

**PRELIMINARY**  
**FOUNDATION INVESTIGATION AND DESIGN REPORT**  
**N-E RAMP/N – WELLINGTON RAMP OVER GUELPH STREET**  
**HIGHWAY 7-NEW, KITCHENER TO GUELPH**  
**G.W.P. 408-88-00**

**Geocres Number: 40P8-155**

**Report to**

**Ministry of Transportation Ontario**  
**West Region**

Thurber Engineering Ltd.  
2010 Winston Park Drive, Suite 103  
Oakville, Ontario  
L6H 5R7  
Phone: (905) 829 8666  
Fax: (905) 829 1166

December 2, 2008  
File: 15-64-17

\\Torserver1\Projects\15\64\17 Hwy 7 New\Structures\N-EW  
Ramp over Guelph St-2,4\15-64-17 N-E over Guelph St-  
FIDR-Final.doc

## TABLE OF CONTENTS

### **PART 1      FACTUAL INFORMATION**

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	Pavement structure.....	3
5.2	Fill .....	3
5.3	Sand.....	4
5.4	Silty Clay Till.....	5
5.5	Sandy Silt Till .....	6
5.6	Groundwater Conditions .....	6
6	MISCELLANEOUS .....	7

### **PART 2 ENGINEERING DISCUSSION AND RECOMMENDATIONS**

7	GENERAL.....	9
8	STRUCTURE FOUNDATIONS.....	9
8.1	Spread Footings on Native Soil .....	10
8.2	Spread Footings on Engineered Fill.....	11
8.3	Steel H-Piles.....	11
8.3.1	Axial Resistance .....	12
8.3.2	Downdrag .....	13
8.4	Abutment Design Considerations .....	13
8.5	Frost Cover.....	13
8.6	Recommended Foundation .....	13
9	BRIDGE APPROACHES AND EMBANKMENTS .....	13
10	CONSTRUCTION CONCERNS .....	14
11	INVESTIGATION FOR DETAIL DESIGN.....	15
12	CLOSURE .....	15

## Appendices

Appendix A	Record of Borehole Sheets
Appendix B	Laboratory Test Results
Appendix C	Foundation Comparison
Appendix D	Figure
Appendix E	Slope Stability Output
Appendix F	Site Photograph
Appendix G	Drawing titled "Borehole Locations and Soil Strata"

**PRELIMINARY  
FOUNDATION INVESTIGATION AND DESIGN REPORT  
N-E RAMP/N – WELLINGTON RAMP OVER GUELPH STREET  
HIGHWAY 7-NEW, KITCHENER TO GUELPH  
G.W.P. 408-88-00**

**Geocres Number: 40P8-155**

**PART 1: FACTUAL INFORMATION**

**1 INTRODUCTION**

This report presents the factual findings obtained from a preliminary foundation investigation conducted at the site of the proposed N-E entrance ramp to Highway 7 and N-E/W ramp to Wellington Street over Guelph Street in the Regional Municipality of Waterloo. The proposed ramps are part of the Highway 7-New project.

The purpose of the 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, a stratigraphic profile, laboratory test results and a written description of the subsurface conditions. A model of the subsurface conditions under the potential foundation footprint was developed from the data obtained in the course of the investigation.

The information collected in the course of the investigation and presented in this report is intended for preliminary design purposes only. Additional site investigation, field testing and engineering analysis will be required at the detail design stage. The extent of the additional investigation will depend, in part, on the final location and General Arrangement of the structure.

Thurber carried out the investigation for the Ministry of Transportation Ontario, Southwestern Region (MTO) under Purchase Order Number 3006-E-0123.

**2 SITE DESCRIPTION**

The site lies approximately 340 m to the north of existing Wellington Street and approximately 60 m west of Kitchener-Waterloo Expressway. At this location, the proposed N-E entrance ramp to Highway 7 and N-E/W ramp to Wellington Street will cross over the existing Guelph Street. The lands surrounding the site are generally industrial and commercial. The site is generally flat.

Based on the Ontario Geological Survey Special Volume 2, The Physiography of Southern Ontario, Third Edition by Chapman and Putnam, the site lies within the physiographic region known as the

Waterloo Hills, characterized by ridges of sandy till and kames or kame moraines, with outwash sands occupying the intervening hollows.

Two photographs of the site, looking at the south side of Guelph Street, east of KWE are included in Appendix F and show the general nature of the surrounding land.

### 3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing at this site was carried out on June 5, 6 and 10 to 12, 2008. Two boreholes, numbered 08-002 and 08-004, were drilled approximately at the north and south abutments of a possible single-span structure arrangement. The depths of Boreholes 08-002 and 08-004 were 15.5 m and 17.0 m (Elevations 292.8 and 291.8), respectively. The Record of Borehole sheets for the boreholes are included in Appendix A. The approximate locations of the two boreholes are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix G.

Prior to commencing the site investigation, clearance was obtained from utility companies having plant in the area.

The boreholes were drilled using hollow stem auger equipment operated by a CME75 truck-mounted drill rig. 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. In Borehole 08-002, drilled at the proposed north abutment, a standpipe piezometer consisting of 19 mm diameter PVC pipe with a slotted screen was installed and enclosed in filter sand to permit longer term groundwater level monitoring. The location and completion details of the piezometer are shown in Table 3.1. Borehole 08-004 was grouted with benseal upon completion. The borehole completion details are shown in Table 3.1.

The completion of the borehole and the standpipe piezometer were carried out in accordance with the requirements of O. Reg. 903 (as amended by O. Reg. 372/07).

**Table 3.1 – Borehole Completion Details**

Foundation Unit	Borehole Location	Piezometer Tip Depth/ Elevation (m)	Completion Details
North Abutment	08-002	15.3/293.0	Piezometer with 1.5 m slotted screen installed with sand filter to 13.1 m, holeplug from 13.1 m to 12.5 m, grout from 12.5 m to 0.9 m, sand from 0.9 m to 0.6 m, holeplug from 0.6 to 0.15, then concrete to surface.
South Abutment	08-004	No Installation	Benseal to 5.8 m, holeplug from 5.8 m to 75 mm, then asphalt to surface.

A member of Thurber'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 Thurber's laboratory for further examination and testing.

#### **4 LABORATORY TESTING**

The recovered soil samples were subjected to Visual Identification (VI) and to natural moisture content determination. The results of this testing are shown on the Record of Borehole sheets in Appendix A. Selected samples were also subjected to gradation analysis (sieve and hydrometer) and Atterberg Limits testing where appropriate. The results of this testing program are shown on the Record of Borehole sheets in Appendix A and on the figures contained 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 along the proposed alignment are presented in this appendix and on the "Borehole Locations and Soil Strata" drawing in Appendix G. 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 pavement structure, sand and gravel fill and fill of variable proportions of mixed soils (sand, silt and clay) overlying native sand, silty clay till and sandy silt till.

##### **5.1 Pavement structure**

Pavement structure consisting of approximately 50 and 100 mm of asphalt overlying granular (sand and gravel fill) road base was encountered in both boreholes drilled on Guelph Street.

The thickness of the granular road base measured in the boreholes ranged from 0.6 m to 0.7 m with the underside at Elevations 307.5 and 308.1 in Boreholes 08-002 and 08-004, respectively.

##### **5.2 Fill**

Layers of fill were encountered below the road base in both boreholes. The fill generally consists of variable proportions of mixture of various soils:

- Brown to grey sand and gravel fill containing trace gravel, trace to some silt, and occasional clayey silt seams.
- A layer of various proportions of mixed fill soils (sand, silt and clay) containing trace gravel was encountered at 0.7 m depth (Elevation 308.1) in Borehole 08-004. This layer was 0.8 m thick.

- A layer of grey silty sand fill was contacted in Borehole 08-004 at 1.5 m depth (Elevation 307.3). Gas odour was noted in this fill layer.
- A layer of dark grey silty clay fill was contacted below the sand and gravel at 0.8 m depth (Elevation 307.5) in Borehole 08-002.

Total thicknesses of the fill were 2.8 m and 2.9 m. The depths to the base of the fill were at 2.9 m and 3.0 m (Elevations 305.4 and 305.8) in Boreholes 08-002 and 08-004, respectively.

The cohesive fill is classified as firm to very stiff, based on SPT 'N' values ranging between 5 and 20 blows for 0.3 m of penetration. The cohesionless fill is classified as compact to dense, based on an SPT 'N' values ranging from 10 to 43 blows per 0.3 m of penetration. The natural moisture content ranged from 10 to 22%.

Grain size distribution curve for fill sample is presented on the Record of Borehole sheet and on Figure B1 Appendix B. The results of the laboratory tests are summarized as follows:

Soil Particles	(%)
Gravel	3
Sand	49
Silt	27
Clay	22

### 5.3 Sand

Native grey sand containing trace to some silt, trace gravel and trace of clay was contacted in Borehole 08-004 12.2 m depth (Elevation 296.6).

Thickness of the sand layer was 1.5 m. The depths to the base of the sand layer 13.7 m (Elevation 295.1).

The sand is classified as very dense, based on SPT 'N' value of 90 blows for 0.3 m of penetration. The natural moisture content measured was 20%.

Grain size distribution curve for a sand sample, is presented on the Record of Borehole sheets and on Figure B2 of Appendix B. The results of the laboratory test are summarized as follows:

Soil Particles	(%)
Gravel	2
Sand	85
Silt & Clay	13

#### 5.4 Silty Clay Till

Native grey silty clay till containing trace sand and trace gravel was contacted below the fill in both boreholes. Depths and elevations where silty clay till was contacted are shown in Table 5.1

**Table 5.2 – Depths and Elevations of Native Silty Clay Till**

Foundation Unit	Borehole	Depth below existing ground surface (m)	Elevation (m)	Thickness (m)
North Abutment	08-002	2.9 to 11.1	305.4 to 297.2	8.2
South Abutment	08-004	3.0 to 12.2	305.8 to 296.6	9.2

A 700-mm thick layer of silt was contacted within the silty clay till at 11.0 m depth (Elevation 297.8) in Borehole 08-004.

The cohesive layer is very stiff to hard in consistency, based on SPT 'N' values ranging from 20 to 68 blows per 0.3 m of penetration. The moisture content varied from 19% to 22%.

Grain size distribution curves for selected silty clay till samples are presented on the Record of Borehole sheets and on Figure B3 Appendix B. Atterberg Limits test results are presented on Figure B6 of Appendix B. The results of the laboratory tests are summarized as follows:

Soil Particles	(%)
Gravel	0 to 1
Sand	1 to 4
Silt	21 to 44
Clay	54 to 78

Liquid Limit	41 to 51
Plastic Limit	18 to 22

The above results show that the silty clay till is of medium to high plasticity with a group symbol of CI-CH.

It should be noted that glacial tills are known to contain cobbles and boulders.



### 5.5 Sandy Silt Till

Grey sandy silt till containing trace gravel, some clay and occasional cobbles was encountered in Boreholes 08-002 and 08-004 at 11.1 m and 13.7 m depths (Elevations 297.2 and 295.1), respectively.

Boreholes 08-002 and 08-004 were terminated within the sandy silt till layer at 15.5 m and 17.0 m depths (Elevations 292.8 and 291.8), respectively.

SPT 'N' values measured in the sandy silt till ranged from 107 blows per 0.3 m of penetration to higher than 100 blows per 0.2 m of penetration, indicating a very dense relative density. The natural moisture contents generally lay in the range of 8 to 10%.

Grain size distribution curves for three samples of the till are presented on the Record of Borehole sheets and on Figure B4 of Appendix B. The results of the laboratory tests are summarized as follows:

Soil Particles	(%)
Gravel	1 to 5
Sand	29 to 43
Silt	42 to 55
Clay	10 to 14

Although not encountered in the boreholes, this glacial till layer may contain cobbles and boulders which may account for some high SPT 'N' values and resistance to augering.

### 5.6 Groundwater Conditions

Water levels were observed in the boreholes during and upon completion of drilling. A standpipe piezometer was installed in Borehole 08-002 (north abutment) to monitor water levels after completion of drilling. The water levels measured in the piezometer are summarized in Table 5.2, along with the measurements in the open boreholes upon completion of drilling.

**Table 5.2 – Water Level Measurements**

Foundation Unit	Borehole	Date (2008)	Water Level (m)		Comment
			Depth	Elevation	
North Abutment	08-002	June 6	5.5	302.8	During drilling In piezometer In piezometer
		July 16	5.5	302.8	
		August 20	5.5	302.8	
South Abutment	08-004	June 11	1.5	307.3	During drilling

The piezometric reading indicates that the groundwater level is near Elevation 302.8 m.

The above values are short-term readings and seasonal fluctuations of the groundwater level are to be expected. In particular, the groundwater level may be at a higher elevation after the spring snowmelt or after periods of heavy rainfall.

## 6 MISCELLANEOUS

All-Terrain Drilling of Waterloo, Ontario supplied a truck-mounted CME75 drill rig and conducted the drilling, sampling and in-situ testing operations.

The drilling and sampling operations in the field were supervised on a full time basis by Mr. Stephane Loranger, C.E.T. of Thurber, under the direction of Mr. Alastair E. Gorman, P.Eng and Mr. Mark Farrant, P. Eng.

The coordinates for the boreholes and the ground surface Elevations were obtained by Thurber Engineering Ltd. using GPS equipment.

Overall supervision of the field program was conducted by Mr. Alastair E. Gorman, P.Eng. and Mr. M. Farrant, P. Eng. Interpretation of the data and preparation of the report were carried out by Mr. Alastair E. Gorman, P.Eng. and Ms. R. Palomeque Reyna, P.Eng.

Dr. P.K. Chatterji, P.Eng. a Designated Principal Contact for MTO Foundations projects, reviewed the report.

Thurber Engineering Ltd.

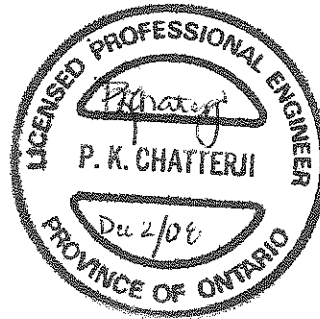
Rocio Palomeque Reyna, P.Eng.  
Geotechnical Engineer



Alastair E. Gorman, P.Eng.,  
Senior Foundations Engineer



P.K. Chatterji, P.Eng.,  
Review Principal, Designated MTO Contact



**PRELIMINARY  
FOUNDATION INVESTIGATION AND DESIGN REPORT  
N-E RAMP/N – WELLINGTON RAMP OVER GUELPH STREET  
HIGHWAY 7-NEW, KITCHENER TO GUELPH  
G.W.P. 408-88-00**

**Geocres Number: 40P8-155**

**PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS**

**7 GENERAL**

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

Based on the Plates 2A and 2B of the E.A:

- The proposed grades of N-E Ramp/N-Wellington Ramp over Guelph Street will be at approximate Elevations ranging from 323.0 to 325.0.
- The existing ground surface within the proposed ramp structures is near Elevations 308.3 to 308.8. Hence, N-E Ramp embankments will be on a fill about 16.0 m to 17.0 m high, relative to the surrounding grade.

It is anticipated that a single span structure will be constructed. 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 this investigation.

**8 STRUCTURE FOUNDATIONS**

The stratigraphy identified in the preliminary investigation consisted primarily of pavement structure and fill of various materials (sand, sand and gravel, silty sand and silty clay) overlying native layers of very stiff to hard silty clay till, very dense sand and very dense sandy silt till. The groundwater level measured in the piezometer was 5.5 m (Elevation 302.8) below the ground surface.

In the preparation of the preliminary geotechnical design recommendations, consideration was given to the following foundation types:

- Spread footings bearing on native soil
- Spread footings on engineered fill
- Steel H-piles driven into the very dense soil

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

### 8.1 Spread Footings on Native Soil

Spread footings bearing on native soil generally are the least expensive form of construction.

The existing fill is not considered to be suitable for the support of spread footings and the footings must be placed on the underlying native soils.

The design of spread footings bearing on native undisturbed very stiff to hard silty clay till must be in accordance with the elevations and bearing resistances given in Table 8.1.

**Table 8.1 – Bearing Resistances for Spread Footings**

Element	Depth (m)	Elev.	ULS <sub>r</sub> (kPa)	SLS (kPa)	Soil
North Abutment (BH 08-002)	3.0	305.3	300	200	Very stiff silty clay till
	4.3	304.0	450	300	Very stiff to hard silty clay till
South Abutment (BH 08-004)	3.2	305.6	300	200	Very stiff silty clay till
	4.3 or below	304.5 or below	450	300	Very stiff to hard silty clay till

The bearing resistances in Table 8.1 are for vertical, concentric loading. In the case of eccentric or inclined loading, the bearing resistance must be adjusted as shown in the CHBDC (2006) Clause 6.7.3 and Clause 6.7.4.

The geotechnical SLS resistance values given above are based on an estimated total settlement not exceeding 25 mm. This settlement is expected to be substantially complete by the end of construction. Differential settlement is not expected to exceed 20 mm across the width of the structure or between foundation elements.

Founding elevations presented in Table 8.1 are generally above groundwater level measured in the piezometer. Water was also observed at 1.5 m below ground surface in Borehole 08-004 during drilling. If water is observed during footing excavation or if excavation extends below the groundwater level, groundwater control will be required to

construct the footings in the dry, to prevent sloughing of the sides and to prevent disturbance of the footing bases due to the inflow of groundwater.

### 8.2 Spread Footings on Engineered Fill

Spread footings can also be founded on Granular "A" engineered fill pads. These would be most useful in the case of perched abutments on footings.

If an engineered fill pad is used, subexcavation of any topsoil, organics, unsuitable, surficial fill soils or other deleterious materials must be stripped from the footprint of the foundation to expose competent native subgrade material. The engineered fill must bear on native very stiff silty clay till and the highest permitted founding elevations at which engineered fill pads may be placed, are given in Table 8.2.

**Table 8.2 – Founding Elevations for Engineered Fill Pads**

<b>North Abutment (BH 08-002)</b>	<b>South Abutment (BH 08-004)</b>
305.4	305.8

Typically, spread footings on pads of engineered granular fill at least 2 m thick may be designed for the following geotechnical resistances:

- Factored ULS 900 kPa
- SLS 350 kPa

These resistance values are for concentric, vertical loads only. In the case of eccentric or inclined loading, the geotechnical resistance must be calculated as illustrated in the CHBDC Clause 6.7.3 and Clause 6.7.4.

For footings designed on the basis of the geotechnical resistance values given above, total settlement under a footing is expected to not exceed 25 mm. This settlement is expected to be substantially complete by the end of construction. Differential settlement is not expected to exceed 20 mm across the width of the structure or between foundation elements.

The Granular A pad must be compacted to 100% of Standard proctor maximum dry density (SPMDD) at optimum moisture content  $\pm 2\%$ . The geometry of the fill pad must conform to the general requirements shown in Figure 1 in Appendix D.

### 8.3 Steel H-Piles

The soil stratigraphy encountered at this site is considered to be suitable for the support of foundations on driven steel piles.

It is recommended that the H-piles be driven to achieve resistance in the very dense sandy silt till encountered at this site. The depths and elevations at which the H-piles are expected to develop the required resistance are given in Table 8.3.

**Table 8.3 – Estimated Pile Tip Elevation**

Foundation Unit	Pile Tip Depth (m)	Pile Tip Elevation
North Abutment (BH 08-002)	13.8	294.5
South Abutment (BH 08-004)	14.8	294.0

### 8.3.1 Axial Resistance

For preliminary design, the vertical, axial, factored geotechnical resistance at Ultimate Limit States (ULS) and geotechnical resistance at Serviceability Limit States (SLS) for two pile sections when driven into the very dense sandy silt till are presented in Tables 8.4.

**Table 8.4 – Axial Resistance of Two Pile Sections Founded on Very Dense Soils**

Pile Section	Geotechnical Resistance (kN)	
	Factored ULS	SLS
HP 310 X 110	1,600	1,400
HP 360 X 132	1,800	1,600

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

Installation of the piles must be in accordance with SP 903S01 and must be controlled using the Hiley Formula and an ultimate resistance of 3,200 kN for an HP 310 X 110 pile and 3,600 kN for the HP 360 X 132 pile.

These are preliminary recommendations and may change during detail design based on the final alignment, final bridge arrangement and the results of the site investigation and field testing to be completed at that time.

Due to the possible presence of cobbles and boulders in the glacial sandy silt till layer at the expected founding layer, the tips of all driven piles should be fitted with steel H-Pile driving shoes in accordance with OPSD 3000.100.

Higher geotechnical resistances may be achieved by installing the piles to a greater depth. For piles extending below Elevations 293.0 and 292.0 (approximately) at the north and south abutments, respectively a greater depth of exploration is required and must be addressed during the detail design phase.

### **8.3.2 Downdrag**

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

### **8.4 Abutment Design Considerations**

From a geotechnical perspective, the conditions at this site are considered to be suitable for the design of conventional, semi-integral or integral abutments.

### **8.5 Frost Cover**

The design depth of frost penetration for this site is 1.4 m. All footing bases and undersides of pile caps/abutment stems must be provided with at least 1.4 m of soil cover.

### **8.6 Recommended Foundation**

From a geotechnical perspective, and based on current information, the recommended abutment foundation consists of steel H-piles driven into the very dense native sandy silt till, despite the higher cost noted in Appendix C.

## **9 BRIDGE APPROACHES AND EMBANKMENTS**

Based on the two boreholes drilled at the site, the approach embankments will be constructed over very stiff silty clay till and may incorporate the sand and gravel fill of the existing embankment.

Preliminary analysis indicates that at the abutments, settlement in the order of 45 to 55 mm is estimated in the foundation soils under the loading imposed by approximately 17.0 m of the approach fill. Due to the stiff to hard nature of the foundation soils, these settlements, as well as the settlements of the fill itself, will be essentially completed when construction of the fill is completed. Further settlement analysis should be conducted during the detail phase design.

The 16.0 m to 17.0 m high embankments likely to be constructed will be stable at side slopes of 2H:1V if constructed using SSM or granular fill. 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 grade to shed run-off water.

For the purpose of preliminary embankment stability analyses, the commercially available slope stability program GSLOPE developed by Mitre Software Inc. was used. The Bishop's simplified method for stability analysis was employed.

Global stability analyses were conducted for a 17 m high, 2H:1V SSM or earth fill embankment. The stability of the embankment was also checked under seismic loading assuming an acceleration



of 0.08g. The computed factors of safety are as shown in Table 9.1. Slope stability computation outputs are included in Appendix E.

**Table 9.1 Computed Factors of Safety**

Location / Material	Condition	Factor of Safety	Figure (Appendix E)
<b>17 m High</b>			
Earth Fill	Normal	1.6	1
Earth Fill	Seismic = 0.08g	1.3	2

These factors of safety are considered to be acceptable for the proposed embankment bearing on the very stiff to hard foundation soils present at this site.

The global, internal and surficial stability of the approach embankment fills should be further evaluated during the detail design phase.

The proposed N-E entrance ramp grade shown on EA Plates 2A and 2B may be at approximately 20 m above the groundwater table measured in the piezometer. During detail design, when the grade has been finalized, permanent drainage (if necessary) and slope protection requirements must be addressed.

## 10 CONSTRUCTION CONCERNS

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

1. Pile refusal at higher elevation.

Although there was little direct evidence of their presence during drilling, glacial till deposits inherently contain boulders. It is possible that a pile will achieve refusal at a higher elevation than anticipated due to encountering a boulder. If it is suspected that this is happening, the QVE must immediately bring it to the attention of the CA. If the CA cannot resolve the issue, it must be referred to the design team for resolution.

2. Pile fails to develop specified resistance.

If a pile has not developed the specified resistance after being driven 3 m beyond the anticipated pile tip elevation, stop driving and check the Hiley calculation and all input values. If the calculation still shows that the pile has not reached the specified resistance, the following procedure should be implemented:

- a) Stop driving in that pile group for 48 hours (minimum)
- b) After 48 hours, warm up the hammer on another pile then commence re-driving the subject pile and measure the resistance.
- c) If the pile still does not reach the specified resistance, the QVE must immediately advise the CA who, in turn, should refer the issue to the design team.

### 3. Impact on adjacent structure

During the detail design phase, depending on the outcome of detail settlement analysis due to the placement of 16.0 m to 17.0 m high approach fill for the new structure, it may be necessary to implement a monitoring program for the existing structure.

## 11 INVESTIGATION FOR DETAIL DESIGN

During the detail design phase of the project, additional site investigation and field testing will be required. The following minimum program is recommended:

### 1. Boreholes for structure foundations.

Additional boreholes may be required for the structure foundations, especially if the structure is built off the current N-E entrance ramp to Highway 7 and N-E/W ramp to Wellington Street over Guelph Street alignment and thus removed from the alignment of the current investigation. Exploration off the existing road embankment is recommended.

### 2. Boreholes for approaches.

A minimum of one borehole is recommended in each approach fill on N-E Ramp.

### 3. Impact on existing structure

During the detail design phase, the impact on the existing structure and KWE pavement of placing new fill for embankment widening must be analyzed. It is anticipated that 45 to 55 mm of settlement may be induced by the new fill.

## 12 CLOSURE

Engineering analysis and preparation of the report were carried out by Mr. Alastair E. Gorman, P.Eng and Ms. R. Palomeque Reyna, P.Eng.

The report was reviewed by Dr. P.K. Chatterji, P. Eng., a Designated Principal Contact for MTO Foundations Projects.

Thurber Engineering Ltd.

Rocio Palomeque Reyna, P.Eng., M.Eng.  
Geotechnical Engineer



Alastair E. Gorman, P.Eng.,  
Senior Foundations Engineer



P. K. Chatterji, P.Eng.,  
Review Principal



## **Appendix A**

### **Record of Borehole Sheets**

## SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

### 1. TEXTURAL CLASSIFICATION OF SOILS

CLASSIFICATION	PARTICLE SIZE	VISUAL IDENTIFICATION
Boulders	Greater than 200mm	same
Cobbles	75 to 200mm	same
Gravel	4.75 to 75mm	5 to 75mm
Sand	0.075 to 4.75mm	Not visible particles to 5mm
Silt	0.002 to 0.075mm	Non-plastic particles, not visible to the naked eye
Clay	Less than 0.002mm	Plastic particles, not visible to the naked eye

### 2. COARSE GRAIN SOIL DESCRIPTION (50% greater than 0.075mm)

TERMINOLOGY	PROPORTION
Trace or Occasional	Less than 10%
Some	10 to 20%
Adjective (e.g. silty or sandy)	20 to 35%
And (e.g. sand and gravel)	35 to 50%

### 3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

DESCRIPTIVE TERM	UNDRAINED SHEAR STRENGTH (kPa)	APPROXIMATE SPT <sup>(1)</sup> 'N' VALUE
Very Soft	12 or less	Less than 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	Greater than 200	Greater than 30

NOTE: Hierarchy of Soil Strength Prediction

- 1) Laboratory Triaxial Testing
- 2) Field Insitu Vane Testing
- 3) Laboratory Vane Testing
- 4) SPT value
- 5) Pocket Penetrometer



### 4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

DESCRIPTIVE TERM	SPT "N" VALUE
Very Loose	Less than 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	Greater than 50

### 5. LEGEND FOR RECORDS OF BOREHOLES

SYMBOLS AND ABBREVIATIONS FOR SAMPLE TYPE	SS Split Spoon Sample	WS Wash Sample	AS Auger (Grab) Sample
	TW Thin Wall Shelby Tube Sample	TP Thin Wall Piston Sample	
	PH Sampler Advanced by Hydraulic Pressure	PM Sampler Advanced by Manual Pressure	
	WH Sampler Advanced by Self Static Weight	RC Rock Core	SC Soil Core

$$\text{Sensitivity} = \frac{\text{Undisturbed Shear Strength}}{\text{Remoulded Shear Strength}}$$






 Water Level  
 C<sub>pen</sub> Shear Strength Determination by Pocket Penetrometer

- (1) SPT 'N' Value Standard Penetration Test 'N' Value – refers to the number of blows from a 63.5kg hammer free falling a height of 0.76m to advance a standard 50 mm outside diameter split spoon sampler for 0.3 m depth into undisturbed ground.
- (2) DCPT Dynamic Cone Penetration Test – Continuous penetration of a 50 mm outside diameter, 60° conical steel point attached to "A" size rods driven by a 63.5 kg hammer free falling a height of 0.76 m. The resistance to cone penetration is the number of hammer blows required for each 0.3 m advance of the conical point into undisturbed ground.

# UNIFIED SOILS CLASSIFICATION

MAJOR DIVISIONS		GROUP SYMBOL	TYPICAL DESCRIPTION
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW	Well-graded gravels or gravel-sand mixtures, little or no fines.
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines.
		GM	Silty gravels, gravel-sand-silt mixtures.
		GC	Clayey gravels, gravel-sand-clay mixtures.
	SAND AND SANDY SOILS	SW	Well-graded sands or gravelly sands, little or no fines.
		SP	Poorly-graded sands or gravelly sands, little or no fines.
		SM	Silty sands, sand-silt mixtures.
		SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS	SILTS AND CLAYS $W_L < 50\%$	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays. ( $W_L < 30\%$ ).
		CI	Inorganic clays of medium plasticity, silty clays. ( $30\% < W_L < 50\%$ ).
		OL	Organic silts and organic silty-clays of low plasticity.
	SILTS AND CLAYS $W_L > 50\%$	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
		CH	Inorganic clays of high plasticity, fat clays.
		OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils.
CLAY SHALE			
SANDSTONE			
SILTSTONE			
CLAYSTONE			
COAL			

## EXPLANATION OF ROCK LOGGING TERMS

ROCK WEATHERING CLASSIFICATION		SYMBOLS	
Fresh (FR)	No visible signs of weathering.		
Fresh Jointed (FJ)	Weathering limited to the surface of major discontinuities.		CLAYSTONE
Slightly Weathered (SW)	Penetrative weathering developed on open discontinuity surfaces, but only slight weathering of rock material.		SILTSTONE
Moderately Weathered (MW)	Weathering extends throughout the rock mass, but the rock material is not friable.		SANDSTONE
Highly Weathered (HW)	Weathering extends throughout the rock mass and the rock is partly friable.		COAL
Completely Weathered (CW)	Rock is wholly decomposed and in a friable condition, but the rock texture and structure are preserved.		Bedrock (general)

DISCONTINUITY SPACING		STRENGTH CLASSIFICATION			
Bedding	Bedding Plane Spacing	Rock Strength	Approximate Uniaxial Compressive Strength (MPa)	Field Estimation of Hardness*	
Very thickly bedded	Greater than 2m	Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Thickly bedded	0.6 to 2m				
Medium bedded	0.2 to 0.6m	Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Thinly bedded	60mm to 0.2m				
Very thinly bedded	20 to 60mm	Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Laminated	6 to 20mm				
Thinly Laminated	Less than 6mm	Medium Strong	25.0 to 50.0	3,500 to 7,500	Breaks under single blow of geological hammer.
<b>TERMS</b>					
Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.	Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.	Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
Rock Quality Designation: (RQD)	Total length of sound core recovered in pieces 0.1m in length or larger as a percentage of total core run length.	Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail
Uniaxial Compressive Strength (UCS)	Axial stress required to break the specimen				
Fracture Index: (FI)	Frequency of natural fractures per 0.3m of core run.				

# RECORD OF BOREHOLE No 08-002

1 OF 2

METRIC

G.W.P. 408-88-00 LOCATION N 4 814 728.23 E 225 979.86 ORIGINATED BY ES  
 HWY 7 BOREHOLE TYPE Hollow Stem Augers COMPILED BY WM  
 DATUM Geodetic DATE 2008.06.05 - 2008.06.06 CHECKED BY RPR

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						
308.3								20 40 60 80 100						
0.9 0.1	ASPHALT: (50mm)		1	AS			308							
307.5	SAND and GRAVEL, some silt Grey to Brown Moist (FILL)													
0.8	Silty CLAY, some sand, trace gravel, some organics, occasional black staining Firm to Stiff Dark Grey (FILL)		1	SS	13		307							
306.2			2	SS	5									
2.1	SAND and GRAVEL, trace silt, occasional clayey silt seams Compact Grey Wet (FILL)		3	SS	10		306							
305.4														
2.9	Silty CLAY, trace sand Very Stiff to Hard Dark Grey (TILL)		4	SS	21		305							0 1 44 55
			5	SS	29		304							
			6	SS	41		302							
			7	SS	38		301							0 3 43 54
			8	SS	42		299							

Continued Next Page

+ 3 x 3 Numbers refer to  
Sensitivity 20  
15-5  
10 (%) STRAIN AT FAILURE



# RECORD OF BOREHOLE No 08-002

2 OF 2

METRIC

G.W.P. 408-88-00 LOCATION N 4 814 728.23 E 225 979.86 ORIGINATED BY ES  
 HWY 7 BOREHOLE TYPE Hollow Stem Augers COMPILED BY WM  
 DATUM Geodetic DATE 2008.06.05 - 2008.06.06 CHECKED BY RPR

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		
	Continued From Previous Page							SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						
								PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT w <sub>p</sub> — w — w <sub>L</sub> WATER CONTENT (%)						
297.2	Silty CLAY, trace gravel Hard Grey (TILL)		9	SS	68		298							
11.1	Sandy SILT, trace gravel, trace to some clay Very Dense Grey Moist (TILL)						297							
			10	SS	107		296							5 43 42 10
			11	SS	100/ 225		295							
							294							
292.8			12	SS	100/ 275		293							1 33 54 12
15.5	END OF BOREHOLE AT 15.5m. BOREHOLE OPEN AND WATER LEVEL AT 5.5m UPON COMPLETION OF DRILLING. Piezometer installation consists of 19mm diameter schedule 40 PVC pipe with a 1.52m slotted screen.  WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) 2008.07.16 5.5 302.8 2008.08.20 5.5 302.8													

## METRIC

CHECKED BY RPE

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity

# RECORD OF BOREHOLE No 08-004

2 OF 2

METRIC

G.W.P. 408-88-00 LOCATION N 4 814 712.58 E 225 985.09 ORIGINATED BY SLL  
 HWY 7 BOREHOLE TYPE Hollow Stem Augers COMPILED BY WM  
 DATUM Geodetic DATE 2008.06.10 - 2008.06.12 CHECKED BY RPR

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			SHEAR STRENGTH kPa					
	Continued From Previous Page							20 40 60 80 100	PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>		
								20 40 60 80 100	WATER CONTENT (%)				
									○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE				
										</			

+ 3 . x 3 : Numbers refer to  
Sensitivity 20  
15 5  
10 (%) STRAIN AT FAILURE

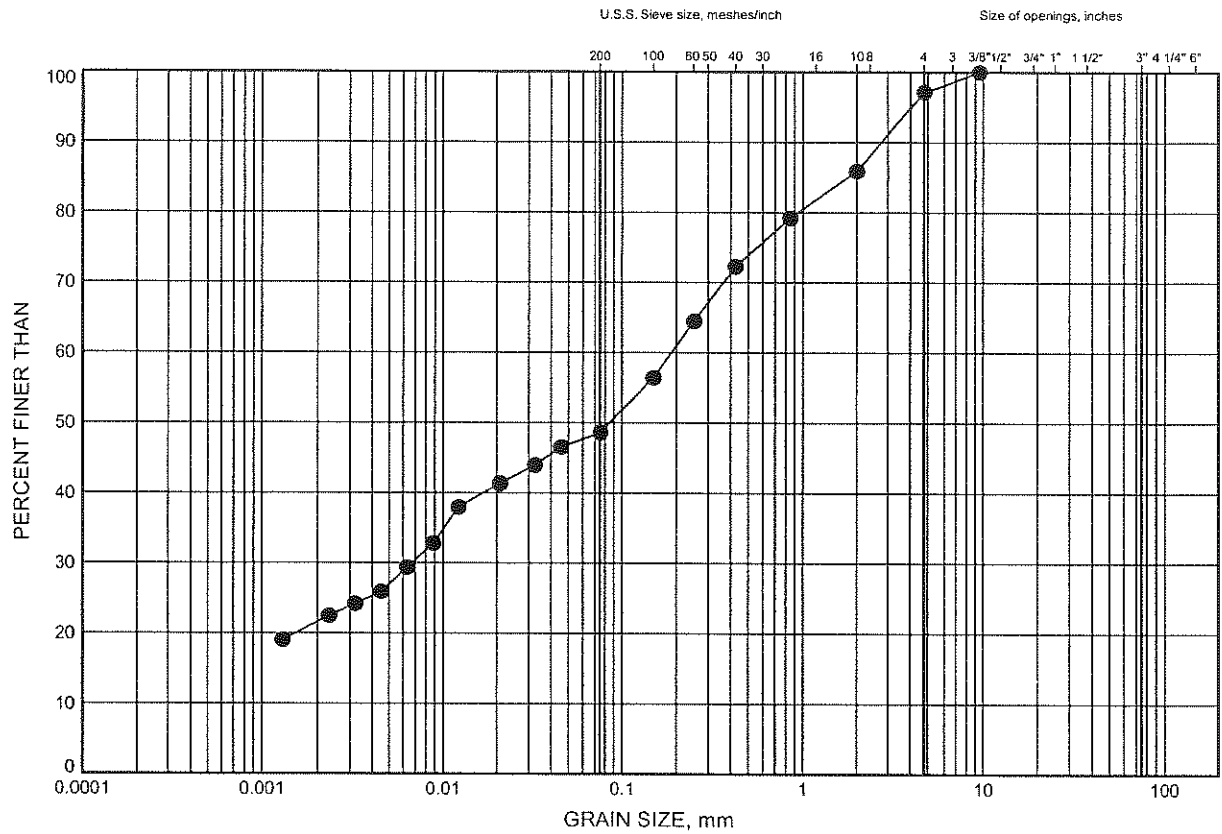
## **Appendix B**

### **Laboratory Test Results**

# Highway 7 - New GRAIN SIZE DISTRIBUTION

FIGURE B1

Sand, Silt and Clay FILL



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

## LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	08-004	1.07	307.73



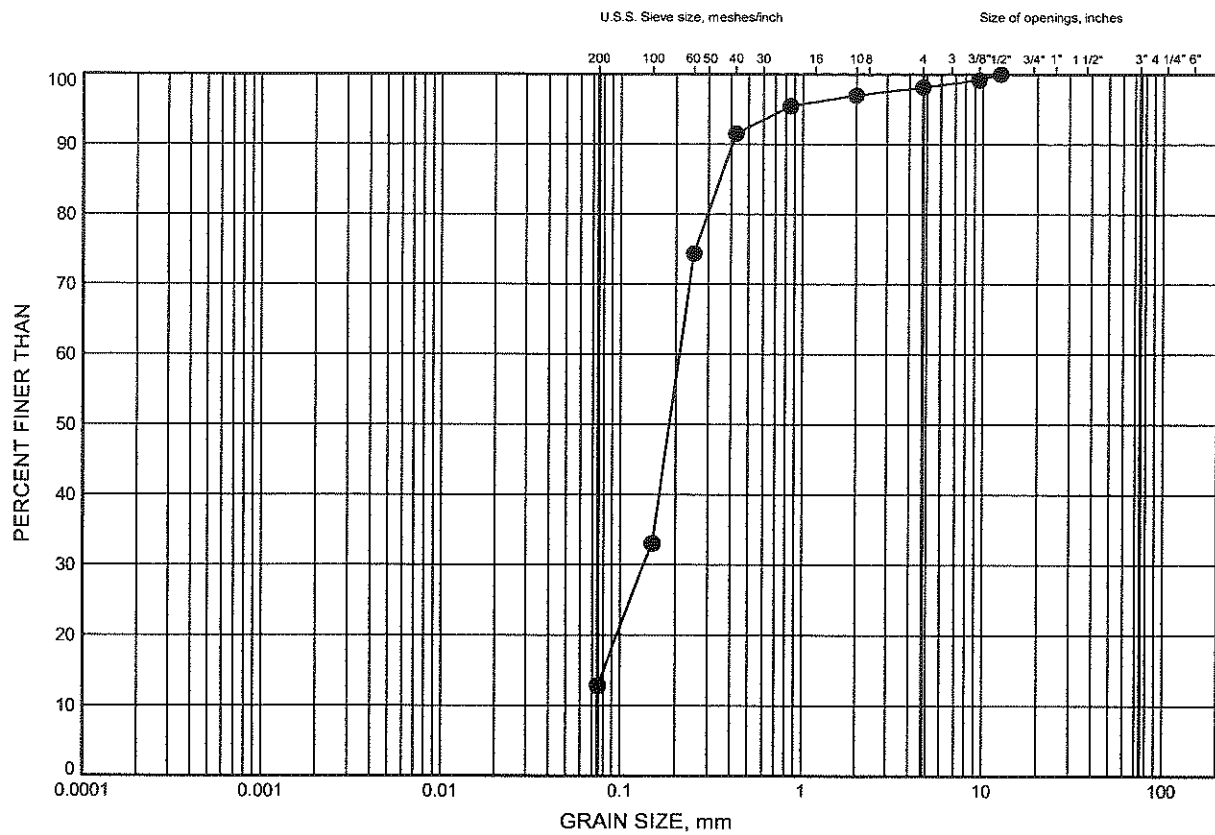
GRAIN SIZE DISTRIBUTION - THURBER 6417R.GPJ 12/3/08

W.P.# 408-88-00.....  
Prepared By AN.....  
Checked By RPR.....

# Highway 7 - New GRAIN SIZE DISTRIBUTION

FIGURE B2

Sand



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

## LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	08-004	12.48	296.32

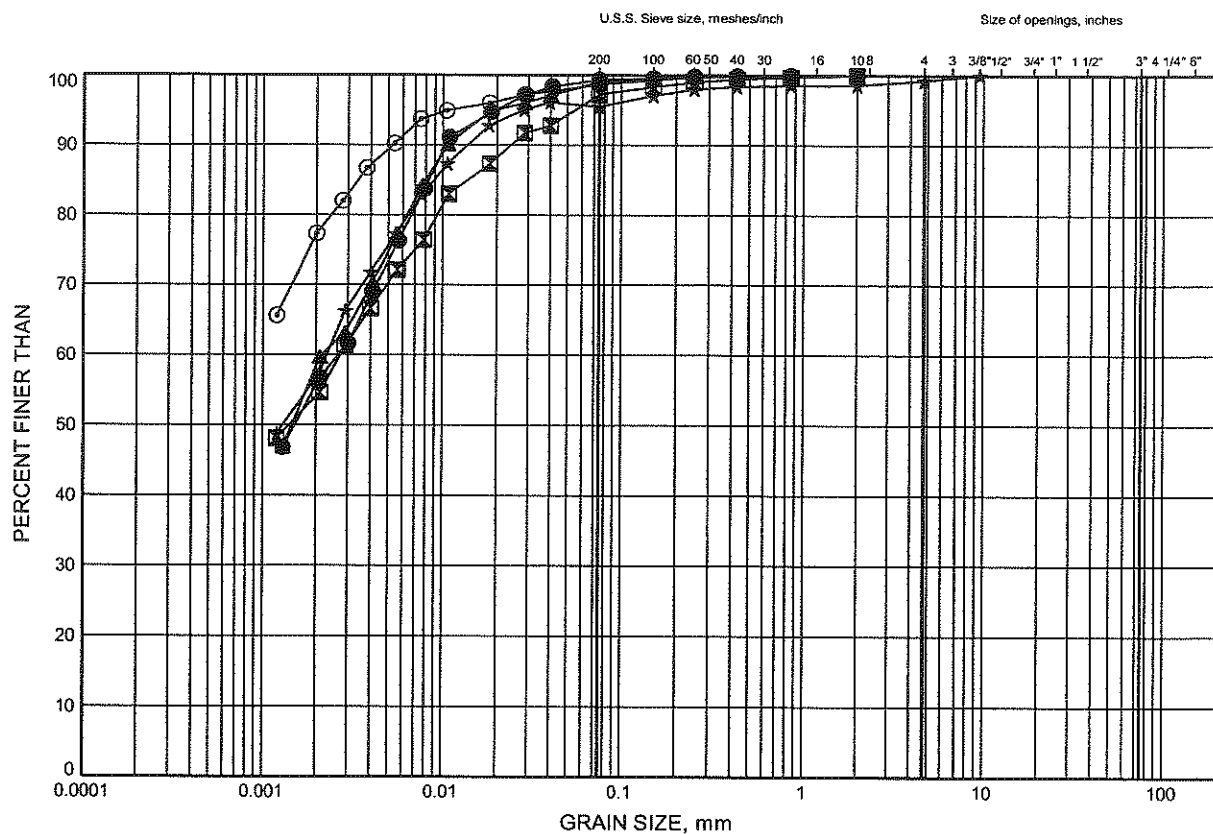


W.P.# 408-88-00  
Prepared By AN  
Checked By RPR

# Highway 7 - New GRAIN SIZE DISTRIBUTION

FIGURE B3

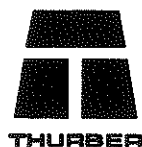
## Silty Clay TILL



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

### LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	08-002	3.35	304.95
⊠	08-002	7.92	300.38
▲	08-004	4.88	303.92
★	08-004	7.92	300.88
⊙	08-004	10.82	297.98

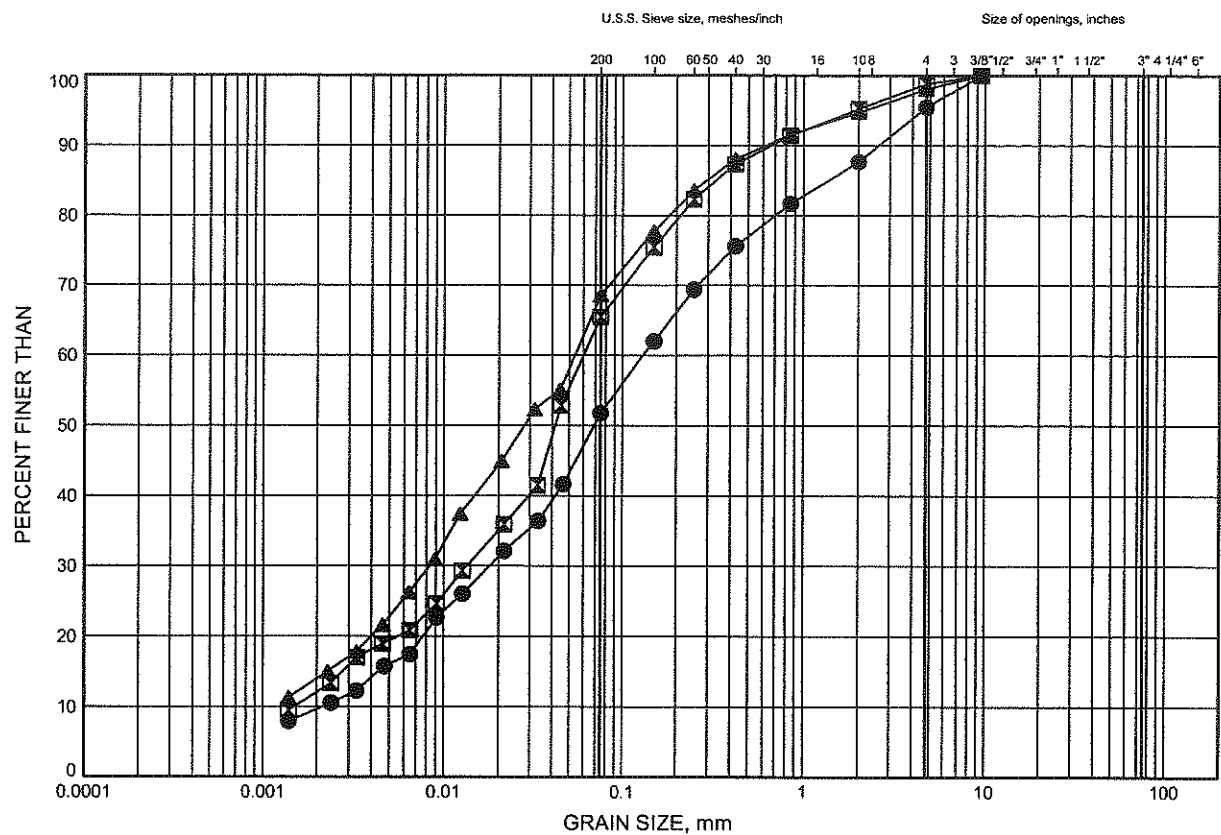


W.P.# .408-88-00.....  
Prepared By .AN.....  
Checked By .RPR.....

Highway 7 - New  
GRAIN SIZE DISTRIBUTION

FIGURE B4

Sandy Silt TILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	08-002	12.42	295.88
⊠	08-002	15.36	292.94
▲	08-004	15.39	293.41



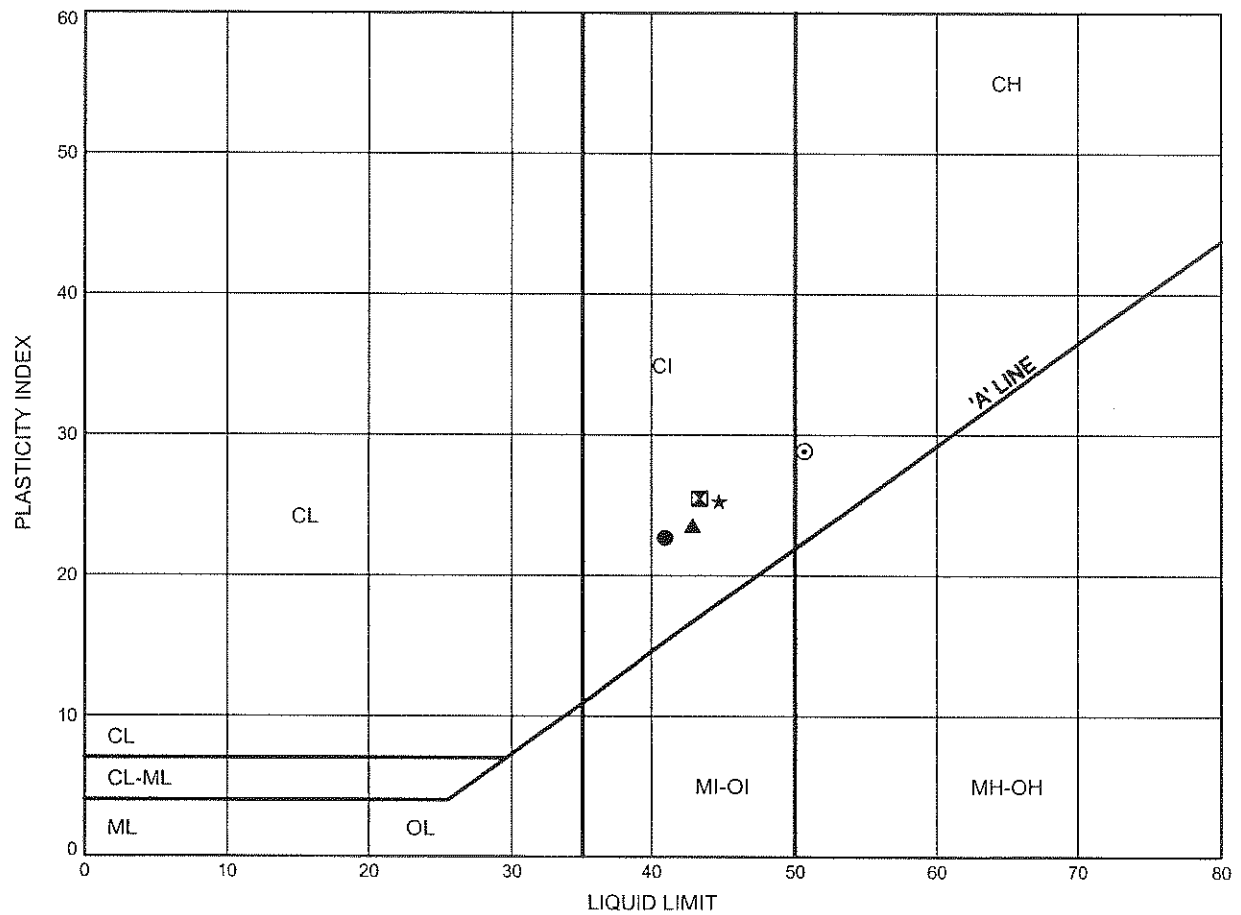
W.P.# 408-88-00  
Prepared By AN  
Checked By RPR



# Highway 7 - New ATTERBERG LIMITS TEST RESULTS

FIGURE B5

Silty Clay TILL



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	08-002	3.35	304.95
⊠	08-002	7.92	300.38
▲	08-004	4.88	303.92
★	08-004	7.92	300.88
⊙	08-004	10.82	297.98

Date December 2008  
Project 408-88-00



Prep'd AN  
Chkd. RPR

## **Appendix C**

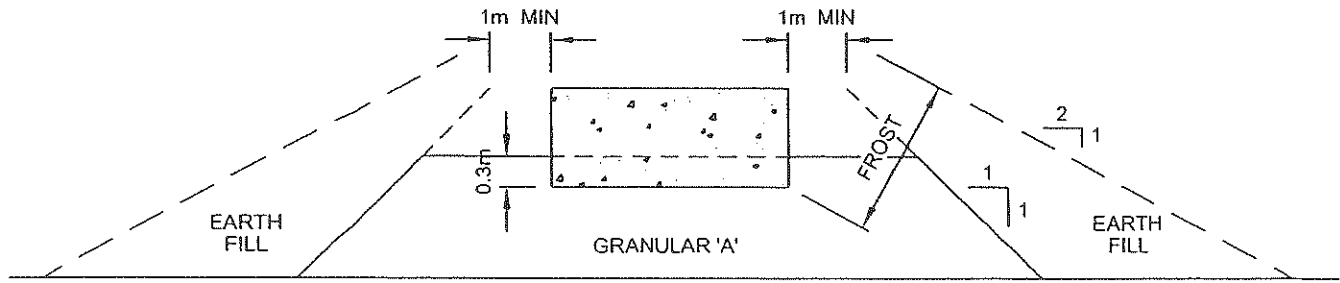
### **Foundation Comparison**

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

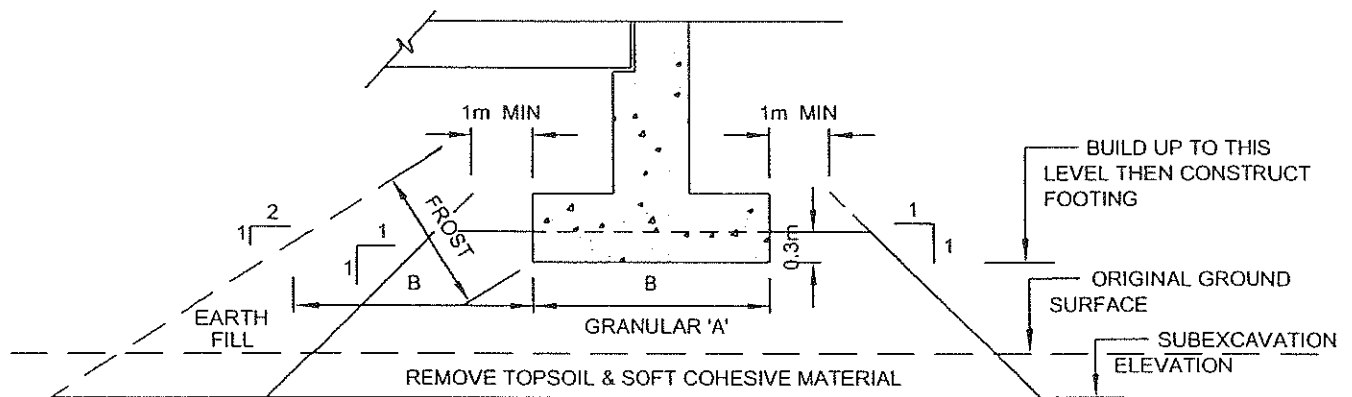
Foundation Element	Spread Footings	Spread Footings on Engineered Fill	Driven Piles
Abutments	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. Generally less costly construction than deep foundation elements.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Lower geotechnical resistance available due to founding on compact soils near the surface.</li> <li>ii. Dewatering may be required, depending on depth of excavation.</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. Generally less costly construction than deep foundation elements.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Better geotechnical resistance than spread footings on native, but still influenced by the compact soils at the surface.</li> <li>ii. Dewatering may be required, depending on depth of excavation.</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. High geotechnical resistance may be developed by driving the piles into very dense soils.</li> <li>ii. Comparatively short abutment stem possible</li> <li>iii. Permits integral abutment design</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Higher unit cost compared to footings.</li> <li>ii. Very dense soils at shallow depth will limit length of pile and geotechnical resistance that can be developed.</li> </ul>
	<b>NOT RECOMMENDED</b>	<b>NOT RECOMMENDED</b>	<b>RECOMMENDED</b>

## Appendix D

### Figure



CROSS-SECTION



LONGITUDINAL SECTION

NOT TO SCALE

NOTES:

1. REMOVE TOPSOIL AND SOFT SILTY CLAY SUBSOIL UNDER FOOTPRINT OF COMPACTED GRANULAR 'A'.
2. PLACE GRANULAR 'A' AND EARTH FILL TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO O.P.S.S. 501.
3. CONSTRUCT CONCRETE FOOTING.
4. PLACE REMAINDER OF GRANULAR 'A' AND EARTH FILL AS REQUIRED.
5. SOURCE M.T.C. 1982.

ENGINEER	AEG
DRAWN	SS
DATE	April , 2004
APPROVED	PKC
SCALE	NTS

ABUTMENT ON COMPACTED FILL SHOWING  
GRANULAR A CORE



DWG. NO.

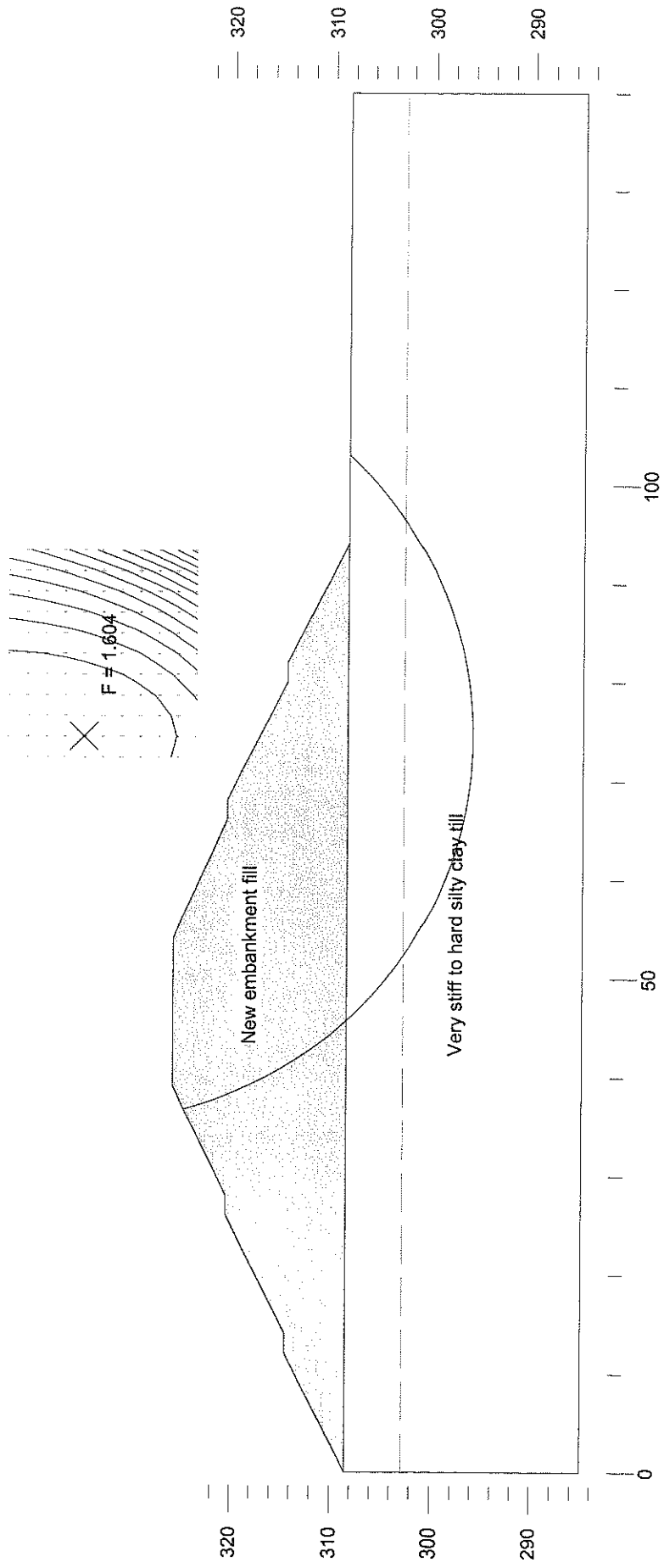
FIGURE 1

## **Appendix E**

### **Slope Stability Output**

Thurber Engineering Ltd. - Toronto  
 15-64-17 Highway 7 - New  
 N-E Entrance Ramp to Hwy 7 over Guelph Road  
 August 27, 2008  
 North and South Abutments  
 17.0 m high

Earth Fill	Gamma C	Phi	Piezo
Silty Clay Till	kN/m <sup>3</sup>	deg	Surf.
	21	30	1
	18.5	0	1
		100	



Thurber Engineering Ltd. - Toronto  
 15-64-17 Highway 7 - New  
 N-E Entrance Ramp to Hwy 7 over Guelph Road  
 August 27, 2008  
 North and South Abutments  
 17.0 m high

Earth Fill	Gamma C	Phi	Piezo
	kN/m <sup>3</sup>	deg	Surf.
Silty Clay Till	21	30	1
	18.5	0	1
Seismic coefficient = 0.08			

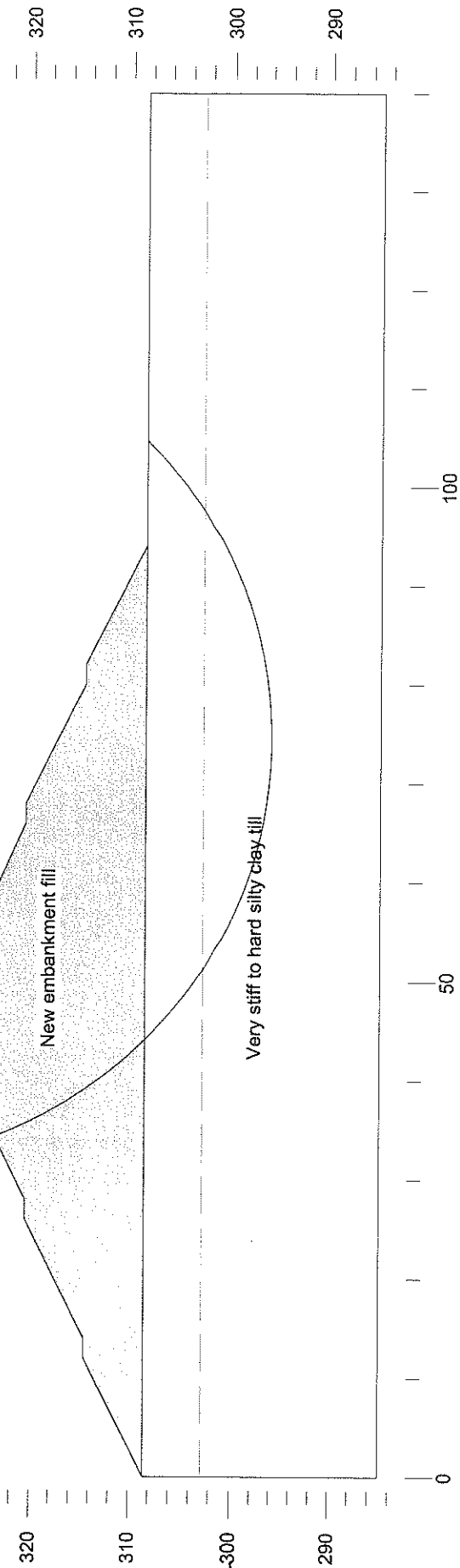
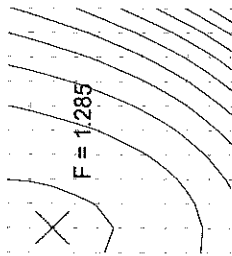


FIGURE 2



N-E ramp/N-Wellington Ramp over Guelph St.  
Highway 7-New, Kitchener to Guelph

---

## **Appendix F**

### **Site Photograph**

N-E ramp/N-Wellington Ramp over Guelph St.  
Highway 7-New, Kitchener to Guelph

---



Photo 1. Looking at the south side of Guelph Street, east of KWE. Borehole 08-004

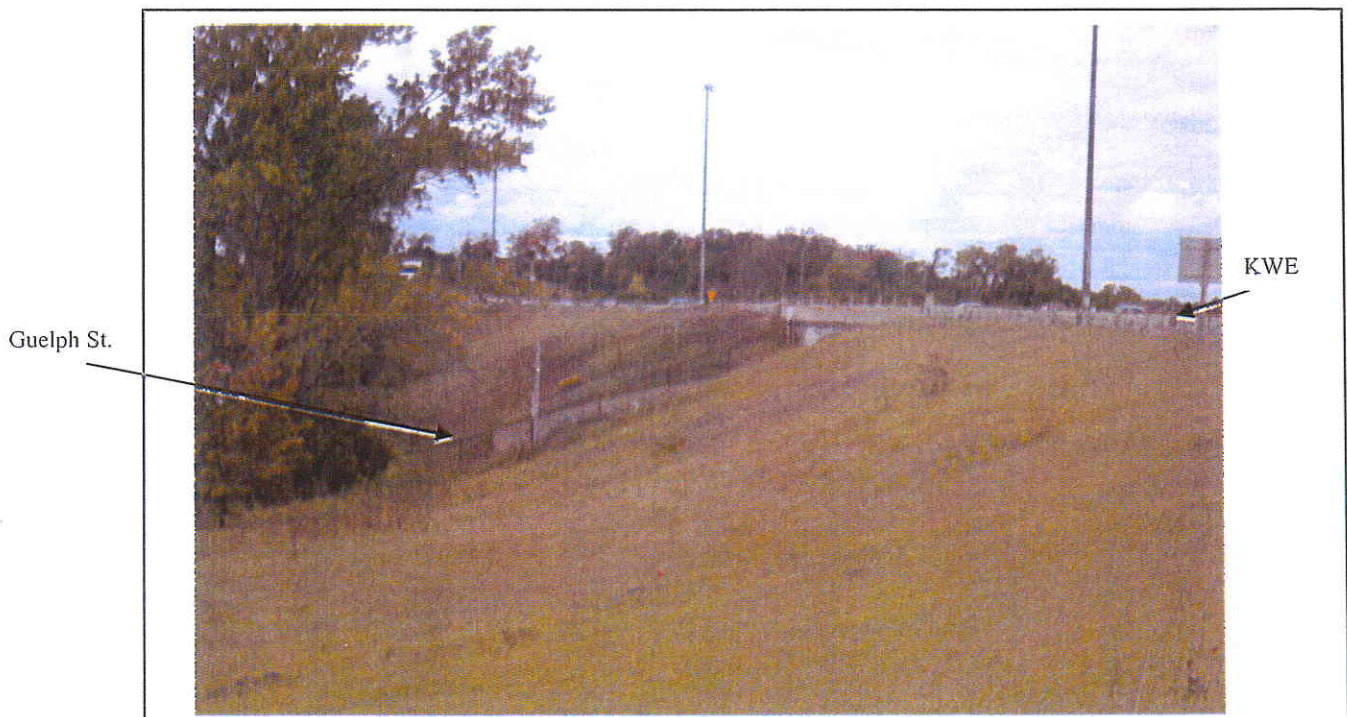


Photo 2. Looking at the south side of Guelph Street, east of KWE. Borehole 08-004

N-E ramp/N-Wellington Ramp over Guelph St.  
Highway 7-New, Kitchener to Guelph

---

## **Appendix G**

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

