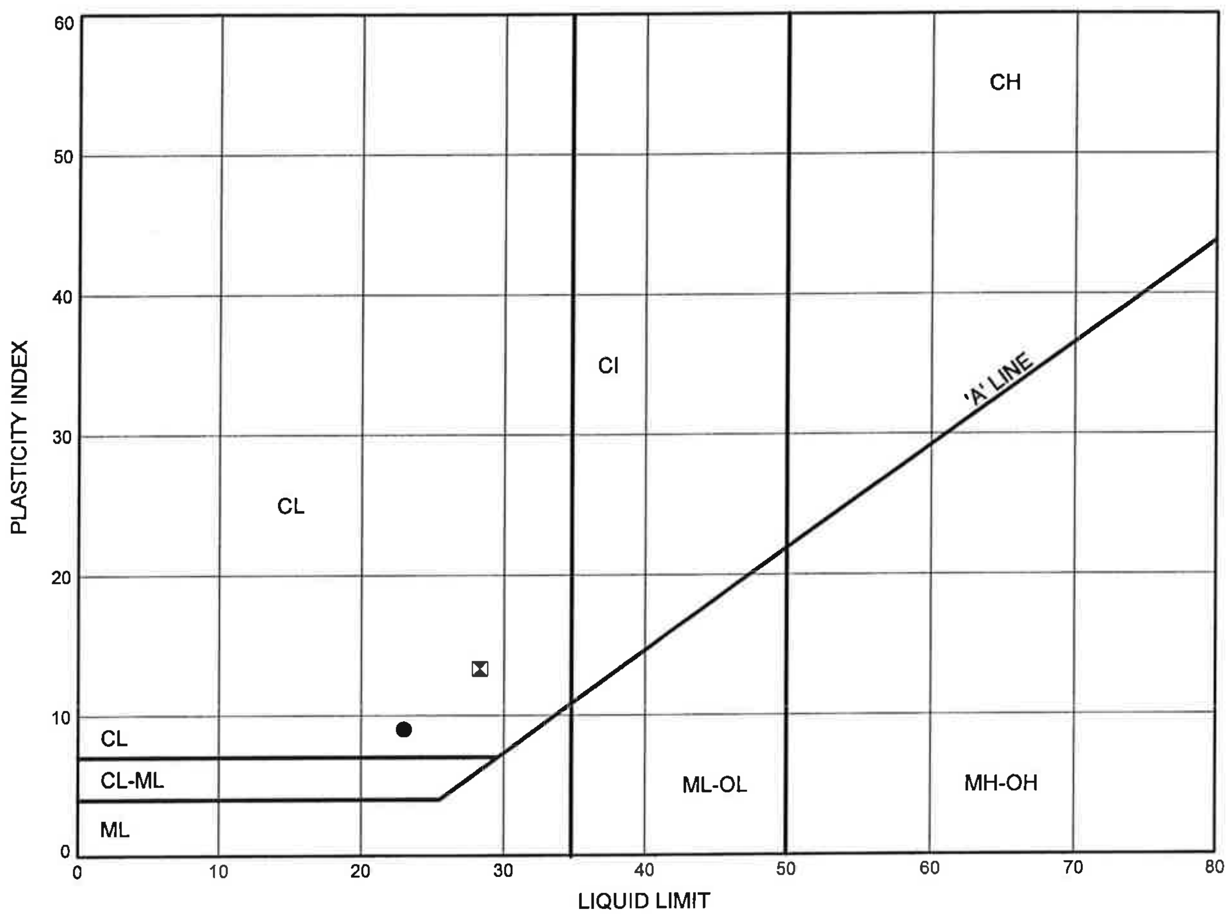
Prep'd DB

Chkd. RA

GSD 1-00-0350 HWY 410 VALLEYWOOD BLVD.GPJ 20/09/05

ATTERBERG LIMITS TEST RESULTS	FIGURE B2

Clayey Silt Till



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	VB2	4.7	254.5
☒	VB3	9.3	249.9

Date September 2005

Project 108-00-01

Prep'd DB

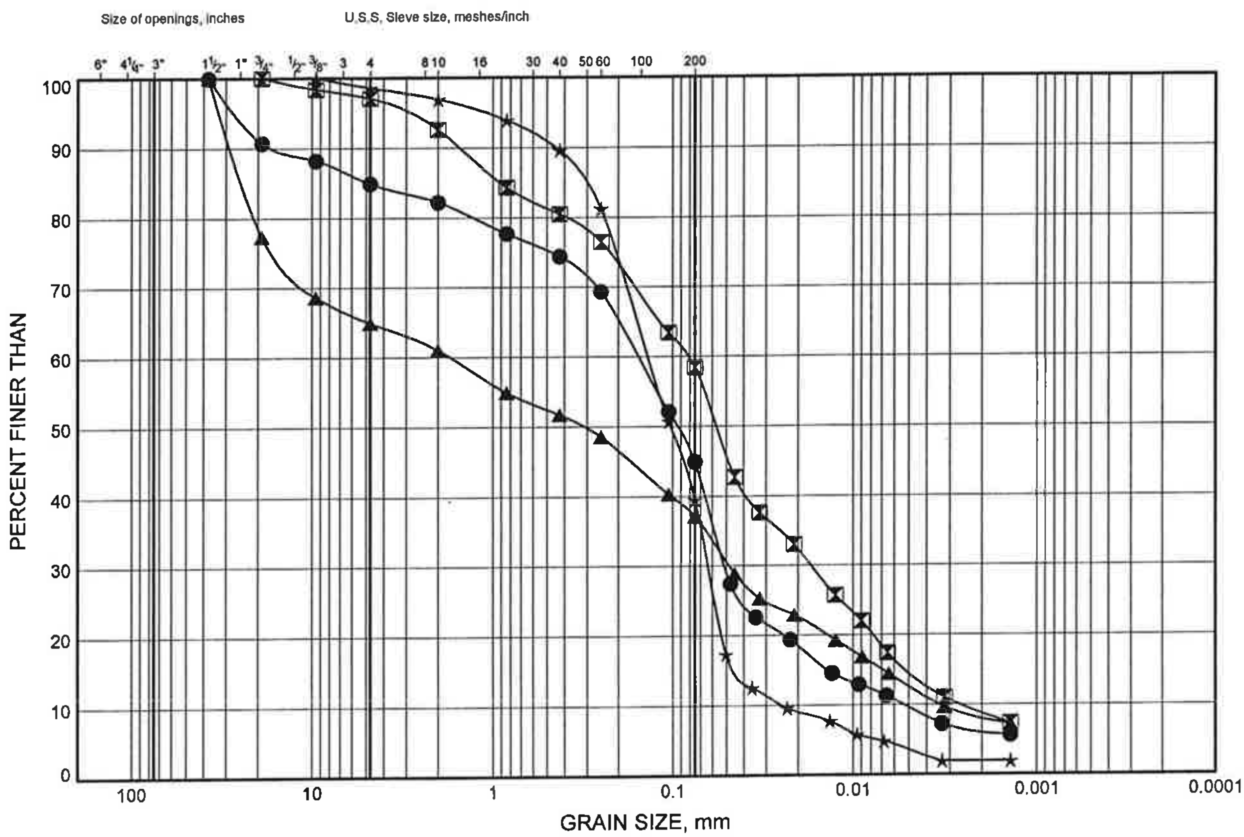
Chkd. RA

ALTR 1-00-0350 HWY 410 VALLEYWOOD BLVD.GPJ 20/09/05

GRAIN SIZE DISTRIBUTION

FIGURE B3

Sand and Silt Till



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

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	VB2	12.3	246.9
⊠	VB3	18.5	240.7
▲	VB4	13.9	245.0
★	VB6	13.9	243.9

Date September 2005

Project 108-00-01

Prep'd DB

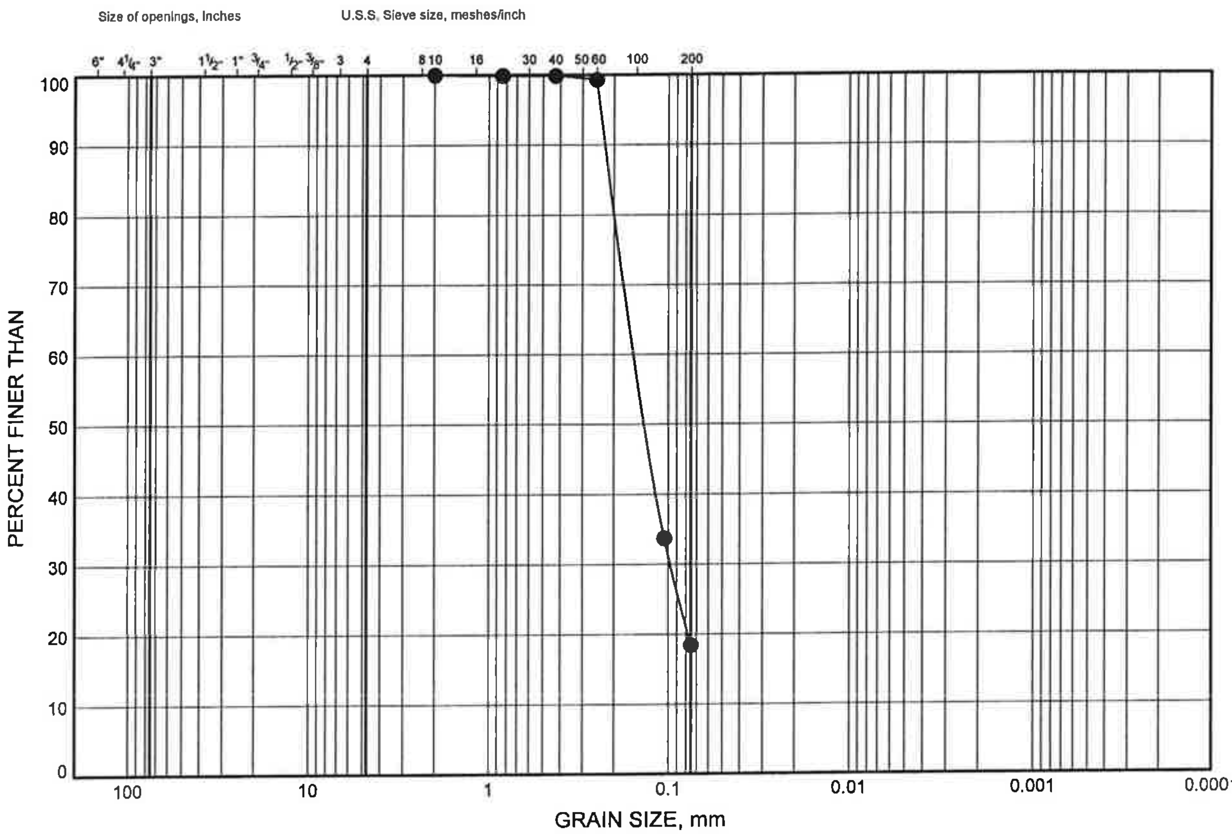
Chkd. RA

GSD 1-00-0350 HWY 410 VALLEYWOOD BLVD.GPJ 20/09/05

GRAIN SIZE DISTRIBUTION

FIGURE B4

Silty Fine Sand



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

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	VB2	18.5	240.7

Date September 2005

Project 108-00-01

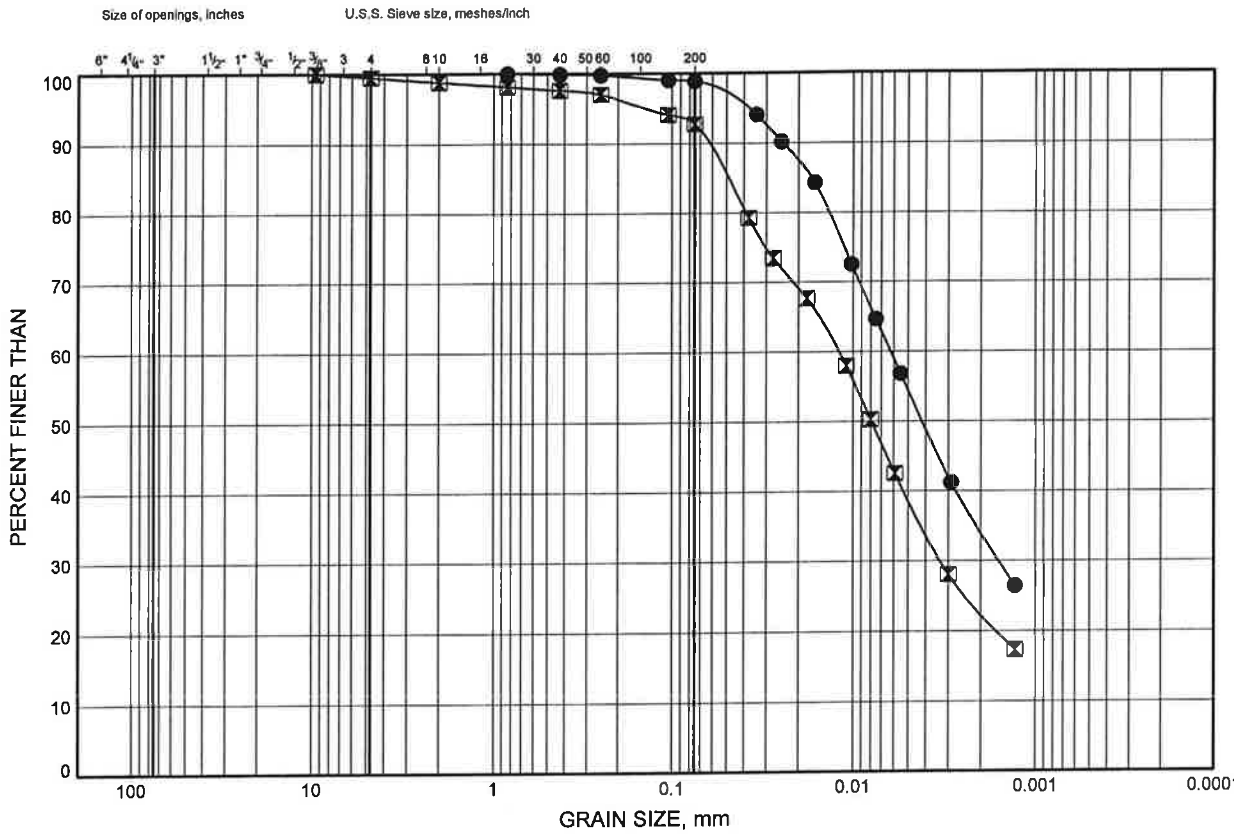
Prep'd DB

Chkd. RA

GRAIN SIZE DISTRIBUTION

FIGURE B5

Clayey Silt



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

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	VB3	24.6	234.6
⊠	VB4	23.1	235.7

Date September 2005

Project 108-00-01

Prep'd DB

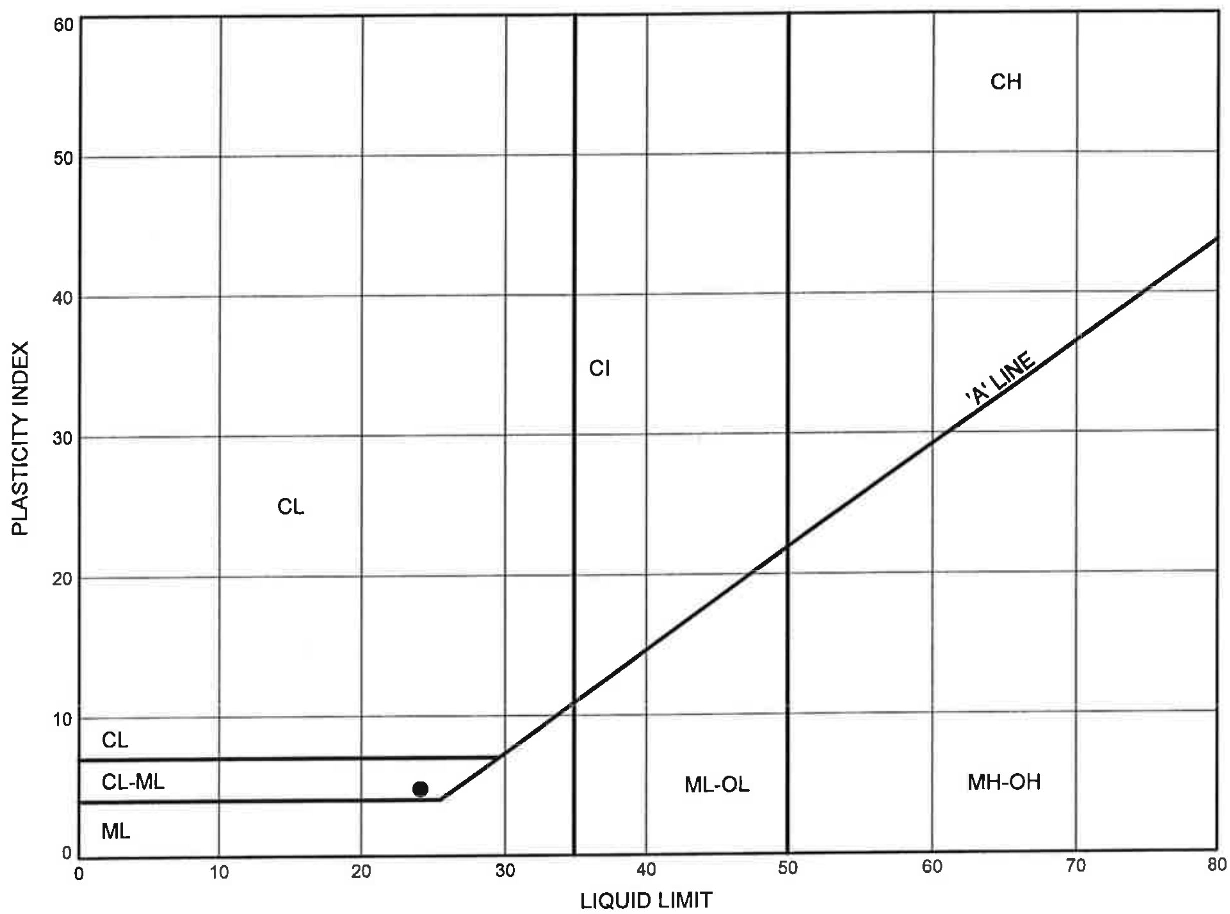
Chkd. RA

GSD 1-00-0350 HWY 410 VALLEYWOOD BLVD.GPJ 23/01/07

ATTERBERG LIMITS TEST RESULTS

FIGURE B6

Clayey Silt



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	VB3	24.6	234.6

Date September 2005

Project 108-00-01

Prep'd DB

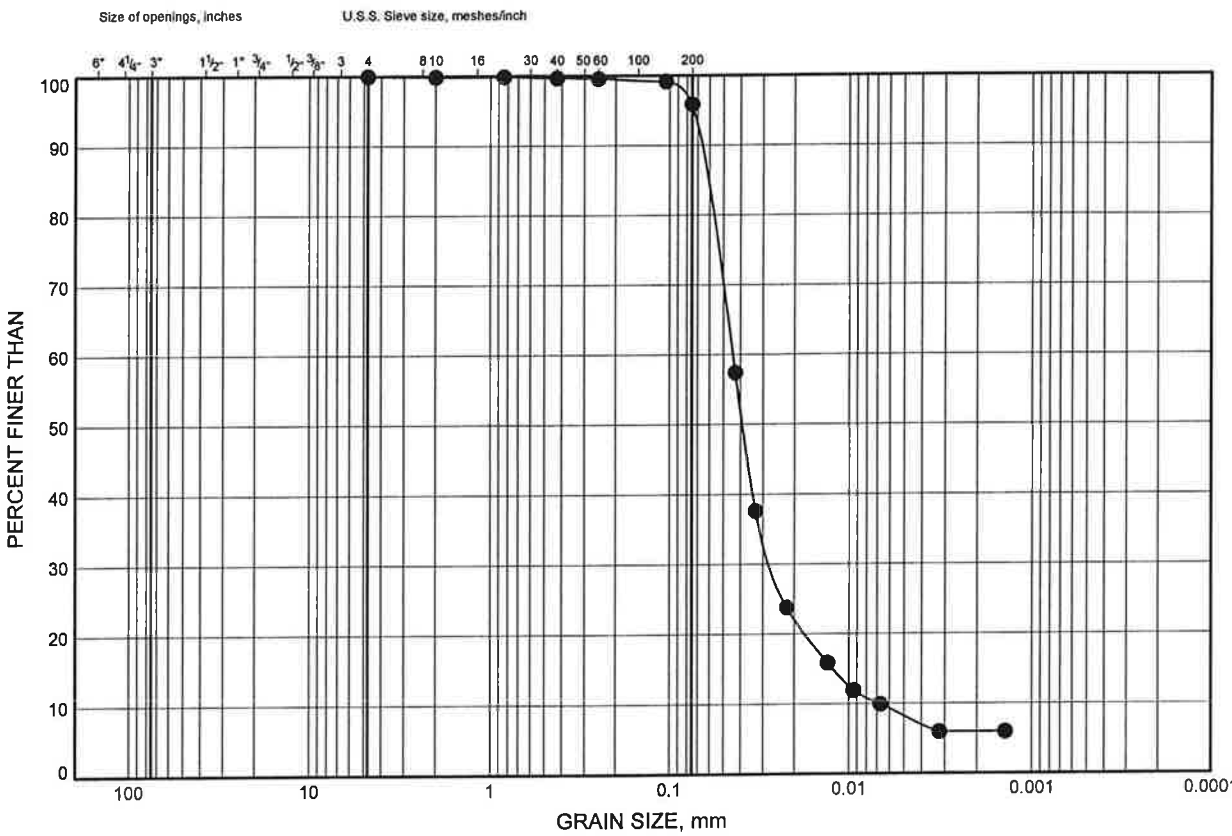
Chkd. RA

ALTR 1-00-0350 HWY 410 VALLEYWOOD BLVD.GPJ 20/09/05

GRAIN SIZE DISTRIBUTION

FIGURE B7

Silt Till



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

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	VB5	26.1	232.0

Date September 2005

Project 108-00-01

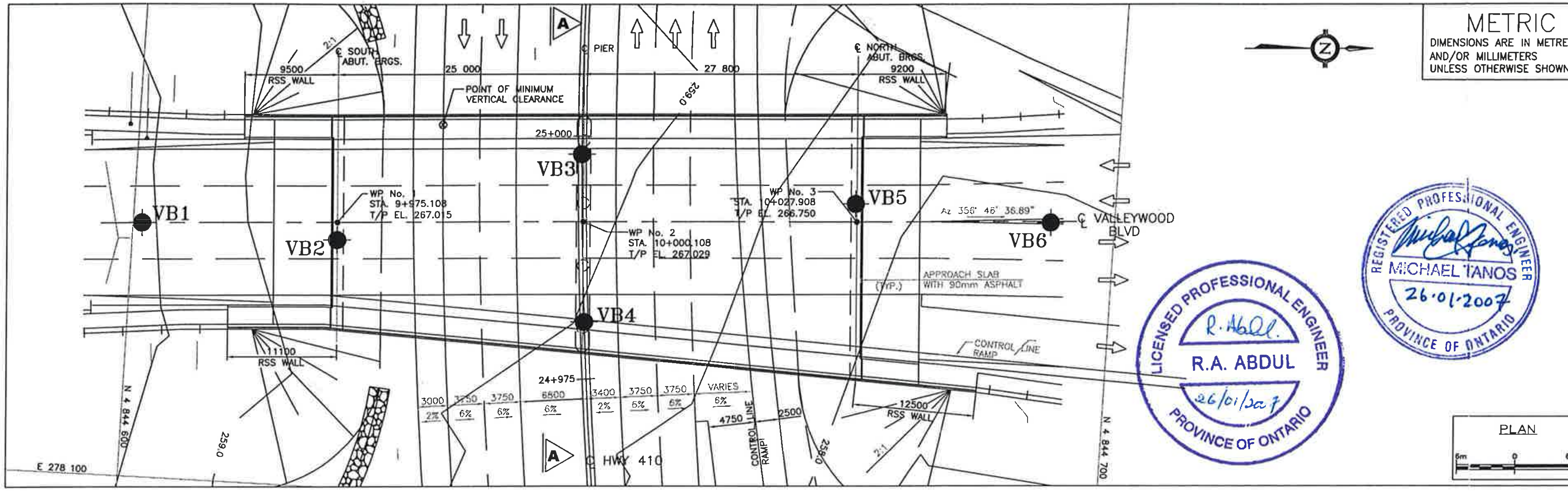
Prep'd DB

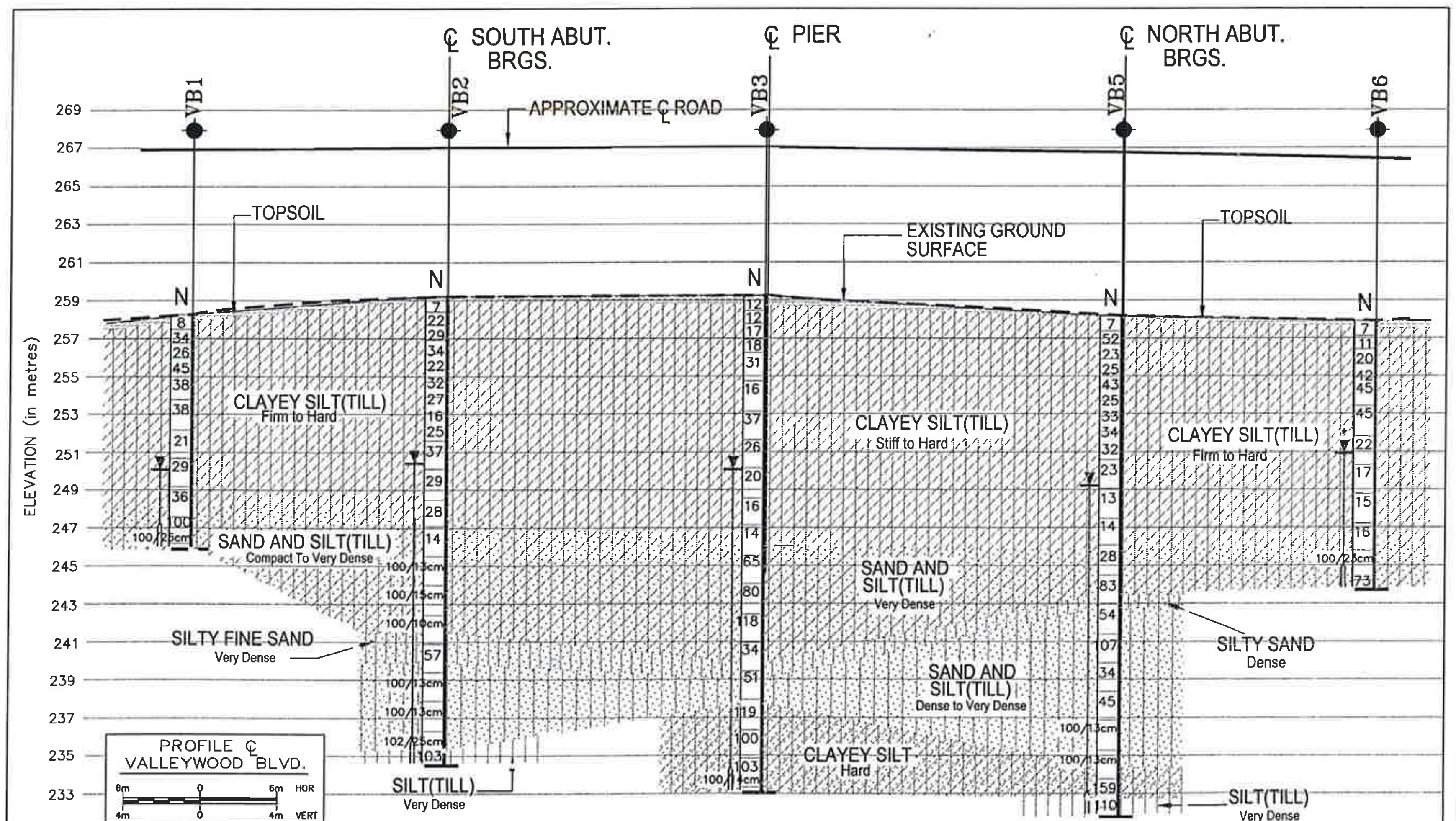
Chkd. RA

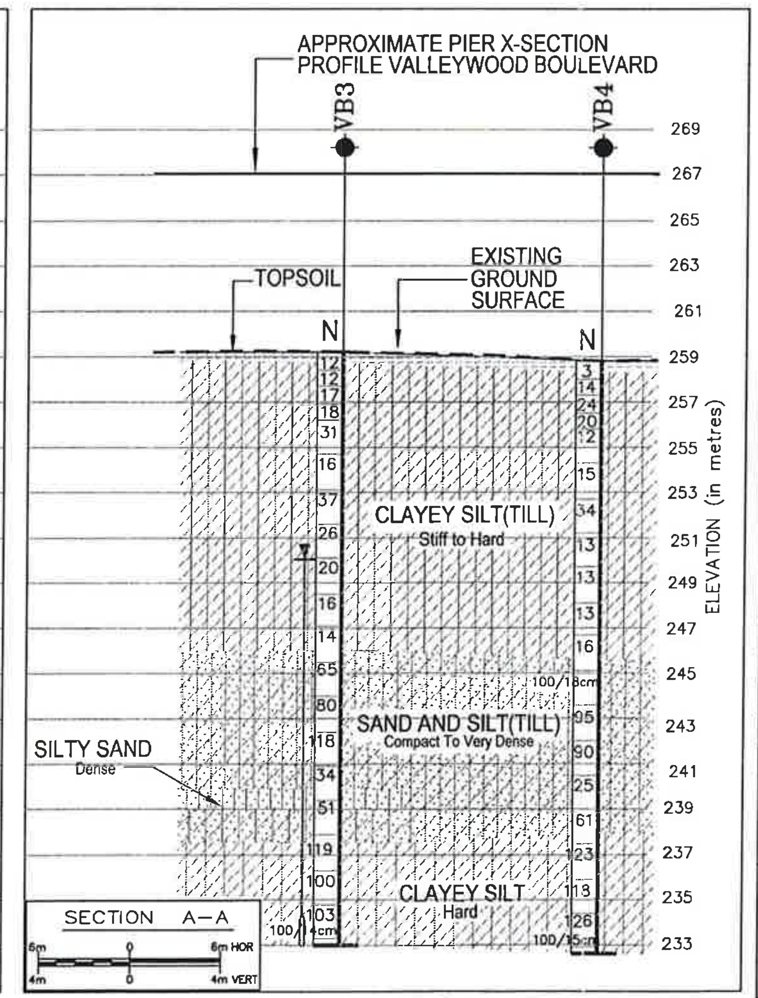
APPENDIX C

Drawing titled
"Borehole Locations and Soil Strata"

Terraprobe Limited







LICENSED PROFESSIONAL ENGINEER R.A. ABDUL 26/01/2007 PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER MICHAEL TANOS 26.01.2007 PROVINCE OF ONTARIO

DRAWING NOT TO BE SCALED 100mm ON ORIGINAL DRAWING

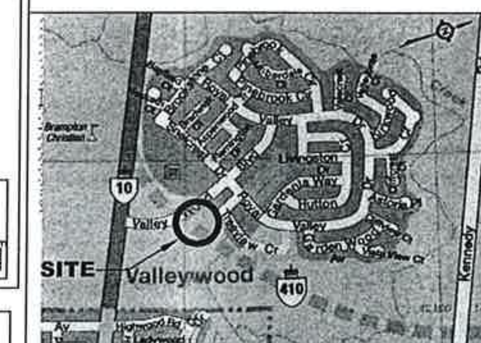
CONT. W.P. 108–00–01

HIGHWAY 410 – PHASE III VALLEYWOOD BOULEVARD – UNDERPASS
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET 364

Giffels An Ingenium Group Company

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



KEY PLAN

LEGEND

	Bore Hole
	Dynamic Cone Penetration Test
	Bore Hole And 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 2005–09 * 2005–04
	Piezometer
90%	Rock Quality Designation
A/R	Auger Refusal

No	ELEVATION	COORDINATES NORTHING	COORDINATES EASTING
VB1	258.3	4844600.9	278073.8
VB2	259.2	4844620.9	278074.5
VB3	259.2	4844645.2	278064.4
VB4	258.8	4844646.5	278081.6
VB5	258.1	4844673.3	278067.9
VB6	257.8	4844693.5	278068.7

NOTE

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore holes the boundaries are assumed from geological evidence.

REVISIONS

DATE	BY	DESCRIPTION

DESIGN R.A	CODE CHBDC2000	LOAD	DATE SEPT.2005		
DRAWN P.S	CHK R.A	SITE: 24–745	STRUCT	DWG 2	

APPENDIX D

Foundation Comparison

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COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

Foundation Element	Driven Piles	Augered Caissons	Footing on Native Soil	Footing on Engineered Fill
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. Higher unit cost compared to other footing options such as driven piles. ii. Precludes consideration on an integral abutment structure. iii. Relatively high construction effort required to install caissons compared to driven piles	***Advantages:*** i. Less costly compared to driven piles. ***Disadvantages:*** i. Precludes consideration of an integral abutment structure. ii. Relatively long abutment stems required at both abutments.	***Advantages:*** i. Length of abutment stems can be reduced. ***Disadvantages:*** i. Precludes consideration of an integral abutment structure.
Pier	***Advantages:*** i. High geotechnical resistances available. ii. 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. Higher unit cost compared to other footing options such as driven piles. ii. Relatively high construction effort required to install caissons compared to driven piles.	***Advantages:*** i. Less costly compared to driven piles. ***Disadvantages:*** i. Low geotechnical resistance will result in an uneconomically large footing size. ii. Potential for unacceptable settlements and differential settlement.	***Advantages:*** N/A ***Disadvantages:*** i. Requires a relatively deep excavation (high construction effort) in order to construct an engineered fill pad.

APPENDIX E

Figures

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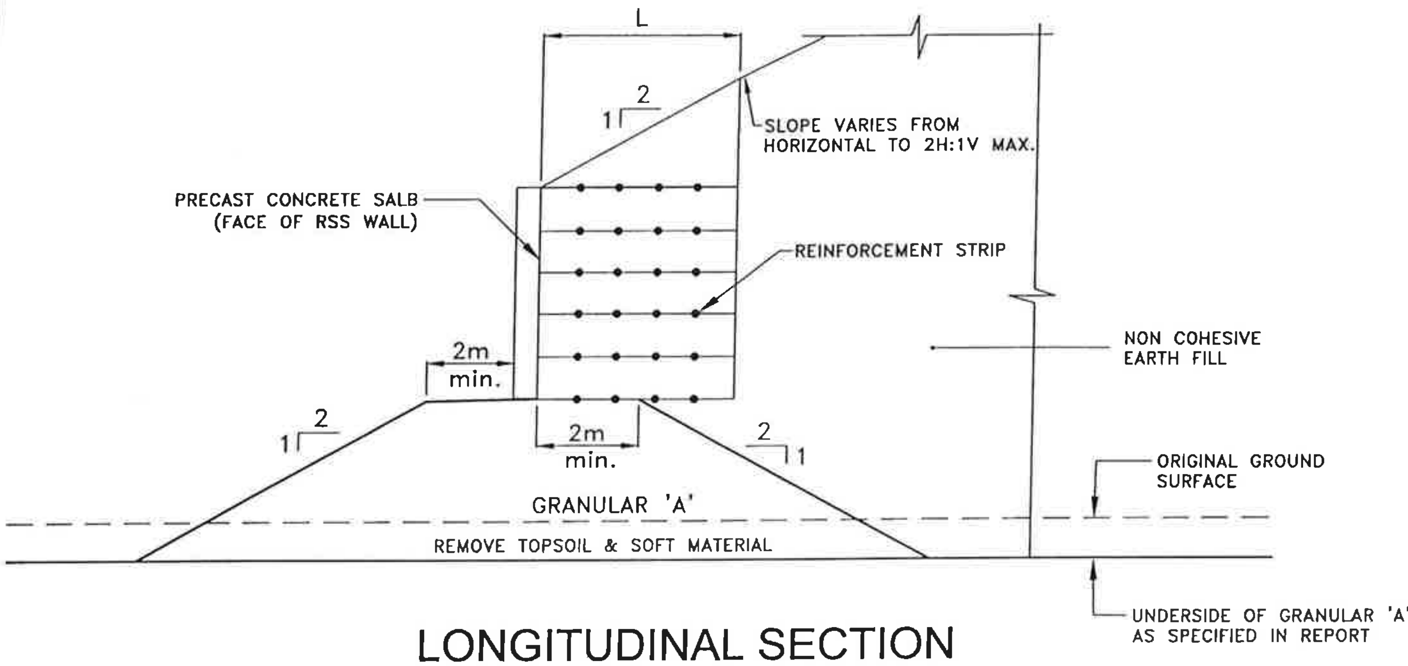

LONGITUDINAL SECTION

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

TERRAPROBE

File No. 1-00-0350

FIGURE E1

APPENDIX F

Suggested NSSP Wording

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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 operations. The soil conditions are described in the Foundation Investigation Report prepared for this site".

If a pile encounters refusal on cobbles and 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.