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FOUNDATION INVESTIGATION REPORT
HWY 404/ELGIN MILLS ROAD INTERCHANGE
THE REGIONAL MUNICIPALITY OF YORK

Ref. No. G-20.0702A
September 2001

Prepared for:

The Regional Municipality of York
c/o R.V. Anderson Associates Limited
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Distribution

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ON

FOUNDATION INVESTIGATION

HWY 404/ELGIN MILLS ROAD INTERCHANGE

THE REGIONAL MUNICIPALITY OF YORK

1.0 INTRODUCTION

The foundation investigation described in this report was conducted at the request of R.V. Anderson Associates Limited (R.V.A.), on behalf of the Regional Municipality of York, at the site of the interchange of Hwy 404 and Elgin Mills Road in the Region of York. The objectives of the study were to establish the subsurface conditions at the locations of the various components of the project, in order to provide geotechnical design input and construction related comments for the proposed works. Specifically, the following were to be addressed:

- Soil stratigraphy, groundwater conditions;
- design parameters for the foundation of the bridge over Hwy 404, the culvert extensions and a retaining wall, including geotechnical resistances at ULS and SLS, total and differential deformations associated with the geotechnical resistance at SLS and lateral earth pressures;
- anticipated construction conditions.

2.0 PROJECT DESCRIPTION

The project involves the detailed design of the bridge structure at the Elgin Mills Road and Hwy 404 interchange, a new culvert structure located on Elgin Mills Road at Sta 4+450±, the extension of a culvert under Hwy 404 at Sta 20+700± and a retaining wall along a 100± m long section of the N-EW ramp, between approximate Sta 10+400 and 10+500.

The existing Hwy 404 underpass at Elgin Mills Road consists of a two (2) lane bridge structure, constructed in 2 spans and bearing on spread footings. This bridge will be replaced with a new structure having 5 lanes, a ramp lane and sidewalks on each side.

We understand that the new bridge structure over Hwy 404 will be constructed in two stages. First, a structure to carry the future westbound traffic on Elgin Mills Road located north of the existing structure will be constructed. This will be followed by the demolition of the existing structure and its replacement with a new bridge to carry the future eastbound traffic.

The new culvert under Elgin Mills Road at Sta 4+450± will be a rigid framed open bottom single cell structure to replace an existing CSP.

An existing culvert under Hwy 404 at approximate Sta 20+709 is a concrete box structure and will be extended below the N-EW and E-N ramps.

The retaining wall proposed to be constructed along the west side of the N-EW ramp will be designed as a mechanically stabilized earth structure.

The widening of the approach embankments for the Elgin Mills overpass will be carried out at both sides of the existing road.

3.0 METHOD OF INVESTIGATION

A total of fifteen (15) new boreholes were put down between December 13, 2000 and January 5, 2001, at the approximate locations shown on Enclosure 1 attached to this report, using both truck and track mounted power auger drill rigs The boreholes were drilled with solid stem continuous flight augers. Standard penetration tests (SPT) were performed in all boreholes.

In the field, the boreholes were laid out with reference to stakes placed at maximum 100 m intervals by a survey crew retained by R.V.A. (Hunt Surveys). Some of the boreholes, however, were drilled at the specific stake locations. Between stakes, the borehole locations were established by measuring from the nearest stake available.

Field engineers from our office supervised the drilling and logged the boreholes. In the boreholes, soil samples were taken at 0.75 m intervals of depth in the upper 6 m, and at 1.5 m intervals thereafter. The samples were visually identified in the field and transported to our laboratory, where they were re-examined by a senior engineer. Representative samples were selected for geotechnical laboratory analyses, which included natural moisture content determinations, particle size distributions and consistency limits. The results of the laboratory tests are summarized on the borehole log sheets (Enclosures 2 to 16) and the grading curves are shown plotted in Figures 1 to 8 of this report.

The elevations of the ground surface at the locations of the boreholes drilled beside the stakes were provided by the surveyor. The elevations of the ground surface at the locations of Boreholes 107, 109, 116 and 117 were estimated based on the ground elevations of the nearest stakes. The elevations of these boreholes are, therefore, approximate.

In addition to the new boreholes drilled for this project, we have also reviewed the existing borehole information available from the geotechnical investigation performed by Geo-Canada Ltd. for the reconstruction of Elgin Mills Road (Ref. No. G-98.1203) and the results of previous investigations carried out by others in 1962, 1970 and 1977, for the crossing of Elgin Mills Road and Hwy 404 (WP No. 160-74-28). A drawing showing the results of these latter boreholes (Enclosure 20) as well as logs of the previously drilled boreholes by Geo-Canada Ltd. (Boreholes 28 and 31, Enclosures 17 and 18) are also attached to this report.

4.0 SITE DESCRIPTION AND GEOLOGY

The study area is relatively flat. The Rouge River crosses under Elgin Mills Road at a distance of approximately 300 m west of Hwy 404. There is another creek crossing under

Hwy 404 at a distance of 400 ± m north of Elgin Mills Road. The existing Elgin Mills Road overpass consists of a 2-lane bridge structure.

Physiographically, the site is located in the Halton till plain. This till plain has a gently undulating, rolling surface and limited relief. The majority of the till plain is covered by glacial till deposits, which generally consist of dense, stony and bouldery sandy silt and silty sand. Surficial sands, silts and clays cover large, mostly discontinuous areas in the region. These were laid down by melt waters of the receding glaciers towards the end of the last ice age.

The depth to the surface of the Paleozoic shale bedrock of Georgian Bay Formation underlying the site is known to be in excess of 100 m.

5.0 SUBSURFACE CONDITIONS

In general, competent subsurface conditions were encountered across the site at relatively shallow depths. A variety of soil types were encountered in the boreholes, including fill and possible fill, clayey silt, sandy silt, silty clay, sand, silty sand, gravelly sand, sand and silt, sand and gravel and tills. The tills generally have a sandy and silty texture with variable amounts of clay and gravel size particles, but clayey silt tills were also encountered in places. The presence of cobbles and boulders should always be anticipated in the tills.

Groundwater was encountered in eleven (11) of the fifteen (15) boreholes at depths ranging from 0.8 to 5.5 m shortly after the boreholes were drilled. For the long term groundwater observation, seven (7) new piezometers were installed.

In the following paragraphs, the major soil deposits encountered in the boreholes are briefly described. For specific details of subsurface conditions, reference should be made to the individual borehole log sheets, which are attached to this report as Enclosures 2 to 18. Enclosure No. 19 shows inferred geological profiles through the proposed east and west abutments and the central pier locations.

5.1 Topsoil

Topsoil was encountered in all boreholes, except Boreholes 103, 104 and 113. The thickness of the topsoil, as measured at the borehole locations, ranged from 50 to 200 mm.

5.2 Fill and Possible Fill

Fill and possible fill were encountered in Boreholes 104 and 111 with thicknesses of approximately 0.6 m and 1.4 m respectively. The fill consists of sandy silt and silty sand, and contains trace of organic matter and rootlets. The measured natural moisture content of samples from the fill was in the range of 28 to 39%. A grading curve for the fill sample from Borehole 111 composed of 1% gravel, 78% sand and 21% of silt and clay is shown on Figure 8.

5.3 Native Fine Grained, Cohesive Soils

These deposits, which include silty clay, clayey silt and clayey silt till, were encountered in most of the boreholes, generally below the topsoil layer and with thicknesses up to 3.9 m. These are low to medium plasticity soils of generally soft to very stiff, but mostly firm to stiff consistency. The upper deposits, which extend to between 0.8 and 2.2 m below grade (El. between 223.8 and 233.5 m), generally contain traces of organic matter. Figure 7 shows a typical grading curve for a deposit of clayey silt till. The tested sample contains 1% gravel, 6% sand, 71% silt and 22% clay. Atterberg limit tests performed on representative samples gave liquid limits between 25 and 35%, plastic limits from 13 to 19% and plasticity indices of 12 to 16%. The natural moisture content ranged between 11 and 42%.

Due to their mode of formation, the presence of cobbles and boulders should always be anticipated in the glacial till deposits.

5.4 Native Fine Grained, Non-Cohesive Soils

These deposits, which include sand and silt, sand and silt till, sandy silt and sandy silt till, were encountered in most of the boreholes at variable depths. These are broadly graded deposits, likely of common origin but with minor variations in the relative proportions of sand and silt. Figures 1, 2 and 3 show the test results of several grain size analyses on these materials. Generally, they contain 0 to 13% gravel, 13 to 53% sand, 42 to 87% silt and clay.

A wide range of standard penetration blow counts (SPT 'N' values) were recorded in these materials, from 11 to over 100 blows/0.3 m. However, these materials are generally dense to very dense. The measured natural moisture contents ranged from 3 to 19%.

5.5 Native Coarse Grained, Granular Soils

Included in this soil group are the sand, gravelly sand, sand and gravel, silty sand and silty sand till found in a few boreholes. The results of grain size analyses on several representative samples (Figures 4 to 6) show that these soils are composed of 1 to 57% gravel, 37 to 61% sand, 6 to 41% silt and clay size particles. These deposits are generally dense to very dense (SPT 'N' values from 34 to greater than 100 blows/0.3 m). The measured natural moisture contents ranged from 8 to 16%.

As mentioned in Section 5.3 of this report, the presence of cobbles and boulders should always be anticipated in the glacial till deposits.

5.6 Groundwater Conditions

Four (4) boreholes were found to be dry during and shortly after drilling. Boreholes 106, 111, 114, 115 and 117 caved in on completion of drilling. Groundwater was encountered within eleven (11) boreholes, ranging in depth from 0.8 to 5.5 m shortly after the boreholes were drilled. These observations in the open uncased boreholes may not be representative of the true groundwater conditions.

Piezometers were installed in Boreholes 101, 104, 105, 111, 112, 113 and 116. The groundwater levels in these and two previously drilled boreholes (Boreholes 28 and 31) were measured on completion of the drilling and on January 16 and January 29, 2001. The measurements obtained on the two latter dates are shown in the following table:

BH No.	Piezometer Installed on	W.L. on January 16, 2001		W.L. on January 29, 2001	
		Below Grade (m)	Elevation (m)	Below Grade (m)	Elevation (m)
101	Jan. 5/01	1.5	230.1	1.6	230.0
104	Jan. 3/01	0*	230.5	0*	230.5
105	Jan. 2/01	2.1	230.0	2.2	229.9
111	Dec. 18/00	0.7	222.7	0.8	222.6
112	Dec. 18/00	0.4	224.2	0.4	224.2
113	Dec. 18/00	1.8	227.6	1.7	227.7
116	Dec. 13/00	0.1	230.3	0.1	230.3
28	Aug. 11/99	5.0	224.0	4.7	224.3
31	Aug. 11/99	1.5	229.8	1.7	229.6

* Water level at ground surface.

These observed water levels are also shown on the individual borehole log sheets.

The groundwater table can be expected to fluctuate seasonally and in response to major weather events.

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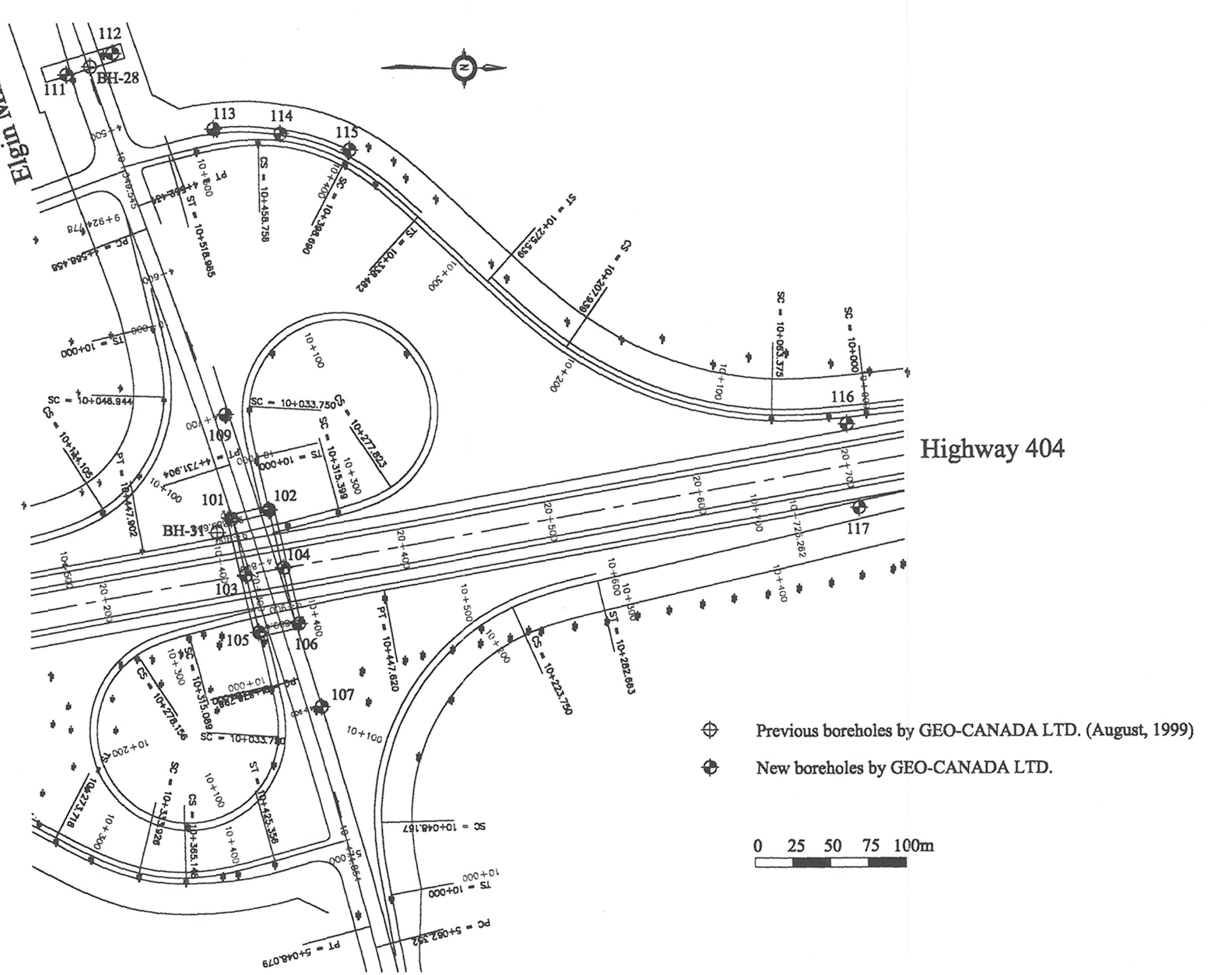
ENCLOSURES



Borehole Location Plan
Hwy. 404 / Elgin Mills Road Interchange

G-20.0702A

Encl. No. 1

July 2001

LOG OF BOREHOLE 101

CLIENT : R.V. Anderson Associates Ltd.
PROJECT : Elgin Mills Road Reconstruction
LOCATION : West abutment, South side
DATUM ELEVATION : Geodetic

DRILLING DATA
Method : Solid Stem Augering
Diameter : 110 mm
Date : Jan. 5, 2001

REF. NO. : G-20.0702A
ENCL. NO. : 2

(m) ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	'N' BLOWS / 0.3 M	GROUND WATER CONDITIONS	GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
231.6	Ground Surface					
0.0	~150mm TOPSOIL CLAYEY SILT with organics and rootlets, brown very stiff	1	SS	16	Seal	
230.8 / 0.8	SILTY SAND some gravel, trace of clay brown dense	2	SS	34		17 42 41
		3	SS	40	W.L. 230.0 m 29/01/01	
229.3 / 2.3	SANDY SILT (Glacial Till) some gravel, trace of clay brown very dense	4	SS	77/275mm		
	grey	5	SS	89/250mm		
		6	SS	50/100mm		
	lenses of silt	7	SS	50/100mm	Seal	5 23 52 20
	lenses of sand	8	SS	50/75mm		
		9	SS	50/50mm		
		10	SS	50/75mm		
222.4		11	SS	50/100mm	Piezometer	
9.2	END OF BOREHOLE					

Date	W.L.
05/01/01	230.1m
16/01/01	230.1m

Elevation scale: 228, 226, 224

RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE
PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L
WATER CONTENT (%): 10 20 30
UNIT WEIGHT
REMARKS AND GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

LOG OF BOREHOLE 102

CLIENT : R.V. Anderson Associates Ltd.
PROJECT : Elgin Mills Road Reconstruction
LOCATION : West abutment, North side
DATUM ELEVATION : Geodetic

DRILLING DATA
Method : Solid Stem Augering
Diameter : 110 mm
Date : Jan. 5, 2001

REF. NO. : G-20.0702A
ENCL. NO. : 3

(m) ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	'N' BLOWS/0.3 M	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
232.0	Ground Surface						
0.0	~150mm TOPSOIL POSSIBLE FILL sandy silt some gravel, trace of organics brown compact to dense	1	SS	10			
		2	SS	32			
230.8 / 1.2	SANDY SILT (Glacial Till) trace of gravel and clay mottled grey to brown very dense	3	SS	50/150mm			2 35 54 9
					W.L. 230.0 m 05/01/01		
		4	SS	50/100mm			
		5	SS	50/125mm			
		6	SS	50/75mm		228	7 32 52 9
		7	SS	50/125mm			
226.7 / 5.3	SILT (Glacial Till) some gravel and sand, trace of clay very dense	8	SS	50/75mm			13 14 73
						226	
		9	SS	50/100mm			
		10	SS	50/100mm		224	
		11	SS	50/50mm			
222.8 / 9.2	END OF BOREHOLE						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE
PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L
WATER CONTENT (%): 10 20 30
UNIT WEIGHT

LOG OF BOREHOLE 103

CLIENT : R.V. Anderson Associates Ltd.
PROJECT : Elgin Mills Road Reconstruction
LOCATION : South pier
DATUM ELEVATION : Geodetic

DRILLING DATA
Method : Solid Stem Augering
Diameter : 110 mm
Date : Jan. 5, 2001

REF. NO. : G-20.0702A
ENCL. NO. : 4

(m) ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	'N' BLOWS / 0.3 M	ELEVATION SCALE	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
230.4	Ground Surface					
0.0	GRAVELLY SAND some silt and clay brown, wet dense to very dense	1	SS	37	230	
		2	SS	77/275mm		35 49 16
229.0 / 1.4	SILTY SAND (Glacial Till) trace of gravel, trace to some clay brown to grey very dense	3	SS	50/100mm		
		4	SS	50/125mm	228	
		5	SS	50/50mm		
226.6 / 3.8	CLAYEY SILT (Glacial Till) trace of gravel and sand grey hard	6	SS	50/125mm		1 6 71 22
		7	SS	50/150mm	226	
		8	SS	50/150mm		
224.3 / 6.1	SANDY SILT (Glacial Till) trace to some clay, trace of gravel grey very dense	9	SS	50/125mm	224	
		10	SS	50/75mm		
					222	
		11	SS	60/125mm		
221.1 / 9.3	END OF BOREHOLE Borehole was dry upon completion					

Column headings also shown: STRATA PLOT; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L, WATER CONTENT (%) 10 20 30; UNIT WEIGHT

LOG OF BOREHOLE 104

CLIENT : R.V. Anderson Associates Ltd.
PROJECT : Elgin Mills Road Reconstruction
LOCATION : North pier
DATUM ELEVATION : Geodetic

DRILLING DATA
Method : Solid Stem Augering
Diameter : 110 mm
Date : Jan. 3, 2000

REF. NO. : G-20.0702A
ENCL. NO. : 5

(m) ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	'N' BLOWS / 0.3 M	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
230.5	Ground Surface				W.L. 230.5 m 29/01/01	230.5		
0.0 / 229.9	FILL sandy silt some gravel, trace of organics, mottled grey compact	1	SS	14	Seal	230		
0.6	SILTY SAND (Glacial Till) some to trace gravel trace of clay brown very dense	2	SS	61			○	16 45 33 6
		3	SS	60/150mm			○	
227.7		4	SS	68		228	○	1 61 38
2.8	SANDY SILT (Glacial Till) trace of clay, some gravel grey very dense	5	SS	50/75mm			○	
		6	SS	50/75mm			○	13 74 13
		7	SS	50/75mm	Seal	226	○	
		8	SS	50/75mm				
		9	SS	50/125mm		224	○	
		10	SS	50/125mm			○	
221.2		11	SS	58/150mm	Piezometer	222	○	
9.3	END OF BOREHOLE							

Date W.L.
03/01/01 225.0m
16/01/01 230.5m

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE
PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30
UNIT WEIGHT

LOG OF BOREHOLE 105

CLIENT : R.V. Anderson Associates Ltd.
PROJECT : Elgin Mills Road Reconstruction
LOCATION : East abutment, South side
DATUM ELEVATION : Geodetic

DRILLING DATA
Method : Solid Stem Augering
Diameter : 110 mm
Date : Jan. 2, 2001

REF. NO. : G-20.0702A
ENCL. NO. : 6

(m) ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	'N' BLOWS / 0.3 M	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
232.1	Ground Surface						
0.0	~75mm TOPSOIL SILTY CLAY with organics and rootlets dark brown, firm	1	SS	8	Seal	232	
231.3 / 0.8	SANDY SILT trace of gravel brown, wet compact to v. dense	2	SS	25			
		3	SS	68			0 34 66
					W.L. 229.9 m 29/01/01		
		4	SS	59			
229.3 / 2.8	SAND and GRAVEL trace of silt brown, wet very dense	5	SS	78/275mm			57 37 6
228.5 / 3.6	SAND fine to medium some gravel, trace of silt, brown, wet very dense	6	SS	76		228	
227.8 / 4.3	SILTY SAND (Glacial Till) trace of clay, some gravel grey, wet very dense	7	SS	50/125mm	Seal		
		8	SS	50/125mm			
		9	SS	50/100mm		226	12 47 41
		10	SS	50/50mm		224	
		11	SS	50/75mm	Piezometer		
222.9 / 9.2	END OF BOREHOLE						

Dynamic Cone Penetration Resistance Plot: 20 40 60 80 100
Shear Strength: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE
Plastic Limit W_P, Natural Moisture Content W, Liquid Limit W_L
Water Content (%): 10 20 30
Unit Weight

Date	W.L.
03/01/01	230.7m
16/01/01	230.0m

Geo-Canada Ltd.

LOG OF BOREHOLE 106

CLIENT : R.V. Anderson Associates Ltd.
PROJECT : Elgin Mills Road Reconstruction
LOCATION : East abutment, North side
DATUM ELEVATION : Geodetic

DRILLING DATA
Method : Solid Stem Augering
Diameter : 110 mm
Date : Dec. 15, 2000

REF. NO. : G-20.0702A
ENCL. NO. : 7

(m) ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	'N' BLOWS/0.3 M	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
232.9	Ground Surface						
0.0	~125mm TOPSOIL SILTY CLAY, trace gravel, some sand, organics, dark brown stiff	1	SS	14			
232.1 / 0.8	SILTY SAND to SANDY SILT brown, compact	2	SS	26		232	
231.6 / 1.3	SAND and SILT (Glacial Till) some to trace of gravel, trace of clay dense to very dense	3	SS	34			10 43 40 7
		4	SS	50/138mm			
						230	
	brown	5	SS	50/125mm			
	grey	6	SS	50/100mm			4 54 42
		7	SS	50/125mm	Caved 228.0 m 15/12/00		
		8	SS	55/138mm			
		9	SS	50/125mm			
						226	
		10	SS	50/100mm			
						224	
223.7		11	SS	50/100mm			
9.2	END OF BOREHOLE Borehole was dry upon completion						

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100
SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE
PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L
WATER CONTENT (%) 10 20 30
UNIT WEIGHT

LOG OF BOREHOLE 107

CLIENT : R.V. Anderson Associates Ltd.
PROJECT : Elgin Mills Road Reconstruction
LOCATION : Sta. 4+900
DATUM ELEVATION : Geodetic

DRILLING DATA
Method : Solid Stem Augering
Diameter : 110 mm
Date : Jan. 2, 2001

REF. NO. : G-20.0702A
ENCL. NO. : 8

(m) ELEV / DEPTH	SOIL PROFILE DESCRIPTION	NUMBER	TYPE	'N' BLOWS/0.3 M	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS
234.9	Ground Surface						
0.0	~50mm TOPSOIL SILTY CLAY trace of organics and rootlets dark brown stiff	1	SS	11			
		2	SS	10		234	
233.5 1.4	SILTY SAND trace of gravel brown, wet dense to very dense	3	SS	34			
		4	SS	93/275mm			
		5	SS	50/125mm		232	
		6	SS	50/125mm			
230.8 4.1	END OF BOREHOLE Borehole was dry upon completion						

LOG OF BOREHOLE 109

CLIENT : R.V. Anderson Associates Ltd.
PROJECT : Elgin Mills Road Reconstruction
LOCATION : Sta. 4+700
DATUM ELEVATION : Geodetic

DRILLING DATA
Method : Solid Stem Augering
Diameter : 110 mm
Date : Jan. 5, 2001

REF. NO. : G-20.0702A
ENCL. NO. : 9

(m) ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	'N' BLOWS/0.3 M	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
231.9	Ground Surface							
0.0	~200mm TOPSOIL CLAYEY SILT some organics and gravel, brown, stiff	1	SS	10				
231.1 / 0.8	SILT and SAND some to trace gravel brown, wet compact to v. dense	2	SS	20				0 54 46
					W.L. 230.4 m 05/01/01			
		3	SS	36		230		
		4	SS	50/100mm				
		5	SS	50/125mm				
		6	SS	50/100mm		228		
		7	SS	59				
227.0 / 4.9	END OF BOREHOLE							

Chart headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT

LOG OF BOREHOLE 111

CLIENT : R.V. Anderson Associates Ltd.
PROJECT : Elgin Mills Road Reconstruction
LOCATION : Sta. 4+450 South
DATUM ELEVATION : Geodetic

DRILLING DATA
Method : Solid Stem Augering
Diameter : 110 mm
Date : Dec. 18, 2000

REF. NO. : G-20.0702A
ENCL. NO. : 10

(m) ELEV / DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	'N' BLOWS / 0.3 M	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH	WATER CONTENT (%) W_P W W_L (10 20 30)	UNIT WEIGHT	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
223.4	Ground Surface										
0.0	~50mm TOPSOIL POSSIBLE FILL silty sand trace of gravel, rootlets and organics dark brown loose to compact		1	SS	6	Seal			o		
			2	SS	15	W.L. 222.6 m 29/01/01			o		1 78 21
222.0 / 1.4	SILT and SAND (Glacial Till) some clay and gravel grey very dense		3	SS	76		222		o		
			4	SS	50/125mm	Piezometer			o		
220.2			5	SS	50/125mm	Caved 220.5 m 18/12/00			o		
3.2	END OF BOREHOLE										

Date	W.L.
18/12/00	222.6m
16/01/01	222.7m

Geo-Canada Ltd.

LOG OF BOREHOLE 112

CLIENT : R.V. Anderson Associates Ltd.
PROJECT : Elgin Mills Road Reconstruction
LOCATION : Sta. 4+450 North
DATUM ELEVATION : Geodetic

DRILLING DATA
Method : Solid Stem Augering
Diameter : 110 mm
Date : Dec. 18, 2000

REF. NO. : G-20.0702A
ENCL. NO. : 11

(m) ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	'N' BLOWS / 0.3 M	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
224.6	Ground Surface							
0.0 / 223.8	~75mm TOPSOIL SILTY CLAY, trace ot some sand and gravel dark brown, soft	1	SS	3	Seal; W.L. 224.2 m 29/01/01	224	○	
0.8 / 223.1	GRAVELLY SAND trace of silt grey, wet very dense	2	SS	55			○	
1.5 / 221.1	SAND and SILT (Glacial Till) some clay, trace of gravel grey very dense	3	SS	96			○	2 36 44 18
		4	SS	50/125mm	Piezometer	222	○	
		5	SS	92/275mm			○	
3.5	END OF BOREHOLE							

Date	W.L.
18/12/00	222.8m
16/01/01	224.2m

LOG OF BOREHOLE 113

CLIENT : R.V. Anderson Associates Ltd.
PROJECT : Elgin Mills Road Reconstruction
LOCATION : N-EW Ramp Sta. 10+475 approx.
DATUM ELEVATION : Geodetic

DRILLING DATA
Method : Solid Stem Augering
Diameter : 110 mm
Date : Dec. 18, 2000

REF. NO. : G-20.0702A
ENCL. NO. : 12

(m) ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	'N' BLOWS/0.3 M	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
229.4	Ground Surface							
0.0	SANDY SILT some rootlets and trace of organics to 0.6m, trace of gravel brown compact to dense	1	SS	14	Seal			
		2	SS	23				3 30 67
						228		
		3	SS	35	W.L. 227.7 m 29/01/01			
227.3 / 2.1	SILTY SAND trace of silt brown very dense	4	SS	50/125mm				
		5	SS	72		226		
225.8 / 3.6	SANDY SILT (Glacial Till) clayey some gravel brown to grey very dense	6	SS	50/125mm				
					Piezometer			
		7	SS	92				
224.4 / 5.0	END OF BOREHOLE Date W.L. 18/12/00 226.7m 16/01/01 227.6m							

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE
PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30
UNIT WEIGHT

LOG OF BOREHOLE 114

CLIENT : R.V. Anderson Associates Ltd.
PROJECT : Elgin Mills Road Reconstruction
LOCATION : N-EW Ramp Sta. 10+445 approx.
DATUM ELEVATION : Geodetic

DRILLING DATA
Method : Solid Stem Augering
Diameter : 110 mm
Date : Dec. 13, 2000

REF. NO. : G-20.0702A
ENCL. NO. : 13

(m) ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	'N' BLOWS/0.3 M	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
228.9	Ground Surface						
0.0	~75mm TOPSOIL over SILTY CLAY, some organics and rootlets, dark brown, very soft	1	SS	4			
228.1 / 0.8	SAND and SILT (Glacial Till), some clay, trace of gravel, brown, wet, compact to very dense	2	SS	15		228	7 46 36 11
		3	SS	38			
		4	SS	50/125mm			
		5	SS	50/125mm		226	
		6	SS	50/125mm	W.L. 224.9 m 13/12/00; Caved 224.5 m 13/12/00		
223.9 / 5.0	grey	7	SS	50/125mm		224	
	END OF BOREHOLE						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH: O UNCONFINED; + FIELD VANE; ● QUICK TRIAXIAL; × LAB VANE
PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L
WATER CONTENT (%): 10 20 30
UNIT WEIGHT

LOG OF BOREHOLE 115

CLIENT : R.V. Anderson Associates Ltd.
PROJECT : Elgin Mills Road Reconstruction
LOCATION : N-EW Ramp Sta. 10+400 approx.
DATUM ELEVATION : Geodetic

DRILLING DATA
Method : Solid Stem Augering
Diameter : 110 mm
Date : Dec. 13, 2000

REF. NO. : G-20.0702A
ENCL. NO. : 14

(m) ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	'N' BLOWS / 0.3 M	GROUND WATER CONDITIONS	ELEVATION SCALE	NATURAL MOISTURE CONTENT W (%)	GR	SA	SI	CL
227.1	Ground Surface										
0.0	~50mm TOPSOIL over CLAYEY SILT to SILTY CLAY some organics and rootlets dark brown frim to stiff	1	SS	6			~41				
		2	SS	10		226	~31				
225.9 / 1.2	SILTY SAND to SANDY SILT trace of clay, some gravel brown, wet dense to very dense	3	SS	45			~25				
		4	SS	50/100mm			~25				
224.5 / 2.6	SAND and SILT (Glacial Till) some clay, trace of gravel very dense	5	SS	50/125mm		224	~24	3	36	42	19
		6	SS	50/125mm			~24				
		7	SS	50/125mm	Caved 223.7 m 13/12/00		~24				
222.4 / 4.7	END OF BOREHOLE Borehole was dry upon completion										

LOG OF BOREHOLE 116

CLIENT : R.V. Anderson Associates Ltd.
PROJECT : Elgin Mills Road Reconstruction
LOCATION : N-EW Ramp Sta. 10+020
DATUM ELEVATION : Geodetic

DRILLING DATA
Method : Solid Stem Augering
Diameter : 110 mm
Date : Dec. 13, 2000

REF. NO. : G-20.0702A
ENCL. NO. : 15

(m) ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLES: NUMBER	TYPE	'N' BLOWS / 0.3 M	GROUND WATER CONDITIONS	ELEVATION SCALE
230.4	Ground Surface					
0.0	~125mm TOPSOIL over CLAYEY SILT to SILTY CLAY some rootlets and sand, some to trace gravel brown firm to very stiff	1	SS	7	W.L. 230.3 m 29/01/01; Seal	230
		2	SS	20		
228.2		3	SS	26		
2.2	CLAYEY SILT (Glacial Till) some gravel wet very stiff	4	SS	19		228
226.5		5	SS	16		
3.9	SANDY SILT (Glacial Till) trace of clay some gravel wet very dense	6	SS	70		226
225.4		7	SS	69	Piezometer	
5.0	END OF BOREHOLE					

Date W.L.
13/12/00 227.1m
16/01/01 230.3m

Column headings (plots): DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT; REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

LOG OF BOREHOLE 117

CLIENT : R.V. Anderson Associates Ltd.
PROJECT : Elgin Mills Road Reconstruction
LOCATION : E-N Ramp Sta. 10+465 approx. @ ℄
DATUM ELEVATION : Geodetic

DRILLING DATA
Method : Solid Stem Augering
Diameter : 110 mm
Date : Dec. 15, 2000

REF. NO. : G-20.0702A
ENCL. NO. : 16

(m) ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	'N' BLOWS / 0.3 M	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
231.6	Ground Surface						
0.0	~150mm TOPSOIL over SANDY SILT to SILTY SAND (Glacial Till) trace of clay, some gravel brown compact	1	SS	11			
		2	SS	21			
		3	SS	28	W.L. 229.8 m 15/12/00	230	10 35 55
229.3 / 2.3	SILT (Glacial Till) some sand, trace of clay brown compact to v. dense	4	SS	23			0 17 78 5
		5	SS	50/125mm	Caved 228.6 m 15/12/00		
228.0 / 3.6	GRAVELLY SAND some gravel and silt with fine sand at bottom browon to grey, wet compact to v. dense	6	SS	15		228	
		7	SS	50/50mm			
226.8 / 4.8	END OF BOREHOLE						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE
PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L
WATER CONTENT (%): 10 20 30
UNIT WEIGHT

LOG OF BOREHOLE 28

CLIENT : R. V. Anderson
PROJECT : Widening of Elgin Mills Rd.
LOCATION : Between Bayview Ave. and Woodbine Ave.
DATUM ELEVATION : Geodetic

DRILLING DATA
Method : Augering
Diameter : 100mm
Date : 11/08/99

REF. NO. : G-981203
ENCL. NO. : 17

(m) ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	'N' BLOWS/0.3 M	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
229.0	Ground Surface						
0.0 / 228.4	70mm ASPHALT FILL gravelly sand some silt brown, damp, compact	1	SS	23			24 59 17
0.6 / 226.7	FILL sandy silt some clay compact black to brown damp grey moist	2	SS	12		228	
		3	SS	26			
2.3 / 223.7	FILL sandy clay some silt and gravel trace of cobbles brown, moist firm to stiff	4	SS	13		226	
		5	SS	7			
	some sand and topsoil rootlets	6	SS	6			
		7	SS	4	Seal	224	
5.3 / 222.4	SANDY SILT trace of clay brown, damp compact to v. dense	8	SS	22	W.L. 223.5 m 11/08/99		
		9	SS	72	Piezometer		
6.6	END OF BOREHOLE Date W.L. 16/08/99 223.7m 23/08/99 223.6m						

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100
SHEAR STRENGTH: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE
PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L
WATER CONTENT (%) 10 20 30
UNIT WEIGHT

LOG OF BOREHOLE 31

CLIENT : R. V. Anderson
PROJECT : Widening of Elgin Mills Rd.
LOCATION : Between Bayview Ave. and Woodbine Ave.
DATUM ELEVATION : Geodetic

DRILLING DATA
Method : Augering
Diameter : 100mm
Date : 11/08/99

REF. NO. : G-981203
ENCL. NO. : 18

(m) ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES – NUMBER	TYPE	'N' BLOWS / 0.3 M	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
231.3	Ground Surface									
0.0	100mm TOPSOIL SANDY SILT (POSSIBLE FILL) trace of clay, brown	1	SS	19						
230.5 / 0.8	SILTY SAND brown, damp compact	2	SS	19						
230.1 / 1.2	SANDY SILT (Glacial Till) brown, damp dense	3	SS	47		230				
229.0 / 2.3	SANDY SILT trace of gravel and clay, grey, damp very dense	4	SS	50/150mm			2	46	42	10
228.3 / 3.0	SANDY SILT (Glacial Till) trace of gravel some clay grey, damp very dense	5	SS	50/125mm		228	3	44	38	15
		6	SS	50/50mm						
		7	SS	50/150mm						
		8	SS	50/75mm		226				
		9	SS	50/125mm		224				
					Seal					
		10	SS	50/75mm	Piezometer					
					Caved 222.2 m 11/08/99	222				
221.1		11	SS	50/125mm						
10.2	END OF BOREHOLE Borehole was dry upon completion Date W.L. 16/08/99 229.4m 23/08/99 229.4m									

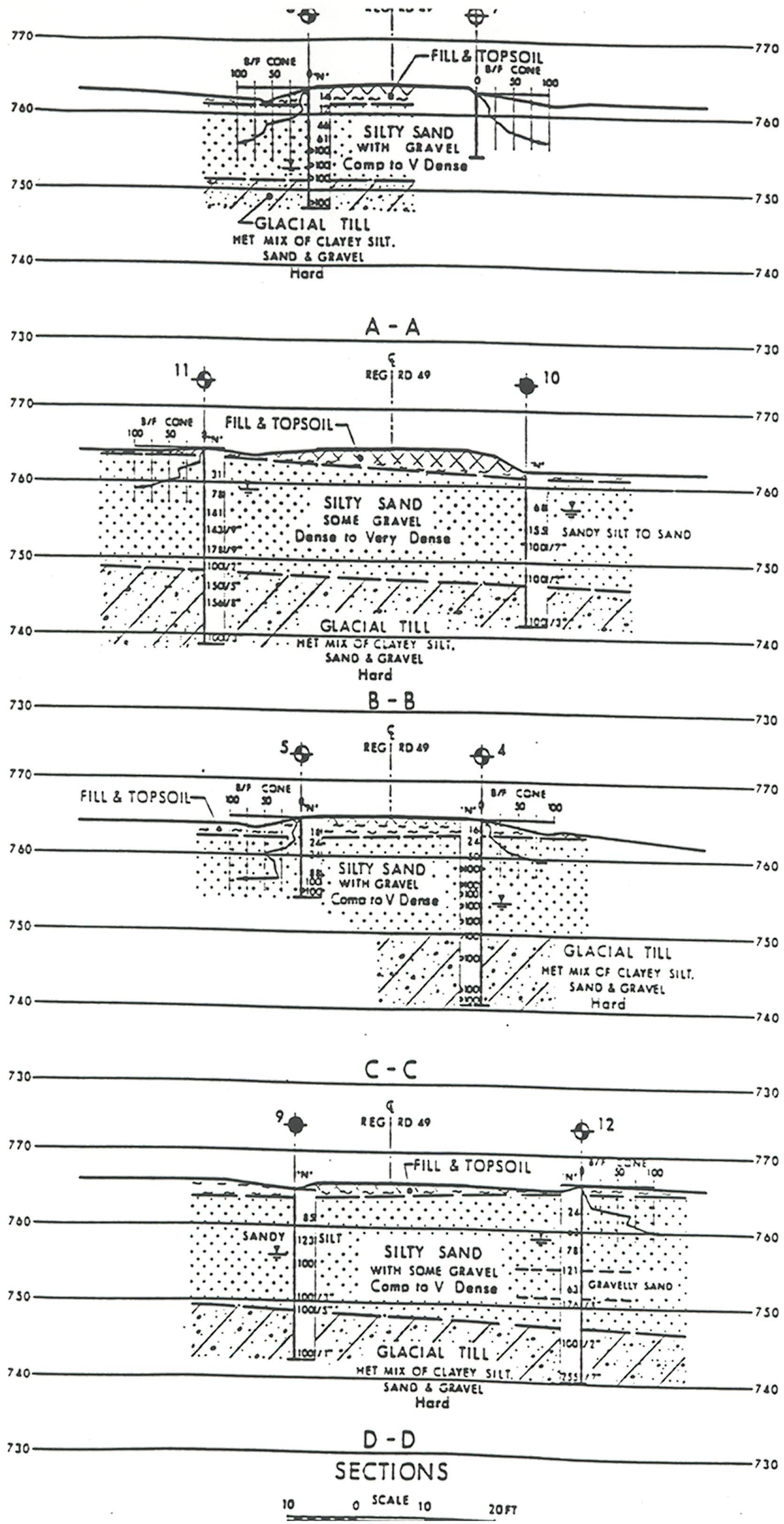
DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE
PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L
WATER CONTENT (%): 10 20 30
UNIT WEIGHT

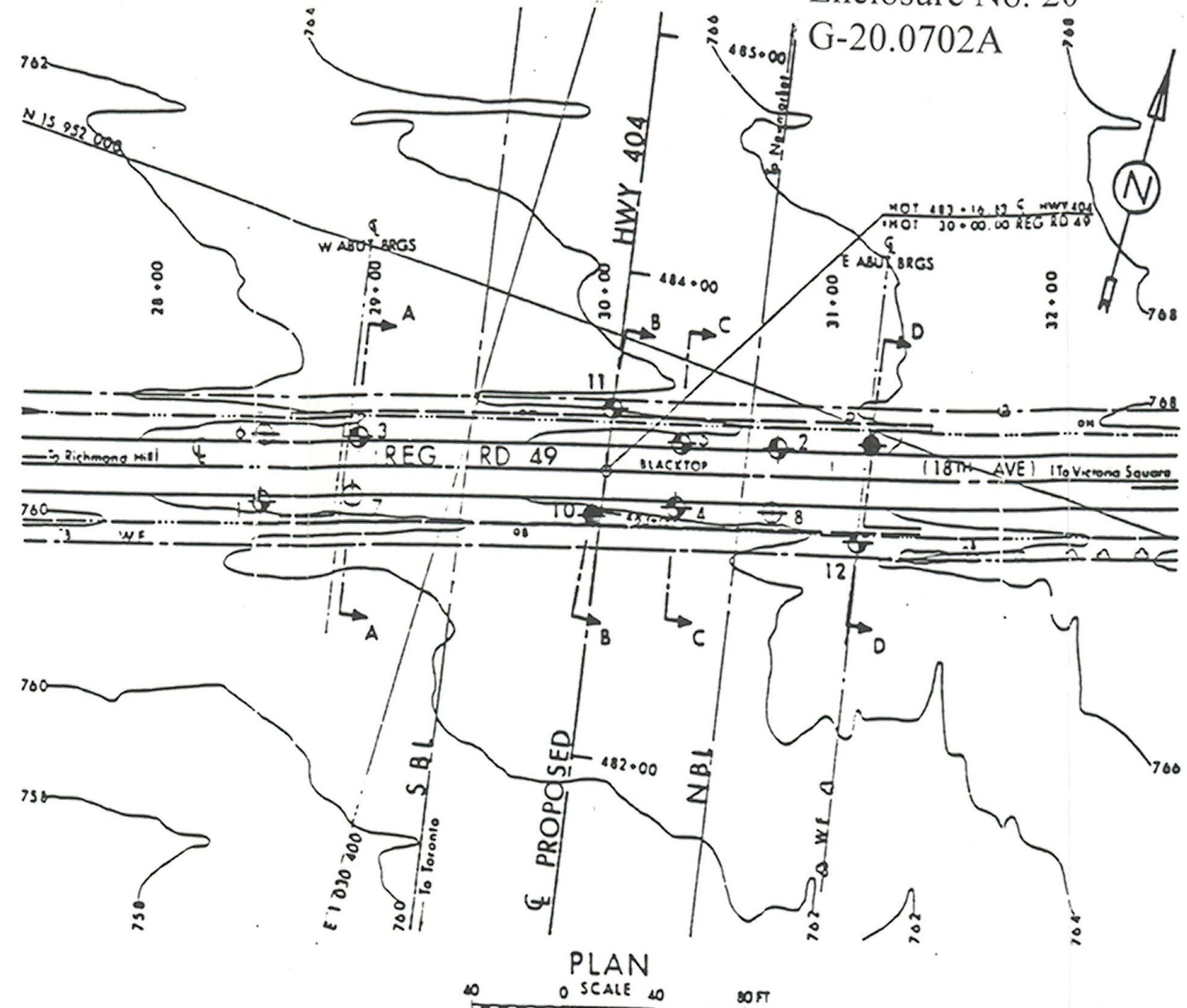
Enclosure No. 20
G-20.0702A

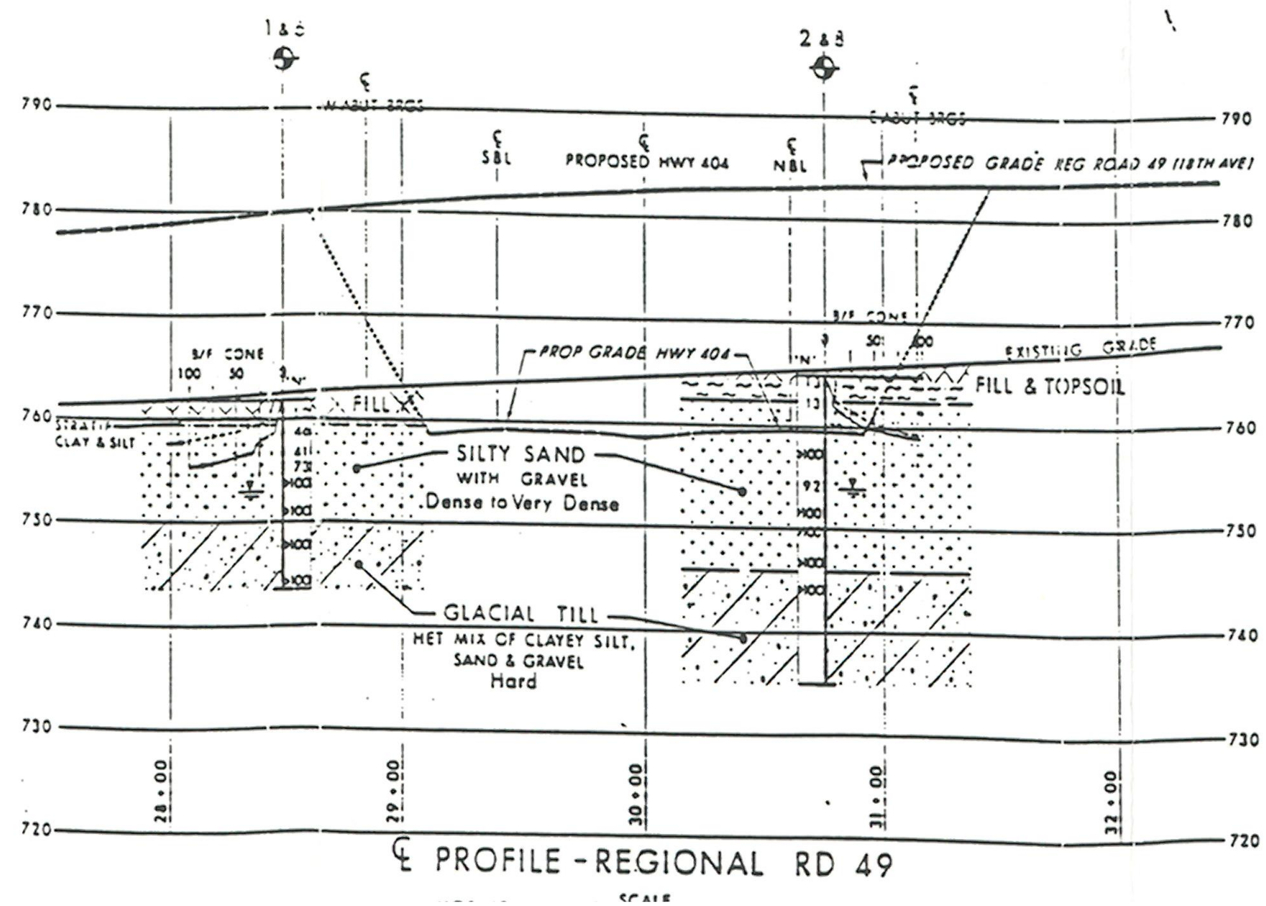
CONT No 78-45
WP No 160-74-28

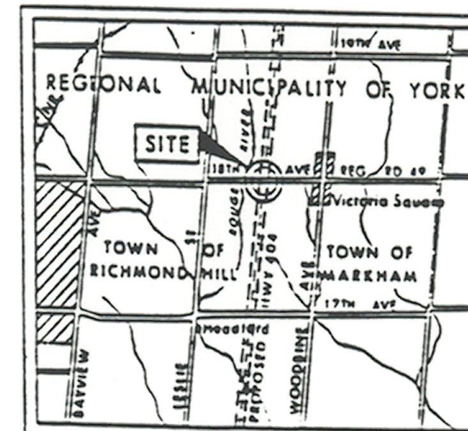
REG RD 49 U'PASS (18TH AVE)
3·8 MILES NORTH OF HWY 7
BORE HOLE LOCATIONS & SOIL STRATA

SHEET 298









KEY PLAN

1 0·5 0 1 2 Miles

LEGEND

Bore Hole

Dynamic Cone Penetration Test (Cone)

Bore Hole & Cone

"N" Blows/ft (Std Pen Test 350 ft lbs energy)

CONE Blows/ft (60° Cone, 350 ft lbs energy)

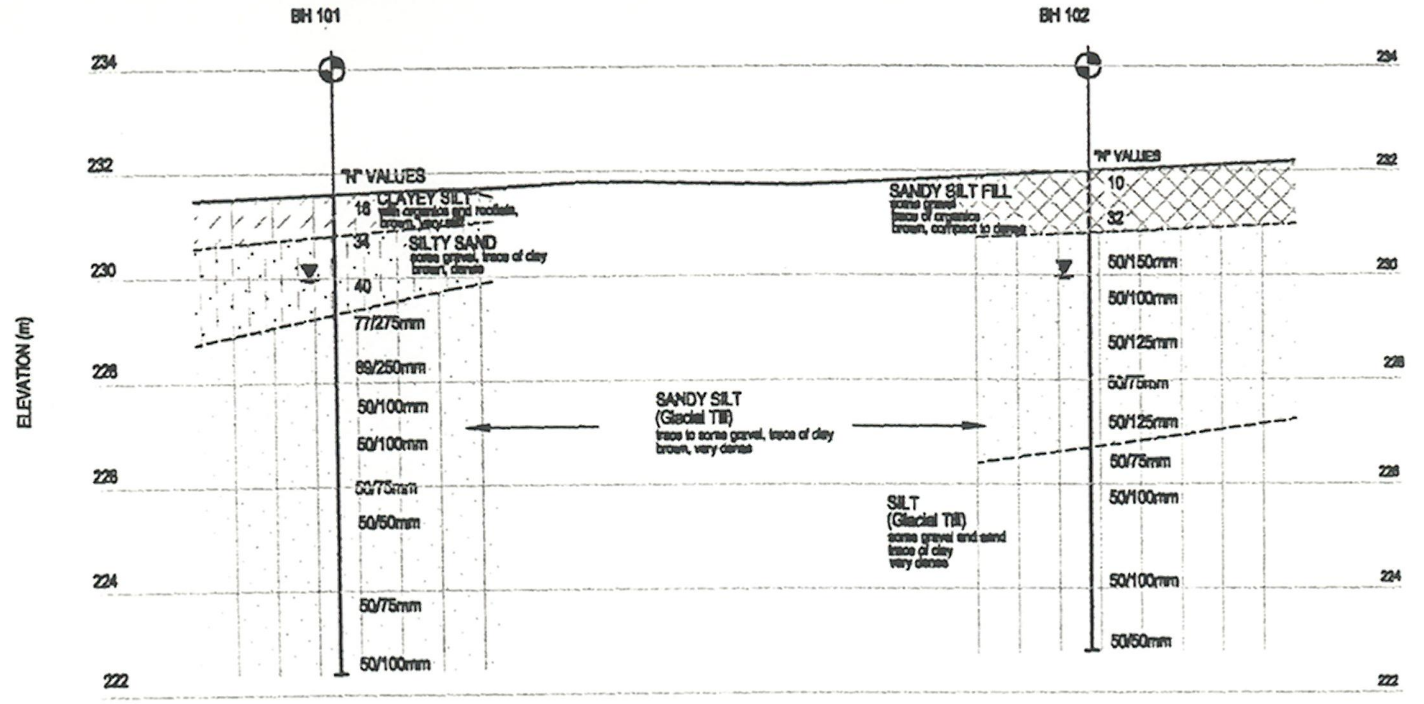
WL at time of investigation
Bore Holes 1 to 8 June 1962
" " 9 & 10 Nov 1970
" " 11 & 12 Feb 1977

No	ELEVATION	CO-ORDINATES NORTH	EAST	
1	762·0	15 951 885	1 030 320	FRANKI June 1962
2	765·0	15 951 980	1 030 532	
3	763·5	15 951 924	1 030 356	
4	765·0	15 951 942	1 030 495	
5	765·0	15 951 967	1 030 490	
6	763·0	15 951 912	1 030 314	
7	763·0	15 951 900	1 030 360	
8	765·5	15 951 954	1 030 537	
9	764·9	15 951 995	1 030 570	MTC Nov 1970
10	762·0	15 951 926	1 030 462	
11	764·6	15 951 971	1 030 457	Feb 1977
12	766·3	15 951 954	1 030 576	

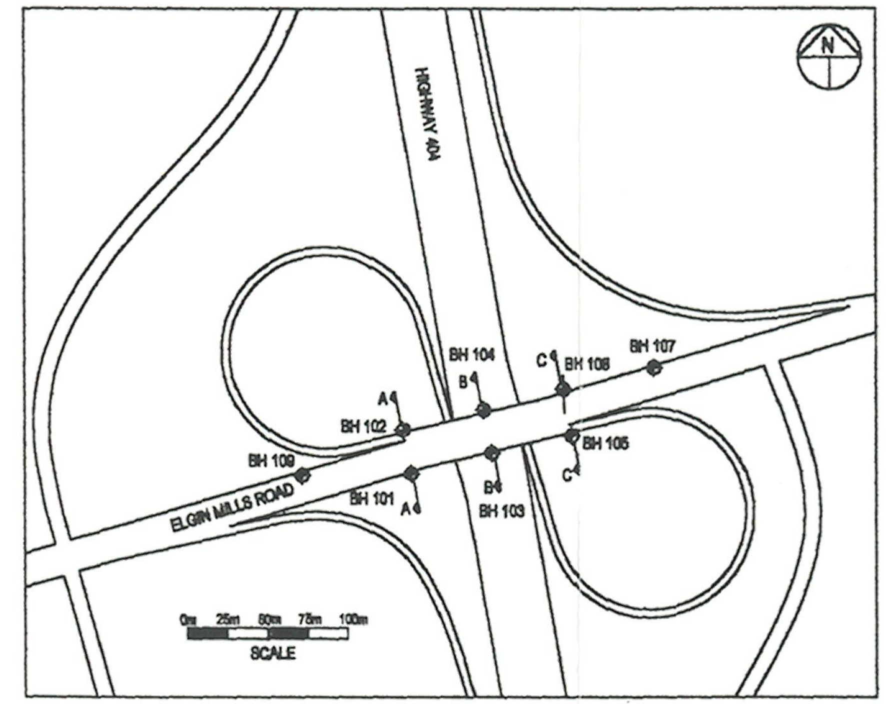
NOTE:
The complete foundation investigation file for this project may be examined at the Engineering Materials Office, Downsview. Information contained in this file and any supplementary files is specifically excluded in accordance with the conditions of Section 102-2 of Form 100.

-NOTE-
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

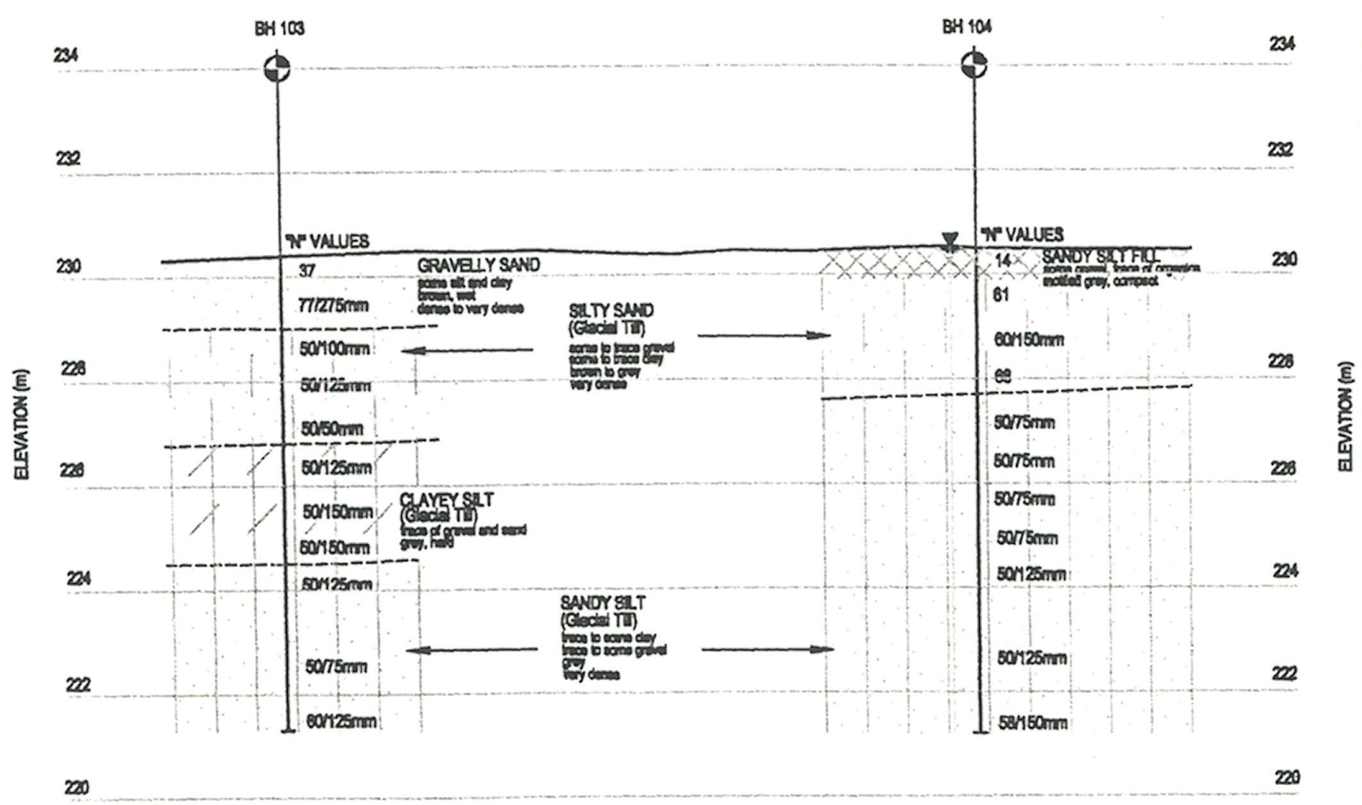
REVISIONS		
DATE	BY	DESCRIPTION



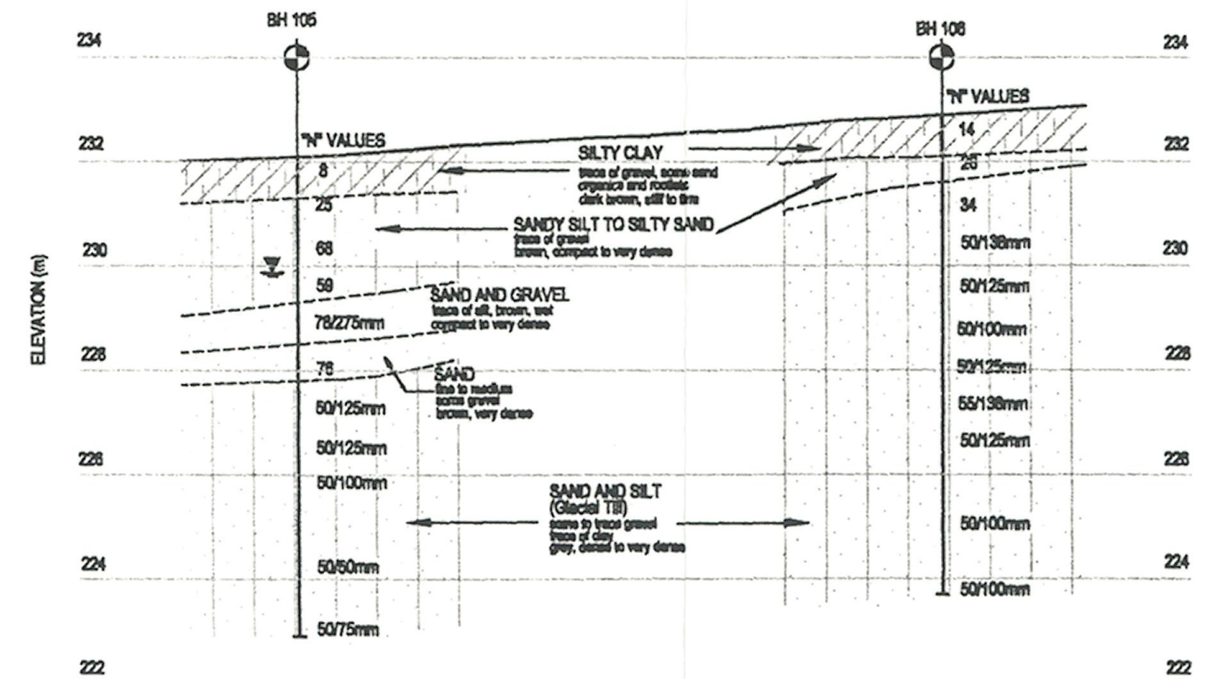
WEST ABUTMENT PROFILE (A-A)



PLAN



PIER PROFILE (B-B)



EAST ABUTMENT PROFILE (C-C)

NOTE: The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are asssumed from geological evidence.

HORIZONTAL
0 2m 4m 6m 8m 10m
0 1m 2m 3m 4m 5m
VERTICAL

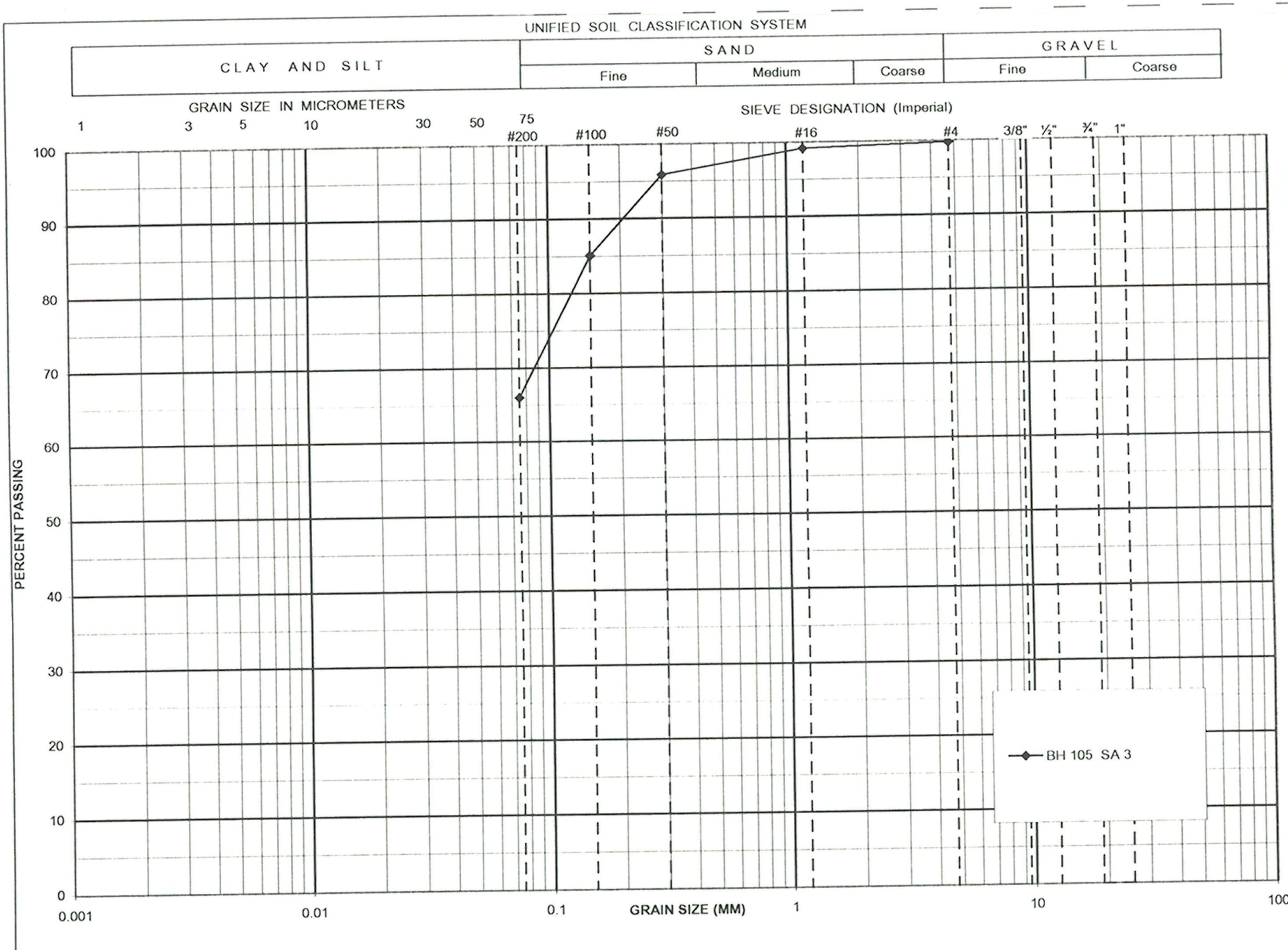
HIGHWAY 404 - ELGIN MILLS INTERCHANGE
INFERRED GEOLOGICAL PROFILES

G-20.0702A

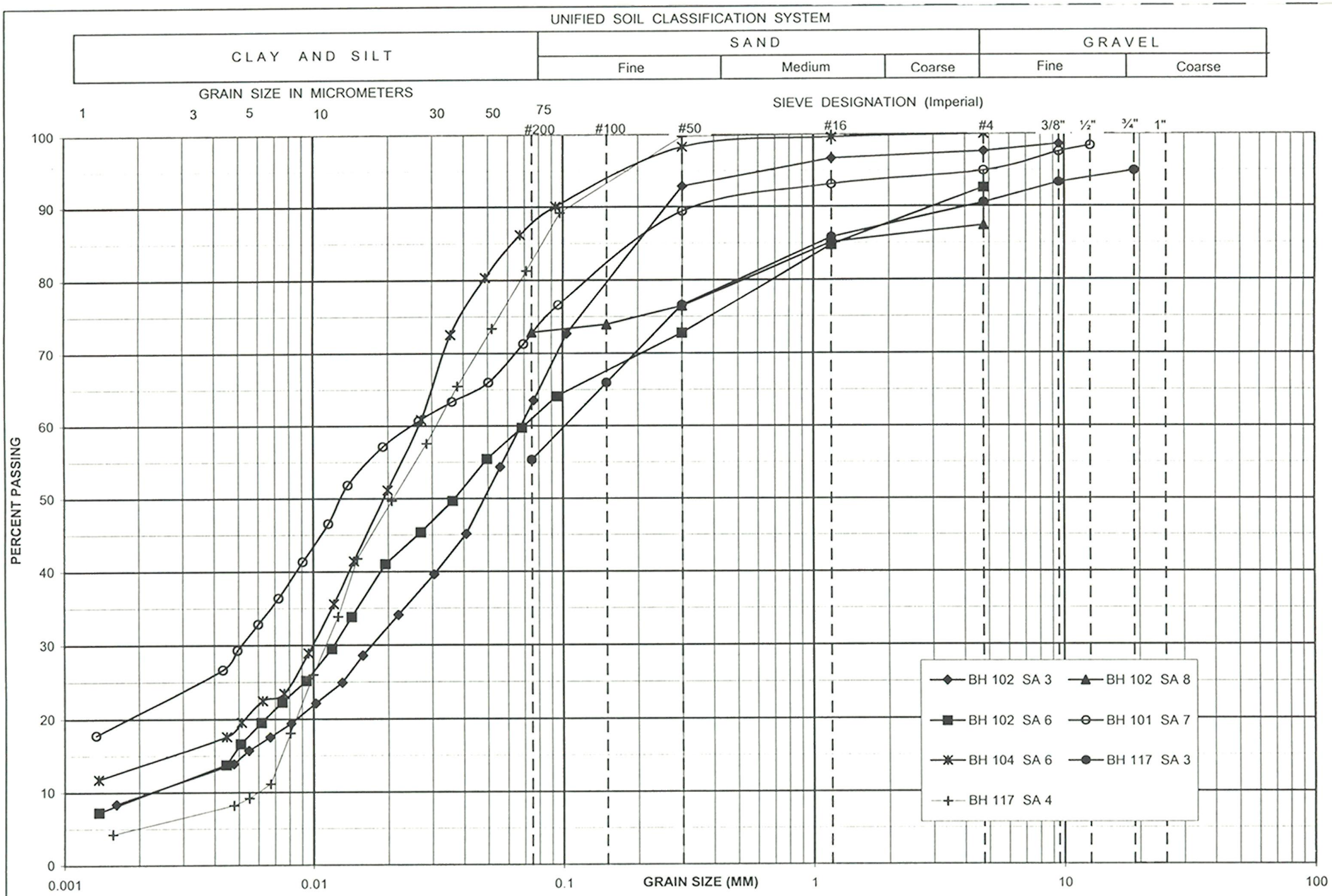
Enclosure No. 19

July, 2001

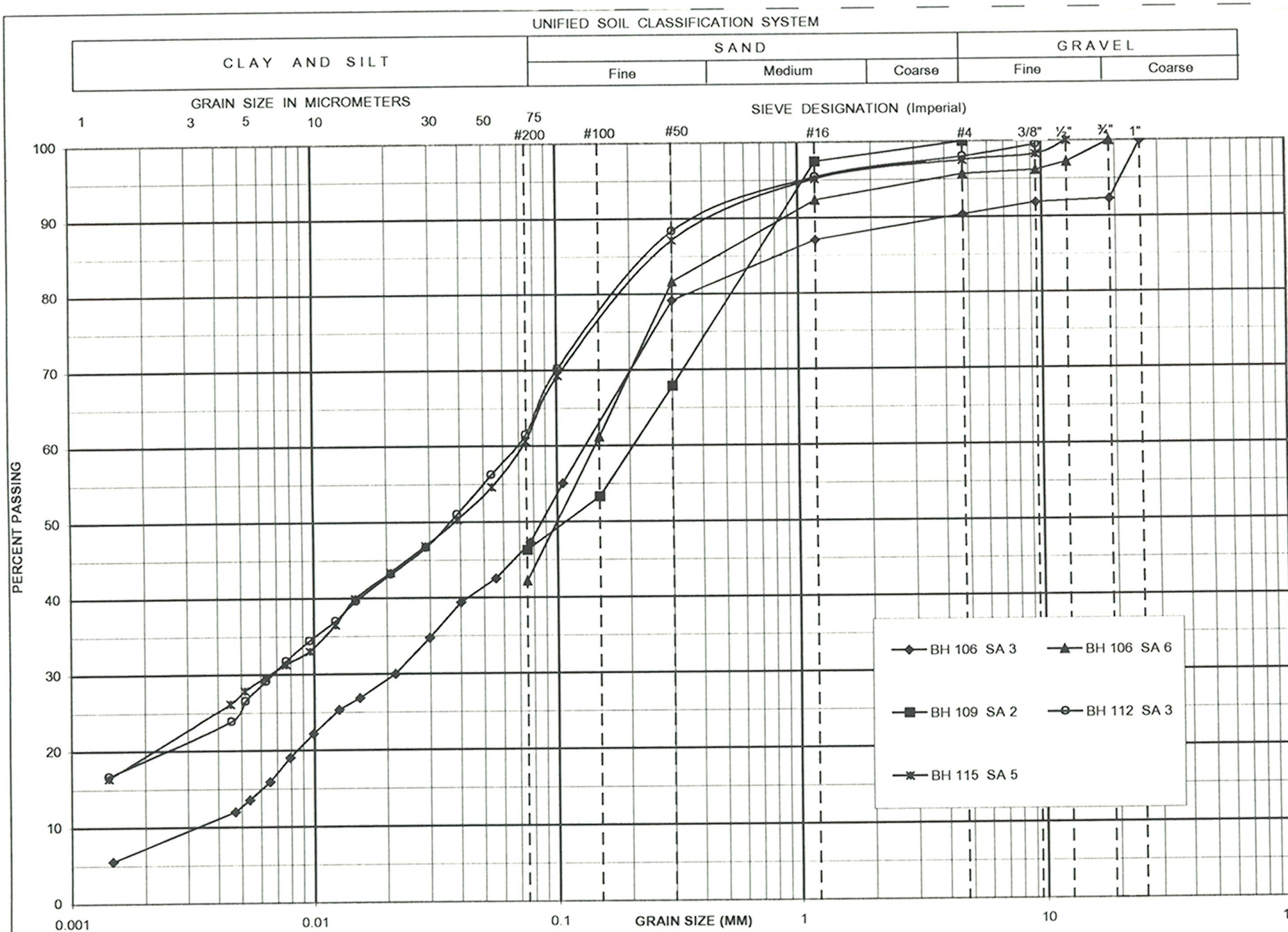
FIGURES



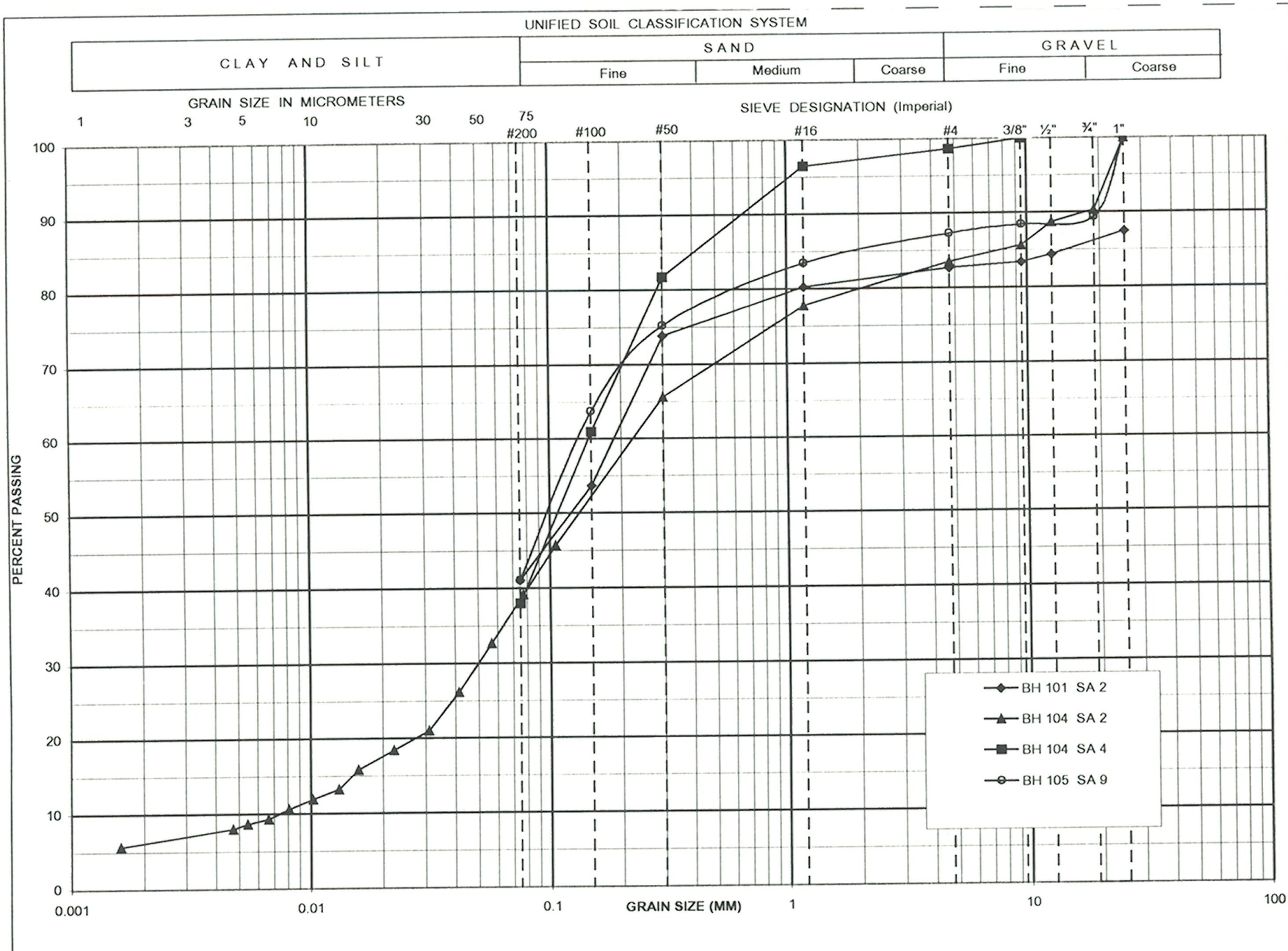
GEO - CANADA	GRAIN SIZE DISTRIBUTION SANDY SILT	FIGURE No.	1
		REF. No.	G-20.0702A
		DATE	February, 2001



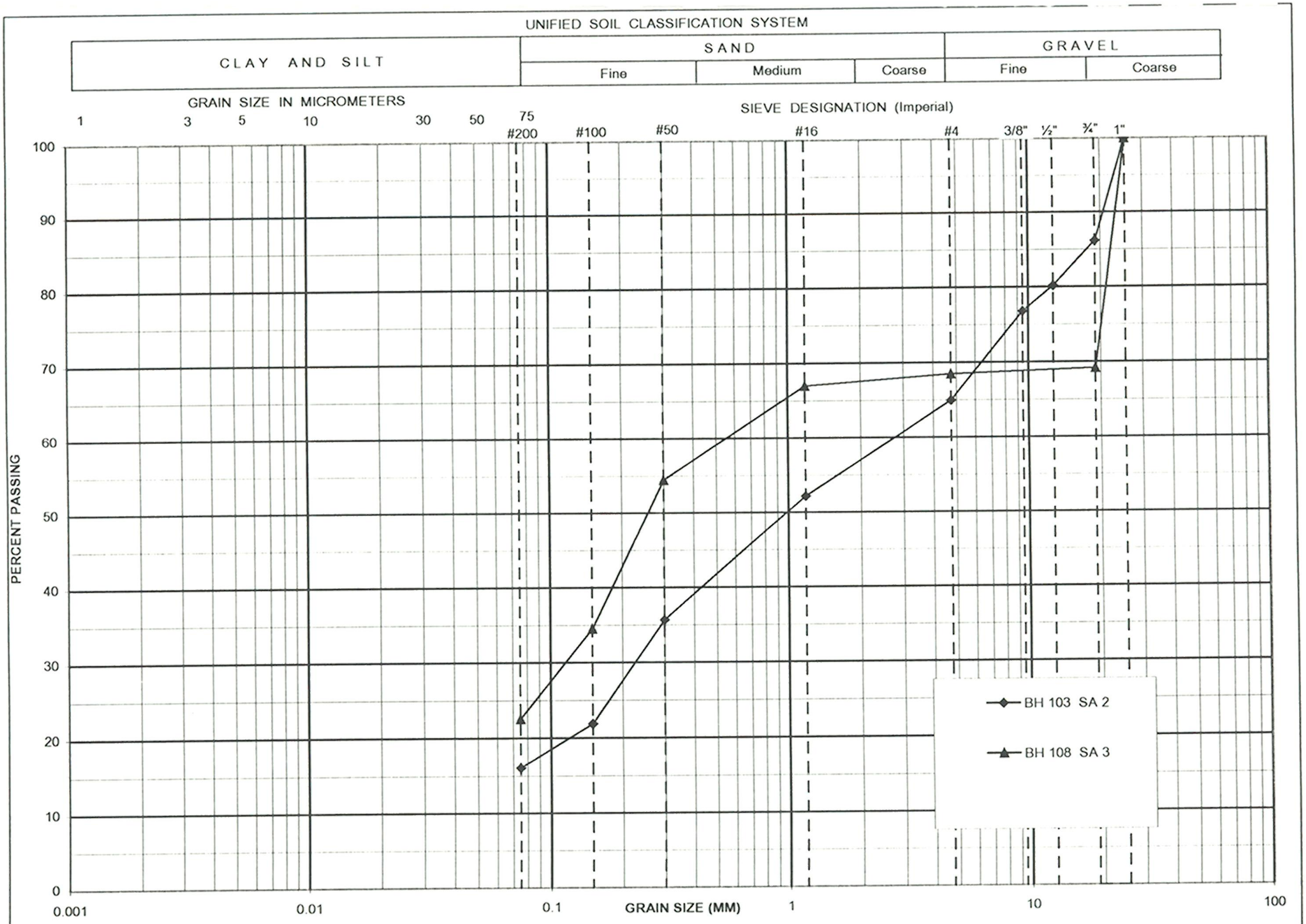
GEO - CANADA	GRAIN SIZE DISTRIBUTION SANDY SILT TILL AND SILT TILL	FIGURE No.	2
		REF. No.	G-20.0702A
		DATE	September, 2001



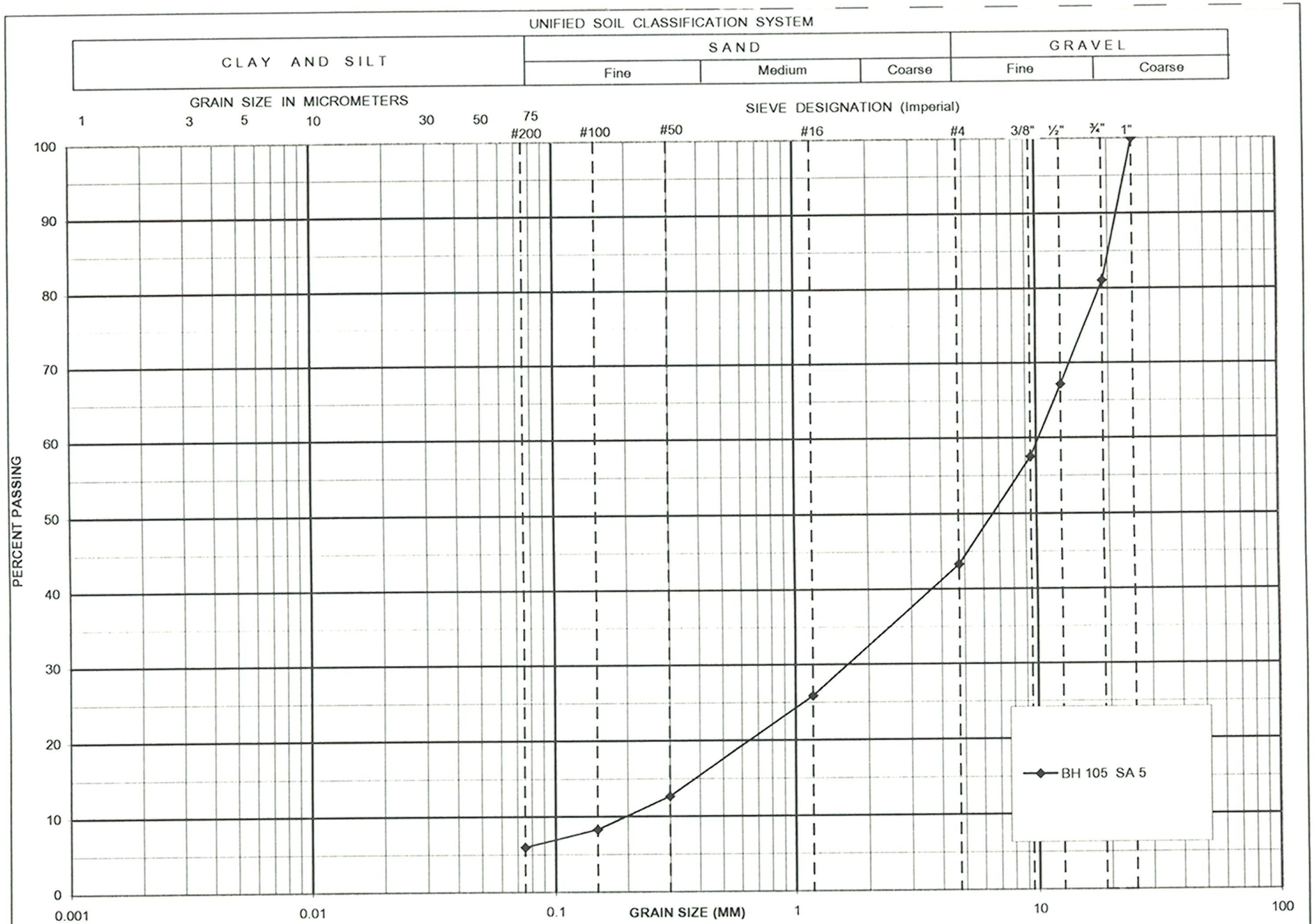
GEO - CANADA	GRAIN SIZE DISTRIBUTION SAND AND SILT TILL / SAND AND SILT	FIGURE No.	3
		REF. No.	G-20.0702A
		DATE	February, 2001



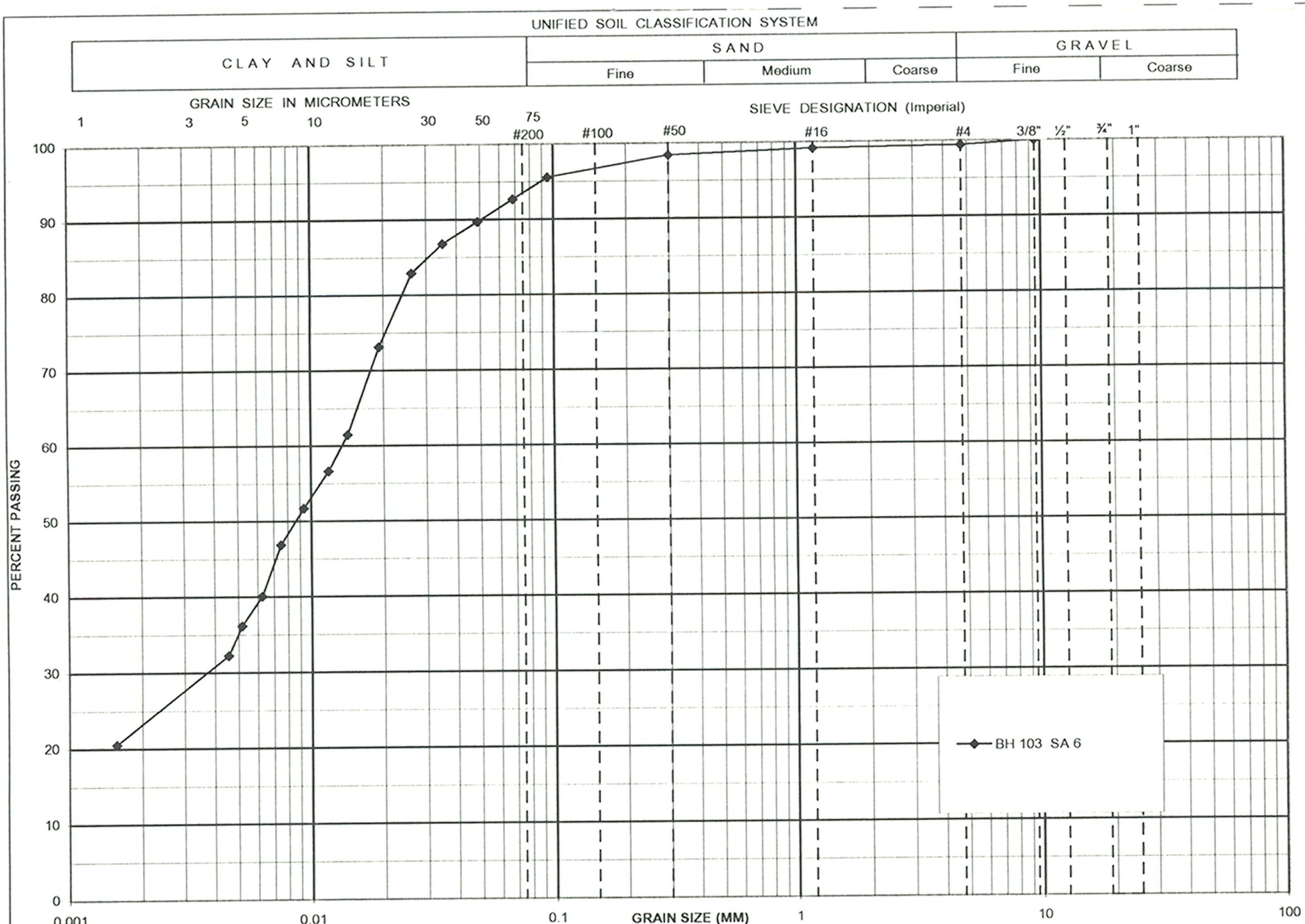
GEO - CANADA	GRAIN SIZE DISTRIBUTION SILTY SAND AND SILTY SAND TILL	FIGURE No.	4
		REF. No.	G-20.0702A
		DATE	February, 2001



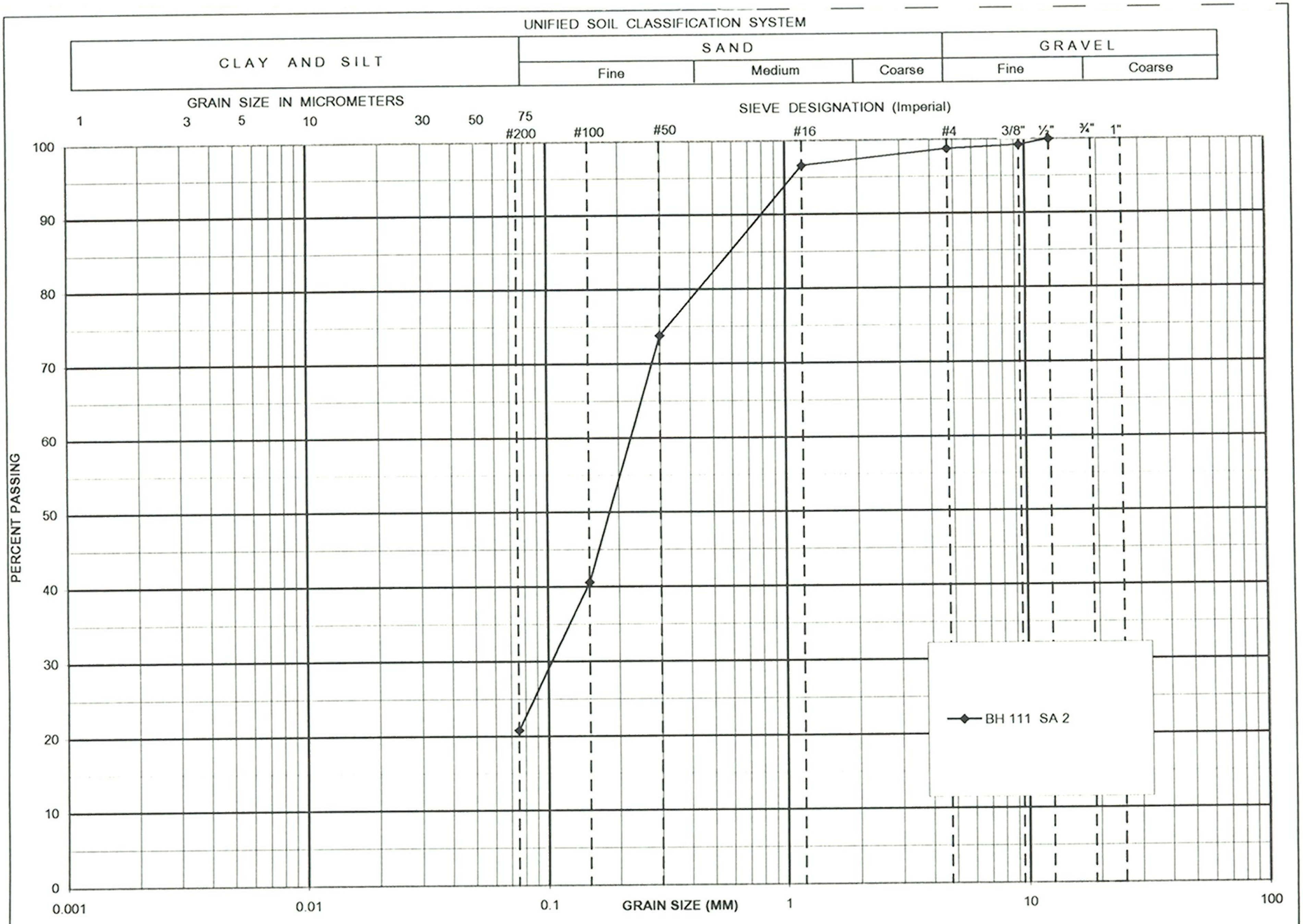
GEO - CANADA	GRAIN SIZE DISTRIBUTION GRAVELLY SAND TILL AND GRAVELLY SAND	FIGURE No.	5
		REF. No.	G-20.0702A
		DATE	February, 2001



GEO - CANADA	GRAIN SIZE DISTRIBUTION SAND AND GRAVEL	FIGURE No.	6
		REF. No.	G-20.0702A
		DATE	February, 2001



GEO - CANADA	GRAIN SIZE DISTRIBUTION CLAYEY SILT TILL	FIGURE No. 7 REF. No. G-20.0702A DATE February, 2001



GEO - CANADA	GRAIN SIZE DISTRIBUTION FILL	FIGURE No.	8
		REF. No.	G-20.0702A
		DATE	February, 2001

APPENDIX 'A'

EXPLANATION OF TERMS USED IN REPORT

N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS $\bar{N}$.

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

c_u (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	>200
	VERY SOFT	*SOFT*	*FIRM*	*STIFF*	*VERY STIFF*	*HARD*

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 - 5	5 - 10	10 - 30	30 - 50	>50
	VERY LOOSE	*LOOSE*	*COMPACT*	*DENSE*	*VERY DENSE*

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND / OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (R Q D), FOR MODIFIED RECOVERY, IS:

R Q D (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	*POOR*	*FAIR*	*GOOD*	*EXCELLENT*

JOINTING AND BEDDING:

SPACING	50mm	50 - 300mm	0.3m - 1m	1m - 3m	>3m
JOINTING	*VERY CLOSE*	*CLOSE*	*MOD. CLOSE*	*WIDE*	*VERY WIDE*
BEDDING	*VERY THIN*	*THIN*	*MEDIUM*	*THICK*	*VERY THICK*

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T W ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T W ADVANCED MANUALLY
T W	THINWALL OPEN	F S	FOIL SAMPLE

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY $= \frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES
ρ_w	kg/m^3	DENSITY OF WATER
γ_w	kN/m^3	UNIT WEIGHT OF WATER
ρ	kg/m^3	DENSITY OF SOIL
γ	kN/m^3	UNIT WEIGHT OF SOIL
ρ_d	kg/m^3	DENSITY OF DRY SOIL
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL
e	1, %	VOID RATIO
n	1, %	POROSITY
w	1, %	WATER CONTENT
S_r	%	DEGREE OF SATURATION
w_L	%	LIQUID LIMIT
w_p	%	PLASTIC LIMIT
w_s	%	SHRINKAGE LIMIT
I_p	%	PLASTICITY INDEX $= w_L - w_p$
I_L	1	LIQUIDITY INDEX $= \frac{w - w_p}{I_p}$
I_C	1	CONSISTENCY INDEX $= \frac{w_L - w}{I_p}$
e_{max}	1, %	VOID RATIO IN LOOSEST STATE
e_{min}	1, %	VOID RATIO IN DENSEST STATE
I_D	1	DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$
D	mm	GRAIN DIAMETER
D_n	mm	n PERCENT - DIAMETER
C_u	1	UNIFORMITY COEFFICIENT
h	m	HYDRAULIC HEAD OR POTENTIAL
q	m^3/s	RATE OF DISCHARGE
v	m/s	DISCHARGE VELOCITY
i	1	HYDRAULIC GRADIENT
k	m/s	HYDRAULIC CONDUCTIVITY
j	kN/m^3	SEEPAGE FORCE

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FOUNDATION DESIGN REPORT
HWY 404/ELGIN MILLS ROAD INTERCHANGE
THE REGIONAL MUNICIPALITY OF YORK

Ref. No. G-20.0702A
September 2001

Prepared for:

The Regional Municipality of York
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FIGURES

APPENDIX

6.0 DISCUSSION AND RECOMMENDATIONS

6.1 Bridge Foundations

The existing Hwy 404 underpass at Elgin Mills Road consists of a two (2) lane bridge structure, constructed in 2 spans and bearing on spread footings. This bridge will be replaced with a new structure having 5 lanes, a ramp lane and sidewalks on each side.

We understand that the new bridge structure over Hwy 404 will be constructed in two stages. First, a structure to carry the future westbound traffic on Elgin Mills Road located north of the existing structure will be constructed. This will be followed by the demolition of the existing structure and its replacement with a new bridge to carry the future eastbound traffic. We further understand that the new bridge grade will be about 1.8 m higher than the existing grade.

Boreholes 101 through 106 were drilled for the proposed bridge, which will be a two-span structure with precast concrete girders and cast-in-place deck. The centre pier will be located within the median of Hwy 404.

Based on drawing Sheet 299, Contract No.73-45, WP No.160-74-28, dated August 1977, the centre piers of the existing bridge are founded on a spread footing approximately 11 m long by 4.6 m wide. The above mentioned drawing also shows that the abutments are also founded on spread footings, approximately 13.4m long by 3.8m wide.

6.1.1 Abutments

6.1.1.1 Shallow Spread Footings on Native Soils

The abutments of the structure can be supported on shallow spread footings on competent native soils. For the footings, the following geotechnical resistances, as calculated by static analysis, are recommended:

Table 6.1.1.1.1

Foundation Element		Highest Foundation Elevation (m)	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Resistance at SLS (kPa)	Soil Type
West Abutment	South Side BH 31 & 101 (east bound bridge)	229.8	650	400	Sandy Silt Till/ Silty Sand
		229.0	1000	600	Sandy Silt Till
	North Side - BH 102 (west bound bridge)	230.5	1000	600	Sandy Silt Till
East Abutment	South Side - BH 105 (east bound bridge)	230.6	800	500	Sandy Silt
	North Side - BH 106 (west bound bridge)	231.3	600	400	Sand & Silt Till
		230.6	1000	600	Sand & Silt Till

For the calculation of the factored geotechnical resistances at ULS a bearing resistance factor of 0.5 was applied to the ultimate geotechnical resistance, in accordance with Clause 6-6.2.2 of the O.H.B.D.C.

The maximum total and differential settlements at the above S.L.S. values are estimated to be less than 25 mm and 20 mm respectively. The above bearing values should be reduced for inclined or eccentrically loaded footings in accordance with Sections 6-8.4.2 and 6-8.5.3 of the Ontario Highway Bridge Design Code, 3rd Edition. All footing bases should be carefully cleaned by hand and inspected by qualified geotechnical personnel before placing the concrete. Allowance should be made to place a 100 to 150 mm thick lean concrete mudmat in all footing excavations to minimize disturbance. Following the construction of the footings, backfill should be placed to a sufficient height above the footing (i.e. at least 1.2 m) to prevent disturbance and frost penetration.

The unfactored horizontal resistance against sliding between concrete and approved till surface can be calculated using a friction angle of 29 degrees. Additional resistance can be obtained from the horizontal passive resistance of the undisturbed soil in front of the foundation element, assuming an ultimate K_p value of 3.5. In the calculation of the available passive resistance, the resistance offered by the upper 1.2 m of soil (i.e. within the frost zone) should not be included. To the ultimate passive resistance, a resistance factor of 0.5 should be applied.

All footings should be provided with a minimum of 1.2 m of earth cover or equivalent insulation for frost protection.

6.1.1.2 Footing for Perched Abutments on Engineered Fill

Perched abutments can be supported on spread footings bearing on a compacted granular core, the base of which is established on undisturbed, native, competent soils at or below the elevations shown in the table below.

Table 6.1.1.2.1

General Area	Borehole No.	Existing Ground Elevation (m)	Recommended Stripping Depth (m)	Recommended Stripping Elevation (m)	Soil Type
West Abutment East Bound Bridge	31 101	231.3 231.6	1.2 1.1	230.1 230.5	Sandy Silt Till Silty Sand
West Abutment West Bound Bridge	102	232.0	1.3	230.7	Sandy Silt Till
East Abutment East Bound Bridge	105	232.1	1.0	231.1	Sandy Silt
East Abutment West Bound Bridge	106	232.9	1.1	231.8	Silty Sand to Sandy Silt

The granular core must be placed directly on sufficiently competent, undisturbed native soils. The minimum thickness of the Granular A pad below the footing should be 1.5 m below. A geotechnical resistance at S.L.S. of 350 kPa and a factored geotechnical resistance at U.L.S. of 850 kPa may be used for design purposes. Details of the granular core are shown on Figure 9. The degree of compaction of the granular pad shall be 100% of the standard Proctor maximum dry density (SPMDD), and the material must be placed at a moisture content within ± 2% of optimum. Lift thicknesses must not exceed 200 mm in loose height. The granular pad shall be constructed using

OPSS Granular A material. The unfactored horizontal resistance against sliding between the foundation and the properly compacted granular pad can be calculated using an angle of friction of 35 degrees. For frost protection, the footings should have a permanent earth cover of at least 1.2 m.

6.1.1.3 Deep Foundations for Perched Abutments

Even though the subsurface conditions do not warrant the use of deep foundations, the abutments could be perched within the approach fills and supported on end bearing steel H-piles driven into the very dense native soil strata, particularly if an 'integral abutment' type bridge is to be constructed.

Due to the anticipated very hard driving conditions and the presence of cobbles and boulders, the use of a heavy section, such as HP 310x110 with reinforced tips, is recommended in order to achieve sufficient penetration into the very dense till. Oversize materials (e.g. greater than 75 mm nominal diameter) should not be used in the embankment fills through which piles would be driven. The following axial pile resistances are recommended:

Pile Section	**Axial Pile Resistance**		**Maximum Driving Energy kJ.**	**Recommended Pile Tip Elevation (m)**
	Factored U.L.S. (kN)	**S.L.S. (kN)**		
310 HP 110	1650	1175	70	at or below elev. 227.0

The minimum geotechnical pile length beneath the pile cap and/or the highway grade should be 8.0 m. The minimum permissible pile length should also be discussed with the Structural Engineer.

The factored horizontal resistance of the above pile section can be taken to be 120 kN at U.L.S. and 50 kN at S.L.S.

The piles should be driven with a hammer capable of delivering a minimum energy of 48,000 Joules per blow. The maximum driving energy, however, should not exceed 70,000 Joules, as indicated in the above table.

Due to the anticipated very hard driving conditions and the presence of cobbles and boulders, the pile sections should be equipped with standard MTO reinforced tips (welded flange plates as per OPSD 3301.00). Pre-augering of a portion of the pile trajectories should be included as a unit price contingency item in the bridge contract. The pile tips should be at or below El. 227.0 m.

The piles should be driven to the elevation recommended in the previous paragraphs and thereafter, in accordance with the standard MTO practice driving should be controlled using the Hiley Formula. The estimated ultimate resistance of the piles by the Hiley Formula can be calculated by dividing the recommended axial resistance at ULS by a resistance factor of 0.5 as per current MTO practice. With this criterion, the estimated ultimate axial resistance as per Hiley Formula is approximately 3300 kN (i.e. 1650 ÷ 0.5 = 3300 kN).

All pile driving should be carried out in accordance with SP 903S01. We do not anticipate structural/foundation damage to the existing structure due to the vibrations caused by pile driving for the new structure. We recommend, however, that a vibration specialist and/or a pile driving contractor be consulted in this matter. In this regard, conducting vibration monitoring will likely be necessary.

For frost protection, all pile caps should have a permanent earth cover of at least 1.2 m.

In case of an "Integral Abutment Bridge" design, the top 3.0 m of the pile, below the underside of the pile cap, should be installed into two (2) pre-drilled, 600 and 800 mm inside diameter concentric corrugated steel pipes (CSP), leaving an annular space between the pipes. Details of the installation are shown on Figure 10. The gradation of the sand used in the pre-augered holes should be as follows:

MTO Sieve Designation		Percentage Passing by Mass
2mm	# 10	100%
600μm	# 30	80% - 100%
425μm	# 40	40% - 80%
250μm	# 60	5% - 25%
150μm	# 100	0% - 6%

Although the design shown on Figure 10 is used mostly with retained soil systems, it is also recommended for conventional integral abutments because this design will ensure the long term flexibility of the foundations.

6.2 Lateral Pile Capacity

Spring constants for the evaluation of lateral pile deflections for steel piles subjected to horizontal loading and spaced no closer than 1.5 m on centre, may be calculated by multiplying the following coefficient of horizontal subgrade reaction (K_s) values by the spring influence length and the pile width i.e.;

Spring constant = (K_s) b L

where:

K_s = coefficient of horizontal subgrade reaction [MN/m^3]

b = pile width [m]

L = influence length of spring element [m]

z = depth below ground surface in meters

Please note that the K_s values are assumed to increase linearly with depth.

Area	Material Type	Applicable Depth Limits (m) below underside pile cap	K_s (MN/m^3)
Abutments	Loose Sand Fill	0-3m	0
	Compact to dense Silty Sand, Sandy Silt and Tills	>3m	15z
Piers	Very dense Silty Sand Till, Sandy Silt Till and Hard Clayey Silt Till	0 - 9	35z

In no case should the normal contact pressure at the pile-soil interface at the factored ULS condition exceed p_{max}, where:

$p_{max} = 3\ K_p\ \gamma'\ z\ f_r$

$K_p = 3.69$ for native soils

$\gamma' = 11$ kN/m^3

z = depth of pile penetration below underside of pile cap

f_r = resistance factor = 0.5

Calculation of pile head deflections (SLS analysis) can be assessed using finite-difference computer analysis methods.

6.2.1 Central Pier Foundations

Boreholes 103 and 104 indicate that the central piers can be supported on normal spread footing foundations. The footings should be placed on competent (very dense) glacial till at or below the elevations shown in the following table:

Table 6.2.1.1

Foundation Elevation/ BH No.	Existing Ground Surface Elevation (m)	Recommended Highest Foundation (Base) Depth Below Existing Ground Surface (m)	Recommended Highest Foundation (Base) Elevation (m)	Factored Geotechnical Resistance at ULS* (kPa)	Geotechnical Resistance at SLS (kPa)	Soil Type
Central Pier East Bound Bridge BH 103	230.4	1.4	229.0	1000	600	Silty Sand Till
Central Pier West Bound Bridge BH 104	230.5	1.5	229.0	1000	600	Silty Sand Till

* Incorporating a resistance factor of 0.5 as per OHBDC.

The serviceability condition is based on the premise that the maximum total and differential settlements are 25 mm and 20 mm respectively. This can be achieved provided that the founding subgrade is undisturbed during the construction.

Under inclined loading conditions, the bearing resistance at ULS should be reduced in accordance with Clause 6-8.4.2 and 6-8.5.3 of OHBDC.

The unfactored resistance against sliding between the concrete and approved till surface can be calculated using a friction angle of 29 degrees.

For frost protection, the footings should have a permanent earth cover of at least 1.2 m or equivalent artificial insulation.

All footing bases should be carefully prepared and then inspected and approved by qualified geotechnical personnel, before paving the concrete. Allowance should be made to place a 100 to 150 mm thick lean concrete mudmat on all footing excavations to minimize disturbance. Following the construction of the footings, backfill should be placed to a sufficient height above the footing (i.e. at least 1.2 m) to prevent disturbance and frost penetration.

6.3 Horizontal Earth Pressures

Backfill behind the abutments and retaining walls should consist of non frost susceptible, free draining granular materials in accordance with the Ontario Ministry of Transportation Standards.

Provided that the backfill placed behind the abutments and any retaining walls consists of free draining OPSS Granular A or B material constructed in accordance with OPSD 3501.00, and adequate provisions are made for an appropriate drainage scheme, the following geotechnical parameters may be used for computation of earth pressures:

Parameter	Backfill Material	
	Granular A	**Granular B**
Unit weight γ, (kN/m^3)	22	21
Angle of internal friction, ϕ (degrees)	35°	32°
Coefficient of active earth pressure (K_a)	0.27	0.31
Coefficient of earth pressure at rest (K_o)	0.43	0.47
Coefficient of passive earth pressure (K_p)	3.69	3.25

The unfactored horizontal earth pressure "p" acting at a depth "z" below a nearly horizontal finished grade can be calculated as:

$p = K(\gamma z + q)$ (kN/m^2)

where

K = (K_a) for unrestrained wall (e.g. cantilevered with no diaphragm)

K = (K_o) for restrained wall

q = unit surcharge pressure (kN/m^2)

The earth pressure coefficient adopted will depend on whether the retaining structure is restrained or movements can be allowed such that the active state of earth pressure can develop. If the abutment is restrained and does not allow lateral yielding, then at rest, pressures should be used as per Clause C6-7.1 of the OHBDC, 3rd Edition. The effect of compaction should also be taken into consideration in the selection of the appropriate earth pressure coefficients in accordance with Clause 6-7.4.3 of OHBDC.

Vibratory equipment for use behind abutments and retaining walls should be restricted on size as per current MTO practice.

As an alternative to conventional retaining wall, MTO's Retained Soil System may be used.

The following should be included in the Contract Documents:

- identify longitudinal extent in plan of the Retained Soil System;
- identify in plan transverse space constraints (top of wall and bottom of wall);
- identify elevation of top of wall and bottom of wall;
- include NSSP for Retained Soil Systems in contract documents.

The Retained Soil System should be of high performance and high appearance.

6.4 Approach Embankments

Considering the competent nature of the subsoils, no deep seated rotational or translational types of slope instability are anticipated under the proposed approach fill heights of 8 m or less for the bridge structure. The embankments can, therefore, be constructed with 2H:1V side and end slopes. Should the height of the embankments exceed 8 m, a 2 m wide berm should be constructed at mid height of the embankment. The slopes of the embankments should be provided with erosion protection against surface water runoff.

Before placing the new fills, all topsoil and deleterious organic matter should be removed from the surface within the fill area, and the subgrade should be proof-rolled. Any loose/soft areas detected during proof-rolling should be subexcavated and replaced with fill materials that are consistent with the local soils and compacted to a minimum 98% SPMDD.

The approach embankments for the proposed new bridge should be constructed using inorganic fill materials, which should be placed in maximum 200 mm lifts and compacted to a minimum 95% SPMDD at a placement moisture content within ±2% of optimum. The degree of compaction within the top 0.5 m of the fill (i.e. the subgrade immediately beneath the granular base) should be increased to 98%.

Provided that all organic and otherwise unsuitable materials are removed within an envelope given by an imaginary slope not steeper than 1:1 from the toe of the proposed embankment, and the subgrade is properly compacted from the surface as detailed above, the settlement of foundation materials (i.e. not including the settlement of the embankment material under its own weight) should not exceed 15 mm. The settlement of the embankment fill under its own weight can be expected to be about 45 mm for an embankment of 8 m in height. Such settlements are considered acceptable.

Proper benching should be applied to the existing embankment as per MTO procedures and OPSD 208.01.

Proper erosion control measures should be implemented both during the construction and permanently. This can be achieved by immediate seeding and sodding.

Groundwater level was recorded more than 1.5 m below the ground surface and, therefore, we do not anticipate major problems due to groundwater seepage during stripping of the subgrade and backfilling for the construction of the embankment fills.

6.5 Temporary Grade Separation

In order to facilitate demolition of the existing bridge structure, it is anticipated that the approach fill behind the existing abutment will be removed to expose the abutments. This may necessitate the use of a temporary grade separation structure. This structure could be constructed of mechanically stabilized earth (such as a wrapped-face geogrid system or equivalent) or, alternatively, conventional shoring methods could be used.

If utilized, temporary shoring should be designed in accordance with the requirements of the 3rd Edition of the Canadian Foundation Engineering Manual. It is anticipated that a soldier pile and lagging wall could be constructed for this purpose. A rectangular shaped earth pressure envelope can be assumed behind this wall. The unfactored horizontal earth pressure "p" acting at a depth "z" (in metres) below a nearly horizontal grade can be calculated as:

$$p = 4.4z \ (kN/m^2)$$

Surcharge loading (where applicable) should be added to the above pressure distribution. All excavations, shoring and backfilling should be carried out in conformance with the Safety regulations of the Province, as well as the following specifications:

SP 539S01 - Protection Schemes

SP 902S01 - Excavation and Backfilling to Structures

The main soil deposits encountered in the boreholes can be classified as :

Fill (above water table): Type 3

Silt and sand till, silty sand till, sandy silt till (above water table): Type 2

Silty sand (above water table): Type 3

Silty clay, clayey silt (above water table): Type 2

Gravelly sand (above water table): Type 3

The shoring should be designed so that the lateral movement of any portion of the roadway protection system will not exceed the established criterion for the structure performance level. In this case, the Performance Level should be 2.

6.6 Culvert Foundations

6.6.1 Culvert Under Elgin Mills Road

A new rigid framed, open bottom, single cell culvert is proposed under Elgin Mills Road at the Rouge River crossing, at Sta 4+450 ±.

The new culvert will be located near the site of the existing CSP. The foundation grade of the new culvert is proposed to be at El. 221.8 ± m (i.e. 1.6 m below the existing stream bed of the Rouge River).

Boreholes 111 and 112, put down in the vicinity of the south and north ends of the existing culvert respectively, encountered very dense silt and sand till at the proposed foundation level. Footings founded on this stratum can be designed for a factored geotechnical resistance of 600 kPa at U.L.S. and 400 kPa at S.L.S. An on-site inspection of the foundation grade must be performed to confirm the foregoing geotechnical resistances of the soils. Refer to Paragraph 6.1.1.1. for the horizontal shear resistance at the interface between the foundation and soil.

The silt and sand till at the proposed foundation level is prone of softening (dilation) when exposed to weathering and machinery traffic. As such, it is recommended that once the bearing soils have been approved, a 150 mm thick 'mud slab' of 5 MPa concrete be immediately placed to protect the footing bases.

6.6.2 Culvert Under Hwy 404

The existing concrete box culvert under Hwy 404 at approximate Sta 20+709 will be extended to below the N-EW and E-N ramps. The invert elevation of the existing culvert is approximate El. 229.9 m. Boreholes 116 and 117 drilled at the west and east sides of the existing culvert encountered firm to very stiff clayey silt to silty clay and compact sandy silt to silty sand till respectively at the existing invert level. These strata should provide adequate support for the extensions of the culvert.

Footings founded on undisturbed native soils at or below El. 229.6 m can be designed for a factored geotechnical resistance of 300 kPa at U.L.S. and 200 kPa at S.L.S. The foundation grades should be inspected by a qualified geotechnical engineer to confirm the foregoing geotechnical resistances.

6.6.3 General Notes

The culverts should be designed to withstand the appropriate weight of fill and traffic loadings, and frost pressures (where adequate frost cover is not provided). Provided that the culverts are founded on a properly prepared subgrade, the maximum total settlement at the S.L.S. values given in Sections 6.5.1 and 6.5.2 is estimated to be 25 mm. Granular backfill beside and above the box culverts should conform to OPSD 803.02.

Depending on the proposed gradient of the culverts and in view of the anticipated 25 mm of settlement, camber may be required, especially for the culvert under Hwy 404. Depending on the length of the proposed structures, water tight joint treatment may be required within the new sections, as well as at the interface of the existing and new sections.

For frost protection, the footings should have a permanent earth cover of at least 1.2 m or a box culvert may be structurally designed to resist frost forces. The depth of foundations, including those for cut-off walls and retaining walls, should be determined on the basis of scour depths, whichever is greater. In computing frost protection, only half the thickness of rip-rap should be provided.

As an alternative to conventional retaining walls, consideration could be given to MTO's Retained Soil System in which case the designer will have to include the geometric, performance and appearance requirements.

Erosion protection should be provided at the culvert inlet (including the sides) and outlet, especially if head and wing walls are not incorporated. Where clay seal will be used, this should be at least 0.6 m thick. The embankment slope around the culvert inlet may need to be protected against erosion by means of a properly designed work blanket. A filter blanket at the outlet will also be required. In case of an open-invert structure, proper erosion measures should be taken inside the culvert.

6.7 Retaining Wall Between Approximate Sta 10+400 and 10+500, Ramp N-EW

Due to the proximity of the Rouge River to a portion of the N-EW ramp, a mechanically stabilized earth retaining wall is proposed at the above-captioned section. Design details of the wall are not available at this time, however, in accordance with the longitudinal profile provided to us, the maximum height of the wall will not exceed 4.5 ± m.

Based on the results of Boreholes 113, 114 and 115, the base of the retaining wall can be founded on the sandy silt, sand and silt and silty sand to sandy silt at a depth of approximately 1.2 m below existing grade, with geotechnical resistances of up to 200 kPa at SLS and up to 300 kPa at factored ULS. Higher geotechnical resistances (up to 350 kPa and 550 kPa at SLS and factored ULS respectively) can be achieved at a depth of approximately 1.5 m below grade (i.e. at El. 227.9 m, 227.4 m and 225.6 m at Borehole Locations 113, 114 and 115 respectively).

Lateral earth pressures and materials to be used behind the retaining structures were given in Section 6.3 of this report under the heading 'Horizontal Earth Pressures'.

The expected settlements due to the stress increase on the foundation soils at the above SLS resistances should be less than 25 mm. Settlement of the embankment itself will depend on the material type, but for preliminary design purposes, a value of 0.6% of the embankment fill height can be assumed.

The materials to be used for the construction of the reinforced earth retaining wall are unknown at this time. Generally, the use of high internal friction granular material will result in shorter internal reinforcement elements.

If the high water of the Rouge River rises to above the footing elevations, then depending on the configuration of the road and geometry, the use of subdrains may be desirable at foundation level behind the retaining wall. The subdrain should consist of 100 mm diameter perforated pipe placed in 400 mm wide trenches, the invert of which extends 450 mm below the granular sub-base. The pipe should be surrounded and the trench backfilled with 19 mm clear stone. The stone should be completely wrapped in a non-woven filter fabric material.

The effective filtration opening size (F.O.S.) of the filter fabric should be 120 microns or less.

6.8 Frost Protection

Design frost protection for the general area is 1.2 m. Therefore, a permanent soil cover of 1.2 m of its thermal equivalent of artificial insulation is required for frost protection of foundations.

6.9 Construction Conditions

Excavations for the foundation units will be generally shallow and, due to the competent nature of the soil types underlying the site, we do not anticipate major or unusual construction problems. Temporary excavations above the groundwater table should be stable at 1:1 slopes. Dewatering will be needed for the construction of spread footings. This can be achieved by gravity drainage into perimeter ditches and pumping from filtered sumps, if the water level is less than 0.5 m above the foundation grade at the time of construction. Otherwise, well points will be required.

The rate of groundwater flow can be computed by assuming the following coefficients of permeability:

Soil Type	Coefficient of Permeability k (cm/sec)
Silty Sand, Silty Sand Till	10^{-4}
Sandy Silt, Sand and Silt	10^{-5}
Gravelly Sand	10^{-2} to 10^{-3}

Effective surface water diversion works should be provided to keep out surface water from the excavations during construction of spread footings, especially for the construction of the culverts.

7.0 CLOSURE

The Limitations of Report, as quoted in Appendix 'B', is an integral part of this report.

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PROVINCE OF ONTARIO

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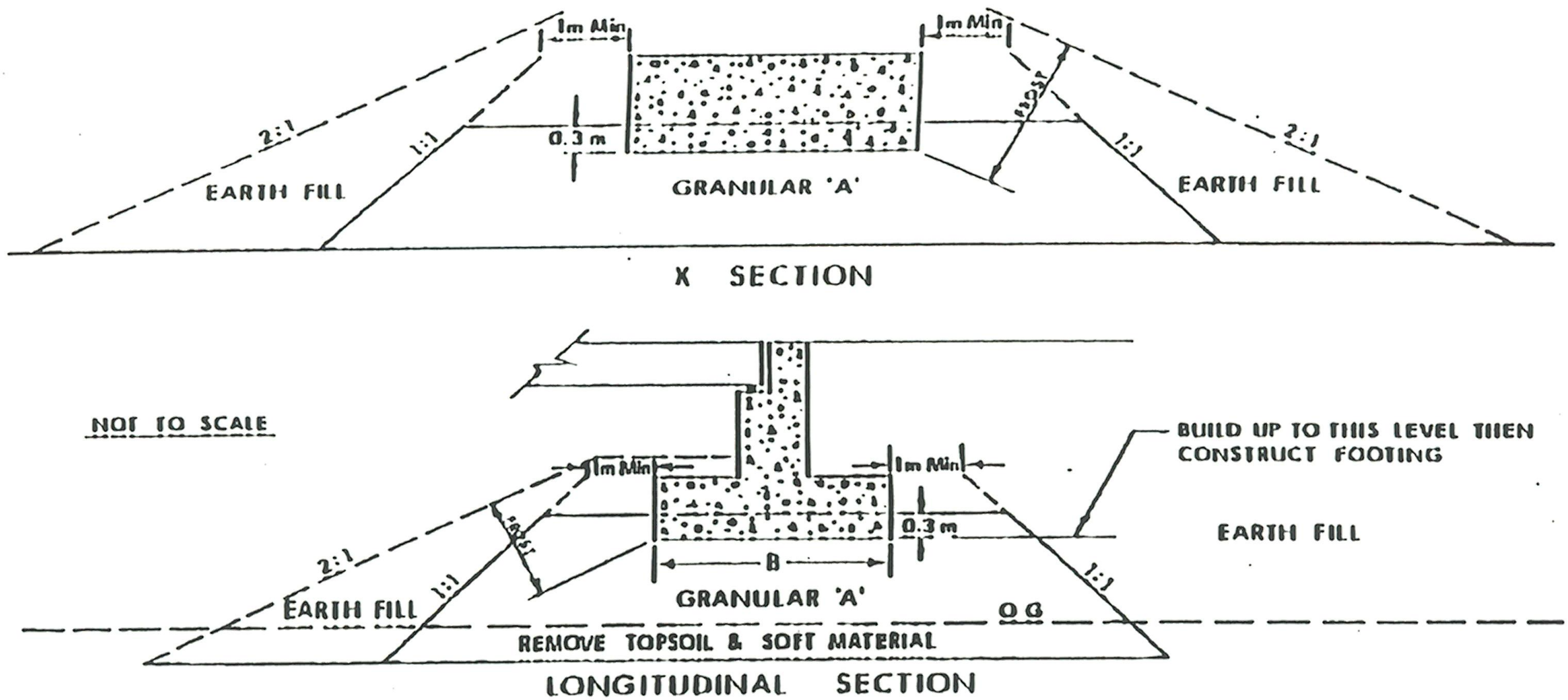
Encl.

D22-reports/G-20 0702A RVA

FIGURES



NOTES:

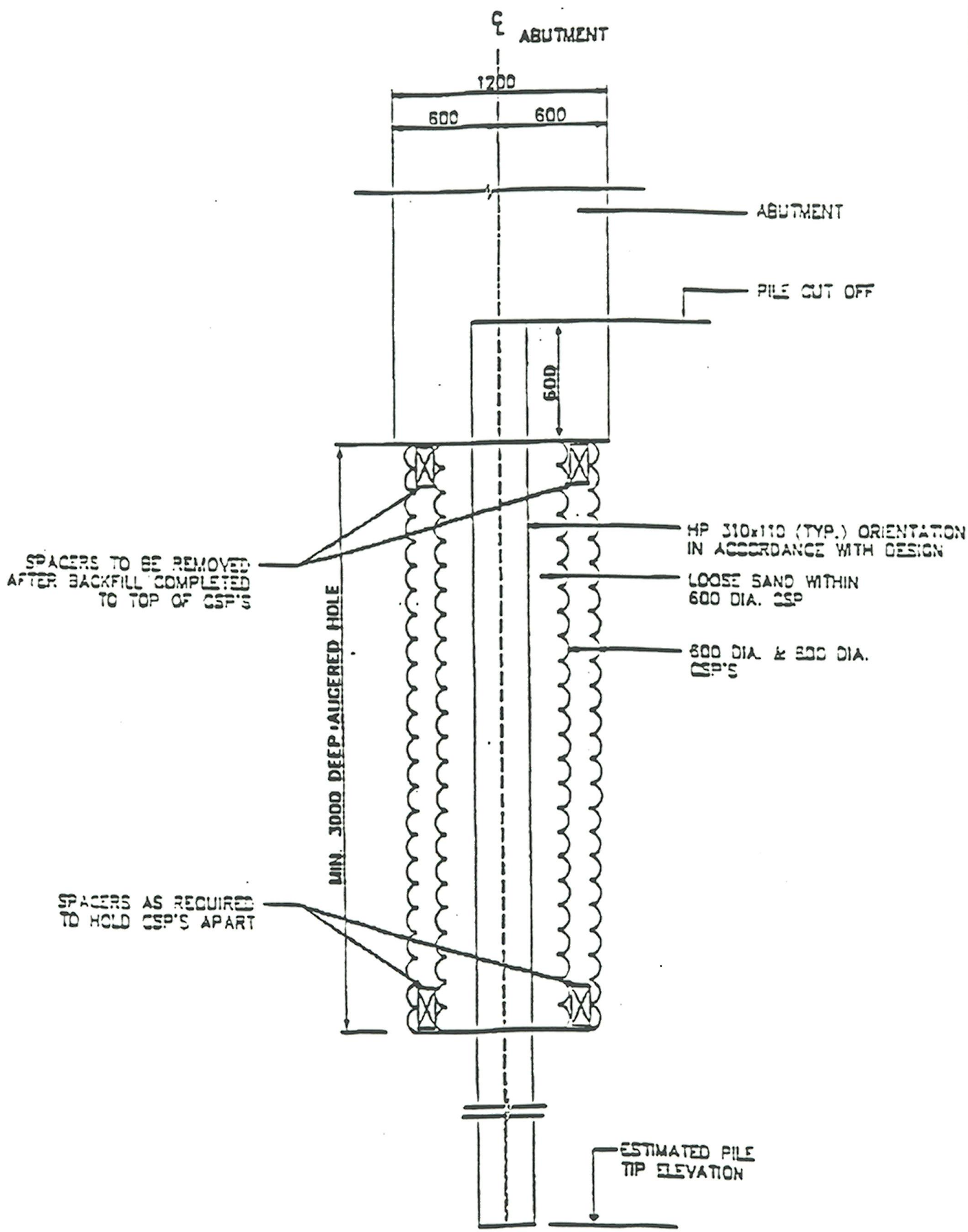
1 - REMOVE TOPSOIL &/OR SOFT SUBSOIL UNDER AREA OF COMPACTED GRANULAR 'A' & EARTH FILL.

2 - PLACE GRANULAR 'A' & EARTH FILL TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO CURRENT M T O STANDARDS.

3 - CONSTRUCT CONCRETE FOOTING.

4 - PLACE REMAINDER OF GRANULAR 'A' & EARTH FILL AS REQUIRED.

Ministry of Transportation Ontario	ABUTMENT ON COMPACTED FILL SHOWING GRANULAR 'A' CORE	G-20.0702A FIGURE No. 9



INTEGRAL ABUTMENT CSP DETAIL	FIG. No. 10 G-20.0702A

APPENDIX ‘B’

LIMITATIONS OF REPORT

The conclusions and recommendations given in this report are based on information determined at the borehole locations. The information contained herein in no way reflects on the environmental aspects of the project, unless otherwise stated. Subsurface and groundwater conditions between and beyond the boreholes may differ from those encountered at the testhole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the borehole locations and may not be suitable for other purposes, such as grading, excavating, planning, development, etc.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report.

The comments made in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of boreholes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusions as to how the subsurface conditions may affect their work. This work has been undertaken in accordance with normally accepted geotechnical engineering practices.

Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. Shaheen & Peaker Limited accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.