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PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT WILLOW CREEK BRIDGE STRUCTURE SITE 30-139 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-17

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PRELIMINARY FOUNDATION INVESTIGATION REPORT
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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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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 existing Willow Creek bridge which is located at the northern end of Barrie. Existing subsurface data for this site from an investigation conducted for the Department of Highways, Ontario (DHO) in 1970 were used to determine the subsurface conditions for this preliminary design study. The DHO report was entitled "*Foundation Investigation Report for the Proposed Extension to the Structure at the Crossing of Hwy. 400 and Willow Creek – Township of Vespra – County of Simcoe (GEOCRES File No. 31D-166)*", dated January 1971.

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 three-span Willow Creek bridge is located about 500 m south of the existing overpass structure at the Highway 11 interchange. The MTO has designated the Willow Creek bridge as Structure Site 30-139.

At this structure site, the Highway 400 grade is at about Elevation 232 m and the original ground surface was at about Elevation 230 m. At the bridge structure, the Willow Creek channel is about 15 m wide, with its base at about Elevation 229 m.

The existing three-span bridge was constructed in the late 1950s, and widened in the 1970s under Contract 77-112. According to the General Arrangement drawing for the structure widening, provided by Morrison Hershfield (the structural designers for this study), the existing structure is founded on driven steel H-piles. The underside of the abutment and pier pile caps is at about Elevation 228.5 m. The H-pile tips are at about Elevation 190 m.

3.0 INVESTIGATION PROCEDURES

The Department of Highways, Ontario (DHO) carried out a subsurface investigation at this site in November 1970. At that time, four boreholes were advanced in the vicinity of the abutments and piers for the then-proposed widening of the bridge. Boreholes 1 and 2 were located near the south abutment and pier, respectively; Boreholes 3 and 4 were located near the north pier and abutment, respectively. The boreholes were advanced to depths ranging from about 14 m to 34 m below ground surface, which varied from about Elevation 229.5 m to 231 m.

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. Dynamic cone penetration tests were carried out in each borehole. In-situ vane shear testing was also carried out to determine the undrained shear strength of cohesive soils, where practicable. The groundwater conditions in the open borehole were observed during and following the drilling operations. Laboratory tests, consisting of water contents, Atterberg limits, bulk unit weights, grain size distributions, undrained shear strengths, and organic content determinations, 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 1971 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 surficial soils in the Peterborough Drumlin Field consist primarily of gravelly sand till or sand and gravel deposits. Drumlins (glacially-shaped hills) are more frequent in the southern portion of the section of the Peterborough Drumlin Field traversed by Highway 400. Deposits of silt, clay or peat may be found in the low-lying areas between drumlins.

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

4.2 Site Stratigraphy

The detailed subsurface soil and groundwater conditions encountered in the boreholes are given on the Record of Borehole sheets 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 located near the south abutment and pier, respectively; Boreholes 3 and 4 were located near the north pier and abutment, respectively. The boreholes were advanced to depths ranging from about 14 m to 34 m. The approximate locations and ground surface elevations for these borings are shown on the attached Drawing 1.

In summary, the soils below Highway 400 grade at this site consist of very loose to dense silty sand overlying soft to firm silty clay, which in turn overlies a lower deposit of very dense silty sand. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Upper Silty Sand

The 1970 boreholes encountered a stratum of silty sand extending from ground surface to about 19.5 m depth (Elevation 210 m). Trace organics were encountered in the upper 3 m of the deposit. Layers of clayey silt were found between about 3 m and 6 m depth in Boreholes 1 and 4, and below about 4 m depth in Borehole 3. Typical grain size distribution curves for the silty sand are plotted on Figure 1 in Appendix A. The test results from Borehole 3 show the grading for clayey silt layers that were encountered within the silty sand.

The measured Standard Penetration Test (SPT) 'N' values ranged from 2 to 50 blows per 0.3 m of penetration, but were typically between about 5 and 20 blows per 0.3 m of penetration. The SPT results indicate that the silty sand has a predominantly loose to compact relative density.

4.2.2 Silty Clay to Clay

Below the silty sand at about Elevation 210 m, the boreholes encountered a 10 m thick deposit of silty clay to clay. The results of grain size distribution testing carried out on samples of this deposit are plotted on Figure 2 in Appendix A.

Atterberg Limits testing carried out on two samples of this material measured plastic limits of 15 and 22 per cent, liquid limits of 30 and 54 per cent, and plasticity indices of 15 and 32 per cent. These test results indicate that the clay is inorganic and of low to high plasticity. The natural moisture contents measured on the same samples of silty clay were 33 and 53 per cent, typically near the liquid limit of the material. In-situ vane shear testing measured undrained shear strengths of about 20 kPa to 35 kPa, indicating that the silty clay has a soft to firm consistency.

4.2.3 Lower Silty Sand

A lower deposit of silty sand underlies the silty clay deposit at about Elevation 199 m to 200 m. The deposit was not fully penetrated by any of the boreholes, but was at least 4 m thick as encountered in Borehole 2. Measured SPT 'N' values ranged from 20 to 133 blows per 0.3 m of penetration, but were typically greater than 100 blows per 0.3 m of penetration. These SPT results indicate that the lower silty sand generally has a very dense relative density.

4.3 Groundwater Conditions

The water levels observed in the open boreholes following completion of the November 1970 drilling operations were measured to be between Elevation 228.9 m and 229.2 m, approximately 0.3 m to 1.5 m below the then-existing ground surface and about 3 m below the current Highway 400 grade. The river water level was observed at the time of the investigation to be at about Elevation 229.2 m. 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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DJE/LCC/ASP/FJH/lcc
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PART B

PRELIMINARY FOUNDATION DESIGN REPORT
WILLOW CREEK BRIDGE
STRUCTURE SITE 30-139
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 Willow Creek bridge, 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 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 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, necessitating widening or replacement of the existing Willow Creek bridge structure.

At the structure site, the Willow Creek channel is about 15 m wide, with its base at about Elevation 229 m; the sides of the channel are formed of driven steel sheet-piling. Based on the general arrangement and layout drawings, the existing three-span structure is founded on driven steel H-piles. The underside of the abutment and pier pile caps is at about Elevation 228.5 m, and the tips of the H-piles are at about Elevation 190 m.

5.2 Bridge Foundation Options

The subsoils encountered in the boreholes put down during the 1970 investigation consist of very loose to dense silty sand overlying soft to firm silty clay, which in turn overlies a lower deposit of very dense silty sand.

Shallow foundations are not recommended for widening or replacement of the existing bridge, due to the presence of the relatively loose, water-bearing upper silty sand deposit and the underlying soft to firm silty clay deposit. It is recommended that the foundation elements for the new or widened structures be supported on steel H-piles driven to found within the lower, very dense silty sand deposit which underlies the site. Preliminary recommendations for driven steel H-pile foundations are provided in the following section.

5.3 Driven Steel H-Piles

Based on the results of the 1970 boreholes, the top of the very dense lower silty sand deposit is at about Elevation 200 m to 199 m. For preliminary design, a pile tip level of Elevation 190 m (similar to the existing pile tip elevation) may be assumed for the widening or replacement. Assuming that the new pile caps are placed at the same elevations as the existing pile caps, the piles would be about 35 m to 40 m long. It is pointed out that additional borehole investigation will be required at the detailed design stage in order to confirm the soil conditions at and below the pile tip elevations.

5.3.1 Axial Geotechnical Resistance

For preliminary design, the factored axial resistance at Ultimate Limit States (ULS) for steel HP 310 x 110 H-piles driven to found within the very dense lower silty sand deposit at or below the tip elevation given above may be taken as 1,400 kN.

The additional load due to embankment construction will induce consolidation settlement of the silty clay soils under the widened portions of the embankments. At the abutments and at piers which are located within the embankments, this consolidation will induce a downward movement of the soils adjacent to the piles, and negative skin friction will develop along the portion of the pile shaft embedded within the silty clay. The negative skin friction load on a single pile may be taken as 100 kN. This value is unfactored; an appropriate load factor should be applied in the structural analysis. To minimize the amount of negative skin friction that may develop on a pile, consideration could be given to placing the new approach embankment fill as early as possible to allow as much settlement as possible to occur prior to the driving of the piles. The embankment could also be surcharged to increase the magnitude of settlement prior to pile installation.

The axial resistance at Serviceability Limit States (SLS) for 25 mm of settlement may be taken as 1,000 kN.

To achieve the above design resistances, the piles should be driven using a hammer with rated energy not exceeding 60 kJ. Provision should be made to re-tap selected piles to confirm the set after adjacent piles have been driven, in accordance with MTO's current Special Provision.

5.3.2 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:

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

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

For cohesive soils:

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

where B is the pile diameter (m) and
k_{s1} is the constant of horizontal subgrade reaction, as given below

The piles will be driven through embankment fill, loose to compact silty sand, and soft to firm silty clay. For the embankment fill and loose to compact silty sand above the groundwater level (above about Elevation 229 m), the range in value of n_h may be taken as 5 MPa/m to 15 MPa/m in the structural analysis. For the loose to compact silty sand soils below the groundwater table, between Elevations 229 m and 210 m, the range in value of n_h may be taken as 2 MPa/m to 10 MPa/m. Between Elevations 210 m and 200 m, the range in value of k_{s1} may be taken as 5 MPa/m to 20 MPa/m.

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

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

5.3.3 Frost Protection

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

5.4 Lateral Earth Pressures

The lateral pressures acting on the structure abutments and any 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 II**
Soil unit weight:	22 kN/m³	21 kN/m³
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.

5.5 Embankment Design

Based on the topographic information on the Engineering and Title Records plates and on site reconnaissance, the existing approach embankment side slopes are formed at a gradient of about 2 horizontal to 1 vertical (2H:1V). For the highway widening, the new side slopes should be formed at a maximum gradient of 2H:1V. The widening of the embankment 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. To ensure the stability of the abutment foreslopes, erosion and scour protection should be placed at the banks of the Willow Creek channel.

During detailed design, once the configuration of the structures and embankments is established, the recommended slope configuration will have to be reviewed to confirm the stability, and the embankment settlement will have to be assessed.

5.6 Design and Construction Considerations

5.6.1 Groundwater and Surface Water Control

The new pier pile caps will be located adjacent to Willow Creek, and the excavations for both the abutment and pier pile caps will extend into the water-bearing upper silty sand soils. A dewatering scheme will be necessary to facilitate pile cap excavation, pile driving and concrete placement in dry conditions. It is noted that excavation without groundwater control could result in loss of ground and consequent undermining of the existing pile caps.

The groundwater level should be lowered to at least 0.5 m below the pile cap underside prior to excavation and pile driving. In this regard, a well point or eductor system could be employed in the pile cap areas. Alternatively, interlocking steel sheeting may be driven around the foundation areas to a depth below founding level equal to the height of the groundwater above the founding level. It must be ensured that a full enclosure is achieved around the pile cap areas in order for the steel sheeting option to provide the required groundwater control.

5.6.2 Excavation

The pile cap excavations will extend a minimum of 1.5 m below lowest surrounding grade, and would be carried out through the existing embankment fill and the generally loose to compact, water-bearing upper silty sand deposit. 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. Any fill and cohesionless native soils would be classified as Type 3 soils, assuming that proper groundwater control is in place prior to excavation. Temporary open-cut slopes should therefore be maintained no steeper than 1 horizontal to 1 vertical (1H:1V). Where space restrictions dictate, pile cap excavations could also be carried out within a braced excavation.

5.6.3 Vibrations

If widening is adopted, given the close proximity of the new construction to the existing structure, it is recommended that vibration monitoring be carried out during pile installation. A Non-Standard Special Provision (NSSP) should be included in the Contract Documents to require the Contractor to monitor vibrations on the existing bridge and maintain the measured vibration levels below a peak particle velocity of 50 mm per second.

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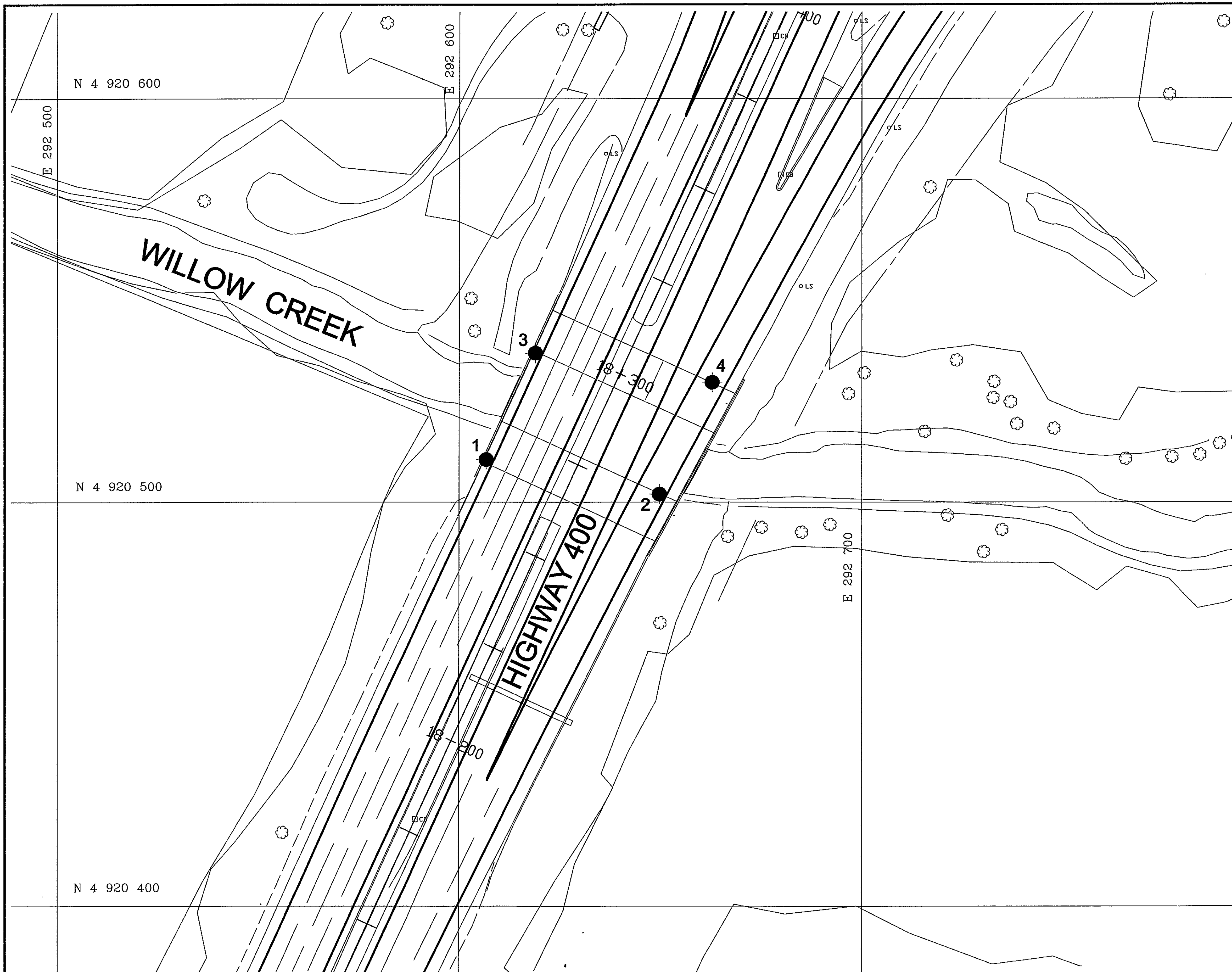
DIST HWY 400
CONT. No.
GWP No. 30-95-00

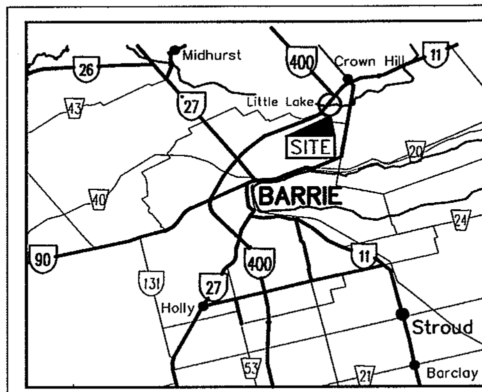
WILLOW CREEK BRIDGE
HWY 400
BOREHOLE LOCATION PLAN

SHEET

Golder Associates

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA





KEY PLAN

0 2.0 4.0 6.0 8.0 10.0 20.0
SCALE IN KILOMETRES

LEGEND

- Borehole, previous investigation
- Borehole, present investigation

No.	ELEVATION	LOCATION NORTHING	LOCATION EASTING
1	230.7	4,920,511	292,607
2	229.4	4,920,502	292,650
3	229.5	4,920,537	292,619
4	230.8	4,920,530	292,663

REFERENCE

This drawing was created from digital file "50214.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 30-139
DRAWN: MHW	CHKD. LCC	APPD. ASP	DWG. 1

P1143F22.DWG

APPENDIX A

RECORDS OF BOREHOLES
1970 SUBSURFACE INVESTIGATION

FORM OB-MT-126 (REV. 1969) OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 1

FOUNDATION SECTION

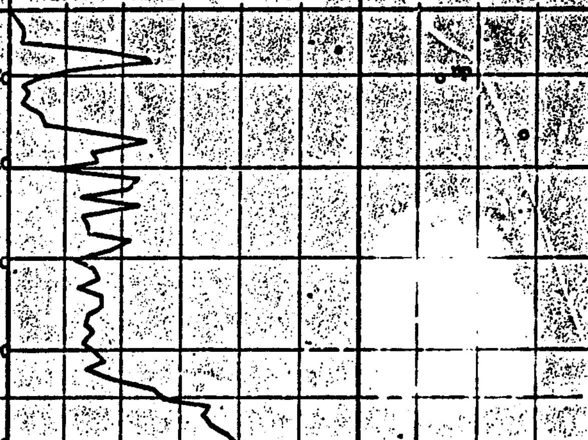
JOB 70-11097 LOCATION Hwy. 400 & Willow Creek Sta. 371 + 24 70' Lt. ORIGINATED BY VK
W.P. 105-70-08 BORING DATE Nov. 25, 1970 COMPILED BY VK
DATUM Geodetic BOREHOLE TYPE Washboring with BX & NX Casing CHECKED BY

	ELEV. / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	REMARKS GR. SA. SI. CL.
230.7m (0.0m)	757.0	Ground Level					
	0.0	with organic matter	1	SS	21		
			2	SS	7	750	0 70 19 5
		with layers of clayey silt	3	SS	4		
			4	SS	4		0 9 56 35
		Silty fine sand Loose to compact	5	SS	8	740	
			6	SS	16		
			7	SS	12	730	0 91 (9)
			8	SS	15	720	
218.1m (12.6m)	725.5 / 41.5	End of Borehole	9	SS	21		
216.4m (14.3m)	710.0 / 47.0	End of D.C.P.				710	

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100
SHEAR STRENGTH P.S.F. ○ UNCONFINED ✦ FIELD VANE ● QUICK TRIAXIAL × LAB. VANE 400 800 1200 1600 2000

LIQUID LIMIT w_L PLASTIC LIMIT w_P WATER CONTENT w — WATER CONTENT % 10 20 30

BULK DENSITY γ P.C.F.



20 / 15 ✦ 5 / 10 % STRAIN AT FAILURE

FORM OB-MT-126 (REV. 1969)

OFFICE REPORT ON SOIL EXPLORATION

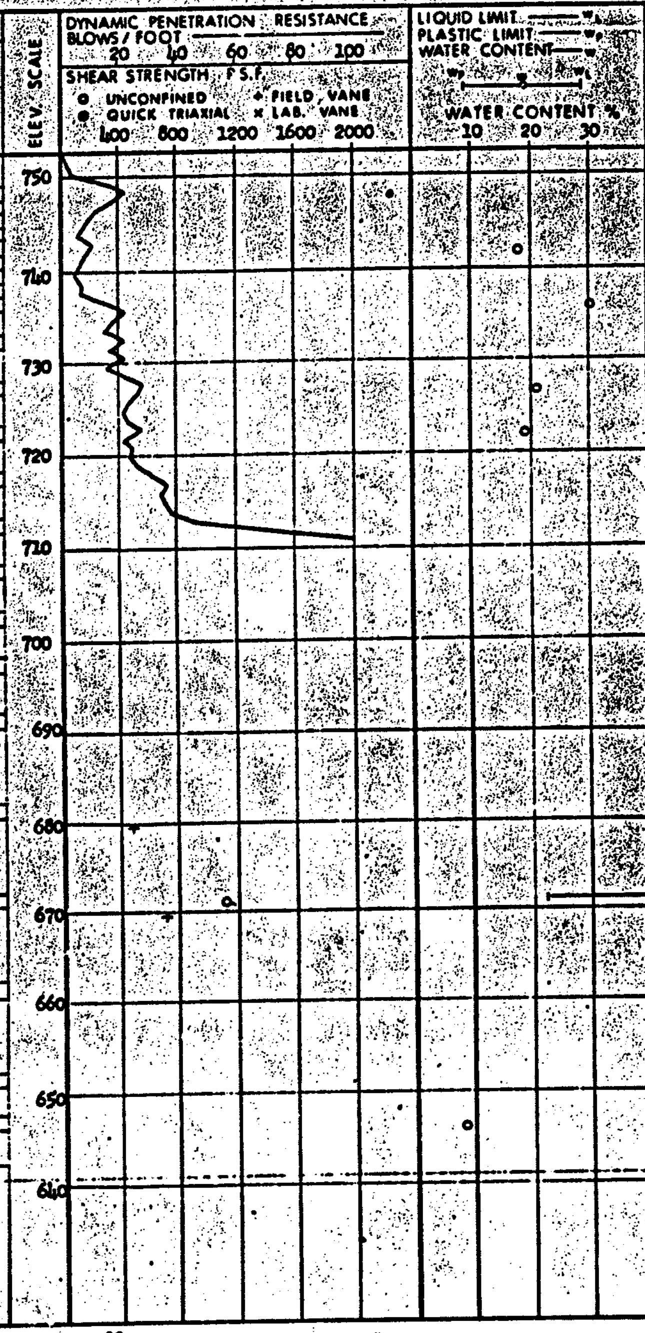
DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 2

FOUNDATION SECTION

JOB 70-11097 LOCATION Hwy. 400 & Willow Cr. Sta. 371 + 61 70' Rt. ORIGINATED BY VK
W.P. 105-70-08 BORING DATE Nov. 5, 1970 COMPILED BY VK
DATUM Geodetic BOREHOLE TYPE Washboring with BX & NX Casing, Pendrill CHECKED BY

ELEV. DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS/FOOT	BULK DENSITY P.C.F.	REMARKS GR SA SI CL
229.4m 752.8	Ground Level					
(0.0m) 0.0						
	with organic matter	1	SS	19		
		2	SS	14		
		3	SS	7		5 62 27 6
		4	SS	3		
		5	SS	2		
		6	SS	8		
		7	SS	6		
	Silty fine sand	8	SS	10		0 62 34 4
	Loose to Dense	9	SS	14		
		10	SS	36		
		11	SS	9		
		12	SS	7		
		13	SS	50		
		14	SS	38		
209.9m 688.8						
(19.5m) 64.0						
		15	TW	PH		
	Clayey silt to clay				104.5	0 5 32 63
		16	TW	PH		
	Soft to Firm					
		17	TW	PH		
199.6m 654.8						
(29.9m) 98.0		18	SS	20		
	Silty sand					
	Compact to Very Dense	19	SS	101		8 61 (31)
		20	SS	133		
195.5m 641.3						
(34.0m) 111.5	End of Borehole					



20
1.5 ± 3 % STRAIN AT FAILURE
10

FORM OB-MT-126 (REV 1969) OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 3

FOUNDATION SECTION

JOB 70-11097 LOCATION Hwy. 400 & Willow Cr. Sta. 372 + 24 70' Lt. ORIGINATED BY VK

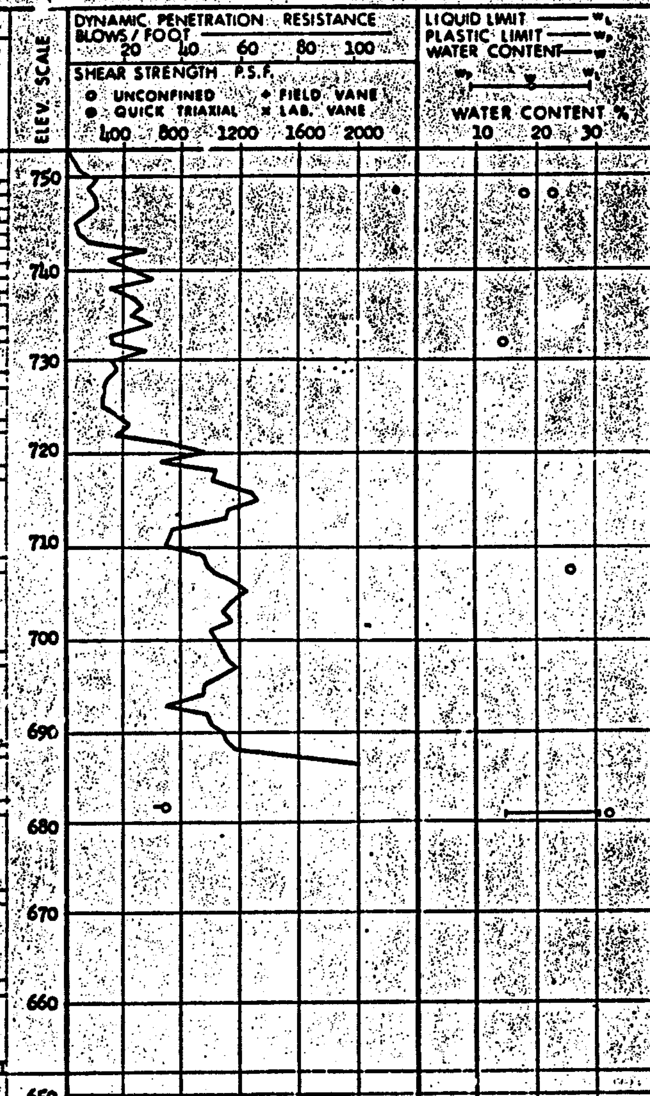
W.P. 105-70-08 BORING DATE Nov. 23, 1970 COMPILED BY VK

DATUM Geodetic BOREHOLE TYPE Washboring with NX & BX Casing CHECKED BY

	ELEV. DEPTH	DESCRIPTION
229.5m (0.0m)	753.0 / 0.0	Ground Level; with organic mater
		Silty sand, layers of clayey silt up to 10" thick below elev. 740. Loose to Compact
210.0m (19.5m)	689.0 / 64.0	Clayey silt. Firm
199.6m	655.0	
198.9m	652.5	Silty Sand, Very Dense
(30.6m)	100.5	End of Borehole

SAMPLES

NUMBER	TYPE	BLOWS/FOOT
1	SS	7
2	SS	3
3	SS	12
4	SS	18
5	SS	20
6	SS	5
7	SS	4
8	SS	6
9	SS	16
10	SS	6
11	SS	13
12	TW	PH
13	TW	PH
14	TW	PH
15	TW	PH
16	SS	111



BULK DENSITY γ P.C.F.	REMARKS GR SA SI CL
	▼752.0
	0 58 38 4
	0 18 62 20
	0 90 (10)
	0 5 71 24
118	0 1 49 50

$\underset{10}{15 \overset{20}{+} 5}$ % STRAIN AT FAILURE

FORM OB-MT-126 (REV 1969)

OFFICE REPORT ON SOIL EXPLORATION

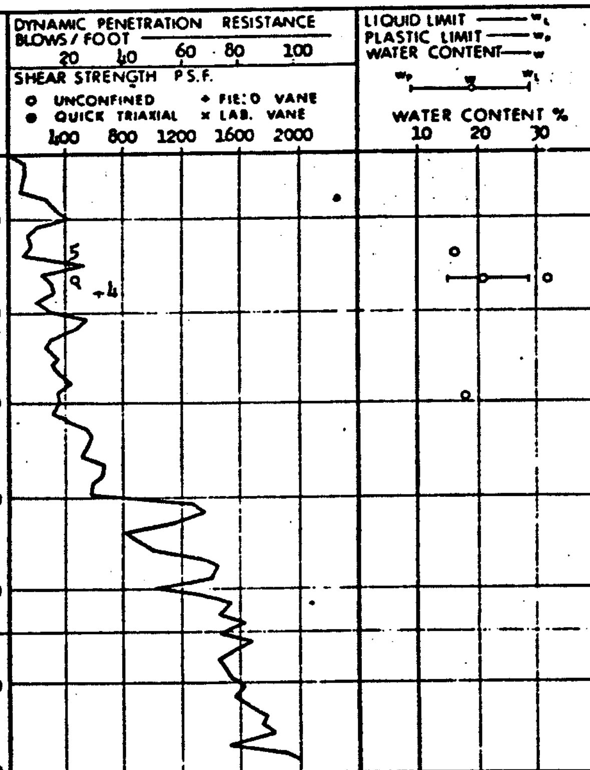
DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 4

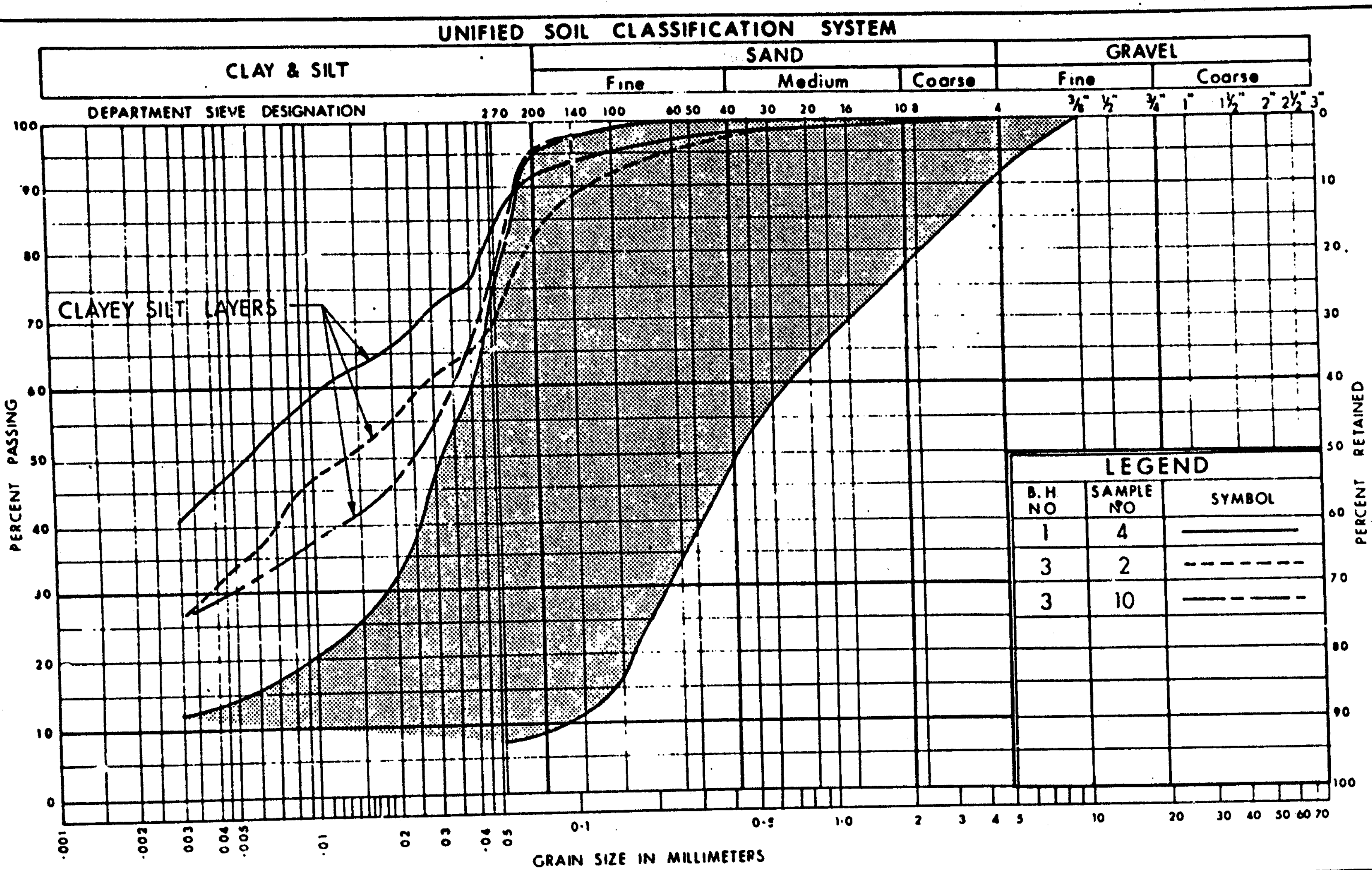
FOUNDATION SECTION

JOB 70-11097	LOCATION Hwy. 400 & Willow Cr. Sta. 372 + 60 70' Rt.	ORIGINATED BY VK
W.P. 105-70-08	BORING DATE Nov. 23, 1970	COMPILED BY VK
DATUM Geodetic	BOREHOLE TYPE Washboring with BX & NX Casing	CHECKED BY

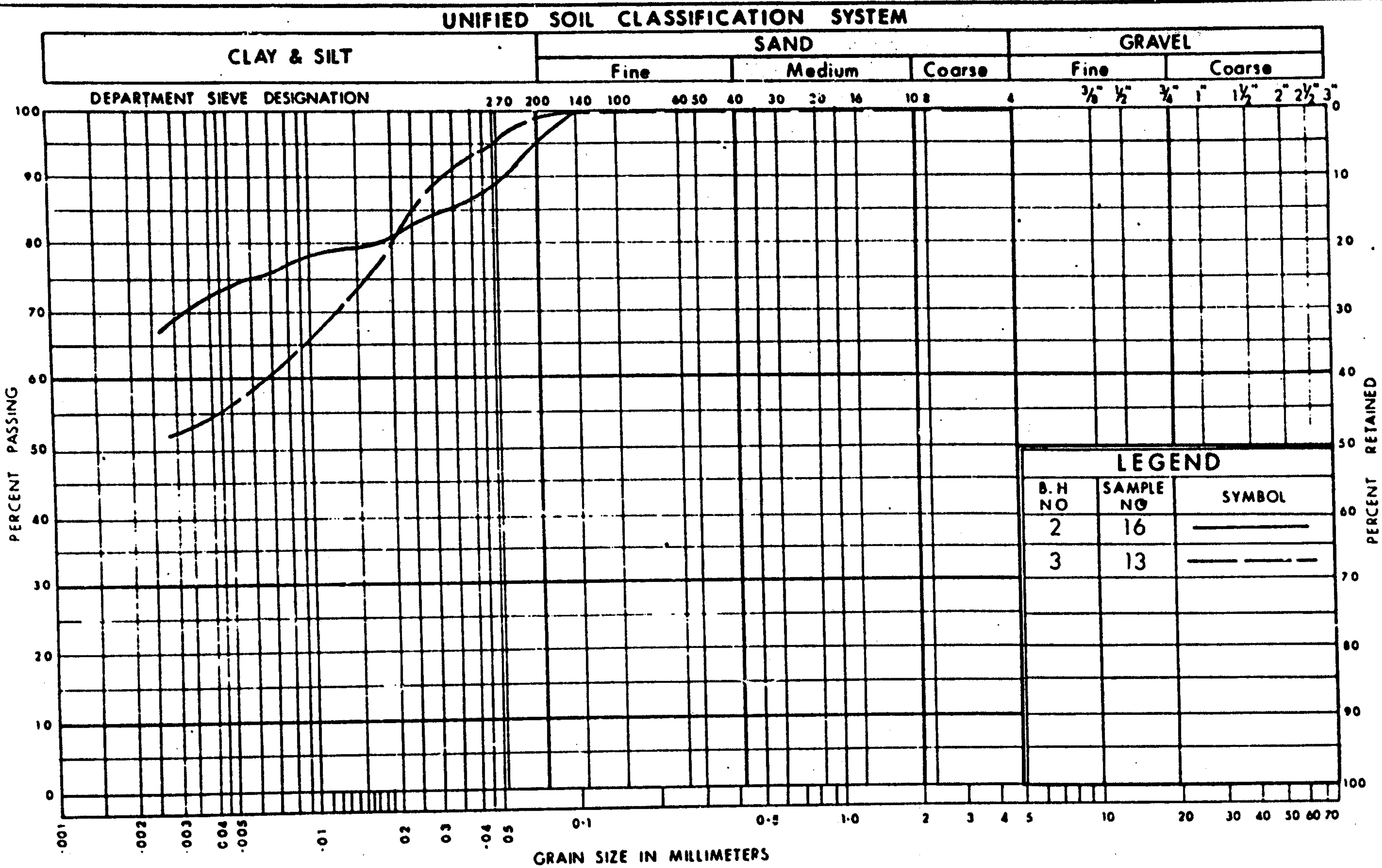
ELEV. DEPTH	SOIL PROFILE DESCRIPTION	STRAT. PLOT	SAMPLES NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
230.8m (0.0m) 757.2	Ground Level								
0.0	with organic matter		1	SS	10				▼ 752.0
			2	SS	27	750			
	layers of clayey silt		3	TW	PH				0 5 84 11
	soft to firm		4	TW	PH			118	0 68 29 3
			5	TW	PH	740			
			6	SS	18				
			7	SS	16	730			2 72 (26)
			8	SS	19				
	Silty fine sand					720			
	Loose to Compact		9	SS	29				
						710			
215.1m (15.7m) 705.7 51.5	End of Borehole		10	SS	7				
						700			
210.8m (20.0m) 691.7 65.5	End of D.C.P.					690			



20
15 ✛ 5 % STRAIN AT FAILURE
10



DEPARTMENT OF HIGHWAYS MATERIALS and TESTING DIVISION	W.P No. 105 - 70 - 08
GRAIN SIZE DISTRIBUTION SILTY SAND	JOB No. 70 - 11097
	FIG. 1



DEPARTMENT OF HIGHWAYS
MATERIALS and TESTING DIVISION

GRAIN SIZE DISTRIBUTION

CLAYEY SILT TO CLAY

W.P No. 105 - 70 - 08

JOB No. 70 - 11097

FIG. 2