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**FOUNDATION INVESTIGATION AND DESIGN REPORT
PROPOSED RETAINING WALL
QEW WIDENING BETWEEN GLENDALE AVENUE
AND MOUNTAIN ROAD INTERCHANGES
REGIONAL MUNICIPALITY OF NIAGARA FALLS
G.W.P. 281-99-00**

Submitted to:

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PART A

**FOUNDATION INVESTIGATION REPORT
PROPOSED RETAINING WALL
QEW WIDENING BETWEEN GLENDALE AVENUE
AND MOUNTAIN ROAD INTERCHANGES
REGIONAL MUNICIPALITY OF NIAGARA FALLS
G.W.P. 281-99-00, AGREEMENT NO. 2005-A-000197**

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1.0 INTRODUCTION

Golder Associates Ltd. has been retained by McCormick Rankin Corporation (McCormick Rankin) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services associated with the widening of a 4 km length of the Queen Elizabeth Way (QEW) between the Glendale Avenue and Mountain Road interchanges in Niagara Falls, Ontario. Foundation engineering services are required for the widening of the existing CN Rail bridge over the QEW, the extension and / or replacement of ten structural culverts, new retaining and noise barrier walls, and proposed high mast lighting.

This report addresses the proposed retaining wall along the Niagara-bound QEW lanes, between approximately Stations 11+200 and 11+360 of Niagara Township. A subsurface investigation has been carried out, in which three boreholes were advanced and in-situ and laboratory testing were conducted, to determine the subsurface conditions along the proposed retaining wall.

The terms of reference for the scope of work are outlined in Golder Associates' Proposal No. P01-8048, dated April 2000. The work has been carried out in accordance with Golder Associates' Quality Control Plan for Foundation Design Services, dated April 2000.

2.0 SITE DESCRIPTION

The proposed retaining wall is located along the west side of the Niagara-bound QEW lanes, north of the CN Rail bridge, between approximately Stations 11+200 and 11+360 of Niagara Township. The QEW grade rises southward between these limits, from about Elevation 135 m to 140 m. Six Mile Creek meanders at the toe of the highway embankment in this area, approximately 5 m to 10 m west of the proposed wall alignment. The Six Mile Creek bed is at about Elevation 129.5 m, and the adjacent floodplain varies from about Elevation 130 m to 132 m.

The existing ground surface along the proposed wall alignment is at about Elevation 132 m to 133 m between Stations 11+200 and 11+300. Between Stations 11+300 and 11+360, the existing ground surface along the proposed wall rises steadily up the creek valley slope, from about Elevation 133 m to 138.5 m.

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out at this site on January 29 and 30, 2001, at which time three boreholes were drilled along the proposed wall alignment. Borehole RW-1 was drilled at the south end of the proposed retaining wall alignment, from the top of the bank adjacent to Six Mile Creek, to a depth of about 3 m where bedrock was encountered. Boreholes RW-2 and RW-3 were drilled in the middle and at the north end of the retaining wall alignment, respectively, from the base of the highway embankment immediately adjacent to Six Mile Creek. These boreholes were advanced to about 8 m and 14 m depth; bedrock was encountered at about 8 m depth in Borehole RW-2.

The investigation was carried out using a bombardier-mounted D-50 drill rig, supplied and operated by Master Soil Investigations Ltd. of Weston, Ontario. The boreholes were advanced using solid stem augers. Samples of the overburden and of weathered bedrock, where encountered, were obtained at 0.75 m to 1.5 m intervals of depth using 50 mm outside diameter split-spoon samplers, in accordance with the Standard Penetration Test (SPT) procedure. The water levels in the open boreholes were observed throughout the drilling operations, and a piezometer was installed in two of the boreholes to permit monitoring of the groundwater level at the site.

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

The borehole locations and ground surface elevations were provided by J.D. Barnes Ltd., Ontario Land Surveyors. The borehole elevations are referenced to geodetic datum, and the northing and easting coordinates are referenced to the MTM NAD83 survey system. The borehole locations, together with elevations and northing and easting coordinates, are shown on the attached Drawing 1.

4.0 GENERAL SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geology

The proposed retaining wall site is located along the Niagara Escarpment, which separates the lower Iroquois Plain to the north from the Haldimand Clay Plain physiographic region, located south of the escarpment. In the Niagara region, the escarpment base is located at about Elevation 105 m, and the top reaches about Elevation 190 m. The escarpment itself consists of dolostone, limestone, sandstone and shale bedrock, mantled by relatively thin deposits of silty clay till, sandy silt till, sands, and silts. The depth to bedrock on the escarpment is shallow, varying typically between 1 m and 6 m.

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 and Figures 1 to 4 following the text of this report. The stratigraphic boundaries shown on the borehole records 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 the northern and central portions of the proposed wall alignment, the subsoils consist of highway embankment fill overlying clayey silt and silty sand to sandy silt strata. These alluvial deposits are underlain by a deposit of hard clayey silt till and / or very dense, sand and silt to silt till overlying shale bedrock. At the south end of the retaining wall, which is located above the Six Mile Creek valley, the subsoils consist of hard silty clay till overlying shale bedrock.

A more detailed description of the subsurface conditions encountered in Boreholes RW-1 to RW-3 is provided in the following sections. The locations and elevations of the boreholes, together with the interpreted stratigraphic profile along the proposed wall, are shown on the attached Drawing 1.

4.2.1 Embankment Fill

About 1.2 m of embankment fill was encountered in Borehole RW2, which was drilled near the toe of the highway embankment. The fill consists of clayey silt containing trace to some sand

and gravel, and trace organics, rootlets and asphalt fragments. The fill is very stiff, based on one measured Standard Penetration Test (SPT) 'N' value of 20 blows per 0.3 m of penetration.

4.2.2 Alluvium

A layer of clayey silt containing trace to some sand and gravel was encountered below the embankment fill in Borehole RW-2, and extended from ground surface in Borehole RW-3. These two boreholes were drilled within the Six Mile Creek valley, located in the northern and central portions of the proposed retaining wall alignment; the clayey silt is therefore considered to be an alluvial or floodplain deposit. The encountered clayey silt was 1.7 m and 2.1 m thick, extending to Elevations 129.9 m and 128.1 m in Boreholes RW-2 and RW-3, respectively. The upper 1.4 m of the clayey silt encountered in Borehole RW-3 contained trace to some organics, rootlets, fibres and wood fragments, typical of a floodplain deposit.

An Atterberg limits test carried out on one sample of the clayey silt measured a plastic limit of 18 per cent, a liquid limit of 34 per cent, and a plasticity index of 16 per cent; these test results are plotted on Figure 1, and indicate that the clayey silt is inorganic and of low plasticity. The measured SPT 'N' values ranged from 12 to 28 blows per 0.3 m of penetration, indicating that the clayey silt has a stiff to very stiff consistency.

Below the clayey silt layer in Borehole RW-2, extending between Elevation 129.9 m and 128.4 m, a 1.5 m thick layer of silty sand to sandy silt alluvium is present. The result of a grain size distribution test is shown on Figure 2. The measured SPT 'N' values were 11 and 27 blows per 0.3 m of penetration, indicating that this silty sand to sandy silt alluvium layer has a compact relative density.

4.2.3 Clayey Silt / Silty Clay Till and Sand and Silt to Silt Till

Within the Six Mile Creek valley in the northern and central portions of the proposed retaining wall alignment, a till deposit is present below the alluvium. The surface of the till deposit was encountered at Elevation 128.4 m in Borehole RW-2, and at Elevation 128.1 m in Borehole RW-3. In the boreholes drilled within the Six Mile Creek valley, the encountered till deposit grades in composition from a clayey silt to a sand and silt to a silt, containing some sand. The clayey silt portion of the deposit is of very low plasticity. The results of one Atterberg limits test on a sample of the clayey silt till from Borehole RW-3 are shown on the borehole record and on Figure 3. A plastic limit of 15 per cent and a liquid limit of 19 per cent were measured, resulting in a plasticity index of about 4 per cent. An additional Atterberg limits test was carried out on a

sample of the silt till, as shown on the record for Borehole RW-3; this test indicated that the material is non-plastic. The result of a grain size distribution test on the silt till is shown on Figure 4. The measured SPT 'N' values were in all cases greater than 100 blows per 0.3 m of penetration, indicating that the clayey silt till has a hard consistency, and the sand and silt to silt till has a very dense relative density.

At the south end of the retaining wall, above the Six Mile Creek valley, Borehole RW-1 encountered about 3 m of silty clay till extending from ground surface to Elevation 136.6 m. The silty clay till contained trace to some sand and trace gravel. Atterberg limits testing on one sample of the silty clay till measured a plastic limit of 18 per cent, a liquid limit of 44 per cent, and a plasticity index of 25 per cent. These results, shown on the record of Borehole RW-1 and plotted on Figure 3, indicate that the silty clay material is inorganic and of intermediate plasticity. The measured SPT 'N' values ranged from 55 to 78 blows per 0.3 m of penetration, indicating that this silty clay till has a hard consistency.

4.2.4 Bedrock

Shale bedrock was encountered below the silty clay till in Borehole RW-1, above the Six Mile Creek valley at the south end of the proposed retaining wall. At this location, the surface of the shale was encountered at about Elevation 136.6 m. Within the Six Mile Creek valley, shale bedrock was encountered below the sand and silt till in Borehole RW-2; bedrock was not encountered in Borehole RW-3. The surface of the shale within the creek valley declines from about Elevation 125.5 m in the vicinity of Borehole RW-2, to below Elevation 116.5 m in the vicinity of Borehole RW-3 at the south end of the retaining wall.

The shale was penetrated by augering and split-spoon sampling. The samples recovered consisted of weathered shale of a red-brown colour. Refusal to split-spoon and auger penetration occurred at about Elevation 136.3 m in Borehole RW-1, and at about Elevation 124.4 m in Borehole RW-2. It is inferred that refusal occurred on a harder sandstone layer, likely on the Grimsby Formation which is expected at about this elevation.

4.3 Groundwater Conditions

Water levels were noted in the open boreholes during and upon completion of the drilling operation; these levels are noted on the attached Records of Boreholes. Borehole RW-1, drilled from above the Six Mile Creek valley, was dry on completion of drilling operations. The water

levels in open Boreholes RW-2 and RW-3, drilled within the floodplain adjacent to Six Mile Creek, were at 2.3 m and 1.5 m depth (Elevation 130.5 m and 128.7 m), respectively.

Piezometers were sealed in Boreholes RW-1 and RW-2 to permit monitoring of the groundwater levels at the site. Details of the piezometer installations and the groundwater level measurements are shown on the attached Records of Boreholes. The water levels in the piezometers were measured in February and August 2001, and are summarized in the table below. These measurements indicate that the groundwater level slopes downward from south to north, similar to the topography at the site and that the groundwater level in the floodplain area is slightly above the level in Six Mile Creek.

<i>Borehole Number</i>	<i>February 9, 2001</i>		<i>August 9, 2001</i>	
	<i>Depth to Water</i>	<i>Groundwater Elevation</i>	<i>Depth to Water</i>	<i>Groundwater Elevation</i>
RW-1	1.2 m	138.3 m	Dry	Below 137.3 m
RW-2	2.5 m	130.3 m	2.7 m	130.1 m

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.

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PART B

**FOUNDATION DESIGN REPORT
PROPOSED RETAINING WALL
QEW WIDENING BETWEEN GLENDALE AVENUE
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5.0 ENGINEERING RECOMMENDATIONS

5.1 General

This section of the report provides recommendations on the geotechnical aspects of design of the proposed retaining wall, based on interpretation of the factual information obtained during the subsurface investigation. It should be noted that the interpretation and recommendations are intended for use only by the design engineer. Where comments are made regarding 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.

In order to accommodate the widening of the QEW adjacent to Six Mile Creek, a 3 m to 3.5 m high retaining wall will be required between Stations 11+200 and 11+360 of Niagara Township. The wall will be located along the west side of the Niagara-bound lanes, near the toe of the existing highway embankment. Within the limits of the retaining wall, the existing QEW grade rises southward from about Elevation 135 m to 140 m. The floor of the Six Mile Creek valley varies from about 130 m to 132 m, declining northward. The existing ground surface along the proposed wall alignment is at about Elevation 132 m to 133 m between Stations 11+200 and 11+300. Between Stations 11+300 and 11+360, the existing ground surface along the wall alignment rises up the creek valley slope, from about Elevation 133 m to 138.5 m.

The results of the boreholes put down during the investigation indicate that there is competent till material overlying shale bedrock. However, within the Six Mile Creek valley, clayey silt floodplain deposits and compact, wet silty sand to sandy silt are present above the till. The clayey silt floodplain deposits encountered in Borehole RW3 contain variable amounts of organic matter. This borehole was put down about 10 m away from the proposed wall alignment which extends partway up the existing embankment slope at this location. It is feasible that fill materials are present overlying the floodplain deposits at the actual wall alignment.

Based on these subsurface conditions, consideration could be given to the use of a retained soil system (RSS) wall at this site. Alternatively, a cantilever retaining wall could be adopted, supported on spread footings on the hard till at the south end of the wall; in the central and northern portions of the wall, the footings would have to be stepped down to reach the very dense till. To reduce the excavation requirements within the Six Mile Creek valley, a third possible alternative would be a soldier pile and concrete panel retaining wall, in which the soldier piles would be extended through the floodplain deposits into the very dense till.

5.2 Shallow Foundations

Recommendations for shallow foundations for RSS and cantilever retaining wall systems at this site are provided in the following subsections. Spread footings for concrete cantilever walls must be placed on inorganic native soils below any fill and, within the Six Mile Creek valley, below any floodplain deposits which contain organic matter. In this regard, it should be noted that significant excavation below the existing ground surface may be required in order to reach the recommended founding levels, particularly for the cantilever wall option within the Six Mile Creek valley. The RSS wall is generally more tolerant of settlement and consideration could be given to supporting the wall on the fill / floodplain deposits thereby reducing the excavation requirements.

5.2.1 Geotechnical Resistance – Retained Soil System (RSS) Wall

A mechanically-reinforced soil retaining wall system (retained soil system or RSS wall) consists of granular fill placed and compacted in layers, and reinforced with metal or fabric strips or grids. A facing material, typically pre-cast concrete panels mechanically fastened to the reinforcing strips or grids, is used to form the face of the reinforced soil structure and to prevent the loss of fill material.

Use of this system is considered appropriate for the proposed 3 m to 3.5 m high retaining wall adjacent to Six Mile Creek. The factored geotechnical resistance at Ultimate Limit States (ULS) and the geotechnical resistance at Serviceability Limit States (SLS) are given in the following table, for two alternative founding conditions. The first alternative is for placing the RSS wall directly on the existing floodplain and fill deposits and accepting the total and differential settlements which may occur. Since the fill materials and organic content within the floodplain deposits are variable, it is not possible to quantify the magnitude of settlement which could occur. It is expected, however, that the total settlement would not exceed 75 mm for the proposed wall heights.

Since removal of the organic deposits may be required as part of the embankment widening, the second alternative involves subexcavation of the organic deposits within the floodplain and the fill materials, and replacement with compacted granular fill placed to raise the grade to the wall founding level. The subgrade should be inspected by qualified geotechnical personnel prior to placement of the granular materials.

	<i>Station</i>	<i>Founding Soils</i>	<i>Factored Geotechnical Resistance at ULS</i>	<i>Geotechnical Resistance at SLS</i>
Alternative 1	11+200 to 11+325	Fill and clayey silt floodplain deposits at Elevation 132 m to 134 m (existing ground surface)	150 kPa	X
	11+325 to 11+360	Silty clay till (variable elevation)	200 kPa	200 kPa
Alternative 2	11+200 to 11+260	Compacted granular fill on stiff clayey silt	200 kPa	150 kPa
	11+260 to 11+325	Compacted granular fill on very stiff clayey silt	200 kPa	200 kPa
	11+325 to 11+360	Hard silty clay till	300 kPa	500 kPa

The design parameters for the founding soils along the RSS wall are given in the table below, where:

- c' is the apparent effective cohesion (kPa);
 N' is the effective angle of internal friction ($^{\circ}$); and
 γ is the total unit weight of the soil (kN/m^3).

<i>Station</i>	<i>Soil Type</i>	<i>Founding Soils Design Parameters</i>			<i>Coefficient of Friction (Sliding)</i>	<i>Design Groundwater Level</i>
		c'	ϕ'	γ		
11+200 to 11+260	Stiff clayey silt	X	28	19	0.40	130 m
11+260 to 11+325	Clayey silt fill or very stiff clayey silt	X	30	21	0.45	131 m
11+325 to 11+360	Hard silty clay till	X	30	21	0.45	134 m to 136 m

The internal stability of the mechanically-reinforced soil walls should be checked by the RSS supplier / designer. The Factor of Safety related to global stability for properly designed and constructed RSS walls at this site will be greater than 1.3.

5.2.2 Geotechnical Resistance - Concrete Cantilever Walls

The following table provides maximum founding elevations, factored geotechnical resistances at ULS and geotechnical resistances at SLS for the soil types present along the proposed wall. It should be noted that the SLS capacities provided are for a total settlement of 25 mm. A minimum of 1.2 m of earth cover should be provided for protection against frost penetration.

<i>Station</i>	<i>Maximum Founding Elevation</i>	<i>Founding Soil Conditions</i>	<i>Assumed Footing Width</i>	<i>Factored Geotechnical Resistance at ULS</i>	<i>Geotechnical Resistance at SLS</i>	<i>Coefficient Of Friction (Sliding)</i>
11+200 to 11+260	128 m	Hard clayey silt till	3 m	850 kPa	550 kPa	0.55
11+260 to 11+325	130 m – 131 m	Very stiff clayey silt	3 m	350 kPa	200 kPa	0.4
11+325 to 11+360	133 m – 138 m	Hard silty clay till	3 m	350 kPa	350 kPa	0.4

The geotechnical resistances provided are given for loads that will be applied perpendicular to the surface of the footings. The inclination of the load should be taken into account in accordance with OHBDC (Section 6-8.4.2).

Resistance to lateral forces / sliding resistance between the concrete footing and the subsoils should be calculated in accordance with Section 6-8.4.3 of the OHBDC. The coefficients of friction, $\tan \delta$, are given in the above table.

5.3 Deep Foundations

Because of the presence of the relatively weak surficial (floodplain) soils in the creek valley, consideration could be given to the use of a soldier pile and concrete panel wall supported on augered piles, to minimize the extent of excavation required for spread footing construction.

5.3.1 Geotechnical Resistance - Caissons

A soldier pile and concrete panel wall would be supported on steel H-piles founded within the very dense sand and silt to silt till which occurs below about Elevation 127 m to 128.5 m within the Six Mile Creek valley. Outside the floodplain area, if the same type of wall is used, the soldier piles would be founded within the silty clay till and bedrock. Given the relatively low height of the wall (a maximum of 3 m to 3.5 m), it is anticipated that the wall can be cantilevered without a requirement for anchors. Suitable drainage and insulation should be provided to the back of this type of wall.

For design, the factored axial resistance at Ultimate Limit States (ULS) for steel piles socketted in concrete within the very dense sand and silt to silt till with a base Elevation of 124 m may be taken as 850 kN, assuming a 0.76 m diameter socket. The geotechnical resistance at SLS may be taken as 850 kN. The same axial resistances may be used for steel piles socketted within the silty clay till / shale bedrock with base at least 4 m below the base of the wall.

The coefficients of passive lateral earth pressure, K_p , are provided in the following table for determining the lateral resistance of the soldier piles. The passive toe restraint to the soldier piles may be determined using a triangular earth pressure distribution, acting over an equivalent width equal to three times the pile socket diameter.

<i>Location</i>	<i>Soil Type</i>	<i>K_p</i>
11+200 to 11+325	Fill	3.0
	Stiff to very stiff clayey silt	2.8
	Clayey silt till	4.2
	Sand and silt to silt till	4.2
11+325 to 11+360	Silty clay till	3.3
	Shale bedrock	4.2

The caissons will be extended below the groundwater level at the site. Consequently, precautions will be required to control groundwater seepage into the caisson hole and to ensure that the founding soils are not disturbed.

5.4 Lateral Earth Pressures

The lateral pressures acting on the proposed retaining wall will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill, on the freedom of lateral movement of the wall, and on the drainage conditions behind the wall. The following recommendations are made concerning the design of the proposed retaining wall, in accordance with the OHBDC:

- Select free-draining granular fill meeting the specifications of OPSS Granular 'A' or Granular 'B' but with less than 5 per cent passing the 200 sieve should be used as backfill behind the wall. This fill should be compacted in loose lifts not greater than 200 mm in thickness to 95 per cent of the material's Standard Proctor maximum dry density in accordance with OPSS 501. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill.
- A compaction surcharge equal to 16 kPa should be included in the lateral earth pressures for the structural design of the wall in accordance with OHBDC Figure 6-7.4.3. Compaction equipment should be used in accordance with OPSS 501.06.
- 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 from OHBDC Figure 6-7.4.1) or within the wedge-shaped zone defined by a line drawn at 1.2 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (Case II from OHBDC Figure 6-7.4.4).

- 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/m ³
Coefficients of lateral earth pressure:	
Active, K_a	0.35
At rest, K_o	0.50

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

	<i>Granular 'A'</i>	<i>Granular 'B' Type 2</i>
Soil unit weight:	22 kN/m ³	21 kN/m ³
Coefficients of lateral earth pressure:		
Active, K_a	0.27	0.31
At rest, K_o	0.43	0.47

- If the wall support allows lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the wall support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design.

It should be noted that the above design recommendations and parameters assume level backfill and ground surface behind the retaining wall. Where there is sloping ground behind the wall, the coefficient of lateral earth pressure must be adjusted to account for the slope.

5.5 Design and Construction Considerations

5.5.1 Excavation

At the proposed retaining wall location, excavation for shallow spread footings will extend at least 1.2 m below the proposed grade, and up to 3 m below the existing ground surface in the Six Mile Creek valley in order to reach suitable founding soils. The required excavation depth will depend on the type of wall proposed, and on the consequent distance of the cut into the existing embankment side slope.

Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act for Construction Activities. Generally, temporary open-cut slopes should be maintained no steeper than 1 horizontal to 1 vertical (1H:1V) in the native material. For temporary open-cut slopes through fill and floodplain materials, side slopes should be maintained no steeper than 1.5H:1V. Where space restrictions dictate, it may be necessary to carry out the cut within a braced excavation.

The soils comprising the excavation subgrade will be sensitive to disturbance from ponded water, construction traffic and frost. All foundation excavations should be inspected by experienced geotechnical personnel prior to fill or concrete placement to ensure that soil having adequate bearing capacity has been reached and that the bearing surfaces have been properly prepared. Provision should be made to sub-excavate below founding level where unsuitable subgrade soils are encountered, and to replace the removed material with compacted granular soil or lean mix concrete. In addition, provision should be made for placement of a mud coat once the subgrade has been inspected and approved.

5.5.2 Dewatering

The portion of the wall between about Station 11+200 and 11+325 will be located within the Six Mile Creek valley, adjacent to the creek. A dewatering scheme may be necessary in this area, depending upon the creek water level at the time of construction, to control the water flow and allow footing construction in dry conditions.

Groundwater seepage into shallow footing excavations will occur through the floodplain deposits and clayey silt where the excavations are extended below the groundwater level. It is expected that the quantity of seepage from this source can be handled by pumping from properly filtered sumps or a filtered drain placed at the base of the excavation. Greater quantities could occur if water-bearing granular interlayers are intercepted within the floodplain deposits, such as the silty sand alluvium encountered in Borehole RW-2.

As discussed in Section 5.3.1, deep foundations at this site would be extended below the groundwater level in the creek valley, through cohesionless alluvial deposits at some locations and into till deposits. The use of a liner, likely in conjunction with drilling mud, will be required to control groundwater seepage into the caisson hole and to ensure that the founding soils are not disturbed.

GOLDER ASSOCIATES LTD.

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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

	c_u, s_u	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

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.

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 stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

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

PROJECT 001-1127F				RECORD OF BOREHOLE No RW1				1 OF 1		METRIC		
W.P. 281-99-00				LOCATION N 4,778,671; E 333,762				ORIGINATED BY GM				
DIST Central HWY QEW				BOREHOLE TYPE 108mm dia. Solid Stem Augers				COMPILED BY LCC				
DATUM Geodetic				DATE January 29, 2001				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 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x REMOULDED		WATER CONTENT (%)		
139.5 0.0	GROUND SURFACE Silty Clay, trace to some sand, trace gravel (Till) Hard Brown to reddish brown Moist		1	SS	78							
			2	SS	70							
			3	SS	55							
136.6 3.2	Shale (Bedrock) Weathered Red-brown END OF BOREHOLE Refusal to split-spoon and auger penetration at 3.2m depth (Elev. 136.3m) Notes: 1. Borehole dry on completion of drilling. 2. Water level in piezometer at 1.2m depth (Elev. 138.3m) on February 9, 2001. 3. Piezometer dry on August 9, 2001.		4	SS	35/15							

PROJECT 001-1127F				RECORD OF BOREHOLE No RW2				1 OF 1		METRIC					
W.P. 281-99-00				LOCATION N 4,778,789; E 333,710				ORIGINATED BY GM							
DIST Central HWY QEW				BOREHOLE TYPE 108mm dia. Solid Stem Augers				COMPILED BY LCC							
DATUM Geodetic				DATE January 29, 2001				CHECKED BY ASP							
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED							
132.8 0.0	GROUND SURFACE Clayey Silt, trace to some sand and gravel, trace organics, rootlets, asphalt (Fill) Very stiff Brown to black Moist		1	SS	20										
131.6 1.2	Clayey Silt, trace to some sand, trace gravel Very stiff Brown Moist		2	SS	26										
			3	SS	28										
129.9 2.9	Silty Sand to Sandy Silt, trace gravel Compact Red-brown Moist to wet		4	SS	11										
			5	SS	27										
128.4 4.4	Sand and Silt, trace gravel, trace clay (Till) Very dense Red-brown Moist		6	SS	56/08										
			7	SS	50/08										
125.5 7.3	Shale (Bedrock) Weathered Red-brown		8	SS	102/13										
124.4 8.4	END OF BOREHOLE Auger Refusal Notes: 1. Water level in open borehole on completion of drilling at 2.3m depth (Elev. 130.5m). 2. Water level in piezometer at 2.5m depth (Elev. 130.3m) on February 9, 2001, and at 2.7m depth (Elev. 130.1m) on August 9, 2001.														

ON_MOT 001-1127.GPJ ON_MOT.GDT 12/9/01

RECORD OF BOREHOLE No RW3

1 OF 2

METRIC

PROJECT 001-1127F

W.P. 281-99-00

LOCATION N 4,778,796; E 333,661

ORIGINATED BY GM

DIST Central HWY QEW

BOREHOLE TYPE 108mm dia. Solid Stem Augers

COMPILED BY LCC

DATUM Geodetic

DATE January 30, 2001

CHECKED BY ASP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
								○ UNCONFINED	+ FIELD VANE						
130.2	GROUND SURFACE						20 40 60 80 100	20 40 60 80 100	20 40 60					GR SA SI CL	
0.0	Clayey Silt, trace to some sand and gravel, trace to some organics, fibres, wood fragments and rootlets Stiff Black/brown Moist		1	SS	12										
128.8	Clayey Silt, trace sand and gravel Stiff Grey-brown Moist		2	SS	12										
128.1	Clayey Silt with sand to some sand, trace gravel (Till) Hard Reddish brown Moist		3	SS	80/15										
			4	SS	95/10										
126.8	Silt, some sand, trace clay and gravel, occasional boulders(Till) Very dense Red-brown Moist to wet		5	SS	65/10								Non-Plastic	1 20 73 6	
3.4			6	SS	90/10										
			7	SS	75/08										
	Contains clayey silt till interlayers.		8	SS	101/15										
			9	SS	102/15										
			10	SS	85/15										
			11	SS	100/15										
	Augers grinding from 12.8m to 13.8m depth.		12	SS	90/13										
116.4															
13.8															

Contains clayey silt till interlayers.

Augers grinding from 12.8m to 13.8m depth.

Continued Next Page

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

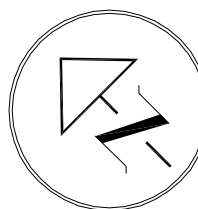
ONL MOT 001-1127.GPJ ONL MOT.GDT 21/12/01

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

ON_MOT 001-1127.GPJ ON_MOT.GDT 21/12/01

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

DIST. HWY. QEW
CONT No.
GWP No. 281-99-00

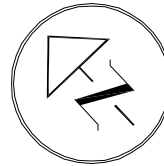
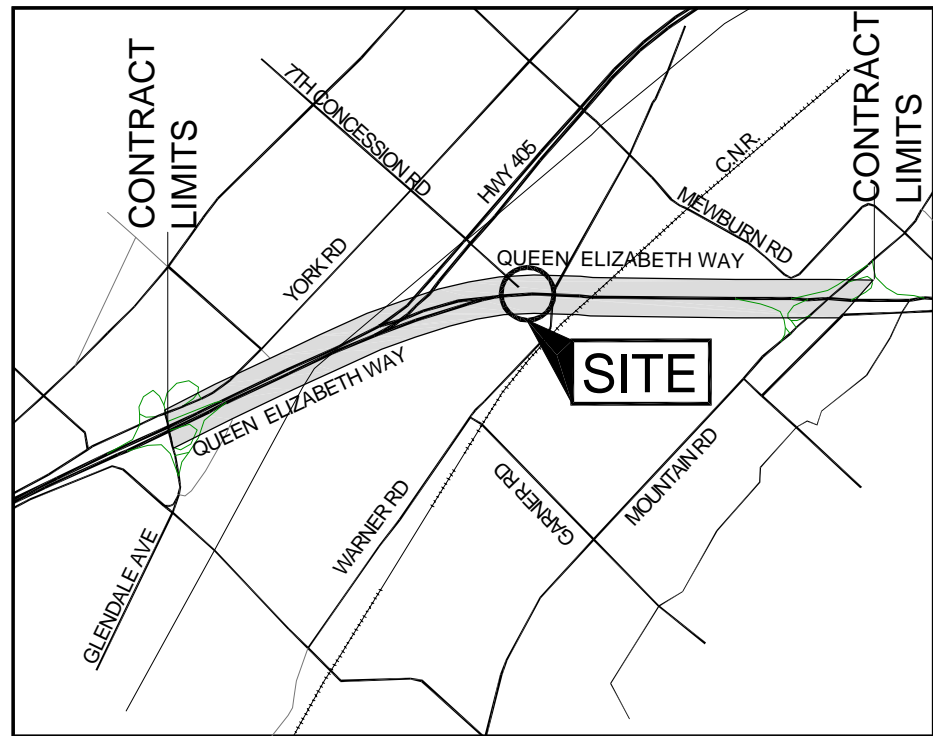


RETAINING WALL
BOREHOLE LOCATIONS

SHEET



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN

LEGEND



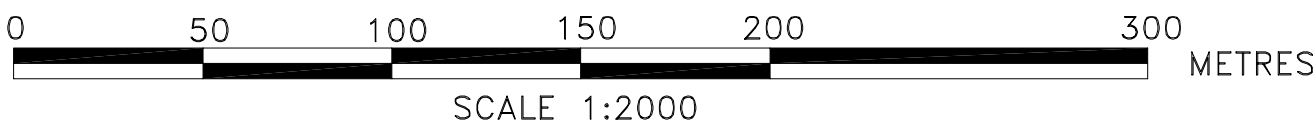
Borehole

No.	ELEVATION	LOCATION	
		NORTHING	EASTING
RW-1	139.5	4778671	333762
RW-2	132.8	4778739	333710
RW-3	130.2	4778796	333661

REFERENCE

This drawing was prepared using Base Map and Proposed Alignment files provided by McCormick Rankin Corporation, dated September 2001.

PLAN



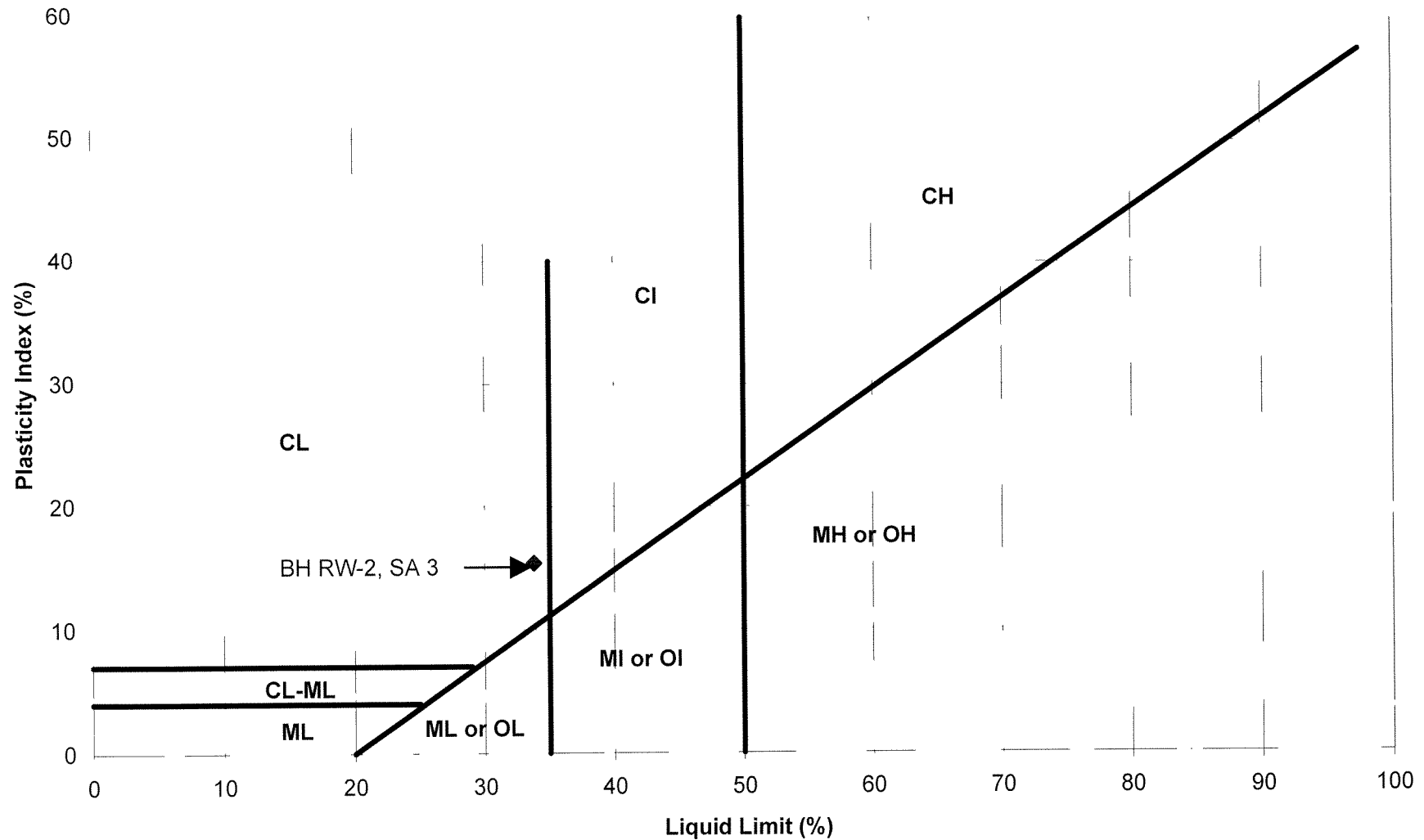
PLOT DATE: February 21, 2003
DRAWN BY: P127F-301/LWS

NO.	DATE	BY	REVISION

Geocres No.			
HWY. QEW	PROJECT NO. 001-1127F-3		DIST. CENTRAL
SUBM'D. LCC	CHKD. ASP	DATE: OCT. 2001	SITE:
DRAWN: JFC	CHKD. LCC	APPD. ASP	DWG. 1

**PLASTICITY CHART
CLAYEY SILT (FLOODPLAIN DEPOSIT)**

FIGURE 1



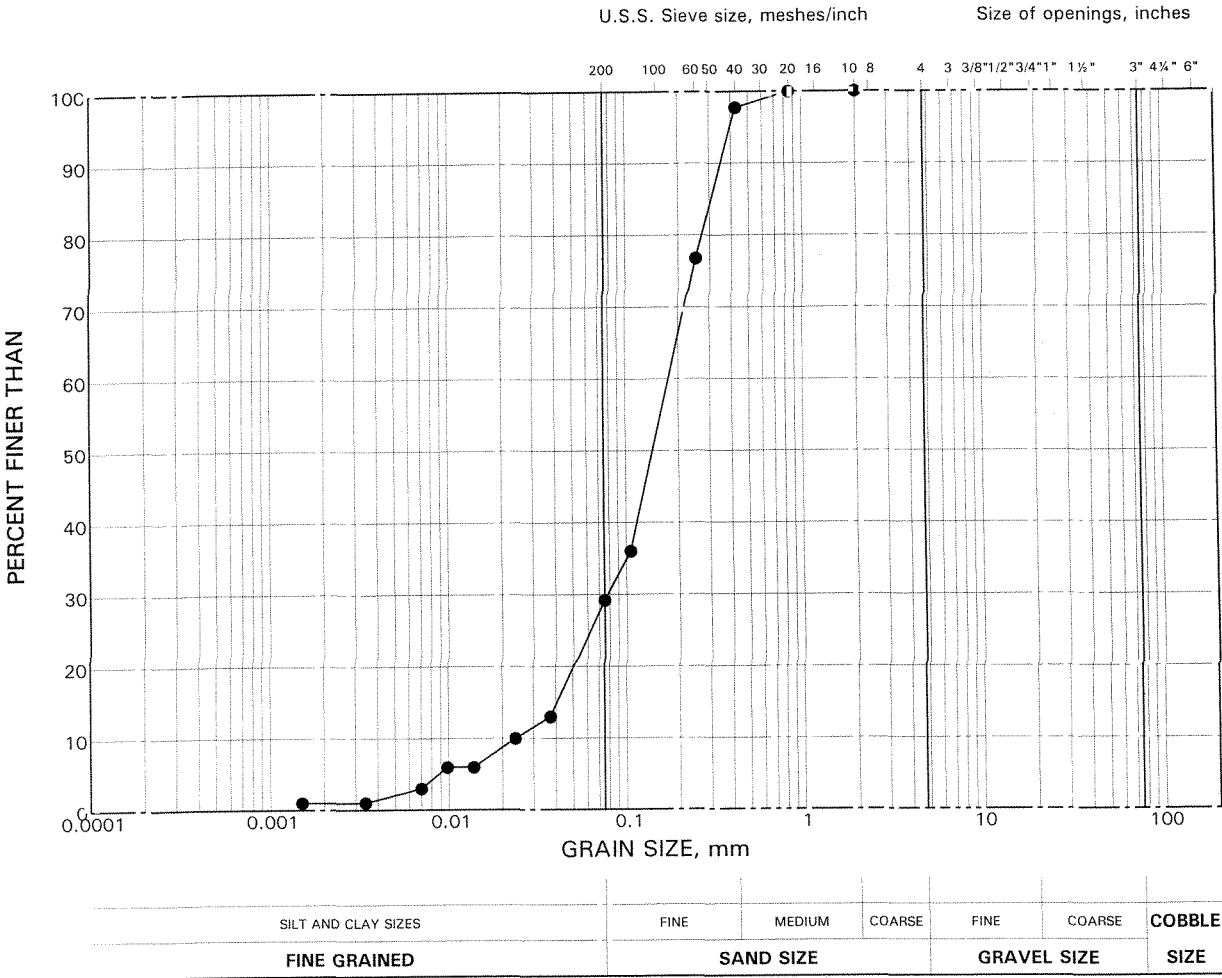
Date: September 2001
Project: 001-1127F-3

Golder Associates

Drawn: LCC
Checked: LCC

GRAIN SIZE DISTRIBUTION Silty Sand (Alluvium)

FIGURE 2

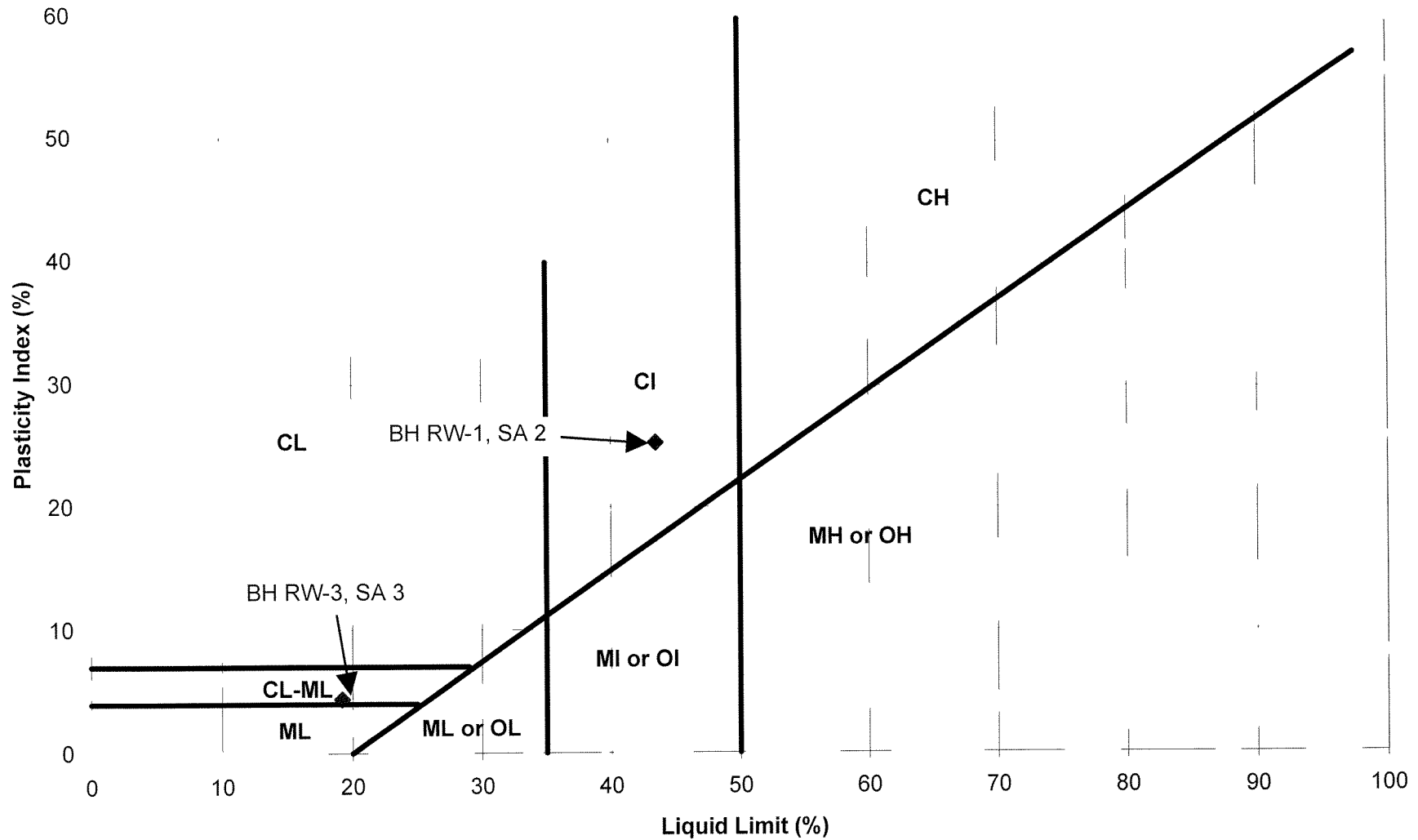


LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
•	RW2	5	128.5

**PLASTICITY CHART
CLAYEY SILT TO SILTY CLAY TILL**

FIGURE 3



Date: September 2001 _____

Project: 001-1127F-3

Golder Associates

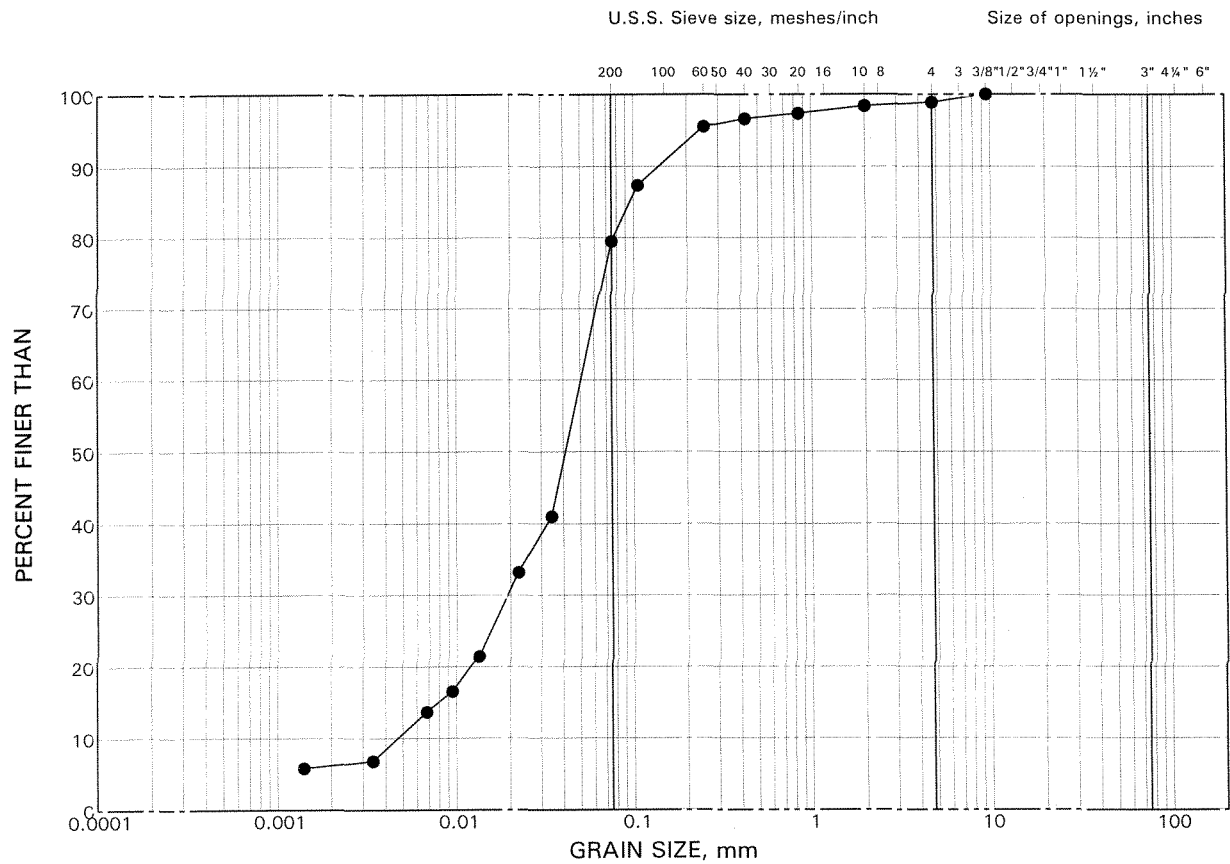
Drawn: LCC

Checked: LCC

GRAIN SIZE DISTRIBUTION

Silt Till

FIGURE 4



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
•	RW3	5	126.3