FOUNDATION INVESTIGATION AND DESIGN REPORT
DOANE ROAD UNDERPASS
HIGHWAY 404 EXTENSION
FROM GREEN LANE TO WOODBINE AVENUE/RAVENSHOE ROAD
ONTARIO
G.W.P. 2109-05-00

Geocres Number: 31D-483

Report to

Philips Engineering / Hatch Mott MacDonald Joint Venture

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted at the location of a proposed underpass carrying Doane Road over the proposed Highway 404 extension in the Regional Municipality of York, Ontario.

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

Thurber carried out the investigation as a sub-consultant to Philips Engineering/Hatch Mott MacDonald Joint Venture under the Ministry of Transportation Ontario (MTO) Agreement Number 2007-E-0027.

2 SITE DESCRIPTION

The proposed Doane Road underpass structure will be located on the existing Doane Road alignment, approximately 2.0 km north of Mount Albert Road and approximately 900.0 m west of the intersection of Doane Road and Woodbine Avenue (York Regional Road 8), in the Town of East Gwillimbury, Regional Municipality of York.

Currently, the topography along Doane Road slopes downward to the southeast. The ground surface within the proposed structure varies from west to east from Elevations 265.6 to 261.7.

The lands around the site are generally undeveloped and/or agricultural. Vegetation consists mainly of tall grass, shrubs and a few mature trees. There are farmsteads to the north and south of Doane Road.

The site lies within the physiographic region known as The Peterborough Drumlin Field, characterized by drumlinized till. The till is typically sandy with shallow coverings of silt and fine sand.

Photographs in Appendix G show:

1. View of the site looking at Boreholes 08-50 and 08-52 drilled on the south side of Doane Road.
2. View of the site looking at Borehole 08-53, WBL of Doane Road.
3. View of the site looking at Borehole 08-54 drilled on the south side of Doane Road.
4. View of the site looking at Borehole 08-55 drilled on Doane Road.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project were carried out from October 20 to 24, 2008 and consisted of drilling and sampling a total of eight boreholes (numbered 08-48 to 08-55). Six boreholes (Boreholes 08-49 to 08-54) were drilled at the proposed foundation elements (embankments and pier) to depths ranging from 9.3 m to 12.3 m (Elevations 248.7 to 256.8). One borehole was drilled at each approach embankment. Termination depths for the approach boreholes (Boreholes 08-48 and 08-55) were 7.9 m and 9.3 m (Elevations 257.8 and 252.4 m), respectively.

Permission to access and drill on adjacent lands located on the north side of Doane Road had not been granted by the land owner at the time of investigation. In light of this situation and after discussions with MTO's Foundation office, Boreholes 08-49, 08-51 and 08-53 were moved from their original locations approximately 5.0 to 10.0 m to the south and drilled on the existing Doane Road.

The approximate borehole locations are shown on the Borehole Locations and Soil Strata Drawing in Appendix H. The coordinates and elevations of the boreholes are given on the drawing and on the individual Record of Borehole Sheets in Appendix A.

The borehole locations were marked in the field and utility clearances were obtained prior to drilling.

Drilling was carried out using a track mounted CME 55 drill rig. A combination of solid and hollow stem auger drilling techniques were used to advance the boreholes. Samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT) in the overburden soils.

The drilling and sampling operations were supervised on a full time basis by a member of Thurber's technical staff. The supervisor logged the boreholes and processed the recovered soil samples for transport to Thurber's laboratory for further examination and testing.

Groundwater conditions in the open boreholes were observed throughout the drilling operations. Standpipe piezometers consisting of 19 mm diameter PVC pipe with slotted screens were installed and enclosed in filter sand in three boreholes to permit longer term groundwater level monitoring. The locations and completion details of the piezometers are shown in Table 3.1.

Table 3.1 – Borehole Completion Details

Foundation Unit	Borehole	Piezometer Tip Depth/ Elevation (m)	Completion Details
West Approach	08-48	None installed	Borehole backfilled with holeplug to 0.07 m, then asphalt to surface.
West Abutment	08-49	None installed	Borehole backfilled with holeplug to 0.05 m, then asphalt to surface.
	08-50	9.1/255.2	Sand from 9.1 m to 7.3 m, holeplug from 7.3 m to surface.
Pier	08-51	8.1/255.5	Sand from 8.1 m to 5.8 m, holeplug from 5.8 m to 0.15 m, then concrete to surface.
	08-52	None installed	Borehole backfilled with holeplug to 3.0 m, then auger cuttings to surface.
East Abutment	08-53	None installed	Borehole backfilled with holeplug to 0.07 m, then asphalt to surface.
	08-54	9.8/251.2	Sand from 9.8 m to 7.6 m, holeplug from 7.6 m to surface.
East Approach	08-55	None installed	Borehole backfilled with holeplug to 0.2 m, auger cuttings from 0.2 m to 0.05 m then asphalt to surface.

4 LABORATORY TESTING

The recovered soil samples were subjected to Visual Identification (VI) and to natural moisture content determination. Selected samples were also subjected to grain size distribution analyses (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 are presented in these sheets and on the "Borehole Locations and Soil Strata Drawing" and "Stratigraphic Sections" 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 soil stratigraphy encountered across the site consists of fill of variable composition (sand, sand and gravel, silty sand and clayey silt) overlying native deposits of sand and silt till, gravelly sand, clayey silt and silt till. Asphalt was encountered at the surface in boreholes drilled on Doane Road. More detailed descriptions of the individual strata are presented below.

5.1 Pavement Structure

Pavement structure consisting of approximately 25 mm to 50 mm of asphalt overlying granular (sand and gravel fill) road base was encountered in Boreholes 08-48, 08-49, 08-51, 08-53 and 08-55 drilled on existing Doane Road lanes and shoulders.

5.2 Fill

Fill was contacted below the pavement structure in most of the boreholes, except in Boreholes 08-50, 08-52 and 08-54. The fill generally consists of brown to dark brown sand and silty sand containing trace to some gravel and some silt. In Borehole 08-53, drilled at the proposed east abutment, a cohesive layer of silty clay fill containing trace sand and occasional silt seams was contacted below the cohesionless fill at 0.8 m depth (Elevation 261.4).

The thickness of the fill ranged from 0.55 m to 2.25 m. The depth to the base of the fill varied from 0.6 to 2.3 (Elevations 260.4 to 265.5).

SPT 'N' values recorded in the cohesionless fill ranged from 13 to 37 blows per 0.3 m penetration indicating a loose to dense relative density. In Borehole 08-55, an SPT 'N' value of 73 blows per 0.3 m of penetration indicating a very dense relative density was measured below the asphalt layer.

In the silty fill layer, the SPT 'N' values were 7 and 12 blows per 0.3 m of penetration, indicating a firm to stiff consistency.

The moisture content of the fill ranged from 5% to 23%.

Grain size distribution curve for a sample of silty clay fill tested is presented on the Record of Borehole sheet and on Figure B1. Atterberg Limit test results are presented on Figure B9 of Appendix B. The results of the laboratory test are summarized as follows:

Soil Particles	(%)
Gravel	0
Sand	4
Silt	68
Clay	28

Index Property	(%)
Liquid Limit	27
Plastic Limit	16

The above results show that the silty clay fill is typically of low plasticity with a group symbol of CL.

5.3 Sand and Silt Till

Layers of native brown to grey sand and silt till containing trace to some clay, trace to some gravel and occasional cobbles were observed across the site in Boreholes 08-48 to 08-54 at depths and elevations as indicated in Table 5.1.

Table 5.1 – Locations of Native Sand and Silt Till

Foundation Unit	Borehole	Depth below existing ground surface (m)	Elevation (m)	Thickness (m)
West Approach	08-48	0.8 to 2.4	264.9 to 263.2	1.6
West Abutment	08-49	0.6 to 3.0	265.5 to 263.0	2.4
	08-50	0.05 to 2.1	264.3 to 262.2	2.1
Pier	08-51	2.3 to 12.3 (borehole termination depth)	261.4 to 251.3	At least 10.0
	08-52	0.05 to 9.5 (borehole termination depth)	262.3 to 252.8	At least 9.5
East Abutment	08-53	1.8 to 10.8 (borehole termination depth)	260.4 to 251.4	At least 9.0
	08-54	0.05 to 2.3 5.6 to 12.3 (borehole termination depth)	260.9 to 258.7 255.4 to 248.7	2.2 At least 6.7

Standard Penetration tests in this sand and silt till deposit gave SPT 'N' values ranging from 5 to 58 blows per 0.3 m of penetration, indicating a loose to very dense relative density. Higher SPT 'N' values ranging from 83 blows per 0.3 m of penetration to 100 blows per 0.075 m of penetration were measured below 2.3 m depth (Elevation 261.4) in Borehole 08-51, below 3.4 m and 4.6 m depth (Elevations 259.0 and 257.5) in Boreholes 08-52 and 08-53 and below 7.6 m depth (Elevation 253.4) in Borehole 08-54.

The moisture content of samples from the sand and silt till deposit varies between 8% and 22%.

Grain size distribution curves for sand and silt till samples tested are presented on the Record of Borehole sheet and on Figures B2 and B3. A sample of sand and silt containing some clay was tested for Atterberg Limits and the test results are presented on Figure B10 of Appendix B.

The results of the laboratory tests are summarized as follows:

Soil Particles	(%)
Gravel	0 to 8
Sand	32 to 48
Silt	45 to 57
Clay	4 to 13

Index Property	(%)
Liquid Limit	17
Plastic Limit	12

The above results show that the clayey zones in the sand and silt till are typically of low plasticity with group symbols of CL-ML.

Glacial tills inherently contain cobbles and boulders which may account for some high blow counts and resistance to augering.

5.4 Gravelly Sand

A layer of native brown gravelly sand containing some silt and clay was encountered in Borehole 08-48 at 2.4 m depth (Elevation 263.2). Cobbles were encountered within the gravelly sand layer at 3.5 m depth (Elevation 262.1). Thickness of the gravelly sand layer was 1.7 m. The depth to the base of the gravelly sand was 4.1 m (Elevation 261.5).

SPT 'N' values measured in the gravelly sand were 35 and 50 blows per 0.3 m of penetration, indicating a dense relative density. Moisture contents were 2% and 10%.

Grain size distribution curve for a gravelly sand sample tested is presented on the Record of Borehole sheets and on Figure B4. The results of the laboratory test are summarized as follows:

Soil Particles	Gravelly Sand (%)
Gravel	23
Sand	64
Silt & Clay	13

5.5 Clayey Silt Till

Native brown to grey clayey silt till containing trace of sand was contacted at 2.3 m depth (Elevation 258.7) in Borehole 08-054, drilled at the proposed east abutment. Thickness of the clayey silt was 3.3 m. The depth to the base of the clayey silt was 5.6 m (Elevation 255.4).

SPT 'N' values measured in the clayey silt till were 14 and 26 blows per 0.3 m of penetration, indicating a stiff to very stiff consistency. Moisture content ranged from 19% to 20%.

A grain size distribution curve for a clayey silt till sample tested is presented on the Record of Borehole sheet and on Figure B5. Atterberg Limit test results are presented on Figure B11 of Appendix B. The results of the laboratory test are summarized as follows:

Soil Particles	(%)
Gravel	0
Sand	1
Silt	76
Clay	22

Index Property	(%)
Liquid Limit	26
Plastic Limit	19

The above results show that the clayey silt till is typically of low to medium plasticity with group symbols of ML-CL.

5.6 Silt Till

Layers of native brown to grey silt till containing some sand to sandy, trace to some clay, trace to some gravel and occasional cobbles were observed in Boreholes 08-48 to 08-50 and 08-55 at depths and elevations as indicated in Table 5.1.

Table 5.1 – Locations of Native Silt Till

Foundation Unit	Borehole	Depth below existing ground surface (m)	Elevation (m)	Thickness (m)
West Approach	08-48	4.1 to 7.9 (borehole termination depth)	261.5 to 257.8	At least 3.8
West Abutment	08-49	3.0 to 9.3 (borehole termination depth)	263.0 to 256.8	At least 6.3
	08-50	2.1 to 9.4 (borehole termination depth)	262.2 to 255.0	At least 7.3
East Approach	08-55	1.1 to 9.3 (borehole termination depth)	260.5 to 252.4	At least 8.2

Layers of sand and silty sand were contacted within the silt till at 4.1 m depth (Elevations 262.0 and 260.3) in Boreholes 08-49 and 08-50.

Standard Penetration tests in this deposit gave SPT 'N' values ranging from 24 to 92 blows per 0.3 m of penetration, indicating a compact to very dense relative density. Locally in Borehole 08-55, loose conditions were measured at 1.5 m depth (Elevation 260.2). SPT 'N' values higher than 100 blows per 0.3 m of penetration were measured generally below 4.0 m depth (approximate Elevation 261.0) in Boreholes 08-48 to 08-50 drilled at the west abutment and west approach and at 7.5 m depth (Elevation 254.2.) in Borehole 08-55 drilled at the east approach.

The moisture content of samples from the sand and silt till deposit varies between 3% and 22%.

Grain size distribution curves for the silt till samples tested are presented on the Record of Borehole sheet and on Figures B6 and B7. Grain size distribution curve for the layer of silty sand tested is presented on the Record of Borehole sheet and on Figure B8. A sample of silt till containing some clay was tested for Atterberg Limits and the test results are presented on Figure B12 of Appendix B.

The results of the laboratory tests are summarized as follows:

Soil Particles	Silt Till (%)	Silty Sand (%)
Gravel	0 to 6	1
Sand	2 to 28	68
Silt	54 to 81	26
Clay	7 to 17	5

Index Property	(%)
Liquid Limit	18
Plastic Limit	12

The above results show that the clayey zones in the silt till are typically of low plasticity with group symbols of CL-ML.

Glacial tills inherently contain cobbles and boulders which may account for some high blow counts.

5.7 Water Levels

Water levels were observed in the boreholes during and upon completion of drilling. Standpipe piezometers were installed in three boreholes to monitor water levels after completion of drilling. The water levels measured in the piezometers are summarized in Table 5.3, along with the measurements in the boreholes upon completion of drilling.

Table 5.3 – Water Level Measurements

Foundation Unit	Borehole	Date	Water Level (m)		Comment
			Depth	Elevation	
West Approach	08-48	October 22, 2008	6.5	259.1	Open borehole
West Abutment	08-49	October 22, 2008	7.1	259.0	Open borehole
	08-50	October 20, 2008 October 24, 2008 November 28, 2008 February 6, 2009 February 20, 2009 March 20, 2009 April 22, 2009 September 2, 2009	5.0 4.4 4.9 0.1 0.2* 1.0 1.1 2.6	259.4 260.0 259.5 264.3 264.6 263.4 263.3 261.8	Open borehole In piezometer In piezometer In piezometer In piezometer In piezometer In piezometer In piezometer
Pier	08-51	November 28, 2008	4.1	259.5	In piezometer
	08-52	October 21, 2008	4.6	257.8	Open borehole
East Abutment	08-53	October 23, 2008	3.0	259.1	Open borehole
	08-54	October 24, 2008 November 28, 2008 February 6, 2009 February 20, 2009 March 20, 2009 April 22, 2009 September 2, 2009	2.1 3.7 Ground surface 0.4* 0.7* 0.6* 0.6	258.9 257.3 261.0 261.4 261.7 261.6 260.4	Open borehole In piezometer In piezometer In piezometer In piezometer In piezometer In piezometer
East Approach	08-55	October 22, 2008	4.2	257.5	Open borehole

* Water level above ground surface (artesian condition)

The piezometric readings of the current investigation indicate that the groundwater level at the site is high and the water level decreases from west to east from Elevations 264.6 to 261.7.

Water levels were observed approximately 0.2 m to 0.7 m above the existing ground surface (artesian conditions) during the later Winter/early Spring season in Boreholes 08-50 and 08-054, near Elevations 264.6 and 261.7, respectively.

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

Borehole locations were selected by Thurber Engineering Ltd. Surveyors from J. D. Barnes obtained the co-ordinates and the ground surface elevations at each borehole.

Thurber obtained utility clearances for the borehole locations prior to drilling.

DBW Drilling of Ajax Ontario supplied track mounted CME 55 drill rig and conducted the drilling, sampling and in-situ testing operations.

The field program was supervised on a full time basis by Ms. Eckie Siu of Thurber.

Routine laboratory testing was carried out by Thurber Engineering Ltd.

Overall supervision of the field program was conducted by Mr. Alastair E. Gorman, P.Eng. and Ms. R. Palomeque Reyna, 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.

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

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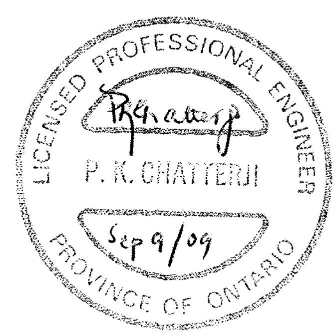

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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

7 GENERAL

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

It is understood that the proposed underpass is to carry Doane Road over the extension of Highway 404 located in the Regional Municipality of York, Ontario.

The natural ground surface at the structure site slopes downward from west to east from approximate Elevations 264.6 m to 261.7 m.

Based on the preliminary General Arrangement (GA) drawing provided by Hatch Mott MacDonald, a two-span structure supported on two abutments and one pier is proposed. The west and east spans will be 40.3 m long. The bridge is approximately 28.0 m to 29.0 m wide. It is understood that the finished grade of Highway 404 will be at about Elevations 263.0 m to 264.0 m.

At the west abutment, the proposed finished grade at the structure will be about Elevation 270.7 m and the original ground surface is near Elevation 265.6 m, resulting in a west approach embankment up to 5.1 m high. At the east abutment, the finished grade will be at Elevation 271.4 m and the original ground surface is at about Elevation 261.7 m, resulting in an east approach embankment up to 9.7 m high.

It is understood that an interchange may be constructed at this site in the future. The recommendations in this report address the foundation design for the bridge that is understood to be compatible with the proposed interchange. Future foundation investigation may be required for high fills after the ramp configuration has been finalized.

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

8 STRUCTURE FOUNDATIONS

The stratigraphy encountered at the abutments and pier locations consist of pavement structure and/or sand fill overlying extensive deposits of native loose to very dense sand and silt till, stiff to very stiff clayey silt till and compact to very dense silt till. Interbedded layers of sand, silty sand and gravelly sand were observed within the till deposits.

The piezometric readings indicate that water level is high at this site and the water level varies from west to east from Elevations 264.6 to 261.7, and as a result most of the overburden sands, silts and clayey silt tills are below the groundwater level. Artesian conditions were encountered during the Winter/Spring season at the west and east abutments (Boreholes 08-50 and 08-054), with water level measured 0.2 m to 0.7 m above ground surface (Elevations 264.6 and 261.7).

Initial consideration was given to the following foundation types:

- Spread footings on native soils
- Spread footings on engineered fill
- Augered Caissons (drilled shafts)
- Driven piles

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

8.1 Spread Footings on Native Ground

At both abutments and at the pier, thickness of the fill varies from 0.6 m to 2.3 m. The existing fill is not considered to be suitable for the support of spread footings. Therefore it is recommended that spread footings be extended below this fill layer and bear on the underlying competent native soils.

Provided a minimum footing width of 2 m is maintained, the design of spread footings bearing on native undisturbed compact to very dense sand and silt till and silt till must be in accordance with the elevations and bearing resistances given in Table 8.1.

Table 8.1 – Highest Permitted Founding Elevations and Bearing Resistance for Spread Footing on Native Soils

Foundation Unit	Borehole	Depth below existing ground surface (m)	Founding Elevation (m)	ULS_f (kPa)	SLS (kPa)	Soil
West Abutment	08-49	3.8	262.3	600	400	Compact to very dense silt till and sand and silt till
	08-50	2.1				
Pier	08-51	3.6	260.0	600	400	Dense to very dense sand and silt till
	08-52	2.4				
East Abutment	08-53	4.1	258.0	450	300	Very dense sand and silt till
	08-54	3.0				Very stiff clayey silt till

*Above the proposed Hwy 404 grade at Elevation 262.3

The geotechnical resistances quoted above 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.

Founding elevations for the west and east abutments indicated in Table 8.1 will be in cohesionless soils, approximately 2.3 m to 3.7 m below the groundwater table. The founding elevation for the pier will be approximately 0.5 m above the groundwater table. Prior dewatering will therefore be required for footing construction.

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 settlements are not expected to exceed 20 mm across the width of the structure.

The sliding resistance of mass concrete poured on the native sand and silt till and silt till may be computed on the basis of an ultimate coefficient of friction of 0.55. This is an "ultimate" value and requires a degree of sliding movement to occur to fully mobilize the resistance.

The bases of the foundation excavations should be inspected by a geotechnical engineer to confirm that the exposed surface conforms to the design requirements and has been adequately prepared to receive concrete. Where subexcavation is required to remove unsuitable material from below the design founding level, the founding surface should be re-established using engineered fill or mass concrete of the same class of concrete as used in the footing. The engineered fill must consist of OPSS Granular "A" placed in 150 mm lifts, compacted to 100% of its SPMDD at ±2% of optimum moisture content.

An effective dewatering plan must be in place prior to the start of footing excavation so that the concrete footings are poured in the dry. A permit to take water (PTTW) may be required for this dewatering process.

Spread footings on native soils are not recommended at this site due to the presence of high water levels and artesian conditions observed and the associated dewatering that will be required required prior to footing excavation.

8.2 Spread Footings on Engineered Fill

Consideration was also given to placing spread footings on engineered fill pads. The GA indicates that proposed Highway 404 grade will be near Elevations 263.0 m to 264.0 m. .

If an engineered fill pad is used at this site, all fill, topsoil or other deleterious materials must be stripped from the footprint of the engineered fill to expose competent native subgrade material. At this site, the engineered fill will bear on native compact to dense sand and silt till or stiff to very stiff clayey silt till. The highest permitted founding elevations at which engineered fill should be placed, are given in Table 8.2.

Table 8.2 – Highest Permitted Base Elevations for Engineered Fill

Foundation Unit	Borehole	Highest Permitted Engineered Fill Base		
		Depth below existing ground surface (m)	Fill Base Elevation (m)	Soil
West Abutment	08-49	0.8	265.3*	Compact sand and silt till
	08-50	0.7	263.7*	Compact sand and silt till
Pier	08-51	2.3	261.3	Very dense sand and silt till
	08-52	1.9	260.5	Compact sand and silt till
East Abutment	08-53	2.6	259.5	Compact sand and silt till
	08-54	2.3	258.7	Stiff to very stiff clayey silt till

*Above the proposed Hwy 404 grade near Elevations 263.0 to 264.0.

The engineered fill must consist of OPSS Granular “A” placed in 150 mm lifts and compacted to 100% of its SPMDD at ±2% of optimum moisture content and generally conforming to the geometry illustrated in Figure 1 in Appendix D. The thickness of engineered fill must be a minimum of 2.0 m.

Provided a minimum footing width of 2 m is maintained footings bearing on the well compacted engineered fill may be designed for the following values:

- Factored geotechnical resistance of 900 kPa at Ultimate Limit States (ULS)
- Geotechnical resistance of 350 kPa at Serviceability Limit States (SLS)

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 settlements are not expected to exceed 20 mm across the width of the structure.

The lateral resistance of the footings founded on engineered fill may be computed using an unfactored friction of 0.7. This is an "ultimate" value and requires a degree of sliding movement to occur to fully mobilize the resistance.

The highest permitted engineered fill base elevations for the east and west abutments given in Table 8.2 are 0.9 m to 3.0 m below the groundwater table on this site and the pier base elevations are approximately 1.0 m to1.8 m above groundwater table.

If a temporary excavation required to construct the engineered fill pads extends below the water table, a dewatering plan must be in place to construct the fill pad and the footing in the dry and to prevent sloughing of the sides or disturbance of the base of the excavation due to the inflow of groundwater. The dewatering method adopted must depress the groundwater level to at least 0.5 m below the base of excavation and must maintain a stable, unwatered excavation throughout the duration of the fill pad and footing construction. Dewatering must remain operational and effective until the footing is constructed and backfilled or until the engineered fill pad is completed to a level at least 0.5 m above the groundwater level.

Due to the high water table conditions and the associated dewatering requirements, footings on engineered fill are not recommended at this site.

8.3 Augered Caissons (Drilled Shafts)

At this site augered caissons are not recommended due to potential installation difficulties through a deep deposit of cohesionless soil under water. Sealing of the caisson liner into the founding stratum will be difficult and base boiling may also be encountered. Accordingly, this alternative has not been developed further.

8.4 Driven Piles

The subsurface conditions at this site are considered suitable for the design of foundations supported on driven steel H-piles. The results of the investigation indicate that driven H-piles at the abutments and at the pier will develop geotechnical resistance in the very dense sand and silt till and silt till. Based on an HP 310 X 110 pile, a minimum embedment depth of 6 m is required. In this regard at the pier, pile installation would probably require pre-augering to achieve sufficient embedment.

The elevations at which the piles are expected to develop the required resistance are given in Table 8.3.

Table 8.3 – Estimated Pile Tip Elevation

Foundation Unit	Borehole	Anticipated Pile Tip Elevation To Develop Required Resistance	Anticipated Pile Length below original ground (m)	Comments
West Abutment	08-49	259.0	7.1	-
	08-50	258.0	6.4	A minimum pile length of 6 m must be achieved. Depending on final design, pre-augering may be required at the dense till to achieve sufficient pile embedment.
Pier	08-51	259.0	4.6	
	08-52	256.5	5.9	
East Abutment	08-53	255.0	7.1	-
	08-54	251.5	9.5	-

The pile tip elevations shown in Table 8.5 should be used for estimating purposes only. The actual pile tip elevations will be controlled as described in Section 8.4.4 Pile Driving.

8.4.1 Axial Resistance

The vertical, axial, factored geotechnical resistances at Ultimate Limit States (ULS) and geotechnical resistances at Serviceability Limit States (SLS) for two pile sections when driven into the very dense glacial till soils are presented in Table 8.4.

Table 8.4 – Axial Resistance of Pile Sections Founded on Very Dense Glacial Till Soils

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

8.4.2 Pile Tips

Due to the possible presence of cobbles and boulders in the glacial sand and silt till and silt till layers 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.

8.4.3 Pile Installation

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

The Contract Documents must contain a NSSP alerting the Bidders to:

- The presence of cobbles and boulders in the expected bearing stratum.
- The possibility of piles within a group achieving the specified resistance at different elevations.
- The possibility of some piles meeting refusal on a large boulder.

Suggested texts for the NSSP's are included in Appendix E.

8.4.4 Pile Driving

Pile driving must be controlled by the Hiley Formula and an ultimate pile resistance to be specified by the designer in accordance with Clause 3.3.2 (b) Construction Stage of the Structural Manual. The Hiley formula need not be used until the piles are 2.0 m above the design pile tip elevations. The appropriate pile driving note is "Piles to be driven in accordance with Standard SS 103-11 using an ultimate resistance of "R" kN per pile". "R" must have the minimum values shown in Table 8.5.

Table 8.5 – Ultimate Geotechnical Resistance of Piles

Pile	Ultimate Resistance (R) (kN)
HP 310x110	3,200
HP 360x132	3,600

The NSSP should require the QVE to terminate driving before the pile is damaged by overdriving.

To facilitate pile installation, embankment fill through which piles will be driven must not contain oversize material, i.e. no particles exceeding 75 mm in size.

Embankment fill must be placed prior to pile driving.

Boreholes 08-50 and 08-54 show water level from 0.2 m to 0.7 m above the ground surface, elevations 264.6 to 261.7 (artesian condition). Artesian pressure has the potential to cause flow up the pile shaft, with accompanying loss of fines.

At this site, however, Doane Road will be constructed on 5.1 m to 9.7 m high approach embankments and piles will be driven through these approach fills. Since these fills will be above the artesian water elevation, no upward flow around the piles due to artesian pressure is anticipated.

8.4.5 Downdrag

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

8.4.6 Integral Abutment Considerations

The ground conditions at this site are considered suitable for an integral abutment design. The use of H-piles at the abutments allows for the design of an integral abutment structure.

After each pile is driven, the space between the pile and the CSP should be filled with sand. An NSSP must be included in the contract drawings specifying the gradation of the sand according to Table 8.6.

Table 8.6 – Integral Abutment Sand Grading

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

8.4.7 Lateral Resistance

For the cohesionless soils encountered, the lateral resistance of the pile may be calculated using a value for the coefficient of horizontal subgrade reaction (k_s) and ultimate lateral resistance (p_{ult}) as follows:

$$k_s = n_h \,.\, z / D \qquad (\text{kN/m}^3)$$

$$p_{ult} = 3 \,.\, \gamma \,.\, z \,.\, K_p \qquad (\text{kPa})$$

where
- z = depth of embedment of pile in metres
- D = pile width in metres
- n_h = value from Table 8.7
- γ = unit weight (Table 8.7)
- K_p = passive earth pressure coefficient (Table 8.7)

The lateral resistance of the piles for cohesive clayey silt soils, contacted in Borehole 08-54, may be calculated using a value for the coefficient of horizontal subgrade reaction (k_s) and ultimate lateral resistance (p_{ult}) as follows:

k_s = $67*S_u/D$ (kN/m^3)

p_{ult} = $9*S_u$ (kPa) at and below a depth of 3*D (m) reduced to zero at the ground surface

where

D = pile width in metres

S_u = undrained shear strength (kPa) (Table 8.7)

The above equations and recommended parameters may be used to analyze the interaction between a pile and the surrounding soil. The lateral pressures obtained from the analysis should not exceed the ultimate lateral resistance.

The spring constant, K, for analysis may be obtained by the expression, $K = k_s*L*D$ (kN/m), where k_s is the coefficient of horizontal subgrade reaction (kN/m^3), D is the pile width (m) and L is the length (m) of the pile segment or element used in the analysis. The ultimate lateral resistance on any one segment of pile, P_{ult}, may be obtained from the expression, $P_{ult} = p_{ult}*L*D$. This represents the ultimate load at which the pile fails and will not support any additional lateral load at greater displacements. It is recommended, however, that the total lateral resistance in one pile be limited to no more than 150 kN at ULS and 50 kN at SLS. Parameters for lateral pile resistance are shown in Table 8.7.

Table 8.7 – Parameters for Lateral Pile Resistance

Location	Depth	Elevation	n_h (kN/m^3)	S_u kPa	K_p	Unit Weight (kN/m^3)	Soil Conditions
West Abutment	0.0 to 1.0	OGL to 265.1	1,500	-	3.0	21	Compact sand fill
	1.0 to 4.0	265.1 to 262.1	6,500	-	3.2	21	Compact to very dense sand and silt till
	Below 4.0	Below 262.1	10,000	-	3.4	11*	Very dense sand and silt till, silt till
East Abutment	0.0 to 2.0	OGL to 259.0	2,200	-	3.0	21	Loose to compact sand and silt till
	2.0 to 6.0	259.0 to 255.0	-	100	2.8	10*	Very stiff to very stiff clayey silt till
	6.0 – 7.5	255.0 to 253.5	7,000	-	3.2	11*	Dense sand and silt till
	Below 7.5	Below 253.5	10,000	-	3.4	11*	Very dense sand and silt till

*Buoyant unit weight below the water table.

Pile interaction should be considered with reference to CHBDC Clause 6.8.9.2.

For lateral soil/pile group interaction analysis, the modulus of subgrade reaction (k_s) may have to be reduced based on the pile spacing.

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

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

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

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

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

Intermediate values may be obtained by interpolation.

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

8.5 Recommended Foundation

From a geotechnical perspective, and based on current information, it is recommended that foundations for the bridge abutments and pier be supported on steel H-piles driven into the very dense glacial till soil.

8.6 Frost Cover

The design depth of frost penetration at this site is 1.4 m.

Frost protection must be provided for the undersides of all foundation elements and must consist of a minimum of 1.4 m of soil cover.

9 RETAINED SOIL SYSTEMS

If Retained Soil System (RSS) walls are incorporated in the design, the soil conditions encountered at the site are considered suitable for the support of RSS walls at the east and west approaches/abutments. Details of RSS walls were not provided at the time of preparation of this report.

The borehole information indicates that the foundation conditions at the possible RSS wall locations comprised compact to dense sand and silt till overlying an extensive deposit of compact to very dense silt till. In borehole 08-53, a 1.8-m thick layer of sand and clay fill was contacted surficially. In Borehole 08-54, a layer of stiff to very stiff clayey silt was encountered within the sand and silt till.

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

The performance of a RSS is dependent on, among other factors, the characteristics of its foundation. Failure to provide an adequate foundation may lead to settlement and distortion of the RSS and, in severe cases, to possible failure of the system. It is critical that the RSS walls are not subject to settlement due to compression of the foundation soils and embankment fill. The foundation of the entire RSS mass must be considered, i.e. from the face of the wall to the furthest extent of the reinforcement.

To provide an acceptable foundation performance, the RSS mass must be founded on the native undisturbed soils. The highest base levels for the underside of the wall and the soil type at the base levels are indicated in Table 9.1.

Table 9.1 – Maximum Elevation at Underside of Wall Base or Granular A Fill

Foundation Element	Borehole	Depth (m)	Elevation (m)	Soil
West	08-49	0.8	265.3*	Compact sand and silt till
	08-50	0.7	263.7*	Compact sand and silt till
East	08-53	2.9	259.2	Compact sand and silt till
	08-54	2.3	258.7	Stiff to very stiff clayey silt till

*Above the proposed Hwy 404 grade near Elevations 263.0 to 264.0.

A RSS wall founded on native compact sand and silt till or stiff to very stiff clayey silt till at or below elevations shown in Table 9.1 should be designed for a factored bearing resistance of 300 kPa at ULS and a bearing resistance of 200 kPa at SLS.

Alternatively, the RSS may be founded on engineered fill founded on the native soils contacted at the above elevations. Engineered fill placed under the RSS mass to achieve the design founding level should consists of OPSS Granular "A" compacted to 100% of its SPMDD at a moisture content within 2% of optimum. The engineered pad must extend at least 500 mm beyond the limits of the RSS mass and levelling strip.

The geotechnical resistances provided above are for concentric, vertical loading. The effects of load inclination and eccentricity need to be taken into account according to the CHBDC 2006 Section 6.7.

The entire block of reinforced earth must be designed against various modes of failure including sliding and overturning. Sliding resistance along the base of the wall on engineered granular fill may be estimated using an ultimate friction coefficient of 0.55. For an RSS block founded on native cohesive or cohesionless soils, coefficient of sliding friction of 0.45 may be used.

Topsoil, loose fill, and any soft/wet native material should be stripped from the footprint of the RSS. The native soil under the RSS foundation should be proofrolled to detect and replace any soft areas.

The highest permitted east and west wall base elevations given in Table 9.1 are 0.9 m to 3.0 m below the groundwater table on this site. If a temporary excavation required to construct the RSS walls extends below the water table, a dewatering plan must be in place to construct the underside of the wall base in the dry and to prevent sloughing of the sides or disturbance of the base of the excavation due to the inflow of groundwater.

The supplier of the proprietary RSS system must demonstrate that it will meet the Ministry's specifications for performance and appearance. The RSS supplier/designer may specify more stringent criteria or other requirements related to the particular design. The internal stability of the RSS wall should be analyzed by the supplier/designer of the proprietary product selected for this site.

If a RSS wall system is selected, the global stability must be analyzed after the location of the wall is known. The global stability of the RSS wall is dependent on the characteristics of the embankment fill and the foundation soils, the geometry of the embankment and location of the RSS within the embankment. Global stability should not be a concern for a RSS wall founded on the native soils at this site.

10 EXCAVATION

All excavation must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the fill is classed as Type 3 soil. Due to the loose to compact conditions, the native soils within the probable depth of excavation at this site may be classed as Type 3 soils, and Type 4 below the water table.

Excavation of the cohesionless native foundation soils below the groundwater level without prior dewatering is not recommended since the inflow of groundwater will cause boiling and sloughing of the soil below the water table making it difficult to maintain a dry, sound base on which to work.

Prior to excavation below the natural groundwater level, the groundwater must be depressed to a level below the deepest excavation level sufficient to maintain a stable base and prevent soil disturbance by construction traffic.

The excavation of the cohesionless foundation soils and backfilling for foundations must be carried out in accordance with SP 902S01.

Bidders must be alerted to the fact that excavation must be carried out through till soils, which may include cobbles and boulders.

11 UNWATERING

The groundwater level is high at this site. Piezometers installed in boreholes revealed that the groundwater level is near elevations 264.6 to 261.7. However, seepage may occur from perched zones in the granular fill, or from sand and silt pockets/lenses in the underlying heterogeneous till deposits.

If spread footings are selected for foundations, it is anticipated that excavation will be close to or extend below groundwater table and dewatering to lower the groundwater level below the footing excavation base will be required. The Contractor should also be prepared to pump from sumps to remove any remaining seepage water or surface water collecting in an excavation. Placement of concrete or compacting granular engineered fill must be done in the dry. Unwatering must remain operational and effective until the footing is constructed and backfilled.

The design of the dewatering system that may be required is the responsibility of the Contractor and the Contract Documents must alert him to this responsibility and the need to engage a dewatering specialist. While the responsibility for dewatering remains with the Contractor, suitable systems that might be employed for shallow excavations below water table, include pumping from filtered sumps. Possible use of vacuum wellpoints will be required for deeper excavations penetrating more than 0.5 m below the groundwater level.

12 APPROACH EMBANKMENTS

Approach embankment construction using either granular fill or SSM is feasible on the foundation soils encountered at this site. At the west abutment, settlement in the order of 10 mm to 20 mm is estimated in the foundation soils under the loading imposed by approximately 5.1 m of the approach fill. Similarly at the east abutment, 40 mm to 50 mm of settlement is estimated in the foundation soils under the loading imposed by 9.7 m of approach fill. Due to the non cohesive nature of the foundation soils, these settlements will be immediate and essentially completed when construction of the fill is completed. However, it is recommended that the approach embankments be constructed at least one month in advance of pile driving in order to reduce the possible horizontal displacement of piles.

At the east abutment, post construction settlement of the fill mass is estimated to be as high as 0.5% of the embankment height, approximately 50 mm. Based on the above settlement estimates, it is considered prudent to overbuild the approach embankment to account for a total settlement of 100 mm.

The global, internal and surficial stability of the approach embankment fills will depend on the slope geometry and also to a large degree on the material used to construct the embankments. Embankments constructed using granular material, select subgrade material or non-cohesive earth fill will have stable side slopes at inclinations of up to 2H:1V.

For the purpose of 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 granular 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 12.1. Slope stability computation outputs are included in Appendix F.

Table 12.1 Computed Factors of Safety

Location / Material	Condition	Factor of Safety	Figure (Appendix F)
West Approach – 5m			
Earth Fill	Normal	1.6	1
Earth Fill	Seismic = 0.08g	1.3	2
East Approach - 10 m			
Earth Fill	Normal	1.4	3
Earth Fill	Seismic = 0.08g	1.1	4

In each case of normal loading, the factor of safety against global failure was equal to or greater than 1.4. Under the assumed seismic loading, the minimum factor of safety calculated was 1.1. These factors of safety are considered to be acceptable for the proposed embankment bearing on non-cohesive soil.

It is recommended that all topsoil or deleterious material be stripped prior to constructing the approach fills. Embankment construction should be in accordance with OPSS 206, as amended by Special Provision "Amendment to OPSS 206, December 1993", dated November 2002.

Where earth fill embankments are higher than 8 m, mid-height berms should be incorporated in each 8 m vertical interval. 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.

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

13 BACKFILL TO ABUTMENTS

In the case of integral or semi-integral abutments, backfill to the abutment must be granular material.

In the case of a conventional abutment, granular backfill is recommended.

The backfill to the abutment walls must be in accordance with OPSS 902 as amended by Special Provision 902S01. Granular backfill must be placed to the extents shown in OPSD 3101.150. All granular material should meet the requirements of SP 110F13 Amendment to OPSS 1010, March 1993.

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

The design of the abutment must incorporate a subdrain as shown in OPSD 3101.150 or OPSD 3101.200, as applicable.

14 EARTH PRESSURE

Earth pressures acting on the structure may be assumed to be triangular and to be governed by the characteristics of the abutment backfill. For a fully drained condition, the pressures should be computed in accordance with the CHBDC but generally are given by the expression:

$$P_h = K^*(\gamma h + q)$$

Where:

P_h = horizontal pressure on the wall at depth h (kPa)

K = earth pressure coefficient (see Table 14.1 below)

γ = unit weight of retained soil (see Table 14.1 below)

h = depth below top of fill where pressure is computed (m)

q = value of any surcharge (kPa)

In accordance with Clause 6.9.3 of the CHBDC, a compaction surcharge should be added. The magnitude should be 12 kPa at the top of fill and decreasing to 0 kPa at a depth of 2.0 m for Granular B Type I or 1.7 m for Granular A or Granular B Type II.

Earth pressure coefficients for backfill to the abutment wall are dependent on the material used as backfill. Typical values are shown in Table 14.1.

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

The factors in Table 14.1 are "ultimate" values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.9.1 (a) in the Commentary to the Canadian Highway Bridge Design Code.

Table 14.1 – Earth Pressure Coefficient (K)

Condition	Earth Pressure Coefficient (K)			
	OPSS Granular A or OPSS Granular B Type II $\phi = 35°, \gamma = 22.8$ kN/m^3		OPSS Granular B Type I $\phi = 32°, \gamma = 21.2$ kN/m^3	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (Unrestrained Wall)	0.27	0.40*	0.31	0.48*
At rest (Restrained Wall)	0.43	-	0.47	-
Passive (Movement Towards Soil Mass)	3.7	-	3.3	-

* For wing walls.

15 SEISMIC CONSIDERATIONS

15.1 Seismic Design Parameters

The site is treated as lying in Seismic Zone 1. The following seismic parameters should be used for design:

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

The soil profile type at this site has been classified as Type II. Therefore, according to Table 4.4 of the CHBDC, a Site Coefficient "S" (ground motion amplification factor) of 1.2 should be used in seismic design.

15.2 Liquefaction Potential

The potential for liquefaction of the foundations soils was assessed using the Seed and Idriss (1971) method[1].

Using this method, it is estimated that under the existing conditions the foundation soils at both abutments and pier are not prone to liquefaction.

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

If the structure is supported on steel piles, the foundation loads will be transferred by the steel piles to very dense sand and silt till. It is not considered likely that the vertical geotechnical resistance of the piles will be compromised.

The embankments themselves will be constructed above the groundwater level and are not considered to be in danger of undergoing liquefaction.

15.3 Retaining Wall Dynamic Earth Pressures

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

In calculating the active, passive and at rest earth pressure coefficients the angle of friction between the wall and backfill material is assumed to be 0.5 ϕ. For the design of retaining walls, the coefficients of horizontal earth pressure in Table 15.1 may be used.

Table 15.1 – Earth Pressure Coefficient (K) for Earthquake Loading

Wall Condition	Granular A or Granular B Type II $\phi = 35°$; $\delta = 17.5°$ $\gamma = 22.8$ kN/m^3		OPSS Granular B Type I $\phi = 32°$; $\delta = 16°$ $\gamma = 21.2$ kN/m^3	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (K_{AE})*	0.3	0.47	0.34	0.58
Passive (K_{PE})	3.6	-	3.2	-
At Rest (K_{OE})**	0.53	-	0.58	-

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

16 CONSTRUCTION CONCERNS

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

1. **Pile refusal at higher elevation.**

   Glacial till deposits inherently contain boulders and there was some evidence of their presence during drilling. 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 2 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 retap 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.

In some circumstances, particularly at the pier, the piles may achieve refusal prior to being driven to sufficient depth. In this case, the contractor must pre-auger to within 1m of the anticipated founding elevation prior to driving piles.

3. Artesian water flow during pile driving.

Groundwater levels were measured above the existing ground level. At this site, the piles will be driven through the east and west approach fills. Since these fills will be above the artesian water elevation, no upward flow around the piles due to artesian pressure is anticipated.

However, if artesian groundwater flow is observed during pile driving, or any other construction activities, the contractor or QVE must immediately advise the CA. If the CA agrees there are concerns regarding the artesian flow, the issue should be referred to the design team.

4. Destabilization of excavations

If excavation is carried out 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.

17 CLOSURE

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

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

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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[1] '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_{pen} 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

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

SYMBOLS

CLAYSTONE

SILTSTONE

SANDSTONE

COAL

Bedrock (general)

DISCONTINUITY SPACING

Bedding	**Bedding Plane Spacing**
Very thickly bedded	Greater than 2m
Thickly bedded	0.6 to 2m
Medium bedded	0.2 to 0.6m
Thinly bedded	60mm to 0.2m
Very thinly bedded	20 to 60mm
Laminated	6 to 20mm
Thinly Laminated	Less than 6mm

TERMS

Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.
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.
Uniaxial Compressive Strength (UCS)	Axial stress required to break the specimen
Fracture Index: (FI)	Frequency of natural fractures per 0.3m of core run.

STRENGTH CLASSIFICATION

Rock Strength	**Approximate Uniaxial Compressive Strength** (MPa)	(psi)	**Field Estimation of Hardness***
Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Medium Strong	25.0 to 50.0	3,500 to 7,500	Breaks under single blow of geological hammer.
Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail

RECORD OF BOREHOLE No 08-48

METRIC

G.W.P.	2109-05-00
HWY	404
DATUM	Geodetic
LOCATION	Doane Rd. N 4 887 093.56 E 310 301.81
BOREHOLE TYPE	Solid Stem Augers
DATE	2008.10.22 - 2008.10.22
ORIGINATED BY	ES
COMPILED BY	AN
CHECKED BY	RPR

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
265.6 / 0.0	ASPHALT (50mm)						
	SAND, some silt to silty, some gravel, Compact, Brown to Dark Brown, Moist (FILL)	1	SS	13		265	
264.9 / 0.8	SAND and SILT, some clay, trace gravel, occasional oxide staining, Loose to Compact, Brown, Moist (TILL)	2	SS	8			
		3	SS	13		264	1 36 52 11
263.2 / 2.4	Gravelly SAND, some silt, some clay, Dense, Brown, Moist	4	SS	35		263	0 46 50 4
		5	SS	50			23 64 13 (SI+CL)
	Coobles at 3.5m					262	
261.5 / 4.1	SILT, some sand to sandy, Very Dense, Grey, Wet (TILL)	6	SS	140		261	
					∇	260	
		7	SS	180/ .275		259	0 28 63 9
		8	SS	185/ .250		258	
257.8 / 7.9	END OF BOREHOLE AT 7.9m. BOREHOLE OPEN AND WATER LEVEL AT 6.5m UPON COMPLETION OF DRILLING. BOREHOLE BACKFILLED WITH HOLEPLUG TO 0.07m THEN ASPHALT TO SURFACE.						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X LAB VANE; 40 80 120 160 200
PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60
UNIT WEIGHT γ kN/m³

+ ³, × ³ : Numbers refer to Sensitivity   15 -⊕- 5 (20, 10) (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 08-49

METRIC

G.W.P. 2109-05-00
LOCATION Doane Rd. N 4 887 108.93 E 310 311.68
ORIGINATED BY ES

HWY 404
BOREHOLE TYPE Solid Stem Augers
COMPILED BY AN

DATUM Geodetic
DATE 2008.10.22 - 2008.10.22
CHECKED BY RPR

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
266.1 / 0.0	**ASPHALT** (25mm)					
	SAND, some gravel, Compact, Brown, Moist (FILL)	1	SS	23		
265.5 / 0.6	**SAND** and **SILT**, some clay, trace gravel, occasional oxidize staining, Compact to Dense, Brown, Moist (TILL)	2	SS	25		1 35 52 12
		3	SS	28		
		4	SS	33		
263.0 / 3.0	**SILT**, some sand, trace clay, occasional cobbles, occasional oxide staining, Compact to Very Dense, Brown, Moist (TILL)	5	SS	24		0 2 81 17
	Layer of sand (800mm), Wet					
	Brown to Grey	6	SS	135		0 39 56 5
	Some sand	7	SS	188/ .275		0 19 73 8
					∇ (water level)	
		8	SS	100/ .150		
256.8		9	SS	100/ .125		
9.3	END OF BOREHOLE AT 9.3m. BOREHOLE OPEN TO 8.2m AND WATER LEVEL AT 7.1m UPON COMPLETION OF DRILLING. BOREHOLE BACKFILLED WITH					

ELEVATION SCALE: 266, 265, 264, 263, 262, 261, 260, 259, 258, 257

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE — 40 80 120 160 200

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³

Continued Next Page

+³, ×³: Numbers refer to Sensitivity — 20 / 15 ◇ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 08-49

METRIC

G.W.P. 2109-05-00 LOCATION Doane Rd. N 4 887 108.93 E 310 311.68 ORIGINATED BY ES

HWY 404 BOREHOLE TYPE Solid Stem Augers COMPILED BY AN

DATUM Geodetic DATE 2008.10.22 - 2008.10.22 CHECKED BY RPR

SOIL PROFILE		SAMPLES						DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X LAB VANE (40 80 120 160 200)	PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				
	Continued From Previous Page										
	HOLEPLUG TO 0.05m THEN ASPHALT TO SURFACE.										

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$+^3$, $\times^3$: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 08-50

METRIC

G.W.P. 2109-05-00	LOCATION Doane Rd. N 4 887 090.43 E 310 327.05		ORIGINATED BY ES
HWY 404	BOREHOLE TYPE Hollow Stem Augers		COMPILED BY AN
DATUM Geodetic	DATE 2008.10.20 - 2008.10.20		CHECKED BY RPR

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE — 40 80 120 160 200

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
264.4	
0.0	**TOPSOIL** (50mm)
	SAND and **SILT**, trace clay, trace roots Compact Dark Brown to Brown Moist (TILL) Possible cobbles at 0.4m Layer of sand (200mm) Occasional cobbles and boulders Very Dense
262.2 / 2.1	**SILT**, some sand, trace to some clay Very Dense Brown Moist (TILL) Layer of silty sand (400mm) Grey Wet
255.0 / 9.4	END OF BOREHOLE AT 9.4m. BOREHOLE OPEN AND WATER LEVEL AT 5.0m UPON COMPLETION OF DRILLING.

SAMPLES

NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
1	SS	10				
2	SS	26				
3	SS	58				
4	SS	77	0	15	73	12
5	SS	92				
6	SS	100/ .200	1	68	26	5
7	SS	100/ .225	1	14	78	7
8	SS	100/ .200				
9	SS	180/ .225				

ELEVATION SCALE: 264, 263, 262, 261, 260, 259, 258, 257, 256

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity

15 20 5 10 (%) STRAIN AT FAILURE

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Ministry of Transportation Ontario

RECORD OF BOREHOLE No 08-50

METRIC

G.W.P. 2109-05-00 | LOCATION Doane Rd. N 4 887 090.43 E 310 327.05 | ORIGINATED BY ES

HWY 404 | BOREHOLE TYPE Hollow Stem Augers | COMPILED BY AN

DATUM Geodetic | DATE 2008.10.20 - 2008.10.20 | CHECKED BY RPR

SOIL PROFILE			SAMPLES						DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X LAB VANE; 40 80 120 160 200)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE					
	Continued From Previous Page											
	Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.											

WATER LEVEL READINGS:

DATE	DEPTH (m)	ELEV. (m)
2008.10.24	4.4	260.0
2008.11.28	4.9	259.5
2009.02.06	0.1	264.3
2009.02.20	0.2*	264.6
2009.03.20	1.0	263.4
2009.04.22	1.1	263.3
2009.09.02	2.6	261.8

* (above ground surface)

+ ³, × ³: Numbers refer to Sensitivity   20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 08-51

METRIC

G.W.P. 2109-05-00
LOCATION Doane Rd. N 4 887 113.65 E 310 353.28
ORIGINATED BY ES

HWY 404
BOREHOLE TYPE Solid Stem Augers
COMPILED BY AN

DATUM Geodetic
DATE 2008.10.23 - 2008.10.23
CHECKED BY RPR

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
263.6					
0.0	**ASPHALT** (25mm)				
	Silty **SAND**, trace to some gravel, Compact, Brown, Moist (FILL), Occasional cobbles at 0.9m	1	SS	27	
		2	SS	15	
		3	SS	37	
261.4 / 2.3	**SAND** and **SILT**, trace to some clay, trace gravel, occasional oxide staining, Very Dense, Brown, Moist (TILL)	4	SS	116	3 34 51 12
		5	SS	115	
	Layer of fine sand (1.2m), Brown to Grey	6	SS	133	0 56 38 6
		7	SS	100/ .150	0 32 56 11
	Layer of fine sand (0.9m)	8	SS	83	
		9	SS	100/ .75	4 43 46 7
	Hard augering, auger grinding				

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE (263, 262, 261, 260, 259, 258, 257, 256, 255, 254); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 40 80 120 160 200); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (20 40 60); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

ONTMT4S 0596.GPJ 8/26/09

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 08-51

METRIC

G.W.P. 2109-05-00 LOCATION Doane Rd. N 4 887 113.65 E 310 353.28 ORIGINATED BY ES

HWY 404 BOREHOLE TYPE Solid Stem Augers COMPILED BY AN

DATUM Geodetic DATE 2008.10.23 - 2008.10.23 CHECKED BY RPR

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 40 80 120 160 200	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page										
	SAND and SILT, trace gravel, occasional sand pockets Very Dense Grey (TILL)		10	SS	100/ .75		253		○		
							252				
251.3 12.3			11	SS	100/ .100				○		
	END OF BOREHOLE AT 12.3m. BOREHOLE OPEN TO 8.1m 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)
2008.11.28	4.1	259.5

$+^3, \times^3$: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 08-52

METRIC

G.W.P.	2109-05-00	LOCATION	Doane Rd. N 4 887 100.05 E 310 368.44	ORIGINATED BY	ES
HWY	404	BOREHOLE TYPE	Hollow Stem Augers	COMPILED BY	AN
DATUM	Geodetic	DATE	2008.10.20 - 2008.10.21	CHECKED BY	RPR

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
262.4							
0.0	TOPSOIL (50mm)						
	SAND and SILT, trace to some clay, trace gravel, occasional rootlets, occasional oxide staining Loose Dark Brown to Brown Moist (TILL)	1	SS	10		262	
		2	SS	5			3 34 51 13
	Compact Grey	3	SS	12		261	
	Dense Brown to Grey Wet	4	SS	41		260	0 48 48 5
	Occasional cobbles Very Dense	5	SS	96		259	
	Brown to Grey	6	SS	100/ .125	∇	258	
		7	SS	100/ .125		257, 256	
		8	SS	100/ .125		255, 254	
252.8		9	SS	175/ .250		253	1 33 57 9
9.5	END OF BOREHOLE AT 9.5m. WATER LEVEL AT 4.6m UPON COMPLETION OF DRILLING.						

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DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 40 80 120 160 200
PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60
UNIT WEIGHT γ kN/m³

+³, ×³: Numbers refer to Sensitivity   15-⊕-5 (20, 10) (%) STRAIN AT FAILURE

ONTMT4S 0596.GPJ 8/26/09

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 08-52

METRIC

G.W.P. 2109-05-00 LOCATION Doane Rd. N 4 887 100.05 E 310 368.44 ORIGINATED BY ES

HWY 404 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN

DATUM Geodetic DATE 2008.10.20 - 2008.10.21 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 40 80 120 160 200	PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	Continued From Previous Page										
	BOREHOLE BACKFILLED WITH HOLEPLUG TO 3.0m THEN AUGER CUTTINGS TO SURFACE.										

$+^3, \times^3$: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 08-53

METRIC

G.W.P. 2109-05-00 LOCATION Doane Rd. N 4 887 127.36 E 310 398.80 ORIGINATED BY ES
HWY 404 BOREHOLE TYPE Solid Stem Augers COMPILED BY AN
DATUM Geodetic DATE 2008.10.23 - 2008.10.23 CHECKED BY RPR

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
262.1							
0.0	**ASPHALT** (50mm)						
	SAND, some gravel, some silt to silty Compact Brown to Dark Brown Moist (FILL)	1	SS	25		262	
261.4 0.8	Silty **CLAY**, trace sand, occasional silt seams, occasional oxide staining Firm to Stiff Brown (FILL)	2	SS	12		261	
260.4 1.8	**SAND and SILT**, trace to some clay, trace gravel, occasional sand seams Loose to Compact Brown to Grey Moist (TILL)	3	SS	7		260	0 4 68 28
		4	SS	10			
		5	SS	14	∇	259	6 36 45 13
	Very Dense	6	SS	97		258	
	Occasional sand pockets	7	SS	100/ .175		257 / 256	4 33 53 9
	Layer of fine sand (500mm) Occasional cobbles Grey	8	SS	178/ .225		255 / 254	
		9	SS	100/ .100		253	4 40 48 8

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE: 40 80 120 160 200
PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L
WATER CONTENT (%): 20 40 60
UNIT WEIGHT γ kN/m³

Continued Next Page

+ ³, × ³: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 08-53

METRIC

G.W.P.	2109-05-00	LOCATION	Doane Rd. N 4 887 127.36 E 310 398.80	ORIGINATED BY	ES
HWY	404	BOREHOLE TYPE	Solid Stem Augers	COMPILED BY	AN
DATUM	Geodetic	DATE	2008.10.23 - 2008.10.23	CHECKED BY	RPR

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X LAB VANE; 40 80 120 160 200)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page										
	SAND and **SILT**, some clay, trace gravel Very Dense Grey Moist						252				
251.4 10.8	(TILL)		10	SS	100/ .100				o		
	END OF BOREHOLE AT 10.8m. BOREHOLE OPEN TO 4.8m AND WATER LEVEL AT 3.0m UPON COMPLETION OF DRILLING. BOREHOLE BACKFILLED WITH HOLEPLUG TO 0.07m THEN ASPHALT TO SURFACE.										

$+^3, \times^3$: Numbers refer to Sensitivity

15-⊕-5 (20 above, 10 below) (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 08-54

METRIC

G.W.P. 2109-05-00 | LOCATION Doane Rd. N 4 887 112.68 E 310 413.77 | ORIGINATED BY ES

HWY 404 | BOREHOLE TYPE Hollow Stem Augers | COMPILED BY AN

DATUM Geodetic | DATE 2008.10.21 - 2008.10.24 | CHECKED BY RPR

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
261.0					
0.0	**TOPSOIL** (50mm)				
	SAND and **SILT**, trace clay, occasional oxide staining Loose Dark Brown to Brown Moist (TILL)	1	SS	8	
		2	SS	9	
		3	SS	7	
258.7 2.3	Clayey **SILT**, trace sand Stiff to Very Stiff Brown to Grey (TILL)	4	SS	14	0 1 77 22
		5	SS	26	
		6	SS	14	
255.4 5.6	**SAND** and **SILT**, trace gravel, trace clay Dense to Very Dense Grey Moist (TILL)	7	SS	34	
	Possible boulder at 7.3m. Layer of fine sand (400mm).	8	SS	158/ .125	8 41 44 7
		9	SS	100/ .150	

Elevation scale: 261, 260, 259, 258, 257, 256, 255, 254, 253, 252

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: 40 80 120 160 200
O UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB VANE

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

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

15 ⊕ 5 (20 above, 10 below) (%) STRAIN AT FAILURE

ONTMT4S 0596.GPJ 9/9/09

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 08-54

METRIC

G.W.P. 2109-05-00	LOCATION Doane Rd. N 4 887 112.68 E 310 413.77	ORIGINATED BY ES
HWY 404	BOREHOLE TYPE Hollow Stem Augers	COMPILED BY AN
DATUM Geodetic	DATE 2008.10.21 - 2008.10.24	CHECKED BY RPR

ELEV / DEPTH	DESCRIPTION (Continued From Previous Page)	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
	SILT, some sand, some clay, trace gravel Very Dense Grey Moist (TILL)						251						
			10	SS	100/ .100			o		3	17	65	15
							250						
							249						
248.7			11	SS	100/ .100			o					
12.3	END OF BOREHOLE AT 12.3m. BOREHOLE OPEN TO 9.7m AND WATER LEVEL AT 2.1m UPON COMPLETION OF DRILLING. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.												

Chart headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 40 80 120 160 200; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; REMARKS & GRAIN SIZE DISTRIBUTION (%)

WATER LEVEL READINGS:

DATE	DEPTH (m)	ELEV. (m)
2008.11.28	3.7	257.3
2009.02.06	0.0	261.0
2009.02.20	0.4*	261.4
2009.03.20	0.7*	261.7
2009.04.22	0.6*	261.6
2009.09.02	0.6	260.4

* (above ground surface)

+ 3, × 3: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 08-55

METRIC

G.W.P. 2109-05-00 | LOCATION Doane Rd. N 4 887 131.71 E 310 424.00 | ORIGINATED BY ES

HWY 404 | BOREHOLE TYPE Solid Stem Augers | COMPILED BY AN

DATUM Geodetic | DATE 2008.10.22 - 2008.10.22 | CHECKED BY RPR

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	WATER CONTENT (%)	GR	SA	SI	CL
261.7										
0.0	**ASPHALT** (50mm)									
	SAND, some gravel, some silt to silty, occasional oxide staining Compact to Very Dense Brown Moist (FILL)	1	SS	73		o				
260.5		2	SS	19		o				
1.1	**SILT**, some clay, some sand, occasional oxide staining Loose to Compact Brown Moist (TILL)	3	SS	8		o	0	13	74	13
	Occasional coarse sand pockets	4	SS	15		o	1	42	45	12
		5	SS	18		o				
					∇					
		6	SS	18		o				
	Sandy Grey	7	SS	19		⊢o⊣	6	25	55	15
	Occasional cobbles Very Dense	8	SS	118		o				
252.4		9	SS	100/ .150		o				
9.3	END OF BOREHOLE AT 9.3m. BOREHOLE OPEN AND WATER LEVEL AT 4.2m UPON COMPLETION OF DRILLING. BOREHOLE BACKFILLED WITH									

Elevation scale: 261, 260, 259, 258, 257, 256, 255, 254, 253

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: o UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 40 80 120 160 200

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

THURBER

RECORD OF BOREHOLE No 08-55

METRIC

G.W.P. 2109-05-00	LOCATION Doane Rd. N 4 887 131.71 E 310 424.00	ORIGINATED BY ES
HWY 404	BOREHOLE TYPE Solid Stem Augers	COMPILED BY AN
DATUM Geodetic	DATE 2008.10.22 - 2008.10.22	CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X LAB VANE (40 80 120 160 200)	PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	Continued From Previous Page										
	HOLEPLUG TO 0.2m, AUGER CUTTINGS TO 0.05m THEN ASPHALT TO SURFACE.										

$+^3, \times^3$: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

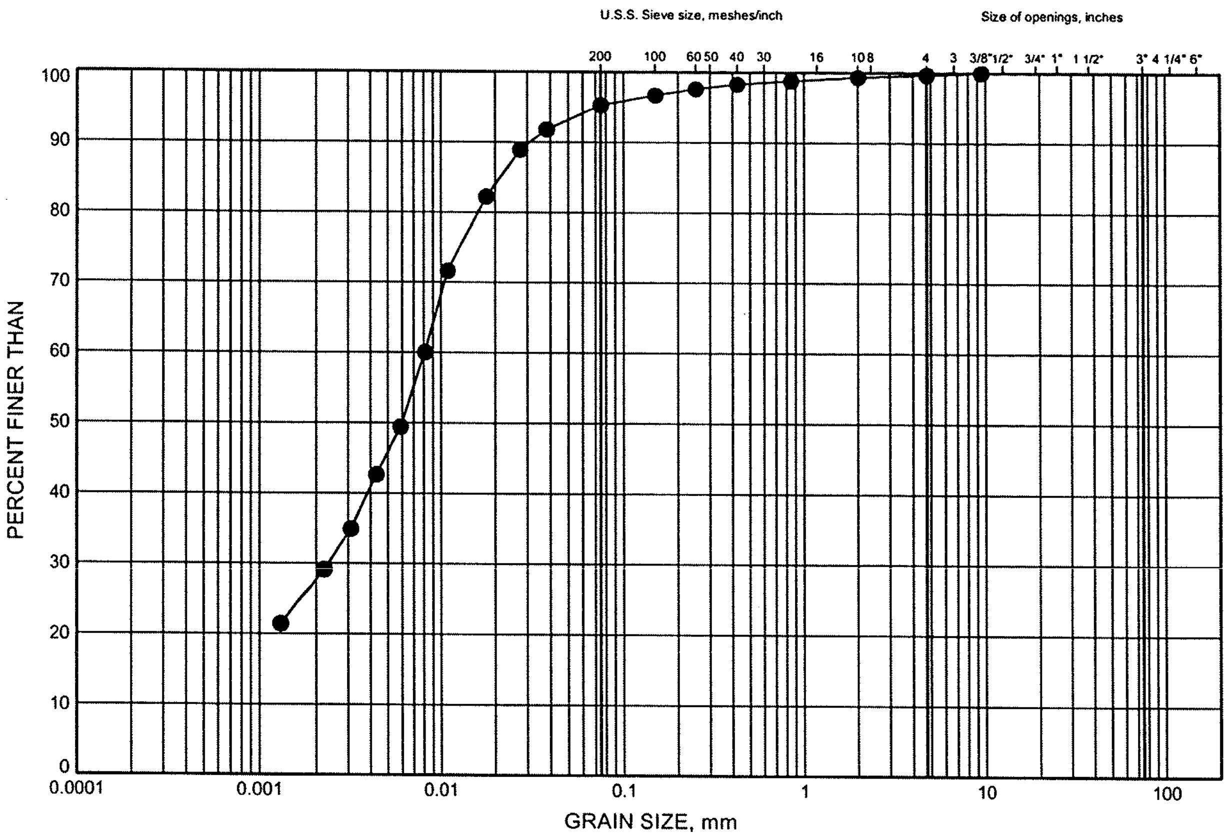
Appendix B

Laboratory Test Results

Hwy 404 Extension
GRAIN SIZE DISTRIBUTION

FIGURE B1

Silty Clay (FILL)



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	08-53	1.83	260.31

W.P.# 19-1605-96
Prepared By AN
Checked By RPR

THURBER

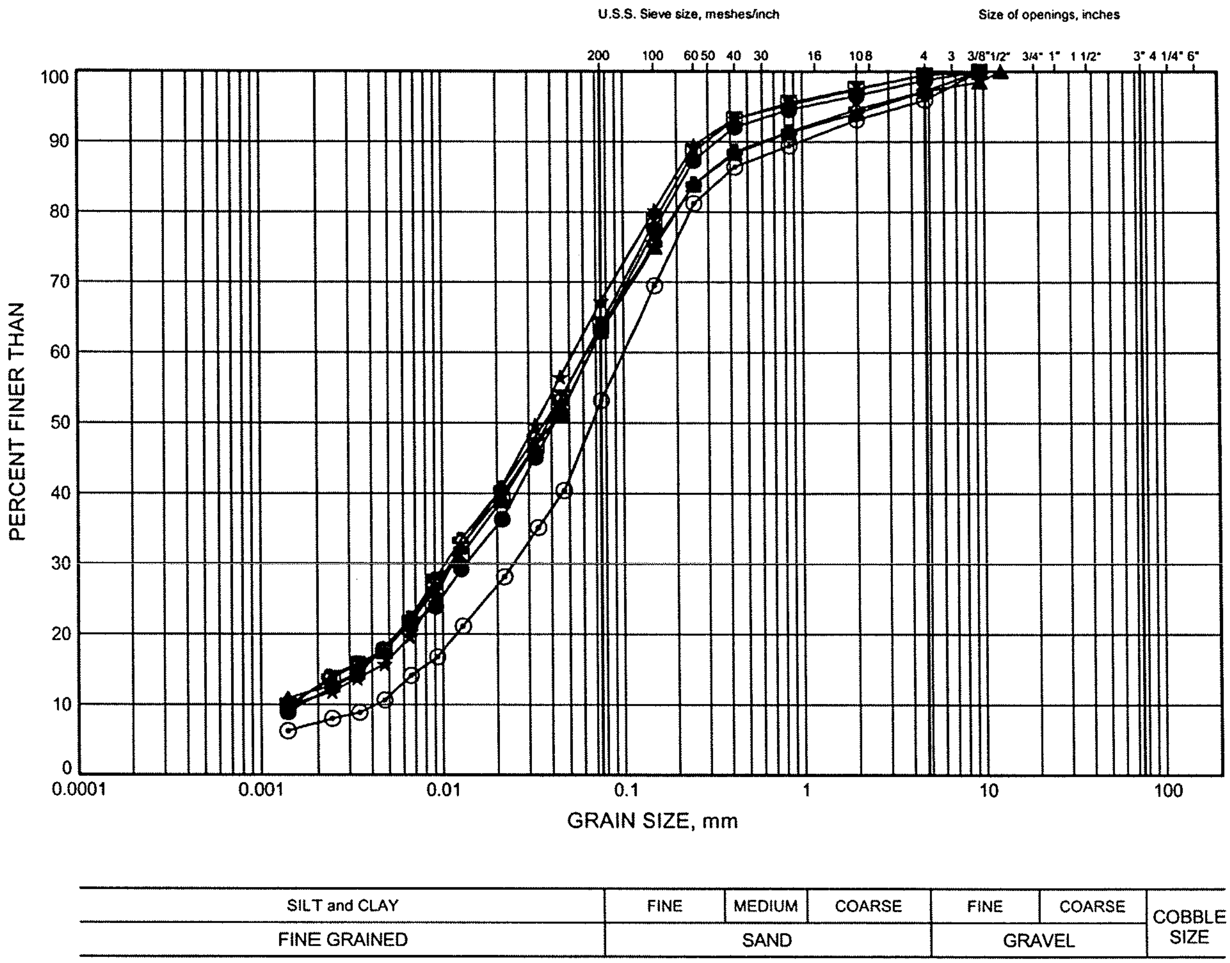
GRAIN SIZE DISTRIBUTION - THURBER 0596.GPJ 1/22/09

Hwy 404 Extension

GRAIN SIZE DISTRIBUTION

FIGURE B2

Sand and Silt (TILL)



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	08-48	1.83	263.80
⊠	08-49	1.07	265.02
▲	08-51	2.59	261.05
★	08-51	6.17	257.47
⊙	08-51	9.18	254.46
✿	08-52	1.07	261.31

W.P.# 19-1605-96

Prepared By AN

Checked By RPR

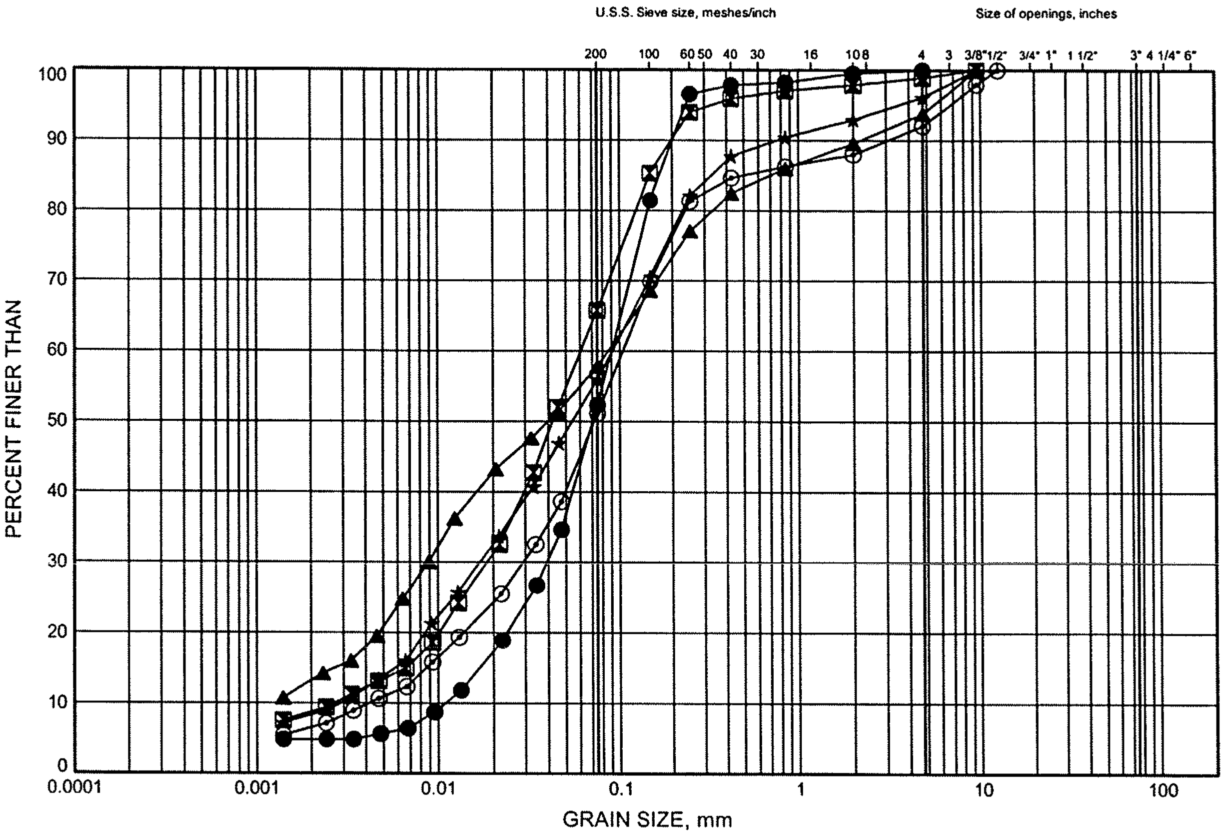
THURBER

Hwy 404 Extension

GRAIN SIZE DISTRIBUTION

FIGURE B3

Sand and Silt (TILL)



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	08-52	2.59	259.79
⊠	08-52	9.35	253.03
▲	08-53	3.47	258.67
★	08-53	9.26	252.88
⊙	08-54	7.83	253.17

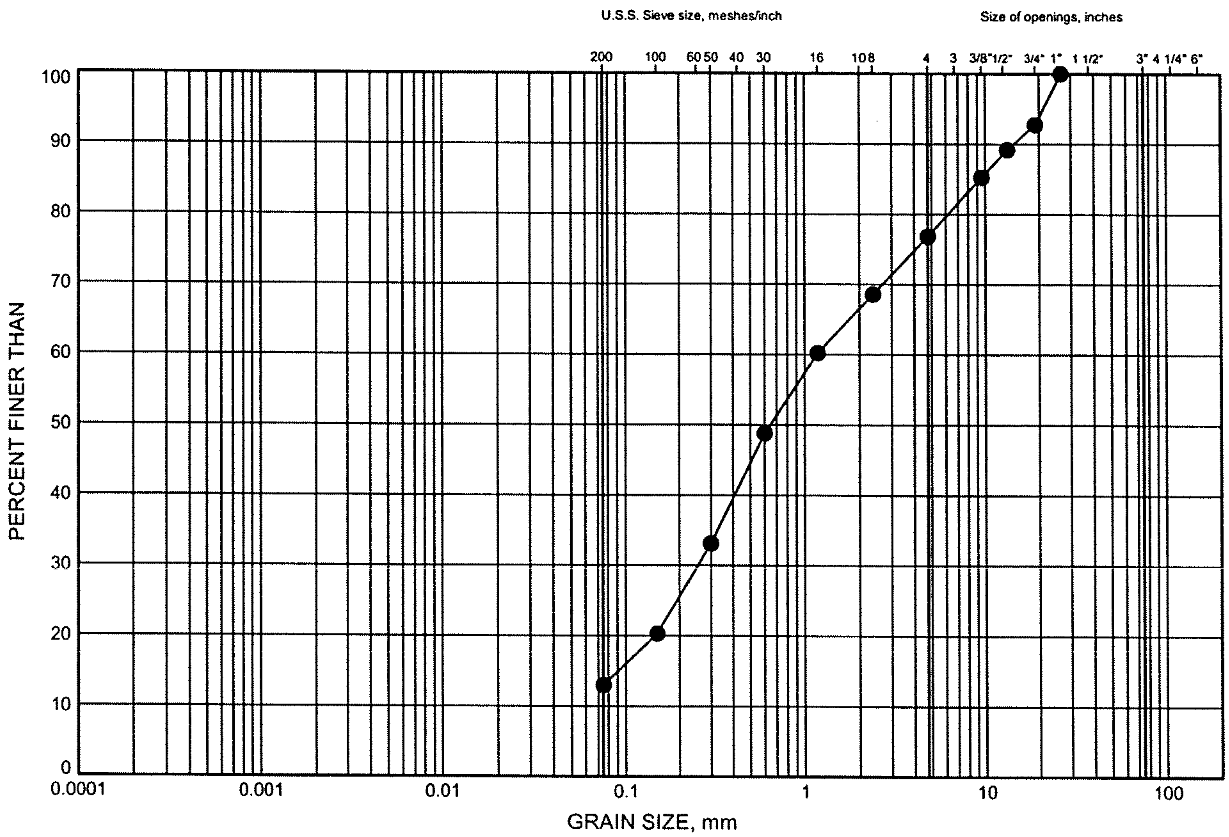
W.P.# 19-1605-96
Prepared By AN
Checked By RPR

THURBER

GRAIN SIZE DISTRIBUTION - THURBER 0596.GPJ 12/12/08

Hwy 404 Extension GRAIN SIZE DISTRIBUTION	FIGURE B4

Gravelly Sand



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	08-48	3.35	262.28

W.P.# 19-1605-96

Prepared By AN

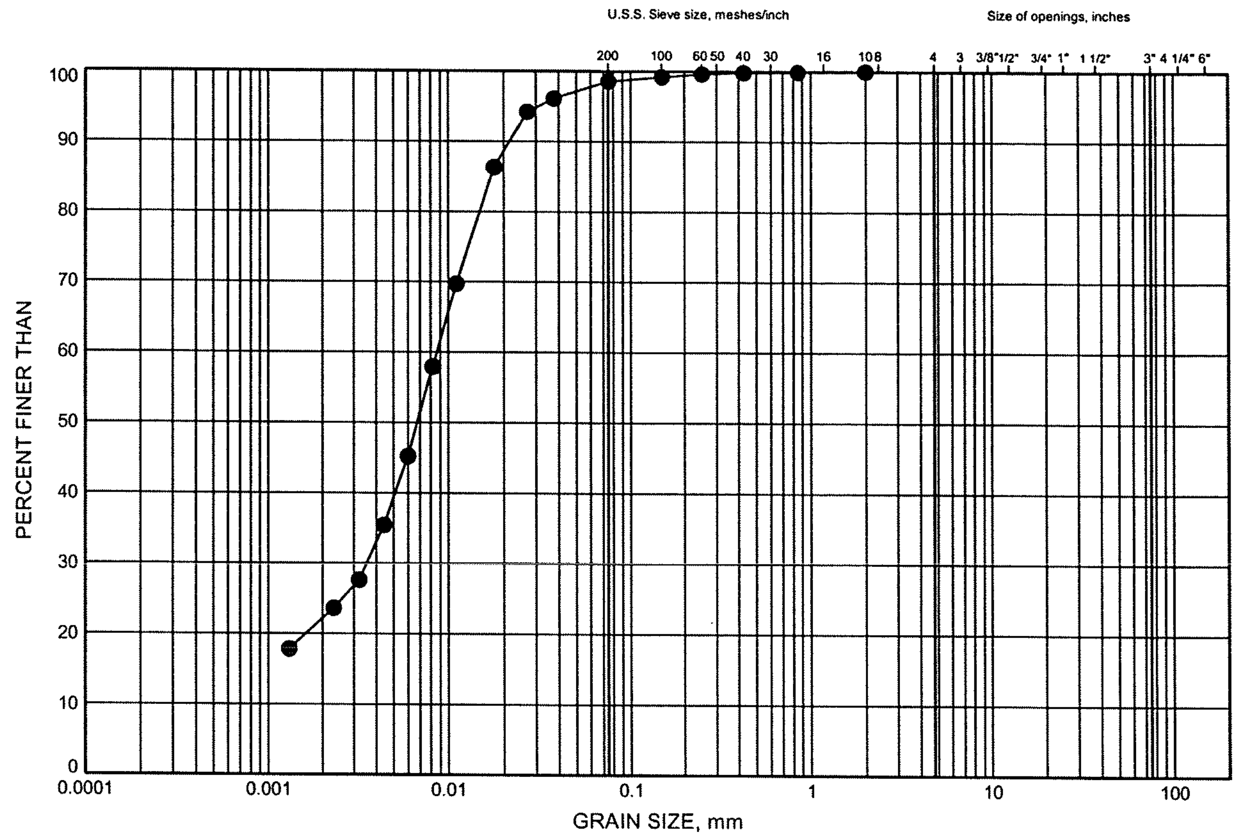
Checked By RPR

THURBER

Hwy 404 Extension
GRAIN SIZE DISTRIBUTION

FIGURE B5

Clayey Silt (TILL)



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	08-54	2.59	258.41

W.P.# 19-1605-96
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Checked By RPR

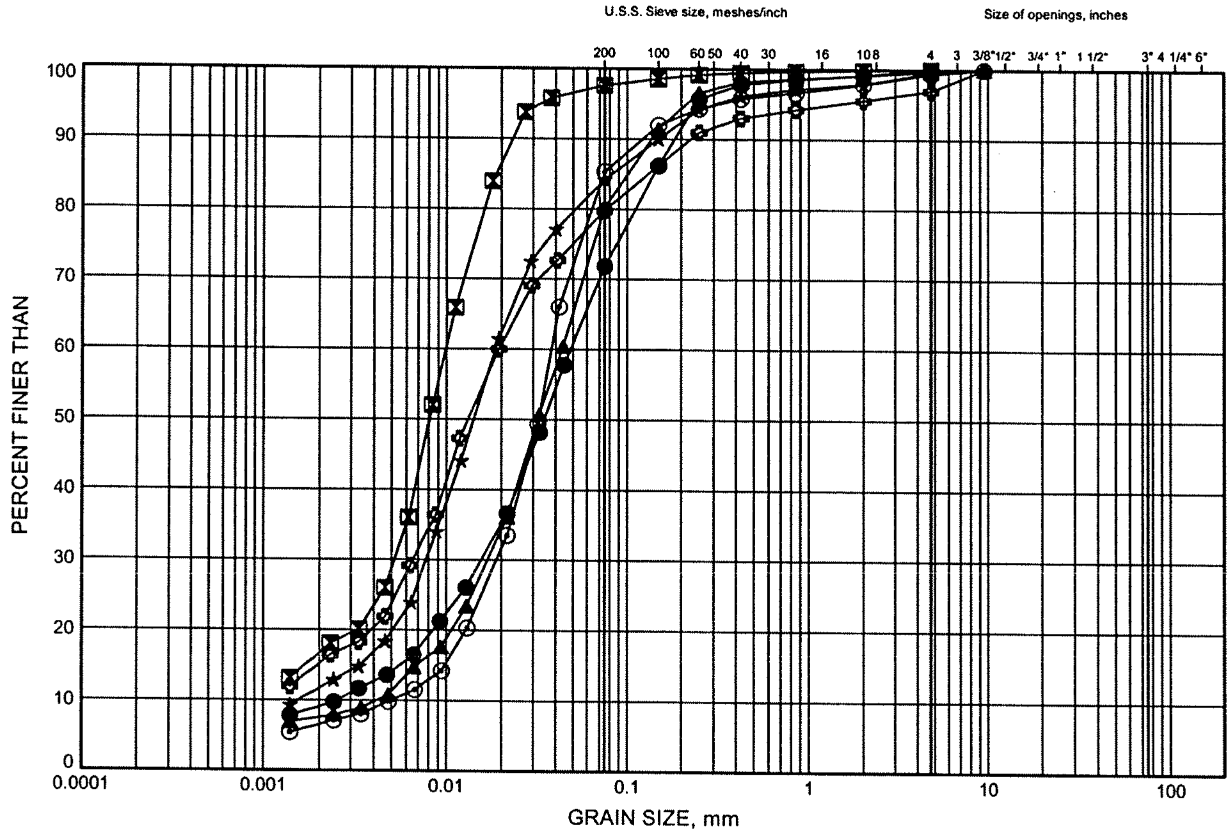
THURBER

Hwy 404 Extension

GRAIN SIZE DISTRIBUTION

FIGURE B6

Silt (TILL)



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	08-48	6.23	259.40
⊠	08-49	3.35	262.74
▲	08-49	6.40	259.69
★	08-50	2.59	261.79
⊙	08-50	6.29	258.09
✪	08-54	10.72	250.28

W.P.# 19-1605-96

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Checked By RPR

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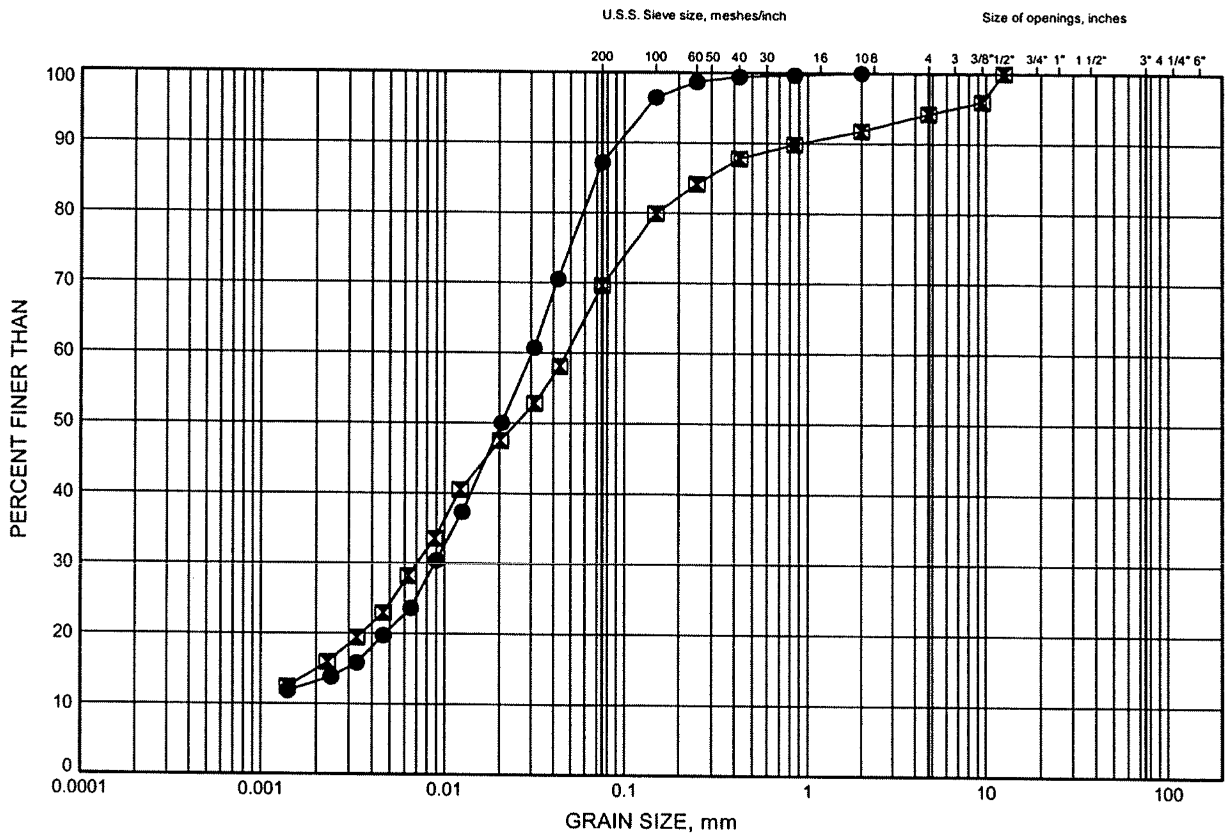
GRAIN SIZE DISTRIBUTION - THURBER 0596.GPJ 12/15/08

Hwy 404 Extension

GRAIN SIZE DISTRIBUTION

FIGURE B7

Silt (TILL)



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	08-55	1.83	259.84
⊠	08-55	6.40	255.27

W.P.# 19-1605-96

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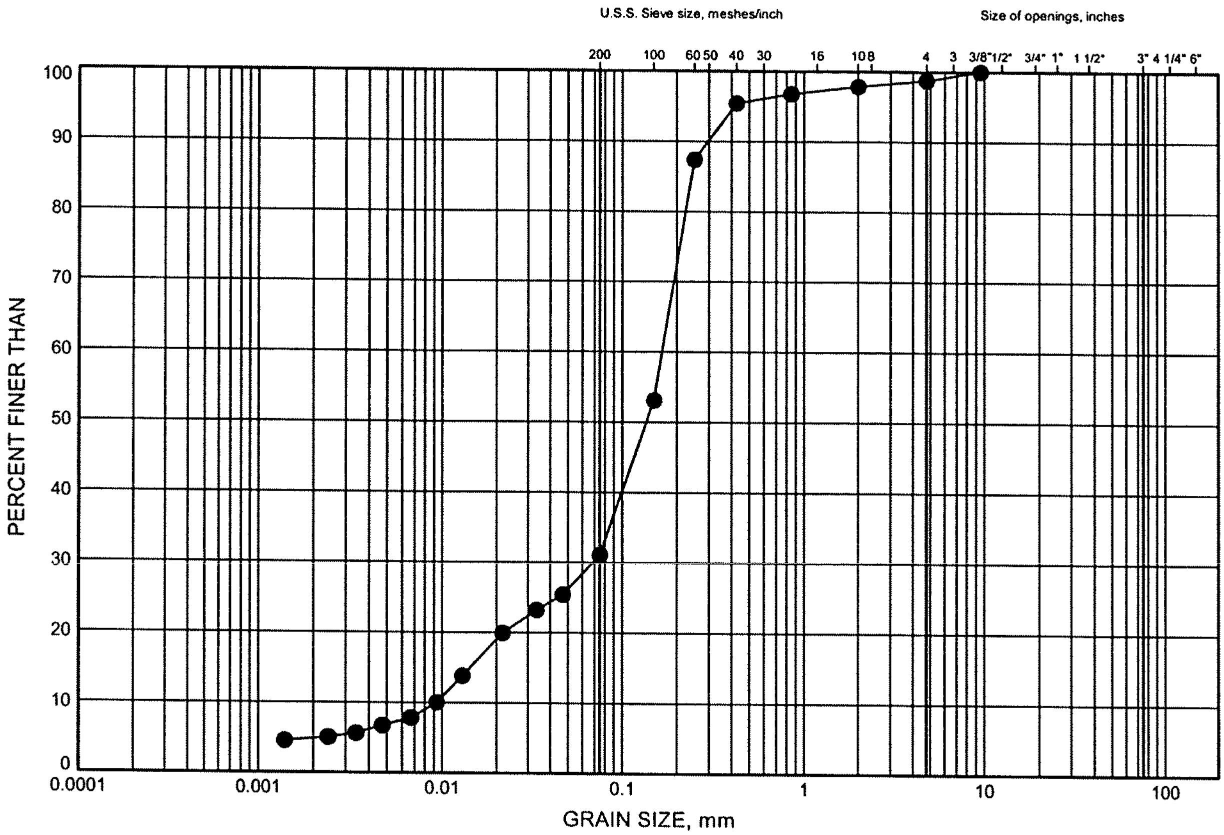
GRAIN SIZE DISTRIBUTION - THURBER 0596.GPJ 12/15/08

Hwy 404 Extension

GRAIN SIZE DISTRIBUTION

FIGURE B8

Silty Sand



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	08-50	4.62	259.76

GRAIN SIZE DISTRIBUTION - THURBER 0596.GPJ 12/15/08

W.P.# 19-1605-96

Prepared By AN

Checked By RPR

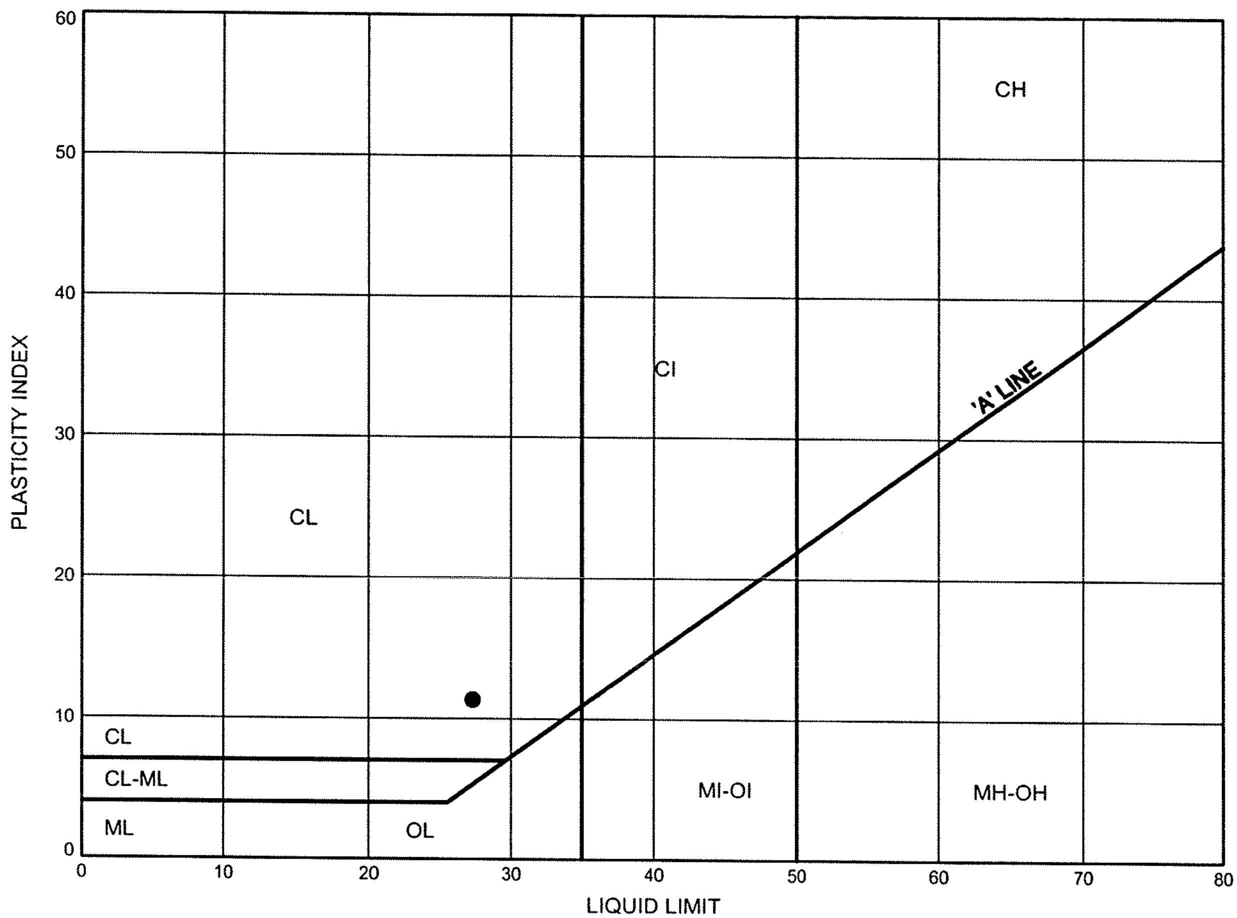
THURBER

Hwy 404 Extension

ATTERBERG LIMITS TEST RESULTS

FIGURE B9

Silty Clay (FILL)



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	08-53	1.83	260.31

THURBALT 0596.GPJ 1/22/09

Date January 2009

Project 19-1605-96

THURBER

Prep'd AN

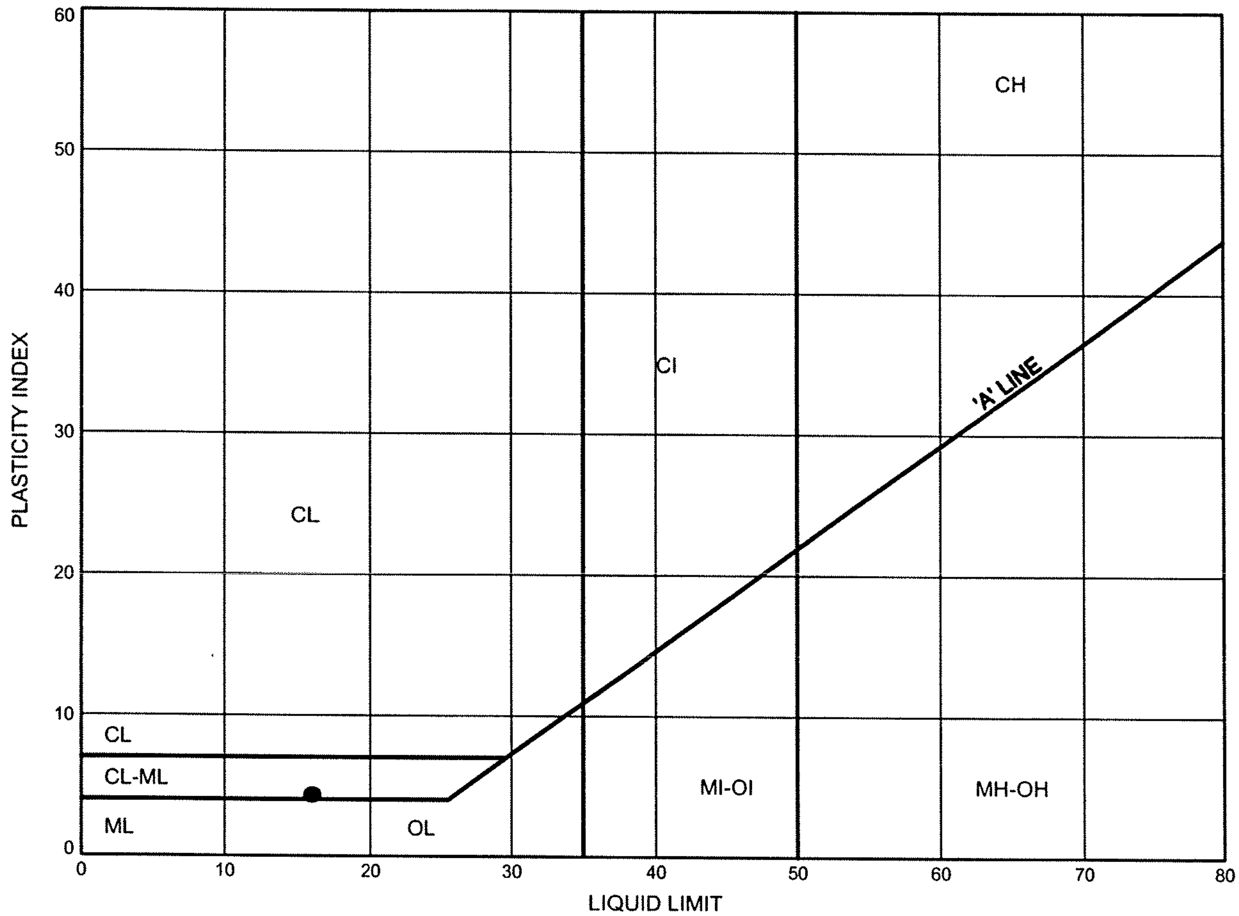
Chkd. RPR

Hwy 404 Extension

ATTERBERG LIMITS TEST RESULTS

FIGURE B10

Sand and Silt (TILL)



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	08-53	3.54	258.60

THURBALT 0596.GPJ 12/15/08

Date December 2008

Project 19-1605-96

THURBER

Prep'd AN

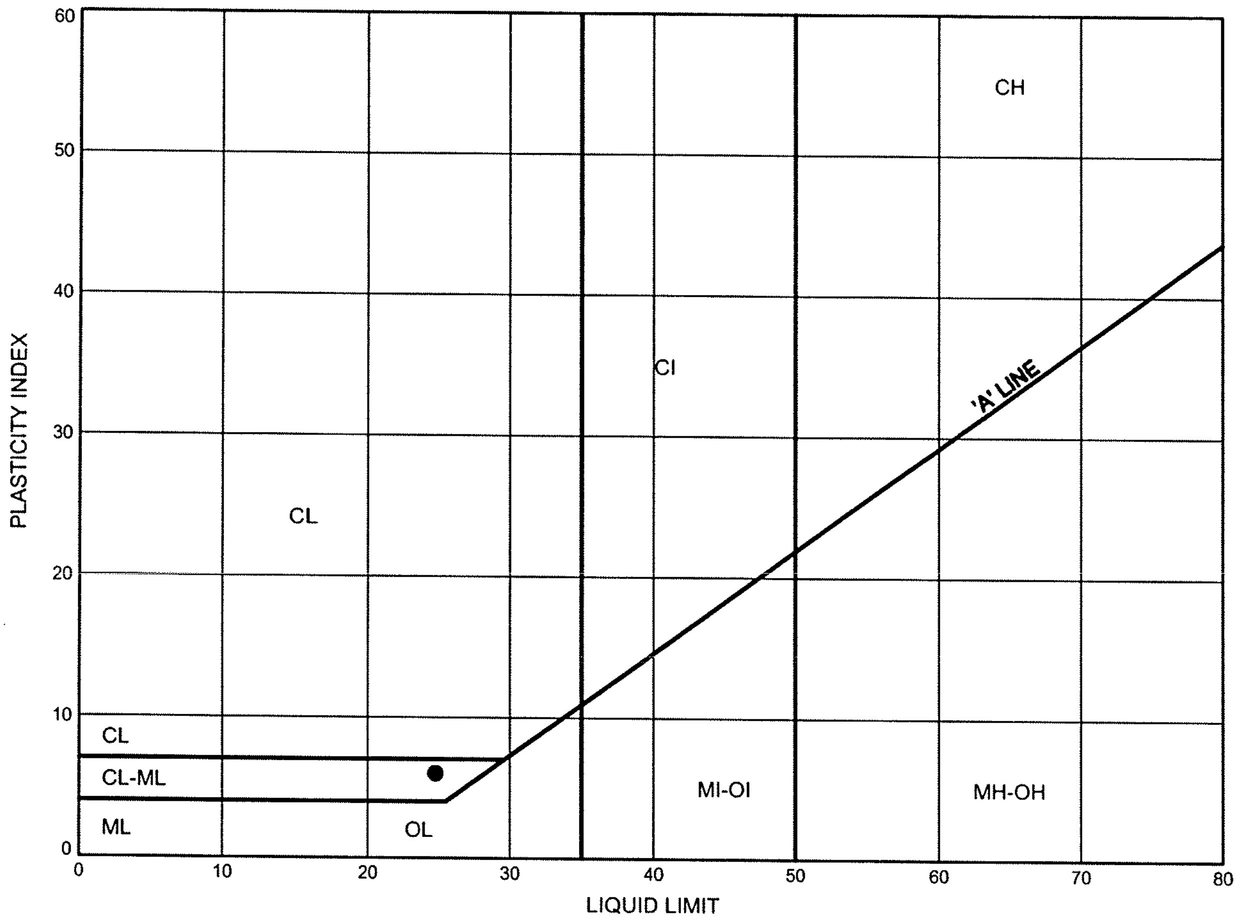
Chkd. RPR

Hwy 404 Extension

ATTERBERG LIMITS TEST RESULTS

FIGURE B11

Clayey Silt (TILL)



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	08-54	2.59	258.41

Date December 2008

Project 19-1605-96

THURBER

Prep'd AN

Chkd. RPR

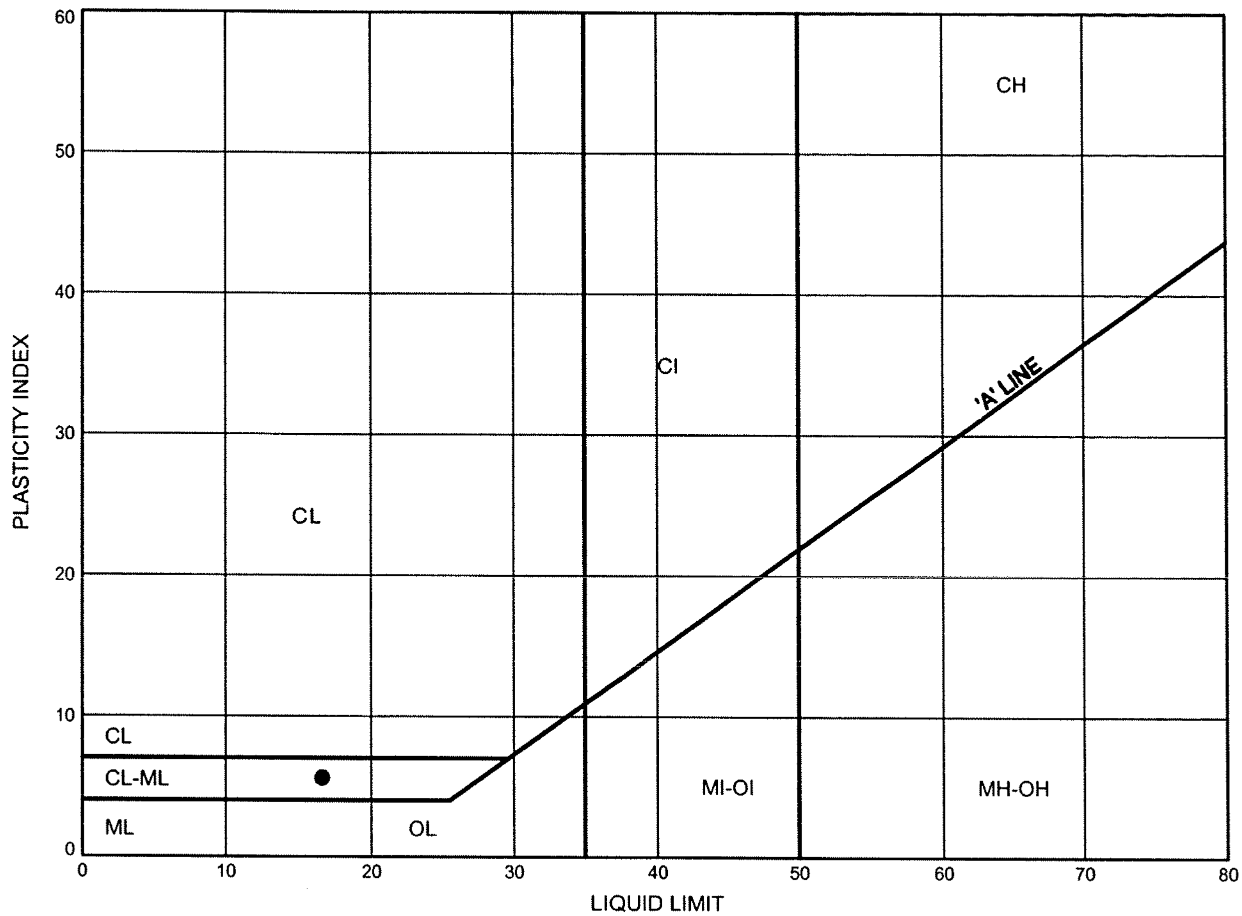
THURBALT 0596.GPJ 12/15/08

Hwy 404 Extension

ATTERBERG LIMITS TEST RESULTS

FIGURE B12

Silt (TILL)



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	08-55	6.40	255.27

Date December 2008

Project 19-1605-96

THURBER

Prep'd AN

Chkd. RPR

THURBALT 0596.GPJ 12/15/08

Appendix C

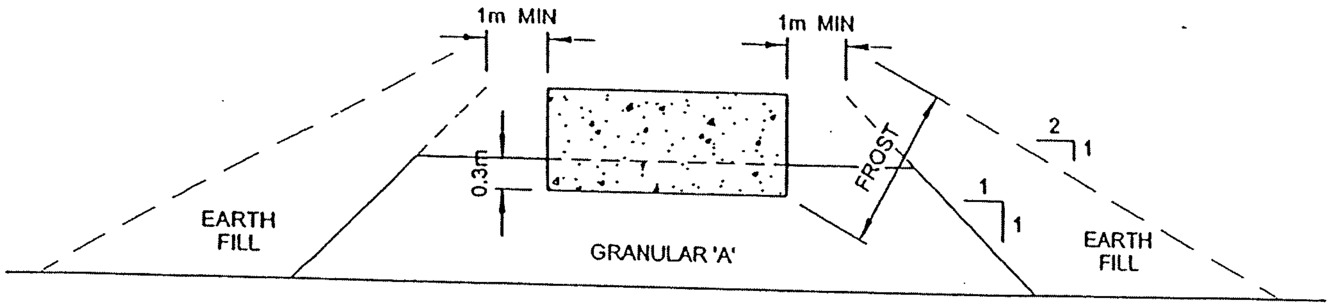
Foundation Comparison

COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

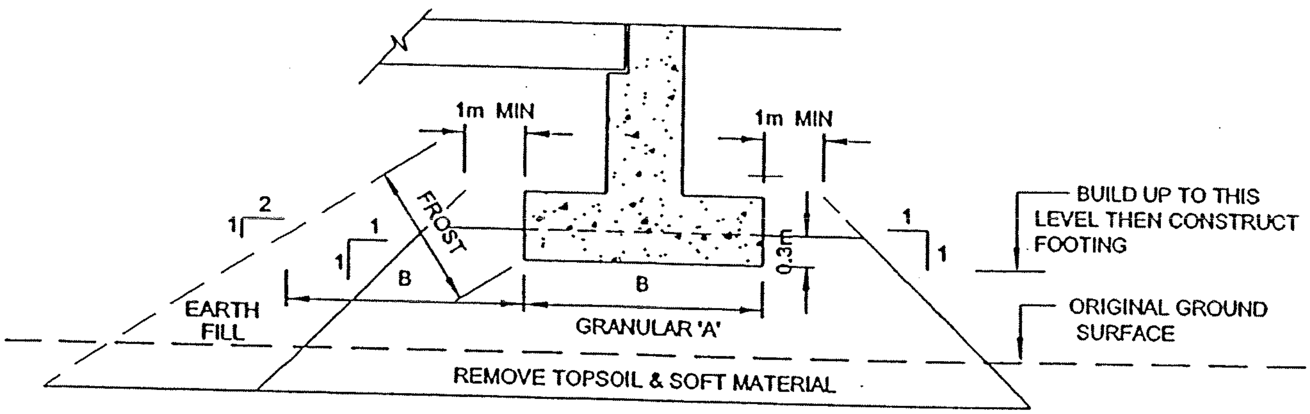
Footing on Native Soil	Footing on Engineered Fill	Driven Piles
Advantages: i. Economical to install.	***Advantages:*** i. Possibility of shortening the abutment height. ii. Higher geotechnical resistance than is available on native soil. iii. Lower cost compared to deep foundations. iv. Allows use of perched abutments. v. Allows choice of semi-integral abutment.	***Advantages:*** i. High geotechnical resistance available by driving piles to achieve resistance in the very dense soil. ii. Allows choice of conventional, integral or semi-integral abutment design. iii. Readily installed.
Disadvantages: i. Comparatively longer abutment stem. ii. Deep excavation and dewatering required. iii. Potential for settlements.	***Disadvantages:*** i. Lower geotechnical resistance than piles. ii. High cost of constructing engineered fill. iii. Potential settlements.	***Disadvantages:*** i. Higher unit cost compared to footings. ii. Construction concerns related to the possibility of piles being obstructed by a boulder during driving.
NOT RECOMMENDED	**FEASIBLE, BUT NOT RECOMMENDED**	**RECOMMENDED AT THE ABUTMENTS AND PIERS**

Appendix D

Figure



CROSS-SECTION



LONGITUDINAL SECTION

NOT TO SCALE

NOTES:

1. REMOVE TOPSOIL AND OR SOFT SUBSOIL UNDER AREA OF COMPACTED GRANULAR 'A' AND EARTH FILL.
2. PLACE GRANULAR 'A' AND EARTH FILL TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO 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	ABUTMENT ON COMPACTED FILL SHOWING GRANULAR A CORE	THURBER
DRAWN SS		
DATE April , 2004		
APPROVED PKC		DWG. NO. FIGURE 1
SCALE NTS		

TED35146.DWG

Appendix E

List of SPs and OPSS, and Suggested Text for Selected NSSP

1. List of Special Provisions and OPSS Documents Referenced in this Report

- OPSS 572
- OPSS 902 as amended by Special Provision 902S01.
- SP 110F13 Amendment to OPSS 1010, March 1993
- SSP 105S10
- OPSD 3501.000
- OPSD 3505.000
- OPSD 3501.000
- OPSD 3505.000

OPSS 206, as amended by Special Provision "Amendment to OPSS 206, December 1993", dated November 2002.

2. Suggested text for a NSSP on Pile Installation

The till may contain cobbles and boulders, particularly below Elevation 259.0 m at the west abutment and Elevations 257.0 and 255.0 at the pier and at the east abutment, respectively. The presence of cobbles and boulders will potentially have an impact on the installation of piles at the site. Some possible impacts that must be taken into consideration include, but are not necessarily limited to:

- The cobbles and boulders may impede the driving of the piles resulting in more arduous driving in the very dense soils.
- Some piles may meet refusal on boulders that are large enough not to be dislodged or broken by the pile driving.
- As a result of the presence of boulders, piles may meet refusal at varying depths.
- Pile driving must be controlled according to the criteria specified for the site.

CSP FOR INTEGRAL ABUTMENT - Item No.

Special Provision

Scope

This specification covers the requirements for the installation of the CSP's, including sand fill and polystyrene sheets, at the integral abutments.

References

This specification refers to the following standards, specifications or publications:

Ontario Provincial Standard Specifications, Construction:

OPSS 906	Structural Steel
OPSS 909	Prestressed Concrete - Precast Members

Ontario Provincial Standard Specifications, General:

OPSS 180	Management and Disposal of Excess Materials

Ontario Provincial Standard Specifications, Material:

OPSS 1605	Expanded Extruded Polystyrene
OPSS 1801	Corrugated Steel Pipe Products

Canadian Standards Association Standards:

CSA G164-M	Galvanizing of Irregularly-Shaped Articles

Ministry of Transportation Publications

MTO Manual of Designated Sources of Materials

Definitions

For the purposes of this specification, the following definitions apply:

Abutment Stem: means the cast-in-place concrete component of the abutment placed over the top of the piles and forming the bearing seat for the girders.

CSP: means helical corrugated steel pipe.

Design Engineer: means the Engineer who produces the design and/or working drawings, and who has a minimum of five (5) years in the design and/or construction of bridges.

Submission and Design Requirements

Submissions

All submissions shall bear the seal and signature of the Design Engineer.

At least two weeks prior to commencement of installation of the abutment, the Contractor shall submit to the Contract Administrator, for information purposes only, three (3) sets of the working drawings.

The Contractor shall have a copy of the submitted working drawings on site at all times.

Working Drawing Requirements

Working drawings shall include at least the following:

1. Layout and Elevations of the CSP's;
2. Source of the sand fill, and description of placing method and equipment;
3. Location and details of all temporary bracing, including permanent and temporary spacers, for the piles, CSP's and abutment stems;
4. Detailed construction sequence for the work, including installation and removal of the temporary bracing.

Design Requirements

The Contractor shall be responsible for the complete detailed design of the construction sequence for the work, including the installation and removal of all temporary bracing. The general sequence of construction shall be as shown on the Contract drawings.

The Contractor shall be responsible for the complete detailed design of all temporary bracing, including temporary and permanent spacers, required to maintain the piles, CSP's, abutment stems and girders in their specified positions through all stages of construction until concrete in deck has reached a compressive strength of 25 MPa. All temporary bracing, except spacers identified as permanent on the Contract drawings, shall be removed.

Temporary bracing for prestressed, precast girders shall meet the requirements of OPSS 909. Temporary bracing for structural steel girders shall meet the requirements of OPSS 906.

Material

Corrugated Steel Pipe

CSP shall be in accordance with OPSS 1801, and shall be from a supplier listed under DSM # 4.60.80. The CSP shall be of the diameter and wall thickness specified on the Contract drawings, and shall be galvanized in accordance with CSA G164-M.

Permanent Spacers and Associated Hardware

Permanent spacers and associated hardware left in place shall not consist of wood and corrodible material.

Sand Fill

The sand fill for backfilling the inner CSP shall meet the gradation requirements of Table 1 below:

Table 1 - Sand Fill Gradation Requirements

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

Expanded Extruded Polystyrene

Expanded extruded polystyrene shall be in accordance with OPSS 1605, and shall be from a supplier listed under DSM # 3.30.30.

Construction

General

The sequence of construction for installing the concrete pads, CSP's, sand fill and abutment stems, including the installation and removal of the temporary bracing, shall be in accordance with the working drawings.

The Contractor shall not proceed with the abutment backfill above the level of the bottom of the CSP's without written permission from the Contract Administrator.

Corrugated Steel Pipe

CSP's shall be supplied in the lengths and with the end treatments, either square or skew, as specified on the Contract drawings; field cutting and splicing of CSP's will not be permitted. Cut ends shall be neat and free of burrs. The planes defined by the end treatments of each CSP shall be parallel to each other.

Handling and storage of CSP's shall be in accordance with the manufacturer's recommendations. Damaged CSP's shall be rejected. Localized areas of damaged galvanizing on otherwise acceptable CSP's shall be repaired by two coats of zinc-rich paint.

The Contractor shall set the inner and outer CSP over each pile in the abutment into the concrete pad, following the batter of the pile, while the concrete in the concrete pad is still plastic. The CSP's shall extend at least 150 mm into the concrete pad.

The Contractor shall ensure the full perimeter of the tops of all CSP's at each abutment are at the Elevation shown on the working drawings.

After the CSP's have been set, the Contractor shall take all measures necessary to prevent the ingress of water, backfill and debris into the CSP's.

Sand Fill

The sand fill shall be placed dry of optimum and free-flowing, completely filling the volume between the inner CSP and pile. No additional compaction effort other than the action of placing the sand fill itself shall be applied to the sand fill.

The placing of the sand fill shall be carried out in a manner such as to not damage and displace the CSP's.

After the sand fill has been placed to the top of each inner CSP, the Contractor shall take all measures necessary to prevent the ingress of water and other liquids into the sand fill until after the concrete in the abutment stem has been placed and cured.

Expanded Extruded Polystyrene

The expanded extruded polystyrene sheets shall completely cover the area under the abutment stem as shown on the Contract drawings. The sheets shall be placed in one piece for the width of the abutment stem, with butt joints perpendicular to the centre-line of abutment bearings. The minimum length of sheet shall be 500 mm.

Joints between sheets within 500 mm of a pile centre-line will not be permitted. At each pile location, a minimum 1000 mm long sheet shall be centred on the pile and a 500 mm diameter hole neatly cut in the sheet so as to fit over the pile in one piece, fully spanning the annular space between the double CSP's.

The Contractor shall adjust the backfill to ensure full and uniform contact of the sheets with the backfill and the full perimeter of the tops of the CSP's. The vertical step at joints between sheets shall not exceed 5 mm.

The Contractor shall protect the sheets from damage during installation of the reinforcing for the abutment stem, and shall secure the sheets from "floating" during placing of the concrete in the abutment stem. Only hardware approved by the Owner shall be used to secure the sheets. All hardware used to secure the sheets shall be installed so as not to project above the top surface of the sheets into the abutment stem.

Temporary Bracing

Temporary bracing shall be installed and removed in accordance with the working drawings.

The temporary bracing shall not distort, nor pierce the walls of, the CSP's. Welding to the CSP's will not be permitted.

Concrete anchors shall be removed and the holes filled with non-shrink grout.

Tolerances

The CSP's at each pile shall be constructed to the following tolerances:

Criteria	Tolerance
Maximum deviation of inner and outer CSP from pile centroid.	± 25 mm
Maximum deviation from specified spacing between inner and outer CSP's.	± 25 mm
Maximum deviation of any point on the top perimeter of the CSP's from the specified Elevation.	± 10 mm

Quality Assurance

Prior to placing the CSP's, the Contractor shall establish reference points at each abutment and determine the location of the centroid of each pile in the abutment with respect to these reference points. The Contractor shall maintain the reference points until written permission to proceed with the backfill above the level of the bottom of the CSP's has been given by the Contract Administrator.

Measurement for Payment

There will be no measurement for this item.

Basis of Payment

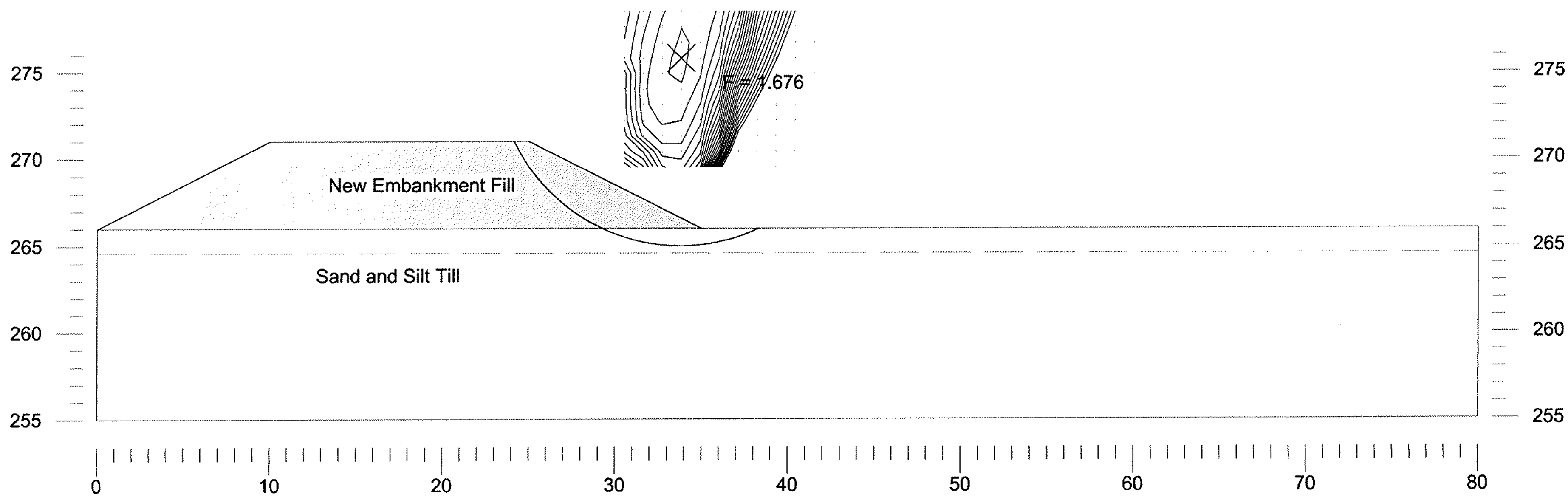
Payment at the contract price for the above items shall be full compensation for all labour, equipment and material required to do the work.

Appendix F

Slope Stability Output

	Gamma kN/m3	C kPa	Phi deg	Piezo Surf.
Earth Fill	21	0	30	1
Sand/Silt Till	20	0	31	1

Thurber Engineering Ltd. - Toronto
19-1605-96
Highway 404 Extension
September 8, 2009
West Abutment
5.0 m high



9/9/2009 12:57:07 PM H:\19\1605\96HWY4~1\REPORT~1\DOANER~1\WA-1A.GSL Thurber Engineering Ltd. - Toronto F = 1.676

FIGURE 1

	Gamma kN/m3	C kPa	Phi deg	Piezo Surf.
Earth Fill	21	0	30	1
Sand/Silt Till	20	0	31	1

Seismic coefficient = 0.08

Thurber Engineering Ltd. - Toronto
19-1605-96
Highway 404 Extension
September 8, 2009
West Abutment
5.0 m high - Seismic

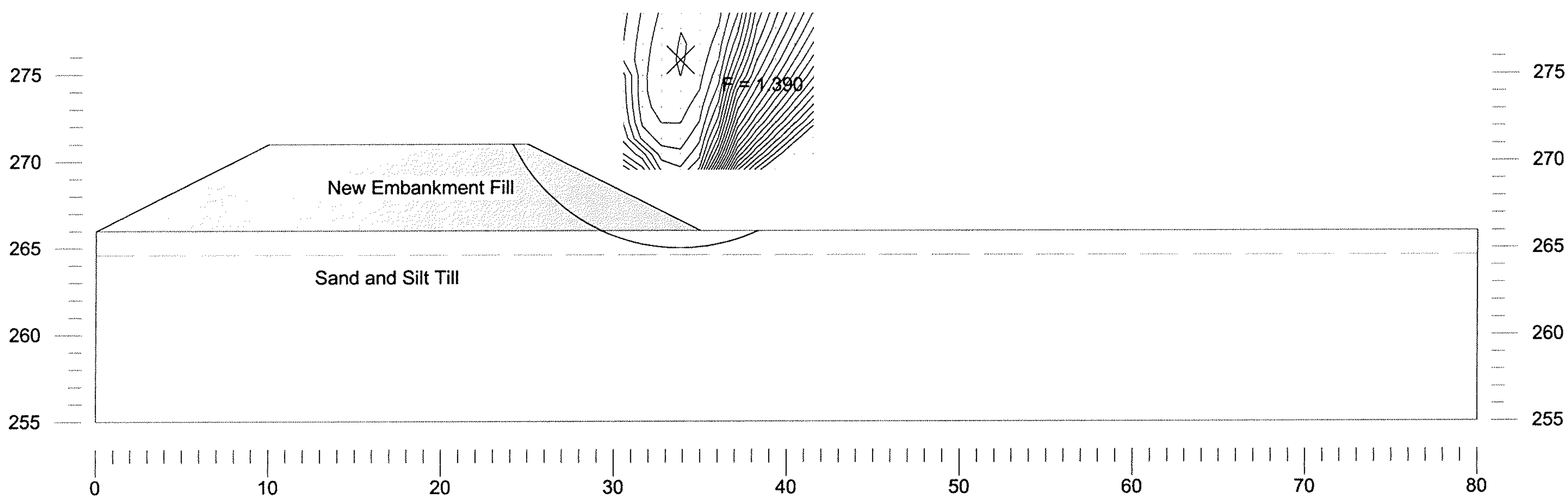


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FIGURE 2

	Gamma kN/m3	C kPa	Phi deg	Piezo Surf.
Earth Fill	21	0	30	1
Sand/Silt Till	20	0	30	1

Thurber Engineering Ltd. - Toronto
19-1605-96
Highway 404 Extension
September 8, 2009
East Abutment
10.0 m high

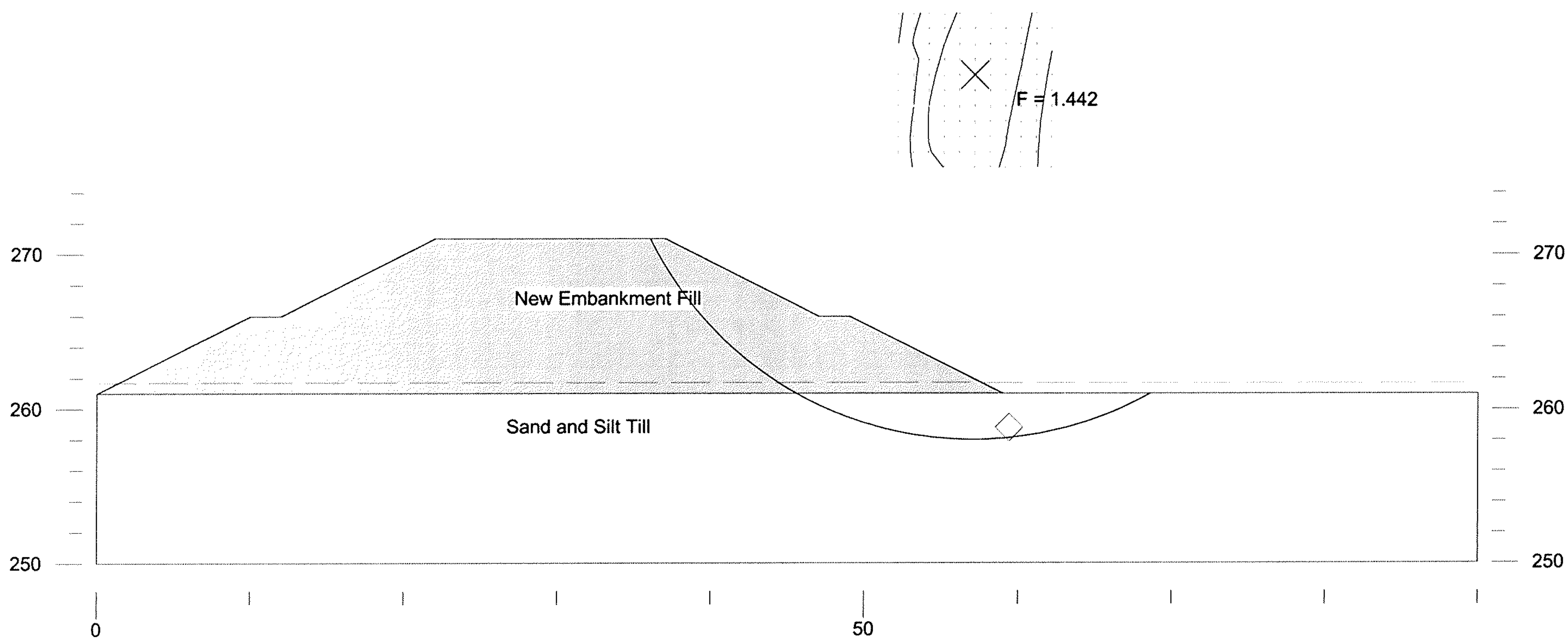

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FIGURE 3

	Gamma kN/m3	C kPa	Phi deg	Piezo Surf.
Earth Fill	21	0	30	1
Sand/Silt Till	20	0	30	1

Seismic coefficient = 0.08

Thurber Engineering Ltd. - Toronto
19-1605-96
Highway 404 Extension
September 8, 2009
East Abutment
10.0 m high

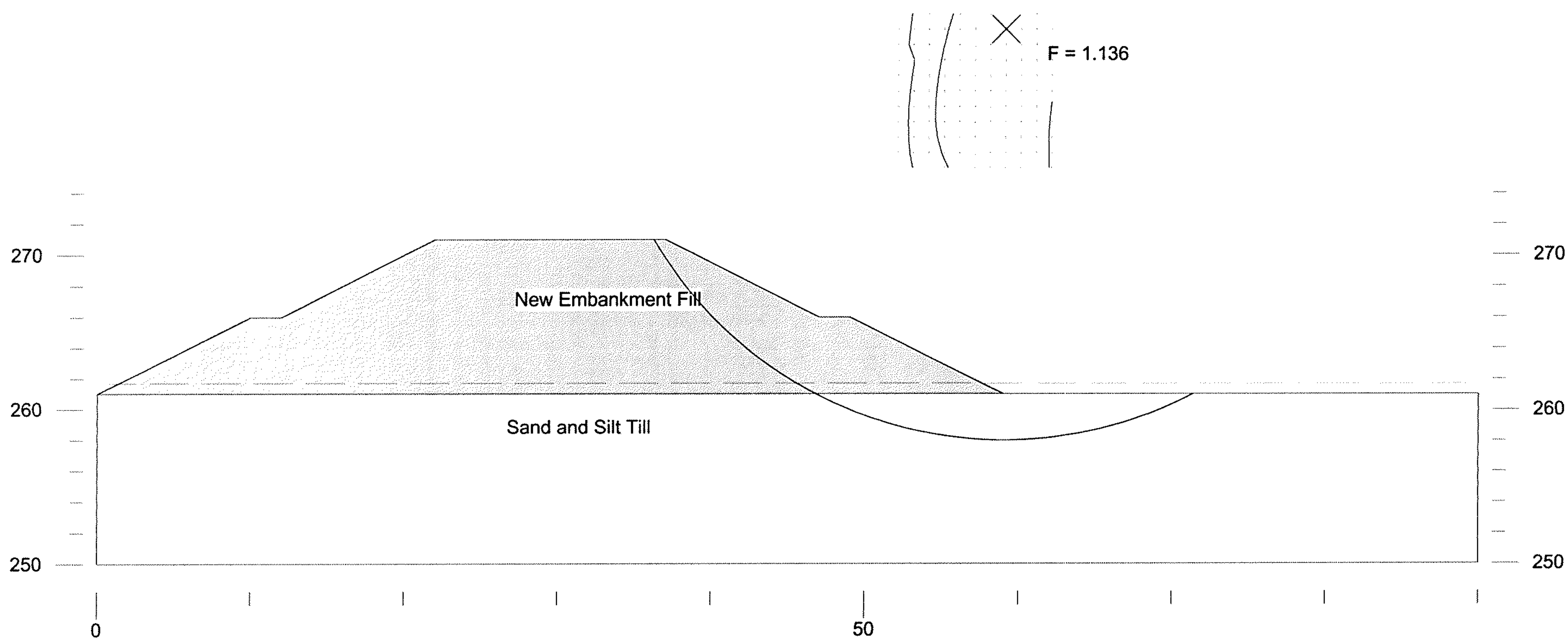

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FIGURE 4

Appendix G

Site Photographs

Photograph 1 – View of the site looking at Boreholes 08-50 and 08-52 drilled on the south side of Doane Road

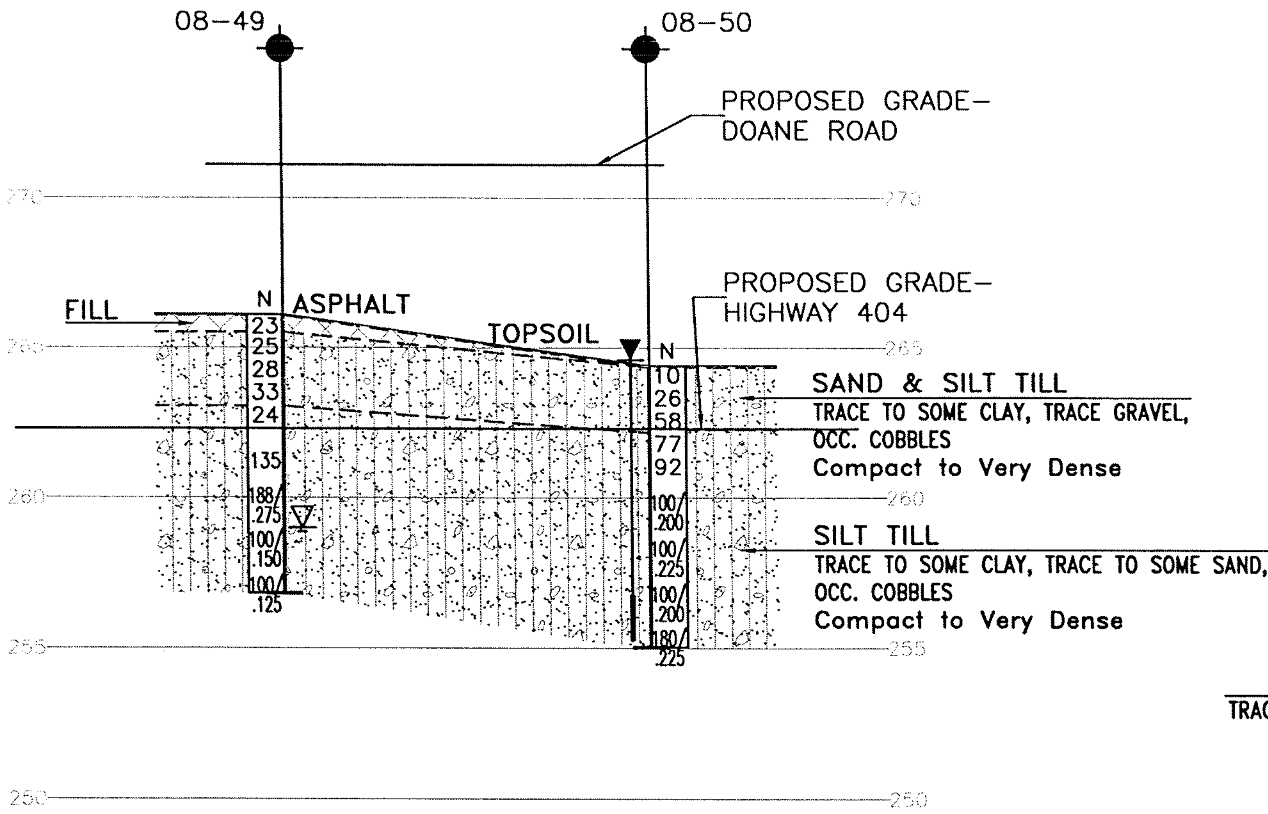
Photograph 2 – View of the site looking at Borehole 08-53, WBL of Doane Road

Photograph 3 – View of the site looking at Borehole 08-54 drilled on the south side of Doane Road

Photograph 4 – View of the site looking at Borehole 08-55 drilled on Doane Road

Appendix H

Drawing
Borehole Locations and Soil Strata



SECTION A–A

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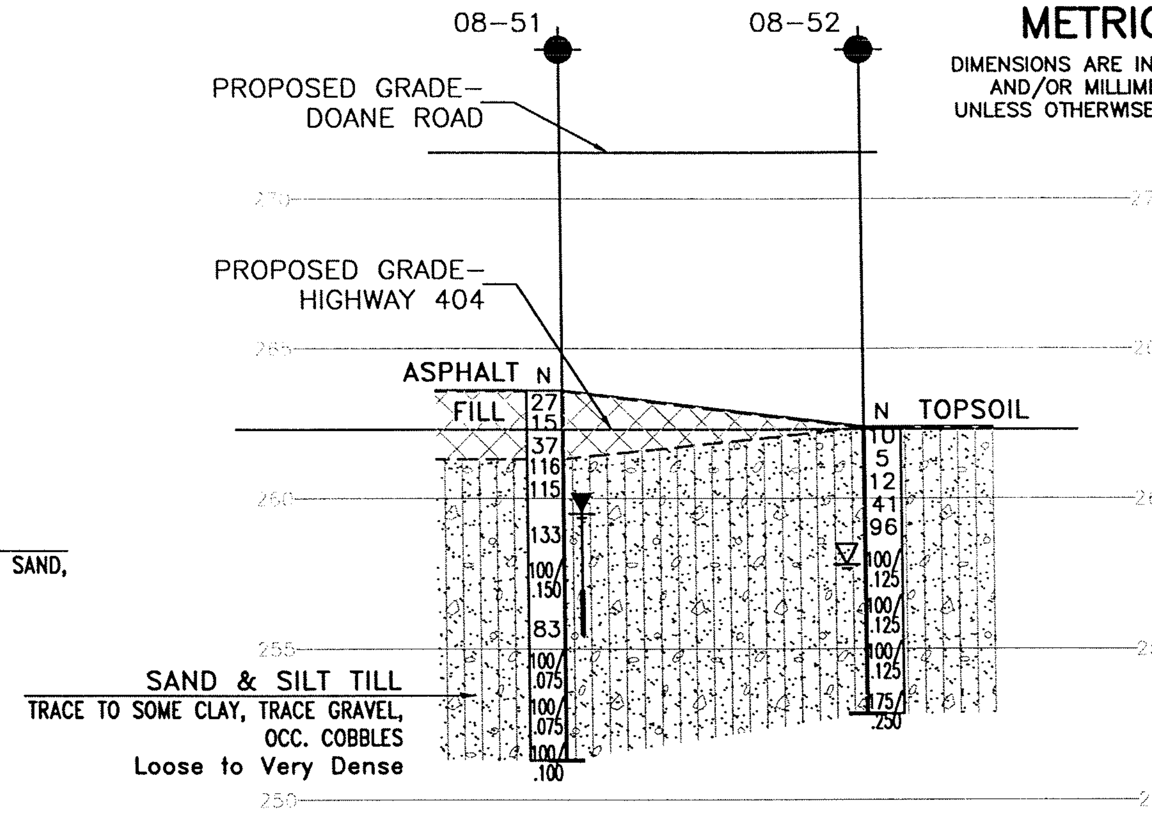
V 1:250



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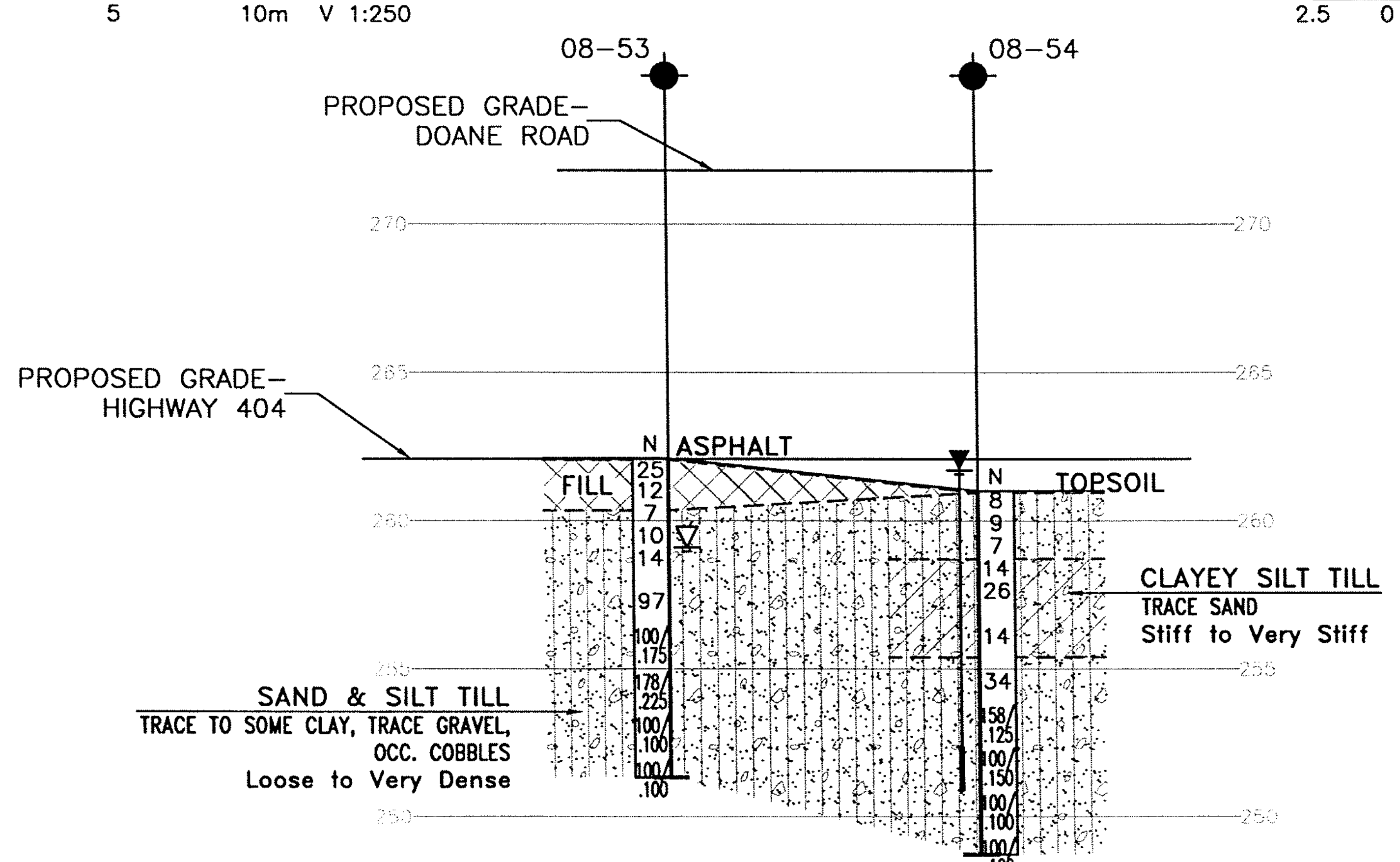

SECTION C–C

H 1:500
V 1:250

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

CONT No
GWP No 2109–05–00

HIGHWAY 404 EXTENSION
BRIDGE FOUNDATIONS
DOANE ROAD UNDERPASS
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

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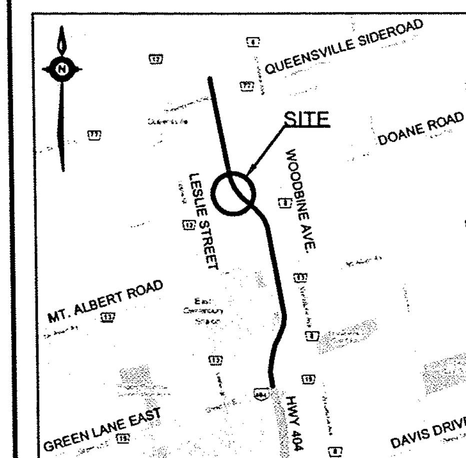

KEYPLAN

LEGEND

	Borehole
	Borehole and Cone
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
08–48	265.6	4 887 093.6	310 301.8
08–49	266.1	4 887 108.9	310 311.7
08–50	264.4	4 887 090.4	310 327.1
08–51	263.6	4 887 113.7	310 353.3
08–52	262.4	4 887 100.1	310 368.4
08–53	262.1	4 887 127.4	310 398.8
08–54	261.0	4 887 112.7	310 413.8
08–55	261.7	4 887 131.7	310 424.0

-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.

GEOCRES No. 31D - 483

LICENSED PROFESSIONAL ENGINEER
R. Palomeque Reyna
100083209
Sep/09/09
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
Sep 9/09
PROVINCE OF ONTARIO

REVISIONS	DATE	BY	DESCRIPTION		
DESIGN RPR	CHK PKC	CODE	LOAD	DATE AUG. 2009	
DRAWN MFA	CHK AEG	SITE	STRUCT	DWG	

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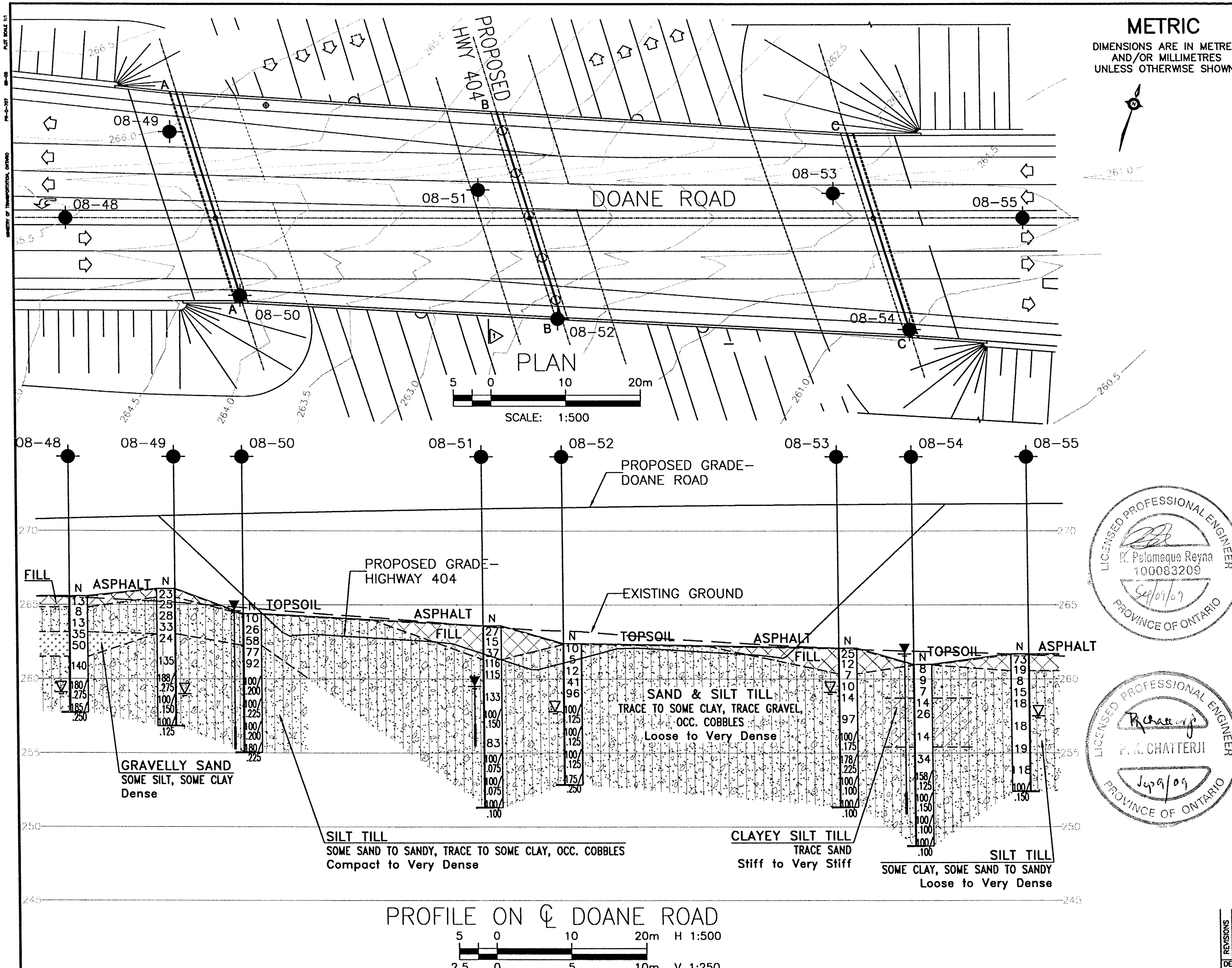

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

CONT No
GWP No 2109-05-00

HIGHWAY 404 EXTENSION
BRIDGE FOUNDATIONS
DOANE ROAD UNDERPASS
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

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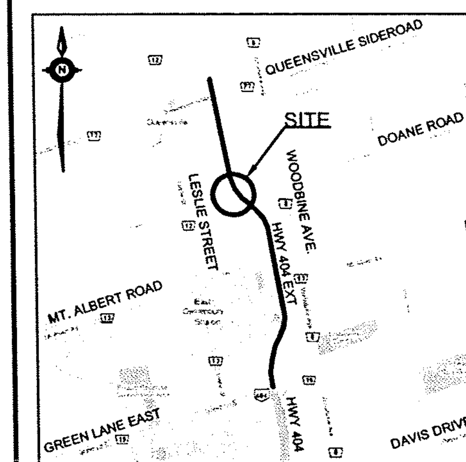

KEYPLAN

LEGEND

	Borehole
	Borehole and Cone
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
08-48	265.6	4 887 093.6	310 301.8
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08-50	264.4	4 887 090.4	310 327.1
08-51	263.6	4 887 113.7	310 353.3
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08-55	261.7	4 887 131.7	310 424.0

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GEOCRES No. 31D - 483

REVISIONS	DATE	BY	DESCRIPTION
DESIGN RPR	CHK PKC	CODE	LOAD	DATE SEP. 2009
DRAWN MFA	CHK AEG	SITE	STRUCT	DWG