

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
KILLARNEY BEACH ROAD UNDERPASS
STRUCTURE SITE 30-212
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

DISTRIBUTION:

- | | | | |
|---|----------------|---|--|
| 1 | Copy (Unbound) | - | URS Cole, Sherman, Thornhill, Ontario |
| 2 | Copies (Bound) | - | URS Cole, Sherman, Thornhill, Ontario |
| 3 | Copies | - | MTO Southwestern Region, London, Ontario |
| 1 | Copy | - | MTO Foundations Section, Downsview, Ontario |
| 2 | Copies | - | Golder Associates Ltd., Mississauga, Ontario |



January 2002

001-1143F-2

TABLE OF CONTENTS

<u>SECTION</u>	<u>PAGE</u>
PART A - PRELIMINARY FOUNDATION INVESTIGATION REPORT	
1.0 INTRODUCTION	1
2.0 SITE DESCRIPTION	2
3.0 INVESTIGATION PROCEDURES	3
4.0 SITE GEOLOGY AND STRATIGRAPHY	4
4.1 Regional Geological Conditions	4
4.2 Site Stratigraphy	4
4.2.1 Fill and Topsoil	5
4.2.2 Silty Clay Till	5
4.2.3 Silty Sand to Sand and Silt Interlayer	6
4.2.4 Clayey Silt Till	6
4.2.5 Silty Sand Till	7
4.3 Groundwater Conditions	7
PART B - PRELIMINARY FOUNDATION DESIGN REPORT	
5.0 ENGINEERING RECOMMENDATIONS	9
5.1 General	9
5.2 Bridge Foundation Options	9
5.3 Spread Footings	10
5.3.1 Axial Geotechnical Resistance	10
5.3.2 Resistance to Lateral Loads	11
5.3.3 Frost Protection	11
5.4 Driven Steel H-Piles	11
5.4.1 Axial Geotechnical Resistance	12
5.4.2 Resistance to Lateral Loads	12
5.4.3 Frost Protection	13
5.5 Lateral Earth Pressures	13
5.6 Embankment Design	14
5.7 Design and Construction Considerations	15
5.7.1 Groundwater Control	15
5.7.2 Excavation	15
5.7.3 Settlement	16
5.7.4 Obstructions	16

In Order
Following
Page 16

Lists of Abbreviations and Symbols
Records of Boreholes B2-1 and B2-2
Drawing 1
Figures 1 and 2

TABLE OF CONTENTS (CONTINUED)

LIST OF DRAWINGS

Drawing 1 Killarney Beach Road Underpass, Highway 400, Borehole Location Plan

LIST OF FIGURES

Figure 1 Grain Size Distribution Test Result, Silty Clay Till
Figure 2 Grain Size Distribution Test Result, Sand and Silt

PART A

**PRELIMINARY FOUNDATION INVESTIGATION REPORT
KILLARNEY BEACH ROAD UNDERPASS
STRUCTURE SITE 30-212
HIGHWAY 400 WIDENING FROM 1 KM SOUTH
OF HIGHWAY 89 TO HIGHWAY 11
G.W.P. 30-95-00**

TABLE OF CONTENTS

<u>SECTION</u>	<u>PAGE</u>
PART A - PRELIMINARY FOUNDATION INVESTIGATION REPORT	
1.0 INTRODUCTION	1
2.0 SITE DESCRIPTION	2
3.0 INVESTIGATION PROCEDURES	3
4.0 SITE GEOLOGY AND STRATIGRAPHY	4
4.1 Regional Geological Conditions	4
4.2 Site Stratigraphy	4
4.2.1 Fill and Topsoil	5
4.2.2 Silty Clay Till	5
4.2.3 Silty Sand to Sand and Silt Interlayer	6
4.2.4 Clayey Silt Till	6
4.2.5 Silty Sand Till	7
4.3 Groundwater Conditions	7

Lists of Abbreviations and Symbols

Records of Boreholes B2-1 and B2-2

Drawing 1

Figures 1 and 2

LIST OF DRAWINGS

Drawing 1 Killarney Beach Road Underpass, Highway 400, Borehole Location Plan

LIST OF FIGURES

Figure 1 Grain Size Distribution Test Result, Silty Clay Till

Figure 2 Grain Size Distribution Test Result, Sand and Silt

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 replacement of the existing Killarney Beach Road underpass 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 Killarney Beach Road (Innisfil Fourth Line, also formerly Churchill Sideroad) underpass structure is located about 4 km north of the Highway 89 interchange and 5.5 km south of the Innisfil Beach Road (Simcoe Road 21) interchange in the Town of Innisfil, in Simcoe County. The MTO has designated this underpass as Structure Site No. 30-212.

At this site, the original ground surface varies from about Elevation 285 m to 287 m. The Highway 400 grade is at about Elevation 285.5 m to 287 m, rising northward. Killarney Beach Road has been constructed in fill, with approach embankments up to about 7 m in height. The local road grade over Highway 400 is at about Elevation 292.5 m.

According to the general layout drawing for the existing single-span underpass structure, 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 284.5 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 B2-1 and B2-2 were advanced in the vicinity of the east and west abutments to depths of about 11 m and 9.5 m, respectively, below the Highway 400 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 water levels in the open boreholes were observed during and following the drilling operations, and a piezometer was installed in Borehole B2-1, in the vicinity of the west abutment, to permit monitoring of the groundwater level 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, 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, which is located about 1 km north of Highway 89, 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 physiographic region, in which the Killarney 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 Figures 1 and 2. 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 B2-1 and B2-2 were advanced in the vicinity of the east and west abutments, respectively, from approximately Highway 400 grade. The locations and ground surface elevations for these borings are shown on the attached Drawing 1.

In summary, at the toe of the existing embankments, the boreholes encountered fill and topsoil, overlying a variable till deposit consisting of stiff to hard silty clay to clayey silt grading to very dense silty sand with depth. A significant interlayer of silty sand to sand and silt was encountered within this till deposit. A detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Fill and Topsoil

A layer of fill, 1.5 m to 2.1 m thick, was encountered in the boreholes. The fill ranges in composition from silty sand to sand and gravel, containing minor quantities of organics, clay and, in Borehole B2-2, asphalt pieces. The measured Standard Penetration Test (SPT) 'N' values ranged from 7 to 25 blows per 0.3 m of penetration, indicating that the fill has a loose to compact relative density.

A layer of topsoil was found underlying the fill in both boreholes; this topsoil is 800 mm and 300 mm thick in Boreholes B2-1 and B2-2, respectively. An organic content test on a sample of this topsoil gave an organic content of 7.3 per cent.

4.2.2 Silty Clay Till

An upper deposit of silty clay till, between 0.8 m and 3.6 m in thickness, was penetrated below the fill and topsoil in both boreholes. The top of the silty clay till was encountered between Elevation 283.9 m and 283.5 m. In Borehole B2-1 on the east side of the highway, the base of the silty clay till was encountered at about Elevation 283 m, overlying a silty sand to sand and silt interlayer. In Borehole B2-2 on the west side of the highway, the base of the silty clay till was encountered at about Elevation 280 m, directly overlying the lower hard clayey silt till.

The silty clay till contains trace to some sand and gravel. The results of a grain size distribution test carried out on a representative sample are shown on Figure 1. The natural moisture contents measured on samples of the silty clay till ranged from 22 to 24 per cent. Atterberg Limits testing on two samples measured plastic limits of 15 and 19 per cent, liquid limits of 35 and 39 per cent, and plasticity indices of about 20 per cent. The results of the Atterberg Limits testing indicate that this silty clay till is inorganic and of intermediate plasticity.

The SPT 'N' values measured in the silty clay till ranged from 15 to 23 blows per 0.3 m of penetration, indicating that this till has a stiff to very stiff consistency.

4.2.3 Silty Sand to Sand and Silt Interlayer

A 3.5 m thick layer of silty sand to sand and silt, containing trace clay and gravel, was encountered below a thin upper portion of silty clay till in Borehole B2-1, between about Elevation 283 m and 279.5 m. No corresponding layer was encountered in Borehole B2-2. It is considered that the silty sand to sand and silt may represent an interlayer or lens within the till deposit.

The results of a grain size distribution test carried out on a representative sample are shown on Figure 2. The natural moisture contents measured on two samples of the silty sand to sand and silt till were 13 and 14 per cent. The measured SPT 'N' values ranged from 9 to 82 blows per 0.3 m of penetration; however, it is considered that the measurement of 9 blows could have been impacted by groundwater inflow into the borehole. This interlayer is therefore considered to have a dense to very dense relative density, based on the other measured SPT 'N' values of 44 to 82 blows per 0.3 m of penetration.

4.2.4 Clayey Silt Till

A deposit of clayey silt till, containing trace to some sand and gravel, was encountered in the boreholes below the silty clay till and, where present, the silty sand to sand and silt interlayer. The surface of the clayey silt till deposit was encountered in the boreholes between Elevation 280 m and 279.5 m. The till extended to the maximum depth investigated in Borehole B2-1 (to about Elevation 275 m), and graded to silty sand till at about Elevation 277 m in Borehole B2-2.

Natural moisture contents of 7 and 8 per cent were measured on two samples of the clayey silt till. Atterberg Limits testing on one sample measured a plastic limit of 9 per cent, a liquid limit of 16 per cent, and a plasticity index of 7 per cent. The results of the Atterberg Limits testing indicate that this clayey silt till is inorganic and of low plasticity.

The SPT 'N' values measured in the clayey silt till were greater than 100 blows per 0.3 m of penetration, indicating that this deposit has a hard consistency.

4.2.5 Silty Sand Till

Silty sand till, containing trace gravel, was encountered at the base of Borehole B2-2. This layer was explored for a depth of 0.7 m. The measured natural moisture content on one sample of this till was 13 per cent. The measured SPT 'N' value of greater 100 blows per 0.3 m of penetration indicates that this till has a very dense relative density.

4.3 Groundwater Conditions

Water levels were observed in the open boreholes following completion of the drilling operations in October 2000. In Borehole B2-1, the measured water level was at 2.1 m depth (Elevation 284.1 m), about 1 m above the top of the water-bearing silty sand to sand and silt interlayer within the till. In Borehole B2-2, the water level was measured at 6 m depth (Elevation 279.9 m), at the top of the clayey silt portion of the till deposit.

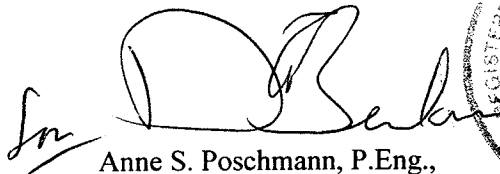
The groundwater level measured in the piezometer sealed at depth in Borehole B2-2 was at 1.9 m depth (Elevation 284 m) in January 2001 and at 1.1 m depth (Elevation 284.8 m) in March 2001. These readings most probably represent the piezometric groundwater level at the site.

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

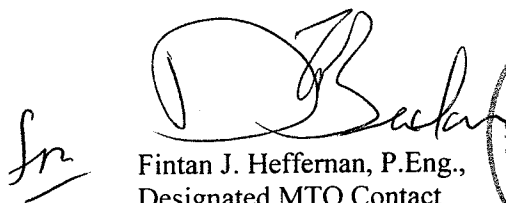
GOLDER ASSOCIATES LTD.



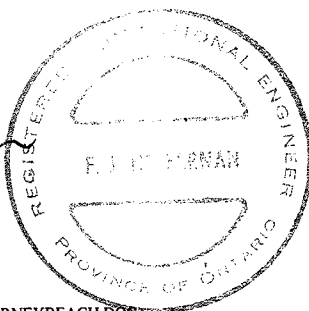
Lisa C. Coyne, P.Eng.,
Geotechnical Engineer



Anne S. Poschmann, P.Eng.,
Principal



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



LCC/JLS/ASP/FJH/clg

N:\ACTIVE\1100\001-1143F\RPT02-02\AN-KILLARNEYBEACH.DOC

PART B

**PRELIMINARY FOUNDATION DESIGN REPORT
KILLARNEY BEACH ROAD UNDERPASS
STRUCTURE SITE 30-212
HIGHWAY 400 WIDENING FROM 1 KM SOUTH
OF HIGHWAY 89 TO HIGHWAY 11
G.W.P. 30-95-00**

5.0 ENGINEERING RECOMMENDATIONS

5.1 General

This section of the report provides preliminary foundation design recommendations for the replacement of the existing Killarney Beach Road underpass 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. Replacement of the existing Killarney Beach Road underpass structure will therefore be necessary.

Based on the available general layout drawing for the existing single-span structure, the abutments and associated retaining walls are supported on spread footings founded at about Elevation 284.5 m. The Highway 400 grade is at about Elevation 285.5 m to 287 m, while the surrounding grade varies from about Elevation 285 m to 287 m. Killarney Beach Road has been constructed on embankment fill, with its grade at about Elevation 292.6 m over Highway 400.

5.2 Bridge Foundation Options

The subsoils at the site consist of fill and topsoil overlying an upper, stiff to very stiff silty clay till deposit which is underlain in the borehole on the east side of Highway 400 by a loose to very dense silty sand interlayer. These deposits are in turn underlain by a deposit of hard clayey silt till grading with depth to very dense silty sand till. The top of the hard / very dense till was encountered in the boreholes at about Elevation 280 m to 279.5 m, about 5 m to 7 m below Highway 400 grade. The measured groundwater level was as high as 1 m below ground surface, at approximately Elevation 285 m.

Consideration could be given to the use of perched abutments for the replacement structure, founded on spread footings which are placed on a compacted granular pad within the approach

embankment fill. Spread footings placed on native soils will be more difficult to adopt for the replacement structure, due to the relatively high groundwater table and the variable thickness of the stiff to very stiff silty clay till deposit across the site. However, if the design can accommodate lower design bearing values, consideration could be given to placing the footings on this stiff to very stiff silty clay till. If this option is adopted, further investigation will be required for final design to assess the variability of the silty clay till and the extent of the silty sand / sand and silt interlayer. Prior groundwater control will be required if this deposit is present at the proposed founding level. It is noted that it is not considered practicable to found spread footings on the hard clayey silt till deposit, as the top of this deposit is about 5 m to 7 m below the Highway 400 grade.

Alternatively, the abutments and any piers could be supported on steel H-piles driven to found within the hard lower clayey silt till. The soil conditions at this site are favourable for the use of an integral abutment replacement structure.

Preliminary recommendations for spread footings, including perched abutments, and for deep foundations are provided in the following sections.

5.3 Spread Footings

For preliminary design of the abutment and pier footings for the replacement structure, spread footings could be founded on the stiff to very stiff silty clay till or the silty sand to sand and silt interlayer where present, at or below Elevation 283.5 m. A minimum soil cover of 1.5 m should be provided. Any associated wing wall or retaining wall footings could 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 material would be necessary as a subgrade above Elevation 283.5 m.

Alternatively, consideration could be given to the use of abutment footings perched within the approach embankments.

5.3.1 Axial Geotechnical Resistance

Spread footings placed on the properly prepared silty clay till or, where present, silty sand to sand and silt interlayer, at or below the design elevation indicated above may be designed using a factored geotechnical resistance at Ultimate Limit States (ULS) of 300 kPa, assuming a 3 m wide footing. The settlement of the footings will be dependent on the footing size and configuration,

the applied loads, and the thickness and distribution of the stiff to very stiff silty clay till deposit. For preliminary design purposes, the geotechnical resistance at Serviceability Limit States (SLS) may be taken as 150 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' core, 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).

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. The angle of friction between the concrete and the undisturbed founding soils should be taken as 22 degrees; the corresponding coefficient of friction, $\tan \delta$, would then be 0.4. 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

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 new underpass structure on steel H-piles driven to found within the hard lower clayey silt till deposit. The surface of the clayey silt till was encountered in the boreholes at about Elevation 280 m, some 6 m below Highway 400 grade and

12 m below Killarney Beach Road grade. A design pile tip level of Elevation 278 m may be assumed 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 Elevation 278 m 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.

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:

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

If necessary, the pile caps could be perched within the embankment fill to provide a greater driven length. In this case, the piles would extend through the embankment fill, the stiff to very stiff upper silty clay till, and the generally dense to very dense silty sand to sand and silt interlayer, where present. For well-compacted cohesionless embankment fill and for the silty sand to sand and silt interlayer, the range in value of n_h may be taken as 5 MPa/m to 10 MPa/m in the structural analysis. For the stiff to very stiff silty clay till, below Elevation 283.5 m, the range in value of k_{s1} may be taken as 20 MPa/m to 45 MPa/m in the structural analysis.

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

The 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 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 Design

Based on the topographic information on the Engineering and Title Records plates and on site reconnaissance, the existing Killarney Beach Road embankment side slopes are formed at a

gradient of about 2 horizontal to 1 vertical (2H:1V). If any widening of the local road embankment will be required, the new side slopes also should be formed at a maximum gradient of 2H:1V. The widening of the embankment 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.

5.7 Design and Construction Considerations

5.7.1 Groundwater Control

Groundwater seepage into the footing or pile cap excavations will occur from the fill and from water-bearing lenses or interlayers of granular soil within the till deposit. Pumping from properly-filtered sumps or a filtered drain placed at the base of the excavation should provide sufficient groundwater control during excavation to foundation level within the stiff to very stiff silty clay till. Where the in-situ integrity of the silty sand to sand and silt interlayer is to be maintained, dewatering would be required prior to excavation.

The silty subgrade soils at the site are susceptible to disturbance from ponded water and construction traffic. Surface water run-off should be directed away from the footing excavations, and care must be exercised to avoid disturbance, particularly at and below founding level. Provision should be made in the Contract Documents for the placement of a lean concrete mat to protect the soils from such disturbance once the subgrade has been inspected and approved.

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, using proper groundwater control. The footing or pile cap excavations will extend a minimum of 1.5 m below lowest surrounding grade, generally through fill, the upper silty clay till and the silty sand to sand and silt interlayer, where present. The upper silty clay till and silty sand to sand and silt interlayer would be classified as Type 2 to 3 soils. Temporary open-cut slopes should be maintained no steeper than 1 horizontal to 1 vertical (1H:1V). For excavations formed within the silty clay till, the lower 1.2 m of the excavation sides may be formed near-vertical. Where space restrictions dictate, footing or pile cap excavations could also be carried out within a braced excavation.

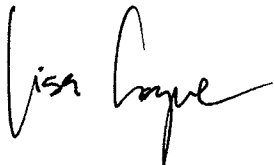
5.7.3 Settlement

Deformation of the ground due to foundation loading will result in settlement of the bridge piers, abutments and superstructure. The resulting differential settlement across the structure is a critical consideration in the design. The magnitude of deformation will be dependent on the type of foundations, the rigidity of the structural elements, the variability and consistency / relative density of the founding soils, and the activities during construction. The total and differential settlement will have to be reassessed during final design.

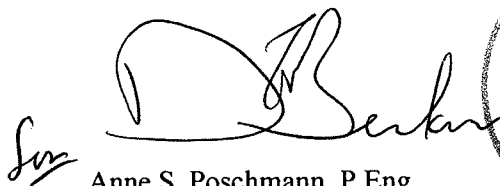
5.7.4 Obstructions

Although no cobbles or boulders were encountered during the borehole investigation, it should be noted that cobbles and boulders are inherent in glacially-derived materials. Cobbles and boulders should therefore be expected during driving of steel H-piles if a deep foundation option is selected.

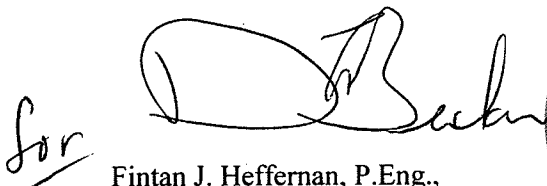
GOLDER ASSOCIATES LTD.



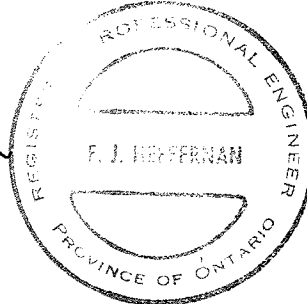
Lisa C. Coyne, P.Eng.,
Geotechnical Engineer



Anne S. Poschmann, P.Eng.,
Principal



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



LCC/JLS/ASP/FJH/clg

N:\ACTIVE\1100\001-1143F\RPT02-02JAN-KILLARNEYBEACH.DOC

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.

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 \times 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 = σ'_p / σ'_{vo}

(e) Shear Strength

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

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

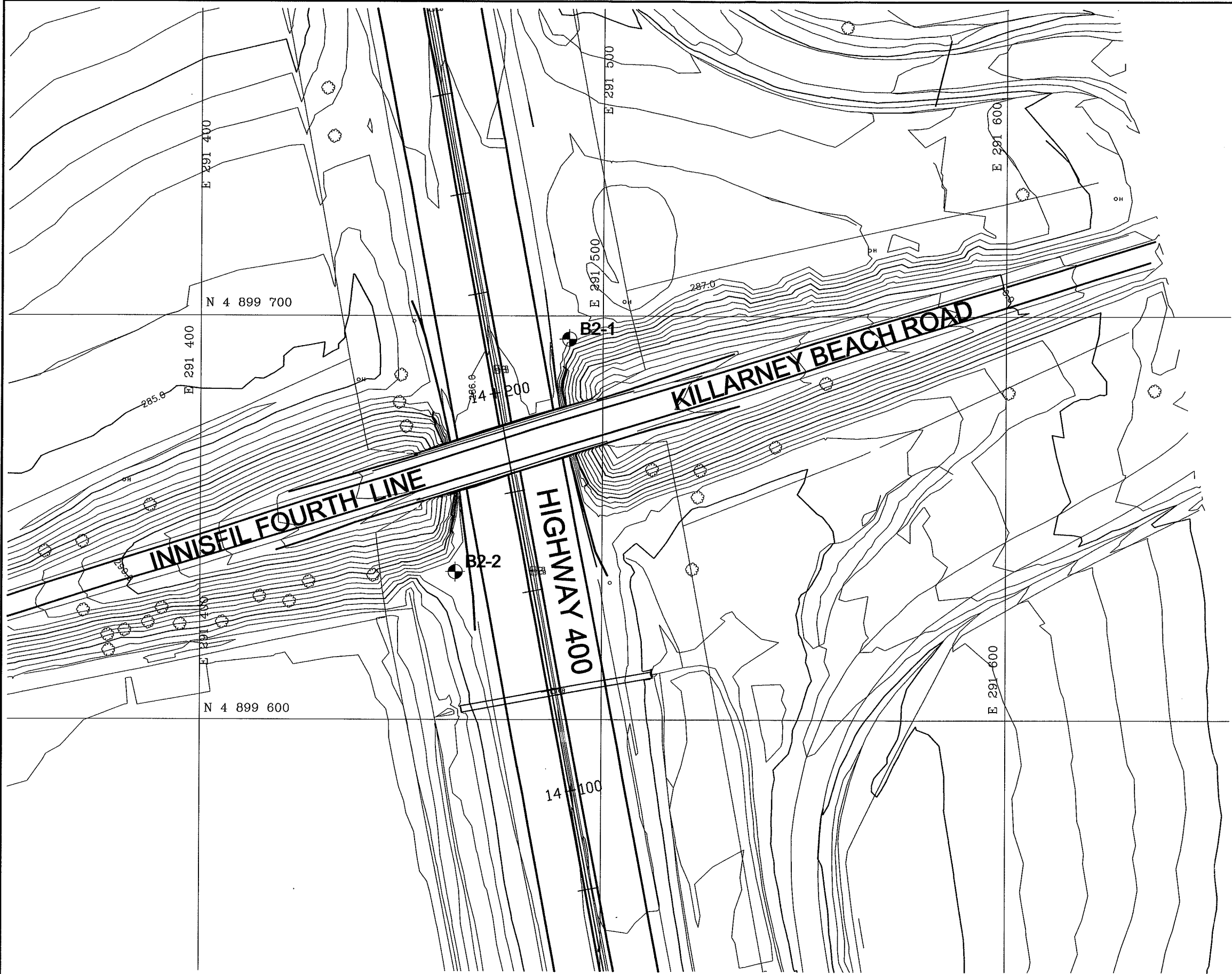
2. Shear strength = (Compressive strength)/2

PROJECT 001-1143F				RECORD OF BOREHOLE No B2-1				1 OF 1		METRIC			
W.P. 30-95-00				LOCATION N 4899694.3; E 291491.8				ORIGINATED BY AZ					
DIST SW HWY 400				BOREHOLE TYPE 108mm DIAMETER SOLID STEM AUGERS				COMPILED BY LCC					
DATUM Geodetic				DATE Oct.23/2000				CHECKED BY ASP					
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED						
286.2	GROUND SURFACE												
0.0	Silty Sand, some organics, trace clay and gravel (Fill) Compact Moist Brown/black		1	SS	18								
284.7	Topsoil		2	SS	13								
1.5													
283.9	Silty Clay, trace sand and gravel (Till) Very stiff Brown Moist		3	SS	21								
2.3													
283.1	Silty Sand to Sand and Silt, trace clay and gravel Loose to very dense Brown Moist to wet		4	SS	9								
3.1			5	SS	51								
			6	SS	44								
			7	SS	82								
279.6	Clayey Silt, some sand, trace gravel. (Till) Hard Grey Moist		8	SS	108/15								
6.6													
			9	SS	100/08								
			10	SS	100/15								
275.2	END OF BOREHOLE												
11.0	Note: Water level in open borehole at 2.1m depth (Elev.284.1m) on completion of drilling operations.												

ON_MOT_0011143F.GPJ_ON_MOT.GDT 14/1/02

PROJECT 001-1143F			RECORD OF BOREHOLE No B2-2			1 OF 1			METRIC						
W.P. 30-95-00			LOCATION N 4899636.5; E 291463.6			ORIGINATED BY PKS									
DIST SW HWY 400			BOREHOLE TYPE 108mm DIAMETER SOLID STEM AUGERS			COMPILED BY LCC									
DATUM Geodetic			DATE Oct.26/2000			CHECKED BY ASP									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED							
285.9	GROUND SURFACE														
0.0	Sand and Gravel, trace silt, trace clay pockets and asphalt pieces (Fill) Loose to compact Moist Brown		1	SS	7										
			2	SS	25										
283.8			3	SS	16										
283.5	Topsoil														
2.4	Silty Clay, trace sand and gravel (Till) Stiff to very stiff Moist Grey		4	SS	15										
			5	SS	15										
			6	SS	23										
			7	SS	15										
279.9															
6.0	Clayey Silt, trace to some sand and gravel (Till) Hard Moist Grey		8	SS	121										
			9	SS	108										
277.1															
8.8	Silty Sand, trace clay and gravel (Till) Very dense Moist Grey														
276.4			10	SS	100/23										
9.5	END OF BOREHOLE														
	Notes: 1. Water level in open borehole at 6m depth (Elev.279.9m) on completion of drilling operations. 2. Water level in piezometer at 1.9m depth (Elev.284.0m) on January 19, 2001, and at 1.1m depth (Elev.284.8m) on March 15, 2001.														

ON_MOT_0011143F.GPJ ON_MOT.GDT 14/1/02



DIST

HWY 400

CONT. No.

GWP No. 30-95-00

KILLARNEY BEACH RD. UNDERPASS

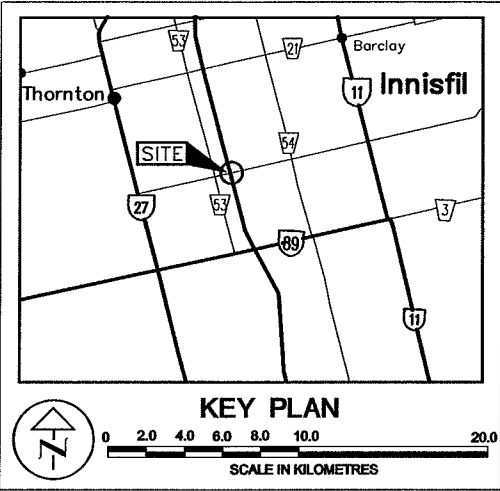
HWY 400

BOREHOLE LOCATION PLAN

SHEET

Golder Associates

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



LEGEND			
	Borehole, previous investigation		
	Borehole, present investigation		
No.	ELEVATION	LOCATION	
		NORTHING	EASTING
B2-1	286.2	4,899,694.3	291,491.8
B2-2	285.9	4,899,636.5	291,463.6

REFERENCE
This drawing was created from digital file "33808, 33809.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 30-212
DRAWN: MHW	CHKD. LCC	APPD. ASP	DWG. 1

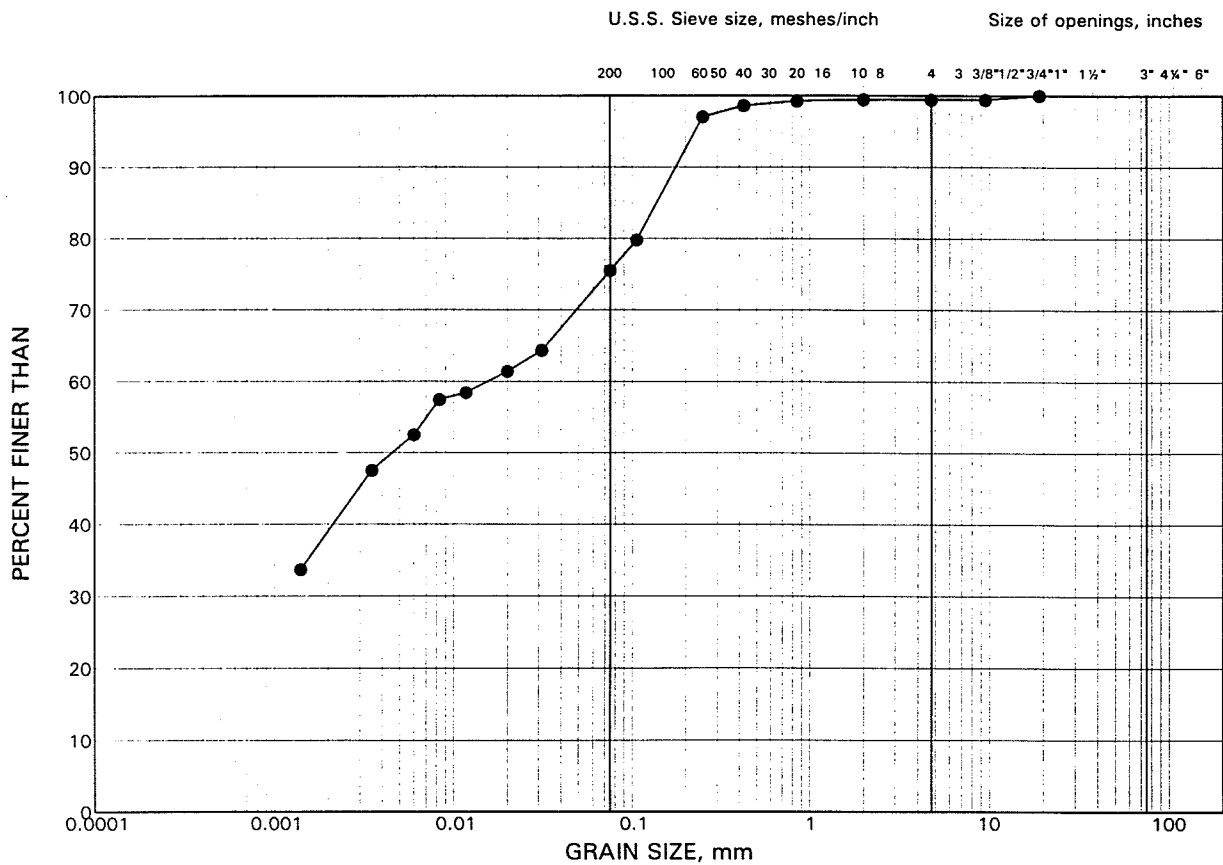
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

P1449F08.DWG

GRAIN SIZE DISTRIBUTION TEST RESULT

Silty Clay Till

FIGURE 1



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

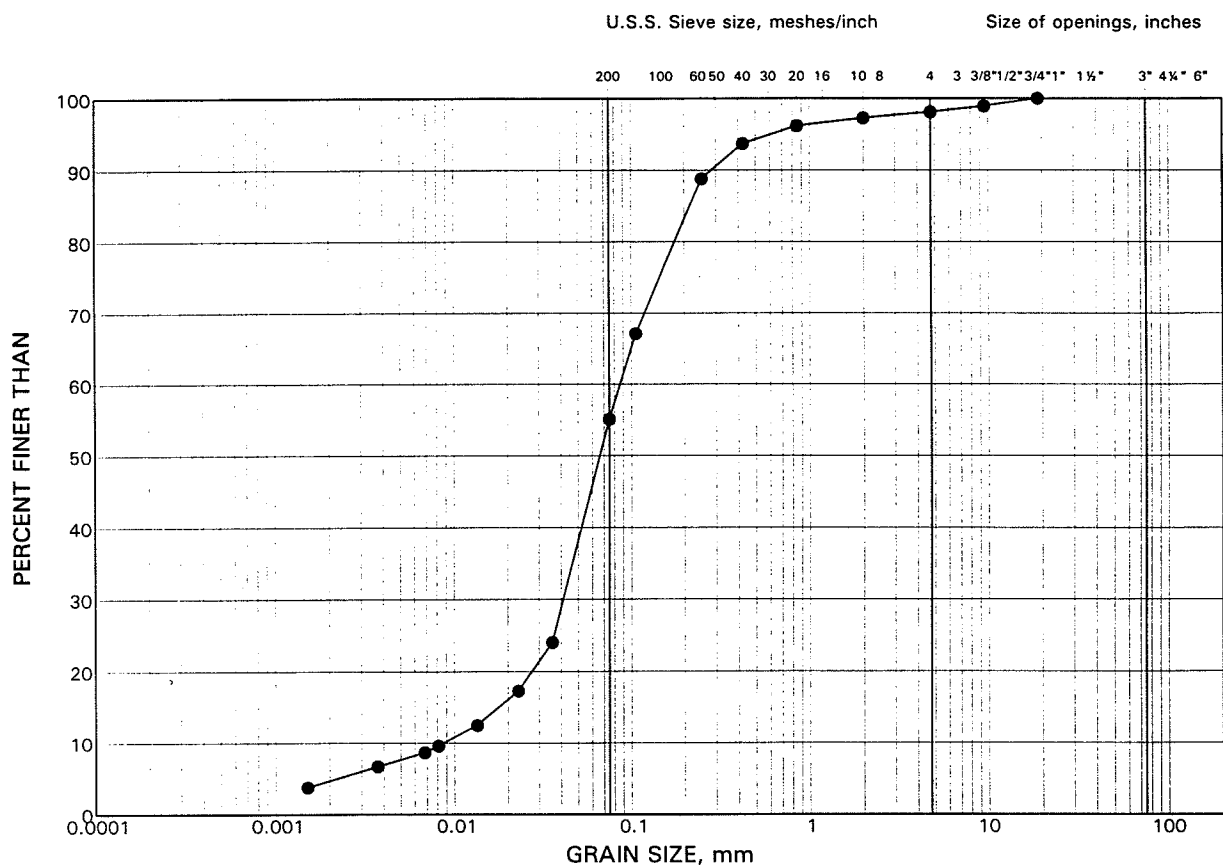
LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
•	B2-2	5	282.5

GRAIN SIZE DISTRIBUTION TEST RESULT

Sand and Silt

FIGURE 2



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
•	B2-1	5	282.1