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Golder Associates Ltd.

2180 Meadowvale Boulevard
Mississauga, Ontario, Canada L5N 5S3
Telephone (905) 567-4444
Fax (905) 567-6561

Golder Associates

REPORT ON

FOUNDATION INVESTIGATION AND DESIGN
GUELPH LINE UNDERPASS
NORTH HALF OF QUEEN ELIZABETH WAY
AND GUELPH LINE INTERCHANGE
REGIONAL MUNICIPALITY OF HALTON
GWP: 47-88-00

Submitted to:

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

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PART A - FOUNDATION INVESTIGATION REPORT

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 underpass at Queen Elizabeth Way (QEW) in the Region of Halton, Ontario. The project consists of the north half of the QEW and Guelph Line interchange and includes the replacement of the existing Guelph Line underpass, the construction of the Guelph Line / North Service Road Overpass, the Roseland Creek culvert extension and the embankments. This report addresses the Guelph Line underpass.

The purpose of the foundation investigation is to determine the subsurface conditions at the site of the proposed underpass 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 Underpass 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 in digital format on August 31, 2000. The following documents have been reference during the preparation of this report.

- GEOCRES 30M5-74 dated December 1955, Preliminary Design Report, QEW / Guelph Line Interchange, W.P. 137-86-00.
- Contract Drawings, Widening of Highway 25 Underpass, W.P. 204-62.

2.0 SITE DESCRIPTION

The site is located at the existing QEW and Guelph Line interchange, and is within the MTO District 4 in the City of Burlington.

The topography of the site area is generally level and gradually slopes downwards towards the south. The existing Guelph Line and underpass have been constructed entirely in fill. Within the project limits, the vegetation cover generally consists of grass, bushes, and one mature tree on the southeast side of the proposed abutment location.

3.0 INVESTIGATION PROCEDURES

The field work for this investigation was carried out between July 18 and September 7, 2000. At this time four boreholes were put down at the site. Boreholes 1 to 4 were put down within the limits of the proposed foundation units and extended into the bedrock.

The investigation was carried out using a bombardier-mounted CME-55 drill rig (for the two holes drilled at the toe of the existing embankment) and using a truck-mounted D-50 drill rig (for the two holes drilled through the embankment) 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 8.4 m and 12.2 m below the existing ground surface. The bedrock was cored in Boreholes 1 and 3. Groundwater conditions in the open boreholes were observed throughout the drilling operations. Piezometers were installed in two 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, rock 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 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 Drawing 1.

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 and 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 variable thickness of embankment fill material to about Elevations 104.4 m to 105.5 m underlain by a 1.6 m to 3.3 m of either clayey silt / sandy silt till or wet sandy silt. The till or sandy silt layer is underlain by shale bedrock from the Queenston Formation containing limestone and siltstone layers.

Locations and elevations of the borings, together with the interpreted stratigraphical profile and sections, are shown on the attached Drawing 1. A detailed description of the subsurface conditions encountered in the boreholes for this investigation is provided in the following sections.

4.2.1 Road Base Fill

A 0.8 m to 0.9 m thick layer of very dense, crushed gravel and sand and gravel fill was encountered beneath the asphalt road surface in Boreholes 2 and 4, which were drilled through the existing bridge embankment.

4.2.2 Silty Clay to Clayey Silt Fill

The 7.3 m to 8.3 m thick embankment encountered in Boreholes 2 and 4 consisted of red-brown to grey, clayey silt with sand, containing some gravel and trace organics. In Borehole 3, drilled at the toe of the embankment, 2.4 m of this fill was present. In general the embankment fill was consistent in composition throughout depth. A 0.2 m thick layer of asphalt was encountered in the fill in Borehole 4 at about Elevation 106.3 m. About 0.6 m of dark brown silty clay with sand and organics was encountered immediately below ground surface in Borehole 1. Grey clayey silt containing trace sand, gravel and organics was encountered at 1.5 m depth in Borehole 1 and was about 1.5 m thick. Standard Penetration testing (SPT) carried out within the fill gave 'N' values between 5 blows and 64 blows per 0.3 m of penetration, indicating a firm to hard consistency. The water content of selected samples of the silty clay / clayey silt fill was measured between about 10 percent and 19 percent.

4.2.3 Silty Sand Fill

A 0.9 m thick layer of compact, dark brown, silty sand fill containing trace clay, gravel and organics was present between the silty clay and clayey silt fill in Borehole 1. The SPT 'N' value for this sample was 22 blows per 0.3 m of penetration and the measured water content was 11 percent.

4.2.4 Sandy Silt

In Borehole 2 only, a 2.0 m thick layer of wet, red-brown, sandy silt containing trace clay and gravel and pockets of silty clay was encountered below the embankment fill at about Elevation 104.7 m. The measured SPT 'N' value was greater than 100 blows per 0.3 m of penetration indicating a very dense state of packing. This deposit is water bearing and directly overlies the bedrock.

4.2.5 Sandy Silt Till

Red-brown sandy silt till was encountered below the fill in Boreholes 1 and 3 at about Elevations 104.5 m and 105.6 m respectively. In Borehole 1, the sandy silt till was about 1.6 m thick overlying the bedrock. In Borehole 3, the sandy silt till was 1.6 m thick and overlies a layer of clayey silt till. The SPT ‘N’ values ranged from 24 blows to greater than 100 blows per 0.3 m of penetration indicating a compact to very dense state of packing. The measured water contents on selected samples of the till were about 15 percent.

4.2.6 Clayey Silt Till

A 1.7 m to 1.8 m thick deposit of red-brown clayey silt to silty clay containing some to with sand, some gravel was encountered in Boreholes 3 and 4. In Borehole 3, the clayey silt till was encountered below the sandy silt till at about Elevation 104.0 m and directly overlies the bedrock. In Borehole 4, the clayey silt till was encountered beneath the embankment fill in Borehole 4 at about Elevation 104.9 m and overlies the bedrock. The SPT ‘N’ values were between 85 blows and greater than 100 blows per 0.3 m of penetration indicating a hard consistency. Atterberg limits testing were carried out on selected samples of the clayey silt till and gave a liquid limit between 18 percent and 26 percent and a plasticity index between 3 percent and 9 percent indicating the clayey silt is inorganic and of low plasticity. The measured water content on selected samples of the till was between about 11 percent and 14 percent, below the plastic limit of the material.

4.2.7 Bedrock

The shale bedrock surface was encountered in all four boreholes at about Elevations 102.2 m to 103.1 m. Hard limestone and siltstone layers between 25 mm and 75 mm thick were encountered throughout the shale and were inferred from augering through the bedrock in Boreholes 2 and 4. The shale bedrock was cored in Boreholes 1 and 3. The upper 1.0 m to 1.5 m of the shale was moderately weathered and below these depths the shale was slightly weathered. Rock Quality Designation (RQD) values were measured between 0 percent and 63 percent and total core recovery was measured between 56 percent and 100 percent. Limestone and siltstone interbeds were also evident throughout the recovered core. A 25 mm clay seam was encountered in the core at about Elevation 101.4 m in Borehole 3.

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 1 and 3 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.

Water was noted in the open hole in Borehole 2 on completion of drilling at Elevation 104.3 m. The water level was measured in the piezometer in Borehole 1 after completion of overburden drilling and rock coring at Elevation 104.7 m. The water level in the open hole in Borehole 3 was measured after completion of overburden drilling and prior to rock coring at Elevation 102.8 m. Borehole 4 was dry upon completion of drilling operations. The water levels in the piezometers in Boreholes 1 and 3 are summarized in the table below.

Borehole	*On Completion of Drilling*		*Water Levels in Piezometers*					
			August 16, 2000		*September 7, 2000*		*September 22, 2000*	
	Depth (m)	*Elevation (m)*	*Depth (m)*	*Elevation (m)*	*Depth (m)*	*Elevation (m)*	*Depth (m)*	*Elevation (m)*
1	2.8	104.7	2.9	104.6	3.5	104.0	--	--
3	5.2	102.8	--	--	--	--	4.2	103.8

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

GOLDER ASSOCIATES LTD.

Sarah E.M. Poot, P. Eng.

Anne S. Poschmann, P.Eng.
Principal

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

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PART B – FOUNDATION DESIGN REPORT

GEULPH LINE UNDERPASS
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 foundation aspects of design of the proposed Guelph Line underpass 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 existing bridge will be replaced with a new longer and wider structure and that the new embankment will tie in with the existing embankments. The QEW will remain at the current grade, and the new bridge structure will be shifted east of its current location.

5.2 Bridge Foundations

The subsoils encountered in the boreholes put down during the present investigation typically consist of fill and very dense / hard tills overlying moderately to slightly weathered shale bedrock. The composition of the till varies from clayey silt, some sand and gravel to sandy silt, some gravel and trace clay. The fill deposits may contain cobbles or boulders. A boulder was encountered just below ground surface in the fill in one borehole during the drilling operation. The water table was measured between about Elevations 103.8 m to 104.7 m in the immediately vicinity of the proposed structure.

Based on the subsurface information above, consideration may 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. It is understood that integral abutment design is being considered for the structure which will involve driving piles through the existing Guelph Line embankment fills. The piles would be driven to practical refusal on or within the shale bedrock. Difficulties may be encountered in advancing the piles through the lower portion of the embankment fill and pre-augering may be required to advance the piles. Consideration may also be given to support on shallow spread footings placed on the very dense / hard till deposits or the surface of the weathered shale bedrock.

5.2.1 Shallow Foundations

Spread footings may be used to support the abutments and the central pier location. It is assumed that footings would be located below the proposed adjacent road grade either in the very dense / hard till or at the surface of the weathered bedrock.

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 hard clayey silt till would be about Elevation 104.5 m. Footings placed on the surface of the weathered shale bedrock would extend to about Elevation 102.2 m. The design values for both cases are given in the table below.

Founding Option	*Factored Geotechnical Resistance*	*Geotechnical Resistance*
	ULS	*SLS*
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 on the till is at about the groundwater level. For the footings on the surface of the weathered shale, the founding level is about 2.5 m below the groundwater level. The above geotechnical resistances assume, however, that appropriate construction procedures are adopted during footing construction to ensure that the very dense / hard till or 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 very dense / 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 abutments and central pier placed on the very dense / hard till at Elevation 104.5 m will be at about the groundwater table level. Footings placed on the surface of the weathered shale at about Elevation 102.2 m will be about 2 m to 2.5 m below the groundwater table and some water inflow into footing excavations should be expected in either case. Pumping from well-filtered sumps placed at the base of the excavation should provide sufficient groundwater control during foundation excavations. Sumps should be maintained outside of the footing area. In the case of the piers where there are space restrictions, longer excavations may be required to maintain sump locations outside the footing area.

The founding soils and weathered bedrock are sensitive to disturbance and softening due to water seepage or ponding. Placement of a mud coat will be required at the base of excavation for the footing area. Exposure without protection of the mud coat will allow water to soften the founding soils or weathered rock. The cleaned excavation base should be inspected by qualified geotechnical personnel prior to placing the mud coat. The mudcoat 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

It is understood that consideration is being given to designing the underpass structure with integral abutments. Driven steel H-piles are considered suitable for the abutment support. Consideration may also be given to the use of caissons.

5.2.2.1 Geotechnical Axial Resistance – Driven Steel H-Piles

For design, the factored axial resistance at Ultimate Limit States (ULS) 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 bedrock surface is variable at the site and ranges between about Elevations 102.2 m and 103.1 m. For design of piled foundations, a minimum pile length of 5 m is typically required. Therefore, for the south abutment piles, the design should be checked for a pile tip level of Elevation 102.7 m to confirm that the minimum length is achievable. For the north abutment, the available driven pile length should be checked against the highest bedrock surface level which is Elevation 103.1 m.

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 / obstructions 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 below the pile cap level will be required to provide, at a minimum, a starting guide for pile driving.

The H-piles should be driven to the above tip elevations and to a final set of no less than 15 blows per 25 mm of penetration using a hammer with rated energy of about 50 kJ but not exceeding 60 kJ. Provision should be made to re-tap the piles to confirm the set after adjacent piles have been driven in accordance with Special Provision 903S01.

It is possible that heavy driving may be encountered above the bedrock surface due to the presence of cobbles and boulders. It should be noted, however, that the above design resistances assume the piles are driven at least to the bedrock surface. For the north abutment, the piles should be driven to at least Elevation 103.1 m at the east limit of the footing and to at least Elevation 102.2 m at the west limit of the footing. For the south abutment, the piles should be driven to at least Elevation 102.7 m. The appropriate notes which should be shown on the General Arrangement drawing are:

- "Piles to be driven to bedrock."
- "Piles to be driven to the elevations as shown."

5.2.2.2 Geotechnical Axial Resistance - Caissons

It is understood that caissons socketted into the shale bedrock are being considered as an alternative for the central pier. The bedrock surface at the four boreholes drilled at the bridge abutments found the bedrock surface between Elevations 102.3 m and 103.1 m. The bedrock elevation at the central pier could be inferred to be in this range; however, this should be confirmed on site by means of a test pit prior to foundation construction at the pier. For the central pier, the caissons will be about 5 m long assuming a rock socket length of 2.5 m which, for caissons larger than 1 m in diameter, results in a length to diameter ratio of less than 5. It is recommended that the caissons for the central pier 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. The caissons should be extended to at least 2 m below the bedrock surface which, for design, may be assumed to be at Elevation 102.5 m for the central pier. Serviceability Limit States (SLS) does not apply for caissons founded within the unweathered shale bedrock at this site.

If caissons are adopted for the abutments, the caissons should be designed based on shaft friction through the overburden and the shale bedrock. For design, an axial geotechnical resistance at ULS of 5 MN and 6.2 MN may be assumed for 1.2 m and 1.5 m diameter caissons, respectively, assuming a rock socket length of 2 m.

5.2.2.3 Resistance of Lateral Loads

The lateral loading could be resisted fully or partially by the use of battered piles. In the case of integral abutments, 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)

Soil Type	*$n_h \cdot z$ (MPa/m)*	*Note*
Clayey Silt Fill	30	K_h constant with depth
Clayey Silt / Sandy Silt Till	60	K_h constant with depth

[**NOTE:** the value of $n_h z$ is equivalent to $k_s/5$ for cohesive soils]

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:

Pile Spacing in Direction of Loading D = Pile Diameter	*Subgrade Reaction Reduction Factor R*
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 minimum of 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 (assuming clean earth fill)	20 kN/m^3
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 Mechanically Reinforced Soil Retaining Wall System

A retaining wall is proposed at the northeast corner of the bridge structure. Based on the plan / profile provided, it is understood that the wall height will not extend the full height of the embankment.

The subsoils encountered in Borehole 3 put down at the northeast corner of the structure during the present investigation consist of fill, dense sandy silt till and hard clayey silt till overlying interbedded shale and limestone bedrock. The fill, where encountered, generally consists of very stiff to hard clayey silt / silty clay, with sand and trace gravel to a depth of about 4 m. The 1.6 m thick upper till layer consists of dense to very dense sandy silt containing trace clay and gravel. The hard clayey silt / silty clay till with sand and trace gravel is underlain by the bedrock. The groundwater table is at about Elevation 103.8 m.

The base of the retaining walls is expected to be founded on either the dense sandy silt till or the hard clayey silt till or will be founded higher up within the embankment fill. It is understood that a mechanically reinforced soil retaining wall system is being considered for the retaining wall.

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. 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.

All topsoil and loose / soft materials should be removed prior to placing the granular for the reinforced soil system. For the reinforced earth mass founded on the dense sandy silt till or the hard clayey silt till, a factored geotechnical resistance at ULS of 600 kPa may be assumed for design. The corresponding geotechnical resistance at SLS may be taken as 400 kPa. Assuming proper placement and compaction of the fills used for widening / raising construction, the above geotechnical resistances may also be used for design of the RSS wall if founded within the embankment fill.

5.5 Embankments

The proposed Guelph Line alignment requires widening to tie into the existing Guelph Line embankments which are about 8 m to 9 m in height.

5.5.1 Embankment Design

The embankment subgrade soils consists of loose to very dense sandy silt to hard clayey silt till or very dense, wet sandy silt 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.

5.5.2 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 both sides 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.

5.6 Excavations and Temporary Cut Slopes

Excavations for footing construction will extend through silty clay / clayey silt fill and till deposits consisting of clayey silt and sandy silt. At the proposed bridge locations the excavations for the spread footings at the abutments will be up to about 11 m in depth below existing Guelph Line grade and about 2 m below the proposed grade for the QEW. Cobbles and boulders are inherent in the glacial deposits as encountered at this site and should be expected during excavation. The excavation bases will be up to 2.5 m below the groundwater level as measured in the piezometers. 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 a fully braced excavation. The excavation for spread footings or pile cap construction in the median will have to be made with vertical supported sides to minimize disruption to traffic on the QEW. Roadway protection for Guelph Line will also be required adjacent to existing bridge abutments during construction of the new widened structure.

Water seepage into the excavations through the fill and till deposits is expected to be minor, except during periods of sustained precipitation. Pumping from well-filtered sumps located at the base of the excavation within the glacial till should provide adequate groundwater control during foundation excavations. The consideration 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 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 roadway protection 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:

$$p = K_a \gamma H$$

where

H = the height of the excavation at any point in metres
K_a = 0.3 for level ground behind excavation
γ = soil unit weight = 20 kN/m^3

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

$$p = K \gamma H$$

where

H = the height of the excavation
K = 0.3 for level ground behind excavation
γ = soil unit weight = 20 kN/m^3

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 / very dense 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 at or below the groundwater level. The soil unit weight should be taken as 20 kN/m^3 and the unit weight of water should be taken as 9.8 kN/m^3. A groundwater level at Elevation 104.7 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 personnel. 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.

Anne S. Poschmann, P.Eng.
Principal

Fintan J. Heffernan, P.Eng.
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

II. PENETRATION RESISTANCE

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

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

PH: Sampler advanced by hydraulic pressure
PM: Sampler advanced by manual pressure
WH: Sampler advanced by static weight of hammer
WR: Sampler advanced by weight of sampler and rod

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

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

(b) Cohesive Soils

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

IV. SOIL TESTS

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

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

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LIST OF SYMBOLS

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

I. GENERAL

π = 3.1416
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_α 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 $= \sigma'_p/\sigma'_{vo}$

(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

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 1

METRIC

PROJECT 991-1105 (3000)
W.P. 47-88-00
LOCATION N 480156.5; E 280194.1
ORIGINATED BY SEP
DIST HWY QEW
BOREHOLE TYPE 114mm SOLID STEM AUGERS
COMPILED BY SEP
DATUM Geodetic
DATE July 29, 2000
CHECKED BY ASP

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
107.46	GROUND SURFACE					
0.00	Silty Clay with sand and organics, trace gravel, Very stiff, Dark brown, Moist (Fill)	1	50 DO	25		
106.86 / 0.60	Silty Sand, trace clay, trace gravel, trace organics, Compact, Dark brown, Moist (Fill)	2	50 DO	22	o	
105.96 / 1.50	Clayey Silt, trace sand and gravel, trace organics, Firm to stiff, Grey, Moist (Fill)	3	50 DO	9		
		4	50 DO	18	o	
104.46 / 3.00	Sandy Silt, some gravel, trace clay, Compact to very dense, Red-brown, Moist (Glacial Till)	5	50 DO	24		
		6	50 DO	130	o	11 29 53 7
102.86 / 4.60	Red-brown, moderetly weathered, weak to very weak, slightly to moderatley weathered SHALE with occasional grey limestone/siltstone interbeds. (QUEENSTON FORMATION) Bedrock cored from 4.7 m to 8.9 m depth. For bedrock coring details refer to Record of Drillhole 1.	7	50 DO	80/15		
98.56 / 8.90	END OF BOREHOLE					

Elevation scale: 107, 106, 105, 104, 103, 102, 101, 100, 99

Notes:
1. Water level measured in piezometer at 2.75 m depth (Elev.104.71 m) after installation.
2. Water level measured in piezometer at 2.9 m depth (Elev.104.56 m) on August 16, 2000.
3.Water level measured in piezometer at 3.5 m depth (Elev.104.0 m) on September 7, 2000.

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

ON_MOT 991-1105.GPJ ON_MOT.GDT 25/9/00

RECORD OF DRILLHOLE: 1

PROJECT: 991-1105 (3000)
LOCATION: N 4801156.5; E 280194.1
INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: July 29, 2000
DRILL RIG: CME 55
DRILLING CONTRACTOR: MASTER SOILS INVESTIGATION

SHEET 1 OF 1
DATUM: Geodetic

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

DEPTH SCALE METRES	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
	Continued from previous page	102.89			
5	Red-brown, weak to very weak, slightly to moderately weathered SHALE (QUEENSTON FORMATION). Occasional grey, moderately strong, slightly weathered limestone to calcareous siltstone interbeds. Bedding joints tend to occur at shale/limestone interbed contacts.	4.57	1		
6			2	B,R,UE B,R,PL	
7				B,R,UE B,PL,SM	
8			3	B,PL,SM B,PL,SM B,PL,SM	
		98.56 8.90		B,PL,UE B,PL,SM	
9	END OF BOREHOLE				
10					
11					
12					
13					
14					

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: SB
CHECKED: ASP

DRILLHOLE 1105ROCK.GPJ GLDR_CAN.GDT 25/9/00 MMZ

RECORD OF BOREHOLE No 2

METRIC

PROJECT 991-1105 (3000)
W.P. 47-88-00 LOCATION N 4801133.0; E 280174.4
DIST HWY QEW BOREHOLE TYPE 114mm SOLID STEM AUGERS
DATUM Geodetic DATE July 19, 2000

ORIGINATED BY SEP
COMPILED BY SEP
CHECKED BY ASP

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE
112.82	GROUND SURFACE					
112.80 / 0.00	Asphalt					
112.42 / 0.40	Crushed Gravel, some sand, Light brown, Dry (Fill)					
112.07 / 0.75	Sand and Gravel, trace silt, Light brown, Dry (Fill)					112
	Clayey Silt with sand, some gravel, Stiff to hard, Red-brown to grey, Moist (Fill)	1	50 DO	29		
		2	50 DO	17		111
		3	50 DO	16		110
		4	50 DO	13		109
		5	50 DO	44		108
						107
		6	50 DO	17		106
	Trace wood fragments at 7.7 m depth.	7	50 DO	45		105
104.72 / 8.10	Sandy Silt, trace clay, trace gravel, pockets of silty clay, Very dense, Red-brown, Wet				∇	104
		8	50 DO	60/.15		103
102.72 / 10.10	Weathered Shale bedrock with hard limestone layers inferred from drilling. (QUEENSTON FORMATION)	~~9~~	~~50 DO~~	~~100/.05~~		102
						101
100.62 / 12.20	END OF BOREHOLE					

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

Notes:
1. Water level measreud in open borehole at 8.55 m depth (Elev.104.27m).

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

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 3

METRIC

PROJECT 991-1105 (3000)			
W.P. 47-88-00	LOCATION N 4801218.4; E 280132.4		ORIGINATED BY SEP
DIST	HWY QEW	BOREHOLE TYPE 114mm SOLID STEM AUGERS	COMPILED BY SEP
DATUM Geodetic	DATE September 7, 2000		CHECKED BY ASP

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)
107.96	GROUND SURFACE				
0.00	Boulder at ground surface Clayey Silt with sand, trace gravel Very stiff to hard Brown, mottled Dry to moist (Fill)	1	50 DO	32	
		2	50 DO	20	o (≈13)
		3	50 DO	64	
105.56 / 2.40	Sandy silt, trace clay, trace gravel, occasional pockets of clayey silt Dense tto very dense Red-brown Moist (Glacial Till)	4	50 DO	43	o (≈15)
		5	50 DO	60/.15	
103.96 / 4.00	Clayey silt to silty clay with sand, trace gravel Hard Red-brown Moist (Glacial Till)	6	50 DO	85	o, W_p–W_L
102.26 / 5.70	Red-brown, weak, slightly to moderatley weathered SHALE with occasional grey (10mm to 60m thick) limestone/siltstone interbeds. (QUEENSTON FORMATION) Bedrock cored from 5.7 m to 8.4 m depth. For bedrock coring details refer to Record of Drillhole 3.	~~7~~	~~50 DO~~		
99.56 / 8.40	END OF BOREHOLE Notes: 1. Water level measured in open borehole prior to rock coring at 5.2 m depth (Elev.102.8 m) 2. Water level measured in piezometer at 4.2 m depth (Elev. 103.8 m) on Sept.22, 2000.				

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE (107, 106, 105, 104, 103, 102, 101, 100); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X REMOULDED, 20 40 60 80 100; PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

PROJECT: 991-1105 (3000)

RECORD OF DRILLHOLE: 3

SHEET 1 OF 1

LOCATION: N 4801218.4; E 2801332.4

DRILLING DATE: August 15-16, 2000

DATUM: Geodetic

DRILL RIG: CME 55

INCLINATION: -90° AZIMUTH: —

DRILLING CONTRACTOR: MASTER SOILS INVESTIGATION

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	COLOUR % RETURN / FLUSH	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY k, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		Continued from previous page		102.26												
6		Red-brown, weak, slightly to moderately weathered massive to thinly bedded SHALE (QUEENSTON FORMATION). Occasional grey, 10mm to 60mm thick siltstone interbeds. Bedding joints tend to occur at shale/siltstone interbed contacts. A 25 mm thick red-brown clay seam was encountered at 6.0 m depth.		5.70	1											
7					2											
8				99.56 8.40	3											
		END OF BOREHOLE														
9																
10																
11																
12																
13																
14																
15																

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: SEP

CHECKED: ASP

RECORD OF BOREHOLE No 4

METRIC

PROJECT 991-1105 (3000)
W.P. 47-88-00 LOCATION N 4801192.0; E 280113.6 ORIGINATED BY SEP
DIST HWY QEW BOREHOLE TYPE 114mm SOLID STEM AUGERS COMPILED BY SEP
DATUM Geodetic DATE July 18, 2000 CHECKED BY ASP

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
114.08	GROUND SURFACE					
0.00 / 113.78	Asphalt					
0.30	Crushed gravel, some sand, Very dense, Light brown, Dry (Fill)	1	50 DO	30/.20		
113.18 / 0.90	Clayey Silt with sand, some gravel, Firm to hard, Red-brown to grey, Moist (Fill)	2	50 DO	12	o	
		3	50 DO	5		
		4	50 DO	19	o	
		5	50 DO	10		
		6	50 DO	27	o	
	Aspahlt layer 0.22 m thick at 7.8 m depth (Elev.106.28m)	7	50 DO	50/.07		
104.88 / 9.20	Clayey Silt, some sand, some gravel, Hard, Red-brown, Dry (Glacial Till)	8	50 DO	81/.28	o	14 11 52 23
		9	50 DO	50/.12		
103.08 / 11.00	Weathered Shale bedrock (QUEENSTON FORMATION), with 50-75 mm thick hard limestone layers inferred from drilling from 10.7 m to 12.2 m depth.	10	50 DO	70/.12		
101.78 / 12.30	END OF BOREHOLE					

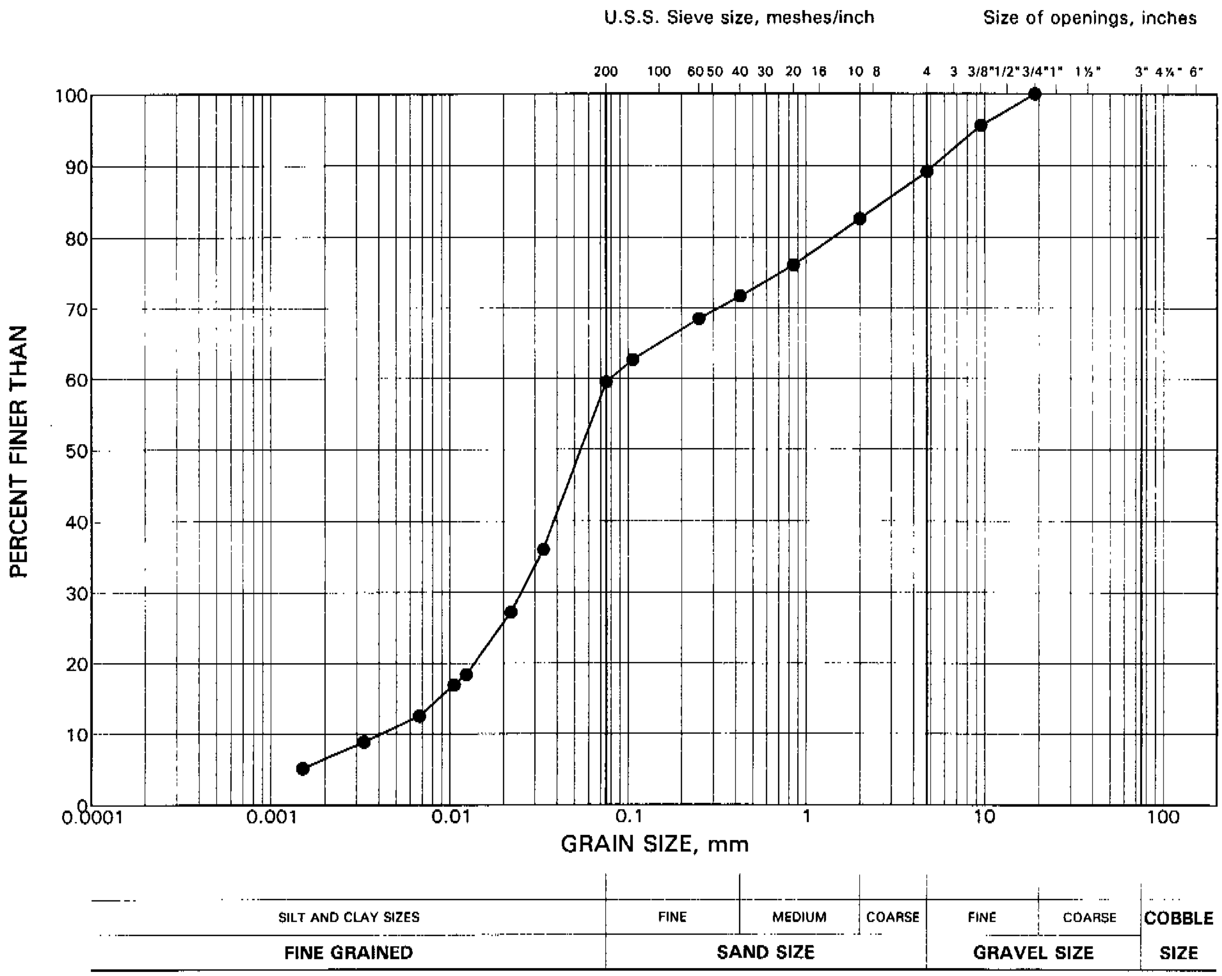
Notes:
1. Open borehole dry upon completion of drilling.

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

OVERSIZE DRAWING(S)

GRAIN SIZE DISTRIBUTION
SANDY SILT (GLACIAL TILL)

FIGURE 1



LEGEND

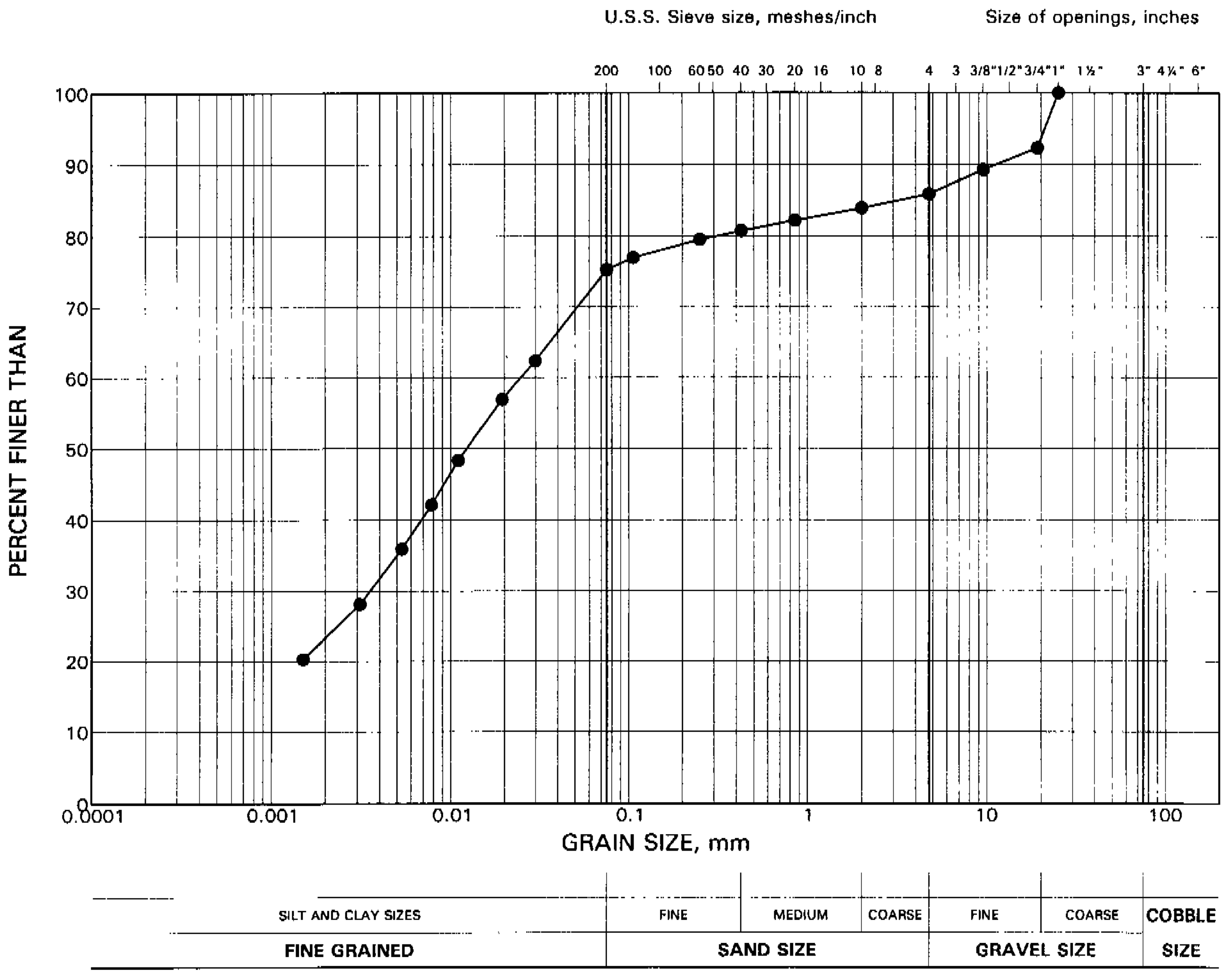
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	1	6	103.2

Project 991-1105A

Golder Associates

GRAIN SIZE DISTRIBUTION
CLAYEY SILT (GLACIAL TILL)

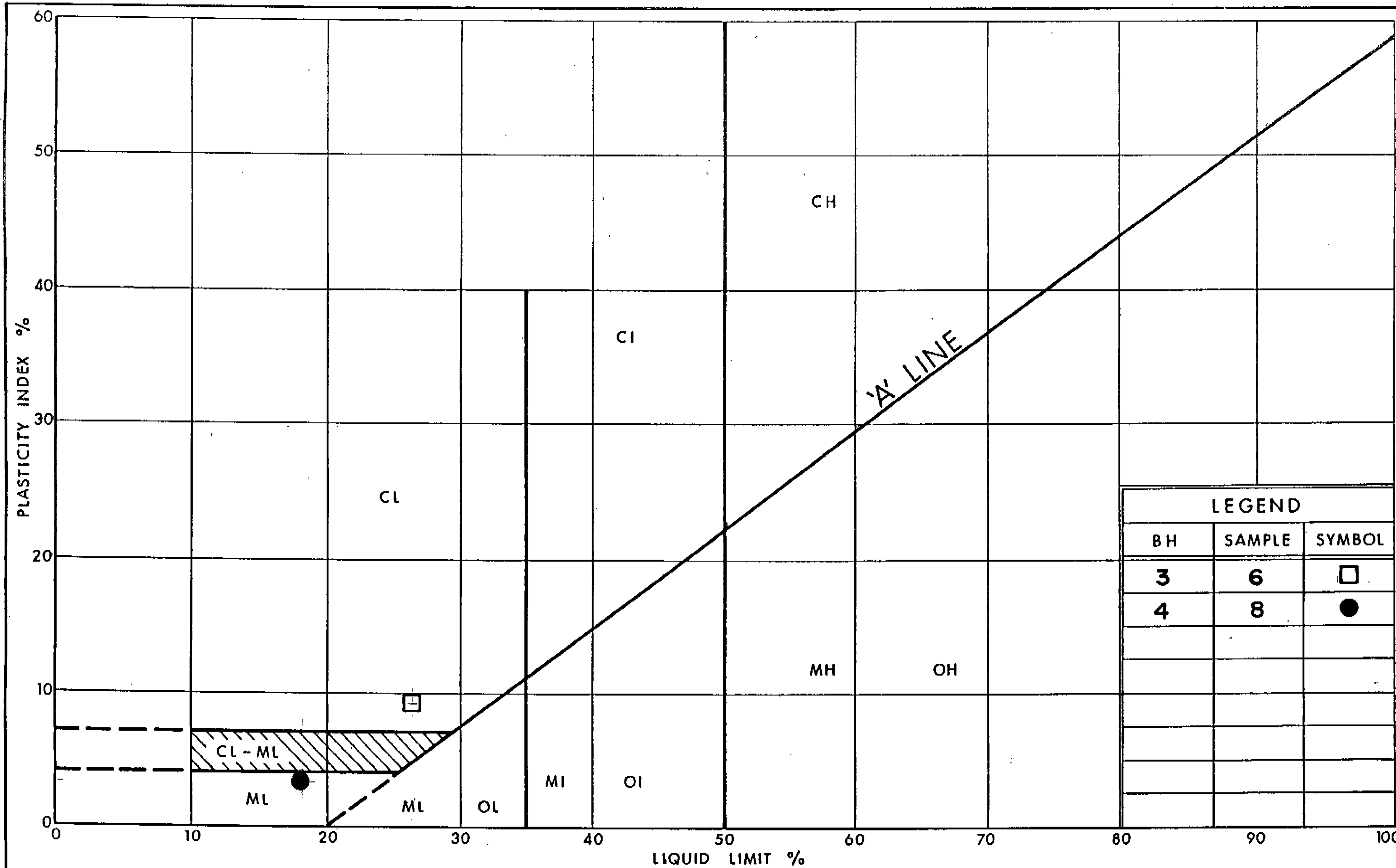
FIGURE 2



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	4	8	104.5

Oct 75, FF-S-21



LEGEND

BH	SAMPLE	SYMBOL
3	6	□
4	8	●

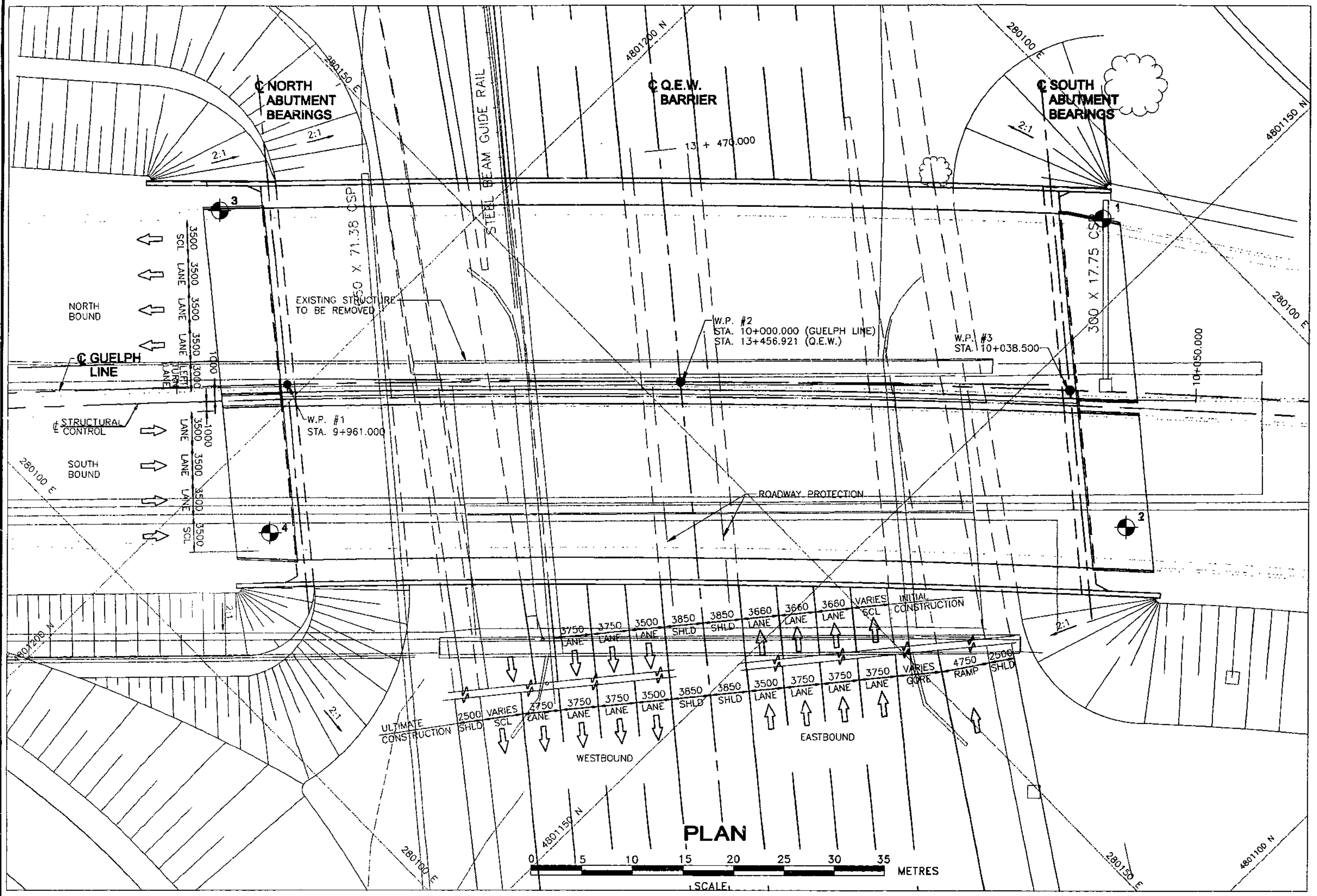
Ministry of Transportation
Ontario

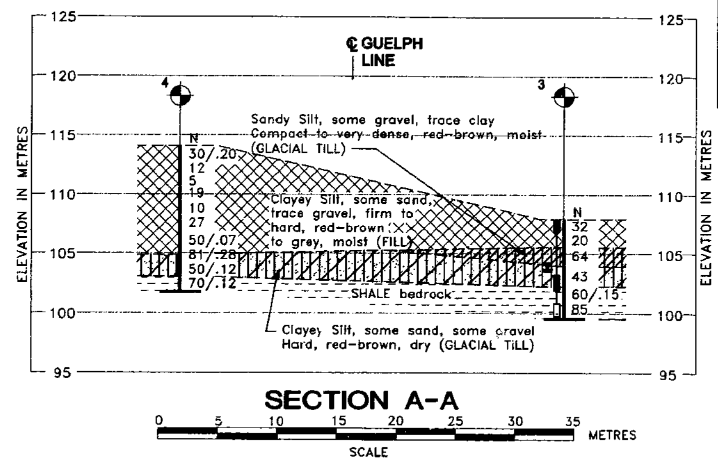
PLASTICITY CHART

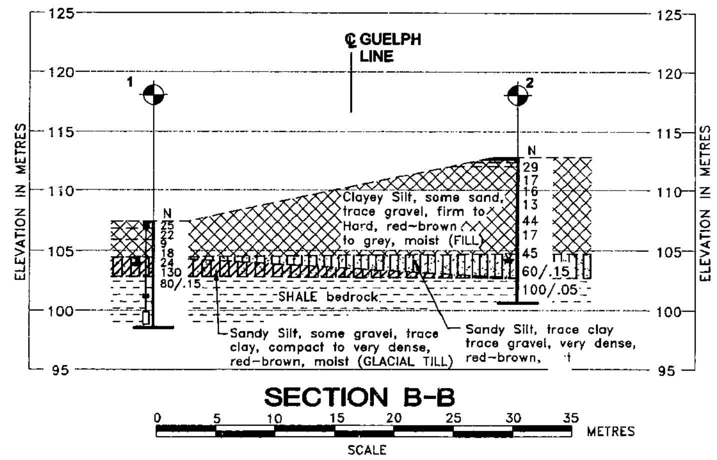
CLAYEY SILT (GLACIAL TILL)

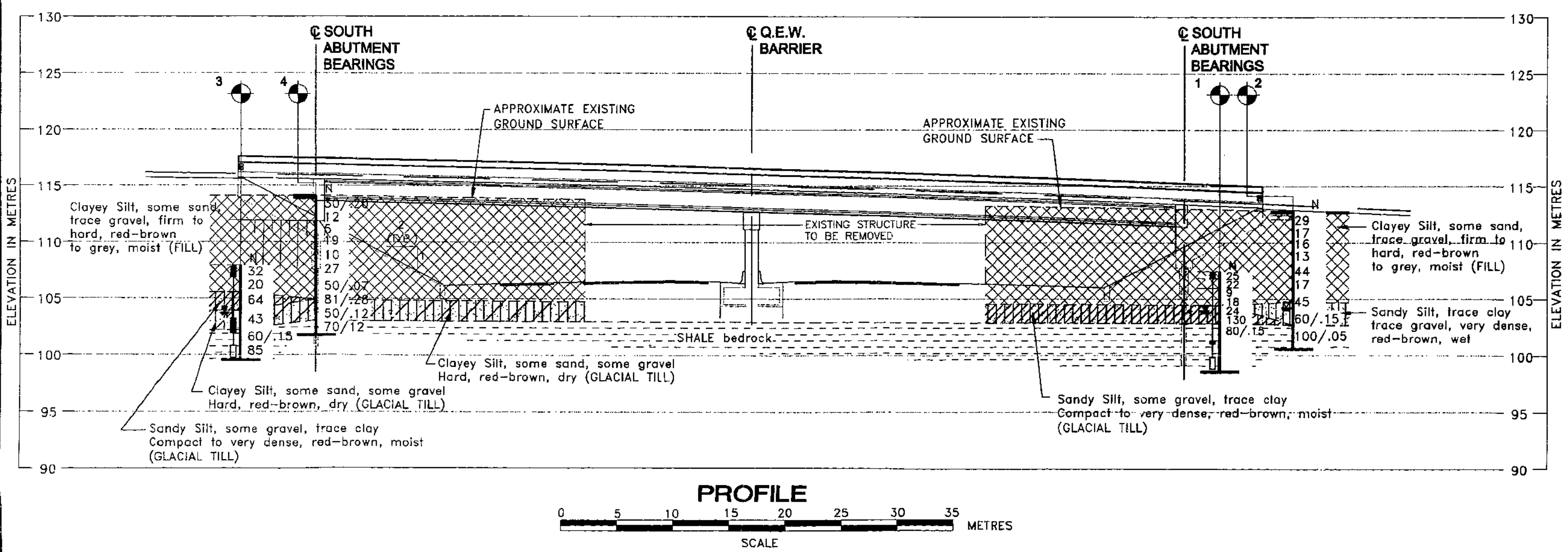
FIG No	3
GWP	47 - 88 - 00

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









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

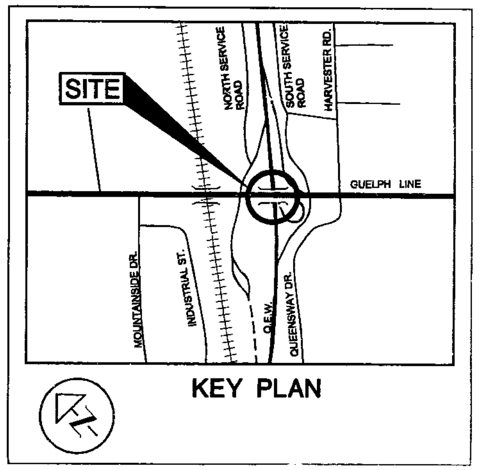
Q.E.W. UNDERPASS AT GUELPH LINE

BOREHOLE LOCATIONS & SOIL STRATA

SHEET

Golder Associates

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 7, 2000
- WL upon completion of drilling

No.	ELEVATION	LOCATION NORTHING	LOCATION EASTING
1	107.46	4801156.5	280194.1
2	112.82	4801133.0	280174.4
3	107.96	4801218.4	280132.4
4	114.08	4801192.0	280113.6

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. 1

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