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 HIGHWAY 89 UNDERPASS STRUCTURE SITE 30-256 HIGHWAY 400 WIDENING FROM 1 KM SOUTH OF HIGHWAY 89 TO HIGHWAY 11 G.W.P. 30-95-00

Submitted to:

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

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TABLE OF CONTENTS

SECTION **PAGE**

TABLE OF CONTENTS

LIST OF DRAWINGS

LIST OF FIGURES

PART A

PRELIMINARY FOUNDATION INVESTIGATION REPORT
HIGHWAY 89 UNDERPASS
STRUCTURE SITE 30-256
HIGHWAY 400 WIDENING FROM 1 KM SOUTH OF HIGHWAY 89 TO HIGHWAY 11
G.W.P. 30-95-00

TABLE OF CONTENTS

SECTION **PAGE**

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 Highway 89 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 Highway 89 underpass structure is located about 11 km north of the Simcoe Road 88 interchange, and about 7 km south of the Innisfil Beach Road interchange, in the Town of Innisfil, Simcoe County. The MTO has designated this underpass as Structure Site No. 30-256.

At this existing structure, the Highway 400 grade is at about Elevation 229 m to 229.5 m. Highway 89 has been constructed in fill, with approach embankments up to about 7 m in height. The Highway 89 grade over Highway 400 is at about Elevation 236 m.

According to the general layout drawing for this existing single-span underpass structure (provided by Morrison Hershfield, the structural designers for this preliminary design study), the abutments and associated wing walls are supported on spread footings which are founded at about Elevation 227.7 m.

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out at this site in December 2000, at which time two boreholes were drilled. Boreholes B1-1 and B1-2 were advanced on the east and west sides of Highway 400 to depths of 28 m and 37 m, respectively, below the Highway 400 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. Borehole B1-1 was advanced using solid stem augers; this borehole was terminated at 28 m depth due to "tightening" of the augers in the soil. In Borehole B1-2, hollow stem augers were used initially to 7.5 m depth, then the borehole was advanced using casing with wash boring. 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 B1-2 to permit monitoring of the groundwater levels at the site.

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

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

4.0 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 Highway 89 site is located within the southern of these two lobes of the Simcoe Lowlands.

The surficial soils in the Peterborough Drumlin Field 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 B1-1 and B1-2 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 Highway 89 embankment fill at the site overlies a deposit of silty sand to sand silt, underlain by a silty clay deposit, which in turn overlies a lower silty sand deposit. A detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Fill

About 1.4 m and 2.3 m of fill and/or reworked soil was encountered in the boreholes drilled at the north toe of the Highway 89 embankments on the east and west sides of Highway 400, respectively. The fill consists of silty sand, containing minor quantities of gravel, pockets of silty clay, and organics. The measured Standard Penetration Test (SPT) 'N' values ranged from 8 to 20 blows per 0.3 m of penetration, indicating that this fill has a loose to compact relative density.

4.2.2 Silty Sand to Sandy Silt

A silty sand to sandy silt deposit, 17.5 m to 19.5 m thick, was encountered below the fill in both boreholes at about Elevation 226 m. The base of the deposit was encountered between Elevation 209 m and 206.5 m. The silty sand to sandy silt contains trace clay and gravel. Layers of silty clay, 25 mm to 100 mm in thickness, were encountered within the silty sand to sandy silt 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 sandy silt soils are wet, with measured natural moisture contents ranging from 19 to 21 per cent.

The measured SPT 'N' values ranged from 13 to 46 blows per 0.3 m of penetration, with an average of 25 blows per 0.3 m of penetration. The silty sand to sandy silt deposit has a compact to dense, but predominantly compact, relative density.

4.2.3 Silty Clay

A deposit of silty clay containing trace sand underlies the silty sand to sandy silt deposit. The top of the silty clay was encountered at about 20 m to 22 m depth (Elevation 209 m to 206.5 m). The deposit was fully penetrated in Borehole B1-2, where it was about 11.5 m thick. In Borehole B1-1, the silty clay is at least 8 m thick.

The natural moisture contents measured on samples of the silty clay ranged from 23 to 36 per cent. Atterberg Limits testing on two samples measured plastic limits of 15 and 18 per cent, liquid limits of 33 and 43 per cent, and plasticity indices of 18 and 25 per cent. The results of the Atterberg Limits testing indicate that the silty clay is inorganic and of low to intermediate plasticity.

The measured SPT 'N' values ranged from 6 to 40 blows, but were typically between 6 and 19 blows per 0.3 m of penetration. Field vane testing carried out within the softer portion of the deposit, below about 25 m in Borehole B1-2, measured undrained shear strengths of 80 kPa to 90 kPa. The results of the SPT and in-situ vane shear strength testing indicate that the silty clay deposit has a generally stiff to very stiff consistency.

4.2.4 Lower Silty Sand

A lower deposit of silty sand, containing thin silty clay layers, was encountered below 33.5 m depth (Elevation 195 m) in Borehole B1-2. The deposit was not fully penetrated by this boring, but it is at least 3.5 m thick. The recovered silty sand samples were wet, with a measured natural moisture content of 21 per cent on one sample.

This lower silty sand deposit is very dense, with measured SPT 'N' values of 52 and 72 blows per 0.3 m of penetration.

4.3 Groundwater Conditions

Recovered samples from both the upper silty sand to sandy silt deposit and the lower silty sand deposit were wet; both deposits comprise aquifers at the site, separated by the silty clay deposit. The water levels were observed in the open boreholes following drilling operations in December 2000. At that time, the measured water levels were at 2.3 m and 2.7 m depth, at or slightly below the top of the silty sand to sandy silt deposit in the boreholes. The groundwater level measured in the piezometer installed in Borehole B1-2 was at 1.8 m depth (Elevation 226.6 m) in January 2001, and at 1.3 m depth (Elevation 227.1 m) in March 2001.

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

GOLDER ASSOCIATES LTD.

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

for Anne S. Poschmann, P.Eng.,
Principal

REGISTERED PROFESSIONAL ENGINEER
A S POSCHMANN
PROVINCE OF ONTARIO

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

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PART B

PRELIMINARY FOUNDATION DESIGN REPORT
HIGHWAY 89 UNDERPASS
STRUCTURE SITE 30-256
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 Highway 89 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, necessitating replacement of the existing Highway 89 underpass structure.

Based on the general layout drawing for the existing single-span structure, the abutments and associated wing walls are supported on spread footings founded at about Elevation 227.7 m. The Highway 400 grade is at about Elevation 229 m to 229.5 m, and Highway 89 has been constructed in fill with approach embankments up to about 7 m in height. The Highway 89 grade over Highway 400 is at about Elevation 236 m.

5.2 Bridge Foundation Options

The soils at the site consist of fill overlying a generally compact silty sand to sandy silt deposit, which is about 18 m to 20 m thick. This upper cohesionless deposit is water-bearing, with the measured groundwater level at about Elevation 227 m, near the top of the deposit and about 2 m to 2.5 m below Highway 400 grade. This silty sand to sandy silt overlies a generally stiff to very stiff silty clay deposit, which is in turn underlain by a lower, very dense silty sand deposit. The top of this very dense silty sand is at about Elevation 195 m.

Based on these subsurface conditions, it is recommended that the new structure be founded on spread footings placed on the upper silty sand to sandy silt stratum. The spread footings should be placed as high as possible, to minimize the requirements for dewatering and the potential for disturbance to the founding soils due to the high groundwater level. In this regard, consideration could also be given to the use of perched abutments, founded on spread footings placed on a compacted granular pad within the approach fill.

Consideration could also be given to the use of deep foundations, such as steel "H" piles driven to found within the very dense, lower silty sand deposit at this site. It is noted that this lower silty sand deposit is almost 40 m below the Highway 89 grade.

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

5.3 Spread Footings

For preliminary design of the bridge abutment and pier footings, spread footings may be placed at a design founding level of Elevation 227 m, to be founded below the fill / reworked soil, on the generally compact silty sand to sandy silt deposit. 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 for support of the wall footings. It is noted that the design founding level given herein is affected by the base of fill elevation as encountered in Borehole B1-2, drilled at the north toe of the Highway 89 embankment. This founding elevation is coincident with the groundwater level measured in Borehole B1-2 in March 2001. If, during the detailed design stage of subsurface investigation, native soil is encountered higher at the proposed foundation locations, consideration could be given to raising the design founding level to minimize the dewatering requirements and the potential for disturbance of the founding soils.

To avoid the difficulties associated with groundwater control and potential disturbance to the silty sand to sandy silt soils, consideration should also be given to the use of abutment footings perched on the embankment fill.

5.3.1 Axial Geotechnical Resistance

Spread footings placed on the properly prepared silty sand to sandy silt deposit at Elevation 227 m may be designed using a factored geotechnical resistance at Ultimate Limit States (ULS) of 500 kPa, assuming a 4 m wide footing. The settlement of footings founded on the silty sand to sandy silt will be dependent on the footing size and configuration, on the degree of disturbance caused by construction activities, and on the applied loads. The majority of this settlement is expected to take place during the construction period; for preliminary design purposes, the geotechnical resistance at Serviceability Limit States (SLS) may be taken as 250 kPa for a design founding level of Elevation 227 m. 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 founding soils should be taken as 24 degrees; the corresponding coefficient of friction, tan δ, would then be 0.45. 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 abutments and pier of the replacement structure on steel H-piles driven to found within the very dense lower silty sand deposit, which was encountered below Elevation 195 m in Borehole B1-2. For preliminary design, a pile tip elevation of 190 m may be assumed.

5.4.1 Axial Geotechnical Resistance

For preliminary design of deep foundations, the factored axial resistance at ULS for steel HP 310 x 110 H-piles driven into the very dense lower silty sand may be taken as 900 kN. The axial resistance at SLS for 25 mm of settlement, for a single pile, may be taken as 800 kN.

To achieve the above design resistances, the piles should be driven to a tip elevation of at least 190 m. 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)

Based on the borehole data, the subsoils in front of the upper portion of the piles will consist of embankment fill overlying generally compact silty sand to sand silt soils. For the embankment fill, the range in value of n_h may be taken as 5 MPa/m to 10 MPa/m in the structural analysis. For the water-bearing, generally compact silty sand to sandy silt deposit, the range in value of n_h may also be taken as 5 MPa/m to 10 MPa/m. 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:

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

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 wall. 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 Highway 89 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 should be formed at a maximum gradient of 2H:1V. The construction 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

The abutment and pier footing excavations will extend close to water-bearing silty sand to sandy silt soils (unless the abutment footings are designed to be perched on a granular pad within the approach embankment fill). A dewatering scheme will be necessary to facilitate footing excavation and concrete placement in dry conditions. In this regard, it will be necessary to lower the groundwater level to at least 0.5 m below the design founding level prior to excavation and construction of the footings.

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

5.7.2 Excavation

Footing excavations will extend a minimum of 1.5 m below lowest surrounding grade, through existing fill and into generally compact, water-bearing silty sand to sandy silt soils. 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. With proper dewatering or a temporary cut-off wall, the silty sand to sandy silt deposits at this site would be classified as Type 3 soil. Temporary open-cut slopes in the dewatered soil should be maintained no steeper than 1 horizontal to 1 vertical (1H:1V). Where space restrictions dictate, the foundation works could also be carried out within a braced excavation.

5.7.3 Settlement

Foundation loading will result in deformation of the ground and consequent settlement of the foundation elements and superstructure. Differential settlement could occur between the abutments and pier locations, depending on the variability and consistency/relative density of the foundation soils, on the types of foundations selected, and on the degree of disturbance caused by activities during construction. The potential for differential settlement should be reassessed during the detailed design stage, once the proposed bridge configuration is established.

GOLDER ASSOCIATES LTD.

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

for Anne S. Poschmann, P.Eng.,
Principal

REGISTERED PROFESSIONAL ENGINEER
A. S. POSCHMANN
PROVINCE OF ONTARIO

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

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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 δ
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

RECORD OF BOREHOLE No B1-1

METRIC

PROJECT 001-1143F
W.P. 30-95-00
LOCATION N 4895635.8; E 292452.1
ORIGINATED BY PKS
DIST SW HWY 400
BOREHOLE TYPE 108mm DIAMETER SOLID STEM AUGERS
COMPILED BY LCC
DATUM Geodetic
DATE Dec.14-18/2000
CHECKED BY ASP

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
228.9	GROUND SURFACE									
0.0	Silty Sand, trace to some gravel with pockets of clay (Fill) Loose to compact Moist Brown	1	SS	8						
		2	SS	20	228					
227.7 / 1.2	Silty Sand to Sandy Silt, trace gravel, trace clay, containing silty clay layers Compact to dense Wet Brown to grey	3	SS	13	227	o				
		4	SS	39	226	o	0	55	43	2
		5	SS	46						
	Becoming grey below 3.8m depth	6	SS	27	225					
		7	SS	32	224	o	0	24	74	2
					223					
		8	SS	27	222					
		9	SS	30	221	o				
					220					
		10	SS	37	219					
	Increasing frequency of silty clay layers, 25mm to 100mm in thickness, below 10.7m depth.	11	SS	33	218	o				
					217					
		12	SS	34	216					
		13	SS	22	215					
					214					

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ 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; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued Next Page

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

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No B1-1

METRIC

PROJECT 001-1143F

W.P. 30-95-00

DIST SW HWY 400

DATUM Geodetic

LOCATION N 4895635.8; E 292452.1

BOREHOLE TYPE 108mm DIAMETER SOLID STEM AUGERS

DATE Dec.14-18/2000

ORIGINATED BY PKS

COMPILED BY LCC

CHECKED BY ASP

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%)
	— CONTINUED FROM PREVIOUS PAGE —							
	Silty Sand to Sandy Silt, trace gravel, trace clay, containing silty clay layers Compact to dense Wet Brown to grey		14	SS	29		213	
			15	SS	38		212, 211, 210	
209.1 / 19.8	Silty Clay, trace sand Stiff to hard Wet Grey		16	SS	40		209, 208, 207	
			17	SS	12		206, 205, 204	
200.9 / 28.0			18	SS	19		203, 202, 201	
	END OF BOREHOLE Notes: 1.Hole terminated due to tightening of soil around augers and resulting difficulties in advancing/withdrawing augers. 2.Water level in open borehole on December 15 and 18, 2000 at 2.7m depth (Elev.226.2m)							

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

GR SA SI CL

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

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

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No B1-2

METRIC

PROJECT 001-1143F

W.P. 30-95-00

LOCATION N 4895623.5; E 292394.6

ORIGINATED BY GPD

DIST SW HWY 400

BOREHOLE TYPE 108mm ID HOLLOW STEM AUGERS AND CASING

COMPILED BY LCC

DATUM Geodetic

DATE Dec.14-18/2000

CHECKED BY ASP

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
228.4	GROUND SURFACE									
0.0	Silty Sand, trace gravel, trace organics (Fill) Loose to compact Moist Brown				228					
		1	SS	14	227					
		2	SS	8						
226.1 / 2.3	Silty Sand to Sandy Silt, trace clay Compact Wet Brown	3	SS	13	226	o				
		4	SS	23	225	o	0	71	29	0
		5	SS	16	224					
		6	SS	13		o	0	34	65	1
					223					
		7	SS	21	222					
					221					
		8	SS	19	220	o				
		9	SS	18	219					
					218					
		10	SS	21	217	o				
		11	SS	19	216					
					215					
		12	SS	14	214	o				

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE; 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; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity

$\circ^{3\%}$ STRAIN AT FAILURE

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

Ministry of Transportation Ontario

RECORD OF BOREHOLE No B1-2

METRIC

PROJECT	001-1143F
W.P.	30-95-00
LOCATION	N 4895623.5; E 292394.6
DIST	SW
HWY	400
BOREHOLE TYPE	108mm ID HOLLOW STEM AUGERS AND CASING
DATUM	Geodetic
DATE	Dec.14-18/2000
ORIGINATED BY	GPD
COMPILED BY	LCC
CHECKED BY	ASP

SOIL PROFILE — CONTINUED FROM PREVIOUS PAGE —

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES
	Silty Sand to Sandy Silt, trace clay Compact Wet Brown	13	SS	13
		14	SS	21
206.6		15	SS	18
21.8	Silty Clay, trace sand Firm to very stiff Moist Grey	16	SS	18
		17	SS	8
		18	SS	6

ELEVATION SCALE: 213, 212, 211, 210, 209, 208, 207, 206, 205, 204, 203, 202, 201, 200, 199

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 (value shown: 43)

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

Continued Next Page

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

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No B1-2

METRIC

PROJECT 001-1143F

W.P. 30-95-00

LOCATION N 4895623.5; E 292394.6

ORIGINATED BY GPD

DIST SW HWY 400

BOREHOLE TYPE 108mm ID HOLLOW STEM AUGERS AND CASING

COMPILED BY LCC

DATUM Geodetic

DATE Dec.14-18/2000

CHECKED BY ASP

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION (— CONTINUED FROM PREVIOUS PAGE —)	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS
	Silty Clay, trace sand Firm to very stiff Moist Grey	19	SS	9	198 / 197 / 196 / 195		
194.9 33.5	Silty Sand containing silty clay layers Very dense Wet Grey	20	SS	52	194 / 193 / 192		
		21	SS	72			
191.4 37.0	END OF BOREHOLE						

Notes:
1. Water level in open borehole at 2.3m depth (Elev.226.1m) on completion of drilling operations.
2. Water level in piezometer at 1.8m depth (Elev.226.6m) on January 19, 2001, and at 1.3m depth (Elev.227.1m) on March 15, 2001.

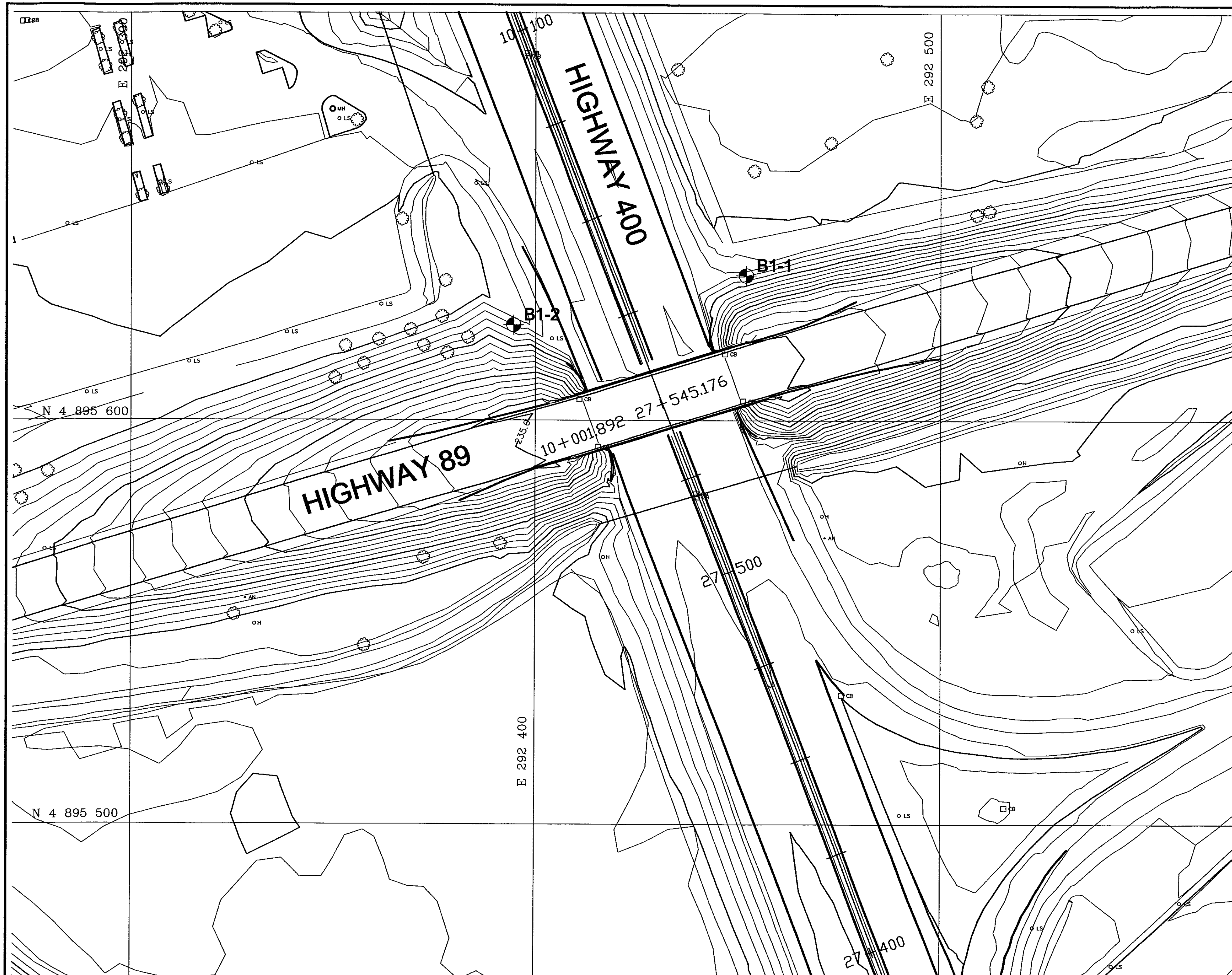
Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X REMOULDED (20, 40, 60, 80, 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10, 20, 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

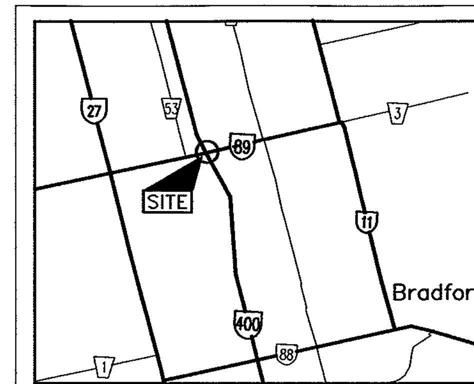
+³, X³: Numbers refer to Sensitivity   O³% STRAIN AT FAILURE

DIST HWY 400 CONT. No. GWP No. 30-95-00	
HIGHWAY 89 UNDERPASS HWY 400 BOREHOLE LOCATION PLAN	SHEET

Golder Associates

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA





KEY PLAN

0 2.0 4.0 6.0 8.0 10.0 20.0
SCALE IN KILOMETRES

LEGEND

Borehole, previous investigation

Borehole, present investigation

No.	ELEVATION	LOCATION NORTHING	LOCATION EASTING
B1-1	228.9	4,895,635.8	292,452.1
B1-2	228.4	4,895,623.5	292,394.6

REFERENCE

This drawing was created from digital file "33811.dwg" provided by URS Cole Sherman

0 10 20 30 40 50 100m
SCALE IN METRES

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

NO.	DATE	BY	REVISION

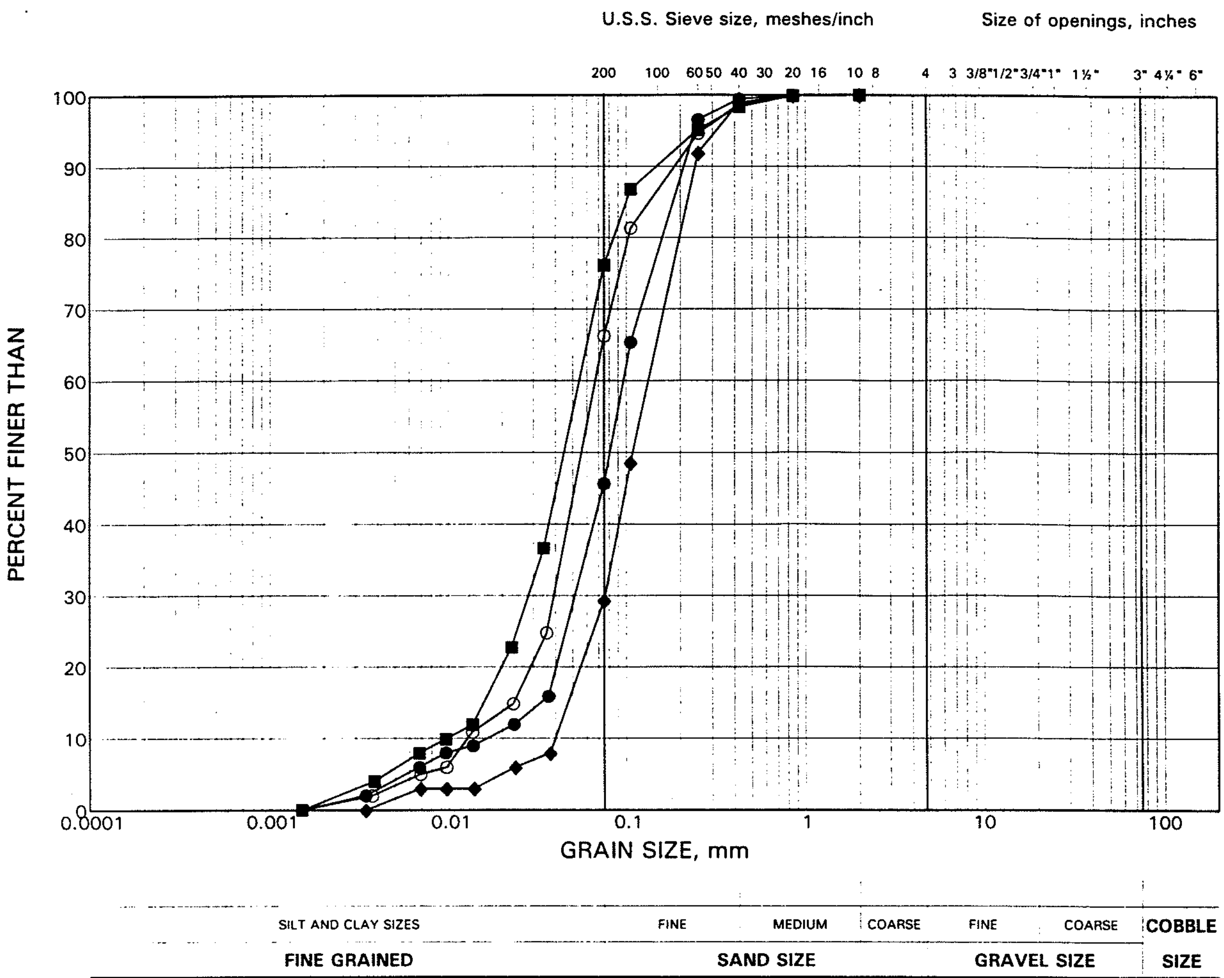
Geocres No.

HWY. No. 400	PROJECT NO.: 001-1143F		
SUBM'D. LCC	CHKD: ASP	DATE: JANUARY 2001	SITE 30-256
DRAWN: MHW	CHKD. LCC	APPD. ASP	DWG. 1

P1143F01.DWG

GRAIN SIZE DISTRIBUTION TEST RESULTS
Silty Sand to Sandy Silt Deposit

FIGURE 1



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	B1-1	4	226.3
■	B1-1	7	224.0
◆	B1-2	4	225.0
○	B1-2	6	223.5

Project 001-1143F

Golder Associates