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**PRELIMINARY FOUNDATION
INVESTIGATION AND DESIGN REPORT
ESSA ROAD (SIMCOE ROAD 30) OVERPASS
STRUCTURE SITE 30-178
HIGHWAY 400 WIDENING FROM 1 KM SOUTH
OF HIGHWAY 89 TO HIGHWAY 11
G.W.P. 30-95-00. AGREEMENT NO. 3005-A-000074**

Submitted to:

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

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January 2002



001-1143F-8

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PART A

**PRELIMINARY FOUNDATION INVESTIGATION REPORT
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Appendix A Records of Boreholes and Test Results – 1970 Investigation

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 widening and / or replacement of the Essa Road (Simcoe Road 30, formerly Highway 27) overpass structure in Barrie, Ontario. Existing subsurface data for this site from an investigation conducted by the Department of Highways, Ontario in 1970 (*“Foundation Investigation Report for Proposed Extensions to the Overpass Structures at the Crossing of Highway 400 and Highway 27, City of Barrie”*, dated December 1970 – GEOCRE File No. 31D-169) were used to determine the subsurface conditions 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 Essa Road (Simcoe Road 30, formerly Highway 27) overpass is located about 3 km north of the Molson Park Drive interchange and 2.5 km south of Dunlop Street (Simcoe Road 90, formerly Highway 90) in Barrie, Ontario. The MTO has designated this overpass as Structure Site 30-178.

The Highway 400 embankment is up to 6.5 m high, with the highway grade at about Elevation 253.5 m to 253 m at the structure site, declining northward. The Essa Road grade is at about Elevation 246.5 m.

The existing overpass structures were widened in the early 1970s under Contract 71-150. According to the general layout drawing for this contract, 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. The spread footings are founded at about Elevation 244.8 m and 245.3 m at the north and south abutments, respectively.

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was previously carried out at this site by the Department of Highways, Ontario (DHO) in October 1970. At that time, two boreholes were advanced to determine the subsurface conditions for widening of the then-existing northbound and southbound lane overpass structures. Boreholes 1 and 2 were extended to 12 m and 20 m depth on the east and west sides of Highway 400, respectively.

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 groundwater conditions in the open boreholes were observed during and following the drilling operations. Laboratory index and classification tests, consisting of natural moisture content determinations, Atterberg Limits testing and grain size distribution analyses, were carried out on selected soil samples.

The borehole locations and elevations, referenced to the geodetic datum, were established by the DHO. Approximate northing and easting co-ordinates consistent with the MTM NAD83 survey system, currently in use on this project, have been determined by Golder Associates based on the borehole locations given in the 1970 report. The approximate borehole locations 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 Essa Road site is located within the Simcoe Lowlands physiographic region, just north of the Peterborough Drumlin Field boundary.

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

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

4.2 Site Stratigraphy

The detailed subsurface soil and groundwater conditions encountered in the 1970 boreholes, and the results of in-situ and laboratory testing, are given on the Record of Borehole sheets and figures contained in Appendix A. 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 1 and 2 were advanced on the east and west sides of Highway 400, from approximately Essa Road grade. The approximate locations and ground surface elevations for these boreholes are shown on the attached Drawing 1.

In summary, the subsoils at the site consist of surficial deposits of loose to very dense sand and very stiff clayey silt, overlying a deposit of compact to very dense silty sand till. A detailed description of the subsurface conditions encountered in the boreholes is provided in the following section.

4.2.1 Sand

A surficial sand deposit, 2.3 m to 8.5 m in thickness, was encountered in both boreholes. The sand extended to about Elevation 240.5 m in Borehole 1, and to about Elevation 238.5 m in Borehole 2, on the east and west sides of Highway 400, respectively. The sand contains trace to some silt and trace gravel; grain size distribution test results for samples of this surficial sand layer are presented on Figure 1 in Appendix A. A layer of clayey silt was encountered within the sand; this sub-layer is discussed in Section 4.2.2.

The sand layer was moist, with measured natural water contents of 5 to 8 per cent. The measured Standard Penetration Test (SPT) 'N' values ranged from 6 to 65 blows, but were typically between about 20 and 40 blows per 0.3 m of penetration. The sand therefore has a predominantly compact to dense relative density.

4.2.2 Clayey Silt

A 1.8 m thick layer of clayey silt was encountered within the sand deposit in Borehole 2, on the west side of Highway 400. The surface of the clayey silt layer was at about Elevation 245.5 m.

The clayey silt was moist, with measured natural water contents of 17 and 23 per cent. Atterberg Limits testing on both samples of this material measured plastic limits of 16 and 21 per cent, liquid limits of 25 and 26 per cent, and plasticity indices of 5 and 9 per cent; the results of the Atterberg Limits testing indicate that the clayey silt is inorganic and of low plasticity. Measured SPT 'N' values of about 27 blows per 0.3 m of penetration for both samples indicate that the clayey silt layer has a very stiff consistency.

4.2.3 Silty Sand Till

The 1970 boreholes encountered a silty sand till deposit below the surficial sand deposit. The surface of the silty sand till was encountered at Elevation 240.5 m and 238.5 m in the boreholes on the east and west sides of Highway 400, respectively. The till extended to the maximum depth investigated (approximately 20 m depth, corresponding to about Elevation 227.5 m).

The silty sand till contains trace clay and gravel. An envelope of grain size distribution test results for samples of the till is shown on Figure 2 in Appendix A.

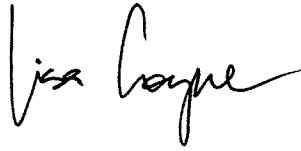
The measured natural moisture contents of two samples of the silty sand till were 13 and 17 per cent. The measured SPT 'N' values ranged from 14 to 79 blows per 0.3 m of penetration, but were typically between 14 and 50 blows per 0.3 m of penetration, indicating that the till has a predominantly compact to dense relative density.

4.3 Groundwater Conditions

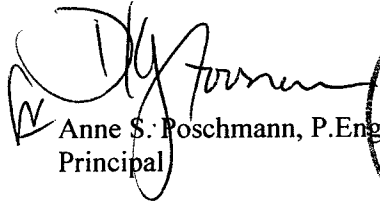
The groundwater conditions were observed in the open boreholes during the October 1970 drilling investigation. The water levels were measured at Elevation 245 m and 243 m on the east and west sides of Highway 400, respectively. These water levels are between 1.5 m and 3.5 m below the Essa Road grade.

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

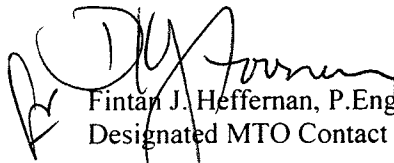
GOLDER ASSOCIATES LTD.



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LCC/ASP/FJH/clg/spc

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PART B

**PRELIMINARY FOUNDATION DESIGN REPORT
ESSA ROAD (SIMCOE ROAD 30) OVERPASS
STRUCTURE SITE 30-178
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 Essa Road (Simcoe Road 30, formerly Highway 27) overpass structure, associated with the widening of Highway 400. The recommendations are preliminary only and are based on interpretation of the factual data obtained from a limited number of boreholes advanced during a 1970 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. Widening and / or replacement of the existing Essa Road overpass structure will therefore be necessary.

Based on the available general layout drawing for the existing single-span overpass, the abutments and associated retaining and wing walls are supported on spread footings. The north and south abutment footings are founded at about Elevation 244.8 m and 245.3 m, respectively. The Essa Road grade is at about Elevation 246.5 m, and the Highway 400 grade is at about Elevation 253.5 m to 253 m at the structure site, declining northward.

5.2 Bridge Foundation Options

The soils below the Essa Road level consist of a surficial deposit of predominantly compact to dense sand, containing a very stiff clayey silt interlayer, overlying a compact to very dense silty sand till deposit. The water levels measured in the open boreholes in October 1970 were at Elevation 245 m and 243 m on the east and west sides of Highway 400, respectively, between 1.5 m and 3.5 m below the Essa Road grade, but they could be higher particularly during wet weather periods.

Based on these subsurface conditions and on the existing overpass foundation conditions, the widened structures could be founded on spread footings, placed on the generally compact to

dense surficial sand deposit to match the founding levels for the existing spread footings. If a new replacement structure is to be constructed, 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 the structure could be supported on deep foundations, such as driven steel H-piles.

Preliminary recommendations for spread footings, including perched abutments, and for deep foundations are provided in the following sections.

5.3 Spread Footings

For preliminary design of the abutment foundations, spread footings may be placed at or below Elevation 245.3 m, on the generally compact to dense sand deposit or the very stiff clayey silt interlayer that exists at Borehole 2. Where the existing structure is being widened, the new footings should match the existing founding levels. A minimum of 1.5 m of soil cover should be provided to the footings. Specific precautions must be taken during the excavation and construction period to avoid disturbance and loosening of granular subsoil due to inadequate groundwater control.

Alternatively, if the existing structure is to be replaced, 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 at or below the design elevation given above may be designed for a factored geotechnical resistance at Ultimate Limit States (ULS) of 500 kPa, assuming a 2.5 m wide footing.

Differential settlement will occur between the new footings and the existing structure. Due to the granular nature of the subsoils, the majority of this settlement is expected to occur during or shortly after the construction period; however, some longer-term consolidation settlement will occur in the clayey silt interlayer, if this is present at the locations of the new foundation elements. The settlement of the footings will be dependent on the footing size and configuration, on the applied loads, and on the distribution and thickness of any cohesive interlayers. For preliminary design purposes, the geotechnical resistance at Serviceability Limit States (SLS) may be taken as 300 kPa. The geotechnical resistances at ULS and SLS will have to be reviewed

following the detailed design stage of subsurface investigation, once the piezometric groundwater conditions at the site are confirmed and 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.

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 sand founding soils should be taken as 27 degrees; the corresponding coefficient of friction, $\tan \delta$, would then be 0.5. It should be assumed that the clayey silt interlayer will be present on the west side of the highway for preliminary design purposes; the angle of friction between the concrete and the clayey silt interlayer should be taken as 24 degrees, with a corresponding coefficient of friction of 0.45. Where "perched" abutment footings are adopted, the angle of friction between the concrete footings and the compacted Granular 'A' pad should be taken as 30 degrees; the corresponding coefficient of friction would be 0.58.

5.3.3 Frost Protection

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

5.4 Driven Steel H-Piles

Consideration could be given to supporting a replacement structure or structurally-separate widening on steel H-piles. The recommendations that follow assume that the piles would be

driven to about Elevation 229 m, terminating in the silty sand till deposit; however, the available subsurface information is not sufficient to confirm this assumption. Further borehole investigation will be required in the detailed design stage to determine the probable founding elevation. It is assumed that the pile caps would be perched above the Essa Road grade, to minimize the potential requirement for appropriate groundwater control at the site.

5.4.1 Axial Geotechnical Resistance

For preliminary design of deep foundations at this site, the factored axial resistance at ULS for steel HP 310 x 110 H-piles driven to the design elevation given above may be taken as 1,000 kN. The axial resistance at SLS for a single pile, for 25 mm of settlement, may be taken as 800 kN. 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:

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

It is expected that the steel H-piles would be driven through embankment fill and the compact to dense sand layer, into the compact to dense silty sand till. For well-compacted granular embankment fill and native sand above the water table (i.e. above Elevation 245 m), the range in value of n_h may be taken as 10 MPa/m to 20 MPa/m in the structural analysis. For the predominantly compact to dense soils below the water table (i.e. below Elevation 245 m), the range in value of n_h may be taken as 5 MPa/m to 10 MPa/m in the structural analysis. 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:

<i>Pile Spacing in Direction of Loading $d = \text{Pile Diameter}$</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/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 Embankment Design

Based on the topographic information on the Engineering and Title Records plates and on site reconnaissance, the existing Highway 400 embankment side slopes are formed at a gradient of about 2 horizontal to 1 vertical (2H:1V). For the embankment widening, the new side slopes should be formed at a maximum gradient of 2H:1V. The embankment widening should be carried out using conventional fill placement and compaction practices, and benching of the existing embankment side slopes should be carried out to key in the new fill.

5.7 Design and Construction Considerations

5.7.1 Groundwater Control

The water levels observed in the open boreholes in the October 1970 investigation were at Elevation 245 m to 243 m, about 1.5 m to 3.5 m below the Essa Road grade and near the design founding elevation for spread footings. Provision should, however, be made for higher groundwater levels during the construction period. Groundwater control will be required to prevent disturbance to the subgrade soils for spread footings founded on the native soils. To this end, it is recommended that the groundwater level be lowered prior to excavation and maintained 0.5 m below the founding level for the footings during the construction period.

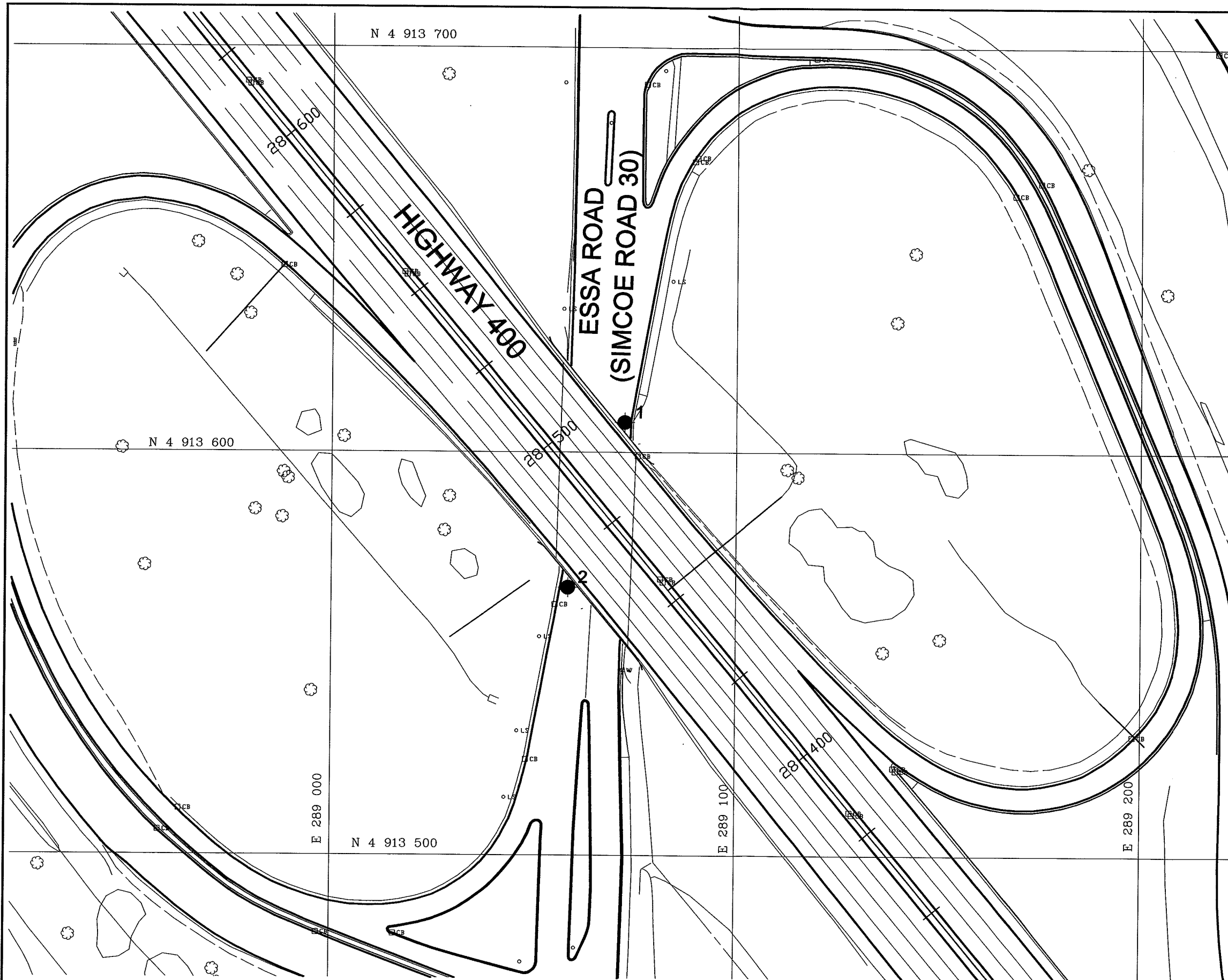
The stabilized groundwater conditions at the Essa Road site will have to be determined during the detailed design stage; further assessment of the groundwater control requirements (such as well points or properly filtered sumps) will be necessary at that time.

5.7.2 Excavation

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

5.7.3 Settlement

Deformation of the ground due to foundation loading will result in settlement of the bridge abutments and superstructure. This settlement will be differential with respect to the existing overpass structure, and also could vary along the new structure depending on the variability and consistency/relative density of the founding soils. The potential for differential settlement should be reassessed during the detailed design stage once the proposed bridge configuration is established.



DIST HWY 400
CONT. No.
GWP No. 30-95-00

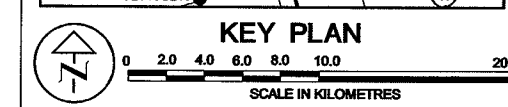
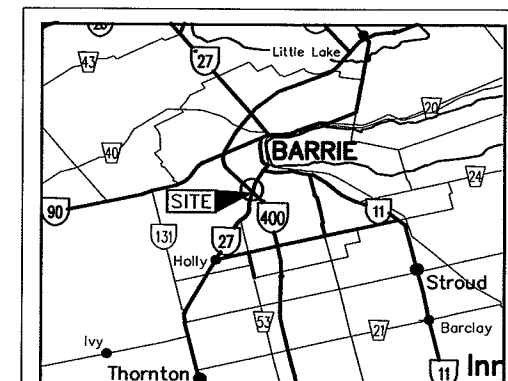


ESSA ROAD OVERPASS
HWY 400
BOREHOLE LOCATION PLAN

SHEET



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



LEGEND

- Borehole, previous investigation
- Borehole, present investigation

No.	ELEVATION	LOCATION	
		NORTHING	EASTING
1	245.8	4,913,607	289,072
2	247.1	4,913,566	289,058

REFERENCE

This drawing was created from digital file "50207.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.: 001-1143F	
SUBM'D. LCC	CHKD: ASP	DATE: JANUARY 2001	SITE 30-178
DRAWN: MHW	CHKD. LCC	APPD. ASP	DWG. 1

APPENDIX A

RECORDS OF BOREHOLES AND TEST RESULTS – 1970 INVESTIGATION

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 1

FOUNDATION SECTION

JOB 70-18091
W.P. 105-40-09
DATUM GoodvilleLOCATION Sta. 705 + 73.69' E. of Rwy. 100
BORING DATE Oct. 18, 1970
BOREHOLE TYPE Cent. flight augerORIGINATED BY TE
COMPILED BY TE
CHECKED BY TE

SOIL PROFILE		SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT		LIQUID LIMIT — PLASTIC LIMIT — WATER CONTENT —		BULK DENSITY P.C.F.	REMARKS	
ELEV. DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS/FOOT		50	100	60	80			100
245.8m (0.0m)	Ground surface											
0.0	Thin to medium sand, trace of silt. Loose to Very Dense Brown	1	SS	8								
		2	SS	16								
		3	SS	21								
		4	SS	42								
240.5m (5.3m)		5	SS	65								
7.5	Med. to coarse silt, sand and gravel, trace of clay. Glacial Till Compact to Very Dense Grey	6	SS	49								
		7	SS	19								
		8	SS	39								
		9	SS	27								
233.5m (12.3m)		10	SS	34								
10.5	End of Borehole											

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 2

FOUNDATION SECTION

JOB 70-11091

LOCATION Sta. 706 + 03 69' 12. Rev. 100

ORIGINATED BY TE

W.P. 100-80-09

BORING DATE Oct. 25/70

COMPILED BY TE

ENTRANCE On Water

BOREHOLE TYPE Cone, Flight Auger

CHECKED BY TE

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT				LIQUID LIMIT — % PLASTIC LIMIT — % WATER CONTENT %			BULK DENSITY PCF	REMARKS
DEPTH	DESCRIPTION	STRAT. NO.	NUMBER	TYPE	BLOWS/FOOT		20	40	60	80	100	10	20	30	
0.0	247.2m (0.0m)														
0.0	fine to medium sand with silt.		1	SS	42										2-18 (20)
0.5	clayey silt with trace of sand and gravel.		2	SS	27										
1.0	Very Stiff		3	SS	27										
1.0	240.5m (3.7m)		4	SS	42										242.9m
1.5			5	SS	27										2-18 (20)
2.0			6	SS	20										
2.5			7	SS	20										
3.0	238.7m (8.5m)		8	SS	16										2-18 (20)
3.5			9	SS	51										
4.0	Bot. mix. of silt, sand and gravel, trace of clay.		10	SS	14										
4.5	Compact to Very Dense Grey														
5.0															
5.5															
6.0	227.4m (19.8m)														
6.5															
7.0															
7.5															
8.0															
8.5															
9.0															
9.5															
10.0	End of Borehole														

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT			SAND			GRAVEL		
DEPARTMENT SIEVE DESIGNATION			Fine	Medium	Coarse	Fine	Coarse	
20	40	60	75	100	150	20	40	60
40	80	100	200	250	280	40	80	100
80	100	150	200	250	280	80	100	150
100	150	200	250	280	300	100	150	200
150	200	250	280	300	325	150	200	250
200	250	280	300	325	350	200	250	300
250	280	300	325	350	375	250	300	350
300	325	350	375	400	425	300	350	400
350	375	400	425	450	475	350	400	450
400	425	450	475	500	525	400	450	500
450	475	500	525	550	575	450	500	550
500	525	550	575	600	625	500	550	600
550	575	600	625	650	675	550	600	650
600	625	650	675	700	725	600	650	700
650	675	700	725	750	775	650	700	750
700	725	750	775	800	825	700	750	800
750	775	800	825	850	875	750	800	850
800	825	850	875	900	925	800	850	900
850	875	900	925	950	975	850	900	950
900	925	950	975	1000	1025	900	950	1000
950	975	1000	1025	1050	1075	950	1000	1050
1000	1025	1050	1075	1100	1125	1000	1050	1100
1050	1075	1100	1125	1150	1175	1050	1100	1150
1100	1125	1150	1175	1200	1225	1100	1150	1200
1150	1175	1200	1225	1250	1275	1150	1200	1250
1200	1225	1250	1275	1300	1325	1200	1250	1300
1250	1275	1300	1325	1350	1375	1250	1300	1350
1300	1325	1350	1375	1400	1425	1300	1350	1400
1350	1375	1400	1425	1450	1475	1350	1400	1450
1400	1425	1450	1475	1500	1525	1400	1450	1500
1450	1475	1500	1525	1550	1575	1450	1500	1550
1500	1525	1550	1575	1600	1625	1500	1550	1600
1550	1575	1600	1625	1650	1675	1550	1600	1650
1600	1625	1650	1675	1700	1725	1600	1650	1700
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1700	1725	1750	1775	1800	1825	1700	1750	1800
1750	1775	1800	1825	1850	1875	1750	1800	1850
1800	1825	1850	1875	1900	1925	1800	1850	1900
1850	1875	1900	1925	1950	1975	1850	1900	1950
1900	1925	1950	1975	2000	2025	1900	1950	2000
1950	1975	2000	2025	2050	2075	1950	2000	2050
2000	2025	2050	2075	2100	2125	2000	2050	2100
2050	2075	2100	2125	2150	2175	2050	2100	2150
2100	2125	2150	2175	2200	2225	2100	2150	2200
2150	2175	2200	2225	2250	2275	2150	2200	2250
2200	2225	2250	2275	2300	2325	2200	2250	2300
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4150	4175	4200	4225	4250	4275	4150	4200	4250
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5050	5075	5100	5125	5150	5175	5050	5100	5150
5100	5125	5150	5175	5200	5225	5100	5150	5200
5150	5175	5200	5225	5250	5275	5150	5200	5250
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6150	6175	6200	6225	6250	6275	6150	6200	6250
6200	6225	6250	6275	6300	6325	6200	6250	6300
6250	6275	6300	6325	6350	6375	6250	6300	6350
6300	6325	6350	6375	6400	6425	6300	6350	6400
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6450	6475	6500	6525	6550	6575	6450	6500	6550
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6800	6825	6850	6875	6900	6925	6800	6850	6900
6850	6875	6900	6925	6950	6975	6850	6900	6950
6900	6925	6950	6975	7000	7025	6900	6950	7000
6950	6975	7000	7025	7050	7075	6950	70	

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT

SAND

GRAVEL

DEPARTMENT SEVE DESIGNATION

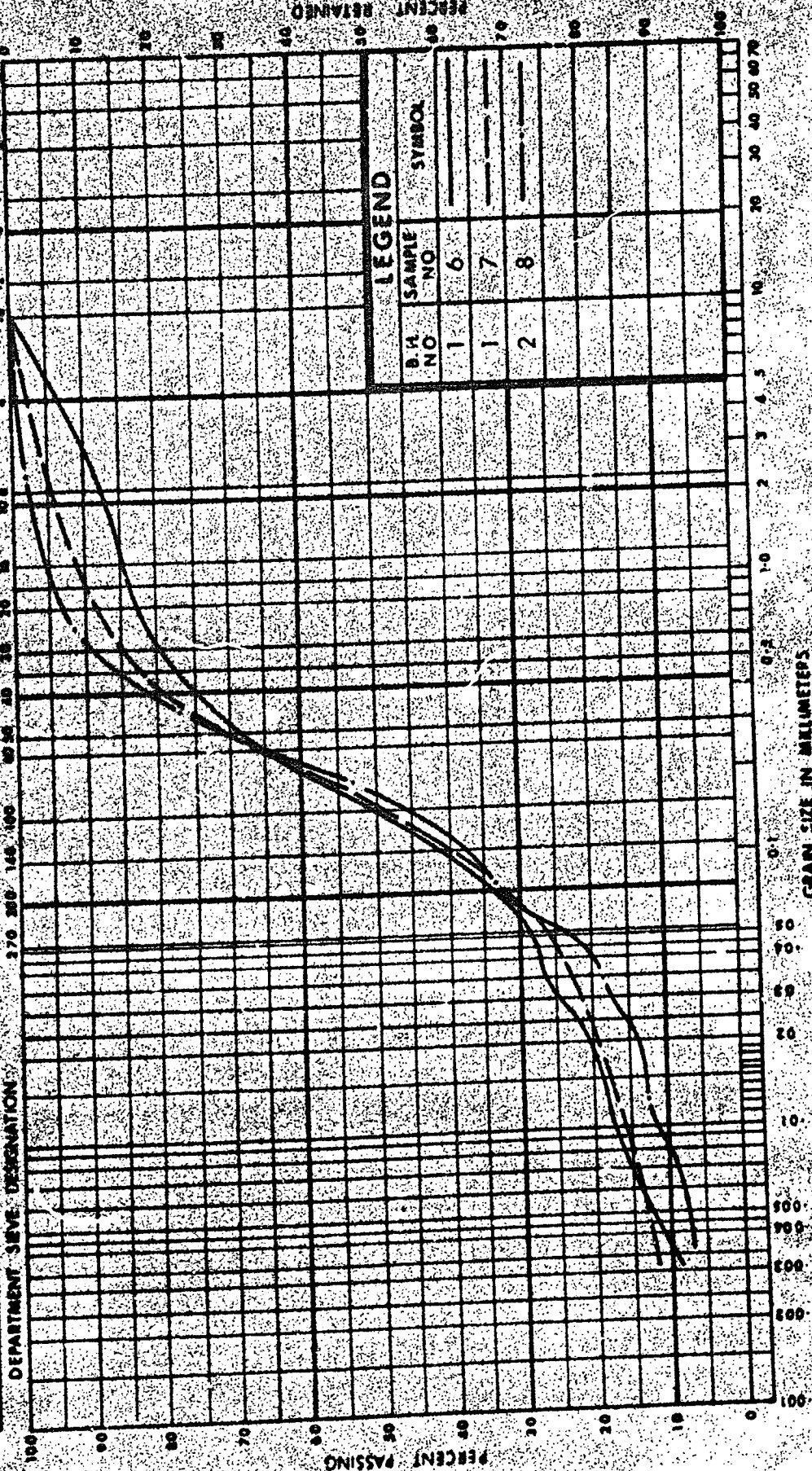
Fine

Medium

Coarse

Fine

Coarse



GRAIN SIZE DISTRIBUTION

GLACIAL TILL

DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION



W.P. No. 105-70-09

Job No. 70-11091

FIG. 2