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PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT VICTORIA STREET UNDERPASS STRUCTURE SITE 30-208 HIGHWAY 400 WIDENING FROM 1 KM SOUTH OF HIGHWAY 89 TO HIGHWAY 11 G.W.P. 30-95-00, AGREEMENT NO. 3005-A-000074

Submitted to:

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PART A - PRELIMINARY FOUNDATION INVESTIGATION REPORT

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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 Victoria Street (Innisfil Tenth Line, formerly Stroud 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 Victoria Street (Innisfil Tenth Line, also formerly known as Stroud Road) underpass structure is located about 2.8 km north of the Innisfil Beach Road (Simcoe Road 21) interchange and 2.8 km south of the Molson Park Drive interchange, in the Town of Innisfil, Simcoe County. The MTO has designated this underpass as Structure Site No. 30-208.

In the vicinity of this structure site, Highway 400 has been constructed in a 3 m to 9 m deep cut. The Highway 400 grade declines northward from about Elevation 304 m to 303.5 m within the structure limits. Victoria Street is near the original ground surface, with its grade at about Elevation 310 m over Highway 400.

The two-span underpass structure was constructed in the late 1940s under Contract 46-48. According to the general layout drawing for this contract, which was provided by Morrison Hershfield (the structural designers for this preliminary study), the abutments, associated wing walls and the pier are supported on spread footings founded at about Elevation 302.5 m to 302.2 m, declining toward the east. The existing spread footings are approximately 2.7 m wide at the abutments, and 1.8 m wide at the centre pier.

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out at this site in October 2000, at which time two boreholes were drilled. Boreholes B6-1 and B6-2 were advanced on the east and west sides of Highway 400 to depths of 5 m and 8 m, respectively, below the Highway 400 cut grade.

The investigation was carried out using bombardier-mounted D-50 and B-57 drill rigs supplied and operated by Master Soil Investigations Ltd. of Weston, Ontario. Both 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 throughout the drilling operations, and a piezometer was installed in Borehole B6-1 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, 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 Victoria Street 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 B6-1 and B6-2 were advanced on the east and west sides of Highway 400, respectively, from approximately Highway 400 grade. The locations and ground surface elevations for these borings are shown on the attached Drawing 1.

In summary, the soils below the Highway 400 cut at this site consist of a very dense deposit of silty sand to silt. On the east side of the highway, 1.2 m of silty clay till was encountered underlying the cohesionless deposit, while on the west side of the highway, the silty sand to silt deposit grades to sand and gravel at about 7 m depth (approximately Elevation 297.5 m). A detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Topsoil and Silty Sand, Some Organics

About 300 mm of topsoil was encountered in Borehole B6-1 on the east side of Highway 400. On the west side of the highway, Borehole B6-2 encountered an upper 600 mm layer of silty sand containing some organics. This silty sand is loose, with a measured Standard Penetration Test (SPT) 'N' value of 7 blows per 0.3 m of penetration.

4.2.2 Silty Sand to Silt, Some Sand

A cohesionless soil deposit, about 3.5 m to 6.5 m thick, was encountered below the topsoil and loose, organic silty sand. The deposit ranges in composition from silty sand to silt containing some sand. Layers of silty clay, up to 75 mm in thickness, were encountered within the silty sand to silt deposit in some of the samples. The grain size distribution test results obtained on representative samples of this deposit are shown on Figure 1.

The silty sand to silt soils were generally moist, with measured natural moisture contents ranging from 5 to 15 per cent. The soils became wet below about 3.2 m depth (Elevation 301.7 m) in Borehole B6-2.

The measured SPT 'N' values ranged from 50 to 190 blows per 0.3 m of penetration, but were typically greater than 100 blows per 0.3 m of penetration. The silty sand to silt deposit therefore has a very dense relative density.

4.2.3 Silty Clay Till

A deposit of silty clay till containing trace quantities of sand and gravel, as well as silt and sand seams, underlies the silty sand to silt deposit in Borehole B6-1. The top of the silty clay till was encountered at 3.7 m depth (Elevation 300.1 m). The deposit was not fully penetrated by the boring, but it is at least 1.2 m thick.

Atterberg Limits testing on a sample of the till measured a plastic limit of 16 per cent, a liquid limit of 28 per cent, and a plasticity index of 12 per cent. The results of the Atterberg Limits testing indicate that the silty clay till is inorganic and of low plasticity. The natural moisture content measured on a sample of the till was 23 per cent; this water content is considered to have been influenced by the presence of sand and silt seams within the till.

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

4.2.4 Sand and Gravel

A 0.7 m thick layer of sand and gravel was encountered at the base of Borehole B6-2, underlying the silty sand to silt deposit. This sand and gravel layer is very dense, with a measured SPT 'N' value of greater than 100 blows per 0.3 m of penetration.

4.3 Groundwater Conditions

Recovered samples from the silty sand to silt deposit were generally moist, although the samples became wet below about 3.2 m depth in Borehole B6-1. This increase in moisture content likely reflects perched water atop the silty clay till that was encountered in this borehole. It is possible that a shallow surficial aquifer exists within the sandy silt to silt deposit during wet periods of the year. However, both boreholes were dry on completion of the drilling operations in October 2000, and the piezometer sealed within the sandy silt to silt deposit in Borehole B6-1 was dry when measured in March 2001.

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PART B

PRELIMINARY FOUNDATION DESIGN REPORT
VICTORIA STREET UNDERPASS
STRUCTURE SITE 30-208
HIGHWAY 400 WIDENING FROM 1 KM SOUTH OF HIGHWAY 89 TO HIGHWAY 11
G.W.P. 30-95-00, AGREEMENT NO. 3005-A-000074

5.0 ENGINEERING RECOMMENDATIONS

5.1 General

This section of the report provides preliminary foundation design recommendations for the replacement of the existing Victoria Street (Innisfil Tenth Line) 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 Victoria Street underpass structure will therefore be necessary.

Based on the general layout drawing for the existing two-span structure, the pier, abutments and associated retaining and wing walls are supported on spread footings founded at about Elevation 302.5 m to 302.2 m, declining eastward. Highway 400 has been constructed in cut, with its grade at about Elevation 304 m to 303.5 m within the structure limits. Victoria Street is near the original ground surface at the site, with its grade at about Elevation 310 m over Highway 400.

5.2 Bridge Foundation Options

The soils below the Highway 400 level consist of a thin layer of topsoil and organic silty sand overlying a deposit of very dense silty sand to silt, which in turn overlies deposits of hard silty clay till and very dense sand and gravel.

Based on these subsurface conditions, it is recommended that the new structure be founded on spread footings placed on the silty sand to silt stratum. 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, if integral abutments are under consideration for the replacement structure, the abutments could be supported on steel H-piles driven to found within the very dense silty sand to silt deposit. In this case, it is assumed that the pile cap would be perched above the existing Highway 400 grade in order to achieve the required pile length.

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

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 bridge abutment and pier footings, spread footings may be founded on the very dense silty sand to silt deposit at or below Elevation 303 m; a minimum soil cover of 1.5 m should be provided. 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 within the approach embankments.

5.3.1 Axial Geotechnical Resistance

Spread footings placed on the properly prepared silty sand to silt deposit at the design elevation given above may be designed using a factored geotechnical resistance at Ultimate Limit States (ULS) of 1,000 kPa, assuming a 3 m wide footing. The settlement of footings founded on the silty sand to sandy silt will be dependent on the footing size and configuration, and on the applied loads. The majority of this settlement will take place during construction itself; for preliminary design purposes, the geotechnical resistance at Serviceability Limit States (SLS) may be taken as 800 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 (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 silty sand to silt founding soils should be taken as 24 degrees; the corresponding coefficient of friction, tan δ, 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 new underpass structure on steel H-piles driven to found within the very dense silty sand to silt deposit or the hard silty clay deposit. Based on the results of Boreholes B6-1 and B6-2, which were drilled from Highway 400 grade, it is anticipated that an adequate driving resistance would not be achieved until at least Elevation 300 m, i.e. 3 m of penetration into the deposit. This would put the pile founding level more than 10 m below Victoria Street grade and 4 m below Highway 400 grade.

It is noted that additional borehole investigation will be required at the proposed abutment locations during detailed design in order to determine the composition and consistency of the upper soils through which the piles would be driven.

5.4.1 Axial Geotechnical Resistance

For preliminary design, a pile founding level of Elevation 299 m may be assumed. The factored axial resistance at ULS for steel HP 310 x 110 H-piles driven to this design tip elevation may be taken as 1,400 kN. The axial resistance at SLS for a single pile, for 25 mm of settlement, may be taken as 1,200 kN.

As a guide, to achieve the above design resistances, the piles should be driven to a final set of no less than 10 blows per 25 mm of penetration using a hammer with rated energy of about 50 kJ, and not exceeding 60 kJ. The actual set criteria should be established based on the Contractor's pile driving equipment. 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:

$$k_h = \frac{n_h z}{B}$$

where

n_h is the constant of subgrade reaction
z is the depth (m)
B is the pile diameter (m)

It is noted that the soils above the Highway 400 cut level were not investigated during this preliminary subsurface investigation stage. Based on regional geological mapping, it is expected that the surficial subsoils will consist of silts, sands and gravels. For these materials, the range in value n_h may be taken as 10 MPa/m to 20 MPa/m in the structural analysis. This estimated range of values will require confirmation following the detailed design stage of 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:

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

5.4.3 Frost Protection

The pile caps should be provided with 1.5 m 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 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 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 Permanent Cut Slopes

Boreholes B6-1 and B6-2 were advanced from the Highway 400 cut grade at about Elevation 304 m to 305 m, and so no borehole information is available regarding the soils which will comprise the permanent cut slopes along the east and west sides of Highway 400. Based on regional geological mapping, it is anticipated that the cut will be formed in silt, sand and gravel soils, similar to that encountered below Elevation 304 m.

Based on the topographic information on the Engineering and Title Records plates and on site reconnaissance, the existing slopes are generally formed at a gradient of 2 to 3 horizontal to 1 vertical (2H:1V to 3H:1V). For preliminary design purposes, a maximum gradient of 2.5H:1V may be assumed for the new permanent cut slopes. This design recommendation will have to be confirmed during the detailed design stage of the subsurface investigation program.

It is noted that the fine sands and silts which will comprise the cut slopes will be readily erodible. The slope surfaces should be grassed, and drainage should be provided from the top of the slope to the ditch line.

5.7 Design and Construction Considerations

5.7.1 Dewatering

Groundwater seepage into the footing or pile cap excavations 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 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 placement of a lean concrete mat to protect the soils from such disturbance.

5.7.2 Excavation

The footing or pile cap excavations will extend a minimum of 1.5 m below lowest surrounding grade, generally through very dense silty sand to silt soils below the Highway 400 grade. 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 soils below the Highway 400 grade would be classified as Type 2 to 3 soil; for the soils below Victoria Street grade, the type and consistency / relative density will have to be confirmed during the detailed design stage. Temporary open-cut slopes should be maintained no steeper than 1 horizontal to 1 vertical (1H:1V). Where space restrictions dictate, footing or pile cap excavations could also be carried out within a braced excavation.

5.7.3 Obstructions

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

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LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Dynamic Cone Penetration Resistance; N_d:

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

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

Piezo-Cone Penetration Test (CPT)

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

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

(b) Cohesive Soils

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

IV. SOIL TESTS

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

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

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LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

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

* Density symbol is ρ. Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)

(a) Index Properties (con't.)

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

(c) Hydraulic Properties

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

(d) Consolidation (one-dimensional)

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

(e) Shear Strength

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

Notes: 1. $\tau = c' + \sigma' \tan \phi'$
2. Shear strength = (Compressive strength)/2

Ministry of Transportation
Ontario

Foundation Design

RECORD OF BOREHOLE No B6-1

METRIC

PROJECT 001-1143F

W.P. 30-95-00 LOCATION N 4907794.9; E 290179.1 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

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
303.8	GROUND SURFACE									
303.5 / 0.0	Topsoil									
0.3	Sandy Silt to Silt, some sand, containing thin clay layers below 3.1m depth; Very dense; Brown; Moist becoming wet at 3.2m depth				303					
		1	SS	87						
		2	SS	125	302					
		3	SS	190			0	19	78	3
		4	SS	99/.15	301					
300.1 / 3.7	Silty Clay, trace sand and gravel (Till), containing silt and sand seams; Hard; Brown; Moist				300					
		5	SS	165						
298.9 / 4.9		6	SS	128/.15	299					

END OF BOREHOLE

Notes:
1. Borehole dry on completion of drilling operations.
2. Piezometer dry on March 15, 2001.

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

$+^3, \times^3$: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

Ministry of Transportation Ontario

RECORD OF BOREHOLE No B6-2

METRIC

PROJECT 001-1143F			ORIGINATED BY PKS
W.P. 30-95-00	LOCATION N 4907743.3; E 290135.1		COMPILED BY LCC
DIST SW	HWY 400	BOREHOLE TYPE 108mm DIAMETER SOLID STEM AUGERS	CHECKED BY ASP
DATUM Geodetic	DATE Oct.26/2000		

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
304.9	GROUND SURFACE						
0.0	Silty Sand, trace gravel, some organics Loose Moist Brown	1	SS	7			
304.3 0.6	Silty Sand Very dense Moist Brown	2	SS	50	304		
		3	SS	100	303	o	0 84 16 0
		4	SS	147	302		
		5	SS	114		o	
		6	SS	100/.15	301		
		7	SS	95/.23	300	o	
	Contains 50mm to 75mm thick layers of clayey silt below 4.9m depth				299		
		8	SS	105		o	
297.6 7.3	Sand and Gravel Very dense Moist Brown				298		
296.9		9	SS	105/.25	297		
8.0	END OF BOREHOLE Note: Borehole dry on completion of drilling.						

Column headings: GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L, WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³

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

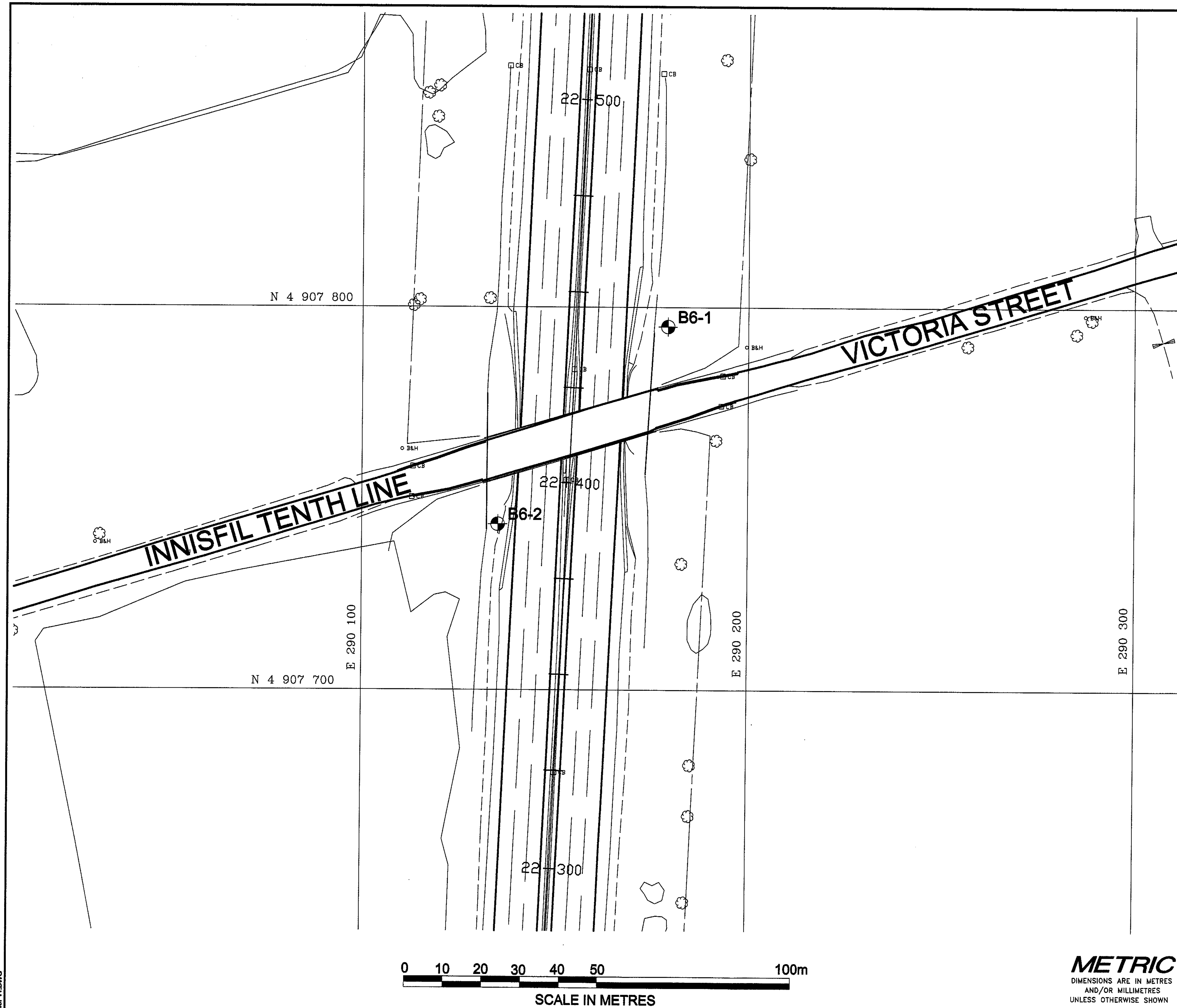
DIST HWY 400
CONT. No.
GWP No. 30-95-00

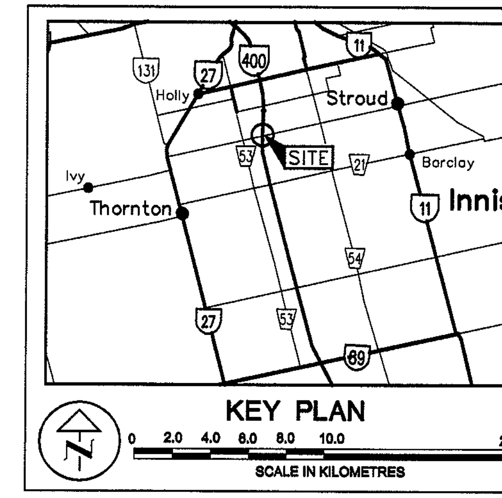
SHEET

INNISFIL TENTH LINE UNDERPASS
HWY 400
BOREHOLE LOCATION PLAN

Golder Associates

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA





KEY PLAN

LEGEND

- Borehole, previous investigation
- Borehole, present investigation

No.	ELEVATION	LOCATION NORTHING	LOCATION EASTING
B6-1	303.8	4,907,794.9	290,179.1
B6-2	304.9	4,907,743.3	290,135.1

REFERENCE
This drawing was created from digital file "50203.dwg" provided by URS Cole Sherman

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

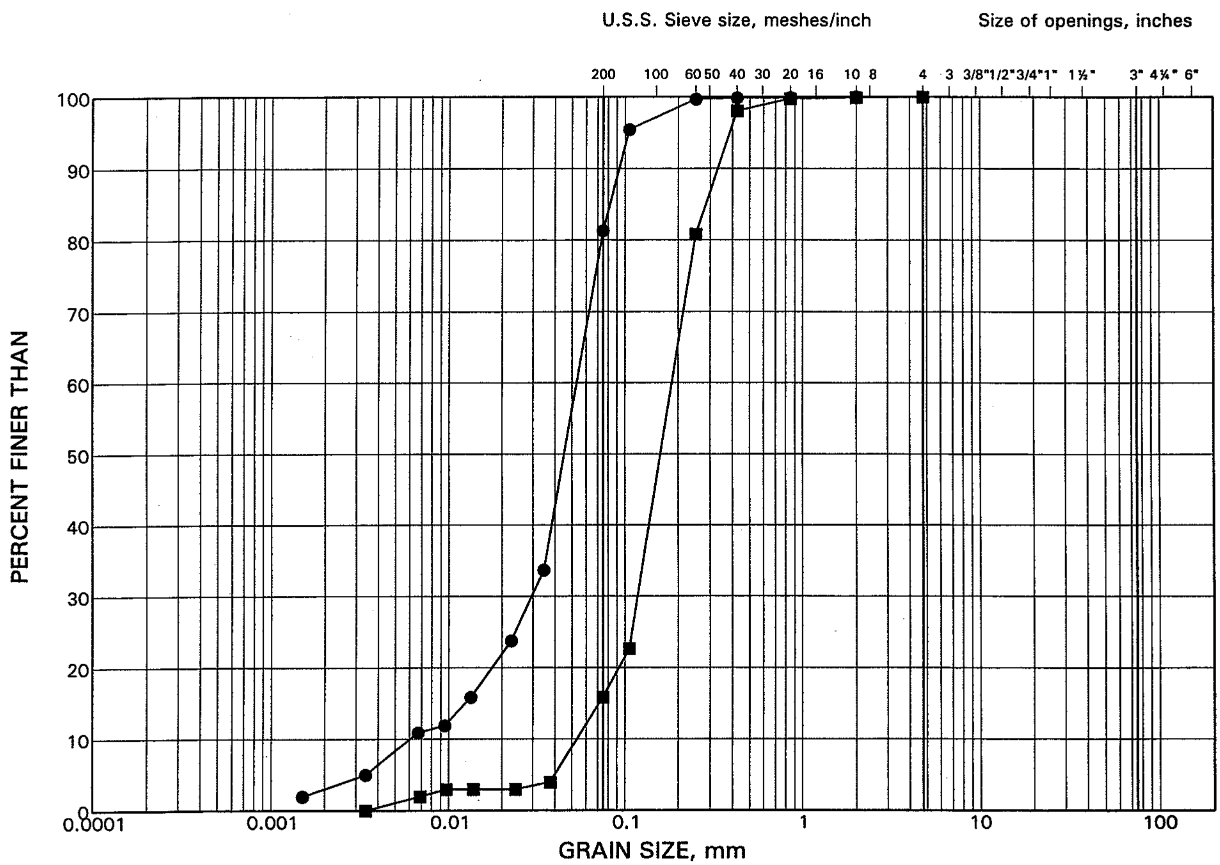
NO.	DATE	BY	REVISION

Geocres No.

HWY. No. 400		PROJECT NO.: 001-1143F	
SUBM'D. LCC	CHKD: ASP	DATE: JANUARY 2001	SITE 30-208
DRAWN: MHW	CHKD. LCC	APPD. ASP	DWG. 1

P1143F1.DWG

GRAIN SIZE DISTRIBUTION TEST RESULTS Silty Sand to Silt, some sand	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)
●	B6-1	3	301.3
■	B6-2	3	303.2

Project 001-1143F

Golder Associates