

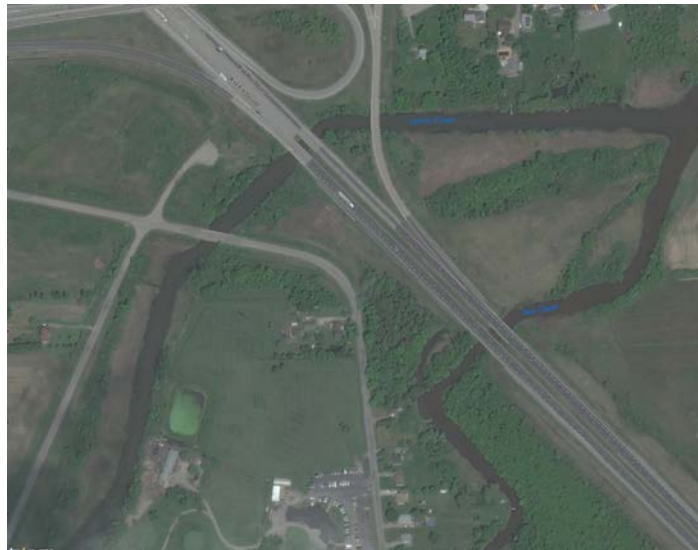


December 15, 2014

## PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

### Tee Creek Bridges, QEW Structure Replacements at Black Creek, Lyons Creek, Seventh Street and Tee Creek Regional Municipality of Niagara GWP 2177-08-00

**Submitted to:**  
URS Canada Inc.  
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REPORT

**GEOCREs No. 30M3-280**

**Report Number:** 12-1111-0088-4

**Distribution:**

3 Copies - MTO - Central Region  
1 Copy - MTO – Foundations Section  
2 Copies - URS Canada Inc.  
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REPORT - TEE CREEK BRIDGES**

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# **PART A**

**PRELIMINARY FOUNDATION INVESTIGATION REPORT  
TEE CREEK BRIDGES**

**QEW STRUCTURE REPLACEMENTS AT BLACK CREEK, LYONS CREEK,  
SEVENTH STREET AND TEE CREEK, REGIONAL MUNICIPALITY OF  
NIAGARA**

**G.W.P 2177-08-00**



### 1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by URS Canada Inc. (URS) on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary foundation engineering services for the replacement/rehabilitation of seven structures (one at Seventh Street, and two at each of Lyons Creek, Tee Creek and Black Creek) on the Queen Elizabeth Way (QEW) in the Regional Municipality of Niagara, Ontario.

The terms of reference and scope of work for the foundation engineering services are outlined in MTO's Request for Proposal (RFP) for Assignment No. 2011-E-0045 dated June 2011, and in Section 5.8 of the *Technical Proposal* for this assignment and MTO's Additional scope of work for Tee Creek Bridges over QEW dated October 5, 2012. The Scope of Work for the foundation engineering services in support of the replacement/rehabilitation of the Tee Creek bridges is presented in Golder's scope change letter, dated October 17, 2012.

This report addresses the results of the subsurface investigation carried out for the proposed replacement/rehabilitation of the Tee Creek bridges.

This preliminary Foundation Investigation Report is for planning purposes only and the Design/Build proponent shall satisfy himself as to the sufficiency of the available information and supplement the information as needed to meet the requirements for detail design.

### 2.0 SITE DESCRIPTION

The Tee Creek bridges carry the QEW southbound (Fort Erie bound) and northbound (Toronto bound) over Tee Creek, which is south of Lyons Creek Road, north of the Town of Fort Erie, within the Regional Municipality of Niagara, Ontario.

Tee Creek is a relatively shallow watercourse, approximately 20 m wide, and flows from southwest to northeast with the high water level at the existing bridge site at about Elevation 171.2 m, as referenced in a previous borehole investigation report<sup>1</sup>.

In general, the topography along this section of the QEW is relatively flat. The area is sparsely treed and is surrounded by residential/recreational properties. The existing ground surface at the borehole locations on the QEW ranges between Elevations 174.2 m and 174.0 m referenced to Geodetic datum.

The existing Tee Creek bridges approach embankments are up to about 4.5 m to 5 m high at the north and south approaches for both bridges.

The existing Tee Creek northbound lane (NBL) structure is a variable depth cast-in-place concrete T-beam bridge with a 19.66 m centre span and two 5.26 m cantilevered end spans, for a total length of 30.18 m. The bridge has a roadway width of 11.6 m measured between barrier walls. Based on a review of the design drawings for the NBL structure, the existing piers at this structure are supported on piles. It is our understanding that this structure will be replaced.

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<sup>1</sup> Geocres No. 30M02-174



The Tee Creek southbound lane (SBL) structure is a three span AASHTO pre-stressed concrete girder bridge with a 19.66 m centre span and 9.83 m end spans, and a total length of 39.32 m. The roadway width is 13 m measured between the inside faces of barrier walls. The existing pier caps and foundations were salvaged when the original variable depth T-Beam superstructure was replaced in 1981 with the current AASHTO girder superstructure. New abutments were constructed and supported on steel HP 310x79 piles driven to bedrock, to support the new end spans. The bridge is in overall good condition and will be maintained and rehabilitated.

### 3.0 INVESTIGATION PROCEDURES

#### 3.1 Previous Investigations

As part of the QEW construction and widening in the vicinity of Tee Creek in the 1980's a subsurface investigation was carried and the results of the investigation are reported in:

**MTO GEOCREs No. 30M03-174:** Report titled "Foundation Investigation Report for Tee Creek/Q.E.W Crossing S.B. Bridge (Fort Erie Bound) W. P. 45-80-01 Site No. 34-67 District 4, Hamilton", prepared by Department of Highways – Ontario, dated September 30, 1980.

A total of three boreholes, designated as Boreholes 1, 2, and 3 were drilled near the Tee Creek SBL bridge (Fort Erie Bound) at the approximate locations shown in the Drawing 1 and the Record of Borehole sheets for these three boreholes are attached in Appendix A.

The GEOCREs sourced boreholes used in this report have been re-numbered to include the MTO GEOCREs No. followed by the original borehole designation to differentiate them from the boreholes advanced for the current investigation. Therefore, the boreholes from MTO GEOCREs 30M03-174 have been renamed to 174-X, where X is the original borehole number.

#### 3.2 Current Investigation

The field work for this subsurface investigation was carried out between June 21 and July 5, 2013, during which time four Boreholes (13-07 to 13-10) were advanced behind the approach slabs of the existing structures. The boreholes were advanced using a track-mounted CME-55 drill rig supplied and operated by Geo-Environmental Drilling Inc. of Milton, Ontario. The boreholes were advanced through the overburden using 108 mm Inside Diameter (I.D.) hollow stem augers. Soil samples were obtained at 0.75 m and 3.0 m intervals of depth using a 50 mm Outside Diameter (O.D.) split-spoon sampler driven by an automatic hammer in accordance with the Standard Penetration Test (SPT) procedure (ASTM D1586, Standard Test Method for Standard Penetration Test). In situ field vane tests were carried out within lacustrine deposits using an MTO 'N' size Field Vane.

The borehole at the location of the southwest abutment of the Tee Creek SBL structure (Borehole 13-08) was advanced to a depth of 6.7 m below the QEW pavement surface, terminating at about Elevation 167.5 m to confirm the thickness of the embankment fill at the southwest abutment. The subsurface information at the southwest abutment is supplemented by the information obtained from Borehole 174-2, drilled during the previous 1980 investigation. Boreholes 13-07, 13-09 and 13-10 were advanced to auger refusal on inferred bedrock at depths between 30.1 m and 32.2 m below the QEW pavement surface between Elevations 144.1 m and 141.9 m.



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The groundwater conditions were observed within the hollow stem augers during and upon completion of the drilling operations and water level readings are indicated on the Record of Borehole sheets contained in Appendix A. All boreholes were backfilled with bentonite pellets and capped with asphalt patches upon completion, in accordance with Ontario Regulation 903 (as amended).

The field work was supervised on a full-time basis by members of Golder's staff who located the boreholes in the field, completed utility clearances, directed the drilling, sampling, and in situ testing operations, and logged the boreholes. The soil samples were identified in the field, placed in labelled containers and transported to Golder's laboratory in Mississauga for further visual examination and then to Golder's Cambridge laboratory for testing. Index and classification tests consisting of water content and organic determinations, Atterberg limits and grain size distribution were carried out on selected soil samples. The geotechnical laboratory testing was completed according to applicable MTO LS standards. The as-drilled borehole locations and ground surface elevations were determined in the field by Callon Dietz, Ontario Land Surveyors. The borehole locations (referenced to the MTM NAD83 co-ordinate system) and ground surface elevations (referenced to Geodetic datum) and drilled depths are summarized below and are shown on Drawing 1.

Foundation Element	Borehole Number	Location (MTM NAD83)		Ground Surface Elevation (m)	Borehole Depth (m)
		Northing (m)	Easting (m)		
Tee Creek Bridge SBL – North Abutment	13-07	4,765,574.3	336,686.4	174.2	32.2
Tee Creek Bridge SBL – South Abutment	13-08	4,765,532.1	336,722.7	174.2	6.7
Tee Creek Bridge NBL – North Abutment	13-09	4,765,591.0	336,717.3	174.0	32.1
Tee Creek Bridge NBL – South Abutment	13-10	4,765,559.9	336,740.9	174.2	30.1

The boreholes from the 1980 investigation are shown on Drawing 1 and the locations should be considered approximate as the borehole locations were plotted on the current drawing, using the QEW centreline and local site features as references.

## 4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

### 4.1 Regional Geology

This section of QEW is located in the Haldimand Clay Plain physiographic region as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, 1984)<sup>2</sup>.

<sup>2</sup> Chapman, L. J. and Putnam, D. F., 1984. *The Physiography of Southern Ontario*, Ontario Geological Society, Special Volume 2, Third Edition.



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The Haldimand Clay Plain physiographic region is a broad undulating plain of glaciolacustrine surface sediments which covers an area of about 3,500 square km. The region mostly contains lacustrine clay deposits overlying clay till which is turn underlain by shales and dolostone of the Salina formation.

### 4.2 Subsurface Conditions

As part of this subsurface investigation, four boreholes were advanced in the vicinity of the existing Tee Creek bridges. The detailed subsurface soil and groundwater conditions as encountered in the boreholes advanced during the current field investigation, together with the results of the in situ and laboratory tests carried out on selected soil samples are presented on the Record of Borehole sheets contained in Appendix A. Copies of the previous Record of Borehole sheets from the 1980 investigation (174-1, 174-2 and 174-3) are also provided in Appendix A and are used to supplement the most recent investigation. The results of geotechnical laboratory testing from the current investigation are also presented on Figures B1 to B7 contained in Appendix B. The results of the in situ field tests (i.e. SPT 'N'-values and field vane results) as presented on the Record of Borehole sheets and in Section 4.2 are uncorrected. The stratigraphic boundaries shown on the Record of Borehole sheets and on the interpreted stratigraphic profiles on Drawing 1 are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations. The interpreted stratigraphy shown on Drawing 1 is a simplification of the subsurface conditions.

In summary, the subsoil conditions encountered at the site consist of cohesionless fill and organic layers overlying a relatively thick deposit of clayey silt which has occasional pockets of cohesive till within its lower portion. The cohesive deposit is underlain by deposits of silt to silty sand. Boreholes 13-07, 13-09 and 13-10 were terminated upon practical refusal to advance the augers on inferred bedrock.

Boreholes 174-2 and 174-3 from the 1980 investigation encountered a deposit of sand and gravel above the inferred bedrock. The bedrock in Borehole 174-2 was confirmed to be dolomite by rock coring.

A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

#### 4.2.1 Asphalt/Concrete

An approximately 200 mm and 225 mm layer of asphalt was encountered immediately below the road level at about Elevation 174.2 m in Boreholes 13-07 and 13-10 that were advanced through the existing pavement structure. The asphalt layer is underlain by 300 mm of concrete in Borehole 13-10.

#### 4.2.2 Cohesionless Fill

A 1.0 m to 1.4 m thick layer of cohesionless fill comprised of brown to grey sand and gravel, sand, and silty sand was encountered below the asphalt/concrete layer(s) in Boreholes 13-07 and 13-10 and at the ground surface in Borehole 13-09 and extended to depths ranging between 1.4 m and 1.5 m (Elevations 172.8 m and 172.6 m). The fill generally contained trace to some silt and trace clay. An approximately 1.4 m and 1.3 m thick layer of cohesionless fill comprised of silty sand and gravel was also encountered at about Elevations 174.2 m and 174.1 m below the existing ground surface in Boreholes 174-1 and 174-2, respectively, from the 1980 investigation.





## PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT - TEE CREEK BRIDGES

The SPT 'N'-values measured within the cohesionless fill deposit range from 5 blows to 20 blows per 0.3 m of penetration, indicating loose to compact relative density.

The natural water content measured on six samples of the cohesionless fill encountered during the current investigation ranges from about 4 per cent to 16 per cent. The results of grain size distribution tests completed on two samples of the fill are shown on Figure B1 in Appendix B.

### 4.2.3 Cohesive Fill

An approximately 2.7 m to 4.5 m thick deposit of cohesive fill, comprised of clayey silt to silty clay was encountered below the ground surface in Borehole 13-08 and underlying the cohesionless fill in Boreholes 13-07, 13-09 and 13-10. The fill deposit generally contains trace to some sand and variable amounts of organics. The fill extended to depths ranging between 4.1 m and 5.6 m (Elevations 170.1 m and 168.4 m).

The SPT 'N'-values measured within the cohesive fill range from 4 blows to 11 blows per 0.3 m of penetration. Two in situ field vane tests carried out within the fill measured undrained shear strengths greater than 96 kPa. The SPT 'N'-values and the field vane testing suggest that the cohesive fill has a soft to stiff consistency.

The natural water content measured on seventeen samples of the cohesive fill range from about 7 per cent to 118 per cent, but generally ranges from 20 per cent to 35 per cent, the highest measured water content being attributed to the presence of trace organics and wood fragments. The organic content measured on a sample of the fill from Borehole 13-08 is about 6 per cent. The results of grain size distribution tests completed on two samples of the cohesive fill are shown on Figure B2 in Appendix B. Atterberg limits tests were carried out on four samples of the cohesive fill deposit and measured liquid limits ranging from about 19 per cent to 46 per cent, plastic limits ranging from about 12 per cent to 23 per cent and plasticity indices ranging from about 7 per cent to 23 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure B3 in Appendix B, and indicate that the material is classified as clayey silt of low plasticity to silty clay of intermediate plasticity.

### 4.2.4 Clayey Silt to Silty Clay to Clay

A relatively thick deposit of cohesive soil comprised of brown to grey clayey silt to silty clay to clay was encountered below the cohesive fill in Boreholes 13-07 to 13-10, below the creek bed in Borehole 174-1 and underlying the cohesionless fill in Boreholes 174-2 and 174-3. The top of the deposit was encountered between Elevations 172.9 m and 168.4 m. Boreholes 13-08 and 174-1 were terminated within this deposit at Elevations 167.5 m and 157.0 m, respectively, penetrating the clayey silt to silty clay to thicknesses of about 2.2 m and 13.1 m. The thickness of the deposit in the other boreholes varies between about 18.7 m and 21.8 m. A sandy silt pocket was encountered within the deposit in Borehole 13-09 and extends from a depth of 18.1 m to 20.1 m (Elevations 155.9 m to 153.9 m). The upper portion of the deposit as encountered in Borehole 174-2 contains trace organics and the deposit generally contains trace to some sand and discontinuous gravel and silt seams.

The SPT 'N'-values measured within this deposit range from 1 blow to 10 blows per 0.3 m of penetration. In situ field vane tests carried out within this deposit during the current investigation measured undrained shear strengths ranging from about 53 kPa to greater than 96 kPa, with sensitivity values between 1 and 3. Field vane, as well as laboratory vane and laboratory unconfined compression tests performed within this deposit during the 1980 investigation, measured undrained shear strength ranging between about 10 kPa and greater than 96 kPa,



but typically between about 30 kPa and 60 kPa. The laboratory and field vane test results indicate that the clayey silt to silty clay to clay deposit has a firm to stiff consistency.

An SPT 'N'-value of 5 blows per 0.3 m of penetration was measured within the sandy silt pocket, indicating a loose relative density.

The natural water content measured for thirty-four samples of the cohesive deposit from the current investigation ranges from about 23 per cent to 36 per cent. The water content for a sample of the sandy silt from the pocket in Borehole 13-09 is about 22 per cent. The results of grain size distribution tests completed on four samples of the clayey silt to silty clay and on a sample of the sandy silt pocket from the current investigation are shown on Figure B4 and Figure B5 in Appendix B, respectively. Atterberg limits tests were carried out on twelve samples from the cohesive deposit and measured liquid limits ranging from about 24 per cent to 55 per cent, plastic limits ranging from about 17 per cent to 26 per cent, and plasticity indices ranging from about 11 per cent to 29 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure B6 in Appendix B, and indicate that the material is classified as clayey silt of low plasticity to clay of high plasticity.

### 4.2.5 Silt to Silty Sand

A deposit of cohesionless soils comprised of silt to sandy silt to silty sand was encountered underlying the cohesive deposit in all boreholes except Boreholes 13-08 and 174-1 which were terminated within the cohesive deposit. The thickness of the deposit varies between 4.5 m and 9.1 m and the deposit extends to auger refusal that was encountered at depths ranging from 30.1 m to 32.2 m (Elevations 144.1 m to 141.9 m) in Boreholes 13-07, 13-09 and 13-10.

The SPT 'N'-values measured within the silt to silty sand deposit range from 5 blows to 61 blows per 0.3 m of penetration, indicating a loose to very dense relative density.

The natural water content measured for eight samples of silt to silty sand from the current investigation ranges from about 17 per cent to 25 per cent. The results of grain size distribution tests completed on five samples of this deposit are shown on Figure B7 in Appendix B.

### 4.2.6 Sand and Gravel

A 1.7 m and 4.3 m thick deposit of sand and gravel was encountered underlying the silt to silty sand deposit in Boreholes 174-2 and 174-3 (1980 investigation), and extended to depths of about 29.6 m and 31.7 m (Elevations 144.6 m and 142.4 m), respectively.

An SPT 'N'-value of 17 blows per 0.3 m of penetration was measured within the deposit in Borehole 174-3, indicating a compact relative density.

### 4.2.7 Bedrock/Refusal

In all boreholes except Boreholes 13-08 and 174-1, the bedrock surface is inferred from refusal to further penetration of the hollow stem augers to be at depths between 29.6 m and 32.2 m below ground surface, corresponding to Elevations 144.6 m and 141.9 m.

Bedrock was confirmed by coring in Borehole 174-2 at about 29.6 m below the ground surface (Elevation 144.6 m) and about 3.2 m of core samples were recovered. The bedrock is described as very hard



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dolomite with occasional soft gypsum and shale interlayers. The total core recovery ranged from 70 per cent to 100 per cent.

### 4.3 Groundwater Conditions

The soil samples obtained in the boreholes were generally moist to wet. The water levels observed in the open boreholes during and upon completion of drilling are shown on the Record of Borehole sheets and are summarized below:

Borehole	Ground Surface Elevation (m)	Depth to Water Level (m)	Groundwater Elevation (m)	Date
13-07	174.2	5.1	169.1	July 5, 2013
13-08	174.2	Dry	--	July 4, 2013
13-10	174.2	5.8	168.4	July 3, 2013

The water levels presented above and on the Record of Borehole sheets may not represent stabilized groundwater conditions at the time of the investigation.

The water level was measured in Borehole 174-1 at Elevation 171.2 m (1 m below ground surface) on April 2, 1980.

The groundwater and creek water levels are expected to fluctuate seasonally in response to changes in precipitation and snow melt, and are expected to be higher during the Spring season and periods of precipitation.



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### 5.0 CLOSURE

This Preliminary Foundation Investigation Report was prepared by Mr. Al Varshoi, M.E.Sc., and Mr. Mehdi Mostakhdemi, P.Eng., a geotechnical engineer with Golder, and reviewed by Mr. Kevin Bentley, M.Sc., P.Eng. an Associate with Golder. Mr. Jorge M.A. Costa, P.Eng., a Designated MTO Foundations Contact and Principal with Golder, conducted an independent review of this report.

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**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN  
REPORT - TEE CREEK BRIDGES**

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# **PART B**

**PRELIMINARY FOUNDATION DESIGN REPORT  
TEE CREEK BRIDGES**

**QEW STRUCTURE REPLACEMENTS AT BLACK CREEK, LYONS CREEK,  
SEVENTH STREET AND TEE CREEK, REGIONAL MUNICIPALITY OF  
NIAGARA  
G.W.P 2177-08-00**



## 6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

### 6.1 General

This section of the report provides preliminary foundation design recommendations for the proposed replacement of the existing QEW NBL bridge and rehabilitation of the existing QEW SBL bridge over Tee Creek. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during this preliminary subsurface investigation. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the preliminary design of the structure foundations. Further investigation and analysis will be required during detail design.

Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project, and for which special provisions may be required in the Contract Documents. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

This preliminary Foundation Design Report is for planning purposed only and the Design/Build proponent shall satisfy himself as to the sufficiency of the available information and supplement the information as needed to meet the requirements for detail design. The Design/Build proponent is solely responsible for selecting the appropriate foundation alternatives for replacement/rehabilitation of the Tee Creek Bridges.

### 6.2 Foundation Options

Based on the planning study completed by URS to date for the rehabilitation/replacement of the Tee Creek bridges, it is understood that the future works will include replacement of the existing NBL bridge with a single span structure with the new abutments to be placed behind the existing piers and the existing SBL bridge will be maintained and rehabilitated. It is further understood that re-alignment and/or grade change of the QEW at the location of the bridges are not under consideration at this time. The approach embankment fill immediately behind the new NBL structure will be up to 4.5 m thick to match the grade of the existing embankment behind the present abutment.

Based on the subsurface conditions at this site, both shallow and deep foundation options have been considered for support of the abutments for the new Tee Creek NBL bridge. The as-built information for the piers of the existing NBL and SBL bridges and details of pile foundations are unknown at this time. From existing drawings, the new abutments for the reconstructed SBL bridge are each supported on 9-HP310x79 steel piles driven to bedrock. A copy of the foundation drawings for the reconstructed SBL bridge is provided in Appendix C.

The location of the new abutment foundations for the NBL bridge should be selected to avoid interference from the existing foundation elements.

A summary of the advantages and disadvantages associated with each option is provided below, and a comparison of the alternative foundation options based on advantages, disadvantages, risks and approximate costs is provided in Table 1 following the text of this report.

- **Spread footings:** Due to the presence of very soft to stiff clayey deposits up to about 25 m thick, the estimated preliminary geotechnical resistance and reaction are not sufficient to support the replacement



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structures on strip or spread footings founded at relatively shallow depths. Therefore, spread footings are not considered as a feasible option and is not discussed further in this report.

- **Steel H-piles driven to found on the bedrock:** Driven steel H-piles are suitable and feasible for support of new abutments (and would permit integral abutment design) and associated wingwalls/retaining walls at this site and allow for integral abutment design. It is assumed that the new pile caps would be “perched” within the approach embankments above the floodplain grade, thus minimizing the depth of excavation and associated requirements for temporary protection systems and dewatering. There is a relatively minor risk associated with penetrating through or the piles “hanging up” on cobbles or boulders within the sand and gravel deposit above the bedrock (although further investigation is recommended in this regard at the detail design stage).
- **Steel pipe piles driven to found on the bedrock:** Driven steel pipe piles could also be considered as a deep foundation option for support of new abutments (and would permit integral abutment design) and associated wing walls/retaining walls at this site. It is assumed that the abutment pile caps would be “perched” within the QEW approach embankments, minimizing the depth of excavation and associated requirements for temporary protection and dewatering. Pipe piles are considered to have a slightly higher risk than H-piles for “hanging up” or being deflected away from their vertical or battered orientation due to the presence of cobbles and/or boulders within the sand and gravel deposit above the bedrock at this site. It should be noted, however, that MTO does not readily accept the use of pipe piles or integral abutment design.
- **Caissons founded in the bedrock:** Caissons founded in the bedrock are feasible for support of the new abutments (although they would preclude integral abutments) at this site. Temporary or permanent liners would be required during caisson construction given the risk of running/flowing soil when excavating through the water-bearing cohesionless deposits. In addition, coring and/or churn drilling techniques are expected to be required to penetrate into the bedrock to the target founding levels.

The following sections provide recommendations for driven steel H-pile or pipe pile foundations, and caisson foundations to support the proposed bridge replacement and widening. Based on the subsurface conditions at the site and the above considerations, and considering the satisfactory performance of the existing structure and its foundations, the preferred option from a geotechnical/foundations perspective is to support the abutments for the new structure on steel H-piles driven to found on the bedrock, in an integral abutment configuration. Deep foundations whether H-piles or caissons, should be constructed in accordance with OPSS 903 (Deep Foundations).

## 6.3 Driven Steel H-Pile or Steel Pipe (Tube) Foundations

### 6.3.1 Founding Elevations

The new abutments and associated wingwalls may be supported on steel H-piles or steel pipe (tube) piles driven to found on or into the bedrock. The surface elevation for the bedrock and its strength characteristics varies in the boreholes, and further investigation will be required at the detail design stage to confirm these preliminary founding elevations. The following pile tip elevations may be used for preliminary design purposes, assuming termination on or just into the bedrock:





## PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT - TEE CREEK BRIDGES

Structure	Foundation Element	Borehole Number(s)	Estimated Design Pile Tip Elevation (m)
Fort Erie Bound	Tee Creek Bridge SBL – North Abutment	13-07	Existing Piles Driven to Bedrock*
	Tee Creek Bridge SBL – South Abutment	13-08	Existing Piles Driven to Bedrock*
Toronto Bound	Tee Creek Bridge NBL – North Abutment	13-09	141.0
	Tee Creek Bridge NBL – South Abutment	13-10	143.5

\*For assessment of existing piles

The pile caps should be placed at a minimum depth of 1.2 m below adjacent final grade for frost protection purposes. The elevations of the underside of the new pile caps are not known at this time.

For the installation of steel H-piles or steel pipe piles, consideration must be given to the potential presence of cobbles and boulders within the soil deposits. In this regard, steel H-piles are preferred over steel pipe piles as pipe piles are considered to pose a higher risk of “hanging up” or being deflected away from their vertical or battered orientation during installation, due to their larger end area. The piles should be reinforced at the tip with driving shoes to reduce the potential for damage to the piles during driving.

As discussed further in Section 6.6 (Construction Considerations), vibration monitoring is not anticipated to be required during deep foundation construction activities, either for the existing bridges or for the nearest buildings.

The long-term settlement associated with the consolidation of the soft to stiff clayey deposits will induce a downward movement of the soils adjacent to the piles and negative skin friction will develop along portions of the pile shafts embedded within or above the soft to stiff clayey layer. For preliminary design purposes, factored downdrag loads of 600 kN for HP 310x110 piles (assuming a negative skin friction factor of 0.25) should be considered to act on piles. The structural capacity of the pile must be sufficient to withstand the combined permanent load plus the downdrag load (if the downdrag loads are greater than the live loads). The magnitude and duration for the settlement and the downdrag loads should be reassessed during detail design, following completion of additional investigation and testing.

Alternatively, the embankment could be constructed to design grade and preloaded for a period of approximately nine months (with the duration to be confirmed during detail design). This latter method is preferred, as it would address concerns with differential settlement in the immediate vicinity of the abutment. If there is no preload, the embankment may have to be constructed using lightweight fill to eliminate the differential settlement.

### 6.3.2 Axial Geotechnical Resistance/Reaction

For preliminary design for HP 310x110 piles or for the assessment of the existing SBL bridge HP 310x79 piles driven to the estimated tip elevations provided in Section 6.3.1, the factored geotechnical axial resistance at ULS may be taken as 1,700 kN, and the geotechnical axial reaction at SLS (for approximately 10 mm of settlement)





## PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT - TEE CREEK BRIDGES

may be taken as 1,500 kN. Similar axial resistances may be used in the design of closed-end, concrete-filled, 324 mm (12 ¾ in.) diameter steel pipe piles having a minimum wall thickness of 9.5 mm (3/8 in.).

The preliminary geotechnical resistances provided above will have to be re-evaluated and modified as necessary during detail design in consideration of the additional subsurface investigation at the new foundation elements.

### 6.4 Caissons

As an alternative to steel H-piles or pipe piles, caissons could be considered for support of the new abutments. Temporary or permanent liners sealed into the bedrock will be required during caisson construction because of the water-bearing cohesionless soils that are present at this site. For the installation of caissons, consideration must be given to the potential presence of cobbles and boulders within the soil deposits.

#### 6.4.1 Founding Elevations

As the surface of the bedrock varies based on the borehole results, and to accommodate some weathering in the upper portion of the bedrock, socketing into the bedrock is recommended. The table below provides caisson founding levels for preliminary design:

Structure	Foundation Element	Borehole Number(s)	Design Caisson Founding Elevation* (m)
Toronto Bound	Tee Creek Bridge NBL – North Abutment	13-09	140.0
	Tee Creek Bridge NBL – South Abutment	13-10	143.0

\* Assumes caissons are socketed a minimum 1 m below slightly weathered bedrock.

#### 6.4.2 Axial Geotechnical Resistance/Reaction

For preliminary design, caissons socketed approximately 1 m or more into the bedrock should be designed, using a factored axial geotechnical resistance at ULS of 5 MPa; for a 1 m diameter caisson, this would equate to a factored geotechnical axial resistance at ULS of 3,900 kN. The geotechnical reaction at SLS (for less than 25 mm of settlement) may be taken as 3,000 kN.

### 6.5 Approach Embankments

#### 6.5.1 Subgrade Preparation and Embankment Construction

It is recommended that all surficial topsoil/organic material or existing fill materials be stripped from the footprint of the new approach embankments. The depth and extent of stripping should be assessed during detail design when additional subsurface information will be available for the widened approach embankment areas.

Additional fill for construction of the embankment widening could consist of clean earth fill or granular fill. Benching of the north and south sides of the existing QEW embankment should be carried out to “key in” the new fill materials for the realignment/widening, in accordance with OPSD 208.010 (*Benching of Earth Slopes*).



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To reduce erosion of the embankment side slopes due to surface water runoff, placement of topsoil and seeding or pegged sod, in accordance with OPSS.PROV 804, is recommended as soon as practicable after construction of the embankments.

### 6.5.2 Approach Embankment Stability

Preliminary slope stability analyses have been performed for the proposed new approach embankments using the commercially available program SLIDE V.6, produced by Rocscience Inc., to check that a minimum factor of safety of 1.3 is achieved for the proposed embankment heights and geometries under static conditions. This minimum factor of safety is considered appropriate for the proposed bridge replacement on this project, considering the design requirements and the available field and laboratory testing data.

The preliminary stability analyses were completed for a maximum 4.5 m high approach embankments, based on the subsurface conditions as encountered in Boreholes 13-07 to 13-10. The following parameters have been used in the preliminary analyses, based on field and laboratory test data as well as accepted correlations:

Soil Deposit	Bulk Unit Weight (kN/m <sup>3</sup> )	Effective Friction Angle	Undrained Shear Strength (kPa)
Embankment fill	21	34°	-
Firm to stiff clayey silt to silty clay	20	28°	30
Loose to very dense silt to silty sand	19	30°	-

The preliminary stability analysis results indicate that a 4.5 m high embankment with side slopes no steeper than 2H:1V will have a factor of safety greater than 1.3 against global instability, assuming appropriate subgrade preparation and proper placement and compaction of the embankment fill materials. An example of the results from the static global stability analyses is provided on Figure 1. This preliminary assessment of the stability of the approach embankments should be reviewed and confirmed based on the additional borehole information obtained within the proposed footprint for the widened QEW approach embankments during detail design.

The stability of the approach embankments as analyzed for a cross-section model perpendicular to the QEW alignment. Once the design configuration of the new bridges is known, global stability of the front slopes of the bridges should be assessed.

### 6.5.3 Approach Embankment Settlement

The new NBL Tee Creek bridge is proposed to be constructed at the location of the existing structures. Preliminary settlement analyses for the anticipated soil conditions below the new/widened approach embankments were carried out using the commercially available computer program *Settle-3D* from Rocscience, using estimated elastic deformation moduli as given below, based on correlations with the SPT "N" values, undrained shear strengths, Atterberg limits testing and engineering judgement from experience with similar soils in this region of Ontario (Bowles, 1984; Kulhawy and Mayne, 1990; Peck et al., 1974).



## PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT - TEE CREEK BRIDGES

Soil Deposit	Bulk Unit Weight (kN/m <sup>3</sup> )	Elastic Modulus (MPa)	Preconsolidation Pressure (kPa)	C <sub>c</sub>	C <sub>r</sub>
Embankment fill	21	-	-	-	-
Firm to stiff clayey silt to silty clay	20	10	150*	0.24*	0.05*
Loose to very dense silt to silty sand	19	20	-	-	-

\* Based on the results of two consolidation tests from the 1980 investigation

Based on this preliminary assessment, the settlement of the foundation soils under the new 4.5 m high section of the approach embankments immediately behind the new abutments is estimated to be up to about 350 mm. Approximately 150 mm of this settlement is expected to occur relatively quickly during and immediately following construction of the approach embankments. However, approximately 200 mm of this settlement is associated with longer-term consolidation of the soft to firm portion of the clayey deposits under the new/widened approach embankment loading; it is anticipated that the majority of this settlement would be completed within approximately nine months. This estimated magnitude and duration of settlement should be reassessed following additional investigation (including consolidation testing) during detail design.

The above preliminary settlement estimates do not include compression of the fill itself, which would occur during and after the construction of the embankment depending on the type of materials used. The magnitude of fill compression may range from 0.5 to 1 per cent of the height of the embankment, assuming approximately 98 per cent compaction of the embankment fill is achieved, relative to the material's standard Proctor maximum dry density. In the case where granular fill is used for embankment construction, settlement of the fill itself is expected to occur essentially during embankment construction, whereas non-granular earth fill materials are expected to exhibit some additional settlement over time.

## 6.6 Construction Considerations

The following subsections identify future construction considerations that should be considered at this stage as they may impact the planning and preliminary design. Where applicable, Non-Standard Special Provisions (NSSP) should be developed during the future detail design stage for incorporation in the Contract Documents.

### 6.6.1 Excavation and Temporary Protection Systems

The foundation excavations for pile caps would extend through the existing fill and very soft to stiff clayey deposits. If space permits, open-cut excavations into these materials should be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) for Construction Activities. The existing fill and soft/stiff soils should be classified as Type 4 soil, according to the OHSA. Temporary excavations (i.e. those that are open for a relatively short time period) through these materials should be made with side slopes no steeper than 3H:1V, assuming that appropriate groundwater control is in place.

If temporary protection systems are required, the selection and design of the protection system will be the responsibility of the Contractor. However, for conceptual/planning purposes, the temporary protection systems should be designed and constructed in accordance with OPSS.PROV 539 (*Temporary Protection Systems*). The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS.PROV 539. It is considered that either a driven, interlocking sheetpile system or a soldier pile and timber lagging system would be suitable for the temporary excavation support at the abutments.



### 6.6.2 Groundwater Control

While new abutment pile caps would be maintained above the groundwater level at the site, excavations for new pile caps would extend below the groundwater level.

Due to the proximity of the abutments to the edge of the Tee Creek, a groundwater cut-off (cofferdam or similar measure) is recommended to minimize dewatering requirements and potential environmental impacts.

### 6.6.3 Bedrock Excavation and/or Socket Formation

Where excavation into sound bedrock is required, it is expected that hoe-ramming techniques will be needed to reach the design founding elevation. It is recommended that an NSSP be developed at the detail design stage and included in the Contract Documents to warn the contractor that the bedrock at the site is weak to medium strong, that excavation into the bedrock will require appropriate equipment and construction procedures, and that the bedrock excavation must not disturb the existing bridge footings.

Alternatively, if caissons are the selected foundation option and rock sockets are required to provide the necessary foundation capacity, it is recommended that an NSSP be included in the Contract Documents to warn the Contractor that the bedrock is weak to strong. Further, it is expected that socket formation would require coring or churn drilling to advance the hole.

### 6.6.4 Obstructions

The soils at this site are glacially or glacio-fluvially derived and as such should be expected to contain cobbles and boulders, which could affect the installation of deep foundations. Further observation is recommended in the next stage of investigation in support of the detail design. If conditions warrant, an NSSP should be included in the Contract Documents developed during the detail design stage to identify to the contractor the possible presence of cobbles and/or boulders within the overburden soils.

### 6.6.5 Vibration Monitoring During Construction

A maximum peak particle velocity (PPV) of 100 mm/s is generally considered applicable for bridge structures in good condition. Based on vibration monitoring experience, it is considered unlikely that vibrations induced by conventional construction activities such as pile driving, coring/churn drilling, or hoe-ramming will reach this threshold level and, therefore, vibration monitoring for the existing bridges is not expected to be required during construction at this site.

Existing residential buildings are located to the northeast and southwest of the structure site, approximately 200 m from the Tee Creek bridges. Although a lower PPV threshold of 50 mm/s is generally considered applicable for vibration impacts on buildings, the construction zone of influence would likely be less than 100 m. Therefore, vibration monitoring is not expected to be required at the existing buildings adjacent to the bridge site.

## 6.7 Recommendations for Further Work during Detail Design

Additional boreholes will be required within each of the foundation elements and within the approach embankment areas during the future detail design stage of investigation, to further assess and/or confirm the subsurface conditions and the preliminary recommendations provided herein, as follows:

- Abutments:



## PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT - TEE CREEK BRIDGES

- Assessment of the presence of any cohesionless soil lenses or interlayers within the cohesive deposits at the site, which could impact groundwater control requirements for foundation excavations.
- Observation of the presence and frequency of cobbles and/or boulders within the soil deposits, to assess the need for an NSSP to warn the contractor of the presence of such obstructions as they may affect excavations and the installation of driven steel H-pile foundations.
- Assessment of vibration thresholds for the nearby residential buildings, and if warranted development of an NSSP for a vibration monitoring plan.
- Further assessment of the depth and strength of the bedrock at the location of the new abutments.
- Further assessment of the groundwater conditions at the location of each foundation element where excavation would be required.
- Further assess the as-built configuration of the foundations of the existing bridges, determine the as-built location and configuration of the sheetpile foundations.
- Approach embankments:
  - Assessment of the depth and extent of stripping of topsoil/organics, fill materials and loosened or softened native soils within the footprint of the new approach embankments.
  - Further assessment of the thickness and consolidation/elastic compression properties of the soils within the footprint of the widened approach embankments, to confirm the settlement estimates.
  - Further assessment of the engineering parameters and global slope stability of the widened approach embankment.



## PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT - TEE CREEK BRIDGES

### 7.0 CLOSURE

This Preliminary Foundation Design Report was prepared by Mr. Al Varshoi, M.E.Sc., and Mr. Mehdi Mostakhdemi, P.Eng., a geotechnical engineer with Golder, and reviewed by Mr. Kevin J. Bentley a senior geotechnical engineer and associate with Golder. Mr. Jorge M. A. Costa, P.Eng., a Designated MTO Foundations Contact and Principal with Golder, conducted an independent review of this report.

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## PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT - TEE CREEK BRIDGES

### REFERENCES

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- Canadian Geotechnical Society, 2006. *Canadian Foundation Engineering Manual*, 4th Edition. The Canadian Geotechnical Society, BiTech Publisher Ltd., British Columbia.
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- NAVFAC, 1986. *Design Manual DM 7.02: Soil Mechanics, Foundation and Earth Structures*. U.S. Navy. Alexandria, Virginia.
- Ontario Geological Society, 1991. *Geology of Ontario*. Special Volume 4, Part 1. Eds. P.C. Thurston, H.R. Williams, R.H. Sutcliffe and G.M. Stott. Ministry of Northern Development and Mines, Ontario.
- Peck, R.B., Hanson, W.E., and Thornburn, T.H., 1974. *Foundation Engineering*, Second Edition, John Wiley and Sons, New York.

### Ontario Provincial Standard Specifications (OPSS)

- |               |   |
|---------------|---|
| OPSS.PROV 539 | Construction Specification for Temporary Protection Systems |
| OPSS.PROV 804 | Construction Specification for Seed and Cover               |
| OPSS.PROV 903 | Construction Specification for Deep Foundations             |

### Ontario Provincial Standard Drawings (OPSD)

- |               |  |
|---------------|--|
| OPSD 208.010  | Benching of Earth Slopes                                 |
| OPSD 3090.101 | Foundation Frost Penetration Depths for Southern Ontario |





## PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT - TEE CREEK BRIDGES

**TABLE 1 – COMPARISON OF FOUNDATION OPTIONS  
TEE CREEK BRIDGES**

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Estimated Costs
Spread/strip footings	<ul style="list-style-type: none"> <li>Not feasible due to low geotechnical resistances associated with the very soft to stiff clayey deposits</li> </ul>	<ul style="list-style-type: none"> <li>N/A</li> </ul>	<ul style="list-style-type: none"> <li>N/A</li> </ul>	<ul style="list-style-type: none"> <li>N/A</li> </ul>	<ul style="list-style-type: none"> <li>N/A</li> </ul>
Steel H-piles driven to found on bedrock	<ul style="list-style-type: none"> <li>Feasible for support of abutments and wingwalls/ retaining walls</li> </ul>	<ul style="list-style-type: none"> <li>Pile caps could be maintained higher than spread footing, reducing depth of excavation and temporary protection system requirements adjacent to QEW embankment</li> <li>Limited groundwater control required</li> <li>Allows for integral abutment construction</li> </ul>	<ul style="list-style-type: none"> <li>Minor risk of encountering obstructions (cobbles and/or boulders) during pile driving; this could result in piles “hanging up” and lower geotechnical resistances</li> </ul>	<ul style="list-style-type: none"> <li>Conventional construction methods</li> </ul>	<ul style="list-style-type: none"> <li>Lower relative cost compared with caisson option</li> <li>Estimated cost is approximately \$250/m length for pile installation and \$600/m<sup>3</sup> for pile cap construction, plus cost of any temporary protection systems</li> </ul>
Steel pipe (tube) piles, driven to found on bedrock	<ul style="list-style-type: none"> <li>Feasible for support of new abutments and wingwalls/ retaining walls</li> </ul>	<ul style="list-style-type: none"> <li>Abutment pile caps could be maintained higher than spread footing, reducing depth of excavation and temporary protection system requirements adjacent to QEW embankment</li> <li>Limited groundwater control required</li> </ul>	<ul style="list-style-type: none"> <li>Greater risk than for steel H-pile foundations of encountering obstructions (cobbles and/or boulders) during driving; this could result in piles “hanging up” and lower geotechnical resistances</li> <li>Not readily accepted by MTO for integral abutment design</li> </ul>	<ul style="list-style-type: none"> <li>Conventional construction methods</li> </ul>	<ul style="list-style-type: none"> <li>Costs for steel pipe (tube) piles similar to but slightly higher than those for H-piles</li> </ul>





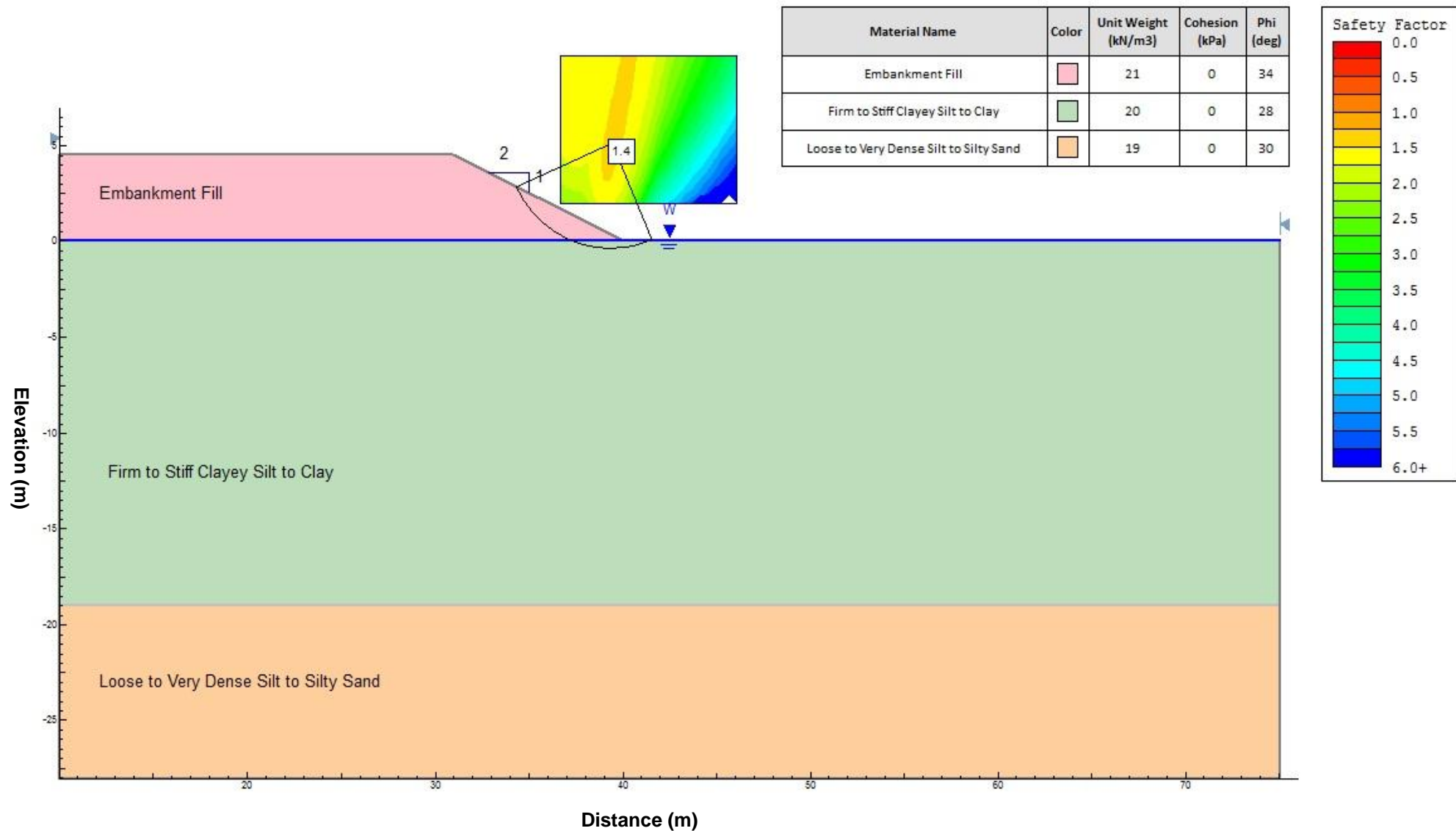
## PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT - TEE CREEK BRIDGES

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Estimated Costs
Caissons founded in bedrock	<ul style="list-style-type: none"><li>• Feasible for support of new abutments</li></ul>	<ul style="list-style-type: none"><li>• Abutment pile caps could be maintained higher than footings, reducing depth of excavation and temporary excavation support requirements adjacent to QEW embankment</li><li>• Higher capacity than piles will require fewer foundation elements</li></ul>	<ul style="list-style-type: none"><li>• Temporary or permanent liners would be required due to risk of running/flowing soils in water-bearing cohesionless deposits</li><li>• Coring and/or churn drilling techniques required to penetrate into the bedrock</li><li>• Precludes use of integral abutments</li></ul>	<ul style="list-style-type: none"><li>• Conventional construction methods with temporary liners required</li><li>• Greater risk than steel piles of encountering obstructions (cobbles, boulders, and/or existing sheetpile foundations) during installation; this could result in caissons not achieving desired elevations and/or lower geotechnical resistances</li></ul>	<ul style="list-style-type: none"><li>• Higher cost compared with shallow foundations or steel H-piles</li></ul>



# Static Global Stability – Tee Creek Bridges Effective Stress Analysis

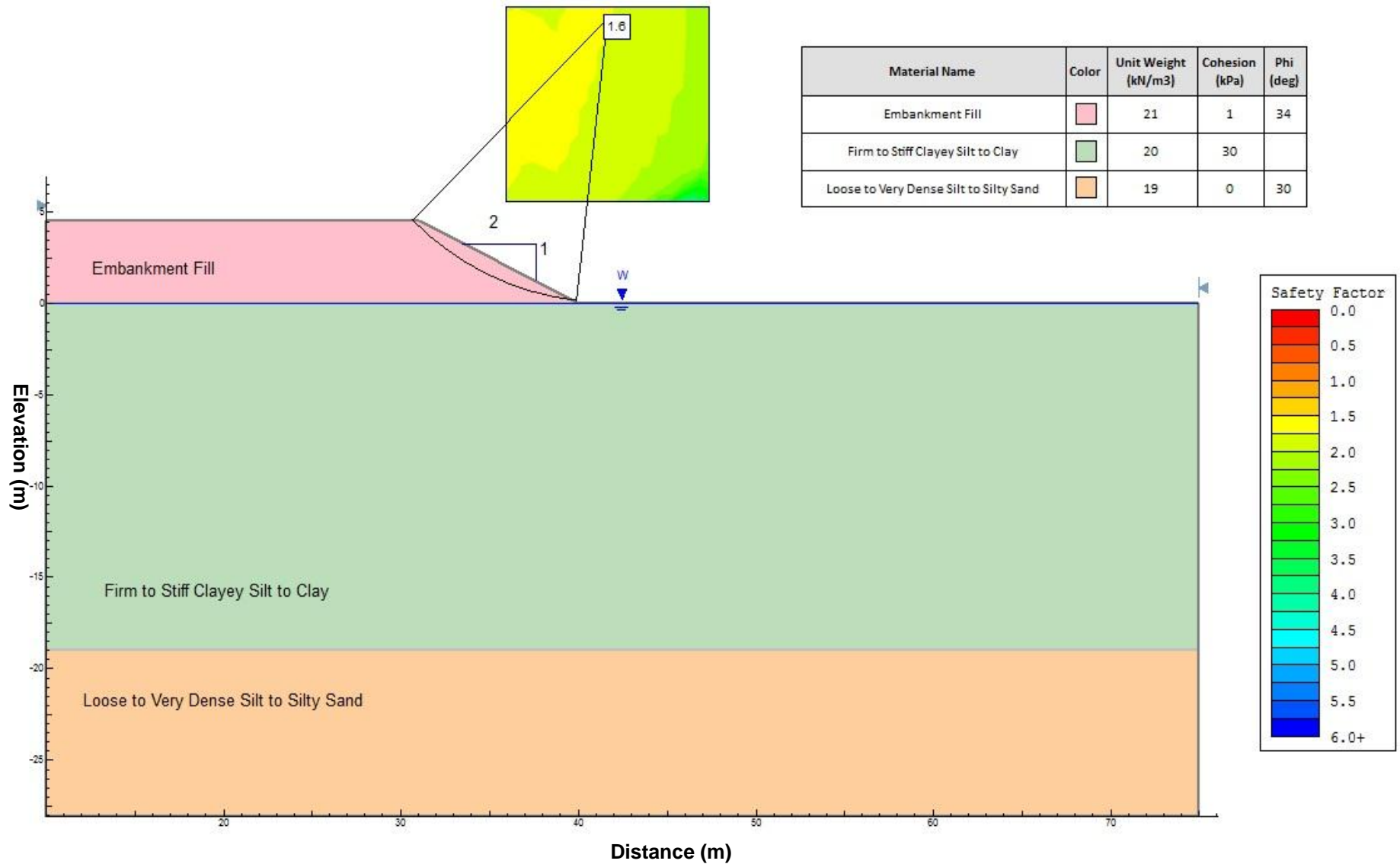
Figure 1





## Static Global Stability – Tee Creek Bridge Total Stress Analysis

Figure 2



**METRIC**  
DIMENSIONS ARE IN METRES AND/OR  
MILLIMETRES UNLESS OTHERWISE SHOWN.  
STATIONS IN KILOMETRES + METRES.

CONT No.  
GWP No. 2177-08-00

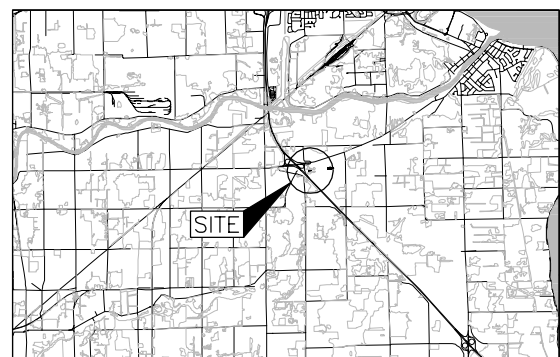


QEW STRUCTURE REPLACEMENT  
TEE CREEK BRIDGES  
BOREHOLE LOCATIONS

SHEET



**Golder Associates Ltd.**  
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN

SCALE  
2 0 2 4 km

LEGEND

- Borehole - Current Investigation
- ⊕ Approximate Borehole Location - Previous Investigation (Geocres No. 30M03-174)

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
13-07	174.2	4765574.3	336686.4
13-08	174.2	4765532.1	336722.7
13-09	174.0	4765591.0	336717.3
13-10	174.2	4765559.9	336740.9
174-1 *	174.2	4765557.5	336706.6
174-2 *	174.2	4765532.1	336719.2
174-3 *	174.1	4765579.2	336699.2

\* Approximate Borehole Coordinates

NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the design configuration as shown elsewhere in the Preliminary Design Report.

The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

The complete Preliminary Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

REFERENCE

Base plans provided in digital format by URS, drawing file nos. X-Base-All.dwg and X-Contours.dwg, received July 30, 2013 and Draft Tee Creek for Golder.dgn, received October 29, 2013.

PLAN  
SCALE  
5 0 5 10 m



NO.	DATE	BY	REVISION
Geocres No. 30M3-280			
HWY.	QEW	PROJECT NO.	12-1111-0088
SUBM'D.	MM	CHKD.	AV
DRAWN:	JFC	CHKD.	MM
DATE:	11/25/2013	APPD.	JMAC
DIST.		SITE:	
DWG.	1		

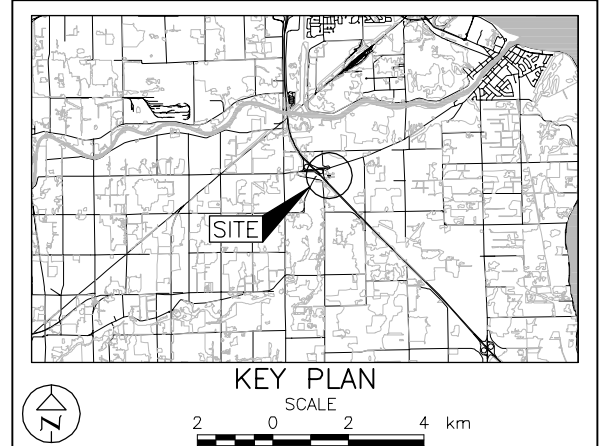
**METRIC**  
DIMENSIONS ARE IN METRES AND/OR  
MILLIMETRES UNLESS OTHERWISE SHOWN.  
STATIONS IN KILOMETRES + METRES.

CONT No.  
GWP No. 2177-08-00




QEW STRUCTURE REPLACEMENT TEE CREEK BRIDGES SOIL STRATA	SHEET
---	-------



**Golder Associates Ltd.**  
MISSISSAUGA, ONTARIO, CANADA



### LEGEND

- |   |   |
|---|---|
|    | Borehole — Current Investigation  |
|    | Approximate Borehole Location — Previous Investigation<br>(Geocres No. 30M03-174) |
| N   | Standard Penetration Test Value   |
| 16  | Blows/0.3m unless otherwise stated<br>(Std. Pen. Test, 475 j/blow)                |
| 100%  | Rock Quality Designation (RQD)  |
|  | WL upon completion of drilling  |
| R   | Refusal   |

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
13-07	174.2	4765574.3	336686.4
13-08	174.2	4765532.1	336722.7
13-09	174.0	4765591.0	336717.3
13-10	174.2	4765559.9	336740.9
174-1 *	174.2	4765557.5	336706.6
174-2 *	174.2	4765532.1	336719.2
174-3 *	174.1	4765579.2	336699.2

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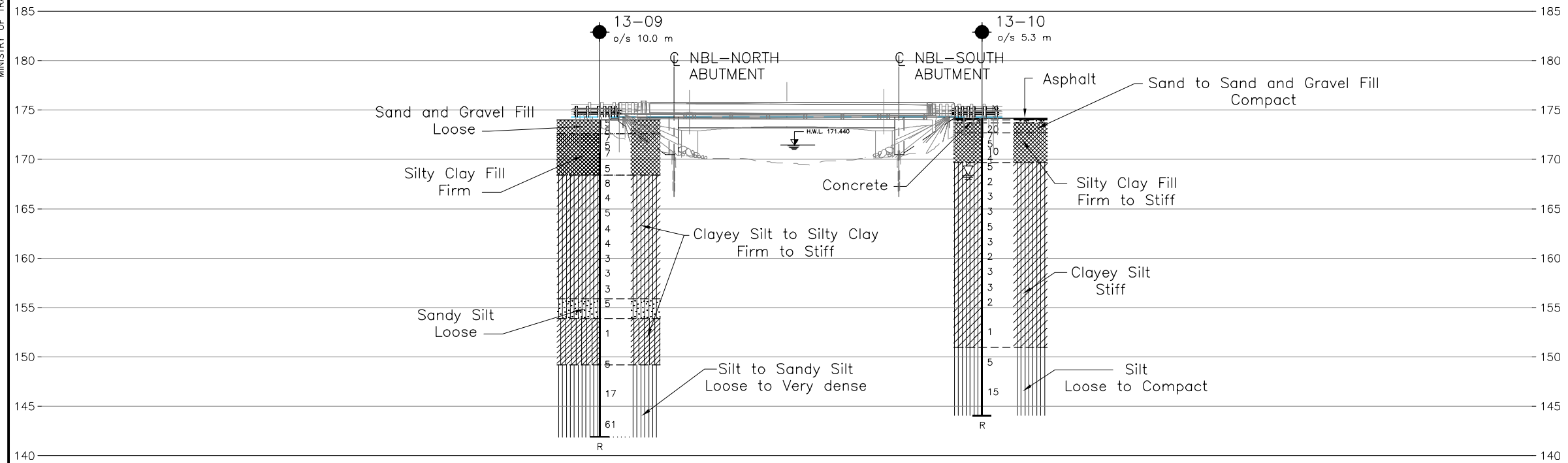
The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

The complete Preliminary Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

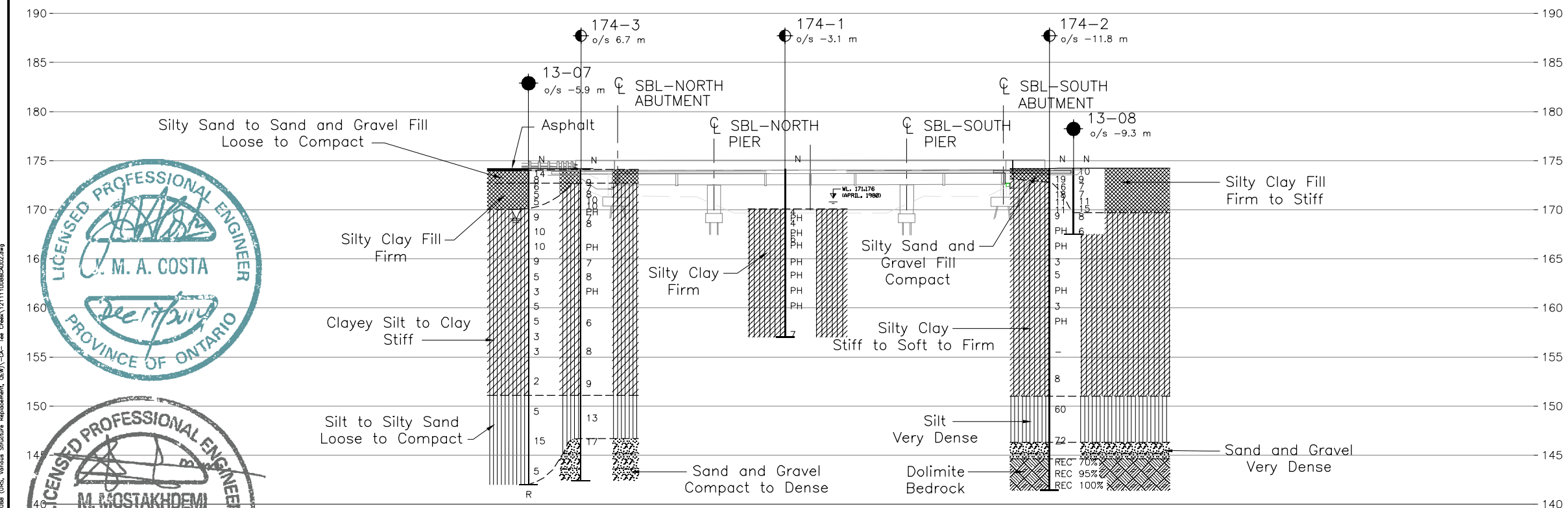
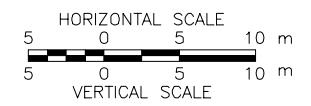
## REFERENCE

Base plans provided in digital format by URS, drawing file nos. X-Base-All.dwg and X-Contours.dwg, received July 30, 2013, Draft Tee Creek for Golder.dgn, received October 29, 2013 and 01\_GA\_Tee Creek-NBL.dwg, received December 10, 2013.

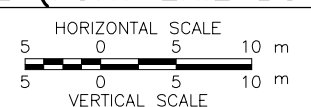
NO.	DATE	BY	REVISION		
Geocres No. 30M3-280					
HWY. QEW			PROJECT NO. 12-1111-0088		DIST.
SUBM'D. MM		CHKD. AV	DATE: 11/25/2013		SITE:
DRAWN: JFC		CHKD. MM	APPD. JMAC		DWG. 2



### PROFILE (TORONTO BOUND LANES)



### B-B' PROFILE (FORT ERIE BOUND LANES)





# **APPENDIX A**

## **Record of Borehole Sheets**





## LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

### I. GENERAL

$\pi$	3.1416
$\ln x$ ,	natural logarithm of x
$\log_{10}$	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
FoS	factor of safety

### II. STRESS AND STRAIN

$\gamma$	shear strain
$\Delta$	change in, e.g. in stress: $\Delta \sigma$
$\varepsilon$	linear strain
$\varepsilon_v$	volumetric strain
$\eta$	coefficient of viscosity
$\nu$	Poisson's ratio
$\sigma$	total stress
$\sigma'$	effective stress ( $\sigma' = \sigma - u$ )
$\sigma'_{vo}$	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
$\sigma_{oct}$	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
$\tau$	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

### III. SOIL PROPERTIES

<b>(a)</b>	<b>Index Properties</b>
$\rho(\gamma)$	bulk density (bulk unit weight)*
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
$\gamma'$	unit weight of submerged soil ( $\gamma' = \gamma - \gamma_w$ )
$D_R$	relative density (specific gravity) of solid particles ( $D_R = \rho_s / \rho_w$ ) (formerly $G_s$ )
e	void ratio
n	porosity
S	degree of saturation

### (a) Index Properties (continued)

w	water content
$w_l$ or LL	liquid limit
$w_p$ or PL	plastic limit
$I_p$ or PI	plasticity index = $(w_l - w_p)$
$w_s$	shrinkage limit
$I_L$	liquidity index = $(w - w_p) / I_p$
$I_C$	consistency index = $(w_l - w) / I_p$
$e_{max}$	void ratio in loosest state
$e_{min}$	void ratio in densest state
$I_D$	density index = $(e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

### (b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

### (c) Consolidation (one-dimensional)

$C_c$	compression index (normally consolidated range)
$C_r$	recompression index (over-consolidated range)
$C_s$	swelling index
$C_\alpha$	secondary compression index
$m_v$	coefficient of volume change
$C_v$	coefficient of consolidation (vertical direction)
$C_h$	coefficient of consolidation (horizontal direction)
$T_v$	time factor (vertical direction)
U	degree of consolidation
$\sigma'_p$	pre-consolidation stress
OCR	over-consolidation ratio = $\sigma'_p / \sigma'_{vo}$

### (d) Shear Strength

$\tau_p, \tau_r$	peak and residual shear strength
$\phi'$	effective angle of internal friction
$\delta$	angle of interface friction
$\mu$	coefficient of friction = $\tan \delta$
$c'$	effective cohesion
$c_u, s_u$	undrained shear strength ( $\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
$p'$	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
$q_u$	compressive strength $(\sigma_1 - \sigma_3)$
$S_t$	sensitivity

\* Density symbol is  $\rho$ . Unit weight symbol is  $\gamma$  where  $\gamma = \rho g$  (i.e. mass density multiplied by acceleration due to gravity)

Notes: 1  
2

$$\tau = c' + \sigma' \tan \phi'$$

$$\text{shear strength} = (\text{compressive strength})/2$$



## LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

### I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
SS	Split-spoon
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

### II. PENETRATION RESISTANCE

#### Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.)

#### Dynamic Cone Penetration Resistance; $N_d$ :

The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

**PH:** Sampler advanced by hydraulic pressure

**PM:** Sampler advanced by manual pressure

**WH:** Sampler advanced by static weight of hammer

**WR:** Sampler advanced by weight of sampler and rod

#### Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm<sup>2</sup> pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance ( $Q_t$ ), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

### III. SOIL DESCRIPTION

#### (a) Non-Cohesive (Cohesionless) Soils

Density Index	N
Relative Density	Blows/300 mm or Blows/ft
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

#### (b) Cohesive Soils Consistency

	$C_u, S_u$	
	kPa	psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

### IV. SOIL TESTS

w	water content
$w_p$	plastic limit
$w_l$	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test <sup>1</sup>
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement <sup>1</sup>
$D_R$	relative density (specific gravity, $G_s$ )
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
$SO_4$	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
$\gamma$	unit weight

**Note:** 1 Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

### V. MINOR SOIL CONSTITUENTS

Per cent by Weight	Modifier	Example
0 to 5	Trace	Trace sand
5 to 12	Trace to Some (or Little)	Trace to some sand
12 to 20	Some	Some sand
20 to 30	(ey) or (y)	Sandy
over 30	And (non-cohesive (cohesionless)) or With (cohesive)	Sand and Gravel Silty Clay with sand / Clayey Silt with sand



PROJECT <u>12-1111-0088</u>		<b>RECORD OF BOREHOLE No 13-07</b>		SHEET 1 OF 3		<b>METRIC</b>	
G.W.P. <u>2177-08-00</u>		LOCATION <u>N 4765574.3 ; E 336686.4</u>		ORIGINATED BY <u>SB</u>			
DIST <u>Central</u> HWY <u>QEW</u>		BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers</u>		COMPILED BY <u>AV</u>			
DATUM <u>Geodetic</u>		DATE <u>July 4 and 5, 2013</u>		CHECKED BY <u>MM</u>			


SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × REMOULDED					
174.2	GROUND SURFACE													
0.0	ASPHALT (225 mm)													
0.2	Sand and gravel (FILL) Compact Grey Moist		1	SS	14									
173.4	Silty sand (FILL) Loose Brown Wet		2	SS	8									
172.8	Silty clay, some sand, trace gravel, trace organics, trace rootlets to a depth of 2.1 m (FILL) Firm Grey Wet		3	SS	6									
170.1	SILTY CLAY to CLAY, trace sand, silt seams throughout Stiff Brown and grey Wet		6	SS	9									
167.0	CLAYEY SILT, trace sand, trace gravel Stiff Brown becoming grey below a depth of 10.7 m Wet		8	SS	10									
			9	SS	9									
			10	SS	5									
			11	SS	3									
			12	SS	5									

Continued Next Page



+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

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PROJECT 12-1111-0088		RECORD OF BOREHOLE No 13-07				SHEET 3 OF 3		METRIC										
G.W.P. 2177-08-00		LOCATION N 4765574.3 ; E 336686.4				ORIGINATED BY SB												
DIST Central HWY QEW		BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers				COMPILED BY AV												
DATUM Geodetic		DATE July 4 and 5, 2013				CHECKED BY MM												
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa										
	--- CONTINUED FROM PREVIOUS PAGE ---						20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					WATER CONTENT (%) 20 40 60						
29.9	Silty SAND, trace to some gravel, trace clay Loose Grey Wet		19	SS	5		144										6 71 22 1	
							143											
142.0	END OF BOREHOLE AUGER REFUSAL						142											
32.2	NOTE:  1. Water level inside auger at a depth of 5.1 m below ground surface (Elev. 169.1 m) upon completion of drilling.																	

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PROJECT 12-1111-0088		RECORD OF BOREHOLE No 13-08		SHEET 1 OF 1		METRIC															
G.W.P. 2177-08-00		LOCATION N 4765532.1 ; E 336722.7		ORIGINATED BY SB																	
DIST Central HWY QEW		BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers		COMPILED BY AV																	
DATUM Geodetic		DATE July 4, 2013		CHECKED BY MM																	
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W <sub>p</sub>	W	W <sub>L</sub>	γ	GR	SA	SI	CL	
174.2 0.0	GROUND SURFACE Silty clay, trace to some sand, trace organics, trace rootlets and vegetation (FILL) Firm to stiff Mottled brown and grey Moist to wet		1	SS	10		174														
			2	SS	9		173														
			3	SS	7		172														
	Containing organics between depths of 2.3 m and 2.9 m ----- organic clayey silt -----		4	SS	7		171														
			5	SS	11		170														
			6	SS	15		169														
169.7 4.5	SILTY CLAY, silt seams throughout, trace rootlets Firm Brown Wet		7	SS	8		169														0 0 40 60
			8	SS	6		168														
167.5 6.7	END OF BOREHOLE  NOTE:  1. Open borehole dry upon completion of drilling.																				

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PROJECT <u>12-1111-0088</u>		<b>RECORD OF BOREHOLE No 13-09</b>		SHEET 1 OF 3		<b>METRIC</b>	
G.W.P. <u>2177-08-00</u>		LOCATION <u>N 4765591.0; E 336717.3</u>		ORIGINATED BY <u>SB</u>			
DIST <u>Central</u> HWY <u>QEW</u>		BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers</u>		COMPILED BY <u>AV</u>			
DATUM <u>Geodetic</u>		DATE <u>June 21 to 24, 2013</u>		CHECKED BY <u>MM</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × REMOULDED					
174.0	GROUND SURFACE													
0.0	Sand and gravel, silt pocket at a depth of 1.0 m (FILL) Loose Brown Moist		1	SS	5									
			2	SS	8									
172.6														
1.4	Silty clay, trace sand, trace organics, trace rootlets (FILL) Firm Grey Wet		3	SS	7									
			4	SS	5									
			5	SS	7									
	Black staining below a depth of 4.6 m.		6	SS	5									
168.4														
5.6	SILTY CLAY, trace sand Firm Mottled brown and grey Wet		7	SS	8									
			8	SS	4									
165.3														
8.7	CLAYEY SILT, trace sand Stiff Grey Wet		9	SS	5									
			10	SS	4									
			11	SS	4									
			12	SS	3									

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+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

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PROJECT 12-1111-0088		RECORD OF BOREHOLE No 13-09		SHEET 2 OF 3		METRIC													
G.W.P. 2177-08-00		LOCATION N 4765591.0; E 336717.3		ORIGINATED BY SB															
DIST Central HWY QEW		BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers		COMPILED BY AV															
DATUM Geodetic		DATE June 21 to 24, 2013		CHECKED BY MM															
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa			WATER CONTENT (%)			γ			GR SA SI CL		
	--- CONTINUED FROM PREVIOUS PAGE ---							20 40 60 80 100	20 40 60 80 100	20 40 60	20 40 60	20 40 60	20 40 60	20 40 60	20 40 60	20 40 60	20 40 60	20 40 60	
								○ UNCONFINED + FIELD VANE	○ UNCONFINED + FIELD VANE	○ UNCONFINED + FIELD VANE	○ UNCONFINED + FIELD VANE	○ UNCONFINED + FIELD VANE	○ UNCONFINED + FIELD VANE	○ UNCONFINED + FIELD VANE	○ UNCONFINED + FIELD VANE	○ UNCONFINED + FIELD VANE	○ UNCONFINED + FIELD VANE	○ UNCONFINED + FIELD VANE	
								● QUICK TRIAXIAL × REMOULDED	● QUICK TRIAXIAL × REMOULDED	● QUICK TRIAXIAL × REMOULDED	● QUICK TRIAXIAL × REMOULDED	● QUICK TRIAXIAL × REMOULDED	● QUICK TRIAXIAL × REMOULDED	● QUICK TRIAXIAL × REMOULDED	● QUICK TRIAXIAL × REMOULDED	● QUICK TRIAXIAL × REMOULDED	● QUICK TRIAXIAL × REMOULDED	● QUICK TRIAXIAL × REMOULDED	
								20 40 60 80 100	20 40 60 80 100	20 40 60	20 40 60	20 40 60	20 40 60	20 40 60	20 40 60	20 40 60	20 40 60	20 40 60	
155.9	CLAYEY SILT, trace sand Stiff Grey Wet		13	SS	3		158												
			14	SS	3		157												
18.1	Sandy SILT, trace to some clay, trace to some gravel Loose Grey Wet		15	SS	5		156												
153.9							155												
20.1	SILTY CLAY, trace sand, trace gravel Stiff Grey Moist						154												
			16	SS	1		153												
149.2							152												
24.8	SILT, trace to some clay, trace sand Loose to compact Brown Wet		17A 17B	SS	5		151												
							150												
							149												
							148												
							147												
			18	SS	17		146												
145.0							145												
29.0	Sandy SILT, some gravel, trace to some clay Very dense Brown Wet																		

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

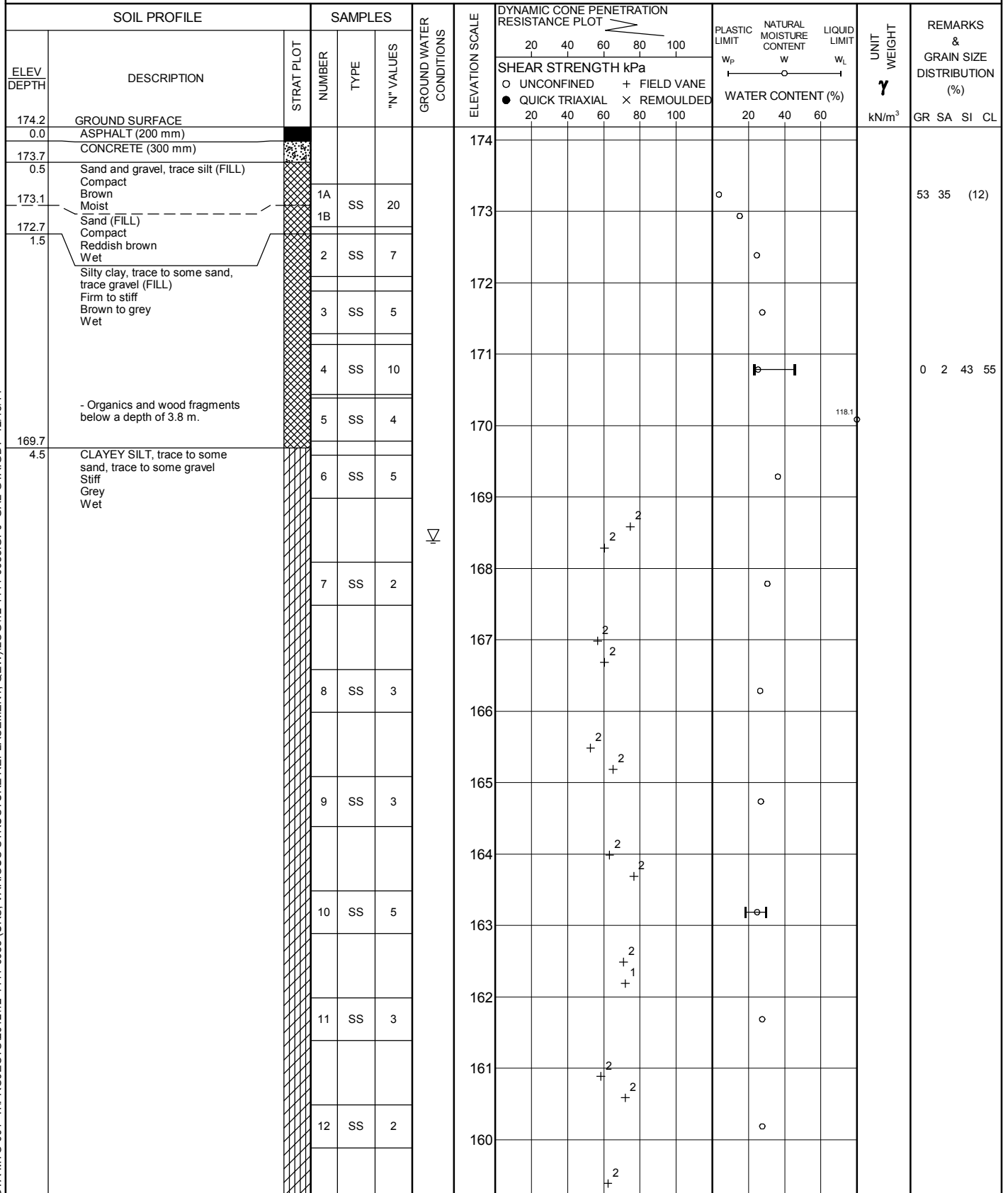
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PROJECT 12-1111-0088			RECORD OF BOREHOLE No 13-09			SHEET 3 OF 3			METRIC															
G.W.P. 2177-08-00			LOCATION N 4765591.0; E 336717.3			ORIGINATED BY SB																		
DIST Central HWY QEW			BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers			COMPILED BY AV																		
DATUM Geodetic			DATE June 21 to 24, 2013			CHECKED BY MM																		
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS			ELEVATION SCALE			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES																			
	--- CONTINUED FROM PREVIOUS PAGE ---																							
	Sandy SILT, some gravel, trace to some clay Very dense Brown Wet		19	SS	61																			
141.9																								
32.1	END OF BOREHOLE AUGER REFUSAL  NOTE:  1. Water level not noted upon completion of drilling.																							

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PROJECT <u>12-1111-0088</u>		<b>RECORD OF BOREHOLE No 13-10</b>		SHEET 1 OF 3		<b>METRIC</b>	
G.W.P. <u>2177-08-00</u>		LOCATION <u>N 4765559.9; E 336740.9</u>		ORIGINATED BY <u>SB</u>			
DIST <u>Central</u> HWY <u>QEW</u>		BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers</u>		COMPILED BY <u>AV</u>			
DATUM <u>Geodetic</u>		DATE <u>July 3, 2013</u>		CHECKED BY <u>MM</u>			



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+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

GTA-MTO 001 T:\PROJECTS\2012\12-1111-0088 (URS, VARIOUS STRUCTURE REPLACEMENT, QEW)\LOG\12-1111-0088.GPJ GAL-GTA.GDT 12/15/14



PROJECT <u>12-1111-0088</u>		<b>RECORD OF BOREHOLE No 13-10</b>				SHEET 3 OF 3		<b>METRIC</b>	
G.W.P. <u>2177-08-00</u>		LOCATION <u>N 4765559.9 ; E 336740.9</u>				ORIGINATED BY <u>SB</u>			
DIST <u>Central</u> HWY <u>QEW</u>		BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers</u>				COMPILED BY <u>AV</u>			
DATUM <u>Geodetic</u>		DATE <u>July 3, 2013</u>				CHECKED BY <u>MM</u>			

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 					PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			UNIT WEIGHT  <b>γ</b> kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20	40	60	80	100	W <sub>p</sub>	W		
144.1 30.1	END OF BOREHOLE AUGER REFUSAL  NOTE:  1. Water level inside auger at a depth of 5.8 m below ground surface (Elev. 168.4 m) upon completion of drilling.	III														

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174-1

# RECORD OF BOREHOLE No 1

W P 45-80-01 LOCATION Sta. 110+14 o/s 12' RT @ O.E.W. (SBL) ORIGINATED BY BL  
 DIST 4 HWY O.E.W. BOREHOLE TYPE Washboring - BW Casing COMPILED BY BL  
 DATUM Geodetic DATE April 2, 1980 CHECKED BY \_\_\_\_\_

174.2

141.2

4.1

OFFICE REPORT ON SOIL EXPLORATION

157.0

17.2

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT  γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100			W <sub>p</sub>	W	W <sub>L</sub>
								SHEAR STRENGTH PSF							WATER CONTENT (%)		
							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE										
571.6	Bridge Deck Surface																
0.0							570										
561.6	Creek Surface																
10.0																	
558.3	Creek Bed						560										
13.3			1	SS	4												
	Silty Clay		2	TW	PH											114	
	Low Plasticity		3	SS	4												
	Reddish Gray		4	TW	PH												
	Firm	Grey	5	SS	6		550									123	
			6	TW	PH												
	Trace of Coarse Sand and Occasional Silt Pockets		7	TW	PH		540									125	
			8	TW	PH												
			9	TW	PH		530									123	
			10	TW	PH												
							520										
515.1	Reddish Grey		11	SS	7												
56.5	End of Borehole																

+3, x5: Numbers refer to  
Sensitivity

20  
15  
10  
+5 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No 2

W P 45-80-01 LOCATION Sta. 110+83 o/s 29' RT @ Q.E.W. (SBL) ORIGINATED BY BL  
DIST 4 HWY Q.E.W. BOREHOLE TYPE Hollow Stem Augers - BW Casing & Cone Test COMPILED BY BL  
DATUM Geodetic DATE April 3, 7 & 9, 1980 CHECKED BY

174.2  
172.9  
1.4

OFFICE REPORT ON SOIL EXPLORATION

151.1  
23.2

146.3

144.6  
29.6

141.4  
32.8

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	VALUES		20	40	60	80	100					
571.6	Roadway Shoulder															
0.0	Fill															
567.1	Silty Sand and Gravel		1	SS	19											
4.5	Reddish Trace of Brown of Organics		2	SS	16											0 1 44 55
	Silty Clay		3	SS	18											0 2 44 54
	Medium Plasticity		4	SS	11											
	Trace of Sand		5	SS	11											
	Stiff		6	SS	9											
	Soft		7	TW	PH										119	
			8	TW	PH											
	Silty Clay		9	SS	3											
	Low to Medium Plasticity		10	SS	5											
	Firm		11	TW	PH											
			12	SS	3											
			13	TW	PH											128
			14	SS	-											
			15	SS	8											
495.6	Silt															
76.0	Trace of Clay and Sand		16	SS	60											4 10 75 11
	Very Dense															
480.1			17	SS	72											0 1 96 3
91.5	Sand and Gravel		18	SS	-											
474.4	Very Dense															
97.2	Bedrock		19	RC	REC 70%											
	Dolomite, Very Hard		20	RC	REC 95%											
	Occasional Soft Gypsum and Shaly Sections		21	RC	REC 100%											
463.9	End of Borehole															
107.7	*Water Level Not Established															

+3, x5: Numbers refer to Sensitivity 20 15 + 5 [%] STRAIN AT FAILURE 10

174-3

# RECORD OF BOREHOLE No 3

W P 43-80-D1 LOCATION Sta. 109+60 o/s 29' LT @ Q.E.W. (SBL)  
 DIST 4 HWY Q.E.W. BOREHOLE TYPE Hollow Stem Augers - NW Casing & Cone Test  
 DATUM Geodetic DATE April 9, 1980  
 ORIGINATED BY BL  
 COMPILED BY BL  
 CHECKED BY

OFFICE REPORT ON SOIL EXPLORATION

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	20 40 60 80 100					
571.1	Roadway Median													
0.0	Fill													
566.6	Silty Sand and Gravel		1	SS	9		570	5'	Augered				0m 1.142	0 12 37 51
4.5			2	SS	7	*							0m 0.72	
	Some Sand		3	SS	8									
			4	SS	10		560							
	Silty Clay		5	SS	10								116	0 0 24 76
	Medium Plasticity		6	TW	PH									
	Trace of Organics		7	SS	7									
	Stiff		8	SS	8		550						128	0m 0.81X
			9	TW	PH									
	Silty Clay		10	SS	7		540							
	Low to Medium Plasticity		11	SS	8									
	Trace of Sand and Organics		12	TW	PH		530						130	0 2 67 31
	Stiff		13	SS	6									
			14	SS	8		520							
			15	SS	9		510							
			16	SS	13		500							
496.1	Silt													
75.0	Some Clay						490							
	Trace of Sand													
	Compact						480							0 5 80 15
481.1														
90.0	Sand and Gravel		16	SS	17		470							
	Compact to Dense													
467.1														
104.0	End of Borehole													
	Refusal to Augers Probable Bedrock													
	*Water Level Not Established													

+3, x<sup>2</sup>: Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE



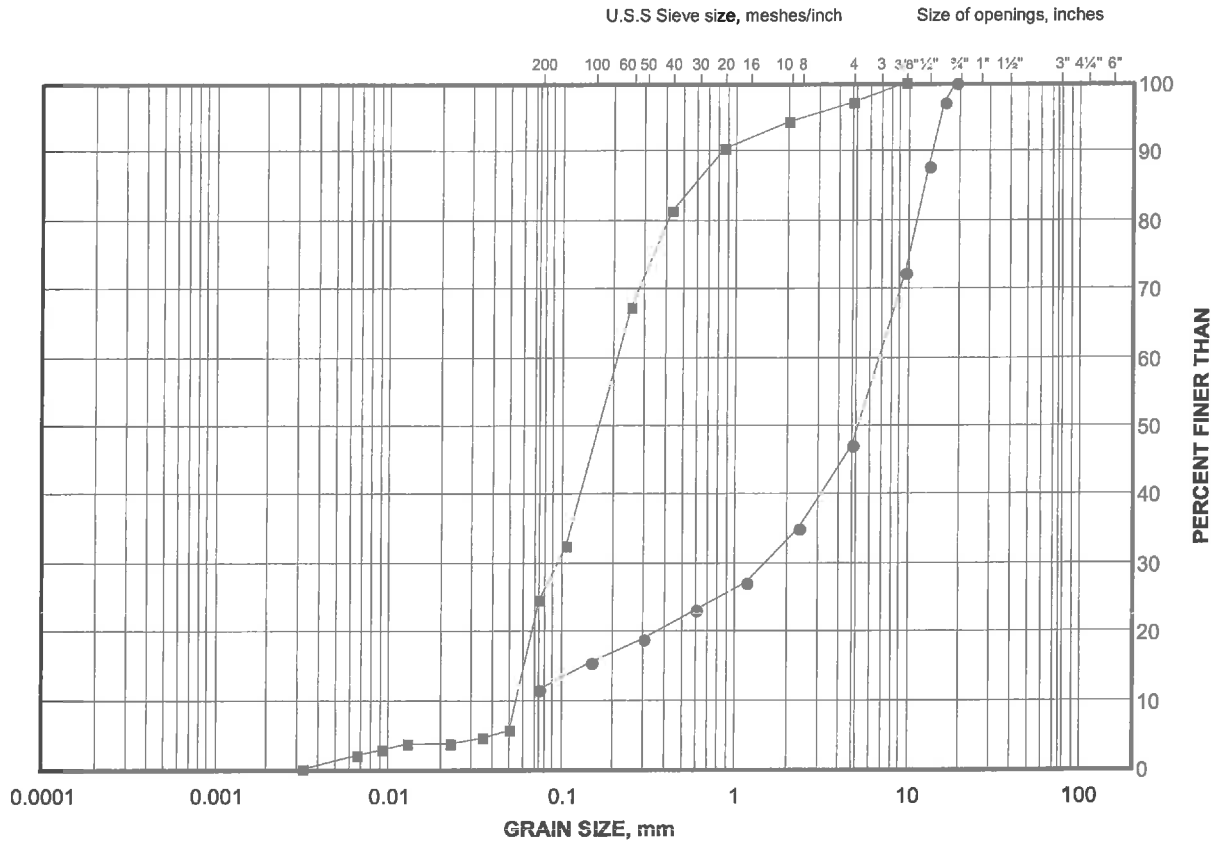
# **APPENDIX B**

## **Laboratory Test Results**

# GRAIN SIZE DISTRIBUTION

Silty Sand to Sand and Gravel Fill

FIGURE B1



SILT AND CLAY SIZES		FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED		SAND SIZE			GRAVEL SIZE		SIZE

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	13-10	1A	173.3
■	13-07	2	173.1

Project Number: 12-1111-0088

Checked By:

**Golder Associates**

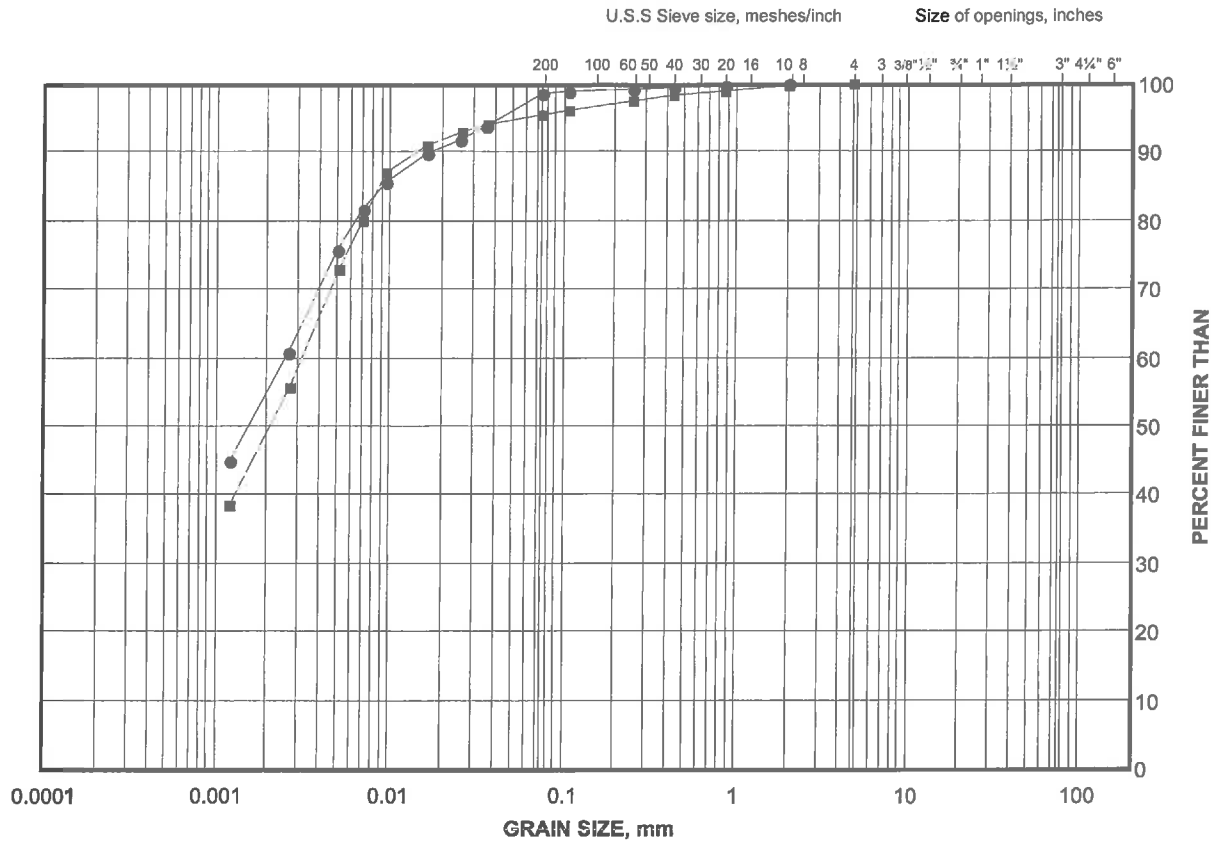
Date: 30-Oct-13



# GRAIN SIZE DISTRIBUTION

Silty Clay Fill

FIGURE B2



SILT AND CLAY SIZES		FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED		SAND SIZE			GRAVEL SIZE		SIZE

## LEGEND

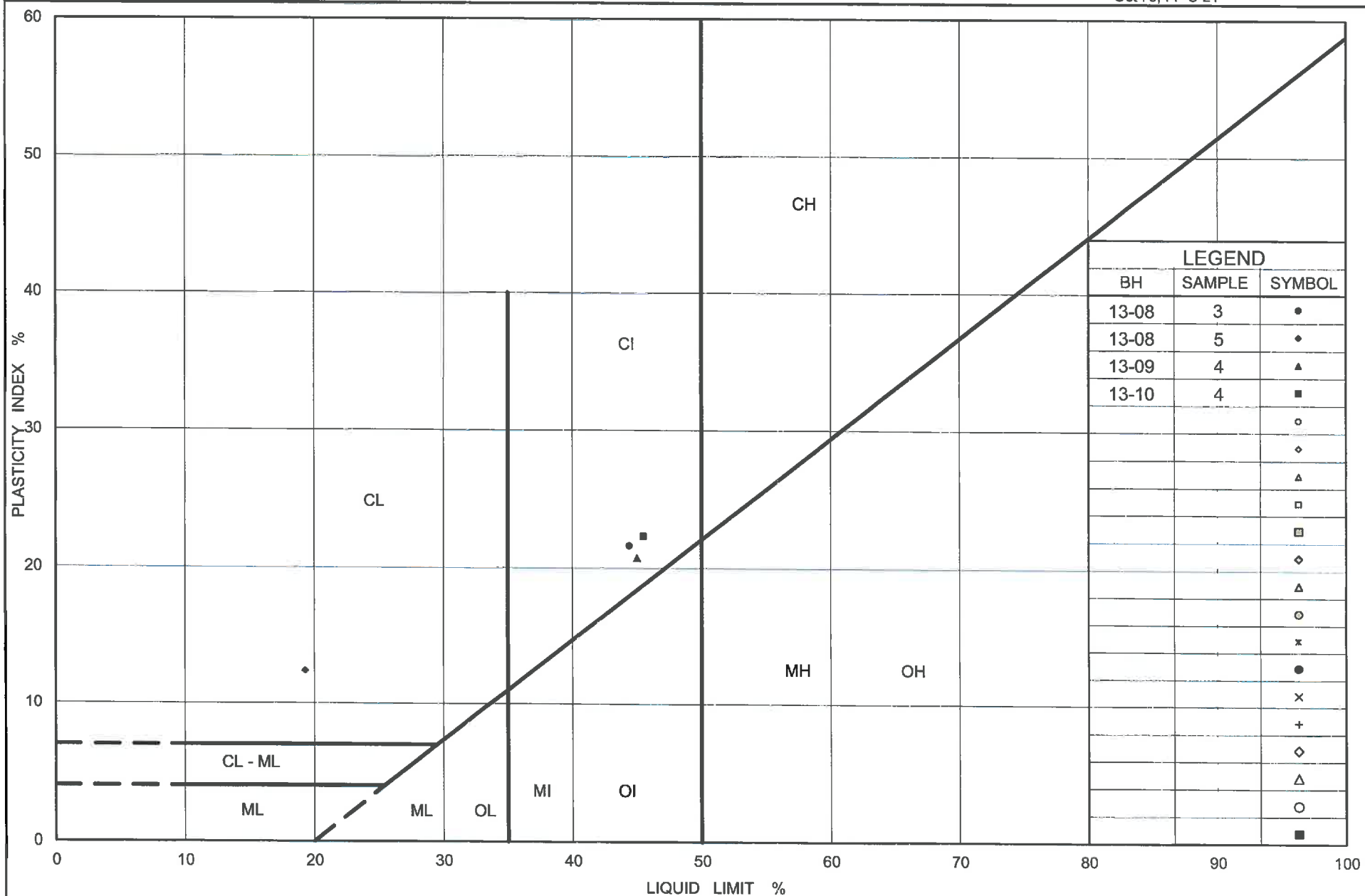
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	13-10	4	170.8
■	13-09	4	171.4

Project Number: 12-1111-0088

Checked By: 

Golder Associates

Date: 30-Oct-13



Ontario

Ministry of  
Transportation

# PLASTICITY CHART Clayey Silt to Silty Clay Fill

Figure No. B3

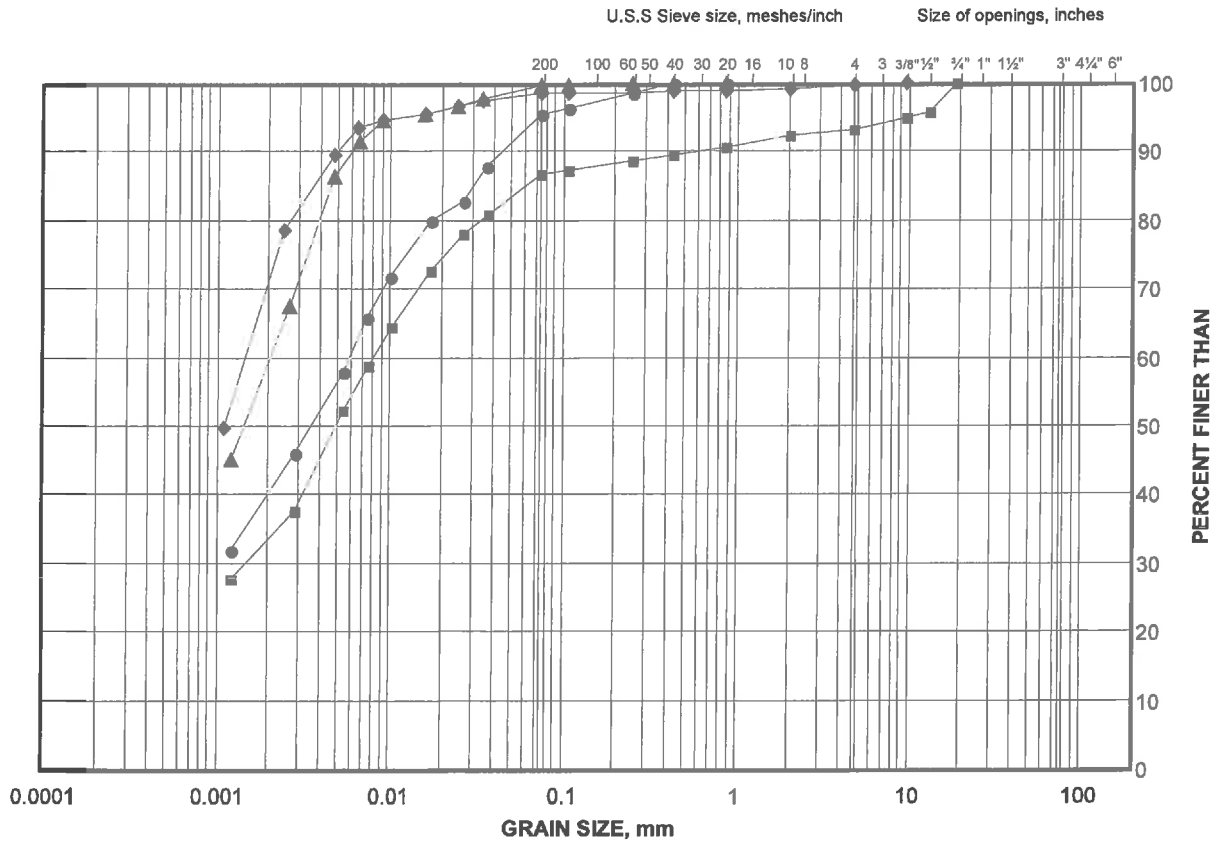
Project No. 12-1111-0088

Checked By: *[Signature]*

# GRAIN SIZE DISTRIBUTION

Clayey Silt to Clay

FIGURE B4



SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED	SAND SIZE			GRAVEL SIZE		SIZE

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	13-09	12	160.0
■	13-10	15	155.6
◆	13-07	6	169.3
▲	13-08	7	169.3

Project Number: 12-1111-0088

Checked By:                     

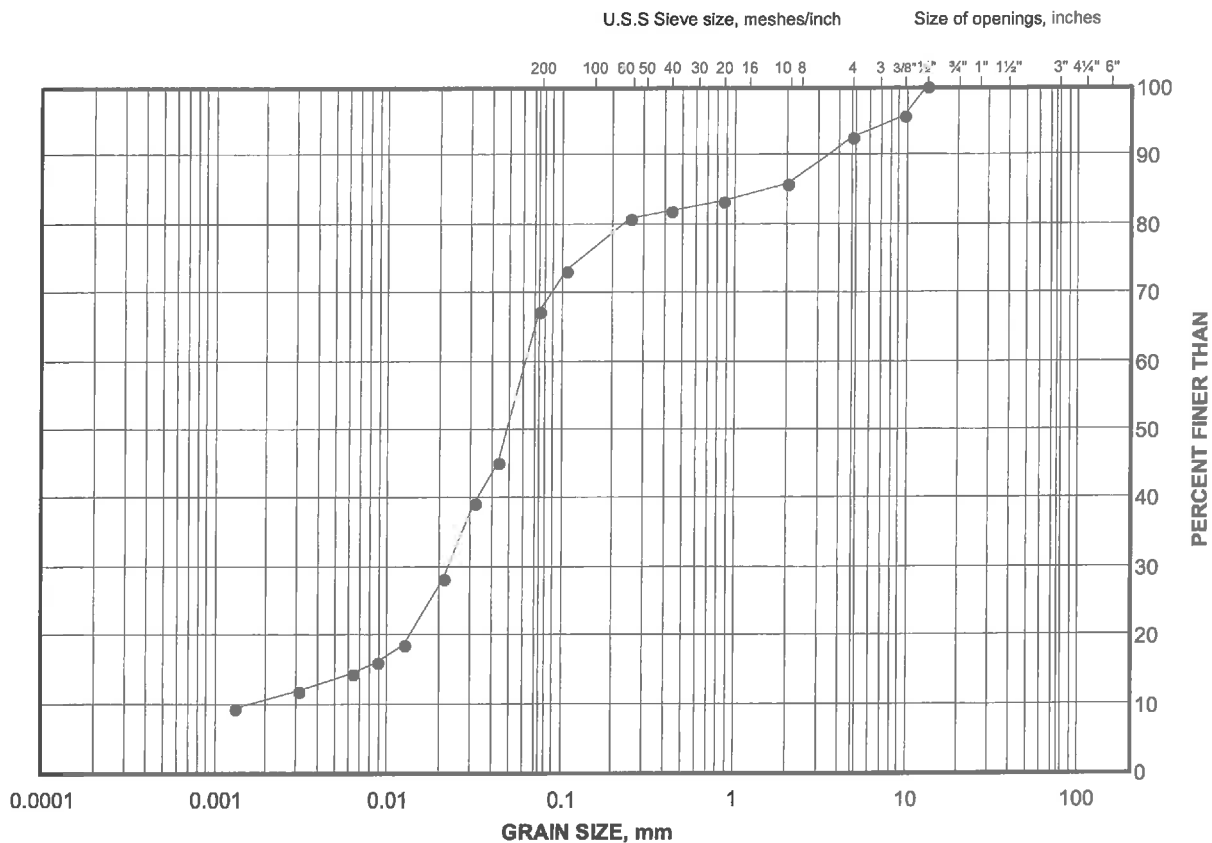
**Golder Associates**

Date: 30-Oct-13

# GRAIN SIZE DISTRIBUTION

Sandy Silt

FIGURE B5



SILT AND CLAY SIZES		FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED		SAND SIZE			GRAVEL SIZE		SIZE

## LEGEND

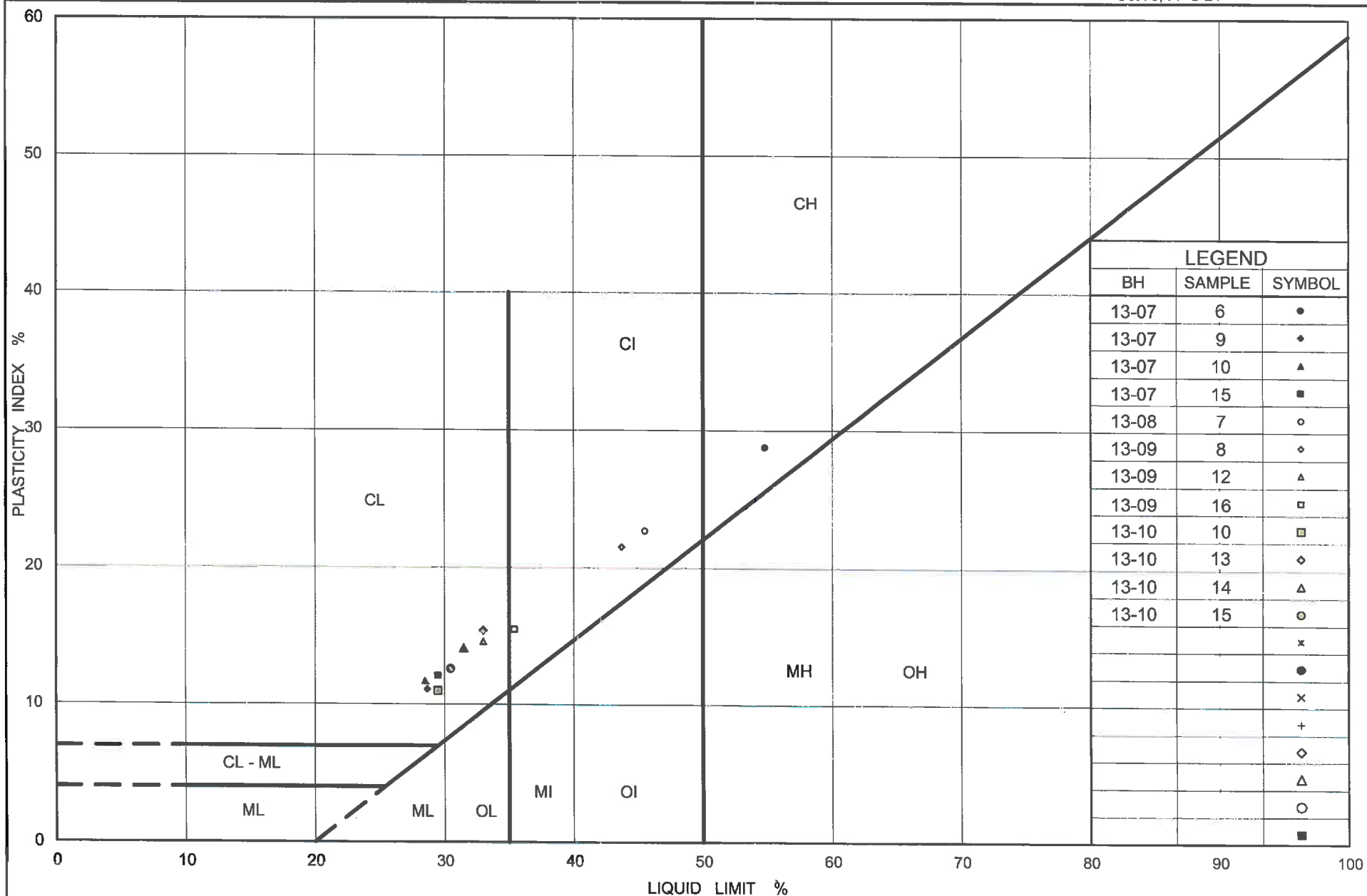
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	13-09	15	155.4

Project Number: 12-1111-0088

Checked By: *[Signature]*

**Golder Associates**

Date: 30-Oct-13



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Transportation

Ontario

## PLASTICITY CHART

Clayey Silt to Clay

Figure No. B6

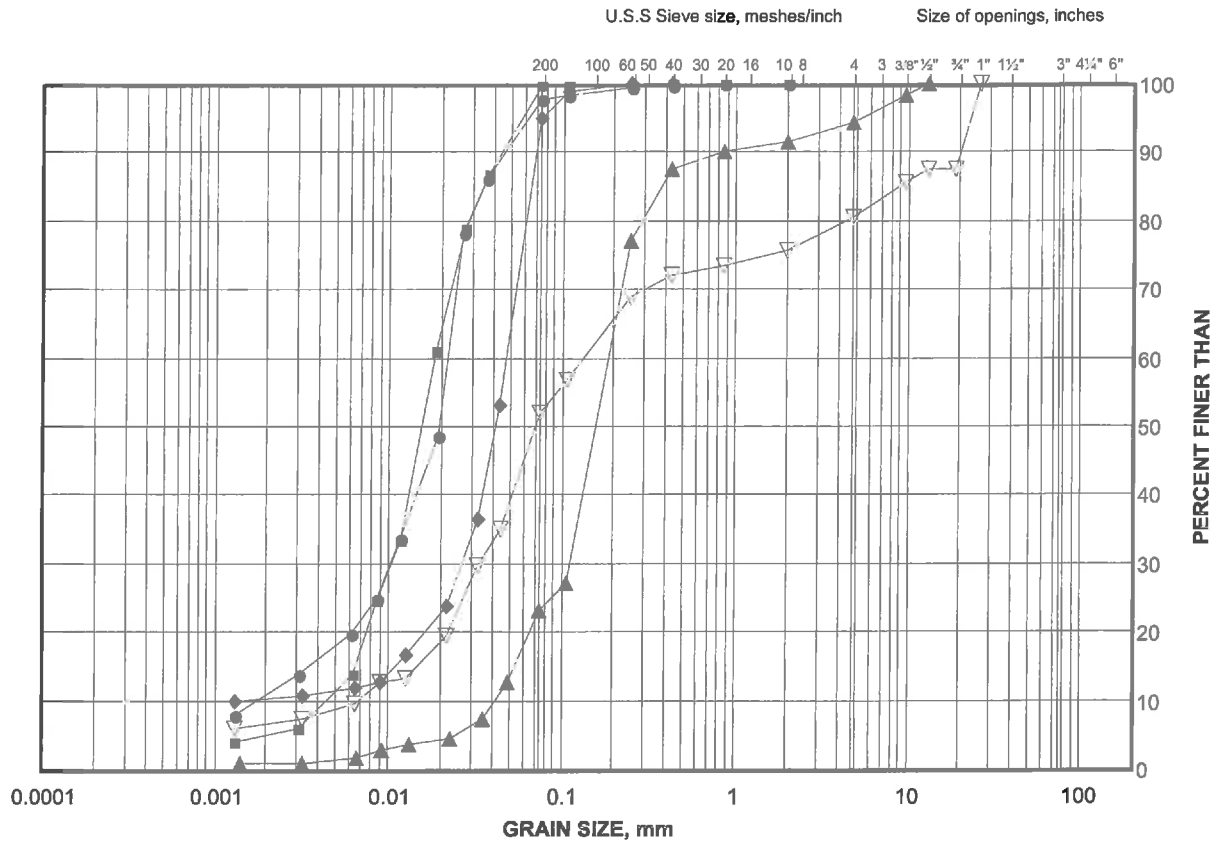
Project No. 12-1111-0088

Checked By: *h.*

# GRAIN SIZE DISTRIBUTION

Silt to Silty Sand

FIGURE B7



SILT AND CLAY SIZES		FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED		SAND SIZE			GRAVEL SIZE		SIZE

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	13-07	17	149.5
■	13-10	17	149.5
◆	13-09	18	146.3
▲	13-07	19	143.4
▽	13-09	19	143.2

Project Number: 12-1111-0088

Checked By: 

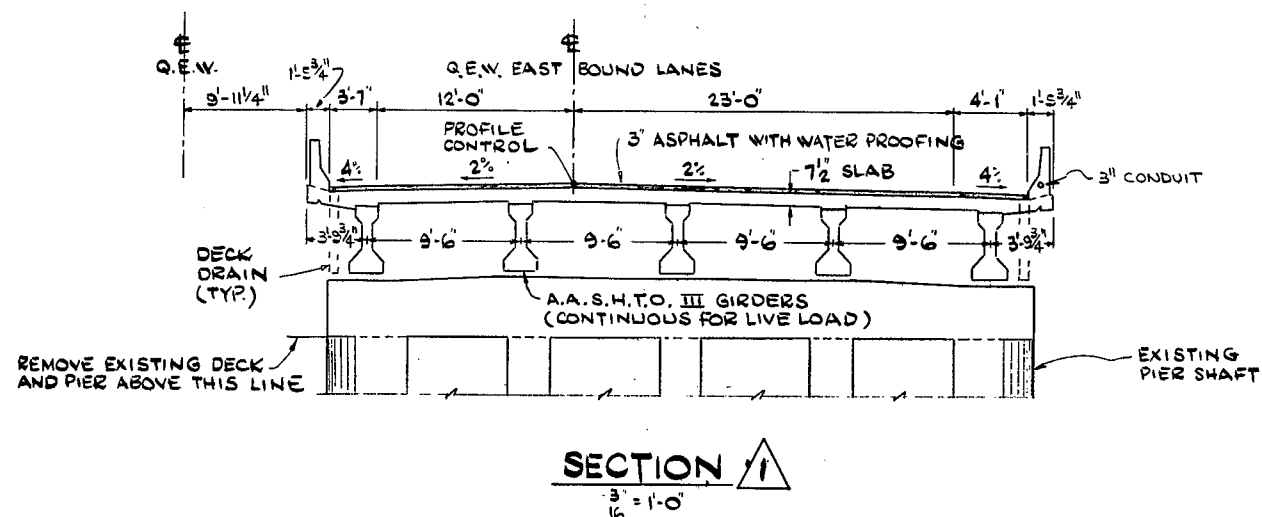
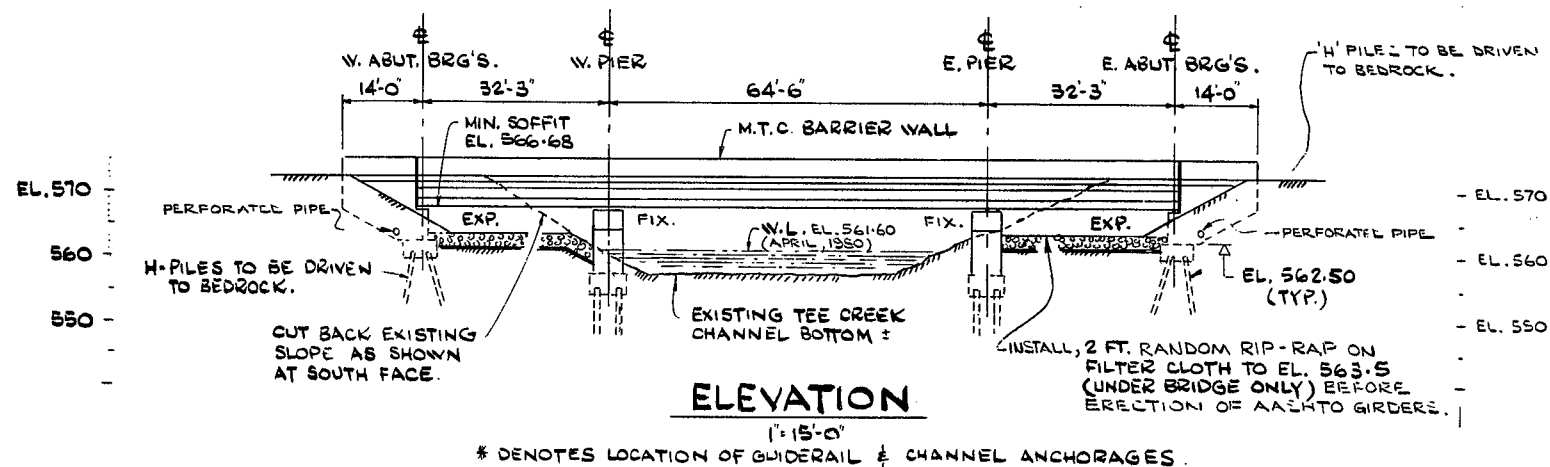
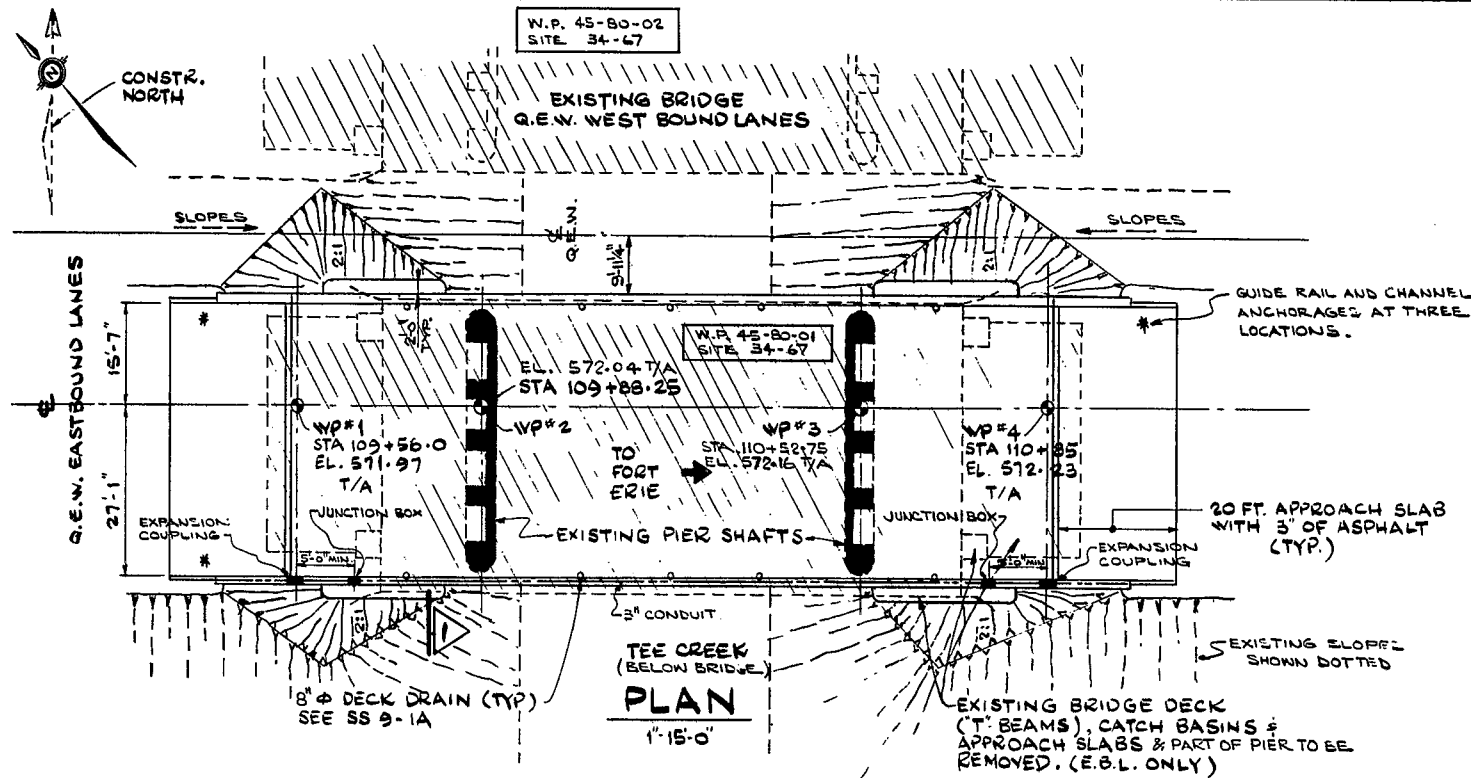
**Golder Associates**

Date: 30-Oct-13



# **APPENDIX C**

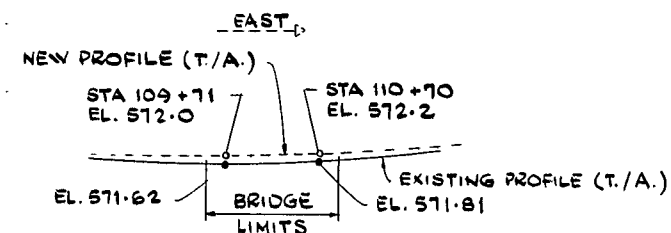
## **Foundation Drawings of the Existing South Bound Lane Bridge**



#### LIST OF DRAWINGS :-

- E.B.L. STRUCTURE DRAWINGS :-
- 34-67-1, GENERAL ARRANGEMENT.
  - 2, BOREHOLE LOCATIONS & SOIL STRATA.
  - 3, FOUNDATION LAYOUT.
  - 4, EAST ABUTMENT LAYOUT.
  - 5, WEST ABUTMENT LAYOUT.
  - 6, WINGWALL DETAILS.
  - 7, PIER CAPPING BEAM & BEARING DETAILS.
  - 8, DECK LAYOUT & SCREED ELEVATIONS.
  - 9, PRESTRESSED GIRDERS.
  - 10, DECK REINFORCEMENT DETAILS.
  - 11, BARRIER WALL.
  - 12, 20 FT APPROACH SLAB.
  - 13, STANDARD DETAILS I
  - 14, STANDARD DETAILS II
  - 15, STANDARD DETAILS III
  - 16, BRIDGE DATE & SITE NUMBER DATA.
  - 17, AS CONSTRUCTED ELEVATIONS & DIMENSIONS.
  - 18, BRIDGE ELECTRICAL DETAILS - TYPE IV
- W.B.L. STRUCTURAL DRAWINGS :-
- 19, GENERAL LAYOUT.
  - 20, SECTIONS AND REPAIR DETAILS.

NOTE RE: DECK REINFORCING :-  
TO ACHIEVE THE MINIMUM CLEAR COVER OF 2" SPECIFIED, THE TOP LAYER OF DECK REINFORCING SHALL BE PLACED PRIOR TO PLACING CONCRETE, WITH A CLEAR COVER OF 2 1/2" ± 1/2" TOLERANCE.



#### PROFILES OF Q.E.W. E.B. LANES

T/A DENOTES TOP OF ASPHALT.  
W.P. DENOTES WORKING POINT.



DIST. No. 4  
CONT No 81-57  
WP No 45-80-01

SUPERSTRUCTURE REPLACEMENT OF  
EXISTING QEW-TEE CREEK BRIDGE  
(E.B.L. STRUCTURE ONLY)  
GENERAL ARRANGEMENT

SHEET  
6

Morrison, Hershfield,  
Burgess & Huggins, Limited  
Consulting Engineers

MHB

#### GENERAL NOTES:

##### 1. CLASS OF CONCRETE

- PRESTRESSED GIRDERS — 35 MPa
- DECK, DIAPHRAGM, PIER CAPPING, BEAM, ABUTMENTS, AND BARRIER WALLS — 30 MPa
- REMAINDER — 20 MPa

##### 2. CLEAR COVER TO REINFORCING STEEL:

- FOOTINGS 3"
- DECK TOP 2", BOTTOM 1 1/2"
- BARRIER WALLS 2"
- APPROACH SLABS 2"
- PIER CAPPING BEAMS, ABUTMENTS & WINGWALLS 2"
- OR AS NOTED ON DRAWINGS.

##### 3. REINFORCING STEEL

- GRADE 400
- REINFORCING BARS WITH THE DESIGNATION "C" AT THE END OF THE BAR MARK SHALL BE EPOXY COATED BARS

##### 4. CONSTRUCTION NOTES:

- THE CONTRACTOR IS RESPONSIBLE FOR FINISHING THE BEARING SEATS DEAD LEVEL TO THE SPECIFIED ELEVATIONS WITH A TOLERANCE OF ± 1/8"
- NO CONCRETE SHALL BE PLACED ABOVE THE ABUTMENT BEARING SEATS UNTIL CONCRETE IN THE DECK HAS BEEN PLACED, UNLESS OTHERWISE NOTED ON DRAWINGS.
- ALL EXPOSED EDGES OF CONCRETE ARE TO BE CHAMFERED 3/4"

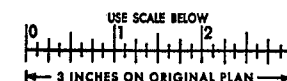
##### 5. CONCRETE QUANTITIES:

- CONCRETE QUANTITIES ARE LISTED BELOW FOR THE APPROPRIATE LUMP SUM TENDER ITEMS :-
- a) CONCRETE IN PIER CAPPING BEAMS, AND CONCRETE IN ABUTMENTS AND WINGWALLS. — 127 CU.YDS
- b) CONCRETE IN DECK AND DIAPHRAGMS. — 176 CU.YDS
- c) CONCRETE IN BARRIER WALLS. — 34 CU.YDS
- d) CONCRETE IN APPROACH SLABS — 53 CU.YDS

6- FOR CONSTRUCTION SEQUENCE SEE DRAWING 19.

7- VERIFY DIMENSIONS AND DETAILS OF EXISTING WORK ON SITE.

FOR REDUCED PLAN

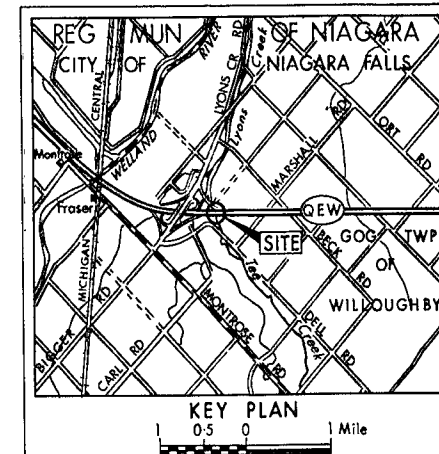


REVISIONS	DATE	BY	DESCRIPTION
DESIGN REH	CHECK A.T.	LOADING HS20-44	DATE 27 OCT 80
DRAWING JB	CHECK S.B.	SITE No 34-67	DWG 1



CONT No 81-57  
WP No 45-80-01

TEE CREEK  
BORE HOLE LOCATIONS & SOIL STRATA  
SHEET  
7



# LEGEND

- Bore Hole
- ⊕ Dynamic Cone Penetration Test (Cone)
- ⊕ Bore Hole & Cone
- 'N' Blows/ft (Std Pen Test 350ft lbs energy)
- CONE Blows/ft (60° Cone, 350ft lbs energy)
- W.L. at time of investigation Apr 1980
- W.L. NOT established for Bore Holes 2 & 3

No	ELEVATION	STATION	OFFSET
1	571.6	110+14	12' RT
2	571.6	110+83	29' RT
3	571.1	109+60	29' LT

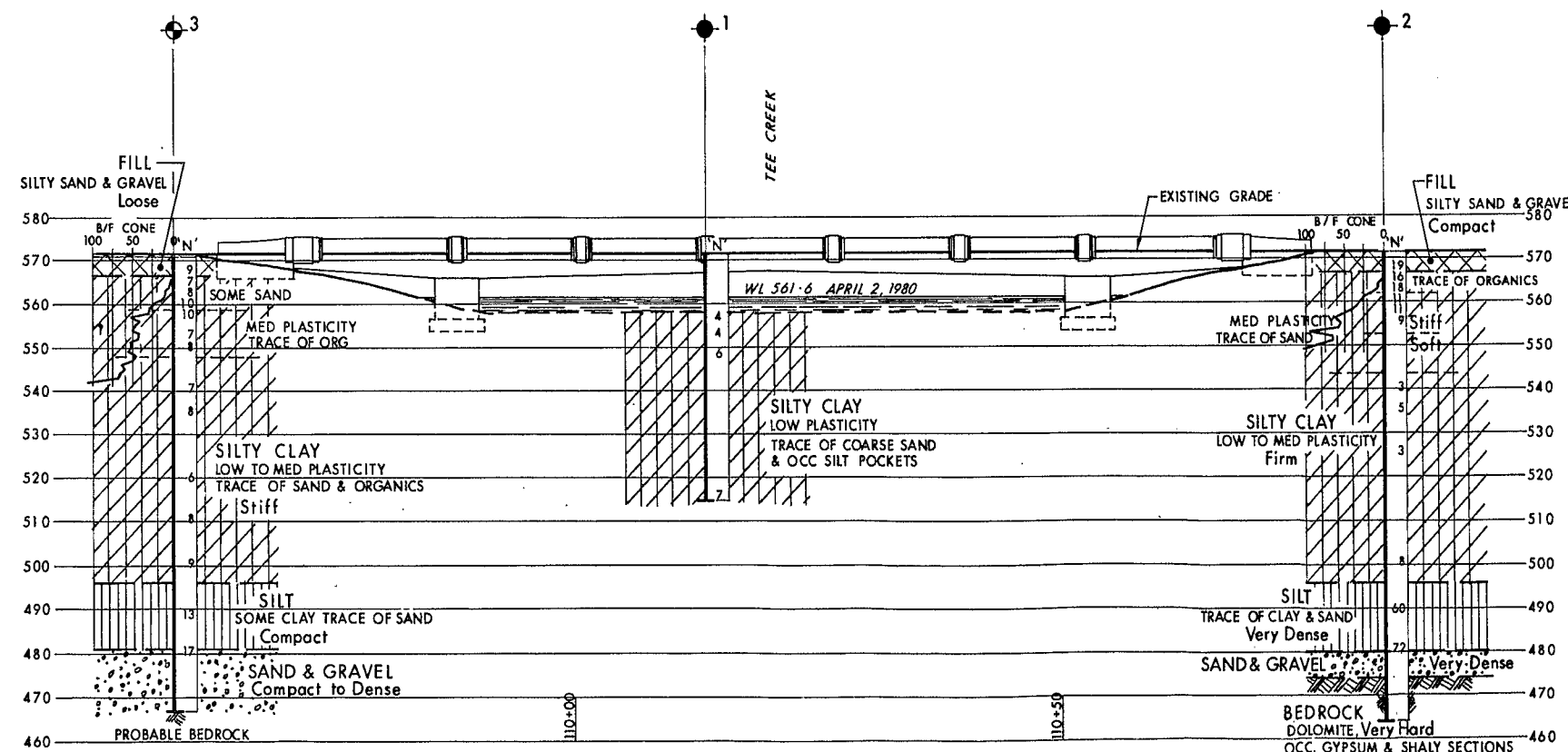
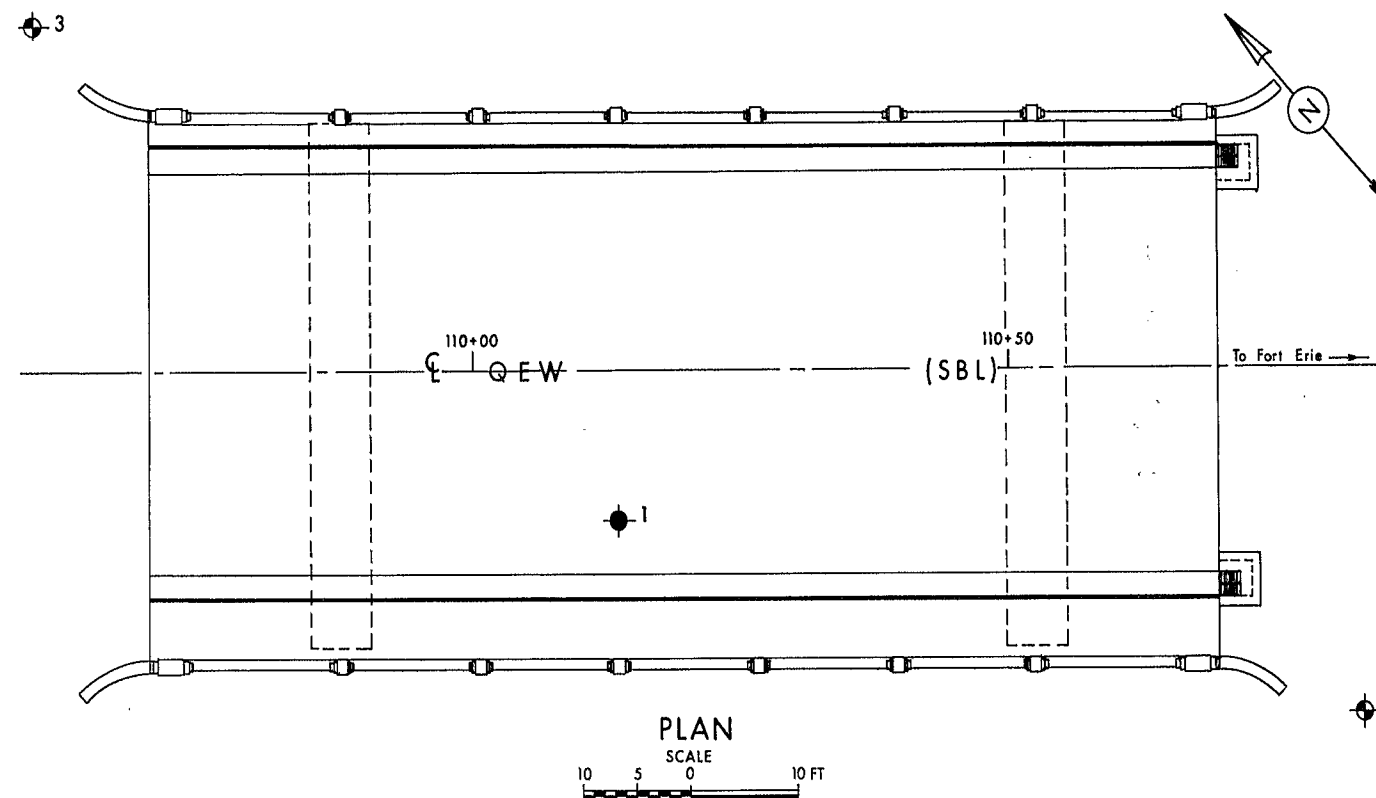
## NOTE

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

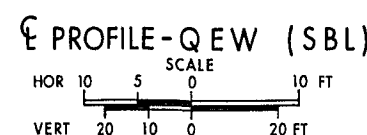
REVISIONS	DATE	BY	DESCRIPTION

GEOCRE No 30 M3-174

HWY No QEW (SBL)	DIST 4
SUBMITT K [CHECKED] DATE June 4, 1980	SITE 34-67
DRAWN R S [CHECKED] APPROVED	DWC 2



NOTE:  
The complete foundation investigation file for this project may be examined at the Engineering Materials Office, Downsview. Information contained in this file and any supplementary files is specifically excluded in accordance with the conditions of Section 102-2 of Form 100.



DIST. No. 4  
CONT No 81-57  
WP No 45-80-01

SHEET  
8

SUPERSTRUCTURE REPLACEMENT OF  
EXISTING Q.E.W. - TEE CREEK BRIDGE  
(E.B.L. STRUCTURE ONLY)  
FOUNDATION LAYOUT.

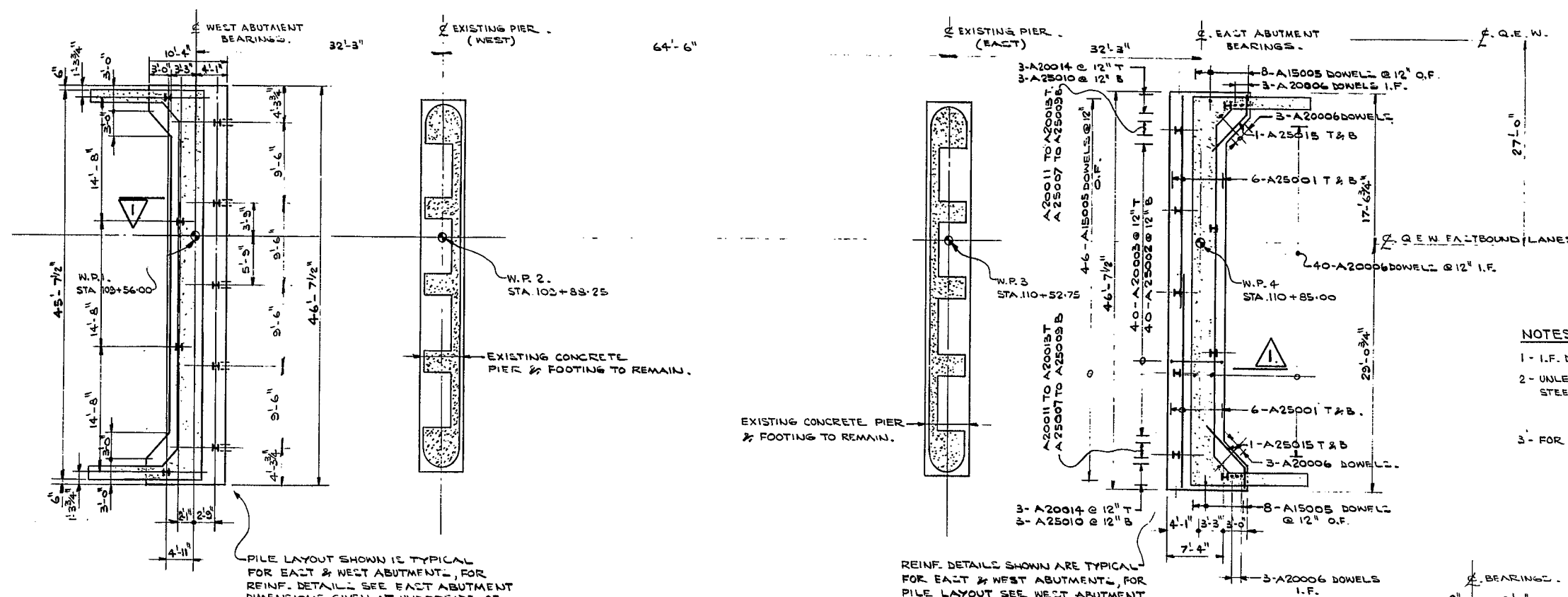
Morrison, Hershfield,  
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Consulting Engineers

MMB

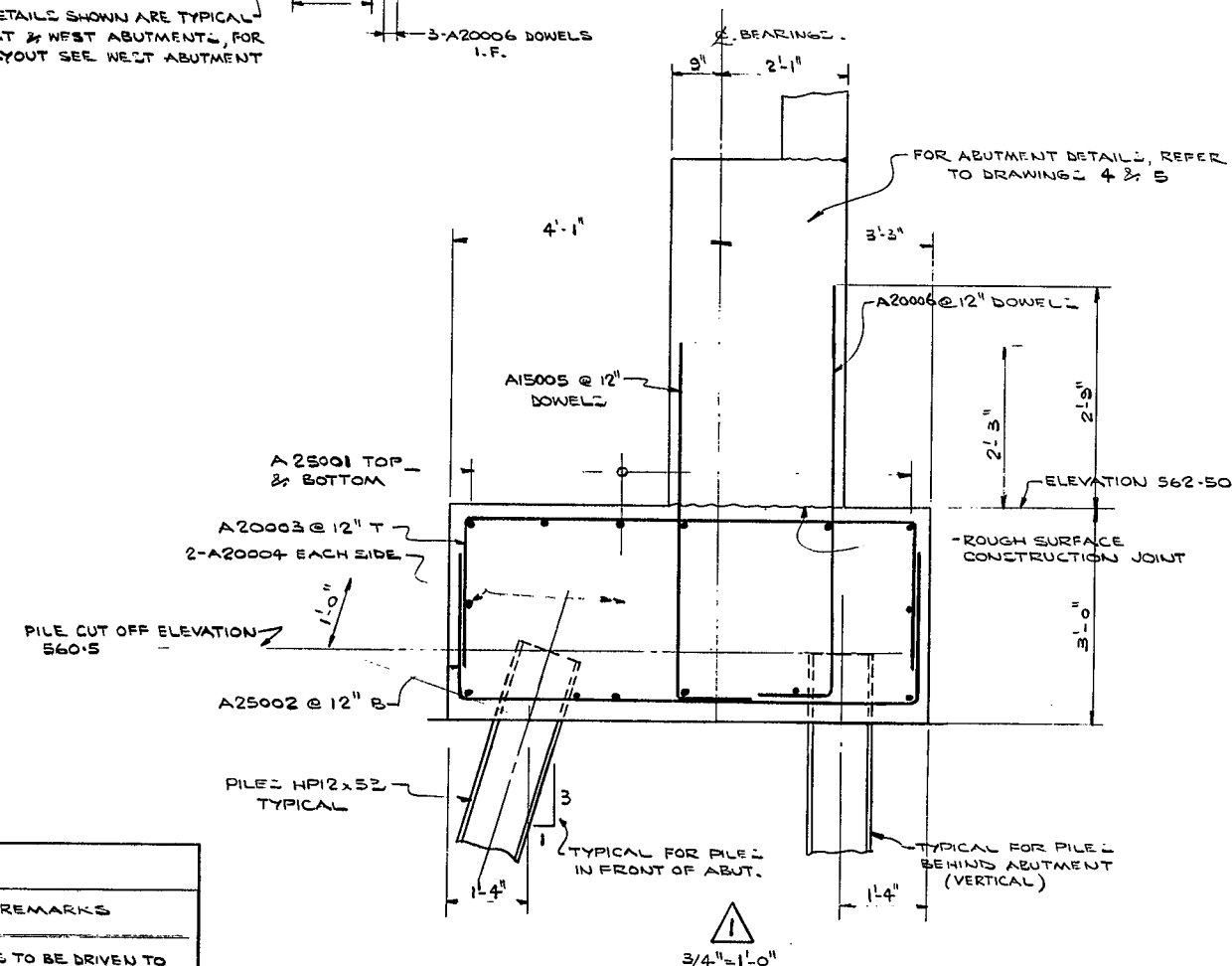


NOTES:-

- 1 - I.F. DENOTES INSIDE FACE ; O.F. DENOTES OUTSIDE FACE .
- 2 - UNLESS OTHERWISE NOTE, MINIMUM LAPs FOR REINFORCING  
STEEL SHALL BE :- 15M = 2'-3"  
20M = 2'-9"  
25M = 4'-0"
- 3 - FOR ABUTMENT AND WINGWALLS SEE DRAWINGS 4, 5 & 6.



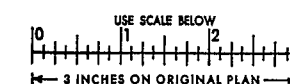
FOUNDATION PLAN.  
1/8" = 1'-0"



PILE DATA.				
LOCATION	Nº REQ'D	APPROX. LENGTH	TYPE	REMARKS
WEST ABUT.	4 S	94 FT. 100 FT.	HP12x53	PILES TO BE DRIVEN TO REFUSAL ON BEDROCK.
WEST PIER	EXISTING	-	-	SEE SS3-1 ON DRAWING IS FOR DRIVING SHOE AND SPLICE DETAILS.
EAST PIER	EXISTING	-	-	PILE LENGTH AS SHOWN ON THE DRAWINGS IS THE THEORETICAL LENGTH BELOW CUT OFF.
EAST ABUT.	4 S	87 FT. 92 FT.	HP12x53	

PILE STEEL TO BE GRADE 44W, TYPICAL.

FOR REDUCED PLAN



REVISIONS	DATE	BY	DESCRIPTION
DESIGN S.B.	CHECK A.T.	LOADING HS20-44	DATE 27 OCT 80
DRAWING BMM	CHECK S.B.	SITE No 34-67	DWG 3

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