

GEOCRES No. 30M5-209DIST. CR REGION W.P. No. 47-88-00CONT. No. W. O. No. STR. SITE No. HWY. No. QEWLOCATION GUELPH LINE / NORTHSERVICE RD OVERPASS

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

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REPORT ON

**FOUNDATION INVESTIGATION AND DESIGN
GUELPH LINE / NORTH SERVICE ROAD OVERPASS
NORTH HALF OF QUEEN ELIZABETH WAY
AND GUELPH LINE INTERCHANGE
REGIONAL MUNICIPALITY OF HALTON
GWP: 47-88-00**

Submitted to:

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PART A – FOUNDATION INVESTIGATION REPORT
GUELPH LINE / NORTH SERVICE ROAD OVERPASS
NORTH HALF OF QUEEN ELEZABETH WAY
AND GUELPH LINE INTERCHANGE
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1.0 INTRODUCTION

Golder Associates Ltd. has been retained by the McCormick Rankin Corporation (McCormick Rankin) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out a foundation investigation at the site of the proposed Guelph Line and North Service Road Grade Separation in the Region of Halton, Ontario. The project consists of the north half of the Queen Elizabeth Way and Guelph Line interchange and includes the replacement of the existing Guelph Line underpass, the Guelph Line Overpass at the North Service Road and grade separation, the Roseland Creek culvert extension and embankments. This report addresses the Guelph Line / North Service Road structure and grade separation.

The purpose of the foundation investigation is to determine the subsurface conditions at the site of the proposed structure by drilling boreholes, and carrying out in-situ tests and laboratory tests on selected samples. The terms of reference for the scope of work are outlined in our Total Project Management proposal P81-1394-1, dated September 1998. The work was carried out in accordance with our Quality Control Plan for Foundation Design Services, Agreement No. 9820-7411-2715, dated January 1999.

The proposed preliminary alignment for the Guelph Line Overpass at North Service Road was presented on profiles provided to us by McCormick Rankin. The General Arrangement plan showing the proposed abutment and pier layout of the underpass structure has been provided to us on digital format on August 31, 2000.

2.0 SITE DESCRIPTION

The bridge site is located north of the Queen Elizabeth Way and immediately south of the existing Guelph Line overpass of the CN Rail (see Drawing 1). The two retaining walls for the grade separation are located east of Guelph Line and bridge and separate the future North Service Road from the East Link and the Guelph Line – North Service Road Link (see Drawing 2), in MTO District 4 in the City of Burlington, in the Region of Halton.

The topography of the site area is generally level and gradually slopes downwards towards the south. The existing Guelph Line has been constructed entirely in fill. Two drainage ditches run parallel to Guelph Line on either side, north of the existing North Service Road. The CN Overhead bridge is situated immediately to the north of the proposed bridge site. Within the project limits, the vegetation cover generally consists of grass, bushes, and mature trees.

3.0 INVESTIGATION PROCEDURES

The field work for this investigation was carried out between July 17 and 20, 2000. At this time seven (7) boreholes were put down at the site. Boreholes 9 and 10 were put down within the limits of the proposed foundation units. Boreholes 12 to 14 were put down along the length of the retaining wall that separates the North Service Road from the East Link. Boreholes 15, 16 and 10+035 NSR were put down along the length of the retaining wall the separates the North Service Road from the North Service Road – Guelph Line Link.

The investigation was carried out using a truck-mounted D-90 drill rig (for the bridge foundation holes) and a bombardier-mounted CME-55 drill rig (for the retaining wall holes) supplied and operated by Master Soil Investigation of North York. In the boreholes, samples of the overburden were obtained at regular intervals of depth of 0.75 m to 1.5 m using 50 mm outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedures. The boreholes were extended to depths of between 4.6 m and 16.7 m below the existing ground surface. Groundwater conditions in the open boreholes were observed throughout the drilling operations. Piezometers were installed in four boreholes to permit monitoring of the groundwater levels at the site. The piezometers consisted of a 200 mm long slotted tip threaded into 12 mm diameter PVC rigid tubing.

The field work was supervised on a full-time basis by a member of our engineering staff who located the boreholes in the field, directed the drilling, sampling, coring and in-situ testing operations, and logged the boreholes. The soil samples were identified in the field, placed in labeled containers and transported back to our laboratory in Mississauga for further examination. Index and classification tests consisting of grain size analyses, Atterberg Limits tests and water content determinations were carried out on selected samples.

The borehole locations were surveyed and staked in the field by Bennett Young Limited, professional land surveyors. Based on the information provided, the northing and easting co-ordinates of the borehole locations are given in UTM, and the borehole elevations are referenced to the Geodetic Datum. The co-ordinates of the boreholes are indicated on the Record of Borehole sheets and the locations of the boreholes are shown on Drawings 1 and 2.

4.0 GENERAL SITE GEOLOGY AND STRATIGRAPHY

4.1 Site Geology

The site is located in the physiographic region known as the Iroquois Plain. The Iroquois Plain is generally composed of a shallow cover of sand and till covering portions between Hamilton and Toronto (Chapman and Putnam, "The Physiography of Southern Ontario", 3rd Edition, 1984). The surface topography slopes gradually and fairly uniformly towards Lake Ontario. The native overburden at the site area is a silty clay till which is underlain by bedrock comprised of red shale (with limestone interbeds) of the Queenston Formation. The depth to bedrock at this site is shallow, varying typically between 2 m to 5 m below original ground surface.

4.2 Site Stratigraphy

The detailed subsurface soil and groundwater conditions encountered in the boreholes, 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 borehole sheets are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations.

In summary, the subsoils at the site generally consist of a surficial layer of topsoil or road base fill underlain by a 1.1 m to 9.5 m thick deposit of clayey silt / silty clay or sandy silt fill. The deeper fills are associated with the embankments. The fill is underlain by a 1.2 m to 3.6 m thick deposit of hard clayey silt glacial till. The till is directly underlain by weathered shale bedrock of the Queenston Formation.

Locations and elevations of the borings, together with the interpreted stratigraphical profile and section for the bridge site, are shown on the attached Drawing 1. Drawing 2 shows the locations of the borings at the retaining walls. A detailed description of the subsurface conditions encountered in the boreholes for this investigation is provided in the following sections.

4.2.1 Topsoil

A surficial layer of topsoil between 100 mm and 300 mm thick was encountered in all the boreholes drilled for the retaining walls (Boreholes 12 through 16 and 10+035 NSR). An increased amount of vegetation exists near the drainage channels on either side of Guelph Line.

4.2.2 Road Base Fill

Road base material, between 0.6 m and 0.9 m thick, consisting of crushed gravel, some sand followed by compact silty sand, trace gravel was encountered below the asphalt road surface in Boreholes 9 and 10, drilled through the existing Guelph Line embankment.

4.2.3 Clayey Silt / Silty Clay to Sandy Silt Fill

A 1.2 m to 9.5 m thick deposit of red-brown to grey clayey silt containing some to with sand, trace to some gravel and trace organics was encountered below the topsoil or road base fill at the location of all the boreholes, except in Boreholes 13, 14 and 16. The upper 1.0 m of the fill in Borehole 10+035 NSR consisted of silty clay containing trace sand and gravel and trace organics. Standard Penetration Testing (SPT) carried out within the clayey silt / silty clay fill gave 'N' values of between 10 blows and 54 blows per 0.3 m of penetration, which indicates a stiff to hard consistency. The upper 3.4 m of the clayey silt fill in Borehole 9 had 'N' values between 4 blows and 14 blows per 0.3 m of penetration indicating a soft to stiff consistency. The natural water content of selected samples of the fill were measured at between 10 percent and 16 percent.

In Borehole 13, 1.1 m of dark brown sandy silt containing trace clay and organics was encountered below the topsoil. The SPT 'N' value for the sandy silt fill was 10 blows per 0.3 m of penetration indicating a loose state of packing. The measured water content on a selected sample of the sandy silt fill was about 25 percent.

4.2.4 Clayey Silt Till

A 1.2 m to 3.6 m thick deposit of clayey silt till was encountered below the fill or topsoil in all of the boreholes. Trace shale and limestone fragments were noted within the clayey silt till. The

clayey silt till was fully penetrated in all the boreholes and directly overlies the weathered shale bedrock.

Standard Penetration Testing (SPT) carried out within the clayey silt till gave 'N' values of 40 blows to greater than 100 blows per 0.3 m of penetration, indicating a hard consistency. Grain size distribution curves for selected samples of the clayey silt till are shown on Figure 1. Atterberg Limits testing was carried out on selected samples of the upper clayey silt till. The liquid limits were between 18 percent and 21 percent and the plasticity were 4 percent to 5 percent. The test results are shown on the plasticity chart on Figure 2 and indicate that the clayey silt till is inorganic and of low plasticity. The natural water content measured on selected samples of the clayey silt till ranged from about 6 percent to 14 percent, below the measured plastic limit of the material.

4.2.5 Bedrock

The shale bedrock surface was encountered in all boreholes. At the bridge abutments (Boreholes 9 and 10) the bedrock was encountered at Elevations 106.0 m and 106.6 m, respectively. At the location of the North Service Road to Guelph Line Link (Boreholes 15, 16 and 10+035 NSR) the bedrock was encountered at about Elevations 106.2 m to 108.0 m (rising towards the east). At the location of the North Service Road to East Link retaining wall (Boreholes 12 through 14) the bedrock was encountered between about Elevations 105.5 m and 106.7 m. Hard limestone layers were encountered throughout the shale and were inferred from augering through the bedrock in Boreholes 12 through 16 and 10+035 NSR.

The shale bedrock was cored in Boreholes 9 and 10. RQD values were measured between 17 percent and 65 percent and total core recovery was measured between 75 percent and 100 percent. Limestone and siltstone interbeds were also evident throughout the recovered core and were between 0.03 m and 0.2 m in thickness. A 0.1 m thick clay seam was present at about Elevation 102.3 m in Borehole 10.

4.3 Groundwater Conditions

Water levels were noted in the open boreholes during and upon completion of the drilling operation; these levels are shown on the attached Record of Borehole sheets. Piezometers were sealed in Boreholes 10, 14, 16 and 10+035 NSR to permit the monitoring of the groundwater levels at the site. Details of the piezometer installations and water level measurements are shown on the attached Record of Borehole sheets.

All of the open boreholes were dry upon completion of drilling operations except in Borehole 15 where the water level was measured at about Elevation 106.6 m in the open borehole. A summary of the water level monitoring results are provided in the following table. All piezometer tips were installed within the bedrock.


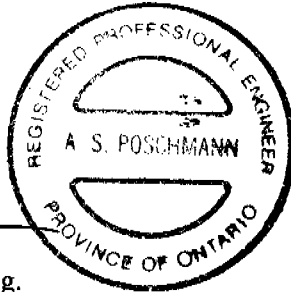
Borehole	On Completion of Drilling		Water Levels in Piezometers					
			July 25, 2000		August 16, 2000		September 5, 2000	
	Depth (m)	Elevation (m)	Depth (m)	Elevation (m)	Depth (m)	Elevation (m)	Depth (m)	Elevation (m)
10	Dry	--	--	--	--	--	12.7	105.6
14	Dry	--	0.9	107.2	0.4	107.7	--	--
16	Dry	--	0.9	109.9	0.4	110.4	--	--
10+035NSR	Dry	--	1.1	109.4	0.9	109.6	--	--

It should be noted that groundwater levels are expected to fluctuate seasonally and are expected to be higher during wet periods of the year.

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PART B – FOUNDATION DESIGN REPORT

GEULPH LINE / NORTH SERVICE ROAD OVERPASS
NORTH HALF OF QUEEN ELIZABETH WAY
AND GUELPH LINE INTERCHANGE
REGIONAL MUNICIPALITY OF HALTON
GWP: 47-88-00

5.0 ENGINEERING RECOMMENDATIONS

5.1 General

This section of the report provides our recommendations on the geotechnical aspects of design of proposed Guelph Line Overpass at the North Service Road and the grade separation based on our interpretation of the factual information obtained during the investigation. It should be noted that the interpretation and recommendations are intended for use only by the design engineer. 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 method and scheduling.

It is understood that the proposed North Service Road will be carried underneath the existing Guelph Line embankment which is about 10 m in height. Two retaining walls will separate the proposed North Service Road from the East Link and Guelph Line Link, respectively. The retaining walls are expected to have a maximum height of about 10 m, to match the existing / future grade of Guelph Line.

5.2 Bridge Foundations

The subsoils encountered in the boreholes put down during the present investigation typically consist of fill and hard clayey silt till overlying shale bedrock. The embankment fill generally consists of soft to hard, clayey silt with sand and trace gravel to depths of about 9.5 m. The till consists of hard clayey silt with sand and some gravel. The groundwater table is at about Elevations 105.6 m in the immediately vicinity of the proposed abutments.

Based on the subsurface information above, consideration may be given to support on shallow spread footings placed on the hard till deposit or the surface of the weathered shale. Consideration may also be given to supporting the structure on steel piles driven to practical refusal within the weathered shale or on caissons socketted into the shale bedrock. Difficulties may be encountered in advancing the piles through the lower portion of the embankment fill and the native soils and pre-augering will likely be required to advance the piles to achieve the minimum pile length of 5 m.

5.2.1 Shallow Foundations

Shallow spread footings may be used to support the abutments for the proposed bridge. It is assumed that footings would be located below the proposed adjacent North Service Road grade either in the native hard till or at the surface of the weathered bedrock. Also considered is the possibility of shallow spread footings placed higher within the embankment fill. In this case, removal of fill material to expose the native soil at about Elevation 108.0 m will be required and the placement of mass concrete to the founding level will be necessary to achieve the stated geotechnical resistance.

5.2.1.1 Geotechnical Resistance

For the configuration of the bridge as shown on the General Arrangement drawings, the highest founding level for spread footings founded on the native hard clayey silt till would be about Elevation 108 m. Footings placed on the surface of the weathered shale bedrock would extend to about Elevation 106 m. The design bearing capacities for both cases are given in the table below.

<i>Founding Option</i>	<i>Factored Geotechnical Resistance</i>	<i>Geotechnical Resistance</i>
	<i>ULS</i>	<i>SLS</i>
Spread Footings of Hard Clayey Silt Till	600 kPa	400 kPa
Spread Footings on Surface of Weathered Shale Bedrock	750 kPa	500 kPa

This founding level for the footings placed on the bedrock is near the groundwater level. The above geotechnical resistances assume, however, that appropriate construction procedures are adopted to handle any seepage inflow during footing construction to ensure that the hard till / weathered bedrock is not softened / disturbed prior to concrete placement.

5.2.1.2 Resistance to Lateral Forces

Resistance to lateral forces / sliding resistance between the concrete spread footings and subsoil should be calculated in accordance with Section 6-8.4.3 of the OHBDC assuming the following unfactored coefficients of friction between the concrete and the founding soils, which consider potential groundwater effects.

Footings on hard till	0.55
Footings on weathered shale	0.45

5.2.1.3 Frost Protection

All footings should be provided with a minimum of 1.2 m of earth cover for frost protection purposes.

5.2.1.4 Construction Considerations

The founding level for the bridge abutments placed on the hard till at about Elevation 108 m or at the surface of the weathered shale at about Elevation 106 m should be at above the groundwater level; however, some water seepage into footing excavations could be expected in either case. Pumping from well-filtered sumps placed at the base of the excavation within the till deposit should provide sufficient groundwater control during foundation excavations. Sumps should be maintained outside of the footing area. It should be noted that the surface of the weathered shale bedrock is variable across the site.

The founding soils and weathered bedrock are sensitive to disturbance and softening due to water seepage or ponding. Any water should be directed away from the footing area at all times. Placement of a lean concrete mud coat will be required at the base of the excavation for the footing area. The cleaned excavation base should be inspected by qualified geotechnical personnel. The mud coat should be placed within four hours after footing inspection. It should be noted that the water levels could be higher during wet periods of the year.

5.2.2 Deep Foundations

Consideration could be given to the use driven piles or caissons for support of the bridge abutments.

5.2.2.1 Factored Geotechnical Resistance – Steel H-Piles

Steel H-piles driven to practical refusal on the weathered shale bedrock at about Elevation 106 m are considered feasible for support of the abutments. For design, the factored axial resistance at Ultimate Limit States for HP 310 x 110 piles driven to practical refusal on shale bedrock may be

taken at 2,000 kN. The axial resistance at Serviceability Limit States (SLS) for 25 mm of settlement may be taken at 1,600 kN.

The SPT 'N' values within the clayey silt / sandy silt till through which the piles are to be driven were generally greater than 100 blows per 0.3 m of penetration. Driving of the piles could be difficult given the denseness of the subsoils and possible boulders in the overlying fill. Stiffening of the pile toe with MTO flange plates will be required for protection during driving. It is considered that pre-augering for a depth of at least 1 m will be required to provide, at a minimum, a starting guide for pile driving.

5.2.2.2 Factored Geotechnical Resistance - Caissons

The use of caissons socketted into the shale bedrock may also be considered as an alternative for the foundations. 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. For design, an axial geotechnical resistance at ULS of 3.4 MN and 5.3 MN may be assumed for 1.2 m and 1.5 m diameter caissons, respectively, assuming a rock socket length of 2 m. Serviceability Limit States (SLS) does not apply for caissons founded within the unweathered shale bedrock at this site.

5.2.2.3 Resistance of Lateral Loads

The lateral loading could be resisted fully or partially by the use of battered piles. If integral abutments are considered, the vertical piles must provide the resistance to the lateral loading. In this case, the horizontal reaction to the pile can be estimated using the following table where:

- K_h = coefficient of horizontal subgrade reaction (MPa/m) = $n_h (z/d)$
 d = pile width or diameter (m)
 n_h = constant of horizontal subgrade reaction (MPa/m)
 z = depth below adjacent road grade (m)

<i>Soil Type</i>	<i>$N_h z$ (MPa/m)</i>	<i>Note</i>
Clayey Silt Till	30	K_h constant with depth
Clayey Silt Till	60	K_h constant with depth

Group action for lateral loading should 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 lateral subgrade reaction in the direction of loading by a reduction factor R as follows:

<i>Pile Spacing in Direction of Loading D = Pile Diameter</i>	<i>Subgrade Reaction Reduction Factor R</i>
8d	1.00
6d	0.70
4d	0.40
3d	0.25

5.2.2.4 Frost Protection

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

5.3 Lateral Earth Pressures

The lateral pressures acting on the bridge abutments will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill and on the subsequent lateral movement of the structure. The following recommendations are made concerning the design of the abutments and the retaining walls in accordance with OHBDC:

- Select free-draining granular fill meeting the specifications of OPSS Granular 'A' or Granular 'B' but with less than 5 percent passing the 200 sieve should be used as backfill behind the walls. All granular fill should be compacted in lifts of loose thickness not greater than 200 mm to 95 percent of the material's Standard Proctor maximum dry density.
- Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill.
- The granular fill may be placed either in a zone with width equal to at least 1.2 m behind the back of the stem (Case I) or within the wedge-shaped zone defined by a 60 degree line extending up and back from the bottom of the rear face of the footing (Case II).
- If the wall support allows lateral yielding of the stem (unrestrained structure), active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding (restrained structure), at-rest pressures should be assumed for geotechnical design.

- A compaction surcharge equal to 16 kPa should be included in the lateral earth pressures for the structural design of the abutment wall in accordance with OHBDC Figure 6-7.4.3. Compaction equipment should be used in accordance with OPSS 501.06.
- For Case I, the pressures are based on the existing and proposed embankment fill materials and the following parameters (unfactored) may be assumed:

Soil unit weight 20 kN/m³
(assuming clean earth fill)

Coefficients of lateral earth pressure:

‘active’ 0.43
‘at rest’ 0.50

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

	Granular ‘A’	Granular ‘B’ Type II
Soil Unit Weight	22 kN/m ³	21 kN/m ³
Coefficients of Lateral Earth Pressure		
‘active’	0.27	0.31
‘at rest’	0.43	0.47

It should be noted that the above design parameters assume level backfill and ground surface behind the wall. Other aspects of the abutment granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD-3501.00.

5.4 Embankments

The proposed Guelph Line alignment requires widening of the existing Guelph Line embankments, which are currently about 10 m in height. It is understood that grade raise of up to 3.5 m is also required; this will increase the embankment height to greater than 13 m.

5.4.1 Embankment Design

The embankment subgrade soils consist of hard clayey silt till underlain by weathered shale bedrock. Providing that the embankment subgrade is properly prepared, the embankment with side slopes maintained at 2 horizontal to 1 vertical would be stable. Settlement below the embankment is expected to be negligible. Settlement of the new embankment fill itself will occur and it should

be noted that the settlement of the widened portion of the embankment will be differential with respect to the existing embankment.

Keying of the new embankment fill into the existing embankment side slopes will help to reduce the impact of differential settlement; generally the differential settlement has greatest impact on the travelled road surface and this should be addressed in the pavement design. The use of granular fill for the embankment widening will reduce the differential settlement since the majority of settlement of granular fills will occur during construction. The majority of the settlement of cohesive embankment fills would occur after construction.

Given the height of the proposed embankment, a mid-height berm with a platform width of 2.0 m will be required on both sides of the embankment for the approaches. The 2.0 m wide platform is not required in front of the abutments since there is a reduced embankment height and concrete slope paving will be present.

5.4.2 Embankment Construction

Topsoil and fill deposits should be stripped from below the fill embankment areas and all subgrade soils proof-rolled to aid fill placement. Placement of additional embankment fill material will be required on the east side of the existing embankment for the road widening. The newly placed embankment fill should be keyed into the existing embankment by benching in accordance with OPSD 208.01. Construction of the embankment above the prepared subgrade may be carried out using clean earth fill meeting the specifications of OPSS 212 or Select Subgrade Material meeting the specifications of OPSS 1010, depending on material availability. All embankment fill should be placed in regular lifts with loose thickness not exceeding 300 mm, and be compacted to at least 95 percent of the material's Standard Proctor maximum dry density. The final lift prior to placement of the granular subbase or base course should be compacted to 100 percent of the Standard Proctor maximum dry density. Inspection and field density testing should be carried out by qualified geotechnical personnel during all fill placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved. The permanent soil slopes of the embankment should be maintained not steeper than 2 horizontal to 1 vertical (2H:1V).

Vegetation cover should be established on all soil slopes to protect embankment fill against surficial erosion, as per OPSS 572.

5.5 Retaining Walls – Grade Separation

The two retaining walls for the grade separation are located east of Guelph Line and separate the future North Service Road from the East Link and the Guelph Line – North Service Road Link. Based on the plan / profile provided, it is understood that the maximum height of the walls will be about 10 m, where the walls join with the proposed Guelph Line. Wing walls will be located adjacent to the structure and parallel to the North Service Road.

The subsoils encountered in the boreholes put down during the present investigation for the walls typically consist of fill and hard clayey silt till overlying interbedded shale and limestone bedrock. The fill, where encountered, generally consists of stiff to hard clayey silt / silty clay, some sand and trace gravel or loose sandy silt to depths of about 1.5 m to 2.0 m and is overlain by a thin layer of topsoil. The till consists of hard clayey silt with sand and some gravel. The groundwater table varies from between Elevations 107.7 m and 110.4 m over the lengths of the walls. A drainage ditch runs parallel to and east of Guelph Line.

The base of the retaining walls is expected to be founded on either the hard clayey silt till or on the surface of the weathered shale bedrock. The clayey silt till is generally encountered just below the topsoil to a maximum depth of 2 m below ground surface. The weathered shale is generally encountered between 2 m and 5 m below ground surface. The ground water level was generally measured within 1.0 m depth below ground surface.

It is understood that two concepts are considered for the retaining walls:

- mechanically reinforced soil retaining wall system; and
- cantilever cast-in-place concrete retaining wall.

To provide protection against frost, the backfill material to a horizontal minimum distance of 1.2 m behind the face of the wall must consist of granular fill such as OPSS Granular A or Granular B Type II.

<i>Fill Materials</i>	<i>Granular A and Granular B, Type II</i>	<i>Granular B, Type II</i>
Soil Unit Weight	22 kN/m ³	21 kN/m ³
Angle of friction (unfactored)	35°	32°

5.5.1 Mechanically Reinforced Soil Retaining Wall System

A mechanically reinforced soil retaining wall system consists of soil reinforced with metal or fabric strips or grids integrated with suitable granular fill which is placed and compacted in layers. All topsoil, existing fill and loose / soft materials should be removed prior to placing the granular for the reinforced soil system and the granular fill material should be placed on native undisturbed soils; i.e. the till or weathered shale.

A facing material, typically pre-cast concrete panels mechanically fastened to the reinforcing strips or grids is used to form the face of the reinforced soil structure and to prevent the loss of fill material.

The maximum height of the wall is anticipated to be about 13 m; however, at the location of the drainage channel, which intercepts both walls, the footing may be required to be taken deeper. For the reinforced earth mass founded on the hard clayey silt till or weathered shale bedrock, a factored geotechnical resistance at ULS of 600 kPa and 750 kPa may be assumed for the hard clayey silt till and the weathered shale, respectively. The corresponding geotechnical resistance at SLS may be taken as 400 kPa and 500 kPa.

5.5.2 Cantilever Cast-in-place Concrete Retaining Wall

For the cantilever wall option, the concrete walls may be supported on spread footings founded within the till deposits. Geotechnical guidelines with respect to recommended geotechnical resistances at ULS and SLS, as well as the highest acceptable founding levels, are provided in the

following table. These values are based on a 3 m wide strip footing and a total settlement of about 25 mm.

<i>Founding Unit</i>	<i>Factored Geotechnical Resistance</i>	<i>Geotechnical Resistance</i>	<i>Highest Acceptable Founding Elevation* (m)</i>
	<i>ULS</i>	<i>SLS</i>	
NSR – East Link	600 kPa	400 kPa	108 to 109
NSR – Guelph Line	600 kPa	400 kPa	108 to 110

NOTE: * must be adjusted, if necessary, to provide a minimum 1.2 m of earth frost cover

Resistance to lateral forces / sliding resistance between the concrete footings and subsoils should be calculated in accordance with Section 6-8.4.3 of the OHBDC assuming an unfactored coefficient of friction of 0.55 between the concrete and the glacial till founding soils to calculate the resistance of the wall footing against sliding.

The geotechnical resistances provided are given under the assumption that the loads will be applied perpendicular to the surface of the footings. The inclination of the load should be taken into account in accordance with OHBDC when the load is not applied perpendicular to the surface of the footing.

The total settlement for footings under the design geotechnical resistances at SLS will be about 25 mm; however, the actual settlement will be dependent on the actual footing size and applied loads. The above guidelines should be reviewed if the footing dimensions differ from the dimensions assumed in this report.

5.6 Excavations and Temporary Cut Slopes

Excavations for footing construction will extend through silty clay / clayey silt to sandy silt fill and hard clayey sit till deposits. At the proposed bridge, the excavations for the footings will be up to about 13 m in depth below existing Guelph Line grade and up to about 5 m below the proposed grade for the North Service Road. Cobbles and boulders could be encountered in the fill at this site. Temporary open cut slopes should be maintained no steeper than 1 horizontal to 1 vertical (1H:1V). Where space restrictions dictate, the excavation could also be carried out within the fully braced

excavation. Roadway protection for Guelph Line will also be required during construction of the new structure to accommodate the staged construction.

Groundwater seepage inflow into the excavations through the clayey silt till deposit and the wet silt layer (where encountered) may occur but is expected to be minor, except during periods of sustained precipitation. Pumping from well-filtered sumps located at the base of the excavation should provide adequate groundwater control during foundation excavations. The considerations with respect to protection of the founding soils, however, as given in Section 5.2.1.4 must be recognized. Sumps should be maintained outside the actual footing limits. Surface water run-off should be directed away from the excavations at all times. The appropriate NSSP should be included in the contract documents. Where the drainage channel is intercepted, water should be diverted away from the footing excavations.

Where space is restricted and will not permit open cuts for footing / wall construction, a temporary support system should be installed to support the sides of the excavation and permit the use of vertical cuts. The temporary support system could consist of soldier piles and lagging where the piles would be socketted into pre-augered holes extended into the weathered shale bedrock below the excavation base. Some cobbles and boulders should be expected during augering for the soldier pile installation. Support to the soldier pile and lagging wall system could be in the form of struts and walers in the case of footing excavations or rakers and anchors in the case of retaining wall excavations.

The design of braced soldier pile and lagging walls should be based on a rectangular earth pressure distribution using the design parameters given below. Where the support to the wall is provided by anchors or rakers, the wall design should be based on a triangular earth pressure distribution using the design parameters given below. The raker / anchor support must be designed to accommodate the loads applied from pressures and surcharge pressures from area, line or point loads as well as the impact of sloping ground behind the system.

Unfactored triangular earth pressure distribution (p in kN/m^2 ; increasing with depth), can be calculated as follows:

$$\begin{aligned} \text{where} \quad p &= K_a \gamma H \\ H &= \text{the height of the excavation at any point in metres} \\ K_a &= 0.3 \text{ for level ground behind excavation} \\ \gamma &= \text{soil unit weight} = 20 \text{ kN/m}^3 \end{aligned}$$

Unfactored rectangular earth pressure distribution (p in kN/m^2 ; constant with depth), can be calculated as follows:

$$\begin{aligned} \text{where} \quad p &= K \gamma H \\ H &= \text{the height of the excavation} \\ K &= 0.3 \text{ for level ground behind excavation} \\ \gamma &= \text{soil unit weight} = 20 \text{ kN/m}^3 \end{aligned}$$

Passive toe restraint to the soldier piles may be determined using a triangular pressure distribution acting over an equivalent width equal to three times the pile socket diameter. The coefficient of passive lateral earth pressure, K_p , for the socket within the hard till or weathered bedrock may be taken as 8.7.

For the bridge footing excavations, the soldier piles will be socketted into the weathered shale bedrock. The soil unit weight should be taken as 20 kN/m^3 . A groundwater level at about Elevation 105.6 m can be assumed at the bridge footing locations.

Where roadway protection is required, grouted rock anchors are feasible and may be designed based on the following ultimate bond stresses as between grout and rock:

300 kPa – over the upper 1.5 m of bedrock
600 kPa – below the upper 1.5 m of bedrock

The ultimate rock anchor capacity calculated from the above adhesion values should be reduced by a factor of safety of at least 1.5. The maximum permissible stress in the anchor tendon or bar under

the design load should not exceed 0.625 of the guaranteed ultimate tensile strength of the tendon or bar.

A performance test should be carried out on at least one anchor to confirm the design and the Contractor's installation method. The performance test should be carried out to 1.5 times the design working load. In addition, each anchor should be proof tested to 1.25 times its working load. The tensile stress in the anchor bar during test loading should not exceed 0.8 of the guaranteed ultimate tensile strength of the bar. Anchor installation and testing should be carried out under the full-time inspection of a geotechnical engineer. Anchor installation and preloading should be complete before the excavation proceeds below the anchor elevation.

All excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Ontario Occupational Health & Safety Act. The native soils at this site would be classified as Type I soil.

GOLDER ASSOCIATES LTD.



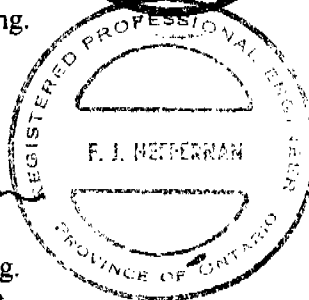
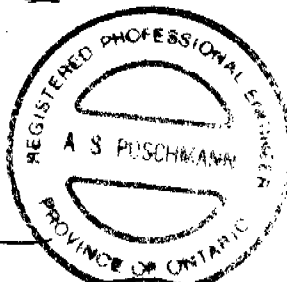
Sarah E.M. Poot, P. Eng.



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Designated MTO Contact



SEMP/ASP/FJH/clg

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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

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

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.)

Consistency

	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

(b) Cohesive Soils

Dynamic Cone Penetration Resistance; N_d :

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

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

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

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test ¹
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO ₄	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

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

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LIST OF SYMBOLS

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

I GENERAL

π	= 3.1416
$\ln x$	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. in 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 stresses (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 (con't.)

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)

(c) 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

(d) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (overconsolidated 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	Overconsolidation ratio = σ'_p / σ'_{vo}

(e) 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

PROJECT 991-1105 (3000)		RECORD OF BOREHOLE No 9		1 OF 2	METRIC
W.P. 47-88-00	LOCATION N 4801343.0; E 279950.8	ORIGINATED BY SEP			
DIST _____ HWY QEW	BOREHOLE TYPE 114mm SOLID STEM AUGERS	COMPILED BY SEP			
DATUM Geodetic	DATE July 17, 2000	CHECKED BY ASP			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			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		W _p	W	W _L		
119.11	GROUND SURFACE													
118.09	Asphalt													
0.20	Crushed gravel, some sand													
118.51	Light brown													
0.60	Dry (Fill)		1	50 DO	14									
	Clayey Silt, some sand, trace gravel													
	Soft to firm		2	50 DO	7									
	Red-brown to grey													
	Moist (Fill)		3	50 DO	4									
			4	50 DO	5									
115.11														
4.00	Clayey Silt with sand, some gravel													
	Very stiff to hard		5	50 DO	25									
	Red-brown													
	Moist (Fill)		6	50 DO	20									
			7	50 DO	21									
			8	50 DO	31									
109.01														
10.10	Clayey Silt with sand, some gravel, shale and limestone fragments													
	Hard		9	50 DO	70/0.1									
	Red-brown													
	Dry (Glacial Till)		10	50 DO	50/0.05									
106.01														
13.10	Red-brown, very weak, highly weathered SHALE with occasional grey limestone interbeds (Queenston Formation).													
	Bedrock cored from 14.1 m to 16.7 m depth.													
	For bedrock coring details refer to Record of Drillhole 9.													

ON_MOT_991-1105.GPJ ON_MOT_GDT_26/9/00

Continued Next Page

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

ON MOT 991-1105.GPJ ON MOT.GDT 26/9/00

+³, X³; Numbers refer to Sensitivity ○^{3%} STRAIN AT FAILURE

PROJECT: 991-1105 (3000)

RECORD OF DRILLHOLE: 9

SHEET 1 OF 1

LOCATION: N 4801343.0; E 279950.8

DRILLING DATE: July 17, 2000

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55

DRILLING CONTRACTOR: MASTER SOILS INVESTIGATION

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH	COLOUR % RETURN	FR-FRACTURE	F-FAULT	SM-SMOOTH	FL-FLEXURED	BC-BROKEN CORE	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION				
									CL-CLEAVAGE	J-JOINT	R-ROUGH	UE-UNEVEN	MB-MECH. BREAK						
									SH-SHEAR	P-POLISHED	ST-STEPPED	W-WAVY	B-BEDDING						
									VN-VEIN	S-SLICKENSIDED	PL-PLANAR	C-CURVED							
									RECOVERY		R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY k, cm/sec				
									TOTAL CORE %	SOLID CORE %				TYPE AND SURFACE DESCRIPTION					
									888888	888888	888888	888888	888888						
		Continued from previous page		105.04															
		Red-brown, very weak, highly weathered SHALE (QUEENSTON FORMATION). Grey, laminated, moderate to slightly weathered, fine grained limestone interbeds at 14.1 m, 14.7 m and 15.3 m depth (50 mm to 175 mm thick).		14.07															
15																			
16																			
					2														
				102.41															
17		END OF BOREHOLE		16.70															
18																			
19																			
20																			
21																			
22																			
23																			
24																			

DRILLHOLE 1105ROCK.GPJ GLDR CAN GDT 25/9/00 MMZ

DEPTH SCALE

1 : 50



LOGGED: SEP

CHECKED: ASP

PROJECT 991-1105 (3000)				RECORD OF BOREHOLE No 10				1 OF 2		METRIC						
W.P. 47-88-00				LOCATION N 4801337.0; E 279979.2				ORIGINATED BY SEP								
DIST HWY QEW				BOREHOLE TYPE 114mm SOLID STEM AUGERS				COMPILED BY SEP								
DATUM Geodetic				DATE July 20, 2000				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			20	40						60	80
118.31	GROUND SURFACE															
0.00	Asphalt															
117.71	Crushed gravel, some sand															
117.41	Light brown															
0.90	Dry (Fill)															
	Silty Sand, trace gravel, trace clay		1	50 DO	12											
	Compact															
	Brown															
	Moist (Fill)		2	50 DO	14											
	Clayey Silt with sand, some gravel															
	Stiff to hard															
	Red-brown to grey		3	50 DO	25											
	Moist (Fill)															
			4	50 DO	16											
			5	50 DO	14											
			6	50 DO	16											
			7	50 DO	23											
			8	50 DO	46											
108.21																
10.10	Clayey Silt with sand, some gravel, shale fragments															
	Hard															
	Red-brown		9	50 DO	100/15											
	Dry (Glacial Till)															
106.61																
11.70	Red-brown, moderately weathered, weak, laminated SHALE bedrock (Queenston Formation), with grey weathered limestone interbeds from 10.7 m to 13.3 m depth.															
	Bedrock cored from 13.3 m to 16.0 m depth.		10	50 DO	100/12											
	For bedrock coring details refer to Record of Drillhole 10.															

ON_MOT_991-1105.GPJ ON_MOT.GDT 26/9/00

Continued Next Page

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

PROJECT 991-1105 (3000)				RECORD OF BOREHOLE No 10				2 OF 2		METRIC				
W.P. 47-88-00		LOCATION N 4801337.0; E 279979.2				ORIGINATED BY SEP								
DIST HWY QEW		BOREHOLE TYPE 114mm SOLID STEM AUGERS				COMPILED BY SEP								
DATUM Geodetic		DATE July 20, 2000				CHECKED BY ASP								
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC NATURAL LIQUID UNIT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	20 40 60 80 100	W _p W W _L	WATER CONTENT (%)	10 20 30	γ	GR SA SI CL
102.31 16.00	END OF BOREHOLE Notes: 1. Open Borehole dry upon completion of drilling and prior to rock coring. 2. Water level measured in piezometer at 12.7m depth (Elev. 105.6 m) on Sept. 5/00.						103							

ON MOT 991-1105.GPJ ON MOT.GDT 28/9/00

PROJECT: 991-1105 (3000)

RECORD OF DRILLHOLE: 10

SHEET 1 OF 1

LOCATION: N 4801337.0; E 279979.2

DRILLING DATE: July 20, 2000

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55

DRILLING CONTRACTOR: MASTER SOILS INVESTIGATION

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV.		RUN No.	PENETRATION RATE (mm/min)	FLUSH % RETURN	COLOUR % RETURN	FR-FRACTURE F-FAULT SM-SMOOTH FL-FLEXURED BC-BROKEN CORE CL-CLEAVAGE J-JOINT R-ROUGH UE-UNEVEN MB-MECH. BREAK SH-SHEAR P-POLISHED ST-STEPPED W-WAVY B-BEDDING VN-VEIN S-SLICKENSIDED PL-PLANAR C-CURVED										DIAMETRAL PORE PRESSURE INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																														
				DEPTH (m)	RECOVERY					R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY k, cm/sec																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																					
					TOTAL CORE %							SOLID CORE %	TYPE AND SURFACE DESCRIPTION	DIP W.F.L. CORE AXIS	T ₁	T ₂	T ₃	T ₄																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
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DRILLHOLE 1105ROCK GPJ GLDR CAN GDT 25/9/00 MMZ

DEPTH SCALE

1 : 50



LOGGED: SEP

CHECKED: ASP

PROJECT 991-1105 (3000)			RECORD OF BOREHOLE No 12			1 OF 1			METRIC													
W.P. 47-88-00			LOCATION N 4801371.9; E 280021.7			ORIGINATED BY SEP																
DIST HWY QEW			BOREHOLE TYPE 114mm SOLID STEM AUGERS			COMPILED BY SEP																
DATUM Geodetic			DATE July 18, 2000			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 x REMOULDED					WATER CONTENT (%) w _p — w — w _L			γ			GR SA SI CL			
110.69	GROUND SURFACE							20 40 60 80 100														
0.99 0.10	Topsoil																					
	Clayey Silt, some sand, trace gravel, occasional cobbles, trace organics Very stiff to hard Brown to red-brown Moist (Fill)		1	50 DO	26		110															
109.11																						
1.58	Clayey Silt, some sand, some gravel Hard Red-brown Moist (Glacial Till)		2	50 DO	47		109															
			3	50 DO	67		108															
			4	50 DO	100		107															
			5	50 DO	111		106															
105.49																						
5.20	Weathered SHALE bedrock with hard limestone layers inferred from drilling, (Queenston Formation).		6	50 DO	120.02		105															
							104															
103.09	END OF BOREHOLE																					
7.60	Note: 1. Open Borehole dry upon completion of drilling.																					

ON MOT 991-1105.GPJ ON MOT.GDT 26/9/00

PROJECT 991-1105 (3000)			RECORD OF BOREHOLE No 13			1 OF 1			METRIC								
W.P. 47-88-00			LOCATION N 4801385.5; E 280084.5			ORIGINATED BY SEP											
DIST _____ HWY QEW			BOREHOLE TYPE 114mm SOLID STEM AUGERS			COMPILED BY SEP											
DATUM Geodetic			DATE July 18, 2000			CHECKED BY ASP											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
109.54	GROUND SURFACE						20	40	60	80	100						
109.27	Topsoil																
0.27	Sandy Silt, trace clay and organics Loose Dark brown Moist (Fill)		1	50 DO	10												
108.14	Clayey Silt, some sand, some gravel Hard Red-brown Moist (Glacial Till)		2	50 DO	90												
1.40			3	50 DO	114												
106.74	Weathered SHALE bedrock (Queenston Formation), with hard limestone layers, inferred from drilling from 3.3 m to 4.4 m depth and from 5.2 m to 6.1 m depth.		4	50 DO	100/0												
2.80			5	50 DO	100/0												
103.44			6	50 DO	100/0												
6.10	END OF BOREHOLE																
	Note: 1. Open Borehole dry upon completion of drilling.																

ON MOT 991-1105.GPJ ON MOT.GDT 26/9/00

PROJECT 991-1105 (3000)				RECORD OF BOREHOLE No 14				1 OF 1		METRIC			
W.P. 47-88-00		LOCATION N 4801390.3; E 280145.3				ORIGINATED BY SEP							
DIST HWY QEW		BOREHOLE TYPE 114mm SOLID STEM AUGERS				COMPILED BY SEP							
DATUM Geodetic		DATE July 20, 2000				CHECKED BY ASP							
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	20 40 60 80 100	W _p W W _L	WATER CONTENT (%)	UNIT WEIGHT γ	GR SA SI CL
108.08	GROUND SURFACE						108						
0.00	Topsoil												
107.78													
0.30	Clayey Silt, some sand, some gravel, pockets of grey silt Hard Red-brown Moist (Glacial Till)		1	50 DO	57		107						
			2	50 DO	46		106						
106.18													
1.90	Weathered SHALE bedrock (Queenston Formation), with hard limestone layers inferred from drilling at 3.8 m and 4.0 m depth.		3	50 DO	100/00		105						
			4	50 DO	100/00		104						
103.48													
4.60	END OF BOREHOLE												
<p>Note:</p> <p>1. Open Borehole dry upon completion of drilling.</p> <p>2. Water level measured in piezometer at 0.9 m depth (Elev. 107.18 m) on July 25, 2000.</p> <p>3. Water level measured in piezometer at 0.38 m depth (Elev. 107.7 m) on August 16, 2000.</p>													

ON MOT 991-1105.GPJ ON MOT.GDT 26/9/00

PROJECT 991-1105 (3000)				RECORD OF BOREHOLE No 15				1 OF 1		METRIC														
W.P. 47-88-00				LOCATION N 4801427.0; E 280028.9				ORIGINATED BY SEP																
DIST HWY QEW				BOREHOLE TYPE 114mm SOLID STEM AUGERS				COMPILED BY SEP																
DATUM Geodetic				DATE July 18, 2000				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			20	40						60	80	100	20	40	60	80	100	10	20
110.39	GROUND SURFACE																							
0.00	Topsoil		1	50 DO	8																			
110.09	Clayey Silt, some sand and gravel Stiff to hard Red-brown Moist (Fill)		2	50 DO	28																			
0.30																								
108.89	Clayey Silt with sand, some gravel, shale fragments Hard Red-brown Dry (Glacial Till)		3	50 DO	100/12																			
1.50																								
107.69	Weathered SHALE bedrock (Queenston Formation), with hard limestone layers inferred from drilling from 6.1 m to 7.6 m depth.		4	50 DO	30/05																			
				5	50 DO	125/12																		
				6	50 DO	109/1																		
			7	50 DO	130/06																			
102.79	END OF BOREHOLE																							
7.60	Note: 1. Water level measured in open borehole at 3.8 m depth (Elev. 106.59 m).																							

ON MOT 991-1105.GPJ ON MOT.GDT 26/9/00

PROJECT 991-1105 (3000)				RECORD OF BOREHOLE No 16				1 OF 1		METRIC			
W.P. 47-88-00		LOCATION N 4801475.0; E 280067.8				ORIGINATED BY SEP							
DIST HWY QEW		BOREHOLE TYPE 114mm SOLID STEM AUGERS				COMPILED BY SEP							
DATUM Geodetic		DATE July 20, 2000				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 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20 40 60 80 100					
110.75	GROUND SURFACE												
110.66	Topsoil												
0.20	Clayey Silt, some sand, trace gravel Hard Red-brown Moist (Glacial Till)		1	50 DO	40								
			2	50 DO	98								
			3	50 DO	60								
108.05	A 0.1 m thick layer of wet, red-brown, Silt, at 2.5 m depth (Elev. 108.2 m).		4	50 DO	100								
2.70	Weathered SHALE bedrock (Queenston Formation), with hard limestone layers inferred from drilling from 3.8 m to 4.0 m depth.												
105.95	END OF BOREHOLE												
4.80	Note: 1. Open Borehole dry upon completion of drilling. 2. Water level measured in piezometer at 0.9 m depth (Elev. 109.85 m) on July 25, 2000. 3. Water level measured in piezometer at 0.4 m depth (Elev. 110.35 m) on August 16, 2000.												

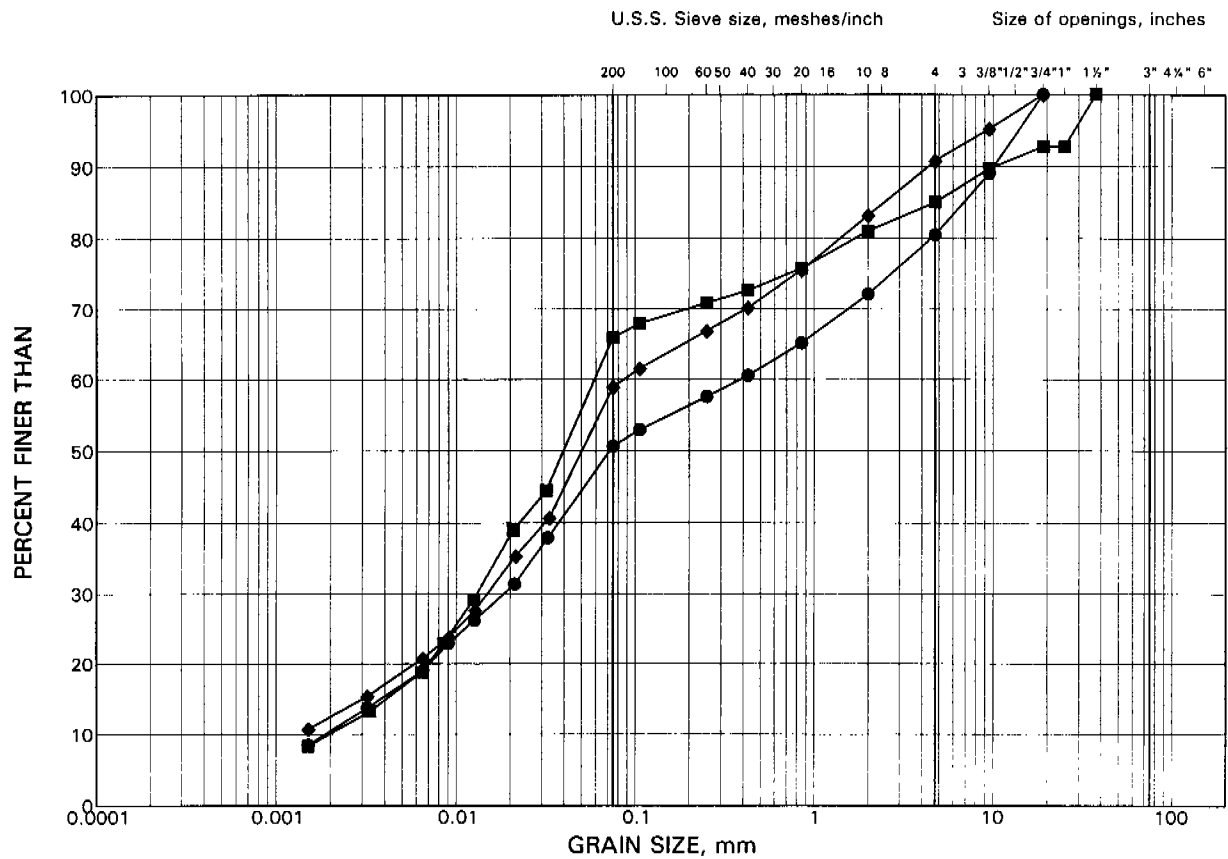
ON MOT 991-1105.GPJ ON MOT.GDT 26/9/00

PROJECT <u>991-1105 (3000)</u>		RECORD OF BOREHOLE No 10+035 (NSR)		1 OF 1		METRIC	
W.P. <u>47-88-00</u>		LOCATION <u>N 4801367.0; E 279986.8</u>		ORIGINATED BY <u>SEP</u>			
DIST <u> </u> HWY <u>QEW</u>		BOREHOLE TYPE <u>114mm SOLID STEM AUGERS</u>		COMPILED BY <u>SEP</u>			
DATUM <u>Geodetic</u>		DATE <u>July 18, 2000</u>		CHECKED BY <u>ASP</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	W _p	W	W _L		
110.50	GROUND SURFACE													
0.00	Topsoil													
0.15	Silty Clay, trace sand and gravel, trace organics Stiff Brown Moist (Fill)		1	50 DO	10									
109.30	Clayey Silt, some sand, trace gravel hard Red-brown Moist (Fill)		2	50 DO	54									
108.40	Clayey Silt with sand, trace gravel, limestone fragments Hard Red-brown Moist (Glacial Till)		3	50 DO	73									
2.10			4	50 DO	99/15									9 32 48 13
106.20														
4.30	Weathered, red-brown SHALE bedrock (Queenston Formation), with hard limestone layers inferred from drilling.		5	50 DO	109/15									
104.40														
6.10	END OF BOREHOLE Note: 1. Open Borehole dry upon completion of drilling. 2. Water level measured in piezometer at 1.15 m depth (Elev. 109.35 m) on July 25, 2000. 3. Water level measured in piezometer at 0.92 m depth (Elev. 109.58 m) on August 16, 2000.													

GRAIN SIZE DISTRIBUTION CLAYEY SILT (GLACIAL TILL)

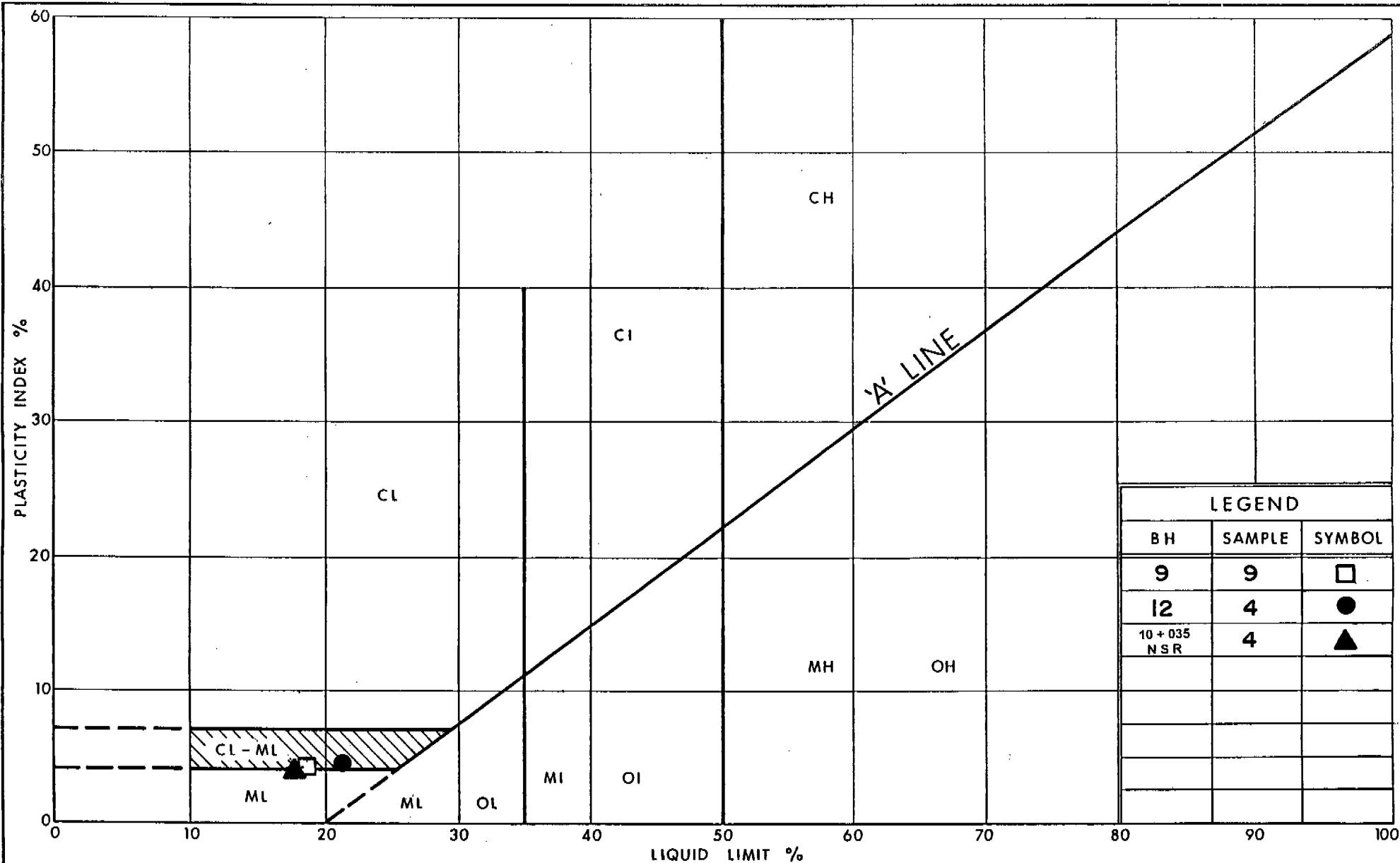
FIGURE 1



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	9	9	108.2
■	12	4	107.6
◆	10+035 NSR	4	107.5



Ontario

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PLASTICITY CHART CLAYEY SILT (GLACIAL TILL)

FIG No 2

GWP 47 - 88 - 00

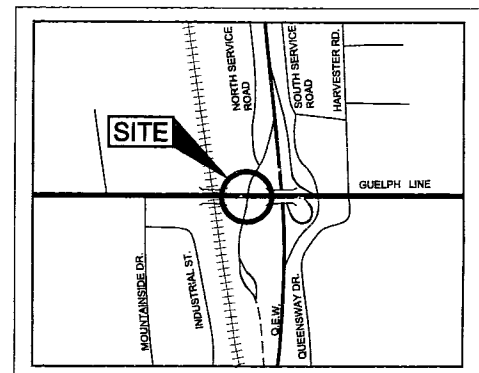
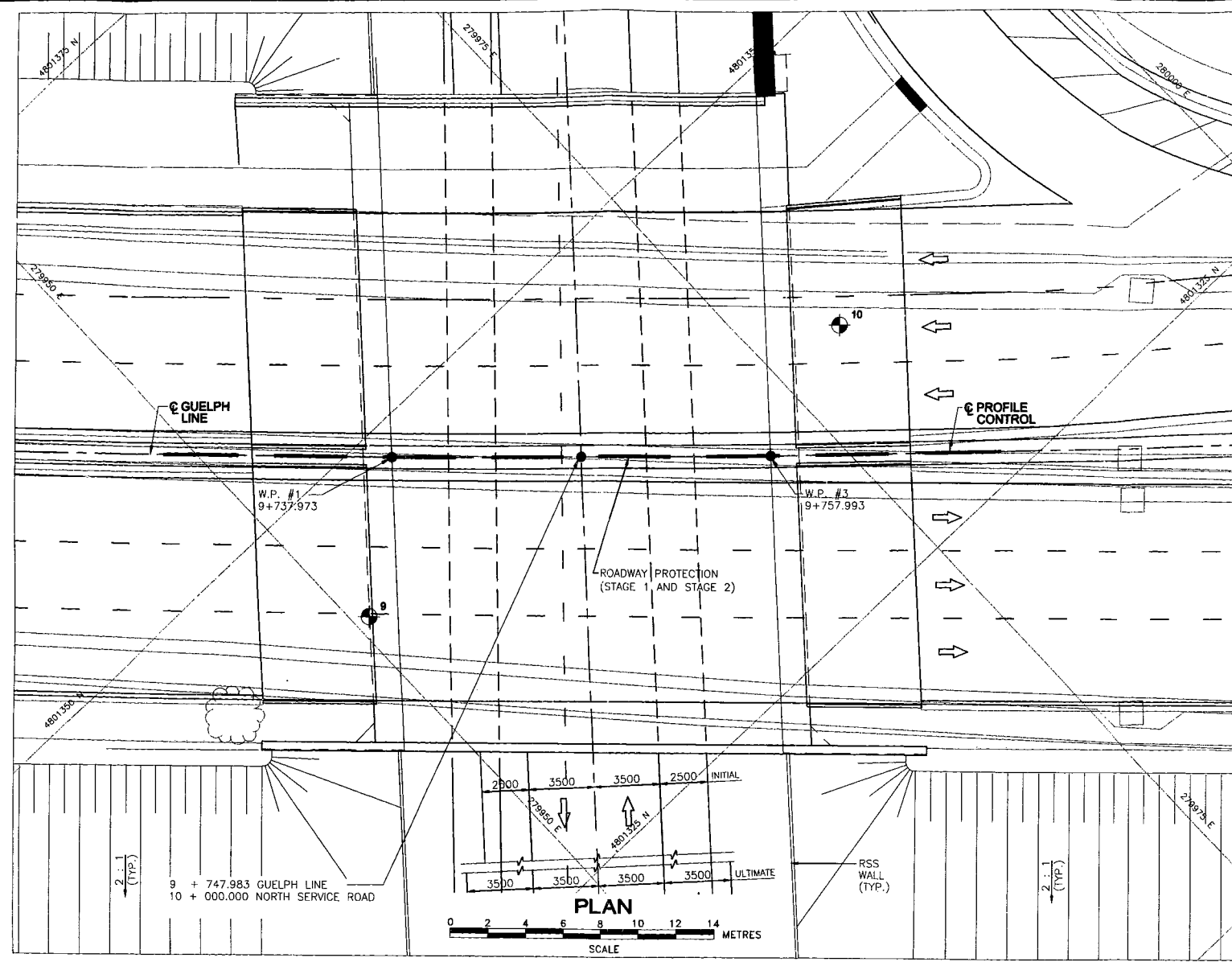
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

DIST 4
CONT No.
WP No. 47-88-00

**GUELPH LINE OVERPASS
AT NORTH SERVICE ROAD**
BOREHOLE LOCATIONS & SOIL STRATA



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN

LEGEND

- Borehole - Current Golder Associates Ltd. Investigation
- Seal
- Piezometer
- N Blows/0.3m (Std. Pen. Test, 475 j/blow)
- WL in piezometer on September 5, 2000
- WL upon completion of drilling

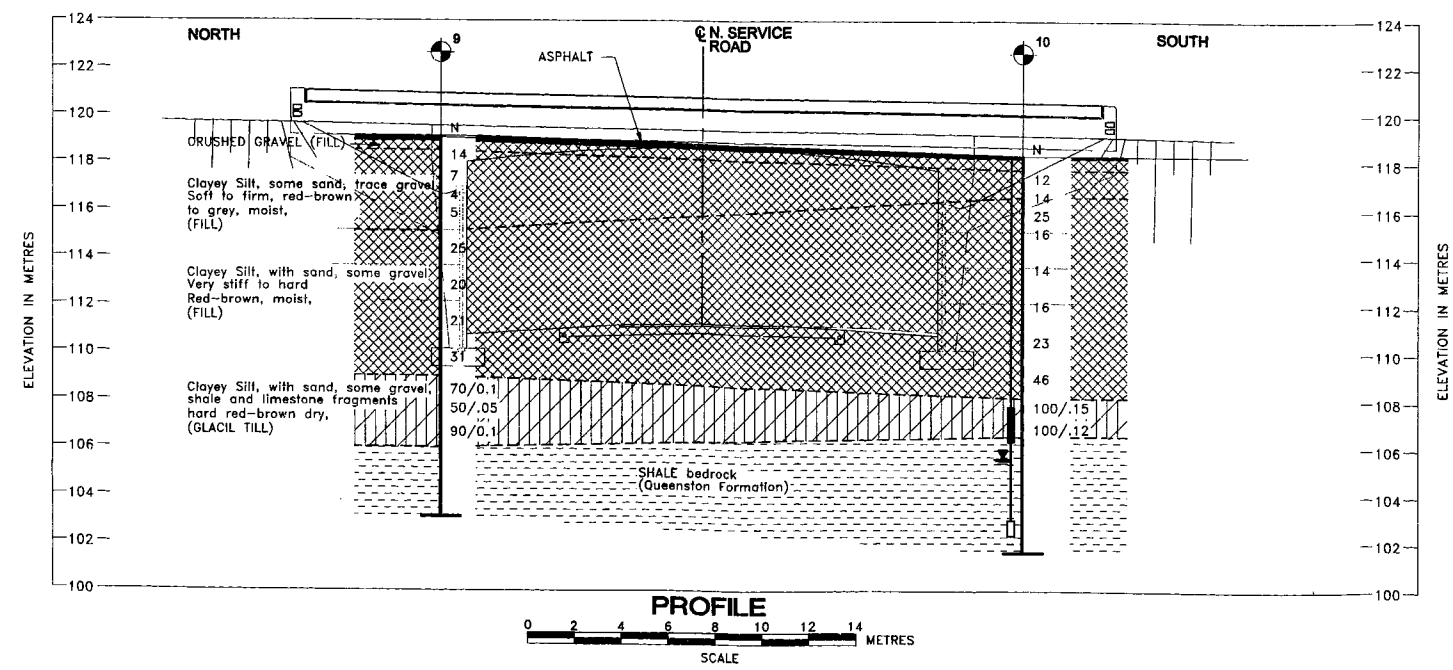
No.	ELEVATION	LOCATION	
		NORTHING	EASTING
9	119.11	4801343.0	279950.8
10	118.31	4801337.0	279979.2

NOTES

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

REFERENCE

This drawing was created from digital file "S3899-20.dwg" titled "GUELPH LINE OVERPASS AT NORTH SERVICE ROAD" provided by McCormick Rankin Corp. on August 31, 2000



NO.	DATE	BY	REVISION

Geocres No.

GUELPH LINE	PROJECT NO.: 991-1105	DIST. 4
SUBM'D. SEP	CHK'D: ASP	DATE: 2000 09 05
DRAWN: JFC	CHKD. SEP	APPD. DWG. 1

01.050018.DWG

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

DIST 4 Q.E.W.
CONT No.
WP No. 47-88-00

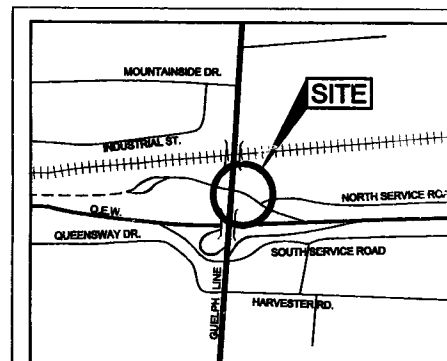
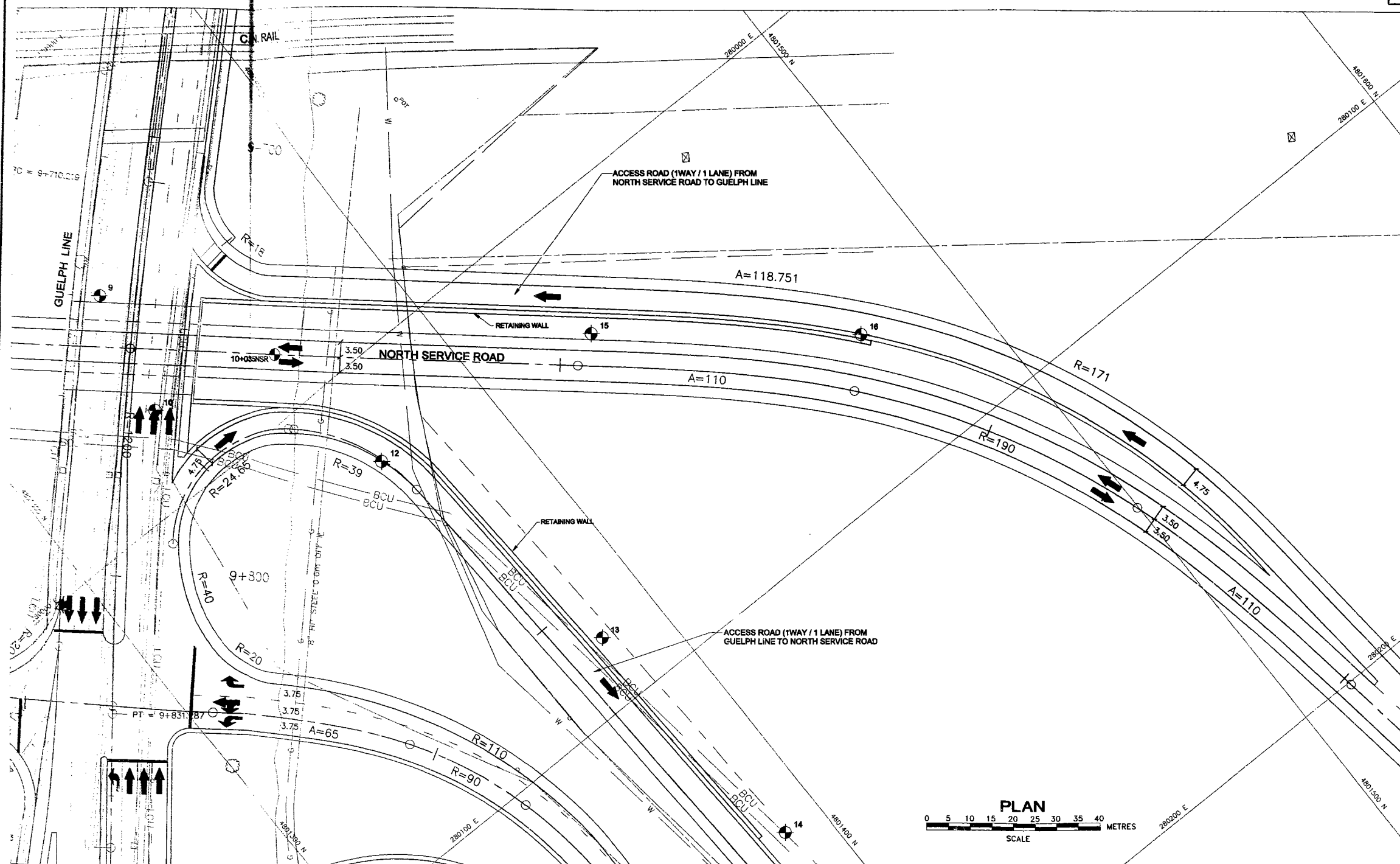


GUELPH LINE/NORTH SERVICE RD.
RETAINING WALL
BOREHOLE LOCATIONS

SHEET



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN

LEGEND

Borehole - Current Golder Associates Ltd. Investigation

No.	ELEVATION	LOCATION	
		NORTHING	EASTING
9	119.11	4801343.0	279950.8
10	118.31	4801337.0	279979.2
12	110.69	4801371.9	280021.7
13	109.54	4801385.5	280084.5
14	108.08	4801390.3	280145.3
15	110.39	4801427.0	280028.9
16	110.75	4801475.0	280067.8
10+035NSR	110.5	4801367.0	279986.8

NOTES

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

REFERENCE

This drawing was created from digital file "S3899-01.dwg" titled "Q.E.W. UNDERPASS AT GUELPH LINE" provided by McCormick Rankin Corp. on August 31, 2000

NO.	DATE	BY	REVISION

Geocres No.

Q.E.W.	PROJECT NO.: 991-1105	DIST. 4
SUBM'D. SEP	CHKD: ASP	DATE: 2000 09 05
DRAWN: JFC	CHKD: SEP	APPD. DWG. 2