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**PRELIMINARY FOUNDATION  
INVESTIGATION AND DESIGN REPORT  
DUNLOP STREET UNDERPASS  
STRUCTURE SITE 30-175  
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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January 2002



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**PART A**

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Record of Borehole B11-1

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Figure 2                      Grain Size Distribution Test Results – Sandy Silt with Clayey Silt Interlayers

## **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 Dunlop Street underpass structure in Barrie, Ontario. A foundation investigation has been carried out, in which one borehole was 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. Under the original scope of work, two boreholes were to be advanced at this site for preliminary investigation purposes. However, during drilling of Borehole B10-1 at the Tiffin Street site, the City of Barrie informed Golder Associates of the presence of trichloroethylene (TCE) contamination in the area encompassing the Barrie/Collingwood Railway (former CN Rail), Tiffin Street and Dunlop Street sites. Based on review of the then-available subsurface information for the Essa Road and Anne Street sites to the south and north of this area, respectively, and following discussions with the MTO Foundations Group, the scope of work was revised as follows:

- Borehole B10-1, which had already been advanced at the Tiffin Street site, would be used to provide preliminary subsurface information for both the Tiffin Street site and the Barrie/Collingwood Railway (former CN Rail) site, less than 200 m to the south. No further drilling would be carried out at these two sites during the preliminary foundation investigation stage.
- One borehole would be drilled at the Dunlop Street (Simcoe Road 90, formerly Highway 90) site. This borehole was to be advanced by telescoping casing, to minimize the potential for aquifer cross-contamination.

The above revised scope of work was approved by the MTO Foundations Section.

## **2.0 SITE DESCRIPTION**

The existing Dunlop Street (Simcoe Road 90, formerly Highway 90) underpass is located about 2.5 km north of the Essa Road (Simcoe Road 30, formerly Highway 27) interchange, and about 2.5 km south of the Bayfield Street (Highway 26) interchange, in Barrie, Ontario. The MTO has designated this underpass as Structure Site 30-175.

At the site, Highway 400 has been constructed near the original ground surface, with its grade at about Elevation 231.5 m, rising toward both the north and south. Dunlop Street has been constructed on embankment fill up to about 5.5 m high; the local road grade is at about Elevation 237 m over the highway.

The existing two-span underpass was constructed in the early 1950s under Contract 50-116. According to the general layout drawing for the existing structure, which was provided by Morrison Hershfield (the structural designers for this preliminary study), the pier, abutments and associated wing walls and retaining walls are supported on spread footings which are founded at about Elevation 229 m to 229.3 m.

### **3.0 INVESTIGATION PROCEDURES**

A subsurface investigation was carried out at this site in January 2001, at which time one borehole was drilled using a bombardier-mounted D-50 drill rig, supplied and operated by Master Soil Investigations Ltd. of Weston, Ontario. Borehole B11-1 was located on the west side of Highway 400, north of Dunlop Street. This borehole was advanced from slightly below the Highway 400 grade to about 40 m depth (approximately Elevation 191 m).

The borehole was advanced by telescoping casing in order to minimize the potential for drilling-induced movement of any contaminants at the site. The borehole was initially advanced to about 11 m depth using hollow stem augers. A bentonite seal was placed over the interval between 9 m and 11 m, the hollow stem augers were withdrawn, and 'N'-size casing was installed and socketted within the bentonite seal at about 10.7 m depth. Drilling below this depth was carried out using 'B'-size casing.

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 level in the open borehole was observed throughout the drilling operations, and a piezometer was installed to permit monitoring of the groundwater level at the site. The piezometer was installed in a second shallow borehole, located about 4 m west of the deep borehole.

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 location and elevation were surveyed by Callon Dietz, Ontario Land Surveyors. The borehole elevation is referenced to geodetic datum, and the northing and easting coordinates are referenced to the MTM NAD83 survey system. The borehole location, ground surface elevation, 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, 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 Dunlop Street site is located within this physiographic region.

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 borehole and the results of in-situ and laboratory testing are given on the Record of Borehole sheets and Figures 1 and 2. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations.

Borehole B11-1 was advanced in the northwest quadrant of the structure site, from approximately Highway 400 grade. The location and elevation for this boring are shown on the attached Drawing 1. In summary, below a thin layer of topsoil and reworked silty clay, the native soils below the Highway 400 level consist of interlayered deposits of stiff to very stiff clayey silt to silty clay, and compact to dense silty sand and sandy silt. Very dense silty sand, with measured SPT 'N' values greater than 100 blows per 0.3 m of penetration, was encountered at about 39 m depth (Elevation 192.5 m).

A more detailed description of the subsurface conditions encountered in the borehole is provided in the following sections. It is noted that no odour, staining or other evidence of contamination was observed in the recovered samples, or in the auger cuttings produced during drilling of Borehole B11-1. Further, the City of Barrie confirmed during the investigation that the TCE contaminant plume is not known to extend to the Dunlop Street structure area.

#### **4.2.1 Upper Clayey Silt to Silty Clay**

A 10 m thick deposit of clayey silt to silty clay was encountered below about 200 mm of topsoil and 600 mm of silty clay fill/reworked soil. The deposit extends from about Elevation 230.5 m to 220.5 m. The clayey silt to silty clay contains trace sand, as well as significant interlayers of wet silty sand; a 1.7 m layer of compact to dense silty sand was encountered at about Elevation 227 m, and a 0.4 m thick layer of silty sand was encountered at about Elevation 224.5 m. A grain size distribution test result obtained for a sample of the clayey silt to silty clay material is shown on Figure 1.

The recovered samples were moist, with measured natural water contents ranging from 15 to 20 per cent. Atterberg limits testing carried out on three samples of the clayey silt to silty clay measured plastic limits of 13 to 16 per cent, liquid limits of 22 to 27 per cent, and plasticity indices of 7 to 12 per cent. The limit test results indicate that the soil is inorganic and of low plasticity.

A Standard Penetration Test (SPT) 'N' value of 19 blows per 0.3 m of penetration was measured at the surface of this deposit, possibly indicative of a stiffer "crust". Measured SPT 'N' values of 4 to 19 blows per 0.3 m of penetration were measured below Elevation 230 m, indicating that the clayey silt to silty clay deposit has a firm to very stiff consistency.

#### **4.2.2 Sandy Silt**

A 7 m thick sandy silt deposit was encountered at about Elevation 200.5 m, below the upper clayey silt to silty clay. The sandy silt contains thin interlayers of silty clay. The result from a grain size distribution test carried out on a representative sample of this layered deposit is shown on Figure 2.

The recovered samples of sandy silt were wet, with measured natural water contents of 14 to 17 per cent. The measured SPT 'N' values ranged from 31 to 48 blows per 0.3 m of penetration, indicating that the sandy silt has a dense relative density.

#### **4.2.3 Lower Silty Clay**

A lower deposit of silty clay was encountered in Borehole B11-1 at about 18 m depth (Elevation 213 m), extending to about 39 m depth (Elevation 192.5 m). This silty clay contains trace sand; two relatively thin interlayers of wet silty sand to sandy silt were encountered within this deposit, as noted on the borehole record. The recovered silty clay samples were moist, with measured natural water contents of 16 to 24 per cent.

The SPT 'N' values measured in the silty clay deposit ranged from 15 to 30 blows per 0.3 m of penetration, indicating that this deposit has a very stiff consistency. One measured SPT 'N' value of 54 blows per 0.3 m of penetration was obtained; however, this measurement reflects the very dense relative density of the sandy silt interlayer encountered at about Elevation 202 m.

#### **4.2.4 Silty Sand**

A lower deposit of silty sand was encountered at 38.7 m depth (Elevation 192.5 m). The recovered sample of silty sand was wet, with a measured SPT 'N' value of 175 blows for 150 mm of penetration. Based on this SPT 'N' value, this lower silty sand has a very dense relative density.

### 4.3 Groundwater Conditions

The water level in Borehole B11-1 was at between about 6 m and 9 m depth (Elevation 225 m to 222.5 m) during the drilling operations. The groundwater level in the piezometer, which is sealed within the silty sand interlayer in the upper clayey silt to silty clay deposit, was measured at 1.3 m depth (about Elevation 230 m) in March 2001. The groundwater levels associated with the sandy silt and lower silty sand deposits were not determined during this preliminary investigation.

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

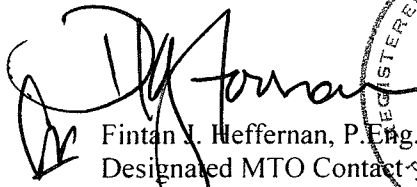
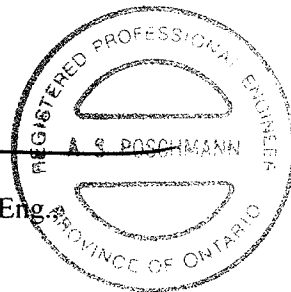
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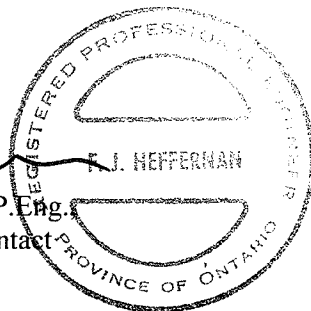
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**PART B**

**PRELIMINARY FOUNDATION DESIGN REPORT  
DUNLOP STREET UNDERPASS  
STRUCTURE SITE 30-175  
HIGHWAY 400 WIDENING FROM 1 KM SOUTH  
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## **5.0 ENGINEERING RECOMMENDATIONS**

### **5.1 General**

This section of the report provides preliminary foundation design recommendations for the replacement of the existing Dunlop Street 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 one borehole advanced to 40 m depth 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 structure 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 Dunlop Street underpass structure will therefore be necessary.

Based on the general layout drawing for the existing double-span structure, it appears that the pier, abutments and associated wing walls and retaining walls are supported on spread footings which are founded at about Elevation 229 m to 229.3 m. The Highway 400 grade is at about Elevation 231.5 m, near the original ground surface. Dunlop Street has been constructed on embankment fill up to about 5.5 m high, with its grade at about Elevation 237 m over the highway.

### **5.2 Bridge Foundation Options**

The native soils below the Dunlop Street grade consist of interlayered deposits of firm to very stiff clayey silt to silty clay, and compact to dense silty sand and sandy silt. Very dense silty sand was encountered at about 39 m depth (Elevation 192.5 m).

Consideration could be given to founding the replacement structure on spread footings, either placed on the native upper clayey silt to silty clay deposit or "perched" on a compacted granular pad within the approach embankment fill. However, settlement of the predominantly firm upper clayey silt to silty clay stratum will occur and the magnitude of this settlement could preclude the use of spread footings for the replacement structure.

It is recommended that consideration be given to the use of deep foundations, such as steel “H” piles, for support of the new structure. Such piles would be driven to found within the very dense, lower silty sand deposit, some 40 m below the Highway 400 grade at this site. The soil conditions at this site are favourable for the use of an integral abutment replacement structure.

Preliminary recommendations for spread footings and for driven steel “H” pile foundations are provided in the following sections.

### **5.3 Spread Footings**

For preliminary design of the abutment and pier foundations, spread footings may be placed on the firm to very stiff clayey silt to silty clay deposit at or below Elevation 229 m. Alternatively, consideration could be given to the use of abutment footings perched on a compacted granular pad within the approach embankment fill.

#### **5.3.1 Axial Geotechnical Resistance**

Spread footings placed on the properly prepared clayey silt to silty clay deposit at or below Elevation 229 m may be designed for a factored geotechnical resistance at Ultimate Limit States (ULS) of 200 kPa, assuming a 3 m wide footing.

The settlement of these footings will be dependent on the footing size, configuration, and applied loads. In addition, settlement of the footings will occur due to consolidation of the founding soils under the additional embankment loading if there is a widening or raising of the existing embankment grade. For preliminary design purposes, the geotechnical resistance at Serviceability Limit States (SLS), for 25 mm of settlement, may be taken as 130 kPa for footings placed at or below Elevation 229 m. It is estimated that the abutment footings could experience up to about 25 mm of additional settlement due to raising/widening of the approach embankments on the existing Dunlop Street alignment. The abutment footings, therefore, could experience up to 50 mm of settlement with the magnitude dependent on the proposed embankment configuration. It should be noted that the pier footing would not be influenced by the embankment loading and settlement of this footing would be only about 25 mm.

Due to the potential differential settlement between the abutment and pier footings, the option of shallow spread footings may not be feasible. The settlement should be confirmed at the final design stage, once the footing size, configuration and loadings as well as the proposed embankment configuration are known, to assess whether this spread footing option is feasible.

Additional field and laboratory testing should be carried out to confirm the strength and to determine the compressibility characteristics of the firm to stiff clayey silt to silty clay stratum to refine the settlement predictions.

For abutment spread footings placed within the approach embankments on a compacted Granular 'A' pad, a factored geotechnical resistance at ULS of 900 kPa may be assumed for preliminary design. The geotechnical resistance at SLS will depend on the thickness of Granular 'A' and the consistency and thickness of the underlying soils. A value of 350 kPa may be assumed for preliminary design; however, this must be reviewed at the final design stage. The concerns with respect to settlement of the abutment footings due to embankment loading as described above will apply to perched abutments as well. Since the footing load will be spread through the Granular "A" pad, the magnitude of differential settlement between the abutments and the pier will be less with this perched abutment approach and may therefore be the preferred option.

The geotechnical resistances provided herein are given under the assumption that the loads will be applied perpendicular to the surface of the footings; where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with the Ontario Highway Bridge Design Code (OHBD).

### **5.3.2 Resistance to Lateral Loads**

Resistance to lateral forces / sliding resistance between the concrete footing and the subsoils should be calculated in accordance with Section 6-8.4.3 of the OHBD. The angle of friction between the concrete and the firm to stiff clayey silt to silty clay may be taken as 22 degrees. The corresponding coefficient of friction,  $\tan \delta$ , would then be 0.4. Where "perched" footings are adopted, the angle of friction between the concrete and the compacted Granular 'A' pad may be taken as 30 degrees, with a corresponding coefficient of friction equal to 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**

Based on the results of Borehole B11-1, the surface of the very dense lower silty sand deposit is at about Elevation 192.5 m. For preliminary design, a pile tip level of Elevation 190 m may be assumed for the structure replacement. The piles would be about 40 m to 45 m long, depending on the pile cap level adopted. It is noted that additional borehole investigation will be required at the detailed design stage in order to confirm the design pile tip elevations.

### **5.4.1 Axial Geotechnical Resistance**

For preliminary design, the factored axial resistance at Ultimate Limit States (ULS) for steel HP 310 x 110 H-piles driven into the lower silty sand deposit may be taken as 1,200 kN. The axial resistance at Serviceability Limit States (SLS) for 25 mm of settlement may be taken as 1,200 kN.

To achieve the above design resistances, the piles should be driven to a tip level of at least Elevation 190 m. The rated energy of the hammer should not exceed 60 kJ. Provision should be made to re-tap selected piles to confirm the set after adjacent piles have been driven, in accordance with MTO's current Special Provision.

With the Dunlop Street underpass lengthened, the existing embankment will likely have to be raised and widened to accommodate the new structure. This additional fill will induce consolidation settlement of the clayey silt to silty clay soils. At the abutment locations, this consolidation will induce a downward movement of the soils adjacent to the piles, and negative skin friction will develop along the upper portion of the pile shaft embedded within the clayey silt to silty clay. Down to a certain point along the pile, called the neutral point, the settlement of the soil will be larger than the downward movement of the pile. The shear stresses mobilized along the pile down to the neutral point act downward, causing a downdrag load. Below this point, the downward movement of the pile is larger than the soil settlement and the mobilized shear stresses act upward on the pile as positive skin friction. At this site, with the pile tip resting on the very dense sand deposit, downward movement of the pile will occur due to yielding at the tip as well as pile compression.

The magnitude of the downdrag load acting on the pile is a function of the skin friction that develops between the pile and the clayey silt to silty clay, the surface area of the pile within these deposits, and the embankment loading (which is only partial filling in this case). The load calculated in this manner is a nominal (unfactored) load. The structural engineer must multiply this load by a load factor of 1.25 and include it as part of the dead load effects acting on the pile, as described in the OHBDC. For preliminary design, the negative skin friction load on a single pile may be taken as 250 kN. The downdrag load will have to be reassessed during the detailed design stage, using shear strength and consolidation data which should be determined for the shallow subsoils at the abutment and pier locations.

To minimize the amount of negative skin friction that may develop on a pile, consideration could be given to placing the new fill for any widened sections of the Dunlop Street embankment as early as possible to maximize the amount of settlement that occurs prior to the driving of the piles.

#### 5.4.2 Resistance to Lateral Loads

The lateral loading could be resisted fully or partially by the use of battered piles. If vertical piles are used, the resistance to lateral loading will have to be derived from the soil in front of the piles. If integral abutments are under consideration, there may also be a requirement for the piles to move sufficiently to accommodate the bridge deck deflections.

The resistance to lateral loading in front of the pile may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction,  $k_h$ , is based on the equations given below.

For cohesionless soils:

$$k_h = \frac{n_h z}{B} \quad \text{where} \quad \begin{array}{l} n_h \text{ is the constant of subgrade reaction} \\ z \text{ is the depth (m)} \\ B \text{ is the pile diameter (m)} \end{array}$$

For cohesive soils:

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

The piles will be driven through embankment fill, firm to very stiff clayey silt to silty clay, and compact to dense, wet silty sand to sandy silt. The following ranges in value of  $n_h$  and  $k_{sl}$  may be assumed in the structural analysis; these values will have to be confirmed at the detailed design stage.

<i>Soil Type</i>	<i><math>n_h</math></i>	<i><math>k_{sl}</math></i>
Embankment fill	5 to 15 MPa/m	—
Firm to very stiff clayey silt to silty clay	—	10 to 35 MPa/m
Dense, wet sandy silt	5 to 10 MPa/m	—
Very stiff silty clay	—	25 to 60 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:

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

### 5.4.3 Frost Protection

The pile caps should be provided with a minimum 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 <sup>3</sup>
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:

	<b>Granular 'A'</b>	<b>Granular 'B'</b>
		<b>Type II</b>
Soil unit weight:	22 kN/m <sup>3</sup>	21 kN/m <sup>3</sup>
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 Dunlop Street embankment side slopes are formed at a gradient of about 2 horizontal to 1 vertical (2H:1V). If any widening of the local road is required, the new side slopes should be formed at a maximum gradient of 2H:1V. The embankment widening should be carried out using conventional fill placement and compaction practices; and benching of the existing embankment side slopes should be carried out to key in the new fill.

The embankment stability and settlement should be further assessed as part of the detailed design, once the embankment configuration has been finalized and the shear strength and consolidation characteristics of the upper clayey silt to silty clay stratum have been established.

## **5.7 Design and Construction Considerations**

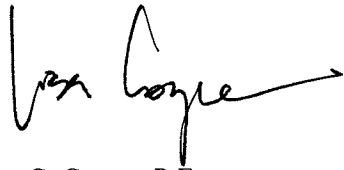
### **5.7.1 Groundwater Control**

Groundwater seepage into the footing 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 should be directed away from the footing excavations.



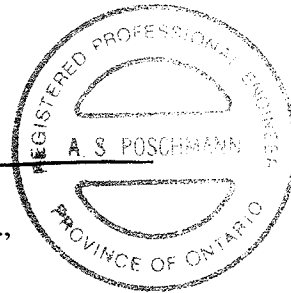
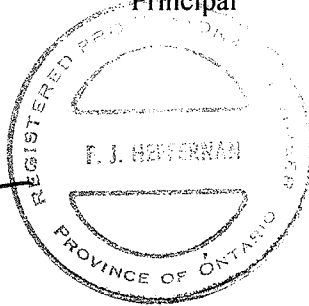
### 5.7.2 Excavation

Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act for Construction Activities. The footing excavations will extend a minimum of 1.5 m below lowest surrounding grade, through existing fill and into the firm to very stiff clayey silt to silty clay deposit. The fill and clayey silt to silty clay would be classified as Type 2 to 3 soils. Temporary open-cut slopes should therefore be maintained no steeper than 1 horizontal to 1 vertical (1H:1V). Where space restrictions dictate, footing excavations could also be carried out within a braced excavation.

#### GOLDER ASSOCIATES LTD.



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## LIST OF ABBREVIATIONS

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

### I. SAMPLE TYPE

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

### III. SOIL DESCRIPTION

#### (a) Cohesionless Soils

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

### II. PENETRATION RESISTANCE

#### Standard Penetration Resistance (SPT), N:

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

#### Consistency

	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

#### (b) Cohesive Soils

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

### IV. SOIL TESTS

w	water content
$w_p$	plastic limit
$w_l$	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test <sup>1</sup>
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement <sup>1</sup>
$D_R$	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 <sub>4</sub>	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (L.V.-laboratory vane test)
$\gamma$	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

$\pi$	= 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

$\gamma$	shear strain
$\Delta$	change in, e.g. in stress: $\Delta \sigma$
$\epsilon$	linear strain
$\epsilon_v$	volumetric strain
$\eta$	coefficient of viscosity
$\nu$	Poisson's ratio
$\sigma$	total stress
$\sigma'$	effective stress ( $\sigma' = \sigma - u$ )
$\sigma'_{vo}$	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stresses (major, intermediate, minor)
$\sigma_{oct}$	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
$\tau$	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
$\gamma'$	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 $\rho$ . Unit weight symbol is $\gamma$ 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
$C_\alpha$	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
$\sigma'_p$	pre-consolidation pressure
OCR	Overconsolidation ratio $= \sigma'_p / \sigma'_{vo}$

#### (e) Shear Strength

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

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

PROJECT 001-1143F				RECORD OF BOREHOLE No B11-1				1 OF 3		METRIC		
W.P. 30-95-00				LOCATION N 4915592.6; E 287961.2				ORIGINATED BY SB/PKS				
DIST SW HWY 400				BOREHOLE TYPE SEE NOTE 1				COMPILED BY LCC				
DATUM Geodetic				DATE Jan.17-26/2001				CHECKED BY ASP				
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT		UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		20 40 60 80 100	20 40 60 80 100	w <sub>p</sub>	w		
231.2	GROUND SURFACE											
0.0	Topsoil											
0.2	Silty Clay, trace sand (Fill) Brown											
230.4												
0.8	Clayey Silt to Silty Clay, trace sand Firm to very stiff Moist Grey		1	SS	19							
			2	SS	4							
			3	SS	8							
			4	SS	7							
227.1												
4.1	Silty Sand, trace clay and gravel Compact to dense Moist Grey		5	SS	26							
			6	SS	37							
225.4												
5.8	Silty Clay, trace sand Stiff Moist Grey  Contains a layer of silty sand from 6.6m to 7m depth (Elev.224.6m to 224.2m)		7	SS	14							
			8	SS	10							
			9	SS	11							
220.5												
10.7	Sandy Silt containing silty clay interlayers Dense Wet Grey		10	SS	31							
			11	SS	42							
			12	SS	48							

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+ 3, X 3: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

PROJECT 001-1143F				RECORD OF BOREHOLE No B11-1				2 OF 3		METRIC					
W.P. 30-95-00		LOCATION N 4915592.6; E 287961.2		ORIGINATED BY SB/PKS											
DIST SW HWY 400		BOREHOLE TYPE SEE NOTE 1		COMPILED BY LCC											
DATUM Geodetic		DATE Jan. 17-26/2001		CHECKED BY ASP											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa 20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X REMOULDED							
--- CONTINUED FROM PREVIOUS PAGE ---															
213.2	Sandy Silt containing silty clay interlayers Dense Wet Grey		13	SS	39		216								
							215								
			14	SS	40		214								
18.0	Silty Clay, trace sand Very stiff Moist Grey		15	SS	16		213								
							212								
			16	SS	30		211								
							210								
			17	SS	19		209								
							208								
			18	SS	15		207								
							206								
							205								
			19	SS	22		204								
							203								
							202								
			20	SS	54										
	Silty sand layer encountered between 19.8m and 20.1m depth (Elev. 211.4m to 211.1m)														
	Sandy Silt layer encountered between 29.3m and 29.6m depth (Elev. 201.9m to 201.6m)														

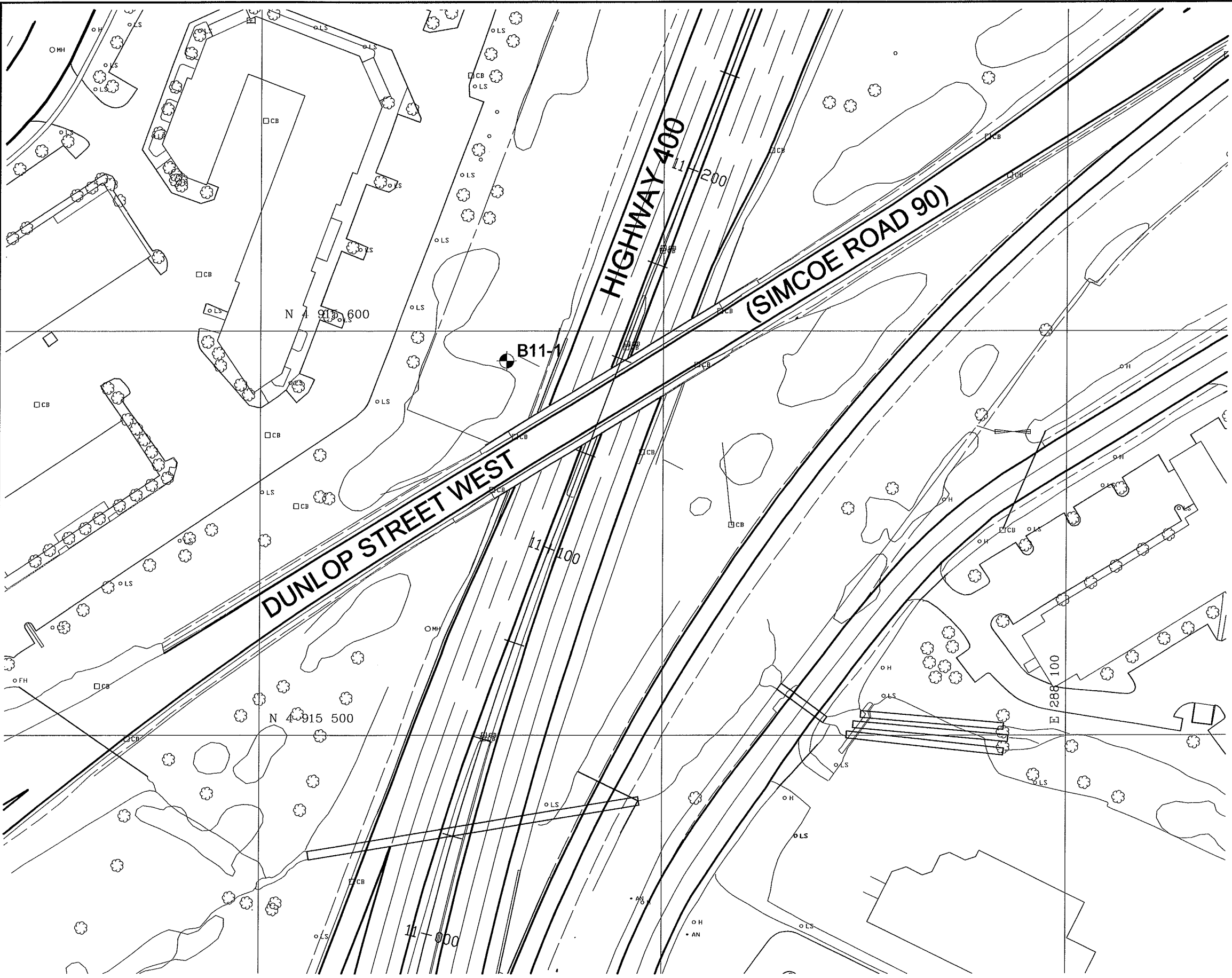
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+ 3, X 3: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

PROJECT 001-1143F			RECORD OF BOREHOLE No B11-1			3 OF 3			METRIC									
W.P. 30-95-00			LOCATION N 4915592.6; E 287961.2			ORIGINATED BY SB/PKS												
DIST SW HWY 400			BOREHOLE TYPE SEE NOTE 1			COMPILED BY LCC												
DATUM Geodetic			DATE Jan.17-26/2001			CHECKED BY ASP												
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED			WATER CONTENT (%) W <sub>p</sub> W W <sub>L</sub>			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
-- CONTINUED FROM PREVIOUS PAGE --																		
	Silty Clay, trace sand Very stiff Moist Grey						201											
							200											
			21	SS	24		199											
							198											
							197											
							196											
							195											
							194											
							193											
192.5																		
38.7	Silty Sand Very dense Brown Wet						192											
191.4																		
39.8	END OF BOREHOLE		22	SS	175/15													
Notes: 1. Hollow stem augers used to advance to 10.7m depth. After sampling, bentonite seal was placed between 11.3m and 8.8m depth, and augers were withdrawn. "N" casing was installed to 10.7m depth, then "B" casing was used for the remainder of the borehole. 2. During drilling operations, water level in open borehole was typically between 6.2m and 8.8m depth (Elev.225.0m to 222.4m). 3. Piezometer installed in second borehole drilled 4m west. Water level in piezometer measured at 1.3m depth (Elev.229.9m) on March 15, 2001.																		

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DIST HWY 400

CONT. No.

GWP No. 30-95-00

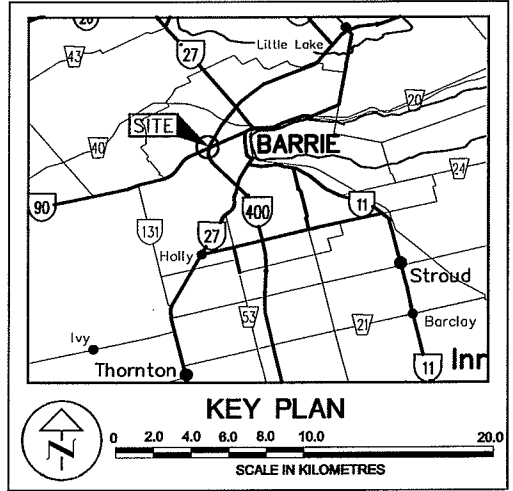
DUNLOP STREET UNDERPASS

HWY 400

BOREHOLE LOCATION PLAN

Golder Associates

Golder Associates Ltd.  
MISSISSAUGA, ONTARIO, CANADA

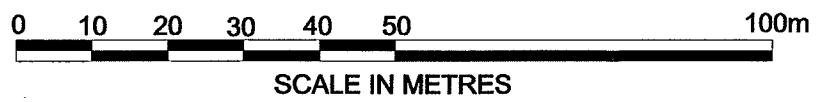


LEGEND			
	Borehole, previous investigation		
	Borehole, present investigation		
No.	ELEVATION	LOCATION	
		NORTHING	EASTING
B11-1	231.2	4,915,592.6	287,961.2

REFERENCE  
This drawing was created from digital file "50209.dwg"  
provided by URS Cole Sherman

NO.	DATE	BY	REVISION
Geocres No.			
HWY. No. 400		PROJECT NO.: 001-1143F	
SUBM'D. LCC	CHKD: ASP	DATE: JANUARY 2001	SITE 30-175
DRAWN: MHW	CHKD. LCC	APPD. ASP	DWG. 1

P143F16.DWG

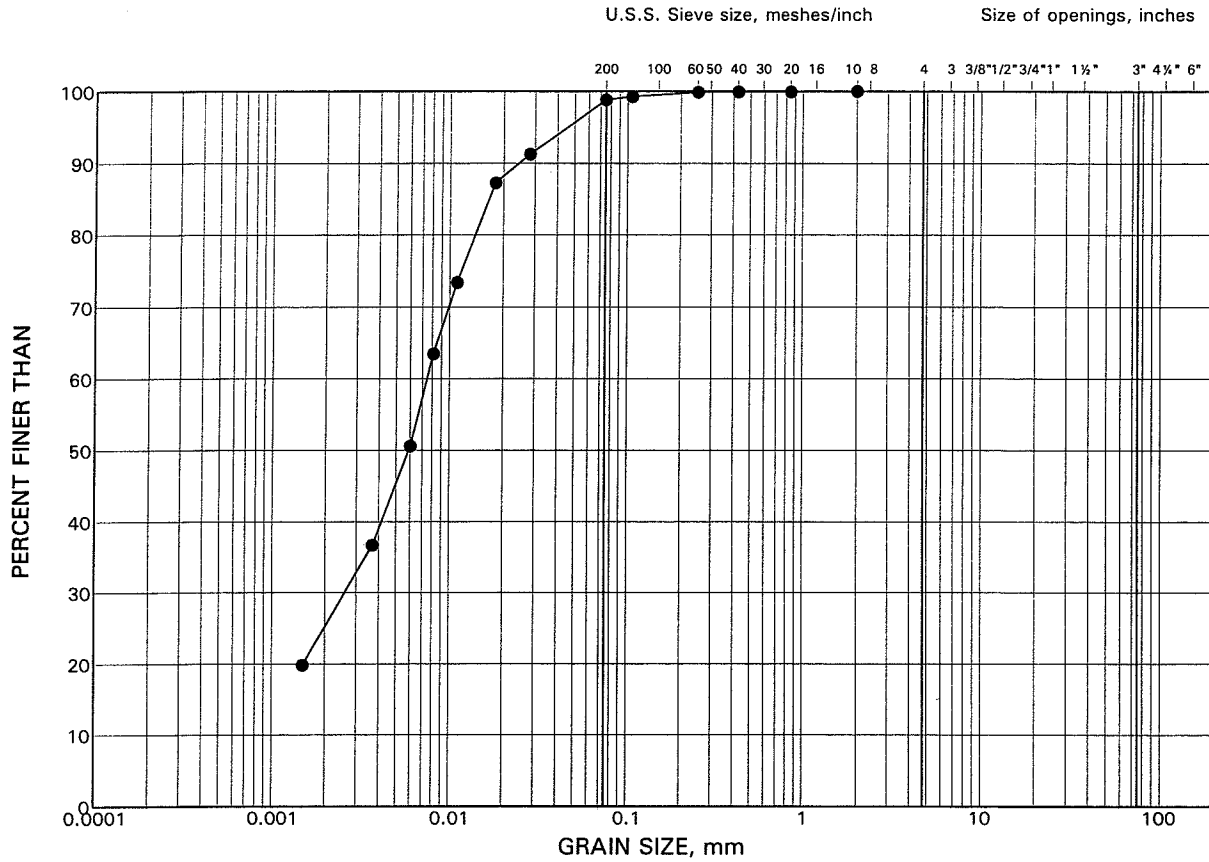


**METRIC**  
DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES  
UNLESS OTHERWISE SHOWN

# GRAIN SIZE DISTRIBUTION

Clayey Silt to Silty Clay

FIGURE 1



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

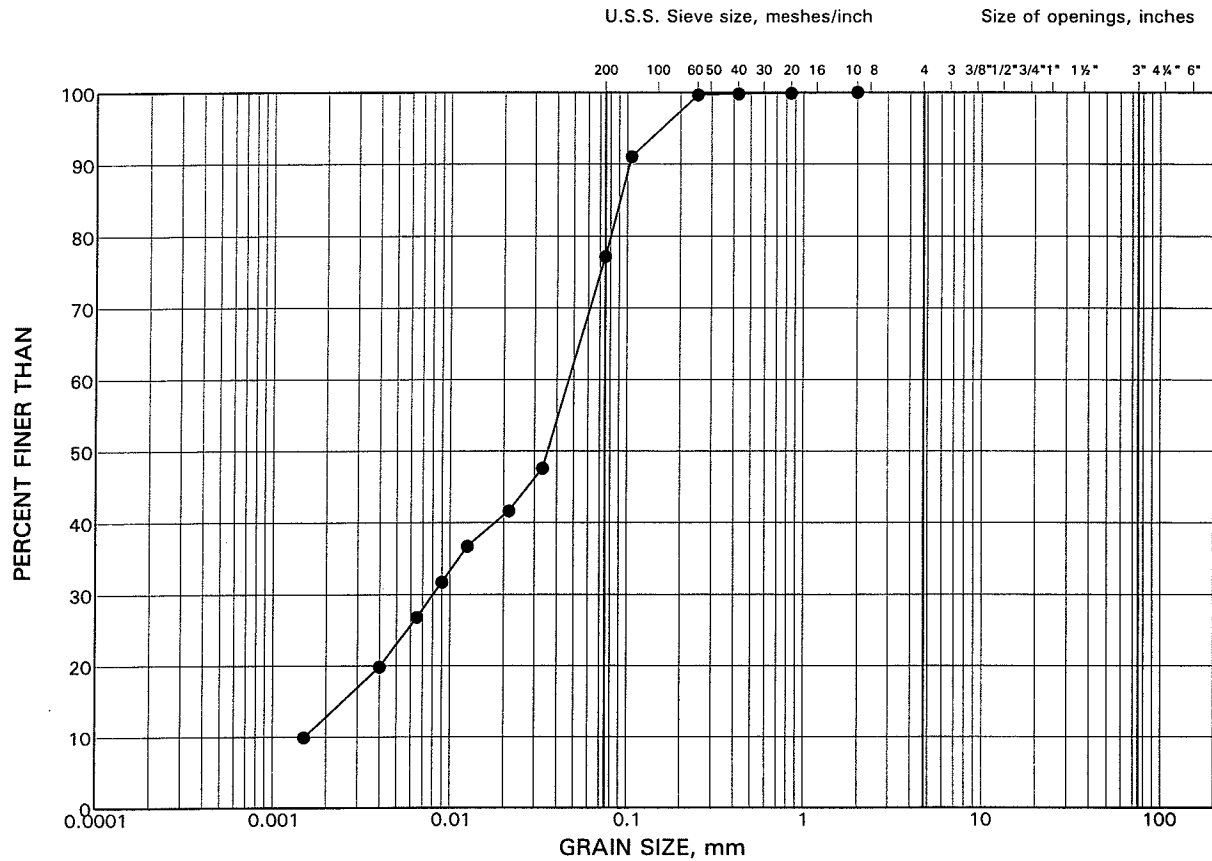
## LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
•	B11-1	4	227.8

# GRAIN SIZE DISTRIBUTION

Sandy Silt with Clayey Silt Interlayers

FIGURE 2



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

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
•	B11-1	12	217.2