



# Terraprobe

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

**Geocres No:  
30M12-319**

**FOUNDATION INVESTIGATION & DESIGN REPORT  
HEART LAKE ROAD 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. 104-00-01, SITE: 24-742**

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## TABLE OF CONTENTS

### Part 1

1	INTRODUCTION .....	1
2	SITE DESCRIPTION .....	1
3	SITE INVESTIGATION AND FIELD TESTING .....	2
4	LABORATORY TESTING .....	3
5	DESCRIPTION OF SUBSURFACE CONDITIONS .....	3
5.1	Silty Sand Fill .....	3
5.2	Clayey Silt Fill .....	4
5.3	Clayey Silt Till .....	4
5.4	Sand (Possible Till) .....	5
5.5	Water Levels .....	5

### Part 2

6	GENERAL .....	7
7	STRUCTURE FOUNDATIONS .....	7
7.1	Spread Footings .....	8
7.2	Augered Caissons (Drilled Shafts) .....	8
7.2.1	Axial Resistance (Drilled Shafts) .....	9
7.3	Prebored H-Piles .....	9
7.3.1	Axial Resistance .....	10
7.3.2	Downdrag .....	10
7.3.3	Integral Abutment Considerations .....	11
7.3.4	Lateral Resistance .....	11
7.3.5	Pile Tips .....	13
7.3.6	Pile Installation .....	13
7.3.7	Pile Driving .....	13
7.4	Recommended Foundation .....	13
7.5	Frost Cover .....	13
8	EXCAVATION AND BACKFILL .....	13
8.1	General .....	13
8.2	Foundations .....	14
9	GROUNDWATER CONTROL .....	14
10	APPROACH EMBANKMENTS .....	14
11	TEMPORARY CUT SLOPE .....	15



12	RETAINED SOIL SYSTEMS.....	16
12.1	Foundation .....	16
12.2	Global Stability .....	17
13	BACKFILL TO ABUTMENTS .....	18
14	EARTH PRESSURE .....	19
15	SEISMIC CONSIDERATIONS .....	20
15.1	Seismic Design Parameters .....	20
15.2	Liquefaction Potential .....	21
15.3	Retaining Wall Dynamic Earth Pressures .....	21
16	CONSTRUCTION CONCERNS .....	21

### Appendices

Appendix A	Record of Borehole Sheets
Appendix B	Laboratory Test Results
Appendix C	Drawing titled "Borehole Locations and Soil Strata"
Appendix D	Foundation Comparison
Appendix E	Figures
Appendix F	Suggested NSSP wording



**FOUNDATION INVESTIGATION AND DESIGN REPORT**  
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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 Heart Lake Road 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. (Giffels), 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 on Heart Lake Road about 420± m north of the Heart Lake Road/Mayfield Road intersection in the Town of Caledon. Heart Lake Road is a two lane asphalt paved road with granular shoulders and ditches on both sides. At the site the topography is flat and vegetation is light consisting mainly of grass and occasional large trees. North of the Mayfield Road intersection the profile grade of Heart Lake Road rises gradually by about 6.5 ± m over a horizontal distance of 420 m becoming relatively level further north of the bridge site.



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<sup>1</sup>. 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 on March 07 and 08, 2005 and consisted of drilling and sampling six boreholes to depths ranging from 6.3 m to 10.8 m. The boreholes were numbered HLR1, HLR2, HLR5, HLR6, HLR7 and HLR8 and their approximate locations are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix C.

The borehole locations, coordinates and geodetic elevations were established in the field by surveyors from Shiu Geomatics Limited based on drawings provided by Giffels. Utility clearances were obtained by Terraprobe 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 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.

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 in sand 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
HLR1	6.4/262.1	Piezometer with 1.5 m slotted screen installed with filter sand to 4.6 m, bentonite seal from 4.6 m to 3.7 m, drill cuttings from 3.7 m to 0.9 m and bentonite seal from 0.9 m to ground surface.

<sup>1</sup> Chapman and Putnam, "The Physiography of South Ontario", 3<sup>rd</sup> Edition, 1984.



**Table 3.1 – Piezometer Installation Details (Cont'd)**

HLR2	10.7/257.8	Piezometer with 1.5 m slotted screen installed with filter sand to 8.5 m, bentonite seal from 8.5 m to 7.6 m, drill cuttings from 7.6 m to 0.9 m and bentonite seal from 0.9 m to ground surface.
HLR6	10.7/258.0	Piezometer with 1.5 m slotted screen installed with filter sand to 9.1 m, bentonite seal from 9.1 m to 8.2 m, drill cuttings from 8.2 m to 0.9 m and bentonite seal from 0.9 m to ground surface.
HLR7	9.1/260.0	Piezometer with 1.5 m slotted screen installed with filter sand to 7.6 m, bentonite seal from 7.6 m to 6.7 m, drill cuttings from 6.7 m to 0.9 m and bentonite seal from 0.9 m to ground surface.

A member of Terraprobe's technical staff supervised the drilling and sampling operations on a full time basis. The supervisor 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 fill and overburden deposits of stiff to hard clayey silt till and very dense sand.

##### **5.1 Silty Sand Fill**

Silty sand fill was encountered in some of the boreholes at the site. This fill is approximately 0.6 m to 0.7 m thick and extends to elevations ranging from 267.8 m to 268.6 m.



The grain size distribution curve of a sample of this fill material is illustrated in Figure B1. The results show a grain size distribution consisting of 14% gravel, 56% sand, 22% silt and 8% clay size particles.

Standard Penetration tests in this fill material yielded 'N' values ranging from 35 to 50 blows for less than 0.3 m penetration. Since the drilling was conducted in relatively cold weather (March) it is our opinion that these relatively high 'N' values may signify frozen ground conditions.

The moisture content of samples from this deposit ranged from 7% to 23% by weight.

## 5.2 Clayey Silt Fill

Clayey silt fill was encountered in two boreholes at depths extending to 0.7 m (Elev. 268.1 m) and 1.4 m (Elev. 267.1 m) below ground surface.

Refer to Figure B2 for the grain size distribution curve of a sample of this fill material. The results show a grain size distribution consisting of 0% gravel, 29% sand, 48% silt and 23% clay size particles.

Standard Penetration tests in this fill material yielded 'N' values of 11 and 45 blows for less than 0.3 m penetration. The higher 'N' value (45 blows) was recorded in a surficial layer of this material probably signifying a frozen condition. Based on an 'N' value of 11 blows for 0.3 m penetration the clayey silt fill is considered to have a stiff consistency.

The moisture content of samples from this deposit ranged from 19% to 21% by weight.

## 5.3 Clayey Silt Till

Across the site a major deposit of clayey silt till was encountered. In the foundation boreholes this deposit was fully penetrated and it extends to depths ranging from 7.0 m to 8.4 m below ground surface or to elevations ranging from 262.1 m to 260.1 m. The approach boreholes were terminated in this deposit at depths of 6.3 m (Elev. 263.0 m) and 6.6 m (Elev. 261.9 m) below ground surface.

The grain size distribution curves of tested samples of this clayey silt till are presented in Figure B3. These results show a grain size distribution consisting of 0-8% gravel, 32-40% sand, 36-54% silt and 14-18% clay size particles. Till soils are also known to contain cobbles and boulders due to their mode of deposition.

Three samples were also subjected to Atterberg Limits tests and the results are presented in Figure B4. The index values from these tests are summarized below:

Liquid Limit:	20-21%
Plastic Limit:	6-7%
Plasticity Index:	13-15%
Natural Moisture Content:	9-11%



These values are characteristic of clayey soils of low plasticity.

Standard Penetration tests in this clayey silt till layer yielded 'N' values ranging from 10 to more than 100 blows for 0.3 m penetration indicating a stiff to hard consistency.

The moisture content of samples from this deposit ranged from 6% to 19% by weight and the bulk unit weight of samples ranged between 20 kN/m<sup>3</sup> and 23.1 kN/m<sup>3</sup>.

#### 5.4 Sand (Possible Till)

Across the site the foundation boreholes encountered a layer of sand with some silt, trace gravel and trace clay. This layer extends to borehole termination depths of 9.2 m and 10.8 m below ground surface or to elevations ranging from 259.9 m to 257.7 m.

The results of grain size distribution tests conducted on a sample from this stratum are illustrated in Figure B5. The results show a grain size distribution consisting of 1% gravel, 81% sand, 13% silt and 5% clay size particles. Cobbles and boulders can also be expected in till soils.

Standard Penetration tests in this stratum gave 'N' values more than 100 blows for 0.3 m penetration indicating a very dense relative density.

The moisture content of samples from this stratum ranged from 1% to 6% by weight.

#### 5.5 Water Levels

A standpipe piezometer was installed in Boreholes HLR1, HLR2, HLR6 and HLR7. 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)
HLR1	April 18, 2005	Dry	-
	Sept. 09, 2005	Dry	-
HLR2	April 18, 2005	Dry	-
	Sept. 09, 2005	Dry	-
HLR6	April 18, 2005	Dry	-
	Sept. 09, 2005	10.8*	257.9
HLR7	April 18, 2005	Dry	-
	Sept. 09, 2005	Dry	-

\* Wet at Base (no free water in standpipe)





These observations suggest that the local groundwater level at the site is likely to exist below the depths of investigation i.e. below Elev. 257.9 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 Heart Lake Road over the proposed four lanes of Highway 410. The span length between the abutments and pier will be  $32.7\pm$  m and the structure will be about  $13.7\pm$  m wide. RSS walls are proposed for the abutment (false abutment) and wing walls.

The proposed finished grades at the structure will be about Elev.  $272.6\pm$  m at the north abutment and Elev.  $272.2\pm$  m at the south abutment. Assuming an integral abutment concept, the approach fill heights will be about  $7\pm$  m and  $9\pm$  m immediately adjacent to the north and south abutments respectively.

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 surficial layers of silty sand and clayey silt fill material underlain by stiff to hard clayey silt till and very dense sand. The groundwater level was not encountered within the depths of investigation and is believed to exist below Elev. 257.9 m.

Consideration was given to the following foundation types:

- Spread footings
- Augered Caissons (drilled shafts)
- Prebored H-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 Heart Lake Road, spread footings are feasible at the pier and abutment locations.

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 & Pier Locations**

Borehole Location	Existing Ground Surface Elev. (m)	Recommended Bottom of Footing Level Below Existing Ground Surface (m)	Footing Elevation (m)	*Factored Geotech. Resistance at ULS (kPa)	*Geotech. Resistance at SLS (kPa)	Subgrade Material
North Abutment HLR 7	269.1	1.5 – 3.0	267.6 – 266.1	375	250	Clayey Silt Till
		3.0 – 7.0	266.1 – 262.1	525	350	Clayey Silt Till
		Below 7.0	Below 262.1	750	525	Sand (Poss. Till)
South Abutment HLR 2	268.5	1.5 – 3.0	267.0 – 265.5	375	250	Clayey Silt Till
		3.0 – 8.4	265.5 – 260.1	525	350	Clayey Silt Till
		Below 8.4	Below 260.1	750	525	Sand (Poss. Till)
Pier HLR 5	268.8	1.5 – 3.0	267.3 – 265.8	375	250	Clayey Silt Till
		3.0 – 7.3	265.8 – 261.5	525	350	Clayey Silt Till
		Below 7.3	Below 261.5	750	525	Sand (Poss. Till)
Pier HLR 6	268.7	0.7 – 2.3	268.0 – 266.4	375	250	Clayey Silt Till
		2.3 – 7.3	266.4 – 261.4	525	350	Clayey Silt Till
		Below 7.3	Below 261.4	750	525	Sand (Poss. Till)

\* Assumes that the distance between the footing edge and the face of the slope is not less than twice the footing width.

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 the subgrade soils should be evaluated in accordance with the CHBDC, 2000. Assume an ultimate coefficient of friction of 0.6 for the very stiff to hard clayey silt till and an ultimate coefficient of friction of 0.7 for the very dense sand.

## 7.2 Augered Caissons (Drilled Shafts)

The abutments may be supported by augered caissons (drilled shafts) designed to bear on the very dense sand encountered across the site.

It is recommended that the caissons be constructed by using temporary liners to support the sidewalls and to allow hand cleaning and inspection of the bearing surface.



A minimum caisson diameter of 900 mm, and as governed by applicable regulations, is required to allow down-the-hole hand cleaning and inspection.

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

#### **7.2.1 Axial Resistance (Drilled Shafts)**

Axial geotechnical resistances of 2700 kPa ULS and 1800 kPa SLS are recommended for a 900 mm diameter caisson founded on the very dense sand encountered at Elev. 262.1± m (north abutment) and Elev. 260.1±m (south abutment).

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 10 mm, a significant portion of which will be complete by the end of construction.

#### **7.3 Prebored H-Piles**

It is recognised that an integral abutment bridge (which requires pile foundations) offers significant long term advantages. Consideration was therefore given to the use of a prebored H-pile arrangement for supporting the structure.

The site in its present condition is not suitable for driven piles but an integral abutment structure may be designed at this site if the abutment foundations are prepared as follows.

- Excavate the original ground to the base of the RSS. Pre-auger to Elev. 262.7 m and Elev. 260.3 m at the north and south abutments respectively with a minimum auger diameter of about 0.76 m. Depth of preaugered hole must be at least 3 m below the base of the RSS. Use a steel casing to effectively seal any water ingress and prevent any cave-in of overburden soils into the drilled hole.
- Properly clean the augered hole, inspect and approve base.
- Place the steel H-pile in the augered hole, centre and drive the pile to effective refusal. The tip need not be reinforced and if desired a lighter section such as HP 310x79 can be used rather than HP 310x110.
- Backfill the hole around the H-pile with concrete to a minimum depth of 3 m i.e. to the base of the RSS. Concrete must be placed by the tremie method.
- Proceed with normal integral abutment construction by installing the CSP's and filling with sand meeting MTO's grading requirements.



### 7.3.1 Axial Resistance

It is recommended that the base of the prebored holes for the H-piles be founded on the hard clayey silt till encountered at Elev. 262.7 m and Elev. 260.3 m at the north and south abutments respectively. For a 0.76 m diameter prebored H-pile arrangement axial geotechnical resistances of 1500 kPa ULS and 1000 kPa SLS are recommended.

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 10 mm, a significant portion of which will be complete by the end of construction.

The steel H-piles will be driven from the bottom of the preaugered holes and are likely to encounter effective refusal in the underlying very dense sand layer at the estimated tip elevations given in Table 7.2 below. The actual pile tip elevations will be controlled as described in Section 7.3.7 Pile Installation.

**Table 7.2 – Estimated Pile Tip Elevations**

Location	Reference Borehole	Estimated Pile Tip Elevation (m)	Founding Stratum
North Abutment	HLR7	260.5±	Sand (Poss. Till)
South Abutment	HLR2	258.0±	Sand (Poss. Till)

The structural resistance of the prebored H-pile arrangement combination should be checked by the structural designer.

Based on the proposed construction sequence the piles will be driven a relatively small distance before reaching effective refusal and the possibility of damage to the H-section will be significantly reduced. Therefore the pile sections need not be reinforced.

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 Downdrag

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



### 7.3.3 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 3.5 m to 5.5 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 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 and the concrete placed in the preaugered hole, 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 7.3.

**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%

### 7.3.4 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:

$$\begin{aligned}
 k_s &= n_h z / D \text{ [cohesionless soils]} & (\text{kN/m}^3) \\
 k_s &= 67 S_u / D \text{ [cohesive soils]} & (\text{kN/m}^3) \\
 p_{ult} &= 3 \gamma z K_p \text{ [cohesionless soils]} & (\text{kPa}) \\
 p_{ult} &= 9 S_u \text{ [cohesive soils]} & (\text{kPa})
 \end{aligned}$$

where

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

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 100 kN (ULS) is recommended for HP 310x110 pile sections.



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<sup>3</sup>),  $D$  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 (m)	Soil Type	Bulk Unit Weight (kN/m <sup>3</sup> )	Angle of Internal Friction ( $\phi$ ) Degrees	Undrained Shear Strength ( $S_u$ ) (kPa)	** Recommended $n_h$ Value (kN/m <sup>3</sup> )
North Abutment HLR7	269.2 – 265.7	Granular Fill	21	32	-	6000
	265.7 – 262.1	Clayey Silt Till	21	0	200	-
	262.1 – 259.9	Sand	21	35	-	12000
South Abutment HLR2	268.8 – 263.3	Granular Fill	21	32	-	6000
	263.3 – 260.1	Clayey Silt Till	21	0	200	-
	260.1 – 257.7	Sand	21	35	-	12000

\* Assumes that the native soils will be excavated to Elev. 265.7 m and Elev. 263.3 m at the north and south abutments respectively and the piles will lie within granular fill comprising the RSS Core.

\*\* Values estimated based on Table 20.3 data, Canadian Foundation Engineering Manual, 3<sup>rd</sup> edition, 1992.

Since the prebored H-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.



### **7.3.5 Pile Tips**

The pile tips are unlikely to be damaged since the piles will be driven a relatively short distance before reaching effective refusal in the very dense sand. Therefore, driving shoes are not considered to be a requirement.

### **7.3.6 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.

### **7.3.7 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 a minimum value of 3,200 kN for an HP 310x110 pile section.

Hiley formula calculations should be carried out from the base of the prebored hole i.e. Elev. 260.1 m at the south abutment and Elev. 262.1 m at the north abutment.

## **7.4 Recommended Foundation**

The use of a prebored H-pile arrangement at the abutments allows for the design of an integral abutment structure. From a geotechnical point of view, it is recommended that this arrangement be used for supporting the abutments. A spread footing is recommended for supporting the pier.

## **7.5 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. Excavations above the water table may be sloped at 1.5H:1V





## **8.2 Foundations**

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

## **9 GROUNDWATER CONTROL**

Since the groundwater level is estimated to be below Elev. 257.9 m it is unlikely that excavations will encounter groundwater at this site. However, provision should be made for controlling any surface water run-off into excavations as well as minor subsurface seepage from any wet sand seams within the overburden.

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. Since the original ground will be excavated at both abutments to facilitate the construction of an integral abutment structure it is envisaged that the embankment heights will be about  $7 \pm$  m and  $9 \pm$  m at the north and south abutments respectively. Further beyond the bridge abutments the embankment fill will be about  $5 \pm$  m high.

The total post construction settlement due to loads imposed by  $7 \pm$  m to  $9 \pm$  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 70 mm to 90 mm for  $7 \pm$  m and  $9 \pm$  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 1.5 and greater were obtained for both long term and short term conditions.

The forward slopes of embankments constructed using granular materials were also analysed. Factors of safety against global failure of 1.5 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 572.

## 11 TEMPORARY CUT SLOPE

It is understood that a temporary detour of Heart Lake Road will be required for approximately 6 months during construction of the structure. The profile grade of the detour within the limits of the structure varies from about Elev. 269.5± m at about Sta. 9+950 to about Elev. 270.5± m at Sta. 10+040. During bridge construction, excavations next to the temporary detour will range from about 4 m to 5 m at the abutment locations and about 7 m at the pier.

Global stability analysis was conducted on temporary cut slopes assuming excavations will be made to Elev. 261± m at the pier location and Elev. 265.5± m and Elev. 263± m at the north and south abutments respectively. Factors of safety against global failure of 1.4 and greater were obtained for both long term and short term conditions for temporary cut slopes inclined at 1.5 H:1 V. It should be noted that a dense sand layer was encountered at this site at elevations ranging from 260.1 m to 262.1 m. Excavations made in this material will require a cut slope inclination of 2 H:1 V.

The following requirements must be met for this open temporary cut:

- No traffic, construction equipment, stockpiles (including snow) or other construction supplies is permitted at the top of the cut slope within a distance of at least 1.5 m from the top of the cut.



- Exposed soil along the slope must be protected from surface erosion using waterproof tarps or plastic sheeting. If ravelling of the slope face becomes an issue, the exposed cut should be faced with shotcrete.
- Construction activities should be scheduled so that the length of time the temporary cut slope is left open is reduced to the extent practical.
- Erosion control measures must be implemented as appropriate such that runoff from the site is reduced to the extent practical.
- Surface water must be diverted away from the excavation and from the top of the slope. This can be accomplished by superelevating this section of the detour alignment so that surface water runoff drains away from the top of the slope. Alternative schemes that can also be considered include installation of a curb and gutter arrangement or paving the shoulder and the drainage swale with hot mix asphalt concrete.
- The general condition of the slope must be inspected weekly by a Geotechnical Engineer to confirm adequate stability.

## 12 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.

### 12.1 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.

The levelling pad for the RSS wall may be formed directly on the native hard clayey silt till or 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 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		
NE & NW Wall	265.5 for 5.5 m	7.1	185	300	200	HLR7 Clayey Silt Till	
	268.5 for 6.0 m	4.1	107	225	150	HLR7 Clayey Silt Till	Subexcavate to Elev. 267.5 m, replace with 1 m of compacted Gran. A to base of wall.
North Abutment Face	265.5	7.1	185	300	200	HLR7 Clayey Silt Till	
SE & SW Wall	263.1 for 7.5 m	9.1	237	375	250	HLR2 Clayey Silt Till	
	266.0 for 6.0 m	6.2	161	300	200	HLR2 Clayey Silt Till	
	268.5 for 6.0 m	3.7	96	225	150	HRL2 Fill	Subexcavate to Elev. 267 m, replace with 1.5 m of compacted Gran. A to base of wall.
South Abutment Face	263.1	9.1	237	375	250	HLR2 Clayey Silt Till	

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 friction of sliding resistance of cast in-situ concrete levelling pad on Granular A = 0.7; hard clayey silt till = 0.60
- 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.

## 12.2 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.



RSS walls are likely to be used for the abutment and its wing walls. It is envisaged that the RSS will be founded either on native clayey silt till or on about 1 m to 1.5 m of earth fill.

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,  $\phi$ , of 30°, cohesion of 0, and unit weight,  $\gamma$ , of 19 kN/m<sup>3</sup>).
- Fill behind the RSS is horizontal.

Analysis carried out on RSS walls located at the base of the embankment yielded a factor of safety greater than 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 90% of the wall height are required to achieve an acceptable factor of safety of 1.5.

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.

### 13 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.

#### 14 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)

$\gamma$  = 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 14.1.



**Table 14.1 – Earth Pressure Coefficients**

Wall Condition	Earth Pressure Coefficient (K)					
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ$ ; $\gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I $\phi = 32^\circ$ ; $\gamma = 21.2 \text{ kN/m}^3$		Rock Fill $\phi = 42^\circ$ ; $\gamma = 19.0 \text{ kN/m}^3$	
	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.

## 15 SEISMIC CONSIDERATIONS

### 15.1 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.



## 15.2 Liquefaction Potential

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

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.

## 15.3 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  $0.5 \phi$ . For the design of retaining walls, the coefficients of horizontal earth pressure in Table 15.1 may be used:

**Table 15.1 – Earth Pressure Coefficient for Earthquake Loading**

Earth Pressure Coefficient (K) for Earthquake Loading						
Wall Condition	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ$ ; $\delta = 17.5^\circ$ $\gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I $\phi = 32^\circ$ ; $\delta = 16^\circ$ $\gamma = 21.2 \text{ kN/m}^3$		Rock Fill $\phi = 42^\circ$ ; $\delta = 21^\circ$ $\gamma = 19.0 \text{ kN/m}^3$	
	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

## 16 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 being encountered during preaugering which may require extending the excavation deeper.
- the possibility of piles encountering boulders when driving.
- the nature and geotechnical properties of non-cohesive earth fill used in the approach fills.





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# APPENDICES

**Terraprobe Limited**



## 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:**

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

## 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
$\sigma$	kPa	TOTAL NORMAL STRESS
$\sigma'$	kPa	EFFECTIVE NORMAL STRESS
$\tau$	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
$\epsilon$	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
$\mu$	1	COEFFICIENT OF FRICTION

### MECHANICAL PROPERTIES OF SOIL

$m_v$	kPa <sup>-1</sup>	COEFFICIENT OF VOLUME CHANGE
$C_c$	1	COMPRESSION INDEX
$C_s$	1	SWELLING INDEX
$C_{\alpha}$	1	RATE OF SECONDARY CONSOLIDATION
$C_v$	m <sup>2</sup> /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
$T_v$	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
$\sigma'_{vo}$	kPa	EFFECTIVE OVERBURDEN PRESSURE
$\sigma'_p$	kPa	PRECONSOLIDATION PRESSURE
$\tau_f$	kPa	SHEAR STRENGTH
$c'$	kPa	EFFECTIVE COHESION INTERCEPT
$\phi'$	- °	EFFECTIVE ANGLE OF INTERNAL FRICTION
$c_u$	kPa	APPARENT COHESION INTERCEPT
$\phi_u$	- °	APPARENT ANGLE OF INTERNAL FRICTION
$\tau_R$	kPa	RESIDUAL SHEAR STRENGTH
$\tau_r$	kPa	REMOULDED SHEAR STRENGTH
$S_t$	1	SENSITIVITY = $c_u / \tau_r$

## PHYSICAL PROPERTIES OF SOIL

$\rho_s$	kg/m <sup>3</sup>	DENSITY OF SOLID PARTICLES	$e$	1, %	VOID RATIO	$e_{max}$	1, %	VOID RATIO IN DENSEST STATE
$\gamma_s$	kN/m <sup>3</sup>	UNIT WEIGHT OF SOLID PARTICLES	$n$	1, %	POROSITY	$I_D$	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
$\rho_w$	kg/m <sup>3</sup>	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
$\gamma_w$	kN/m <sup>3</sup>	UNIT WEIGHT OF WATER	$S_r$	%	DEGREE OF SATURATION	$D_n$	mm	n PERCENT - DIAMETER
$\rho$	kg/m <sup>3</sup>	DENSITY OF SOIL	$w_L$	%	LIQUID LIMIT	$C_u$	1	UNIFORMITY COEFFICIENT
$\gamma$	kN/m <sup>3</sup>	UNIT WEIGHT OF SOIL	$w_p$	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
$\rho_d$	kg/m <sup>3</sup>	DENSITY OF DRY SOIL	$w_s$	%	SHRINKAGE LIMIT	q	m <sup>2</sup> /s	RATE OF DISCHARGE
$\gamma_d$	kN/m <sup>3</sup>	UNIT WEIGHT OF DRY SOIL	$I_p$	%	PLASTICITY INDEX = $(w_L - w_p)$	v	m/s	DISCHARGE VELOCITY
$\rho_{sat}$	kg/m <sup>3</sup>	DENSITY OF SATURATED SOIL	$I_L$	1	LIQUIDITY INDEX = $(w - w_p)/I_p$	i	1	HYDRAULIC GRADIENT
$\gamma_{sat}$	kN/m <sup>3</sup>	UNIT WEIGHT OF SATURATED SOIL	$I_c$	1	CONSISTENCY INDEX = $(w_L - w)/I_p$	k	m/s	HYDRAULIC CONDUCTIVITY
$\rho'$	kg/m <sup>3</sup>	DENSITY OF SUBMERGED SOIL	$e_{max}$	1, %	VOID RATIO IN LOOSEST STATE	j	kN/m <sup>3</sup>	SEEPAGE FORCE
$\gamma'$	kN/m <sup>3</sup>	UNIT WEIGHT OF SUBMERGED SOIL						

## **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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# **APPENDIX A**

## **Record of Borehole Sheets**

**TERRAPROBE LIMITED**



# RECORD OF BOREHOLE No HLR1

1 OF 1

METRIC

W.P. 104-00-01 LOCATION Coords: N:4846173.0 E:280127.7 (Heart Lake Road) ORIGINATED BY MS  
DIST HWY 410 Heart Lake Rd. BOREHOLE TYPE Solid Stem Augers COMPILED BY DB  
DATUM Geodetic DATE 07.03.05 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										
								20 40 60 80 100										
							20 40 60 80 100				10 20 30				GR SA SI CL			
268.5	Ground Surface																	
0.0	FILL - Silty Sand, some gravel, trace clay, brown, frozen		1	SS	35											14 56 22 8		
267.8																		
0.7			CLAYEY SILT - Sandy, some gravel, damp, very stiff to hard, brown  (GLACIAL TILL)	2	SS	17										21.3		
				3	SS	33												
				4	SS	57												
				5	SS	68											21.7	
	6	SS	49															
			7	SS	53													
261.9	End of Borehole																	
6.6	Piezometer installation consists of 19mm dia. Schedule 40 PVC pipe with a 1.5m slotted screen.  Water Level Readings:  Date      Depth(m)      Elevation(m) 18/04/05      Dry      - 09/09/05      Dry      -																	

# RECORD OF BOREHOLE No HLR2

1 OF 1

METRIC

W.P. 104-00-01 LOCATION Coords: N:4846192.7 E:280121.5 (Heart Lake Road) ORIGINATED BY MS  
DIST HWY 410 Heart Lake Rd BOREHOLE TYPE Solid Stem Augers COMPILED BY DB  
DATUM Geodetic DATE 07.03.05 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
268.5	Ground Surface													
0.0	FILL - Silty Sand, some gravel, trace clay, brown, frozen		1	SS	50/ 15cm		268							
267.9														
0.6	FILL - Clayey Silt, Sandy, damp, stiff, brown		2	SS	11		267							0 29 48 23
267.1														
1.4	CLAYEY SILT - Sandy, trace gravel, occasional sandy silt partings, damp, hard, brown (GLACIAL TILL)		3	SS	34		266							6 38 39 17
			4	SS	42		265							
			5	SS	79		264							
			6	SS	82/ 28cm		263							
			7	SS	100/ 8cm		262							0 32 54 14
			8	SS	100/ 0cm		261							
260.1							260							
8.4	SAND some silt, trace gravel, trace clay, dry, very dense, brown (POSSIBLE TILL)		9	SS	100/ 15cm		259							1 81 13 5
257.7							258							
10.8	End of Borehole		10	SS	100/ 10cm									
	Piezometer installation consists of 19mm dia. Schedule 40 PVC pipe with a 1.5m slotted screen.  Water Level Readings:  Date      Depth(m)      Elevation(m) 18/04/05      Dry      - 09/09/05      Dry      -													

ONTARIO MOT 1-00-0350 HWY 410 HLR.GPJ ONTARIO MOT.GDT 23/01/07

# RECORD OF BOREHOLE No HLR5

1 OF 1

METRIC

W.P. 104-00-01 LOCATION Coords: N:4846216.0 E:280084.5 (Heart Lake Road) ORIGINATED BY MS  
DIST HWY 410 Heart Lake Rd. BOREHOLE TYPE Solid Stem Augers COMPILED BY DB  
DATUM Geodetic DATE 08.03.05 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								20	40	60	80	100					
268.8	Ground Surface																
0.0	FILL - Clayey Silt, Sandy, trace gravel, brown, frozen		1	SS	45												
268.1			2	SS	13												
0.7			3	SS	40												
	CLAYEY SILT - Sandy, trace gravel, occasional silty sand seams and partings, damp, hard, brown  (GLACIAL TILL)		4	SS	46												
			5	SS	100/ 25cm												
			6	SS	99												
			7	SS	100/ 25cm												
			8	SS	100/ 25cm												
			9	SS	100/ 13cm												
261.5			10	SS	100/ 10cm												
7.3	SAND some silt, trace gravel, trace clay, dry, very dense, brown  (POSSIBLE TILL)		11	SS	100/ 15cm												
			12	SS	100/ 3cm												
258.0			13	SS	100/ 8cm												
10.8	End of Borehole																
	* Borehole dry (not stabilized) and hole open to full depth on completion of drilling.																

ONTARIO MOT 1-00-0350 HWY 410 HLR.GPJ ONTARIO MOT.GDT 25/10/05

+ 3 × 3: Numbers refer to  
Sensitivity

○ 3% STRAIN AT FAILURE



# RECORD OF BOREHOLE No HLR6

1 OF 1

METRIC

W.P. 104-00-01 LOCATION Coords: N:4846221.9 E:280092.8 (Heart Lake Road) ORIGINATED BY MS  
DIST HWY 410 Heart Lake Rd. BOREHOLE TYPE Solid Stem Augers COMPILED BY DB  
DATUM Geodetic DATE 08.03.05 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL						
268.7	Ground Surface						20	40	60	80	100				GR SA SI CL	
0.0	FILL - Silty Sand, some gravel, moist, brown		1	AS												
268.0							268									
0.7	CLAYEY SILT some sand, trace gravel, occasional silty sand seams and partings, damp, hard, brown  (GLACIAL TILL)		2	SS	32											
			3	SS	32		267									
			4	SS	72		266							22.0		
			5	SS	50/ 10cm		265									
			6	SS	100/ 8cm		264									
			7	SS	100/ 8cm		263							23.1		
			8	SS	100/ 13cm		262									
			9	SS	100/ 15cm		261									
261.4			10	SS	100/ 8cm		260									
7.3	SAND some silt, trace gravel, trace clay, damp, very dense, brown  (POSSIBLE TILL)		11	SS	100/ 10cm		259									
			12	SS	100/ 13cm		258									
257.9			13	SS	100/ 8cm											
10.8	End of Borehole															
	Piezometer installation consists of 19mm dia. Schedule 40 PVC pipe with a 1.5m slotted screen.  Water Level Readings:  Date      Depth(m)      Elevation(m) 18/04/05      Dry      * 09/05/05      wet at base      257.9															

ONTARIO MOT 1-00-0350 HWY 410 HLR.GPJ ONTARIO MOT.GDT 23/01/07

RECORD OF BOREHOLE No HLR7

1 OF 1

METRIC

W.P. 104-00-01 LOCATION Coords: N:4846238.8 E:280062.3 (Heart Lake Road) ORIGINATED BY MS  
DIST HWY 410 Heart Lake Rd. BOREHOLE TYPE Solid Stem Augers COMPILED BY DB  
DATUM Geodetic DATE 07.03.05 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			UNIT WEIGHT  γ  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)				
								○ UNCONFINED   + FIELD VANE	● QUICK TRIAXIAL   × LAB VANE	W <sub>p</sub>	W	W <sub>L</sub>		
269.1	Ground Surface						20   40   60   80   100						GR   SA   SI   CL	
0.0	FILL - Silty Sand, trace to some gravel, moist, brown, frozen		1	SS	50/ 15cm		269							
268.5														
0.6			2	SS	10		268							
			3	SS	36		267							
	CLAYEY SILT - Sandy, trace gravel, damp, hard, brown  (GLACIAL TILL)	stiff	4	SS	48		266						22.0	
			5	SS	75		265						22.3	
			6	SS	100/ 15cm		264							
			7	SS	100/ 20cm		263							4   37   41   18
262.1							262							
7.0	SAND some silt, trace gravel, trace clay, dry to damp, very dense, brown  (POSSIBLE TILL)		8	SS	101/ 25cm		261							
							260							
259.9	End of Borehole		9	SS	100/ 8cm									
9.2	Piezometer installation consists of 19mm dia. Schedule 40 PVC pipe with a 1.5m slotted screen.  Water Level Readings:  Date      Depth(m)      Elevation(m) 16/04/05      Dry      - 09/09/05      Dry      -													


ONTARIO MOT 1-00-0350 HWY 410 HLR.GPJ ONTARIO MOT.GDT 23/01/07

# RECORD OF BOREHOLE No HLR8

1 OF 1

METRIC

W.P. 104-00-01 LOCATION Coords: N:4846257.0 E:280053.9 (Heart Lake Road) ORIGINATED BY MS  
DIST HWY 410 Heart Lake Rd. BOREHOLE TYPE Solid Stem Augers COMPILED BY DB  
DATUM Geodetic DATE 08.03.05 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE									
269.3	Ground Surface						20	40	60	80	100						
0.0	FILL - Silty Sand, trace to some gravel, moist, frozen, brown		1	SS	37												
268.6																	
0.7			CLAYEY SILT trace to some sand, trace gravel, damp, very stiff to hard, brown  (GLACIAL TILL)	2	SS	20											
				3	SS	42											
				4	SS	54											
				5	SS	51											
				6	SS	100/ 15cm											
263.0	End of Borehole		7	SS	57/ 15cm												
6.3	* Borehole dry (not stabilized) and hole open to full depth on completion of drilling.																

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

# **APPENDIX B**

## **Laboratory Test Results**

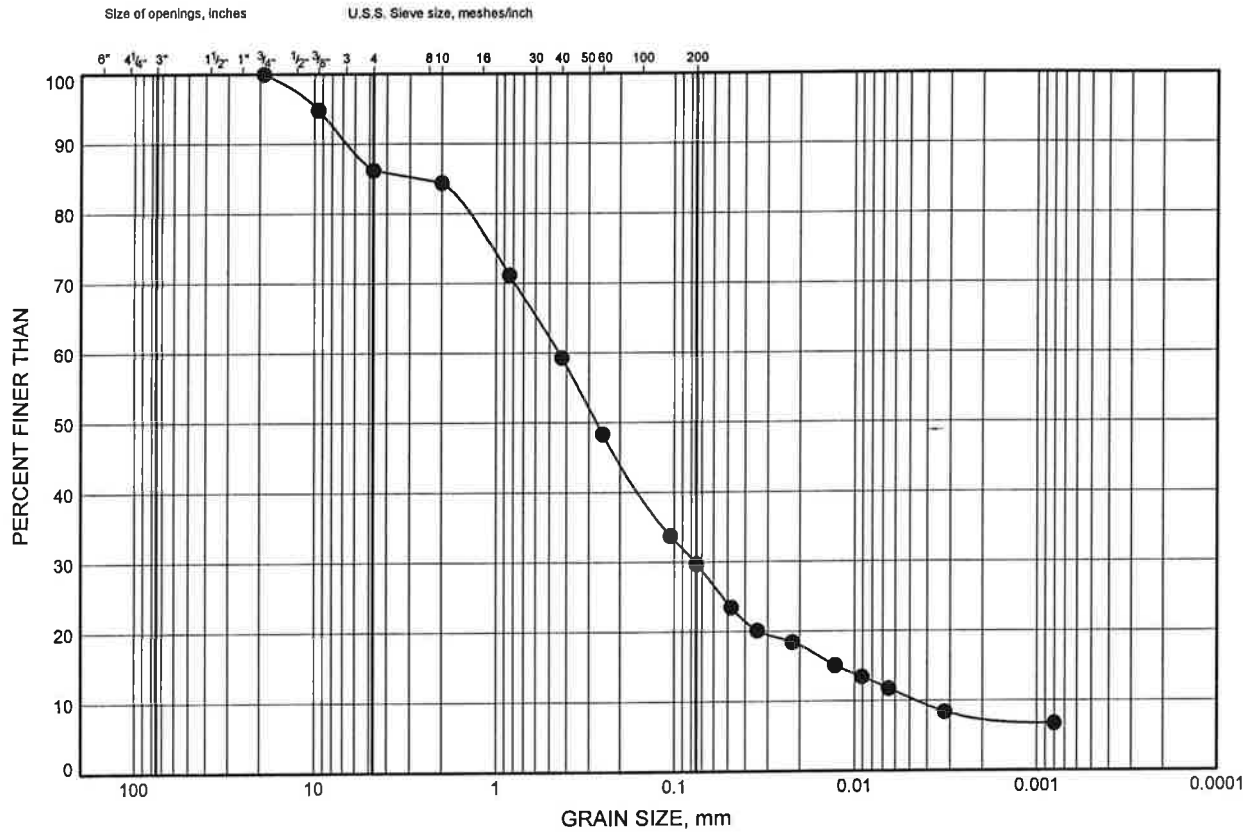
**Terraprobe Limited**



# GRAIN SIZE DISTRIBUTION

FIGURE B1

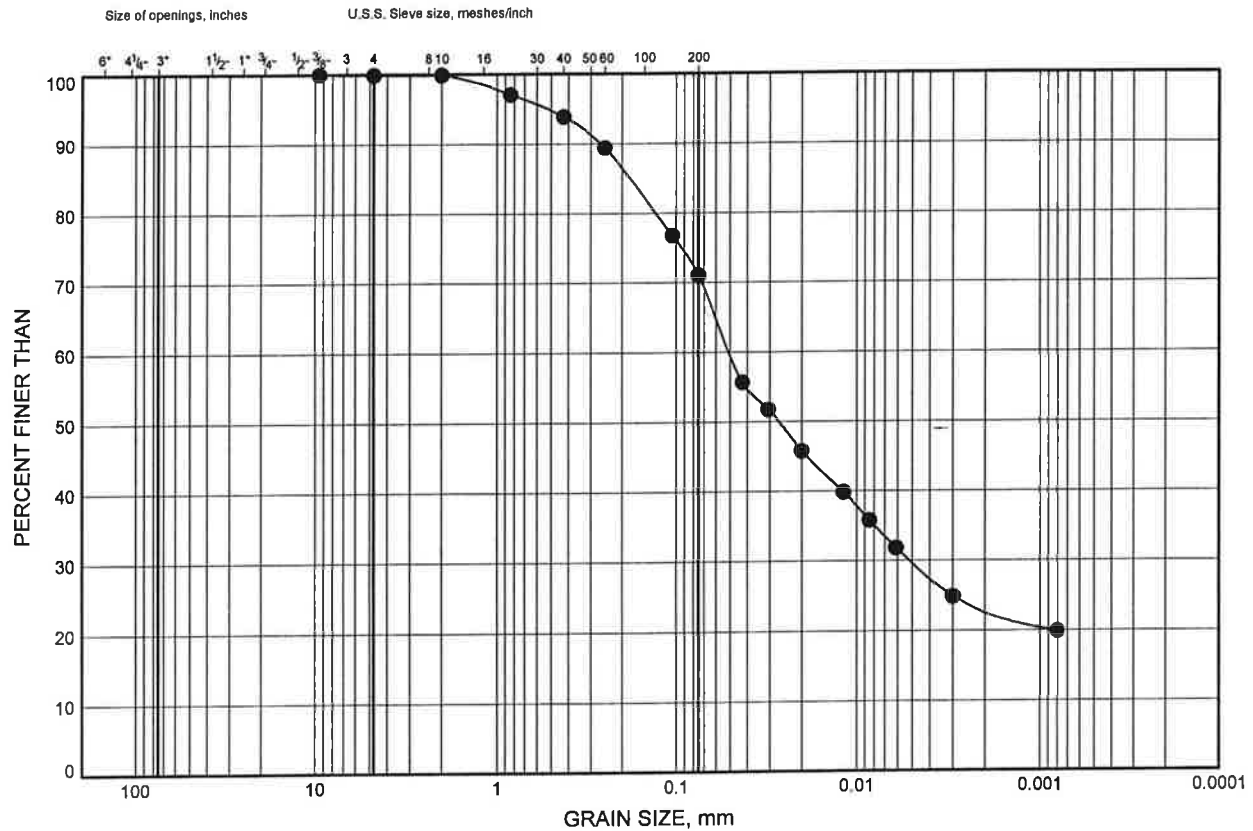
## Silty Sand (Fill)



# GRAIN SIZE DISTRIBUTION

FIGURE B2

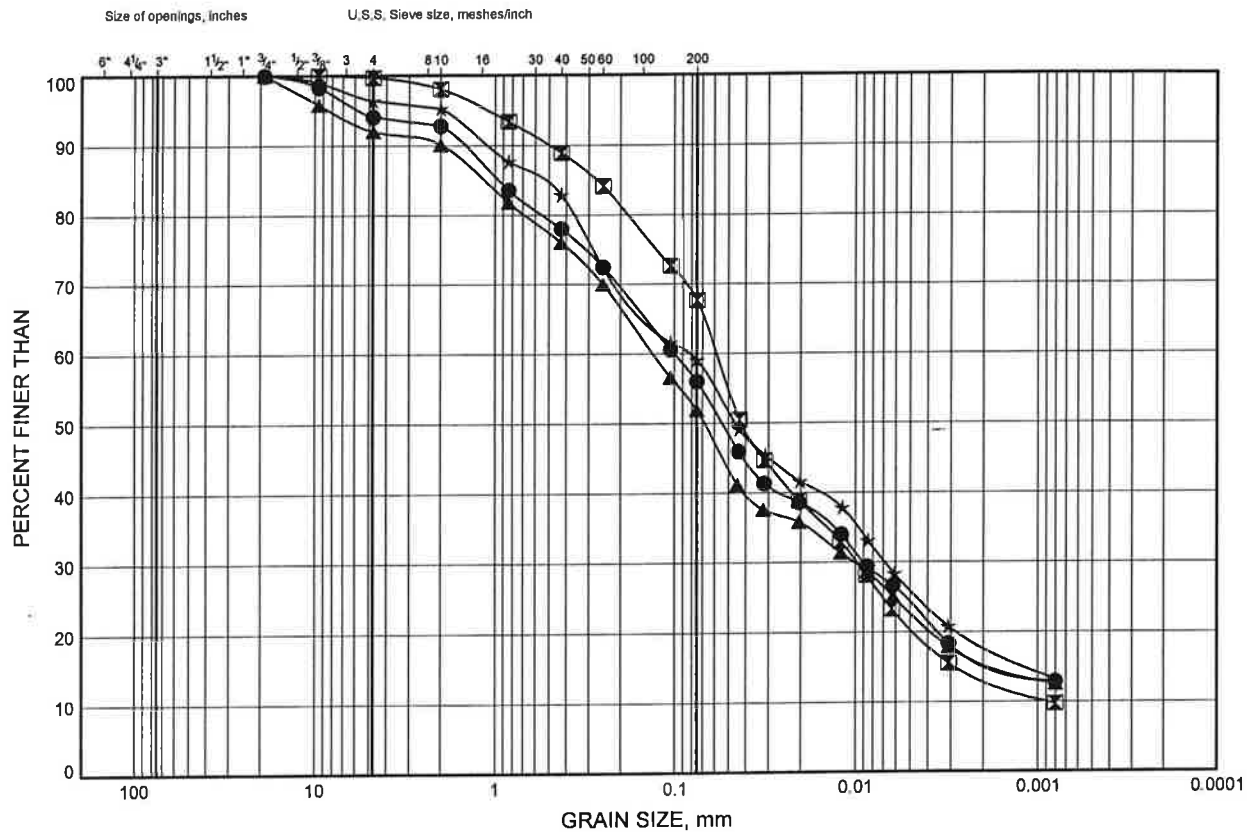
## Clayey Silt (Fill)



# GRAIN SIZE DISTRIBUTION

FIGURE B3

## Clayey Silt Till



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

SYMBOL    BOREHOLE    DEPTH (m)    ELEVATION (m)

●	HLR2	2.5	266.6
☒	HLR2	6.3	262.2
▲	HLR5	4.7	264.1
★	HLR7	6.2	262.9

Date October 2005  
Project 104-00-01

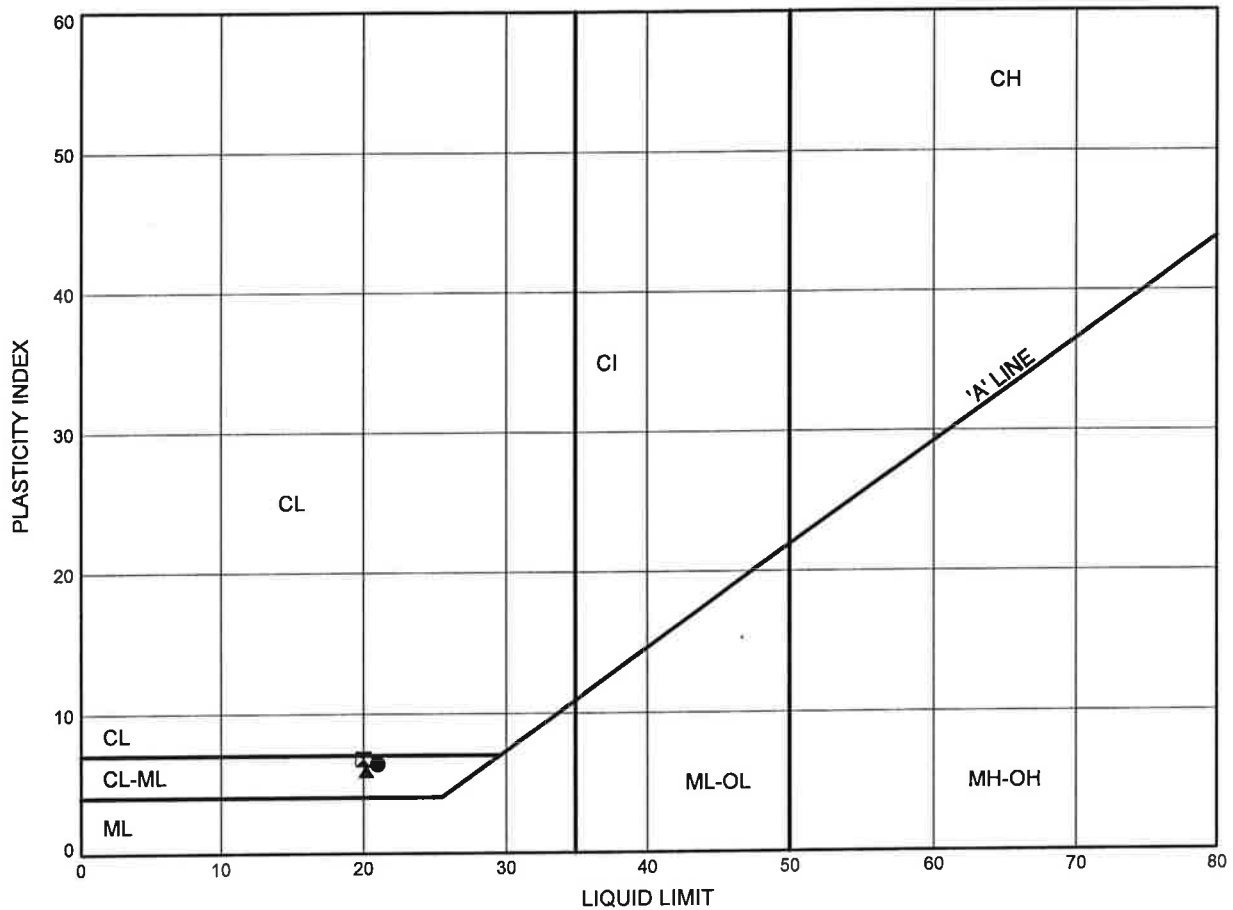


Prep'd DB  
Chkd. RA

# 

FIGURE B4

### 



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	HLR2	2.5	266.0
⊠	HLR5	4.7	264.1
▲	HLR7	6.2	262.9

Date October 2005  
 Project 104-00-01



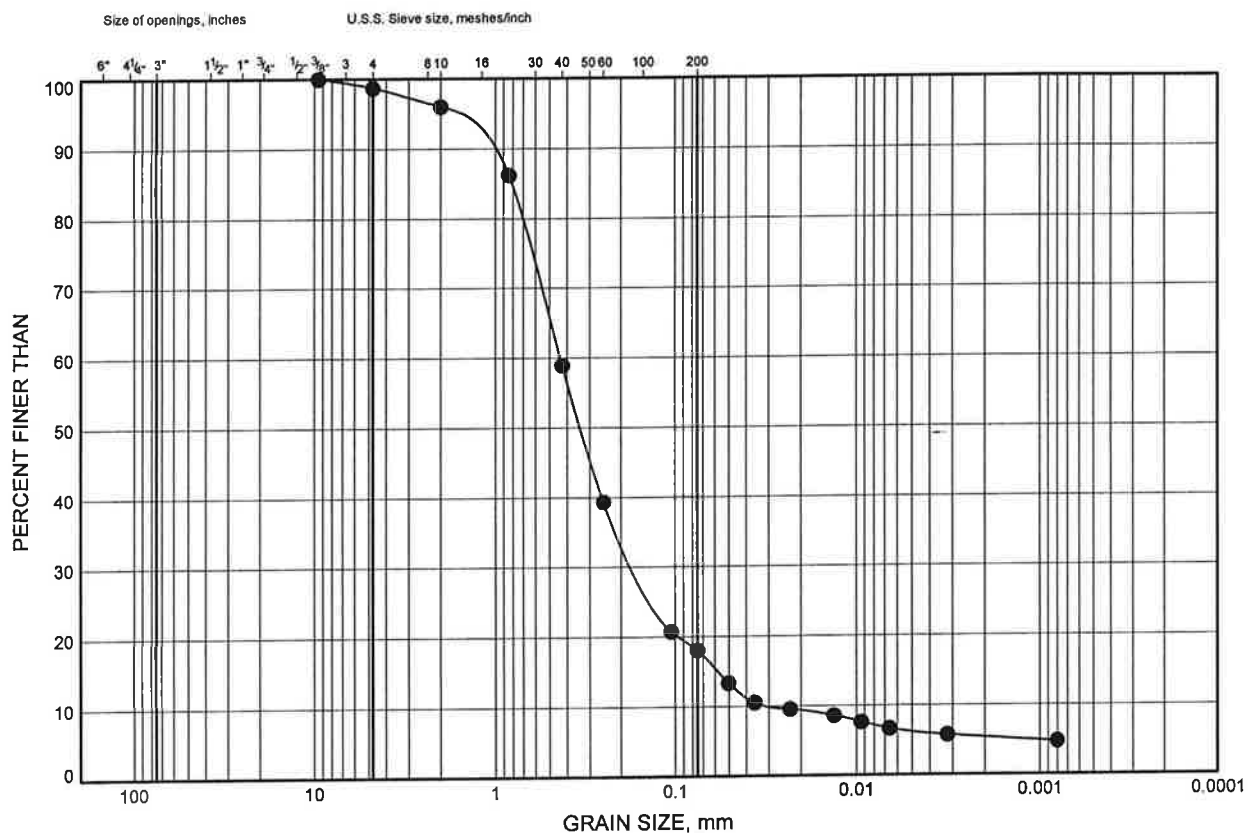
Prep'd DB  
 Chkd. RA



# GRAIN SIZE DISTRIBUTION

FIGURE B5

## Sand (Possible Till)



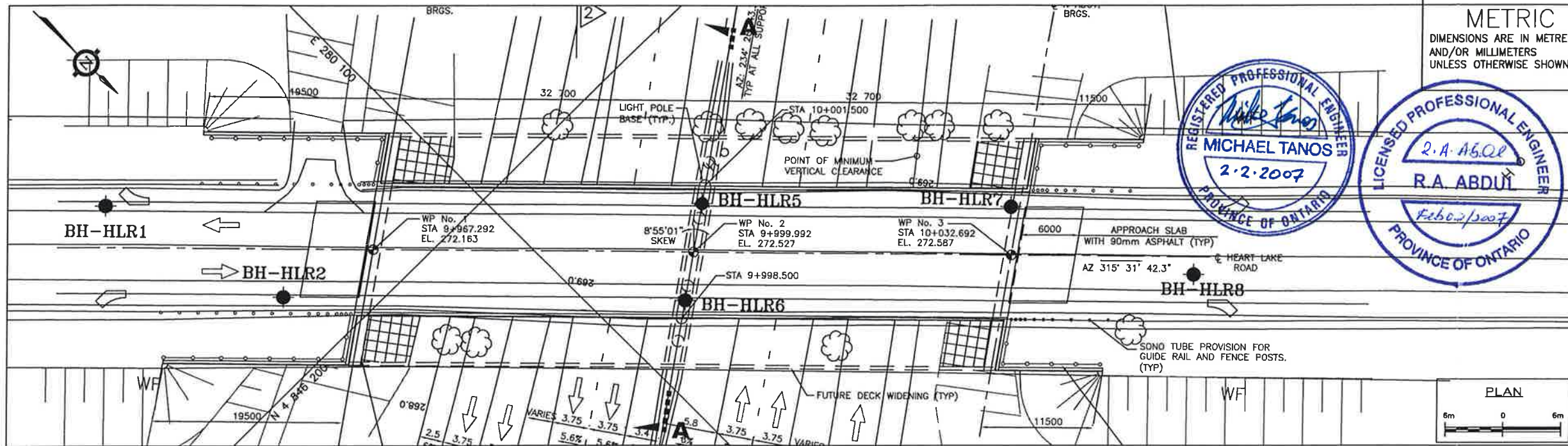
# **APPENDIX C**

**Drawing titled  
“Borehole Locations and Soil Strata”**

**Terraprobe Limited**







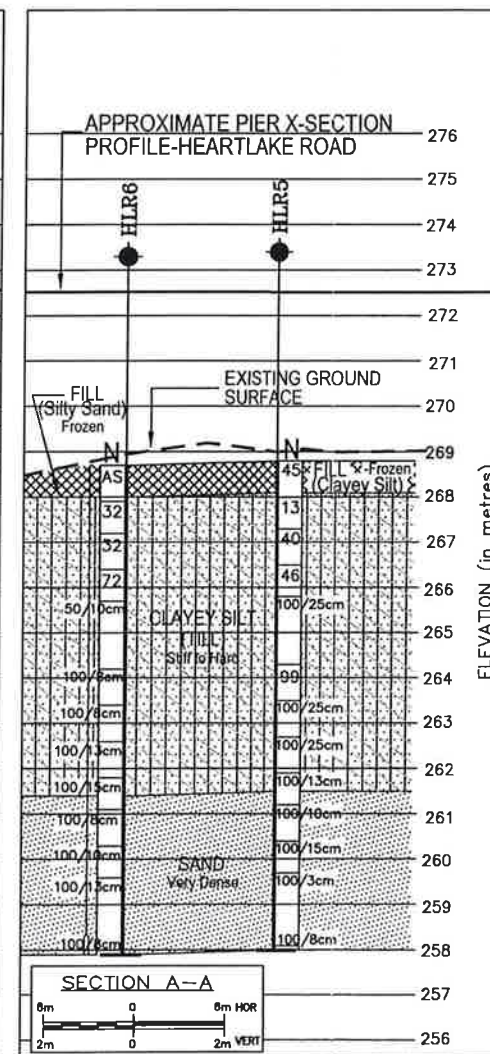
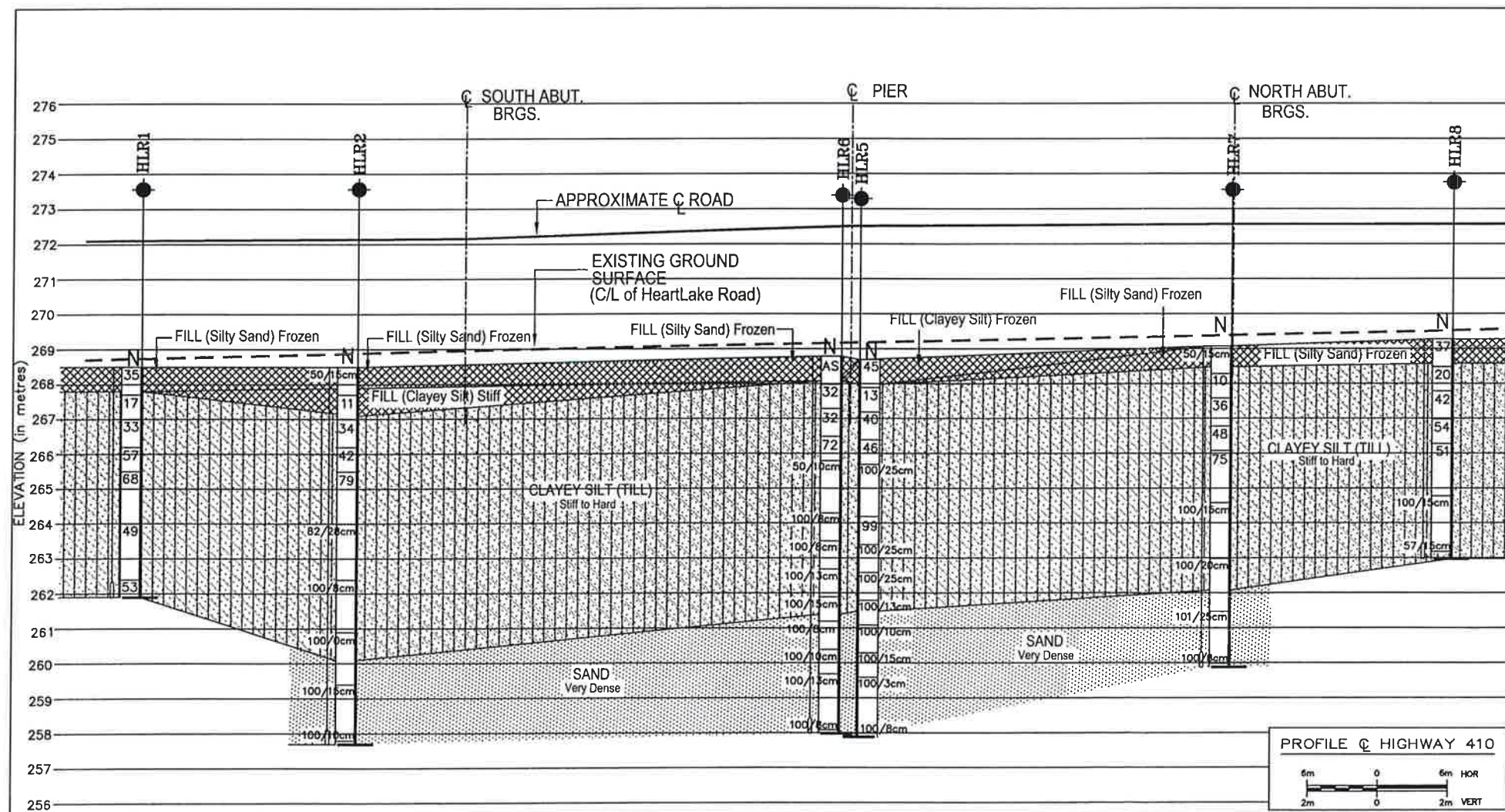
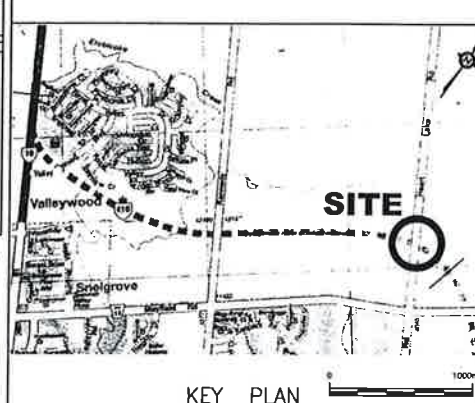
CONT No  
WP No 104-00-01

HIGHWAY 410-PHASE III  
HEARTLAKE ROAD UNDERPASS  
BOREHOLE LOCATIONS  
AND SOIL STRATA

SHEET  
302

**Giffels**  
An Ingenium Group Company

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



- LEGEND
- 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 2005, 09
  - ⬇ Piezometer
  - 90% Rock Quality Designation
  - A/R Auger Refusal

No	ELEVATION	COORDINATES	
		NORTHING	EASTING
HLR1	268.5	4846173.0	280127.7
HLR2	268.5	4846192.7	280121.5
HLR5	268.8	4846216.0	280084.5
HLR6	268.7	4846221.9	280092.8
HLR7	269.1	4846238.8	280062.3
HLR8	269.3	4846257.0	280053.9

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			DESCRIPTION		
	DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
DESIGN	R.A.	CODE	CHBDC2000	LOAD	DATE	OCT.2005
DRAWN	P.S.	CHK	R.A.	SITE	24-742	STRUCT DWG 2

DRAWING NOT TO BE SCALED  
100mm ON ORIGINAL DRAWING



# **APPENDIX D**

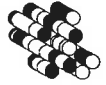
## **Foundation Comparison**

**Terraprobe Limited**



**COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT**

Foundation Element	Prebored H-Piles	Augered Caissons	Footings on Native Soil
North & South Abutments	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. High geotechnical resistances available.</li> <li>ii. Allows choice of integral or semi-integral abutment design.</li> <li>iii. Readily installed</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Higher unit cost compared to footings.</li> <li>ii. Construction concerns related to the possibility of piles being obstructed by a boulder during driving.</li> <li>iii. Requires preboring before pile driving operations.</li> <li>iv. Requires a high quality of construction to ensure minimum disturbance of founding stratum.</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. High geotechnical resistances available.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Higher unit cost compared to other footing options such as spread footings.</li> <li>ii. Precludes consideration of an integral abutment structure.</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. High geotechnical resistances available.</li> <li>ii. Less costly compared to other options such as prebored H-piles.</li> <li>iii. Does not require specialized construction techniques compared to prebored H-piles.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Precludes consideration of an integral abutment structure.</li> </ul>
Pier	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. High geotechnical resistances available.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Higher unit cost compared to spread footings.</li> <li>ii. Construction concerns related to the possibility of piles being obstructed by a boulder during driving.</li> <li>iii. May require preboring due to hard/dense subsurface conditions.</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. High geotechnical resistances available.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Higher unit cost compared to other footing options such as spread footings.</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. High geotechnical resistances available.</li> <li>ii. Less costly compared to other options.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. N/A</li> </ul>

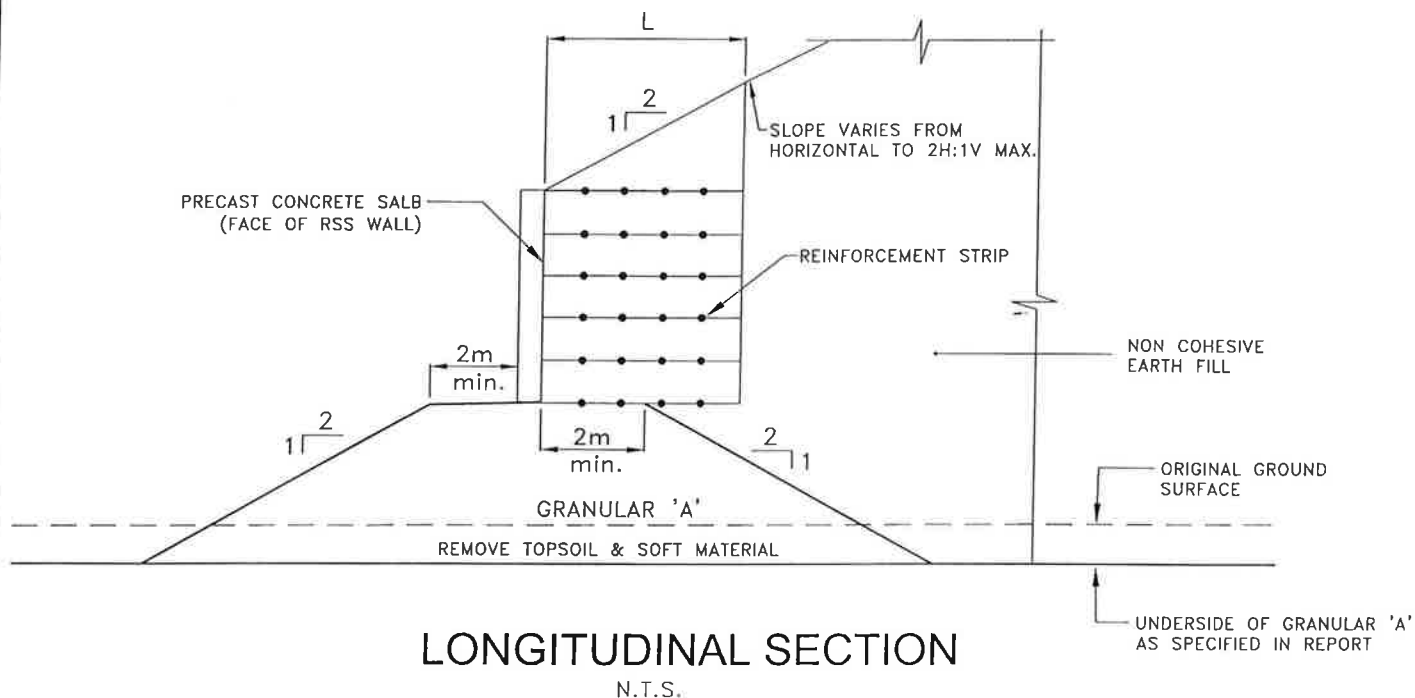


# APPENDIX E

## Figures

**Terraprobe Limited**





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 preboring and 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.

