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PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT SUNNIDALE ROAD UNDERPASS STRUCTURE SITE 30-173 HIGHWAY 400 WIDENING FROM 1 KM SOUTH OF HIGHWAY 89 TO HIGHWAY 11 G.W.P. 30-95-00, AGREEMENT NO. 3005-A-000074

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PART A - PRELIMINARY FOUNDATION INVESTIGATION REPORT

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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 1 km south of Highway 89, northerly 30 km to Highway 11, in Simcoe County, Ontario. Foundation engineering services are required for the widening and / or replacement of eighteen existing overpass and underpass structures, as well as five structural culverts.

This report addresses the replacement of the Sunnidale Road underpass structure in Barrie, Ontario. A subsurface investigation has been carried out, in which two boreholes were advanced, and in-situ and laboratory testing was conducted, to determine the subsurface conditions at the site for this preliminary design study.

The terms of reference for the scope of work are outlined in Golder Associates' Proposal No. P01-1192, dated June 2000.

2.0 SITE DESCRIPTION

The existing Sunnidale Road underpass is located about 1.5 km north of the Dunlop Street (Simcoe Road 90, formerly Highway 90) interchange and 800 m south of the Bayfield Street (Highway 26) interchange, in Barrie, Ontario. The MTO has designated this underpass as Structure Site 30-173.

In the immediate vicinity of the structure site, Highway 400 has been constructed in a cut up to 6 m deep. The Highway 400 grade is at about Elevation 248.5 m to 249 m, rising toward the north. The Sunnidale Road grade rises westward, from about Elevation 255 m to 257 m over Highway 400.

The existing single-span underpass was constructed in the early 1950s under Contracts 50-11 and 50-66. 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 and associated wing walls and retaining walls are supported on spread footings which are founded at about Elevation 247.2 m.

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out at this site in February 2001, at which time two boreholes were drilled. Boreholes B13-1 and B13-2 were advanced on the west and east sides of Highway 400, respectively, from Sunnidale Road grade. The boreholes were extended to between 12.5 m and 13.5 m depth, about 5 m to 6 m below the Highway 400 grade.

The investigation was carried out using a truck-mounted D-90 drill rig supplied and operated by Master Soil Investigations Ltd. of Weston, Ontario. Both boreholes were 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 piezometer was installed in Borehole B13-2 to permit monitoring of the groundwater level at this location.

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

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

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

This 30 km section of Highway 400 traverses, from south to north, the following physiographic regions as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, Third Edition, 1984): the Simcoe Lowlands; the Peterborough Drumlin Field; a second lobe of the Simcoe Lowlands; and the Simcoe Uplands. Along Highway 400, the Simcoe Lowlands are present from the southern limit of the project to just south of Innisfil Creek, and again from Essa Road (Simcoe Road 30, formerly Highway 27) to about 1 km north of Dunlop Street (Simcoe Road 90, formerly Highway 90). The Peterborough Drumlin Field occupies the belt between these lobes of the Simcoe Lowlands, extending from just south of Innisfil Creek, which is located about 1 km north of Highway 89, to Essa Road. The Simcoe Uplands extend from about 1 km north of Dunlop Street to beyond the northern limit of the project at Highway 11.

The two sections where Highway 400 crosses the Simcoe Lowlands consist of two lobes of a sand plain which include the shores of Kempenfelt Bay, the Nottawasaga River and Innisfil Creek. The surficial soils of these sections of the Simcoe Lowlands consist primarily of sand, although silt, clay or peat may be found in low-lying areas.

The surficial soils in the Peterborough Drumlin Field consist primarily of gravelly sand till or sand and gravel deposits. Drumlins (glacially-shaped hills) are more frequent in the southern portion of the section of the Peterborough Drumlin Field traversed by Highway 400. Deposits of silt, clay or peat may be found in the low-lying areas between drumlins.

The surficial soils in the Simcoe Uplands physiographic region, in which the Sunnidale Road site is located, are primarily sandy silt till deposits, known to contain boulders. Low-lying areas may be infilled with shallow sand and gravel deposits, which are shoreline deposits of a former glacial lake that once flooded the area.

4.2 Site Stratigraphy

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

Boreholes B13-1 and B13-2 were advanced on the west and east sides of Highway 400, from Sunnidale Road grade. The locations and ground surface elevations for these boreholes are shown on the attached Drawing 1.

In summary, the soils below the Sunnidale Road approach fill (behind the existing abutment walls) consist of a thin layer of silty sand, overlying silty sand to clayey silt till. In Borehole B13-2, on the east side of Highway 400, the silty sand till is underlain by a deposit of very dense silty sand to sand. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Approach Fill

The boreholes were advanced through the approach fill behind the abutments. This fill extended from just below the pavement structure to approximately 7 m depth, to about Elevation 249.5 m on the west side of Highway 400, and about Elevation 248 m on the east side of the highway.

The approach fill is comprised of sand and gravel; the presence of cobbles was noted in Borehole B13-1. The measured Standard Penetration Test (SPT) 'N' values ranged from 9 to 62 blows, but were typically between 10 and 30 blows per 0.3 m of penetration, indicating that the fill has a predominantly compact relative density.

4.2.2 Surficial Silty Sand

A 0.7 m to 1.1 m thick layer of silty sand was encountered below the approach fill. This silty sand contains trace clay and gravel, and trace quantities of wood fragments and organics. It is considered that this deposit could represent a fill or reworked native soil, due to the presence of wood fragments in the samples. The samples were moist, with measured natural water contents of 8 and 10 per cent. The measured SPT 'N' values were 19 and 31 blows, indicating that this silty sand has a compact relative density.

4.2.3 Silty Sand to Clayey Silt Till

The surficial silty sand layer overlies a deposit of silty sand till. The top of this deposit was encountered at about Elevation 248.5 m and 247.5 m in the boreholes on the west and east sides of the highway, respectively. The silty sand till is about 1.5 m to 2 m thick at the borehole locations; on the west side of the highway, the silty sand till grades to a clayey silt till, while on the east side of the highway, the silty sand till overlies a deposit of silty sand to sand.

The silty sand till contains trace clay and trace to some gravel. A grain size distribution test result for a representative sample of this till is shown on Figure 1. This silty sand till was moist, with two measured water contents of 7 and 8 per cent. The measured SPT ‘N’ values ranged from 21 to greater than 100 blows, but were typically over 80 blows per 0.3 m of penetration. The relative density therefore ranges from compact to very dense, but is predominantly very dense.

The clayey silt portion of the till, encountered below about Elevation 246.5 m in Borehole B13-1 on the west side of the highway, contains trace to some sand and gravel. The measured natural water content was 9 per cent, and Atterberg Limits testing measured plastic and liquid limits of 11 and 14 per cent, and a plasticity index of 3 per cent. The till is therefore inorganic and of low plasticity. It has a hard consistency, with measured SPT 'N' values of greater than 100 blows per 0.3 m of penetration.

4.2.4 Silty Sand to Sand, Some Silt

A silty sand to sand layer was encountered below the silty sand till in the borehole on the east side of the highway. The surface of this layer was found to be at about Elevation 246 m. The silty sand to sand was not fully penetrated by the boring, which was terminated at Elevation 241.7 m; it is at least 4.3 m thick. This material may represent an interlayer or lens, of limited lateral extent, within the till deposit, as it was not encountered within the investigated depth on the west side of the highway.

The layer ranges in composition from silty sand containing trace gravel, to sand containing some silt and trace gravel; in one sample, thin silty clay layers were noted within the cohesionless soil. A grain size distribution test result is shown on Figure 2. The samples were wet, with measured natural water contents ranging from 17 to 19 per cent. The measured SPT 'N' values ranged from 69 to greater than 100 blows per 0.3 m of penetration, indicating that this material has a very dense relative density.

4.3 Groundwater Conditions

Borehole B13-1 on the west side of the highway was dry on completion of the drilling operations. In Borehole B13-2 on the east side of the highway, in which wet silty sand was encountered below Elevation 246 m, the water level on completion of drilling was at 11 m depth, or about Elevation 244 m.

The groundwater level in the piezometer in Borehole B13-2, located east of Highway 400, was measured in March 2001 at about Elevation 246 m, about 9 m below the Sunnidale Road grade and 2.5 m below the Highway 400 grade. This is approximately coincident with the top of the silty sand to silt layer encountered in this boring.

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

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PART B

PRELIMINARY FOUNDATION DESIGN REPORT
SUNNIDALE ROAD UNDERPASS
STRUCTURE SITE 30-173
HIGHWAY 400 WIDENING FROM 1 KM SOUTH OF HIGHWAY 89 TO HIGHWAY 11
G.W.P. 30-95-00, AGREEMENT NO. 3005-A-000074

5.0 ENGINEERING RECOMMENDATIONS

5.1 General

This section of the report provides preliminary foundation design recommendations for the widening and / or replacement of the existing Sunnidale 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 during the subsurface investigation at this site. The interpretation and recommendations provided are intended for planning purposes only, to provide the information necessary at this stage of the study. As such, where comments are made on construction they are provided only in order to highlight those aspects which could affect the planning of the project. Further foundation investigation will be required at this structure site as part of the detailed design stage of the project.

It is understood that Highway 400 will be widened from its existing six-lane configuration to an interim configuration of eight lanes, and an ultimate configuration of ten lanes, and that an alternative for a twelve-lane express / collector system is under consideration between Molson Park Drive and Duckworth Street in Barrie. Throughout the project length, it is expected that the existing highway platform will be widened by between 13 m and 30 m. Replacement of the existing Sunnidale Road underpass structure will therefore be necessary.

Based on the available general layout drawing for the existing single-span structure, the abutments and associated wing walls and retaining walls are supported on spread footings which are founded at about Elevation 247.2 m. Highway 400 has been constructed in cut, with its grade at about Elevation 248.5 m to 249 m, rising toward the north. The Sunnidale Road grade rises westward, from about Elevation 255 m to 257 m over Highway 400.

5.2 Bridge Foundation Options

The soils below the embankment approach fill (encountered behind the abutments in Boreholes B13-1 and B13-2) consist of a thin layer of silty sand (probably fill), overlying silty sand till to clayey silt till. On the east side of the highway, the silty sand till is underlain by a layer of very dense silty sand to sand. It is noted that the native soils above the Highway 400 cut were not investigated during this stage of drilling. The groundwater level associated with the lower, very dense silty sand to sand layer was found to be at about Elevation 246 m in March 2001, approximately 2.5 m to 3 m below the Highway 400 cut grade.

Based on these subsurface conditions, the new structure can be founded on spread footings placed in the dense silty sand till deposit. Consideration could also be given to the use of perched abutments, founded on spread footings placed on a compacted granular pad within the approach embankments. Alternatively, deep foundations, such as driven steel H-piles, could be adopted at this site. The soil conditions at this site are favourable for the use of an integral abutment replacement structure.

Preliminary recommendations for spread footings, including perched abutments, and for driven steel H-pile foundations are provided in the following sections. It should be noted that the boreholes put down during this preliminary phase of field work are located within the excavation area for the existing bridge and encountered fill materials extending to below the Highway 400 cut level. If perched footings or perched pile caps for deep foundations are considered viable options, the native subsoil conditions between the Sunnidale Road and Highway 400 grades will have to be investigated during the detailed design stage.

5.3 Spread Footings

For preliminary design of the abutment and pier foundations, spread footings may be placed on the silty sand till at or below Elevation 248 m on the west side of the highway, and at or below Elevation 247 m on the east side of the highway; for any pier footings, a design founding level of Elevation 247 m should be assumed at this preliminary design stage. A minimum soil cover of 1.5 m should be provided at all footing locations. Any associated wing wall or retaining wall footings may be stepped upward within the till away from the abutments, such that a minimum soil cover of 1.5 m is maintained above the underside of the footings.

Alternatively, consideration could be given to the use of abutment footings perched within the approach embankments.

5.3.1 Axial Geotechnical Resistance

Spread footings placed on a properly prepared base within the silty sand till deposit at the preliminary design elevations given above may be designed for a factored geotechnical resistance at Ultimate Limit States (ULS) of 800 kPa, assuming a 3 m wide footing. The settlement of footings founded on the silty sand till soils will be dependent on the footing size and configuration, and on the applied loads. The majority of the settlement should occur during construction. For preliminary design purposes, the geotechnical resistance at Serviceability Limit States (SLS) may be taken as 500 kPa provided proper groundwater control is exercised during construction to prevent reduction of the existing in situ density of the foundation subsoil. The geotechnical resistances at ULS and SLS will have to be reviewed during detailed design, once the footing size, configuration and loadings are known.

For spread footings placed within the approach embankments on a compacted Granular 'A' core, a factored geotechnical resistance at ULS of 900 kPa may be assumed for preliminary design. The geotechnical resistance at SLS will depend on the thickness of Granular 'A' and the consistency and thickness of the underlying soils; a value of 350 kPa may be assumed for preliminary design. These resistances will have to be confirmed during detailed design, once the composition and consistency of the upper soils at the site are confirmed.

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

5.3.2 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footing and the subsoils should be calculated in accordance with Section 6-8.4.3 of the OHBDC. The angle of friction between the concrete and the undisturbed, very dense silty sand till founding soils should be taken as 24 degrees; the corresponding coefficient of friction, tan δ, would then be 0.45. Where "perched" abutment footings are adopted, the angle of friction between the concrete footings and the compacted Granular 'A' pad should be taken as 30 degrees; the corresponding coefficient of friction would be 0.58.

5.3.3 Frost Protection

All footings should be provided with a minimum of 1.5 m of soil cover for frost protection.

5.4 Driven Steel H-Piles

Consideration could be given to supporting the abutments of the replacement structure on steel H-piles driven to found within the very dense silty sand deposit on the east side and the very dense/hard till deposits on the west side. The surface of these deposits was encountered in the borings at about Elevation 248.5 m and 246.5 m on the west and east sides of the highway, respectively (only slightly below the Highway 400 grade). Based on the nature of these deposits, a suitable driving resistance may be achieved at about Elevation 246 m on the west side, within the hard clayey silt till. On the east side, suitable driving resistance will likely not be achieved until the piles have advanced well into the sand deposit. For preliminary design, pile tip levels at Elevation 246 m and 242 m may be assumed for the west and east sides, respectively. If necessary, the pile caps could be perched within the approach embankments above Highway 400 grade in order to achieve the required length for abutment flexibility purposes.

5.4.1 Axial Geotechnical Resistance

For preliminary design of deep foundations on the west side of the highway, the factored axial resistance at ULS for steel HP 310 x 110 H-piles driven to found within the very dense silty sand till / hard clayey silt till may be taken as 1,600 kN. The axial resistance at SLS for 25 mm of settlement may be taken as 1,400 kN.

On the east side of the highway, where at least 4.5 m of water-bearing silty sand to sand was encountered below a thin till layer in Borehole B13-2, the factored axial resistance at ULS for driven steel HP 310 x 110 piles may be taken as 800 kN. The axial resistance at SLS for 25 mm of settlement may be taken as 800 kN. It is noted that this silty sand to sand layer may represent an interlayer, of limited lateral extent, within the till. The presence or absence of this interlayer at the east abutment location, its thickness and the nature and consistency of the underlying soils, will require investigation during the detailed design stage. This information should be used to confirm / refine the load carrying capacity.

To achieve the above design resistances, the piles should be driven to at least the above design tip elevations. Provision should be made to re-tap selected piles to confirm the set after adjacent piles have been driven, in accordance with MTO's current Special Provision.

5.4.2 Resistance to Lateral Loads

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

The resistance to lateral loading in front of the pile may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, k_h, is based on the following equation:

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

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

Based on borehole data obtained to date, the subsoils in front of the piles will consist of sand and gravel fills, sand till and silty sand deposits. For the compact to dense sand and gravel fill above Elevation 249 m, the range in value of n_h may be taken as 7 MPa/m to 20 MPa/m in the structural analysis. The range in value of n_h may be taken as 10 MPa/m to 25 MPa/m between Elevations 249 m and 246 m, and as 6 MPa/m to 12 MPa/m below Elevation 246 m. These values will have to be confirmed following the detailed design stage of the subsurface investigation.

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

Pile Spacing in Direction of Loading d = Pile Diameter	*Subgrade Reaction Reduction Factor R*
8d	1.00
6d	0.70
4d	0.40
3d	0.25

5.4.3 Frost Protection

All pile caps should be provided with 1.5 m of soil cover for frost protection.

5.5 Lateral Earth Pressures

The lateral pressures acting on the bridge abutments and associated retaining walls will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill, on the freedom of lateral movement of the structure, and on the drainage conditions behind the 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 abutment support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design.

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

5.6 Design of Permanent Cut Slopes

In the vicinity of the Sunnidale Road underpass, the Highway 400 cut is up to about 6 m in depth. Boreholes B13-1 and B13-2 were advanced through the approach fill behind the existing abutments, and so no borehole information is available regarding the native soils which will comprise the permanent cut slopes along the east and west sides of the widened Highway 400. Based on regional geological mapping, it is anticipated that the cut will be formed in silty sand to clayey silt till, similar to that encountered below the highway grade.

Based on the topographic information on the Engineering and Title Records plates and on site reconnaissance, the existing cut slopes are formed at a gradient of 1.7 to 2.5 horizontal to 1 vertical (1.7H:1V to 2.5H:1V). For preliminary design purposes, a gradient no steeper than 2H:1V may be assumed for the new permanent cut slopes. This design recommendation will have to be confirmed during the detailed design stage of the subsurface investigation program.

The groundwater level measured in Borehole B13-2 on the east side of the highway is at about Elevation 246 m, about 2.5 m below the Highway 400 grade. Based on this preliminary information, no major subsurface seepage concerns are anticipated for the permanent cut slopes. However, surficial erosion protection will be required to maintain stability over the long term.

5.7 Design and Construction Considerations

5.7.1 Groundwater Control

Based on the available groundwater level readings, foundation excavations for footings and/or pile caps should be above the groundwater level. However, provision should be made for appropriate groundwater and surface water control during wet periods to prevent loosening and loss of in-situ support within the foundation subsoil.

The silty sand till soils in which the footing excavations will be formed are susceptible to disturbance from ponded water and construction traffic. Provision should be made for a lean concrete mat placed at subgrade level, to protect the soils from such disturbance.

5.7.2 Excavation

Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act for Construction Activities. If spread footings or pile caps placed below the Highway 400 grade are selected for the replacement structure, the footing excavations will extend a minimum of 1.5 m below lowest surrounding grade, through compact silty sand and into compact to very dense silty sand till. The silty sand would be classified as Type 3 soil, while the silty sand till soils below Highway 400 grade would be classified as Type 2 soil. It is recommended that temporary open-cut slopes, where adopted, 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.

5.7.3 Settlement

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

5.7.4 Obstructions

Cobbles were observed within the approach fill but not within the native subsoils in the boreholes advanced at this site. However, it is noted that cobbles and boulders are inherent in glacial soils, and should therefore be expected during footing excavation and temporary shoring system installation, if such a system is required at the site. Where boulders are encountered within footing excavations, they should be removed and the sub-excavated areas should be backfilled with well-compacted Granular 'A' or lean concrete.

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LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Dynamic Cone Penetration Resistance; N_d:

The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH:	Sampler advanced by hydraulic pressure
PM:	Sampler advanced by manual pressure
WH:	Sampler advanced by static weight of hammer
WR:	Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

(b) Cohesive Soils

Consistency	c_u,s_u kPa	c_u,s_u psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

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

S:\FINALDATA\ABBREV\2000\LOFA-D00.DOC

LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$ bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$ dry density (dry unit weight)
$\rho_w(\gamma_w)$ density (unit weight) of water
$\rho_s(\gamma_s)$ density (unit weight) of solid particles
γ' unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R relative density (specific gravity)of solid particles ($D_R = \rho_s/\rho_w$) (formerly G_s)
e void ratio
n porosity
S degree of saturation

* Density symbol is ρ. Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)

(a) Index Properties (con't.)

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

(c) Hydraulic Properties

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

(d) Consolidation (one-dimensional)

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

(e) Shear Strength

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

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

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No B13-1

METRIC

PROJECT	001-1143F		
W.P.	30-95-00	LOCATION	N 4916980.9; E 288547.3
DIST	SW	HWY	400
BOREHOLE TYPE	108mm ID HOLLOW STEM AUGERS		
DATUM	Geodetic	DATE	Feb.6/2001
ORIGINATED BY	PKS		
COMPILED BY	LCC		
CHECKED BY	ASP		

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
256.8	GROUND SURFACE
0.0	Asphalt
0.2	Sand and Gravel, some cobbles (Fill) Compact to dense Brown Moist
249.5 / 7.3	Silty Sand, trace gravel Compact Brown Moist
248.4 / 8.4	Silty Sand, trace clay, trace gravel (Till) Very dense Brown Moist
246.3 / 10.5	Clayey Silt, trace to some sand and gravel (Till) Hard Brown Moist
244.4 / 12.4	END OF BOREHOLE Notes: 1. Borehole dry on completion of drilling. 2. Borehole backfilled with bentonite and surface cold patch on sand bedding.

SAMPLES

NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
1	SS	32				
2	SS	21				
3	SS	9				
4	SS	19				
5	SS	104				
6	SS	100/.15	4	57	30	9
7	SS	101				
8	SS	102				
9	SS	100/.18				

Column headings: GROUND WATER CONDITIONS; ELEVATION SCALE (256, 255, 254, 253, 252, 251, 250, 249, 248, 247, 246, 245); 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

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

RECORD OF BOREHOLE No B13-2

METRIC

PROJECT 001-1143F

W.P. 30-95-00 LOCATION N 4916967.4; E 288585.1 ORIGINATED BY PKS

DIST SW HWY 400 BOREHOLE TYPE 108mm ID HOLLOW STEM AUGERS COMPILED BY LCC

DATUM Geodetic DATE Feb.6-7/2001 CHECKED BY ASP

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
255.1	GROUND SURFACE					
0.0	Asphalt					
0.2	Sand and Gravel (Fill) Compact to very dense Brown Moist	1	SS	62		
		2	SS	11		
		3	SS	15		
		4	SS	25		
248.2 / 6.9	Silty Sand, trace clay, some gravel, trace wood and organics Dense Brown Moist	5	SS	31	~10	
247.5 / 7.6	Silty Sand, trace clay, trace to some gravel (Till) Compact to very dense Brown Moist	6	SS	21	~8	
		7	SS	84		
246.0 / 9.1	Silty Sand to Sand, some silt, trace gravel Very dense Wet Brown	8	SS	69	~17	0 85 15 0
		9	SS	129		
		10	SS	100/.15	~19	
		11	SS	108		
		12	SS	96	~19	
241.7 / 13.4	Thin silty clay layers present in Sample 13.	13	SS	110		
	END OF BOREHOLE					

Notes:
1. Water level on completion of drilling at 11m depth (Elev.244.1m).
2. Water level in piezometer measured at 9m depth (Elev.246.1m) on March 15, 2001.

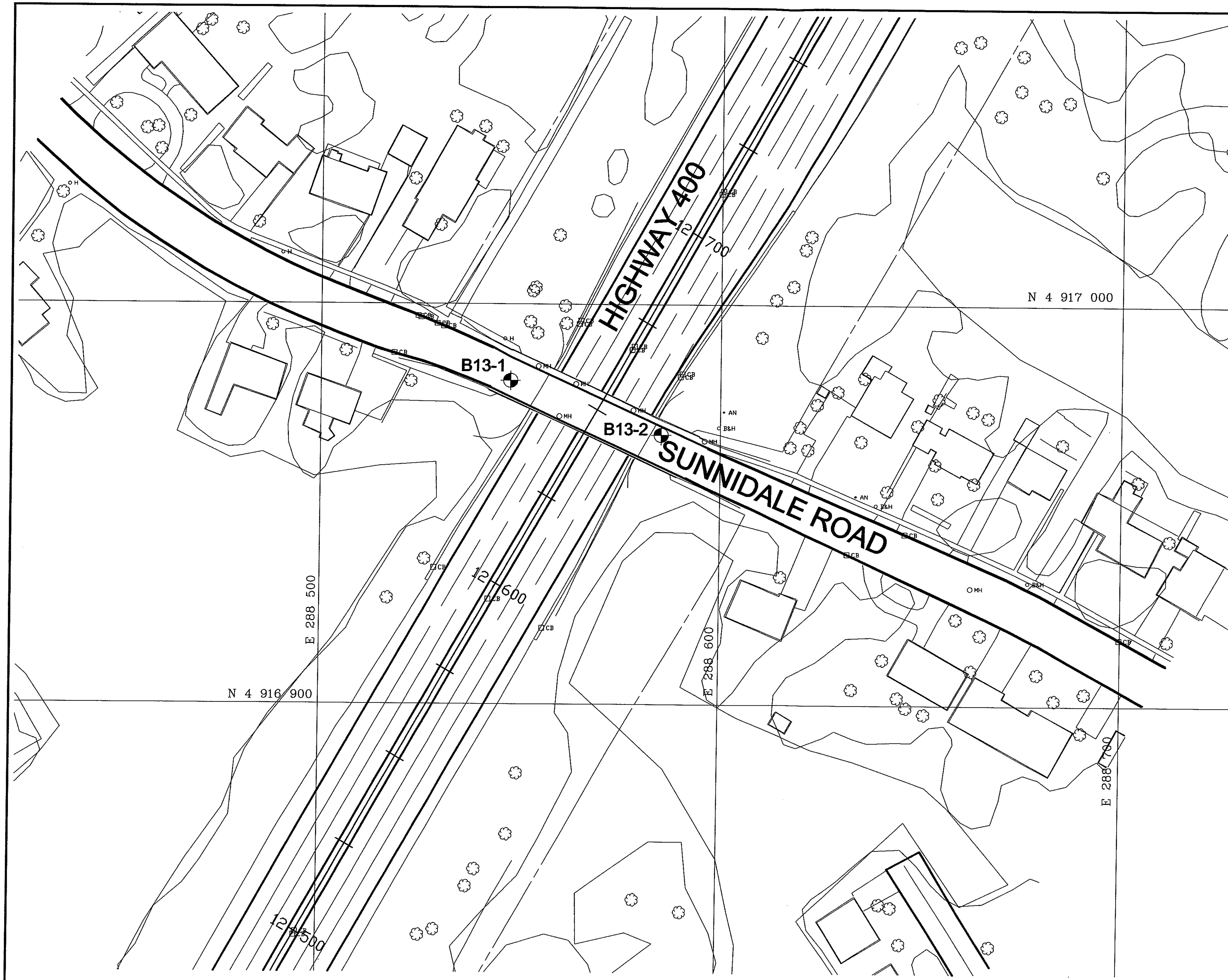
+3, ×3: Numbers refer to Sensitivity   ○3% STRAIN AT FAILURE

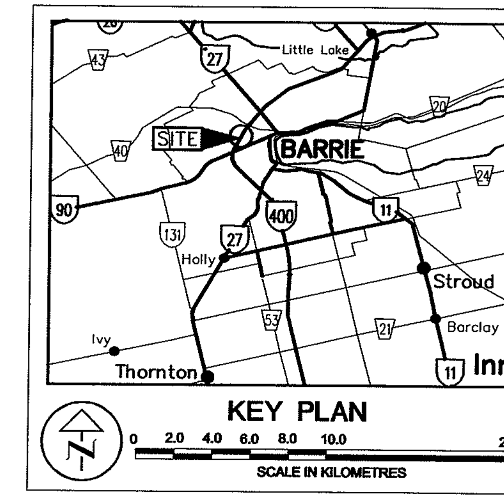
Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; UNIT WEIGHT γ kN/m³

DIST / CONT. No. / GWP No. 30-95-00	HWY 400
SUNNIDALE ROAD UNDERPASS HWY 400 BOREHOLE LOCATION PLAN	SHEET

Golder Associates

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA





LEGEND

- Borehole, previous investigation
- Borehole, present investigation

No.	ELEVATION	LOCATION	
		NORTHING	EASTING
B13-1	256.8	4,916,980.9	288,547.3
B13-2	255.1	4,916,967.4	288,585.1

REFERENCE

This drawing was created from digital file "50210.dwg" provided by URS Cole Sherman

0 10 20 30 40 50 100m
SCALE IN METRES

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

NO.	DATE	BY	REVISION

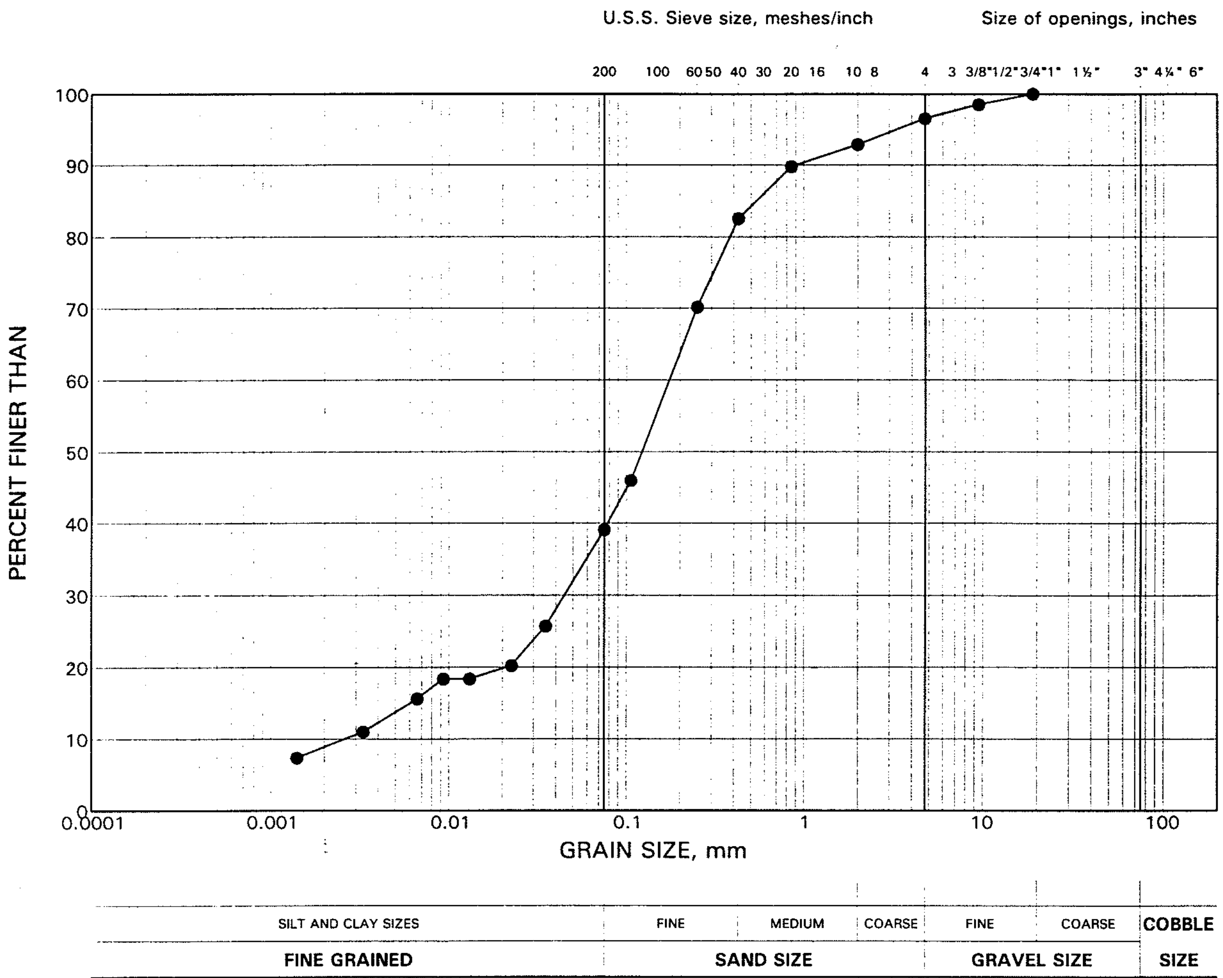
Geocres No.

HWY. No. 400		PROJECT NO.: 001-1143F	
SUBM'D. LCC	CHKD: ASP	DATE: JANUARY 2001	SITE 30-173
DRAWN: MHW	CHKD. LCC	APPD. ASP	DWG. 1

P1143F18.DWG

GRAIN SIZE DISTRIBUTION TEST RESULT
Silty Sand Till

FIGURE 1



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	B13-1	6	247.5

Project 001-1143F

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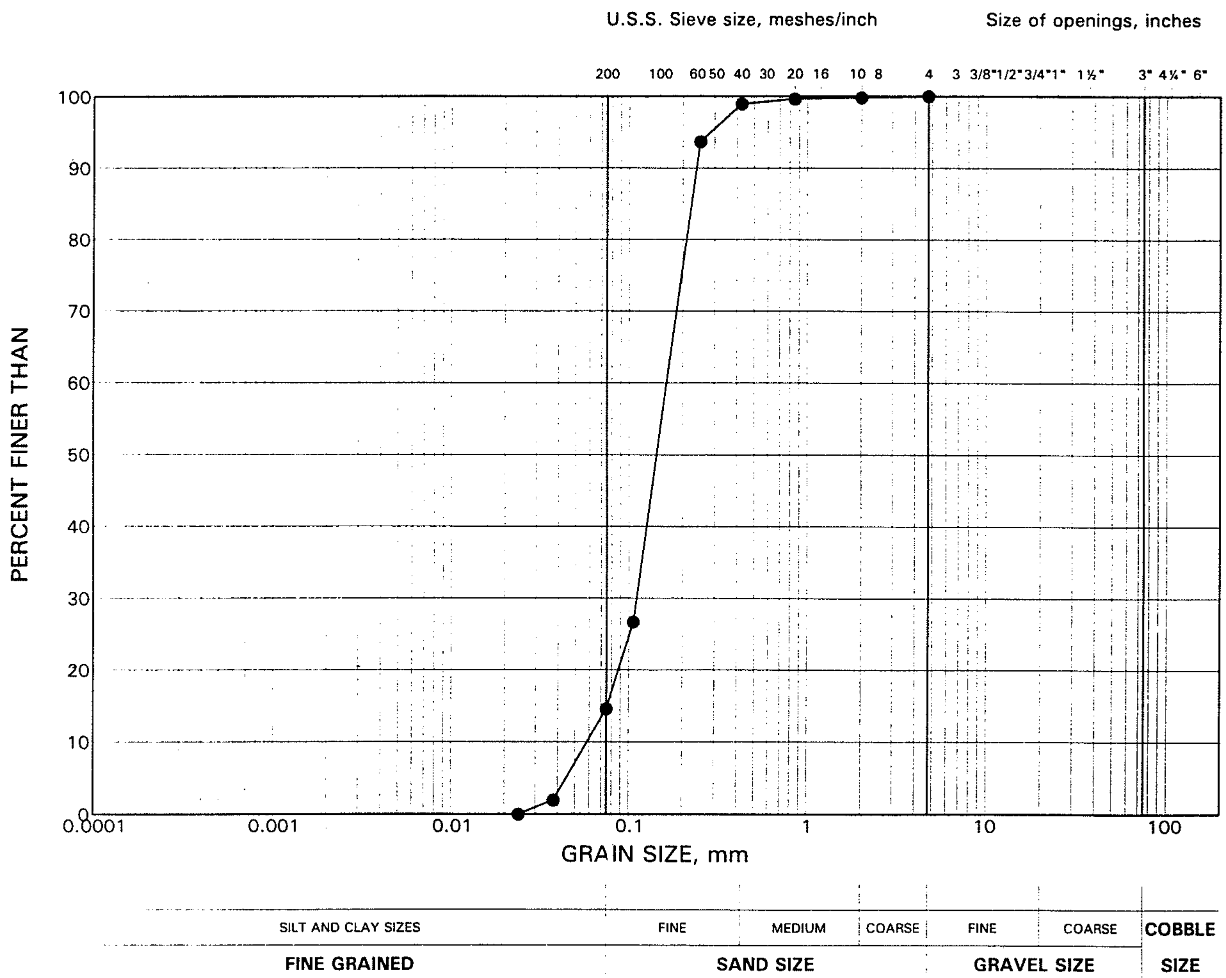
GRAIN SIZE DISTRIBUTION TEST RESULT
Sand, some silt

FIGURE 2



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	B13-2	8	245.6

Project 001-1143F

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