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REPORT ON

**DETAIL DESIGN
FOUNDATION INVESTIGATION AND DESIGN
QEW / FOURTH LINE UNDERPASS
QUEEN ELIZABETH WAY
THIRD LINE TO 1 KM EAST OF TRAFALGAR ROAD
G.W.P 189-00-00, SITE NO. 10-159
MINISTRY OF TRANSPORTATION, ONTARIO
OAKVILLE, ONTARIO**

Submitted to:

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**PART A
DETAIL DESIGN
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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by URS Canada Inc. (URS) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out a foundation investigation for the detail design of the proposed new structure of Fourth Line over the Queen Elizabeth Way (QEW) in Oakville, Ontario. This work forms part of the overall project which includes widening of the QEW, twinning of the Sixteen Mile Creek bridge structure and construction of new culverts, retaining walls and high mast lighting.

The terms of reference for the scope of work are outlined in Golder's proposal P01-1104, dated March 2000, that forms part of the Consultant's Agreement (Number 2005-A-000219) for this project. A digital file of the General Arrangement plan for the Fourth Line structure showing the proposed abutment and pier layout configuration was provided to Golder by URS in October 2006.

2.0 SITE DESCRIPTION

The site is located on the QEW at Fourth Line between Dorval Drive and Third Line in Oakville, Ontario. The proposed widening of the QEW, as well as addition of High Occupancy Vehicle (HOV) lanes along this stretch of the QEW, will require replacement of the Fourth Line structure over the QEW. The new structure will be located along the same alignment as the existing Fourth Line bridge.

The existing rigid frame structure, built in the 1950s, is a 56 m long single span bridge carrying two lanes of traffic over the QEW. The existing bridge is founded on shallow spread footings. The bridge approach embankments rise up to about 7 m above the surrounding land which is generally flat and has grass cover.

3.0 INVESTIGATION PROCEDURES

The fieldwork at the Fourth Line site was carried out between November 28 and December 6, 2001, at which time nine (9) boreholes were advanced. The locations of these boreholes in plan are shown on Drawing 1.

The field investigation was carried out using a track-mounted CME 75 drill rig and truck-mounted CME 75 drill rig supplied and operated by Geo-Environmental Drilling Ltd. of Milton, Ontario. The boreholes were advanced through the overburden using 100 mm outside diameter (O.D.) continuous flight solid stem augers. Soil samples were obtained at intervals of 0.75 m to 1.5 m in depth, using 50 mm outer diameter (O.D.) split-spoon samplers in accordance with Standard Penetration Test (SPT) procedures. Samples of the bedrock were obtained using an 'NQ' size rock core barrel.

The boreholes were advanced to depths ranging from 3.1 m to 14.2 m below the existing ground surface (including rock coring). Boreholes F4, F5, F8 and F9 were extended into the bedrock by coring for lengths ranging from 3.1 m to 4.6 m after augering a short depth into the bedrock. The groundwater conditions in the open boreholes were observed during the drilling operations. In order to permit monitoring of the groundwater levels at the site, piezometers were installed in Boreholes F1, F2 and F10 and sealed at a selected depth within the bedrock/residual soil. The piezometers installed in Boreholes F1 and F2 consist of a 25 mm outside diameter rigid PVC tubing with 0.3 m long slotted tip and the piezometer installed in Borehole F10 consists of a 50 mm outside diameter rigid PVC tubing with a 1.5 m long slotted tip. All boreholes were backfilled completely with a mixture of cuttings and bentonite pellets in accordance with Reg. 903 in place at the time of drilling. The piezometer installation details and water level readings are described on the Record of Borehole sheets that follow the text of this report.

The fieldwork was monitored throughout by members of our engineering and technical staff, who located the boreholes, arranged for the clearance of underground service locations, supervised the 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 and staked in the field by Callon Dietz Inc. and are referenced to MTM NAD 83 co-ordinate system for location and to 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 by members of our engineering staff relative to the staked locations and/or existing bridge features.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

The site is located in the physiographic region known as the Iroquois Plain. The Iroquois Plain is generally composed of shallow deposits of sand and till covering portions between Hamilton and Toronto¹. The surface topography slopes down gradually and fairly uniformly towards Lake Ontario. The overburden in the general area of the site consists of a shallow cover of clayey silt till and residual soil which is underlain by bedrock comprised of red shale of the Queenston Formation.

4.2 Subsoil Conditions

The detailed subsurface soil, bedrock and groundwater conditions as encountered in the boreholes advanced during the current investigations, together with the results of the laboratory tests carried out on selected soil samples, are given on the attached Record of Borehole sheets 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 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 proposed bridge location is shown on Drawings 1 and 2.

In general, at the bridge site, the subsoils consist of a thin layer of topsoil underlain by fill at the toes of the road embankment slopes or asphalt underlain by fill within the road embankment. The fill was underlain by clayey silt to silty clay residual soil and shale bedrock of the Queenston Formation. Details of the subsurface conditions are given below.

4.2.1 Topsoil

Between 0.1 m and 0.2 m of topsoil was encountered at the existing ground surface in Boreholes F1, F2, F6, F7 and F10, located at the toes of the road embankment slopes. The ground surface at these borehole locations ranges between Elevation 110.0 m and 110.6 m.

¹ Chapman, L.J. and Putnam, D.F., 1984. The Physiography of Southern Ontario, 3rd Edition (Ontario Geological Survey, Special Volume 2). Ontario Ministry of Natural Resources.

4.2.2 Embankment Fill

Between 0.1 m and 0.2 m of asphalt was encountered at the existing ground surface in Boreholes F4, F5, F8 and F9, located on the existing Fourth Line paved section. The road grade ranges between Elevation 116.9 m and 117.2 m at these borehole locations.

Fill materials were encountered in all the boreholes below the topsoil or below the asphalt. Typically, in the boreholes drilled on Fourth Line, a 0.6 m to 2.7 m thick layer of silty sand and gravel fill was encountered immediately below the asphalt. Below this road base granular fill, the boreholes penetrated a 4.1 m to 7.1 m thick layer of fill comprised of clayey silt to silty clay. In Borehole F8, there was a 2.7 m thick layer of sand and gravel fill below the clayey silt to silty clay fill. Occasional rootlets and/or asphalt fragments were encountered within the fill in Borehole F9. In the boreholes located at the toes of the embankment slopes, the fill varied in thickness between 0.5 m and 0.7 m and was comprised of brown silty clay.

SPT 'N' values measured within the silty sand and gravel fill ranged between 12 and 29 blows per 0.3 m of penetration indicating a compact relative density. In Borehole F8, one 'N' value of 84 blows per 180 mm of penetration was recorded within the granular fill underlying the clayey silt to silty clay fill, which is likely the result of encountering a cobble or small boulder within the fill at this depth.

SPT 'N' values measured within the clayey silt to silty clay fill ranged between 4 and 24, indicating a firm to very stiff consistency. In general, the 'N' values were less than about 15 blows indicating a firm to stiff consistency. One instance of hard augering was noted within the clayey fill deposit in Borehole F4, which could indicate the presence of cobbles/boulders.

The natural water contents measured on the samples of the fill were between 17 percent and 21 percent.

4.2.3 Clayey Silt to Silty Clay (Residual Soil)

A 0.4 m to 1.2 m thick deposit of residual soil was encountered below the fill in Boreholes F1, F2, F5, F6 and F9. The residual soil is derived through weathering of the shale bedrock and is typically comprised of clayey silt to silty clay containing trace sand and shale fragments. The surface of the deposit was encountered between Elevation 109.1 m and 109.8 m.

SPT 'N' values measured within the clayey silt to silty clay residual soil layer ranged between 17 and 48 blows per 0.3 m of penetration indicating that the deposit has a very stiff to hard consistency. Grain size distribution curves for three selected samples from the residual soil are presented on Figure 1. The grain size curve for a sample of the weathered bedrock from Borehole F5 is also shown on this figure.

The results of Atterberg limits testing carried out on three samples of the residual soil are presented on Figure 2. The test results gave liquid limits ranging between 31 and 42 percent, plastic limits between 7 and 19 percent and plasticity indices between 14 and 23 percent. This indicates that the material is classified as a clayey silt of low plasticity to a silty clay of intermediate plasticity.

The natural water content measured on samples of the clayey silt to silty clay residual soil ranged between 10 and 15 percent.

4.2.4 Bedrock

Shale bedrock of the Queenston Formation was encountered in all of the boreholes. Generally, the surface of the bedrock is at about Elevation 109.4 m in the vicinity of the north abutment and at about Elevation 108.5 m in the vicinity of the south abutment, and ranges between Elevation 109.3 m and Elevation 107.9 m as presented in Table 1.

The bedrock core samples obtained consist of reddish-grey, slightly to highly weathered, thinly layered, fine grained, calcareous shale. Seams or layers of grey, fresh to slightly weathered limestone and siltstone, 25 mm to 250 mm in thickness, were present within the shale bedrock. The Total Core Recovery (TCR) was between 90 percent and 100 percent and the Rock Quality Designation (RQD) measured on the core samples ranged from about 40 percent to 80 percent. This indicates rock mass variable in quality, ranging from poor to good; the quality tended to decrease with depth into the rock formation. The upper 0.8 m to 1.5 m of the shale is typically moderately to highly weathered and the rock is slightly to moderately weathered below this depth.

Point load strength tests were performed on selected samples of the rock core from Boreholes F4, F5, F8 and F9. Diametral (i.e. horizontal or perpendicular to the core axis) point load strength index values are shown on the Record of Drillhole Sheets. Diametral point load index values on samples of the shale range from 0.20 MPa to 1.07 MPa which correspond to estimated unconfined compressive strengths (UCS) between 5 MPa and 21 MPa. The axial point load index values on the samples of the shale range from 0.96 MPa to 2.82 MPa corresponding to estimated UCS values between 22 MPa and 61 MPa. Using the Intact Rock Strength Classification table (CFEM 1993, Table 3.4), these values indicate that the shale is classified as very weak to strong; typically being stronger in the axial (i.e. vertical) direction. The results of the point load testing are presented in Table 2.

One diametral point load index value measured on a sample of the siltstone/limestone interlayers was 3.72 MPa corresponding to an estimated UCS value of 83 MPa. Using the Intact Rock Strength Classification table, this sample is classified as strong. The thickness and depth of these interlayers within the rock core obtained are shown on the Record of Drillhole Sheets. The

interlayers are typically less than 100 mm thick; however, interlayers up to 250 mm thick were encountered in the boreholes at this site.

4.2.5 Groundwater Conditions

The water levels were noted during and after the drilling and coring operations in the boreholes. Piezometers were installed in Boreholes F1, F2 and F10 and sealed into the shale bedrock/residual soil. Details of the piezometer installations and groundwater levels are shown in the Record of Borehole Sheets and presented in Table 3.

Based on the water level measurements, the groundwater table appears to slope downward towards the south from about Elevation 108.5 m on the north side of the QEW to about Elevation 107.8 m on the south side. It should be noted that groundwater levels in the area are subject to seasonal fluctuations due to precipitation events.


4.3 Closure


The senior field technician supervising the drilling program in 2001 was Mr. Suresh Bainey. This report was prepared by Ms. Sarah Poot, P.Eng., and the technical aspects were reviewed by Ms. Anne Poschmann, P.Eng. A quality control review of the report was provided by Mr. Jorge Costa, P.Eng., a Designated MTO Contact for Golder.

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SEP/ASP/JMAC/lb

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PART B

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5.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides foundation design recommendations for the proposed QEW/Fourth Line Underpass as part of the QEW/Third Line to Trafalgar Road reconstruction project. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during 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.

The Fourth Line replacement structure will be 76 m in length and about 29 m wide (i.e. about 20 m wider than the existing structure) to accommodate four lanes of traffic. We understand that the existing bridge will be removed prior to construction of the new bridge. During this time, Fourth Line is expected to be closed to traffic. Integral abutments are being considered for design of the structure with the central pier supported on a shallow foundation. The existing road grade at the bridge is about 7 m above the surrounding ground surface at the toes of the embankments which is at about Elevation 110 m. The general arrangement drawing was provided to us by URS in October 2006.

5.1 Foundation Alternatives

Table 4 summarizes the advantages, disadvantages, relative costs and risks/consequences of the deep and shallow foundation alternatives considered for this site. Discussion on the alternatives is given below.

Spread footings placed on/within the shale bedrock are feasible for support of the bridge for both the abutments and the central pier. The bedrock is at about 3 m depth below the existing QEW grade. At the abutments, excavation to the bedrock surface will be required extending through the existing fill and will be up to about 9 m below the Fourth Line grade. Alternatively, at the abutments, consideration could be given to the use of spread footings perched within the embankment fill at a higher elevation. In this case, excavation of the existing fill would still be required in order to provide an adequate thickness of Granular 'A' core pad beneath the new footing.

Deep foundations, such as steel H-piles or caissons, are also considered feasible for this site, especially if integral abutments are preferred. Given the relatively shallow depth of native

overburden, in conjunction with the proposed underside of the abutments at Elevation 112.6 m at the south end of the structure and 112.9 m at the north end of the structure, driving of steel H-piles to the surface of the shale bedrock would not achieve the minimum required pile length for integral abutment design. Therefore, in order to achieve the minimum pile length, piles would either have to be driven within pre-drilled/pre-augered holes and socketted into the shale bedrock or driven within a backfilled trench excavated into the bedrock.

Caissons socketted into the shale bedrock are also feasible at this site.

In the case of both the pre-augered holes for piles and the caissons, the presence of strong rock layers within the shale could create some difficulties during installation. Non-standard special provisions (NSSPs) would be developed to alert the contractor of the presence of these strong layers to ensure that the proper equipment is available on site to form the socket within the rock as discussed in subsequent sections of this report.

From a foundations perspective, the favoured option is the use of spread footings on shale bedrock to support the abutments and piers. Founding the bridge elements on spread footings on the shale bedrock would eliminate potentially difficult pile/caisson installation. Spread footings placed on the shale bedrock as opposed to perched within the fill will avoid the requirement for sub-excavation and replacement with granular and eliminate any issues associated with different fill material below the abutment footings. Further, with all foundation elements founded on the same material, the potential for differential settlement between foundation elements (abutments and pier) would be reduced.

5.2 Shallow Foundations

Consideration could be given to constructing spread footings founded on the surface of the weathered/fractured shale bedrock or on the shale bedrock below this zone. Alternatively, spread footings founded within the embankment fill (i.e. perched) are also feasible. Design recommendations for these alternatives are given below.

5.2.1 Geotechnical Resistance

For spread footings founded within the shale bedrock at or below Elevation 107.0 m (i.e. below the upper weathered portion of the bedrock), a factored geotechnical resistance at Ultimate Limit States (ULS) of 1,500 kPa may be used for design. The geotechnical resistance at Serviceability Limit States (SLS) for 25 mm of settlement is greater than the ULS value and therefore the ULS value will govern.

The geotechnical resistance of the shale bedrock was estimated based on the results of laboratory point load testing, Rock Quality Designation (RQD) and extent of weathering, as well as the presence of hard layers and clay seams based on the visual examination of the rock mass from NQ core samples. This information is summarized on the Drillhole logs and Table 2 of this report.

Consideration could be given to a founding level within the weathered/fractured shale at Elevation 109.0 m for the north abutment and Elevation 108.0 m for the south abutment. For the pier, a design founding level at Elevation 108.m should be assumed. In this case, a factored axial geotechnical resistance at ULS of 750 kPa and axial geotechnical resistance at SLS of 600 kPa may be used for design.

If the abutment founding level is preferred to be at a higher elevation and/or for associated wing walls, consideration could be given to constructing abutments perched within the embankment fill. For spread footings perched within the embankment, the footings must be founded on a compacted Granular 'A' pad with a thickness of at least one footing width (i.e. 3 m thick pad for a 3 m footing width). Compaction requirements of Special Provision SP105S10 should be followed and, if warranted, an NSSP developed to ensure that 100% compaction is achieved. Given the variable consistency of the existing embankment fill and the fact that the proposed abutment footings would be wider than the existing footings, it would be necessary to remove the full depth of existing fill and replace with suitable fill across the full footing width to ensure that consistent founding conditions extend under the full area of the new footing.

Assuming that the full depth of the exiting fill is removed and replace with a Granular 'A' pad compacted to at least 100 percent of the Standard Proctor maximum dry density of the material, the footing design may be carried out using a factored geotechnical resistance at ULS of 900 kPa, and a geotechnical resistance at SLS of 350 kPa.

For spread footing design, the geotechnical resistances given above assume that the loads will be applied perpendicular to the surface of the footing. Where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with Section 6.7.4 of the *Canadian Highway Bridge Design Code (CHBDC)* and its *Commentary*, using the curves for non-cohesive soils.

5.2.2 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footings and the subgrade should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The coefficient of friction,

$\tan \phi'$, for cast-in-place concrete footings on undisturbed, properly prepared subgrade may be taken as given below.

Subgrade	Coefficient of Friction ($\tan \phi'$)
Compacted Granular 'A' pad	0.60
Shale bedrock	0.62

These represent unfactored values; in accordance with the CHBDC, a factor of 0.8 is to be applied in calculating the horizontal resistance.

5.2.3 Frost Protection

The spread footings founded on compacted fill or shale bedrock should be provided with a minimum of 1.2 m of soil cover for frost protection. If the required soil cover cannot be provided, consideration could be given to the use of rigid polystyrene foam insulation down the abutment stem to the top of the footing and out from the abutment wall for a length out from the footing equal to the depth of frost penetration. As a guideline, 25 mm of rigid polystyrene foam insulation may be used for every 0.3 m reduction in soil cover. Details of insulation placement can be given if this is to be used as an alternative to a soil cover.

5.3 Deep Foundations - Steel H-Piles

Steel H-piles driven to bedrock could be considered at this site; however, since practical refusal to driving would likely be met on shale bedrock between about Elevation 108 m and Elevation 109 m, it may not be possible to obtain the minimum required pile lengths along the entire footing. Typically, pile lengths of at least 5 m are required for the integral abutment configuration (which includes 3 m of sand-filled CSP pipe to impart sufficient flexibility of the piles to accommodate bridge deck deflections for an integral abutment structure). A minimum length of 3 m is required for driven piles in a non-integral configuration.

The proposed highest level of the underside of the pile caps at the abutments are Elevation 112.6 m and Elevation 112.9 m at the south and north abutments, respectively, which would result in pile lengths of less than 5 m. Therefore, in order to achieve the minimum pile length, consideration could be given to full length pre-augering through the overburden, residual soil and bedrock with driving or socketting of the H-piles at the bottom of the pre-augered hole. Driven piles are not a practical alternative for the central pier. Design recommendations for these two alternatives at the abutments are given in the following sections.

5.3.1 Geotechnical Axial Resistance

In order to achieve the minimum pile length, pre-augering through the embankment fill, residual soil and shale bedrock for the full depth of the pile will be required to ensure that an adequate pile length is achieved. Steel H-piles could then be installed within the pre-augered hole and driven out the bottom of the hole to refusal within the shale bedrock, below the weathered/fractured zone. For design, the factored geotechnical axial resistance at ULS for HP 310x110 piles driven to refusal on shale bedrock is 2,000 kN. The geotechnical axial resistance at SLS for 25 mm of settlement for design is 1,600 kN.

Alternatively, the H-pile could be socketted within the shale below the weathered/fractured zone. It is assumed that the load will be transferred down the H-pile (which is placed in the augered hole) and over the full socket diameter of at least 600 mm at the base. It is also assumed that the socket length will be 1 m, formed of mass concrete (minimum 30 MPa). For this option, a factored geotechnical axial resistance at ULS of 3,000 kPa for end-bearing may be used, resulting in a resistance of 850 kN for 600 mm diameter sockets and 1,900 kN for 0.9 m diameter sockets.

This alternative provides for an increased length of pile, if required, to overcome minimum pile length constraints, and has been treated as a “caisson” relying on end-bearing. However, the geotechnical axial resistance at ULS given above is smaller than that for the non-socketted alternative as the pile is resting on the bedrock rather than having been driven to seat into the bedrock.

As discussed in Section 5.2.1, the factored geotechnical axial resistance of the shale bedrock was estimated based on the result of laboratory point load testing presented in Table 2, RQD and extent of weathering, as well as the presence of hard layers and clay seams based on the visual examination of the rock mass from NQ core samples.

For integral abutments, the piles should be installed within 600 mm diameter CSP, the top 3 m of which is filled with loose sand. The sand should conform to the gradation given in the NSSP attached in Appendix A.

In order to achieve a minimum pile length of 5 m, as well as meet the above requirements for end-bearing on the good quality shale, the design tip elevations are presented in Table 5.

Pile installation should be in accordance with SP903S01. The piles should be fitted with Titus Ejector rock points or equivalent and appropriate driving procedures must be adopted to ensure adequate/proper seating of the piles without damaging the piles. An example of an NSSP for

inclusion in the Contract Documents is included in Appendix A for reference. For piles driven into the bedrock, the following note should be included on the drawings:

- “Piles to be driven to bedrock.”

5.3.2 Resistance to Lateral Loads

Lateral loading could be resisted fully or partially by the use of battered steel H-piles. If vertical piles are used, the resistance to lateral loading will have to be derived from the soil in front of the piles.

The evaluation of the piles subjected to lateral loads should take into account such factors as the relative rigidity of the pile to the surrounding soil, the fixity condition at the head of the pile (pile cap level), the structural capacity of the pile to withstand bending moment, the soil resistance that can be mobilized, the tolerable lateral deflection at the head of the pile and the pile group effects.

The pile should be modelled as a beam-column supported by springs equivalent to the passive soil reaction distributed along the shaft. The passive resistance developed for lateral deformations typical of bridge foundations is generally much less than the passive pressure associated with a full passive resistance. This full passive resistance is calculated from earth pressure theories assuming unlimited deformation of the soil. The lateral resistance of the pile may be limited by the factored structural flexural resistance of the pile rather than the resistance of the soil.

Therefore, in order to develop the full passive resistance, the pile would have to deflect a ‘large’ amount. For piles ‘fixed’ within the pile cap, the magnitude of possible deflection is further reduced and the horizontal geotechnical resistance of the pile is some fraction of the full passive resistance occurring at relatively small horizontal displacements.

It can be assumed, based on the shear strength of the soil, that the pile can be considered a laterally supported compression member. The horizontal load capacity of vertical piles may be limited in three different ways:

- The capacity of the soil may be exceeded, resulting in large horizontal movements of the piles and failure of the foundation;
- The bending moments may generate excessive bending stresses in the pile material, resulting in structural failure of the piles; or
- The deflections of the pile heads may be too large to be compatible with the superstructure.

CFEM (1992) gives two methods by which to assess the lateral capacity of a pile. The first is Brom's Method (1964), which examines failure criteria (i.e. ultimate horizontal resistance) for two types of piles – 'short piles' where the lateral capacity of the soil adjacent to the pile is fully mobilized and 'long piles' where the bending resistance of the pile is fully mobilized.

The second method examines the lateral deflections of the pile by using the horizontal subgrade reaction theory where the soil around a pile is modelled using a series of springs. The spring constant is called the coefficient of horizontal subgrade reaction, k_h (kN/m³ or kPa/m). The value of k_h is used as an input parameter into the elastic soil-structure interaction model.

The resistance to lateral loading in front of a vertical pile may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, k_h (kPa/m), is based on the equation for cohesionless soils given below:

$$k_h = \frac{n_h z}{B} \quad \text{Where} \quad \begin{array}{l} n_h \text{ is the constant of horizontal subgrade reaction (kPa/m)} \\ z \text{ is the depth (m)} \\ B \text{ is the pile diameter/width (m)} \end{array}$$

and for cohesive soils:

$$k_h = \frac{67 s_u}{B} \quad \text{Where} \quad \begin{array}{l} s_u \text{ is the undrained shear strength of the soil (kPa)} \\ B \text{ is the pile diameter width (m)} \end{array}$$

The values of n_h and s_u to be assumed in the structural analysis are given in Table 6. The different values reflect 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. The horizontal passive resistance of HP 310 x 110 piles at this site should be assumed based on Table C6.8.7.1 of the CHBDC as 260 kN (ULS, factored) and 200 kN (SLS, for 10 mm horizontal deflection at the pile cap level).

Based on the above discussion, it is considered that both the structural and geotechnical resistances of the piles should be evaluated to establish the governing case. For the proposed piles (HP 310x110) driven to bedrock at this site, the horizontal resistance at ULS will be controlled by structural limitations such as the yield moment (M_{YIELD}) of the pile (i.e. Brom's 1964 method). At SLS, the horizontal resistance of the piles will be controlled by deflections and the horizontal resistance of the pile should be calculated based on the coefficient of horizontal subgrade reaction (k_h) of the soil.

The upper zone of soil down to a depth below the pile cap equal to about $1.5 \times B$ (after Brom's 1964), where B = pile diameter, should be neglected in the calculation of lateral resistance of the pile to account for disturbance effects during installation.

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

5.3.3 Frost Protection

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

5.4 Caissons

5.4.1 Geotechnical Axial Resistance

As an alternative to driven or socketted H-piles, consideration could be given to the use of caissons for support of the abutments as well as the centre pier. The load carrying capacity for caissons depends on the total length of the caissons, the length of the rock socket and the diameter of the caissons. Where the length to diameter ratio is less than 5, it is recommended that the caissons be designed as end-bearing units. Regardless, caissons should be socketted into the shale bedrock a minimum of 2 m. A factored geotechnical axial resistance at ULS of 3,000 kPa may be used for design of caissons founded in the shale bedrock at or below Elevation 105.0 m. This results in a capacity of 1,900 kN for 0.9 m diameter sockets and 3,400 kN for 1.2 m diameter sockets. The geotechnical axial resistance at SLS for 25 mm of settlement will be greater than the ULS value and, as such, the ULS value will govern design. If a greater capacity is required, the caissons must be lengthened to a minimum of 5 diameters and a factored unit shaft friction value of 750 kPa per metre of length may be used for design.

The foundation recommendations are made on the basis of the information obtained at the borehole locations. In accordance with SP903S01, the Contractor's QVE should inspect the

caissons to determine that the conditions encountered are consistent with the information obtained from the borings, to ensure that the minimum bedrock socket lengths are achieved, and to confirm that the base of the caisson foundations have been adequately prepared/cleaned of loose drill cuttings.

Temporary liners may be required through the fill and residual soil deposits to prevent loss of ground. Groundwater seepage into the caisson excavations through the bedrock is anticipated. Surface water should be directed away from the caisson excavations to prevent road drainage from entering the excavations.

5.4.2 Resistance to Lateral Loads

The resistance to lateral loading for the caissons should be in accordance with Section 5.3.2. The horizontal passive resistance of 0.9 m diameter caissons socketted into the shale bedrock at this site should be assumed as 500 kN at ULS and 350 kN at SLS.

5.4.3 Frost Protection

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

5.5 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. Seismic (earthquake) loading must also be taken into account in the design.

The following recommendations are made concerning the design of the 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 1010) 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 3101.150 and 3121.150.

- 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 proposed embankment fill materials and the existing overburden soils and the following parameters (unfactored) may be used assuming the use of earth fill:

	Earth Fill
Soil unit weight:	21 kN/m ³
Coefficients of static lateral earth pressure:	
Active, K_a	0.31
At rest, K_o	0.47

- 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 follows, in accordance with Section C6.9.1(a) of the *CHBDC Commentary*:

- rotation (i.e. ratio of wall movement to wall height) 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.

5.5.1 Seismic Considerations

Seismic (earthquake) loading must also be taken into account in the design in accordance with Section 4.6 of the *CHBDC*. The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e. an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

$$K \gamma' d + (K_{AE} - K) \gamma' H$$

Where	K =	either the static active earth pressure coefficient (K_a) or the static at rest earth pressure coefficient (K_o);
	K_{AE} =	the seismic active earth pressure coefficient;
	γ' =	the effective unit weight of the soil (kN/m^3)
		<ul style="list-style-type: none"> • taken as soil unit weights given above for fill materials; • taken as 21 kN/m^3 for residual soil and 20 kN/m^3 for the existing fill material, where encountered;
	d =	the depth below the top of the wall (m); and
	H =	the height of the wall above the toe (m).

In this regard, the following should be taken into account in the lateral earth pressures.

- Seismic loading will result in increased lateral earth pressures acting on the abutment stem and retaining walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure. According to the National Building Code of Canada, this site is located in Seismic Zone $Z_a = 1$. The site-specific zonal acceleration ratio for Oakville in accordance with Section C4.4.4.1 of the *CHBDC Commentary* (2001) is 0.05. Based on the type of soil deposits at the site, a 30 percent amplification of the ground motion was assessed in accordance with Section 4.1.8.4 and Table 4.1.8.4 B of the NBC (2006) for Class D soils and 5 percent damped spectral acceleration $S_a(0.2)$ less than or equal to 0.25. This amplification resulted in an increase in the peak horizontal ground acceleration from 0.05g to 0.065g. The seismic lateral earth pressure coefficients given below have thus been derived based on a design zonal acceleration ratio $A = 0.065$.
- In accordance with Sections 4.6.4 and C.4.6.4 of the *CHBDC* and its *Commentary*, for structures which allow lateral yielding, the horizontal seismic coefficient, k_h , used in the calculation of the seismic active pressure coefficient, is taken as 0.5 times the zonal acceleration ratio (i.e. $k_h = 0.03$). For structures that do not allow lateral yielding, k_h is taken as 1.5 times the zonal acceleration ratio (i.e. $k_h = 0.10$). The seismic active earth pressure

coefficient is also dependent on the vertical component of the earthquake acceleration, k_v . Three discrete values of vertical acceleration are typically selected for analysis, corresponding to $k_v = +2/3 k_h$, $k_v = 0$, and $k_v = -2/3 k_h$.

- The following seismic active pressure coefficients (K_{AE}) for the two cases (Case I and Case II) may be used in design; these coefficients reflect the maximum K_{AE} obtained using the k_h and three values of k_v as described above. It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is flat.

SEISMIC ACTIVE PRESSURE COEFFICIENTS, K_{AE}

	Case I	Case II	
	Earth Fill	Granular A	Granular B Type II
Yielding wall	0.30	0.26	0.26
Non-yielding wall	0.34	0.30	0.30

Note: The above KAE values for yielding walls are applicable provided that the wall can move up to 250A (mm), where A is the design zonal acceleration ratio of 0.065. This corresponds to displacements of up to 16 mm at this site.

- These CHBDC seismic KAE values include the effect of wall friction ($\delta=\phi'/2$) and are less than the static values of K_a and K_o reported above for the very low zonal acceleration ratio for this site. Therefore the contribution of the dynamic component in the active lateral earth pressures acting on the abutment stem or retaining walls at this site is not significant and the static lateral earth pressures are adequate for design.

5.6 Approach Embankment Design

The Fourth Line roadway is at about Elevation 117 m at the abutments; the surrounding land and QEW grade is about Elevation 110 m resulting in embankments up to about 7 m high. The following sections give the results of the stability and settlement analysis carried out for the site.

5.6.1 Stability

Limit equilibrium slope stability analyses were performed using the commercially available program GeoStudio 2004 (Version 6.20), produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis. For all analyses, the factor of safety of numerous potential failure surfaces was computed in order to establish the minimum factor of safety. The factor of safety is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. A target minimum factor of safety of 1.3 is normally used for the design of embankment slopes under static conditions. This factor of safety is considered adequate for the embankments at this site considering the design requirements and the field data available.

The stability analyses were performed to check that the target minimum factor of safety was achieved for the design embankment height, excavation depths and geometries. In general, circular slip surfaces were analyzed in the design.

Engineering properties of the site soils used as input to the stability analyses have been estimated based on our experience and judgment and are presented below.

Soil Type	Unit Weight (kN/m ³)	Undrained Shear Strength (kPa)	Angle of Internal Friction
New Earth Fill (assume granular material)	21	n/a	35°
Existing Embankment Fill (typically clayey silt to silty clay)	20	n/a	32°
Clayey Silt/Silty Clay (Residual Soil)	21	100	35°
Weathered Shale (Bedrock)	22	n/a	35°

The results of the slope stability analyses indicate that the 6 m to 7 m high approach embankments with side slopes oriented at 2 horizontal to 1 vertical (2H:1V) will have a minimum factor of safety greater than 1.3 against deep-seated slope instability, assuming appropriate subgrade preparation and proper placement and compaction of embankment fill materials. This factor of safety is considered appropriate for the embankments at this site. The results of the analysis are shown on Figure 3.

5.6.2 Settlement

Apart from the area at and immediately behind the abutments, it is expected that the fill from the existing embankment fills will be left in place and the widening of the embankments will be constructed using granular earth fill. Behind the abutments, select granular fill will be used (see Section 5.5).

Settlement of the approach embankments occurs as a result of compression of new embankment fill itself, as well as consolidation of the underlying soils on which the approaches will be founded. Since the embankment height will be less than 1 m at the abutments, settlement of the underlying soils will mainly take place in the areas where the embankment is being widened. Settlement of the underlying soils beneath the existing and the widened portion of the embankments is expected to be less than 25 mm. The new fill must be keyed into the existing embankment as per OPSD 208.01.

Provided that the embankment widening material consists of granular earth fill, the settlement of the embankment fill itself is expected to be less than 25 mm, and the settlement is expected to occur during construction. If cohesive earth fill (i.e. fill containing more than 20% passing the No. 200 sieve) is used for embankment widening, the majority of settlement would occur after construction.

In summary, about 25 mm of settlement is expected to occur during embankment construction operation if granular earth fill is used for the embankment widening. This settlement will be differential with respect to the existing embankment.

5.6.3 Subgrade Preparation and Embankment Construction

All topsoil, organic matter and softened / loosened soils should be removed from below the widened approach embankment areas and wasted / re-used as landscaping fill. The new fill should be keyed into the existing fill by benching into the existing embankment side slopes or cut slopes in accordance with OPSD 208.01.

All subgrade soils should be proof-rolled prior to fill placement and embankment fill should be placed in accordance with SP206S03 (dated July 2006). The final lift prior to placement of the granular subbase and base courses should be compacted to not less than 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.

5.7 Liquefaction Potential and Seismic Analysis

5.7.1 Analysis Methods

The liquefaction potential of granular soils under seismic loading is assessed using the empirical method outlined in Section C.4.6.2 of the *CHBDC Commentary(2001)* based on papers by Seed and Idriss (1971) and Seed et al. (1984), which compares the cyclic resistance ratio (CRR) of the soils to the cyclic stress ratio (CSR) caused by an earthquake. The CRR is determined based on correlations with the normalized penetration resistance and fines content of soil together with the characteristic earthquake magnitude for liquefaction assessment (that is indirectly related to the number of significant stress cycles or duration of strong shaking). The CRR is corrected for earthquake magnitude and overburden stress effects. The CSR at a given depth is related to the peak ground acceleration, the ratio of the total to effective overburden stress at that depth, and soil flexibility. A factor of 0.65 is used to convert the maximum CSR to an equivalent CSR of uniform cycles (Section C4.6.2 of the *CHBDC Commentary*).

In general, geologically young, loose, saturated deposits of sand, silty sands, and non-plastic silt are susceptible to liquefaction.

5.7.1.1 Liquefaction-Induced Settlements and Lateral Movements

Where liquefaction is identified to be a problem using the methods described above, vertical settlements of the soil under the earthquake loading may occur due to the contraction of the sand deposit. The anticipated post-earthquake settlements are estimated using a relationship developed by Tokimatsu and Seed (1987) where the anticipated post-earthquake volume change is related to the SPT 'N' and CSR.

The lateral movements can be estimated using relationships proposed by Makdisi and Seed (1978). If unacceptable lateral movements are anticipated, soil improvement methods should be considered and could include densification, removal and re-compaction, grouting, or permanent drainage so that the pore water pressure rise necessary to trigger liquefaction is controlled.

5.7.1.2 Embankment Stability under Seismic Conditions

If liquefaction of the subsoils under an embankment loading is not anticipated, the stability of the embankment slope may be assessed using conventional pseudo-static methods of slope stability analysis under earthquake-induced peak ground acceleration. A calculated factor of safety of 1.0 is considered appropriate; however, a factor of safety less than 1.0 does not indicate full-scale failure of the embankment slope due to the application of the peak ground acceleration in one direction for a short period of time. In this case, other methods, such as the Newmark sliding block method may be used to assess the magnitude of the ground movement.

Where liquefaction is triggered in the underlying soil deposit, the stability of the embankment is analyzed using post-liquefaction, residual shear strength parameters in the liquefied layers using the correlation proposed by Seed and Harder (1990) which is correlated to SPT 'N' values. If, under these conditions, the embankment is estimated to have a factor of safety less than 1.0 under static conditions (i.e. without inertia effects), the embankment is considered to be susceptible to a flow slide. Flow slides are characterized by very large lateral and vertical displacements of the embankment. If under residual strength conditions, the static factor of safety is greater than 1.0, lateral displacements may still occur, and these are estimated using the Newmark method, which relates the horizontal acceleration necessary to induce a factor of safety equal to 1.0 in the embankment (i.e. yield acceleration) to the anticipated displacements. If the yield acceleration is greater than the maximum acceleration for this site, then no remedial measures are required. If the yield acceleration is less than the maximum acceleration and the computed movements are unacceptable, soil improvement methods may be necessary to improve soil conditions.

5.7.2 Results of Analysis

The liquefaction susceptibility of the soil deposits underlying the proposed roadway embankments and the consequent stability of the embankment under seismic loading conditions for the QEW/Fourth Line Underpass site has been assessed. The peak zonal acceleration used for this site (Oakville) is 0.065g, which is based on a zonal acceleration of 0.05 g multiplied by an amplification factor of 1.3 for the types of soils found at the site. Typically, for free-draining soils, the seismic loading is applied to the long-term (drained) conditions.

Using the methods outlined in Section 5.6.1, the soils at this site have a very low risk of liquefaction. This assessment corresponds to a characteristic earthquake of magnitude 7 representing approximately 10 to 15 effective cycles of loading and has been established based on historical earthquake data and de-aggregation of seismic risk carried out for other projects in the general region, and taking into consideration that smaller magnitude events (i.e. $\leq M5$) do not contribute to liquefaction damage.

A factor of safety greater than 1.0 against embankment instability under seismic conditions is obtained. The results of the embankment slope stability under an earthquake-induced peak ground acceleration equal to 0.065 g using the commercially available program GeoStudio 2004 (Version 6.20), produced by Geo-Slope International Ltd. are shown on Figure 4.

5.8 Design and Construction Considerations

The following sections are intended to highlight items or operations that may pose construction difficulties, such as excavations and temporary cut slopes, bedrock excavation, groundwater and surface water control and obstructions.

5.8.1 Excavations and Temporary Cut Slopes

It is anticipated that the bulk of the excavations at the site for pile/caisson cap or spread footing construction 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 and good construction practice. The existing fills are typically classified as Type 3 soil, according to the OHSA handbook. The very stiff to hard clayey silt/silty clay residual soil and the weathered shale bedrock are considered as Type 1 or 2 soils according to OHSA. Temporary excavations (i.e. those which are only open for a relatively short period of time) through the overburden soils (native and fill) should be made with side slopes no greater than 1.5H:1V above the groundwater level and no greater than 2H:1V below the groundwater level. Excavations through the weathered bedrock can be made using vertical slopes.

The base of the excavations, in the residual soil or on the bedrock, should be covered with a thin mud coat consisting of lean mix concrete (5 MPa strength) and placed within 4 hours of reaching and preparing the excavation base. This will protect the founding soil/rock from exposure to the elements, ponded water and from construction traffic, as the residual soil and shale are particularly susceptible to weathering upon exposure. An NSSP should be included in the contract to address this issue (under the mass concrete item); a sample NSSP has been included in Appendix A.

Temporary shoring will be required for the median pier construction and possibly for the abutment footings, depending on which foundation alternative is chosen. Consideration could be given to the use of a braced soldier pile and lagging system for support. It may be necessary to socket the soldier piles into the bedrock to provide sufficient lateral resistance at the pile toe to allow the excavation to be advanced. The temporary excavation support system should be designed and constructed in accordance with Special Provision SP105S19. The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in SP105S19.

5.8.2 Groundwater and Surface Water Control

The groundwater level (measured in the piezometers) at this site is between about Elevation 107.8 m and 108.5 m. At the abutments and central pier, it is anticipated that the groundwater inflow to excavations made for pile cap or spread footing construction can be adequately controlled by sumping from properly filtered sumps at the base of the excavations. All surface water should be directed away from the excavations at all times and, especially, highway drainage should be prevented from entering the excavations.

5.8.3 Obstructions

Shale fragments were noted during drilling through the residual soil deposits at this site. It is probable that less weathered limestone/siltstone layers or slabs may also be encountered within the residual soil..

Difficulty will be experienced augering and/or driving piles, as well as making open cut excavations, through the hard limestone/siltstone layers within the shale bedrock. At this site, the limestone/siltstone layers within the shale were measured in the cores to be up to 250 mm in thickness and have a strength of greater than 80 MPa. An NSSP should be included in the contract to alert the Contractor to have appropriate equipment on site to advance through these layers; a sample NSSP has been included in Appendix A.


5.9 Closure

This report was prepared by Ms. Sarah Poot, P.Eng., a geotechnical engineer with Golder. The technical aspects were reviewed by Ms. Anne Poschmann, P.Eng., a Senior Principal with Golder; Mr. Jorge Costa, P.Eng., Principal with Golder and the Designated MTO Contact, conducted a quality control review of the report.

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TABLE 1
BEDROCK SURFACE ELEVATION
QEW/FOURTH LINE UNDERPASS
G.W.P. 189-00-00, SITE NO. 10-159
OAKVILLE, ONTARIO

Borehole	Borehole	Ground Surface Elevation (m)	Depth to Bedrock Surface(m)	Bedrock Surface Elevation (m)
South Approach	F1	110.0	1.2	108.8
South Abutment	F4	116.9	7.9	109.0
	F5	117.0	9.1	107.9
	F6	110.1	1.4	108.7
North Abutment	F7	110.1	0.6	109.5
	F8	116.9	7.6	109.3
	F9	117.2	8.7	108.5
	F10	110.5	0.9	109.6
North Approach	F2	110.6	1.2	109.4

Prepared By: SEP
Checked By: JMAC

TABLE 2
POINT LOAD STRENGTH TEST RESULTS
QEW/FOURTH LINE UNDERPASS
G.W.P. 189-00-00, SITE NO. 10-159
OAKVILLE, ONTARIO

Borehole Number	Sample Depth (ft)	Rock Type	Test Type	Core ⁽²⁾ Diameter (mm)	Ram Pressure (kPa)	Load (P) (kN)	Is Axial (MPa)	Is Diametral (MPa)	Is (50mm) (MPa)	Approx. ⁽¹⁾ UCS (MPa)
F4	11.0	Siltstone-Limestone	D	46	5816.3	7.9		3.725	3.587	83
F4	11.3	Queenston Shale	A	33	4108.3	5.6	2.819		2.673	61
F4	11.3	Queenston Shale	A	38	4005.6	5.4	2.387		2.336	54
F4	11.3	Queenston Shale	D	47	342.1	0.5		0.210	0.204	5
F4	11.9	Queenston Shale	A	36	3286.7	4.5	2.024		1.967	45
F4	11.9	Queenston Shale	D	48	342.1	0.5		0.201	0.198	5
F5	10.2	Queenston Shale	A	42	1780.3	2.4	0.960		0.961	22
F5	10.2	Queenston Shale	D	47	342.1	0.5		0.210	0.204	5
F5	10.9	Queenston Shale	D	47	684.3	0.9		0.420	0.408	9
F8	11.4	Queenston Shale	D	47	855.3	1.2		0.525	0.510	12
F8	12.1	Queenston Shale	A	42	3355.1	4.5	1.809		1.811	42
F8	12.1	Queenston Shale	D	47	1094.8	1.5		0.672	0.653	15
F9	10.0	Queenston Shale	D	46	342.1	0.5		0.219	0.211	5
F9	10.6	Queenston Shale	A	46	2396.5	3.2	1.205		1.226	28
F9	10.6	Queenston Shale	D	36	1026.4	1.4		1.073	0.926	21

⁽¹⁾ $I_{S50} \times 23$ (actual value will have to be confirmed by UCS testing), from ISRM ("Suggested Methods for Determining Point Load Strength", International Society for Rock Mechanics Commission on Testing Methods, Int. J. Rock. Mech. Min. Sci. and Geomechanical Abstr., Vol 22, No. 2 1985, pp. 51-60.

⁽²⁾ Actual distance between point load cones at time of failure.

Compiled By: SEP
Reviewed By: JMAC

TABLE 3
GROUNDWATER ELEVATION
QEW/FOURTH LINE UNDERPASS
G.W.P. 189-00-00, SITE NO. 10-159
OAKVILLE, ONTARIO

Borehole	Borehole	Ground Surface Elevation (m)	Depth to Groundwater (m)	Groundwater Elevation (m)
South Approach	F1	110.0	2.2	107.8 ¹
South Abutment	F4	116.9	--	Dry ²
	F5	117.0	--	Dry ²
	F6	110.1	3.5	106.6 ²
North Abutment	F7	110.1	1.7	108.4 ²
	F8	116.9	--	Dry
	F9	117.2	--	Dry
	F10	110.5	2.1	108.4 ¹
North Approach	F2	110.6	2.1	108.5 ¹

- Notes 1. Measured in piezometer on June 10, 2002.
2. Measured in the open borehole upon completion of soil drilling and prior to rock coring.

Prepared By: SEP
 Checked By: JMAC

TABLE 4
EVALUATION OF FOUNDATION ALTERNATIVES
QEW/FOURTH LINE UNDERPASS
G.W.P. 189-00-00, SITE NO. 10-159
OAKVILLE, ONTARIO

Footing Option	Advantages	Disadvantages	Relative/Estimated Costs (See Note 2)	Risks/Consequences
Spread Footings on Fill (Perched Abutments)	<ul style="list-style-type: none"> Excavation to bedrock surface not required. Groundwater control not required. 	<ul style="list-style-type: none"> Excavation of existing fill required to provide uniform material under footing. 	<ul style="list-style-type: none"> Lower cost than spread footings on shale due to decreased excavation depth. ~\$60,000 	<ul style="list-style-type: none"> Some risk of additional settlement below widened footings on different fill types.
Spread Footings on Shale Bedrock	<ul style="list-style-type: none"> Increased resistance over spread footings on fill. Minimal settlement of footings on rock. Groundwater control likely not required. 	<ul style="list-style-type: none"> Excavation required up to 3 m below road grade. Space may not permit open cut and temporary shoring may be required adjacent to highway excavations. 	<ul style="list-style-type: none"> Increased cost of ground-water control and temporary shoring (if required). ~\$75,000 	<ul style="list-style-type: none"> Low risk.
Piles socketted into Shale Bedrock	<ul style="list-style-type: none"> Minor excavation required for pile cap construction. Can be used for integral abutments. 	<ul style="list-style-type: none"> Minimum pile length may not be achievable with driving alone– augering/socketting into shale will be required. 	<ul style="list-style-type: none"> Increased costs of augering/socketting into shale to get minimum pile length. ~\$50,000 	<ul style="list-style-type: none"> Augering/socketting of pile into bedrock may be difficult due to hard layers within the shale.
Caissons in Shale Bedrock	<ul style="list-style-type: none"> Relatively straightforward construction for central pier in highway median. Minor excavation required for caisson cap construction. Relatively fewer caissons required as compared to steel H-piles. 	<ul style="list-style-type: none"> Temporary shoring may be required in median for caisson cap construction. Temporary liners may be required. Cannot be used in an integral abutment system. 	<ul style="list-style-type: none"> Typically more expensive than driven steel H-piles since more equipment and materials required. ~\$50,000 	<ul style="list-style-type: none"> Socketting of caisson into bedrock may be difficult due to hard layers within the shale.

Notes:

1. To be read in conjunction with accompanying report.
2. Cost for excavating through existing embankment not included as similar for all options. Cost for the central pier not included.

Prepared By: SEP
Checked By: JMAC

TABLE 5
DESIGN PILE TIP ELEVATION
QEW/FOURTH LINE UNDERPASS
G.W.P. 189-00-00, SITE NO. 10-159
OAKVILLE, ONTARIO

Foundation Unit	Elevation of Bedrock Surface (m)	Design Pile Tip Elevation for Piles Driven within a Pre- drilled Hole (m)	Design Pile Tip Elevation for Socketted Piles (m)
South Abutment	107.9 to 109.0	107.0	106.0
North Abutment	108.5 to 109.3	107.5	106.5

Note: No borehole drilled at central pier in accordance with Golder's proposal P01-1104 dated March 2000. Bedrock surface elevations are based on pile tip elevations interpolated between those at the south and north abutments.

Prepared By: SEP
Checked By: JMAC

TABLE 6
HORIZONTAL SUBGRADE REACTION PARAMETERS
QEW/FOURTH LINE UNDERPASS
G.W.P. 189-00-00, SITE NO. 10-159
OAKVILLE, ONTARIO

Foundation Element	Relevant Boreholes	Soil Unit	Elevation (m)	n_h (MPa/m)	s_u (kPa)
South Abutment	F4, F5	CSP Backfill	Where applicable	4.4	--
		Existing Clayey Silt/Silty Clay Fill	Above Elev. 109.0 m	--	40
		Clayey Silt Residual Soil	Elev. 109.0 m to 108.0 m	--	100
		Weathered Shale Bedrock	Elev. 108.0 m to 107.0 m	--	100
		Shale Bedrock	Below Elev. 107.0 m	--	150
North Abutment	F8, F9	CSP Backfill	Where applicable	4.4	--
		Existing Clayey Silt/Silty Clay Fill	Above Elev. 109.5 m	--	40
		Silty Clay Residual Soil	Elev. 109.5 m to 108.5 m	--	100
		Weathered Shale Bedrock	Elev. 108.5 m to 107.5 m	--	100
		Shale Bedrock	Below Elev. 107.5 m	--	150

Prepared By: SEP
Checked By: JMAC

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, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezcone Penetration Test (CPT)

An electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm² 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	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.

LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. stress: $\Delta\sigma$
ϵ	linear strain
ϵ_v	volumetric strain
η	coefficient of viscosity
ν	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

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_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 $= \tan \delta$
c'	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 + \sigma_3)/2$ or $(\sigma'_1 + \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 + \sigma_3)$
S_t	sensitivity

Notes: 1 $\tau = c' + \sigma' \tan \phi'$
2 Shear strength = (Compressive strength)/2

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 To	
⊥ - Perpendicular To	

PROJECT		111-1128		RECORD OF BOREHOLE No F1		1 OF 1 METRIC															
W.P.		189-00-00		LOCATION		N 4810964.0 ; E 287930.0															
DIST		4		HWY		QEW															
BOREHOLE TYPE		CME 75, 100mm O.D. Solid Stem Auger		COMPILED BY		SEP															
DATUM		Geodetic		DATE		November 29, 2001															
CHECKED BY		ASP																			
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20	40	60	80	100	W _p	W	W _L	γ	GR	SA	SI	CL
110.0		GROUND SURFACE																			
109.2	0.8	TOPSOIL Silty Clay (FILL) Brown																			
108.8	1.2	CLAYEY SILT with red shale fragments (RESIDUAL SOIL) Hard Red-brown and grey Moist Weathered, red-brown and grey SHALE BEDROCK (Queenston Formation) with occasional grey limestone/siltstone layers Becoming wet at 3.0 m depth. Augers grinding at 1.7 m and 2.3 m for 25 mm.		1	SS	44		109													
				2	SS	50/.15		108													
				3	SS	50/.08		107													
106.2	3.8	Augers grinding from 3.4 m to 3.8 m depth. END OF BOREHOLE Auger Refusal Notes: 1. Open borehole dry upon completion of drilling. 2. Water level in piezometer at 2.3 m depth (Elev. 107.7 m) on January 8, 2002 and at 2.2m depth (Elev. 107.8 m) on June 10, 2002.																			

PROJECT		111-1128		RECORD OF BOREHOLE No F2		1 OF 1 METRIC														
W.P.		189-00-00		LOCATION		N 4811011.0; E 287833.0														
DIST		4		HWY		QEW														
BOREHOLE TYPE		CME 75, 100mm O.D. Solid Stem Auger		COMPILED BY		SEP														
DATUM		Geodetic		DATE		November 29, 2001														
CHECKED BY		ASP																		
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa			WATER CONTENT (%)			γ			GR SA SI CL		
110.6	0.0	GROUND SURFACE							20 40 60 80 100	20 40 60 80 100	10 20 30									
109.8	1.2	TOPSOIL Silty Clay (FILL) Brown Moist						110												
109.4		CLAYEY SILT with red-brown and grey shale fragments (RESIDUAL SOIL)		1	SS	44		109												
		Hard Red-brown Moist		2	SS	50/10		108												
		Weathered, red-brown and grey SHALE BEDROCK (Queenston Formation) with occasional grey limestone/siltstone layers		3	SS	98/23		107												
105.9	4.7	END OF BOREHOLE		4	SS	50/10		106												
Notes:																				
1. Open borehole dry upon completion of drilling.																				
2. Water level in piezometer at 2.3 m depth (Elev. 108.3 m) on January 8, 2002 and at 2.1 m depth (Elev. 108.5 m) on June 10, 2002.																				

PROJECT 011-1128			RECORD OF BOREHOLE No F4			1 OF 1 METRIC											
W.P. 189-00-00			LOCATION N 4810966.0 ; E 287912.0			ORIGINATED BY SB											
DIST 4 HWY QEW			BOREHOLE TYPE CME 75, 100mm I.D. Hollow Stem Auger			COMPILED BY SEP											
DATUM Geodetic			DATE November 28, 2001			CHECKED BY ASP											
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					WATER CONTENT (%) W _p W W _L			γ	GR SA SI CL
116.9	GROUND SURFACE							20 40 60 80 100									
0.0	ASPHALT																
	Sand and gravel (FILL)																
	Brown																
	Moist																
116.1																	
0.8	Clayey Silt to Silty Clay with sand and gravel, occasional shale fragments (FILL)		1	SS	12		116										
	Firm to stiff																
	Brown and grey		2	SS	7		115										
	Moist																
			3	SS	7		114										
			4	SS	4		113										
			5	SS	6		112										
			6	SS	8		111										
110.8	Hard augering from 5.8 m depth.																
6.1	Silty Clay, trace sand and gravel and occasional shale fragments (FILL)		7	SS	9		110										
	Stiff																
	Red brown																
	Moist																
109.0																	
7.9	Moderately to highly weathered, red-brown SHALE BEDROCK (Queenston Formation), with occasional grey limestone/siltstone beds.		8	SS	24		109										
	Hard augering below 8.5 m depth.																
	Bedrock cored from 9.4 m to 13.6 m depth.		9	SS	50/15		108										
	For bedrock coring details see Record of Drillhole F4.																
							107										
							106										
							105										
							104										
103.3	END OF BOREHOLE																
13.6	Notes: 1. Open borehole dry upon completion of augering and prior to rock coring.																

PROJECT: 011-1128

RECORD OF DRILLHOLE: F4

SHEET 2 OF 2

LOCATION: N 4810966.0 ;E 287912.0

DRILLING DATE: November 28, 2001

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 75

DRILLING CONTRACTOR: Geo-Environmental Drilling Ltd.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV.	RUN No.	PENETRATION RATE (m/min)	COLOUR % RETURN	FLUSH	JN - Joint FLT - Fault SHR - Shear VN - Vein CJ - Conjugate	BD - Bedding FO - Foliation CO - Contact OR - Orthogonal CL - Cleavage	PL - Planar CU - Curved UN - Undulating ST - Stepped IR - Irregular	PO - Polished K - Slickensided SM - Smooth Ro - Rough MB - Mechanical Break	BR - Broken Rock NOTE: For additional abbreviations refer to list of abbreviations & symbols.	NOTES WATER LEVELS INSTRUMENTATION												
				DEPTH										RECOVERY			R.Q.D. %	FRACT INDEX PER 0.3 m	DISCONTINUITY DATA			HYDRAULIC CONDUCTIVITY			Diameter Point Load Index (MPa)	RMC -Q AVG.
				(m)										TOTAL CORE %	SOLID CORE %	B Angle			DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	K, cm/sec	10	10	10		
		Refer to Previous Page		107.50																						
10	NQ	Slightly to highly weathered, thinly bedded, fine grained, very weak to strong red-brown, calcareous SHALE. (Queenston Formation).		9.40	1																					
		Occasional, strong, grey limestone/shaley limestone beds at the following depths:																								
		Depth (m) Thickness (mm)																								
		9.5 75																								
		10.3 50																								
11		10.4 150																								
		11.5 75																								
		11.7 175																								
		12.4 50																								
		12.6 25																								
		12.9 50																								
		13.2 100																								
12					2																					
13					3																					
14		END OF BOREHOLE		103.30 13.60																						
15																										
16																										
17																										
18																										
19																										

DEPTH SCALE

1 : 50



LOGGED: SB

CHECKED: SEP

MIS-RCK 002 1128ROCK.GPJ GAL-MISS.GDT 4/25/07 MMZ

[illegible]

PROJECT: 011-1128

RECORD OF DRILLHOLE: F5

SHEET 2 OF 2

LOCATION: N 4810965.0 ;E 287905.0

DRILLING DATE: December 5, 2001

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 75

DRILLING CONTRACTOR: Geo-Environmental Drilling Ltd.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH	RECOVERY				FRACT INDEX PER 0.3 m	DISCONTINUITY DATA				HYDRAULIC CONDUCTIVITY			Diametral Point Load Index (MPa)	RMC -Q AVG.	NOTES WATER LEVELS INSTRUMENTATION				
								JN - Joint FLT - Fault SHR - Shear VN - Vein CJ - Conjugate	BD- Bedding FO- Foliation CO- Contact OR- Orthogonal CL - Cleavage	PL - Planar CU - Curved UN - Undulating ST - Stepped IR - Irregular	PO - Polished K - Slickensided SM - Smooth Ro - Rough MB - Mechanical Break		BR - Broken Rock	NOTE: For additional abbreviations refer to list of abbreviations & symbols.	TOTAL CORE %	SOLID CORE %	R.Q.D. %	B Angle	DIP w.r.t. CORE AXIS				TYPE AND SURFACE DESCRIPTION	K, cm/sec	10 10 10 10	10 10 10 10
		Refer to Previous Page		107.60																						
10	NQ	Slightly to highly weathered, thinly bedded fine grained, very weak to strong, red-brown, calcareous SHALE (Queenston Formation).		9.40	1																					
		Occasional strong, grey, limestone/siltstone beds at the following depth:																								
		Depth (m) Thickness (mm)																								
		9.6 65																								
		10.3 50																								
		10.5 75																								
		10.6 25																								
		10.7 25																								
		11.2 40																								
		11.4 75																								
	11.9 25																									
	12.2 40																									
11					2																					
12																										

DEPTH SCALE

1 : 50



LOGGED: PKS

CHECKED: SEP

MIS-RCK 002 1128ROCK.GPJ GAL-MISS.GDT 4/25/07 MMZ

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

PROJECT <u>011-1128</u>		RECORD OF BOREHOLE No F7				1 OF 1 METRIC										
W.P. <u>189-00-00</u>		LOCATION <u>N 4810992.0; E 287842.0</u>				ORIGINATED BY <u>SB</u>										
DIST <u>4</u> HWY <u>QEW</u>		BOREHOLE TYPE <u>CME 75, 100mm O.D. Solid Stem Auger</u>				COMPILED BY <u>SEP</u>										
DATUM <u>Geodetic</u>		DATE <u>November 29, 2001</u>				CHECKED BY <u>ASP</u>										
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
110.1	GROUND SURFACE															
8.0	TOPSOIL															
109.5	Silty clay (FILL) Brown															
0.6	Weathered, red-brown and grey SHALE BEDROCK (Queenston Formation) with occasional limestone siltstone layers		1	SS	86/25											
			2	SS	50/13											
	Augers grinding at depths of 1.2 m and 2.3 m for 25 mm.															
			3	SS	50/08											
	Augers grinding at depths of 3.6 m and 4.0 m for 50 mm.															
105.5			4	SS	80/06											
4.6	END OF BOREHOLE															
	Notes: 1. Water level in open borehole at a depth of 1.7 m (Elev.108.4 m) upon completion of drilling.															

PROJECT 011-1128			RECORD OF BOREHOLE No F8			1 OF 2 METRIC		
W.P. 189-00-00			LOCATION N 4811019.0; E 287858.0			ORIGINATED BY PKS		
DIST 4 HWY QEW			BOREHOLE TYPE CME 75, 100mm I.D. Hollow Stem Auger			COMPILED BY SEP		
DATUM Geodetic			DATE November 30, 2001			CHECKED BY ASP		
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	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 (%)
116.9	GROUND SURFACE							
0.0	ASPHALT							
0.2	Sand and Gravel (FILL)							
116.1								
0.8	Clayey Silt to Silty Clay, some sand and gravel, occasional shale fragments (FILL) Firm to stiff Red-brown Moist		1	SS	8		116	
			2	SS	5		115	
			3	SS	5		114	
			4	SS	6		113	
			5	SS	9		112	
112.0			6	SS	20		111	
4.9	Sand and gravel, trace silt, trace asphalt fragments (FILL) Dense to very dense Brown Moist		7	SS	84/18		110	
109.3							109	
7.6	Moderately to highly weathered, red-brown SHALE BEDROCK (Queenston Formation), with occasional grey limestone/siltstone beds. Bedrock cored from 9.6 m to 14.2 m depth. For bedrock coring details see Record of Drillhole F8.		8	SS	53		108	
			9	SS	75/10		107	
							106	
							105	
							104	
							103	
102.7								
14.2								

Continued Next Page

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

MIS-MTO 001 011-1128.GPJ GAL-MISS.GDT 4/25/07

PROJECT <u>011-1128</u>		RECORD OF BOREHOLE No F8				2 OF 2 METRIC										
W.P. <u>189-00-00</u>		LOCATION <u>N 4811019.0; E 287858.0</u>				ORIGINATED BY <u>PKS</u>										
DIST <u>4</u> HWY <u>QEW</u>		BOREHOLE TYPE <u>CME 75, 100mm I.D. Hollow Stem Auger</u>				COMPILED BY <u>SEP</u>										
DATUM <u>Geodetic</u>		DATE <u>November 30, 2001</u>				CHECKED BY <u>ASP</u>										
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa								
	--- CONTINUED FROM PREVIOUS PAGE ---															
	END OF BOREHOLE Notes: 1. Open borehole dry upon completion of augering and prior to rock coring.															

MIS-MTO 001 011-1128.GPJ GAL-MISS.GDT 4/25/07

SHEET 3 OF 3

DATUM: Geodetic

DRILLING CONTRACTOR: Geo-Environmental Drilling Ltd.

CHECKED: SEP

MIS-RCK 002 1128ROCK.GPJ GAL-MISS.GDT 4/25/07 MMZ

PROJECT 011-1128		RECORD OF BOREHOLE No F9				1 OF 1 METRIC								
W.P. 189-00-00		LOCATION N 4811011.0; E 287859.0				ORIGINATED BY PKS								
DIST 4 HWY QEW		BOREHOLE TYPE CME 75, 100mm O.D. Hollow Stem Auger				COMPILED BY SEP								
DATUM Geodetic		DATE December 3, 2001				CHECKED BY ASP								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
117.2	GROUND SURFACE							20 40 60 80 100	20 40 60 80 100	10 20 30				
0.0	ASPHALT													
0.2	Silty Sand and gravel, pockets of silty clay (FILL) Compact Brown Moist		1	SS	25									
			2	SS	25									
			3	SS	12									
114.3	Clayey Silt to Silty Clay, some sand and gravel, occasional shale fragments (FILL) Firm to very stiff Red-brown Moist		4	SS	8									
2.9	Trace rootlets at 3.0 m depth.		5	SS	12									
	Sand and gravel with asphalt pieces from 4.3 m to 4.4 m depth.		6	SS	15									
			7	SS	9									
109.4	SILTY CLAY with red shale fragments (RESIDUAL SOIL) Hard Red-brown Moist		8	SS	48									4 1 57 38
108.5	Moderately to highly weathered, red-brown SHALE BEDROCK (Queenston Formation), with occasional grey limestone/siltstone beds.		9	SS	75/10									
8.7	Bedrock cored from 9.4 m to 12.5 m depth.													
	For bedrock coring details see Record of Drillhole F9.													
104.7	END OF BOREHOLE													
12.5	Notes: 1. Open borehole dry upon completion of augering and prior to rock coring.													

PROJECT: 011-1128

RECORD OF DRILLHOLE: F9

SHEET 2 OF 2

LOCATION: N 4811011.0 ;E 287859.0

DRILLING DATE: December 4, 2001

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 75

DRILLING CONTRACTOR: Geo-Environmental Drilling Ltd.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	PENETRATION RATE (mm/min)	FLUSH	COLOUR % RETURN	JN - Joint FLT - Fault SHR - Shear VN - Vein CJ - Conjugate										BD - Bedding FO - Foliation CO - Contact OR - Orthogonal CL - Cleavage										PL - Planar CU - Curved UN - Undulating ST - Stepped IR - Irregular										PO - Polished K - Slickensided SM - Smooth Ro - Rough MB - Mechanical Break										BR - Broken Rock	NOTE: For additional abbreviations refer to list of abbreviations & symbols.	NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																															
								RECOVERY		R.Q.D. %	FRACT INDEX PER 0.3 m	DISCONTINUITY DATA										HYDRAULIC CONDUCTIVITY		Diametral Point Load Index (MPa)	RMC -Q AVG.																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																								
								TOTAL CORE %	SOLID CORE %			B Angle	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	K, cm/sec																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																		
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DEPTH SCALE

1 : 50

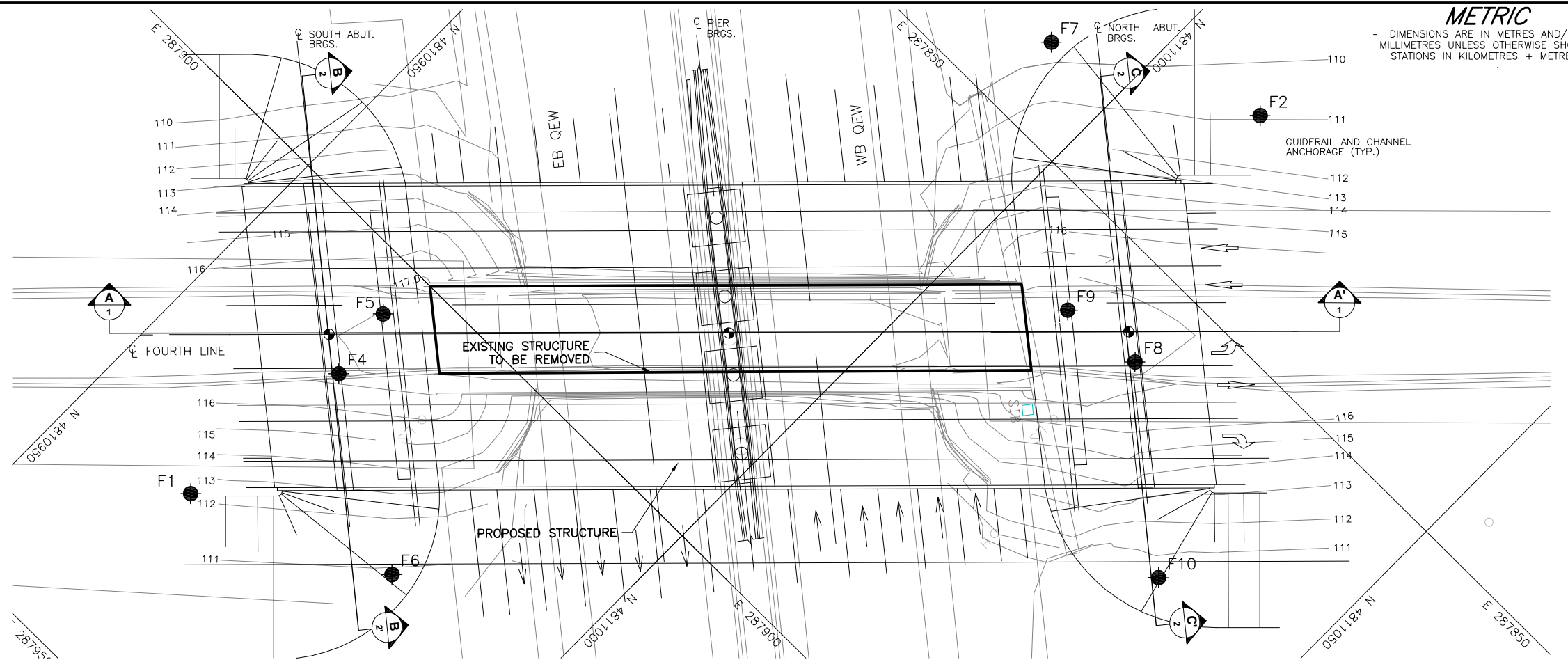


LOGGED: PKS

CHECKED: SEP

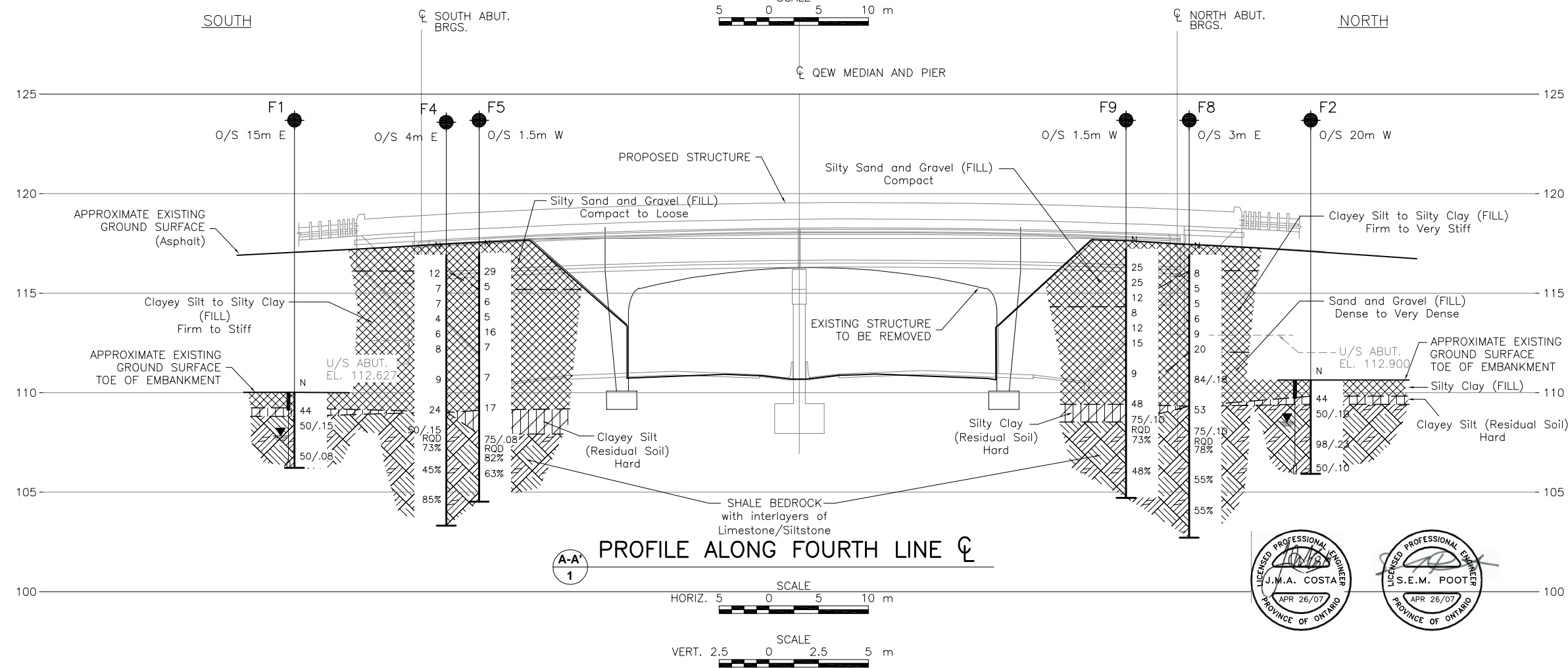
MIS-RCK 002 1128RCK.GPJ GAL-MISS.GDT 4/25/07 MMZ

PROJECT 011-1128		RECORD OF BOREHOLE No F10				1 OF 1 METRIC							
W.P. 189-00-00		LOCATION N 4811035.0; E 287871.0				ORIGINATED BY PKS							
DIST 4 HWY QEW		BOREHOLE TYPE CME 75, 100mm O.D. Solid Stem Auger				COMPILED BY SEP							
DATUM Geodetic		DATE December 6, 2001				CHECKED BY ASP							
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					
110.5	GROUND SURFACE						20 40 60 80 100	20 40 60 80 100	10 20 30				
0.0	Clayey TOPSOIL												
0.2	Clayey Silt, some sand, trace gravel, occasional shale fragments (FILL)												
109.6	Firm Red-brown Moist		1	SS	53								
0.9	Weathered, red-brown SHALE BEDROCK (Queenston Formation) with occasional grey limestone/siltstone layers		2	SS	109								
	Becoming wet below 2.3 m depth.		3	SS	100/15								
107.4	END OF BOREHOLE		4	SS	100/08								
3.1	Notes: 1. Borehole caved at a depth of 2.4 m, base of hole wet upon completion of drilling. 2. Water level in piezometer at a depth of 2.3 m (Elev. 108.2 m) on January 8, 2002 and at a depth of 2.1 m (Elev. 108.4 m) on June 10, 2002.												



PLAN

SCALE
5 0 5 10 m



PROFILE ALONG FOURTH LINE

A-A'
1

SCALE
HORIZ. 5 0 5 10 m

SCALE
VERT. 2.5 0 2.5 5 m

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

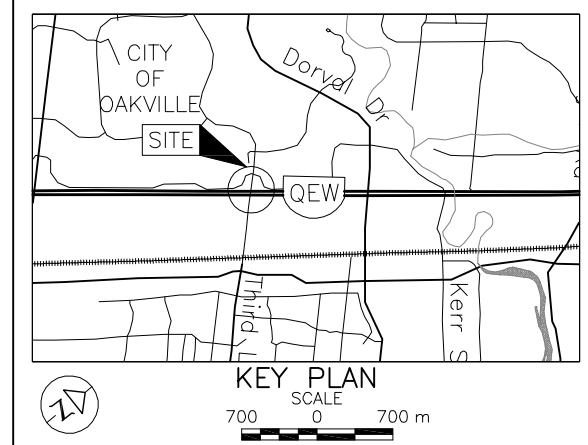
GUIDERAIL AND CHANNEL ANCHORAGE (TYP.)

CONT No.
WP No.189-00-00

FOURTH LINE UNDERPASS
QEW - EBL/WBL
BOREHOLE LOCATIONS AND SOIL STRATA



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN

SCALE
700 0 700 m

LEGEND

- Borehole - Current Investigation
- 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, measured on June 10, 2002

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
F1	110.0	4810964	287930
F2	110.6	4811011	287833
F4	116.9	4810966	287912
F5	117.0	4810965	287905
F6	110.1	4810983	287922
F7	110.1	4810992	287842
F8	116.9	4811019	287858
F9	117.2	4811011	287859
F10	110.5	4811035	287871

NOTES

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

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

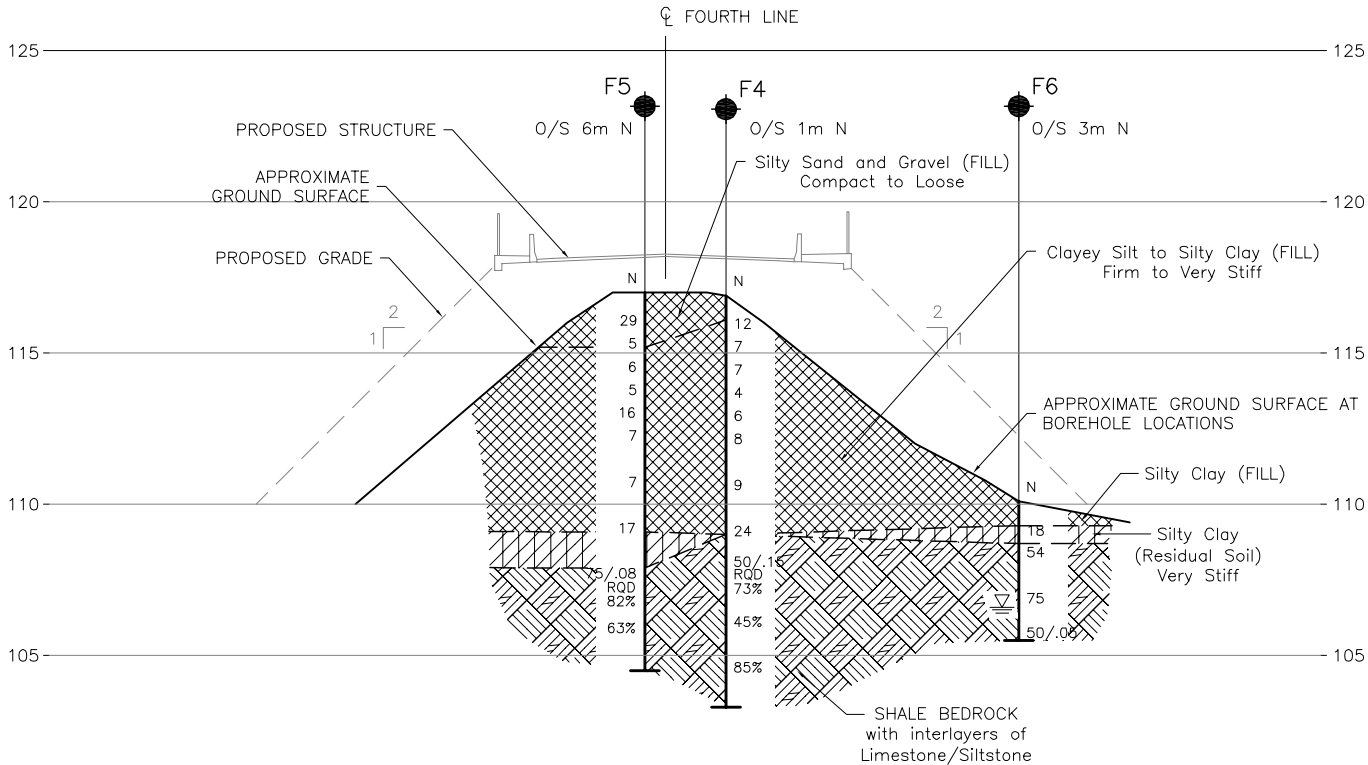
The complete foundation investigation and design report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS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

REFERENCE

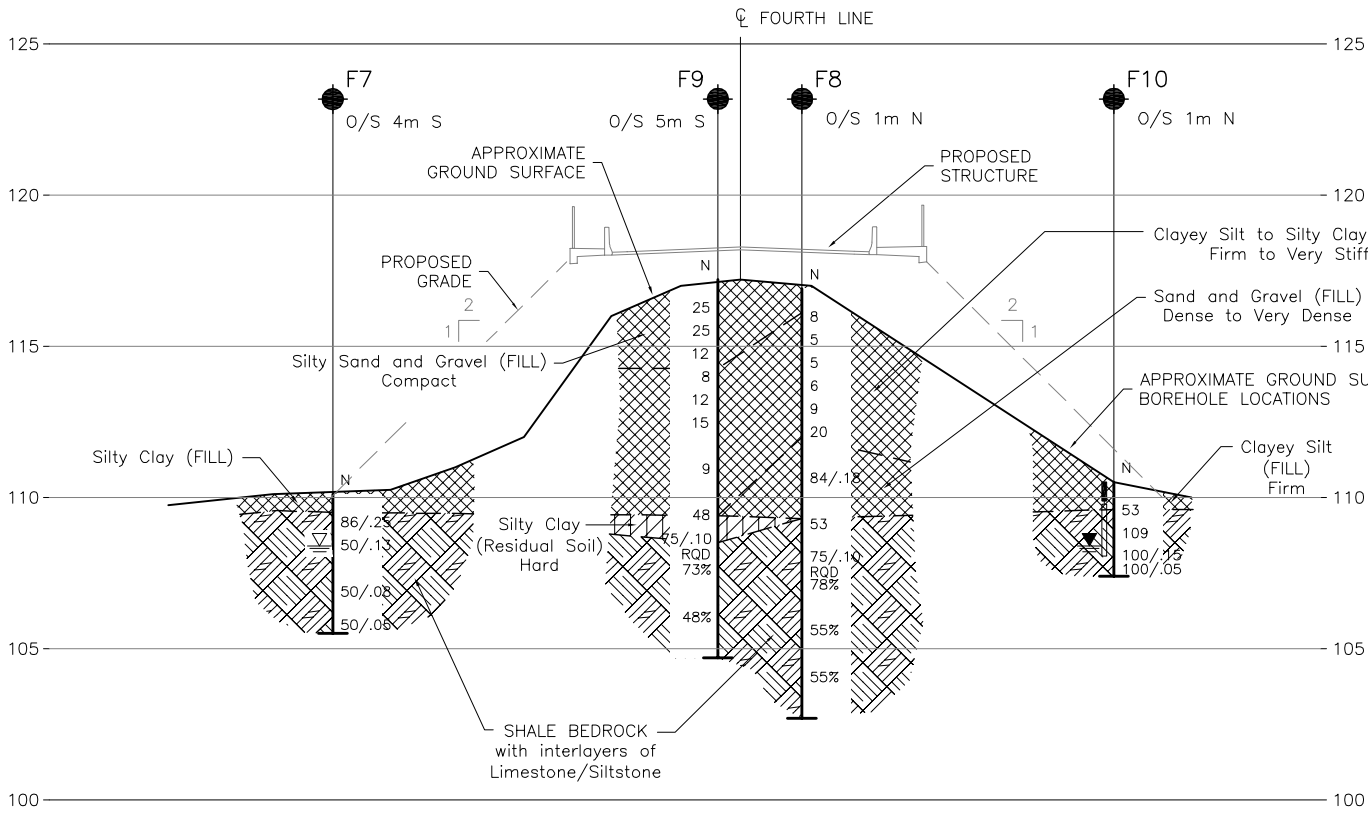
Base plans provided in digital format by URS, drawing file nos. 4thline_ga.dwg, received November 10, 2006 and drawing file no. 00119bgd.dwg, received November 12, 2001.



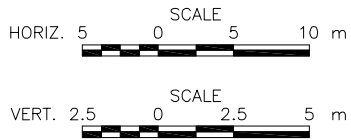
NO.	DATE	BY	REVISION
Geocres No. 30M5-257			
HWY. QEW	PROJECT NO. 011-1128		DIST. 4
SUBM'D. CN	CHKD. CN	DATE: APR 2007	SITE: 10-159
DRAWN: MSM	CHKD. SEP	APPD. JMAC	DWG. 1



SECTION ALONG SOUTH ABUTMENT



SECTION ALONG NORTH ABUTMENT



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

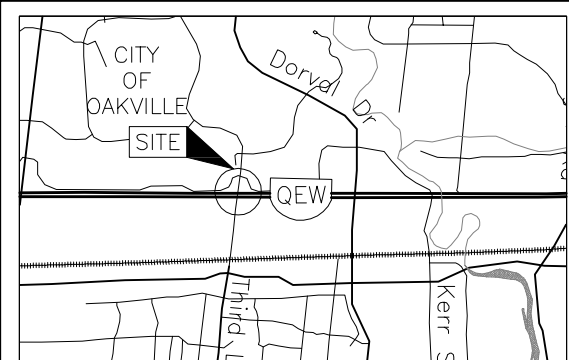
CONT No.
WP No. 189-00-00

FOURTH LINE UNDERPASS
QEW – EBL/WBL
SOIL STRATA

SHEET



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN

SCALE
700 0 700 m

LEGEND

- Borehole – Current Investigation
- 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, measured on June 10, 2002
- WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
F1	110.0	4810964	287930
F2	110.6	4811011	287833
F4	116.9	4810966	287912
F5	117.0	4810965	287905
F6	110.1	4810983	287922
F7	110.1	4810992	287842
F8	116.9	4811019	287858
F9	117.2	4811011	287859
F10	110.5	4811035	287871

NOTES

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

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

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REFERENCE

Base plans provided in digital format by URS, drawing file nos. 4thline_ga.dwg, received November 10, 2006 and drawing file no. 00119bgd.dwg, received November 12, 2001.

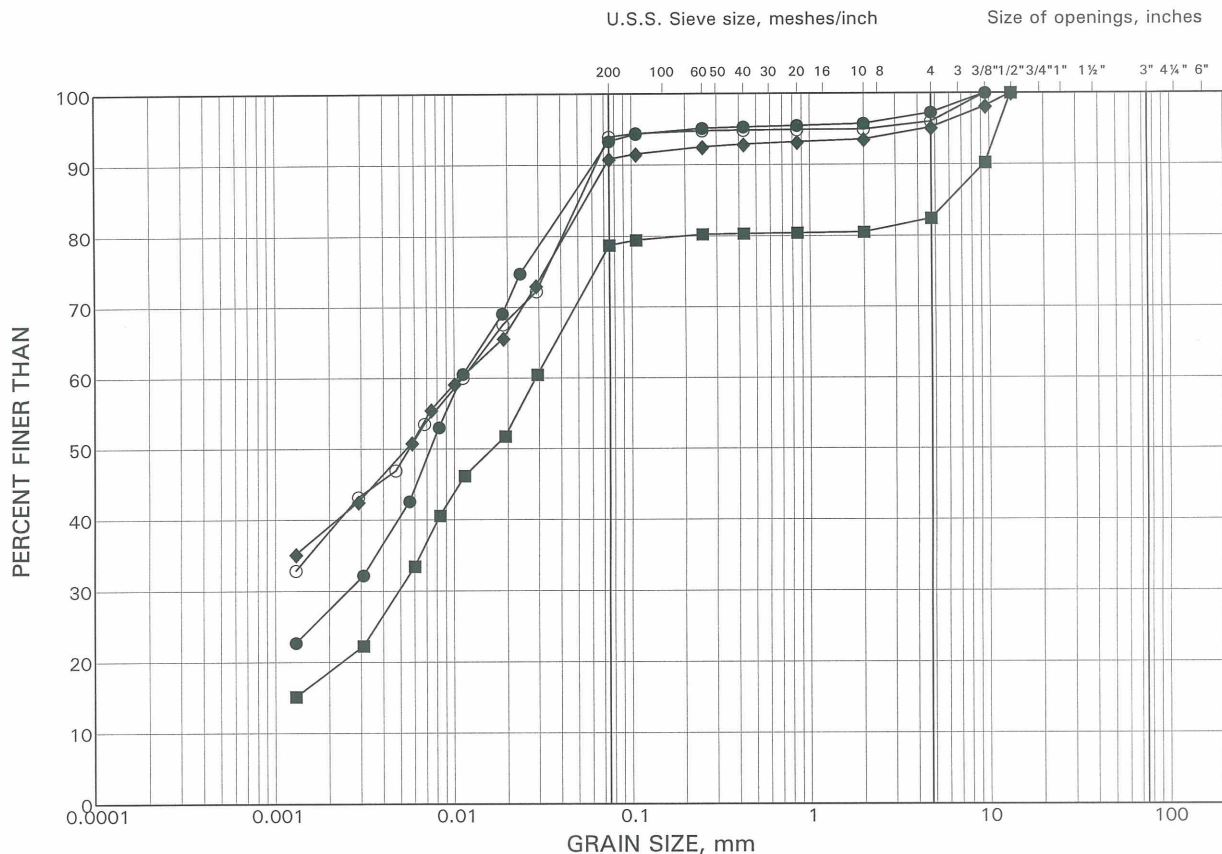


NO.	DATE	BY	REVISION
Geores No. 30M5-257			
HWY.	QEW	PROJECT NO.	011-1128
SUBM'D.	CN	CHKD.	CN
DRAWN:	MSM	CHKD.	SEP
DIST.		4	
SITE:		10-159	
DWG.		2	

GRAIN SIZE DISTRIBUTION

Clayey Silt to Silty Clay (Residual Soil)

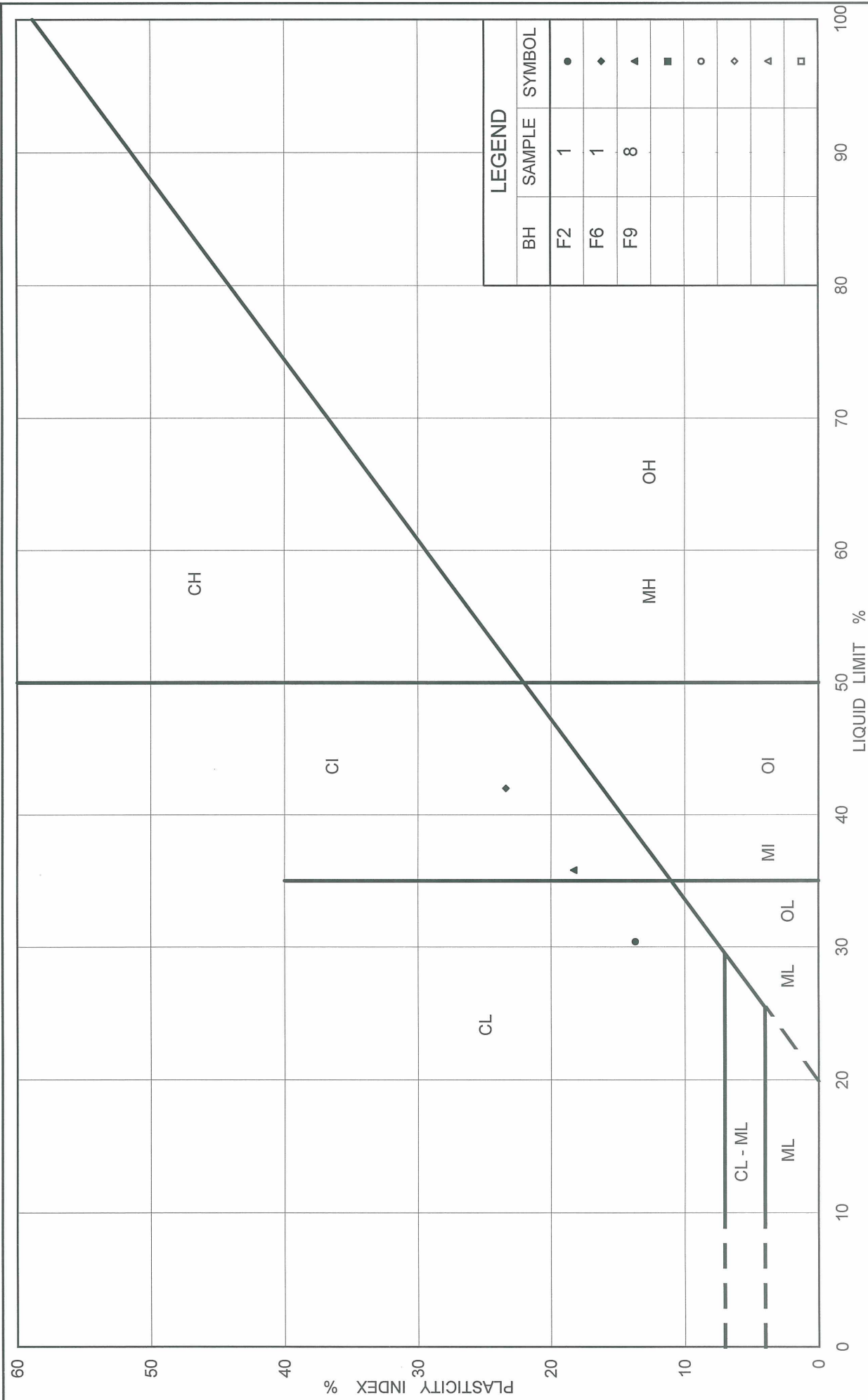
FIGURE 1



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

LEGEND

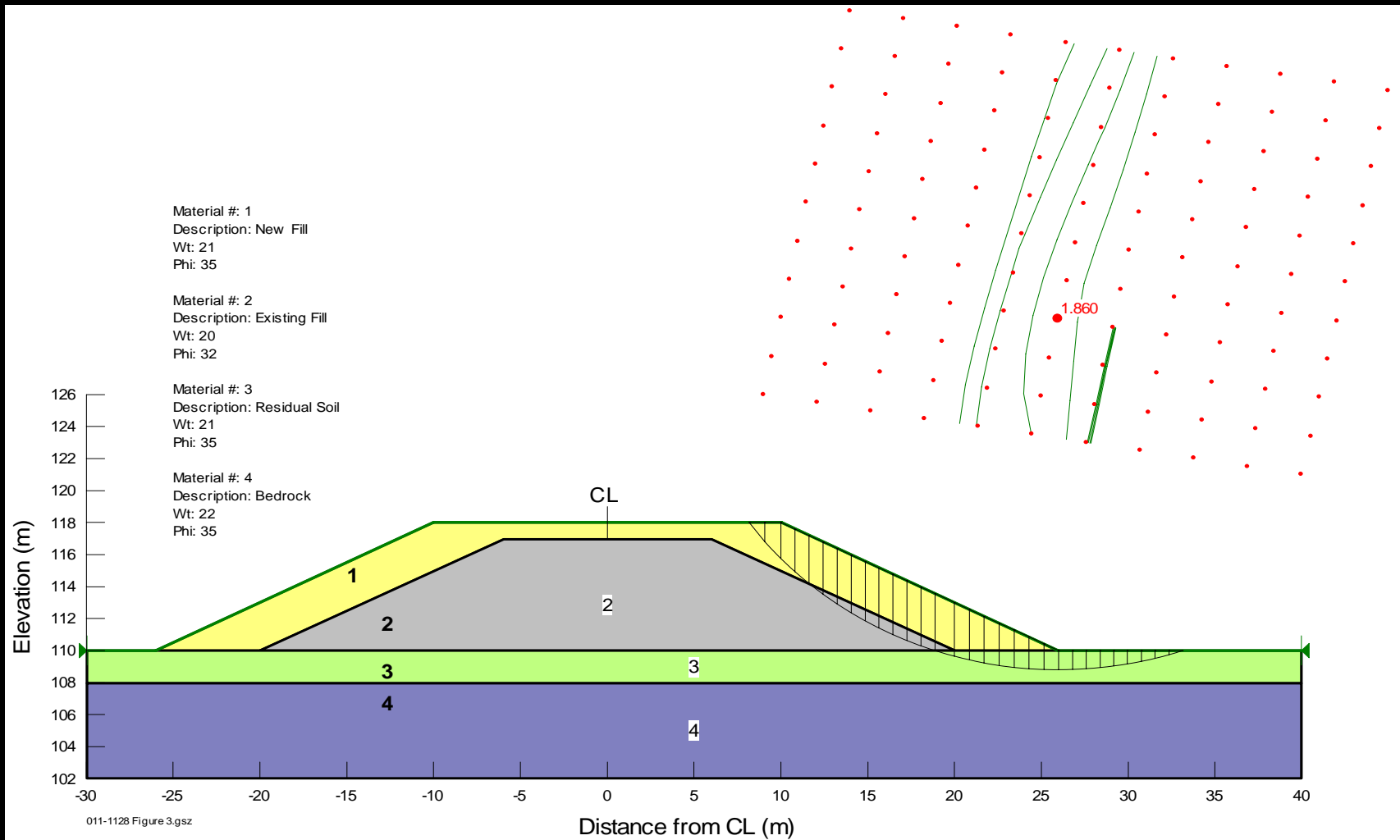
SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	F2	1	109.6
■	F5	9	107.8
◆	F6	1	109.1
○	F9	8	109.3



	<p>PLASTICITY CHART</p> <p>Clayey Silt to Silty Clay (Residual Soil)</p>	FIG No. 2
		Project No. 011-1128
		Checked by

STABILITY ANALYSIS QEW/Fourth Line Approach Embankments

FIGURE 3



Date: April 2007
Project: 011-1128-2

Golder Associates

Drawn: SEP
Checked: JMAC

STABILITY ANALYSIS QEW/Fourth Line Approach Embankments Under Seismic Loading

FIGURE 4

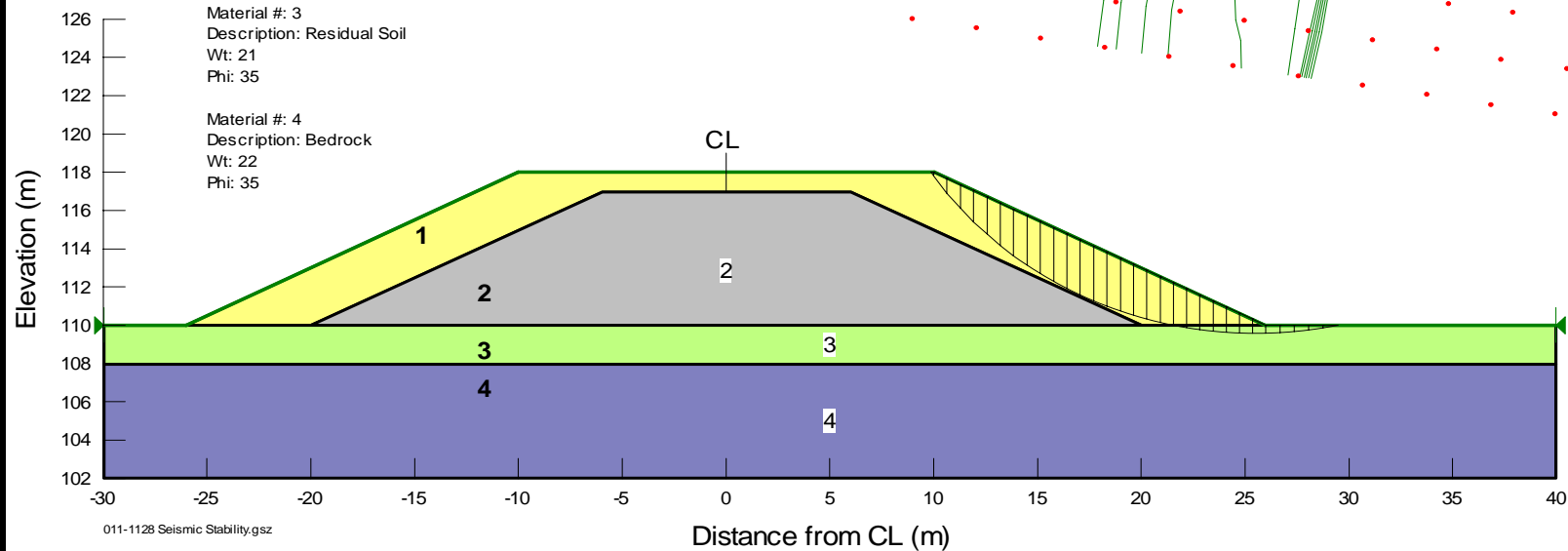
Seismic Load = 0.065g

Material #: 1
Description: New Fill
Wt: 21
Phi: 35

Material #: 2
Description: Existing Fill
Wt: 20
Phi: 32

Material #: 3
Description: Residual Soil
Wt: 21
Phi: 35

Material #: 4
Description: Bedrock
Wt: 22
Phi: 35



Date: April 2007
Project: 011-1128-2

Golder Associates

Drawn: SEP
Checked: JMAC

APPENDIX A
NON-STANDARD SPECIAL PROVISIONS

CSP BACKFILL - Item No.

Special Provision

Scope

The work under this item shall include all supply and placement of backfill to the CSP pipes surrounding the abutment piles.

Construction

The material shall satisfy the following gradation:

<u>MTO Sieve Designation</u>	<u>Percentage Passing (Mass)</u>
2 mm (#10)	100%
600 um (#30)	80% - 100%
425 um (#40)	40% - 80%
250 um (#60)	5% - 25%
150 um (#100)	0% - 6%

Basis of Payment

Payment at the contract price shall include all labour, equipment and materials to carry out the above work.

ROCK POINTS - Item No.

Non-Standard Special Provision

Scope

As part of the work under the above tender item, the Contractor shall supply TITUS Rock Injector Pile Points on HP 310 x 110 Piles for the Fourth Line Underpass structure.

References

OPSS 906 – Structural Steel

Materials

The pile points shall be of the following:

Product

Manufacturer

HPP-R-12

Titus Steel Company Ltd.
6767 Invader Cr.
Mississauga, ON
Tel (905) 564-2446

(Or approved equivalent)

Basis of Payment

Payment at the Contract Price for the above tender items shall be full compensation for all labour, equipment and material to do the work.

MASS CONCRETE - Item No.

Special Provision

Scope

The work under this item shall include all supply and placement of a mass concrete mat for all footings founded on the shale bedrock or residual soil for the Fourth Line Underpass. The purpose of the mass concrete pad is to provide protection of the subgrade from exposure and construction/foot traffic as per the Contract Drawings and Documents.

The work under this item shall also include all supply and placement of mass concrete for H-pile socket.

Construction

Work under this item shall satisfy the following requirements:

Pier Cap or Spread Footings

The surface of the pier/caisson cap or spread footing subgrade soils/bedrock soil shall be exposed and cleaned so that competent founding surface is exposed;

The mass concrete shall have a minimum 28 day strength of 5 MPa;

The mass concrete shall be placed on the exposed clean, subgrade soils/bedrock as per the Contract Drawings and Documents;

The thickness of the mass concrete shall be a minimum of 75 mm.

H-Pile Socket

The mass concrete shall have a minimum 28 day strength of 30 MPa;

The mass concrete shall be placed within a pre-augered hole, cleaned and inspected as per the Contract Drawings and Documents;

The thickness of the mass concrete socket shall be a minimum of 1000 mm.

Basis of Payment

Payment at the contract price shall include all labour, equipment and materials to carry out the above work.

CAISSON PILES - Item No.
HP310x110 PILES - Item No.
EXCAVATION FOR STRUCTURE - Item No.

Special Provision

Scope

The contractor is alerted of the potential presence for harder limestone/siltstone layers or slabs within the residual soil and the presence of strong limestone/siltstone layers within the shale bedrock.

The Contractor shall effect the appropriate construction materials and procedures to install the caisson piles or driven piles to the design tip elevation, maintaining sidewall stability throughout the excavation and concrete placement operation. The Contractor shall effect the appropriate construction materials and procedures for excavations within the residual soil and shale bedrock.

Basis of Payment

The contract price shall include all labour, equipment and materials to carry out the above work.