Golder Associates Ltd.

2390 Argentia Road
Mississauga, Ontario, Canada   L5N 5Z7
Telephone: (905) 567-4444
Fax: (905) 567-6561

REPORT ON

DETAIL DESIGN
FOUNDATION INVESTIGATION AND DESIGN
BURLINGTON STREET OVER QEW
AND RHCE W-S RAMP (BRIDGE 6)
BURLINGTON STREET INTERCHANGE
G.W.P. 443-97-00
MINISTRY OF TRANSPORTATION, ONTARIO
HAMILTON, ONTARIO

Submitted to:

McCormick Rankin Corporation
2655 North Sheridan Way
Mississauga, Ontario L5K 2P8

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AND RHCE W-S RAMP (BRIDGE 6)
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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by McCormick Rankin Corporation (MRC) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out a foundation investigation as part of the detailed design of the Queen Elizabeth Way / Burlington Street Interchange to accommodate the proposed Red Hill Creek Expressway Interchange, in Hamilton, Ontario.

The terms of reference for the scope of work are outlined in Golder's proposal P21-1334, dated November 2002, that forms part of the Consultant's Agreement (Number P.O.2005-A-000482) for this project. This report addresses the proposed bridge carrying Burlington Street over the Queen Elizabeth Way (QEW) and the Red Hill Creek Expressway (RHCE) W-S Ramp (referred to as Bridge 6) as part of the interchange project. The work was carried out in accordance with the Quality Control Plan for this project dated November 2002.

The investigation was supplemented with information contained in the following Golder Associates report:

- Foundation Investigation and Design, Embankments, Queen Elizabeth Way / Red Hill Creek Expressway and Burlington Street Interchanges, Agreement No. 9820-7411-2805, Hamilton, Ontario, dated January 1999.

2.0 SITE DESCRIPTION

The site is located directly adjacent to and west of the existing Burlington Street bridge over the QEW and associated ramps. The south shore of Lake Ontario is located approximately 200 m north of the site (see key plan on Drawing 1). Red Hill Creek is present about 200 m to the south of the site and flows roughly in an west to east direction. The terrain in this area is generally flat-lying and grassy with occasional treed areas. The existing Burlington Street embankment rises about 8 m above the existing ground; the grade of the QEW is at about Elevation 77 m in the vicinity of the bridge. The City of Hamilton Pumping Station is located to the northwest of the bridge site, and a series of pipes, variable in depth and diameter, associated with the pumping station run diagonally through the site.

It should be noted that for description purposes, the north direction is assumed to be towards Lake Ontario.

3.0 INVESTIGATION PROCEDURES

3.1 Foundation Investigation

The field work was carried out between August 27 and September 10, 2003 at which time three (3) boreholes, numbered BR6-1, BR6-2 and BR6-3 were advanced. Boreholes 2 and BESR-5 were advanced at the site as part of the investigation carried out by Golder in 1998. The locations of the boreholes in plan are shown on Drawing 1.

The current field investigation was carried out using a track-mounted CME 75 drill rig supplied and operated by Geo-Environmental Drilling Ltd. of Milton, Ontario. The boreholes were advanced using 210 mm O.D. continuous flight hollow stem augers. Soil samples were obtained at intervals ranging from 0.75 m to 1.5 m in depth, using a 50 mm outer diameter (O.D.) split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures. An automatic hammer was used for purposes of obtaining SPT ‘N’ values. In situ ‘N’ vanes shear strength testing were obtained where appropriate, in the clayey strata. Samples of the bedrock were obtained using an ‘NQ’ size rock core barrel.

The three new boreholes (BR6-1, BR6-2, and BR6-3) were extended into the bedrock by coring and were advanced to between 18.5 and 18.6 m below the existing ground surface (including rock coring). The groundwater conditions in the open boreholes were observed during the drilling operations and piezometers were installed in Boreholes BR6-2 and BR6-3 to permit monitoring of the groundwater level at these locations. The piezometers consist of a 25 mm outside diameter rigid PVC tubing with a 0.3 m long slotted tip that is sealed at a selected depth within the boreholes. The installation details and water level readings are described on the Record of Borehole sheets that follow the text of this report.

The field work was supervised throughout by members of our engineering and technical staff who located the boreholes, arranged for the clearance of underground service locations, supervised the drilling, sampling and in-situ testing operations, logged the boreholes, and examined and cared for the soil and rock samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to our Mississauga geotechnical laboratory where the samples underwent further visual examination and laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards as appropriate. Classification testing (water content, Atterberg limits and grain size distribution) was carried out on selected samples. Point load testing was carried out on samples of the rock core.

The boreholes were laid out in the field by J.D. Barnes Surveying Ltd. using the NAD 83 MTM (Zone 12) co-ordinate system and the geodetic datum for elevation. Where the boreholes were shifted at the time of drilling, the northings, eastings and elevations of the as-drilled boreholes were measured in the field relative to the staked locations by members of our engineering staff.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

The QEW in this area follows the shoreline of Lake Ontario and lies mainly in the Iroquois Plain physiographic region. The Iroquois Plain is generally composed of shallow sandy materials deposited on the bed of the glacial Lake Iroquois. The area is also referred to as the Niagara Fruit Belt (Chapman and Putnam, "The Physiography of Southern Ontario", 3rd Edition, 1984). The bedrock in the area of the site is shale of the Queenston Formation, the bedrock is typically at depths of 10 m or deeper below ground surface. There are infilled bedrock valleys known to exist in the general area; in particular at the Burlington Skyway.

The overburden at the site consists predominantly of two main till sheets laid during two distinct glacial events; the Halton till and the Wentworth till. The Wentworth till is predominantly sandy silt till and is the lower till sheet at the site. The Halton till is present over the lower till and is predominantly clayey silt to silty clay with low plasticity.

4.2 Subsoil Conditions

The detailed subsurface soil, bedrock and groundwater conditions as encountered in the boreholes advanced during this investigation and in the 1998 investigation, together with the results of the laboratory tests carried out on selected soil samples, are given on the attached Record of Borehole sheets and in Appendix A following the text of this report. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests (SPTs). These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Further, subsurface conditions will vary between and beyond the borehole locations. The inferred soil stratigraphy based on the results of the boreholes at the bridge location are shown on Drawing 1.

In general, the subsoils at the site consist of a surficial layer of silty sand to silty clay fill overlain by a thin discontinuous layer of topsoil. The surficial fill deposits are underlain by a thick deposit of grey clayey silt till. The clayey silt till is underlain by a thin deposit of sandy silt till containing shale and limestone fragments in turn underlain by clayey silt residual soil overlying shale bedrock of the Queenston Formation. In the boreholes at the site where bedrock was proven by coring, the total overburden thickness was about 15 m. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Topsoil

Topsoil was encountered at the existing ground surface in Boreholes BR6-2 and BR6-3 on the north and south sides of the QEW and in BESR-5, located about 70 m south of the QEW. The surface of the topsoil ranged between Elevation 76.8 m and 78.1 m and the thickness is between 0.1 m to 0.3 m.

4.2.2 Fill

Fill materials were encountered, either immediately below the topsoil or at ground surface in all the boreholes. The fill ranges in composition from sandy silt containing some clay and trace gravel to clayey silt to silty clay containing some sand and gravel to silty sand. The fill ranges in colour from brown to grey to black and contains trace to some organics and rootlets. Brick fragments were noted in Borehole BR6-3 and BESR-5, located at the north and south ends of the site. The fill extends to between 2.3 and 3.5 m depth below ground surface at the borehole locations.

At the south end of the site, the fill appears to be associated with the existing Burlington Street or associated ramp alignments (Boreholes BR6-1, BR6-2, BESR-5). At the north end of the site, the fill may be associated with the QEW or utility works in the area. Although not encountered in the boreholes, fill materials are inherently variable and could contain boulders/cobbles and rubble.

Standard Penetration Testing (SPT) measured ‘N’ values within the granular fills range between 14 and 24 blows per 0.3 m of penetration, indicating a compact relative density. SPT measured ‘N’ values within the cohesive fills ranged between 4 and 27 blows per 0.3 m of penetration indicating a soft to very stiff consistency. The natural water content measured on samples of the fill ranged from 9 to 24 percent.

4.2.3 Clayey Silt Till

A deposit of clayey silt till was encountered underlying the fill in all the boreholes. The clayey silt till contained trace to some sand and gravel. This deposit is considered to be the “Halton” till sheet. The surface of the clayey silt till was encountered between Elevations 73.4 m to 74.6 m in the boreholes and the thickness varied from 8.7 m to 9.3 m. This deposit was not fully penetrated in Borehole BESR-5.

It should be noted that although cobbles and/or boulders were not noted in the boreholes within the clayey silt till deposit at this site, cobbles and boulders are common in glacially derived materials.

Standard Penetration Testing (SPT) measured ‘N’ values ranged between 9 and 69 blows per 0.3 m of penetration and were typically less than 40 blows, indicating a stiff to hard consistency. In general, the stiff portion of the deposit (N values less than 15 blows per 0.3 m of penetration) was encountered between about Elevation 68 m and 71 m. In situ field vane testing was carried out within the stiff portion of the clayey silt till deposit in Borehole BR6-3 and the measured undrained shear strength ranged between 62 kPa and 100 kPa also confirming a stiff consistency. One grain size distribution test result on a sample obtained from Borehole BR6-2 is shown on Figure 1.

Atterberg limits testing was carried out on three samples of the clayey silt till deposit. The liquid limits ranged from 30 to 37 percent and the plastic limits ranged from 14 to 18 percent giving an average plasticity index of 17 percent. The results of the testing indicate that the deposit is a clayey silt of low to intermediate plasticity. The results of the Atterberg limits tests are plotted on the plasticity chart on Figure 2.

The natural water contents measured on selected samples of the clayey silt till deposit ranged between 13 to 21 percent, with an average of 17.

4.2.4 Sandy Silt Till

In Boreholes BR6-1, BR6-2, BR6-3, and 2, a deposit of red-brown sandy silt till was encountered below the grey clayey silt till. This deposit contains varying amounts of clay, gravel, shale and limestone pieces. This deposit is considered to be the “Wentworth” till sheet. The surface of the deposit was encountered between Elevations 64.6 m and 65.6 m and ranged between 2.1 m and 2.8 m in thickness.

Although cobbles and/or boulders were not observed within the till deposit in the boreholes put down at this site, boulders are common in glacially derived materials and have been encountered in this deposit in boreholes elsewhere at the interchange site.

The measured SPT ‘N’ values within the till deposit were greater than 100 blows per 0.3 m of penetration, indicating a very dense state of packing. One grain size distribution test result on a sample obtained from Borehole BR6-2 is shown on Figure 3. The natural water content measured on selected samples of the sandy silt till deposit varied between 8 and 12 percent.

4.2.5 Clayey Silt Residual Soil

In Boreholes BR6-1, BR6-2 and BR6-3, a thin layer of residual soil was encountered below the sandy silt till. The residual soil is derived through weathering of the shale bedrock and is essentially comprised of clayey silt. This deposit generally has a till-like structure but can contain zones of rock-like structure. The surface of the deposit was encountered between Elevation 62.9 m and 63.1 m. This deposit ranged from 1.4 m to 1.6 m in thickness.

The measured SPT 'N' values within the residual soil deposit were greater than 100 blows per 0.3 m of penetration, indicating a hard consistency. The natural water content measured on two samples of this deposit were between 8 and 9 percent.

4.2.6 Bedrock

In Boreholes BR6-1, BR6-2 and BR6-3 shale bedrock was encountered below the residual soil and in Borehole 2, the bedrock was encountered below the sandy silt till. The bedrock surface was encountered between Elevations 61.4 m and 61.8 m. The bedrock in Borehole 2 was confirmed by augering and split spoon sampling and in Boreholes BR6-1, BR6-2 and BR6-3, bedrock was confirmed by coring for a depth of 3.2 m to 3.4 m.

The bedrock samples obtained consist of reddish-grey, highly to slightly weathered, thinly layered, fine grained, very weak to medium strong calcareous shale of the Queenston Formation. Seams and layers of medium to very strong limestone/siltstone were present within the shale bedrock. These seams were measured to be up to 230 mm thick, but were typically less than 50 mm thick. The depth and thickness of these seams are given on the Record of Drillhole Sheets. The Total Core Recovery was between 98 percent and 100 percent. The Rock Quality Designation (RQD) measured on the core samples in the boreholes ranged from about 85 to 100 percent, indicating a rock mass of good to excellent quality.

Point load strength tests were performed on selected samples of the rock core from the boreholes. Diametral point load strength index values are shown on the Record of Drillhole Sheets. Diametral point load index values on core samples of the shale range from 0.17 MPa to 2.0 MPa which corresponds to an estimated unconfined compressive strength (UCS) ranging from 4 MPa to 48 MPa. The axial point load index values on core samples of the shale were between 0.86 MPa and 2.16 MPa corresponding to approximate UCS values of between 20 MPa and 50 MPa. Using the Intact Rock Strength Classification table, these values indicate that the shale is classified as very weak to medium strong. On the limestone/siltstone core samples, the diametral point load index values were between 1.77 MPa and 4.13 MPa corresponding to approximate UCS values between 41 MPa and 95 MPa. The axial point load index values were between 1.9 MPa and 5.8 MPa corresponding to approximate UCS values between 45 MPa and 133 MPa. This indicates that the limestone/siltstone interlayers are classified as medium strong to very strong.

4.2.7 Groundwater Conditions

The water levels were noted during and after the drilling and coring operations in the boreholes. Piezometers installed in Boreholes BR6-2 and 2 were sealed into the bedrock and the piezometer installed in Borehole BR6-3 was sealed within the residual soil deposit. Details of the piezometer installations are shown in the Record of Borehole Sheets following the text of this report. The water levels in the open holes upon completion of drilling and in the piezometers approximately one month to five months after installation are summarized in the table below.

Borehole	***Installations***	***Ground Surface Elevation (m)***	***Ground Water Level Depth (m)***	***Ground Water Level Elevation (m)***	***Date***
BESR-5	Open borehole	78.1	n/a	Dry	n/a
BR6-1	Open borehole	76.6	1.5	75.1	n/a
BR6-2	Piezometer	76.8	2.0	74.8	October 22, 2003
BR6-3	Piezometer	76.9	2.0	74.9	October 22, 2003
2	Piezometer	76.3	2.0	74.3	November 10, 1998

The groundwater table is likely controlled by the water level in Lake Ontario and is expected to slope slightly downwards towards the lake. It should be noted that groundwater levels in the area are subject to seasonal fluctuations and periods of precipitation.

4.3 Closure

This report was prepared by Miss Sarah Poot, P.Eng., a senior geotechnical engineer. The technical aspects were reviewed by Ms. Anne S. Poschmann, P. Eng., a Principal with Golder Associates Ltd. Mr. Fintan J. Heffernan, P. Eng., a Designated MTO Contact for Golder, conducted a quality control review of the report.

GOLDER ASSOCIATES LTD.

LICENSED PROFESSIONAL ENGINEER
S. E. M. POOT
PROVINCE OF ONTARIO

Sarah E. M. Poot, P.Eng.,
Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
F. J. HEFFERNAN
PROVINCE OF ONTARIO

Fintan J. Heffernan, P.Eng.,
Designated MTO Contact

Anne S. Poschmann, P.Eng.,
Principal

SEP/ASP/FJH/sep
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PART B

FOUNDATION DESIGN
BURLINGTON STREET OVER QEW
AND RHCE W-S RAMP (BRIDGE 6)
BURLINGTON STREET INTERCHANGE
G.W.P. 443-97-00
MINISTRY OF TRANSPORTATION, ONTARIO
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5.0 ENGINEERING RECOMMENDATIONS

This section of the report provides foundation design recommendations for the proposed Bridge 6 as part of the QEW/Burlington Street Interchange. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during two phases of the subsurface investigation at this site. The interpretation and recommendations provided are intended only to provide the designers with sufficient information to assess the feasible foundation alternatives and to design the proposed structure foundations. As such, where comments are made on construction they are provided only in order to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods, scheduling and the like.

It is understood that the proposed bridge will carry the re-aligned Burlington Street over the Queen Elizabeth Way and the proposed Red Hill Creek Expressway west-to-south (W-S) ramp. The existing Queen Elizabeth Way will be widened as part of the overall project. The proposed bridge is a 3-span structure with spans of 39 m, 45 m and 39 m in length. Retaining walls are proposed beyond the limits of the wing walls at the north approach; this report addresses the northeast wing wall/retaining wall as well as the first 20 m of the wing wall/retaining wall on the northwest side of the bridge. The remainder of the northwest retaining wall (i.e. Retaining Wall 4) is addressed under separate cover by Golder (High Fills/Retaining Wall report, dated April 2005). The proposed embankments will be up to about 8.2 m and 7.6 m in height at the north and south approaches, respectively.

Several existing utility pipes run diagonally below the site. The location of these pipes with respect to the bridge foundation elements is shown on Drawing 1 and discussed in Section 5.6.1. The pipe size, material, condition and other details are given in the following table:

Utility	*Diameter (mm)*	*Type*	*Location Relative to Foundation Elements*	*Proposed Works*
Electrical Conduits	9x90	n/a	• Crosses under south abutment	• To be abandoned
Trash discharge main	300	concrete	• 4 m away from southeast corner of abutment, • 7 m away from west side of south pier	• To be abandoned
Watermain	300	concrete	• 7 m away from southeast corner of abutment, • 4 m away from west side of south pier	• To be abandoned

Utility	*Diameter (mm)*	*Type*	*Location Relative to Foundation Elements*	*Proposed Works*
Overflow drain	1200	concrete	• 5 m away from southeast corner of abutment and west side of south pier	• To be left in place, can be re-habilitated if necessary • Vibration not an issue during bridge construction
Discharge	1500	Non-reinforced concrete	• Located under west end of south pier	• To be abandoned
Discharge	2100	Reinforced concrete	• Located under east end of south pier • Pipe is in good condition • Steel liner to be extended through this area prior to bridge construction • Invert of pipe at Elevation 70 m	• Vibration a concern during south pier construction
Discharge	2700	Steel-lined tunnel	• 4 m east of north pier • Invert of pipe about 11 m below the bedrock surface (El. 51 m)	• Tunnel construction underway • Vibration a concern during north pier construction

The general arrangement drawing was provided to us by MRC in October 2004 and was used in preparation of the foundation drawing (Drawing 1).

5.1 General

Various alternatives for the bridge abutment foundations were considered and a summary of the advantages/disadvantages, costs and risks/consequences of each alternative is presented in Table 1, following the text of this report. Spread footings founded on the competent sandy silt till are not recommended for support of the bridge due to the deep excavation required, the need for groundwater control, and the temporary shoring that would be required. Spread footings founded at shallower depth within the clayey silt till are not recommended due to the low axial resistance and potential settlement of the underlying clayey silt till. However, shallow spread footings on the clayey silt till are recommended for the retaining walls if one of the settlement mitigation schemes are implemented. It is considered that piles driven to found on the very dense sandy silt till or caissons socketted into the very dense sandy silt till or bedrock for support of the piers, abutments and wing walls are the most feasible options from a geotechnical/foundation perspective.

Shallow foundation could, however, be considered for support of the retaining walls at the north abutment as discussed in Section 5.5.

5.2 Deep Foundations

Steel H-piles founded within the very dense sandy silt till or caissons founded within the very dense sandy silt till or on/within the bedrock may be used for support of the bridge abutments and piers.

The existing pipes are located within the clayey silt till at about Elevation 70 m; the deep foundations would be extended a minimum of 5 m below the pipe inverts to reach the founding stratum. Due to the close proximity of existing pipes, care must be taken to ensure that the pipes are not damaged or disturbed during pile driving or augering for caissons. At this site, pre-augering is required through the upper very stiff to hard portion of the clayey silt till, adjacent to the pipe, to Elevation 69 m. This pre-augering is required for the driven piles in the area of the existing utility pipelines in order to reduce the amount of vibrations and provide guidance to the pile to minimize the potential for deviation in the alignment. A separation distance of at least 1 m should be provided between the piles and the pipes. In addition, the portion of the deposit just below the pipe inverts has a stiff consistency and could be subject to settlement due to vibrations during pile driving/caisson augering and particularly during setting of the pile into the very dense sandy silt till. Special considerations with respect to the pile set criteria must be applied (as discussed in Section 5.2.1.1).

The excavation for caisson construction has the potential of changing the stress conditions in the soils surrounding the pipes if the excavation is in relatively close proximity to the pipes. A minimum distance of three caisson diameters should be maintained between the outer edges of the caissons and the existing pipes to minimize the potential for impacting the pipes. Given the proximity of the pipes to the south abutment, caissons are not feasible at this location.

A non-standard special provision (NSSP) should be included in the Contract Documents indicating the depth of pre-augering required, the piles which require pre-augering, and the vibration monitoring required during pile installation (see also Section 5.8). The pre-augering information should also be shown on the contract drawings.

5.2.1 Steel H-Pile Foundations

Steel H-piles driven through the stiff to hard clayey silt till and founded within the very dense sandy silt till (where SPT ‘N’ values are greater than 100 blows per 0.3 m of penetration) may be used for support of the abutments, piers and wing walls. In areas where the foundation units are adjacent to the existing pipes that are to be maintained, pre-augering will be required to Elevation 69 m to penetrate through the very stiff to hard clayey silt till adjacent to the pipes in order to minimize potential vibrations, to provide guidance to the pile and to reduce the potential for deviation in alignment.

It should be noted it is possible that the pipes were constructed in open cut, and the open cut trenches backfilled with earth material. In this case, a temporary liner may be required through the fill materials (i.e. previous open cut trenches) to prevent loss of materials into the pre-augered hole.

Where the piles are located at a distance greater than 3 m from the existing pipes, piles could be driven to found within the sandy silt till deposit without pre-augering; however, this distance may have to be altered in the field depending on the results of the vibration monitoring. Since the combined thickness of the very dense sandy silt till and the hard clayey silt residual soil overlying the bedrock is greater than 3 m, it is likely that practical refusal for the piles may be met within the very dense/hard deposits prior to reaching the bedrock surface.

If the piles are pre-augered, the pre-augered hole should be backfilled with loose sand similar to that used in backfilling of the CSP pipes in integral abutments.

For design, the following pile tip levels may be assumed for piles terminated within the very dense sandy silt till (assumed minimum 2 m penetration) or just into the bedrock. There should be provision made in the contract for dealing with varying pile lengths.

Foundation Location	***Relevant Borehole***	***Design Pile Tip Elevation (m)***	
		Very Dense Sandy Silt Till (see Note 1)	Shale Bedrock
South Abutment	BR6-1	63.5	61.0
Pier #1	BR6-2	63.0	61.0
Pier #2	BR6-2,BR6-3	63.0	61.0
North Abutment	BR6-3, 2	62.5	61.0

Notes: 1. Assumes 2 m penetration into the very dense sandy silt till deposit before reaching practical refusal.

Vibration monitoring would have to be carried out during pile installation to ensure that the vibration levels on the existing pipes are maintained within tolerable ranges as discussed in Section 5.8. In addition, the set criteria may have to be adjusted depending on the results of the monitoring.

5.2.1.1 Axial Geotechnical Resistance

For HP 310 x 110 piles driven to practical refusal within the very dense sandy silt till or driven to found on the bedrock, the factored axial resistance at Ultimate Limit States (ULS) and the axial geotechnical resistance at Serviceability Limit States (SLS) which may be used for design are given below:

Founding Stratum	***Factored Axial Resistance (ULS)***	***Axial Resistance (SLS)***
Sandy Silt Till	1,400 kN	1,100 kN
Shale Bedrock	2,000 kN	1,600 kN

It is anticipated that driven steel H-piles will hang up within the very dense sandy silt till and for this option, therefore, the pile capacities given for piles terminating in the very dense sandy silt till should be used.

If additional pile capacity is required, the piles that are sufficient distance away from the existing pipes could be founded on the shale bedrock provided that pre-augering is carried out to the surface of the bedrock. In this case, the pile capacities for piles terminating in the shale bedrock should be used. After the pre-augering is complete the piles are required to be driven to reach an appropriate set within the founding material below the base of the pre-augered hole. The piles that require pre-augering will be shown on the Contract Drawings.

Pile installation should be in accordance with SP903S01. The piles should be stiffened with MTO driving shoes or Titus "H" Standard Bearing Pile Points for protection during driving in accordance with SS103-12 and SP903S01 (Clause 903.05.03). For piles driven to found within the very dense till, the set should be established in accordance with the dynamic formula (Hiley) or by the application of the wave equation analysis procedure. The following note should be shown on the Contract drawing assuming that a resistance factor of 0.5 (in accordance with MTO Foundations requirements) is applied to the use of the Hiley:

– "Piles to be driven in accordance with Standard SS 103-11 using an ultimate capacity of 2,800 kN per pile but must be driven below EL 65.5 m at the north abutment, EL 64.5 m at the south abutment and EL 65 m at the south pier."

For the piles which are pre-augered and then driven to found on the shale bedrock, the following note should be used for the drawings:

– "Piles to be driven to bedrock."

5.2.1.2 Downdrag Load (Negative Skin Friction)

The embankment loading will cause consolidation settlement of the underlying “stiff” clayey silt till deposit. This stiff zone occurs over a limited thickness and the settlement is expected to be about 25 mm under the proposed embankment loading (as detailed in Section 5.4.3). Negative skin friction or downdrag loads will need to be taken into account during design of the piles supporting the abutments as a consequence of this consolidation settlement. The abutment pile structural design should be based on the full downdrag load acting on the piles within and above the stiff till zone. The estimated unfactored downdrag load acting on the HP 310x110 piles may be taken as 275 kN per pile at the abutment locations. Downdrag loads do not apply to the pier locations since no load is being imposed over the surrounding ground.

The load calculated in this manner is an unfactored load. The structural capacity of the piles must be checked for the factored dead and downdrag loads in accordance with Section 6.8.4 of the CHBDC for ULS conditions. The piles are designed either as end-bearing on the bedrock or based on combined shaft (within the lower portions of the clay till) and end-bearing within the very dense sand till. For these conditions (basically classified as non-yielding foundations), the settlement of the piles is largely governed by compression of the pile and will not be greater than 25 mm under the combined SLS and downdrag loading.

One method to reduce downdrag loads would be to construct a preload embankment in the abutment areas and allow the settlement to occur prior to installing the piles. If preloading to the full embankment height is possible over the entire abutment area, then downdrag loads would not have to be considered. If, due to construction staging proximity to the QEW, only partial preloading is possible in the abutment areas, then the full downdrag loading would have to be considered. More details regarding preloading are given in Section 5.4.4.

If sub-excavation of the fill/floodplain deposits is carried out at the north abutment, settlement of the underlying ‘stiff’ clayey silt till will still occur under the embankment loading and therefore, the full downdrag loading would have to be considered in the design. More details regarding sub-excavation are given in Section 5.4.4.

5.2.1.3 Resistance to Lateral Loads

Lateral loading could be resisted fully or partially by the use of battered steel H-piles. However, due to the close proximity of the buried pipes battered piles may not be able to be used in some locations. If vertical piles are used, the resistance to lateral loading will have to be derived from the soil in front of the piles.

The resistance to lateral loading in front of the pile may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, k_h, is based on the equations given below.

For cohesionless soils:

$$k_h = \frac{n_h z}{B}$$

Where

n_h is the constant of horizontal subgrade reaction, as given below
z is the depth (m)
B is the pile diameter/width (m)

For cohesive soils:

$$k_h = \frac{k_{s1}}{5B}$$

Where

B is the pile diameter/width (m)
k_{s1} is the constant of horizontal subgrade reaction, as given below

The following ranges for the value of n_h and k_{s1} may be assumed in the structural analysis. The range in values reflects the variability in the subsurface conditions as well as the two extremes of design: the requirement for flexibility in the case of integral abutments and the requirement for lateral support in the case of non-integral abutments and the pier.

Soil Unit	***Elevation***	***n_h (MPa/m)***	***k_{s1} (MPa/m)***
Backfill to pre-augered hole	Over the pre-augered length	2 to 5	--
Fill (loose to compact sandy silt to silty sand or soft to stiff clayey silt to silty clay)	77 to 74 m	3 to 7	10 to 20
Very stiff to hard clayey silt till	74 to 71 m	--	30 to 50
Stiff clayey silt till layer	71 to 68 m	--	15 to 30
Very stiff to hard clayey silt till	68 to 65 m	--	30 to 50
Very dense sandy silt till and hard clayey silt residual soil	65 to 62 m	9 to 15	--

A maximum lateral resistance of 180 kN at ULS for HP 310 x 110 piles and 80 kN at SLS is recommended.

Group action for lateral loading should also be considered when the pile spacing in the direction of the loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction in the direction of loading by a reduction factor, R, as follows:

Pile Spacing in Direction of Loading d = Pile Diameter	***Subgrade Reaction Reduction Factor***
8d	1.00
6d	0.70
4d	0.40
3d	0.25

The subgrade reaction reduction factor should be interpolated for pile spacings in between those shown on the above table.

5.2.1.4 Frost Protection

The pile caps should be provided with a minimum of 1.2 m of soil cover for frost protection.

5.2.2 Caissons

As an alternative to pile foundations, caissons socketted into the very dense sandy silt till deposit or into the shale bedrock could be used for support of the bridge abutments and piers. It should be noted that although the sandy silt till overlying the bedrock is relatively thin, this deposit is known to contain cobbles and boulders which may pose difficulties in advancing the caissons / temporary liners through to the bedrock surface.

In addition, socketting the caissons into the shale bedrock may require rock coring or churn drilling techniques due to the presence of harder limestone/siltstone layers. Significant vibrations could be induced during this process and as such, consideration could be given to founding the caissons on the surface of the bedrock.

The following design base elevations may be used at the bridge abutments and piers for caissons founded on the surface of or just into the sandy silt till and for caissons on the surface of the bedrock or socketted at least 2 m into the bedrock:

Foundation Location	***Relevant Boreholes***	***Design Caisson Founding Elevation (m)***		
		Very Dense Till	Surface of Bedrock	2m Socket into Bedrock
South Abutment	BR6-1	65.5	61.5	59.5
Pier #1	BR6-2	65.0		
Pier #2	BR6-2,BR6-3	65.0		
North Abutment	BR6-3, 2	64.5		

The groundwater level is at about 2 m below the ground surface and temporary liners will be required for groundwater control during caisson socketting.

5.2.2.1 Axial Geotechnical Resistance

The caissons will derive their axial resistance in part from end-bearing and in part from shaft friction. The factored axial geotechnical resistance at ULS and axial geotechnical resistance at SLS that may be used for design are given in the table below:

Caisson Diameter(m)	*Axial Resistance*			
	Very Dense Sandy Silt Till or Surface of Shale Bedrock		*Shale Bedrock (minimum 2 m socket)*	
	ULS	**SLS**	**ULS**	**SLS**
0.9	3,200 kN	2,800 kN	4,000 kN	n/a
1.5	6,600 kN	4,500 kN	8,000 kN	n/a
1.8	8,200 kN	5,600 kN	10,000 kN	n/a

For caissons founded at least 2 m into the shale bedrock, the resistance required to achieve 25 mm of settlement is greater than that given for ULS and therefore SLS conditions do not apply.

5.2.2.2 Downdrag Load (Negative Skin Friction)

The estimated unfactored downdrag load acting on the caissons at the abutments may be taken as shown in the table below:

Caisson Diameter (m)	*Unfactored Downdrag Load (kN)*
0.9	600
1.5	1,000
1.8	1,200

Other requirements for structural design with respect to downdrag load on the caissons should be in accordance with Section 5.2.1.2.

5.2.2.3 Resistance to Lateral Loads

The resistance to lateral loading for the caissons should be in accordance with Section 5.2.1.3, with the upper limit as determined through the use of the horizontal subgrade reaction formulas. The recommended maximum lateral resistance for the caissons is as follows:

Caisson Diameter (m)	*Factored Lateral Resistance at ULS (kN)*	*Lateral Resistance at SLS (kN)*
0.9	550	250
1.5	950	425
1.8	1100	500

5.2.2.4 Frost Protection

If pile caps are used, they should be provided with a minimum of 1.2 m of soil cover for frost protection.

5.3 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment stems and any associated wing walls / retaining walls will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill, on the magnitude of surcharge including construction loadings, on the freedom of lateral movement of the structure, and on the drainage conditions behind the walls. For this site location, the geotechnical seismic considerations do not impact on the design since it is within the lowest seismic zone given in CHBDC.

The following recommendations are made concerning the design of the abutment and retaining walls. It should be noted that these design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- Select free-draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) Granular ‘A’ or Granular ‘B’ Type II should be used as backfill behind the walls. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3501.00 and 3504.00.

- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem, in accordance with *CHBDC* Section 6.9.3 and Figure 6.9.3. Compaction equipment should be used in accordance with OPSS 501.06. Other surcharge loadings should be accounted for in the design, as required.

- The granular fill may be placed either in a zone with width equal to at least 1.2 m behind the back of the wall stem (Case I in Figure C6.9.1(l) of the *Commentary to the CHBDC*) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (Case II in Figure C6.9.1(l) of the *Commentary to the CHBDC)*.

- For Case I, the pressures are based on the existing and proposed embankment fill materials and the following parameters (unfactored) may be used:

	Earth Fill
Soil unit weight:	21 kN/m^3
Coefficients of static lateral earth pressure:	
Active, K_a	0.33
At rest, K_o	0.50

- For Case II, the pressures are based on the granular fill as placed and the following parameters (unfactored) may be assumed:

	GRANULAR 'A'	**GRANULAR 'B' TYPE II**
Soil unit weight:	22 kN/m^3	21 kN/m^3
Coefficients of static lateral earth pressure:		
Active, K_a	0.27	0.27
At rest, K_o	0.43	0.43

If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design. The movement to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure, may be taken as:

- rotation of approximately 0.002 about the base of a vertical wall;
- horizontal translation of 0.001 times the height of the wall; or
- a combination of both.

A restrained structure is typically culverts or rigid frame bridge where the rotational or horizontal movement is not sufficient to mobilize an active earth pressure condition. For this condition, an at-rest pressure plus any compaction surcharge should be included in the design of the structure.

5.4 Embankment Design and Construction

The proposed grade of the bridge varies from about Elevation 85.0 m to 84.0 m at the north and south approaches, respectively. The existing ground surface at the bridge site is at about Elevation 77.0 m resulting in approach embankments between 7 m and 8 m in height. As discussed in Section 5.1, wing walls are required at both abutments and retaining walls are proposed to retain the earth embankments behind the abutment on the north side of the bridge.

5.4.1 Subgrade Preparation and Embankment Construction

It is our understanding that it is not normal practice to carry out topsoil stripping from below embankments which are greater than 1.2 m in height. However, at this site, we recommend that all topsoil, organic matter and softened / loosened soils should be removed from below the approach embankment areas and disposed of off-site or re-used as landscaping. For quantity estimation purposes at this site, a topsoil thickness of 0.3 m should be assumed.

All subgrade soils should be proof-rolled prior to fill placement and embankment fill should be placed in accordance with SP206S03 (dated January 2004). The final lift prior to placement of the granular subbase and base courses should be compacted to 100 percent of the Standard Proctor maximum dry density. Inspection and field density testing should be carried out by qualified personnel during placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

To reduce surface water erosion on the embankment side slopes, topsoil and seeding should be carried out as soon as possible. If this slope protection is not in place before winter, then alternate protection measures, such as covering the slope with straw or gravel sheeting to prevent erosion, will be required to reduce the potential for remedial works on the side slopes in the spring prior to topsoil and seeding.

5.4.2 Approach Embankment Stability

Limit equilibrium slope stability analyses were performed using the commercially available program SLOPE/W, produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis, to check that a minimum factor of safety of 1.3 is achieved for the proposed approach embankment height and geometry under static conditions. This minimum factor of safety is considered appropriate for the embankments at this site considering the design requirements and the available field and laboratory testing data.

Static slope stability analyses that examine the global stability of the approach embankments were carried out using the following parameters based on field and laboratory test data and accepted correlations:

Soil Deposit	***Bulk Unit Weight***	***Effective Friction Angle***	***Undrained Shear Strength***
Embankment Fill	21 kN/m^3	32°	–
Very loose to compact Sandy Silt Fill Soft to very stiff Clayey Silt Fill	19 kN/m^3	29°	–
Very Stiff to Hard Clayey Silt Till	20 kN/m^3	–	100 kPa
Stiff Clayey Silt Till	19 kN/m^3	–	50 kPa
Sandy Silt Till	21 kN/m^3	33°	–

The analyses indicate that a factor of safety of greater than 1.3 for a deep-seated failure surface is obtained for up to 8 m high approach embankments with side slopes at the proposed 2 horizontal to 1 vertical (2H:1V) profile.

5.4.3 Approach Embankment Settlement

Settlement of the approach embankment as a result of the embankment loading can be expected mainly due to consolidation of the surficial fills and the underlying clayey silt till deposit encountered in the area of the approach embankments. In order to estimate the magnitude and rate of settlement, analyses were carried out in part using the commercially available computer program Unisettle Version 3.2 in conjunction with hand calculations.

The immediate compression of the fill/floodplain deposits was modelled by estimating an elastic modulus of deformation based on the SPT 'N' values and empirical correlations correlations found in literature (Bowles and Kulhawy and Mayne). Across most of the site, the SPT 'N' values within the fill ranges between 4 and 27 with an average of about 14 blows per 0.3 m of penetration. The fill at the south approach area is up to 3.5 m thick and is associated with the existing Burlington Street and associated ramps. The fill at the north approach area is up to 2.9 m thick and is associated with the QEW or utility works in the area and contains a higher quantity of black organics (based on visual classification). This fill layer is anticipated to settle in response to the addition of the embankment loading.

The fill materials present below the original ground surface extend to Elevation 73.5 m and 74.3 m at the north and south approaches, respectively. The settlement of the fill is 50 mm at the north abutment and less than 25 mm at the south abutment, depending on the thickness and consistency/relative density and the organic content of the material (as discussed in Section 4.2.1). Based on the variability of composition of the fill material, the majority of this settlement is expected to occur during construction, however, some settlement will occur following construction over the long-term. Approximately 25 mm of post-construction settlement is likely to occur at the north abutment.

Settlement of the stiff clayey silt till layer, encountered between about Elevation 68 m and 71 m, under the up to 8 m high embankments is expected to occur. The following correlation relating in situ shear strength to preconsolidation pressure (Mesri) was used to determine whether the settlement is within the overconsolidated range or the normally consolidated range:

$$s_u = 0.22\sigma_p'$$

where: s_u = average mobilized undrained shear strength

σ_p' = preconsolidation pressure

Based on the depth and thickness of this stiff layer, the shear strength as low as 60 kPa (based on the measured SPT 'N' values), the proposed embankment loading will be within the overconsolidated range and it is estimated that the settlement of this layer will be about 25 mm and will occur over the long-term.

Settlement within the till deposits between the base of the fill deposit and Elevation 71, and below Elevation 68 m, is expected to be nominal.

The total long-term settlement of the subsoils (fill and clayey silt till) anticipated as a result of the embankment loading is expected to be 50 mm at the north abutment and 25 mm at the south abutment.

In addition to the settlement of the underlying soils, settlement of the embankment fill itself will also occur. If the embankment fill material consists of granular soil, the settlement is expected to be less than 25 mm and will occur rapidly (i.e. during construction). If the embankment is constructed using earth fill containing plastic soil such as clayey silt or silty clay, the settlement is still anticipated to be about 25 mm; however, some of the settlement will occur after the fill embankment is in place (i.e. post-construction).

5.4.4 Mitigation of Settlement

It is understood that the maximum settlement of about 25 mm at the structure location and 50 mm for the approach embankment beyond the structure is acceptable to limit subsequent maintenance on the new roadway pavement structure. The settlement of the fill and clayey silt till is anticipated to be up to 75 mm at the north approach embankment area with the majority of the settlement occurring within the fill. Up to 50 mm of this settlement is expected to occur over the long-term. If this settlement cannot be tolerated, then consideration could be given to sub-excavation of the fill materials or preloading the north approach embankment area in order to reduce these settlements.

5.4.4.1 Sub-excavation

In this regard, sub-excavation of the fill material in the north approach embankment area would have to be carried out to Elevation 73.5 m to remove the soft/loose fill containing the organics. This would require excavation up to 1.0 m below the water table. If this fill is removed, the settlement will be reduced from 75 mm to 25 mm. It should be noted that downdrag loads on the piles will still have to be considered as a result of the settlement of the underlying “stiff” clayey silt till layer.

The excavated material would have to be replaced with compacted granular fill (Granular ‘A’ or Select Subgrade Material). The limits of the subexcavation would have to cover an area delineated by a line extended from the base of the walls outwards at a 1H:1V slope. The excavation cut slopes should be made no steeper than 1.5H:1V. Requirements with respect to excavation, dewatering and temporary excavations should be as described in Section 5.7.

In all cases, it is recommended that the approach slab construction and paving be delayed for a period of 3 months after the bridge is constructed and the embankment completed.

5.4.4.2 Preloading

Alternatively, if the schedule permits and the required space is available, consideration could be given to preloading the approach embankment area for a period of 6 months to reduce the long-term settlements to these acceptable limits. It is estimated that about half of the ong-term settlement would occur during the first 2 months, with the remaining settlement occurring over the next 4 months. The preload embankments must be constructed with side slopes at no steeper than 2H:1V, and monitoring of settlement during and after the preload period should be carried out. A special provision for monitoring using settlement plates will be included in the contract (see Appendix B).

If the embankment is constructed and time is permitted for preloading and consolidation prior to installing the abutment piles, then downdrag loads on the piles/caissons as a result of settlement (as discussed in Sections 5.2.1.2 and 5.2.2.3) would not have to be considered in the design. This applies only if there is sufficient space to construct the preload embankment to the full height over the abutment area. In this case, due to construction staging and the proposed grade raise, it may not be possible to construct this full height preload embankment and therefore, the full downdrag loads should be considered in the design.

It should be noted that in general, the addition of a surcharge load will reduce the length of the preload period. However, the addition of a 2 m surcharge on top of the preload will not be stable for the anticipated embankment heights.

5.4.4.3 Other Settlement Mitigation Options

Since the majority of the settlement is expected to occur within the fill deposits, the use of other settlement mitigation schemes such as wick drains or lightweight (EPS) fill are not recommended. Wick drains are not appropriate for reducing settlement times within the fill for this site given the variability and composition of the fill materials. Wick drains could reduce the settlement time within the "stiff" clayey silt till layer due to the depth of the layer if preloading time was limited. In addition, due to the depth and limited thickness of the stiff clayey silt till layer, substantial pre-drilling would be required for wick drain installation.

Lightweight fill is considered to be cost prohibitive for the quantity that would be required to reduce settlement to the acceptable limits.

5.5 Retaining Walls

As discussed in Section 5.1, retaining walls are required on the northwest and northeast corners of the bridge. The northeast wall extends approximately 13 m behind the abutment wall. The northwest wall extends for about 400 m beyond the end of the abutment. In this report, however, only the first 20 m of the northwest wall beyond the end of the abutment is addressed. The walls are about 8 m in height at these locations and are required to separate the embankment from the pumping station to the northwest and from the existing embankment to the east.

These structures could consist of either a mechanically-reinforced soil retaining wall system (retained soil system or RSS wall) or a pile-supported concrete wall. The advantages, disadvantages, relative costs and risks/ consequences for the different wall options are summarized in Table 2.

In order to minimize the differential settlement between the RSS wall and the pile supported bridge abutments, RSS walls should only be considered if sub-excavation of the fill material is carried out. If sub-excavation and/or preloading cannot be carried out, pile-supported concrete walls should be considered. Design recommendations for piled walls should be as per Section 5.2. The following sections discuss the design recommendations for RSS type walls.

5.5.1 Settlement and Stability

The settlement of the wall is governed by the embankment loading on the underlying fill and also the stiff clayey silt till. As discussed in Section 5.4.3, settlement underneath the wall is expected to be up 75 mm unless sub-excavation of the fill or preloading of the embankment prior to wall construction is carried out.

If preloading is carried out, the settlement will be reduced to less than 25 mm which is within tolerable limits for RSS walls. If sub-excavation of the fill is carried out, the settlement will be reduced to 25 mm, which is also within tolerable limits for RSS walls. Both of these settlement mitigation options (discussed in Section 5.4.4), are acceptable from a foundation perspective as giving the most effective design in terms of limiting the settlement.

Additional measures such as delaying the construction of the pavement on the RSS wall may be required to minimize the differential settlement between the barrier on the abutment and the barrier on the RSS wall.

If sub-excavation and/or preloading cannot be carried out, then a pile-supported concrete wall will have to be used.

Static slope stability analyses was carried out using the methodology and parameters given in Section 5.4.2. The analyses indicate that a factor of safety of greater than 1.3 for a deep-seated failure surface is obtained for up to 8 m high wall. It should be noted that the internal stability of the mechanically-reinforced soil walls should be checked by the RSS supplier / designer.

5.5.2 Geotechnical Resistance

An RSS wall typically consists of granular fill placed and compacted in layers, and reinforced with metal or fabric strips or grids. A facing material, typically pre-cast concrete panels mechanically fastened to the reinforcing strips or grids, is used to form the face of the reinforced soil structure and to prevent the loss of fill material and is supported on a strip footing. A typical RSS wall has a front facing supported on a strip footing placed at shallow depth below the ground surface in front of the wall. The facing footing must be founded below any topsoil, loose fill or unsuitable native soils.

Assuming that the RSS wall acts as a unit and utilizes the full width of the reinforced soil mass, which is taken as two-thirds of the height of the wall, a factored geotechnical resistance at ULS of 500 kPa and a geotechnical resistance at SLS (for 25 mm of settlement) of 350 kPa may be used for assessment of the reinforced mass founded on the properly prepared very stiff to hard clayey silt till deposits. These resistances values assume that the north approach embankment area (under the wall) has been preloaded for 6 months or that the unsuitable fill material at the north approach embankment area has been removed (to Elevation 73.5 m) and backfilled up to the original grade (Elevation 77 m) under the full area of the reinforced earth mass. With this approach, the facing footing, and RSS mass, will be supported on the sub-excavation backfill.

The resistance to lateral forces / sliding resistance between the compacted granular fill and the subgrade should be calculated in accordance with Section 6.7.5 of the CHBDC. The coefficient of friction, tan φ', between the compacted granular fill of the RSS wall and the compacted granular fill that replaces the sub-excavated material may be taken as 0.70. This represents an unfactored value; in accordance with the CHBDC, a factor of 0.8 is to be applied in calculating the horizontal resistance.

Frost protection is not required for RSS walls.

5.6 Construction Considerations

5.6.1 Existing Pipes

As per the general arrangement drawing, the south pier of Bridge 6 is directly overtop the existing 1500 mm diameter and 2100 mm diameter pipes. The 1500 mm pipe is to be abandoned and as such, there is no concern with respect to vibration of this pipe. The 2100 mm pipe is to remain in place and it is understood that the steel liner to this reinforced concrete pipe will be extended prior to footing construction. For this pipe, vibration and damage is a key concern. The 1200 mm pipe is about 5 m west of the pier and it is expected that any damage sustained to this pipe can be repaired as required.

5.6.1.1 Settlement

At the south abutment, the settlement of the fill is expected to be less than 25 mm and the settlement of the "stiff" clayey silt till layer is expected to be 25 mm. The pipes will not be affected by settlement of the fill. The pipes will settle up to about 25 mm as a result of consolidation of the clayey silt till below the base of the pipes. In addition, some additional settlement of the clayey silt till below the base of the pipes may be caused by vibration if pre-augering to the bedrock surface is carried out. This additional settlement not anticipated to be more than about 25 mm. It is anticipated that these pipes could tolerate up to 50 mm of settlement; however, the TPM consultant should contact the municipality to confirm this assumption.

5.6.1.2 Vibration

Deep foundations founded within the very dense sandy silt till or the bedrock would extend some 5 m to 10 m below the pipe inverts. The soils at and immediately below the existing pipes have a "stiff" consistency with SPT 'N' values ranging between 9 and 15 blows per 0.3 m of penetration and will be affected by vibration during pile driving/caisson augering.

The excavation for caisson construction has the potential of causing stress relief in the soils surrounding the pipes if the excavation is in relatively close proximity to the pipes. This could impact the structural integrity of the pipes.

The minimum separation distance from the pile/caisson to the pipes will be dependant on the future use of the existing pipes. Care must be taken to ensure that the existing pipes are not damaged during pile driving or augering for caissons. In addition, care must be taken to ensure that the pipe bedding is not intercepted or that precautions are taken to prevent undermining of the existing pipes.

For the case of driven piles founded on the bedrock, pre-augering to the bedrock will be required in order to both minimize the vibrations during driving and provide a bit of guidance to the pile to minimize deviation in alignment. In this regard, a distance of at least 1 m should be maintained between the pile and the pipe (as discussed in Section 5.2). For the case of caissons, a minimum distance of three caisson diameters be maintained between the outer edges of the caisson and the existing pipes.

In both cases (piles and caissons), vibration monitoring to assess the impact of the vibrations on the existing 2100 mm pipe will be required and is discussed in further detail in Section 5.8. The 1200 mm pipe will be rehabilitated after bridge construction and as a result, it is considered that vibration monitoring of this pipe is not required.

5.6.2 Proposed 2700 mm Diameter Tunnel

The proposed 2700 mm pipe is located in plan in the vicinity of the north pier and the north abutment. However, the crown of the 2700 mm diameter tunnel is proposed to be at about Elevation 54 m, which is about 7 m to 8 m below the surface of the bedrock. If driven piles are used for support, there would not be any adverse impact expected unless there is substantial loading and closely spaced piles. In the case of caissons founded on the surface of the bedrock or socketted 2 m into the bedrock, the potential for transfer of load to the underlying pipe will depend on the caisson diameter and applied load at the founding level.

5.6.3 Obstructions

It should be noted that although boulders were not noted in the Boreholes drilled for Bridge 6, cobbles/ boulders were encountered within the clayey silt till and sandy silt till deposits elsewhere across the site and boulders should be expected within the glacially derived till materials. In addition, the fill materials could contain boulders or rubble. Difficulty may be experienced augering and/or driving of piles through boulders at this site. Provision should be made for coring or down-hole hammers for advancing the caissons or pre-augering through this deposit, where boulders are encountered.

5.7 Excavations and Temporary Cut Slopes

It is anticipated that the abutment pile caps will be constructed at or about the existing grade level. If sub-excavation is being considered as part of the settlement mitigation scheme discussed in Section 5.4.4, excavations will extend through fill materials consisting of loose to compact silty sand to sandy silt and soft to very stiff silty clay to clayey silt to Elevation 73.4 at the north approach which is about 1.0 m below the water level. The base of the excavation will be within the very stiff to hard clayey silt till. This clayey silt till deposit is susceptible to disturbance from ponded water and construction traffic. Precautions such as the placement of a 75 mm thick lean concrete (less than 1 MPa compressive strength) “mudcoat” may be required to provide suitable working conditions. The mudcoat should be placed within 4 hours of reaching the base of the excavation. The contractor should be made aware that trafficking over the exposed clayey material may not be possible and is not desirable and an Operational Constraint should be included in the contract in this regard.

It is anticipated that the bulk of the excavations at the site can be made in open cut. Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act (OHSA) for Construction Activities. The surficial fills are classified as Type 3 soil, according to the OHSA. Temporary excavations (i.e. those which are only open for a relatively short period) extended through the fill to the surface of the very stiff to hard clayey silt till should be made with side slopes no steeper than 1.5 horizontal to 1 vertical (1.5H:1V) through these materials above the water table and 3H:1V below the water table.

The ground water level at the site is typically at about Elevation 74.5 m and is between 1.5 m and 2.0 m below the ground surface. It is anticipated that for the open-cut excavations, the groundwater can be adequately controlled by sumping from properly filtered sumps.

Excavation support for roadway protection will be required at the north pier and possibly elsewhere at this site. Where required, the temporary excavation support system should be designed and constructed in accordance with MTO's Special Provision 539S01. The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in SP 539S01.

5.8 Monitoring

5.8.1 Settlement Monitoring

If preloading of the approaches/abutment areas is carried out to mitigate the settlement (as discussed in Section 5.4.4.2), settlement monitoring will be required. The monitoring program should consist of the installation of a series of settlement plates within the embankment fill which would be surveyed at regular intervals during and after construction, for the duration of the preloading period. In addition, the monitoring program should consists of the installation of vibrating wire piezometers to measure excess pore pressures to compliment the settlement plates. The locations and specifications of the monitoring points should be specified in the contract.

The monitoring should be carried out by a monitoring specialist retained by MTO who would be responsible for obtaining the baseline and subsequent survey and piezometer readings and reporting of the data. The instruments should be installed by the contractor, including extension of the steel rods and PVC sleeves of the settlement plates during filling. The monitoring specialist would need to be on site during extension of the rods to obtain accurate measurements of rod length.

The non-standard special provisions for settlement plates, vibrating wire piezometers and general monitoring (including the installation of temporary survey benchmarks) are presented in Appendix C. Detailed layout of the settlement plates will be prepared at the contract stage.

5.8.2 Vibration Monitoring

The proposed structure foundations will be in close proximity to the existing pipes (as discussed in Section 5.2). Vibration monitoring should be carried out during construction (particularly pile/caisson installation) to ensure that vibration levels on the existing pipes are maintained below tolerable levels. A non-standard special provision should be included in the contract for this purpose as discussed in Section 5.2. A draft of this special provision is given in Appendix B.

A maximum peak particle velocity (PPV) of 50 mm/s would be recommended for both the steel-lined 2700 mm diameter tunnel and 2100 mm diameter steel-lined reinforced concrete pipes. In order to monitor vibrations on these structures, the monitoring unit should be placed on the pipe, as close as possible to the construction activities. In this regard, the pipe may have to be exposed by either daylighting methods or borehole drilling methods. If the pipe cannot be exposed, then the unit could be buried in the ground close to the pipe. However, in this case, the maximum PPV may have to be adjusted since vibration levels measured in the ground would be higher than the PPV measured on the pipe.

In addition, the piles furthest from the existing pipes should be driven first, in order to check vibration levels on the pipes, and if necessary, alter driving/pre-augering procedure for the piles closest to the pipes.

5.9 Closure

This report was prepared by Miss Sarah Poot, P.Eng., a senior geotechnical engineer. The technical aspects were reviewed by Ms. Anne S. Poschmann, P. Eng., a Principal with Golder Associates Ltd. Mr. Fintan J. Heffernan, P. Eng., a Designated MTO Contact for Golder, conducted a quality control review of the report.

GOLDER ASSOCIATES LTD.

LICENSED PROFESSIONAL ENGINEER
S. E. M. POOT
PROVINCE OF ONTARIO

Sarah E. M. Poot., P.Eng.,
Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
F. J. HEFFERNAN
PROVINCE OF ONTARIO

Fintan J. Heffernan, P.Eng.,
Designated MTO Contact

Anne S. Poschmann, P.Eng.,
Principal

SEP/JPD/ASP/FJH/sd
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TABLE 1
EVALUATION OF FOUNDATION ALTERNATIVES
BRIDGE 6 - BURLINGTON STREET OVER QEW AND RHCE W-S RAMP

Foundation Option	***NF***	***Advantages***	***Disadvantages***	***Relative Costs***	***Risks/Consequences***
Spread Footings on upper crust of stiff to hard clayey silt till	X	Minimized deep excavations.	Low geotechnical resistance. Groundwater control may be required. Differential settlement between abutment/pier foundations due to consolidation of underlying stiff clayey silt till layer under the embankment loading.	Lower relative costs than piled foundations.	Not recommended due to potential for differential settlements anticipated between abutments and piers.
Spread Footings on very dense sandy silt till	X		Deep (10 m to 14 m) excavation required. Groundwater control and temporary shoring required.	Increased cost for groundwater control and temporary shoring compared to shallower footings.	Not recommended due to significant depth of deep excavations.
Steel H Piles driven through stiff to hard clayey silt till just into very dense sandy silt till		Relatively straight forward construction except where foundation elements are adjacent to existing and proposed pipes.	Lower capacity than piles on bedrock. Where piles are adjacent to pipes, pre-augering below pipe invert level may be required to minimize vibrations due to driving and minimize potential for disturbance of the pipe bedding. Battered piles may not be possible in some areas due to proximity of pipes.	Lower relative costs than piles driven to bedrock. Increase in cost associated with pre-drilling where required.	Pile locations adjacent to existing pipes would require pre-augering to permit pile installation.
Steel H Piles driven to shale bedrock		Increased capacity over piles terminated in overburden.	Difficulties anticipated with driving through very dense till deposit; piles will 'hang-up' within the till, therefore pre-augering to the bedrock required at all pile locations. Battered piles may not be possible due to proximity of pipes.	Increase in cost associated with pre-drilling at all pile locations.	All piles would require pre-augering to permit pile installation.
Caissons socketted into very dense sandy silt till		Less likelihood of encountering boulders to reach founding level in till.	Lower capacity than caissons socketted into bedrock. Temporary liners may be required for groundwater control. Need to maintain adequate distance from existing pipes to minimize potential of affecting integrity.	Lower relative costs than caissons socketted into bedrock.	
Caissons socketted into shale bedrock		Higher capacities than caissons on till.	Temporary liners may be required for groundwater control. Socketting into bedrock may require rock coring or churn drilling techniques. Need to maintain adequate distance from existing pipes to minimize potential of affecting integrity.	Increased cost of socketting into bedrock.	Difficulty may be encountered socketting liner in borehole to seal off water; tremie concreting may be required. Difficulties may be met with advancing caissons through the sandy silt till if boulders are encountered.

NF: Indicates that the founding option is considered not feasible.

TABLE 2
EVALUATION OF RETAINING WALL ALTERNATIVES
NORTH APPROACH AREA OF BRIDGE 6

Foundation Option	***NF***	***Advantages***	***Disadvantages***	***Relative Costs***	***Risks/Consequences***
Retained Soil System (RSS) Wall founded on fill	X	Minimal excavation required.	Settlement of fill as well of underlying clayey silt till deposit up to 100 mm.	Less expensive than having to sub-excavate and replace prior to building wall.	Not recommended due to large magnitudes of settlement unless sub-excavation is carried out.
Retained Soil System (RSS) Wall founded on compacted granular fill after sub-excavation of existing fills.		Settlement of fill no longer relevant. Settlement of clayey silt till will still occur but within tolerable levels.	Up to 3.5 m of excavation required, partially below groundwater table. Settlement of underlying clayey silt till will occur due to embankment loading and may be differential along the length of the wall or between the wall and the bridge.	Costs of excavation and groundwater control (sumps) as well as replacement backfill.	Some settlement of the wall will still occur as a result of the underlying clayey silt till layer. Some differential settlement between bridge and wall.
Concrete Cantilever Wall founded on shallow spread footings on till	X		Up to 3.5 m of excavation required, partially below groundwater table. Settlement of underlying clayey silt till will occur due to embankment loading and may be differential along the length of the wall or between the wall and the bridge.	Costs of excavation and groundwater control (sumps).	Not recommended due to differential settlements between bridge and wall.
Concrete Cantilever Wall founded on deep foundations		Minimize settlement between bridge abutment and walls.	Must consider downdrag on piles.	More costly foundation treatment compared to RSS walls. Concrete walls typically more expensive than RSS walls.	Settlement of road embankment will still occur; differential with respect to the wall.

NF: Indicates that the founding option is considered not feasible.

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
SS	Split-spoon
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
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, 60º 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^2 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) Cohesionless Soils

Density Index (Relative Density)	N 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	c_u, s_u 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[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
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)
γ	unit weight

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

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LIST OF SYMBOLS

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

I. General

π	3.1416
in x,	natural logarithm of x
$\log_{10}$	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
F	factor of safety
V	volume
W	weight

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\ \sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
v	poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma$-u)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1+\sigma_2+\sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	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
γ'	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_1	liquid limit
w_p	plastic limit
l_p	plasticity index $= (w_1 - w_p)$
w_s	shrinkage limit
I_L	liquidity index $= (w - w_p)/I_p$
I_C	consistency index $= (w_1 - w) / I_p$
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_a	coefficient of secondary consolidation
m_v	coefficient of volume change
c_v	coefficient of consolidation
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation pressure
OCR	over-consolidation ratio $= \sigma'_p/\sigma'_{vo}$

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = tan δ
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

Notes:
1 $\tau = c' + \sigma' \tan \phi'$
2 shear strength = (compressive strength)/2
* density symbol is ρ. Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING STATE

Fresh: no visible sign of weathering.

Faintly weathered: weathering limited to the surface of major discontinuities.

Slightly weathered: penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.

Moderately weathered: weathering extends throughout the rock mass but the rock material is not friable.

Highly weathered: weathering extends throughout rock mass and the rock material is partly friable.

Completely weathered: rock is wholly decomposed and in a friable condition but the rock texture and structure are preserved.

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very thickly bedded	> 2 m
Thickly bedded	0.6 m to 2m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	< 6 mm

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	> 3 m
Wide	1 - 3 m
Moderately close	0.3 - 1 m
Close	50 - 300 mm
Very close	< 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	> 60 mm
Coarse Grained	2 - 60 mm
Medium Grained	60 microns - 2 mm
Fine Grained	2 - 60 microns
Very Fine Grained	< 2 microns

Note: * Grains >60 microns diameter are visible to the naked eye.

CORE CONDITION

Total Core Recovery

The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)

The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, measured relative to the length of the total core run. RQD varies from 0% for completely broken core to 100% for core in solid sticks.

DISCONTINUITY DATA

Fracture Index

A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

Dip with Respect to (W.R.T.) Core Axis

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90° angle is horizontal.

Description and Notes

An abbreviated description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes or mechanically induced features caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

B - Bedding	P - Polished
FO - Foliation/Schistosity	S - Slickensided
CL - Cleavage	SM - Smooth
SH - Shear Plane/Zone	R - Ridged/Rough
VN - Vein	ST - Stepped
F - Fault	PL - Planar
CO - Contact	FL - Flexured
J - Joint	UE - Uneven
FR - Fracture	W - Wavy
MF - Mechanical Fracture	C - Curved
$\parallel$ - Parallel To	
$\perp$ - Perpendicular To	

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RECORD OF BOREHOLE No BR6-1

METRIC

PROJECT 021-1162
W.P. 443-97-00
DIST ____ HWY QEW
DATUM Geodetic

LOCATION N 4791041.4 ;E 282928.9
BOREHOLE TYPE CME 75 Bombardier; 210mm O.D. Hollow Stem Auger
DATE September 3, 2003

ORIGINATED BY PKS
COMPILED BY KG
CHECKED BY SEP

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER
76.6	GROUND SURFACE				
0.0	Clayey SILT, some sand and gravel, organics (FILL); Very Stiff to soft; Brown, grey, and red; Moist	1	SS	22	
		2	SS	6	∇
		3	SS	4	
74.3 / 2.3	Clayey SILT, some sand, trace gravel, occasional shale fragments (TILL); Hard to stiff; Brown to grey; Moist	4	SS	26	
		5	SS	34	
		6	SS	34	
		7	SS	22	
		8	SS	10	
		9	SS	16	
		10	SS	18	
	Occasional shale and limestone fragments below 10.7 m depth; Becoming red below 10.7 m depth	11	SS	93	
65.6 / 11.0	Sandy SILT, trace clay and gravel, shale and limestone fragments (TILL); Very dense; Red; Moist	12	SS	100/0.15	
62.9 / 13.7	Clayey SILT, some sand, shale fragments (Residual Soil); Hard; Red; Moist	13	SS	100/0.28	

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE (76, 75, 74, 73, 72, 71, 70, 69, 68, 67, 66, 65, 64, 63, 62); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED, 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued Next Page

+³, ×³: Numbers refer to Sensitivity   O³% STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No BR6-1

METRIC

PROJECT 021-1162			
W.P. 443-97-00	LOCATION N 4791041.4 ;E 282928.9		ORIGINATED BY PKS
DIST	HWY QEW	BOREHOLE TYPE CME 75 Bombardier; 210mm O.D. Hollow Stem Auger	COMPILED BY KG
DATUM Geodetic	DATE September 3, 2003		CHECKED BY SEP

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X REMOULDED) 20 40 60 80 100	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
61.4 / 15.2	Highly to slightly weathered, reddish-grey, calcareous SHALE BEDROCK (Queenston Formation) with occasional grey siltstone layers. Bedrock cored from 15.2m to 18.6m depth. For bedrock coring details see Record of Drillhole BR6-1		~~11~~	~~SS~~	~~30/0.1~~		61				
							60				
							59				
58.0 / 18.6	End of Borehole Notes: 1. Water level at 1.5 m depth (Elev. 75.1 m) in open borehole on September 3, 2003.						58				

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

$O^{3\%}$ STRAIN AT FAILURE

PROJECT: 021-1162

RECORD OF DRILLHOLE: BR6-1

LOCATION: N 4791041.4 ;E 282928.9

DRILLING DATE: September 08, 2003

DATUM: Geodetic

DRILL RIG: Bomb CME 75

INCLINATION: -90° AZIMUTH: ---

DRILLING CONTRACTOR: Geo-Environmental Drilling Ltd.

FR/FX-FRACTURE F-FAULT SM-SMOOTH FL-FLEXURED BC-BROKEN CORE
CL-CLEAVAGE J-JOINT R-ROUGH UE-UNEVEN MB-MECH. BREAK
SH-SHEAR P-POLISHED ST-STEPPED W-WAVY B-BEDDING
VN-VEIN S-SLICKENSIDED PL-PLANAR C-CURVED

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		Refer to previous page	61.40			
	NQ	Highly to slightly weathered, thinly layered, reddish-grey, very fine grained, very weak to medium strong, calcareous SHALE BEDROCK (Queenston Formation), occasional seams of grey, medium to very strong siltstone; layers greater than 25 mm listed below:	15.20	1		
16		Depth (m) Thickness (mm) 15.5 230 16.2 75 16.4 25 16.5 25 18.5 25		2		
17		All fractures are bedding, rough, planar, except where noted			F, SM, PL	
18				3		
			58.00			
		End of Drillhole	18.60			
19						
20						
21						
22						
23						
24						
25						

Column headings also include: SYMBOLIC LOG; PENETRATION RATE (m/min); FLUSH; COLOUR % RETURN; RECOVERY (TOTAL CORE % 80 60 40 20; SOLID CORE % 80 60 40 20); R.Q.D. % 80 60 40 20; FRACT. INDEX PER 0.3 5 10 15 20; DIP w.r.t. CORE AXIS 0 30 60 90; HYDRAULIC CONDUCTIVITY K, cm/sec 10^{-6} 10^{-5} 10^{-4} 10^{-3}; DIAMETRAL POINT LOAD INDEX (MPa) 2 4 6.

MISS_ROCK 0211162EARCK.GPJ GLDR_CAN.GDT 21/4/05 TM

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: PS
CHECKED: SEP

Golder Associates

RECORD OF BOREHOLE No BR6-2

METRIC

PROJECT 021-1162			
W.P. 443-97-00	LOCATION N 4791041.7 ;E 282980.0		ORIGINATED BY PKS
DIST	HWY QEW	BOREHOLE TYPE CME 75 Bombardier; 210mm O.D. Hollow Stem Auger	COMPILED BY KG
DATUM Geodetic	DATE September 3, 2003		CHECKED BY SEP

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X REMOULDED; 20 40 60 80 100
PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30
UNIT WEIGHT γ kN/m³
REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
76.8	GROUND SURFACE
0.0 / 0.1	Sandy TOPSOIL
	Silty CLAY, trace to some sand and gravel, trace rootlets (FILL) Very stiff to firm Brown, grey, red Moist
74.5 / 2.3	Clayey SILT, some clay, trace gravel (TILL) Hard to stiff Brownish-grey to grey Moist
	Containing shale and limestone fragments below 7.6 m depth
65.2 / 11.6	Sandy SILT, some gravel, trace clay, shale and limestone fragments Very dense Red Moist (Till)
63.1 / 13.7	Clayey SILT, some sand and red shale fragments (Residual Soil) Hard Red Moist

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS GR SA SI CL
1	SS	27	
2	SS	14	
3	SS	7	
4	SS	35	
5	SS	31	
6	SS	40	5 18 54 23
7	SS	20	
8	SS	9	
9	SS	18	
10	SS	15	
11	SS	21	
12	SS	100/0.1	29 24 44 3
13	SS	100/0.2	

ELEVATION SCALE: 76, 75, 74, 73, 72, 71, 70, 69, 68, 67, 66, 65, 64, 63, 62

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity O $^{3\%}$ STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No BR6-2

METRIC

PROJECT 021-1162			
W.P. 443-97-00	LOCATION N 4791041.7 ;E 282980.0		ORIGINATED BY PKS
DIST	HWY QEW	BOREHOLE TYPE CME 75 Bombardier; 210mm O.D. Hollow Stem Auger	COMPILED BY KG
DATUM Geodetic		DATE September 3, 2003	CHECKED BY SEP

SOIL PROFILE: ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X REMOULDED) 20 40 60 80 100	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
			~~11~~	~~SS~~	~~30/0.1~~						
61.5 / 15.3	Highly to slightly weathered, reddish-grey SHALE BEDROCK (Queenston Formation) with occasional siltstone layers Bedrock cored from 15.3 m to 18.6 m depth For Bedrock coring details see Record of Drillhole BR6-2						61 60 59				
58.2 / 18.6	End of Borehole Notes: 1. Water level at 1.5 m depth in open borehole on August 8, 2003 2. Water level in piezometer at 2.0 m depth (Elev. 74.8 m) on October 22, 2003.										

MISS_MTO 0211162EAMTO.GPJ ON_MOT.GDT 21/4/05

+³, X³: Numbers refer to Sensitivity   O³% STRAIN AT FAILURE

RECORD OF DRILLHOLE: BR6-2

PROJECT: 021-1162
SHEET 1 OF 1

LOCATION: N 4791041.7 ;E 282980.0
DATUM: Geodetic

INCLINATION: -90°   AZIMUTH: ---

DRILLING DATE: September 08, 2003
DRILL RIG: Bomb CME 75
DRILLING CONTRACTOR: Geo-Environmental Drilling Ltd.

FR/FX-FRACTURE, F-FAULT, SM-SMOOTH, FL-FLEXURED, BC-BROKEN CORE
CL-CLEAVAGE, J-JOINT, R-ROUGH, UE-UNEVEN, MB-MECH. BREAK
SH-SHEAR, P-POLISHED, ST-STEPPED, W-WAVY, B-BEDDING
VN-VEIN, S-SLICKENSIDED, PL-PLANAR, C-CURVED

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	TYPE AND SURFACE DESCRIPTION
		Refer to previous page	61.60		
	NQ	Highly to slightly weathered, thinly layered, reddish-grey, very fine grained, very weak to medium strong, calcareous SHALE BEDROCK (Queenston Formation), occasional seams of grey, medium to very strong siltstone; layers greater than 25 mm listed below:	15.20	1	
16		Depth (m) / Thickness (mm): 15.8 / 25; 15.9 / 25; 16.7 / 50; 17.5 / 200		2	B, SM-R, PL
					MB FR, R, PL
17		All fractures are bedding, rough, planar, except where noted		3	
18					MB
					MB
			58.20 18.60		
		End of Drillhole			

Other column headings: SYMBOLIC LOG; PENETRATION RATE (m/min); COLOUR % RETURN; FLUSH; RECOVERY: TOTAL CORE %, SOLID CORE %; R.Q.D. %; FRACT. INDEX PER 0.3; DISCONTINUITY DATA: DIP w.r.t. CORE AXIS; HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}); DIAMETRAL POINT LOAD INDEX (MPa); NOTES WATER LEVELS INSTRUMENTATION

Depth scale: 16, 17, 18, 19, 20, 21, 22, 23, 24, 25

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: PS
CHECKED: SEP

Foundation Design

Golder Associates

RECORD OF BOREHOLE No BR6-3

METRIC

PROJECT 021-1162

W.P. 443-97-00 LOCATION N 4791042.8 ;E 283033.3 ORIGINATED BY PKS

DIST HWY QEW BOREHOLE TYPE CME 75 Bombardier; 210mm O.D. Hollow Stem Auger COMPILED BY KG

DATUM Geodetic DATE September 10, 2003 CHECKED BY SEP

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES
76.9	GROUND SURFACE			
0.0	Sandy TOPSOIL			
0.2	Silty SAND to Silty CLAY, some sand, trace to some gravel, organics, red brick fragments (FILL) Compact Brownish-grey Moist	1	SS	22
		2	SS	14
75.4 / 1.5	Clayey SILT, trace gravel, black organics (FILL) Firm Grey Moist	3	SS	4
74.6 / 2.3	Clayey SILT, some sand, trace gravel (TILL) Very stiff to hard Mottled brown and grey to grey Moist	4	SS	24
		5	SS	36
		6	SS	39
		7	SS	25
	Containing shale fragments below 6.1 m depth	8	SS	9
		9	SS	18
	Containing shale and limestone fragments below 8.1 m depth	10	SS	16
		11	SS	20
65.6 / 11.3	Sandy SILT, trace clay, trace gravel with shale and limestone fragments (TILL) Very dense Red Moist	12	SS	100/0.1[illegible]
63.0 / 13.9	CLAYEY SILT, some sand, red shale fragments (Residual Soil) Hard Red Moist	13	SS	100/0.1[illegible]

Column headings: GROUND WATER CONDITIONS; ELEVATION SCALE (76, 75, 74, 73, 72, 71, 70, 69, 68, 67, 66, 65, 64, 63, 62); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Shear strength plot annotations: 1.7 (at elev. 70), 2.0 (at elev. 69)

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No BR6-3

METRIC

PROJECT 021-1162
W.P. 443-97-00
LOCATION N 4791042.8 ;E 283033.3
ORIGINATED BY PKS
DIST HWY QEW
BOREHOLE TYPE CME 75 Bombardier; 210mm O.D. Hollow Stem Auger
COMPILED BY KG
DATUM Geodetic
DATE September 10, 2003
CHECKED BY SEP

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X REMOULDED) 20 40 60 80 100	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
			14	SS	100/0.0						
61.6 / 15.3	Highly to slightly weathered, reddish-grey calcareous SHALE BEDROCK (Queenston Formation) with occasional siltstone layers Bedrock cored from 15.3 m to 18.4 m depth For coring details see Record of Drillhole BR6-3						61 60 59				
58.4 / 18.5	End of Borehole Notes: 1. Water level of piezometer at 2.0 m depth (Elev. 74.9 m) on October 22, 2003.										

$+^3, \times^3$: Numbers refer to Sensitivity $O^{3\%}$ STRAIN AT FAILURE

RECORD OF DRILLHOLE: BR6-3

PROJECT: 021-1162
SHEET 1 OF 1

LOCATION: N 4791042.8 ;E 283033.3
DRILLING DATE: September 11, 2003
DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---
DRILL RIG: Bomb CME 75
DRILLING CONTRACTOR: Geo-Environmental Drilling Ltd.

FR/FX-FRACTURE, F-FAULT, SM-SMOOTH, FL-FLEXURED, BC-BROKEN CORE, CL-CLEAVAGE, J-JOINT, R-ROUGH, UE-UNEVEN, MB-MECH. BREAK, SH-SHEAR, P-POLISHED, ST-STEPPED, W-WAVY, B-BEDDING, VN-VEIN, S-SLICKENSIDED, PL-PLANAR, C-CURVED

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	COLOUR % RETURN / FLUSH	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		Refer to previous page		61.50												
16	NQ	Highly to slightly weathered, thinly layered, reddish-grey, very fine grained, very weak to medium strong, calcareous *SHALE BEDROCK (Queenston* Formation), occasional seams of grey, medium to very strong siltstone; layers greater than 25 mm listed below: Depth (m) Thickness (mm) 16.2 50		15.40	1											
17		*All fractures are bedding, rough, planar, except where noted*			2											
18				58.41 18.50												
		End of Drillhole														
19																
20																
21																
22																
23																
24																
25																

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: PS
CHECKED: SEP

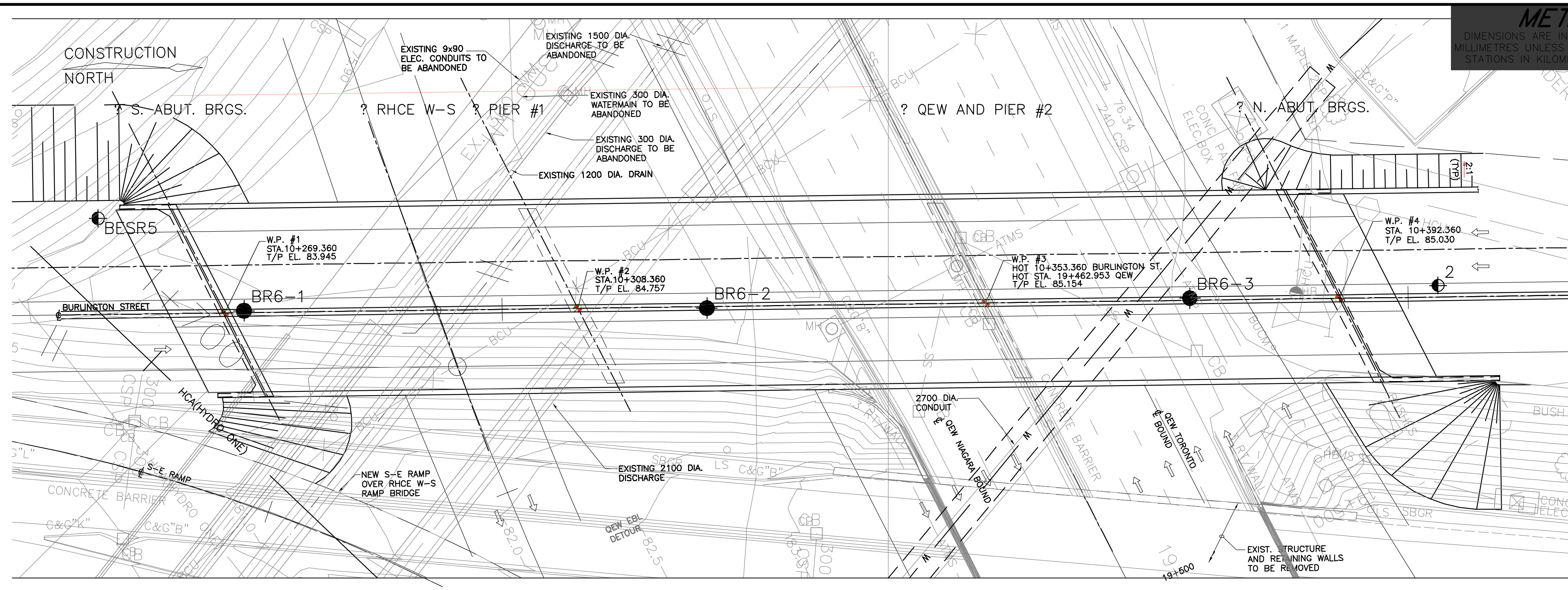
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No. 2005–2008
WP No. 443–97–00

BURLINGTON STREET OVER QEW AND RHCE W–S RAMP (BRIDGE 6)
BOREHOLE LOCATIONS AND SOIL STRATA

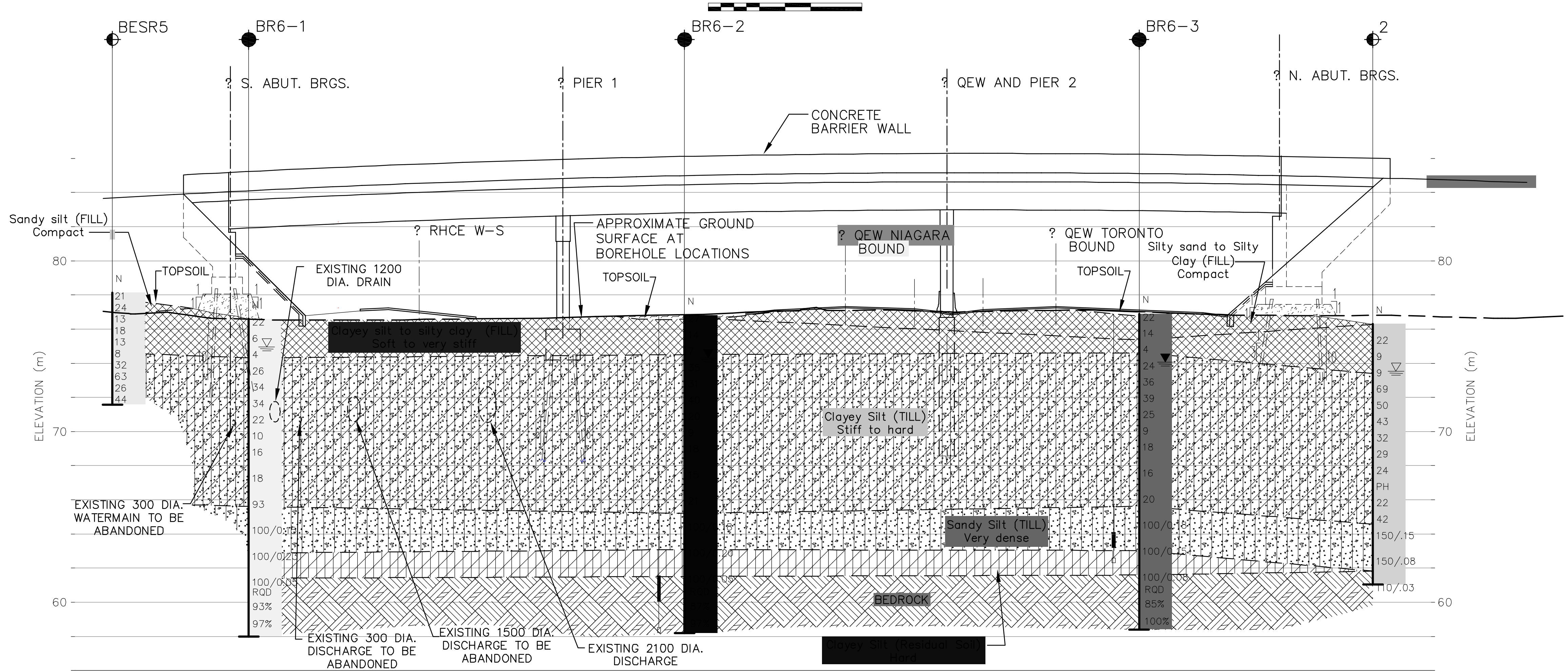
SHEET

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



PLAN

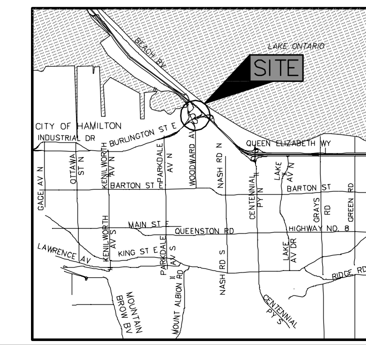
SCALE
6 0 6 12 m



PROFILE ALONG CENTERLINE

HORZ. SCALE
6 0 6 12 m

VERT. SCALE
3 0 3 6 m



KEY PLAN

LEGEND

- Borehole – Current Investigation, Golder 2003
- Borehole – Previous Investigation, Golder 1998
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- WL in piezometer
- WL upon completion of drilling

No.	ELEVATION	CO–ORDINATES	
		NORTHING	EASTING
BESR5	78.1	4791051.6	282912.8
2	76.3	4791044.2	283060.7
BR6–1	76.6	4791041.4	282928.9
BR6–2	76.8	4791041.7	282980.0
BR6–3	76.9	4791042.8	283033.3

NOTES

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

The complete 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.

This drawing is for subsurface information only. The proposed structure details are shown for illustration purposes only and may may not be consistent with the final design configuration as shown elsewhere in the Contract Documents.

REFERENCE

Base plans provided in digital format by McCORMICK RANKIN CORPORATION, drawing file no. 'S5132–308GA001.dwg', received Feb. 9, 2005.

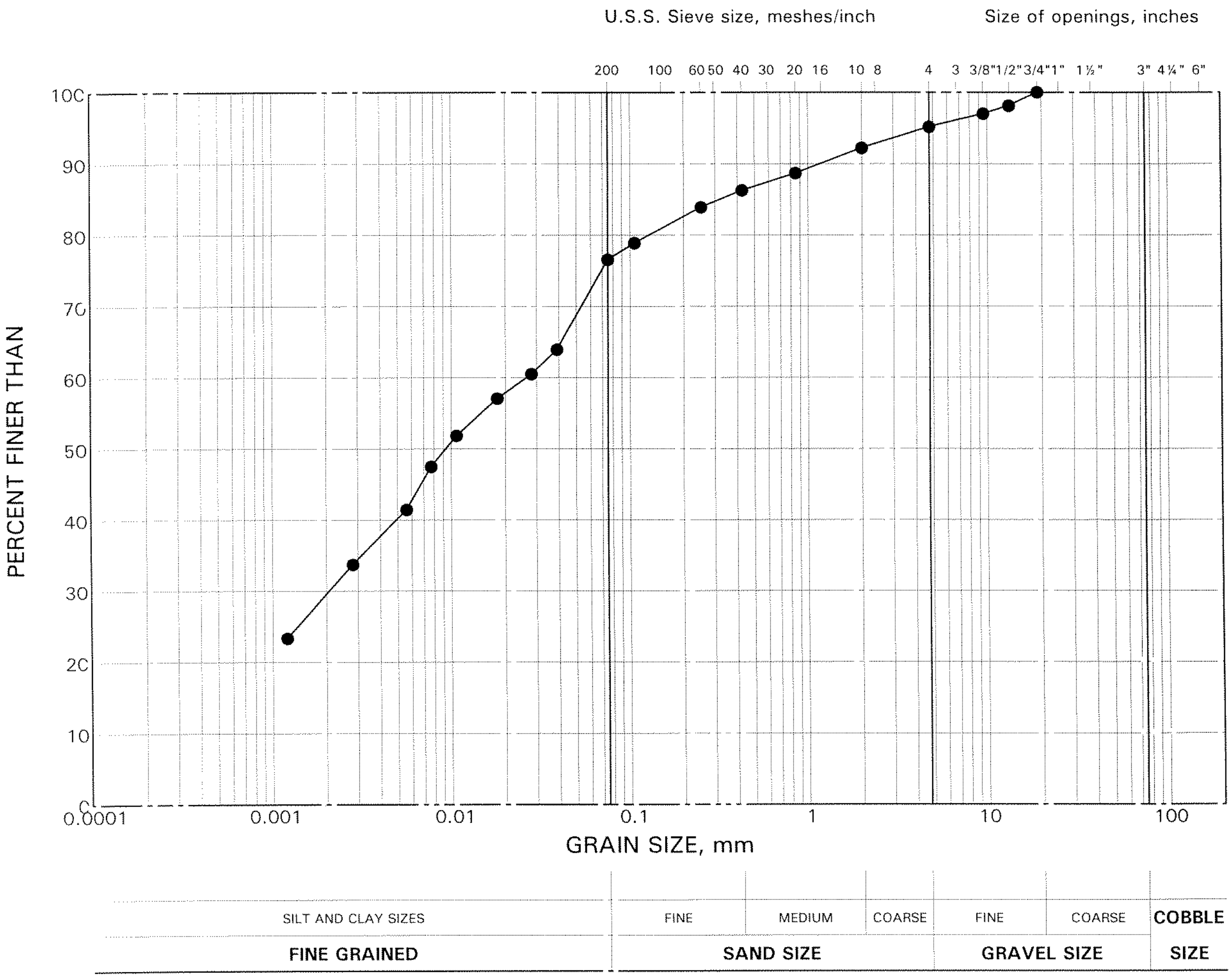
NO.	DATE	BY	REVISION

Geocres No.

HWY. QEW		PROJECT NO. 021–1162(BR6)	DIST.
SUBM'D.	CHKD. SEP	DATE: APRIL 2005	SITE:
DRAWN: JDR	CHKD. ASP	APPD. FJH	DWG. 1

GRAIN SIZE DISTRIBUTION
Clayey Silt (Till)

FIGURE 1



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	BR6-2	6	72.7

Date April, 2005

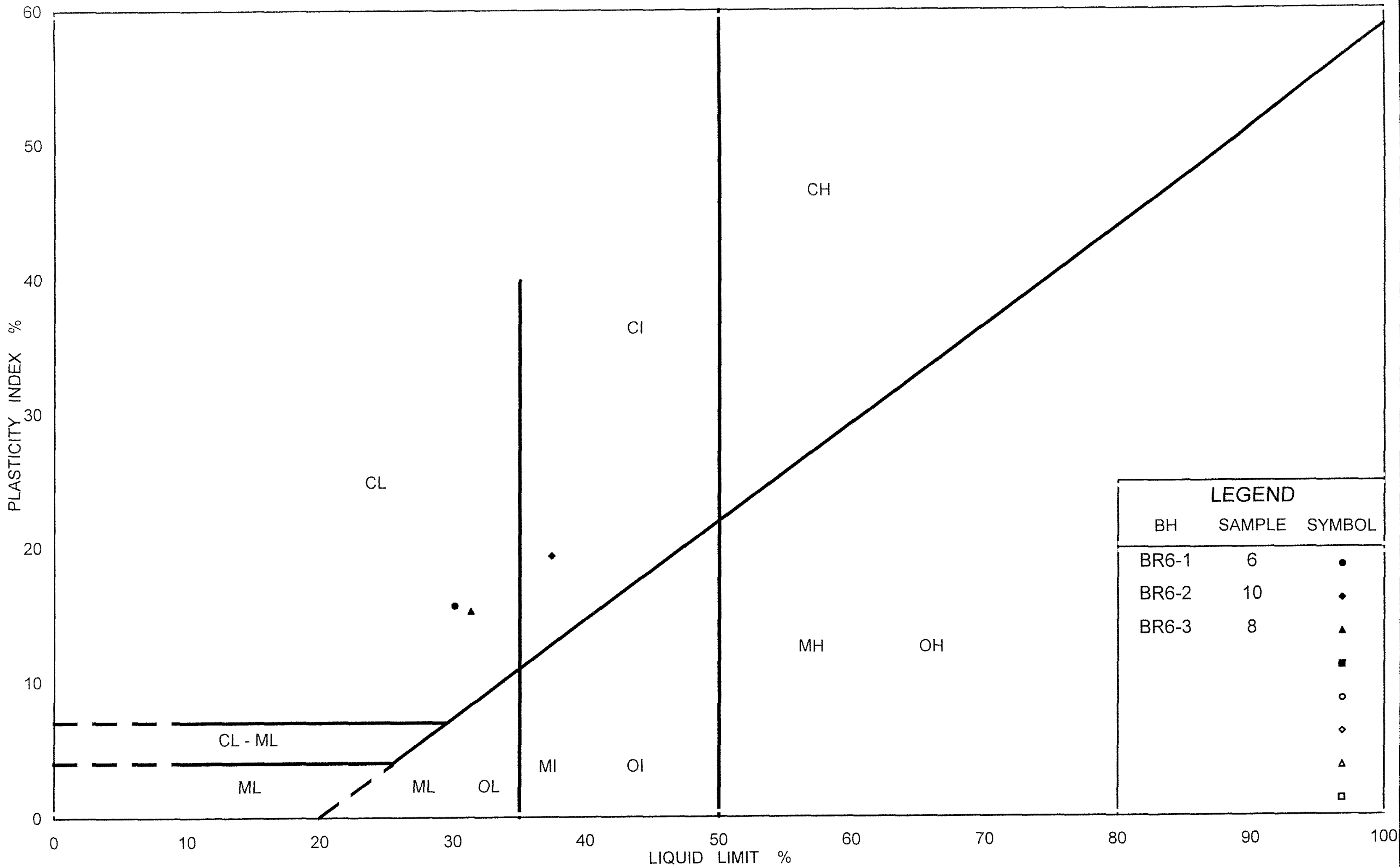
Project 021-1162-BR6

Golder Associates

Prepared by LG

Checked by ______

Oct 75 FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART
Clayey Silt (Till)

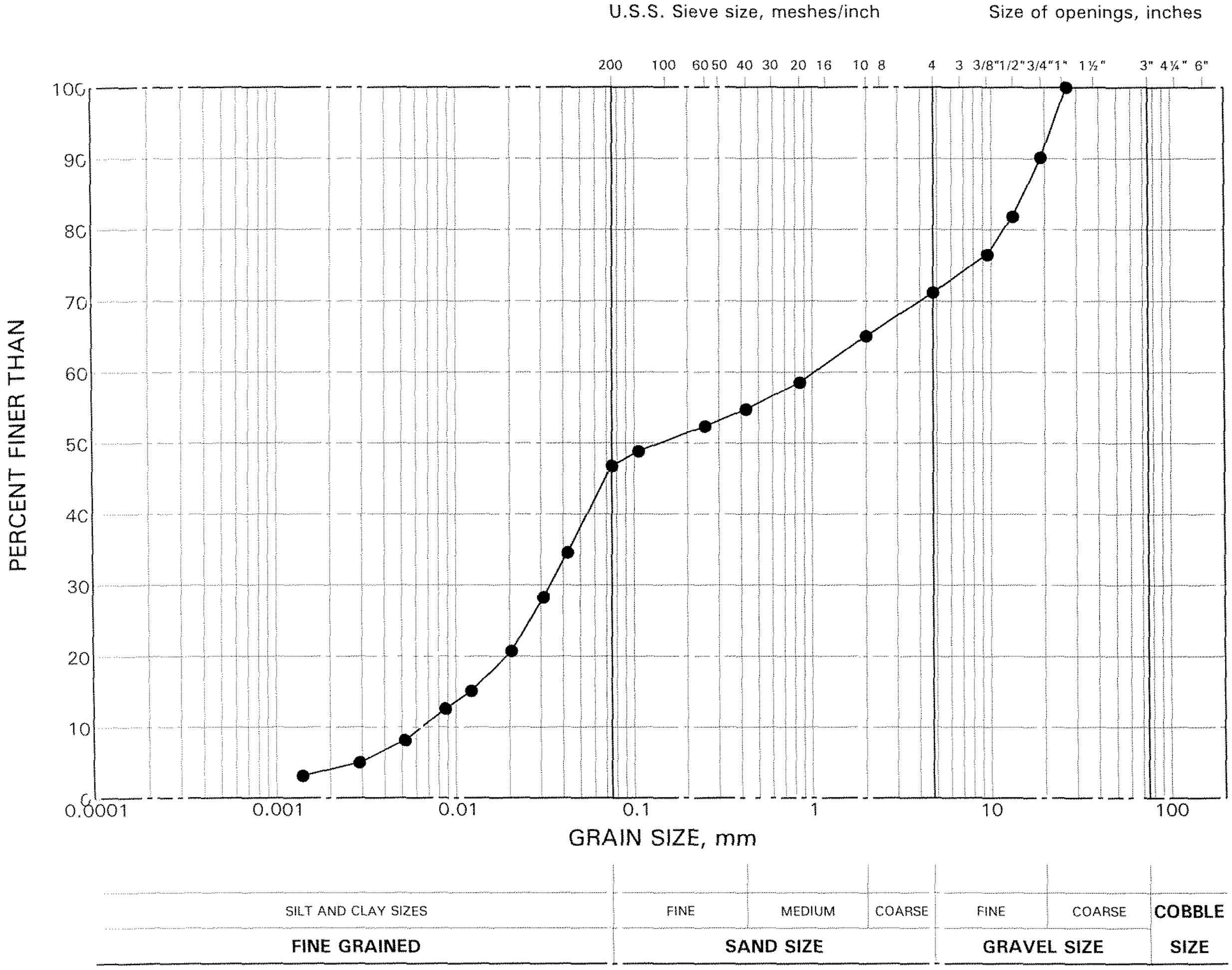
FIG No. 2

Project No. 021-1162-BR6

Date: April, 2005

GRAIN SIZE DISTRIBUTION
Sandy Silt (Till)

FIGURE 3



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	BR6-2	12	64.5

Date April, 2005

Project 021-1162-BR6

Golder Associates

Prepared by LG

Checked by ________

APPENDIX A

RECORD OF BOREHOLE SHEETS
GOLDER ASSOCIATES 1999

RECORD OF BOREHOLE 2

W.P.
DIST. 4; HWY. QEW
LOCATION:

BORING DATE: JUNE 26/98

SHEET 1 OF 2
DATUM: GEODETIC
PROJECT: 981-1108

Boring method: CME 55 BOMBARDIER, SOLID STEM AUGERS

Depth Scale (m)	Description	Elev. / Depth (m)	Sample Number	Type	Blows/0.3m	Piezometer or Standpipe Installation
0	GROUND SURFACE	76.30 / 0.00				
	Clayey Silt, trace to some sand and gravel, trace shale fragments Very stiff to stiff Brown and grey Dry (Fill)		1	50 OD	22	
			2	50 OD	9	
2	Silty Clay, trace to some sand and gravel Stiff Brown and grey Moist (Fill)	74.17 / 2.13	3	50 OD	9	
3	Silty Clay, trace sand and gravel, some silt seams and partings Hard Mottled grey and brown becoming brown Moist (Glacial Till - weathered crust)	73.40 / 2.90	4	50 OD	69	
			5	50 OD	50	
			6	50 OD	43	NATIVE BACKFILL
			7	50 OD	32	
6	Silty Clay, trace to some sand and gravel Very stiff becoming hard below 10m depth Grey becoming reddish grey Moist (Glacial Till)	70.30 / 6.00	8	50 OD	29	
			9	50 OD	24	
			10	TO	PH	
			11	50 OD	22	
10						

Columns also shown (plotted): DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m; SHEAR STRENGTH Cu, kPa (nat V - + Q - ● rem V - ⊕ U - O); HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT (Wp — W — Wl; 10 20 30 40); ADDITIONAL LAB. TESTING.

CONTINUED ON NEXT PAGE

DATA INPUT: PS JAN.25/99

DEPTH SCALE 1 to 50

Golder Associates

LOGGED: DB
CHECKED: SP

SOILM6

W.P.

DIST. 4; HWY. QEW

LOCATION: N 4791062.07; E 283044.75

RECORD OF BOREHOLE 2

BORING DATE: JUNE 26/98

DATUM: GEODETIC

PROJECT: 981-1108

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. / DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	WATER CONTENT	PIEZOMETER OR STANDPIPE INSTALLATION
		CONTINUED FROM PREVIOUS PAGE						
10–11.73	CME 55 BOMBARDIER SOLID STEM AUGERS	Silty Clay, trace to some sand and gravel Very stiff becoming hard below 10m depth Grey becoming reddish grey Moist (Glacial Till)	64.57 / 11.73	12	50 OD	42		NATIVE BACKFILL
11.73–14.48		Sandy Silt, trace clay, trace gravel, trace shale fragments Very dense Reddish brown Moist (Glacial Till)	61.82 / 14.48	13	50 OD	150 /15	o (approx. 11)	BENTONITE SEAL
				14	50 OD	150 /08		SAND FILTER NATIVE BACKFILL
14.48–15.27		Shale Highly weathered Reddish brown Dry (Bedrock)	61.03 / 15.27	15	50 OD	110 /03		
		END OF BOREHOLE						Water level in piezometer at Elev. 74.9m on July 15/98 and at Elev. 74.3m on Nov.10/98.

Chart headings: DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m; SHEAR STRENGTH Cu, kPa (nat V - +, Q - ●, rem V - ⊕, U - O); HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT (Wp, W, Wl; 10 20 30 40); ADDITIONAL LAB. TESTING

DATA INPUT. PS JAN 25/99

DEPTH SCALE 1 to 50

LOGGED: DB

CHECKED: SP

SOILM6

RECORD OF BOREHOLE BESR-5

W.P. 441-97-00
DIST. 4, HWY: QEW
LOCATION: N 4791051.574; E 282912.756

BORING DATE: SEPT.28/98

DATUM: GEODETIC
PROJECT: 981-8033

BORING METHOD: CME 55 BOMBARDIER SOLID STEM AUGERS

DESCRIPTION	ELEV. / DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
GROUND SURFACE	78.13 / 0.00					
Topsoil	77.83 / 0.30	1	50 DO	21		
Sandy Silt, some clay, trace gravel, trace organics and oxidized stains; Compact; Brown to grey brown; Dry; (Fill)		2	50 DO	24		
	76.30 / 1.83	3	50 DO	13		
Silty Clay, some sand, trace gravel, trace organics; Stiff to very stiff; Brown; Dry to moist; (Fill)		4	50 DO	18		
	75.13 / 3.00	5	50 DO	13		
Silty Clay, some sand, some organics, trace brick fragments; Soft to firm; Black; Moist to wet; (Fill)	74.63 / 3.50	6	50 DO	8	MH	
Silty Clay, some sand, trace gravel; Hard; Mottled brown and grey; Moist; (Glacial Till)		7	50 DO	32		
		8	50 DO	63		
		9	50 DO	26		
	71.58 / 6.55	10	50 DO	44		
END OF BOREHOLE						Open hole dry upon completion of drilling.

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m
SHEAR STRENGTH Cu, kPa: nat V - + Q - ● rem V - ⊕ U - O; 20 40 60 80
HYDRAULIC CONDUCTIVITY, k, cm/s
WATER CONTENT, PERCENT: Wp W Wl; 10 20 30 40

DEPTH SCALE 1 to 50

Golder Associates

LOGGED: PKS
CHECKED: SP

DATA INPUT: PS NOV 5/98

ILM6

APPENDIX B

NON STANDARD SPECIAL PROVISIONS

VIBRATION MONITORING - Item No.

Special Provision

Scope

This special provision describes requirements for vibration monitoring during the piling works for the south pier of Bridge 6 and west pier of Bridge 7.

References

The subsurface conditions at the site are described in the following Foundation Investigation Report for G.W.P 443-97-00:

- Bridge 6; Burlington Street S-W and E-S Ramp over RHCE W-S Ramp and Queen Elizabeth Way, Burlington Street Expressway Interchange (dated March 2005).
- Bridge 7; Burlington Street S-E Ramp over Red Hill Creek W-S Ramp, Burlington Street Expressway Interchange (dated March 2005).

Definitions

Quality Verification Engineer (QVE): An Engineer with a minimum of five (5) years experience in the field of installation of piling and vibration monitoring or alternatively has demonstrated expertise by providing satisfactory quality verification services for the work at a minimum of two (2) projects of similar scope to the contract. The Quality Verification Engineer shall be retained by the Contractor to ensure general conformance with the contract documents and issue certificate(s) of conformance.

Submission Requirements

The Contractor shall submit details of the vibration monitoring plan to the Quality Verification Engineer for review. The submittals shall satisfy the specifications and at a minimum contain the following specific information:

Qualifications of vibrations monitoring specialist.
Proposed instrumentation.
Proposed location of instruments on existing 2100 mm diameter discharge pipe.
Proposed frequency of readings.
Proposed methods for adjusting piling methods if readings show excess vibrations.

At least 3 weeks prior to the piling operations, the Contactor shall submit six (6) copies of the submittal to the Contract Administrator. The submittals shall satisfy the specifications and at a minimum contain the above information as provided to the Contractor's Quality Verification Engineer.

Pre-augering

The piles that require pre-augering and the elevation of the base of the pre-augering for these piles is shown on the Contract Drawings.

Backfill to the pre-augered holes should be with specially graded sand shall meet the following gradation requirements:

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

Monitoring

The vibration monitoring equipment shall be placed on the existing steel liner surrounding the pipe such that it will not be disturbed. The location should be as close as possible to the piling works.

The vibrations on the existing pipe shall not exceed 50 mm/s (peak particle velocity).

The Contractor shall take readings on the pipe during the driving/pre-augering of each pile, starting with the pile furthest away from the existing pipe. As a minimum, the readings should be taken and recorded during the first 6 m of driving and during seating of the pile into the till or bedrock.

The results shall be submitted to the Contract Administrator after each pile has been driven/pre-augered prior to continuing with the subsequent piles. As a minimum, the pile number, location, set criteria and driving log must be submitted with vibration monitoring results.

If the results are acceptable, the Contractor may continue with the next piles with readings taken during driving of each pile. The results of subsequent piles should be submitted to the Contract Administrator after each pile has been driven/pre-augered.

If the readings are not within the limits stated above, the Contractor must alter his driving/augering procedures until the vibrations on the existing pipe are within acceptable levels. The above process must be repeated for each pile.

Basis of Payment

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

END OF SECTION

STANDPIPE PIEZOMETERS - ITEM NO.
SETTLEMENT PLATES - ITEM NO.
VIBRATING WIRE PIEZOMETERS - ITEM NO.
VIBRATING WIRE SETTLEMENT CELLS - ITEM NO.

Special Provision **April 2005**

1.0 SETTLEMENT INSTRUMENTATION AND MONITORING - GENERAL

The Contractor shall retain a Geotechnical Consultant with MTO classification of "Geotechnical (Structures and Embankments) - High Complexity", to undertake the supply and installation of geotechnical instruments.

"The Contractor" shall be understood to refer to the Contractor and their Geotechnical Consultant.

1.1 Scope

This non-standard special provision contains the requirements for the supply and installation of the following geotechnical instruments:

- Vibrating Wire Settlement Cells with Vented Reference Reservoirs (SC);
- Settlement Plates (SP);
- Vibrating Wire Piezometers (VWP);
- Standpipe Piezometers (SSP).

1.2 Purpose

1.2.1 The purpose of these instruments is to monitor the progress of settlement and dissipation of excess pore water pressure of the preload embankments.

1.2.2 The timing for removal of the preload and construction of other works will be controlled by the instrumentation readings.

1.2.3 The completed embankments shall remain undisturbed until such time as the monitoring shall indicate that a sufficient degree of consolidation of the foundation soil has been achieved.

1.3 Or equal

The term, "or equal" shall be understood to indicate that the equal product is the same or better than the specified product in function, performance, reliability, quality and general configuration.

1.4 Notification

The Contract Administrator shall be notified a minimum of 15 working days in advance of commencing the installation of instruments.

1.5 Submission Requirements

The Contractor shall submit details of proposed installation methods including locations and types of survey benchmarks, and installation schedule to the Contract Administrator for approval, a minimum of 15 days before the start of instrument installation.

2.0 DRAWINGS

Reference shall be made to the following contract drawings which are contained elsewhere in the Contract:

- Monitoring Section Location Plan
- Typical Monitoring Sections
- Typical Instrument Installation Details

3.0 SITE CONDITIONS

3.1 Subsurface Conditions

The subsurface conditions at the site are described in the following Foundation Investigation Reports for G.W.P 443-97-00:

- Bridges 1A/1B and 2; Burlington Street W-S ramp over Beach Boulevard, Burlington Street Interchange (dated April 2005);
- Bridge 3; Burlington Street W-S ramp over Red Hill Creek, Burlington Street Interchange (dated April 2005);
- Bridge 4; Burlington Street and S-E ramp over Red Hill Creek, Burlington Street Interchange (dated April 2005);
- Bridge 5; Burlington Street S-RHCE ramp over Red Hill Creek, Burlington Street Interchange (dated April 2005);
- Bridge 6; Burlington Street over QEW and RHCE W-S ramp, Burlington Street Interchange (dated April 2005);
- Bridge 7; Burlington Street S-E ramp over RHCE W-S ramp, Burlington Street Interchange (dated April 2005);

- High Fill Embankments, Retaining Walls and Swamp Crossing; Burlington Street Interchange (dated April 2005);

3.2 Equipment Operation and Weather Conditions

All monitoring equipment and associated materials shall be capable of withstanding the range of temperatures possible for their location within the ground or on the surface. Monitoring will be conducted year round (by others).

4.0 INSTALLATIONS

4.2 Survey Bench Marks

4.2.1 The Contractor shall provide non-yielding deep seated survey bench marks and shall establish the geodetic elevation of each such benchmark.

4.2.2 The number and locations of bench marks shall be such that direct sighting is possible from all geotechnical instruments to at least one bench mark.

4.3 Personnel

Instrument installation shall be undertaken under the full time supervision of the Contractor.

4.4 Materials and Equipment

The Contractor shall supply all materials and equipment required for the installation of instrumentation unless otherwise noted. All instrumentation shall be and shall remain in proper working condition until the completion of the contract monitoring period.

4.5 Instrument Location

Prior to the installation of instruments, the Contractor shall accurately survey and stake the location of each instrument and obtain a ground elevation at each instrument location.

4.6 Underground Utilities

The Contractor shall be responsible for locating and protecting all underground utilities prior to drilling boreholes for installing instruments. Any damage to underground utilities caused by the Contractors work shall be repaired by the Contractor at no cost to the Contract Administrator.

4.7 Marking and Labelling

4.7.1 The location of any above ground monitoring fixture shall be made clearly visible to nearby traffic before, during and after embankment construction. Marking shall be of sufficient size to be visible from a reversing vehicle and after heavy snow falls.

4.7.2 Instruments and / or their data cables shall be clearly labelled in the field, each instrument having a unique identifier. The labelling shall remain legible for the entire period of monitoring.

4.8 Protection of Instruments

All instruments shall be adequately protected by the Contractor such that they are not damaged during construction operations, including preload removal. Any instrument damaged by the Contractor's work shall be immediately replaced at the Contractor's cost.

4.9 Accuracy of Surveying

Elevations of the deep benchmarks and all other elevations ~~required to be determined by~~ the Contractor shall be surveyed to an accuracy of plus / minus 2 mm or better.

4.10 Survey Personnel

Surveying to establish the benchmarks and other elevations ~~which the General Contractor~~ is required to determine elsewhere in this contract shall be carried out by a registered surveyor with appropriate equipment and experience. The surveyor shall be retained by the Contractor.

4.11 Boreholes for Installation of Instrumentation

4.11.1 The Contractor shall make a basic stratigraphic log of boreholes as they are being drilled. In-situ or laboratory testing is not required.

4.11.2 Boreholes shall be advanced using conventional drilling ~~methods and shall be~~ as straight and vertical as practical.

4.12 Installation Program

All instrumentation shall be installed prior to embankment construction and following installation of wick drains (by others). All Vibrating Wire Piezometers (VWP) are to be located at the centroid of the nearest triangular grouping of wick drains such that the VWP is at an equal distance from each of the closest wick drain installations.

5.0 MONITORING

5.1 Personnel / Access

Data collection, interpretation and reporting shall be conducted by others, under the direction of the Contract Administrator.

The Contractor shall provide safe access and assistance to others reading all geotechnical instruments.

5.2 Monitoring Program

5.2.1 The Contractor shall meet with the Contract Administrator and staff responsible for the on-going monitoring immediately after installation of all of the instruments and before the start of embankment construction. At this meeting, the Contractor shall hand over to the Contract Administrator all records pertaining to the installation of the instruments and all equipment to be supplied by the Contractor.

5.2.2 Monitoring by others for the baseline readings shall commence within seven days after completion of installation of all of the instruments and shall continue on a schedule to be determined by the Contract Administrator throughout the construction of the embankments and on selected instrumentation for up to twelve months following construction of the embankments.

6.0 PAYMENT

6.1 Measurement and Basis of Payment

The measurement and the basis of payment is outlined in the specific Non-Standard Special Provisions for each type of instrument to be installed.

END OF SECTION

SETTLEMENT PLATES - ITEM NO.

Special Provision **April 2005**

1.0 GENERAL

1.1 <u>Scope</u>

1.1.1 This non-standard special provision contains the requirements for the supply and installation of settlement plates.

1.1.2 The purpose of the settlement plates is to directly monitor settlements of the embankment base. Settlement is measured by survey of the top of the rod with reference to stable, non-settling benchmarks.

1.2 General Procedure

1.2.1 Rods shall be attached to a settlement plate at existing ground level. As embankment construction proceeds, the rods shall be extended above the new ground level.

1.2.2 Sleeves around the rods shall be installed to reduce friction and allow uninhibited movement of the rod with the plate. As embankment construction proceeds, the sleeves shall be extended above the new ground level.

1.2.3 A protective surround shall be extended with the rods and sleeves as embankment construction proceeds.

1.3 Location

The Contractor shall install the settlement plates at the locations shown on the Contract Drawings.

2.0 MATERIALS

2.1 General

The Contractor shall supply all materials and equipment required for the installation of the settlement plates.

2.2 Plate

The Contractor shall supply a steel plate with thickness of at least 6.35 mm. It shall be at least 0.5 m by 0.5 m in plan dimensions.

2.3 Rod

2.3.1 The Contractor shall supply a steel pipe with an outside diameter of at least 25 mm.

2.3.2 The top of the rod shall be capped in such a way that a single survey point can be clearly identified and returned to.

2.4 Friction Reducing Sleeve

The Contractor shall supply a PVC pipe, friction reducing sleeve with an internal diameter slightly larger than the rod diameter.

2.5 Protective Surround

The Contractor shall supply a protective surround for the portion of the rod and sleeve within the embankment. The surround shall consist of a 300 mm diameter corrugated metal pipe (CMP) filled with compacted sand.

2.6 Monitoring Equipment

The elevation of the top of the settlement rods shall be surveyed by an experienced surveyor, retained by the Contractor, to provide the datum readings. The surveyor shall provide suitable equipment capable of surveying settlement rod elevations to an accuracy of ± 2 mm or better.

3.0 INSTALLATION

3.1 General

The Contractor shall install settlement plates as detailed elsewhere in the Contract, in addition to what is stated or emphasized below.

3.2 Settlement Plate

3.2.1 The settlement plate shall be installed horizontally on undisturbed native soil, just below the existing ground.

3.2.2 The elevation of the base of the plate shall be surveyed by the Contractor before backfilling.

3.3 Rod

3.3.1 The rod shall be fixed to the centre of the plate and perpendicular to the plate.

3.3.2 The rod will be extended in 1.5 m increments as the embankment increases in height.

3.3.3 The coupling of the rods shall be such that all sections have the same axis and no separation or contraction will occur at the couplings.

3.4 Friction Reducing Sleeve

The friction reducing sleeve shall extend over the entire length of the rod that is below ground and within the embankment fill.

3.5 Protective Surround

3.5.1 The CMP protective surround shall be extended in 1.5 m increments with the rods.

3.5.2 The settlement rod shall be in the centre of the CMP.

3.5.3 The annulus between the CMP and the friction reducing sleeve shall be filled with compacted sand to a level no higher than the top of the sleeve.

3.6 Installation Details

3.6.1 The elevation, easting and northing of the centre of the base of the plate shall be surveyed by the Contractor.

3.6.2 The elevation, easting and northing of the top of the rod shall be surveyed by the Contractor.

3.6.3 The total distance from the base of the plate of the top of the rod shall be measured and recorded to an accuracy ± 2 mm or better.

4.0 REPORTING

The Contractor shall record and report relevant installation details to the Contract Administrator. These include, but are not limited to:

- Settlement rod and plate location, easting and northing;
- Elevation of plate and rod;
- Distance between base of plate and top of rod;

- Dates of installation and datum readings;
- Installation notes / sketches;
- Description of settlement rods, sleeve, plate.

5.0 PAYMENT

5.1 Measurement for Payment

Measurement of the item, "Settlement Plates", including all appurtenances, is by quantity. The unit of measurement is each.

5.2 Basis of Payment

Payment at the contract price for the above item shall be full compensation for all labour, equipment and material to do the work, including the establishment of the required benchmarks and surveying required to establish the locations and initial base line elevations for each settlement plate and the required reporting.

END OF SECTION

VIBRATING WIRE PIEZOMETERS - ITEM NO.

Special Provision **April 2005**

1.0 GENERAL

1.1 Scope

1.1.1 This non-standard special provision contains the requirements for the supply and installation of vibrating wire piezometers (VWP). The installation shall be carried out by the Contractor.

1.1.2 The purpose of the piezometers is to monitor pore water pressure at depths within the foundation soil. The piezometer readings shall help to establish the timing for the removal of preload fills.

1.2 General Procedure

1.2.1 The piezometers shall be installed in boreholes after wick drain installation but prior to embankment construction. The boreholes shall be of sufficient diameter to accommodate installation of one VWP sensor, filter sand, bentonite plug and grout.

1.2.2 Installation details are shown elsewhere in the Contract.

1.2.3 The VWP signal cables shall be extended out of the embankment footprint area through metal (or other suitable material) conduit, buried in trenches, as shown elsewhere in the Contract.

1.2.4 Boreholes containing VWP sensors shall be at least 3 m from other boreholes.

1.2.5 Boreholes containing VWP sensors shall be equidistant from nearby wick drains located at the centroid of the nearest triangular grouping of wick drains.

1.3 Locations

1.3.1 The Contractor shall install the VWP sensors at the locations and to the depths shown on the Contract Drawings.

2.0 MATERIALS

2.1 Vibrating Wire Piezometers

2.1.1 The Contractor shall supply VWP borehole piezometers by Slope Indicator model 52611020 (-5 psi to 50 psi) - or equal, compatible with the Slope Indicator VW Data Recorder model 52613500 – or equal. All VW piezometers shall be of the same make.

2.1.2 All piezometers shall be calibrated prior to installation and the calibration data for each piezometers shall be provided to the Contract Administrator.

2.2 Signal Cable

The Contractor shall supply Slope Indicator model 50613524 cable – or equal. The length of cable for each piezometer shall be carefully estimated from the construction drawings to ensure that there is enough signal cable for each piezometer to provide enough slack in the borehole and along the monitoring trenches until each cable is out of the embankment footprint area where they shall be protected from earthmoving equipment.

2.3 Bentonite

2.3.1 The Contractor shall supply bentonite (OPSS 1205) in pellet form in sufficient quantity to form borehole plugs as required.

2.4 Filter Sand

The Contractor shall supply clean washed sand for filter around VWP sensors. The sand shall be Sakcrete washed general purpose sand - or equal.

2.5 Grout

The Contractor shall supply bentonite-cement grout for general borehole backfilling. A suitable grout mix design consists of 23 kg of bentonite (OPSS 1205), 143 litres of water and 40 kg of cement (Type 10 – OPSS 1301).

2.6 Readout Unit

The Contractor shall supply a Slope Indicator Company VW Data Recorder model 52613500 – or equal to store the VW piezometer (and VW settlement cell) readings. The Contractor shall also supply Slope Indicator Company MP Manager and MP Graph software - or equal, to generate plots of pore pressure with time. The readout box and software shall become the property of the Ministry and shall be handed over to the

Contract Administrator at the hand over meeting following completion of instrument installation.

2.7 Trench Burial and Metal Conduit

The signal cable for each piezometer shall be buried in a shallow monitoring trench as shown elsewhere in the contract and taken out of the embankment footprint area. The Contractor shall supply suitable conduits (metal or PVC) to protect the signal cables in the trenches and above ground surface during trench backfill and embankment fill placement. If appropriate, several signal cables may be housed in a single conduit and laid in a common trench.

2.8 Universal Terminal Box

2.8.1 The cable and the "quick connect" at the reading end of all the piezometers for a monitoring section shall be connected to a universal terminal box, Slope Indicator model 57711600 – or equal, equipped with a universal connector, Slope Indicator model 57705001 – or equal.

2.8.2 For the monitoring sections specified in the Contract, a minimum of one such terminal box will be required, for the west embankment. The Contractor shall ensure access to the terminal box at all times including but not limited to snow clearing in the winter.

2.8.3 The terminal boxes shall be locking and waterproof and securely attached to posts. Posts shall be 100 mm x 100 mm, minimum 3 m long wooden posts installed with minimum of 1.2 m embedment.

3.0 INSTALLATION

3.1 General

Installation of the VWPs shall be as per the manufacturer's recommendations in addition to what is stated or emphasized below.

3.2 Borehole Installation

3.2.1 The borehole shall be advanced to 300 mm below the lowest tip elevation using suitable drilling techniques. The sides of the borehole shall be stable and the borehole shall be free of drilling mud and debris.

3.2.2 The piezometer shall be installed as shown elsewhere in the Contract.

4.0 REPORTING

The Contractor shall record and report relevant installation details to the Contract Administrator no later than 3 days after installing all vibrating wire piezometers. These include, but are not limited to:

- VWP location, easting, northing;
- Elevations of VWP sensors;
- Stratigraphic log of subsurface conditions, including drilling method notes;
- Dates of installation;
- Installation notes / sketches;
- Model, make and serial numbers of VWP sensors, readout unit and signal cable;
- Calibration details of VWP sensors.

5.0 PAYMENT

5.1 Measurement of Payment

Measurement for the item "Vibrating Wire Piezometers", including all appurtenances, is by plan quantity. The unit of measurement is each.

5.2 Basis of Payment

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

END OF SECTION