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Golder Associates

# PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT FIFTH CONCESSION (PENVILLE ROAD) UNDERPASS STRUCTURE SITE 30-310 HIGHWAY 400 WIDENING FROM YORK / SIMCOE BOUNDARY TO 1 KM SOUTH OF HIGHWAY 89 G.W.P. 40-00-00

Submitted to:

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December 2001

001-1151-2

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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 replacement of the existing West Gwillimbury Fifth Concession (also known as Penville Road) underpass structure. A foundation investigation has been carried out, in which two boreholes were advanced and in-situ and laboratory testing was conducted, to determine the subsurface conditions at the site for this preliminary design study.

The terms of reference for the scope of work are outlined in 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 Fifth Concession underpass structure is located about 2.5 km south of the Simcoe Road 88 (formerly Highway 88) interchange, in the Township of West Gwillimbury, County of Simcoe. The MTO has designated this underpass as Structure Site No. 30-310.

At the existing structure, the Highway 400 grade is at about Elevation 225 m. West Gwillimbury Fifth Concession has been constructed on embankment fill up to 6.5 m in height, with its grade at about Elevation 231.5 m over Highway 400. 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 223.8 m, below the Highway 400 grade.

## 3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out at this site in December 2000, at which time two boreholes were drilled. Boreholes B2-1 and B2-2 were drilled on the east and west sides of Highway 400, respectively, at the toe of the existing Fifth Concession embankments. These boreholes were advanced from approximately Highway 400 grade to between 31 m and 34 m depth.

The investigation was carried out using a bombardier-mounted B-57 drill rig, supplied and operated by Master Soil Investigations Ltd. of Weston, Ontario. The boreholes were advanced using solid stem augers. Samples of the overburden were obtained at 0.75 m to 3 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 one shallow and one deep piezometer were installed at the borehole locations to permit monitoring of the groundwater levels at the site.

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

The borehole locations and elevations were surveyed by Callon Dietz, Ontario Land Surveyors. The borehole elevations are referenced to geodetic datum, and the northing and easting 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, in which the Fifth Concession site is located, consist primarily of varved clay and silt deposits. Where present, 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 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 B2-1 and B2-2 were drilled on the east and west sides of Highway 400, from approximately Highway 400 grade. The approximate locations and ground surface elevations for these borings are shown on the attached Drawing 1.

In summary, the subsoils below the Fifth Concession embankment fill consist of an extensive deposit of clayey silt till, containing varying proportions of sand and gravel. A layer or deposit of silty sand was encountered at the base of one of the boreholes, underlying the clayey silt till. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

### 4.2.1 Fill

A 600 mm thick layer of clayey silt fill was encountered in Borehole B2-1. This clayey silt contains trace sand, gravel and rootlets. The measured Standard Penetration Test (SPT) 'N' value was 5 blows per 0.3 m of penetration, indicating that the fill has a firm consistency.

### 4.2.2 Clayey Silt Till

Below a thin layer of clayey silt fill at Borehole B2-1, and extending from ground surface at Borehole B2-2, an extensive clayey silt till deposit is present at the site. The deposit extends to 34 m and 29 m depth (Elevations 191 m and 196 m) at the locations of Boreholes B2-1 and B2-2, respectively. The clayey silt till was not fully penetrated in Borehole B2-1. In Borehole B2-2, the till is underlain by a silty sand layer.

The clayey silt till generally contains trace to some sand and trace gravel. Near the base of both boreholes (below Elevation 193 m in Borehole B2-1 and below Elevation 198 m in Borehole B2-2), the till contains a significant proportion of sand, and is described as clayey silt with sand, and trace gravel. The results of grain size distribution tests carried out on two samples from the upper portion of the till deposit are shown on Figure 1.

The natural moisture contents measured on samples of the clayey silt till ranged from 13 to 27 per cent. The measured moisture contents in the upper, stiff to hard portion of the till were generally greater than 17 per cent; in the lower, hard portion of the till, water contents of 13 and 14 per cent were measured. Atterberg limits testing on samples of the upper portion of the clayey silt till deposit measured plastic limits of 12 to 17 per cent (typically about 17 per cent) and liquid limits of 22 to 35 per cent (typically 30 to 35 per cent). The plasticity indices ranged from 10 to 19 per cent. These limits test results indicate that the clayey silt is inorganic and of low plasticity.

The upper 27 m to 28 m of the clayey silt till deposit is predominantly stiff to very stiff. The measured SPT 'N' values over this portion of the deposit ranged from 11 to 37 blows per 0.3 m of penetration, but were typically between 11 and 30 blows per 0.3 m of penetration. In situ vane shear tests carried out in Borehole B2-2, where SPT 'N' values of 11 to 17 blows per 0.3 m of penetration were encountered between Elevation 214 m and 210 m, measured undrained shear strengths of approximately 50 kPa.

Below Elevation 198 m to 197 m, about 27 m to 28 m below Highway 400 grade, the till deposit is hard, with measured SPT 'N' values ranging from 85 to greater than 100 blows per 0.3 m of penetration, but generally greater than 100 blows per 0.3 m of penetration.

### 4.2.3 Silty Sand

A layer of silty sand was encountered at about Elevation 196 m, at the base of Borehole B2-2. This layer is at least 2 m thick, but was not fully penetrated by the boring. The recovered sample was moist, with a measured natural water content of about 15 per cent. The measured SPT 'N' value in this layer was greater than 100 blows per 0.3 m of penetration, indicating that the silty sand has a very dense relative density.

## 4.3 Groundwater Conditions

Shallow and deep piezometers were installed in the boreholes at this site. The groundwater level in the shallow piezometer, installed in Borehole B2-2, was measured in January, March and June 2001 to be between Elevation 224.6 m and 224.0 m; these water levels are 0.5 m to 1.1 m below the ground surface at the borehole location, and within 0.5 m to 1 m below the adjacent Highway 400 grade. The water level in the deep piezometer, which is sealed below Elevation 194 in Borehole B2-1, was frozen at ground surface (Elevation 224.7 m) in January and March 2001. In June 2001, the water level in the piezometer was measured to be at Elevation 225.5 m, about 0.8 m above the ground surface at the borehole location, and about 0.5 m higher than the adjacent Highway 400 grade.

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

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# PART B

# PRELIMINARY FOUNDATION DESIGN REPORT
# FIFTH CONCESSION (PENVILLE ROAD) UNDERPASS
# STRUCTURE SITE 30-310
# HIGHWAY 400 WIDENING
# FROM YORK / SIMCOE BOUNDARY
# TO 1 KM SOUTH OF HIGHWAY 89
# G.W.P. 40-00-00

# 5.0 ENGINEERING RECOMMENDATIONS

## 5.1 General

This section of the report provides preliminary foundation design recommendations for the replacement of the existing West Gwillimbury Fifth Concession (also known as Penville Road) underpass structure, associated with the widening of Highway 400. The recommendations are preliminary only and are based on interpretation of the factual data obtained from a limited number of boreholes advanced 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 will be widened by between 10 m and 29 m. Replacement of the existing Fifth Concession underpass will, therefore, be necessary.

It is understood that the preferred option is for construction of a new underpass approximately 25 m (centreline to centreline) to the north of the existing structure, rather than reconstruction on the existing Fifth Concession alignment. Borehole B2-1 was drilled approximately 25 m north of the existing alignment, roughly on the proposed alignment; however, Borehole B2-1 was drilled on the south side of the existing Fifth Concession, and is therefore located about 45 m south of the proposed realignment. The subsurface conditions have been extrapolated to the proposed site for the purposes of this preliminary study; however, it will be necessary to carry out additional borehole investigation at the detailed design stage in order to confirm the subsurface soil and groundwater conditions at the actual structure location.

## 5.2 Bridge Foundation Options

The soils at the site consist of embankment fill overlying a predominantly stiff to very stiff clayey silt till deposit, which grades to a hard clayey silt till deposit below Elevation 198 m to 197 m (about 27 m to 28 m below the Highway 400 grade).

Based on these subsurface conditions, consideration could be given to founding the replacement structure on spread footings, either placed on the native clayey silt till deposit or "perched" on a compacted granular pad within the approach embankment fill. It should be noted that there is potential for differential settlement to occur between the abutment footings and the piers due to embankment loading. Alternatively, the abutments and pier could be supported on steel H-piles driven to practical refusal within the hard clayey silt till which was encountered in the boreholes below Elevation 198 m to 197 m.

Preliminary recommendations for spread footings and for deep foundations are provided in the following sections. As noted previously, these recommendations are based on boreholes located up to about 45 m away from the proposed structure. Additional boreholes will be required at the detailed design stage to determine if the subsoil conditions at the proposed structure location are consistent with those used in establishing the preliminary geotechnical resistances given herein.

## 5.3 Spread Footings

For preliminary design of the bridge abutment and pier footings, spread footings may be placed at or below a design founding level of Elevation 223.5 m, to be founded on the generally stiff to very stiff clayey silt till 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 may be required for support of the wall footings.

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

According to the results of the preliminary boreholes, the consistency of the clayey silt till founding stratum varies significantly across the site, with lower strength on the north side of Fifth Concession as compared to the south side. For this reason, the lower strength has been used for the spread footing design parameters.

Spread footings placed on the properly prepared clayey silt till stratum at or below the design founding elevation given above may be designed using a factored geotechnical resistance at ULS of 300 kPa, assuming a 4 m wide footing.

For preliminary design, the geotechnical resistance at SLS, may be taken as 200 kPa. It should be noted that settlement of the abutment footings will be influenced by settlement of the subsoils under any increased embankment loading if along the current alignment and by new embankment loading if along a new alignment for Fifth Concession. In addition, the settlement of the footings (both abutments and the pier) will be dependent on the footing size, configuration, and applied loads.

It is estimated that up to 50 mm of consolidation settlement of the stiff clayey silt till deposit as encountered at Borehole B2-1 on the east side of Highway 400 could occur due to a 2 m raising of the approach embankment grade. For the conditions as encountered in Borehole B2-2, where the till deposit is considered to be less compressible, the consolidation settlement is expected to be less than 25 mm. The abutment footings, therefore, could experience up to 50 mm of settlement (with the upper limit expected on the east side of Highway 400) while the pier footing would not be influenced by the embankment loading and its settlement would be less than 25 mm.

If a new Fifth Concession alignment is adopted, it is estimated that construction of a new 6 m high embankment could result in up to 100 mm of settlement. This magnitude of settlement would be applicable to the east abutment footing. At the pier, as above, the settlement would be less than 25 mm, resulting in a differential settlement as high as 75 mm.

The following parameters have been assumed for the settlement assessment based on correlations with Atterberg limits:

- Preconsolidation pressure, $P_c'$ = 100 kPa to 200 kPa greater than existing in situ pressure;
- Compression index, $C_c = 0.2$; and
- Recompression index, $C_r = 0.02$.

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 determine compressibility characteristics of the subsoils 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 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 concerns with respect to settlement of the abutment footings due to embankment loading as described above will apply to perched abutments as well.

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 clayey silt till founding soils should be taken as 22 degrees; the corresponding coefficient of friction, tan $\delta$, would then be 0.4. 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 refusal within the hard clayey silt till, which was encountered in the boreholes below Elevation 198 m to 197 m, about 27 m to 28 m below the Highway 400 grade. For preliminary design, a pile tip level of Elevation 194 m may be assumed. Depending on whether the abutment pile caps are placed near the Highway 400 grade or "perched" within the Fifth Concession approach embankments, the abutment piles will be between 29 m and 35 m in length. The pier piles would be about 29 m in length.

### 5.4.1 Axial Geotechnical Resistance

It is recommended that the piles be terminated within the clayey silt till deposit, and not penetrate into the underlying silty sand deposit where present. Given the restricted penetration into the founding stratum, it has been assumed that a final set of 20 blows per 25 mm of penetration (i.e. practical refusal) will not be achieved. For preliminary design, the factored axial resistance at ULS for steel HP 310 x 110 H-piles driven at least 0.5 m into the hard clayey silt till (to about Elevation 197 m) may be taken as 1,400 kN. For preliminary design, the axial resistance at SLS for a single pile may be taken as 1,400 kN. The SLS capacity will have to be confirmed at the final design stage once the pile group configuration has been established. In addition, the distribution of the lower silty sand deposit will have to be determined during the final design stage in order to confirm the pile tip elevations.

The piles should be driven using a hammer with rated energy of about 50 kJ, and not exceeding 60 kJ. Provision should be made to re-tap selected piles to confirm the set after adjacent piles have been driven, in accordance with MTO's current Special Provision.

The placement of additional embankment fill, whether for a grade raise on the existing alignment or for embankment construction on a new alignment, will induce consolidation settlement of the stiff to very stiff clayey silt till 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 till. 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 till, the surface area of the pile within these deposits, and the settlement induced by embankment loading. 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 200 kN. The downdrag load will have to be reassessed during the detailed design stage, using shear strength and consolidation data which should be determined at the abutment 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 the Fifth Concession embankments 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 following equation:

$$k_h = \frac{k_{s1}}{5B}$$

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 existing embankment fill, assumed to be clean earth fill, and the generally stiff to very stiff clayey silt till, into the hard clayey silt till. The following ranges for the value of $k_{s1}$ may be assumed in the structural analysis; these values will have to be confirmed following the detailed design stage of the subsurface investigation.

| *Soil Unit* | $k_{s1}$ |
|---|---|
| Embankment Fill (assumed to be compacted clean earth fill) | 20 to 45 MPa/m |
| Clayey Silt Till: Above Elevation 198 m | 15 to 60 MPa/m |
| Clayey Silt Till: Below Elevation 198 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 soil of 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 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 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 Embankment Design and Construction

Based on topographic information and site reconnaissance, the existing Fifth Concession 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, or for construction of a new embankment for the realigned Fifth Concession, the new side slopes should be formed at a maximum gradient of 2H:1V. Where widening of the existing embankment occurs, benching of the existing embankment side slopes, in accordance with OPSD 208.01, should be carried out to key in the new fill.

The placement of additional Fifth Concession embankment fill will induce consolidation settlement of the stiff to very stiff clayey silt till soils below the widened or new portions of the embankment, as well as below the existing embankment if grade raising occurs on the existing alignment. Based on correlations between plasticity indices and consolidation parameters, it is estimated that up to 50 mm of settlement could occur within the native clayey silt till soils along the existing Fifth Concession alignment, for a grade raise of about 2 m. The magnitude of settlement is expected to be variable between the east and west approaches given the variability encountered in the subsoils. Up to about 100 mm of settlement could occur under a new 6 m embankment loading if a new Fifth Concession alignment is adopted. These estimated settlements have been determined by extrapolation of data from boreholes located up to 45 m from the proposed new alignment. These estimates will have to be confirmed by further borehole drilling for the proposed structure and embankments once the Fifth Concession alignment is established.

The time for the settlement to occur is dependent on the extent of sandy seams within the clayey silt as well as the applied loading. It is expected that the majority of the settlement would occur within the first two or three years after fill placement. It is recommended that the new embankment fill be placed as early as possible to allow most of the settlement to take place prior to paving of the new lanes. Some maintenance of the driving lanes and shoulders should be expected during this initial period. Alternatively, in order to reduce the amount of consolidation settlement of the underlying clayey silt deposit, consideration could be given to the use of light-weight fill for the widened approach embankments; also, the use of light-weight fill may eliminate downdrag loads on the piles. The use of light-weight fill would require further review during detailed design for this structure site. It is recommended that oedometer testing be carried out during the final design stage of the bridge to establish consolidation parameters and allow a refinement of settlement predictions.

Settlement of the newly-placed fill will also occur as a consequence of consolidation of the fill itself. In order to minimize the differential settlement (due to consolidation of the fill itself) between the existing and new portions of the Fifth Concession embankments, the use of granular fill may be considered for new construction. The majority of settlement of granular fills will occur during construction whereas the majority of settlement of cohesive fills, if used, would occur post-construction.

## 5.7 Design and Construction Considerations

### 5.7.1 Groundwater Control

Groundwater seepage into the footing excavations is expected to occur from surficial fill and from water-bearing lenses or interlayers of granular soil within the clayey silt till deposit. 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.

The soils in which the 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 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 existing embankment fill and into stiff to very stiff clayey silt till. 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 fill and clayey silt till soils would be classified as Type 2 to 3 soil. 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 |

## 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) |
| $\gamma$ | 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: $\Delta\,\sigma$
$\varepsilon$ linear strain
$\varepsilon_v$ 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

# RECORD OF BOREHOLE No B2-1



**METRIC**

PROJECT 001-1151
W.P. 40-00-00
DIST SW HWY 400
DATUM Geodetic

LOCATION N 4881670.2; E 294831.5
BOREHOLE TYPE 108mm Diameter Solid Stem Augers
DATE December 6-7, 2000

ORIGINATED BY PKS
COMPILED BY LCC
CHECKED BY ASP

| ELEV DEPTH | DESCRIPTION | SAMPLE NUMBER | TYPE | "N" VALUES | ELEVATION SCALE | WATER CONTENT (%) | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|
| 224.7 | GROUND SURFACE | | | | | | |
| 0.0 | Clayey Silt, trace sand and gravel, trace rootlets (Fill) Firm Brown Moist | 1 | SS | 5 | | | |
| 224.1 0.6 | Clayey Silt, trace to some sand, trace gravel (Till) Stiff to hard Grey Moist | 2 | SS | 15 | 224 | ~24 | |
| | | 3 | SS | 11 | 223 | | |
| | | 4 | SS | 11 | 222 | ~27 (wP ~17, wL ~35) | |
| | | 5 | SS | 11 | 221 | | |
| | | 6 | SS | 12 | 220 | ~25 | |
| | | 7 | SS | 16 | 219 | | |
| | | 8 | SS | 15 | 218 | ~12 (wP ~11, wL ~22) | 4 22 52 22 |
| | | 9 | SS | 26 | 217 | | |
| | | 10 | SS | 14 | 215 | ~17 | |
| | | 11 | SS | 27 | 214 | | |
| | | 12 | SS | 25 | 212 | ~18 | |
| | | 13 | SS | 18 | 211 | | |
| | | 14 | SS | 20 | 209 | ~23 | |
| | | 15 | SS | 32 | 206 | | |

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 B2-1



**METRIC**

PROJECT 001-1151
W.P. 40-00-00
DIST SW HWY 400
DATUM Geodetic

LOCATION N 4881670.2; E 294831.5
BOREHOLE TYPE 108mm Diameter Solid Stem Augers
DATE December 6-7, 2000

ORIGINATED BY PKS
COMPILED BY LCC
CHECKED BY ASP

SOIL PROFILE | 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, 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 FROM PREVIOUS PAGE —

| ELEV DEPTH | DESCRIPTION | NUMBER | TYPE | "N" VALUES |
|---|---|---|---|---|
| | Clayey Silt, trace to some sand, trace gravel (Till)<br>Stiff to hard<br>Grey<br>Moist | 16 | SS | 30 |
| | | 17 | SS | 37 |
| 197.1<br>27.6 | Clayey Silt, trace sand and gravel (Till)<br>Hard<br>Grey<br>Dry to moist | 18 | SS | 85 |
| | | 19 | SS | 100/.15 |
| 192.7<br>32.0 | Clayey Silt with sand, trace gravel (Till)<br>Hard<br>Grey<br>Moist | 20 | SS | 75/.15 |
| 190.9<br>33.8 | END OF BOREHOLE | | | |

Elevation scale: 204, 203, 202, 201, 200, 199, 198, 197, 196, 195, 194, 193, 192, 191

Notes:
1. The water level in the open borehole during drilling operations was at about 15m depth (Elev.209.7m).
2. The water in the piezometer was frozen at ground surface (Elev.224.7m) on January 18, 2001 and March 20, 2001. The water level in the piezometer was measured at 0.8m above ground surface (Elev.225.5m) on June 19, 2001.

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

# RECORD OF BOREHOLE No B2-2



**METRIC**

PROJECT 001-1151
W.P. 40-00-00
DIST SW HWY 400
DATUM Geodetic

LOCATION N 4881613.9; E 294800.0
BOREHOLE TYPE 108mm Diameter Solid Stem Augers
DATE December 11-14, 2000

ORIGINATED BY PKS
COMPILED BY LCC
CHECKED BY ASP

| ELEV / DEPTH | DESCRIPTION | NUMBER | TYPE | "N" VALUES | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|
| 225.1 | GROUND SURFACE | | | | |
| 0.0 | Clayey Silt, trace sand, trace gravel (Till) Stiff to hard Brown to grey Moist | 1 | SS | 30 | |
| | | 2 | SS | 37 | |
| | | 3 | SS | 31 | 1 6 50 43 |
| | | 4 | SS | 29 | |
| | Becoming grey below 2.7m depth | 5 | SS | 29 | |
| | | 6 | SS | 21 | |
| | | 7 | SS | 33 | |
| | | 8 | SS | 22 | |
| | | 9 | SS | 18 | |
| | | 10 | SS | 27 | |
| | | 11 | SS | 13 | |
| | | 12 | SS | 11 | |
| | | 13 | SS | 17 | |
| | | 14 | SS | 34 | |
| | | 15 | SS | 30 | |

Chart columns: STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (225 to 206); 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.

Continued Next Page

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

# RECORD OF BOREHOLE No B2-2



**METRIC**

PROJECT 001-1151

W.P. 40-00-00

DIST SW HWY 400

DATUM Geodetic

LOCATION N 4881613.9; E 294800.0

BOREHOLE TYPE 108mm Diameter Solid Stem Augers

DATE December 11-14, 2000

ORIGINATED BY PKS

COMPILED BY LCC

CHECKED BY ASP

— CONTINUED FROM PREVIOUS PAGE —

| ELEV / DEPTH | DESCRIPTION | SAMPLE NUMBER | TYPE | "N" VALUES | ELEVATION SCALE |
|---|---|---|---|---|---|
| | Clayey Silt, trace sand, trace gravel (Till)<br>Stiff to hard<br>Brown to grey<br>Moist | | | | 205 |
| | | | | | 204 |
| | | 16 | SS | 30 | |
| | | | | | 203 |
| | | | | | 202 |
| | | | | | 201 |
| | | 17 | SS | 28 | |
| | | | | | 200 |
| | | | | | 199 |
| | | | | | 198 |
| 197.7 / 27.4 | Clayey Silt, with sand and gravel (Till)<br>Hard<br>Grey<br>Moist | 18 | SS | 200/.15 | |
| | | | | | 197 |
| 196.1 / 29.0 | Silty Sand<br>Very dense<br>Grey<br>Moist | | | | 196 |
| | | | | | 195 |
| | | 19 | SS | 185/.23 | |
| 194.2 / 30.9 | END OF BOREHOLE | | | | |

Note:
The groundwater level in the piezometer was measured at 1.1m depth (Elev.224.0m) on January 18, 2001, at 0.5m depth (Elev.224.6m) on March 20, 2001, and at 0.8m depth (Elev.224.2m) on June 19, 2001.

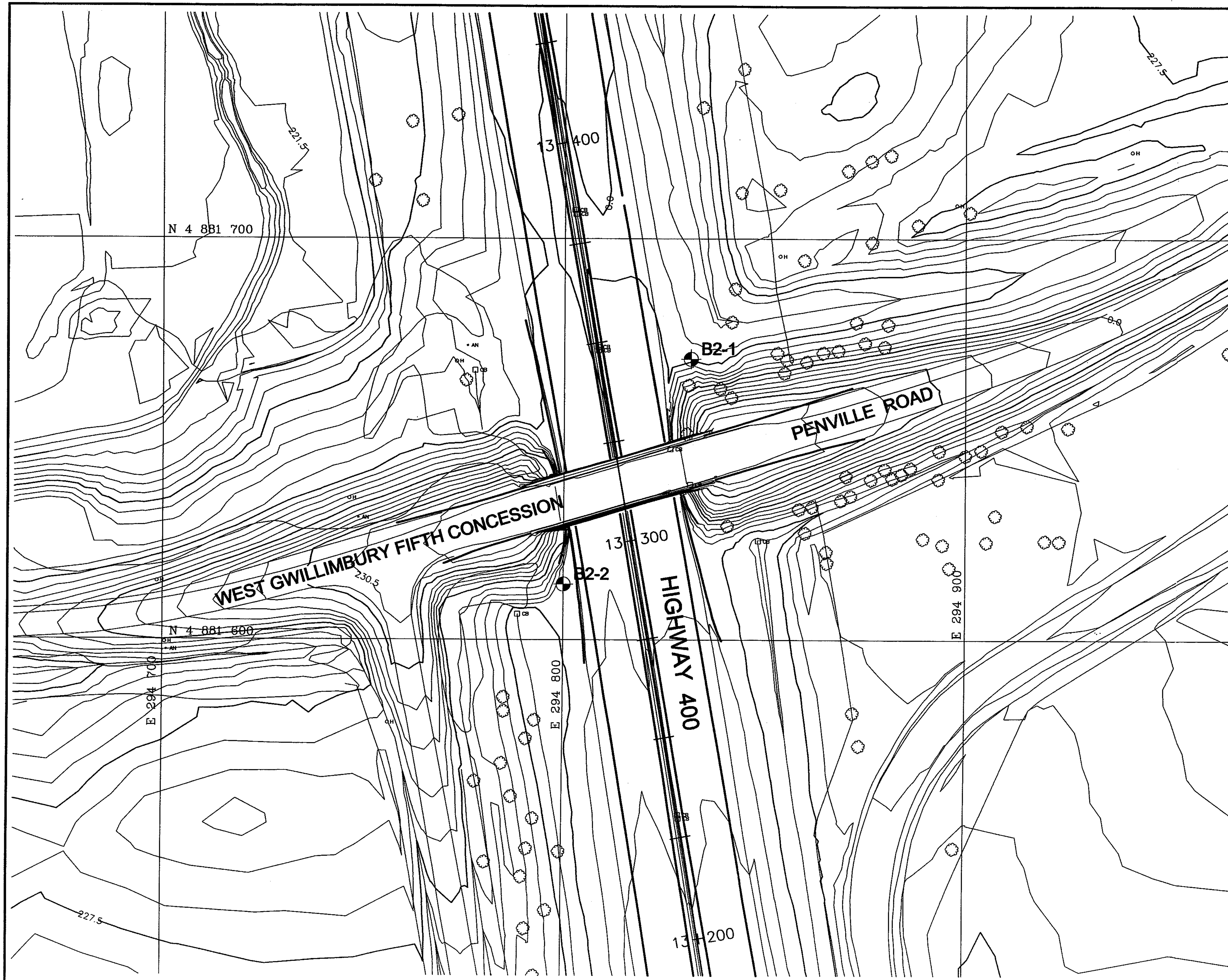
Column headings: SOIL PROFILE (ELEV / DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X REMOULDED; 20, 40, 60,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

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

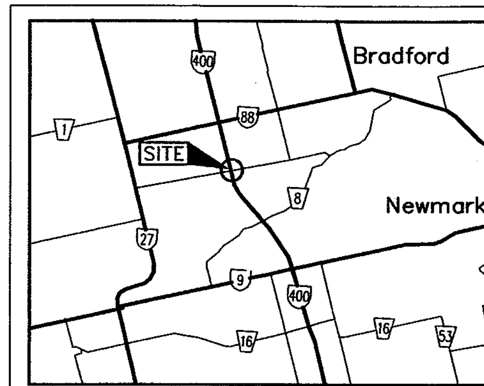
| | |
|---|---|
| DIST HWY 400<br>CONT. No.<br>GWP No. 40-00-00 | |
| FIFTH CONCESSION UNDERPASS<br>HWY 400<br>BOREHOLE LOCATION PLAN | SHEET |

Golder Associates

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA




0 10 20 30 40 50 100m
SCALE IN METRES




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 | EASTING |
| B2-1 | 224.7 | 4,881,670.2 | 294,831.5 |
| B2-2 | 225.1 | 4,881,613.9 | 294,800.0 |

REFERENCE

This drawing was created from digital file "33821.dwg" provided by URS Cole Sherman

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

| NO. | DATE | BY | REVISION |
|---|---|---|---|

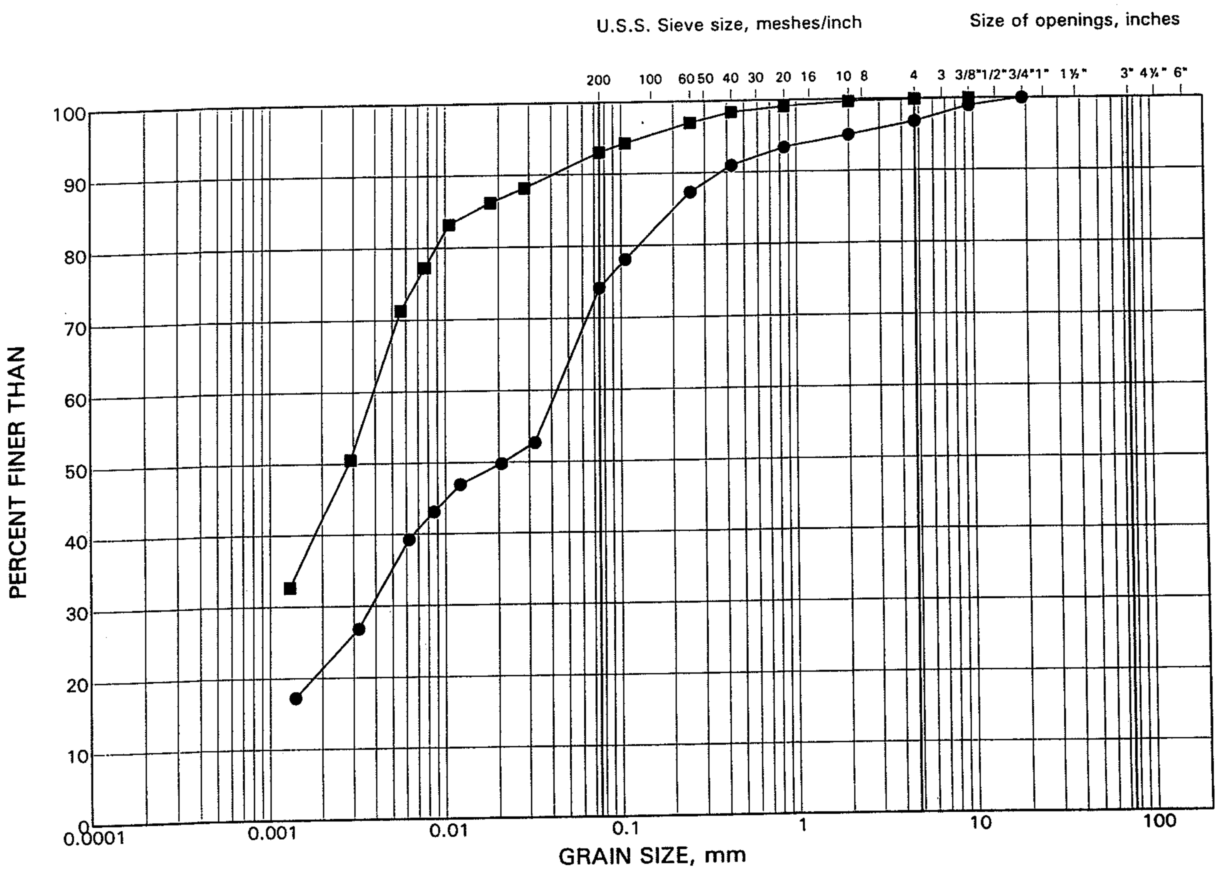
Geocres No.

| | | | |
|---|---|---|---|
| HWY. No. 400 | PROJECT NO.: 001-1151 | | |
| SUBM'D. LCC | CHKD: ASP | DATE: JANUARY 2001 | SITE 30-310 |
| DRAWN: MHW | CHKD. LCC | APPD. ASP | DWG. 1 |

P1151008.DWG

# GRAIN SIZE DISTRIBUTION TEST RESULTS
Clayey 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) |
|---|---|---|---|
| ● | B2-1 | 8 | 218.0 |
| ■ | B2-2 | 3 | 223.0 |

Project 001-1151

**Golder Associates**