GEOCRES No:
31E-239

PRELIMINARY FOUNDATION INVESTIGATION REPORT
PROPOSED SUNSET PASS DRIVE/REGIONAL ROAD 592 BRIDGE
OVER HIGHWAY 11
KATRINE, ONTARIO
W.P. 314-99-00

Prepared For:

STANTEC CONSULTING LTD.

Prepared by:

SHAHEEN & PEAKER LIMITED

Project: SPT1010D
December 3, 2001

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1. INTRODUCTION

Shaheen & Peaker Limited (S&P) was retained by Stantec Consultants Limited to conduct a preliminary foundation investigation for a proposed bridge (overpass structure) that will carry the realigned Sunset Pass Drive and Regional Road 592 over Highway 11. The site is located in Katrine, Ontario, approximately 60 m south of the intersection of the existing Highway 11 with Sunset Pass Drive and Regional Road 592.

The purpose of the investigation was to obtain preliminary information at the site by means of a limited number of boreholes.

The findings of the investigation are presented in this report.

2. SITE DESCRIPTION AND GEOLOGY

The site is located on existing Highway 11 approximately 60 m south of the intersection of the Highway with Sunset Pass Drive and Regional Road 592. Along this stretch, the Highway is an undivided 2-lane roadway with turning lanes. At the proposed bridge site the roadway appears to have been built up with fill; especially the western portion (i.e. the south bound lane) where the presence of several metres of rock fill was noted under the west shoulder.

The existing ground elevation at the proposed bridge site is approximately 321 m. To the north of the site along Highway 11, the grade falls by about 11 m to Elevation 310 m, some 0.5 km away towards a gorge which is about 200 m wide and 6 to 12 m deep. Further north from the gorge, the grade continues to fall towards Magnetawan River, some 0.7 km. further away. The ground elevation near the shores of the River is about 295 m while the River bottom is about 291 m. Further north the grade rises mildly.

To the south of the site, along and in the vicinity of existing Highway 11, the grade rises by about 24 to 35 m to between Elevation 356 and 345 m, some 0.7 km. away, at a rock outcrop that was previously cut to accommodate the existing Highway 11.

Available geological information indicates that the site is located within an area of ice-contact sediments. After the last glacial withdrawal, ice-contact sediments of sand and gravel, followed by glacio-fluvial sediments of deltaic and nearshore sands and gravel, as well as lake bottom silts and clays, were deposited on top of the existing sandy glacial till or directly on the Precambrian bedrock. The area was then inundated by the glacial lake Algonquin, depositing sands, silts and clays in low-lying areas. The bedrock underlying the general area is known to consist of Precambrian (igeneous) gneiss formations encountered at depths ranging from the ground surface to more than 50 m.

3. INVESTIGATION PROCEDURES

The fieldwork for this project was performed during the period April 19 through June 07, 2001 and consisted of three deep boreholes (23.4 m to 42.6 m deep) and two shallow boreholes (8.1 m and 9.6 m deep). The plan locations of the boreholes (numbered RA1 through RA5), along with their inferred stratigraphic profile, are shown on Drawing No. 1.

The boreholes were advanced using a track mounted drilling rig owned and operated by Groundworks Drilling Inc., under the full time supervision of geotechnical personnel from S&P. One shallow borehole (RA1) was extended using solid stem continuous flight augers while the other (RA5) required the use of hollow stem augers as well, because of frequent cave-ins in the cohesionless sand and gravel encountered. The deep boreholes (RA2, RA3 and RA4) also encountered cohesionless soils. This, along with their depth requirements necessitated the employment of casing and washboring methods when the boreholes were advanced below groundwater. Heavy drilling mud was added to counterbalance any hydrostatic head thereby preventing quick conditions and sand back up during and immediately after sampling.

Sampling in the boreholes was conducted at frequent intervals of depth by the Standard Penetration Test method (SPT), as specified in ASTM Method D1586. This consists of freely dropping a 63.5 kg hammer a vertical distance of 0.76 m to drive a 51 mm diameter O.D. split barrel (split spoon) sampler into the ground. The number of blows required to drive the sampler a vertical distance of 0.30 m into the relatively undisturbed ground is recorded as the Standard Penetration Resistance or the N-value of the soil. This value gives an indication of the consistency or compactness of the soil deposit.

Where the consistency of the soil permitted, the undrained shear strength of the soil was measured in-situ by means of field vane tests, using an MTO type field vane and a relatively undisturbed sample was taken by means of a thin walled Shelby tube sampler.

In addition, dynamic cone penetration tests (DCPT) were performed within and/or below the deep boreholes RA3 and RA4. This test consists of continuously driving a 50 mm diameter cone having a 60 degree shaped point into the undisturbed ground using a driving energy of 475J per blow (63.5 kg hammer dropping freely a distance of 0.76 m), similar to the SPT. The number of blows for each 0.3 m of penetration is recorded and this provides an indication of the relative changes in the soil density with depth.

Due to the presence of cobbles and boulders, coring became necessary in Borehole RA3 at depths of approximately 28.9 to 29.4 m and 29.7 to 30.2 m and also in Borehole RA4 from 14.4 to 14.8 m below the ground surface.

Water level observations in the open boreholes were made during the drilling and after the completion of each borehole. A piezometer was also installed in Borehole RA4 thereby enabling us to monitor the groundwater level over a prolonged period of time without interference from surface water. The water level in this piezometer was monitored during a subsequent site visit.

All five boreholes were backfilled to about 8 m below the ground surface with auger cuttings and the upper 8 m of the open boreholes was then grouted using a cement/bentonite mixture.

The results of drilling, in-situ testing and water level observations are given on the Record of Borehole Sheets in Appendix A.

Laboratory tests, consisting of natural moisture content, bulk unit weight, Atterberg Limits and grain-size analyses, were performed on selected soil samples. The results of these tests are presented on the appropriate Record of Borehole Sheets and also in Appendix B.

The borehole locations were established in the field in relation to the already staked out centre-line of the proposed structure. These locations were submitted to Stantec Consulting Ltd., who provided us with geodetic elevations and coordinates.

4. SUBSURFACE CONDITIONS

Boreholes RA1, RA2, RA4 and RA5 encountered a thin veneer of topsoil, which is underlain by a surficial layer of silt, that extends to depths ranging between 1.4 m (Elevation 319.9 m) and 3.7 m (Elevation 316.9 m) below ground surface. Borehole RA3 was extended from the west shoulder of Regional Road 592 and encountered sand fill (embankment fill) extending to a depth of 2.9 m (Elevation 318.9 m). This fill was further underlain by a 0.8 m thick deposit of sand. The sand in Borehole RA3 and silt in Borehole RA4 are underlain by a localized deposit of silty clay that extended to depths of 7.0 m (Elevation 314.8 m) and 5.9 m (Elevation 315.5 m), respectively. A major deposit of fine sand was encountered underlying the surficial silt layer in Boreholes RA1, RA2 and RA5 and beneath the silty clay stratum in Boreholes RA3 and RA4. The thickness of this fine sand stratum ranged from 0.8 m to 32.1 m and it extended to depths from 3.7 m (Elevation 317.6 m) on the west side of Highway 11 at Borehole RA5 to 35.8 m (Elevation 284.8 m) east of Regional Road 592 in Borehole RA2. The fine sand is in turn underlain by a deposit of sand and gravel containing cobbles and boulders. This stratum was encountered in Boreholes RA2, RA3, RA4 and RA5 where it extended to the full depth of the exploration (except in Borehole RA3, where silt was contacted at 39.1 m depth).

Details of the subsurface conditions encountered in the boreholes are presented on the Record of Borehole Sheets in Appendix A. The conditions are briefly described in the following paragraphs.

4.1. TOPSOIL

Boreholes RA1, RA2, RA4 and RA5 encountered a 150 mm thick veneer of topsoil. It should be pointed out that the thickness of topsoil and organic rich soils could be expected to be variable and probably thicker in the low-lying areas.

4.2. FINE SAND (EMBANKMENT FILL)

Borehole RA3 was extended from the west shoulder of Regional Road 592 where it encountered fill that extended to a depth of 2.9 m (Elevation 318.9 m). The fill consists of fine sand (non-cohesive granular soil).

Standard Penetration Tests performed in this fill deposit gave N-values of 12 to 22 blows/0.3 m. Based on this, the fill is considered to be in a compact condition. The measured natural moisture contents of samples from the fill ranged from 13 to 23%. The presence of embankment fill can also be expected under the existing Highway 11 pavement.

4.3 SURFICIAL SILT

Underlying the topsoil, Boreholes RA1, RA2, RA4 and RA5 contacted a surficial deposit of silt to sandy silt with some silty clay seams extending to depths ranging from 1.4 m to 3.7 m below the ground surface or Elevations 319.9 m to 316.9 m. The grain size distribution curves of samples from this deposit are given in Figure 1 in Appendix B. These show 0% gravel, 14 to 15% sand, 78 to 81% silt and 4 to 8% clay size particles. In general, depending on the clay content, the soil exhibits a non-cohesive structure (fine grained granular soil) with some cohesive zones.

Standard Penetration Tests performed in this deposit gave N-values ranging from 2 to 6 blows/0.3 m in the upper 0.7 metre. Below 0.7 metre the N-values ranged from 12 to 28 blows/0.3 m thereby indicating a very loose upper zone underlain by generally compact material with some stiff cohesive zones. The measured natural moisture contents of samples from the upper 0.7 m ranged from 23 to 32%. Below this depth the measured values were 10 to 26%.

4.4 SILTY CLAY

In Borehole RA3, underlying the fill and a 0.8 m thick sand deposit, a clayey silt to silty clay stratum was contacted at a depth of 3.7 m or Elevation 318.1 m. This stratum was also contacted in Borehole RA4 below the surficial silt deposit at a depth of 3.7 m or Elevation 317.7 m. The clayey silt to silty clay deposit extended to depths of 7.0 m (Elevation 314.8 m) and 5.9 m (Elevation 315.5 m) and ranged from 3.3 m to 2.2 m in thickness at Boreholes RA3 and RA4 respectively. This unit was not encountered further beyond at Boreholes RA2 and RA5. It therefore appears that the thickness of this stratum possibly remains relatively constant between boreholes RA3 and RA4 but decreases further beyond.

Although this deposit contains frequent silt seams (i.e. exhibits a laminated, layered structure), it does not exhibit a true varved clay structure of regular yearly seams/layers of clay and silt and/or fine sand.

Standard Penetration tests conducted in this deposit yielded N-values ranging from 6 to 27 blows/0.3 m and field vane tests gave in-situ undrained shear strength values in excess of 100 kPa. These field tests indicate that the consistency of the material can be described as firm within the upper two-thirds zone and stiff below this upper zone in Borehole RA4, and stiff to very stiff in Borehole RA3. The natural measured moisture contents of samples from this deposit ranged between 24% and 37%.

The results of grain-size distribution analyses carried out on two representative samples are given in Figure 2 in Appendix B. They indicate a particle size distribution of 0% gravel, 3 to 4% sand, 60 to 72% silt and 24 to 37% clay size particles.

The deposit is described as a cohesive material and an Atterberg Limits test performed on a representative sample gave the following index values.

Liquid Limit = 32%
Plastic Limit = 22%
Plasticity Index = 10%
Natural Moisture Content = 36%

As shown in Figure 3 in Appendix B, these values indicate a clayey soil of low plasticity. The fact that the measured natural moisture content in Borehole RA4 is above the measured liquid limit value suggests that the deposit in this borehole is probably relatively weak and compressible.

4.5 FINE SAND

The silt in Boreholes RA1, RA2 and RA5 and the silty clay in Boreholes RA3 and RA4 are underlain by a major deposit of fine sand extending to depths of between 3.7 m (Elevation 317.6 m) in Borehole RA5 and 35.8 m (Elevation 284.8 m) in Borehole RA2. The thickness of the deposit ranges from 0.8 m in Borehole RA4 to 32.1 m in Borehole RA2.

The material is generally in the fine sand range with some silt content. The grain-size distribution envelope for seven representative samples is illustrated in Figure 4 of Appendix B. The results show the following grain size distribution.

Gravel = 0%
Sand = 80 – 96%
Silt = 4 – 20%

It is noted that this deposit contained some gravel in Boreholes RA1 and RA2 at depths of about 5 m to 7.5 m and 6 m to 12 m, respectively.

The recorded N-values in this deposit ranged from 14 to in excess of 100 blows/0.3 m. Based on this, the deposit is considered to have a compact to very dense relative density. The measured natural moisture contents of samples from this deposit ranged between 2% and 23 %.

4.6 SAND AND GRAVEL WITH COBBLES AND BOULDERS

Boreholes RA2, RA3, RA4 and RA5 encountered a sand and gravel deposit at depths ranging from 3.7 m to 35.8 m or Elevations 317.6 m to 284.8 m. In Borehole RA3, this deposit extends to a depth of 39.1 m (Elevation 282.7 m) where it is underlain by a deposit of silt while in Boreholes RA2, RA4 and RA5, it extends to the limits (and possibly deeper) of the boreholes whose depths varied from 9.6 m (Elevation 311.7 m) to 42.6 m (Elevation 278.0 m). Cobbles and boulders were encountered in this deposit at Boreholes RA3 and RA4 and coring was required in order to advance the boreholes through them.

The grain size distribution curves of two samples from this deposit are given in Figure 5 in Appendix B. These show 35 to 80% gravel, 19 to 61% sand and 1 to 4% silt and clay size particles.

Because of the oversized particles, it was not possible to obtain reliable standard penetraton test results in this deposit. However, N-values ranged between 14 and in excess of 100 blows for 0.3 m penetration. In many cases full 0.3 m penetration could not be obtained during SPT counts and these low penetrations could be partly attributed to the presence of oversize materials (i.e. coarse gravel, cobbles and boulders). It is noted that two compact zones of this deposit were encountered in Borehole RA4 at depths of about 8.5 m to 10 m and 15 m to 16 m where the recorded N-values are 14 and 29 respectively. The presence of a compact zone was also inferred in Borehole RA3 between about 30 m and 34 m from the results of dynamic cone penetration tests. Based on the recorded N-values, together with the results of dynamic cone penetration tests performed in Boreholes RA3 and RA4, this deposit is inferred to be in a generally dense to very dense condition with some compact zones.

4.7 LOWER SILT

In Borehole RA3, a silt layer was contacted at a depth of 39.1 m (Elevation 282.7 m). The borehole was extended to a depth of 39.7 m below the ground surface or to Elevation 282.1 m where advancing the casing any further was not feasible (possibly due to the presence of a boulder) and the borehole was terminated.

A Standard Penetration test performed in this deposit yielded an N-value of 60 blows for 5 cm, indicating a very dense condition.

Bedrock was not encountered within the depths drilled. The presence of a large boulder (approximately 1.5 m x 4.0 m in plan) was noted about halfway between Boreholes RA1 and RA2, about 5 m to the south. To get an idea whether this represents a bedrock outcrop a borehole was put down about 3.5 m south of the boulder. This borehole was extended to 3.0 m without encountering refusal (ie. no bedrock was contacted within this depth). This can perhaps be further investigated.

4.8 GROUNDWATER CONDITIONS

Groundwater conditions in the open boreholes were observed while drilling and after completing each borehole. In addition, a piezometer was installed in Borehole RA4 to enable prolonged groundwater level measurements, without interference from surface water. The observations and recorded values are shown on the individual Record of Borehole Sheets.

Boreholes RA1 and RA5 were dry upon their completion to depths of 8.1 m and 9.6 m or Elevations 312.7 m and 311.7 m. The piezometer installed in Borehole RA4 to a depth of 7.6 m (Elevation 313.8 m) was also dry on completion and about six weeks thereafter. In Boreholes RA2 and RA3, however, water levels were measured on completion at 1.5 m (Elevation 319.1 m) and 3.4 m (Elevation 318.4 m). These are believed to represent a perched water condition due to accumulation of surface water between the silty clay seams found in the silt layer in Borehole RA2 and on the practically impervious silty clay deposit encountered in Borehole RA3.

Based on these observations together with the moisture contents of the soil samples, it is our opinion that at the time of our investigation the groundwater level at the site was below a depth of at least 8 m below the ground surface or below about Elevation 313 m, but a perched water table prevailed in two of the boreholes located near the central portion of the site.

It should be pointed out that the groundwater table is subject to seasonal fluctuations and fluctuations in response to major weather events.

Yours truly

SHAHEEN & PEAKER LIMITED

REGISTERED PROFESSIONAL ENGINEER
Z. S. OZDEN
PROVINCE OF ONTARIO

Zuhtu Ozden, P.Eng.

for K. R. Peaker, Ph.D., P.Eng.

LICENSED PROFESSIONAL ENGINEER
K. R. PEAKER
PROVINCE OF ONTARIO

DRAWINGS

CONT No
WP No 314-99-00

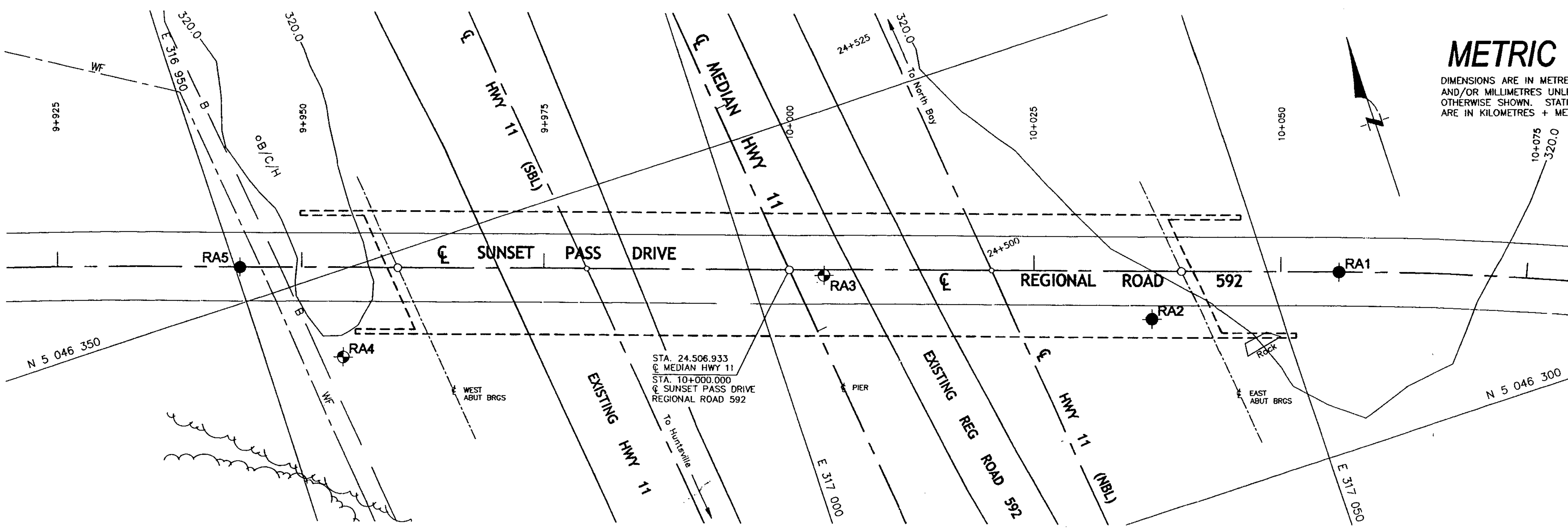
SHEET

HIGHWAY 11
SUNSET PASS DRIVE / REG RD 592
BORE HOLE LOCATIONS & SOIL STRATA

Shaheen & Peaker Limited

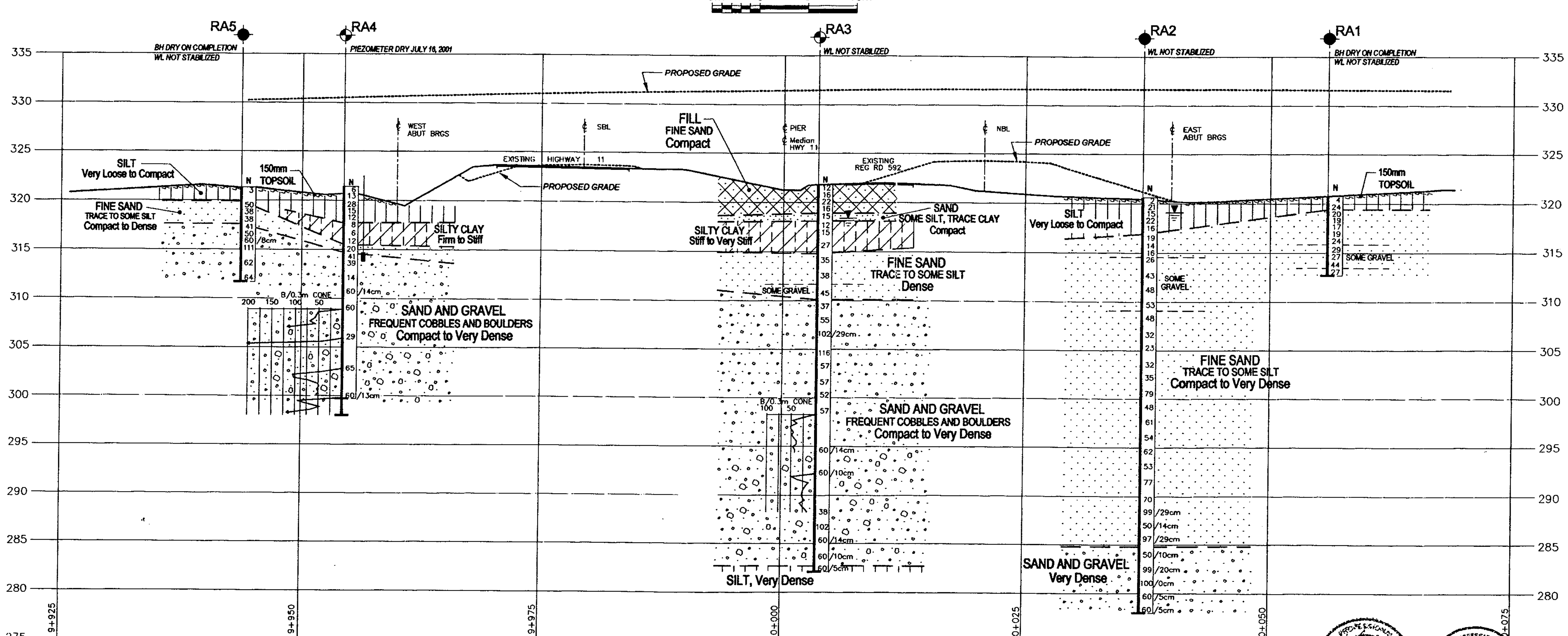
METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS ARE IN KILOMETRES + METRES.



PLAN

SCALE 5m 0 5 10m



℄ PROFILE SUNSET PASS DRIVE / REGIONAL ROAD 592

SCALE 5m 0 5 10m

KEY PLAN

SCALE 1km 0.5 0 1 2km

LEGEND

- Bore Hole
- Dynamic Cone Penetration Test (Cone)
- Bore Hole & Cone
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/0.3m (60° Cone, 475 J/blow)
- W L at time of investigation April and July 2001
- W L in Piezometer
- Piezometer

No	ELEVATION	CO-ORDINATES NORTH	EAST
RA1	320.8	5 046 317.8	317 056.9
RA2	320.6	5 046 319.2	317 037.5
RA3	321.8	5 046 334.0	317 007.1
RA4	321.4	5 046 341.5	316 957.8
RA5	321.3	5 046 353.6	316 950.5

=NOTE=

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

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section GC 2.01 of OPS Gen. Cond.

REV.	DATE	BY	DESCRIPTION

Geocres No.

HWY No 11			DIST 52
SUBM'D ZO	CHECKED ZO	DATE Nov., 2001	SITE
DRAWN JTW	CHECKED JP	APPROVED	DWG 1

MINISTRY OF TRANSPORTATION, ONTARIO PH-D-207 88 10

APPENDIX A
Records of Boreholes

RECORD OF BOREHOLE No RA1

1 OF 1 **METRIC**

W.P. 314-99-00	LOCATION Highway 11/Regional Road 592 - Katrine, ON - Coords: N 5 046 317.8; E 317 056.9	ORIGINATED BY R.A
DIST 52 HWY 11	BOREHOLE TYPE Solid Stem Augers	COMPILED BY G.T
DATUM Geodetic	DATE 24.04.01	CHECKED BY Z.O

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
320.8	Ground Surface						
0.0	150 mm **topsoil**; brown / grey **SILT** very loose / compact, with rootlets to 0.6 m., damp	1	SS	4	*		
		2	SS	24		320	
319.4 / 1.4	**FINE SAND** trace to some silt, compact to dense, brown, damp	3	SS	20		319	0 90 (10)
		4	SS	19		318	
		5	SS	17		317	
		6	SS	19			
		7	SS	24		316	
	------- some gravel	8	SS	29	**	315	** gravel stuck in tip
		9	SS	27		314	
	-------	10	SS	44			
312.7		11	SS	27		313	
8.1	End of borehole *Borehole dry. (Water level not stabilized) and hole open to full depth on completion						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m^3

$+^3, \times^3$: Numbers refer to Sensitivity

15-5 / 20 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No RA2

METRIC

W.P. 314-99-00 | LOCATION Highway 11/Regional Road 592 - Katrine, ON - Coords: N 5 046 319.2; E 317 037.5 | ORIGINATED BY R.A.

DIST 52 | HWY 11 | BOREHOLE TYPE Solid & Hollow Stem Augers, Casing & Washboring | COMPILED BY G.T

DATUM Geodetic | DATE 19.04.01 to 24.04.01 | CHECKED BY Z.O

Column headings: SOIL PROFILE (ELEV / DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20, 40, 60, 80, 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (20, 40, 60); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) (GR SA SI CL)

ELEV / DEPTH	DESCRIPTION
320.6	Ground Surface
0.0	150 mm **Topsoil**
	sandy - - - - very loose - - - -
	SILT with silty clay seams to 2.1 m. compact, grey/brown, moist
316.9 / 3.7	**FINE SAND** trace to some silt, damp to moist
	brown - - - - - - grey
	compact - - - - - - dense to very dense
	some gravel
	- - - - - - -
305.6 / 15.0	

NUMBER	TYPE	"N" VALUES
1	SS	2
2	SS	21
3	SS	15
4	SS	22
5	SS	16
6	SS	19
7	SS	14
8	SS	16
9	SS	26
10	SS	43
11	SS	48
12	SS	53
13	SS	48
14	SS	32

Ground water: ▼* (approx. elev. 319.2)

Elevation scale: 320, 319, 318, 317, 316, 315, 314, 313, 312, 311, 310, 309, 308, 307, 306

Grain size distribution (Sample 5): GR 0, SA 15, SI 81, CL 4

Remarks: April 19 - - - - April 20 (at approx. elev. 312.8)

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

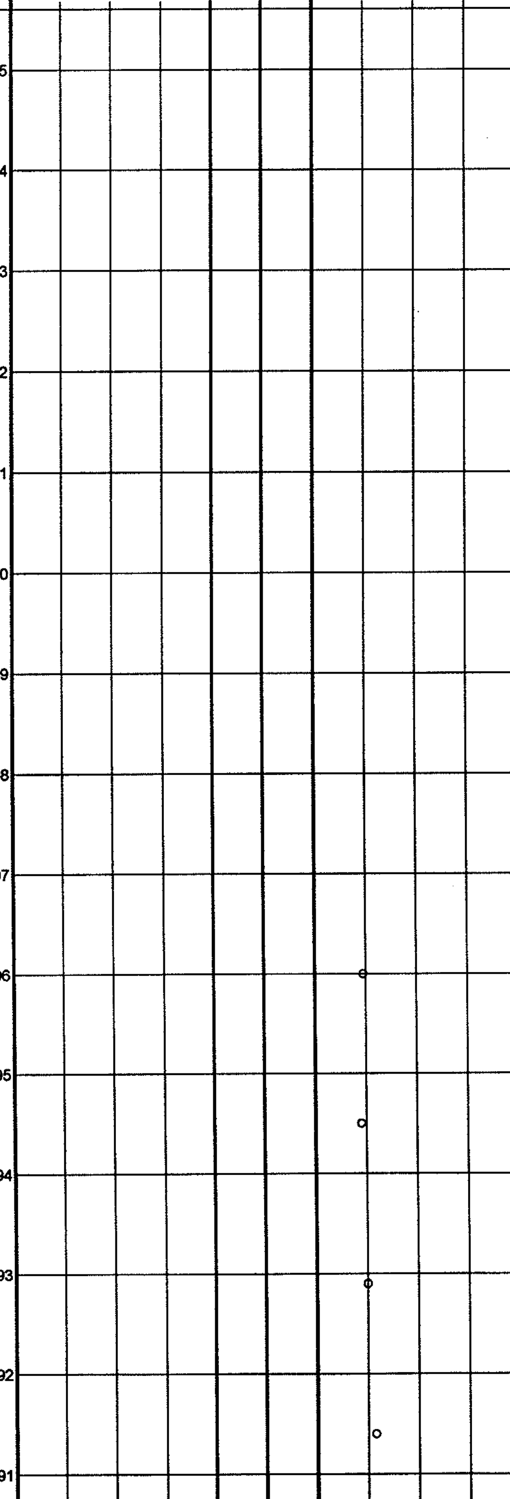
RECORD OF BOREHOLE No RA2

METRIC

W.P. 314-99-00	LOCATION Highway 11/Regional Road 592 - Katrine, ON - Coords: N 5 046 319.2; E 317 037.5	ORIGINATED BY R.A.
DIST 52 HWY 11	BOREHOLE TYPE Solid & Hollow Stem Augers, Casing & Washboring	COMPILED BY G.T
DATUM Geodetic	DATE 19.04.01 to 24.04.01	CHECKED BY Z.O

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
305.6 / 15.0	**FINE SAND** trace to some silt, occasional silt seams, grey; compact - - - dense to very dense; moist - - - wet	15	SS	23	305	
		16	SS	32	304	0 96 (4)
		17	SS	35	303, 302	Commenced washboring
		18	SS	79	301	
		19	SS	48	300, 299	
		20	SS	61	298, 297	
		21	SS	54	296	
		22	SS	62	295	0 93 (7)
		23	SS	53	294, 293	April 20 ------ April 23
		24	SS	77	292, 291	
290.6 / 30.0	Continued Next Page					

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m^3



$+^3, \times^3$: Numbers refer to Sensitivity   15-⌖-5 (20 above, 10 below) (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No RA2

METRIC

W.P. 314-99-00 LOCATION Highway 11/Regional Road 592 - Katrine, ON - Coords: N 5 046 319.2; E 317 037.5 ORIGINATED BY R.A.

DIST 52 HWY 11 BOREHOLE TYPE Solid & Hollow Stem Augers, Casing & Washboring COMPILED BY G.T

DATUM Geodetic DATE 19.04.01 to 24.04.01 CHECKED BY Z.O

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
290.6 / 30.0	**FINE SAND** trace to some silt, very dense, grey, wet					290	
		25	SS	70			
						289	
		26	SS	99/29		288	
		27	SS	50/14		287	
						286	
		28	SS	97/29			0 80 (20)
284.8 / 35.8	**SAND AND GRAVEL** very dense, grey, wet					285	
		~~29~~	~~SS~~	~~50/10~~	**	284	**Fragmented coarse gravel in spoon tip
						283	
		30	SS	99/20		282	
		~~31~~	~~SS~~	~~100/0~~	***	281	*** No recovery
						280	
		~~32~~	~~SS~~	~~60/5~~		279	
278.0		~~33~~	~~SS~~	~~60/5~~	****	278	**** Coarse gravel in spoon tip
42.6	End of borehole *Water level at 1.5 m (not stabilized) and hole open to 18.3 m on completion						Unable to extend casing any further Remove casings 24.04.01

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m^3

$+^3, \times^3$: Numbers refer to Sensitivity

20 / 15 ◈ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No RA3

METRIC

W.P. 314-99-00 LOCATION Highway 11/Regional Road 592 - Katrine, ON - Coords: N 5 046 334.0; E 317 007.1 ORIGINATED BY R.A.

DIST 52 HWY 11 BOREHOLE TYPE Solid & Hollow Stem Augers, Casing & Washboring, NQ Rock Core & D.C.P.T. COMPILED BY G.T

DATUM Geodetic DATE 24.04.01 to 26.04.01 CHECKED BY Z.O

SOIL PROFILE (ELEV / DEPTH, DESCRIPTION, STRAT PLOT) | SAMPLES (NUMBER, TYPE, "N" VALUES) | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 | SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE 20 40 60 80 100 | PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60 | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

- 321.8 / Ground Surface
- 0.0 — **FINE SAND** compact, brown, moist **(FILL)**; some silt
- 318.9 / 2.9 — **SAND**: some silt, trace clay, laminated, compact, brown, moist
- 318.1 / 3.7 — frequent silt seams, stiff; **SILTY CLAY** layered, with silt seams, very stiff, grey/brown; frequent silt seams
- 314.8 / 7.0 — **FINE SAND** trace to some silt, dense, damp, brown; some gravel
- 310.1 / 11.7 — **SAND AND GRAVEL** wet; brown / grey; dense / very dense
- 306.8 / 15.0 — Continued Next Page

Number	Type	"N" Values	Remarks & Grain Size (GR SA SI CL)
1	SS	12	
2	SS	16	
3	SS	22	
4	SS	16	
5	SS	15	
6	SS	12	0 3 60 37
7	SS	15	
8	TW	PH	
9	SS	27	
10	SS	35	0 95 (5)
11	SS	38	
12	SS	45	
13	SS	37	
14	SS	55	April 24 / April 25 Commenced washboring

Elevation scale: 321, 320, 319, 318, 317, 316, 315, 314, 313, 312, 311, 310, 309, 308, 307

$+^3, \times^3$: Numbers refer to Sensitivity

20 / 15 -⊕- 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No RA3

METRIC

W.P. 314-99-00	LOCATION Highway 11/Regional Road 592 - Katrine, ON - Coords: N 5 046 334.0; E 317 007.1	ORIGINATED BY R.A.
DIST 52 HWY 11	BOREHOLE TYPE Solid & Hollow Stem Augers, Casing & Washboring, NQ Rock Core & D.C.P.T.	COMPILED BY G.T
DATUM Geodetic	DATE 24.04.01 to 26.04.01	CHECKED BY Z.O

SOIL PROFILE: ELEV DEPTH | DESCRIPTION | STRAT PLOT

SAMPLES: NUMBER | TYPE | "N" VALUES

GROUND WATER CONDITIONS | ELEVATION SCALE

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE: 20 40 60 80 100

PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

306.8 / 15.0

SAND AND GRAVEL
dense to very dense
grey, wet

------

sand

----

frequent cobbles & boulders

boulder

cobble

291.8 / 30.0

Number	Type	"N" Values	Ground Water	Remarks & Grain Size (GR SA SI CL)
15	SS	102/29	**	**102/29 denotes 102 blows/29 cm
16	SS	116	***	***No recovery
17	SS	57		
18	SS	57		35 61 (4)
19	SS	52		
20	SS	57		
21	SS	60/14	****	****No recovery
22	RC	-		April 25
23	SS	60/10		------
24	RC	-		April 26

Elevation scale: 306, 305, 304, 303, 302, 301, 300, 299, 298, 297, 296, 295, 294, 293, 292

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity

15-⊕-5 (20 above, 10 below) (%) STRAIN AT FAILURE

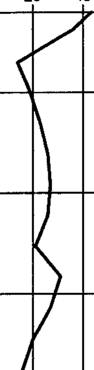
RECORD OF BOREHOLE No RA3

METRIC

W.P. 314-99-00	LOCATION Highway 11/Regional Road 592 - Katrine, ON - Coords: N 5 046 334.0; E 317 007.1	ORIGINATED BY R.A.
DIST 52 HWY 11	BOREHOLE TYPE Solid & Hollow Stem Augers, Casing & Washboring, NQ Rock Core & D.C.P.T.	COMPILED BY G.T
DATUM Geodetic	DATE 24.04.01 to 26.04.01	CHECKED BY Z.O

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLES: NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
291.8 / 30.0	**SAND AND GRAVEL** grey, wet; compact to dense ----- very dense				291, 290, 289		
		25	SS	38	288	o	80 19 (1)
		26	SS	102	287, 286		
		27	SS	60/14	285, 284		
		28	SS	60/10	283	o	
282.7 / 39.1	**SILT** very dense, grey, wet						Unable to extend casing beyond 39.7 m
282.1 / 39.7	End of borehole	29	SS	60/5			

DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100)



SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X LAB VANE (20, 40, 60, 80, 100)

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 20, 40, 60

UNIT WEIGHT γ kN/m³

End of borehole
- *Water level at 3.4 m (not stabilized) and hole open to 7.9 m on completion
- Dynamic Cone Penetration Test performed from 23.4 m to 27.4 m & 29.5 m to 33.5 m soil stratigraphy inferred only.
- Borehole extend by coring from 28.9 m to 29.1 m & 29.7 m to 29.8 m.

$+^3, \times^3$: Numbers refer to Sensitivity   15-⊕-5 (20, 10) (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No RA4

METRIC

W.P. 314-99-00	LOCATION Highway 11/Regional Road 592 - Katrine, ON - Coords: N 5 046 341.5; E 316 957.8	ORIGINATED BY A.J
DIST 52 HWY 11	BOREHOLE TYPE Solid & Hollow Stem Augers, Casing & Washboring, NQ Rock Core & D.C.P.T.	COMPILED BY G.T
DATUM Geodetic	DATE 04.06.01 to 07.06.01	CHECKED BY Z.O

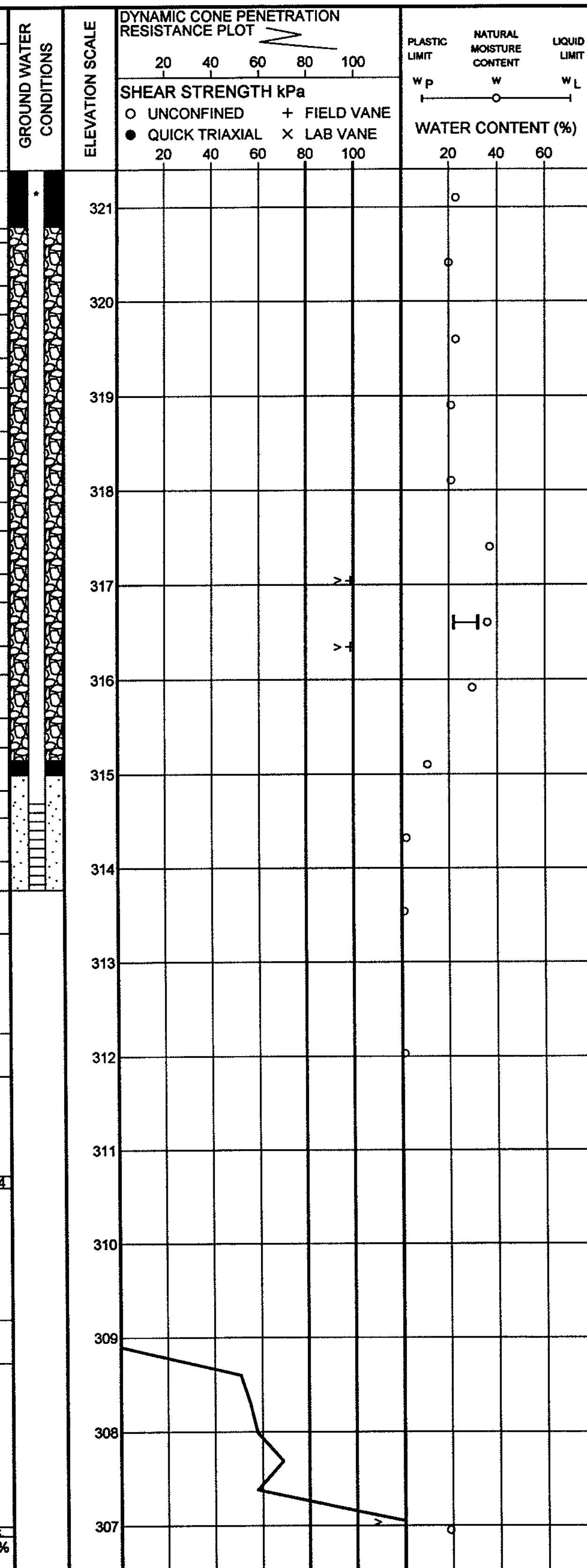
Soil Profile

ELEV / DEPTH	DESCRIPTION
321.4	Ground Surface
0.0	150 mm **Topsoil**
	SILT brown to 3.0 m, grey below, loose to compact, moist; sandy ----------- some silty clay seams, stiff
317.7 / 3.7	**SILTY CLAY** grey, moist to wet; firm ---- stiff
315.5 / 5.9	**FINE SAND** trace to some silt, compact, grey, damp
314.7 / 6.7	**SAND AND GRAVEL** grey; dense ------ compact -------- very dense; ----- frequent cobbles; damp ----- wet; boulder
306.4 / 15.0	

Continued Next Page

Samples

NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	6		
2	SS	13		
3	SS	28		
4	SS	15		
5	SS	12		0 14 78 8
6	SS	8	15.2	
7	SS	6	17.3	0 4 72 24
8	SS	12	16.7	
9	SS	20		0 84 16 0
10	SS	41		
11	SS	39		
12	SS	14		
13	SS	60/14		
14	SS	60		Commenced washboring
15	RC	Rec 100%		June 04 ------ June 05



$+^3, \times^3$: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No RA4

METRIC

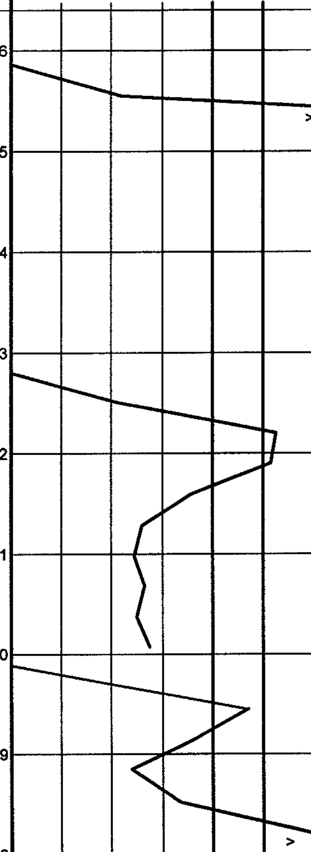
W.P. 314-99-00 | LOCATION Highway 11/Regional Road 592 - Katrine, ON - Coords: N 5 046 341.5; E 316 957.8 | ORIGINATED BY A.J

DIST 52 HWY 11 | BOREHOLE TYPE Solid & Hollow Stem Augers, Casing & Washboring, NQ Rock Core & D.C.P.T. | COMPILED BY G.T

DATUM Geodetic | DATE 04.06.01 to 07.06.01 | CHECKED BY Z.O

SOIL PROFILE | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100) | SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20, 40, 60, 80, 100) | PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L — WATER CONTENT (%) (20, 40, 60) | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS
306.4 15.0	compact ------		16	SS	29	306		
						305		
						304		
	SAND AND GRAVEL frequent cobbles and boulders, very dense to dense, grey, wet		17	SS	65	303	o	June 05 ------ June 06
						302		
						301		
299.7			18	SS	60/13	300	o	June 06 ------ June 07
21.7	End of borehole Unable to advance casing					299		
298.0 23.4	End of Dynamic Cone PenetrationTest Borehole extended by coring from 14.4 m to 14.5 m. *Piezometer installed at 7.6 m Water level at July 16/2001 - dry Dynamic Cone Penetration Test performed from 12.5 m to 14.4 m; 15.5 m to 16.0 m; 18.6 m to 21.3 m & 21.6 m to 23.4 m, soil stratigraphy inferred only. Broken coupling at 15.8 m and 21.5 m. borehole terminated					298		



$+^3, \times^3$: Numbers refer to Sensitivity   15-⌖-5 (20 / 10) (%) STRAIN AT FAILURE

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RECORD OF BOREHOLE No RA5

METRIC

W.P.	314-99-00	LOCATION	Highway 11/Regional Road 592 - Katrine, ON - Coords: N 5 046 353.6; E 316 950.5	ORIGINATED BY	R.A.
DIST 52	HWY 11	BOREHOLE TYPE	Solid & Hollow Stem Augers	COMPILED BY	G.T
DATUM	Geodetic	DATE	27.04.01	CHECKED BY	Z.O

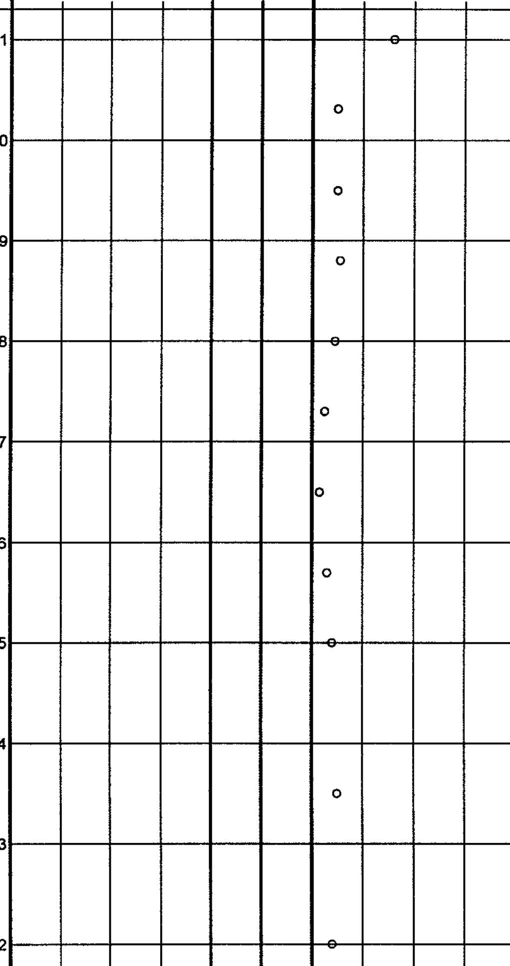
ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLES: NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
321.3	Ground Surface						
0.0	150 mm **Topsoil** **SILT** some sand, rootlets to 0.6 m very loose, brown, moist	1	SS	3	*	321	
		2	SS	-	**	320	**Spoon bouncing on tree roots
319.9 / 1.4	**FINE SAND**: trace to some silt, dense, brown, moist	3	SS	50			
		4	SS	38		319	0 87 (13)
		5	SS	38		318	
317.6 / 3.7	**SAND AND GRAVEL**: brown dense ---- very dense	6	SS	41		317	
		7	SS	50			
		~~8~~	~~SS~~	~~60/8~~	***	316	***Coarse gravel in spoon tip
		9	SS	111		315	
	damp ----- moist					314	
		10	SS	62		313	
311.7		11	SS	64		312	
9.6	End of borehole *Borehole dry (not stabilized) and open to 5.6 m on completion						

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

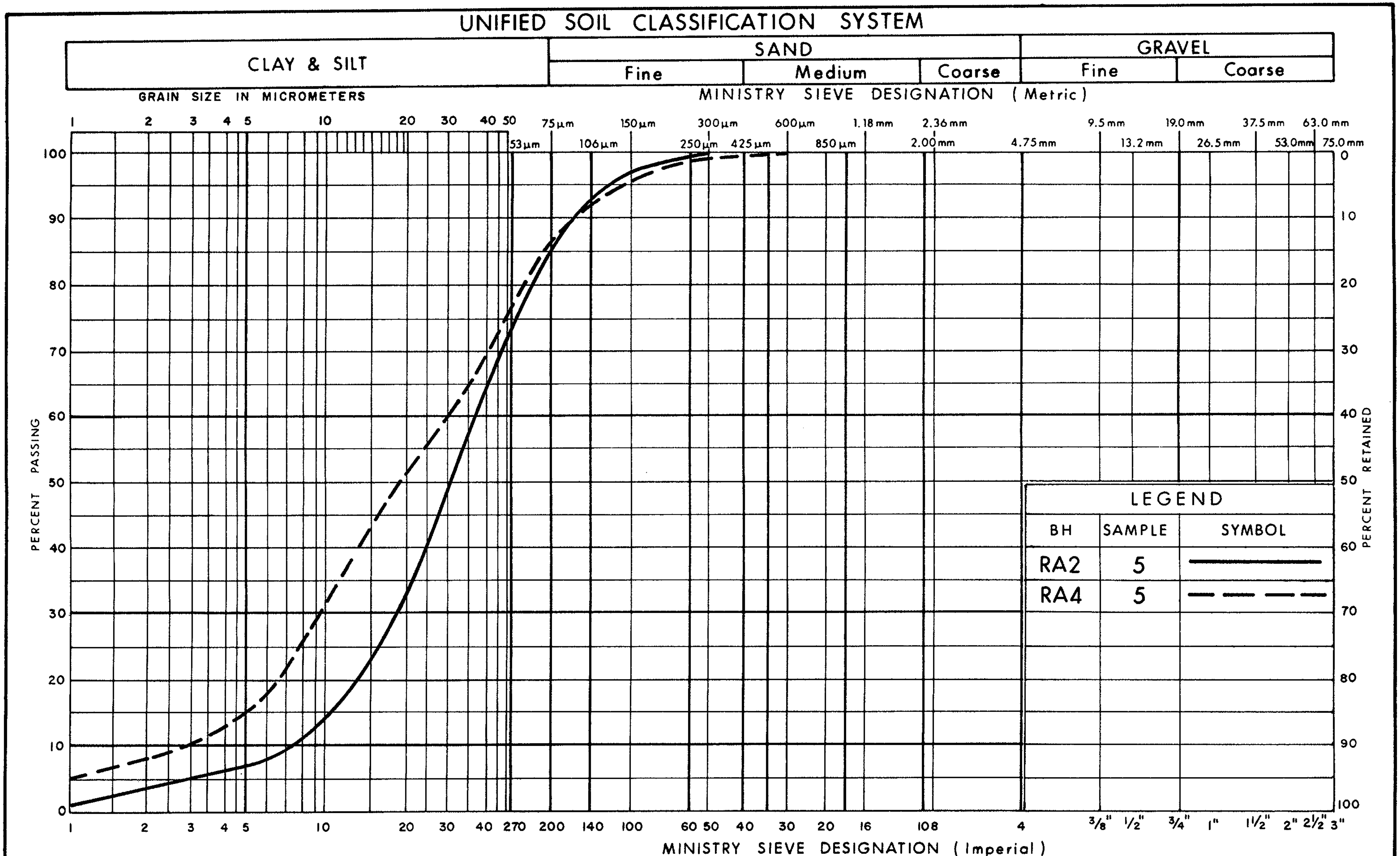
UNIT WEIGHT γ kN/m³



$+^3, \times^3$: Numbers refer to Sensitivity

15-⊕-5 (20 above, 10 below) (%) STRAIN AT FAILURE

APPENDIX B
Laboratory Test Results



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GRAIN SIZE DISTRIBUTION

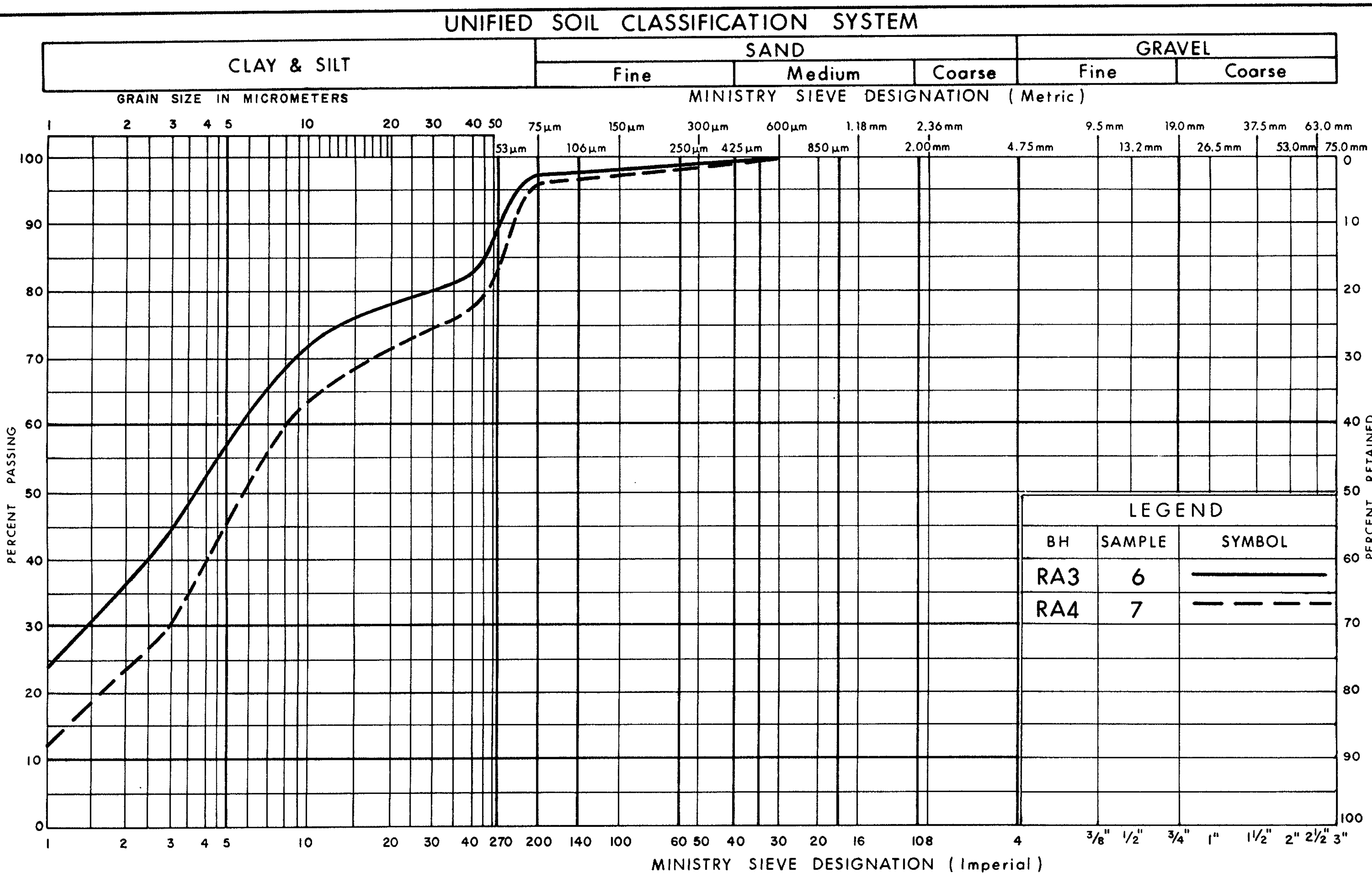
SILT

FIG No 1

W P 314-99-00

SPT 1010D

78 12 M



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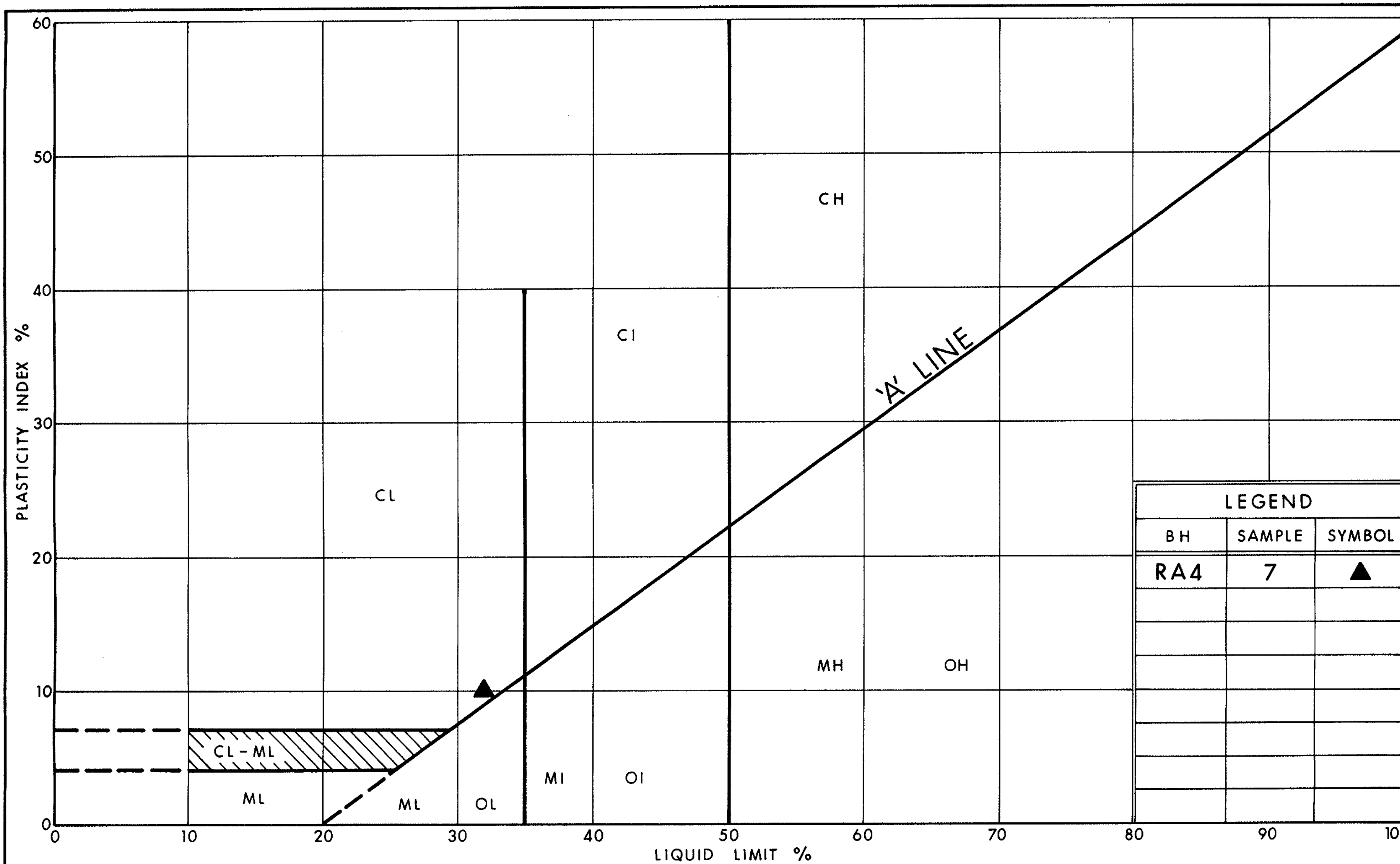
GRAIN SIZE DISTRIBUTION
SILTY CLAY

FIG No 2
W P 314-99-00
SPT 1010D

Oct 75, FF-S-21



LEGEND		
BH	SAMPLE	SYMBOL
RA4	7	▲

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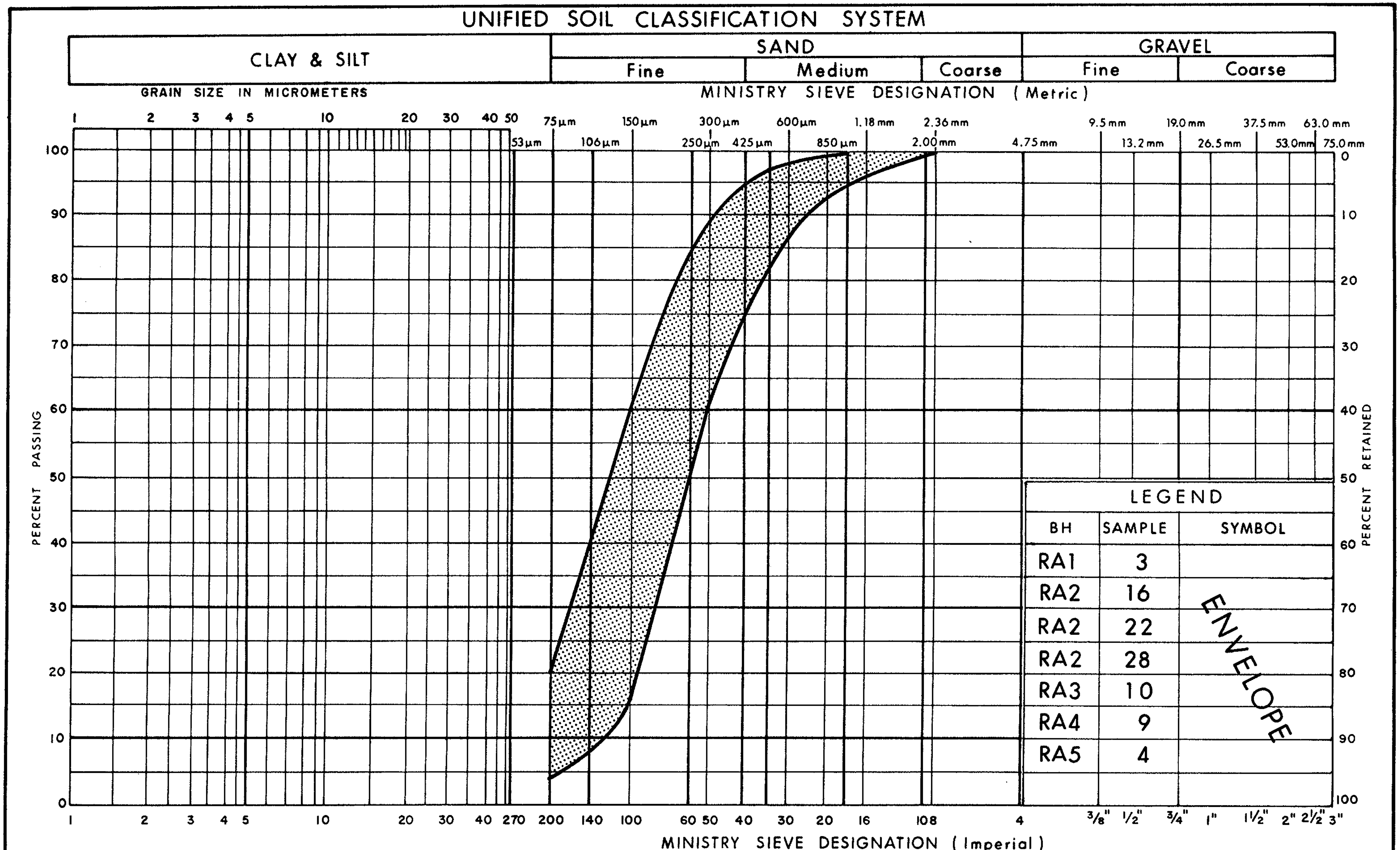
PLASTICITY CHART
SILTY CLAY

FIG No 3
W P 314-99-00
SPT 1010D

78 12 M



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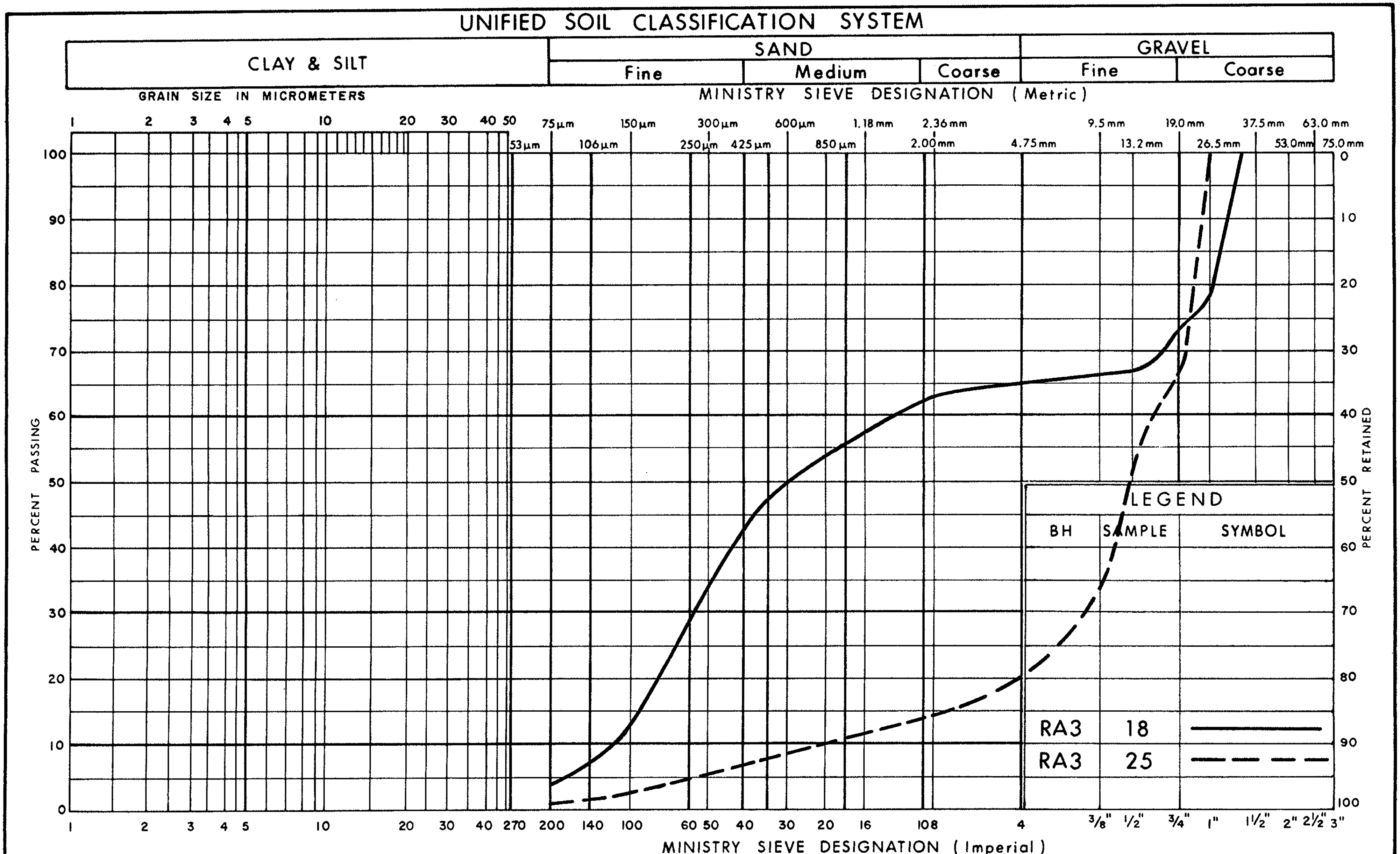
GRAIN SIZE DISTRIBUTION
FINE SAND

FIG No 4

W P 314-99-00

SPT 1010D

78 12 M



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Ontario

GRAIN SIZE DISTRIBUTION
SAND AND GRAVEL

FIG No 5

W P 314-99-00

SPT 1010D

APPENDIX C
Explanation of Terms Used in Report

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/0.3 m)	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:

RQD (%)	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

PRELIMINARY FOUNDATION DESIGN REPORT
PROPOSED SUNSET PASS DRIVE/REGIONAL ROAD 592 BRIDGE
OVER HIGHWAY 11
KATRINE, ONTARIO
W.P. 314-99-00

Prepared For:

STANTEC CONSULTING LTD.

Prepared by:

SHAHEEN & PEAKER LIMITED

Project: SPT1010D
December 3, 2001

250 Galaxy Boulevard
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M9W 5R8
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PRELIMINARY FOUNDATION DESIGN REPORT
PROPOSED SUNSET PASS DRIVE/REGIONAL ROAD 592 BRIDGE
OVER HIGHWAY 11
KATRINE, ONTARIO
W.P. 314-99-00

5. DISCUSSION AND RECOMMENDATIONS

5.1 GENERAL

The existing Highway 11 at Regional Road 592 and Sunset Pass Drive interchanges is a two-lane roadway with turning lanes. Here, Highway 11 will be widened to a four-lane divided roadway with a typically 30 m wide median. The existing two-lane roadway will be utilized as the southbound lanes while the newly built roadway will constitute the northbound lanes. A new roadway will also be constructed that will connect re-aligned Highway 592 to Sunset Pass Drive over the new Highway 11 roadway via a two-span bridge. The anticipated width of this new structure is approximately 12.5 m, while the span lengths will likely be 40 m each, for a total length of 80 m. The top of the pavement elevation for the bridge can be expected to be about 331.7 m on the east side and 330.7 m on the west side thus requiring about 11 m and 9.5 m grade raise at the east and west abutment locations, respectively. As shown on Drawing No. 1, the new bridge will be at a skew.

The ground surface elevations at the borehole locations range from 321.8 to 320.6 m while the top of pavement elevation for the existing Highway 11 is about 323 m.

The boreholes show below some topsoil and fill (Borehole RA3) the presence of a very loose to compact surficial silt deposit which extends to depths ranging between 1.4 and 3.7 m below the ground surface. A localized firm to very stiff silty clay layer was contacted in Boreholes RA3 and RA4. The thickness of this stratum at the borehole locations was measured to be 2.2 and 3.3 m where it extended to depths of 5.9 m and 7.0 m or to Elevations 315.5 and 314.8 m. The silt and the silty clay are underlain by an extensive deposit of compact to very dense fine sand. At the borehole locations, this deposit is about 1 to 32 m thick and extends to depths ranging between about 4 and 36 m below the ground surface or to Elevations 317.6 to 284.8 m. The fine sand is in turn underlain by sand and gravel containing cobbles and boulders. Bedrock was not contacted within the depths explored (i.e. 8 to 43 m). The groundwater table at the time of the investigation was more than 8 m below the ground surface but the presence of a perched water level was noted in two of the boreholes, due to the accumulation of surface water on practically impervious surficial silt clay seams or layers.

5.2 FOUNDATIONS

5.2.1 SPREAD FOOTING FOUNDATIONS

5.2.1.1 SPREAD FOOTING FOUNDATIONS ON NATURAL SUBGRADE

The use of normal spread footing foundations on native ground is feasible at a depth of about 2 m below the existing ground surface at Borehole RA2 (east abutment) but at Borehole RA3 (central pier) and especially at RA4 (west abutment area) a relatively weak and compressible silty clay layer was contacted which will not support adequate loads and which can be expected to undergo consolidation settlements (i.e. excessive total and time-rate differential settlements). For this reason, the footings would have to be extended below this layer in Borehole RA4 or to a sufficient depth in Borehole RA3, as summarized in Table 5.2.1.1.1.

Table 5.2.1.1.1

Borehole/ Location	**Existing Ground Surface Elevation (m)**	**Recommended Highest Footing (Bottom) Level Below Existing Ground Surface (m)**	**Recom-mended Foundation Elevation (m)**	**Factored Geotechnical Bearing Resistance at U.L.S.* kPa**	**Bearing Resistance at S.L.S. kPa**	**Subgrade Material**
RA2/East Abutment	320.6	2.1	318.5	600	250	Silt
RA3/ Central Pier	321.8	5.8	316.0	650	330	Silt/silty clay
RA4/West Abutment	321.4	5.9	315.5	700	350	Fine sand

The elevations given indicate that at Borehole RA3 and RA4 locations deep excavations will be necessary and probably require extensive shoring immediately adjacent to the existing highway. For this reason, the use of spread footing foundations on natural subgrade, although an option, is not the most practical and cost effective solution at this site.

5.2.1.2 SPREAD FOOTING FOUNDATIONS ON IMPROVED SUBGRADE

Consideration could also be given to the use of normal spread footing foundations on engineered fill. In-situ soil improvement methods such as dynamic compaction will probably be unsuitable due to the presence of the silty clay layer in Borehole RA3 and RA4 and also due to the proximity to the Highway. Normal spread footing foundations could be used after removing the existing soils to a sufficient depth (as shown in Table 5.2.1.1.1) and replacing them with engineered fill.

For this purpose, the soil beneath the proposed footing would be removed to the elevations shown in Table 5.2.1.1.1 within an area at least 1 m beyond the perimeter of the proposed footing at the bottom of the excavation. For example, for a footing measuring 5 x 12 m in plan the size of the excavation at the bottom of the excavation would be 5+1+1=7 m by 12+1+1=14 m in plan. The sides of the excavation would be sloped not steeper than 1H:1V (flatter where necessary). This means that at the existing ground level, the area of the excavation would be excessively large, especially where the excavations are deep. For example, at Borehole RA2 location the recommended depth of excavation is about 2 m and therefore, the size of the excavation with 1:1 side slopes would be 7+2+2=11 m by 14+2+2= 18 m in plan view while at Borehole RA3 and RA4 locations it would be 7+6+6=19 m by 14+6+6=26 m or even greater, if flatter side slopes become necessary.

When the excavation reaches the required depth, the subgrade would be evaluated and approved by the Geotechnical Engineer. If necessary, the excavation may need to be deepened to the surface of sufficiently competent soil. The fill used to raise the grade inside the excavation would be Granular 'A' quality materials or similar approved materials, which would be placed in layers not exceeding 200 mm in thickness and uniformly compacted to not less than 100% of its Standard Proctor Maximum Dry Density (SPMDD).

A factored bearing resistance at U.L.S. of 800 kPa and a bearing resistance of S.L.S. equal to 300 kPa can be assigned to the soil prepared in this manner provided that the engineered granular fill below the bottom of the footing is at least 1.5 m thick. The serviceability condition is based on the premise that total and differential settlements will not exceed 25 mm and 20 mm, respectively. For frost protection, the footings must have a frost cover of not less than 1.8 m. Allowance need to be made to deal with a perched water table if necessary during the excavation and backfilling process.

At the abutment locations engineered fill could be extended higher to implement MTO's standard abutment on compacted fill with Granular 'A' core scheme which may be able to utilize somewhat higher soil resistances (e.g. up to 350 kPa for S.L.S. and a factored bearing resistance at U.L.S. of 900 kPa).

From the above discussion, however, it is obvious that very large excavations will be required immediately adjacent to the Highway at the central pier and especially at the west abutment location and therefore the use of spread footing option is believed to be impractical. Therefore, spread footings on improved subgrade although considered an option, are not the most practical and cost effective solution at this site.

5.2.2 DEEP FOUNDATIONS

5.2.2.1 GENERAL

The presence of relatively dry sand (cohesionless soils) followed by wet conditions along with a lack of a well-defined bearing stratum renders the use of drilled and cast-in-place concrete (caisson) foundations impractical. Auger press piles can be considered but these have little resistance to lateral loads and will probably be uneconomical. These options are, therefore, not feasible.

Because of variable soil conditions, in some cases containing oversize materials (e.g. coarse gravel, cobbles and boulders), the use of timber piles and driven pre-cast concrete piles is also not recommended.

5.2.2.2 EXPANDED BASE CONCRETE PILES

Expanded base concrete piles ('Franki' type piles) can be considered. Relatively high resistances can be developed in the cohesionless granular soils prevailing at the site. Depending on the required resistances, the anticipated pile depths can range below 8 and 10 m below the ground surface. For preliminary estimating purposes, the following resistances can be assumed for 406 mm diameter (16-inch) piles at Elevations ranging between 314 and 311 m.

U.L.S. = 1250 kN, S.L.S.= 850 kN

For 508 mm diameter (20-inch) piles, the resistance values can be expected to be about 50% more than those quoted for the 406 m diameter piles (i.e. U.L.S. = 1900 kN and S.L.S. = 1250 kN).

In order to provide lateral resistance, it will be necessary to incorporate reinforcement into the piles. Expanded base concrete piles are installed by specialist contractors. The process is a contractor dependent procedure and the resulting product, including the resulting geotechnical resistances and method of reinforcing, is dependent on the details of method used and on the quality of workmanship and it should, therefore, be discussed with a specialist contractor. This discussion should include method of reinforcing in particular whether a steel H-pile can be incorporated into the system in order to provide a sufficient flex zone, if an 'integral abutment' type bridge is to be constructed. We will also be pleased to discuss further details of this type foundation, should you wish us to do so. In our opinion, this type of pile will probably provide a cost effective and reliable foundation for the prevailing subsurface conditions. Consideration should be given for a pile load test(s).

5.2.2.3 DRIVEN STEEL PILES

Driven steel tube and steel H-piles are available options. Preliminary general arrangement bridge concept indicates skew foundations and therefore an integral abutment type bridge may not be feasible. On the other hand, if an integral abutment type of bridge is to be considered (since the skew is not excessive), then H-piles would need to be utilized.

The borehole results show that the relative density of the soil (as inferred from Standard Penetration and dynamic cone penetration results) is quite variable and in most cases, there is a lack of well-defined competent stratum (e.g. bedrock) for refusal on pile driving. In some cases, the high N-values obtained in the sand and gravel deposit can be attributed to the oversize particles. For example, in Borehole RA3, very high N-values were obtained between about 27 m and 30 m below the ground surface (Elevation 294.5 – 292 m) whereas a dynamic cone penetration test below this elevation gave resistance values on the order of 20 blows/0.3 m. In addition, the sand and gravel deposit contains frequent boulders and some of the piles may 'hang-up' on such obstructions. For these reasons, it is difficult with the available data to predict pile tip elevations. While this difficulty is primarily due to the properties of the sand and gravel deposit encountered at this site, a better understanding of the anticipated pile depths will be realized when additional data from the detailed investigation become available. In our opinion, however, it is possible to obtain the following axial resistances for HP310x110 steel H-piles.

Table 5.2.2.3.1
Recommended Preliminary Axial Resistances
and Tentative Pile Tip Elevations for HP310x110 Steel H-Piles

Support Location	Reference Borehole	Existing Ground Surface Elevation (m)	Estimated Pile Tip Depth/Elevation (m)	Recommended Factored Axial Resistance at U.L.S. (kN)	Recommended Axial Resistance at S.L.S. (kN)
East Abutment	RA2	320.8	30 m/291	1650	1100
Central Pier	RA3	321.8	30 m/292	1650	1100
West Abutment	RA4	321.4	23 m/298	1650	1100

In view of the fact that the presence of frequent cobbles and boulders was noted in the boreholes (e.g. Boreholes RA3 and RA4), the use of a heavy pile section (e.g. HP310x110) with reinforced tips as per MTO specifications (OPSD 3301.00) is recommended.

Steel tube (pipe) piles may also be considered. Tube piles will provide lower resistances, as they will not drive as deep in comparison with steel H-piles but the lower resistances will likely be compensated by the anticipated shorter pile lengths. Steel tube piles of 300 mm nominal diameter (e.g. 324 mm x 9.4 mm size piles) can be expected to provide a Factored Axial Resistance at U.L.S. of about 1100 kN and an Axial Resistance S.L.S. of about 650 kN per pile at depths of between 14 m or Elevation 307 m at Borehole RA4 and 20 m or Elevation 301 m at Borehole RA2. After installation and inspection for possible damage, tube piles will have to be filled with concrete. Again, as a protection against hard driving conditions and the presence of coarse soil particles (cobbles and boulders) a relatively thick steel section should be selected. From previous experience, steel H-piles are preferred over steel tube piles, based on reliability.

In any event, the piles will need to be driven using a suitably heavy hammer capable of delivering a rated energy of at least 55 kilojoules/blow, but not more than 70 kilojoules/blow. The driving of the piles in the field should be controlled by a recognized driving formula, such as the Hiley Formula. The estimated ultimate resistance of the piles by the Hiley Formula can be calculated by dividing the recommended axial resistance at U.L.S. by a resistance factor of 0.5 as per current MTO practice. With this criterion, the estimated ultimate axial resistance for steel H-piles as per Hiley Formula is 3300 kN (i.e. 1650 ÷ 0.5) and for 300 mm nominal diameter steel tube piles, it would be 1100 ÷ 0.5 = 2200 kN.

In accordance with the above criterion, the piles may need to be driven to about 4 m above the design elevation and driving then monitored and controlled by employing the Hiley Dynamic Pile Driving Formula in accordance with MTO Standards SS103-10 and SS103-11.

During the driving process, piles which have already been driven will need to be monitored to determine if heaving occurred due to the effects of driving of adjacent piles. If this phenomenon occurs, the affected piles will need to be re-driven. Re-tapping, to check that relaxation has not occurred, will be necessary. At least 10% of the piles (but not less than two piles) driven at each foundation element should be re-tapped not less than 24 hours after the driving of the pile, as per SP903S01, to check that relaxation has not occurred. If it has, then all the piles should be re-tapped. Furthermore, it may be necessary to stagger the driving of the piles.

All pile driving should be in accordance with special provision SP903S01 – Construction Specification for Piling.

Pile lengths may be significantly different than the quoted values and therefore this aspect will need to be considered in the contract documents and when ordering of the piles.

The minimum spacing between the piles should be in accordance with OHBDC, Clause 6.11-1, current addition. As mentioned before, due to the presence of cobbles and boulders, H-piles should be equipped with reinforced tips as per MTO Standards (OPSD 3301.00). For steel tube piles, the provision of a thick steel toe plate (preferably with reinforcing) is recommended, provided that the pile is inspected for possible damage at the end of installation, before pouring the concrete.

Depending on the findings of the detailed investigation, the, confirmation of anticipated geotechnical resistances with pile load test(s) and/or Pile Driving Analyzer (PDA) may possibly need to be considered.

5.2.3 GENERAL COMMENTS ABOUT FOUNDATIONS

The proposed final embankment elevations adjacent to the west and east abutments are approximately 330.7 m and 331.7 m and this will require raising the grade by approximately 9.5 m and 11.0 m, above the existing ground elevations at the west and east abutments, respectively. The stresses induced by the weight of the fill placed to raise the grade will cause settlements in the underlying foundations soils. In order to pre-induce a significant portion of these settlements for the paved highway near the bridges and to minimize any down-drag loads on the piles due to settlement of soil surrounding them, we recommend that the embankment fills be placed to their final subgrade elevations at least two weeks before pile driving at the east abutment location and at least four weeks at the west abutment location where a weak silty clay stratum was encountered.

For frost protection, all pile caps should have a permanent earth cover of at least 1.8 m.

In cohesionless soils the coefficient of horizontal subgrade reaction can be estimated from:

$$k_s = n_h z/d$$

Where

k_s = coefficient of horizontal subgrade reaction
z = depth
d = pile width
n_h = coefficient related to soil density; see Table 5.2.3.1

Also presented in the same table are estimated values for angle of internal friction and bulk unit weights.

Where the soil is primarily cohesive, the undrained shear strength of the soil is given.

Table 5.2.3.1.

Area/ Reference Borehole No.	Applicable Depth Below Existing Ground Surface (m)	Applicable Elevation (m)	Soil Type	Bulk Unit Weight (kN/m³)	Angle of Internal Friction (φ) Degrees	Recom-mended n_h Value (kN/m³)	Recom-mended Undrained Shear Strength (kPa)
East Abutment/RA2	0.2-3.7	320.4-316.9	Silt	19.0	30	4000	
	3.7-7.0	316.9-313.6	Fine sand	19.5	31	6600	
	7.0-35.8	313.6-284.8	Fine sand	20.0	32	11000	
	35.8-42.6	284.8-278.0	Sand and gravel	21.5	34	11000	
Central Pier/RA3	0-2.9	321.8-318.9	Fine sand fill	19.0	30	5000	
	2.9-3.7	318.8-318.1	Sand	19.5	31	4400	
	3.7-7.0	318-1-314.8	Silty clay	19.0	-	-	100
	7.0-11.7	314.8-310.1	Fine sand	19.5	32	11000	
	11.7-39.1	310.1-282.7	Sand and gravel	21.5	34	11000	
West Abutment/ RA4	0.2-3.7	321.2-317.7	Silt	19.0	30	4000	
	3.7-5.9	317.7-315.5	Silty clay	19.0	-	-	80
	5.9-6.7	315.5-314.7	Fine sand	19.5	31	6600	
	6.7-21.7	314.7-299.7	Sand and gravel	21.5	34	11000	

For preliminary design purposes, the recommended horizontal resistances for HP310x110 steel H-piles are as follows:

Factored Horizontal Resistance at U.L.S. = 130 kN/pile
Horizontal Resistance at S.L.S. = 50 kN/pile

If integral abutments are not constructed then the lateral resistance of the piles can be supplemented, if desired, by the horizontal components of battered piles. In this instance, we recommend that the batter be limited no more than 4:1, as in practice greater batter is difficult to install.

Oversize materials (e.g. greater than 75 mm nominal diameter) should not be used in the embankment fills through which piles would be driven.

5.3 LATERAL EARTH PRESSURES

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

Free-draining backfill materials (i.e. Granular A or Granular B) and the provision of drain pipes and weep holes, etc., should prevent hydrostatic pressure build-up. Computation of earth pressures should be in accordance with O.H.B.D.C. For design purposes, the following parameters (unfactored) can be used.

Compacted Granular 'A'
Unit Weight = 22 kN/m^3
Coefficient of Lateral Earth Pressure:
$K_a = 0.27$
$K_o = 0.43$

Compacted Granular 'B' Type 1
Unit Weight = 21 kN/m^3
Coefficient of Lateral Earth Pressure:
$K_a = 0.31$
$K_o = 0.47$

Rock Fill
Unit Weight = 18.0 kN/m^3
Coefficient of Lateral Earth Pressure:
$K_a = 0.27$
$K_o = 0.43$

These values are based on the assumption that the backfill behind the retaining structure is free-draining and adequate drainage is provided. As well, it is assumed that the ground behind the retaining structure is level.

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 O.H.B.D.C., 3rd Edition. The effect of compaction should also be taken into account in the selection of the appropriate earth pressure coefficients in accordance with Clause 6-7.4.3, O.H.B.D.C., 3rd Edition.

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

5.4 APPROACH EMBANKMENTS

As mentioned before, embankment fills of the order of 9.5 m high on the west side and 11 m on the east side may be required.

Based on the limited borehole and laboratory data and our preliminary analysis carried out using limit state equilibrium (Bishop's simplified method by the computer programme Slope W), no foundation failures are anticipated for the proposed up to 9.5 m high embankments on the west side and 11 m high embankments on the east side, both with 2H:1V slopes, provided that all organic, weak or otherwise unsuitable materials are removed as per MTO standards before placing the fill.

Assuming properly compacted, acceptable inorganic earth fill material, 2 horizontal in 1 vertical side slopes can be used. Proper erosion control measures should be implemented both during the construction and permanently. This can be achieved by immediate seeding or sodding (OPSS 572). For slope heights in excess of 6 m, an at least 2 m wide mid-height berm should be provided, as per MTO Northern Region requirements.

All organic and other unsuitable soils should be removed within an envelope given by an imaginary slope not steeper than 1:1 from the toe of the proposed embankment. Based on the available borehole data for preliminary estimating, the average thickness of the unsuitable soils to be stripped can be assumed to be about 0.3 m. After stripping, the exposed subgrade should be inspected, approved and properly compacted from the surface, using a heavy compactor, suitable for the prevailing site conditions (e.g. silt soils).

The materials used for the construction of the embankment fills should consist of approved, acceptable earth fill. Oversize materials (i.e. nominal diameter in excess of 75 mm) should not be used in embankment fills through which piles would be driven. The fills should be placed in lifts not exceeding 300 mm before compaction and each lift should be uniformly compacted to at least 95% of the material's Standard Proctor dry density. The degree of compaction within the top 0.5 m thick zone of the fill (i.e. subgrade immediately beneath the granular subbase) should be minimum 98%. The settlement of embankment fills prepared as described above should not exceed 60 mm. However, the underlying foundation soils can be expected to settle an additional 100 mm. As mentioned before, we recommend that the embankment fills be placed to the proposed subgrade levels at least two weeks before pile driving at the east abutment and at least four weeks at the west abutment location. If this is done about 80% of the quoted settlements would have taken place before the highway is paved. The residual settlements would be considered acceptable.

5.5 CONSTRUCTION COMMENTS

While at the time of investigation the groundwater table at the site was found at depths greater than 8 m (probably at about 12± m below the ground surface, that is, more than the anticipated depth of excavations), a perched water level was noted in two of the boreholes. Although this water should not present major problems due to its limited quantities, its effects on the inherently dilatent surficial silt deposits should be considered. Dewatering may, therefore, be required to handle this perched water in order to facilitate the construction and to stabilize the soil.

For stripping and compacting beneath the footprint of embankments where wet conditions are encountered, gravity drainage and pumping from open sumps should suffice. In excavations for pile caps, which extend below the water table, dewatering by means of perimeter drainage in oversized excavations and pumping from strategically placed sumps should be sufficient. In excavations for the construction of any spread footings or footing bases (i.e. compacted granular pads under spread footing foundations), in addition to gravity drainage and pumping from filtered sumps as discussed above, additional dewatering effort will likely be necessary. This could consist of draining the perched water collected in the surficial sand and silt into the underlying sand, and/or using strategically placed, properly filtered wells.

5.6 FROST PROTECTION

Design frost penetration for the general area is 1.8 m, therefore, a permanent soil cover of 1.8 m or its thermal equivalent of artificial insulation is required for frost protection of foundations, including pile caps.

6. CLOSURE

The Limitations of Report, as quoted in Appendix D, are an integral part of this report.

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APPENDIX D

Limitations of Report

LIMITATIONS OF REPORT

The conclusions and recommendations given in this report are based on information determined at the testhole locations. The information contained herein in no way reflects on the environment aspects of the project, unless otherwise stated. Subsurface and groundwater conditions between and beyond the testholes 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 testhole locations and should not be used 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 testholes 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.

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