

**PRELIMINARY
FOUNDATION INVESTIGATION AND DESIGN REPORT
N-E RAMP OVER E-S RAMP
AND KITCHENER WATERLOO EXPRESSWAY
HIGHWAY 7-NEW, KITCHENER TO GUELPH
G.W.P. 408-88-00**

Geocres Number: 40P8-160

Report to

**Ministry of Transportation Ontario
Southwestern Region**

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

June 2, 2009
File: 15-64-17

\\Torserver1\Projects\15\64\17 Hwy 7 New\Reports &
Memos\Structures\N-E Ramp over E-S Ramp-KWE -
14&20\15-64-17 NE Ramp over E-S Ramp & KWE FIDR-
FINAL.doc

TABLE OF CONTENTS

PART 1 FACTUAL INFORMATION

1	INTRODUCTION	1
2	SITE DESCRIPTION	2
3	SITE INVESTIGATION AND FIELD TESTING	2
4	LABORATORY TESTING	3
5	DESCRIPTION OF SUBSURFACE CONDITIONS	3
5.1	Topsoil	3
5.2	Fill	4
5.3	Silty Sand and Sandy Silt	4
5.4	Sand	5
5.5	Upper Silty Clay	5
5.6	Upper Sandy Silt Till	6
5.7	Lower Silty Clay	7
5.8	Lower Sandy Silt Till	7
5.9	Groundwater Conditions	8
6	MISCELLANEOUS	9

PART 2 ENGINEERING DISCUSSION AND RECOMMENDATIONS

7	GENERAL	11
8	STRUCTURE FOUNDATIONS	11
8.1	Spread Footings on Native Soil	12
8.2	Spread Footings on Engineered Fill	13
8.3	Steel H-Piles	14
8.3.1	Axial Resistance	14
8.3.2	Downdrag	15
8.4	Augered Caissons on Very Dense Soils (Drilled Shafts)	15
8.5	Abutment Design Considerations	16
8.6	Frost Cover	16
8.7	Recommended Foundation	16
9	BRIDGE APPROACHES AND EMBANKMENTS	16
10	CONSTRUCTION CONCERNS	17

11	INVESTIGATION FOR DETAIL DESIGN.....	18
12	CLOSURE.....	19

Appendices

Appendix A	Record of Borehole Sheets
Appendix B	Laboratory Test Results
Appendix C	Record of Borehole Sheet (Previous Investigation)
Appendix D	Foundation Comparison
Appendix E	Figure
Appendix F	Site photograph
Appendix G	Slope Stability Output
Appendix H	Drawing titled “Borehole Locations and Soil Strata”

**PRELIMINARY
FOUNDATION INVESTIGATION AND DESIGN REPORT
N-E RAMP OVER E-S RAMP
AND KITCHENER WATERLOO EXPRESSWAY
HIGHWAY 7-NEW, KITCHENER TO GUELPH
G.W.P. 408-88-00**

Geocres Number: 40P8-160

PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a preliminary foundation investigation conducted at the site of a proposed high level ramp structure associated with the Highway 7-New/Kitchener Waterloo Expressway (KWE) Interchange. The structure will carry the KWE N-E Hwy 7 Ramp over the Hwy 7 E-S KWE Ramp, Wellington Street and the KWE,

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.

In the preparation of this report, general reference has been made to information on subsurface conditions contained in a previous foundation report. The title of the report is listed as follows:

- Foundation investigation report for proposed Wellington Street Underpass Kitchener-Waterloo Expressway, District #4 (Hamilton), W.J. 66-F-43, W.P. 637-64, Geocres Number 40P8-46, dated August 5, 1966. (Reference 1).

Records of Boreholes 5 and 9 from the previous report are attached in Appendix C for reference.

2 SITE DESCRIPTION

The site lies within the existing Kitchener Waterloo Expressway and Wellington Street interchange in Waterloo, Ontario. At this location, the proposed N-E entrance ramp will cross over existing Wellington Street, Kitchener Waterloo Expressway and the proposed E-S Ramp.

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.

Photographs of the site, looking at the borehole locations 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 July 9 to 11, 16 and 17, 2008. Two boreholes, numbered 08-014 and 08-020, were drilled approximately at the north and south abutments of a possible single-span structure arrangement. The depths of Boreholes 08-014 and 08-020 were 29.1 m and 24.5 m (Elevations 290.3 and 295.5), respectively.

A previous geotechnical investigation was conducted in May 1966. Boreholes 5 and 9 were drilled in close proximity to the proposed structure and were terminated at 12.6 m and 9.5 m (Elevations 306.8 and 310.1), respectively.

The Record of Borehole sheets for the current and previous boreholes are included in Appendices A and C, respectively.

The approximate locations of the boreholes drilled during the present and previous investigations are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix H.

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 track-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 08-020, drilled at the proposed south abutment, a standpipe piezometer consisting of 25 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 08-014 was grouted with grout 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	08-014	No Installation	Borehole backfilled with grout to surface.
South abutment	08-020	24.4/295.6	Piezometer with 1.5 m slotted screen installed with sand filter to 21.0 m, holeplug from 21.0 m to 20.4 m, grout from 20.4 m to 0.2 m, then auger cuttings 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.

Laboratory test results of Boreholes 5 and 9 drilled during the previous investigation (Reference 1) are enclosed in Appendix C.

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 H. An overall description of the stratigraphy is given in the following paragraphs. However, the factual data presented in the Record of Borehole Sheets governs any interpretation of the site conditions.

In general, the site is underlain by topsoil and silt and sand fill overlying native sand, silty sand/sandy silt, silty clay and sandy silt till.

5.1 Topsoil

Topsoil was encountered surficially in the four boreholes. Thickness of topsoil was 125 mm and 600 mm in Boreholes 08-014 and 08-020, respectively. The topsoil thickness may vary between and beyond the borehole locations and the data is not intended for the purpose of estimating quantities

5.2 Fill

Fill was encountered below the topsoil in Boreholes 08-014 and 08-020. The fill consists of brown silty sand and sandy silt containing trace to some gravel and occasional roots. Thickness of the fill was 2.1 m and 1.6 m in Boreholes 08-014 and 08-020, respectively. The depth of the base of the fill extended to 2.2 m (Elevations 317.2 and 317.8).

The fill is classified as loose to dense, based on SPT 'N' values ranging from 8 to 42 blows for 0.3 m of penetration. The natural moisture content ranged from 10 to 19%.

5.3 Silty Sand and Sandy Silt

Layers of silty sand and sandy silt containing trace gravel were contacted in Boreholes 5 and 9 drilled previously at depths and elevations shown in Table 5.1.

Table 5.1 – Depths and Elevations of Native Silty Sand and Sandy Silt

Foundation Unit	Borehole	Depth below existing ground surface (m)	Elevation (m)	Thickness (m)
Pier	5	5.0 to 9.1	314.4 to 310.3	4.1
	9	0.0 to 3.7 5.0 to 9.5 (borehole termination depth)	319.7 to 316.0 314.6 to 310.1	3.7 4.5

The silty sand and sandy silt layers are described as compact to very dense based on SPT 'N' values ranging from 12 to 72 blows per 0.3 m of penetration.

An SPT 'N' value of 4 blows per 0.3 m of penetration, indicating a loose density was measured in Borehole 5 near elevation 313.9.

In Borehole 9, SPT 'N' values higher than 98 blows per 0.15 m of penetration were measured near elevation 311.5. The moisture content varied from 9% to 29%.

Grain size distribution curves for the silty sand and sandy silt layers are presented in Appendix C.

The results of the laboratory tests are summarized as follows:

Soil Particles	(%)
Gravel	2 to 7
Sand	20 to 95
Silt	29 to 75
Clay	3 to 11
Clay & Silt	5 to 27

5.4 Sand

Native sand containing trace of silt was contacted below the topsoil in Borehole 5. Thickness of the sand layer was 5.0 m. The depth to the base of the sand is 5.0 m (Elevation 314.4).

A layer of native grey sand containing trace silt was encountered at 19.2 m (Elevation 300.8) in Borehole 08-020. Thickness of the sand layer was 2.2 m. The depth to the base of the sand is 21.4 m (Elevation 298.6).

The sand is classified as loose to very dense, based on an SPT 'N' values of 7 to 52 blows for 0.3 m of penetration. The natural moisture content ranged from 8% to 17%.

Grain size distribution curves for two sand samples tested are presented on the Record of Borehole sheets and on Figure B1 of Appendix B.

Laboratory test results of previous investigation are presented in Appendix C.

The results of the laboratory tests are summarized as follows:

Soil Particles	(%)
Gravel	0 to 7
Sand	77 to 93
Silt	36
Clay	12
Silt & Clay	7 to 18

5.5 Upper Silty Clay

An upper layer of native brown to grey silty clay containing trace sand and trace gravel was contacted below the fill and silty sand at depths and elevations as indicated in Table 5.2.

Table 5.2 – Depths and Elevations of Native Upper Silty Clay

Foundation Unit	Borehole	Depth below existing ground surface (m)	Elevation (m)	Thickness (m)
North abutment	08-014	2.2 to 7.2	317.2 to 312.2	5.0
Pier	9	3.7 to 5.0	316.0 to 314.6	1.4
South abutment	08-020	2.2 to 4.7	317.8 to 315.3	2.5

The upper silty clay layer is very stiff to hard in consistency, based on SPT 'N' values ranging from 20 to 32 blows per 0.3 m of penetration. The moisture content varied from 19% to 22%.

Grain size distribution curves for selected upper silty clay samples are presented on the Record of Borehole sheets and on Figure B2 Appendix B. Atterberg Limits test results are presented on Figure B4 of Appendix B.

Laboratory test results of previous investigation are presented in Appendix C.

The results of the laboratory tests are summarized as follows:

Soil Particles	(%)
Gravel	0
Sand	5 to 10
Silt	26 to 42
Clay	56 to 64

Liquid Limit	39 to 42
Plastic Limit	18 to 20

The above results show that the upper silty clay is of medium plasticity with a group symbol of CI.

5.6 Upper Sandy Silt Till

Brown to grey upper sandy silt till containing trace gravel and trace clay was encountered in both boreholes at depths and elevations indicated in Table 5.3.

Table 5.3 – Depths and Elevations of Native Upper Sandy Silt Till

Foundation Unit	Borehole	Depth below existing ground surface (m)	Elevation (m)	Thickness (m)
North abutment	08-014	7.2 to 10.4	312.2 to 309.0	3.2
South abutment	08-020	4.7 to 9.5	315.3 to 310.5	4.8

SPT ‘N’ values measured in the sandy silt till ranged from 32 blows per 0.3 m of penetration to higher than 100 blows per 0.225 m of penetration, indicating a dense to very dense relative density. The natural moisture contents generally lay in the range of 10 to 21%.

Grain size distribution curves for selected samples are presented on the Record of Borehole sheets and on Figure B3 of Appendix B. The results of the laboratory tests carried out on sandy silt till samples are summarized as follows:

Soil Particles	(%)
Gravel	0
Sand	14 to 47
Silt	47 to 78
Clay	4 to 8

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.

5.7 Lower Silty Clay

Native grey silty clay containing trace to some sand, trace gravel and occasional sand and silt seams was contacted at lower depths as indicated in Table 5.4.

Table 5.4 – Depths and Elevations of Native Lower Silty Clay

Foundation Unit	Borehole	Depth below existing ground surface (m)	Elevation (m)	Thickness (m)
North abutment	08-014	10.4 to 25.1	309.0 to 294.2	14.8
Pier	5	9.1 to 12.6 (borehole termination depth)	310.3 to 306.8	>3.5
South abutment	08-020	9.5 to 19.2	310.5 to 300.8	9.7

The cohesive layer is very stiff to hard in consistency, based on SPT 'N' values ranging from 29 to 89 blows per 0.3 m of penetration. In Borehole 08-020 at 10.8 m depth (Elevation 309.2), an SPT 'N' value was 8 blows per 0.3 m of penetration, indicating a firm consistency. The moisture content varied from 11% to 28%.

Grain size distribution curves for selected samples of the silty clay are presented on the Record of Borehole sheets and on Figure B1 Appendix B. Atterberg Limits test results are presented on Figure B4 of Appendix B.

Laboratory test results of previous investigation are presented in Appendix C.

The results of the laboratory tests are summarized as follows:

Soil Particles	(%)
Gravel	0
Sand	1 to 4
Silt	29 to 42
Clay	54 to 69

Liquid Limit	39 to 48
Plastic Limit	18 to 20

The above results show that the lower silty clay is of medium plasticity with a group symbol of CI.

5.8 Lower Sandy Silt Till

Grey sandy silt till containing trace gravel, trace to some clay and occasional cobbles was encountered at lower depths and elevations in both boreholes, as indicated in Table 5.5.

Table 5.5 – Depths and Elevations of Native Lower Sandy Silt Till

Foundation Unit	Borehole	Depth below existing ground surface (m)	Elevation (m)	Thickness (m)
North abutment	08-014	25.1 to 29.1** (Borehole termination depth)	294.2 to 290.3	4.0+
South abutment	08-020	21.4 to 24.5 ** (Borehole termination depth)	298.6 to 295.5	3.1+

SPT ‘N’ values measured in the lower sandy silt till were higher than 100 blows per 0.125 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 selected samples are presented on the Record of Borehole sheets and on Figure B3 of Appendix B. The results of the laboratory tests carried out on sandy silt till soil samples are summarized as follows:

Soil Particles	(%)
Gravel	3
Sand	39 to 41
Silt	43
Clay	13 to 15

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.

5.9 Groundwater Conditions

Water levels were observed in the boreholes during and upon completion of drilling. A standpipe piezometer was installed in Borehole 08-020 (south abutment) to monitor water levels after completion of drilling. The water levels measured in the piezometer are summarized in Table 5.6.

Table 5.6 – Water Level Measurements

Foundation Unit	Borehole	Date	Water Level (m)		Comment
			Depth	Elevation	
South abutment	08-020	July 15, 2008	14.6	305.4	In piezometer
		August 20, 2008	14.4	305.6	
Pier	5	May 9, 1966	4.5	314.9	Open borehole
	9	May 10, 1966	3.1	316.6	Open borehole

The piezometric reading indicates that the groundwater level is near Elevation 305.6 m.

Localized groundwater may also be encountered in the upper sandy silt till.

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 track-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.

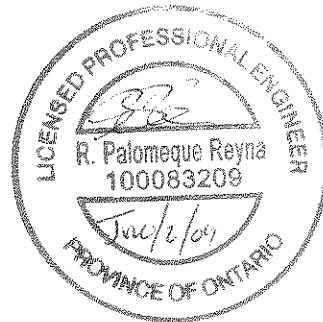
The coordinates for the boreholes and the ground surface elevations were obtained by Thurber Engineering Ltd. using GPS equipment.

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.

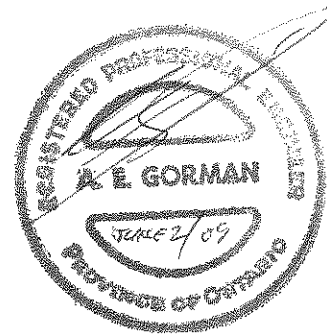
Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations projects, reviewed the report.

Thurber Engineering Ltd.

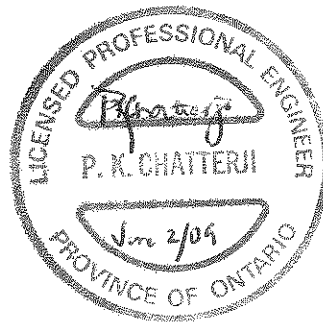
Rocio Palomeque Reyna, P.Eng.
Geotechnical Engineer



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 OVER E-S RAMP
AND KITCHENER WATERLOO EXPRESSWAY
HIGHWAY 7-NEW, KITCHENER TO GUELPH
G.W.P. 408-88-00**

Geocres Number: 40P8-160

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 N-E Ramp to Highway 7 will cross over E-S Ramp from Highway 7, Kitchener Waterloo Expressway and existing Wellington Street. The proposed finished grade of the N-E ramp will be near Elevation 341.0. The existing ground surface within the proposed ramp structure is near Elevations 319.4 to 320.0 at the north and south abutments, respectively. Hence, N-E ramp embankments will potentially be 21.0 m to 22.0 m high relative to the existing surrounding grade.
- The new grade of proposed E-S ramp will be at Elevation 333.0.
- Wellington Street grade will be approximately at Elevation 324.0 at this location.
- It has been assumed that the proposed structure will have at least two spans, with a pier located at the centreline of KWE and centreline of Wellington Street.

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 topsoil and fill overlying native loose to dense sand and sandy silt/silty sand layers, upper and lower layers of very stiff to hard silty clay and upper and lower deposits of dense to very dense sandy silt till. The groundwater level measured in the piezometer was 14.4 m below the ground surface (Elevation 305.6).

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
- Augered Caissons on Very Dense Sandy Silt Till (Drilled Shafts)

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

8.1 Spread Footings on Native Soil

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

The existing fill encountered in Boreholes 08-014 and 08-020 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 or dense to very dense sandy silt 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 _r (kPa)	SLS (kPa)	Soil
North abutment (BH 08-014)	2.2	317.2	300	200	Very stiff silty clay till
	7.6	311.8	600	400	Dense sandy silt till
Pier (BH 5)	0.9	318.5	300	200	Compact sand
	7.3 or below	312.2 or below	600	400	Very dense sandy silt
Pier (BH 9)	1.1	318.5	300	200	Compact silty sand
	5.7	313.9	600	400	Very dense to dense sandy silt/silty sand
South abutment (BH 08-020)	2.4	317.6	450	300	Hard silty clay till
	4.7	315.3	525	350	Dense sandy silt till
	7.6	312.4	600	400	Very dense silty sand 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 above groundwater level measured in the piezometer. However, localized groundwater may be encountered, especially in the upper sandy silt till. Accordingly, the contract must contain provision for groundwater control.

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, all topsoil, or other deleterious materials must be stripped from the footprint of the foundation to expose competent native subgrade material. Subexcavation of existing surficial fill soils will be required. The engineered fill should bear on native very stiff to hard silty clay and the highest permitted founding elevations at which engineered fill pads may be placed, are given in Table 8.2.

Table 8.2 – Highest Founding Depths and Elevations for Engineered Fill Pad

Element	Borehole	Depth (m)	Elev.
North abutment	08-014	2.2	317.2
Pier	5	0.9	318.5
	9	1.1	318.5
South abutment	08-020	2.2	317.8

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. 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 E.

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 lower sandy silt till and sandy silt/silty sand 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	Borehole	Pile Tip Depth (m)	Highest Pile Tip Elevation
North abutment	08-014	26.9	292.5
Pier	9	9.4	310.3
South abutment	08-020	22.5	297.5

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.

8.3.2 Downdrag

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

8.4 Augered Caissons on Very Dense Soils (Drilled Shafts)

Drilled shaft foundations are also suitable for the support of structural loads at this site. Since the caisson is a deep foundation unit, higher geotechnical resistance is available from a caisson in earth than from a similar sized spread footing.

The caissons must be founded in the very dense sandy silt till or sandy silt/silty sand at depths and elevations indicated in Table 8.5. Caisson lengths should be at least 6.0 m and each caisson should extend at least 3.0 m into the “100-blow” soils.

Table 8.5 – Founding Elevations for Augered Caissons

Foundation Unit	Borehole	Founding Depth (m)	Founding Elevation
North abutment	08-014	28.9	290.5
Pier	9	10.8*	308.8*
South abutment	08-020	24.2	295.8

*Additional drilling investigation should be conducted at the pier to confirm the depth and elevation of caisson founding.

Typical preliminary geotechnical resistance has been calculated at the abutments for a range of caisson diameters and founding depths given in Table 8.5. The values are shown in Table 8.6.

Table 8.6 – Vertical Geotechnical Resistance for Caisson Foundations

Caisson Diameter (m)	Typical Preliminary Axial Geotechnical Resistance	
	Factored ULS _r (kN)	SLS (kN)
1.0	6,000	4,500
1.5	10,000	8,000

The vertical geotechnical resistance for caisson foundations has been calculated assuming contributions from factored sidewall resistance in the sockets.

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

The soil providing the resistance, whether it is skin friction or end bearing, must be protected from disturbance.

The caisson installation equipment should be able to dislodge and remove any obstructions in the fill or cobbles and boulders in the till.

The overburden soil at this site is predominantly permeable cohesionless soils under water table and contains cobbles and boulders. These conditions will make caisson installation difficult and sealing of caisson liners into the founding stratum to exclude groundwater will also be difficult.

8.5 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. Integral abutment design will require the use of steel H-piles.

8.6 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, or an equivalent combination of soil cover and extruded polystyrene (EPS) insulation. A 25 mm thickness of EPS is equivalent to 600 mm of soil cover.

8.7 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 lower sandy silt till, despite the higher cost noted in Appendix C. At the assumed pier location, taking account of constructability, a caisson foundation may be the most practical, though driven piles are also feasible.

9 BRIDGE APPROACHES AND EMBANKMENTS

Based on the current and previous boreholes drilled at the site, the approach embankments will be constructed over very stiff to hard silty clay and compact to dense sand and silty sand foundation and may incorporate the sand fill of the existing embankment.

Preliminary analysis indicates that at the abutments, settlement in the order of 85 mm to 95 mm is estimated in the foundation soils under the loading imposed by approximately 22.0 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 21.0 m to 22.0 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 2H:1V SSM or earth fill embankments. The stability of the embankments 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 G.

Table 9.1 Computed Factors of Safety

Location / Material	Condition	Factor of Safety	Figure (Appendix G)
22 m High			
Earth Fill	Normal	1.3	1
Earth Fill	Seismic = 0.08g	1.1	2

These factors of safety are considered to be acceptable for the proposed embankment bearing on cohesive foundation soils.

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

During detail design, when the grade has been finalized, permanent drainage 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. Destabilization of excavations

If excavation is carried out in cohesionless soil without prior implementation of adequate measures to control groundwater and surface water, there is a risk that the sides and/or base of the excavation will be destabilized. This could lead to a risk to personnel working on site, or to a loss of bearing resistance in the soil.

Accordingly, it must be emphasized to the contractor that proper groundwater and surface water control measures must be in place prior to commencing excavation.

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 E-S entrance ramp to Highway 7 over E-S exit ramp from Highway 7 and Kitchener Waterloo Expressway alignment and thus removed from the alignment of the current investigation. Particular attention should be paid to groundwater levels and exploration off the existing road embankment is recommended.

2. Groundwater

Investigation during detail design must address groundwater levels in the upper sand silt till.

3. Boreholes for approaches.

A minimum of one borehole is recommended in each approach 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.

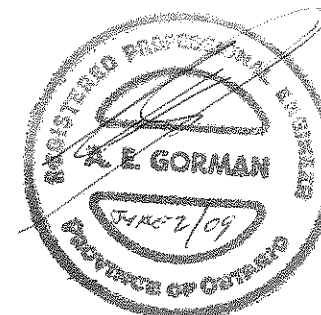
The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

Thurber Engineering Ltd.

Rocío 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

RECORD OF BOREHOLE No 08-014

1 OF 3

METRIC

G.W.P. 408-88-00 LOCATION N 4 814 412.13 E 226 078.93 ORIGINATED BY SLL
 HWY 7 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SA
 DATUM Geodetic DATE 2008.07.16 - 2008.07.17 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa			WATER CONTENT (%)					
								○ UNCONFINED	+ FIELD VANE		○					
								● QUICK TRIAXIAL	x LAB VANE							
319.4							20	40	60	80	100	20	40	60		
0.0	TOPSOIL: (125mm), with roots and rootlets															
0.1	Sandy SILT, trace gravel, occasional rootlets and organics Loose Brown Moist (FILL)		1	SS	8								○			
	Dense		2	SS	38								○			
317.2																
2.2	Silty CLAY, trace gravel Very Stiff Brown		3	SS	20								○			
			4	SS	22								○	—		0 5 39 56
			5	SS	27								○			
			6	SS	32								○			
	Sand seams Hard															
312.2																
7.2	Sandy SILT, trace clay Dense to Very Dense Brown to Grey Wet (TILL)		7	SS	40								○			0 47 47 6
			8	SS	100/ 225								○			0 34 62 4

Continued Next Page

+ ³ × ³ Numbers refer to
Sensitivity

20
15-5
10

(%) STRAIN AT FAILURE

METRIC

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE			"N" VALUES						20 40 60 80 100
							SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100						
Continued From Previous Page							WATER CONTENT (%) 20 40 60	kN/m ³		GR SA SI C			

[illegible]

+ 3, X 3. Numbers refer to Sensitivity

RECORD OF BOREHOLE No 08-014

3 OF 3

METRIC

G.W.P. 408-88-00 LOCATION N 4 814 412.13 E 226 078.93 ORIGINATED BY SLL
 HWY 7 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SA
 DATUM Geodetic DATE 2008.07.16 - 2008.07.17 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL			
	Continued From Previous Page						20 40 60 80 100						
	Silty CLAY, trace gravel, trace sand Hard Grey		15	SS	39								
	Dark Grey		16	SS	89								
	some gravel												
294.2													
25.1	Sandy SILT, trace gravel, some clay Very Dense Grey Moist (TILL)		17	SS	100/ .125								

ONTMT4S 6417R.GPJ 9/11/08

METRIC

+ 3, × 3: Numbers refer to Sensitivity

ONTMT4\$ 6417R.GPJ 9/10/08

METRIC

DATUM Geodetic DATE 2008.07.09 - 2008.07.11 CHECKED BY RPR

Continued Next Page

+ 3, X 3: Numbers refer to Sensitivity

ONTMT4S 6417R.GPJ 9/10/08

RECORD OF BOREHOLE No 08-020

3 OF 3

METRIC

G.W.P. 408-88-00 LOCATION N 4 814 380.98 E 226 219.28 ORIGINATED BY SLL
 HWY 7 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SA
 DATUM Geodetic DATE 2008.07.09 - 2008.07.11 CHECKED BY RPR

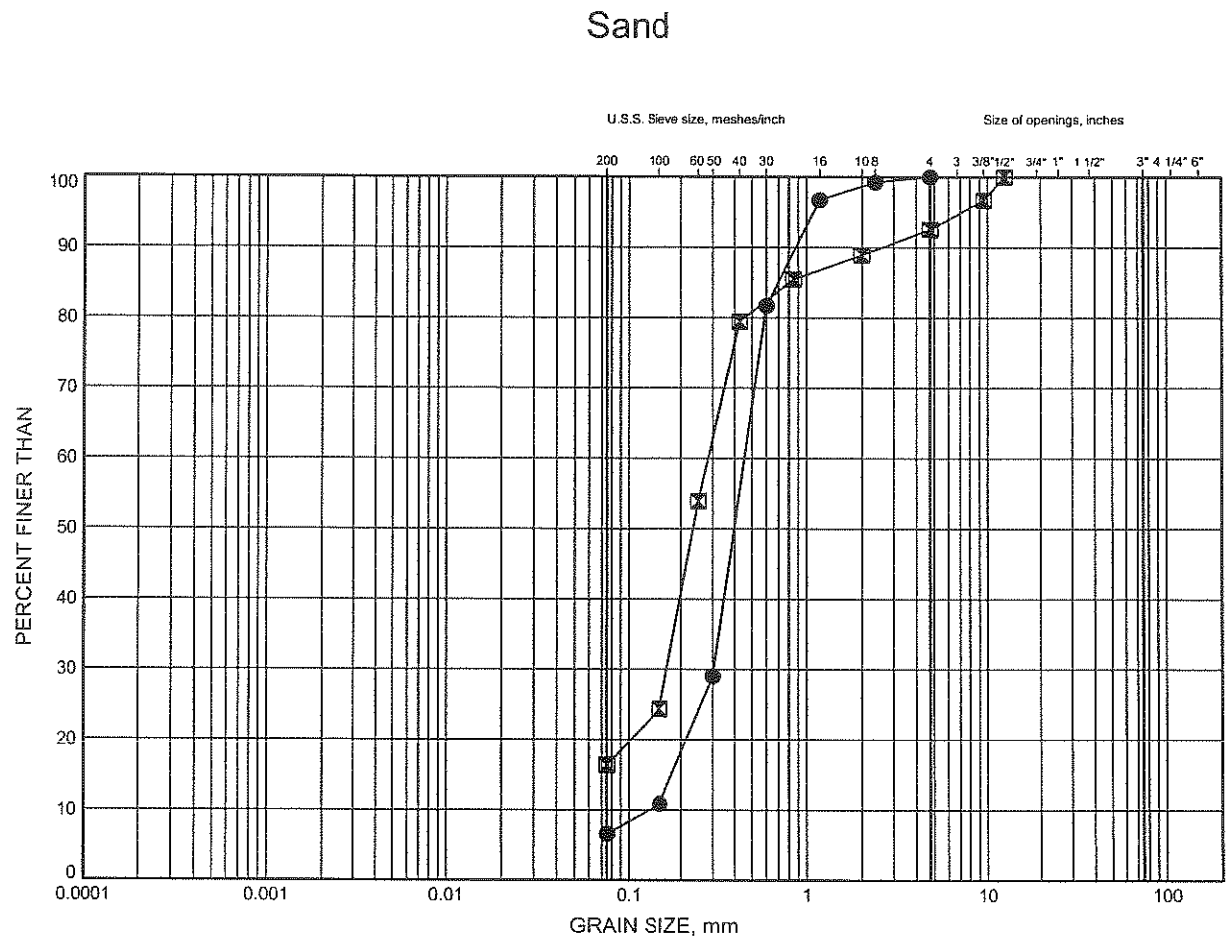
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				
	Continued From Previous Page							20 40 60 80 100				
								○ UNCONFINED + FIELD VANE				
								● QUICK TRIAXIAL × LAB VANE				
								WATER CONTENT (%)				
								20 40 60				
								PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				
								W _p W W _L				

Appendix B

Laboratory Test Results

Highway 7 - New GRAIN SIZE DISTRIBUTION

FIGURE B1



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	08-020	20.12	299.90
⊠	08-020	21.41	298.60

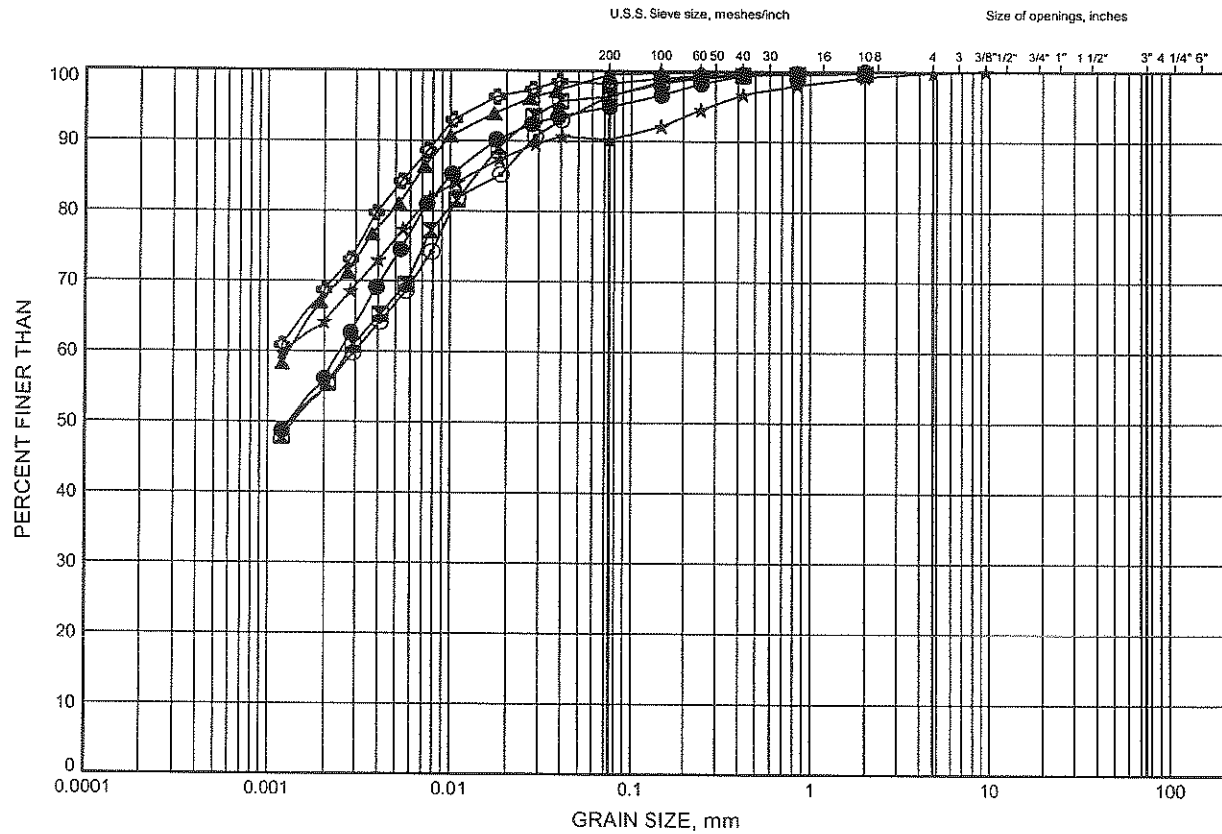


W.P.# .408-88-00.....
 Prepared By .AN.....
 Checked By .RPR.....

Highway 7 - New GRAIN SIZE DISTRIBUTION

FIGURE B2

Silty Clay



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	08-014	3.35	316.04
⊠	08-014	14.02	305.37
▲	08-014	18.59	300.80
★	08-020	3.35	316.66
⊙	08-020	12.50	307.52
⊗	08-020	17.07	302.94

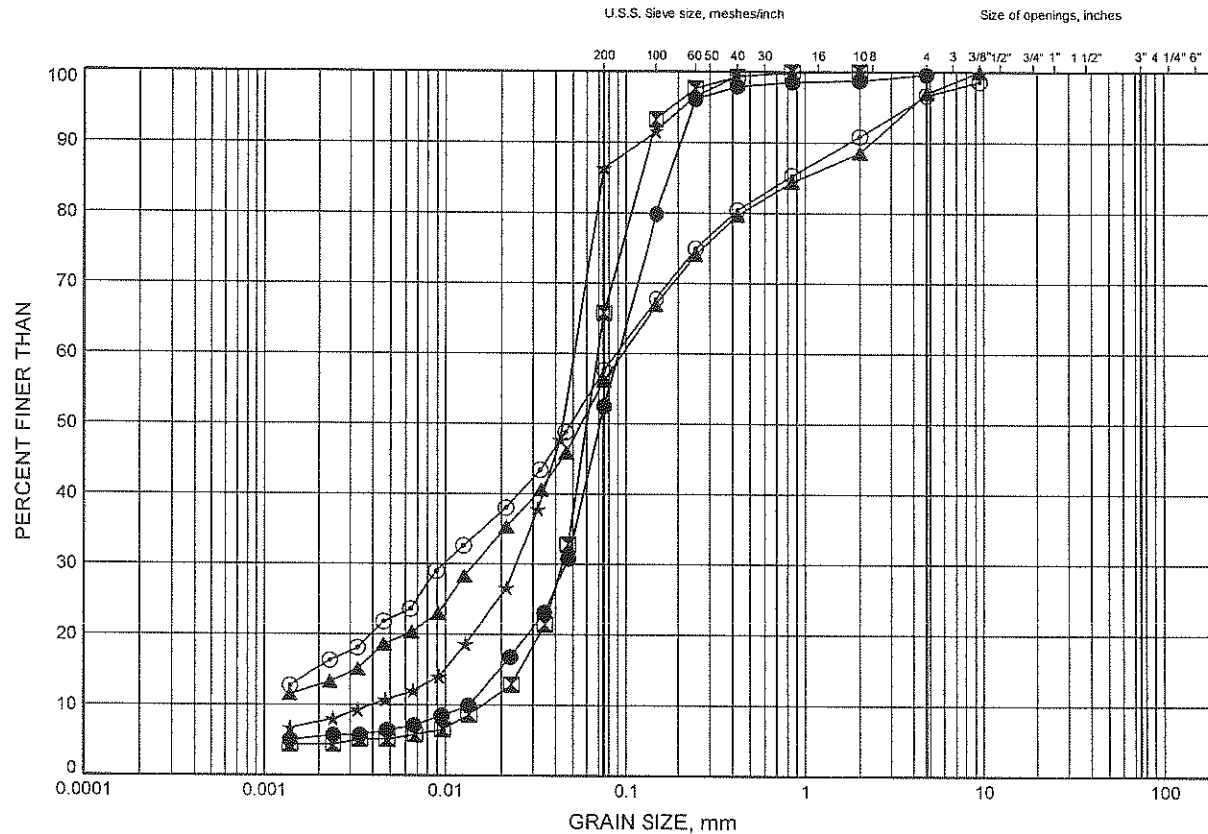


W.P.# 408-88-00.....
Prepared By AN.....
Checked By RPR.....

Highway 7 - New GRAIN SIZE DISTRIBUTION

FIGURE B3

Sandy Silt Till



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	08-014	7.90	311.49
⊠	08-014	9.40	309.99
▲	08-014	25.97	293.42
☆	08-020	6.40	313.61
⊙	08-020	24.46	295.55

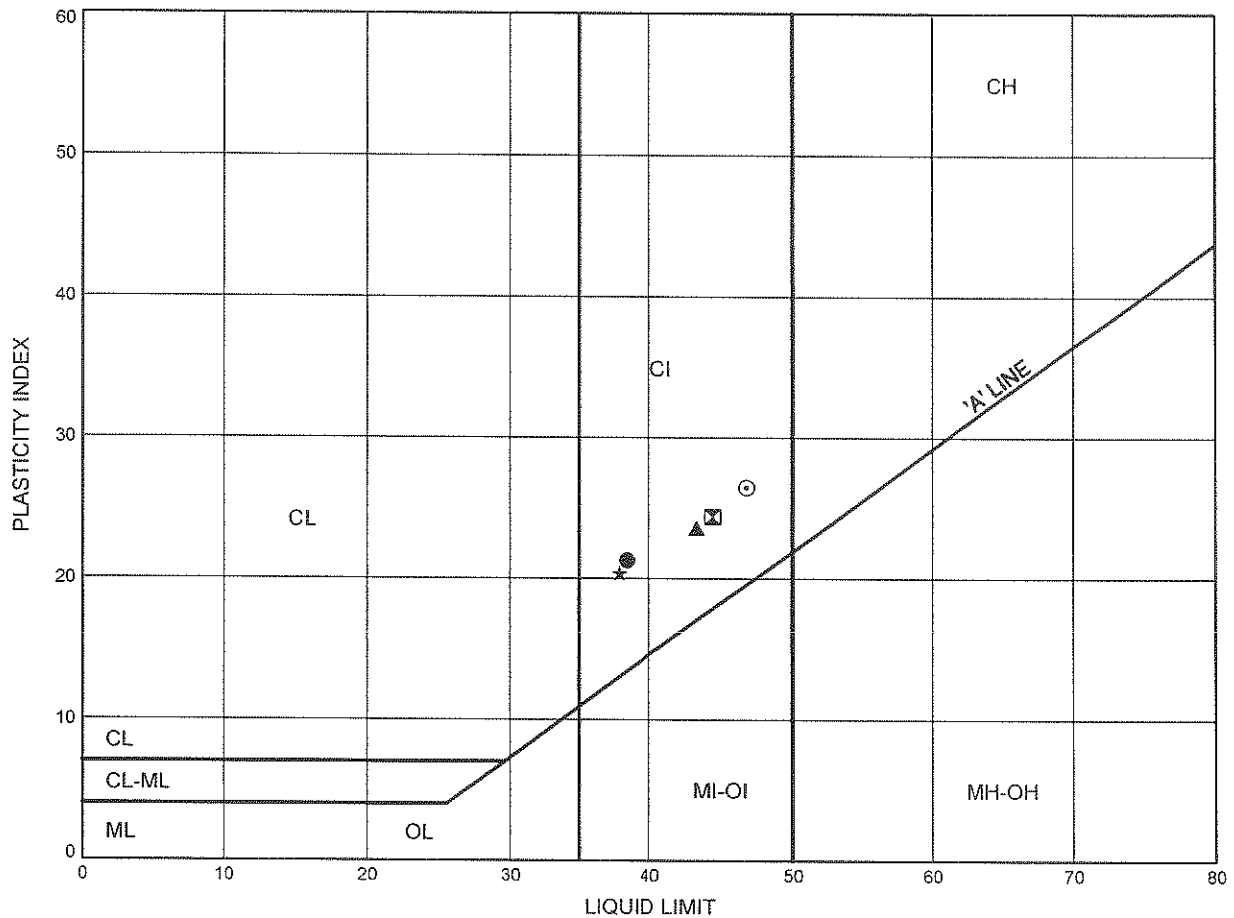


W.P.# 408-88-00
Prepared By MFA
Checked By RPR

Highway 7 - New ATTERBERG LIMITS TEST RESULTS

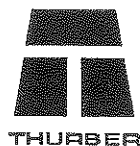
FIGURE B4

Silty Clay



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	08-014	3.35	316.04
⊠	08-014	18.59	300.80
▲	08-020	3.35	316.66
★	08-020	12.50	307.52
⊙	08-020	17.07	302.94

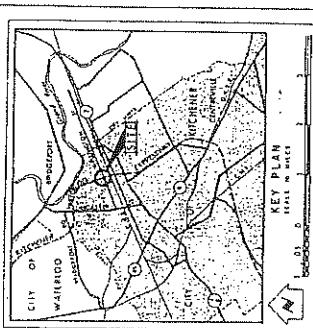
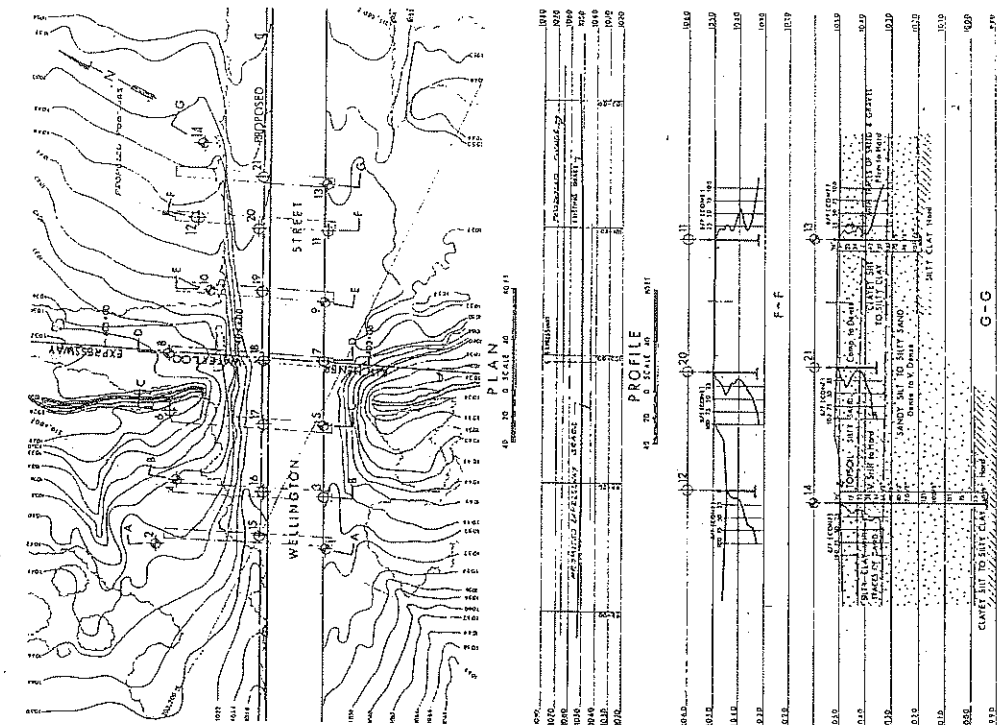
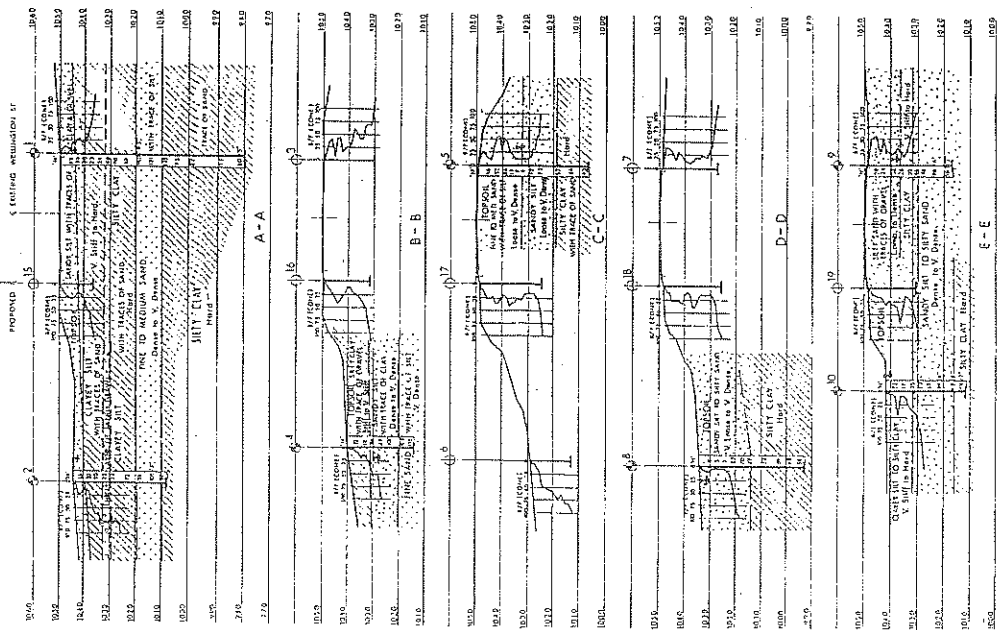
Date September 2008
Project 408-88-00



Prep'd MFA
Chkd. RPR

Appendix C

Record of Borehole Sheet (Previous Investigation)

[illegible]

- NOTE -

The boundaries between the State have been established only at
Gloria Hills Station. Between these the boundaries are assumed
from geological evidence and may be subject to considerable error.

[illegible]

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 5

FOUNDATION SECTION

JOB 66-F-43
W.P. 637-64

LOCATION N 203,193.616 E210,698.331
BORING DATE May 9, 1966

ORIGINATED BY D. Wan
COMPILED BY W.E.

DATUM 1048.04

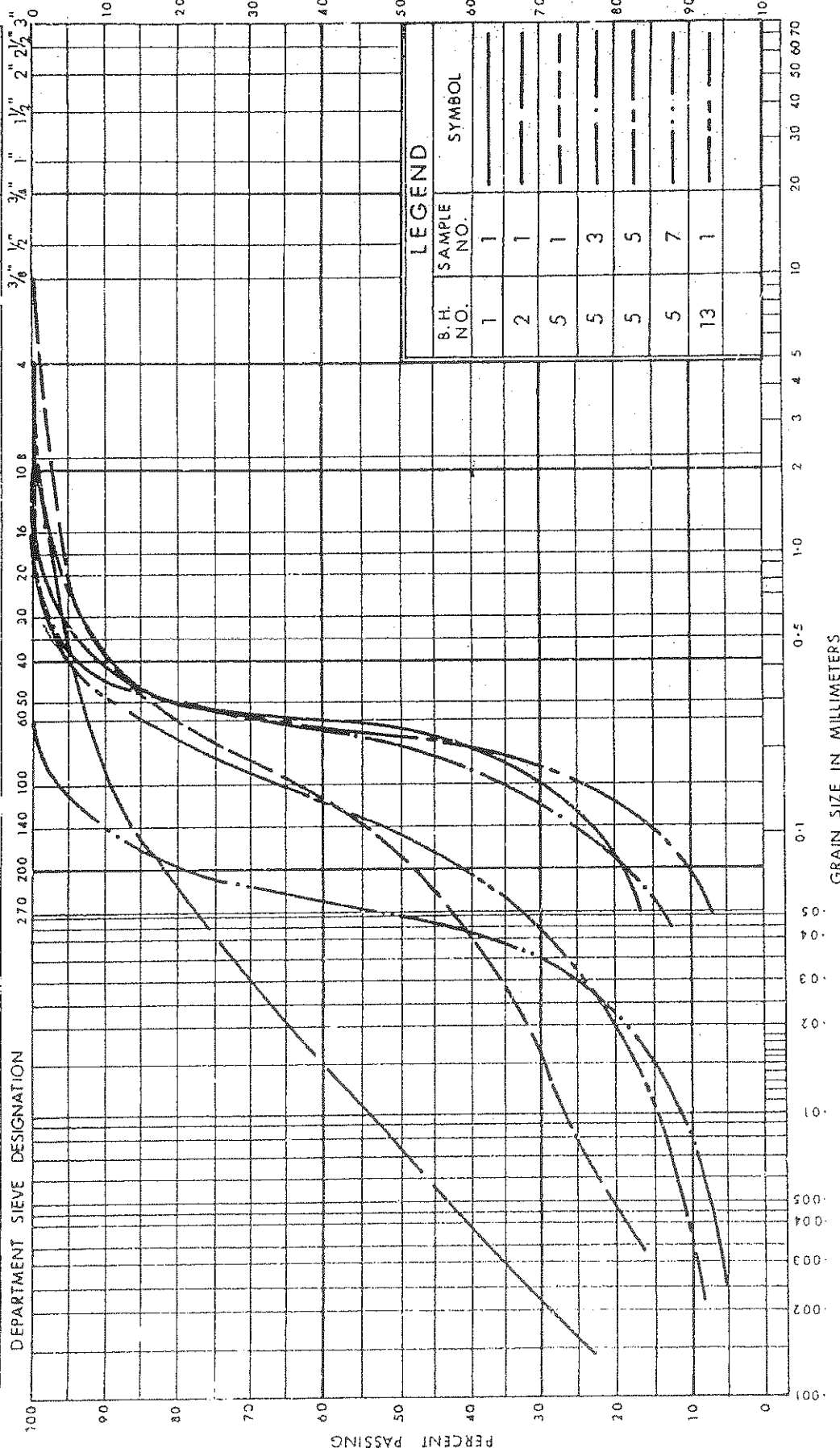
BOREHOLE TYPE Penetration and Washboring

CHECKED BY W.E.

ELEV. DEPTH	SOIL PROFILE DESCRIPTION	STRAT. PLOT	SAMPLES		ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT	LIQUID LIMIT — WL PLASTIC LIMIT — WP WATER CONTENT — W	BULK DENSITY P.C.F.	REMARKS
			NUMBER	TYPE					
1048.04 0.0	Ground Level								
1031.5 16.5	Topsoil Fine to Medium Sand with Traces of Silt Loose to Very Dense		1	SS 16					Gr. 2 Sa. 50 Sl. 36 Cl. 12
			2	SS 52					
			3	SS 46					W.L. Sa. 82 203.1 Sl. Cl. 18
			4	SS 11					Sa. 89 Sl. Cl. 11
			5	SS 7					Sa. 20 Sl. 75 Cl. 5
	Sandy Silt		6	SS 4					
			7	SS 12					
	Loose to Very Dense		8	SS 72					
1018.04 30.0	Silty Clay with Trace of Sand Hard		9	SS 52					
			10	SS 46					
1006.54 41.5	End of Borehole		11	SS 43					

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT			SAND			GRAVEL		
			Fine	Medium	Coarse	Fine	Coarse	
			0.075	0.425	0.850	0.075	0.425	2.0



DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION



ONTARIO

GRAIN SIZE DISTRIBUTION

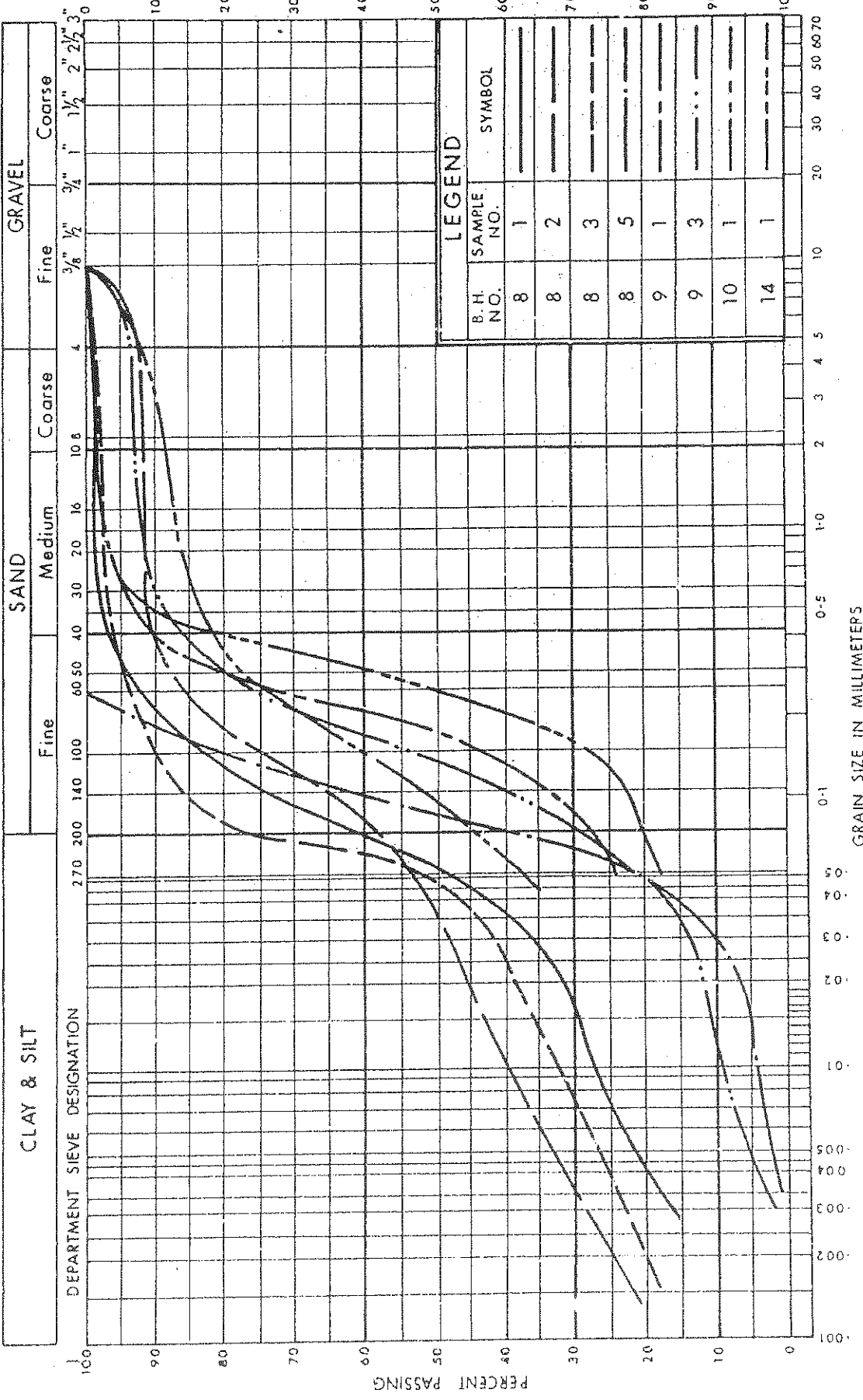
SANDY SILT

WITH TRACE OF CLAY & GRAVEL (LAYER 1)

W.P. No. 637-64

JOB No. 66-F-43

UNIFIED SOIL CLASSIFICATION SYSTEM

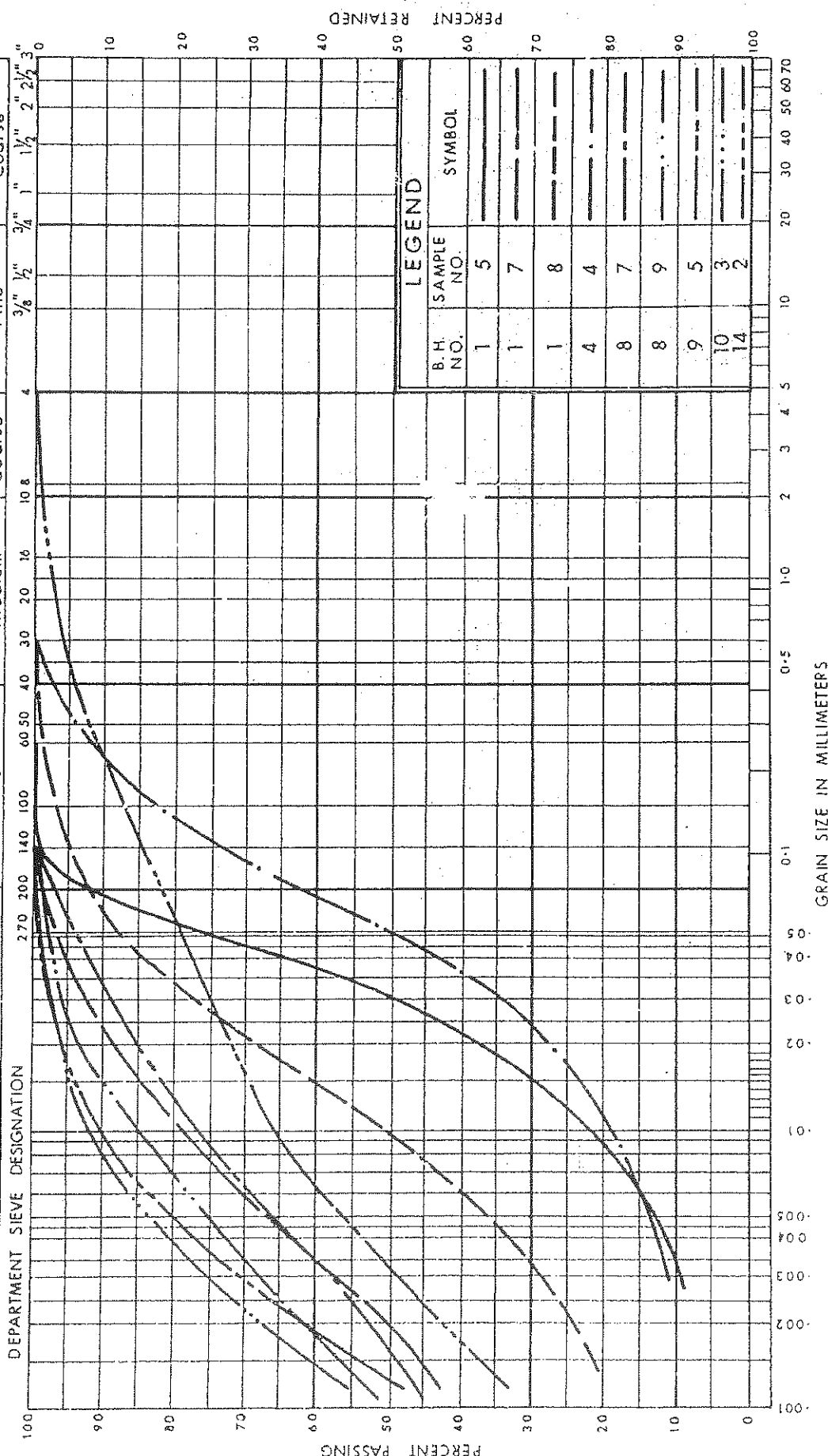


GRAIN SIZE DISTRIBUTION
SANDY SILT
WITH TRACE OF CLAY & GRAVEL (LAYER 1)

W.P. No. 637-64
JOB No. 66-F-43

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT		SAND				GRAVEL			
DEPARTMENT SIEVE DESIGNATION		Fine		Medium		Coarse		Fine	
		100	60	30	16	10	4	3/8	3/4
		200	40	20	10	5	2	1/2	1
		400	20	10	5	2.5	1	3/4	1.5
		600	10	5	2.5	1.5	0.75	1	2
		800	5	2.5	1.5	0.75	0.425	0.75	3
		1000	2.5	1.5	0.75	0.425	0.25	0.425	6
		1250	1.5	0.75	0.425	0.25	0.15	0.25	12
		1500	0.75	0.425	0.25	0.15	0.075	0.15	25
		2000	0.425	0.25	0.15	0.075	0.075	0.075	50
		2500	0.25	0.15	0.075	0.075	0.0425	0.075	100
		3000	0.15	0.075	0.0425	0.0425	0.025	0.0425	200
		3500	0.075	0.0425	0.025	0.025	0.015	0.025	425
		4000	0.0425	0.025	0.015	0.015	0.0075	0.015	850
		4500	0.025	0.015	0.0075	0.0075	0.00425	0.0075	1750
		5000	0.015	0.0075	0.00425	0.00425	0.0025	0.00425	3500
		5500	0.0075	0.00425	0.0025	0.0025	0.0015	0.0025	7000
		6000	0.00425	0.0025	0.0015	0.0015	0.00075	0.0015	14000
		6500	0.0025	0.0015	0.00075	0.00075	0.000425	0.00075	28000
		7000	0.0015	0.00075	0.000425	0.000425	0.00025	0.000425	56000
		7500	0.00075	0.000425	0.00025	0.00025	0.00015	0.00025	112000
		8000	0.000425	0.00025	0.00015	0.00015	0.000075	0.00015	224000
		8500	0.00025	0.00015	0.000075	0.000075	0.0000425	0.000075	448000
		9000	0.00015	0.000075	0.0000425	0.0000425	0.000025	0.0000425	896000
		9500	0.000075	0.0000425	0.000025	0.000025	0.000015	0.000025	1792000
		10000	0.0000425	0.000025	0.000015	0.000015	0.0000075	0.000015	3584000



DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION



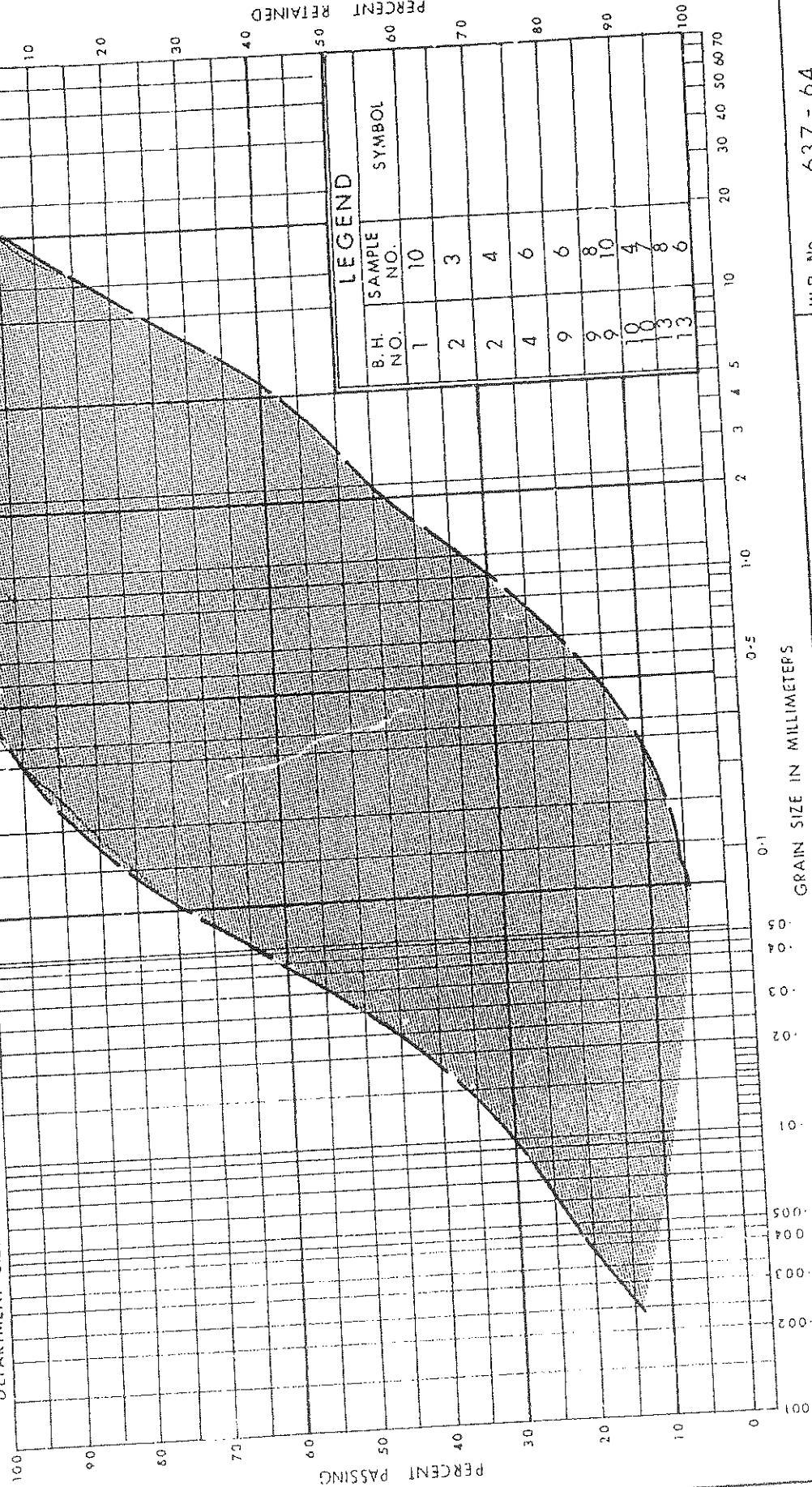
ONTARIO

W.P. No. 637-64
JOB No. 66-F-43

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT			SAND			GRAVEL		
			Fine	Medium	Coarse	Fine	Coarse	
			0.075	0.425	0.850	0.075	0.425	2.000
			75	425	850	75	425	2000

DEPARTMENT SIEVE DESIGNATION



GRAIN SIZE DISTRIBUTION
ENVELOPE FOR FINE TO MEDIUM SAND
WITH TRACE OF SILT & GRAVEL (LAYER 3)

DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

W.P. No. 637-64

JOB No. 66-F-43



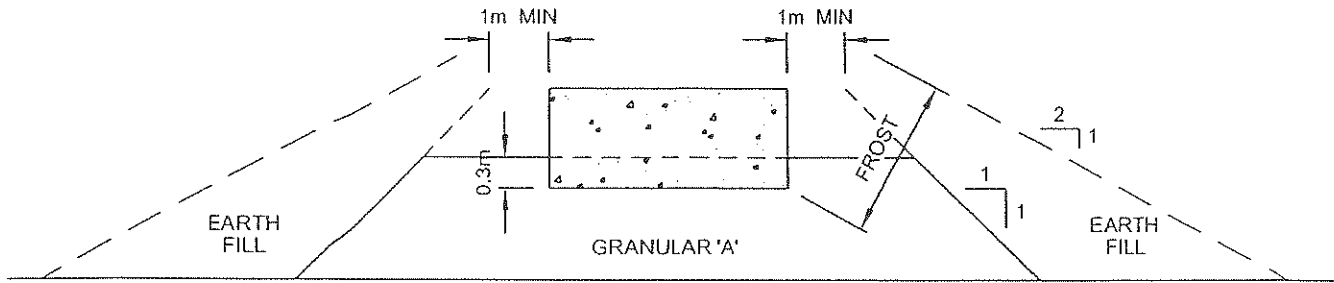
Appendix D
Foundation Comparison

COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

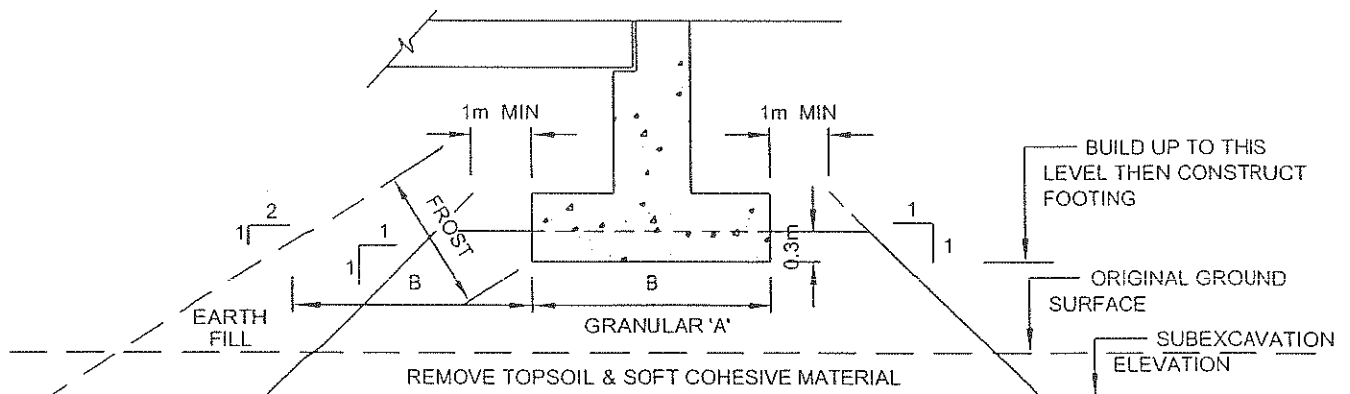
Foundation Element	Spread Footings	Spread Footings on Engineered Fill	Driven Piles	Caissons
Abutments	<p>Advantages:</p> <ul style="list-style-type: none"> i. Generally less costly construction than deep foundation elements. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Lower geotechnical resistance available due to founding on compact soils near the surface. ii. Dewatering may be required, depending on depth of excavation. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. Generally less costly construction than deep foundation elements. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Better geotechnical resistance than spread footings on native, but still influenced by the compact soils at the surface. ii. Dewatering may be required, depending on depth of excavation. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. High geotechnical resistance may be developed by driving the piles into very dense soils. ii. Comparatively short abutment stem possible iii. Permits integral abutment design <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher unit cost compared to footings. ii. Very dense/hard soils at shallow depth will limit length of pile and geotechnical resistance that can be developed. 	<p>Advantages:</p> <ul style="list-style-type: none"> ii. High resistance is available for caissons socketed in bedrock. iii. Subexcavation of fill or variable material is not required. iv. Construction of caissons could continue in freezing weather. <p>Disadvantages:</p> <ul style="list-style-type: none"> iii. Higher cost than spread footings. iv. Dewatering may be required. v. Possibility of boulders being encountered during augering. vi. Potential difficulty in cleaning and inspecting bases.
	NOT RECOMMENDED	NOT RECOMMENDED	RECOMMENDED	FEASIBLE AT THE PIER

Appendix E

Figure



CROSS-SECTION



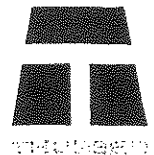
LONGITUDINAL SECTION NOT TO SCALE

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 F
Site photograph

N-E Ramp to HWY 7 over E-S Ramp and KWE
Highway 7-New, Kitchener to Guelph



Photo 1. Looking to the east side of Borehole 08-014, towards the KWE



Photo 2. Picture taken from Wellington Street and KWE bridge, looking northwest. Borehole 08-014

N-E Ramp to HWY 7 over E-S Ramp and KWE
Highway 7-New, Kitchener to Guelph

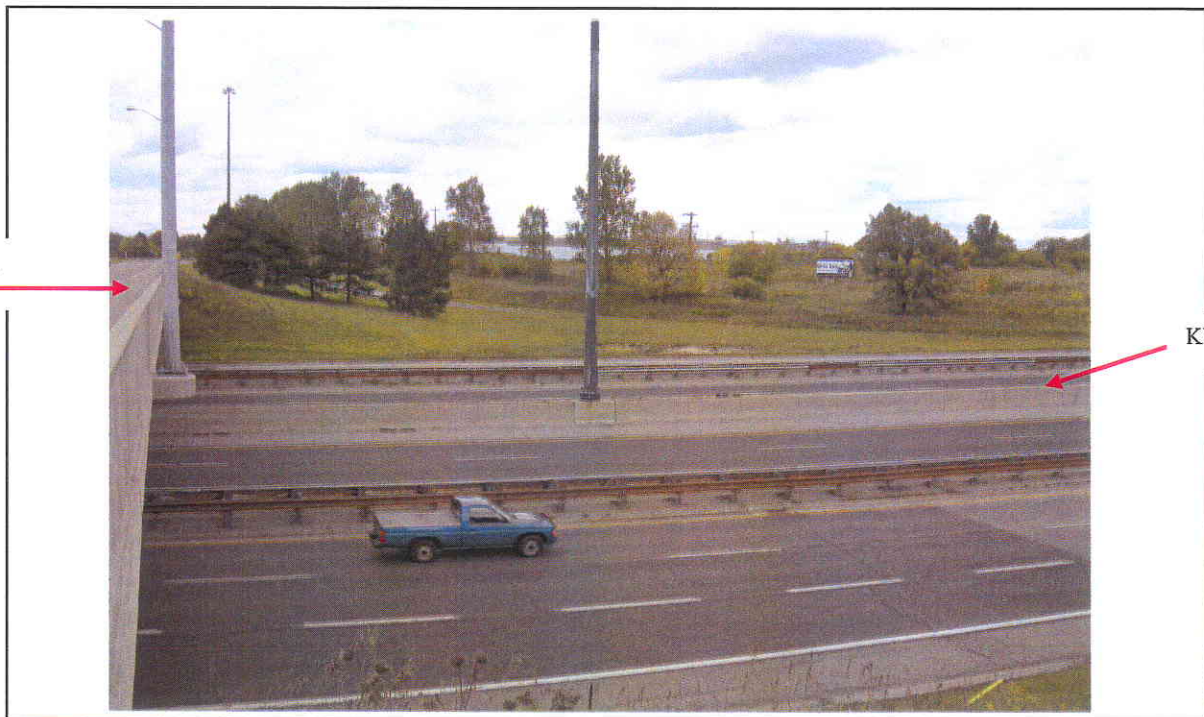
Wellington St.



KWE

Photo 3. Looking to the southeast quadrant of Wellington Street and KWE interchange.
Borehole 08-020

Wellington St.



KWE

Photo 4. Picture taken from Wellington Street and KWE bridge, looking southeast. Borehole 08-020

Appendix G
Slope Stability Output

Thurber Engineering Ltd. - Toronto
 15-64-17 Highway 7 - New
 N-E Ramp over E-S Ramp, Wellington St. & KWE
 August 19, 2008
 North and South Approach Earth Fill
 Embankment height: 22m

Earth Fill	Gamma C	Phi	Piezo
Silty Clay	kN/m ³	deg	Surf.
	21	30	1
	19	0	1

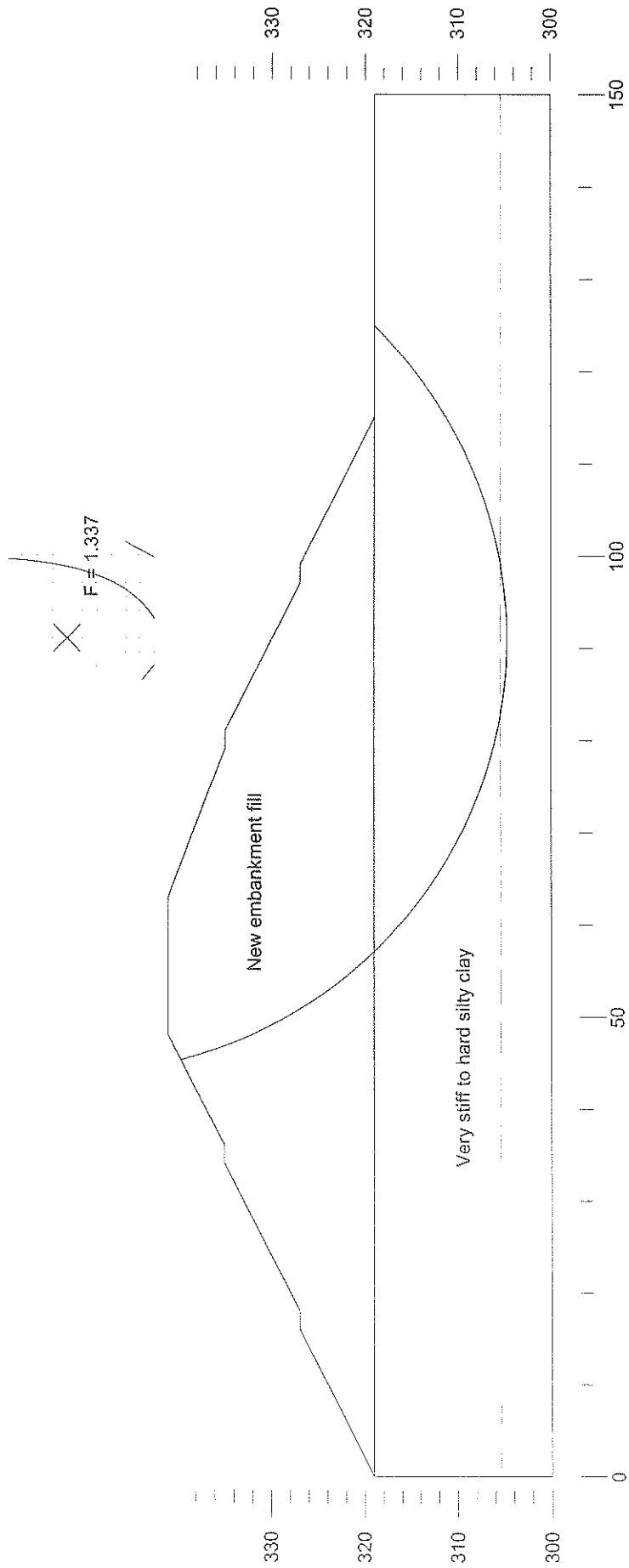


FIGURE 1

Thurber Engineering Ltd. - Toronto
 15-64-17 Highway 7 - New
 N-E Ramp over E-S Ramp, Wellington St. & KWE
 August 19, 2008
 North and South Approach Earth Fill
 Embankment height: 22m

	Gamma C	Phi	Piezo
	kN/m ³	deg	Surf.
Earth Fill	21	30	1
Silty Clay	19	0	1

Seismic coefficient = 0.08

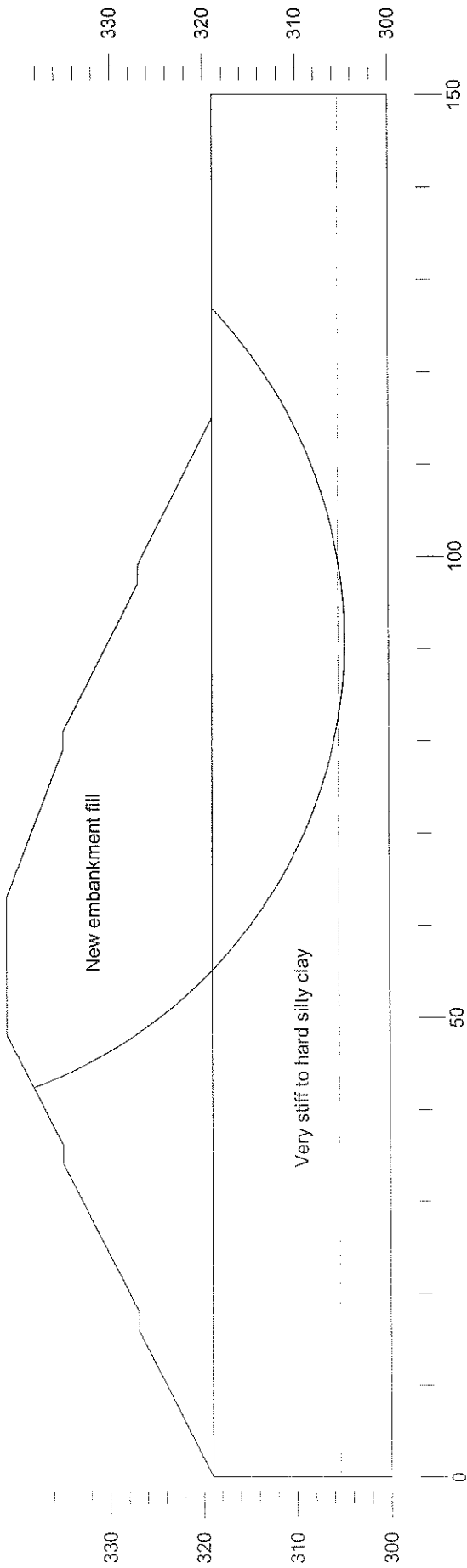
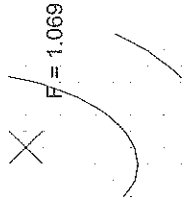
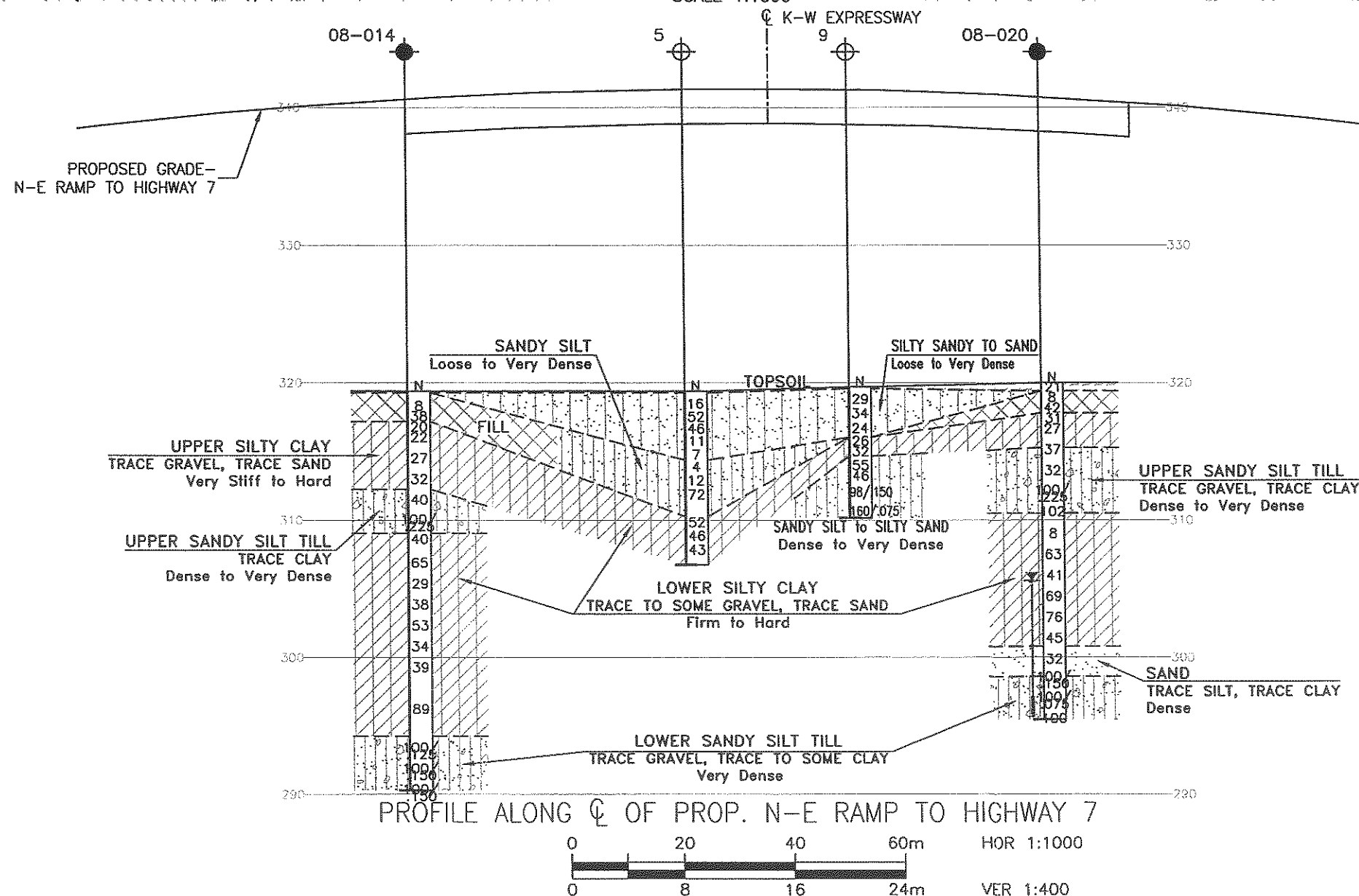
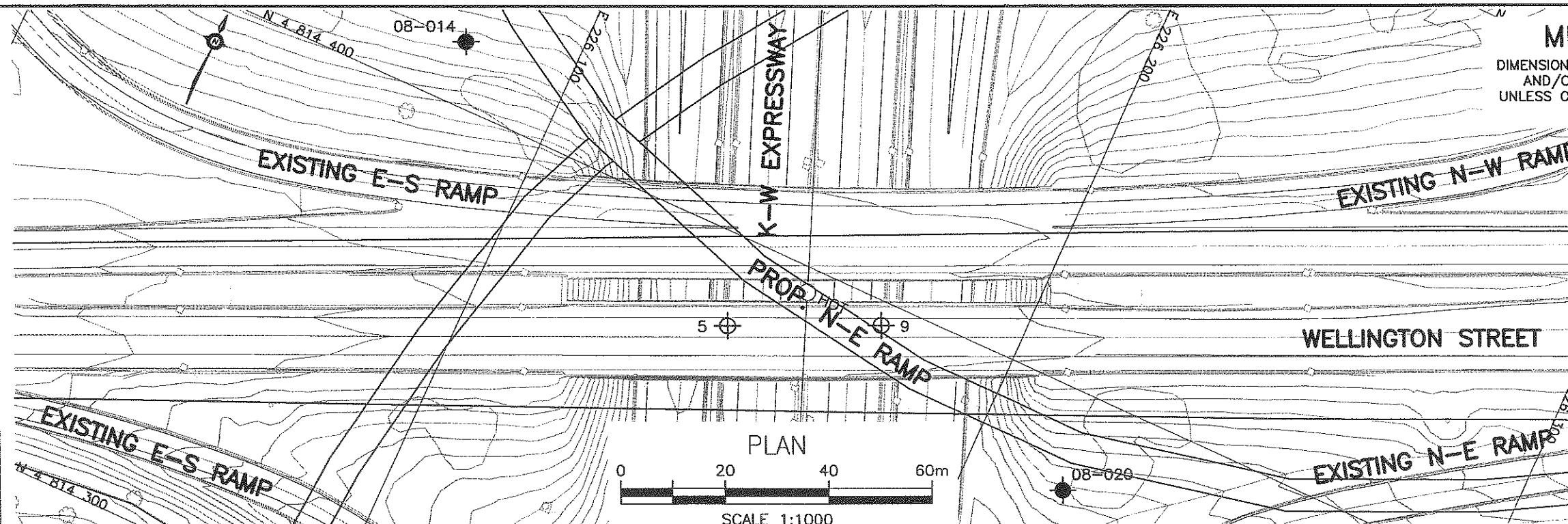


FIGURE 2

Appendix H

Drawing titled “Borehole Locations and Soil Strata”



METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

CONT No
GWP No 408-88-00

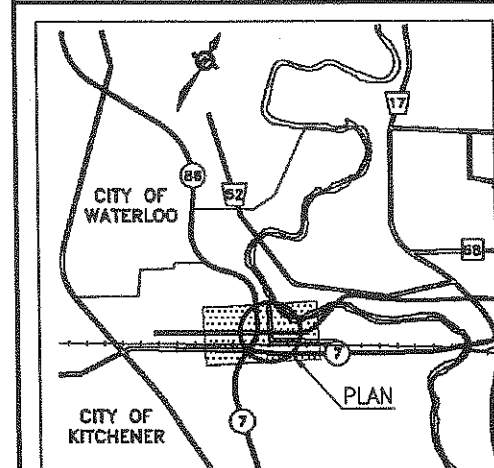
HIGHWAY 7
RECOMMENDED ROUTE
N-E RAMP OVER E-S RAMP & KWE
BOREHOLE LOCATIONS AND SOIL STRATA



SHEET








THURBER ENGINEERING LTD.
 GEOTECHNICAL • ENVIRONMENTAL • MATERIALS



KEYPLAN

LEGEND

- | | |
|---|---------------------------------------|
|  | Borehole |
|  | Previous Borehole by others |
| N | Blows /0.3m (Std Pen Test, 475J/blow) |
| CONE | Blows /0.3m (60° Cone, 475J/blow) |
| PH | Pressure, Hydraulic |
|  | Water Level |
|  | Head Artesian Water |
|  | Piezometer |
| 90% | Rock Quality Designation (RQD) |
| A/R | Auger Refusal |

NO	ELEVATION	NORTHING	EASTING
08-014	319.4	4 814 412.1	226 078.
08-020	320.0	4 814 381.0	226 219.
5	319.4	4 814 382.9	226 147.
9	319.7	4 814 395.5	226 174.

-NOTES-

- 1) The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- 2) This drawing is for subsurface information only. Surface details and features are for conceptual illustration.
- 3) Proposed grades are from Plate 2B of the E.A. Study.

GEOCREs No. 40P8-160

REVISIONS							
	DATE	BY	DESCRIPTION				
DESIGN	AEG	CHK	PKC	CODE	LOAD	DATE JUN. 20	
DEAWN	MEA	CHK	AFG	SITE	STRUCT	LDWG	

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

[illegible]