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**PRELIMINARY FOUNDATION
INVESTIGATION AND DESIGN REPORT
INNISFIL SIXTH LINE OVERPASS
STRUCTURE SITE 30-211
HIGHWAY 400 WIDENING FROM 1 KM SOUTH
OF HIGHWAY 89 TO HIGHWAY 11
G.W.P. 30-95-00**

Submitted to:

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PART A

**PRELIMINARY FOUNDATION INVESTIGATION REPORT
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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 Sixth Line overpass structure. A foundation site 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 Sixth Line overpass structure is located about 7 km north of the Highway 89 interchange and about 8.5 km south of the Molson Park Drive interchange, in the Town of Innisfil, Simcoe County. The MTO has designated this overpass as Structure Site No. 30-211.

At this site, the original ground surface was at about Elevation 294 m to 295 m. Innisfil Sixth Line has been constructed in a cut up to 4 m deep, with its grade at about Elevation 291 m under Highway 400. The Highway 400 grade is at about Elevation 296.5 m at the structure site.

The existing single-span overpass structure was constructed in the early 1950s. According to the general layout drawings for this existing structure, which was provided by Morrison Hershfield (the structural designers for this preliminary study), the abutments and associated retaining walls are supported on spread footings which are founded at about Elevation 289.4 m.

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out at this site in October 2000, at which time two boreholes were drilled. Boreholes B3-1 and B3-2 were advanced in the vicinity of the north and south abutments, on the east and west sides of the highway, respectively. The boreholes were advanced to between 8 m and 11 m below the Innisfil Sixth Line 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. The boreholes were advanced using 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 groundwater conditions in the open boreholes were observed throughout the drilling operations, and a piezometer was installed in Borehole B3-1 to permit monitoring of the groundwater levels at the site.

The field work was supervised on a full-time basis by a member 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 Sixth Line 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 B3-1 and B3-2 were advanced on the east and west sides of Highway 400, respectively, from approximately Innisfil Sixth Line grade. The locations and ground surface elevations for these borings are shown on the attached Drawing 1.

In summary, the site is underlain at the borehole locations by sand and gravel to silty sand fill, overlying clayey silt till. A detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Fill

Beneath the asphalt in Boreholes B3-1 and B3-2, 300 mm to 500 mm of sand and gravel road base fill was encountered.

Underlying this road base fill in Borehole B3-2, a 1.8 m thick layer of silty sand fill, containing some gravel and trace clay, was encountered. It is likely that this silty fill is associated with utility trench backfill, as Borehole B3-2 is located near a catch basin. The measured Standard Penetration Test 'N' values were 11 and 28 blows per 0.3 m of penetration, indicating that the silty sand fill has a compact relative density.

4.2.2 Clayey Silt Till

A deposit of clayey silt till was encountered below the fill in both boreholes. The surface of the till is at Elevation 290.7 m in Borehole B3-1, on the east side of the highway. On the west side of the highway, Borehole B3-2 encountered utility trench backfill; outside of the utility trench areas, it is expected that the surface of the till deposit will be encountered immediately below the road base fill. The till deposit extends to the maximum depth investigated, to about Elevation 283 m and 279.5 m in the boreholes on the east and west sides of Highway 400, respectively. The till deposit is at least 8 m to 11 m thick.

The clayey silt till contains a significant proportion of sand, and trace to some gravel. The result of a grain size distribution test carried out on a representative sample is shown on Figure 1. The natural moisture contents measured on samples of the clayey silt till ranged from 6 to 9 per cent. Atterberg Limits testing was carried out on three samples. The plastic limits ranged from 11 to 12 per cent, the liquid limits from 14 to 15 per cent and the plasticity indices from 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 67 to 138 blows, but were typically greater than 100 blows per 0.3 m of penetration, indicating that the clayey silt till is hard.

4.3 Groundwater Conditions

The groundwater conditions were observed in the open boreholes following drilling operations in October 2000. At that time, the water level measured in Borehole B3-1 was at 6.9 m depth (about Elevation 284.5 m) and rising; the water level measured in Borehole B3-2 was at 4 m depth (about Elevation 287 m). The piezometer which was installed in Borehole B3-1 could not be founded in January or March 2001. This piezometer is presumed to have been destroyed. Therefore, the stabilized groundwater level could not be determined.

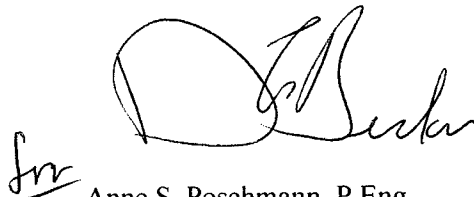
The colour change in the soil from brown to grey at a relatively shallow depth indicates that the piezometric groundwater level is likely in the upper portion of the till.

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

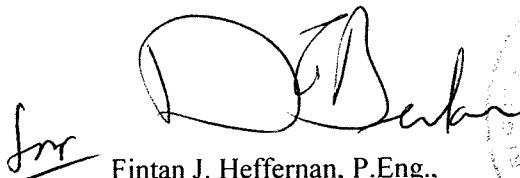
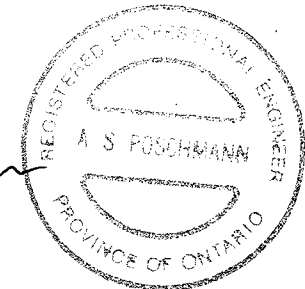
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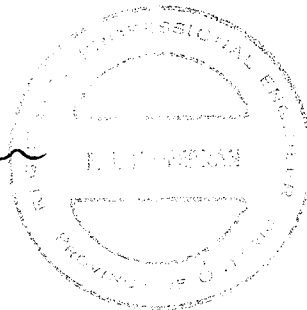
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DJE/LCC/ASP/FJH/lcc

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PART B

**PRELIMINARY FOUNDATION DESIGN REPORT
INNISFIL SIXTH LINE OVERPASS
STRUCTURE SITE 30-211
HIGHWAY 400 WIDENING FROM 1 KM SOUTH
OF HIGHWAY 89 TO HIGHWAY 11
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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 Sixth Line 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 Sixth Line overpass will therefore 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 at about Elevation 289.4 m. The Highway 400 grade is at about Elevation 296.5 m at the structure site, while the surrounding grade is at about Elevation 294 m to 295 m. Innisfil Sixth Line has been constructed in a cut up to 4 m deep, with its grade at about Elevation 291 m under Highway 400.

5.2 Bridge Foundation Options

The native subsoils at the site consist 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 and on the existing foundation 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 Sixth Line grade. Consideration could also be given to the use of perched abutments, founded on spread footings placed on a compacted granular pad within the approach embankments.

Alternatively, for a new overpass or structurally-separate widening, the abutments could be supported on steel H-piles driven to practical refusal within the hard clayey silt till.

It should be noted that the boreholes put down during this preliminary phase of field work were drilled from the Innisfil Sixth Line cut level. If perched footings or integral abutments supported on deep foundations are considered viable options, determination of the subsoil conditions between the Highway 400 grade and the Innisfil Sixth Line cut will be required during the detailed design stage.

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 abutment foundations, spread footings may be placed on the hard clayey silt till deposit at or below Elevation 289.4 m; a minimum of 1.5 m of soil cover should be provided. For widening of the existing overpass structure, the founding level of the existing footings should be matched. Where fill is encountered below footing founding level, it should be removed and replaced with lean concrete. 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. In this case, a well-compacted granular pad will be required to support the wing wall or retaining wall footings above Elevation 289.4 m.

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

5.3.1 Axial Geotechnical Resistance

Spread footings placed within the undisturbed clayey silt till deposit at the design elevation given above may be designed using a factored geotechnical resistance at Ultimate Limit States (ULS) of 900 kPa, assuming a 3 m wide footing. The groundwater conditions at the site will have to be confirmed during the detailed design stage.

The settlement of footings founded on the clayey silt till will be dependent on the footing size and configuration, and on the applied loads. This settlement will be differential with respect to the existing overpass structure where widening of the structure is considered. For preliminary design purposes, the geotechnical resistance at Serviceability Limit States (SLS) may be taken as 600 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, and the groundwater conditions at the site are confirmed.

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 well compacted 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 (OHBDC).

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 OHBDC. The angle of friction between the concrete and the undisturbed founding soils should be taken as 30 degrees; the corresponding coefficient of friction, $\tan \delta$, would then be 0.58. Where "perched" 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

All 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 a new or structurally-separate overpass structure on steel H-piles driven to found within the hard clayey silt till deposit. The pile tip level is expected to be variable at this site and could be as low as Elevation 286 m on the west side and as high as Elevation 288 m on the east side. It is assumed that the abutment pile caps would be perched above the Innisfil Sixth Line grade in order to achieve the required pile length for rotational flexibility purposes. In this regard, it is noted that additional borehole investigation will be

required at the proposed abutment locations during the detailed design stage in order to confirm the soil conditions above the Innisfil Sixth Line cut grade.

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 may be taken as 1,600 kN. The axial resistance at SLS for a single pile, for 25 mm of settlement, may be taken as 1,400 kN. For design, a pile tip level at Elevation 287 m may be assumed.

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 following equation:

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

It is assumed that the abutment pile caps will be perched above the Innisfil Sixth Line cut grade to provide a greater driven length. The native soils above the cut grade were not investigated during this stage of field work. Based on regional geological mapping, it is likely that these soils will consist of clayey silt till, similar to that encountered below the cut grade. For very stiff to hard clayey silt till, as might be expected in the upper portion of the deposit above Elevation 291 m, the range in value of k_{s1} may be taken as 25 MPa/m to 60 MPa/m. Below Elevation 291, the range in value of k_{s1} should be taken as 50 MPa/m to 100 MPa/m in the structural analysis. These values will have to be confirmed following the detailed design stage of the subsurface investigation.

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

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

5.4.3 Frost Protection

All pile caps should be provided with 1.5 m of soil 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, on the freedom of lateral movement of the structure, and on the drainage conditions behind the walls. 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 OPSD 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 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:

	Granular 'A'	Granular 'B'
		Type II
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 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 Sixth Line 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 Sixth Line, a maximum gradient of 2H:1V may be assumed for preliminary design.

5.7 Design and Construction Considerations

5.7.1 Groundwater Control

Groundwater seepage into the footing or pile cap excavations will occur from the surficial fill or from water-bearing lenses or interlayers of granular soil within the clayey silt till deposit, although the quantity is expected to be relatively minor. Pumping from properly-filtered sumps or a filtered drain placed at the base of the excavation should provide sufficient groundwater control during foundation construction 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 or pile cap excavations will be formed are susceptible to disturbance from ponded water and construction traffic. Provision should be made in the Contract Documents for the timely placement of a lean concrete mat to protect the soils from such disturbance.

5.7.2 Excavation

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 footing excavations will extend a minimum of 1.5 m below lowest surrounding grade, generally through hard clayey silt till soils although it is expected that interlayers of granular soil could be encountered within the till. 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, the excavation sides may be maintained near-vertical. Where space restrictions dictate, foundation works could also be carried out within a braced excavation.

5.7.3 Settlement

Deformation of the ground due to foundation loading will result in settlement of the bridge abutments and superstructure. This settlement will be differential with respect to the existing overpass structure, and also could vary along the new structure depending on the variability and consistency/relative density of the founding soils as well as the activities during construction. The potential for differential settlement should be reassessed during the detailed design stage once the proposed bridge configuration is established.

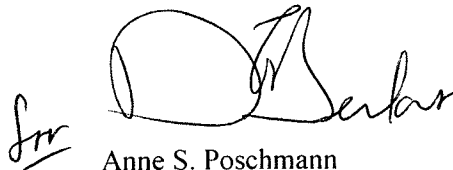
5.7.4 Obstructions

Cobbles were encountered within the clayey silt till deposits as noted on the records for Boreholes B3-1 and B3-2. It is noted that cobbles and boulders are inherent in glacially-derived soils, and should therefore be expected during footing excavation, pile driving and / or temporary shoring system installation. Where boulders are encountered within footing excavations, they should be removed and the sub-excavated areas should be backfilled with well-compacted Granular 'A' or lean concrete.

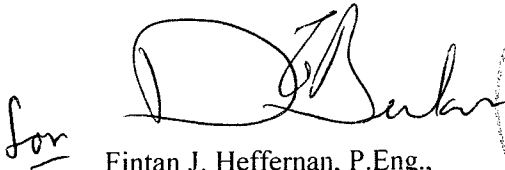
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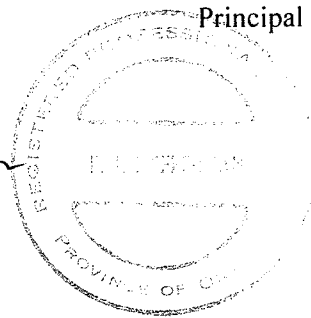
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Anne S. Poschmann
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Fintan J. Heffernan, P.Eng.,
Designated MTO Contact



DJE/LCC/ASP/FJH/lcc

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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 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

IV. SOIL TESTS

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

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

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LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta \sigma$
ϵ	linear strain
ϵ_v	volumetric strain
η	coefficient of viscosity
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stresses (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation
*	Density symbol is ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)

(a) Index Properties (con't.)

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

(c) Hydraulic Properties

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

(d) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (overconsolidated range)
C_s	swelling index
C_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

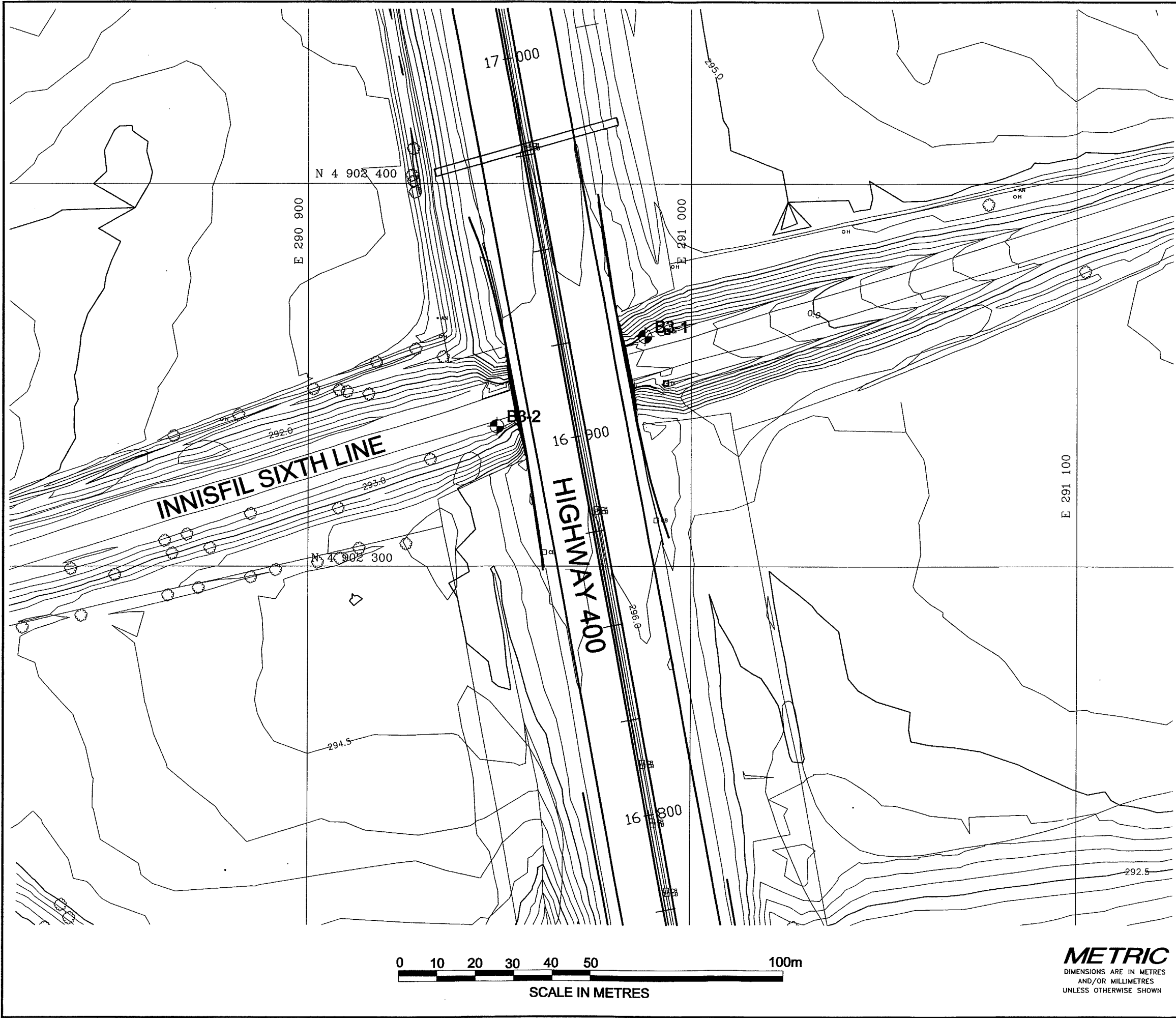
Notes: 1. $\tau = c' + \sigma' \tan \phi'$

2. Shear strength = (Compressive strength)/2

PROJECT 001-1143F				RECORD OF BOREHOLE No B3-1				1 OF 1		METRIC				
W.P. 30-95-00				LOCATION N 4902360.3; E 290986.9				ORIGINATED BY AZ						
DIST SW HWY 400				BOREHOLE TYPE 108mm DIAMETER SOLID STEM AUGERS				COMPILED BY LCC						
DATUM Geodetic				DATE Oct.25/2000				CHECKED BY ASP						
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
291.2	GROUND SURFACE													
0.9	Asphalt													
290.7	Sand and Gravel (Fill)													
0.5	Clayey Silt with sand, trace to some gravel (Till) Hard Brown becoming grey at 1.4m depth Moist		1	SS	67									
			2	SS	130									
			3	SS	138									
			4	SS	104									
			5	SS	138									
			6	SS	135									
			7	SS	100/10									
	Cobble at 6.4m depth													
283.0			8	SS	101									
8.2	END OF BOREHOLE													
	Notes: 1. Water level in open borehole at 7.4m depth (Elev.283.8m) immediately after completion of drilling. Water level rose to 6.9m depth (Elev.284.3m) about 10 minutes after completion of drilling. 2. Piezometer could not be found on January 19 or March 15, 2001; piezometer presumed destroyed.													

ON_MOT 0011143F.GPJ ON_MOT.GDT 14/1/02

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



DIST

HWY 400

CONT. No.

GWP No. 30-95-00

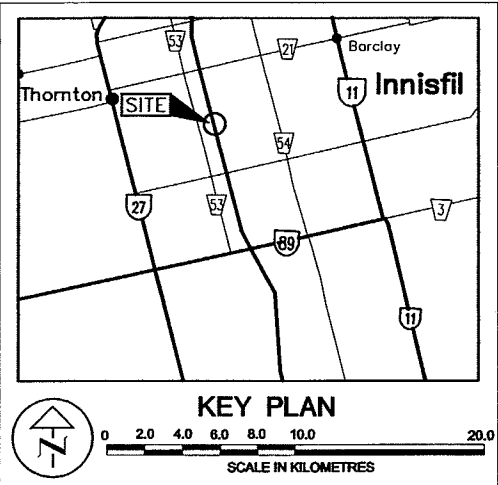
INNISFIL SIXTH LINE OVERPASS

HWY 400

BOREHOLE LOCATION PLAN

SHEET

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



LEGEND			
	Borehole, previous investigation		
	Borehole, present investigation		
No.	ELEVATION	LOCATION	
		NORTHING	EASTING
B3-1	291.2	4,902,360.3	290,986.9
B3-2	290.9	4,902,336.7	290,949.2

REFERENCE
This drawing was created from digital file "33807.dwg"
provided by URS Cole Sherman

NO.	DATE	BY	REVISION
Geocres No.			
HWY. No. 400		PROJECT NO.: 001-1143F	
SUBM'D. LCC	CHKD: ASP	DATE: JANUARY 2001	SITE 30-211
DRAWN: MHW	CHKD. LCC	APPD. ASP	DWG. 1

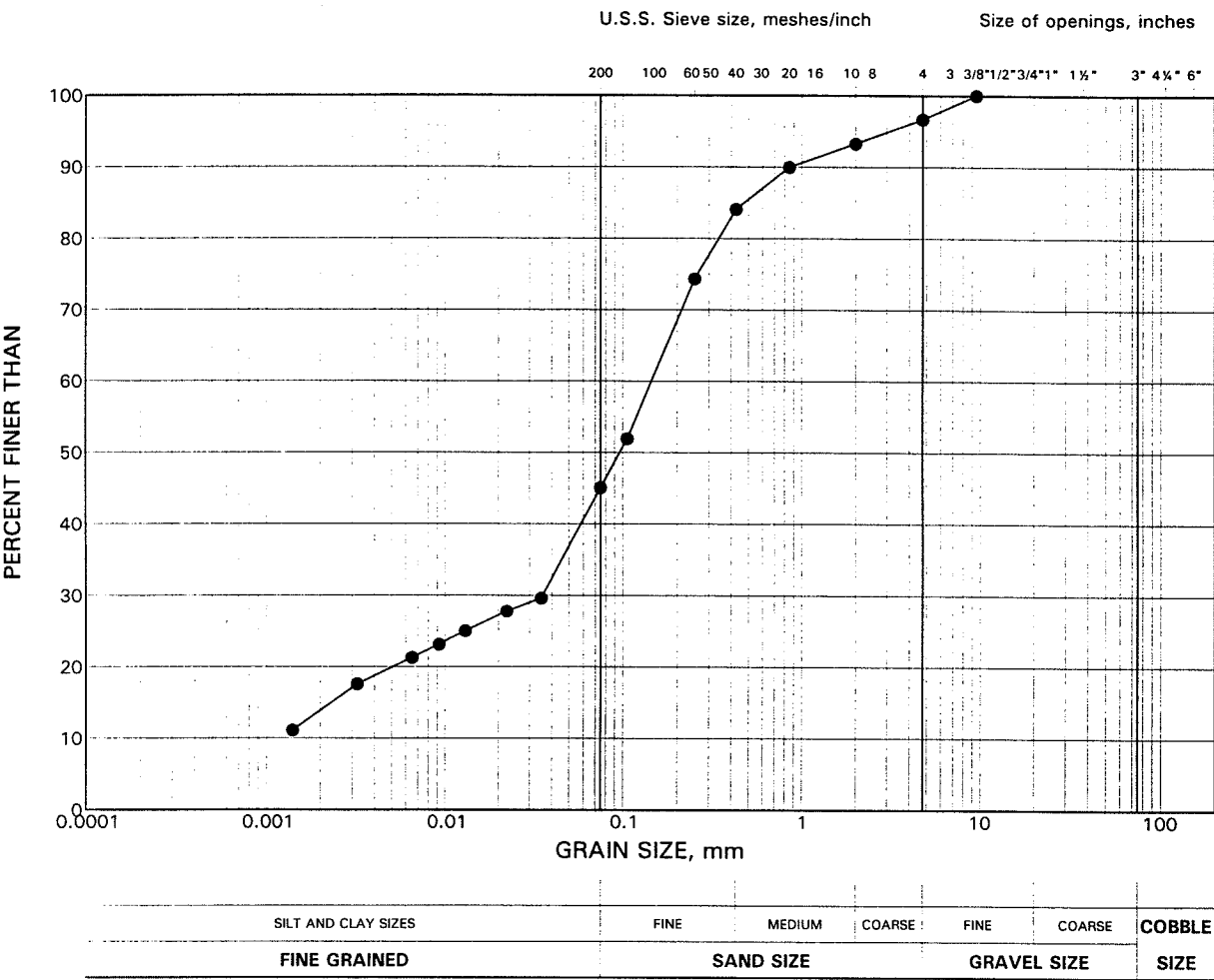
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN



GRAIN SIZE DISTRIBUTION TEST RESULT

Clayey Silt Till

FIGURE 1



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	B3-1	3	288.7