FOUNDATION INVESTIGATION AND DESIGN REPORT
GRAND RIVER CROSSING SBL
HIGHWAY 8 WIDENING, KITCHENER
G.W.P. 277-97-00, SITE: 33-137S

Geocres Number: 40P8-143

Report to

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TABLE OF CONTENTS

Part 1: Factual Information

Part 2: Engineering Discussion and Recommendations

Appendices

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted at the site of a proposed structure at Kitchener, Ontario. The proposed five-span structure will carry the southbound lanes (SBL) of the future widened Highway 8 across the Grand River.

The purpose of the investigation was to explore the subsurface conditions at the site and, based on the data obtained, to provide a borehole location plan, records of boreholes, 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 investigation.

Thurber carried out the investigation as a sub-consultant to Morrison Hershfield, under the Ministry of Transportation Ontario (MTO) Agreement Number 3005-E-0035.

2 SITE DESCRIPTION

The site for the proposed new SBL crossing lies across the valley of the Grand River on the south side of the City of Kitchener and immediately west (downstream) of the existing structure carrying Highway 8 across the river. The existing Highway 8 spans the river channel, as well as the flood plain on the south side of the river, on a five-span structure.

At the site location, the river channel is approximately 60 m wide and the existing bridge spans a distance of approximately 190 m. The water level was measured to be approximately 1.5 m to 2.3 m deep at the locations of boreholes drilled in the river during the current investigation. The south shoreline of the river consists of a generally level floodplain with a gentle slope towards the river channel. The floodplain is mainly vegetated with grass, shrubs and some sparse trees. A gravel trail crosses beneath the existing structure between the south abutment and Pier 4 and continues eastwards and westwards from Highway 8, generally following the alignment of the Grand River. The north shoreline of the river consists of an approximately 18 m high cliff with an approximate slope of 2H : 1V. The slope is vegetated with grass and large trees and portions of the toe of the slope and the riverbank are lined with rip-rap boulders. Residential houses overlooking the river are located at the top of the cliff along Hidden Valley Road, which generally follows the river alignment.

Geologically, the site area is located within the physiographic region known as the Waterloo Hills, which is characterized by sandy hills consisting of ridges of sandy till as well as kames and kame moraines, with outwash sands occupying the intervening hollows. The surficial soils of this region overly Silurian bedrock of the Guelph Formation. Locally, the site lies within the Grand River spillway system, which consists of alluvial terraces containing uniform sandy and gravelly materials, although the steep slope of the north bank of the river can be considered part of a kame moraine system composed mainly of till and sand and gravel deposits.

Photographs of the site are included in Appendix G. Both photographs are taken from the flood plain area on the south side of the Grand River. Photograph #1 is looking northward across the Grand River towards the area of the proposed North Abutment and Piers 1 and 2. Part of the existing Hwy 8 bridge can be seen on the right side of the photograph. Photograph #2 is taken from the area of Piers 3 and 4 and is looking southward across the floodplain towards the area of the proposed South Abutment. Part of the existing Hwy 8 bridge can be seen on the left side of the photograph.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project were carried out between the period of July 27 to November 21, 2006. Twelve boreholes numbered 06-2 to 06-13 pertaining to the five-span structure were drilled to depths ranging from 6.3 m to 19.9 m. Boreholes 06-2 and 06-7 to 06-13 were drilled using truck and track-mounted CME 75 drill rigs in the vicinity of the proposed North and South Abutments and Piers 3 and 4. Boreholes 06-3 to 06-6 were drilled through the riverbed using a barge-mounted CME 75 drill rig in the vicinity of the proposed Piers 1 and 2. The boreholes were drilled as close as was accessible to the foundation elements. The approximate locations of the boreholes are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix F.

Thurber located the borehole locations in the field with reference to the existing Grand River overpass structure. The borehole locations (with the exception of Boreholes 06-3 to 06-6, which were drilled in the Grand River) were subsequently surveyed by Callon Dietz Inc., who provided Thurber with the coordinates and geodetic elevations. Thurber obtained utility clearances prior to drilling.

Prior to drilling boreholes 06-3 to 06-6 in the Grand River, the Department of Fisheries and Oceans Canada determined that at the site of the new SBL crossing, the river contains a species of mussel (Wavy Rayed Lampmussel) that is protected by the Species At Risk Act. Therefore, prior to the commencement of the drilling activities in the river, a mussel relocation program was conducted in order to minimize the impact of the drilling activities on the mussel population at the site. Prior to future construction activities, it is possible that the site location may become repopulated with mussels and therefore a new relocation program may need to be conducted, followed by a post-relocation monitoring program.

A combination of hollow-stem auger drilling techniques and casing and washboring methods 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. One borehole at each foundation element was advanced from 2.9 m up to 6.0 m into bedrock by NQ size diamond coring techniques, with the exception of Borehole 06-2 at the north abutment, which was advanced greater than 3.0 m into refusal soil as defined by SPT 'N' values of greater than 100 blows per 0.3 m.

Groundwater conditions in the open boreholes were observed throughout the drilling operations. At each foundation element (with the exception of Piers 1 and 2, which are located within the river) a standpipe piezometer consisting of 19 mm PVC pipe with a slotted screen was installed and enclosed in filter sand to permit longer term groundwater level monitoring. The locations and completion details of the piezometers are shown in Table 3.1. The boreholes in which no piezometers were installed were grouted with bentonite. Grouting was carried out in accordance with the requirements of MOE Reg. 903. The borehole completion details are shown in Table 3.1.

Table 3.1 – Borehole Completion Details

Location	Details	
	Piezometer Tip Depth/ Elevation (m)	Completion Details
06-2 North Abutment	19.9 / 274.9	Piezometer with 1.5 m slotted screen installed with sand filter to 18.0 m, bentonite seal from 18.0 m to 17.4 m, grout from 17.4 m to 0.9 m and bentonite seal from 0.9 m to ground surface.
06-3 Pier #1	None Installed	Grouted with bentonite to riverbed surface.
06-4 Pier #1	None Installed	Grouted with bentonite to riverbed surface.
06-5 Pier #2	None Installed	Grouted with bentonite to riverbed surface.
06-6 Pier #2	None Installed	Grouted with bentonite to riverbed surface.
06-7 Pier #3	12.1 / 271.2	Piezometer with 1.5 m slotted screen installed with sand filter to 10.1 m, bentonite seal from 10.1 m to 9.7 m, grout from 9.7 m to 0.6 m and bentonite seal from 0.6 m to ground surface.
06-8 Pier #3	None Installed	Grouted with bentonite to ground surface.
06-9 Pier #4	None Installed	Grouted with bentonite to ground surface.
06-10 Pier #4	12.1 / 271.7	Piezometer with 1.5 m slotted screen installed with sand filter to 9.8 m, bentonite seal from 9.8 m to 9.1 m, grout from 9.1 m to 0.5 m and bentonite seal from 0.5 m to ground surface.
06-11 South Abutment	14.0 / 270.5	Piezometer with 1.5 m slotted screen installed with sand filter to 12.2 m, bentonite seal from 12.2 m to 11.6 m and grout from 11.6 m to ground surface.

06-12 South Abutment	None Installed	Grouted with bentonite to ground surface.
06-13 South Approach	None Installed	Grouted with bentonite to ground surface.

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 and rock samples for transport to Thurber's laboratory for further examination and testing.

All rock cores were logged, and the Total Core Recovery (TCR), Rock Quality Designation (RQD) and the Fracture Indices (FI) were determined.

4 LABORATORY TESTING

The recovered soil samples were subjected to Visual Identification (VI) and to natural moisture content determination. The results of this testing are shown on the Record of Borehole sheets in Appendix A. Selected samples were also subjected to gradation analysis and 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. The results of point load tests on rock cores retrieved from the boreholes are shown in Table B1 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 and rock stratigraphy are presented in this appendix and on the "Borehole Locations and Soil Strata" and "Soil Strata" drawings in Appendix F. 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. The factual data from the previous investigation is contained in Appendix C.

In general, the site is underlain by 10.5 m to greater than 19.9 m of overburden soils overlying Limestone bedrock. The overburden soils generally consist of topsoil, granular or clay fill, an upper sand and gravel deposit, sandy silt to silt and sand glacial till, and a lower sand and gravel deposit. Occasional zones of sand and silty clay glacial till were also encountered in the investigation.

5.1 Topsoil and Fill

Across the site 0.1 m to 0.2 m of topsoil was encountered that extends to elevations ranging from 294.7 m to 283.3 m. The topsoil thickness may very between the borehole locations and at other areas of the site.

Borehole 06-2 was drilled in the ditch adjacent to the existing Highway 8 SBL. This borehole encountered a layer of silty clay fill with trace sand and gravel underlying the topsoil. The fill layer extends to a depth of 0.8 m or to an elevation of 294.1 m. The fill is considered to have a stiff consistency based on a Standard Penetration Test 'N' value of 8 blows per 0.3 m penetration. The moisture content of a sample of this material was 19%.

Boreholes 06-11 to 06-13 were drilled in the vicinity of the existing Highway 8 SBL embankment and the gravel trail near the existing south abutment. These boreholes encountered granular fill ranging from sand to sand to gravel underlying the topsoil. A thin layer (75 mm) of topsoil was also encountered beneath the granular fill in Boreholes 06-11 and 06-13. The granular fill and buried topsoil extends to a depth of 0.2 m to 0.7 m or to elevations ranging from 284.1 to 283.5 m. Standard penetration tests conducted in this layer gave 'N' values of 9 to 13 blows per 0.3 m penetration. Based on these results the fill is considered to have a loose to compact relative density. The moisture content of samples from this layer ranged from approximately 7% to 14%.

5.2 Sand

Underlying the topsoil, a layer of sand was encountered in the boreholes located in the floodplain on the south side of the river. The sand also contained some silt and trace gravel. The upper part of the sand was also mixed with topsoil. The sand deposit was approximately 1.4 m to 2.1 m thick and was encountered to depths of 1.5 m to 2.2 m or to elevations of 282.1 m to 281.1 m.

Standard penetration tests in this deposit gave 'N' values from 4 to 33 blows per 0.3 m penetration, indicating that the relative density of the material varies from loose to dense.

The moisture content of samples from this material ranged from approximately 13% to 57%, with the higher values being attributed to the presence of topsoil within the sand.

5.3 Upper Sand and Gravel

An upper deposit of sand and gravel ranging from sandy gravel to gravelly sand extends across most of the site except for at the location of the proposed North Abutment (Borehole 06-2). This deposit ranges in thickness from 0.2 m to 3.2 m and extends to depths of 2.3 m to 4.6 m or to elevations of 281.0 m to 279.1 m. The material contains trace silt and occasional to some cobbles and boulders. The presence of some rip-rap boulders was also observed on the riverbed at Boreholes 06-3 and 06-4 near the north shoreline.

Selected samples of this material were subjected to grain size distribution tests and the results are presented in Figures B4 and B5 in Appendix B.

Standard penetration tests in the upper sand and gravel deposit gave 'N' values from 11 to greater than 50 blows per 0.3 m penetration indicating that the relative density of the material varies from compact to very dense.

The moisture content of samples from this deposit ranged from approximately 4% to 21%.

5.4 Sandy Silt to Silt and Sand Till

Underlying the upper sand and gravel layer, a deposit of glacial till consisting of sandy silt ranging to silt and sand extends across the site. The till also contains trace to some clay, trace to some gravel and cobbles and boulders. The total thickness of the deposit ranges from 1.8 m to greater than 19.1 m, although layers of sand with trace to some silt and trace gravel as well as layers of silty clay till were encountered within this deposit. The deposit extends to depths ranging from 4.7 m to greater than 19.9 m or to elevations ranging from 277.8 m to 273.9 m. Glacial tills inherently contain cobbles and boulders.

Selected samples from this deposit were subjected to grain size distribution tests and the results are presented in Figures B2 and B3 in Appendix B.

SPT 'N' values ranged from 8 to greater than 50 blows per 0.3 m penetration, although were generally between 33 and greater than 50 blows per 0.3 m penetration, indicating that the material has a dense to very dense relative density. Some of the SPT 'N' values may represent tests conducted on cobbles and boulders.

The moisture content of samples from this deposit ranged from approximately 6% to 15%.

5.5 Silty Clay Till

Occasional zones of silty clay glacial till were encountered across the site. The glacial till contains varying amounts of sand, ranging from trace sand to sandy, as well as trace gravel. These zones range in thickness from 0.8 m to 7.6 m and were encountered extending to depths of 8.4 m to 15.2 m or to elevations of 279.6 m to 272.6 m.

Selected samples from this material were subjected to grain size distribution tests and the results are illustrated in Figure B1 in Appendix B. The results of Atterberg Limit tests conducted on selected samples from this material are shown in Figure B6 in Appendix B. All three samples tested plot as "CL".

SPT 'N' values in this material ranged from 24 to more than 50 blows for 0.3 m penetration, indicating a very stiff to hard relative density. Glacial tills inherently contain cobbles and boulders and some of the high SPT 'N' values may represent tests conducted on cobbles and boulders.

The moisture content of samples from this material ranged from approximately 7% to 18%.

5.6 Lower Sand and Gravel

A lower deposit of sand and gravel ranging to gravelly sand extends across the site and the layer overlies the bedrock. This deposit ranges in thickness from 2.0 m to 6.4 m and extends to depths of 10.5 m to 14.6 m or to elevations of 272.0 m to 269.6 m. The material also contains trace silt, occasional to some cobbles and distinct layers of boulders.

Selected samples from this material were subjected to grain size distribution testing and the results are shown in Figures B4 and B5 in Appendix B.

Standard Penetration tests in this deposit gave 'N' values that were more than 50 blows per 0.3 m penetration, indicating that the material has a very dense relative density. Some of the high SPT 'N' values may also represent tests conducted on cobbles and boulders.

The moisture content of samples from this deposit ranges from approximately 6% to 23%.

5.7 Bedrock

The overburden soils described above are underlain by limestone bedrock. Bedrock was proved by coring at the south abutment and at each of the four piers. Table 5.1 summarizes the bedrock depth and the elevations to the top of bedrock where rock was cored and where refusal was encountered on probable bedrock, but the rock was not cored.

The limestone bedrock is generally described as highly to moderately weathered, thinly bedded and grey in colour. Occasional pitted zones and occasional to frequent rubble zones indicate that the rock carries water bearing seams.

TABLE 5.1 – Depth to Bedrock at Foundation Elements

Location	BH Number	Depth to Bedrock (m)	Top of Bedrock Elevation (m)
Pier #1	06-3	10.5 / 9.0*	272.0
	06-4	11.7 / 9.7*	270.8
Pier #2	06-5	10.5 / 8.2*	272.0
	06-6	11.1 / 9.3*	271.4
Pier #3	06-7	12.0**	271.3**
	06-8	13.4	270.3
Pier #4	06-9	11.8	271.6
	06-10	12.2**	271.6**
South Abutment	06-11	14.0**	270.5**
	06-12	14.6	269.6

*Denotes depth to bedrock below river water level / below riverbed level.

**Denotes where refusal was encountered on probable bedrock.

Core recovery in the bedrock was between 55% and 100%. The RQD values generally ranged from 0% to 54% indicating very poor to poor rock quality.

The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, was generally high, ranging from 5 to greater than 10. The Fracture Indices greater than 10 indicate the presence of rubble zones within the rock mass. Some vertical joints were encountered in Borehole 06-6 and they were mostly tight with little to no sand infilling or secondary weathering material.

The estimated unconfined compressive strength of the rock cores tested generally ranges between 41 and 86 MPa indicating a medium strong to strong rock with occasional cores exhibiting higher strength values of 105 to 155 MPa. These estimated rock strength values are based on point load tests that were conducted on rock cores recovered from the boreholes. Due to very poor rock quality in the cores, no point load tests were conducted on samples from Boreholes 06-3 and 06-4. A summary of the Point Load Test Results is presented in Table B1 in Appendix B.

5.8 Water Levels

A standpipe piezometer was installed in a selected borehole at each foundation element except for Piers 1 and 2, which are located in the river. Water levels were measured on separate visits made after the completion of drilling. The water level readings at the foundation elements are presented in Table 5.2.

Based on these observations, local groundwater levels exist at Elevations 283.0 m to 284.9 m. All groundwater observations at this site are short term and the levels are expected to fluctuate seasonally and after severe weather events.

Table 5.2: Water Level Measurements

Date	BH 06-2 N-Abutment	BH 06-7 Pier 3	BH 06-10 Pier 4	BH 06-11 S-Abutment
	Depth/ Elev. (m)	Depth/ Elev. (m)	Depth/ Elev. (m)	Depth/ Elev. (m)
August 1, 2006	-	1.5 / 281.9	-	-
August 9, 2006	-	0.3 / 283.1	0.8 / 283.0	-
August 10, 2006	-	0.4 / 283.0	0.8 / 283.0	1.3 / 283.2
August 11, 2006	-	0.4 / 283.0	0.8 / 283.0	1.2 / 283.3
August 14, 2006	-	0.4 / 283.0	0.9 / 282.9	1.2 / 283.3
August 15, 2006	-	0.4 / 283.0	0.9 / 282.9	1.2 / 283.3
August 16, 2006	-	0.4 / 283.0	0.9 / 282.9	1.2 / 283.3
September 29, 2006	-	0.4 / 283.0	0.7 / 283.1	1.5 / 283.0
January 4, 2007	9.89 / 284.9	-	-	-

6 MISCELLANEOUS

All-Terrain Drilling Limited of Waterloo, Ontario supplied track and truck mounted CME 75 drill rigs and conducted the drilling, sampling and in-situ testing operations for the boreholes drilled on land. Canadian Soil Drilling of Midhurst, Ontario supplied a barge mounted CME 75 drill rig and conducted the drilling, sampling and in-situ testing operations for the boreholes drilled in the Grand River. Water Systems Analysts of Guelph, Ontario conducted the mussel relocation program.

The drilling and sampling operations in the field were supervised on a full time basis by Mr. Stephane Loranger, Mr. George Azzopardi and Mr. Mark Farrant of Thurber.

Mr. Alastair E. Gorman, P.Eng. and Mr. Mark E. Farrant, P.Eng. directed the field operations and prepared the report.

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

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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 structure.

It is understood that the SBL of the widened Highway 8 will cross the Grand River on a new structure to be constructed to the west (downstream) side of the existing structure carrying Highway 8 over the river. The General Arrangement of the new structure is understood to match that of the existing bridge, i.e. five spans. The Grand River will pass between Piers 1 and 3, though most of the channel width will lie between Piers 1 and 2. From the south abutment to the river's edge, the structure spans the flood plain on the south side of the river.

At the south abutment, the finished grade of Highway 8 will be at Elevation 290.7 and the existing ground surface lies at Elevation 284.3. The resulting embankment height above original ground level will, therefore, be in the order of 6.4 m at the south abutment.

At the north abutment, the finished grade of Highway 8 will be at Elevation 295.0 and the existing ground surface, corresponding to the top of the original valley slope, averages Elevation 308 approximately resulting in an approach cut in the order of 13 m deep.

The discussion and recommendations presented in this report are based on our understanding of the project and on the factual data obtained in the course of this investigation. Reference has also been made to the boreholes drilled in a previous investigation by E.M. Peto Associates Ltd. that are included in Appendix C.

8 STRUCTURE FOUNDATIONS

Based on the boreholes drilled at the foundation elements, the site is underlain by limestone bedrock at elevations ranging from 269.6 to 272.0. Immediately overlying the bedrock is a very dense deposit of sand and gravel, overlain in turn by very dense silt till and by more recent, reworked river deposits. A discontinuous layer of hard silty clay till was identified at the south abutment.

Initial consideration was given to the following foundation types:

- Spread footings
- Augered Caissons (drilled shafts)
- Driven piles
- Micropiles

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

8.1 Spread Footings on Native Soil

The soil conditions encountered at this site are considered to be suitable for the use of spread footings. However, a number of factors must be taken into account in the design of spread footing for the new structure, including:

1. The risk of undermining the footings supporting the existing structure.
2. The constructability of foundations in the river for Piers 1 and 2.
3. The risk of the footings being undermined by scour and the requirements to prevent this.

Table 8.1 shows the interpreted elevations of the undersides of the existing footings and design parameters for the new footings.

Table 8.1 Foundation Parameters.

	N. Abut	Pier 1	Pier 2	Pier 3	Pier 4	S. Abut
Elevation of existing underside of footing	289.6	280.4	279.5	280.7	281.2	281.2
Founding elevation for new footing	289.0	279.5	279.3	280.0	281.2	280.5
Difference (m)	-0.6	-0.9	-0.2	-0.7	0.0	-0.7
ULS_f bearing resistance (kPa)	690	1,000	1,000	1,000	750	750
SLS bearing resistance (kPa)	460	N/A[1]	N/A[1]	N/A[1]	N/A[1]	N/A[1]
Groundwater elevation	284.9	282.5[2]	282.5[2]	283.1	283.1	283.0
Minimum thickness of working slab (mm)	150	150	150	150	150	150
Coefficient of sliding friction (ultimate)	0.6	0.7	0.7	0.7	0.7	0.7
Anticipated tip elevation of sheet piling	N/A	272.5	272.3	N/A	N/A	N/A

1) The SLS condition will not govern.
2) River level, subject to fluctuation. Other groundwater levels will fluctuate with the season and recent weather events. The groundwater levels at the piers and south abutment will be strongly influenced by the river level, the north abutment less so.

8.2 Constructability and Footing-Specific Issues

The constructability of the new foundations is potentially influenced by their location relative to the existing foundations and is discussed in the following sections.

8.2.1 North Abutment

Three additional factors that must be taken into account for a spread footing design at this foundation element are:

1. The effect of the slope on available geotechnical resistance
2. The possibility of slope creep
3. The impact of future erosion of the toe of the slope

The first two factors above have been taken into account in the parameters provided in Table 8.1.

With respect to the third factor above, erosion, the overall design of the project must incorporate river bank stabilization. Alternatively, the expected erosion of the slope over the design life of the structure must be determined and the north abutment footing situated such that it will remain stable after the erosion has taken place.

If the space between the new and existing footings is less than 0.6 m (the difference in founding elevations), refer to Section 8.2.5.

8.2.2 Piers 1 and 2

Piers 1 and 2 will lie in the Grand River and the founding soils are represented at Pier 1 by Boreholes 06-3 and 06-4 and at Pier 2 by Boreholes 06-5 and 06-6.

Two issues specific to these piers are:

- Constructability in the river
- Scour protection

The potential depth of scour must be determined by a river hydrologist, making reference to CHBDC (CAN/CSA-S6-06) Sections 1.9.4 and 1.9.5 and appropriate scour protection must be designed to prevent undermining of the footings by the river.

One solution that can be considered to permit construction in the dry, and to provide some protection from scour, is to construct the footing inside a permanent, interlocking, steel sheet pile enclosure. The original drawings for the existing bridge indicate that Piers 1 and 2 of the existing structure were constructed in this manner and they appear to be functioning satisfactorily.

For constructability concerns, the sheet piling must be driven to sufficient depth below the river bed to allow excavation to be carried out inside the sheet pile enclosure without the base of the excavation being destabilized by unbalanced groundwater heads. This can generally be achieved by driving the sheet piling to sufficient depth to reduce the hydraulic exit gradient of seepage to 0.1 or less.

One construction sequence that is considered to be feasible for this site is as follows:

1. Install the interlocking sheet piling to the specified elevation.
2. Pump out the water contained inside the enclosure.
3. Excavate the soil inside the enclosure to the specified elevation, continuing to pump any seepage water.
4. Hand clean all disturbed soil from the base of the excavation and pour a concrete working slab, as specified. It is considered important that the excavation, hand cleaning and pouring of concrete all proceed in one uninterrupted process and be completed as quickly as is feasible and consistent with safety. The contract documents should contain an instruction to the contractor that a schedule is required that allows for completion of excavation, approval by the QVE and placement of the working slab as one continuous process that does not leave the completed excavation base unprotected.
5. After the working slab has been completed, construction of the spread footing and the pier may continue.

During construction, the top of the sheet piling must be high enough to exclude river water from the work area. From a geotechnical perspective, on completion of construction the sheet piling may be cut off flush with the top of the completed footing.

Since the sheet piling will act as a cofferdam during construction, the detailed design of the sheet piling and construction process must be carried out by the Contractor, taking account of safety, his proposed construction methodology and allowing for the highest river level anticipated during the construction period.

If the space between the new and existing footings is less than 0.9 m (the difference in founding elevations) at Pier 1, refer to Section 8.2.5. Pier 2 will not require stepping.

8.2.3 Piers 3 and 4

Piers 3 and 4 will lie in the flood plain of the Grand River and the founding soils are represented at Pier 3 by Boreholes 06-7 and 06-8 and at Pier 4 by Boreholes 06-9 and 06-10.

The footprints of Piers 3 and 4 are underlain by very dense soil considered to be suitable for the support of spread footings. The recommended founding elevations, however, lie approximately 3.5 m below the groundwater level observed during the investigation and the soils through which excavation will be carried out have an estimated permeability of $5x10^{-2}$ cm/sec, based on grain size distribution. It will be necessary, therefore, for the contractor to implement groundwater control measures during construction and prior to excavating below the groundwater level.

The design of the groundwater control system is the responsibility of the contractor. However, two systems that might be considered are:

- Vacuum well-points installed around the proposed excavation
- Interlocking steel sheet piling installed as a cutoff around the foundation excavation

If the space between the new and existing footings is less than 0.7 m (the difference in founding elevations) at Pier 3, refer to Section 8.2.5. Pier 4 will not require stepping.

8.2.4 South Abutment

The south abutment of the structure will lie in the flood plain of the Grand River and the founding soils are represented by Boreholes 06-11 and 06-12.

The footprint of the abutment is underlain by very dense soil considered to be suitable for the support of spread footings. The recommended founding elevation, however, lies approximately 2.5 m below the groundwater level observed during the investigation and the soils through which excavation will be carried out have an estimated permeability of $1x10^{-2}$ cm/sec, based on grain size distribution. It will be necessary, therefore, for the contractor to implement groundwater control measures during construction and prior to excavating below the groundwater level.

The design of the groundwater control system is the responsibility of the contractor. However, two systems that might be considered are:

- Vacuum well-points installed around the proposed excavation
- Interlocking steel sheet piling installed as a cutoff around the foundation excavation

If the space between the new and existing footings is less than 0.7 m (the difference in founding elevations), refer to Section 8.2.5.

8.2.5 Stepping Between Founding Elevations

Where a new footing is founded at a lower elevation than the adjacent, existing footing, undisturbed, original ground must remain below a plane projected downwards from the edge of the existing footing at 45°. This condition can be satisfied if the spacing between the footings is equal to or greater than the difference in founding elevations.

In situations where the spacing between the footings is less than the difference in founding elevations, it is recommended that the east end of the excavation be taken down to the elevation of the existing footing and then be sloped down at 45° to match the required founding elevation. Mass concrete may be placed up to the elevation of the existing footing to provide a uniform elevation on which to construct the new footing.

Sloping of the excavation is not required if the new footing excavation is entirely enclosed in steel sheet pile shoring that is designed to allow no movement or loss of ground under the existing footing.

8.2.6 Inclined Loads

The recommended geotechnical resistances provided above are for concentric, vertical loads. Where eccentric or inclined loads are applied, the resistance used in design must be reduced in accordance with the CHBDC Clause 6.7.3 and Clause 6.7.4.

8.3 Augered Caissons

Consideration was given to supporting the structure on augered caissons (drilled piles). Since the caisson is a deep foundation unit, higher geotechnical resistance is available from a caisson in earth than from a similar sized spread footing. However, at this site caissons have the following disadvantages:

1. They must be installed to greater depth than spread footings in order to develop the higher resistance.
2. The soil providing the resistance, whether it is skin friction or end bearing, must be protected from disturbance.
3. Installation of the caisson to sufficient depth to satisfy (1) above may place the critical sections of the shaft in non-cohesive, possibly bouldery soil below the water table where it will be very difficult to be sure that (2) above is satisfied.
4. The installation costs, particularly in the river, are expected to be higher than for spread footings.

From a geotechnical risk perspective, caissons founded in earth are not recommended at this site.

Caissons founded on bedrock were also considered, but in some boreholes the rock is closely fractured near the surface. As a consequence, deep shafts into the bedrock would be required in order to provide confidence in the founding conditions. Caissons founded in bedrock are not recommended at this site.

8.4 Driven Steel Piles

Driven steel piles will achieve effective refusal in the very dense soils underlying the site.

The estimated founding elevations for steel H-piles and the corresponding pile lengths are shown in Table 8.2.

Table 8.2 – Estimated Pile Lengths

Location	Borehole No.	Elevation of Ground Surface or River Bed	Estimated Pile Tip Elevation	Estimated Length of Pile* (m)
North Abutment	06-2	294.8	291.0	3.8
Pier 1	06-3	281.0	278.0	3.0
	06-4	280.5	277.0	3.5
Pier 2	06-5	280.2	277.0	3.2
	06-6	280.7	277.3	3.2
Pier 3	06-7	283.4	277.5	6.1
	06-8	283.7	275.6	8.1
Pier 4	06-9	283.4	277.0	6.4
	06-10	283.8	277.5	6.3
South Abutment	06-11	284.5	276.8	7.7
	06-12	284.2	276.0	8.2

* From ground surface existing at the time of investigation. In the case of the north abutment, piles will be driven from lower elevations to be determined in the course of the design process.

The actual length of pile will be less than the values in the table as it will be measured from the underside of the pile cap.

Driven piles are considered to be feasible at Piers 3 and 4 and at the south abutment but are not recommended at the north abutment or at Piers 1 and 2 due to the very short length that will actually be driven.

If driven piles are to be used at Pier 1, Pier 2, it is recommended that the minimum length of pile be at least 5 m below the underside of the pile cap. The contract should contain direction that the contractor must be prepared to predrill to a depth of up to 4 m, if necessary, in order to install the piles to the required minimum length without damage. The same provision applies at the north abutment in the case of a conventional or semi-integral abutment.

If driven piles are selected at the north abutment to implement an integral abutment design, then the required length of pile must be determined on the basis of:

- The "free" length of pile required to provide flexibility, and
- Sufficient length to develop the geotechnical resistance

Typically, a free length of at 3 m is required. At this site, however, due to the length of the structure a greater free length may be found to be necessary on the basis of the structural analysis. It is anticipated that the pile will achieve geotechnical resistance approximately 2 m below the free length.

8.4.1 Axial Resistance

An HP 310 X 110 pile may be designed on the basis of:

- 1,800 kN factored geotechnical resistance at ULS
- 1,600 kN geotechnical resistance at SLS

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

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

8.4.2 Downdrag

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

8.4.3 Pile Tips

The tips of all piles should be fitted with cast steel, H-section rock points from an approved manufacturer such as Titus Steel (Standard H-point) or APF hard Bite or approved equivalent.

8.4.4 Pile Installation

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

8.4.5 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 tips are approaching the bearing stratum, i.e. below Elevation 279, except at the north abutment where the elevation will be higher and determined on the basis of the final foundation design. The appropriate pile driving note is "Piles to be driven in accordance with Standard SS 103-11 using an ultimate resistance of 3,600 kN.

8.5 Micropiles

From a foundation feasibility point of view, the foundations could be supported on micropiles socketed into the bedrock.

Typical micropile installation techniques would be capable of penetrating the very dense sand and gravel with cobbles and boulders to penetrate into the bedrock and provide a high capacity foundation. The advantages of this system are that it is less susceptible to disturbance of the excavation base and would avoid any risk of undermining the existing foundations. The disadvantages relate mainly to the cost and to the fact that they are generally installed by a specialty contractor.

If this option has to be explored further, it will be necessary to discuss the design with a contractor in order to develop appropriate resistances to be used in design.

8.6 Pile Lateral Resistance

The geotechnical lateral resistance acting on a pile may be calculated using a value for the coefficient of horizontal subgrade reaction (k_s) and ultimate lateral resistance (p_{ult}) as follows:

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

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

where z = depth of embedment of pile in metres
D = pile width in metres
n_h = coefficient of horizontal subgrade reaction (Table 8.3)
γ = unit weight (Table 8.3)
K_p = passive earth pressure coefficient (Table 8.3)

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

Table 8.3 – Recommended Soil Parameters

Location	**Elevation**	**n_h (kN/m^3)**	**K_p**	**Unit Weight* (kN/m^3)**	**Soil Conditions**
North Abutment	OGL to 293.5	2,000	2.5	20	Clay fill, Silt and sand, loose
	293.5 to 283.9	8,000	3.3	21	Sand, some gravel and cobbles, dense to very dense.
	283.9 to 275.0	5,000	3.0	11	Hard silty clay till. Very dense sand and gravel
South Abutment	OGL to 278.9	15,000	3.3	11	Compacted to dense sand and gravel, compact to very dense sand and silt till
	278.9 to 270.5	15,000	3.3	11	Very dense sand and very dense sand and gravel, hard silty clay

*Buoyant unit weight below the water table.

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

For lateral soil/pile group interaction analysis, the equation for k_s and p_{ult} quoted above may be used in conjunction with appropriate reduction factors.

Where a pile group is oriented ***perpendicular*** to the direction of loading, group action may be considered by reducing values for k_s and p_{ult} 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.

8.7 Recommended Foundation

The recommended foundation system for this structure is abutments and piers supported on spread footings bearing on undisturbed, very dense native soil.

8.8 Abutment Considerations

On the basis of the geotechnical conditions on site and the recommended foundation type, conventional abutment design is recommended. Semi-integral abutments could also be designed, supported on spread footings.

If an integral abutment design is considered to be appropriate, on the basis of other considerations, then driven H-pile foundations must be used at the abutments.

If an integral or semi-integral abutment design is considered, it may require special consideration of the magnitude of movement to be accommodated and detailed analysis of the soil-structure interaction. Such analysis can be developed, if necessary, as the design proceeds.

8.9 Frost Cover

Pile caps and footings on earth must be provided with a minimum of 1.4 m of earth cover over the footing base (founding elevation).

8.10 Erosion Protection

It is recommended that the foundations of Pier 1 and Pier 2 be protected from erosion and undercutting by the river. Protection could be provided by permanently installed steel sheet piling as discussed earlier in this report. Alternatively, a specialist in river hydrology should be consulted regarding these requirements.

The recommendations provided for the north abutment foundation are based on the present location and geometry of the north valley slope, which implicitly assumes that there will be no erosion of the slope in the future. A specialist in river hydrology should be consulted regarding the potential for erosion and, if necessary, either erosion protection must be provided at the toe of the slope or the north abutment foundation must be designed to lie beyond the zone of influence of the erosion predicted in the design life of the structure.

9 EXCAVATION AND BACKFILL

9.1 General

All excavations must be carried out in accordance with the Occupational Health and Safety Act (OHSA) and in accordance with Special Provision 902S01. For the purposes of the OHSA, the native soils at this site that will be excavated in open excavations may be classified as Type 2 soils except for the sand that may be exposed at the north abutment that should be treated as Type 3 soil. Excavation below the groundwater level is not recommended without prior dewatering. Provided dewatering is carried out as described below, temporary excavations may be sloped at 1H:1V.

9.2 Foundations

The excavation 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 very dense, cohesionless soils extending below the water table and that man-made fill or obstructions, cobbles or boulders may be encountered.

The presence of cobbles and boulders in very dense soil may present difficulties for installing steel sheet piling.

Excavations formed to the elevation of the underside of the pile cap, as discussed in Section 8, will lie at or slightly above the groundwater levels recorded during the investigation. The sides and base of the excavation must be maintained in a stable condition and Bidders must be alerted to the fact that groundwater levels and the river level will vary and may be higher at the time of construction.

10 GROUNDWATER AND FLOOD CONTROL

At the time of investigation, the groundwater level at Piers 3 and 4 and the south abutment lay at depths of 0.4 to 1.5 m below the ground surface. The groundwater level will vary and may be higher at the time of construction. At this site, the design of dewatering and protection systems must also take account of the possibility of the Grand River level rising rapidly due to flood conditions. The groundwater and surface (flood) water must be controlled during construction to maintain a stable excavation and to allow concrete to be placed in an unwatered excavation.

The design of the groundwater and surface water control systems is the responsibility of the Contractor. However, suitable systems that might be considered include pumping from filtered sumps for nominal penetration below the groundwater level or the use of a sheeted excavation or vacuum well-points.

At the north abutment, the groundwater level was measured at Elevation 284.9, 9.9 m below ground surface. It is not expected that any excavation at the north abutment will penetrate to that elevation, but the contractor must be prepared to deal with any seepage entering the excavation and to maintain an unwatered condition.

Any accumulation of water from the base of the excavation should be removed prior to placing concrete or compacting granular fill. Placement of concrete or compacting engineered fill must be done in the dry.

It should be noted that dewatering foundation excavations may invoke a requirement to have a Permit to Take Water (PTTW), as issued by MOE. It is recommended that this permit application be submitted prior to awarding the contract in order to avoid delay during construction.

11 STRUCTURE APPROACHES

11.1 South Approach

The south approach will lie on an embankment approximately 6.4 m above the Grand River flood plain. The soils on which the immediate approach embankment will be constructed consist of compact sand fill and compact to very dense sand and gravel.

It is recommended that the immediate approach embankment be constructed of SSM or granular fill and that the inclination of the side slopes not exceed 2H:1V. An embankment built according to this recommendation will possess satisfactory internal and global stability.

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

11.2 North Approach

The north approach to the structure will lie in a cut in the north valley slope that will have a maximum depth of approximately 13 m. The geotechnical design requirements for the mainline cut are addressed in a separate report by Thurber Engineering Ltd. entitled Foundation Investigation and Design Report, Mainline Cut Sta. 13+400 to Sta. 13+650, Highway 8 Widening, Kitchener, G.W.P. 277-97-00".

At the immediate approaches to the bridge, it can be assumed that a 2H:1V cut slopes will be stable.

The global stability of the valley slope below the proposed structure was carried out by the modified Bishop method using GSlope software from Mitre Software. The results of the analysis indicate that to achieve a minimum factor of safety of 1.3, the forward edge of the footing must be at least 7 m behind the face of the slope and the founding elevation must not be above 287.0.

The graphical output of the slope stability analysis is included in Appendix H.

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

12 RETAINED SOIL SYSTEMS

Retained soil system (RSS) walls may be used subject to the requirements presented in this section.

RSS walls must be specified to be "High Performance" and "High Appearance". The contract drawings and documents must 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 must included a reference to the most recent version of the Special Provision RSS walls and any related NSSPs.

12.1 Foundation

The performance of an RSS is dependent, among other factors, on the characteristics of its foundation. Failure to provide an adequate foundation may lead to settlement and distortion of the RSS and, in severe cases, to possible failure of the system. The 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 soil that is compact/very stiff or better. The highest elevations for founding the RSS wall are:

- North abutment 293.0
- South abutment 283.0

The subgrade must be competent and free of organics, soft or deleterious soils.

The RSS mass must be constructed in the dry and the excavation must be unwatered as necessary to achieve the dry conditions.

The following parameters may be used for the design of the RSS foundation on native soil:

	North Abutment	**South Abutment**
Highest founding elevation	293.0	283.0
Factored geotechnical resistance at ULS	450	450
Geotechnical resistance at SLS	300	300
Coefficient of friction for sliding resistance	0.6	0.6

Total settlement under a RSS mass constructed as outlined above is expected to be less than 25 mm and to occur essentially as the RSS is constructed. Differential settlement is not expected to exceed 20 mm in a 6 m span.

If necessary, particularly at the south abutment, engineered fill may be placed at the founding elevations given above and up to the required elevation of the underside of the RSS wall. If a thin pad of engineered fill pad is required to make up differences in elevation from the approved native soil to the underside of wall, it is recommended that the bearing resistances for native soil be used. If the thickness of engineered fill exceeds 2 m, the following parameters may be used for the design of the RSS mass:

- Factored geotechnical resistance of 900 kPa at Ultimate Limit States (ULS)
- Geotechnical resistance of 350 kPa at Serviceability Limit States (SLS)
- Ultimate coefficient of sliding resistance of cast in-situ concrete levelling pad on engineered fill = 0.7
- Ultimate coefficient of sliding resistance of RSS mass on Granular A = 0.6

The RSS is a proprietary system and the supplier must design for internal, sliding and overturning stability and for any other failure modes identified by the supplier.

RSS walls constructed as described above on the very dense native soils at this site will satisfy global stability requirements.

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 but rock backfill can be permitted. A NSSP is required to limit rock fill used as abutment backfill to fragments no greater than 300 mm and to include adequate spalls to fill voids in the rock fill.

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

The backfill to the abutment walls should be in accordance with OPSS 902 as amended by Special Provision 902S01. Granular backfill should be placed to the extents shown in OPSD 3101.150, and rock backfill should be placed to the extents shown in OPSD 3101.200.

All granular material should meet the specifications of Special Provision 110F13 "Amendment to OPSS 1010, March 1993". Compaction equipment to be used adjacent to retaining structures should be restricted in accordance with SSP 105S10.

Some settlement will occur within the mass of the approach fill after the fill has been completed. For design purposes, the settlement at final grade should be assumed to equal 0.5% of the height of the fill for rock fill and 1.0% of the height of the fill for earth fill.

The design of the abutment should incorporate a subdrain as shown in OPSD 3501.000 or OPSD 3505.000, as applicable.

14 EARTH PRESSURE

For cases where backfill to the abutment is placed in accordance with OPSD 3101.150 or OPSD 3101.200, as recommended, the lateral earth pressure will be governed by the properties of the material within the backfill limits shown in the respective OPSD, i.e. a line projected up at 1.5H:1V for granular backfill and 1.25H:1V for rock backfill.

If the support system allows yielding of the wall (unrestrained system), active horizontal earth pressure may be used in the geotechnical design of the structure. If the support system does not allow yielding (restrained system), at-rest horizontal earth pressures should be used.

Earth pressures acting on the structure should be computed in accordance with Clause 6.9 of the CHBDC but generally are given by the expression:

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

P_h = horizontal pressure on the wall (kPa)

K = earth pressure coefficient (see table below)

γ = unit weight of retained soil (see table 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 at a depth of 1.7 m for Granular A or Granular B Type II.

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

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) would result in lower earth pressures acting on the wall. In the case of integral or semi-integral abutments, material with a lower passive pressure coefficient (e.g. Granular B Type I) would result in lower forces acting on the ballast wall as the wall moves toward the soil mass. However, the use of Granular "B" Type I may be restricted if the approach embankment consists of rock fill.

The factors in the Table 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 CHBDC, 2006.

Table 14.1 – Earth Pressure Coefficients

Wall Condition	**Earth Pressure Coefficient (K)**					
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ$; $\gamma = 22.8$ kN/m^3		OPSS Granular B Type I $\phi = 32^\circ$; $\gamma = 21.2$ kN/m^3		Rock Fill $\phi = 42^\circ$; $\gamma = 19.0$ kN/m^3	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (Unrestrained Wall)	0.27	0.40*	0.31	0.48*	0.20	0.28*
At rest (Restrained Wall)	0.43	-	0.47	-	0.33	-
Passive (Movement Towards Soil Mass)	3.70	-	3.30	-	5.0	-

* For wing walls.

15 SEISMIC CONSIDERATIONS

15.1 Seismic Design Parameters

The following seismic parameters should be used for design:

Velocity Related Seismic Zone	0
Zonal Velocity Ratio	0.05
Acceleration Related Seismic Zone	1
Zonal Acceleration Ratio	0.05
Peak Horizontal Acceleration	0.08

The soil profile type at this site has been classified as Type I. Therefore, according to Table 4.4.6.1 of the CHBDC, a Site Coefficient "S" (ground motion amplification factor) of 1.0 should be used in seismic design.

15.2 Liquefaction Potential

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

Using this method and assuming an earthquake of magnitude 7.5, it is estimated that under the existing conditions there is negligible potential for liquefaction of the foundation soils below the abutments. Therefore, the vertical geotechnical resistance of the foundations and embankments will not be compromised.

The embankments themselves will be constructed above the groundwater level and are not considered to be in danger of undergoing liquefaction. Some toe failure may occur but it is expected to be of limited nature and readily repairable.

15.3 Retaining Wall Dynamic Earth Pressures

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

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

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[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.

Table 15.1 – Earth Pressure Coefficient for Earthquake Loading

Earth Pressure Coefficient (K) for Earthquake Loading						
Wall Condition	Granular A or Granular B Type II $\phi = 35^\circ$; $\delta = 17.5^\circ$ $\gamma = 22.8$ kN/m^3		OPSS Granular B Type I $\phi = 32^\circ$; $\delta = 16^\circ$ $\gamma = 21.2$ kN/m^3		Rock Fill $\phi = 42^\circ$; $\delta = 21^\circ$ $\gamma = 19.0$ kN/m^3	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (K_{AE})*	0.3	0.45	0.33	0.54	0.23	0.31
Passive (K_{PE})	6.3	6.3	5.4	5.4	12.0	12.0
At Rest (K_{OE})**	0.59		0.63		0.33	

* 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 the issues discussed below.

Impact on Existing Structure

The recommendations presented in this report have been formulated taking account of the existing bridge foundations and possible impacts of the new construction. However, it is possible that unforeseen circumstances may cause impacts on the existing structure and, from a foundations perspective, the most serious would be settlement under the existing foundations.

It is recommended that the contract documents include a monitoring program for the existing structure. As a minimum, this program should require the contractor to establish a reference point on the west end of each pier cap and abutment of the existing structure and to monitor movement of these points relative to known fixed reference points on a regular basis. The suggested frequency is:

- Three readings on separate days prior to construction to establish a baseline
- Daily while any foundation construction or other subsurface construction is in progress
- Weekly when daily readings are not required.

Reading should be taken at the same time each day, preferably first thing in the morning before solar heating affects the structure.

The vertical and horizontal accuracy of readings should be 2 mm. All readings must be reported to the contract administrator within 24 hours and immediately if any movement exceeds 10 mm.

The contract administrator must be advised of the importance of monitoring and be required to advise the Ministry immediately if the vertical or horizontal movement exceeds 10 mm. These values are selected from foundation considerations and more stringent requirements may be imposed from structural considerations.

Potential Disturbance or Loss of Ground

The construction recommendations provided in this report are aimed at reducing the risk of the founding surface being disturbed or loss of ground occurring under an existing footing but unforeseen circumstances may cause one of these conditions to develop. The QVE must be made aware that it is a contractual requirement that the new foundations be constructed without disturbance to the base of the excavation or loss of ground under the existing footings. If either of these conditions is found to be developing, he must bring it to the attention of the Contract Administrator (CA) immediately. The CA must make a decision as to whether the Contractor needs to take steps to protect the site and whether the designer must be contacted to review the situation.

Unwatering

The contract documents should flag unwatering of the foundation excavations, particularly those in the river, as being potentially difficult and requiring input from dewatering experts.

RSS Walls

The appearance and performance of RSS walls is dependent, in part, on the performance of the foundation. It is important that the wall be treated as a structural element and be provided with a foundation as described in this report.

Installation of Sheet Piles

The site investigation and field testing revealed the presence of very dense sand and gravel containing cobbles and boulders. Installation of sheet piling may be difficult under these conditions and contractors must allow for the possibility of predrilling in some locations.

17 CLOSURE

Engineering analysis and preparation of the report were carried out by Mr. Alastair E. Gorman, 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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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

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(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**		**Field Estimation of Hardness***
	(MPa)	**(psi)**	
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

Ministry of Transportation
Ontario

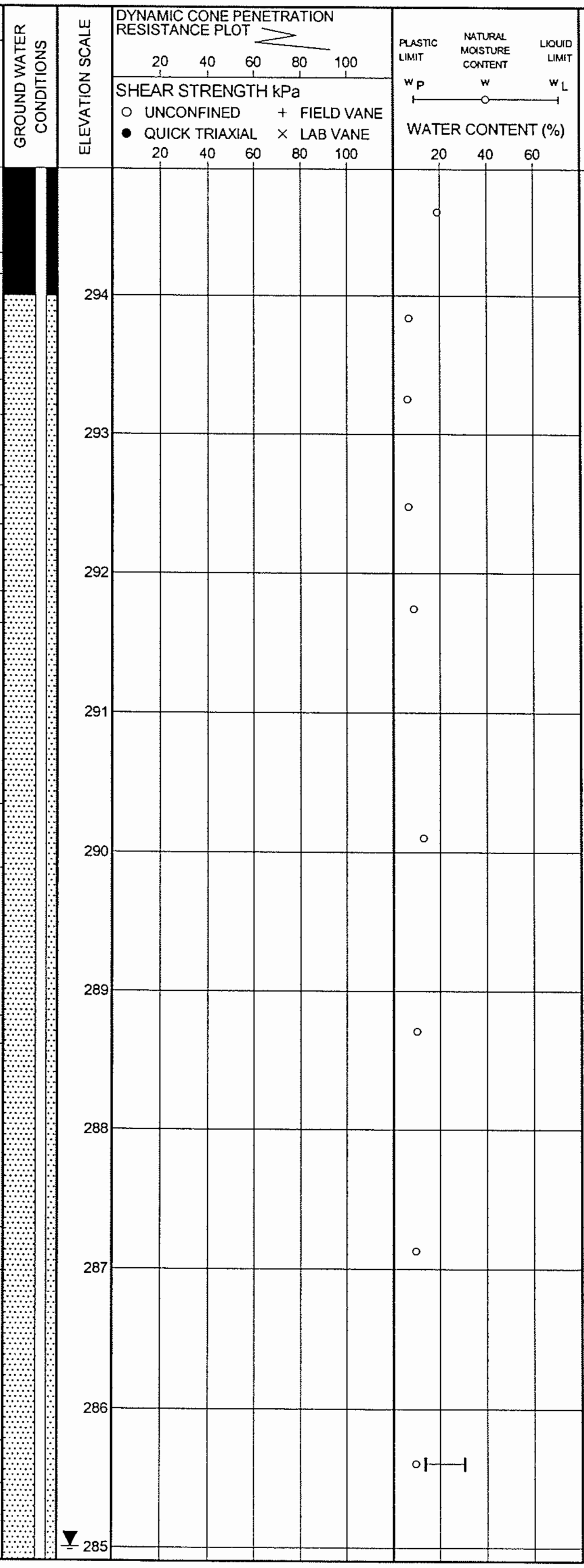
THURBER

RECORD OF BOREHOLE No 06-2

1 OF 3 **METRIC**

G.W.P. 277-97-00	LOCATION Grand River Overpass SBL N 4 809 407.87 E 230 460.97	ORIGINATED BY GA
HWY 8	BOREHOLE TYPE Hollow Stem Augers	COMPILED BY WM
DATUM Geodetic	DATE 28.09.06 - 28.09.06	CHECKED BY MEF

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
294.8					
0.0	**TOPSOIL** (125 mm)				
0.1	Silty **CLAY**, trace to some sand, trace gravel, occasional cobbles Stiff Brown (FILL)	1	SS	8	
294.1 0.8	**SILT and SAND**, some clay, trace gravel Loose to Very Dense Brown Dry (TILL)	2	SS	8	
		3	SS	50/ .100	5 38 41 16
		4	SS	101/ .275	
		5	SS	105/ .225	
290.3 4.6	**SAND**, medium to coarse grained Very Dense Brown Moist	6	SS	100	
288.8 6.0	Sandy **SILT**, trace gravel Very dense Brown Damp to dry (TILL)	7	SS	101/ .200	
287.2 7.6	Silty **CLAY**, some sand to sandy, trace gravel Hard Grey (TILL)(CL)	8	SS	104/ .050	
		9	SS	113	0 20 40 40



UNIT WEIGHT γ kN/m³

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+³, ×³: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 06-2

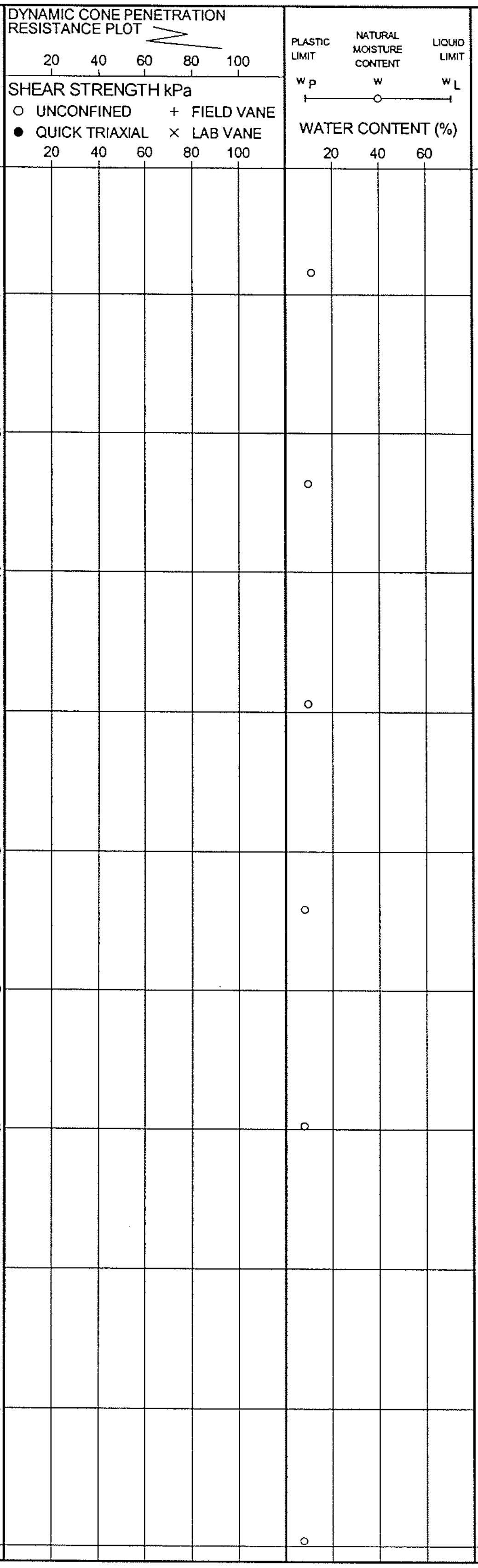
2 OF 3

METRIC

G.W.P.	277-97-00	LOCATION	Grand River Overpass SBL N 4 809 407.87 E 230 460.97	ORIGINATED BY	GA
HWY	8	BOREHOLE TYPE	Hollow Stem Augers	COMPILED BY	WM
DATUM	Geodetic	DATE	28.09.06 - 28.09.06	CHECKED BY	MEF

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	GR	SA	SI	CL
		10	SS	100/ .150	284				
					283				
		11	SS	109/ .150		2	26	52	20
					282				
		12	SS	103/ .250	281				
279.6					280				
15.2	Sandy SILT, some clay, trace gravel Very Dense Brown (TILL)	13	SS	100/ .150					
					279				
		14	SS	100/ .225	278	5	35	45	15
					277				
		15	SS	100/ .275		3	28	54	15
					276				
274.9		16	SS	100/					

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%)



ONTMT4S 7938.GPJ 05/01/07

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+ ³, × ³: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 06-2

3 OF 3 **METRIC**

G.W.P. 277-97-00
LOCATION Grand River Overpass SBL N 4 809 407.87 E 230 460.97
ORIGINATED BY GA
HWY 8
BOREHOLE TYPE Hollow Stem Augers
COMPILED BY WM
DATUM Geodetic
DATE 28.09.06 - 28.09.06
CHECKED BY MEF

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				GR	SA	SI	CL
19.9	END OF BOREHOLE AT 19.89 m Piezometer installation consists of 25mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH(m) ELEV.(m) 04.01.07 9.89 284.9				.075									

ONTMT4S 7938.GPJ 05/01/07

+ ^{3}, × ^{3}: Numbers refer to Sensitivity

$15 \oplus 5$ with 20 above and 10 below: (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

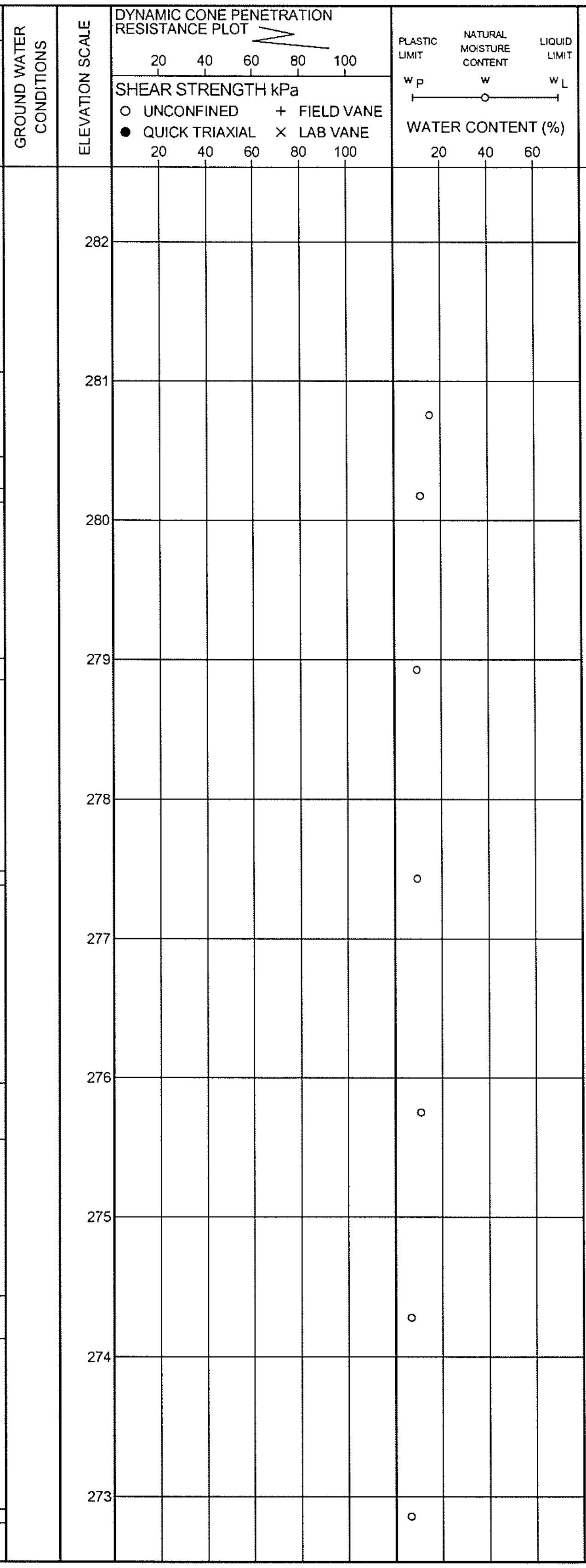
RECORD OF BOREHOLE No 06-3

1 OF 2

METRIC

G.W.P. 277-97-00
LOCATION Grand River Overpass SBL N 4 809 380.57 E 230 500.59
ORIGINATED BY MEF
HWY 8
BOREHOLE TYPE Hollow Stem Augers / NQ Tri-Cone / NQ Core Barrel
COMPILED BY JHL
DATUM Geodetic
DATE 2006-11-15 - 2006-11-16
CHECKED BY MEF

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
282.5								
0.0	**WATER**							
281.0 1.5	Gravelly **SAND**, some cobbles and rip-rap boulders, trace silt Compact Brown Wet	1	SS	22				
280.2 2.3	Sandy **SILT**, some gravel, trace clay, occasional cobbles and boulders Very Dense Grey Wet (TILL)	2	SS	50/ .100	5	50	36	9
		3	SS	60/ .150				
		4	SS	50/ .100				
276.5 6.0	Gravelly **SAND**, some cobbles, trace silt, occasional boulders Very Dense Grey Wet	5	SS	50/ .100				
		6	SS	50/ .150				
		7	SS	50/ .100				
	Rubble zone from 11.45 to 11.76 m							



UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

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+³, ×³: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 06-3

2 OF 2

METRIC

G.W.P. 277-97-00
LOCATION Grand River Overpass SBL N 4 809 380.57 E 230 500.59
ORIGINATED BY MEF
HWY 8
BOREHOLE TYPE Hollow Stem Augers / NQ Tri-Cone / NQ Core Barrel
COMPILED BY JHL
DATUM Geodetic
DATE 2006-11-15 - 2006-11-16
CHECKED BY MEF

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION Continued From Previous Page	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				
271.9 10.5	Highly weathered, thinly bedded, grey, **LIMESTONE BEDROCK** Rubble zone from 10.71 to 11.02 m		1	RUN			272			FI >10 >10	RUN 1# TCR=86%, SCR=43%, RQD=0%, UCS=MPa
271.2 11.3	END OF BOREHOLE AT 11.23 m. BOREHOLE GROUTED WITH BENTONITE TO RIVERBED SURFACE AT 1.47 m.										

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

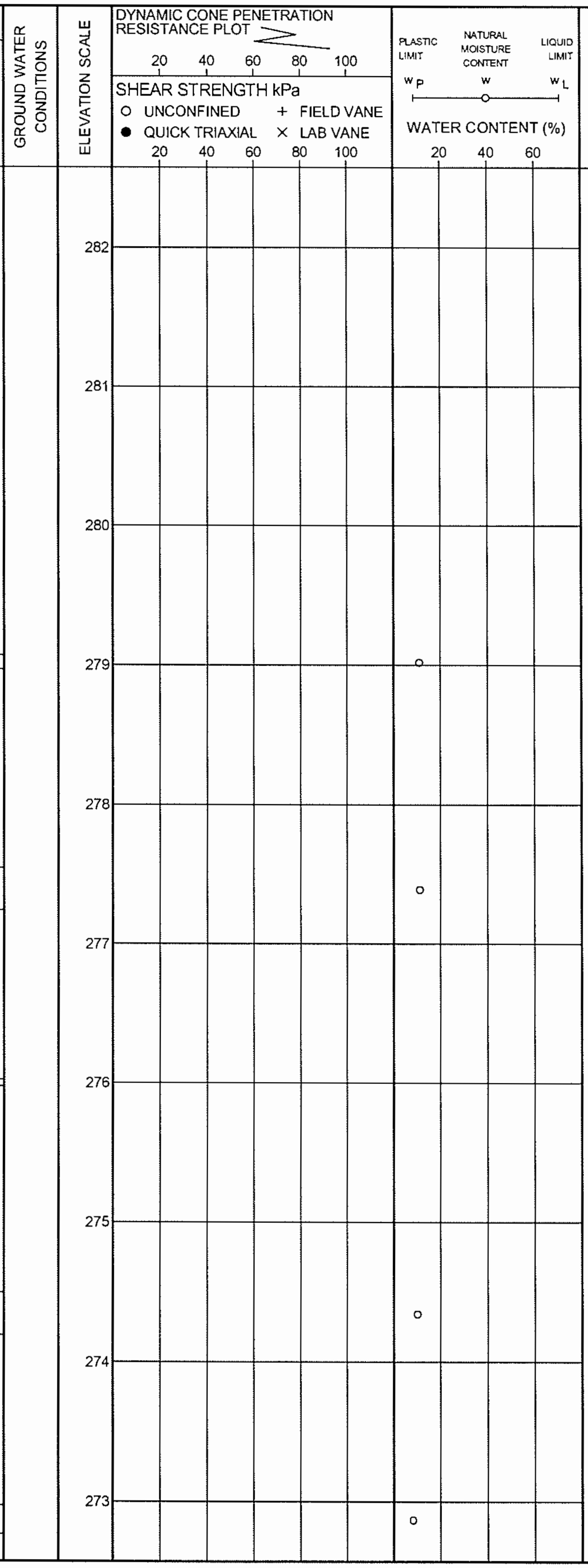
20 15-⊕-5 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 06-4

1 OF 2 **METRIC**

G.W.P. 277-97-00
LOCATION Grand River Overpass SBL N 4 809 369.73 E 230 496.77
ORIGINATED BY MEF
HWY 8
BOREHOLE TYPE Hollow Stem Augers / NQ Tri-Cone / NQ Core Barrel
COMPILED BY JHL
DATUM Geodetic
DATE 2006-11-20 - 2006-11-21
CHECKED BY MEF

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
282.5 0.0	**WATER**							
280.5 2.0	Gravelly **SAND**, some cobbles and rip-rap boulders, trace silt Brown Wet							
279.6 2.9	Sandy **SILT**, some gravel, trace clay Very Dense Grey Wet (TILL)	1	SS	50/ .100				
		2	SS	60/ .150	16	42	33	9
		3	SS	50/ .050				
275.3 7.2	Gravelly **SAND**, trace silt Very Dense Grey Wet	4	SS	50/ .150				
		5	SS	50/ .050				



UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

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+³, ×³: Numbers refer to Sensitivity

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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 06-4

2 OF 2 **METRIC**

G.W.P. 277-97-00	LOCATION Grand River Overpass SBL N 4 809 369.73 E 230 496.77	ORIGINATED BY MEF
HWY 8	BOREHOLE TYPE Hollow Stem Augers / NQ Tri-Cone / NQ Core Barrel	COMPILED BY JHL
DATUM Geodetic	DATE 2006-11-20 - 2006-11-21	CHECKED BY MEF

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE: 20 40 60 80 100
PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%): 20 40 60
UNIT WEIGHT γ kN/m³
REMARKS & GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT	REMARKS
	Continued From Previous Page								
272.0 10.5	Boulder at 10.52 to 10.97 m					272			
271.5 11.0		6	SS	50/ .100			o		
270.8 11.7	Highly weathered, thinly bedded, grey, **LIMESTONE BEDROCK**, with frequent rubble zones, occasional pitted zones					271 270		FI	
		1	RUN			269		>10 8 >10	RUN 1# TCR=94%, SCR=87%, RQD=16%, UCS=MPa
		2	RUN			268		>10 >10 >10 >10	RUN 2# TCR=83%, SCR=62%, RQD=7%, UCS=MPa
		3	RUN			267			RUN 3# TCR=64%, SCR=60%, RQD=0%, UCS=MPa
265.9 16.6	END OF BOREHOLE AT 16.61 m. BOREHOLE GROUTED WITH BENTONITE TO RIVERBED SURFACE AT 2.00 m.					266			

ONTMT4S 7938.GPJ 6/18/07

+ 3, × 3: Numbers refer to Sensitivity

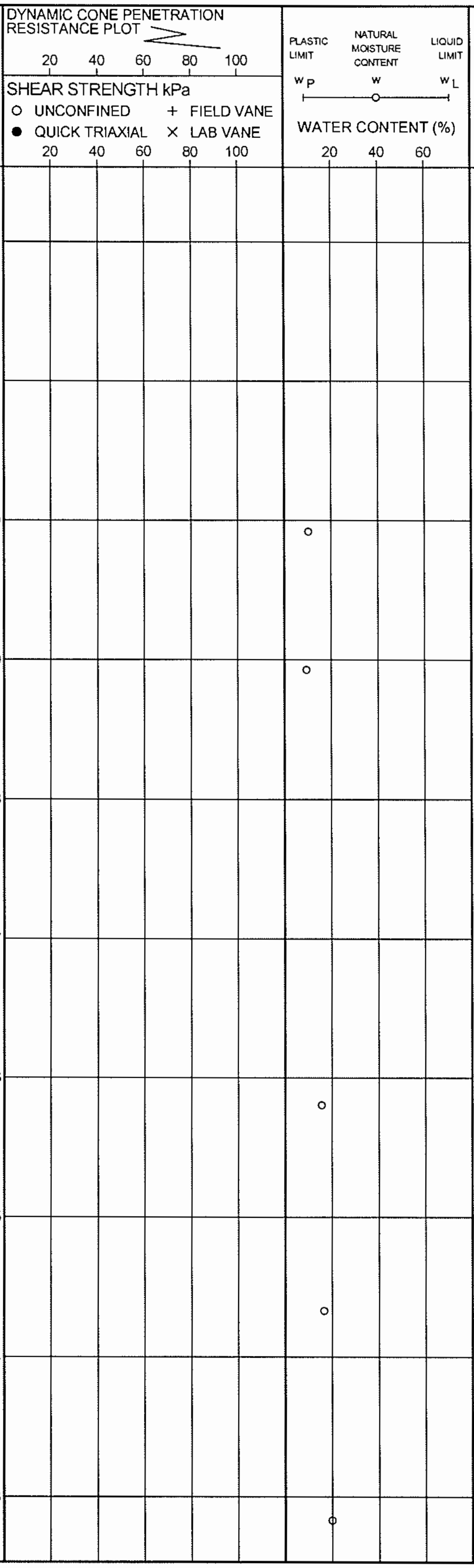
20 15-⊕-5 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 06-5

1 OF 2 **METRIC**

G.W.P. 277-97-00	LOCATION Grand River Overpass SBL N 4 809 359.26 E 230 533.45	ORIGINATED BY MEF
HWY 8	BOREHOLE TYPE Hollow Stem Augers / NQ Tri-Cone / NQ Core Barrel	COMPILED BY JHL
DATUM Geodetic	DATE 2006-11-10 - 2006-11-10	CHECKED BY MEF

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
282.5 0.0	**WATER**							
280.2 2.3	Gravelly **SAND**, some cobbles, trace silt Dense Brown Wet	1	SS	32	57	35	8 (SI+CL)	
279.2 3.2	Sandy **SILT**, some gravel, trace clay, occasional cobbles and boulders Very Dense Grey Wet (TILL)	2	SS	50/ .150	6	39	45	10
277.4 5.1	Gravelly **SAND**, trace silt, occasional cobbles Very Dense Grey Wet	3	SS	50/ .100				
		4	SS	50/ .150				
		5	SS	50/ .050				
		6	SS	60/ .150				



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+ 3, × 3: Numbers refer to Sensitivity

(%) STRAIN AT FAILURE

Ministry of
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Ontario

RECORD OF BOREHOLE No 06-5

2 OF 2 **METRIC**

G.W.P. 277-97-00
LOCATION Grand River Overpass SBL N 4 809 359.26 E 230 533.45
ORIGINATED BY MEF
HWY 8
BOREHOLE TYPE Hollow Stem Augers / NQ Tri-Cone / NQ Core Barrel
COMPILED BY JHL
DATUM Geodetic
DATE 2006-11-10 - 2006-11-10
CHECKED BY MEF

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 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										
271.9 10.5	Highly weathered, thinly bedded, grey, medium strong to strong, **LIMESTONE BEDROCK**, with frequent rubble zones, occasional pitted zones		1	RUN			272 271			FI >10 6 >10 >10	RUN 1# TCR=100%, SCR=78%, RQD=0%, UCS=70MPa
			2	RUN			270			>10 >10 8 >10 >10	RUN 2# TCR=94%, SCR=63%, RQD=0%, UCS=86MPa
269.0 13.4	END OF BOREHOLE AT 13.41 m. BOREHOLE GROUTED WITH BENTONITE TO RIVERBED SURFACE AT 2.31 m.										

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

20
15-⊕-5 (%) STRAIN AT FAILURE
10

Ministry of Transportation
Ontario

THURBER

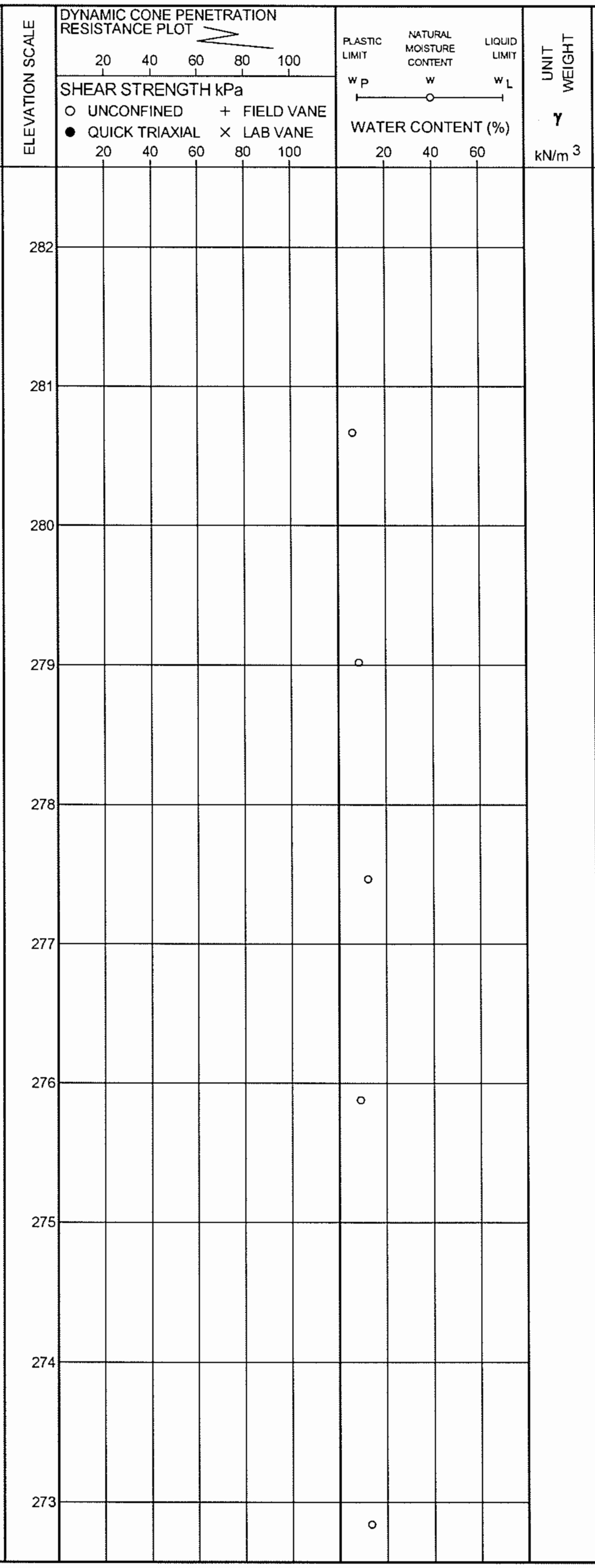
RECORD OF BOREHOLE No 06-6

1 OF 2

METRIC

G.W.P. 277-97-00
LOCATION Grand River Overpass SBL N 4 809 349.51 E 230 527.36
ORIGINATED BY MEF
HWY 8
BOREHOLE TYPE Hollow Stem Augers / NQ Tri-Cone / NQ Core Barrel
COMPILED BY JHL
DATUM Geodetic
DATE 2006-11-13 - 2006-11-14
CHECKED BY MEF

SOIL PROFILE ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES "N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
282.5							
0.0	**WATER**						
280.7 1.8	Gravelly **SAND**, some cobbles, trace silt Very Dense Brown Wet		1	SS	50/ .150		
279.8 2.7	Sandy **SILT**, some gravel, trace clay, occasional cobbles and boulders Very Dense Grey Wet (TILL)		2	SS	50/ .100		
277.8 4.7	Gravelly **SAND**, some cobbles, trace silt, occasional boulders Very Dense Grey Wet		3	SS	50/ .150		
			4	SS	50/ .125		
			5	SS	50/ .075		0 93 7 (SI+CL)
			6	SS	50/ .100		



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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 06-6

2 OF 2

METRIC

G.W.P. 277-97-00
HWY 8
DATUM Geodetic
LOCATION Grand River Overpass SBL N 4 809 349.51 E 230 527.36
BOREHOLE TYPE Hollow Stem Augers / NQ Tri-Cone / NQ Core Barrel
DATE 2006-11-13 - 2006-11-14
ORIGINATED BY MEF
COMPILED BY JHL
CHECKED BY MEF

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page					272				
271.4 11.1	Highly to moderately weathered, thinly bedded, grey, medium strong to strong, **LIMESTONE BEDROCK**, with frequent rubble zones, occasional pitted zones	1	RUN			271			FI 9 >10 6	RUN 1# TCR=75%, SCR=69%, RQD=10%, UCS=47MPa
	Vertical joint from 12.55 to 12.65 m	2	RUN			270			>10 >10	RUN 2# TCR=77%, SCR=64%, RQD=0%, UCS=41MPa
	Vertical joints from 13.31 to 13.41, and 13.72 to 13.87 m	3	RUN			269			>10 >10 >10 7	RUN 3# TCR=100%, SCR=76%, RQD=7%, UCS=78MPa
	Vertical joints from 15.09 to 15.21, and 15.55 to 15.60 m	4	RUN			268 267			>10 >10 8 >10 >10	RUN 4# TCR=77%, SCR=71%, RQD=0%, UCS=MPa
	Vertical joint from 15.90 to 16.00 m	5	RUN						>10 >10	RUN 5# TCR=82%, SCR=65%, RQD=25%, UCS=44MPa
266.1 16.4	Becoming moderately weathered					266			>10 2	
265.4 17.1	END OF BOREHOLE AT 17.15 m. BOREHOLE GROUTED WITH BENTONITE TO RIVERBED SURFACE AT 1.85 m.									

Dynamic cone penetration resistance plot scale: 20 40 60 80 100

Shear strength kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; scale 20 40 60 80 100

PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L; water content scale 20 40 60

+³, ×³: Numbers refer to Sensitivity

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

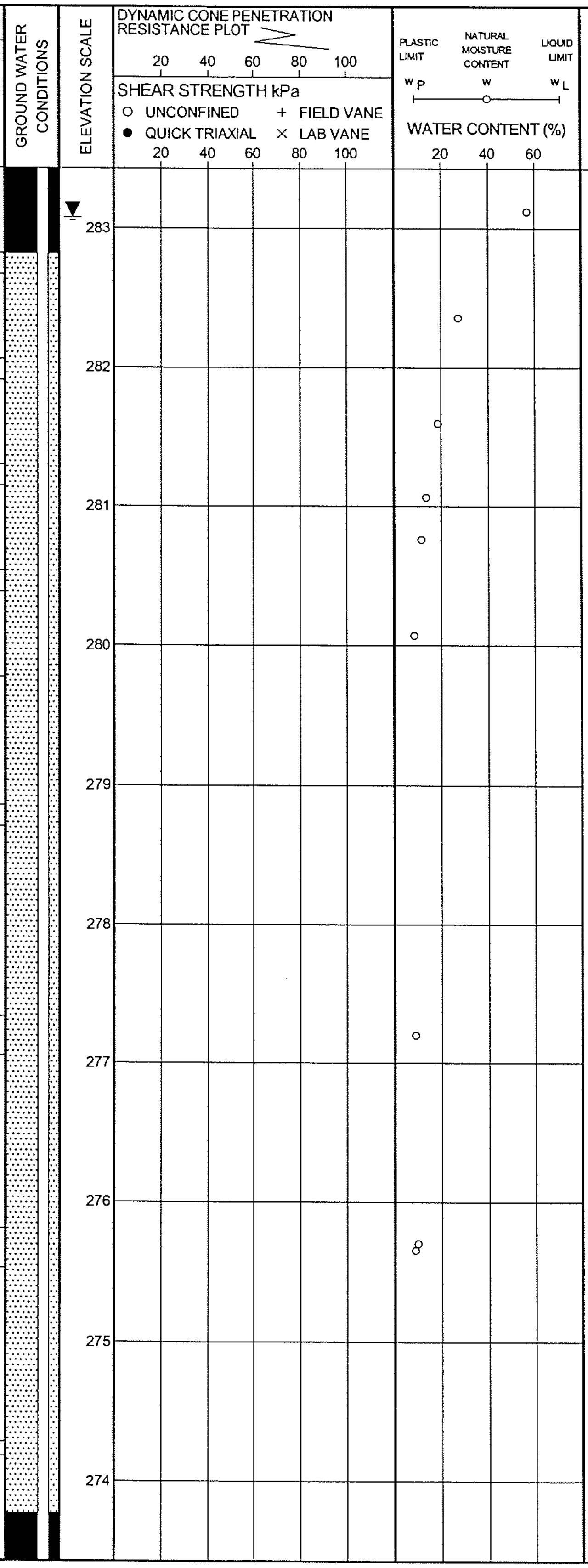
RECORD OF BOREHOLE No 06-7

1 OF 2

METRIC

G.W.P. 277-97-00
LOCATION Grand River Overpass SBL N 4 809 337.06 E 230 569.26
ORIGINATED BY SLL
HWY 8
BOREHOLE TYPE Hollow Stem Augers
COMPILED BY JHL
DATUM Geodetic
DATE 31.07.06 - 31.07.06
CHECKED BY MEF

SOIL PROFILE		SAMPLES			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
283.4								
0.0	**TOPSOIL**: (125 mm)							
0.1	**SAND**, mixed with topsoil, some silt Loose to Compact Dark brown Moist	1	SS	4				
		2	SS	16				
281.9 1.4	**SAND**, some silt, trace gravel, trace roots Loose Dark brown Moist to wet	3	SS	6				
281.1 2.2	**SAND and GRAVEL** Brown Wet							
280.9 2.4	**SILT and SAND**, some clay, trace gravel Dense to very dense Brown Moist (TILL)	4	SS	33				
		5	SS	92	5	39	44	12
	Occasional cobbles							
		6	SS	50/ .150				
	Occasional cobbles and boulders							
277.3 6.1	**SAND and GRAVEL**, trace to some silt, occasional cobbles Very dense Grey Wet	7	SS	50/ .125	42	43	15 (SI+CL)	
		8	SS	50/ .125				
		9	SS	50/ .100	38	59	3 (SI+CL)	



Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 06-7

2 OF 2

METRIC

W.P. 277-97-00
HWY 8
DATUM Geodetic
LOCATION Grand River Overpass SBL N 4 809 337.06 E 230 569.26
BOREHOLE TYPE Hollow Stem Augers
DATE 31.07.06 - 31.07.06
ORIGINATED BY SLL
COMPILED BY JHL
CHECKED BY MEF

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L WATER CONTENT (%) 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				
							273				
			10	SS	68/ .275				○		
							272				
271.3 12.0	Probable **BEDROCK or BOULDERS**		11	SS	50/ .050						
271.1 12.2	END OF BOREHOLE IN PROBABLE BEDROCK OR BOULDERS AT 12.24 m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.										

WATER LEVEL READINGS:

DATE	DEPTH(m)	ELEV.(m)
01.08.06	1.50	281.90
09.08.06	0.31	283.09
10.08.06	0.36	283.04
11.08.06	0.36	283.04
14.08.06	0.41	282.99
15.08.06	0.41	282.99
16.08.06	0.43	282.97
29.09.06	0.35	283.05

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

$\overset{20}{15 \oplus 5}$ 10 (%) STRAIN AT FAILURE

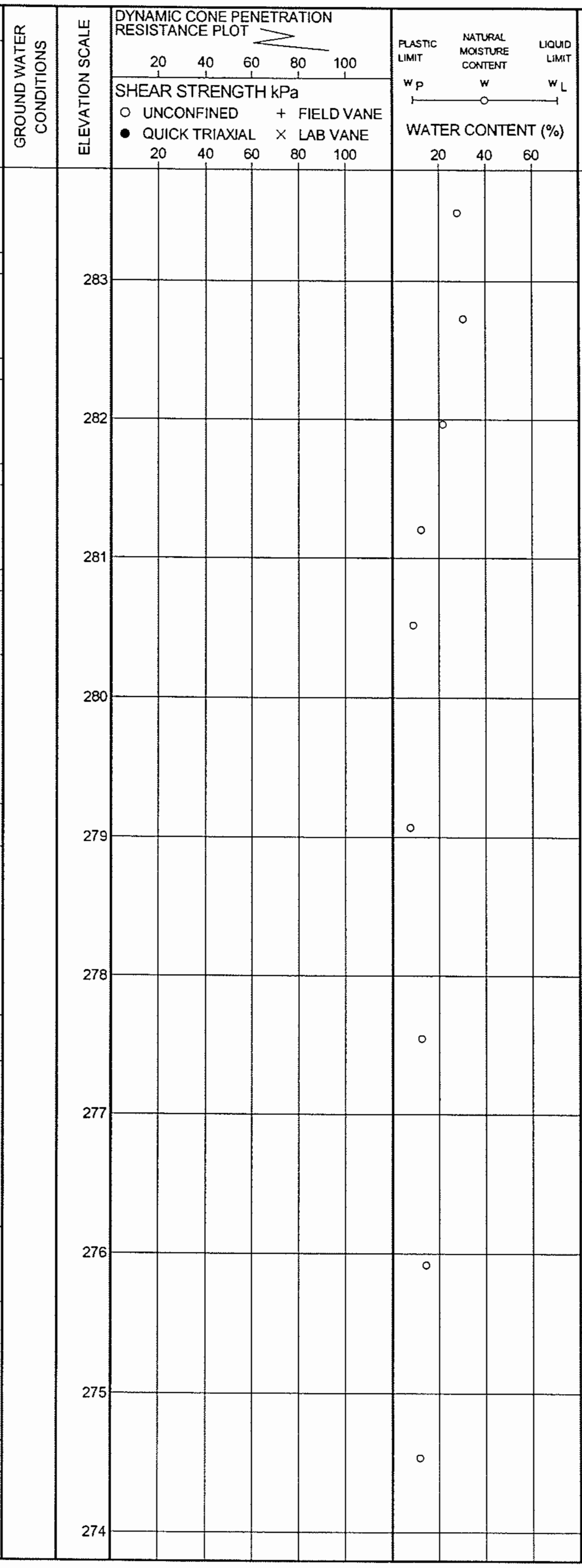
RECORD OF BOREHOLE No 06-8

1 OF 2

METRIC

G.W.P.	277-97-00	LOCATION	Grand River Overpass SBL N 4 809 325.66 E 230 562.08	ORIGINATED BY	MEF/SLL
HWY	8	BOREHOLE TYPE	Hollow Stem Augers / NQ Core Barrel	COMPILED BY	JHL
DATUM	Geodetic	DATE	27.07.06 - 28.07.06	CHECKED BY	MEF

SOIL PROFILE		SAMPLES			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
283.7 0.0	**TOPSOIL**: (150 mm)							
0.2	**SAND**, mixed with topsoil, some silt Loose Brown Dry	1	SS	6				
282.7 1.1	**SAND**, some silt, trace clay Loose to Compact Brown Moist	2	SS	4				
281.9 1.8	Sandy **GRAVEL**, some cobbles, trace silt Dense to Very Dense Brown Wet	3	SS	11				
		4	SS	38				
		5	SS	74	58	34	8 (SI+CL)	
280.2 3.5	Sandy **SILT**, some clay, trace gravel Very dense Grey Dry (TILL)	6	SS	108				
		7	SS	104/ .175	7	28	55	10
	Becoming brown	8	SS	52				
275.6 8.1	**SAND and GRAVEL**, some cobbles, trace silt Very dense Brown Wet	9	SS	100/ .225				



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+ 3, × 3: Numbers refer to Sensitivity

20 15 ⊕ 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 06-8

2 OF 2

METRIC

G.W.P. 277-97-00
LOCATION Grand River Overpass SBL N 4 809 325.66 E 230 562.08
ORIGINATED BY MEF/SLL
HWY 8
BOREHOLE TYPE Hollow Stem Augers / NQ Core Barrel
COMPILED BY JHL
DATUM Geodetic
DATE 27.07.06 - 28.07.06
CHECKED BY MEF

ELEV DEPTH	DESCRIPTION	SAMPLES NUMBER	SAMPLES TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
		10	SS	81	273	○		52 39 9 (SI+CL)
					272			
		11	SS	100/ .125		○		
					271			
270.3 13.4	Highly to moderately weathered, thinly bedded, grey, medium strong to strong, **LIMESTONE BEDROCK**, with occasional rubble zones, occasional pitted zones	1	RUN		270		FI >10	RUN 1# TCR=75%, SCR=58%, RQD=54%, UCS=49MPa
		2	RUN		269		9 6 8 9	RUN 2# TCR=100%, SCR=84%, RQD=21%
		3	RUN		268		>10 10 4 7 7	RUN 3# TCR=98%, SCR=95%, RQD=30%, UCS=105MPa
267.0 16.7	END OF BOREHOLE AT 16.71 m. BOREHOLE GROUTED WITH BENTONITE UPON COMPLETION.							

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100
PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

+ 3, × 3: Numbers refer to Sensitivity

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

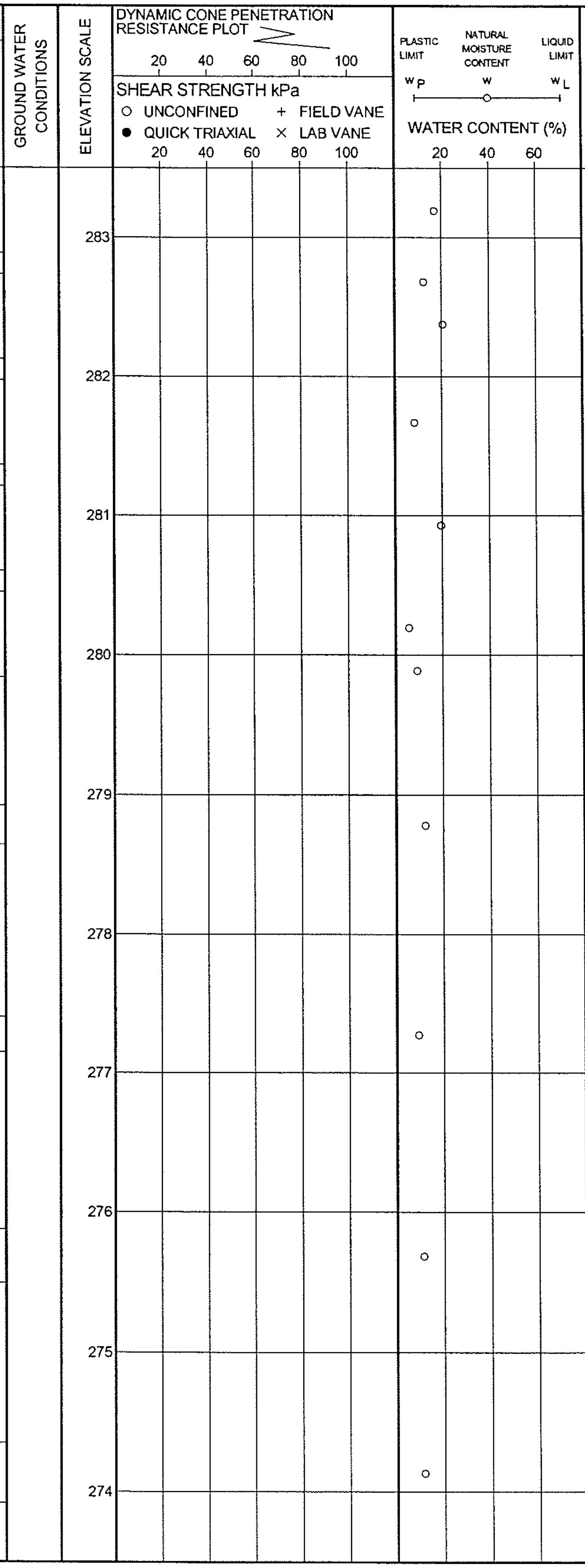
RECORD OF BOREHOLE No 06-9

1 OF 2

METRIC

G.W.P. 277-97-00
HWY 8
DATUM Geodetic
LOCATION Grand River Overpass SBL N 4 809 316.39 E 230 607.09
BOREHOLE TYPE Hollow Stem Augers / NQ Core Barrel
DATE 01.08.06 - 02.08.06
ORIGINATED BY SLL
COMPILED BY JHL
CHECKED BY MEF

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
283.4					
0.0	**TOPSOIL**: (125 mm)				
0.1	**SAND**, some silt, some topsoil, trace gravel Loose to Dense Dark brown Moist	1	SS	4	
		2	SS	33	
281.9 1.5	Sandy **GRAVEL**, trace silt, occasional cobbles Very Dense to Compact Brown Wet	3	SS	53	65 28 7 (SI+CL)
		4	SS	20	
		5	SS	58	
279.9 3.6	Sandy **SILT**, trace gravel Very dense Brown Moist (TILL)				
279.3 4.2	**SAND**, some gravel, trace silt, occasional cobbles Very dense Brown Wet	6	SS	50/ .125	
277.7 5.7	Sandy **SILT**, trace gravel, occasional cobbles Very dense Brown Moist (TILL)	7	SS	50/ .100	
276.1 7.3	**SAND and GRAVEL**, trace silt Very dense Brown Moist	8	SS	76/ .225	50 40 10 (SI+CL)
		9	SS	77/ .275	



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+ 3, × 3: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 06-9

2 OF 2

METRIC

G.W.P.	277-97-00	LOCATION	Grand River Overpass SBL N 4 809 316.39 E 230 607.09	ORIGINATED BY	SLL
HWY	8	BOREHOLE TYPE	Hollow Stem Augers / NQ Core Barrel	COMPILED BY	JHL
DATUM	Geodetic	DATE	01.08.06 - 02.08.06	CHECKED BY	MEF

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES				
		10	SS	88/ 275		273 272		
271.6 11.8	Highly to moderately weathered, thinly bedded, grey, medium strong to strong, **LIMESTONE BEDROCK**, with occasional rubble zones, occasional pitted zones	1	RUN				FI 8 >10	RUN 1# TCR=100%, SCR=38%, RQD=0%
		2	RUN			271 270	5 7 10	RUN 2# TCR=85%, SCR=78%, RQD=27%, UCS=70MPa
		3	RUN			269	2 10 >10	RUN 3# TCR=66%, SCR=21%, RQD=54%, UCS=46MPa
		4	RUN			268	10 5 5	RUN 4# TCR=73%, SCR=63%, RQD=42%, UCS=47MPa
266.9 16.5	END OF BOREHOLE AT 16.48 m. BOREHOLE GROUTED WITH BENTONITE UPON COMPLETION.							

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100

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

ONTMT4S 7938.GPJ 05/01/07

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

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

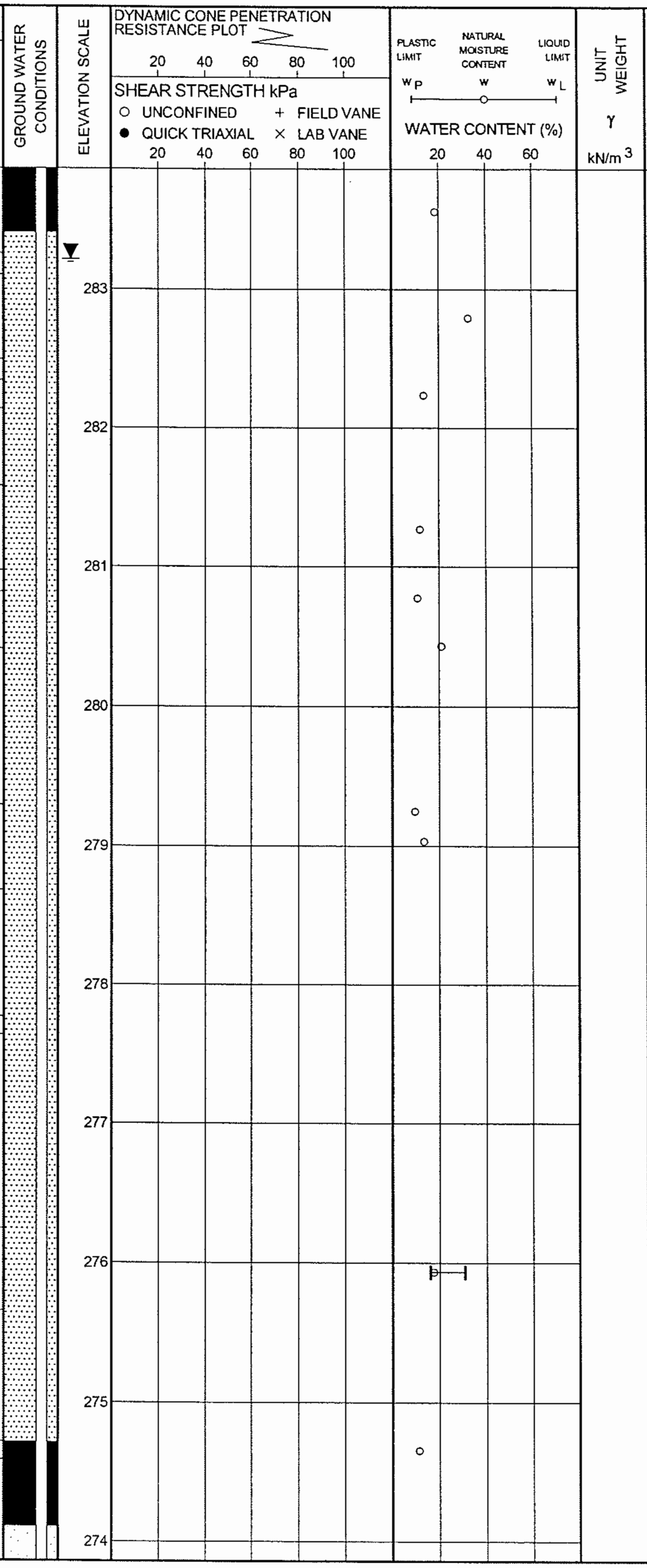
RECORD OF BOREHOLE No 06-10

1 OF 2

METRIC

G.W.P. 277-97-00	LOCATION Grand River Overpass SBL N 4 809 302.31 E 230 599.22	ORIGINATED BY SLL
HWY 8	BOREHOLE TYPE Hollow Stem Augers	COMPILED BY JHL
DATUM Geodetic	DATE 02.08.06 - 03.08.06	CHECKED BY MEF

SOIL PROFILE		SAMPLES			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
283.8								
0.0	TOPSOIL: (150 mm)							
0.2	**SAND**, mixed with topsoil, some silt, trace roots Loose Dark brown Moist	1 2 3	SS SS SS	6 5 50/.050				
282.1 1.7	Sandy **GRAVEL**, trace silt, occasional cobbles Very dense Brown Wet Sand seam at 3.00 to 3.13 m Becoming grey	4 5	SS SS	74 83				
279.1 4.6	Sandy **SILT**, trace gravel Very dense Grey Moist (TILL)	6 7	SS SS	87/.250 50/.125				
277.4 6.4	Silty **CLAY**, trace sand Hard Grey (TILL)(CL)	8	SS	60	0	3	61	36
275.4 8.4	Sandy **SILT**, trace clay, trace gravel, occasional cobbles Very dense Grey Moist to wet: (TILL)	9	SS	50/.125				
273.9 9.9								



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+3, ×3: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 06-10

2 OF 2

METRIC

W.P. 277-97-00
LOCATION Grand River Overpass SBL N 4 809 302.31 E 230 599.22
ORIGINATED BY SLL
HWY 8
BOREHOLE TYPE Hollow Stem Augers
COMPILED BY JHL
DATUM Geodetic
DATE 08.02.06 - 08.03.06
CHECKED BY MEF

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE: 20 40 60 80 100
PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L
WATER CONTENT (%): 20 40 60
UNIT WEIGHT γ kN/m^3

SOIL PROFILE ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES "N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
273.6	**BOULDER**: (300 mm)								
10.2	Gravelly **SAND**, trace silt Very dense Grey Wet	10	SS	50/ .075	273				
					272				
271.6		11	SS	88/ .175		27	69	4 (SI+CL)	
[illegible] 12.2	Probable **BEDROCK or BOULDERS** END OF BOREHOLE IN PROBABLE BEDROCK OR BOULDERS AT 12.21 m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.								

WATER LEVEL READINGS:

DATE	DEPTH(m)	ELEV.(m)
09.08.06	0.75	283.0
10.08.06	0.80	283.0
11.08.06	0.80	283.0
14.08.06	0.85	282.9
15.08.06	0.85	282.9
16.08.06	0.88	282.9
29.09.06	0.65	283.1

+ ^{3}, × ^{3}: Numbers refer to Sensitivity

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

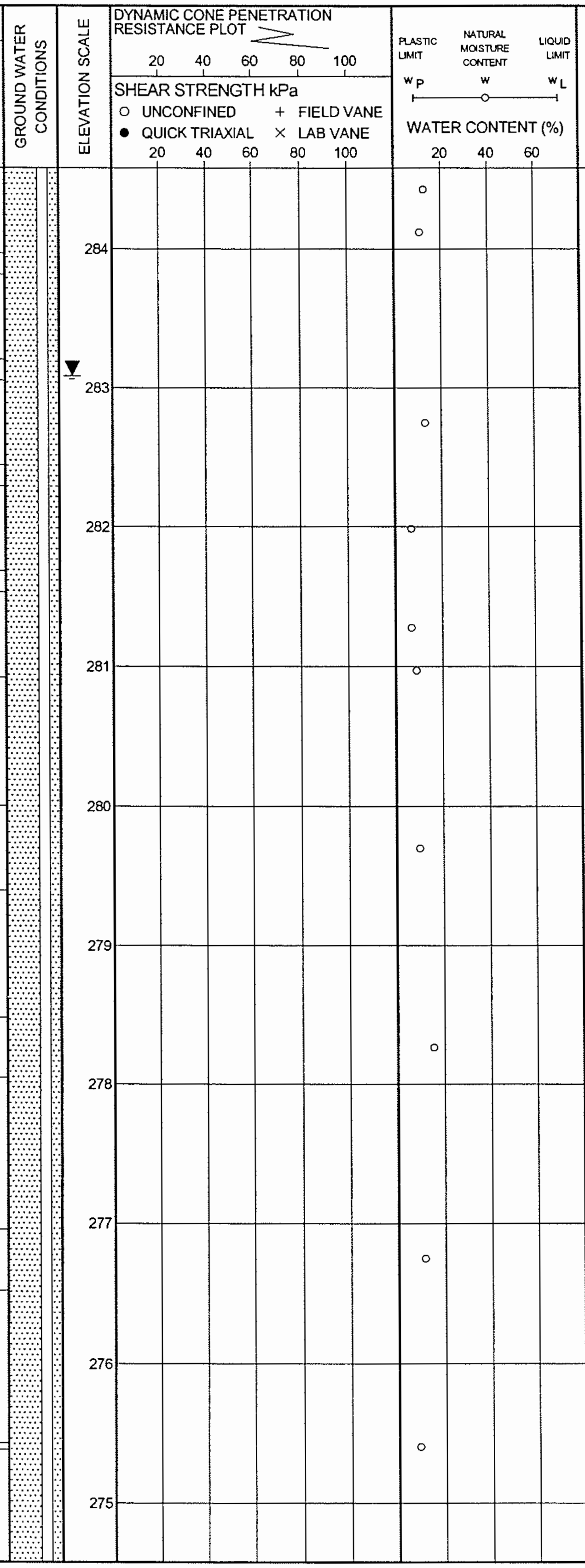
RECORD OF BOREHOLE No 06-11

1 OF 2

METRIC

G.W.P. 277-97-00	LOCATION Grand River Overpass SBL N 4 809 300.78 E 230 631.30		ORIGINATED BY SLL
HWY 8	BOREHOLE TYPE Hollow Stem Augers		COMPILED BY JHL
DATUM Geodetic	DATE 08.08.06 - 09.08.06		CHECKED BY MEF

SOIL PROFILE		SAMPLES			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
284.5								
0.0	**TOPSOIL:** (125 mm)							
0.1	**SAND**, some gravel, trace silt Loose Dark brown Moist (FILL)	1	SS	9				
284.0 0.5	**TOPSOIL:** (75 mm)							
283.9 0.6	**SAND and GRAVEL**, trace silt, occasional cobbles Compact to Dense Brown Moist	2	SS	14				
		3	SS	30				
		4	SS	45				
		5	SS	39	49	42	9 (SI+CL)	
280.9 3.6	**SILT and SAND**, some clay, trace gravel Very Dense Brown Moist (TILL)	6	SS	71	1	37	49	13
278.9 5.6	**SAND**, trace to some silt, some gravel Very dense Grey Moist to wet	7	SS	90/ .275				
		8	SS	88/ .275				
275.9 8.6	Silty **CLAY**, trace sand Hard Grey (TILL)	9	SS	50/ .050				
275.1 9.4	Gravelly **SAND**, trace silt Very Dense Grey							



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

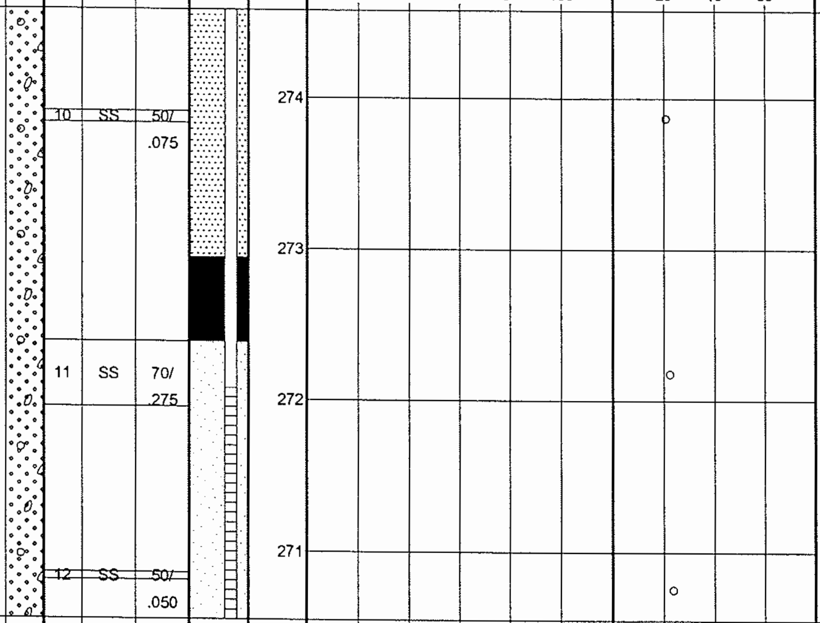
THURBER

RECORD OF BOREHOLE No 06-11

2 OF 2

METRIC

W.P. 277-97-00	LOCATION Grand River Overpass SBL N 4 809 300.78 E 230 631.30		ORIGINATED BY SLL
HWY 8	BOREHOLE TYPE Hollow Stem Augers		COMPILED BY JHL
DATUM Geodetic	DATE 08.08.06 - 08.09.06		CHECKED BY MEF



SOIL PROFILE ELEV DEPTH	DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	GR	SA	SI	CL
	Wet								
		10	SS	50/ .075	274				
					273				
		11	SS	70/ .275	272				
					271				
270.5		12	SS	50/ .050		0	93	2 (SI+CL)	
14.0	END OF BOREHOLE AT 14.02 m. AUGER REFUSAL ON PROBABLE BEDROCK OR BOULDERS. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.								

WATER LEVEL READINGS:

DATE	DEPTH(m)	ELEV.(m)
10.08.06	1.27	283.2
11.08.06	1.21	283.3
14.08.06	1.23	283.3
15.08.06	1.20	283.3
16.08.06	1.24	283.3
29.09.06	1.50	283.0

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB VANE
20 40 60 80 100

PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L

WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

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

STRAT PLOT GROUND WATER CONDITIONS

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

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

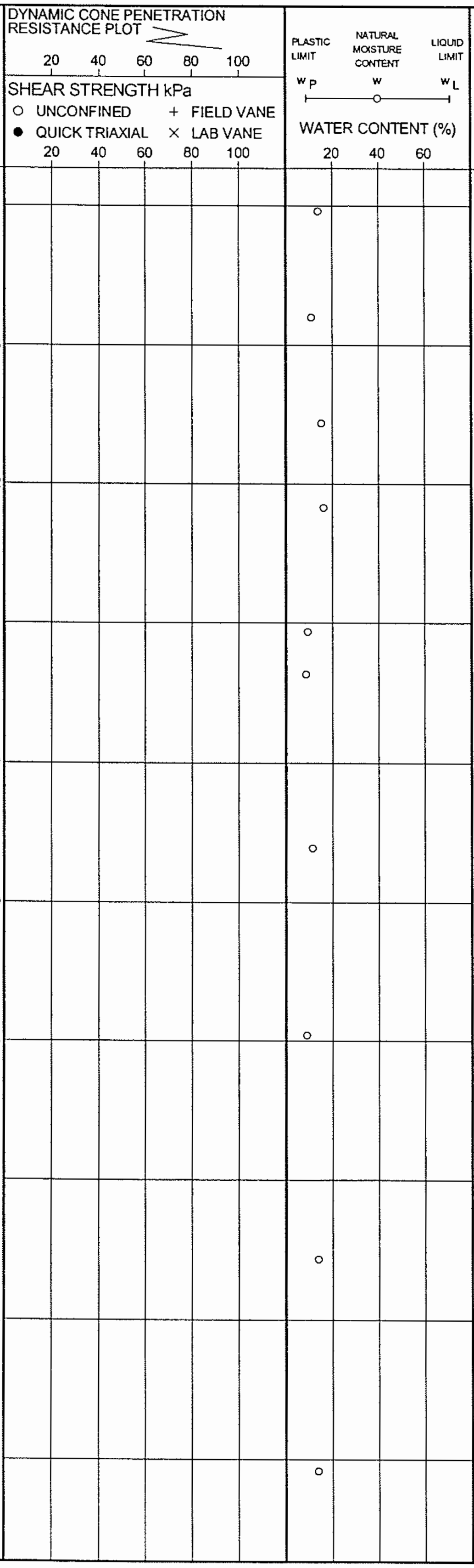
RECORD OF BOREHOLE No 06-12

1 OF 2

METRIC

G.W.P. 277-97-00
LOCATION Grand River Overpass SBL N 4 809 286.20 E 230 626.37
ORIGINATED BY SLL
HWY 8
BOREHOLE TYPE Hollow Stem Augers / NQ Core Barrel
COMPILED BY JHL
DATUM Geodetic
DATE 08.08.06 - 08.08.06
CHECKED BY MEF

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
284.2						
0.0	TOPSOIL: (100 mm)					
0.1	SAND,some silt, trace clay, trace gravel, topsoil stained, trace roots Compact Dark brown Moist (FILL)	1	SS	13	284	
283.5 0.7	**SAND AND GRAVEL**, some silt Compact to Very Dense Brown Moist	2	SS	21	283	
	Occasional cobbles, wet	3	SS	61	282	
		4	SS	50/ .125		
		5	SS	40	281	47 41 12 (SI+CL)
280.6 3.6	Sandy SILT, some clay, trace gravel, occasional cobbles Very Dense Grey Moist (TILL)				280	
		6	SS	74	279	
		7	SS	50/ .100	278	3 32 54 11
					277	
		8	SS	50/ .125	276	
		9	SS	50/ .100	275	



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

THURBER

RECORD OF BOREHOLE No 06-12

2 OF 2

METRIC

G.W.P. 277-97-00
LOCATION Grand River Overpass SBL N 4 809 286.20 E 230 626.37
ORIGINATED BY SLL
HWY 8
BOREHOLE TYPE Hollow Stem Augers / NQ Core Barrel
COMPILED BY JHL
DATUM Geodetic
DATE 08.08.06 - 08.08.06
CHECKED BY MEF

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
274.1 10.1	Silty **CLAY**, trace sand, trace gravel Very stiff Grey (TILL)(CL)					274		
		10	SS	24				1 4 62 33
						273		
272.6 11.6	Gravelly **SAND**, silty, trace clay, occasional cobbles Very dense Grey Wet							
		11	SS	50/ .125		272		
						271		
		12	SS	50/ .125				
						270		
269.6 14.6	Highly to moderately weathered, thinly bedded, grey, strong to very strong, **LIMESTONE BEDROCK**, with occasional rubble zones, occasional pitted zones						FI	
		1	RUN				10	RUN 1# TCR=100%, SCR=43%, RQD=0%
		2	RUN				5	
						269	>10	
		3	RUN				7 7	RUN 2# TCR=92%, SCR=75%, RQD=50%, UCS=73MPa
						268		RUN 3# TCR=60%, SCR=44%, RQD=12%, UCS=86MPa
							>10	
		4	RUN			267	3 6	RUN 4# TCR=55%, SCR=42%, RQD=23%, UCS=155MPa
266.1 18.1	END OF BOREHOLE AT 18.12 m. BOREHOLE GROUTED WITH BENTONITE UPON COMPLETION.							

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100
PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L
WATER CONTENT (%): 20 40 60

+ ³, × ³: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

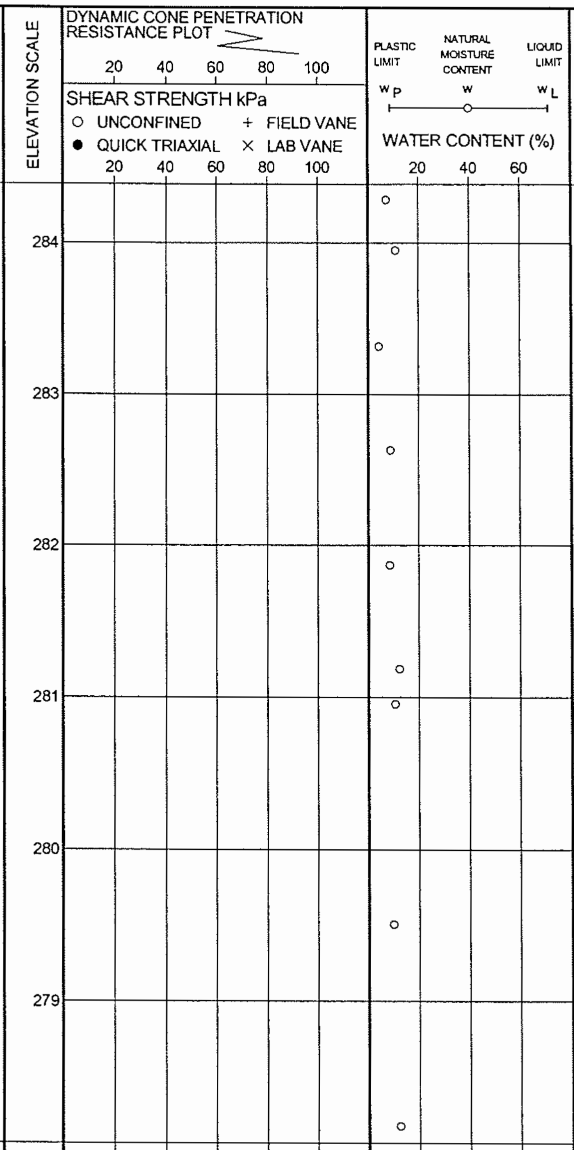
THURBER

RECORD OF BOREHOLE No 06-13

1 OF 1 **METRIC**

G.W.P. 277-97-00	LOCATION Grand River Overpass SBL N 4 809 280.86 E 230 645.36	ORIGINATED BY SLL
HWY 8	BOREHOLE TYPE Hollow Stem Augers	COMPILED BY JHL
DATUM Geodetic	DATE 11.08.06 - 11.08.06	CHECKED BY MEF

SOIL PROFILE ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES "N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
284.3							
0.0 0.0	**TOPSOIL**: (50 mm)						
0.1	**GRAVEL**: (FILL)						
0.2	**TOPSOIL**: (75 mm)						
	SAND and GRAVEL, trace silt Compact to Very Dense Brown Moist	1	SS	25			
		2	SS	34			
		3	SS	100/ .275			
		4	SS	61			38 55 7 (SI+CL)
281.0 3.4	**SILT and SAND**, some clay, trace gravel, occasional cobbles Dense to Very Dense Grey Moist (TILL)	5	SS	30			
		6	SS	36			5 40 44 11
278.0 6.3	END OF BOREHOLE AT 6.33 m. BOREHOLE GROUTED WITH BENTONITE TO SURFACE.	7	SS	50/ .075			



+ 3, × 3: Numbers refer to Sensitivity

15 20 5 10 (%) STRAIN AT FAILURE

Appendix B

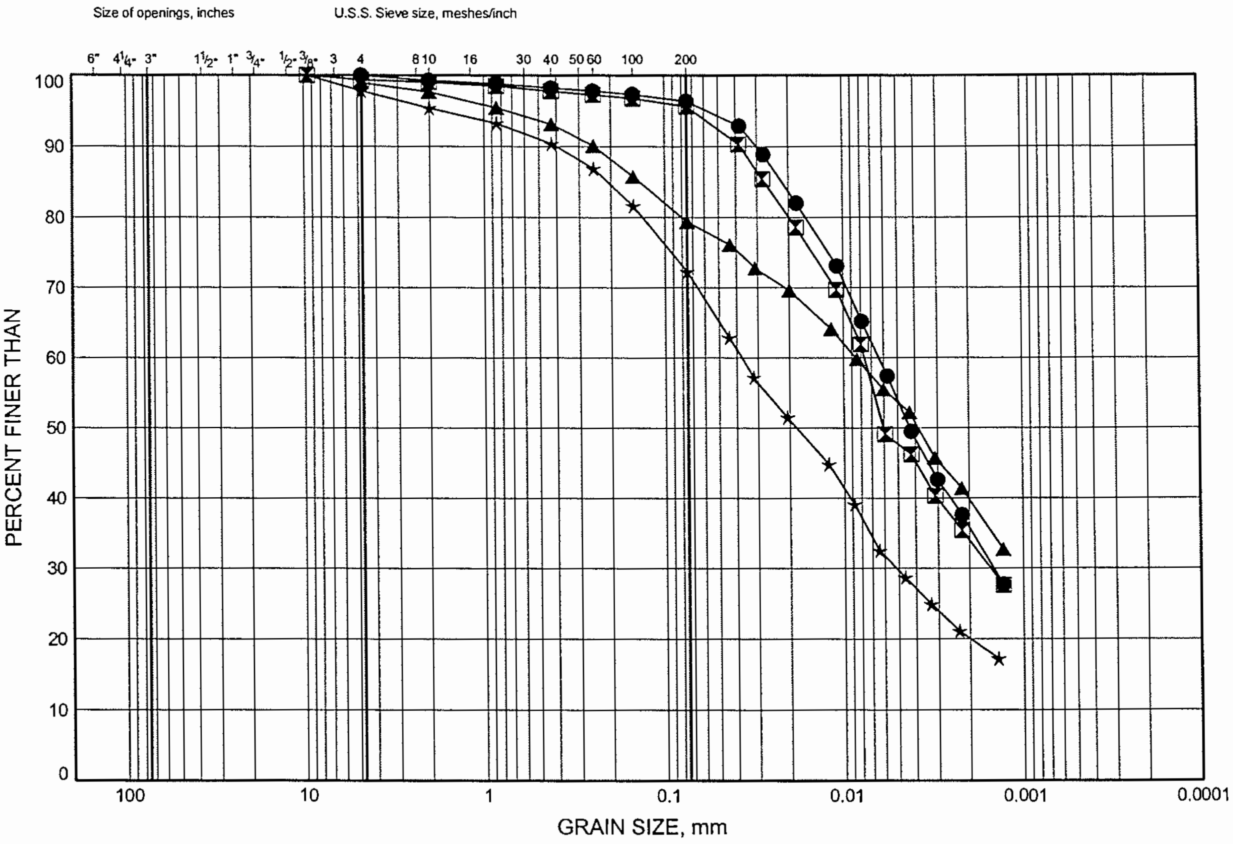
Laboratory Test Results

Highway 8 Widening Over Grand River

GRAIN SIZE DISTRIBUTION

FIGURE B1

SILTY CLAY TO CLAYEY SILT TILL



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	06-10	7.92	275.86
☒	06-12	10.97	273.22
▲	06-2	9.30	285.53
★	06-2	12.50	282.33

Date January 2007

Project 277-97-00

THURBER

Prep'd JHL

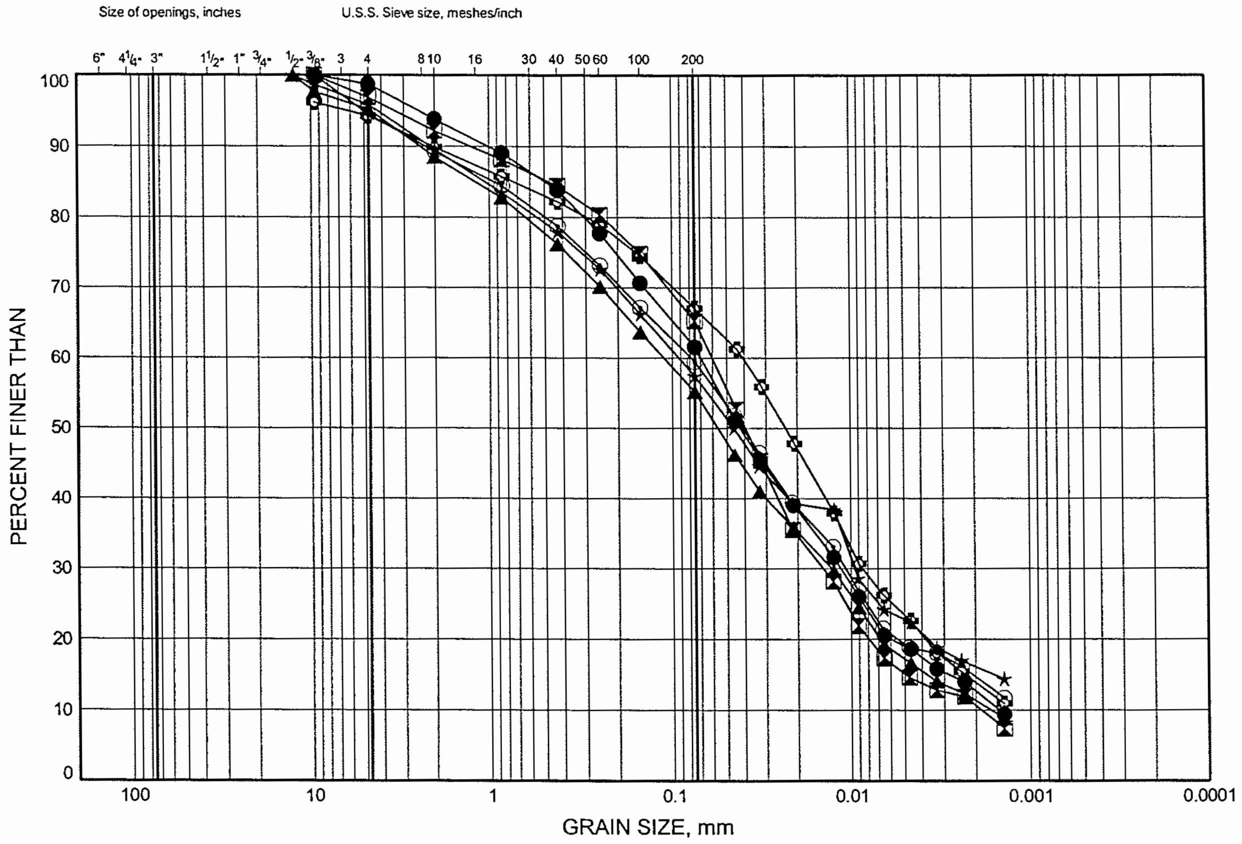
Chkd. MEF

Highway 8 Widening Over Grand River

GRAIN SIZE DISTRIBUTION

FIGURE B2

SANDY SILT TO SILT AND SAND TILL



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	06-11	4.88	279.63
⊠	06-12	6.22	277.97
▲	06-13	4.88	279.43
★	06-2	1.83	293.00
⊙	06-2	17.07	277.76
✪	06-2	18.34	276.49

Date January 2007

Project 277-97-00

Prep'd JHL

Chkd. MEF

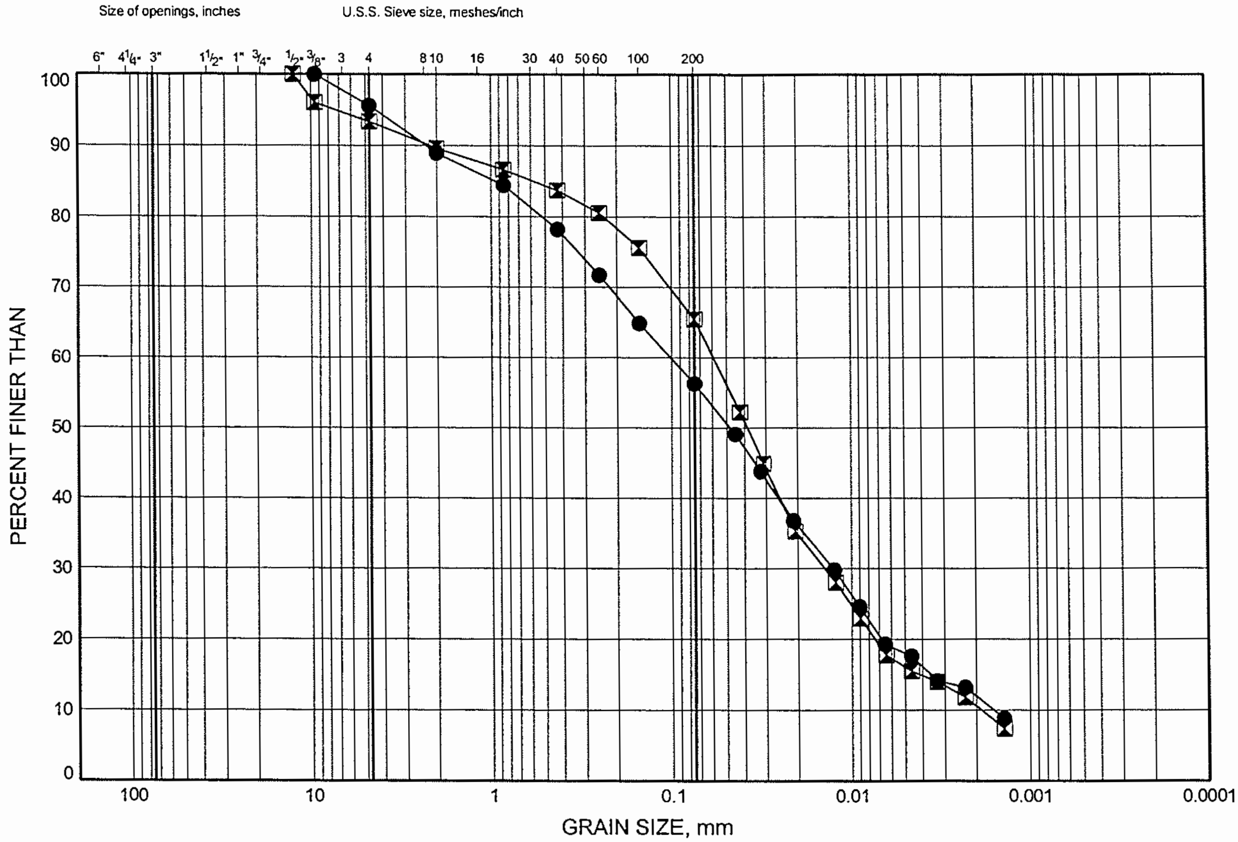
THURBGSD 7938.GPJ 05/01/07

Highway 8 Widening Over Grand River

GRAIN SIZE DISTRIBUTION

FIGURE B3

SANDY SILT TO SILT AND SAND TILL



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	06-7	3.35	280.00
☒	06-8	6.25	277.47

THURBGSD 7938.GPJ 05/01/07

Date January 2007

Project 277-97-00

THURBER

Prep'd JHL

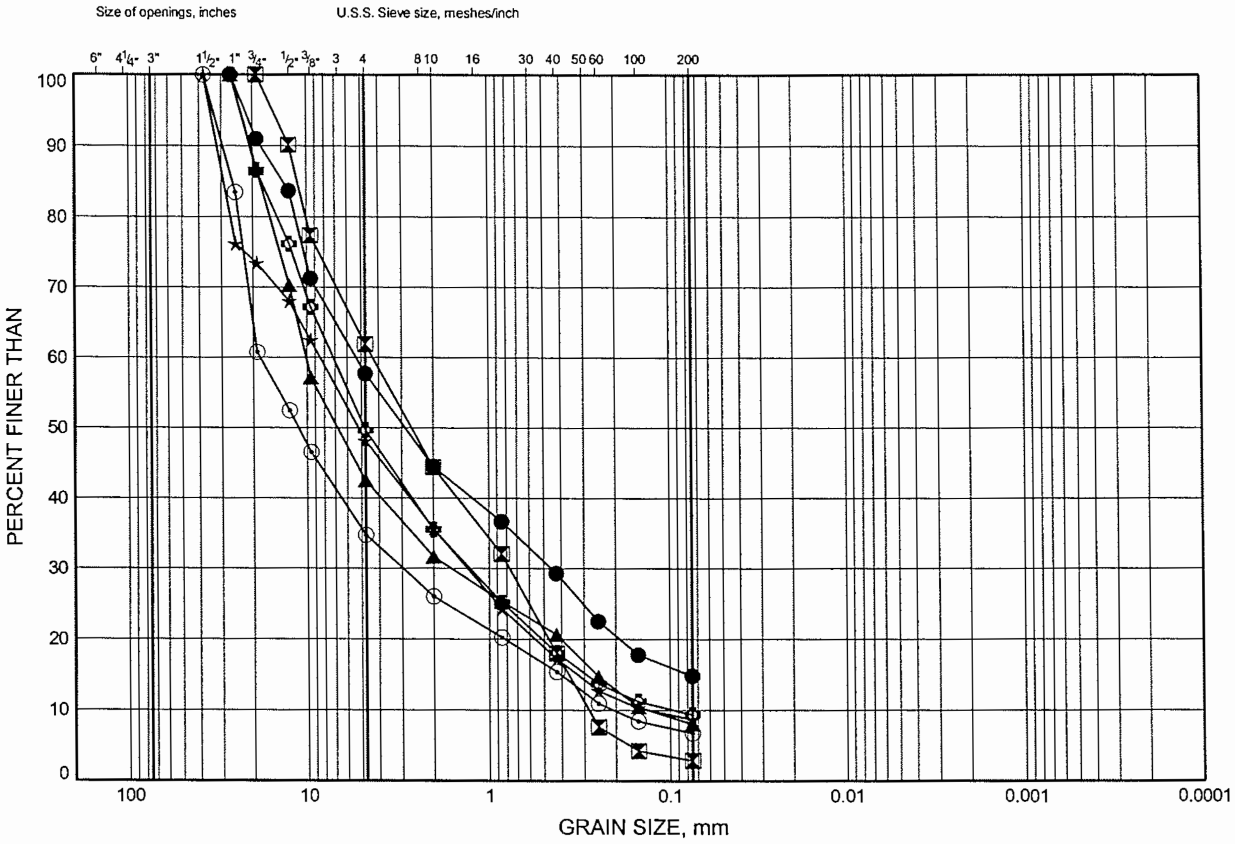
Chkd. MEF

Highway 8 Widening Over Grand River

GRAIN SIZE DISTRIBUTION

FIGURE B4

SANDY GRAVEL TO GRAVELLY SAND



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	06-7	6.22	277.13
⊠	06-7	9.19	274.16
▲	06-8	3.28	280.45
★	06-8	10.90	272.83
⊙	06-9	1.83	281.59
⊕	06-9	7.81	275.61

THURBGSD 7938.GPJ 05/01/07

Date January 2007

Project 277-97-00

THURBER

Prep'd JHL

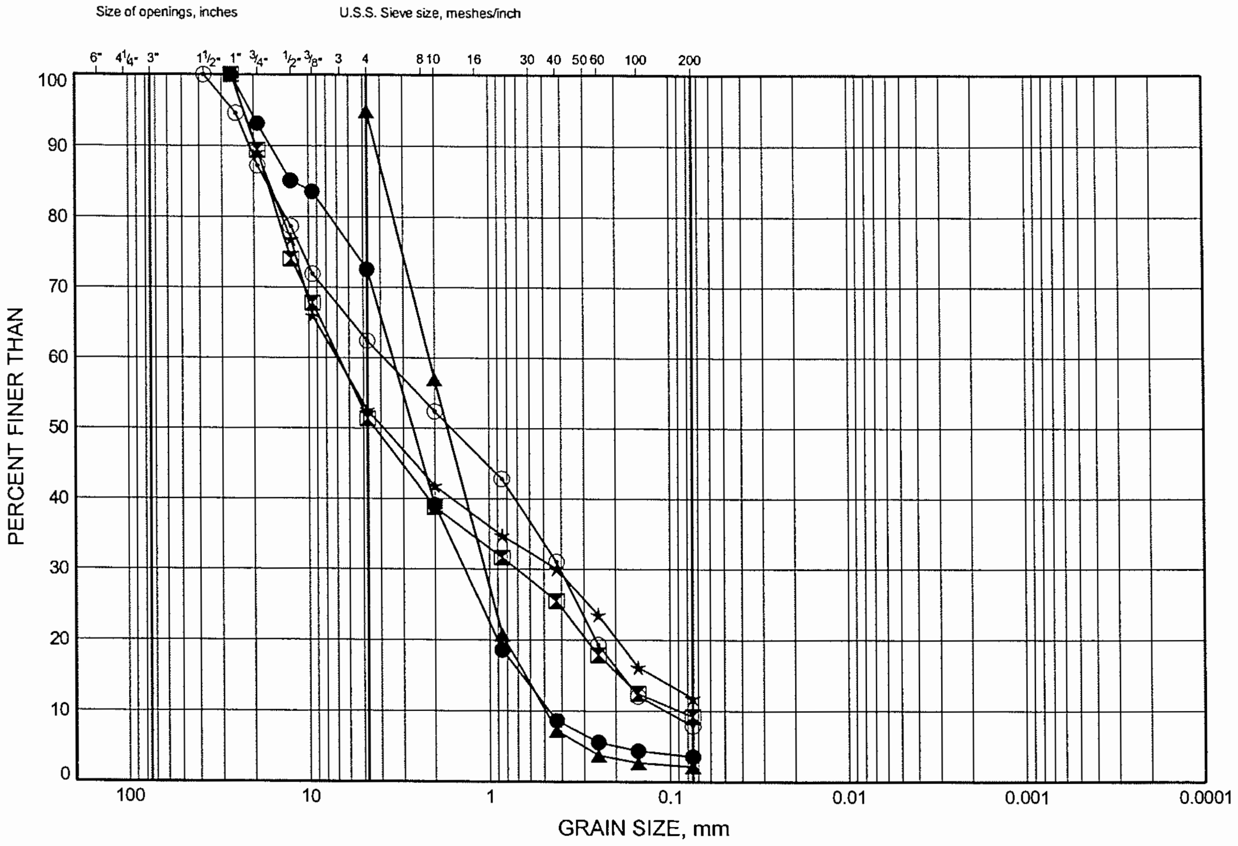
Chkd. MEF

Highway 8 Widening Over Grand River

GRAIN SIZE DISTRIBUTION

FIGURE B5

SANDY GRAVEL TO GRAVELLY SAND



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	06-10	12.04	271.75
⊠	06-11	3.35	281.15
▲	06-11	13.82	270.69
★	06-12	3.18	281.02
⊙	06-13	2.51	281.80

Date January 2007
Project 277-97-00

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Prep'd JHL
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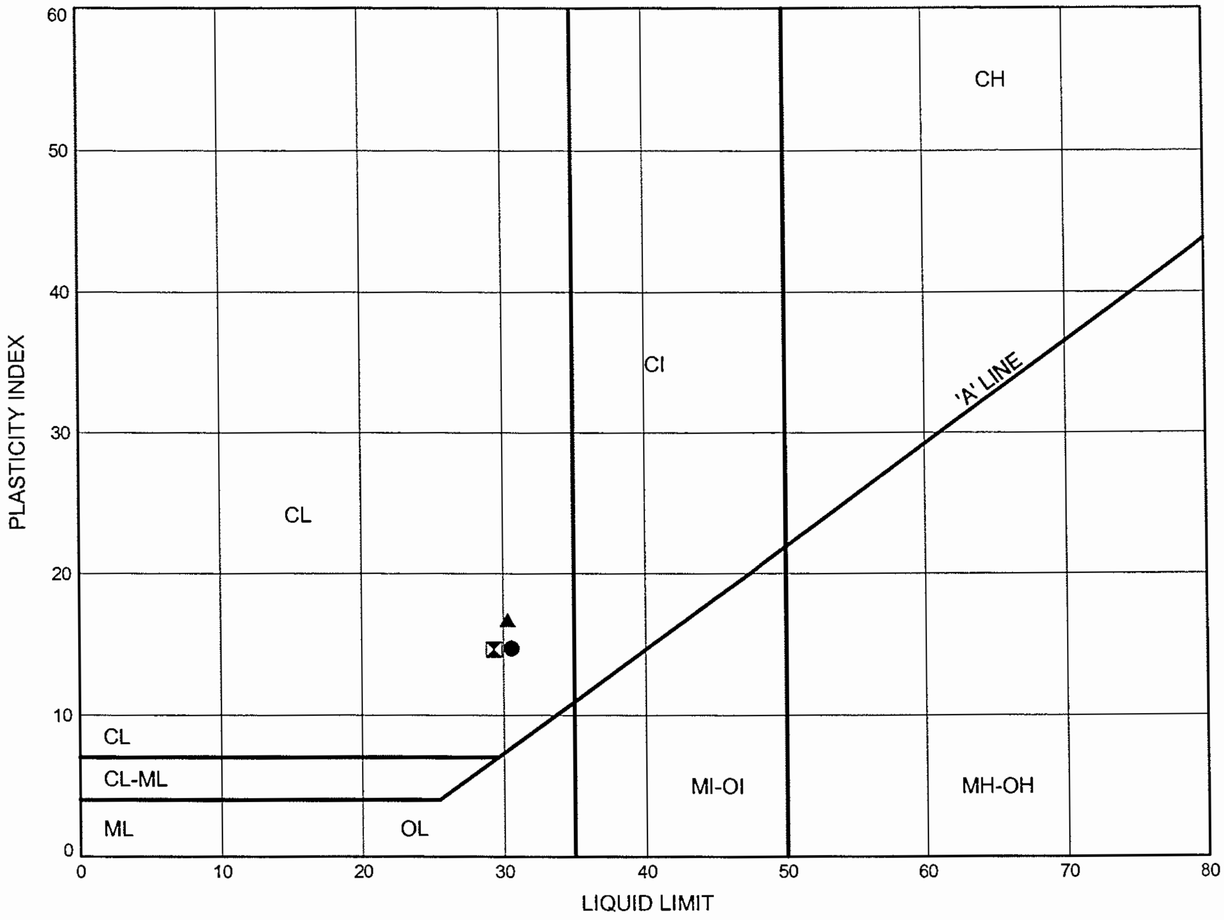
Highway 8 Widening Over Grand River

ATTERBERG LIMITS TEST RESULTS

FIGURE B6

SILTY CLAY TILL



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	06-10	7.92	275.86
☒	06-12	10.97	273.22
▲	06-2	9.30	285.53

THURBALT 7938.GPJ 05/01/07

Date January 2007

Project 277-97-00

THURBER

Prep'd JHL

Chkd. MEF

TABLE B1 - Point Load Test Results
Highway 8 Widening over Grand River

Depth				UCS
feet	Inches	m	Is50	(MPa)
06-5				
35	8	10.87	0.43	10.37
36	7	11.15	4.75	114.05
38	1	11.61	3.02	72.57
39	9	12.12	3.43	82.29
41	0	12.50	4.97	119.23
42	11	13.08	2.16	51.84

Total Rock Core

Average	Minimum	Maximum	
75	10	119	MPa

Run #	Average
1	69.82
2	85.53

Depth				UCS
feet	Inches	m	Is50	(MPa)
06-6				
36	11	11.25	0.97	23.33
38	6	11.73	2.92	69.98
41	7	12.67	1.73	41.47
43	3	13.18	3.24	77.76
53	4	16.26	1.68	40.43
54	4	16.56	1.94	46.66

Total Rock Core

Average	Minimum	Maximum	
50	23	78	MPa

Run #	Average
1	46.66
2	41.47
3	77.76
5	43.54

Depth				UCS
feet	Inches	m	Is50	(MPa)
06-8				
45	5	13.84	4.78	114.81
47	3	14.40	0.87	20.87
48	8	14.83	0.22	5.22
49	0.5	14.95	2.25	54.04
50	6	15.39	1.74	41.75
50	11	15.52	1.52	36.53
50	11	15.52	4.61	110.68
51	9	15.77	6.52	156.55
52	10	16.10	7.44	178.58

Total Rock Core

Average	Minimum	Maximum	
80	5	179	MPa

Run #	Average
1	48.73
3	104.82

Depth				UCS
feet	Inches	m	Is50	(MPa)
06-9				
39	7.5	12.08	6.09	146.12
41	6	12.65	2.61	62.62
42	11	13.08	2.61	62.62
42	11	13.08	1.99	47.74
43	1	13.13	1.33	31.92
45	1	13.74	2.72	65.23
46	9	14.25	1.09	26.09
50	7	15.42	1.52	36.53
52	2	15.90	2.39	57.40

Total Rock Core

Average	Minimum	Maximum	
60	26	146	MPa

Run #	Average
2	70.20
3	45.66
4	46.97

Depth				UCS
feet	Inches	m	Is50	(MPa)
06-12				
48	10	14.88	3.03	72.80
50	9	15.47	1.300	31.20
52	9	16.08	5.136	123.26
53	9	16.38	4.334	104.01
55	2	16.81	6.661	159.86
56	3	17.15	5.634	135.21
56	3	17.15	7.029	168.70

Total Rock Core

Average	Minimum	Maximum	
114	31	169	MPa

Run #	Average
2	72.80
3	86.16
4	154.59

Appendix C

Factual Information from Previous Investigation for Existing Structure

e. m. peto associates, ltd.

SOIL ENGINEERING SERVICE – TORONTO, ONTARIO

BOREHOLE LOG

Job Name Proposed Hwy.#8 Crossing Grand River
Job No. 58119
Borehole No. 2
Client Dept. of Highways of Ontario
Casing BX
Boring Date Oct. 14th – 16th, 1958
Datum D.H.O.
Compiled By C.J.W.
Checked By C.F.P.

SAMPLE CONDITION	SAMPLE TYPE	ABBREVIATIONS
▨ UNDISTURBED	S.S. 2" STANDARD SPLIT TUBE SAMPLE	V.T. IN SITU VANE SHEAR TEST
▧ FAIR	S.L. SPLIT BARREL WITH LINERS	Q/u UNCONFINED COMPRESSIVE STRENGTH
☒ DISTURBED	S.T. THIN-WALLED SHELBY TUBE SAMPLE	W.L. WATER LEVEL IN CASING
■ LOST	W.S. WASH SAMPLE	W.T. GROUND WATER TABLE IN SOIL
	R.C. ROCK CORE	

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth / Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft	WATER LEVELS, SOIL MOISTURE & REMARKS
			0'-0"					
TOPSOIL 0'-0 TO 2"			925.19					
								DUE TO STONE INTERFERENCE
CRUSHED LIMESTONE AND	GREY	VERY DENSE	WL 2'-7" ▵922.6		1 ☒	▵S.S.	74	MOIST NAT. M.C. 9.3%
GRAVEL, MATRIX OF SANDY								W.L. AT 2'-7" ON 15 OCT. 58.
TILL			6'-0"					
SANDY TILL WITH LIMESTONE	GREY	VERY DENSE			2 ☒	S.S	145	MOIST NAT. M.C. 6.9%
FRAGMENTS								
SANDY TILL, GRITS AND	GREY				3 ▧	S.S.	68	MOIST NAT M.C 8.7%
PEBBLES								
			10'-0"					
AS ABOVE WITH GRAVEL	GREY	VERY DENSE			4 ☒	S.S	48	MOIST NAT. M.C 8.5%
								STIFFENS AT 12FT
			14'-0"					DRILLED FROM 14'-0" TO 27'
			911.19					HARD GOING FROM 14'0"
MED COARSE SAND & GRAVEL	OLIVE	VERY DENSE			5 ☒	S.S.	135/10"	WET
N MATRIX OF SANDY CLAY	YELLOW		16'-6" 908.69					
			20'-0"					
CLAYEY SILT, GRITS, PEBBLES	BROWN	VERY DENSE			6 ☒	S.S.	123	WETTER THAN PLASTIC LIMIT
AND GRAVEL								
COARSE TO MED SAND	BROWN	VERY DENSE	25'-0"		7	W.S.	200/4½	
WITH FINE GRAVEL								
			27'-0"					DIA DRILLED FROM 27' TO 37'
								FIRST RUN 27' TO 32' RECOVER
								10" THIS RUN CONSIDERED TO
AS ABOVE WITH BOULDERS								BE BOULDERS.
						BX		SECOND RUN 32' TO 37' RECOVER
			32'-0"			CORE		77%
			893.19					
BANDED LIMESTONE	L. GREY	MED TO SOFT						
		HARDNESS						
		APPROX. 4.	37'-0"					
			888.19					
			HOLE TERMINATED					WATER LEVEL SHOWN AT 2'-(WITH
								DEPTH OF HOLE 22'-3") BELOW
								THIS DEPTH WITH HOLE AT 37'
								ARTESIAN EFFECT NOTED AND
								WATER LEVEL ROSE TO
								ELEVATION 926.23

e. m. peto associates ltd.

SOIL ENGINEERING SERVICE – TORONTO, ONTARIO

BOREHOLE LOG

Job Name Proposed Hwy. #8 Crossing Grand River

Job No. 58119

Borehole No. 3

Client Dept. of Highways of Ontario

Casing BX

Boring Date Oct. 18th–20th, 1958

Datum D.H.O.

Compiled By C.J.W.

Checked By C.F.F.

SAMPLE CONDITION	SAMPLE TYPE	ABBREVIATIONS
▨ UNDISTURBED	S.S. 2" STANDARD SPLIT TUBE SAMPLE	V.T. IN SITU VANE SHEAR TEST
▧ FAIR	S.L. SPLIT BARREL WITH LINERS	Q/u UNCONFINED COMPRESSIVE STRENGTH
☒ DISTURBED	S.T. THIN-WALLED SHELBY TUBE SAMPLE	W.L. WATER LEVEL IN CASING
■ LOST	W.S. WASH SAMPLE	W.T. GROUND WATER TABLE IN SOIL
	R.C. ROCK CORE	

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft.	WATER LEVELS, SOIL MOISTURE & REMARKS
			0'-0"	RIVER WATER LEVEL 18 OCT 1958				
			922.26					
WATER								
SANDY TILL AND GRAVEL	GREY	DENSE	5-0"		1 ☒	S.S.	39	WET NAT. M.C. 8.4%
SANDY TILL GRITS & PEBBLES	GREY	VERY DENSE			2 ▧	S.S.	83	MOIST 7.8%
			10'-0"		3 ■	S.S	100/8"	PROBABLY A LAYER OF BOULDERS
		VERY DENSE	11'-2"					& STONES
			911.10					FROM 11'-2" HARD GOING, HOLE
								DRILLED FROM THIS POINT DOWN
			15'-0"					
CLAYEY COARSE SAND AND GRAVEL UP TO 1½" DIA.	GREY	VERY DENSE			4 ☒	S.S.	130/5"	SATURATED
			20-0"					
AS ABOVE	GREY	VERY DENSE			5 ☒	S.S	100/8"	WET
			25-0"					
AS ABOVE	GREY	VERY DENSE			6 ☒	S.S.	125	VERY WET.
					7 ☒			DIA. DRILLED FROM 26-2 TO
								27'-0" BOULDER. 27 TO 28-9
			28'-9"					DRILLED THR'O CLAYEY GRAVEL
BANDED LIMESTONE	LT GREY	MED. TO	893.51 30-0					
		SOFT						
		HARDNESS				B.X.		DIA DRILLED BX CORE FROM
		APPROX. 4.				CORE		28'-9" TO 33-4, RECOVERY 58%
								AND FROM 33'-4 TO 38'-4
			35-0"					RECOVERY 83%
								ARTESIAN EFFECT NOTED AS
								FOLLOWS FROM 20'-8" DEPTH
			38'-4" 883.93					WATER ROSE TO 923.44 WITH
			HOLE TERMINATED					HOLE AT 38'-4" WATER ROSE
								TO 927.85

e. m. peto associates ltd.

SOIL ENGINEERING SERVICE - TORONTO, ONTARIO

BOREHOLE LOG

Job Name Proposed Hwy. #8 Crossing Grand River   Job No. 58119   Borehole No. 5

Client Dept. of Highways of Ontario   Casing BX   Boring Date Sept. 30th - Oct. 1st, 195

Datum D.H.O.   Compiled By C.J.W.   Checked By C.F.F.

SAMPLE CONDITION

- ▨ UNDISTURBED
- ▧ FAIR
- ☒ DISTURBED
- ■ LOST

SAMPLE TYPE

S.S.	2" STANDARD SPLIT TUBE SAMPLE
S.L.	SPLIT BARREL WITH LINERS
S.T.	THIN-WALLED SHELBY TUBE SAMPLE
W.S.	WASH SAMPLE
R.C.	ROCK CORE

ABBREVIATIONS

V.T.	IN SITU VANE SHEAR TEST
Q/u	UNCONFINED COMPRESSIVE STRENGTH
W.L.	WATER LEVEL IN CASING
W.T.	GROUND WATER TABLE IN SOIL

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft	WATER LEVELS, SOIL MOISTURE & REMARKS
TOP SOIL			0 - 0					
SANDY LOAM WITH GRITS	DK BROWN	LOOSE	931.04		1 ☒	S.S.	6	SLIGHTLY MOIST UP TO 2" φ
STONES AND ORGANIC MATTER.								
SILT, WITH DECOMPOSED	YELLOWISH-	LOOSE			2 ☒	S.S.	6	SLIGHTY MOIST NAT M.C. 9.6 %
SANDSTONE AND FRAGMENTS	BROWN							
OF SCATTERED GRAVEL			5' - 0"					
S[illegible]Y SILT, SOME GRITS	YELLOWISH	LOOSE			3 ☒	SS.	5	MOIST. NAT M.C. 22.4 %
	BROWN							CHANGE OCCURS AT 7'-8"
SILT CHANGING TO MED. SAND	LT. YELLOWISH	LOOSE	WL. 8'-1"		4 ☒	S.S.	3	WET. NAT. M.C. 22.4 %
AND GRITS	BROWN TO		922.96 △			△		STONE CONT. INCREASES AT 9'-2"
	V PALE BROWN		10' - 0"					STONE INTERFERENCE
SILTY COARSE SAND, GRITS		DENSE			5 ☒	S.S.	42	SATURATED M.C. 8.8 %
& PEBBLES.			12' - 0"					WATER IN CASING AT 12'-0"
SANDY TILL WITH GRITS	LT. YELLOWISH	VERY DENSE	919.04		6 ☒	S.S.	109	MOIST. NAT M.C. 8.2 %
AND PEBBLES	BROWN							
			15' - 0"					
SANDY TILL WITH GRITS	GREY	VERY DENSE			7 ☒	SS.	113	MOIST NAT. M.C. 7.5 %.
AND PEBBLES								
								WASH WATER USED FROM 20'-0"
			20' - 0"					
VERY SILTY COARSE SAND	LT YELLOWISH		911.04		8	S.S.	200/4	SATURATED
FINE GRAVEL GRITS AND	BROWN							BOULDERS FROM 20' TO 22'-0"
PEBBLES								
			25' - 0"					
COARSE SAND, FINE GRAVEL	YELLOWISH	VERY DENSE			9 ☒	S.S.	75	SATURATED
GRITS & PEBBLES IN MATRIX	BROWN							
OF SANDY TILL								
			30' - 0"					
MATRIX OF SANDY TILL	GREY	VER DENSE			10 ☒	SS	169	WET
WITH GRITS & PEBBLES	MOTTLED							STONE IN SAMPLER TIP
	BROWN							
			35' - 0"					
MATRIX OF SANDY TILL	LT YELLOWISH	VERY DENSE			11 ☒	SS	61	WET
WITH GRITS & PEBBLES	MOTTLED							
	BROWN							
AS ABOVE CHANGING TO	LT YELLOWISH	VERY DENSE	41' - 0"		12 ☒	SS	89	WET
FINE SANDY SILT	BROWN		890.04					

HOLE TERMINATED

e. m. peto associates ltd.

SOIL ENGINEERING SERVICE – TORONTO, ONTARIO

BOREHOLE LOG

Job Name Proposed Hwy. #8 Crossing Grand River
Job No. 58119
Borehole No. 7
Client Dept. of Highways of Ontario
Casing BX
Boring Date Oct. 2nd – 7th, 1958
Datum D.H.O.
Compiled By C.J.W.
Checked By C.F.F.

SAMPLE CONDITION

- ▨ UNDISTURBED
- ▧ FAIR
- ☒ DISTURBED
- ■ LOST

SAMPLE TYPE

- S.S. 2" STANDARD SPLIT TUBE SAMPLE
- S.L. SPLIT BARREL WITH LINERS
- S.T. THIN-WALLED SHELBY TUBE SAMPLE
- W.S. WASH SAMPLE
- R.C. ROCK CORE

ABBREVIATIONS

- V.T. IN SITU VANE SHEAR TEST
- Q/u UNCONFINED COMPRESSIVE STRENGTH
- W.L. WATER LEVEL IN CASING
- W.T. GROUND WATER TABLE IN SOIL

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth / Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft.	WATER LEVELS, SOIL MOISTURE & REMARKS
			0'-0"					
TOP SOIL 0'-0" TO 11"	BROWN		930.03		1 ☒	S.S.		
SILTY MED-FINE SAND	STRONG	LOOSE			2 ☒	S.S.	6	SLIGHTLY MOIST NAT. M.C. 7.9%
SOME ROOTS	BROWN							
			5'-0"					
S[illegible]Y MED-FINE SAND, GRITS	PALE	LOOSE			3 ☒	SS	9	MOIST NAT. M.C. 11.6%
AND SCATTERED GRAVEL	BROWN		W.L. 6'-11"					GRAVEL CONT. INCREASED 6'-2
SILTY COARSE SAND MANY	YELLOWISH	DENSE	923.11		4 ☒	SS	34	WET NAT. M.C. 8.9%
GRITS & PEBBLES	BROWN							
AS ABOVE CHANGING TO	DK YELLOWISH	VERY DENSE	10'-10"		5 ☒	SS	103	VERY MOIST NAT. M.C. 7.8%
SILTY MED-FINE SAND, GRITS	BROWN		919.19					WASH WATER USED FROM 12'-0"
PEBBLES & OCC. GRAVEL.	LT GREY	VERY DENSE			6 ☒	SS	134	MOIST NAT. M.C. 7.6%
SANDY TILL								
			15'-0"					
AS ABOVE		VERY DENSE			7 ▧	SS	101	MOIST
			17'-0"					VERY HARD AT 17'-0 BOULDERS
			913.03					AND GRAVEL
			20'-0"					
CLAYEY AND SILTY FINE TO	GREY	VERY DENSE			8 ☒	S.S.	78	QUITE MOIST
MED. SAND GRITS & PEBBLES	BROWN							
			25'-0"					
SILTY FINE TO COARSE SAND	PL. BROWN	VERY DENSE			9 ☒	S.S.	54	QUITE MOIST
AND ANGULAR GRAVEL								
								26' TO 30' GRAVEL AND BOULDER
								VERY HARD GOING
			30'-0"					LOST WASH WATER AT 29'-0"
AS ABOVE	PALE GREY	VERY DENSE			10 ☒	S.S.	90/6"	QUITE MOIST
	BROWN							WASH WATER RETURNED 31'-
								DRILLED FROM 31' TO 35'
								BOULDERS AT 32' AND 33'-6"
			35'-0"					
COARSE WITH SOME MED SAND	PALE GREY	VERY DENSE			11 ☒	S.S.	81	WET
AND GRAVEL	BROWN							
SILTY COARSE TO FINE SAND	PALE GREY	VERY DENSE	38'-0"		13 ☒	SS	67	WET
AND GRAVEL	BROWN		892.03					WATER LEVEL 923.11

HOLE TERMINATED

e. m. peto associates ltd.

SOIL ENGINEERING SERVICE – TORONTO, ONTARIO

BOREHOLE LOG

Job Name Proposed Hwy. #8 Crossing Grand River
Job No. 58119
Borehole No. 9
Client Dept. of Highways of Ontario
Casing BX
Boring Date Oct. 6th – 11th, 1958
Datum D.H.O.
Compiled By C.J.W.
Checked By C.F.F.

SAMPLE CONDITION		SAMPLE TYPE		ABBREVIATIONS	
▨	UNDISTURBED	S.S.	2" STANDARD SPLIT TUBE SAMPLE	V.T.	IN SITU VANE SHEAR TEST
▧	FAIR	S.L.	SPLIT BARREL WITH LINERS	Q/u	UNCONFINED COMPRESSIVE STRENGTH
☒	DISTURBED	S.T.	THIN-WALLED SHELBY TUBE SAMPLE	W.L.	WATER LEVEL IN CASING
■	LOST	W.S.	WASH SAMPLE	W.T.	GROUND WATER TABLE IN SOIL
		R.C.	ROCK CORE		

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft	WATER LEVELS, SOIL MOISTURE & REMARKS
			0-0					RIVER LEVEL 922.45 ON 6TH OCT. 58
GRAVEL, PEBBLES, MUD, WATER			922.20					DEPTH OF WATER 3"
SILTY FINE & COARSE SAND AND GRAVEL	PALE GREY BROWN	COMPACT TO DENSE			1 ☒	SS	28	WET NAT. M.C. 15.6%
			5'-0"					
SILTY VERY FINE SAND GRITS AND PEBBLES	PL. BROWNISH-GREY	VERY DENSE			2 ▧	SS	91	MOIST, NAT M.C. 8%
FINE TO MED. SAND, GRITS AND PEBBLES	GREY	VERY DENSE			3 ▧	SS	98	MOIST NAT M.C. 7.7%
			-0					
AS ABOVE	GREY	VERY DENSE			4 ☒	W.S.	160/4½	10'-6 BOULDER DRILLED 11'-2 TO 16'-3 DRILLED THRU BOULDERS & STONES
						W.S.		
SAND & GRITS	GREY		15-0					
			16'-9"					17'-9 TO 22 DRILLED THROUGH
SANDY TILL AND GRAVEL	YELLOWISH BROWN	VERY DENSE	905.45		6 ☒	S.S.	150	WET. BOULDERS AND STONES
			20-0					
			22'-0"					
SILTY COARSE SAND WITH FINE TO ME GRAVEL	GREY	VERY DENSE	900.20		7 ☒	SS	58	WET. 23-6 TO 27 & 28 TO 33-3 DRILLED THROUGH BOULDERS AND STONES
			25' 0					LOST WASH WATER 24' TO 28'
AS ABOVE	GREY	VERY DENSE			8 ☒	SS	112	WET
			30'-0"					
VERY FINE SANDY SILT	LIGHT YELLOWISH BROWN	VERY DENSE			9 ☒	WS		VERY STIFF AT 29-0" PROBABLY LAYER OF BOULDERS, TO 33-3
			33'-3" 888.95					33-3 TO 38-3" DIA. DRILLING 48% RECOVERY
								SECTIONS OF BOTH CORES
BANDED LIMESTONE	LT. GREY					BX. CORE		BADLY PITTED BY WATER ACTION
								38-3 TO 43-3 56% RECOVERY ARTESIAN EFFECT NOTED AS FOLLOWS FROM 22' WATER LEVEL
			43'-3" 878.95					ROSE IN CASING TO EL. 922.62
		HOLE TERMINATED						FROM 23' WATER LEVEL ROSE TO 923.20 WITH HOLE FROM 43'-3" WATER ROSE IN CASING FROM 927.26 TO 927.53

e. m. peto associates ltd.

SOIL ENGINEERING SERVICE - TORONTO, ONTARIO

BOREHOLE LOG

Job Name Proposed Hwy.#8 Crossing Grand River
Job No. 58119
Borehole No. 11
Client Dept. of Highways of Ontario
Casing BX
Boring Date Oct. 15th, 16th & 17th, 1958
Datum D.H.O.
Compiled By C.J.W.
Checked By C.F.F.

SAMPLE CONDITION

- ▨ UNDISTURBED
- ▧ FAIR
- ☒ DISTURBED
- ■ LOST

SAMPLE TYPE

- S.S. 2" STANDARD SPLIT TUBE SAMPLE
- S.L. SPLIT BARREL WITH LINERS
- S.T. THIN-WALLED SHELBY TUBE SAMPLE
- W.S. WASH SAMPLE
- R.C. ROCK CORE

ABBREVIATIONS

- V.T. IN SITU VANE SHEAR TEST
- Q/u UNCONFINED COMPRESSIVE STRENGTH
- W.L. WATER LEVEL IN CASING
- W.T. GROUND WATER TABLE IN SOIL

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth / Elevation	Legend	Sample No and Condition	Sample Type	No of Blows per Ft	WATER LEVELS, SOIL MOISTURE & REMARKS
			0-0					
TOPSOIL 0"-2"			925.81					
COARSE GRAVEL IN MATRIX OF OF ORGANIC CLAYEY SAND	DK GREY	DENSE	W.L. 18OCT58 △ 923.39		1 ☒	SS	37	WET MOIST CONT. 9%
			5-0					
SA DY TILL GRITS AND PI BLES	GREY	VERY DENSE			2 ☒	SS	100/6½	MOIST NAT. M.C. 7.3%
AS ABOVE		VERY DENSE			3 ☒	SS	100/4½	FAIRLY MOIST
			10-0					
					4 ■		100/8½	
MED. TO COARSE SAND AND GRITS & PEBBLES	GREY				5 ☒	WS		PROBABLY GREY SANDY TILL
			15-0					
SANDY TILL GRITS AND PEBBLES	GREY	VERY DENSE			6 ☒	SS	167	VERY MOIST
			20-0					HARD GOING FROM 20'
AS ABOVE WITH GRAVEL UP TO 1½"		VERY DENSE			7 ☒	SS	145	WET
								DIA. DRILLED FROM 21-0 TO 22 RECOVERY 6" DIA. DRILLED 22 TO 27 RECOVERY 20" DIA. DRILLED 27 TO 28' RECOVERY 5" DIA DRILLED 30 T 35 RECOVERY 6" DIA DRILLED 35 TO 36-0 RECOVERY 6" DIA DRILLED 36-0 TO 39'-0" RECOVERY 2'-1" — 63%
			25-0					
MED. FINE SAND	GREY				8 ☒	WS		
MED. FINE GRAVEL	GREY		30-0		9 ☒	WS		
		VERY DENSE			10 ■	SS	50/1"	PROBABLY BOULDER
BOULDERY TILL	GREY	VERY DENSE						
			36-0 889.31					
BANDED LIMESTONE	GREY	MED. TO SOFT HARDNESS APPROX. 4	39-0 886.31			BX CORE		
			HOLE TERMINATED					

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SOIL ENGINEERING SERVICE -- TORONTO, ONTARIO

BOREHOLE LOG

Job Name: Proposed Hwy. #8 Crossing Grand River
Job No.: 58119
Borehole No.: 12
Client: Dept. of Highways of Ontario
Casing: BX
Boring Date: Oct. 14th - 20th, 1958
Datum: D.H.O.
Compiled By: C.J.W.
Checked By: C.F.F.

SAMPLE CONDITION

- ▨ UNDISTURBED
- ▧ FAIR
- ☒ DISTURBED
- ■ LOST

SAMPLE TYPE

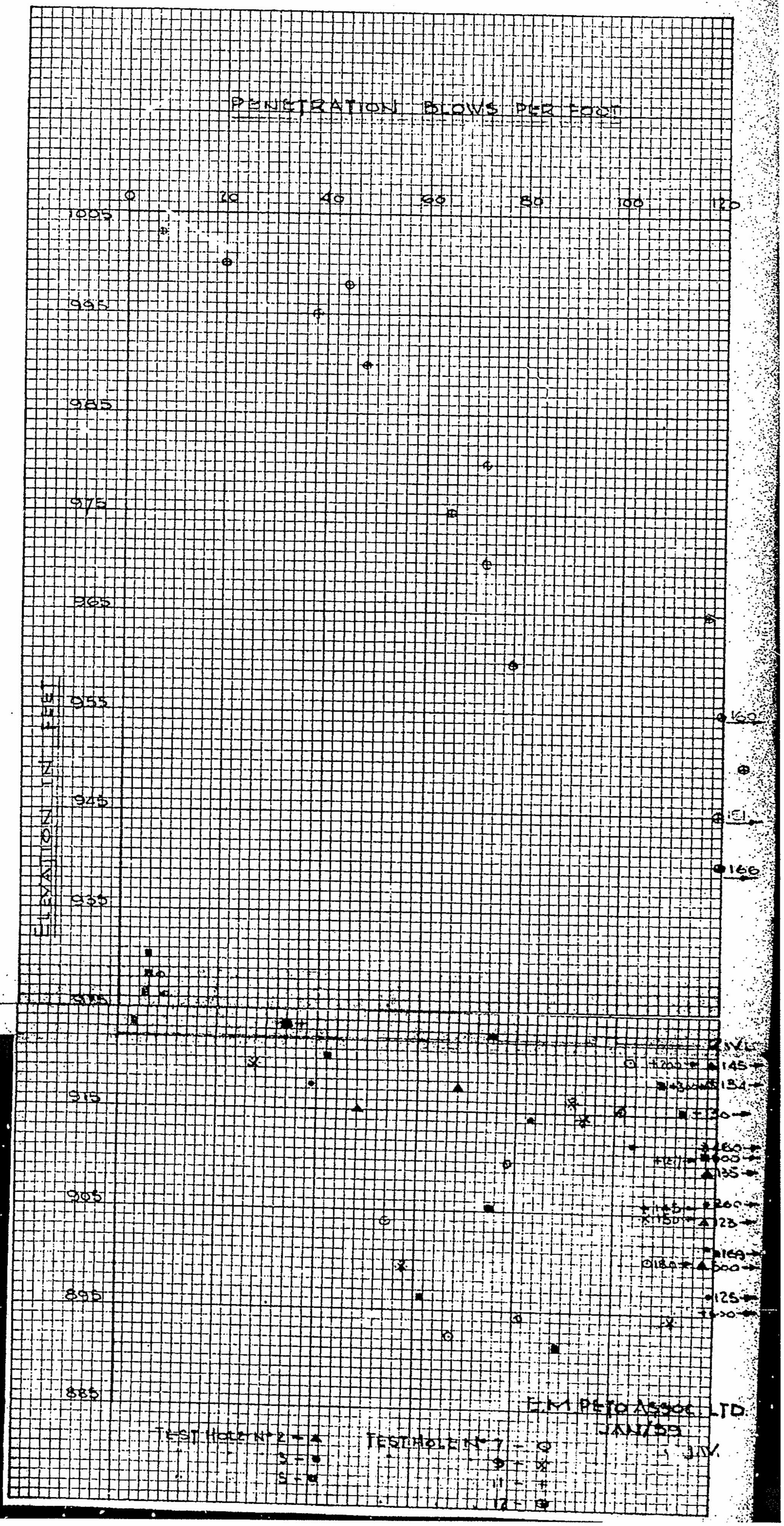
S.S. 2" STANDARD SPLIT TUBE SAMPLE
S.L. SPLIT BARREL WITH LINERS
S.T. THIN-WALLED SHELBY TUBE SAMPLE
W.S. WASH SAMPLE
R.C. ROCK CORE

ABBREVIATIONS

V.T. IN SITU VANE SHEAR TEST
Q/u UNCONFINED COMPRESSIVE STRENGTH
W.L. WATER LEVEL IN CASING
W.T. GROUND WATER TABLE IN SOIL

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft	WATER LEVELS, SOIL MOISTURE & REMARKS
			0-0 1005.43					
SLIGHTLY SILTY MED. FINE SAND WITH BROKEN LIMESTONE FRAGMENTS	BROWN	LOOSE	5-0		1 ☒	S.S.	7	SLIGHTLY MOIST
AS ABOVE		COMPACT			2 ☒	S.S.	20	SLIGHTLY MOIST
SILTY MED-FINE SAND WITH A SEAM OF YELLOWISH-BROWN SANDY LOAM.	BROWN	DENSE	10-0		3 ☒	SS	44	SLIGHTLY MOIST. SOME Ro IN SAMPLE.
MED-FINE SAND GRITS & PEBBLES POCKETS OF S[illegible]Y FINE SAND	PALE-BROWN	DENSE	12-0 993.43 15-0		4 ☒	SS	38	MOIST NAT. M.C. 6.3%
SANDY CLAY WITH LIMESTONE FRAGMENTS GRITS & PEBBLES	BROWN	DENSE	19-0 986.45		5 ☒	S.S.	48	FAIRLY MOIST. NAT. M.C. 8.8%
SANDY TILL, GRITS AND PEBBLES	GREY	VERY DENSE	24-0 981.43		6 ☒	S.S	136	FAIRLY MOIST
SILTY CLAY	DARK GREY-BROWN	V. HARD	30-0		7 ☒	S.S.	72	SHEAR STRENGTH > 7,000 LBS./ DRIER THAN PLASTIC LIMIT MOIST. CONTENT 18.8%
AS ABOVE		V. HARD	35-0		8 ☒	SS	65	SHEAR STRENGTH > 7,000 LBS./ DRIER THAN PLASTIC LIMIT MOIST. CONTENT 18.5%
AS ABOVE		V. HARD	40-0 965.43		9 ▧	S.S.	72	DRIER THAN PLASTIC LIMIT MOIST CONTENT 18.9%
AS ABOVE WITH GRITS AND PEBBLES		V. HARD	43-0 962.43 45-0		10 ☒	SS	117	DUE TO STONE INTERFERENCE MOISTURE AT PLASTIC LIMIT
SILTY FINE SAND WITH LIMESTONE & GRAVEL	GREY	VERY DENSE	50-0		11 ☒	SS	78	WET.
COARSE TO MED. SAND AND FINE GRAVEL	BROWN	VERY DENSE	55-0		12 ☒	WASH SAMPLE	160	WITH [illegible] AT 55' AND HOLE 55 DEEP LOST [illegible] WATER OVERNIGHT
SILTY FINE SAND WITH COARSE TO FINE GRAVEL UP TO 1½" ⌀	GREY	VERY DENSE	5[illegible]-0 94[illegible].43 60-[illegible]		13 ☒	SS	125	WET [illegible]
SANDY TILL, GRITS AND PEBBLES	GREY	VERY DENSE			14 ☒	SS	151	MOIST NAT. M.C. 8.7%
AS ABOVE	GREY	VERY DENSE	66-0 70-0 935.43		15 ☒	SS	166	MOIST NAT. M.C. [illegible] NO FREE WATER LEVEL OBSERVED AT THIS LEVEL

HOLE TERMINATED ON BOULDERS


PENETRATION BLOWS PER FOOT
0
20
40
60
80
100
120
ELEVATION IN FEET
1005
995
985
975
965
955
945
935
925
915
905
895
885
160
151
166
145
134
200
300
230
260
600
151
135
260
145
150
123
169
180
500
125
300
TEST HOLE Nº 2 - ▲
3 - ●
5 - ■
TEST HOLE Nº 7 - ⊙
9 - ✕
11 - +
12 - ⊕
E.M. PETO ASSOC. LTD.
JAN/59
J.V.

e. m. peto associates ltd.
SOIL TESTING LABORATORY

LIQUID LIMIT TEST

FLOW LINE CHARTS

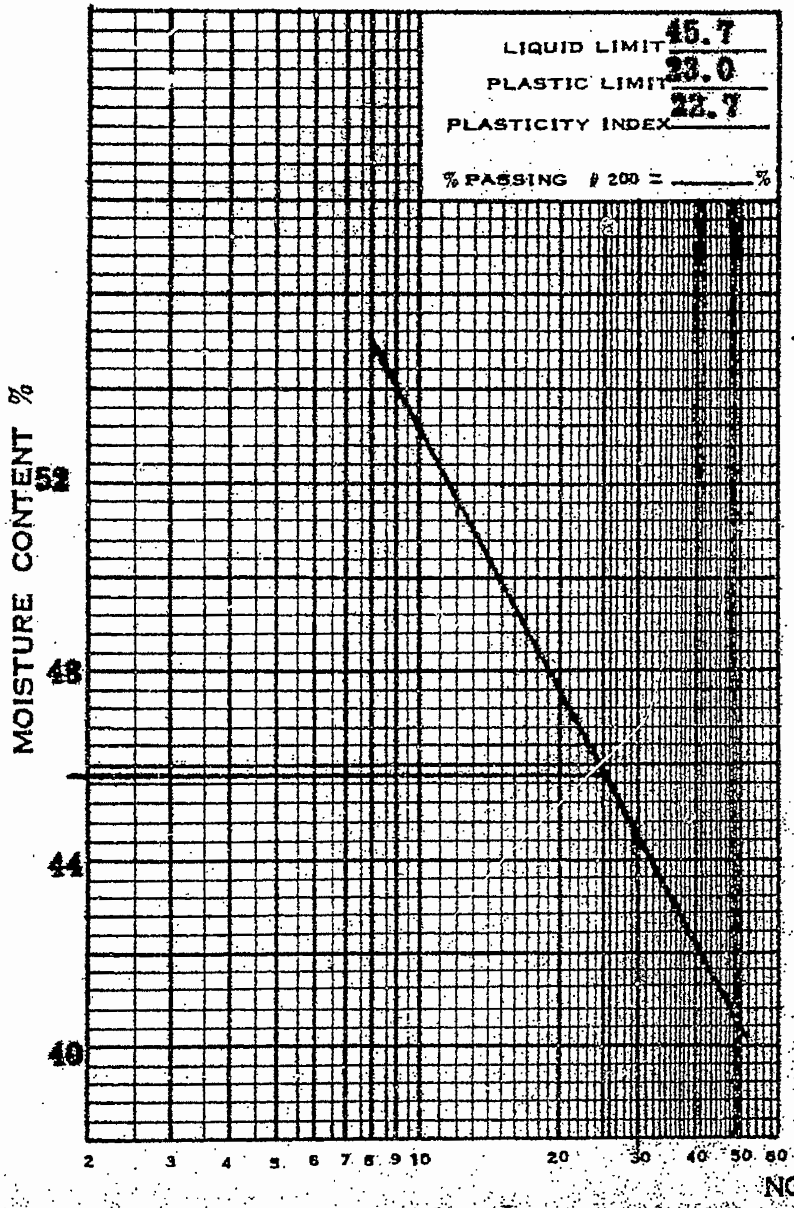
Job No. 58119 Project Proposed Hw. 8 Grand River Crossing

Sample From B. H. 12 Sample 8

Depth 30'-31'

Sample From

Depth

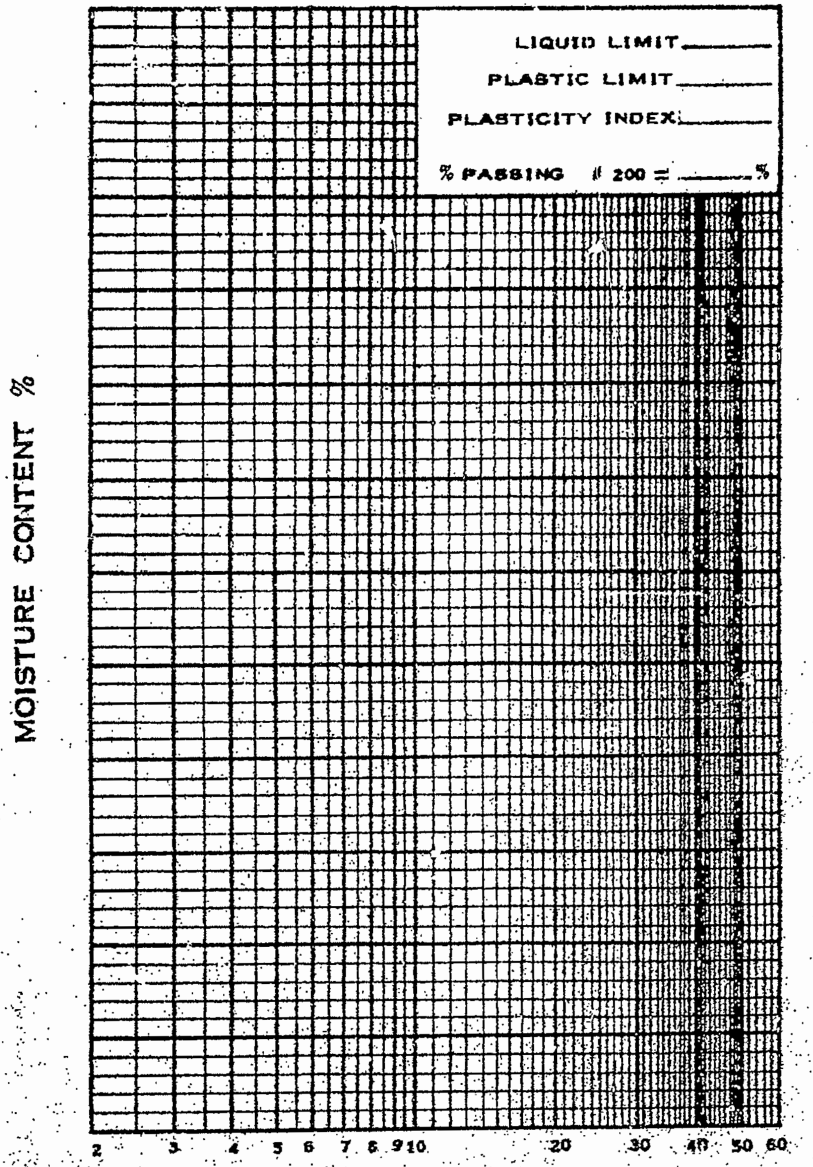


Liquid Limit 45.7

Plastic Limit 23.0

Plasticity Index 22.7

% Passing # 200 = %



Liquid Limit

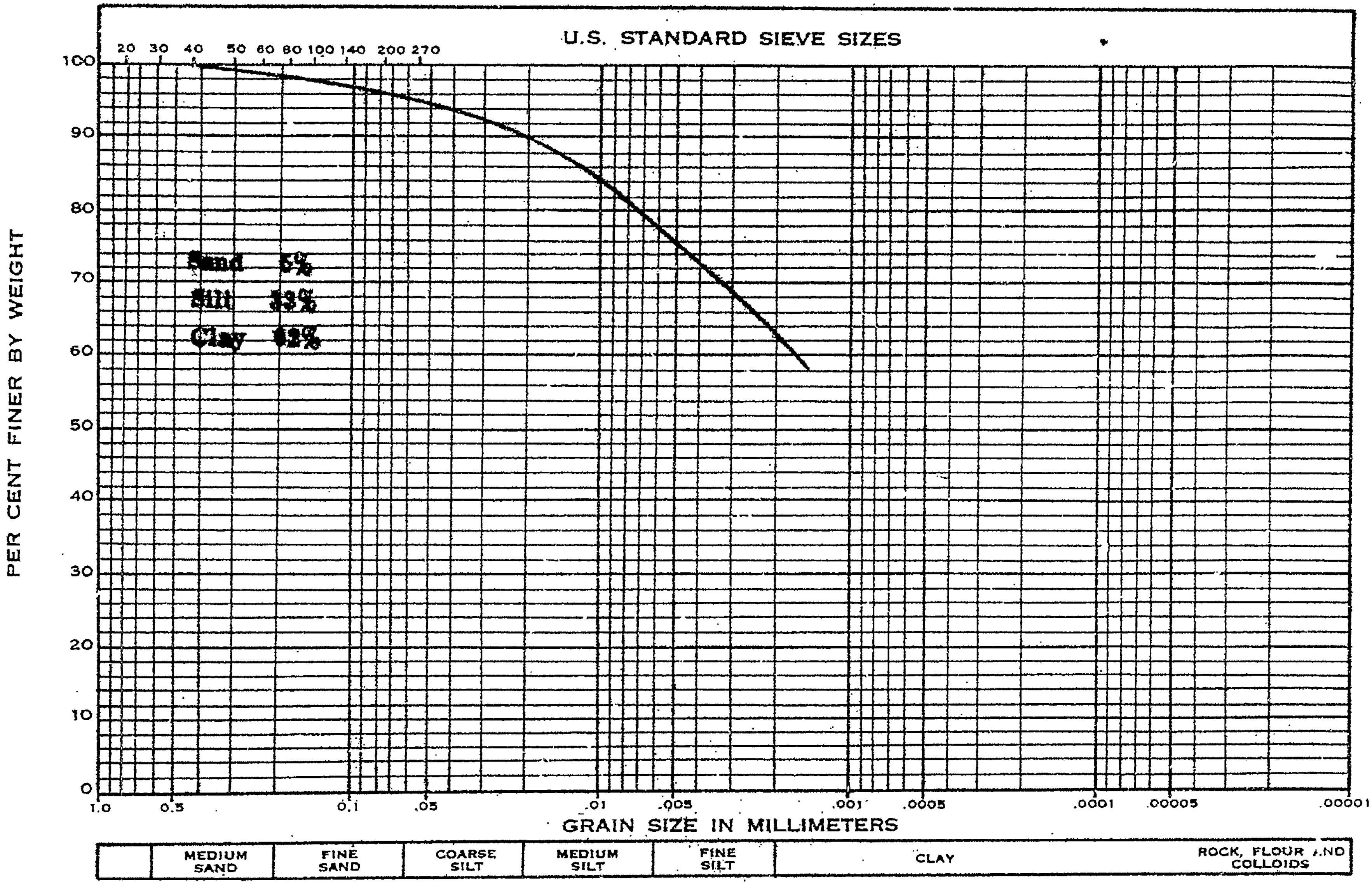
Plastic Limit

Plasticity Index

% Passing # 200 = %

E. M. PETO ASSOCIATES LTD.

HYDROMETER GRAIN SIZE DISTRIBUTION DIAGRAM



M.I.T. CLASSIFICATION

JOB NAME Proposed Hwy. 3 Grand River Crossing #3119 BOREHOLE No. 12 SAMPLE No. 8

DEPTH 30'-31' ELEVATION ______ REMARKS Dark grey brown silty clay

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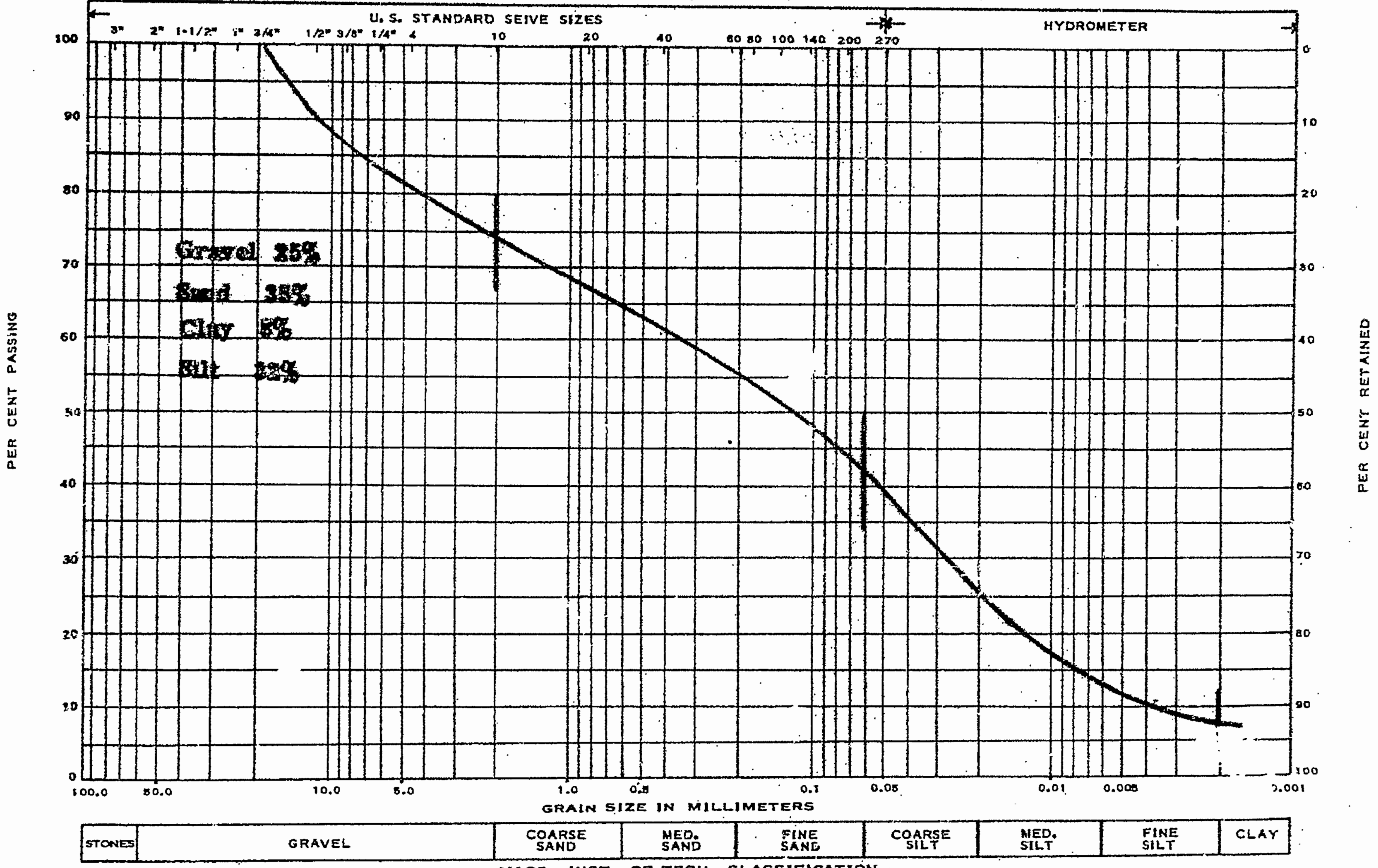
TORONTO, ONTARIO



Gravel 25%
Sand 38%
Clay 5%
Silt 32%

JOB NAME Proposed Hwy. 8 Grand River Crossing JOB NO. 58119 HOLE NO. 5 SAMPLE NO. 6

DEPTH 12'-13' ELEVATION ____ REMARKS Sandy till

GRAIN SIZE DISTRIBUTION

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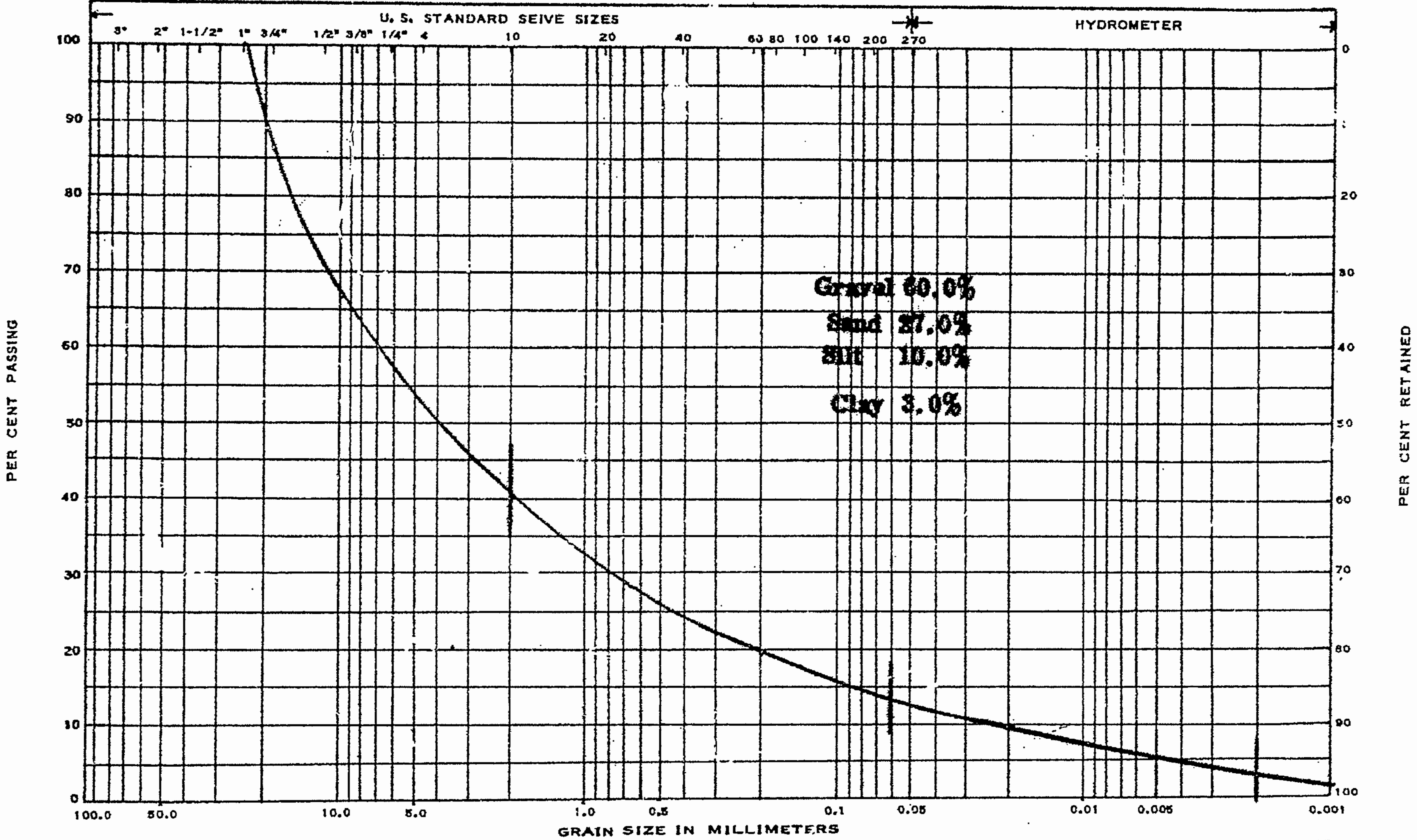

Gravel 60.0%
Sand 27.0%
Silt 10.0%

Clay 3.0%

STONES	GRAVEL	COARSE SAND	MED. SAND	FINE SAND	COARSE SILT	MED. SILT	FINE SILT	CLAY

MASS. INST. OF TECH. CLASSIFICATION

JOB NAME Proposed Hwy. 8 Grand River Crossing JOB NO. 58119 HOLE NO. 5 SAMPLE NO. 9

DEPTH 25'-26' ELEVATION ____ REMARKS Slightly silty sandy gravel

GRAIN SIZE DISTRIBUTION

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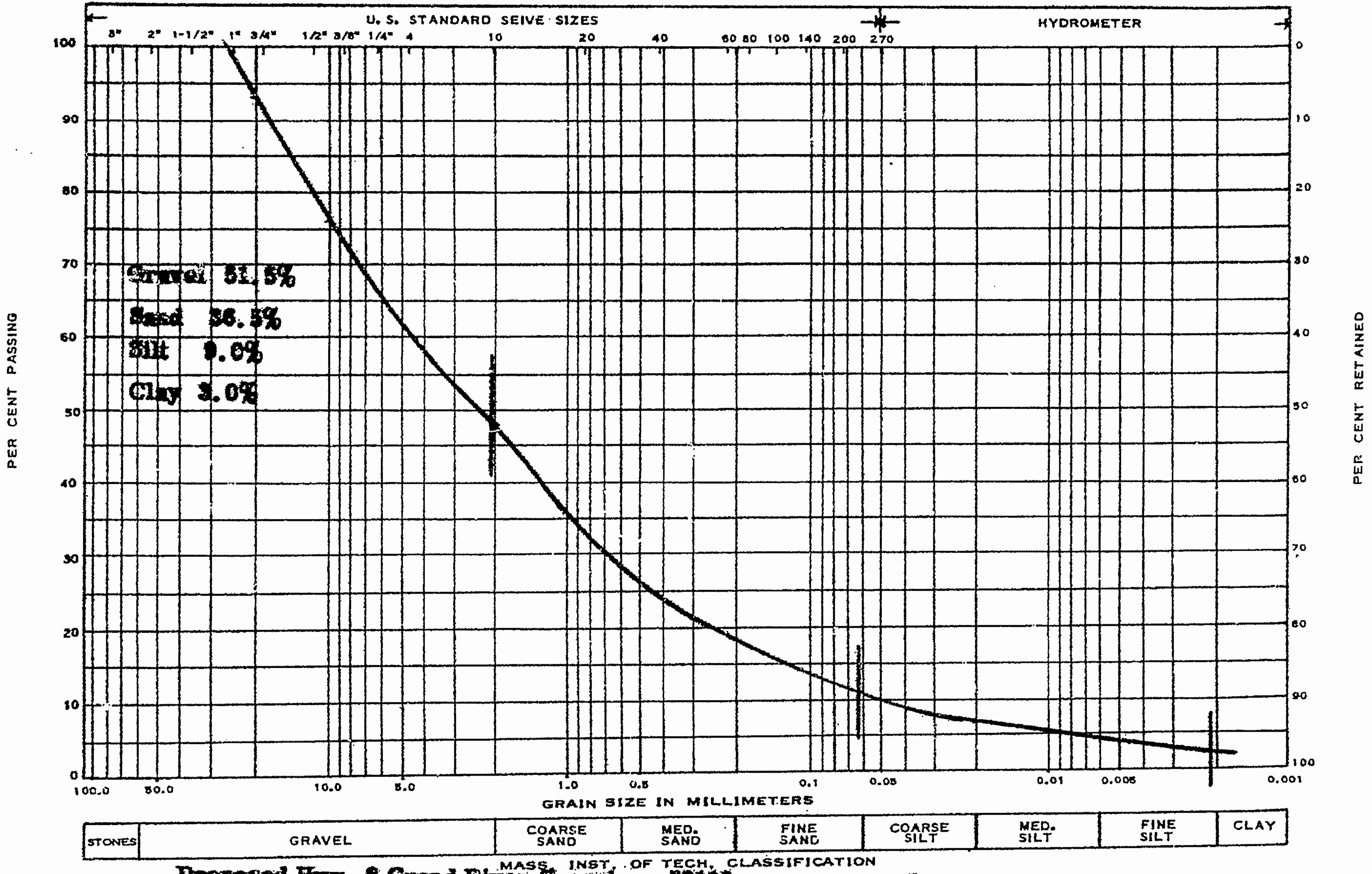
TORONTO, ONTARIO



Gravel 51.5%
Sand 36.5%
Silt 9.0%
Clay 3.0%

JOB NAME Proposed Hwy. 8 Grand River Crossing JOB NO. 58119 HOLE NO. 5 SAMPLE NO. 11

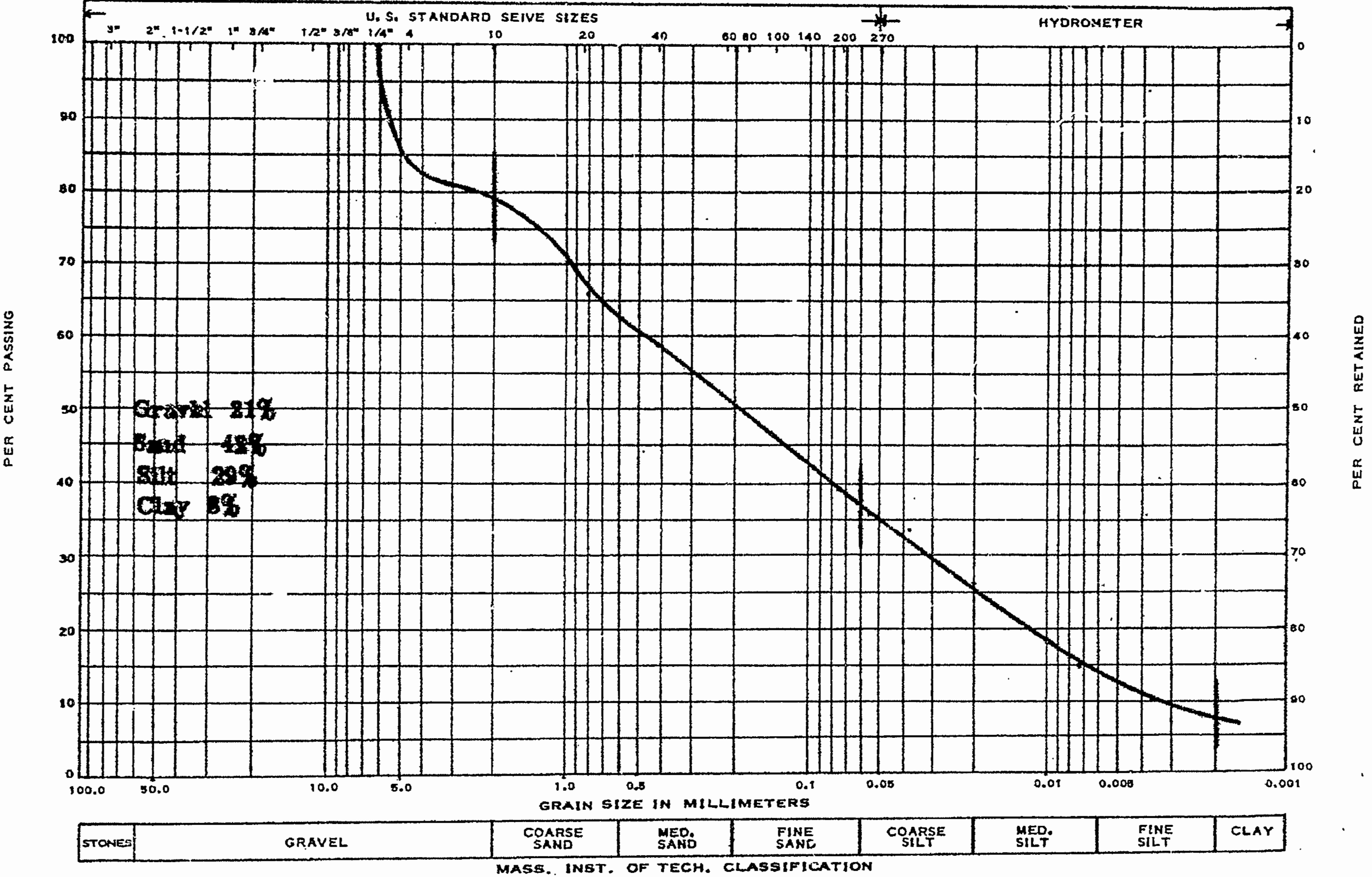
35'-36'

DEPTH______ ELEVATION______ REMARKS Slightly silty sandy gravel

GRAIN SIZE DISTRIBUTION

e. m. peto associates ltd.

TORONTO, ONTARIO



Gravel 21%
Sand 42%
Silt 29%
Clay 8%

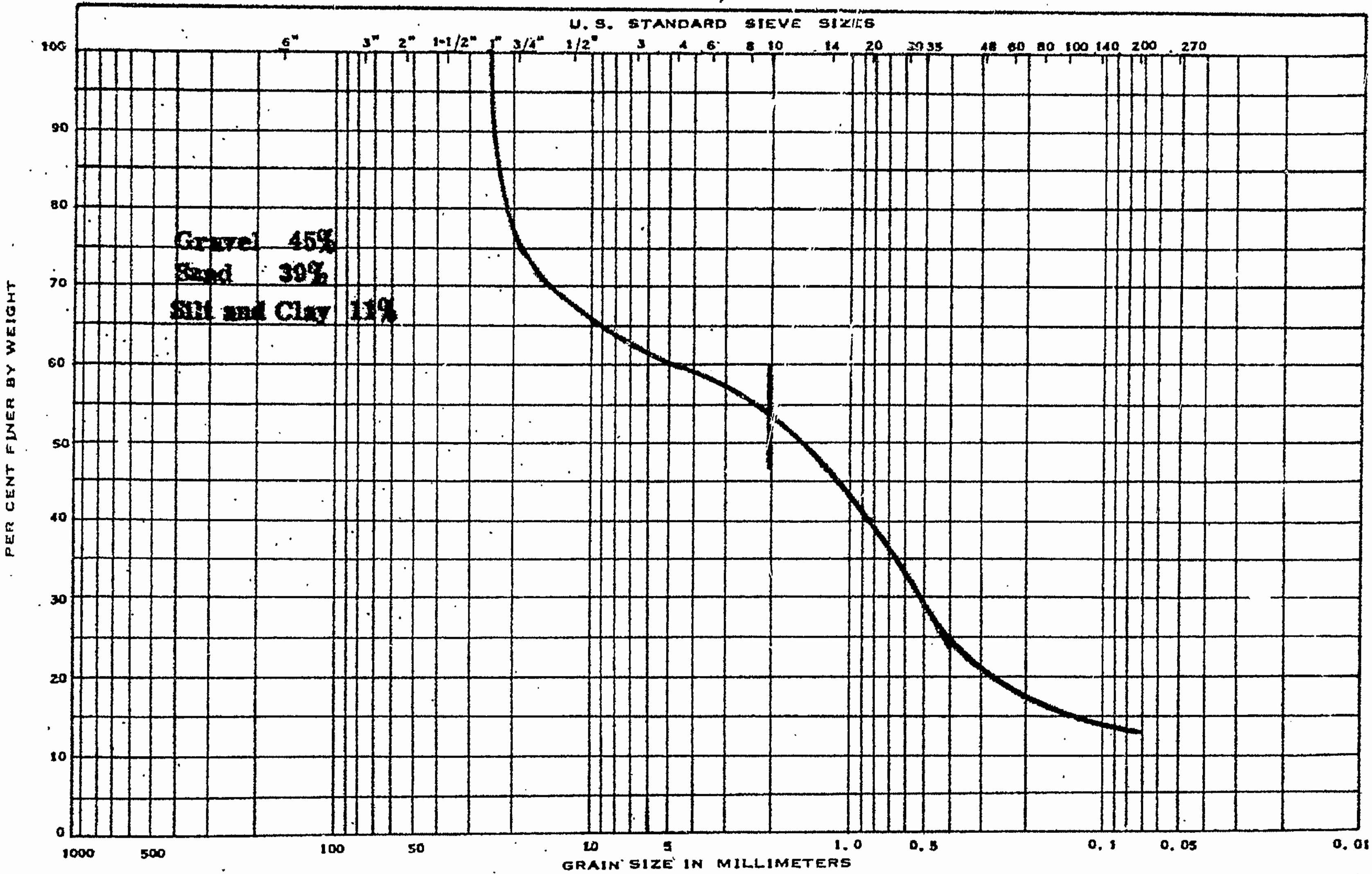
JOB NAME Proposed Hwy. 8 Grand River Crossing JOB NO. 58119 HOLE NO. 7 SAMPLE NO. 6

DEPTH 12' - 13' ELEVATION ______ REMARKS Sandy till

GRAIN SIZE DISTRIBUTION

e. m. peto associates ltd.

TORONTO, ONTARIO



Gravel 45%
Sand 30%
Silt and Clay 11%

BOULDERS	STONES	GRAVEL	COARSE SAND	MED. SAND	FINE SAND	COARSE SILT	MED. SILT

MASS. INST. OF TECH. CLASSIFICATION

JOB NAME Proposed Hwy. 8 Grand River Crossing JOB NO. 58119 HOLE NO. 7 SAMPLE NO. 8

DEPTH 20'-21' ELEVATION ______ REMARKS Sandy gravel

GRAIN SIZE DISTRIBUTION DIAGRAM

COARSE MATERIALS

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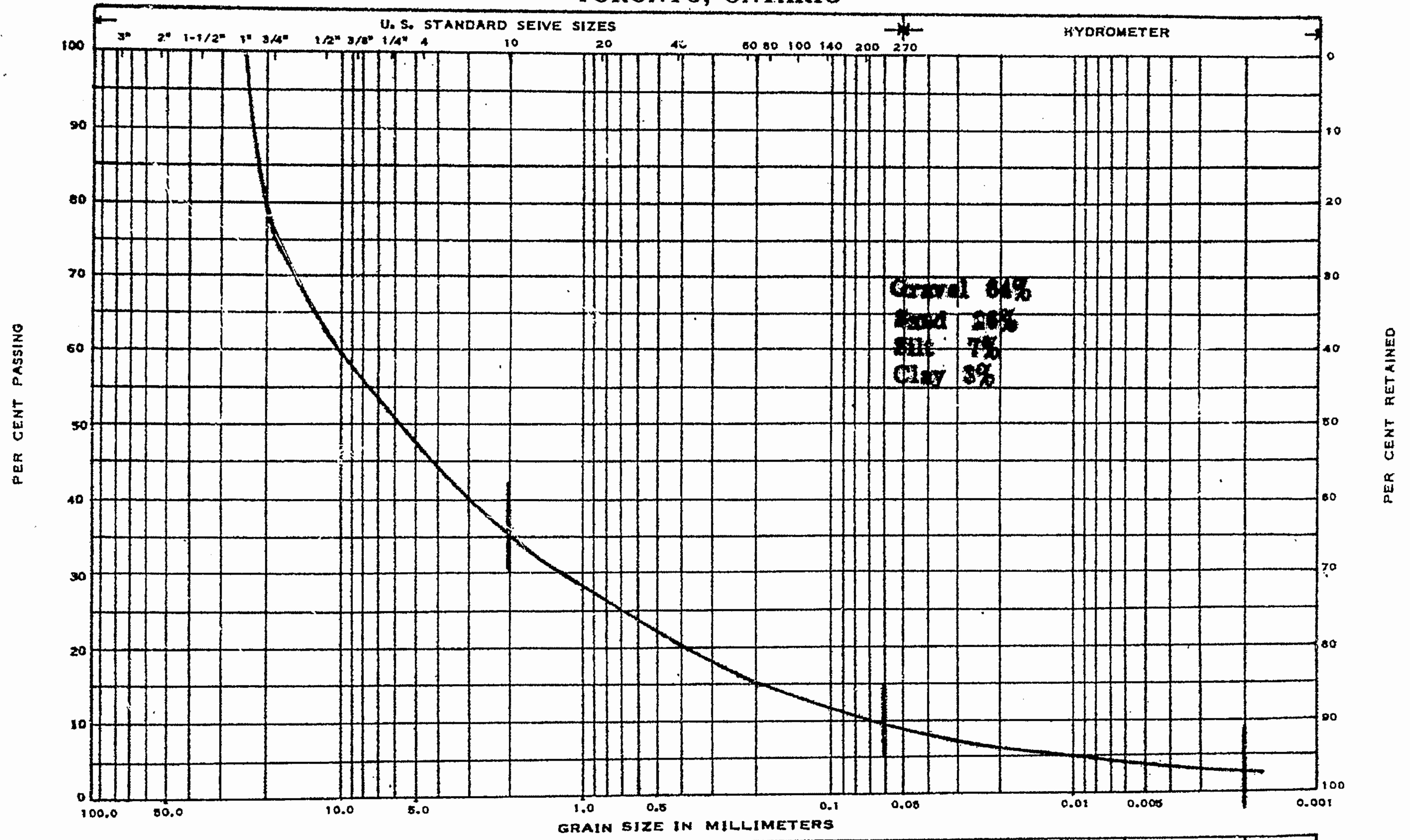
TORONTO, ONTARIO



Gravel 64%
Sand 26%
Silt 7%
Clay 3%

STONES	GRAVEL	COARSE SAND	MED. SAND	FINE SAND	COARSE SILT	MED. SILT	FINE SILT	CLAY

MASS. INST. OF TECH. CLASSIFICATION

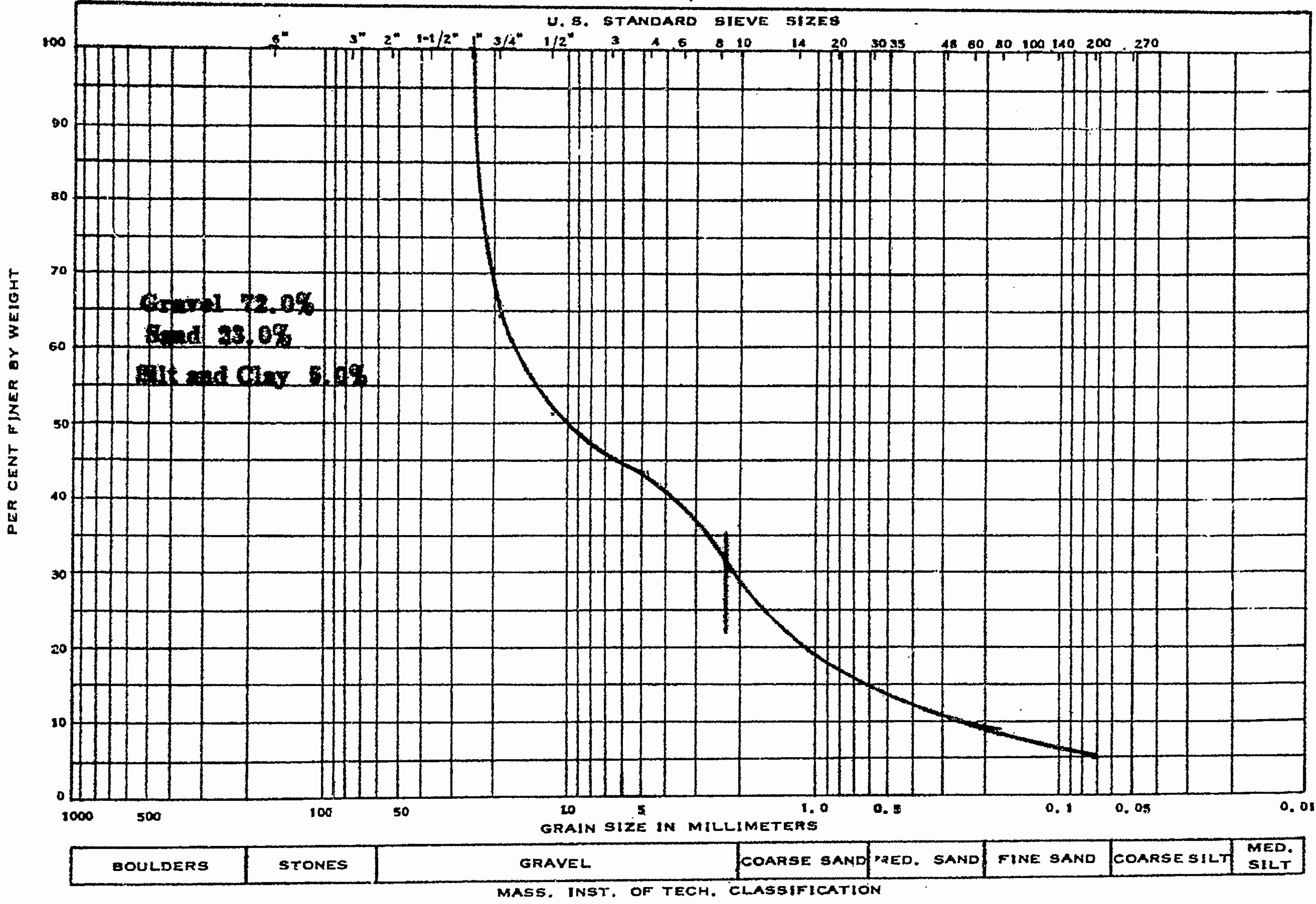
JOB NAME Proposed Hwy. 8 Grand River Crossing JOB NO. 58119 HOLE NO. 7 SAMPLE NO. 9

DEPTH 25'-27' ELEVATION ______ REMARKS Slightly silty sandy gravel

GRAIN SIZE DISTRIBUTION

e. m. peto associates ltd.

TORONTO, ONTARIO



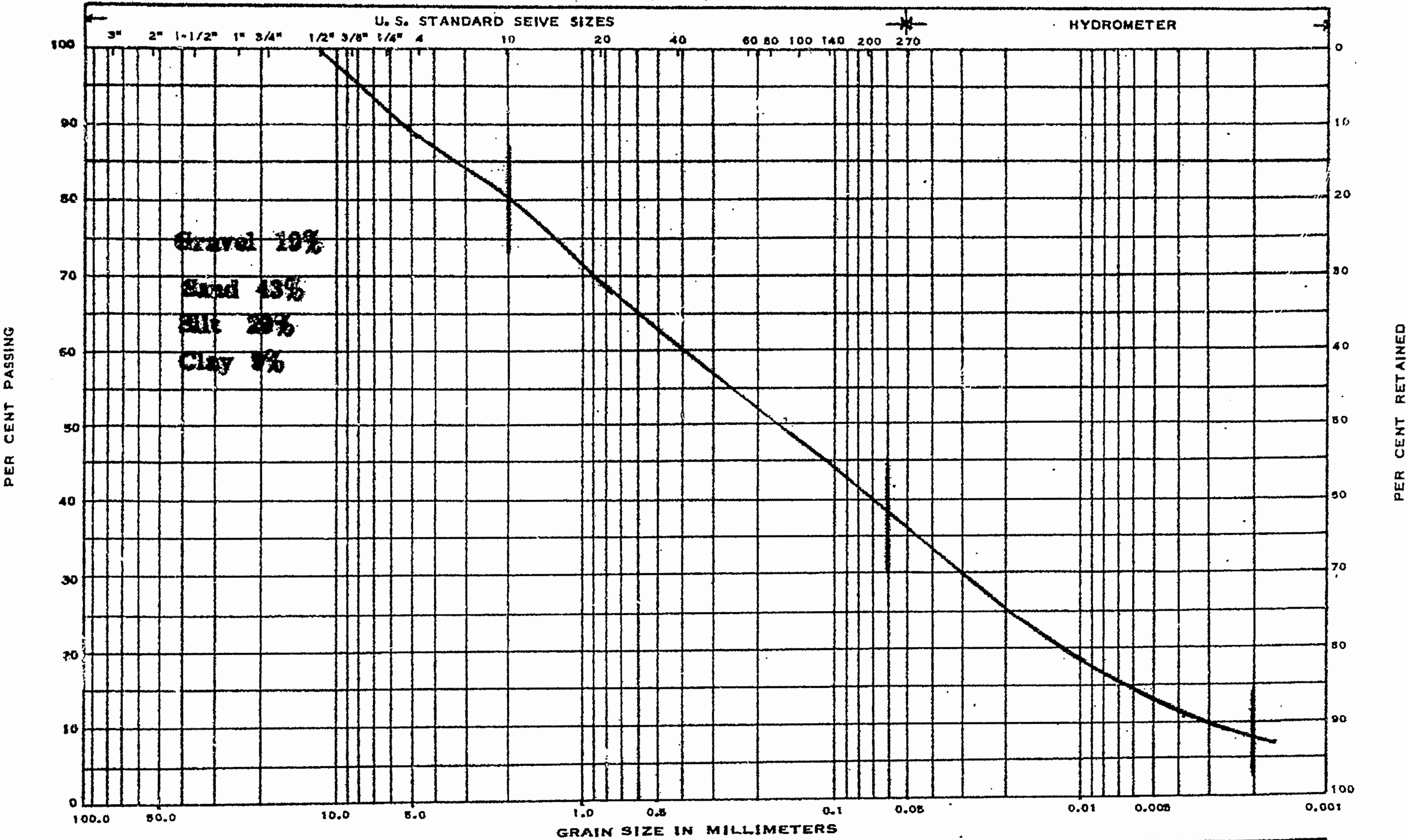
Gravel 72.0%
Sand 23.0%
Silt and Clay 5.0%

JOB NAME Proposed Hwy. 8 Grand River Crossing JOB NO. 58119 HOLE NO. 7 SAMPLE NO. 11

DEPTH 35'-36' ELEVATION ______ REMARKS Sandy gravel

GRAIN SIZE DISTRIBUTION DIAGRAM
COARSE MATERIALS

e. m. peto associates ltd.
TORONTO, ONTARIO



Gravel 19%

Sand 43%

Silt 29%

Clay 9%

STONES	GRAVEL	COARSE SAND	MED. SAND	FINE SAND	COARSE SILT	MED. SILT	FINE SILT	CLAY

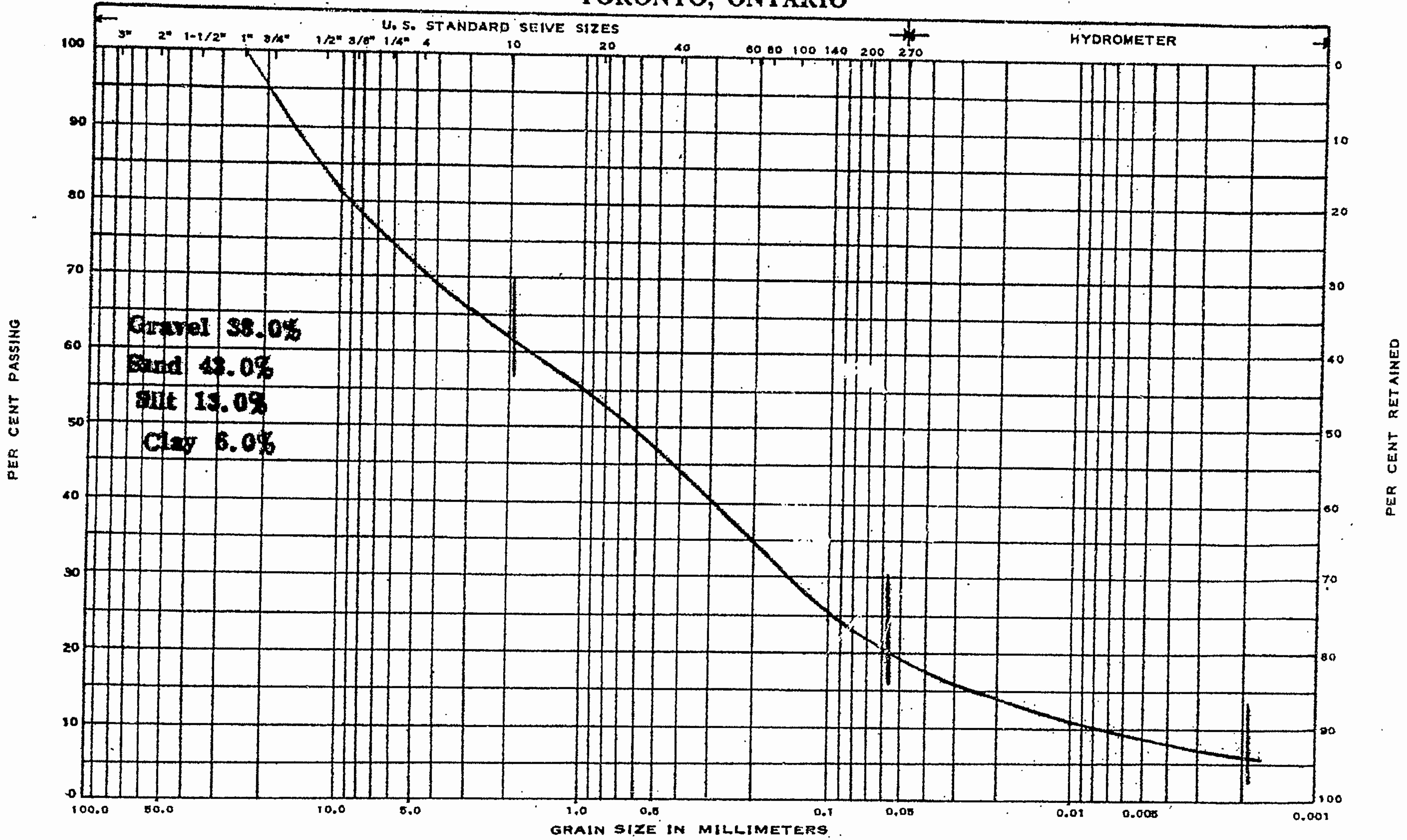
MASS. INST. OF TECH. CLASSIFICATION

JOB NAME Proposed Hwy. 8 Grand River Crossing JOB NO. 58119 HOLE NO. 9 SAMPLE NO. 3

DEPTH 7'-8' ELEVATION______ REMARKS Grey sandy till

GRAIN SIZE DISTRIBUTION

e. n. peto associates ltd.
TORONTO, ONTARIO



Gravel 38.0%
Sand 43.0%
Silt 13.0%
Clay 6.0%

STONES	GRAVEL	COARSE SAND	MED. SAND	FINE SAND	COARSE SILT	MED. SILT	FINE SILT	CLAY

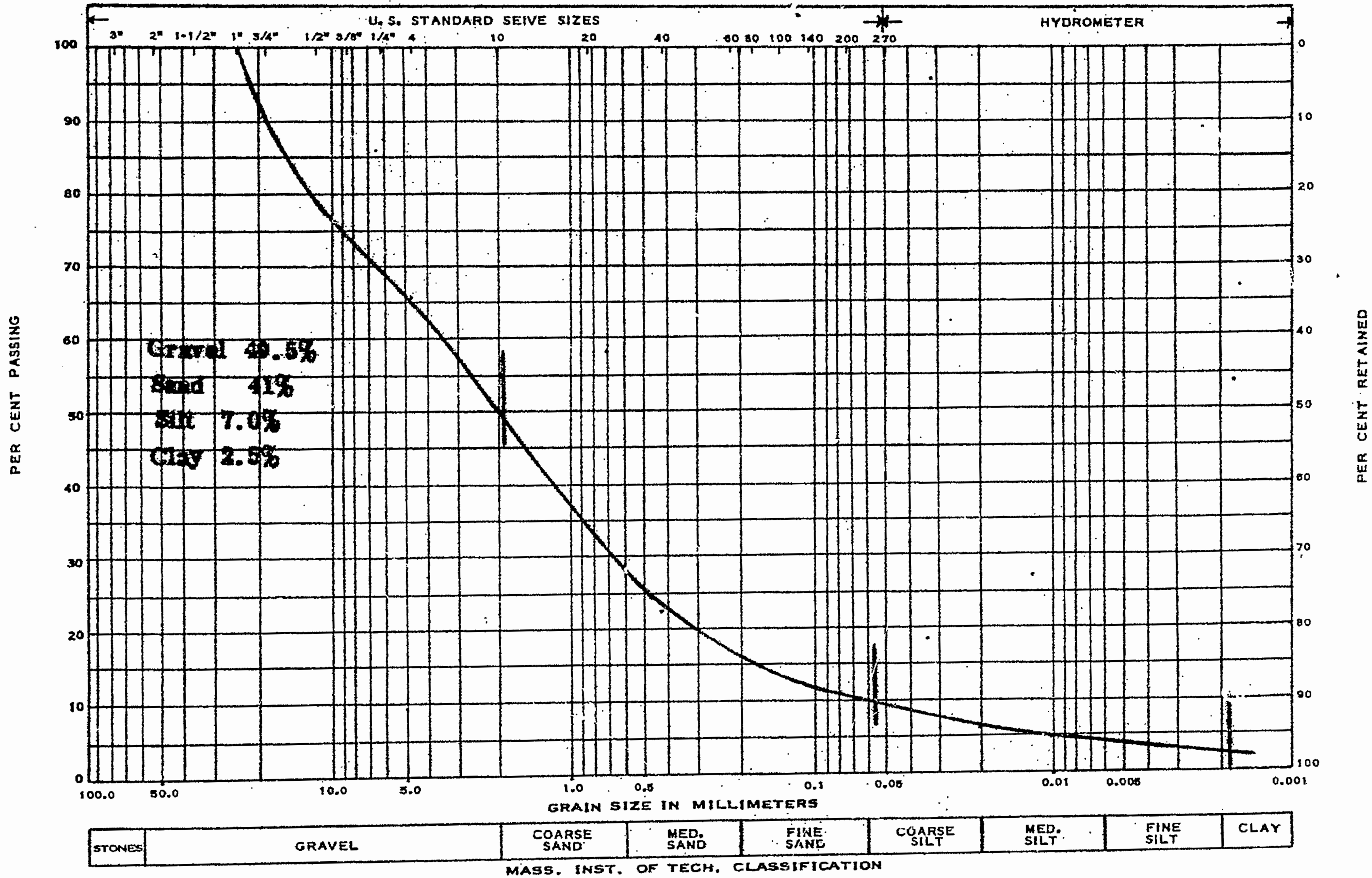
MASS. INST. OF TECH. CLASSIFICATION

JOB NAME Proposed Hwy. 8 Grand River Crossing 58119 HOLE NO. 9 SAMPLE NO. 6

DEPTH 16'9" - 17'9" ELEVATION ______ REMARKS Slightly silty sandy gravel.

GRAIN SIZE DISTRIBUTION

c. m. peto associates ltd.
TORONTO, ONTARIO



Gravel	49.5%
Sand	41%
Silt	7.0%
Clay	2.5%

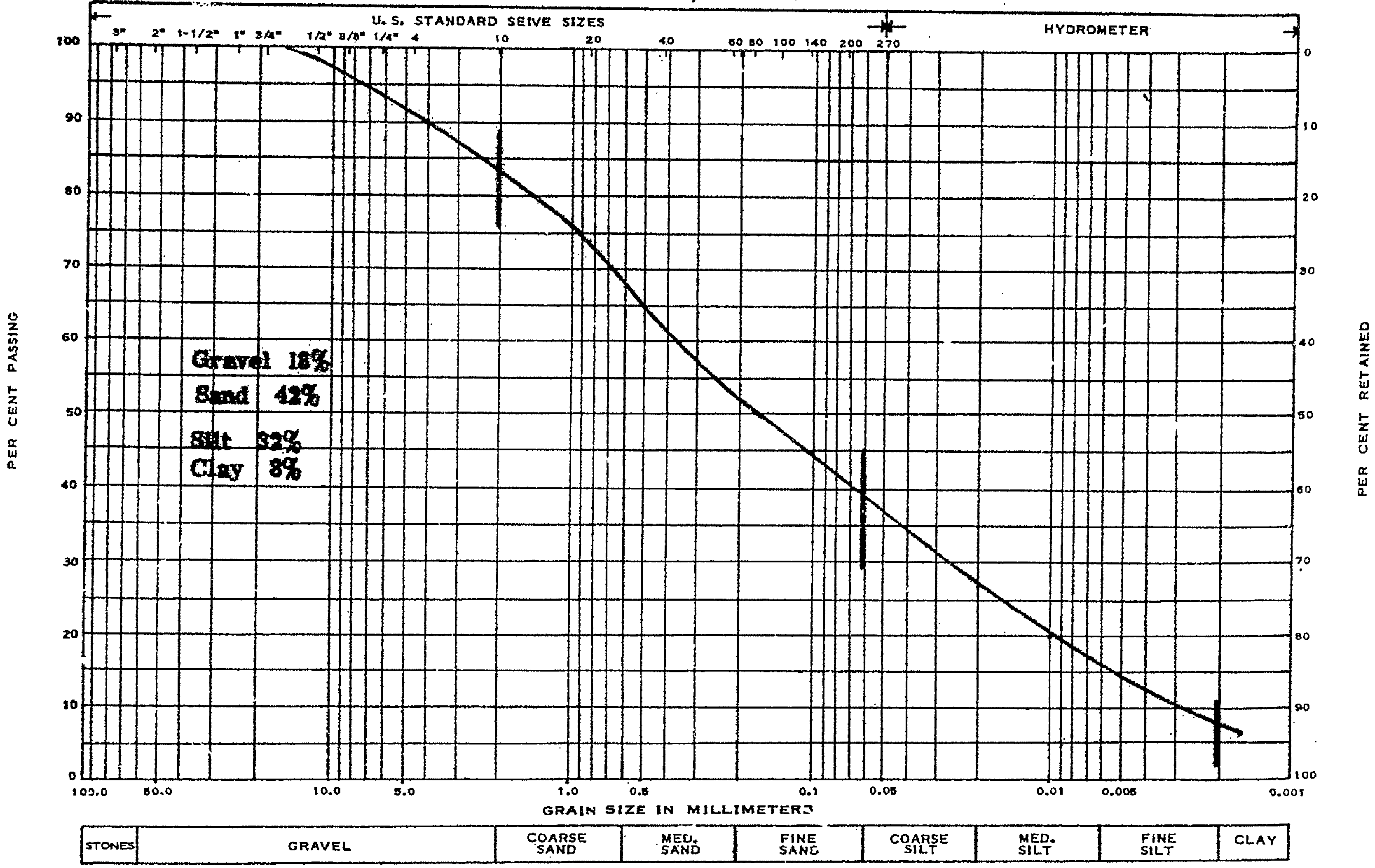
JOB NAME Proposed Hwy. 8 Grand River Crossing 52119 HOLE NO. 9 SAMPLE NO. 7

DEPTH 22'-23' ELEVATION ____ REMARKS Slightly silty sandy gravel.

GRAIN SIZE DISTRIBUTION

e. m. peto associates ltd.

TORONTO, ONTARIO



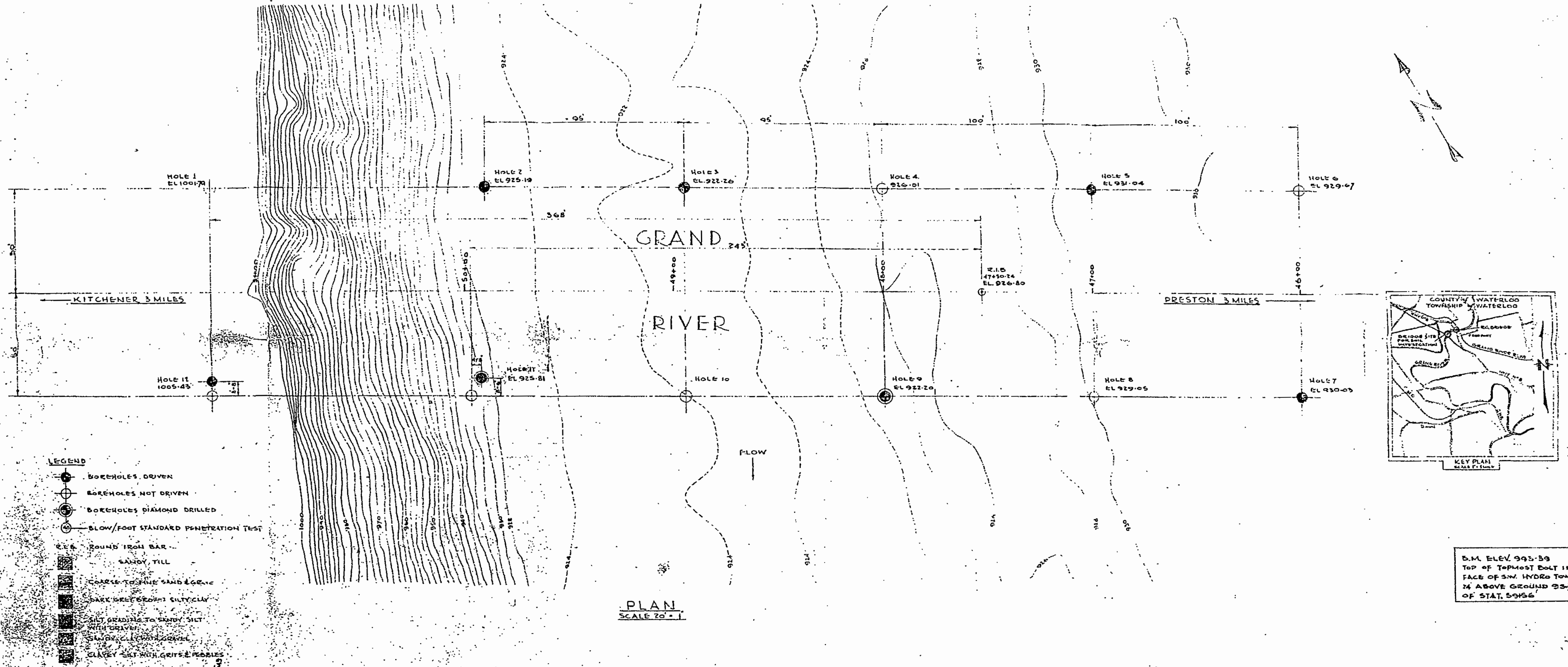
Gravel 18%
Sand 42%
Silt 32%
Clay 8%

MASS. INST. OF TECH. CLASSIFICATION

JOB NAME Proposed Hwy. 8 Grand River Crossing 58119 HOLE NO. 11 SAMPLE NO. 3

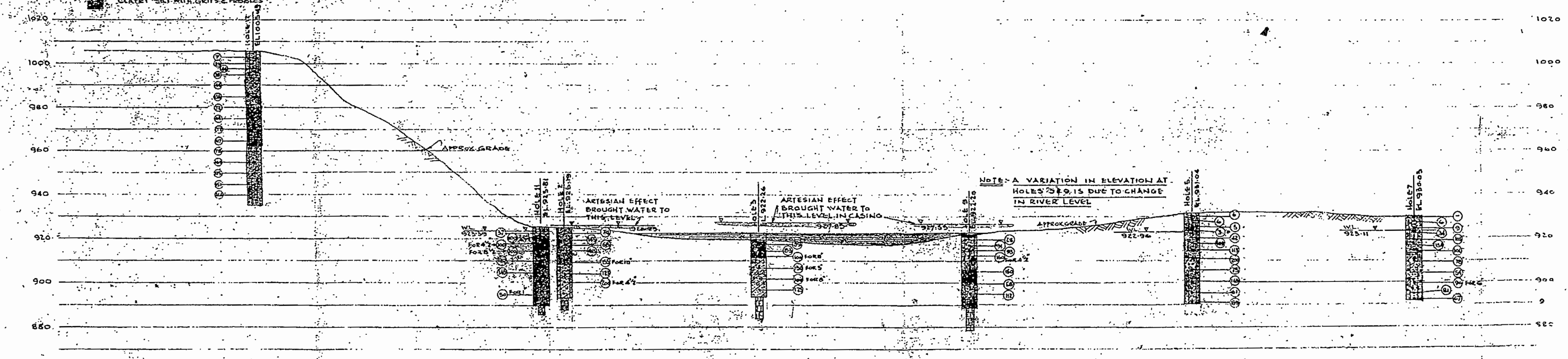
DEPTH 7'-8' ELEVATION ______ REMARKS Grey sandy till.

GRAIN SIZE DISTRIBUTION



B.M. ELEV. 993.39
TOP OF TOPMOST BOLT IN SOUTH
FACE OF S.W. HYDRO TOWER
24' ABOVE GROUND 93.0' RIGHT
OF STAT. 59156

PLAN
SCALE 20' = 1



ARTESIAN EFFECT BROUGHT WATER TO THIS LEVEL

ARTESIAN EFFECT BROUGHT WATER TO THIS LEVEL IN CASING

NOTE:- A VARIATION IN ELEVATION AT HOLES 3 & 9 IS DUE TO CHANGE IN RIVER LEVEL

CROSS SECTION THROUGH HOLES 12, 11, 2, 3, 9, 5 & 7.

PROFILE SCALES:- HOR. 20' = 1
VERT. 20' = 1

NOTE
SEE BOREHOLE LOGS FOR COMPLETE SOIL DETAILS

13 JAN/59	ELEVATIONS CHANGED	G.A.W.	C.F.F.
DATE	REVISION	DWN.	CKD.

e.m. peto & associates ltd.

SOIL SITE INVESTIGATION
AT
PROPOSED HWY 8 CROSSING
OF GRAND RIVER NEAR FREEPORT ONT.
FOR
DEPARTMENT of HIGHWAYS

OUR JOB No. 58119
DATE 30 OCT 19
CLIENTS PLAN No. 62781-1
PER C.L.V.

E. M. PETO

Appendix D

Foundation Comparison

COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

Foundation Element	Footings on Native Soil	H-Piles	Caissons	Micropiles
All	*Advantages:* i. Lower unit cost compared to pile foundations. ii. High geotechnical resistances available on the very dense native soils. *Disadvantages:* i. An integral abutment design is not an available option ii. Possible dewatering requirements iii. Possible scour and undermining problems for piers adjacent to the river. **RECOMMENDED**	*Advantages:* i. High geotechnical resistance available. ii. Relatively short pile lengths required since very dense soils lie at shallow depth. iii. Will allow for the construction of an integral abutment structure. iv. Independent of groundwater conditions. *Disadvantages:* i. Higher unit cost compared to footings. ii. Predrilling may be required in order to install the piles to adequate length. **NOT RECOMMENDED**	*Advantages:* i. High bearing resistances available on very dense soil or bedrock *Disadvantages* i. Difficulties in advancing the caisson shaft to the required depth. ii. Difficulties in obtaining a seal below the liner to pour concrete in dry conditions. iii. Higher cost than other systems. **NOT RECOMMENDED**	*Advantages:* i. High bearing resistances available for micropiles founded in bedrock. ii. Less susceptible to soil disturbance and presence of obstructions than other foundation systems under consideration. *Disadvantages* i. A proprietary system with a limited number of suppliers. ii. Generally much more expensive than the other systems. **NOT RECOMMENDED**

Appendix E

Special Provisions

The following Special Provisions are referenced in this report:

105S10
Amendment to OPSS 206, December 1993
902S01
903S01

Suggested text for a NSSP on Pile Installation should contain the following:

"The soil overlying the bedrock contains cobbles and boulders. The presence of cobbles and boulders will potentially have an impact on the installation of driven piles at the site. Some possible impacts that must be taken into consideration include, but are not necessarily limited to:

- *The pile tips must be protected through the use of rock points*
- *The cobbles and boulders may impede the driving of the piles resulting in more arduous driving to reach bedrock*
- *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"*

Appendix F

Drawings

MINISTRY OF TRANSPORTATION, ONTARIO PR-0-707 88-05 PLOT SCALE 1:1

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

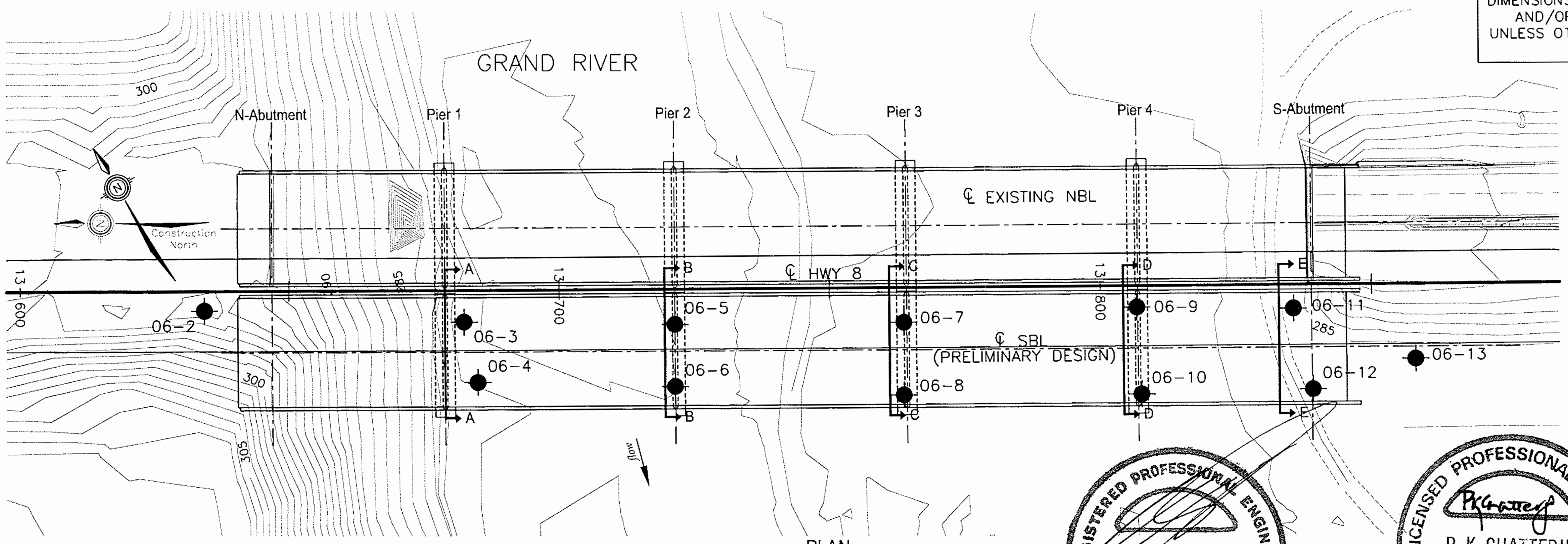
CONT No
GWP No.277-97-00

SHEET

GRAND RIVER CROSSING SBL
HWY 8 WIDENING
KITCHENER
BOREHOLE LOCATIONS AND SOIL STRATA

MORRISON HERSHFIELD

THURBER ENGINEERING LTD.
GEOTECHNICAL • ENVIRONMENTAL • MATERIALS

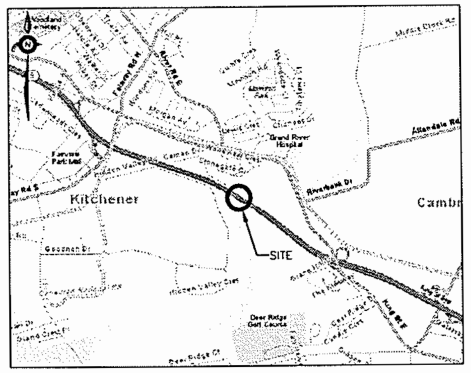


PLAN

5 0 10 20m

REGISTERED PROFESSIONAL ENGINEER
A. E. GORMAN
June 27/07
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
June 27/07
PROVINCE OF ONTARIO

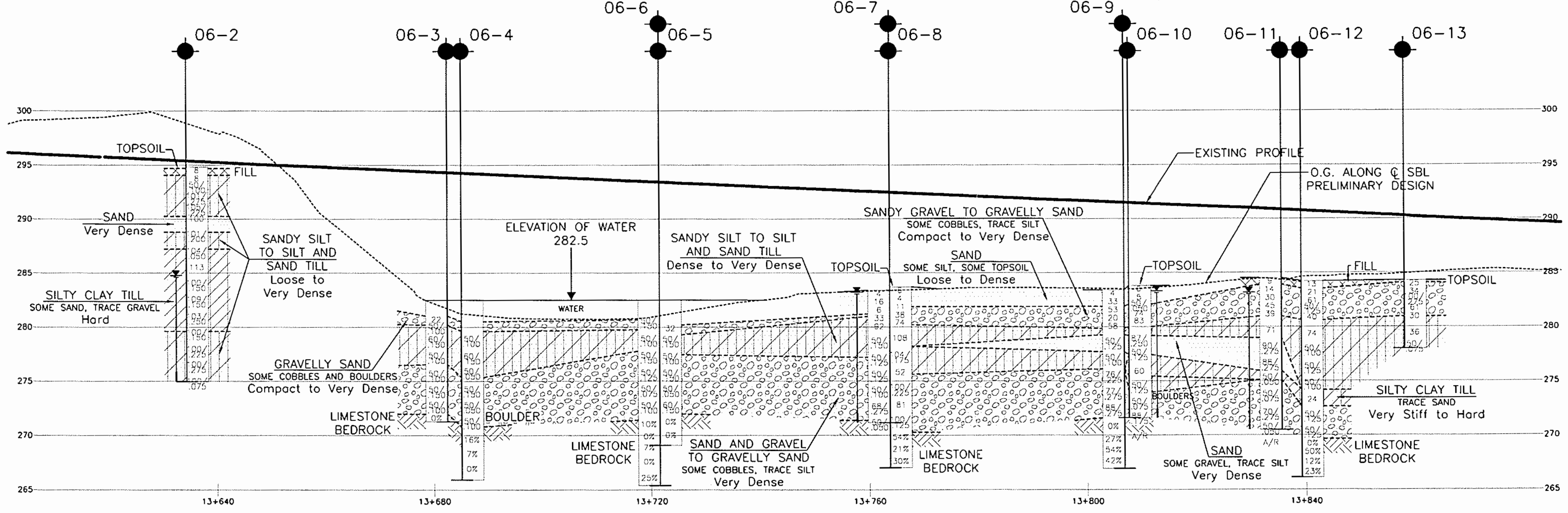


KEYPLAN

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
06-2	294.83	4 809 407.87	230 460.97
06-3	282.50	4 809 380.57	230 500.59
06-4	282.50	4 809 369.73	230 496.77
06-5	282.50	4 809 359.26	230 533.45
06-6	282.50	4 809 349.51	230 527.36
06-7	283.35	4 809 337.06	230 569.26
06-8	283.72	4 809 325.66	230 562.08
06-9	283.42	4 809 316.39	230 607.09
06-10	283.79	4 809 302.31	230 599.22
06-11	284.50	4 809 300.78	230 631.30
06-12	284.19	4 809 286.20	230 626.37
06-13	282.31	4 809 280.86	230 645.36



PROFILE ALONG PRELIMINARY SBL

5 0 10 20m HOR 1:1000
2.5 0 5 10m VERT 1:500

-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. 40P8-143

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS	DATE	BY	DESCRIPTION			
DESIGN AEG	CHK PKC	CODE	LOAD	DATE JAN 2007		
DRAWN JHL	CHK PKC	SITE	STRUCT	DWG		

DRAWING NAME: TE079381GrandRiver
CREATED: DEC 06 MODIFIED:

FILENAME: D:\Job Files\19\479\38 Hwy 8\TE079381Grand River.dwg
PLOTDATE: Jun 18, 2007 - 8:38am

METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

CONT No
GWP No.277-97-00

GRAND RIVER CROSSING SBL
HWY 8 WIDENING
KITCHENER
SOIL STRATA

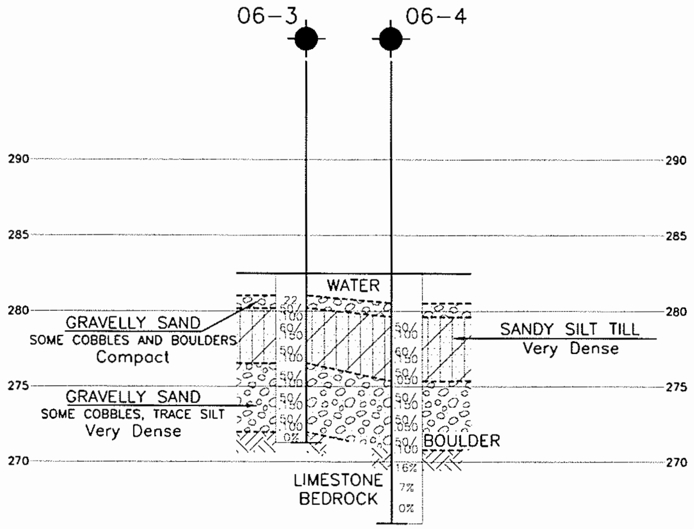
SHEET

MORRISON HERSHFIELD

THURBER ENGINEERING LTD.
GEOTECHNICAL • ENVIRONMENTAL • MATERIALS

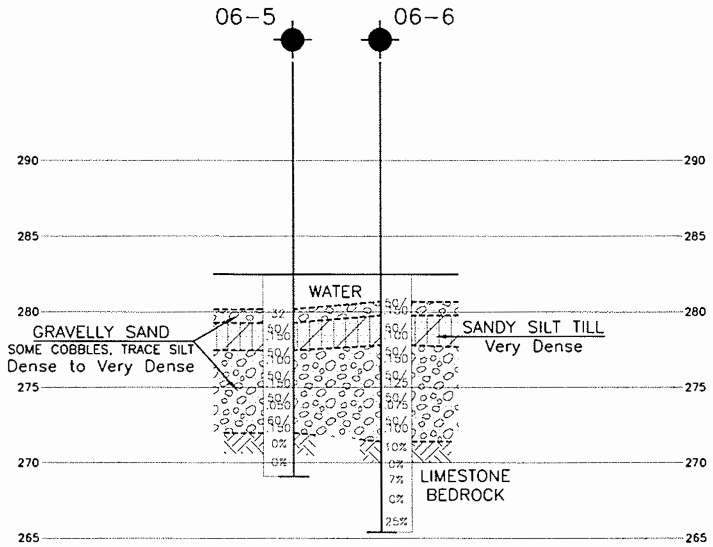
A. E. GORMAN
June 27/07

P. K. CHATTERJI
Jun 27/07



SECTION A-A

HOR 1:1000 — 5 0 10 20m
VERT 1:500 — 2.5 0 5 10m



SECTION B-B

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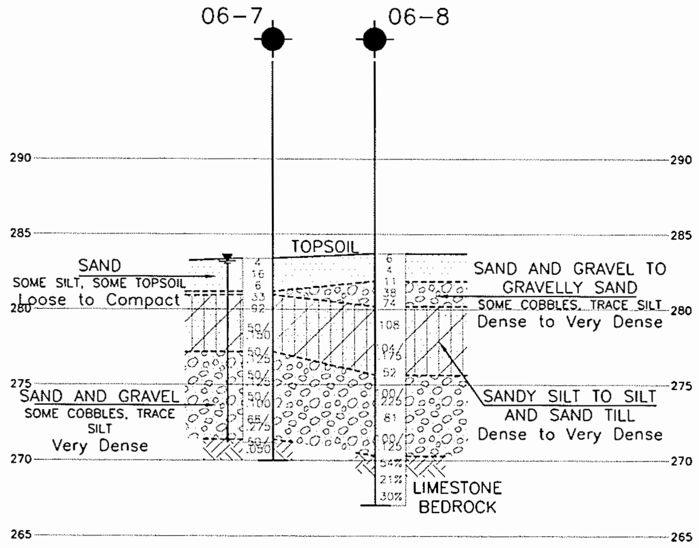

SECTION C-C

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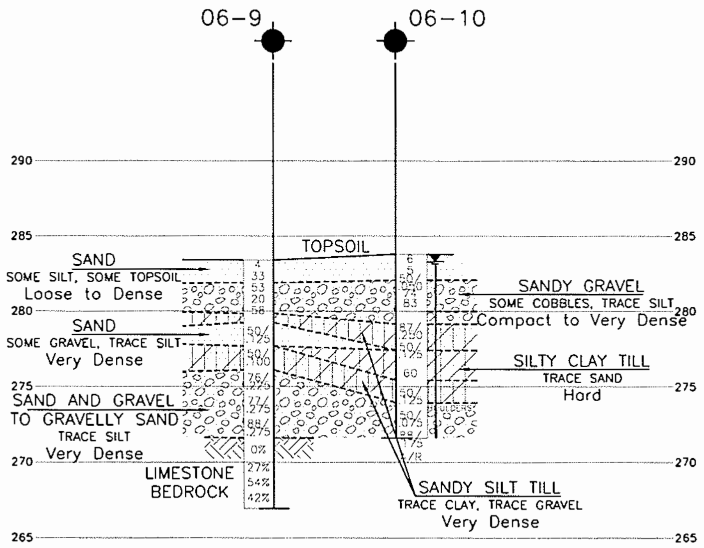
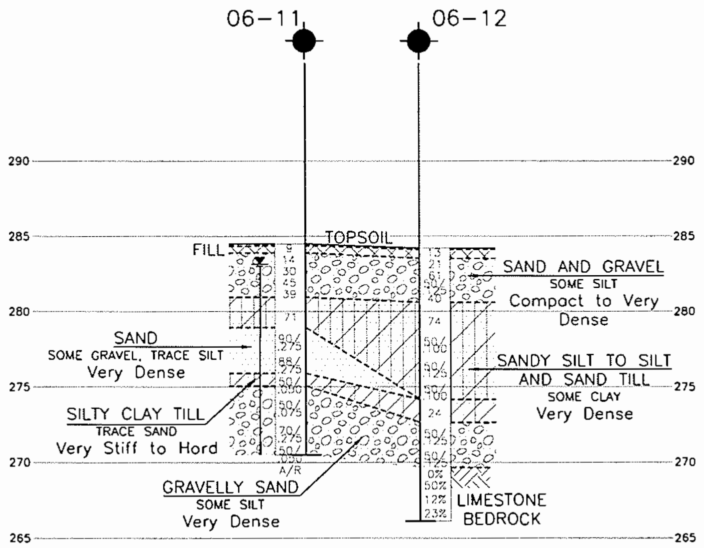
VERT 1:500 — 2.5 0 5 10m



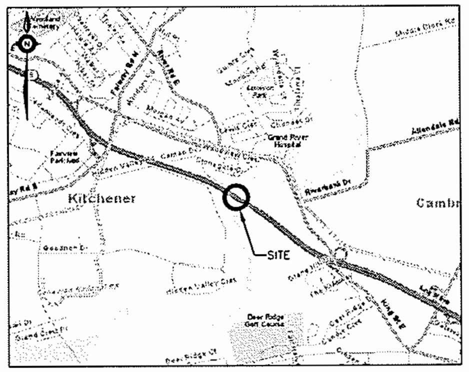
SECTION D-D

HOR 1:1000 — 5 0 10 20m
VERT 1:500 — 2.5 0 5 10m



SECTION E-E

HOR 1:1000 — 5 0 10 20m
VERT 1:500 — 2.5 0 5 10m



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
06-2	294.83	4 809 407.87	230 460.97
06-3	282.50	4 809 380.57	230 500.59
06-4	282.50	4 809 369.73	230 496.77
06-5	282.50	4 809 359.26	230 533.45
06-6	282.50	4 809 349.51	230 527.36
06-7	283.35	4 809 337.06	230 569.26
06-8	283.72	4 809 325.66	230 562.08
06-9	283.42	4 809 316.39	230 607.09
06-10	283.79	4 809 302.31	230 599.22
06-11	284.50	4 809 300.78	230 631.30
06-12	284.19	4 809 286.20	230 626.37
06-13	282.31	4 809 280.86	230 645.36

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GEOCRES No. 40P8-143

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS					
DATE	BY	DESCRIPTION			
DESIGN AEG	CHK PKC	CODE	LOAD	DATE JAN 2007	
DRAWN JHL	CHK PKC	SITE	STRUCT	DWG	

MINISTRY OF TRANSPORTATION, ONTARIO PH-D-707 88-05 PLOT SCALE 1:1

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CREATED: DEC 06
MODIFIED:

FILENAME: D:\Job Files\19\479\38 Hwy 8\TED79381Grand River.dwg
PLOTDATE: Jun 18, 2007 - 8:37am

Appendix G

Site Photographs

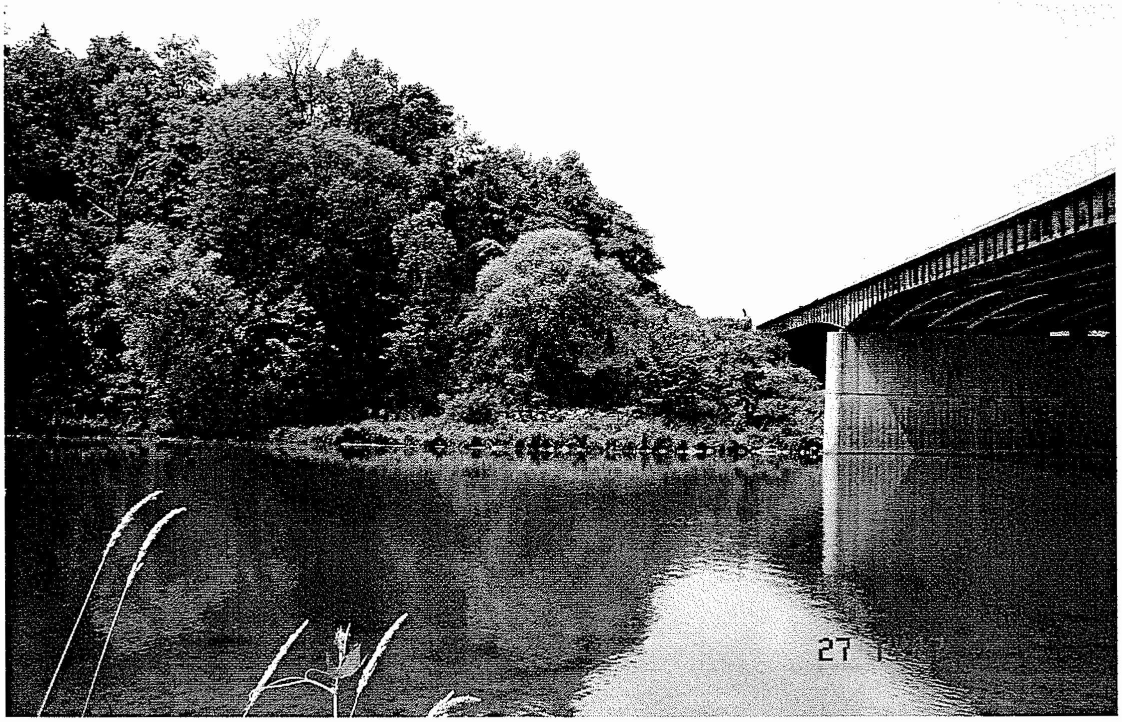


Photo 1, July 2006 – Looking from flood plain on south side of Grand River towards North Abutment and Piers 1 and 2. Existing bridge on right side of photo.

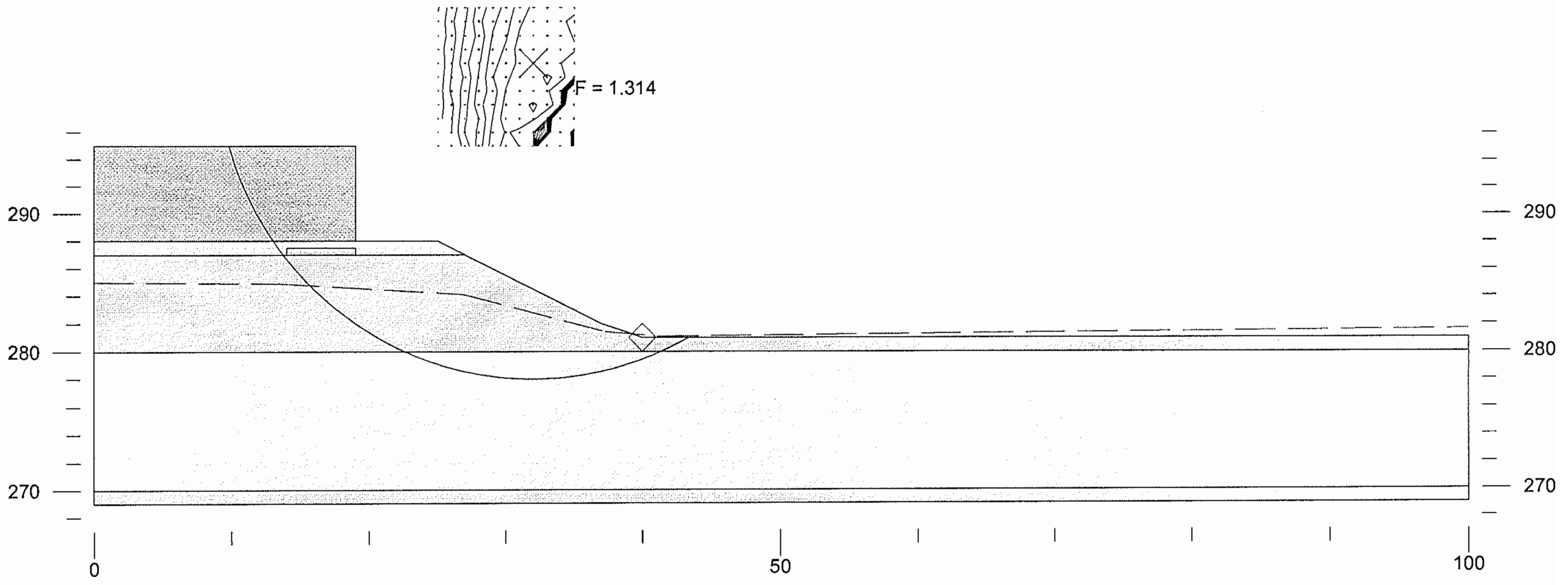
Photo 2, July 2006 – Looking from flood plain on south side of Grand River towards South Abutment. Existing bridge on left side of photo.

Appendix H

Slope Stability Output

	Gamma kN/m3	C kPa	Phi deg	Piezo Surf.
Fill	22	0	33	1
Sandy silt	21	0	35	1
Footing	600	30000	0	1
Silty clay	20	20	33	1
Sandy silt till	22	0	36	1
Bedrock	24	10000	0	1

Thurber Engineering Ltd. - Toronto
19-479-38
Hwy 8 over Grand River
January 5, 2007
North forward slope



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