

**Golder Associates Ltd.**

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



**PRELIMINARY FOUNDATION  
INVESTIGATION AND DESIGN REPORT  
INNISFIL BEACH ROAD OVERPASS  
STRUCTURE SITE 30-210  
HIGHWAY 400 WIDENING FROM 1 KM SOUTH  
OF HIGHWAY 89 TO HIGHWAY 11  
G.W.P. 30-95-00**

Submitted to:

URS Cole, Sherman  
75 Commerce Valley Drive East  
Thornhill, Ontario  
L3T 7N9

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**PART A**

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Figure 1           Grain Size Distribution Test Result, Clayey Silt Till

## **1.0 INTRODUCTION**

Golder Associates Ltd. has been retained by URS Cole, Sherman (Cole, Sherman) on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary foundation engineering services for the ultimate widening of Highway 400 from 1 km south of Highway 89, northerly 30 km to Highway 11, in Simcoe County, Ontario. Foundation engineering services are required for the widening and / or replacement of eighteen existing overpass and underpass structures, as well as five structural culverts.

This report addresses the widening and / or replacement of the existing Innisfil Beach Road (also called Simcoe Road 21 and Thornton Sideroad) overpass structure. A foundation investigation has been carried out, in which two boreholes were advanced and in-situ and laboratory testing was conducted, to determine the subsurface conditions at the site for this preliminary design study.

The terms of reference for the scope of work are outlined in Golder Associates' Proposal No. P01-1192, dated June 2000.

## **2.0 SITE DESCRIPTION**

The existing Innisfil Beach Road overpass structure is located about 9.5 km north of the Highway 89 interchange and about 6 km south of the Molson Park Drive interchange, in the Town of Innisfil, Simcoe County. Innisfil Beach Road is also known locally as Simcoe Road 21 and Thornton Sideroad. The MTO has designated this overpass as Structure Site No. 30-210.

At this existing structure, the Highway 400 grade is at about Elevation 308 m, about 3 m higher than the surrounding grade. Innisfil Beach Road has been constructed in a cut up to about 2.5 m deep, with its grade at about Elevation 302.5 m under Highway 400.

The existing single-span overpass structure was constructed in the early 1950s under Contract 49-65. According to the general layout drawing for this contract, which was provided by Morrison Hershfield (the structural designers for this preliminary study), the abutments are supported on 3.7 m wide spread footings founded below the Innisfil Beach Road grade, at Elevation 300.4 m at the east end, rising to Elevation 300.6 m at the west end of the footings. The associated retaining walls are supported on 1.4 m wide spread footings at the same founding levels.

### **3.0 INVESTIGATION PROCEDURES**

A subsurface investigation was carried out at this site in October 2000, at which time two boreholes were drilled. Borehole B4-1 was advanced on the east side of Highway 400, on the north side of Innisfil Beach Road, to a depth of about 6 m below the local road grade. Borehole B4-2 was advanced in the southeast quadrant of the interchange to a depth of 11 m, extending to about 7.5 m below the Innisfil Beach Road cut grade.

The investigation was carried out using a bombardier-mounted B-57 drill rig supplied and operated by Master Soil Investigations Ltd. of Weston, Ontario. Both boreholes were advanced using 108 mm diameter solid stem augers. Samples of the overburden 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 Borehole B4-2 to permit monitoring of the groundwater levels at the site.

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

The borehole locations and elevations were surveyed by Callon Dietz, Ontario Land Surveyors. The borehole elevations are referenced to geodetic datum, and the northing and easting co-ordinates are referenced to the MTM NAD83 survey system. The borehole locations, together with elevations and northing and easting co-ordinates, are shown on the attached Drawing 1.

## **4.0 SITE GEOLOGY AND STRATIGRAPHY**

### **4.1 Regional Geological Conditions**

This 30 km section of Highway 400 traverses, from south to north, the following physiographic regions as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, Third Edition, 1984): the Simcoe Lowlands; the Peterborough Drumlin Field; a second lobe of the Simcoe Lowlands; and the Simcoe Uplands. Along Highway 400, the Simcoe Lowlands are present from the southern limit of the project to just south of Innisfil Creek (about 1 km north of Highway 89), and again from Essa Road (Simcoe Road 30, formerly Highway 27) to about 1 km north of Dunlop Street (Simcoe Road 90, formerly Highway 90). The Peterborough Drumlin Field occupies the belt between these lobes of the Simcoe Lowlands, extending from just south of Innisfil Creek to Essa Road. The Simcoe Uplands extend from about 1 km north of Dunlop Street to beyond the northern limit of the project at Highway 11.

The two sections where Highway 400 crosses the Simcoe Lowlands consist of two lobes of a sand plain which include the shores of Kempenfelt Bay, the Nottawasaga River and Innisfil Creek. The surficial soils of these sections of the Simcoe Lowlands consist primarily of sand, although silt, clay or peat may be found in low-lying areas.

The surficial soils in the Peterborough Drumlin Field, in which the Innisfil Beach Road site is located, consist primarily of gravelly sand till or sand and gravel deposits. Drumlins (glacially-shaped hills) are more frequent in the southern portion of the section of the Peterborough Drumlin Field traversed by Highway 400. Deposits of silt, clay or peat may be found in the low-lying areas between drumlins.

The surficial soils in the Simcoe Uplands physiographic region are primarily sandy silt till deposits, known to contain occasional boulders. Low-lying areas may be infilled with shallow sand and gravel deposits, which are shoreline deposits of a former glacial lake that once flooded the area.

### **4.2 Site Stratigraphy**

The detailed subsurface soil and groundwater conditions encountered in the boreholes, together with the results of the laboratory testing carried out on selected soil samples, are given on the Record of Borehole sheets and Figure 1. 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.



Boreholes B4-1 and B4-2 were advanced on the east and west sides of Highway 400, respectively. The locations and ground surface elevations for these borings are shown on the attached Drawing 1.

In summary, the subsoils at the site consist of topsoil and fill / reworked soils overlying a deposit of hard clayey silt till, which contains interlayers of silty sand. A detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

#### **4.2.1 Topsoil and Fill / Reworked Soil**

Between 200 mm and 500 mm of topsoil was encountered in the boreholes. In Borehole B4-2, 1.7 m of fill or reworked soil was encountered below the topsoil. This fill consists of clayey silt to silty sand containing trace quantities of organics. Standard Penetration Test (SPT) 'N' values of 11 and 27 blows per 0.3 m of penetration were measured in the fill, indicating that the clayey silt fill has a stiff consistency and the silty sand fill has a compact relative density.

#### **4.2.2 Clayey Silt Till**

A deposit of clayey silt till underlies the topsoil and, where present, the fill or reworked soil. The till deposit extends to the maximum depth investigated (Elevation 294.8 m, approximately 7.5 m below Innisfil Beach Road grade).

The clayey silt till contains a significant proportion of sand and trace to some gravel; the grain size distribution test result for a representative sample of this till is shown on Figure 1. The till deposit grades to a non-plastic silty sand containing trace quantities of clay and gravel, as recovered in one sample from Borehole B4-2, at about Elevation 299 m. The deposit also contains a 600 mm thick interlayer of silty sand containing some gravel, the surface of which was encountered in Borehole B4-1 at about Elevation 300.6 m. Cobbles were inferred within the till deposit from grinding and resistance to augering, as noted on the borehole records.

The natural moisture contents measured on samples of the clayey silt till ranged from 6 to 9 per cent. Atterberg Limits testing measured plastic limits of 10 to 11 per cent, liquid limits of 13 to 14 per cent, and plasticity indices of 3 to 4 per cent. The results of the Atterberg Limits testing indicate that the clayey silt till is inorganic and of low plasticity.

The measured SPT 'N' values ranged from 43 to greater than 100 blows per 0.3 m of penetration, but were typically in excess of 100 blows per 0.3 m of penetration, indicating that this till deposit has a hard consistency.

#### 4.3 Groundwater Conditions

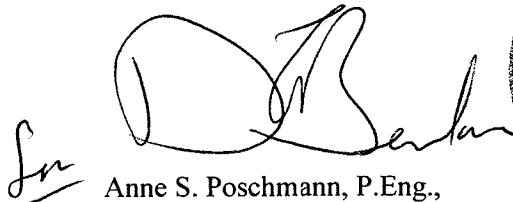
Both boreholes were dry during and on completion of the drilling operations, and recovered samples of the clayey silt till were dry to moist, with measured natural moisture contents between 6 and 9 per cent. The groundwater level in the piezometer installed in Borehole B4-2 was measured at Elevation 297.9 m, approximately 4.5 m below the Innisfil Beach Road grade in March 2001.

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.



Lisa C. Coyne, P.Eng.,  
Geotechnical Engineer



Anne S. Poschmann, P.Eng.,  
Principal



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



LCC/ASP/FJH/clg

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**PART B**

**PRELIMINARY FOUNDATION DESIGN REPORT  
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## **5.0 ENGINEERING RECOMMENDATIONS**

### **5.1 General**

This section of the report provides preliminary foundation design recommendations for the widening and / or replacement of the existing Innisfil Beach Road overpass structure, associated with the widening of Highway 400. The recommendations are preliminary only and are based on interpretation of the factual data obtained from a limited number of boreholes advanced during the subsurface investigation at this site. The interpretation and recommendations provided are intended for planning purposes only, to provide the information necessary at this stage of the study. As such, where comments are made on construction they are provided only in order to highlight those aspects which could affect the planning of the project. Further foundation investigation will be required at this bridge site as part of the detailed design stage of the project.

It is understood that Highway 400 will be widened from its existing six-lane configuration to an interim configuration of eight lanes, and an ultimate configuration of ten lanes, and that an alternative for a twelve-lane express / collector system is under consideration between Molson Park Drive and Duckworth Street in Barrie. Throughout the project length, it is expected that the existing highway platform will be widened by between 13 m and 30 m. Widening and / or replacement of the existing Innisfil Beach Road overpass will be necessary.

Based on the general layout drawing for the existing single-span structure, the abutments and associated retaining walls are supported on spread footings founded between Elevation 300.4 m and 300.6 m, below the Innisfil Beach Road cut grade of about Elevation 302.5 m. The Highway 400 grade at the structure site is at about Elevation 308 m.

### **5.2 Bridge Foundation Options**

The soils at the site consist of topsoil and fill / reworked soils overlying a deposit of hard clayey silt till, having Standard Penetration Test (SPT) 'N' values typically greater than 100 blows per 0.3 m of penetration. Based on these subsurface conditions, it is recommended that the widening or replacement structure be founded on spread footings placed on the hard clayey silt till below the Innisfil Beach Road grade. Consideration could also be given to the use of perched abutments, founded on spread footings placed within the approach embankments.

If the overpass structure is to be replaced, integral abutments could be considered at this site. In that case, the abutments could be supported on driven steel H-pile foundations. The subsoils at and below about Elevation 302 m (approximately 6 m below the Highway 400 grade, at the Innisfil Beach Road cut grade) are generally very hard, with measured Standard Penetration Test

'N' values in excess of 100 blows per 0.3 m of penetration. Heavy pile driving will be encountered within this deposit. In order to achieve the necessary pile length for integral abutments, consideration should be given to perching the pile caps within the Highway 400 approach embankments. Alternatively, pre-augering could be carried out within the hard clayey silt till to ensure an adequate pile length is achieved.

Preliminary recommendations for spread footings, including perched abutments, and for driven steel H-pile foundations are provided in the following sections.

### **5.3 Spread Footings**

For preliminary design of the bridge abutment footings, spread footings may be founded on the hard clayey silt till deposit at or below Elevation 302.5 m; a minimum soil cover of 1.5 m should be provided. Where widening is proposed, the widened footing should match the existing founding level (i.e Elevation 300.4 m to 300.6 m). Any associated wing wall or retaining wall footings may be stepped upward away from the abutments such that a minimum soil cover of 1.5 m is maintained above the underside of the footings.

Alternatively, consideration could be given to the use of abutment footings perched on a granular pad within the approach embankment.

#### **5.3.1 Axial Geotechnical Resistance**

Spread footings placed on the properly prepared clayey silt till deposit at the design elevation given above may be designed using a factored geotechnical resistance at Ultimate Limit States (ULS) of 1,200 kPa, assuming a 3.5 m wide footing. The settlement of footings founded on the clayey silt till will be dependent on the footing size and configuration, and on the applied loads. For preliminary design purposes, the geotechnical resistance at Serviceability Limit States (SLS) may be taken as 900 kPa. The geotechnical resistance at SLS will have to be reviewed following the detailed design stage of subsurface investigation, once the footing size, configuration and loadings are known.

For spread footings placed within the approach embankments on a compacted Granular 'A' pad, a factored geotechnical resistance at ULS of 900 kPa may be assumed for preliminary design. The geotechnical resistance at SLS will depend on the thickness of Granular 'A' and the consistency and thickness of the underlying soils; a value of 350 kPa may be assumed for preliminary design.

The geotechnical resistances provided herein are given under the assumption that the loads will be applied perpendicular to the surface of the footings; where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with the Ontario Highway Bridge Design Code (OHBD C).

### **5.3.2 Resistance to Lateral Loads**

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 OHBD C. The angle of friction between the concrete and the undisturbed clayey silt till founding soils should be taken as 24 degrees; the corresponding coefficient of friction,  $\tan \delta$ , would then be 0.45. Where “perched” abutment footings are adopted, the angle of friction between the concrete footings and the compacted Granular ‘A’ pad should be taken as 30 degrees; the corresponding coefficient of friction would be 0.58.

### **5.3.3 Frost Protection**

The footings should be provided with a minimum of 1.5 m of soil cover for frost protection.

## **5.4 Driven Steel H-Piles**

Consideration could be given to supporting the replacement structure or widenings on steel H-piles driven to found within the hard clayey silt till deposit. The surface of the hard clayey silt till was encountered at about Elevation 302 m in Boreholes B4-1 and B4-2; this level is about 6 m below the Highway 400 grade, at approximately the Innisfil Beach Road cut grade. Because of the hard nature of the till and the presence of cobbles and boulders, hard driving conditions should be anticipated. Consideration should be given to perching the pile caps within the Highway 400 approach embankments in order to maximize the driven pile length. If the piles are driven from the Innisfil Beach Road cut grade, it is expected to be necessary to pre-auger the upper 2 m of the pile length within the till in order to provide a starting guide for driving of the piles. Further, it may be necessary to pre-auger deeper in order to achieve an adequate pile length for integral abutment considerations. As discussed in Section 5.7.3, cobbles and boulders are anticipated within the till deposit, and provision must be made in both the Contract Documents and the contractor’s methods and equipment to handle such obstructions.

For preliminary design where the abutment pile caps are placed at the Innisfil Beach Road cut grade, a pile tip level of Elevation 296 m may be assumed to allow a 5 m pile length to be achieved. If the pile caps are perched within the Highway 400 approach embankments, the pile tip level should be taken at Elevation 299 m for preliminary design.

#### 5.4.1 Axial Geotechnical Resistance

For preliminary design, the factored axial resistance at ULS for steel HP 310 x 110 H-piles driven to found within the hard clayey silt till at or below the tip elevations given above may be taken as 1,400 kN. The axial resistance at SLS for 25 mm of settlement may be taken as 1,200 kN.

To achieve the above design resistances, the piles should be driven 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, and not exceeding 60 kJ. Provision should be made to re-tap selected piles to confirm the set after adjacent piles have been driven, in accordance with MTO's current Special Provision.

#### 5.4.2 Resistance to Lateral Loads

The lateral loading could be resisted fully or partially by the use of battered piles. If vertical piles are used, the resistance to lateral loading will have to be derived from the soil in front of the piles. If integral abutments are under consideration, there may also be a requirement for the piles to move sufficiently to accommodate the bridge deck deflections.

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

For cohesionless soils:

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

For cohesive soils:

$$k_h = \frac{k_{s1}}{5B} \quad \text{where} \quad \begin{array}{l} B \text{ is the pile diameter (m) and} \\ k_{s1} \text{ is the constant of horizontal subgrade} \\ \text{reaction, as given below} \end{array}$$

The piles will be driven through embankment fill (if the pile caps are perched within the Highway 400 approach embankments) and into the clayey silt till. The following ranges for the value of  $n_h$  may be assumed in the structural analysis; these values will have to be confirmed following the detailed design stage of the subsurface investigation.

<i>Soil Unit</i>	<i><math>n_h</math></i>	<i><math>k_{s1}</math></i>
Embankment Fill above Elevation 303.5 m	5 to 15 MPa/m	—
Clayey Silt Till between Elevations 303.5 m and 302 m	—	25 to 60 MPa/m
Clayey Silt Till below Elevation 302 m	—	50 to 100 MPa/m

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

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

#### 5.4.3 Frost Protection

The pile caps should be provided with 1.5 m soil of cover for frost protection.

#### 5.5 Lateral Earth Pressures

The lateral pressures acting on the bridge abutments and associated retaining walls 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, 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 abutments and walls. 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. Other aspects of the abutment granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSS 3501.00 and 3504.00
- 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.
- The granular fill may be placed either in a zone with width equal to at least 1.5 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 1.5 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 <sup>3</sup>
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:

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

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

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

## **5.6 Embankment and Permanent Cut Slope Design**

Based on the topographic information on the Engineering and Title Records plates and on site reconnaissance, the existing embankment side slopes along Highway 400 and permanent cut slopes along Innisfil Beach Road are formed at a gradient of approximately 2 horizontal to 1 vertical (2H:1V). For widening of the Highway 400 embankment, the new side slopes should be formed at a maximum gradient of 2H:1V. The embankment widening should be carried out using conventional fill placement and compaction practices, and benching of the existing embankment side slopes should be carried out to key in the new fill.

If additional permanent cut work is required along Innisfil Beach Road, a maximum gradient of 2H:1V may be assumed for preliminary design.

## **5.7 Design and Construction Considerations**

### **5.7.1 Dewatering**

Groundwater seepage into the footing excavations could occur from water-bearing lenses or interlayers of granular soil within the clayey silt till deposit, although the quantity is expected to be minor. Pumping from properly-filtered sumps or a filtered drain placed at the base of the excavation should provide sufficient groundwater control during foundation works. Surface water run-off, which is expected to be more significant than groundwater seepage, should be directed away from the footing excavations.

The clayey silt till soils in which the footing excavations will be formed are susceptible to disturbance from ponded water and construction traffic. Provision should be made in the Contract Documents for the placement of a lean concrete mat to protect the soils from such disturbance.

### 5.7.2 Excavation

The footing excavations will extend a minimum of 1.5 m below lowest surrounding grade, generally through hard clayey silt till soils although interlayers of granular soil could be encountered within the till. 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. The hard clayey silt till soils would be classified as Type 1 soil, but granular interlayers, if encountered, would be classified as Type 2 soil. Temporary open-cut slopes should therefore be maintained no steeper than 1 horizontal to 1 vertical (1H:1V) to within 1.2 m of the excavation base; below this depth, the excavation sides may be formed near-vertical. Where space restrictions dictate, foundation works could also be carried out within a braced excavation.

### 5.7.3 Obstructions

An obstruction was encountered at 1.4 m depth in Borehole B4-1, and cobbles were noted within the hard clayey silt till deposit in both boreholes. Cobbles and boulders should be expected excavation and pile driving for deep foundations or temporary shoring systems, if these options are adopted.

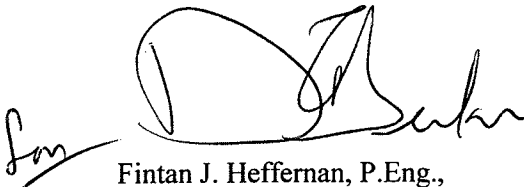
#### GOLDER ASSOCIATES LTD.



Lisa C. Coyne, P.Eng.,  
Geotechnical Engineer



Anne S. Poschmann, P.Eng.,  
Principal



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



LCC/ASP/FJH/clg  
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## LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

### I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
SS	Split-spoon
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

### III. SOIL DESCRIPTION

#### (a) Cohesionless Soils

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

### II. PENETRATION RESISTANCE

#### Standard Penetration Resistance (SPT), N:

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

#### (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

#### 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<sup>2</sup> 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 <sup>1</sup>
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement <sup>1</sup>
$D_R$	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)
$\gamma$	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

$\pi$	= 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

$\gamma$	shear strain
$\Delta$	change in, e.g. in stress: $\Delta \sigma$
$\epsilon$	linear strain
$\epsilon_v$	volumetric strain
$\eta$	coefficient of viscosity
$\nu$	Poisson's ratio
$\sigma$	total stress
$\sigma'$	effective stress ( $\sigma' = \sigma - u$ )
$\sigma'_{vo}$	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stresses (major, intermediate, minor)
$\sigma_{oct}$	mean stress or octahedral stress = $(\sigma_1 + \sigma_2 + \sigma_3)/3$
$\tau$	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
$\gamma'$	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 $\rho$ . Unit weight symbol is $\gamma$ 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
$C_\alpha$	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
$\sigma'_p$	pre-consolidation pressure
OCR	Overconsolidation ratio = $\sigma'_p / \sigma'_{vo}$

#### (e) Shear Strength

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

Notes: 1.  $\tau = c' + \sigma' \tan \phi'$

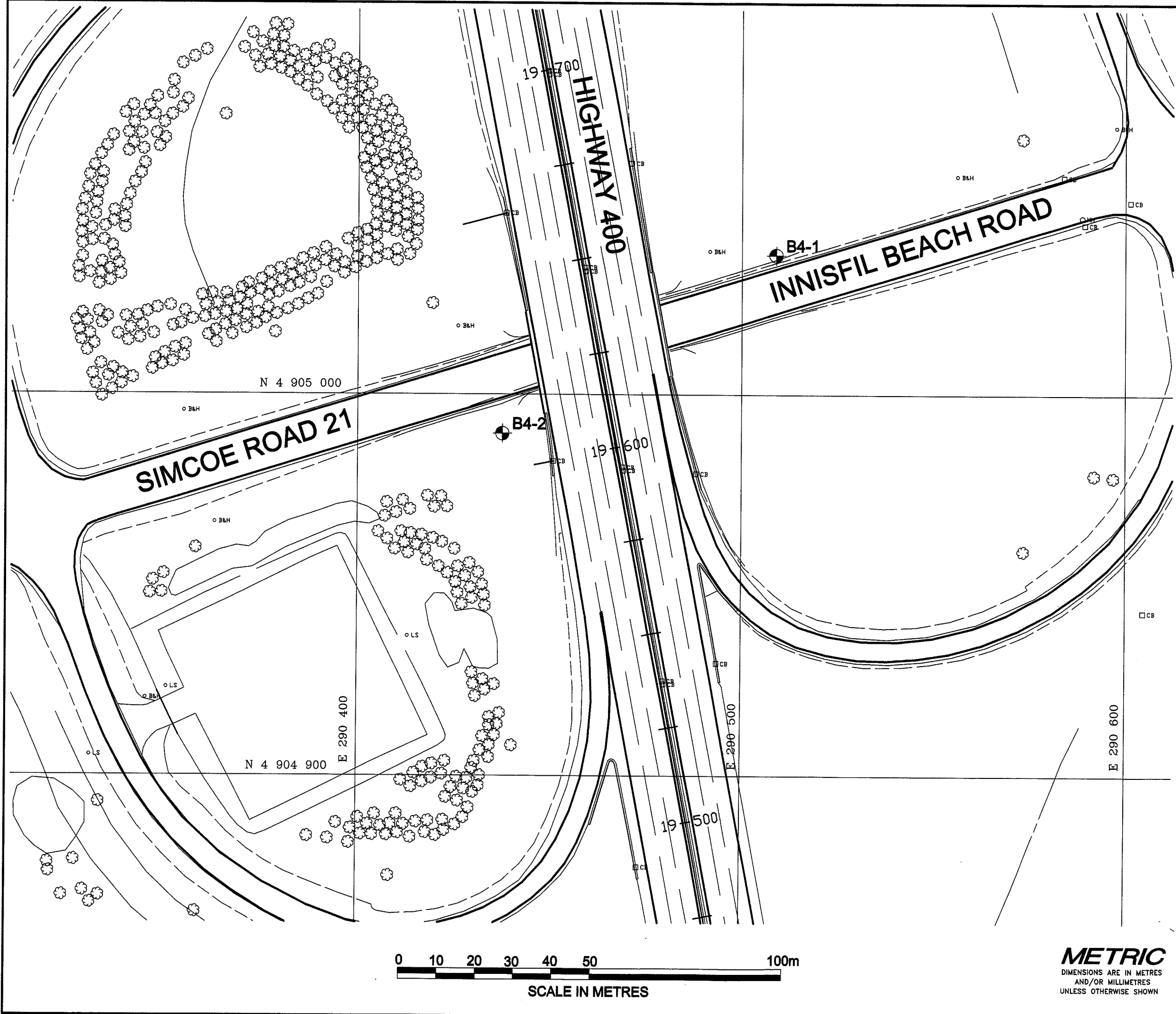
2. Shear strength = (Compressive strength)/2

PROJECT 001-1143F		RECORD OF BOREHOLE No B4-1		1 OF 1		METRIC											
W.P. 30-95-00		LOCATION N 4905036.5; E 290509.0		ORIGINATED BY AZ													
DIST SW HWY 400		BOREHOLE TYPE 108mm DIAMETER SOLID STEM AUGERS		COMPILED BY LCC													
DATUM Geodetic		DATE Oct.24/2000		CHECKED BY ASP													
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa 20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X REMOULDED			WATER CONTENT (%) W <sub>p</sub> W W <sub>L</sub>			γ	GR SA SI CL		
302.7	GROUND SURFACE																
0.0	Topsil																
0.2	Clayey Silt with sand, some gravel (Till) Hard Brown Moist		1	SS	120		302										
			2	SS	125		301										
300.6	Silty Sand, some gravel, trace clay Very dense Brown Dry		3	SS	103/15		300										
300.0	Clayey Silt with sand, some gravel (Till) Hard Brown Moist		4	SS	151/15		299										
2.7			5	SS	101/15		298										
			6	SS	109/15		297										
	Cobbles at 5.5m and 5.8m depth																
296.5			7	SS	103/15												
6.2	END OF BOREHOLE																
Notes: 1. Refusal to auger advance was encountered at 1.4m depth. Borehole was relocated 1m west and drilling continued. 2. Borehole dry on completion of drilling operations.																	

ON\_MOT 0011143F.GPJ ON\_MOT.GDT 14/1/02

PROJECT 001-1143F			RECORD OF BOREHOLE No B4-2			1 OF 1			METRIC								
W.P. 30-95-00			LOCATION N 4904989.7; E 290437.7			ORIGINATED BY AZ											
DIST SW HWY 400			BOREHOLE TYPE 108mm DIAMETER SOLID STEM AUGERS			COMPILED BY LCC											
DATUM Geodetic			DATE Oct.24/2000			CHECKED BY ASP											
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X REMOULDED					WATER CONTENT (%) W <sub>p</sub> W W <sub>L</sub>			UNIT WEIGHT γ kN/m <sup>3</sup>	GR SA SI CL
305.6	GROUND SURFACE							20 40 60 80 100									
0.0	Topsoil																
305.1																	
0.5	Clayey Silt with sand, some organics (Fill) Stiff Dark brown Moist		1	SS	11		305										
304.1							304										
1.5	Silty Sand, trace gravel, clay and organics (Fill) Compact Dark brown Moist		2	SS	27												
303.4							303										
2.2	Clayey Silt with sand, trace gravel (Till) Hard Brown becoming grey at 7.3m depth Moist		3	SS	48												
			4	SS	141/25		302										
	Cobbles at 3.7m depth		5	SS	101/15		301										
			6	SS	115/15		300										
							299										
	Silty sand, trace clay and gravel (Till) encountered in Sample 7.		7	SS	43		298										
			8	SS	210		297										
							296										
			9	SS	240		295										
294.8			10	SS	135/15												
10.8	END OF BOREHOLE																
Notes: 1. Borehole dry on completion of drilling operations. 2. Water level in piezometer measured at 7.7m depth (Elev.297.9m) on March 15, 2001.																	

ON\_MOT\_0011143F.GPJ ON\_MOT.GDT 14/1/02



DIST HWY 400  
CONT. No.  
GWP No. 30-95-00

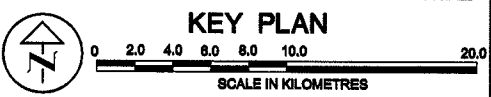
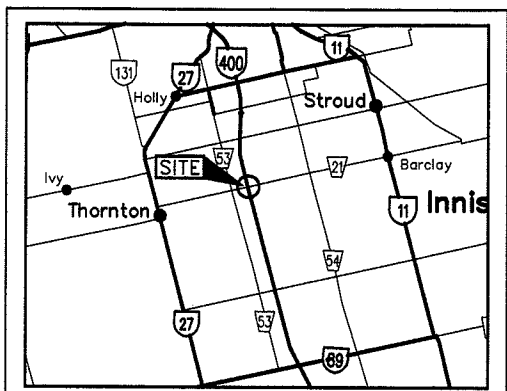
INNISFIL BEACH ROAD OVERPASS  
HWY 400  
BOREHOLE LOCATION PLAN



SHEET



Golder Associates Ltd.  
MISSISSAUGA, ONTARIO, CANADA



LEGEND

- Borehole, previous investigation
- Borehole, present investigation

No.	ELEVATION	LOCATION	
		NORTHING	EASTING
B4-1	302.7	4,905,036.5	290,509.0
B4-2	305.6	4,904,989.7	290,437.7

REFERENCE

This drawing was created from digital file "50201.dwg"  
provided by URS Cole Sherman

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

NO.	DATE	BY	REVISION
Geocres No.			
HWY. No. 400		PROJECT NO.: 001-1143F	
SUBM'D. LCC	CHKD: ASP	DATE: JANUARY 2001	SITE 30-210
DRAWN: MHW	CHKD. LCC	APPD. ASP	DWG. 1

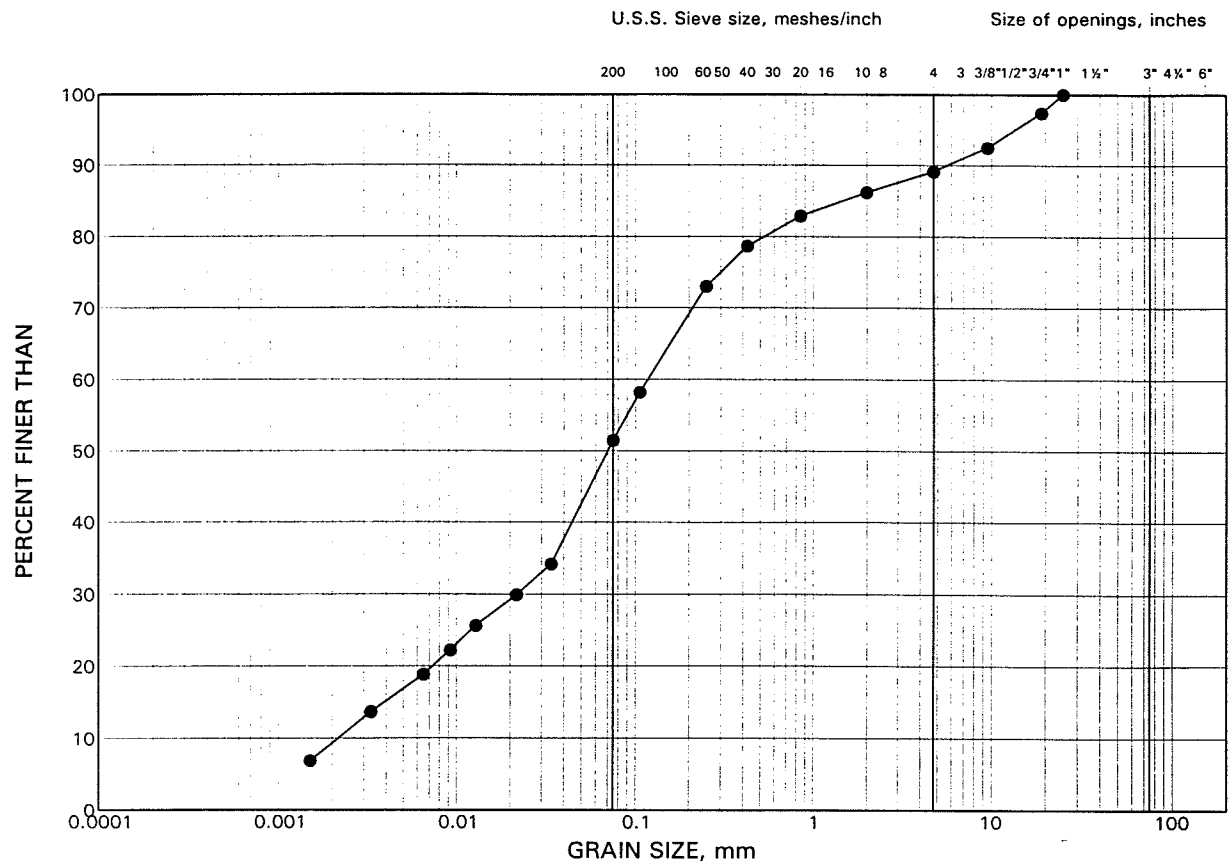
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# GRAIN SIZE DISTRIBUTION TEST RESULT

Clayey Silt Till

FIGURE 1



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

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
•	B4-1	2	300.9