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**PRELIMINARY FOUNDATION
INVESTIGATION AND DESIGN REPORT
SIMCOE ROAD 88 UNDERPASS
STRUCTURE SITE 30-211
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



001-1151-3

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PART A

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Figure 1 Grain Size Distribution Test Result – Clayey Silt to Silty Clay

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 Simcoe Road 88 (formerly Highway 88) 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 Simcoe Road 88 (formerly Highway 88) underpass structure is located about 8.5 km north of the Highway 9 interchange and about 11.5 km south of the Highway 89 interchange, in the Township of West Gwillimbury, County of Simcoe. The MTO has designated this underpass as Structure Site No. 30-211.

At the existing structure, the Highway 400 grade is at about Elevation 242 m to 242.5 m, rising northward. Simcoe Road 88 has been constructed on embankment fill, with its grade at about Elevation 249 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 240.7 m.

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out at this site in March 2001, at which time two boreholes were drilled. Boreholes B3-1 and B3-2 were drilled on the west and east sides of Highway 400, respectively, on the south side at the base of the existing Simcoe Road 88 embankments. These boreholes were advanced to about 20 m and 23 m depth below the Highway 400 grade. Due to the presence of deep snow and drainage ditches, access close to the existing abutments was not possible. The borehole in the southwest quadrant was accessed via the commuter parking lot, and is located on the west side of the drainage ditch.

The investigation was carried out using bombardier-mounted B-57 and D-50 drill rigs, supplied and operated by Master Soil Investigations Ltd. of Weston, Ontario. Borehole B3-1 was advanced using solid stem augers, while Borehole B3-2 was advanced using hollow stem augers. Samples of the overburden were obtained at 0.75 m to 1.5 m intervals of depth using 50 mm outside diameter split-spoon samplers, in accordance with the Standard Penetration Test (SPT) procedure. The water levels in the open boreholes were observed throughout the drilling operations, and a shallow piezometer was installed at both borehole locations to permit monitoring of the groundwater level at the site.

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

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

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

This 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 Simcoe Road 88 site is located, consist primarily of clay and silt deposits. These deposits overlie till with drumlins as found in the Peterborough Drumlin Field. The drumlins 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 B3-1 and B3-2 were drilled on the west and east 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 Simcoe Road 88 embankment fill consist of a deposit of clayey silt to silty clay, overlying hard clayey silt till. In one of the boreholes, the clayey silt till grades to silty sand till, and contains a layer of silty sand. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Fill

Both boreholes were advanced near the south toe of the Simcoe Road 88 embankments. About 2 m and 0.2 m of fill (or reworked native soil) were encountered in Boreholes B3-1 and B3-2, respectively. The fill consists of clayey silt, containing trace to some sand and gravel, as well as trace rootlets and organics. The measured Standard Penetration Test (SPT) 'N' values were 4 and 5 blows per 0.3 m of penetration, indicating that this material has a firm consistency.

4.2.2 Clayey Silt to Silty Clay

An extensive clayey silt to silty clay deposit is present at the site below the fill. The surface of the clayey silt to silty clay was encountered at about Elevation 239 m in Borehole B3-1 on the west side of the highway, and at about Elevation 241.5 m in Borehole B3-2 on the east side of the highway. The clayey silt to silty clay deposit is about 14 m to 16 m thick; its base was encountered at about Elevation 225 m in both boreholes.

The clayey silt to silty clay deposit contains trace sand and gravel. Faint layering was observed in some samples. Sand seams were encountered within the deposit below Elevation 230 m (about 12 m depth) in Borehole B3-2. A grain size distribution test result for a sample of this glaciolacustrine deposit is shown on Figure 1.

The natural moisture contents measured on samples of the clayey silt to silty clay deposit ranged from 20 to 28 per cent. Atterberg limits testing measured plastic limits of 16 to 18 per cent,

liquid limits of 29 to 37 per cent, and plasticity indices of 12 to 20 per cent. The limits test results indicate that the clayey silt to silty clay is inorganic and of low to intermediate plasticity.

The deposit is predominantly stiff to very stiff. The measured SPT 'N' values ranged from 9 to 70 blows per 0.3 m of penetration, but were typically between 10 and 30 blows per 0.3 m of penetration.

4.2.3 Clayey Silt Till to Silty Sand Till

Below the clayey silt to silty clay glaciolacustrine deposit, a deposit of clayey silt till is present. The surface of the till was encountered in both boreholes at about Elevation 225 m. The till extends to the maximum depth investigated, between 20 m and 23 m below Highway 400 grade, corresponding to about Elevation 221 m on the west side of Highway 400 and Elevation 218 m on the east side of the highway.

In Borehole B3-1, a 1 m thick layer of silty sand till was encountered within the clayey silt till below about 17 m depth, between about Elevations 224 m and 223 m. The till deposit is hard / very dense, with measured SPT 'N' values ranging from 73 to greater than 100 blows per 0.3 m of penetration, but generally greater than 100 blows per 0.3 m of penetration.

4.2.4 Silty Sand

A layer of wet silty sand was encountered at the base of Borehole B3-1. This layer is at least 1 m thick, but was not fully penetrated by the boring. The silty sand may represent an interlayer within the clayey silt till, or a deposit underlying the till.

The measured SPT 'N' value in this layer was 17 blows per 0.3 m of penetration. However, this SPT 'N' value is not representative of the in situ density of the silty sand material, due to sample disturbance caused by groundwater inflow to the borehole prior to sampling. It is probable that the silty sand layer has a dense to very dense relative density.

4.3 Groundwater Conditions

The water level in the open boreholes was at Elevations 233.1 m and 241 m in Boreholes B3-1 and B3-2, respectively, during drilling and on completion of the boreholes. These levels correspond to depths of 8.3 m and 0.7 m below ground surface in these boreholes.

Shallow piezometers were installed in both boreholes. The water level in the piezometer installed in Borehole B3-1 was measured to be at Elevation 236.4 m in March 2001, and at Elevation 239.3 m in June 2001. The latter level corresponds to about 2.1 m below the ground surface at the borehole location, and about 2.7 m to 3.2 m below the nearby Highway 400 grade. It is noted that Borehole B3-1 was drilled adjacent to a 2.5 m deep drainage ditch; it is considered that the water level in the piezometer in this borehole is controlled by its proximity to this drainage feature.

The water level in the piezometer installed in Borehole B3-2 was measured to be between Elevation 241.5 m and 241.3 m in March and June 2001, approximately 0.2 m and 0.4 m below the ground surface, and about 0.5 m to 1 m below 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.

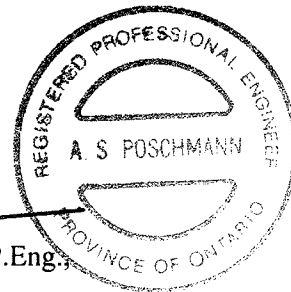
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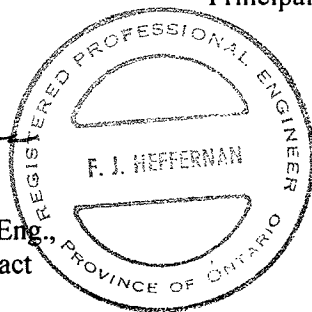
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PART B

PRELIMINARY FOUNDATION DESIGN REPORT

SIMCOE ROAD 88 UNDERPASS

STRUCTURE SITE 30-211

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 Simcoe Road 88 (formerly Highway 88) 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 platform will be widened by between 10 m and 29 m. Replacement of the existing Simcoe Road 88 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 Simcoe Road 88 alignment. Boreholes B3-1 and B3-2 were drilled on the south side of the existing Simcoe Road 88 embankment, and are located about 50 m away from 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 to silty clay deposit, underlain by a hard clayey silt till deposit below Elevation 225 m (about 17 m to 18 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 to silty clay deposit or “perched” on a compacted granular pad within the approach embankment fill. Alternatively, the

abutments and pier could be supported on steel H-piles driven to found within the hard clayey silt till which was encountered in the boreholes below Elevation 225 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 approximately 50 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 about Elevation 237.5 m on the west side of Highway 400, and about Elevation 240 m on the east side of the highway, to extend below the fill and stiff portions of the clayey silt to silty clay and be founded on the very stiff clayey silt to silty clay. 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. It is noted that the design founding level given for the west side of the highway is governed by the base of fill elevation as encountered in Borehole B3-1, which was drilled at the southwest toe of the Simcoe Road 88 embankment. Further subsurface investigation is required at the detailed design stage to establish the surface elevation of the native soil within the proposed foundation limits and to confirm the design founding level.

Alternatively, consideration could be given to the use of abutment footings perched on the embankment fill.

5.3.1 Axial Geotechnical Resistance

Spread footings placed on the properly prepared clayey silt to silty clay stratum at or below the design founding elevations given above may be designed using a factored geotechnical resistance at ULS of 500 kPa, assuming a 4 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, whether the grade is raised on the existing Simcoe

Road 88 alignment or a new alignment is constructed. Further discussion regarding the anticipated settlement is provided in Section 5.6.

For preliminary design purposes, the geotechnical resistance at SLS may be taken as 300 kPa. However, the geotechnical resistance at SLS for the abutment footings on a new alignment will be governed by the consolidation settlement of the soils underlying the new approach embankments. The geotechnical resistance at SLS will have to be reviewed following the detailed design stage of subsurface investigation, once the footing size, embankment configuration and loadings are known. Additional field and laboratory testing should be carried out at that time to determine compressibility characteristics of the subsoils to refine the settlement predictions, particularly if embankment construction on a new alignment is adopted.

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 settlement expected due to consolidation 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 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 properly prepared native clayey silt to silty clay 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 found within the hard clayey silt till, which was encountered in the boreholes below Elevation 225 m, about 17 m to 18 m below the Highway 400 grade. A silty sand layer was encountered in Borehole B3-1 below about Elevation 221.9 m and a silty sand till interlayer was encountered between about Elevations 224.3 m and 223.4 m. The presence of the lower silty sand layers will influence the tip resistance available to the pile where driven into this layer, which was not encountered in Borehole B3-2. Due to the variability in the founding stratum, it is recommended that there be flexibility in the pile length. Additional borehole drilling will be required at the final design stage to further define the variability and extent of the silty sand layer(s).

5.4.1 Axial Geotechnical Resistance

For preliminary design of deep foundations on the east side of Highway 400, the factored axial resistance at ULS for steel HP 310 x 110 H-piles driven to found within the hard clayey silt till below Elevation 225 m may be taken as 1,600 kN. For preliminary design, the axial resistance at SLS for a single pile may be taken as 1,400 kN. The axial resistance at SLS must be confirmed at the final design stage, once the pile group configuration is established. A design tip level of Elevation 220 m may be used for preliminary design.

On the west side of Highway 400, it is recommended that the piles be terminated within the clayey silt till deposit and not penetrate into the underlying silty sand layer which was encountered between Elevation 222 m and 221 m (the extent proven in the drilling to date). Due to the potentially limited pile length within the founding till deposit, the factored axial resistance at ULS for steel HP 310 x 110 piles driven to Elevation 223 m may be taken as 1,400 kN for preliminary design. The settlement of a single pile under a load of 1,400 kN at this site is expected to be less than 25 mm, and the axial resistance at SLS may be taken as 1,400 kN for preliminary design. The axial resistance at SLS will have to be confirmed at the final design stage, once the pile group configuration is established.

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 to silty clay soils; the expected settlement is further discussed in Section 5.6. 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 a portion of the pile shaft embedded within the consolidating clayey silt to silty clay. The magnitude of the downdrag load acting on the pile is a function of the compression of the pile, the skin friction that develops between the pile and the clayey silt to silty clay, the surface area of the pile within these soils, and

the 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 150 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 Simcoe Road 88 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 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:

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

The piles will be driven through embankment fill and the generally stiff to very stiff clayey silt to silty clay, into the hard clayey silt till. The following ranges for the value of n_h and 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.

<i>Soil Unit</i>	<i>n_h</i>	<i>$k_{s,l}$</i>
Embankment Fill (assumed to be compacted granular fill)	5 to 15 MPa/m	—
Clayey Silt to Silty Clay (Above Elevation 225 m)	—	15 to 60 MPa/m
Clayey Silt Till (Below Elevation 225 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:

<i>Pile Spacing in Direction of Loading d = Pile Diameter</i>	<i>Subgrade Reaction Reduction Factor R</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 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/m³

Coefficients of lateral earth pressure:

Active, K_a 0.35

At rest, K_o 0.50

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

	Granular 'A'	Granular 'B'
		Type 2
Soil unit weight:	22 kN/m ³	21 kN/m ³
Coefficients of lateral earth pressure:		
Active, K_a	0.27	0.31
At rest, K_o	0.43	0.47

- If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the wall support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design.

It should be noted that the above design recommendations and parameters assume level backfill and ground surface behind the abutment and retaining walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

5.6 Embankment Design

Based on topographic information and site reconnaissance, the existing Simcoe Road 88 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 Simcoe Road 88, 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 embankment fill will induce consolidation settlement of the stiff to very stiff clayey silt to silty clay 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, the magnitude of this settlement has been estimated using the following parameters:

- Preconsolidation pressure, $P_c' = 200$ kPa greater than existing in situ pressure;
- Compression index, $C_c = 0.25$; and
- Recompression index, $C_r = 0.025$.

It is estimated that up to 25 mm of settlement could occur within these soils due to a grade raise of about 2 m on the existing Simcoe Road 88 alignment. For a new 6 m high embankment, it is estimated that up to about 75 mm of consolidation settlement could occur. The magnitude of settlement is expected to be variable between the east and west approaches given the variability encountered in the subsoils. It is stressed that the estimated settlements have been determined by extrapolation of data from boreholes located about 50 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 Simcoe Road 88 alignment is established.

The time for the settlement to occur is dependent on the extent of sandy seams within the clayey silt to silty clay 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 to silty clay 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 Simcoe Road 88 embankment, 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 to silty clay 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 to silty clay. 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 upper portions of the clayey silt to silty clay deposit 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.

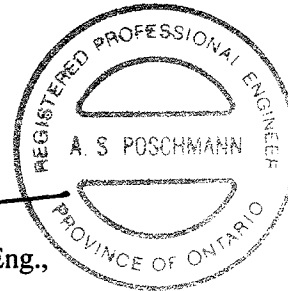
GOLDER ASSOCIATES LTD.



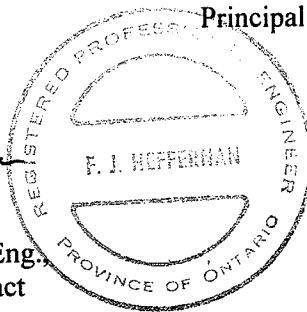
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LCC/ASP/FJH/clg

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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.)

(b) Cohesive Soils

Consistency

	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

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 cm² 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 ¹
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO ₄	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

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

LIST OF SYMBOLS

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

I GENERAL

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

II STRESS AND STRAIN

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

III SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation
*	Density symbol is ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)

(a) Index Properties (con't.)

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

(c) Hydraulic Properties

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

(d) Consolidation (one-dimensional)

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

(e) Shear Strength

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

Notes: 1. $\tau = c' + \sigma' \tan \phi'$

2. Shear strength = (Compressive strength)/2

PROJECT <u>001-1151</u>			RECORD OF BOREHOLE No B3-1			1 OF 2			METRIC		
W.P. <u>40-00-00</u>			LOCATION <u>N 4884346.4; E 294273.5</u>			ORIGINATED BY <u>SB</u>					
DIST <u>SW</u> HWY <u>400</u>			BOREHOLE TYPE <u>108mm Diameter Solid Stem Augers</u>			COMPILED BY <u>LCC</u>					
DATUM <u>Geodetic</u>			DATE <u>March 7-8, 2001</u>			CHECKED BY <u>ASP</u>					

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40						60	80
241.4	GROUND SURFACE															
0.0	Topsoil															
0.3	Clayey Silt, trace to some sand and gravel, trace rootlets and organics (Fill) Firm Grey-brown Moist		1	SS	5											
			2	SS	4											
239.1																
2.3	Clayey Silt to Silty Clay, trace sand and gravel Stiff to hard Brown becoming grey below 6.1m Moist		3	SS	12											
			4	SS	12											
			5	SS	17											
			6	SS	42											
			7	SS	70											
			8	SS	21											
			9	SS	34											
			10	SS	24											
			11	SS	12											
			12	SS	14											
			13	SS	22											
225.0																
16.4	Clayey Silt, trace sand and gravel (Till) Hard Grey		14	SS	141											
224.3																
17.1	Silty Sand, some gravel, trace clay (Till) Very dense Grey Moist															
223.4																
18.0	Clayey Silt, trace sand and gravel (Till) Hard Grey Moist															
221.9																
19.5																

ON_MOT 001-1151.GPJ ON_MOT.GDT 7/12/01

Continued Next Page

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

PROJECT 001-1151				RECORD OF BOREHOLE No B3-1				2 OF 2		METRIC			
W.P. 40-00-00				LOCATION N 4884346.4; E 294273.5				ORIGINATED BY SB					
DIST SW HWY 400				BOREHOLE TYPE 108mm Diameter Solid Stem Augers				COMPILED BY LCC					
DATUM Geodetic				DATE March 7-8, 2001				CHECKED BY ASP					
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC NATURAL LIQUID UNIT WEIGHT REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	20 40 60 80 100	W _p W W _L	WATER CONTENT (%)	γ	GR SA SI CL
221.0	Silty Sand		16	SS	17		221						
20.4	Dense to very dense * Grey Wet Silty Sand Dense to very dense * Grey Wet END OF BOREHOLE												
Notes: 1. Water level in open borehole on completion of drilling at about 8.3m depth (Elev.233.1m). 2. Piezometer was installed in a shallow borehole located about 2m north of the deep borehole. 3. The groundwater level in the piezometer was measured at 5m depth (Elev.236.4m) on March 20, 2001, and at 2.1m depth (Elev.239.3m) on June 19, 2001. * Measured SPT "N" value not considered to be representative due to disturbance caused by groundwater flow through the silty sand prior to sampling.													

ON_MOT 001-1151.GPJ ON_MOT.GDT 7/12/01

PROJECT 001-1151			RECORD OF BOREHOLE No B3-2			1 OF 2		METRIC					
W.P. 40-00-00			LOCATION N 4884372.7; E 294253.7			ORIGINATED BY GPD							
DIST SW HWY 400			BOREHOLE TYPE 162mm Diameter Hollow Stem Augers			COMPILED BY LCC							
DATUM Geodetic			DATE March 7-8, 2001			CHECKED BY ASP							
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID		UNIT WEIGHT		REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	W _p W W _L	WATER CONTENT (%)	γ	GR SA SI CL	
241.7	GROUND SURFACE												
0.0	Clayey Silt, some sand and rootlets (Fill)		1	SS	8		241						
0.2	Firm Dark brown Clayey Silt, trace sand and gravel Stiff to very stiff Brown becoming grey below 3.8m depth Moist		2	SS	12		240						
			3	SS	26		239						
			4	SS	20		238						
			5	SS	20		237						
			6	SS	15		236						
			7	SS	14		235						
			8	SS	9		234						
			9	SS	9		233						
			10	SS	12		232						
			11	SS	20		231						
			12	SS	10		230						
	Occasional sand seams below 12.2m depth		13	SS	14		229						
			14	SS	19		228						
							227						
							226						
224.9	Clayey Silt, trace sand and gravel, occasional sand seams (Fill)						225						
16.8	Hard Grey Moist		15	SS	73		224						
							223						
							222						

ON_MOT 001-1151.GPJ ON_MOT.GDT 7/12/01

Continued Next Page

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

PROJECT 001-1151			RECORD OF BOREHOLE No B3-2			2 OF 2			METRIC				
W.P. 40-00-00			LOCATION N 4884372.7; E 294253.7			ORIGINATED BY GPD							
DIST SW HWY 400			BOREHOLE TYPE 162mm Diameter Hollow Stem Augers			COMPILED BY LCC							
DATUM Geodetic			DATE March 7-8, 2001			CHECKED BY ASP							
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC NATURAL LIQUID UNIT WEIGHT REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	N° VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	W _p W W _L	WATER CONTENT (%)	γ	GR SA SI CL	
--- CONTINUED FROM PREVIOUS PAGE ---													
218.4 23.3	Clayey Silt, trace sand and gravel, occasional sand seams (Till) Hard Grey Moist		16	SS	116		221						
			17	SS	132/23		220						
			18	SS	114		219						
END OF BOREHOLE Notes: 1. Water level in open borehole during drilling operations at about 0.7m depth (Elev.241.0m). 2. The water level in the piezometer was measured at 0.2m depth (Elev.241.5m) on March 20, 2001 and at 0.4m depth (Elev.241.3m) on June 19, 2001.													

ON MOT 001-1151.GPJ ON MOT.GDT 7/12/01

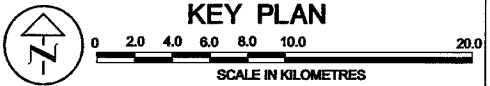
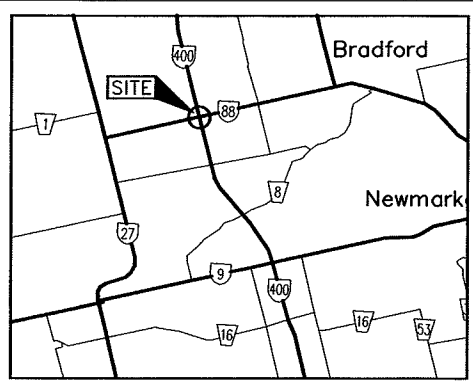
DIST HWY 400
CONT. No.
GWP No. 40-00-00
SIMCOE ROAD 88 UNDERPASS
HWY 400
BOREHOLE LOCATION PLAN



SHEET



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



LEGEND

- Borehole, previous investigation
- Borehole, present investigation

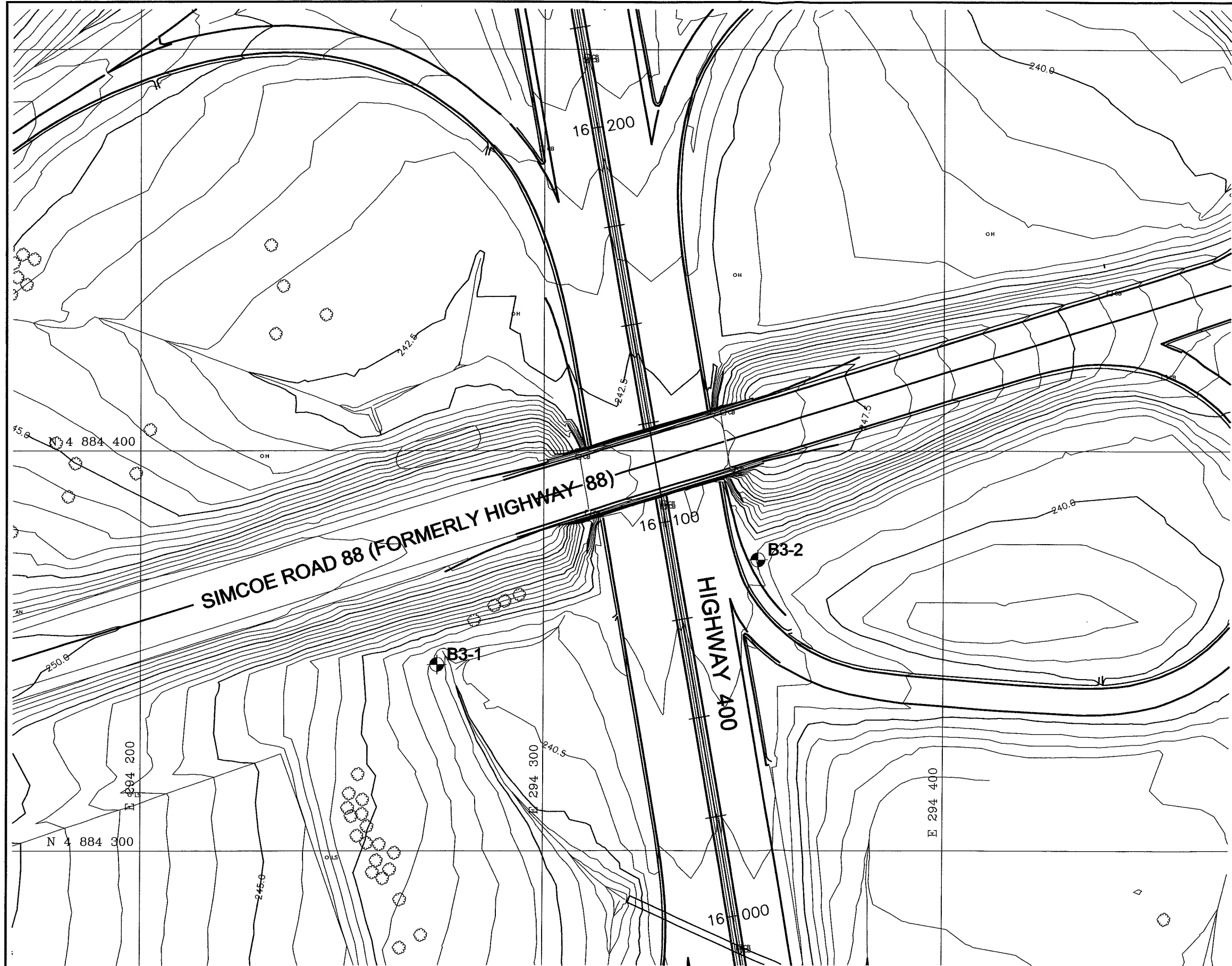
No.	ELEVATION	LOCATION	
		NORTHING	EASTING
B3-1	241.4	4,884,346.4	294,273.5
B3-2	241.7	4,884,372.7	294,253.7

REFERENCE

This drawing was created from digital file "33819.dwg"
provided by URS Cole Sherman

NO.	DATE	BY	REVISION
Geocres No.			
HWY. No. 400		PROJECT NO.: 001-1151	
SUBM'D. LCC	CHKD: ASP	DATE: JANUARY 2001	SITE 30-211
DRAWN: MHW	CHKD. LCC	APPD. ASP	DWG. 1

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

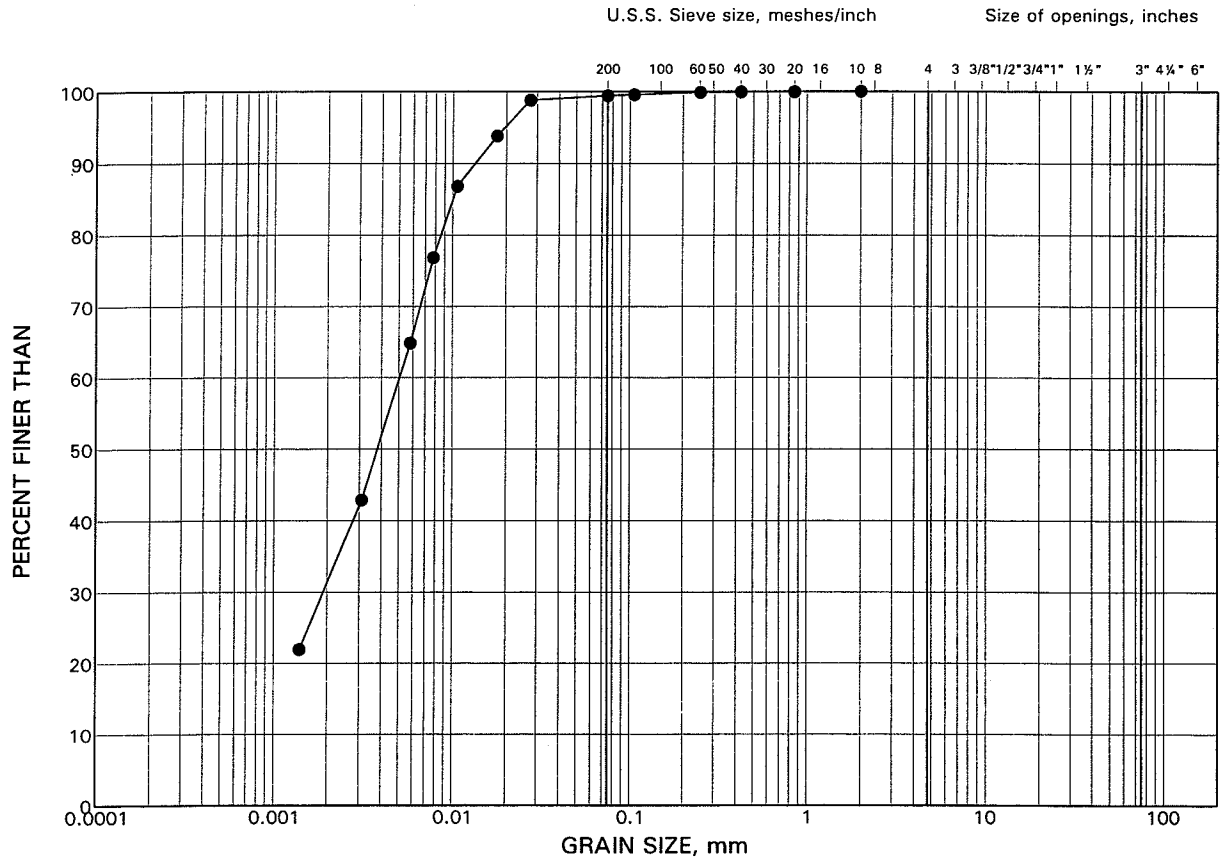


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GRAIN SIZE DISTRIBUTION TEST RESULT

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)
•	B3-1	6	236.2