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 NINTH CONCESSION OVERPASS STRUCTURE SITE 30-308 HIGHWAY 400 WIDENING FROM YORK/SIMCOE BOUNDARY TO 1 KM SOUTH OF HIGHWAY 89 G.W.P. 40-00-00

Submitted to:

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

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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 the York / Simcoe Boundary northerly to 1 km south of Highway 89, in Simcoe County, Ontario. Foundation engineering services are required for the widening and / or replacement of six existing overpass and underpass structures, as well as four structural culverts.

This report addresses the widening or replacement of the existing West Gwillimbury Ninth Concession overpass structure. A foundation investigation has been carried out, in which two boreholes were advanced and in-situ and laboratory testing was conducted, to determine the subsurface conditions at the site for this preliminary design study.

The terms of reference for the scope of work are outlined in the MTO's Request for Quotation (RFQ) dated September 5, 2000, and in Golder Associates' subsequent letters dated December 13, 2000 and February 15, 2001.

2.0 SITE DESCRIPTION

The existing West Gwillimbury Ninth Concession overpass is located about 2.5 km north of the Simcoe Road 88 (formerly Highway 88) interchange and about 8.5 km south of the Highway 89 interchange, in the Township of West Gwillimbury, County of Simcoe. The MTO has designated this overpass as Structure Site No. 30-308.

At the site, Highway 400 has been constructed near the original ground surface, with its grade at about Elevation 277 m to 277.5 m, rising northward. Ninth Concession has been constructed in a cut up to about 6 m deep, with its grade at about Elevation 271.5 m under the highway.

The existing single-span overpass was constructed in the early 1950s under Contract 50-157. According to the general layout drawing for the existing structure, which was provided by Morrison Hershfield (the structural designers for this preliminary study), the abutments are supported on spread footings which are founded at about Elevation 269.9 m, below the Ninth Concession cut grade.

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out at this site in December 2000, at which time two boreholes were drilled to determine the soil and groundwater conditions. Boreholes B4-1 and B4-2 were advanced on the east and west sides of Highway 400, respectively, from the Ninth Concession cut grade. Owing to weather conditions at the time of drilling and access restrictions, boreholes could not be put down from the Highway 400 grade.

The investigation was carried out using a bombardier-mounted D-50 drill rig, supplied and operated by Master Soil Investigations Ltd. of Weston, Ontario. Both boreholes were advanced using solid stem augers, to depths of between 20 m and 23 m below the Ninth Concession cut grade. Samples of the overburden were obtained at 0.75 m to 1.5 m intervals of depth using 50 mm outside diameter split-spoon samplers, in accordance with the Standard Penetration Test (SPT) procedure. The water levels in the open boreholes were observed throughout the drilling operations, and a piezometer was installed in Borehole B4-2 to permit monitoring of the groundwater 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 testing 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 15 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 Schomberg Clay Plains; the Peterborough Drumlin Field; and a second lobe of the Simcoe Lowlands. Along Highway 400, the southern lobe of the Simcoe Lowlands is present at the North Canal / Canal Road site. The Schomberg Clay Plains are present north of this site, to 2 km north of Simcoe Road 88 (formerly Highway 88). The Peterborough Drumlin Field extends from 2 km north of Simcoe Road 88 to about 3 km south of Highway 89. The northern lobe of the Simcoe Lowlands extends from about 3 km south of Highway 89 to beyond the northern limit of this project.

The surficial soils in the Schomberg Clay Plains consist primarily of varved clay and silt deposits. These varved deposits overlie till with drumlins as found in the Peterborough Drumlin Field. The drumlins (glacially-shaped hills) are completely or partially buried by the clay and silt deposits. depending on the size of the drumlin. The varved clay and silt deposits are typically about 5 m thick, although deeper deposits have been found in some locations.

The surficial soils in the Peterborough Drumlin Field, in which the Ninth Concession site is located, consist primarily of gravelly sand till or sand and gravel deposits. Deposits of silt, clay or peat may be found in the low-lying areas between drumlins.

Along Highway 400, the Simcoe Lowlands include the Holland River valley, the shores of Kempenfelt Bay, the Nottawasaga River, and Innisfil Creek. The Holland River valley at the southern end of this project extends southwest from Cook Bay, at the south end of Lake Simcoe; it was once a shallow extension of the lake. The floor of the valley is covered by extensive deposits of loose silts and soft clays, which overlie a till sheet. In localized areas, these silts and clays are overlain by a thin, poorly graded sand of deltaic origin. Because the valley is depressed and poorly drained, a surficial cover of peat has formed in many areas. The surficial soils of the northern lobe of the Simcoe Lowlands consist primarily of sand, although silt, clay or peat may be found in low-lying areas.

4.2 Site Stratigraphy

The detailed subsurface soil and groundwater conditions encountered in the boreholes, and the results of the laboratory testing carried out on selected soil samples, are given on the Record of Borehole sheets and Figure 1. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations.

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

In summary, the soils below the Ninth Concession cut grade consist of road base fill overlying till deposits which grade from sand and silt to silty sand to clayey silt. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Sand and Silt Till

A deposit of sand and silt till was encountered below roadway fill in the Ninth Concession cut. In Borehole B4-1 on the east side of the highway, this till is about 11 m thick, and in Borehole B4-2 on the west side of the highway, the till deposit is about 7 m thick. The sand and silt till contains trace gravel and clay; a grain size distribution test result for a sample of the sand and silt till is shown on Figure 1.

The recovered samples of the sand and silt till were moist. Natural moisture contents measured on samples of the till ranged from 9 to 11 per cent. Atterberg limits testing conducted on four samples of the till measured plastic limits of 10 to 11 per cent, liquid limits of 13 to 14 per cent, and plasticity indices of 3 to 4 per cent.

The measured Standard Penetration Test (SPT) 'N' values ranged from 9 to 74 blows per 0.3 m of penetration, but were typically between about 15 and 50 blows per 0.3 m of penetration. The sand and silt till therefore has a predominantly compact to dense relative density.

4.2.2 Clayey Silt Till

In Borehole B4-2, the sand and silt till grades to a clayey silt till below about Elevation 265 m (approximately 6.5 m below the Ninth Concession cut grade). This clayey silt till deposit is about 13 m in thickness, extending to the end of the borehole at about Elevation 252 m. The clayey silt till contains trace to some sand and trace gravel. A layer of silty sand, about 1.5 m in thickness, was encountered within the till between about Elevation 263.5 m and 262 m. It is noted that a similar 1.5 m thick layer of silty sand was encountered in Borehole B4-1, below Elevation 259.5 m at the base of the sand and silt till deposit.

The recovered samples of the clayey silt till were moist, with measured natural water contents of 9 to 17 per cent. The silty sand interlayer encountered in both boreholes was also moist, with measured natural water contents of 9 to 10 per cent.

The measured SPT 'N' values measured in the clayey silt till in Borehole B4-2 ranged from 27 to greater than 100 blows per 0.3 m of penetration. Above Elevation 257 m, the till has a very stiff to hard consistency. The clayey silt till is hard below Elevation 257 m, where the measured SPT 'N' values were near to or greater than 100 blows per 0.3 m of penetration.

4.2.3 Silty Sand Till

In Borehole B4-1 on the east side of the highway, the upper sand and silt till and silty sand interlayer are underlain by a deposit of silty sand till, containing trace to some gravel and clay. The surface of this deposit was encountered at about Elevation 258 m. The silty sand till was not fully penetrated by the borehole; it is at least 10.5 m in thickness.

The recovered silty sand till samples were moist, with measured natural water contents of 7 to 9 per cent. The measured SPT 'N' values ranged from 30 to greater than 100 blows per 0.3 m of penetration; below Elevation 255 m, the measured SPT 'N' values were generally greater than 100 blows per 0.3 m of penetration. The silty sand till has a dense to very dense, but predominantly very dense relative density.

4.3 Groundwater Conditions

The groundwater level in the piezometer installed in Borehole B4-2 was measured to be at or slightly above the Ninth Concession cut grade. In January 2001, the water level was frozen at Elevation 272.4 m, and in March 2001, the water level was frozen at Elevation 272.1 m. In June 2001, the water level in the piezometer was at Elevation 272.3 m. This groundwater level is associated with the sand and silt till deposit in which the Ninth Concession cut is formed.

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

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PART B

PRELIMINARY FOUNDATION DESIGN REPORT
NINTH CONCESSION OVERPASS
STRUCTURE SITE 30-308
HIGHWAY 400 WIDENING
FROM YORK/SIMCOE BOUNDARY
TO 1 KM SOUTH OF HIGHWAY 89
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5.0 ENGINEERING RECOMMENDATIONS

5.1 General

This section of the report provides preliminary foundation design recommendations for the widening or replacement of the existing West Gwillimbury Ninth Concession overpass structure, associated with the widening of Highway 400. The recommendations are preliminary only and are based on interpretation of the factual data obtained from a limited number of boreholes advanced 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 ultimate configuration of ten lanes. The primary options under consideration involve widening into the median, or using a 22 m wide open median with widening on the outside of the existing highway; depending on which option is adopted, it is expected that the existing highway platform will be widened by between 10 m and 29 m. Widening or replacement of the existing Ninth Concession overpass will, therefore, be necessary.

Based on the general layout drawing for the existing single-span structure, the abutments are supported on spread footings which are founded at about Elevation 269.9 m, below the Ninth Concession cut grade of about Elevation 271.5 m. Highway 400 has been constructed near original ground surface, with its grade at about Elevation 277 m to 277.5 m at the structure site.

5.2 Bridge Foundation Options

The soils below the Ninth Concession cut grade consist of road base fill overlying till deposits which grade from sand and silt to silty sand to clayey silt. The groundwater level associated with the upper sand and silt till deposit is at or slightly above the Ninth Concession cut grade.

Based on these subsurface conditions, consideration could be given to founding the widening or replacement structures on spread footings, either placed on the native sand and silt till deposit or "perched" on a compacted granular pad within the approach fill. It is noted that, if the structure is widened, the existing foundations should be matched. However, for a replacement structure, the perched abutment option is considered advantageous in that groundwater control requirements would be minimized or eliminated in the abutment areas. Spread footings placed on the sand and silt till stratum would be more difficult to adopt for the widening or replacement of the structure, due to the relatively high groundwater table. If excavation is carried out near to or below the groundwater level existing at the time of construction, groundwater control will be required to achieve the design bearing capacities and prevent loosening or disturbance of the foundation soil. In the case of widening of the existing footings, this groundwater control is also essential to maintain the integrity of the founding soils under the existing footings. In this regard, new excavations should not be extended below the existing footings.

Alternatively, the abutments and pier could be supported on steel H-piles driven to found within the hard clayey silt till and very dense silty sand till.

Preliminary recommendations for spread footings and for deep foundations are provided in the following sections.

5.3 Spread Footings

For preliminary design of the abutment foundations, spread footings may be placed at a design founding level of Elevation 269.9 m, to match the existing foundations and be founded on the compact to very dense sand and silt till stratum. 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. As noted elsewhere in this report, the footing level is below the groundwater level and groundwater control in the sand and silt till stratum will be required to achieve the geotechnical resistances recommended herein.

To avoid the difficulties associated with groundwater control and potential disturbance of the founding soils, consideration could 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 sand and silt till stratum at or below Elevation 269.9 m may be designed using a factored geotechnical resistance at ULS of 500 kPa, assuming a 3 m wide footing. The settlement of these footings will be dependent on the footing size and configuration, on the applied loads, and on the degree of disturbance caused during construction. For preliminary design purposes, the geotechnical resistance at SLS may be taken as 300 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 abutment spread footings placed within the approach embankments on a compacted Granular 'A' pad, a factored geotechnical resistance at Ultimate Limit States (ULS) of 900 kPa may be assumed for preliminary design. The geotechnical resistance at Serviceability Limit States (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 properly prepared native sand and silt till 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 replacement structure on steel H-piles driven to found within the hard / very dense lower till deposit. Standard Penetration Test 'N' values greater than 100 blows per 0.3 m of penetration were measured below Elevation 255 m in Borehole B4-1, on the east side of the highway, and below about Elevation 256 m in Borehole B4-2, on the west side of the highway. For preliminary design, a pile tip level of Elevation 252 m may be used. Assuming that the abutment pile caps are placed at or slightly above the Ninth Concession cut grade, to minimize the requirement for groundwater control in the sand and silt till deposit, the piles would be about 20 m long.

5.4.1 Axial Geotechnical Resistance

For preliminary design, the geotechnical resistance at ULS has been determined for a pile driven to found at about Elevation 252 m based on the subsoil conditions encountered in Borehole B4-1, i.e. piles founded within the dense to very dense silty sand till. We have reviewed the ultimate geotechnical resistances as obtained from static pile load tests for piles founded within very dense sands and silts. Although the silty sand till deposit as encountered at this site contains typically more clay that the founding deposits for the static pile load tests, due to the fact that the clay proportion is variable, we consider that this approach to the design is appropriate. A higher capacity is feasible for the piles driven into the clayey silt till as encountered in Borehole B4-2. However, it is recommended that further borehole drilling be carried out at this site during the final design stage to establish where the change in stratigraphy occurs and to confirm the design capacities.

Based on this assessment, the factored axial resistance at ULS for steel HP 310 x 110 piles driven to about Elevation 252 m may be taken as 1,200 kN for preliminary design. The axial resistance at SLS for a single pile may be taken as 1,200 kN for preliminary design. The SLS capacity should be reviewed at the final design stage once the pile group configuration has been established.

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:

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

Where

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

For cohesive soils:

$$k_h = \frac{k_{s1}}{5B}$$ Where B is the pile diameter (m) and k_{s1} is the constant of horizontal subgrade reaction, as given below

The piles will be driven through the upper, compact to very dense sand and silt till, into the lower till deposit, which varies in composition and consistency from dense to very dense silty sand till on the east side of the highway, to very stiff to hard clayey silt till on the west side of the highway. If deep foundations are adopted for support of the widening or replacement structure, it will be necessary to confirm the subsoil type and relative density / consistency within the footprint of the pile cap at the detailed design stage in order to confirm the preliminary recommendations for resistance to lateral loads. The following ranges for the value of n_h and k_{s1} may be assumed in the preliminary structural analysis, subject to confirmation following the detailed design stage of the subsurface investigation.

Borehole	*Soil Unit*	n_h	k_{s1}
B4-1 and B4-2	Sand and Silt Till	5 to 15 MPa/m	–
B4-1	Silty Sand Till: Elevation 258 m to 255 m	5 to 15 MPa/m	
	Silty Sand Till below Elevation 255 m	10 to 20 MPa/m	–
B4-2	Clayey Silt Till: Elevation 262 m to 256 m	–	25 to 60 MPa/m
	Clayey Silt Till below Elevation 256 m	–	50 to 100 MPa/m

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

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 of soil cover for frost protection.

5.5 Lateral Earth Pressures

The lateral pressures acting on the bridge abutments and associated retaining walls will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill, on the freedom of lateral movement of the structure, and on the drainage conditions behind the walls. The following recommendations are made concerning the design of the abutments, in accordance with the OHBDC:

- Select free-draining granular fill meeting the specifications of OPSS Granular 'A' or Granular 'B' but with less than 5 per cent passing the 200 sieve should be used as backfill behind the abutments and walls. This fill should be compacted in loose lifts not greater than 200 mm in thickness to 95 per cent of the material's Standard Proctor maximum dry density in accordance with OPSS 501. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the abutment granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3501.00 and 3504.00.

- A compaction surcharge equal to 16 kPa should be included in the lateral earth pressures for the structural design of the abutment wall, in accordance with OHBDC Figure 6-7.4.3. Compaction equipment should be used in accordance with OPSS 501.06.

- The granular fill may be placed either in a zone with width equal to at least 1.5 m behind the back of the stem (Case I from OHBDC Figure 6-7.4.1) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (Case II from OHBDC Figure 6-7.4.4).

- For Case I, the pressures are based on the existing and proposed embankment fill materials and the following parameters (unfactored) may be assumed:

Soil unit weight:	20 kN/m^3
Coefficients of lateral earth pressure:	
Active, K_a	0.35
At rest, K_o	0.50

- For Case II, the pressures are based on the granular fill as placed and the following parameters (unfactored) may be assumed:

	Granular 'A'	**Granular 'B'** *Type 2*
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 wall 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 Design of Permanent Cut Slopes

In the vicinity of this overpass, the Ninth Concession cut is up to about 6 m in depth, with the road grade at about Elevation 271.5 m. Based on the topographic information on the Engineering and Title Records plates and on site reconnaissance, the existing cut slopes are formed at a gradient of about 2 horizontal to 1 vertical (2H:1V). For preliminary design purposes, a maximum gradient of 2H:1V may be assumed for any new permanent cut slopes.

It is noted that the base of the cut will extend below the groundwater level measured in the sand and silt till deposit. Protection of the lower portion of the cut slope, for example using a drainage blanket, will be necessary to prevent sloughing of the toe of the permanent cut slopes.

5.7 Design and Construction Considerations

5.7.1 Groundwater Control

The founding level for spread footings at Elevation 269.9 m will be below the groundwater level, and groundwater seepage into the footing excavations should be expected to occur through the sand and silt till deposit. Upward seepage through the founding soils will result in loosening and softening of the subgrade. The gradation of the deposit is such that groundwater lowering will be difficult. At a minimum, pumping from properly-filtered sumps or a filtered drain placed at the base of the excavation should be carried out. In addition, the excavation should be carried out in stages such that the final 0.3 m of excavation is completed and lean concrete placed in short strips. Surface water run-off should be directed away from the footing excavations.

5.7.2 Obstructions

Although no cobbles or boulders were encountered during the preliminary subsurface 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 for deep foundations or temporary shoring systems.

5.7.3 Excavation

The footing or pile cap excavations will extend a minimum of 1.5 m below lowest surrounding grade, through the sand and silt till deposit. 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 sand and silt till soils would be classified as Type 2 soil. Temporary open-cut slopes should therefore be maintained no steeper than 1 horizontal to 1 vertical (1H:1V) to within 1.2 m of the excavation base; below this, the excavation walls may be vertical-sided. Where space restrictions dictate, footing excavations could also be carried out within a braced excavation.

GOLDER ASSOCIATES LTD.

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

Anne S. Poschmann, P.Eng.,
Principal

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	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

RECORD OF BOREHOLE No B4-1 — 1 OF 2 — METRIC

PROJECT 001-1151 — LOCATION N 4887127.3; E 293870.9 — ORIGINATED BY SB

W.P. 40-00-00 — BOREHOLE TYPE 108mm Diameter Solid Stem Augers — COMPILED BY LCC

DIST SW HWY 400 — DATE December 21-22, 2000 — CHECKED BY ASP

DATUM Geodetic

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
271.7	GROUND SURFACE					
0.0	Sand and gravel (Fill)	1	SS	22		
271.1 / 0.6	Silty sand, some gravel, trace clay (Fill)	2	SS	9		
270.3 / 1.4	Sand and Silt, trace clay and gravel (Till) Compact to very dense Brown becoming grey below 3m depth Moist	3	SS	52		7 46 37 10
		4	SS	56		
		5	SS	74		
		6	SS	18	∇ (approx. elev. 267.8)	
		7	SS	17		
		8	SS	26		
		9	SS	28		
		10	SS	19		
		11	SS	17		
259.5 / 12.2	Silty Sand, trace gravel Dense Grey Moist	12	SS	39		
258.0 / 13.7	Silty Sand, trace to some gravel, trace to some clay (Till) Dense to very dense Grey Moist	13	SS	80		
		14	SS	30		
		15	SS	119/.23		
		16	SS	123		

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE (271 to 252); 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

Continued Next Page

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

RECORD OF BOREHOLE No B4-1

METRIC

PROJECT 001-1151

W.P. 40-00-00 LOCATION N 4887127.3; E 293870.9 ORIGINATED BY SB

DIST SW HWY 400 BOREHOLE TYPE 108mm Diameter Solid Stem Augers COMPILED BY LCC

DATUM Geodetic DATE December 21-22, 2000 CHECKED BY ASP

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)
	Silty Sand, trace to some gravel, trace to some clay (Till) Dense to very dense Grey Moist	17	SS	48	251	
		18	SS	120	250	o (~7)
					249	
248.4 / 23.3		19	SS	107		
	END OF BOREHOLE Note: Water level in open borehole on completion of drilling at 4.0m depth (Elev.267.7m).					

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

RECORD OF BOREHOLE No B4-2

METRIC

PROJECT 001-1151
W.P. 40-00-00
LOCATION N 4887124.8; E 293822.5
ORIGINATED BY GPD
DIST SW HWY 400
BOREHOLE TYPE 108mm Diameter Solid Stem Augers
COMPILED BY LCC
DATUM Geodetic
DATE December 19-21, 2000
CHECKED BY ASP

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES
272.1	GROUND SURFACE			
0.0	Sand and Silt, trace clay and gravel (Till) Loose to dense Grey Moist	1	SS	26
		2	SS	14
		3	SS	35
		4	SS	27
		5	SS	25
		6	SS	47
		7	SS	9
264.8 / 7.3	Clayey Silt, trace sand and gravel (Till) Stiff Grey Moist	8	SS	12
263.4 / 8.7	Silty Sand, trace clay and gravel Loose Grey Moist	9	SS	6
261.9 / 10.2	Clayey Silt, trace to some sand, trace gravel (Till) Very stiff to hard Grey Moist	10	SS	31
		11	SS	27
		12	SS	33
		13	SS	94
		14	SS	100

Column headings: SOIL PROFILE; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (272 to 253); 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

Continued Next Page

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

RECORD OF BOREHOLE No B4-2

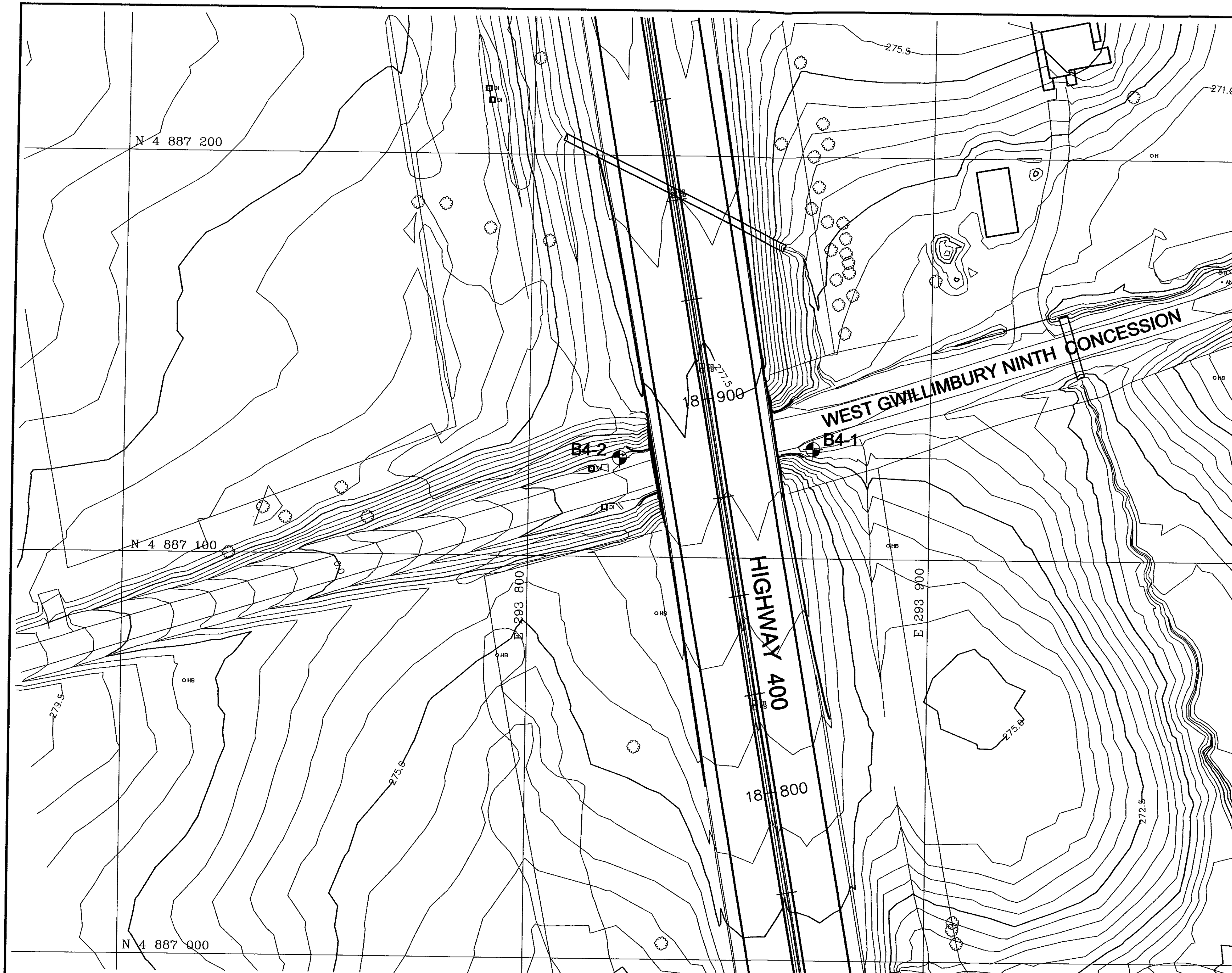
METRIC

PROJECT 001-1151
W.P. 40-00-00
LOCATION N 4887124.8; E 293822.5
ORIGINATED BY GPD
DIST SW HWY 400
BOREHOLE TYPE 108mm Diameter Solid Stem Augers
COMPILED BY LCC
DATUM Geodetic
DATE December 19-21, 2000
CHECKED BY ASP

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	— CONTINUED FROM PREVIOUS PAGE —										
251.7			15	SS	117		252				
20.4	END OF BOREHOLE Notes: 1. Water level in open borehole on completion of drilling at 0.3m depth (Elev.271.8m) 2. The piezometer was installed in a second borehole, drilled 1m west of this deep borehole. 3. The water level in the piezometer was frozen at 0.3m above ground surface (Elev.272.4m) on January 18, 2001, and at ground surface (Elev.272.1m) on March 20, 2001. The water level in the piezometer was 0.2m above ground surface (Elev.272.3m) on June 19, 2001.										

Legend: SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X REMOULDED. PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L.

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



DIST HWY 400
CONT. No.
GWP No. 40-00-00

NINTH CONCESSION OVERPASS
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	LOCATION EASTING
B4-1	271.7	4,887,127.3	293,870.9
B4-2	272.1	4,887,124.8	293,822.5

REFERENCE
This drawing was created from digital file "33817.dwg" provided by URS Cole Sherman

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

NO.	DATE	BY	REVISION

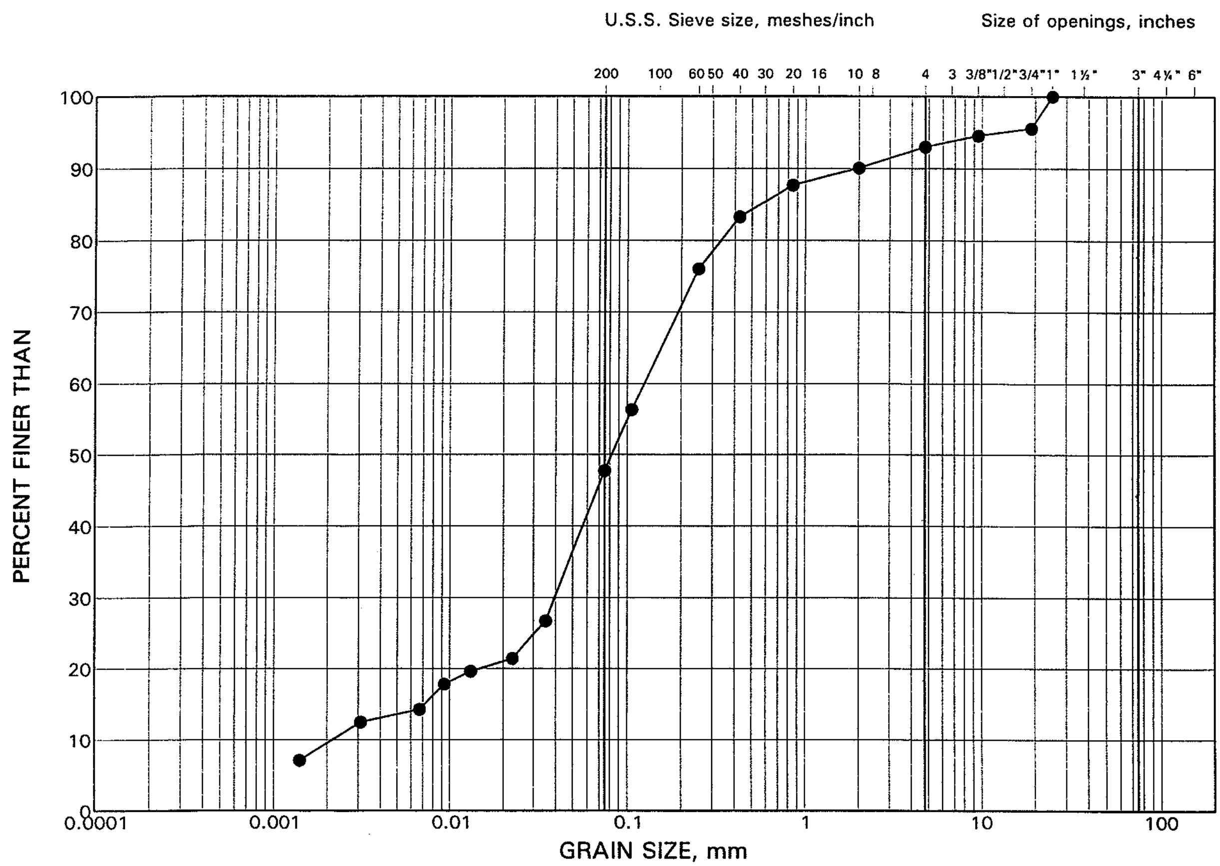
Geocres No.

HWY. No. 400		PROJECT NO.: 001-1151	
SUBM'D. LCC	CHKD: ASP	DATE: JANUARY 2001	SITE 30-308
DRAWN: MHW	CHKD. LCC	APPD. ASP	DWG. 1

P1151004.DWG

GRAIN SIZE DISTRIBUTION TEST RESULT
Sand and Silt Till

FIGURE 1



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	B4-1	3	269.6

Project 001-1151

Golder Associates