

GEOCREDS No:
31E-328

**PRELIMINARY FOUNDATION INVESTIGATION REPORT
PROPOSED HIGHWAY 11 REALIGNMENT
MUNICIPAL SERVICE ROAD BRIDGE
OVER MAGNETAWAN RIVER
KATRINE, ONTARIO
W.P. 314-99-00**

Prepared For:

STANTEC CONSULTING LTD.

Prepared by:

SHAHEEN & PEAKER LIMITED

**Project: SPT1010A
November 8, 2001**

**250 Galaxy Boulevard
Etobicoke, Ontario
M9W 5R8
Tel: (416) 213-1255
Fax: (416) 213-1260**

Table of Contents

1. INTRODUCTION	1
2. SITE DESCRIPTION AND GEOLOGY	1
3. INVESTIGATION PROCEDURES	2
4. SUBSURFACE CONDITIONS	5
4.1. Topsoil	6
4.2. Silty Fine Sand	7
4.3. Clayey Silt	8
4.4. Silt	9
4.5. Gravelly Sand With Silt, Cobbles and Boulders	10
4.6. Groundwater	10

DRAWINGS	DRAWING NO.
BOREHOLE LOCATION PLAN & STRATIGRAPHIC SECTIONS	1
APPENDICES	
RECORDS OF BOREHOLES	APPENDIX A
LABORATORY TEST RESULTS	APPENDIX B
EXPLANATION OF TERMS USED IN REPORT	APPENDIX C

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1. INTRODUCTION

Shaheen & Peaker Limited ("S&P") was retained by Stantec Consultants Limited ("Stantec") to conduct a preliminary foundation investigation for a proposed bridge which will carry a new Municipal Service Road over the Magnetawan River. The site is located immediately south of Katrine, Ontario, where the proposed Municipal Service Road will run about 45 m west of, and practically parallel to the southbound lanes of the proposed Highway 11. The bridge will cross over the Magnetawan River which flows in a northwesterly direction at this location. The crossing will be situated about 0.8 km south of Doe Lake Road and Three Mile Lake Road in Katrine, and about 7½ km south of Burk's Falls.

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

The findings of the investigation are presented in this report.

2. SITE DESCRIPTION AND GEOLOGY

The proposed bridge will be located in a grassed field, with bushes along the river, and there is a heavily forested and higher area approximately 50 m west of the site. The topography is hummocky, and the field is probably utilized for grazing. Drainage is towards the Magnetawan River which meanders both upstream and downstream from the site.

The proposed Municipal Service Road will be a new alignment, two-laned and asphalt paved. The ground surface elevation in the immediate vicinity of the bridge is between Elevation 295 m and 303 m, generally rising towards the north.

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 gravels, 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 (igneous) gneiss formations and is encountered at depths ranging from the ground surface to more than 50 m.

3. INVESTIGATION PROCEDURES

The fieldwork for the proposed bridge was performed during the period of May 22 through May 31, 2001, and consisted of drilling two deep boreholes (numbered RT 2 and RT 3) at each of the two abutment locations and three shallow boreholes (numbered RT 1, RT 1A and RT 4) below the approximately highest point of the approach embankments. Table 3.1 below summarizes the depth of boreholes.

Table 3.1 Depth of Boreholes

Borehole No.	RT 1	RT 1A	RT 2	RT 3	RT 4
Depth (m)	5.3	9.6	28.2	30.1	9.6

The boreholes were located in the field by Shaheen & Peaker Limited using for reference stakes installed at the planned abutment locations by Stantec. Geodetic elevations and horizontal control of the borehole locations (using stations and offset distances, and Ontario grid coordinates) were determined by Stantec after the completion of the field work.

The locations of the boreholes are shown on the Borehole Location Plan, Drawing No. 1.

The boreholes were advanced using a track mounted drilling rig outfitted for soil sampling and rock coring, and owned and operated by Groundworks Drilling Inc. The drilling of each borehole began by means of hollow-stem continuous flight power augers. When the saturated sand material was backing up into the hollow-stem augers,

casing was installed and the boreholes were advanced by the washboring method. In the deeper boreholes diamond-drilling method was also used to penetrate the boulder obstructions. To minimize disturbing the saturated fine sand deposits when taking samples, drilling mud was used at larger depths to counterbalance the external hydrostatic head and the sampler was withdrawn slowly. In spite of such measures, some inevitable disturbance may have reduced the recorded N-values.

Samples in the boreholes were taken at frequent intervals of depth by the Standard Penetration Test method (SPT), in general accordance with ASTM Method D1586. The SPT 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 the blows of the hammer required to drive the sampler into the relatively undisturbed ground by a vertical distance of 0.30 m is recorded as the Standard Penetration Resistance or the N-value of the soil. The N-values indicate the compactness condition of nonplastic/cohesionless soils (gravels, sands and silts) or the consistency of plastic/cohesive soils (clays and clayey soils).

In Boreholes RT 2 and RT 3, the saturated and cohesionless silty fine sand deposits were further explored by Dynamic Cone Penetration Tests (DCPT). In this test a 51 mm diameter, 60 deg. apex cone point, attached to the tip of A-size rods, is driven into the ground, using the same driving energy as in the SPT method. By recording the number of blows to drive the cone/rod assembly into the soil every 0.3 m, a qualitative record of subsoil density is obtained. Although the interpretation of the test results is difficult because soil samples are not obtained by the DCPT method and the penetration resistances are not the same as the N-values, useful information is gained by the continuity of the results and by the elimination of unbalanced hydrostatic effects which influence the SPT values.

Where the consistency of the soil permitted in the cohesive (clayey) deposits, relatively undisturbed samples (TW) were taken with 51 mm dia. thin-walled ("Shelby") tubes which were pushed into the bottom of the borehole by static weight or by hydraulic pressure. The undrained shear strength of the soil was measured in-situ by means of a field vane test using the MTO type field vane equipment.

The boreholes were advanced through boulders by diamond drilling using wire-line core barrel of NQ size (63.5 mm dia.).

For long-term observation of the ground water conditions, a sealed piezometer was installed in Borehole RT 3. Since the ground water level did not attain equilibrium condition in the other boreholes at the time of completion, the water levels were estimated from the condition of the recovered samples and from other field observations.

The soil profile and ground water level encountered in the boreholes, type of samples and sampling depths, N-values and vane test results, together with the coring data – where applicable - are presented on the Record of Borehole Sheets, in Appendix A of this report.

Upon their completion, the boreholes were backfilled to about 8 m below the ground surface with soils brought up by augering (i.e. auger cuttings). The upper 8 m of the open boreholes was then grouted using a cement/bentonite mixture.

The geotechnical index properties of selected representative samples were determined by standardized laboratory methods which included natural moisture content measurements, Atterberg (liquid and plastic) Limits tests, and grain-size analyses. The TW tubes were extruded and the unit weight of the samples was measured. The results of laboratory tests are presented on the appropriate Borehole Log Sheets and, also in Appendix B.

4. SUBSURFACE CONDITIONS

All the five boreholes were drilled in open field within 4 m from the centreline of the proposed underpass and Table 4.1 below presents an overview of the borehole locations.

Table 4.1 Overview of Borehole Locations and Elevations

Borehole No.	RT 1 A	RT 1	RT 2	RT 3	RT 4
Ground Elevation	295.6 m	295.6 m	295.1 m	294.9 m	295.1 m
Location	South Embankment	South Embankment	South Abutment	North Abutment	North Embankment

Distance
between Boreholes: ♦-----1.5 m -----♦----- ~ 20 m ----♦--- ~ 70 m --♦--- ~ 20 m ----♦

The boreholes were drilled on the south and north sides of Magnetawan River. At the borehole locations, the site is relatively flat and the ground surface elevations are within 1 m from one another, ranging from Elevation 295.6 m (Boreholes RT1 and 1A) to Elevation 294.9 m (Borehole RT3). Below a thin veneer of topsoil, a laminated/layered silty fine sand deposit was encountered. This deposit extended to about 5 m depth (Elevation 290.3 m) in the southernmost borehole (RT1A) while in Borehole RT2 located near the proposed south abutment location it was found to extend to about 25 m depth (Elevation 270.3 m). On the north side of the river, in Boreholes RT 3 and RT 4, the silty fine sand deposit is divided below depths of 2.2 and 0.7 m into upper and lower zones by clayey silt and silt strata of about 3 to 5 m thickness.

In Boreholes RT 1 and RT 1A, which were drilled on the south side but further away from the river, at Elevation 290.3 m similar clayey silt and silt strata of a total thickness of 4.3 m (possibly more) were encountered below the 5.3 m thick upper silty fine sand layer. These boreholes extended to a combined depth of only 9.6 m and the lower silty fine sand layer (which probably exists here too) was not encountered.

The silty sand layer was explored to its (probable) full depth in Boreholes RT 2 and RT 3, and at approximately 23 to 25 m depth (≈Elevation 270 to ≈272 m) a gravelly sand deposit, with silt and numerous cobbles and boulders was encountered. The boreholes were terminated in this stratum, at 28 to 30 m depth below existing ground level, corresponding to Elevation ≈267 to ≈265 m.

The ground water table was between 0.5 m and about 2 m depth below ground surface (corresponding to Elevation 294.4 and 293.4 m) at the time of the fieldwork.

The soil deposits encountered in the borings indicate that there are deep flood deposits along the Magnetawan River which probably originate from the predecessor of the present river. As the deep channel – or subterranean valley – was filled up with these sediments, the flow of water probably slowed gradually and the gradation of the sediments became finer. This geological history may explain the presence of silt and clayey silt layers in the upper zones of the soil.

As described on the Record of Borehole sheets, the soil deposits at the site are layered and when the thickness of layers in the recovered samples was observed to be less than 6 mm, the structure of the deposit is characterized as “laminated”. Due to the intermittent (i. e. non-continuous) sampling in the layered deposits encountered at the bridge site, the stratigraphic descriptions and boundaries should be considered approximate and of general nature only.

Summarized subsurface sections are shown on Drawing No. 1 while the details of the subsurface conditions encountered in the boreholes are presented on the Record of Boreholes, in Appendix A. The various soil strata and their characteristics encountered in the boreholes are briefly described in the following subsections of this report.

4.1. TOPSOIL

At three of the four borehole locations (locations RT 1 and RT 1A being considered as one) topsoil was encountered, ranging in thickness from 50 mm (at Borehole RT 4) to 100 mm (at Borehole RT 1). Although in Borehole RT 2 the topsoil was not clearly discernible, roots and finely dispersed organic matter was encountered

to about 0.9 m depth. The topsoil is sandy, and due to the organic content of the topsoil and the upper soil strata the tested samples had relatively high natural moisture contents ranging from 38 to 42%.

4.2 SILTY FINE SAND

Immediately below the topsoil the boreholes contacted a major deposit of silty fine sand which extended to 5.3 m depth (Elevation 290.3 m) in Borehole RT1A, to depths of 24.8 m (Elevation 270.3 m) and 23.3 m (Elevation 271.6 m) in Boreholes RT2 and 3, respectively and to the full depth (9.6 m) – possibly deeper - of Borehole RT4.

In Borehole RT3 and RT4, this deposit is separated at depths of 2.2 and 0.7 m, respectively, into an upper and a lower zone, due to the presence of 3.0 to 5.2 m thick clayey silt and silt deposits. Borehole RT1A was terminated in the silt deposit at a depth of 9.6 m, without encountering the lower zones of the silty fine sand.

The upper zones of this deposit are layered and contain traces of organic matter. The natural moisture contents of samples recovered from these upper zones range from 10 to 36%; the higher values are attributed to the presence of organic matter. The upper silty fine sand is generally very loose to loose, as indicated by N-values which generally range from 1 to 11 blows/0.3 m.

The N-values recorded below this upper zone generally range from 4 to 34 with an average value of about 11 blows for 0.3 m penetration. (When calculating the average N-value, blows of 3 or less were not included because they were probably reduced by groundwater effects and a high value of 58 blows was also disregarded because it was caused by coarse gravel particles.) Generally, the N-values increased with depth especially in Borehole RT3 which was drilled on the north side of the river.

The lower fine sand is entirely below the groundwater table, and is in a wet condition. Many samples behaved like a heavy liquid indicating that the material lacks cohesion. The water contents measured in the laboratory generally ranged from 18% to 25%. A sample with some gravel size particles (Borehole RT2 Sample 16) had 14% water content.

The silty fine sand is a cohesionless, fine-grained granular material. The grain size distribution envelope of samples from the deposit is given in Figure 1, Appendix B. These indicate the following range:

Gravel:	0
Sand:	45-89%
Silt:	11-55%
Clay:	~0%

The presence of occasional fine sandy silt to silt zones or layers was noted in the deposit.

4.3 CLAYEY SILT

A 0.7 to 2.5 m thick layer of grey clayey silt, which is a cohesive material, was encountered, sandwiched between the upper and lower zones of the silty fine sand deposit in Boreholes RT 1A, RT 3 and RT 4. The clayey silt deposit was contacted at depths ranging between 0.7 m and 5.3 m below the ground surface or between Elevations 294.4 and 290.3 m and extended to depths of 3.2 to 6.0 m below the ground surface or to Elevations 291.9 to 289.6 m.

The clayey silt has the following average grain size distribution characteristics, and liquid and plastic limits (see Appendix B, Figures 2 and 3).

Gravel:	0 %	
Sand:	16 %	(range: 13 – 18 %)
Silt:	70 %	(range: 62 – 74%)
Clay:	14 %	(range: 10 – 20 %)

Liquid Limit:	24%	(range: 22 – 26 %)
Plastic Limit:	19 %	(range: 18 – 21 %)
Plasticity Index:	5 %	(range: 4 – 5 %)
Liquidity Index*:	1.6	(range: 0.6 – 2.8)

*Using the water content of the Atterberg Limits test samples.

The average natural water content of the tested samples was 29% and ranged from 22 to 38 %. Some water contents were probably increased by the presence of organic matter in the clayey silt deposit.

N-values recorded in this deposit ranged from 1 to 22 blows/0.3 m, but were generally 3 to 12 blows. The undrained shear strength of the clayey silt was measured in-situ by means of a field vane test which gave a value of about 60 kPa. Based on these results, the consistency of the clayey silt is described as very soft to very stiff but generally stiff. The sensitivity of the clayey silt was measured to be 4 with the field vane test.

4.4 SILT

With the exception of Borehole RT2, a silt layer was encountered at all borehole locations. The silt, which was contacted beneath the clayey silt deposit, is grey, laminated, contains thin layers of clay and fine sand. The thickness of the deposit ranges from 1.5 m to in excess of 3.6 m. It was encountered at depths ranging from 3.2 to 6.0 m below the ground surface (Elevations 291.9-289.6 m) and extended to depths ranging from 5.2 m to in excess of 9.6 m.

Generally, the silt is a non-plastic material although clay laminations could lend some plasticity to it in some areas. The presence of some sandy layers was also noted. Four grain size distribution curves gave the following average results (see Appendix B, Figure 4).

Gravel:	0 %
Sand:	19 % (range: 5 – 43%)
Silt:	81% (range: 56 – 95%)
Clay:	~0 % (range: 0 - ~1%)

The silt is wet and the average natural water content of samples from the deposit was measured to be 24%, ranging between 16 and 33%. The wide range of the water content measurements is due to the layered structure of the deposit: the sandy layers had lower water contents while the clayey layers had higher. Two bulk density measurements gave 19.6 and 19.0 kN/m³ values.

The standard penetration test results yielded an average N-value of 8 blows per 0.3 m penetration (range: 4 to 18 blows). The uniform gradation and cohesionless nature of the silt deposit makes it very susceptible to disturbance by groundwater effects therefore one N-value of 3 blow per 0.3 m was not included in the average. In our opinion, despite the use of drilling mud, disturbance caused by the ground water probably reduced the measured N-values therefore the silt layer can be considered as generally loose to compact.

4.5 GRAVELLY SAND WITH SILT, COBBLES AND BOULDERS

The lowest lying deposit explored in this investigation is a cohesionless material consisting of gravelly sand, with some silt content and frequent cobbles and boulders. It was reached at 24.8 and ~23.3 m depths (i.e. at El. 270.3 and ~ 271.6 m), respectively, in Boreholes RT 2 and RT 3, and was explored to 3.4 m and ~6.9 m depths. The grain size distribution of a sample from the deposit is presented in Figure 5 in Appendix B. The N-values ranged widely between 9 blows for 0.3 m penetration, and 60 blows for 13 cm penetration. The low values were probably caused by hydrostatic unbalance in the casing while the high ones were obtained when the sampler hit a coarse gravel, cobble or boulder. Generally, the boulders could only be penetrated by diamond drilling. In our opinion, the gravelly sand deposit is generally dense to very dense which is also indicated by the dynamic cone penetration tests which were performed in the bottom of both boreholes: within 0.6 m penetration well over one hundred blows were required to drive the cone 0.3 m, and the tests had to be discontinued. These short dynamic cone penetration tests also signaled termination of the exploration, at 28.2 m depth (Borehole RT 2) and 30.2 m depth (Borehole RT 3).

4.6 GROUNDWATER

With the exception of Borehole RT 3 - where a piezometer was installed - the groundwater levels did not stabilize in the boreholes on completion of the boring therefore the depth of the groundwater levels was estimated from the moisture condition of the recovered samples or from observations during drilling.

In the piezometer installed in Borehole RT 3, the water level readings were taken every working day from May 25 until June 1, 2001. The three last readings on May 29th and 31st, and on June 1st, were the same: 0.5 m below ground surface which corresponds to El. 294.4 mm. The estimated depth of water levels in the three remaining boreholes ranged from 0.8 to 2.2 m, i.e. from Elevation 293.4 m to Elevation 294.3 m. The groundwater level may be influenced to some extent by the water level in Magnetawan River. It is of interest to note that the water level in the River was at Elevation 293.38 m in April 2001. (This information was obtained from the General Arrangement drawing prepared by Stantec.)


It should be noted that the groundwater levels can be expected to fluctuate with the season of the year and with weather events.

Yours truly

SHAHEEN & PEAKER LIMITED



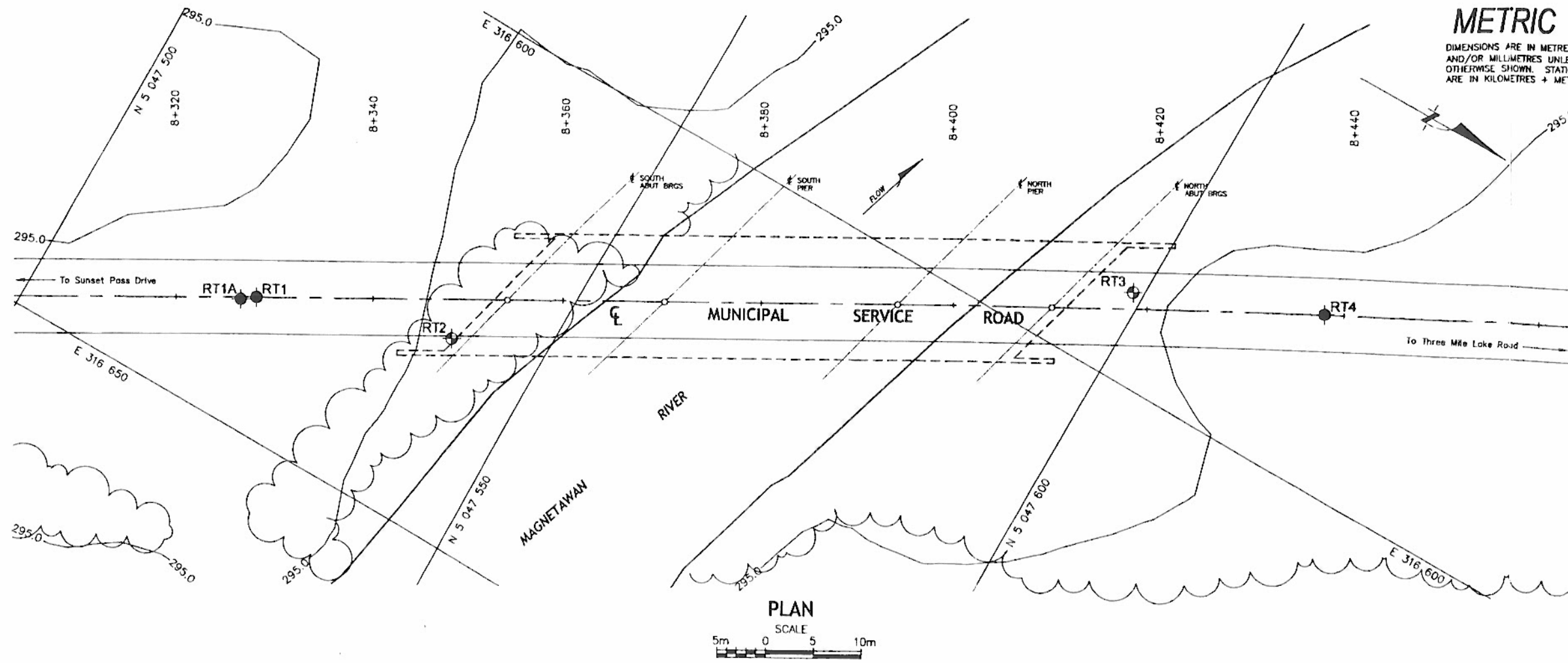
Z. S. Ozden, P.Eng



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



DRAWINGS



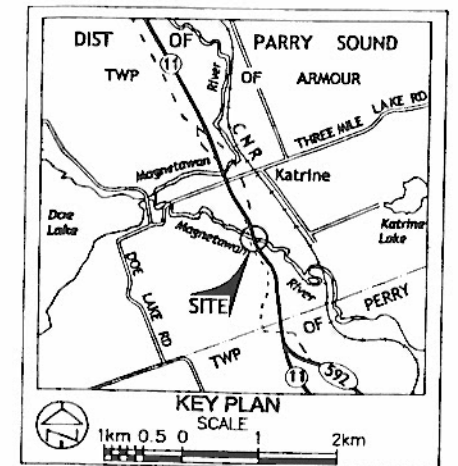
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
ARE IN KILOMETRES + METRES.

CONT No
WP No 314-99-00

HIGHWAY 11
MUNICIPAL SERVICE ROAD
OVER MAGNETAWAN RIVER
BORE HOLE LOCATIONS & SOIL STRATA

SHEET

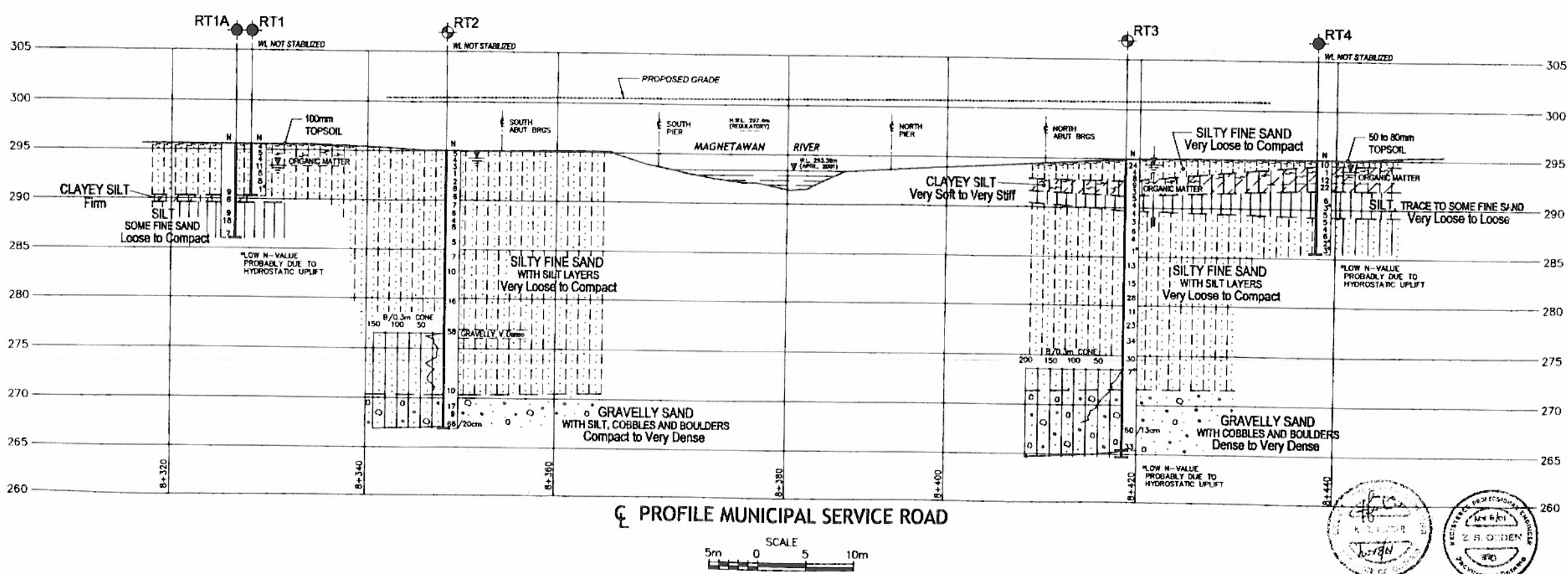
Shaheen & Peaker Limited



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
May and June 2001
- W L in Piezometer
- Piezometer

No	ELEVATION	CO-ORDINATES NORTH	EAST
RT1	295.6	5 047 521.2	316 637.1
RT1A	295.6	5 047 519.9	316 637.9
RT2	295.1	5 047 540.6	316 630.7
RT3	294.9	5 047 598.9	316 591.2
RT4	295.1	5 047 616.9	316 583.3



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
1	Nov 6/01	J.T.W.	Initial Design

Geocres No.

HWY No 11

SUBD LRS CHECKED 20 DATE Nov., 2001 SITE

OWNR JTW CHECKED IP APPROV

APPENDIX A

Records of Boreholes

RECORD OF BOREHOLE No RT1

1 OF 1

METRIC

W.P. 314-99-00 LOCATION Municipal Service Rd. Crossing over Magnetawan River-Coords N 5 047 521.2; E 316 637.1 ORIGINATED BY R.A.
DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augering COMPILED BY G.T.
DATUM Geodetic DATE 25.05.01 CHECKED BY LSR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE										W _p	W	W _L
295.6	Ground Surface						20	40	60	80	100	20	40	60		GR SA SI CL				
0.0	100 mm Topsoil	loose to very loose moist ----- wet loose to compact	1	SS	4	▽										0 60 (40)				
			2	SS	5															
			3	SS	4															
	SILTY FINE SAND with organic matter to 2.2 m, layered		4	SS	11															
			5	SS	8															
			6	SS	8															
			7	SS	1		**													
290.3																**SS7: Low N-value probably due to hydrostatic uplift				
5.3	End of borehole Borehole abandoned because of sand backup in hollow stem augers For continuation of BH RT1 see BH RT1A Ground water not stabilized on completion of boring *Ground water level estimated from moisture condition of soil samples																			

RECORD OF BOREHOLE No RT2

1 OF 3

METRIC

W.P. 314-99-00 LOCATION Municipal Service Rd. Crossing over Magnetawan River-Coords N 5 047 540.6; E 316 630.7 ORIGINATED BY R.A.
DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augering, Washboring, NQ Rock Coring & D.C.P.T. COMPILED BY G.T.
DATUM Geodetic DATE 28.05.01 to 30.05.01 CHECKED BY LSR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
FLYV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE						
295.1	Ground Surface							20 40 60 80 100						
0.0														
	rootlets	damp	1	SS	5		295							
	-----	wet	2	SS	4		294							
			3	SS	3		293							0 70 30 0
			4	SS	1		292							
			5	SS	2		291							
	brown		6	SS	6		290							
	grey organic silt layers		7	SS	6		289							
	-----		8	SS	7		288							
	grey		9	SS	6		287							
			10	SS	4		286							
			11	SS	8		285							
			12	SS	5		284							
			13	SS	7		283							
			14	SS	10		282							
							281							

Continued Next Page

+ 3 × 3 : Numbers refer to
Sensitivity

20
15 10 5
(%) STRAIN AT FAILURE

0 45 (55)
HST Augering
Washboring

RECORD OF BOREHOLE No RT3

1 OF 3

METRIC

W.P. 314-99-00 LOCATION Municipal Service Rd. Crossing over Magnetawan River-Cords N 5 047 598.9; E 316 591.2 ORIGINATED BY R.A.
DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augering, Washboring, NQ Rock Coring & D.C.P.T. COMPILED BY G.T.
DATUM Geodetic DATE 22.05.01 to 24.05.01 CHECKED BY LSR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W _p	W	W _L		
294.9	Ground Surface							20 40 60 80 100						
0.0	80 mm Topsoil	cobbles	1	SS	24									
	SILTY FINE SAND layered, very loose to loose	brown moist wet grey	2	SS	4		294							
	traces of organic matter		3	SS	8		293							
292.7	CLAYEY SILT some sand traces of organic matter, very soft to stiff		4	SS	9		292							0 13 74 13
	grey wet		5	SS	3									
291.2	SILT some fine sand laminated, loose to very loose		6	SS	5		291							
	grey wet		7	SS	4		290							0 13 87 0
289.7			8	SS	4		289							
5.2	SILTY FINE SAND with silt layers, grey wet		9	SS	3		288							Hollow Stem Augering
			10	SS	6		287							Washboring
			11	SS	4		286							
			12	SS	1		285							SS12: Low N-value probably due to hydrostatic uplift
		very loose to loose compact	13	SS	13		284							
			14	SS	15		283							
			15	SS	28		282							
279.9							281							0 83 17 0
							280							

15.0 Continued Next Page

+ 3, x 3. Numbers refer to
Sensitivity 20
15-5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No RT3

2 OF 3

METRIC

W.P. 314-89-00 LOCATION Municipal Service Rd. Crossing over Magnetawan River-Coords N 5 047 598.9; E 316 591.2 ORIGINATED BY R.A.
DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augering, Washboring, NO Rock Coring & D.C.P.T. COMPILED BY G.T.
DATUM Geodetic DATE 22.05.01 to 24.05.01 CHECKED BY LSR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60		
279.9												
15.0			16	SS	11							
							279					
			17	SS	23		278					
	SILTY FINE SAND with silt layers compact to dense grey wet						277					
			18	SS	34		276					
							275					
			19	SS	30		274					
							273					
			20	SS	7		272					
							271					
271.6	probable lower boundary of silty sand deposit						270					
23.3							269					
							268					
	GRAVELLY SAND with cobbles and boulders, dense to very dense, grey, wet		21	SS	60/13		267					
							266					
							265					
265.4			22	SS	33							
29.5	End of borehole											
264.9												

Continued Next Page

+ 3, x 3: Numbers refer to
Sensitivity

20
15 10 5
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No RT3

3 OF 3

METRIC

W.P. 314-99-00 LOCATION Municipal Service Rd. Crossing over Magnetawan River-Coords N 5 047 588.9; E 316 591.2 ORIGINATED BY R.A.
DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augering, Washboring, NQ Rock Coring & D.C.P.T. COMPILED BY G.T.
DATUM Geodetic DATE 22.05.01 to 24.05.01 CHECKED BY LSR

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa							WATER CONTENT (%)	
264.9							20	40	60	80	100	20	40	60	kN/m ³	GR SA SI CL
264.7							20	40	60	80	100	20	40	60		
30.2	End of Dynamic Cone Penetration Test Dynamic Cone Penetration Test performed from 21.3 m to 26.8 m, Soil stratigraphy inferred only Dynamic Cone Penetration Test performed from 29.6 m to 30.2 m Piezometer installed to 6.7 m Stabilized ground water level in piezometer at 0.5 m (May 29, 30 and June 01/2001)															

+ 3, x 3: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No RT4

1 OF 1

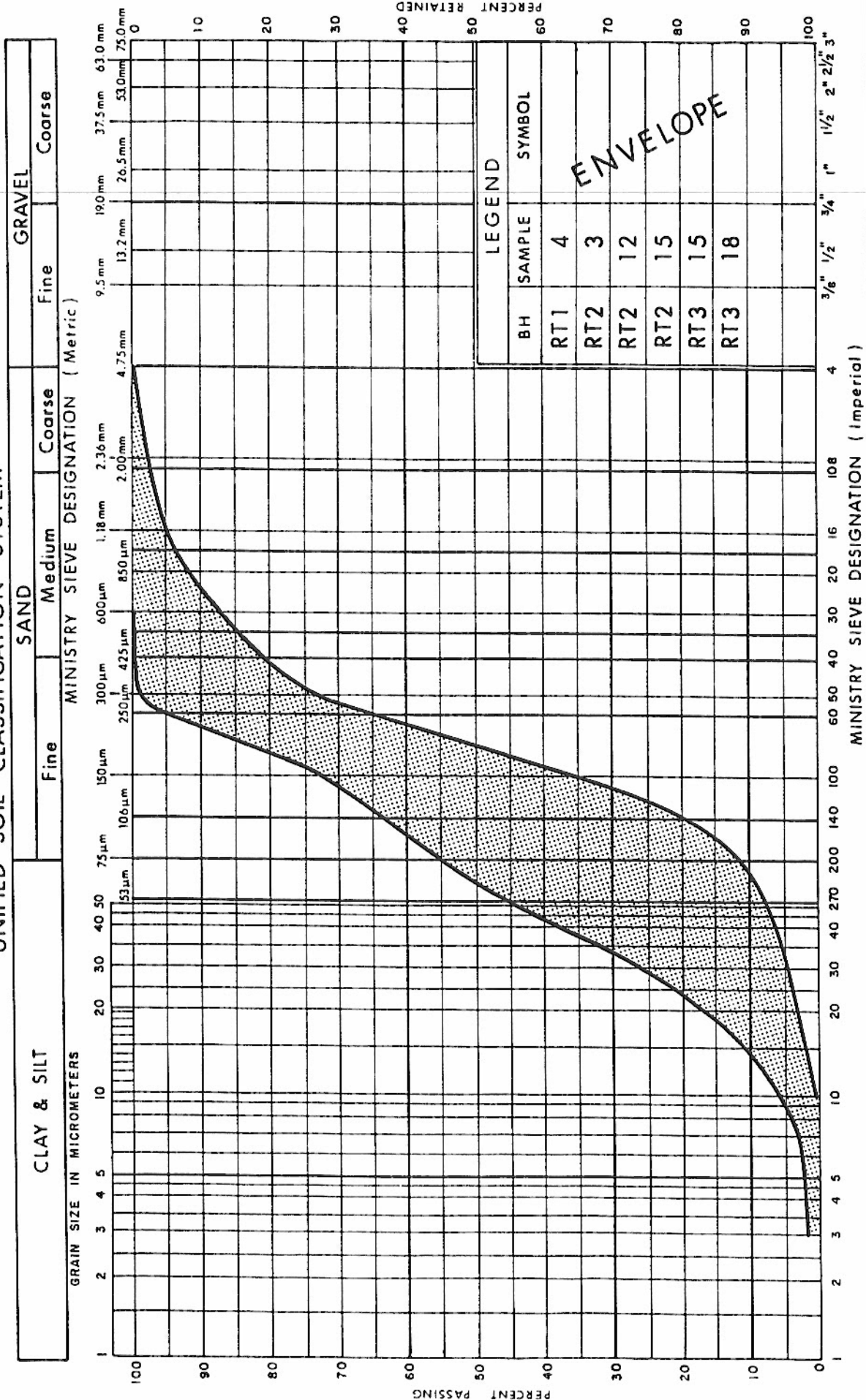
METRIC

W.P. 314-99-00 LOCATION Municipal Service Rd. Crossing over Magnetawan River-Coords N 5 047 616.9, E 316 583.3 ORIGINATED BY R.A.
DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augering COMPILED BY G.T.
DATUM Geodetic DATE 24.05.01 CHECKED BY LSR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
295.1	Ground Surface													
0.0	50 mm Topsoil		1	SS	10		295							
294.4	SILTY FINE SAND compact, brown, damp													
0.7			2	SS	1		294							
	very soft													
	stiff		3	SS	12		293							0 17 73 10
	with organic matter													
	CLAYEY SILT		4	SS	22		292							
	trace sand and gravel, grey, wet													
291.9			5	TW	PH		291						19.6	0 5 95 0
3.2	SILT		6	SS	6		290							
	trace fine sand, loose, grey, wet		7	SS	3**		289							
			8	SS	5		288							0 43 56 1
289.2							287							
5.9			9	SS	5		286							
	SILTY FINE SAND		10	SS	4									** SS7, SS12 and SS13 Low N-value probably due to hydrostatic uplift
	very loose to loose grey, wet		11	SS	6									
			12	SS	2**									
			13	SS	3**									
285.5														
9.6	End of borehole Ground water level not stabilized on completion of boring *Ground water level estimated from moisture condition of SS sampler and soil samples													

APPENDIX B

Laboratory Test Results

Ministry of
Transportation

GRAIN SIZE DISTRIBUTION

SILTY FINE SAND

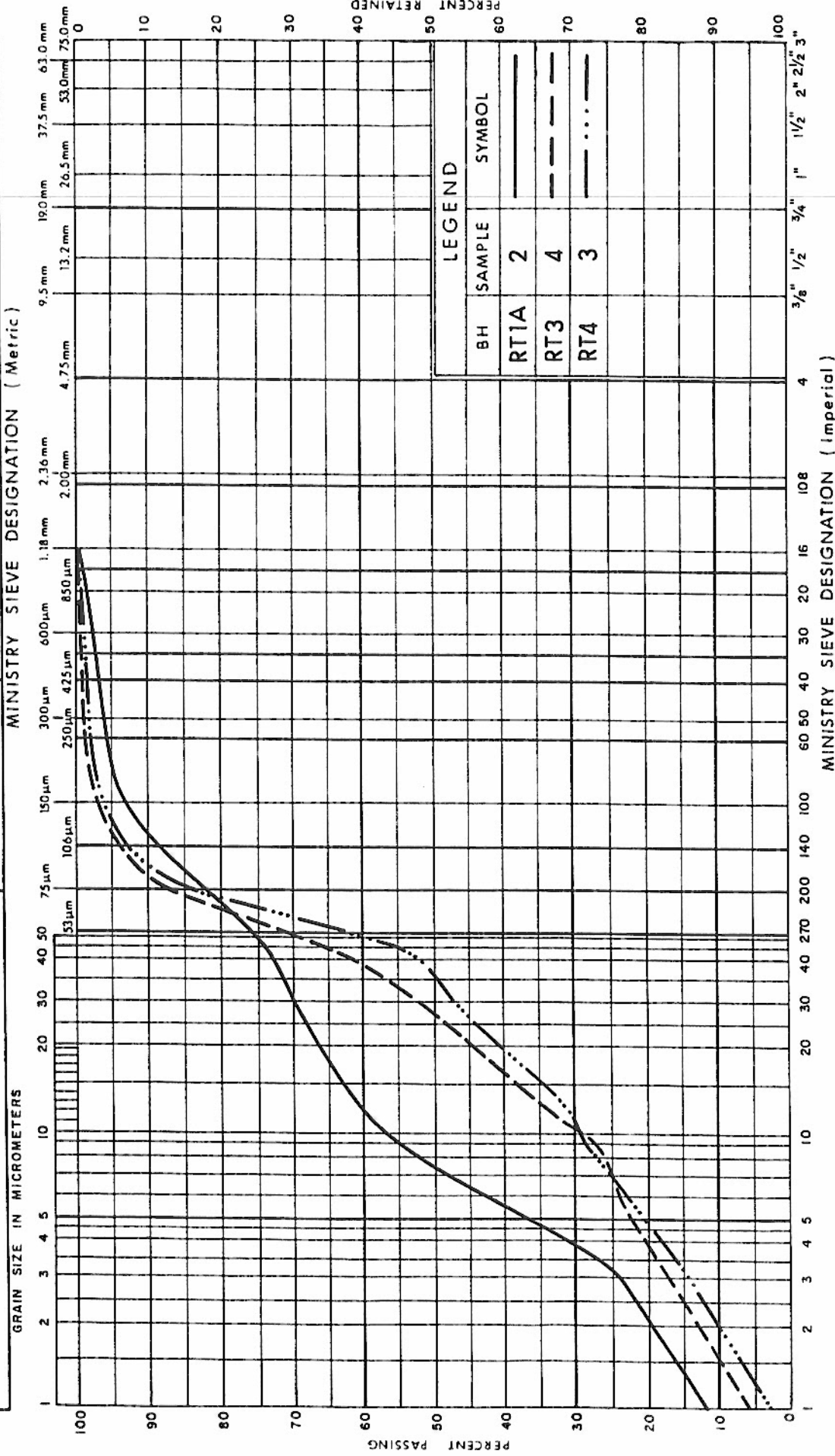
FIG No 1

WP 314-99-00

SPT 1010A

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT		SAND			GRAVEL		
		Fine	Medium	Coarse	Fine	Coarse	



GRAIN SIZE DISTRIBUTION

CLAYEY SILT, TRACE FINE SAND

Ministry of
Transportation



FIG No 2

W P 314-99-00

SPT 1010A

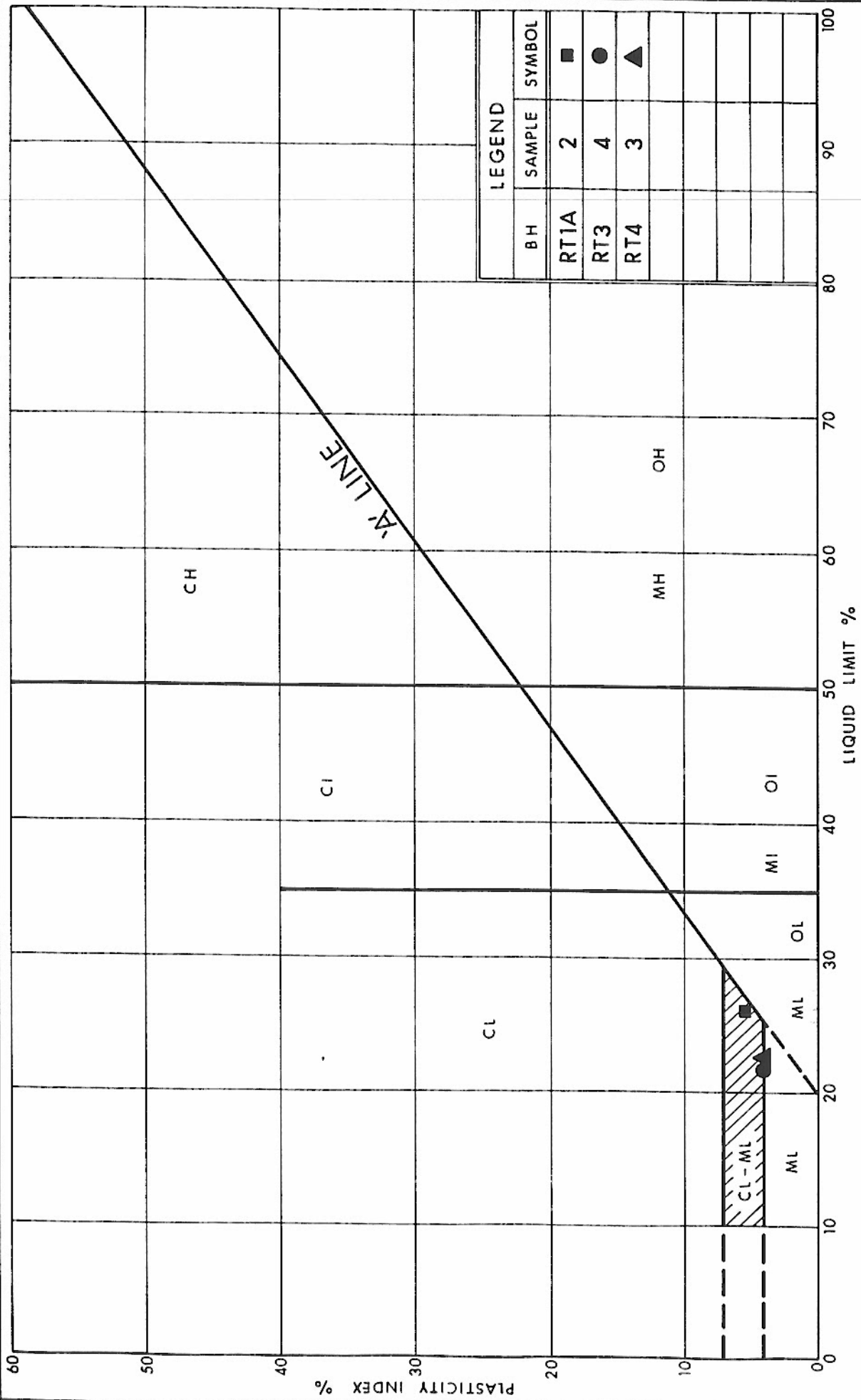


FIG No 3

W P 314-99-00

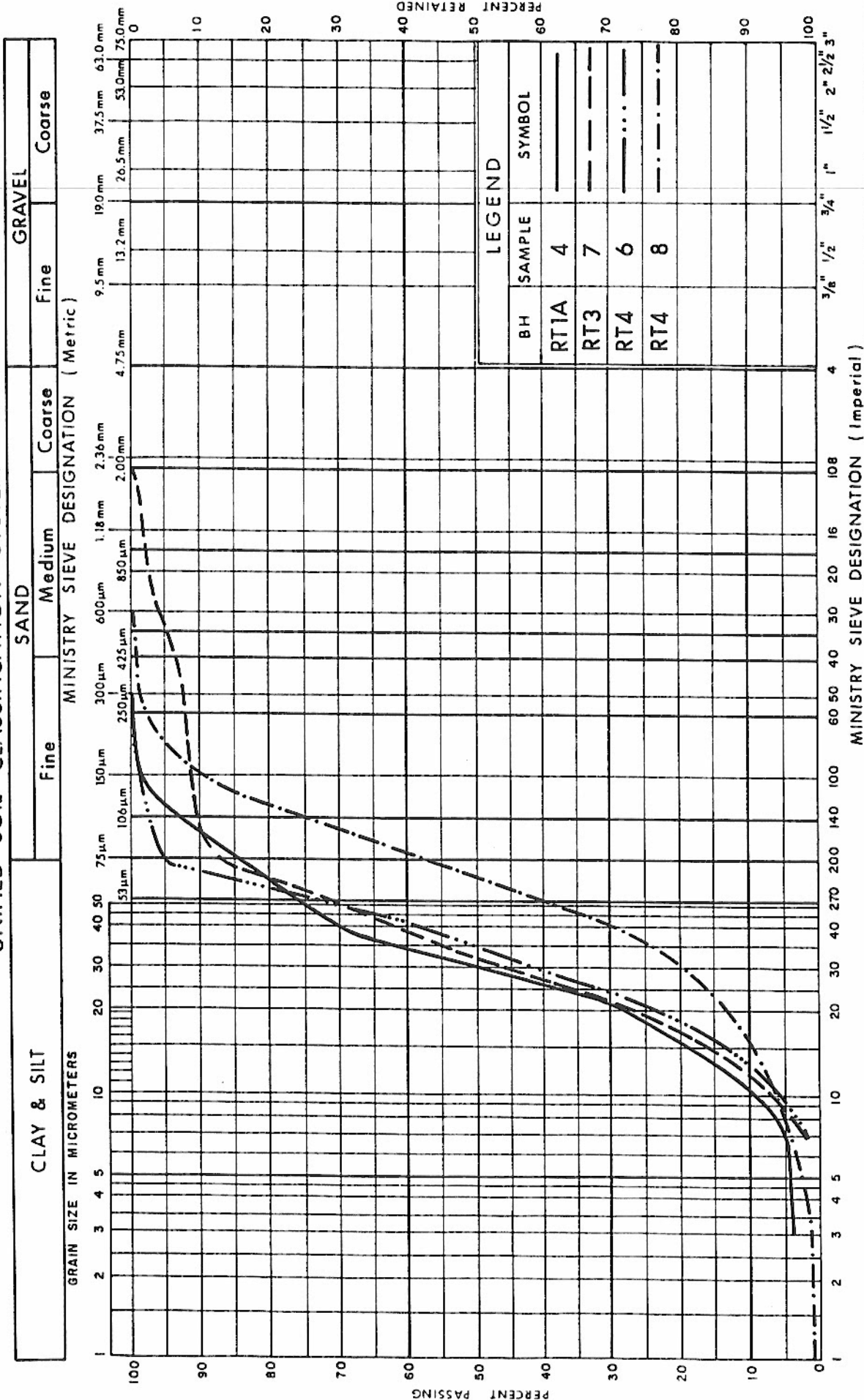
SPT 1010A

Ministry of
Transportation



CLAYEY SILT, TRACE FINE SAND

UNIFIED SOIL CLASSIFICATION SYSTEM



GRAIN SIZE DISTRIBUTION

SILT, SOME FINE SAND

FIG No 4

W P 314-99-00

SPT 1010A

Ministry of
Transportation

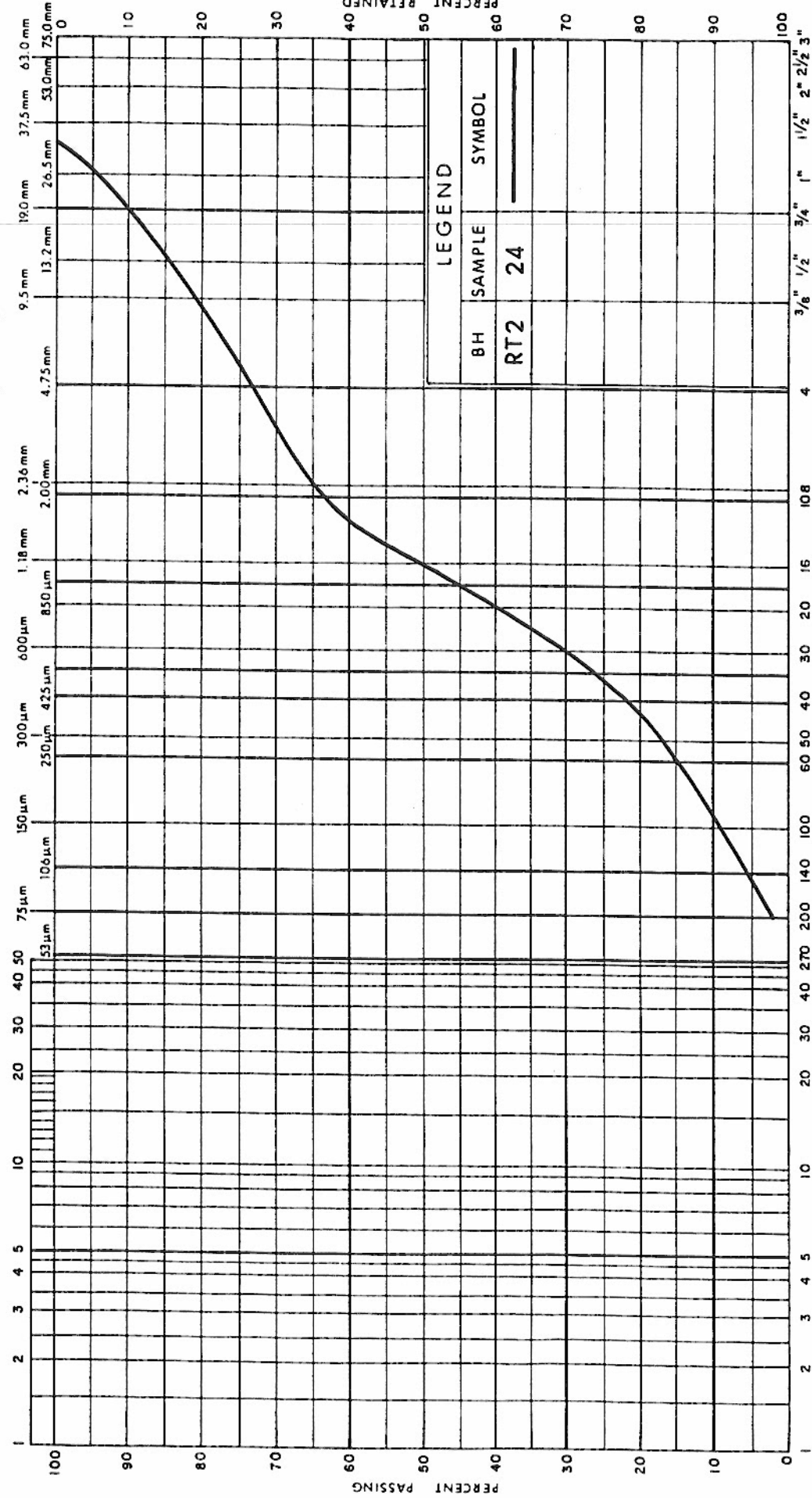


UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT		SAND			GRAVEL		
		Fine	Medium	Coarse	Fine	Coarse	

GRAIN SIZE IN MICROMETERS

MINISTRY SIEVE DESIGNATION (Metric)



LEGEND

BH	SAMPLE	SYMBOL
RT2	24	—

MINISTRY SIEVE DESIGNATION (Imperial)

GRAIN SIZE DISTRIBUTION

GRAVELLY SAND

FIG No 5

W P 314-99-00

SPT 1010A

Ministry of
Transportation



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/0.3m)	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 (RQD), 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
U		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		COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa ⁻¹	COEFFICIENT OF VOLUME CHANGE
C_c		COMPRESSION INDEX
C_s		SWELLING INDEX
C_α		RATE OF SECONDARY CONSOLIDATION
C_v	m ² /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v		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		SENSITIVITY = $\frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m ³	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	e_{min}	1, %	VOID RATIO IN DENSEST STATE
γ_s	kN/m ³	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	I_D		DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
ρ_w	kg/m ³	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kN/m ³	UNIT WEIGHT OF WATER	S_r	%	DEGREE OF SATURATION	D_n	mm	n PERCENT - DIAMETER
ρ	kg/m ³	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u		UNIFORMITY COEFFICIENT
γ	kN/m ³	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
ρ_d	kg/m ³	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m ³ /s	RATE OF DISCHARGE
γ_d	kN/m ³	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $w_L - w_p$	v	m/s	DISCHARGE VELOCITY
ρ_{sat}	kg/m ³	DENSITY OF SATURATED SOIL	I_L		LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	i		HYDRAULIC GRADIENT
γ_{sat}	kN/m ³	UNIT WEIGHT OF SATURATED SOIL	I_C		CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	k	m/s	HYDRAULIC CONDUCTIVITY
ρ'	kg/m ³	DENSITY OF SUBMERGED SOIL	e_{max}	1, %	VOID RATIO IN LOOSEST STATE	j	kg/m ²	SEEPAGE FORCE
γ'	kN/m ³	UNIT WEIGHT OF SUBMERGED SOIL						

**PRELIMINARY FOUNDATION DESIGN REPORT
PROPOSED HIGHWAY 11 REALIGNMENT
MUNICIPAL SERVICE ROAD BRIDGE
OVER MAGNETAWAN RIVER
KATRINE, ONTARIO
W.P. 314-99-00**

Prepared For:

STANTEC CONSULTING LTD.

Prepared by:

SHAHEEN & PEAKER LIMITED

**Project: SPT1010A
November 8, 2001**

**250 Galaxy Boulevard
Etobicoke, Ontario
M9W 5R8
Tel: (416) 213-1255
Fax: (416) 213-1260**

Table of Contents

5.	DISCUSSION AND RECOMMENDATIONS	12
5.1	General.....	12
5.2	Foundations.....	14
5.2.1	Spread Footing Foundations.....	14
5.2.2	Deep Foundations	14
5.3	Lateral Earth Pressures.....	19
5.4	Approach Embankments	20
5.5	Construction Comments	22
5.6	Frost Protection	23
6.	CLOSURE	23

APPENDIX

LIMITATIONS OF REPORT

APPENDIX D

**PRELIMINARY FOUNDATION DESIGN REPORT
PROPOSED HIGHWAY 11 REALIGNMENT
MUNICIPAL SERVICE ROAD BRIDGE OVER MAGNETAWAN RIVER
KATRINE, ONTARIO
W.P. 314-99-00**

5. DISCUSSION AND RECOMMENDATIONS

5.1 GENERAL

The new Highway 11 will consist of a four-lane divided roadway with a typically 30 m wide median. Immediately south of Katrine, the existing Highway 11 will be used for the southbound lanes of the new highway while a new two-lane roadway will be built west of the southbound lanes to serve as a municipal service road which will provide access to the near-by communities. At the location where this service road will cross Magnetawan River a new bridge will have to be built.

According to preliminary arrangement drawings prepared by and received from Stantec, the south and north abutments of the proposed bridge will be at Sta. ~8+354 and ~8+410, respectively. The skew angle between the roadway and the Magnetawan River will be approximately 45°. The planned total length (including the approach slabs at the two ends of the bridge) and total width of the proposed bridge is 68 m and ~12.5 m, respectively. The structure will have three girders spanning 16, 24 and 16 m length. As presently envisaged, the girders will be supported by two abutments with expansion joints and two intermediate piers with fixed supports, and the girders will consist of prestressed precast concrete beams of a structural height of 1400 mm. If the girders will be interconnected to act as continuous beams, the structure will be sensitive to differential settlements. Due to the skew of the structure, the abutment foundations will probably not be designed and constructed as "integral abutments." The elevations of the top of pavement at the supports of the municipal service road bridge are shown in Table 5.1.1 below.

Table 5.1.1
Bridge Pavement Elevations at Support Points

SUPPORT	MUNICIPAL SERVICE ROAD BRIDGE	
	Station – TOP OF PAVEMENT -- Elevation	
South Abutment	8 + 354.200 m	300.290 m
South Pier	8 + 370.200 m	300.450 m
North Pier	8 + 394.200 m	300.690 m
North Abutment	8 + 410.200 m	300.850 m

The existing ground level at the south abutment is about Elevation 295.1 m therefore the south approach embankment will be about 5 m high. The existing ground level is slightly lower – about Elevation 294.4 m - at the north abutment and the road grade is rising to the north therefore the north approach embankment will be about 6 m high. Our discussion and analysis are based on this conceptual design.

The boreholes indicate that the proposed bridge will be built at a site underlain by a 23 to 25 m deep loose to compact silty fine sand deposit which is divided in some areas into an upper and a lower zone by clayey silt and silt strata of about 3 to 5 m (possibly more) thickness. The thick silty fine sand layer, including the clayey silt and silt layers, was probably deposited by the predecessor of the Magnetawan River and by the glacial Lake Algonquin. Below the silty fine sand deposit, at 23 to 25 m depth (~ Elevation 270 to 272 m), a generally dense to very dense gravelly sand deposit was encountered which was explored to a depth of about 5 m near the proposed abutment locations. Bedrock was not encountered within the total explored 28 to 30 m depth of the boreholes.

The ground water table was between 0.5 m and 2 m depth below the existing ground surface at the time of the fieldwork in May – June 2001.

5.2 FOUNDATIONS

5.2.1 SPREAD FOOTING FOUNDATIONS

The findings in the boreholes indicate that the silty fine sand deposit is in a very loose to compact state to about 10 to 12 m depth, and the clayey silt and silt layers encountered are generally of stiff to soft consistency or are loose, respectively. As a result, the allowable bearing pressure is very low which would result in uneconomically large foundations, and construction costs would further increase due to dewatering. Further, spread footings could settle differentially which could be detrimental to a continuous structure. For these reasons, normal spread footings placed on the natural soil deposits or on compacted Granular 'A' pad are not recommended, based on cost and reliability.

5.2.2 DEEP FOUNDATIONS

5.2.2.1 DRIVEN STEEL H-PILES AND STEEL TUBE PILES

Driven steel H-piles and concrete filled steel tube piles are feasible options. The two deep boreholes (RT 2 and RT 3) encountered a generally dense to very dense and bouldery granular deposit at about 25 m depth below ground surface, corresponding to about Elevation 270 m. In our opinion, the piles should develop satisfactory bearing resistance when penetrating into this deposit. Table 5.2.2.1.1 indicates the anticipated tip elevation of the piles. These values should be confirmed during the detailed foundation investigation.

Table 5.2.2.1.1

Type of Pile	Anticipated Penetration into Dense to Very Dense Stratum	Anticipated Pile Tip Elevation	Anticipated Pile Length Below Existing Ground Surface
Steel H-pile	2 to 4 m	El. 268 to 266 m	27 to 29 m
Concrete Filled Tube Pile	1 to 3 m	El. 269 to 267 m	26 to 28 m

The recommended preliminary axial resistances for 310x110 steel H-piles are shown below.

Recommended Factored Axial Resistance at U. L. S: 1650 kN (per pile)
Recommended Axial Resistance at S. L. S: 1100 kN (per pile)

A heavy H-pile section (e.g. HP 310x110) should be selected due to anticipated tough driving conditions through cobbles and boulders until satisfactory resistance is attained. Also, the H-piles should be provided with reinforced tips as per MTO specifications (OPSD 3301.00).

Closed end steel tube piles filled with concrete after driving may also be considered. Such piles would provide a lower resistance than H-piles as shown below but they have the advantage that they can be inspected after driving and prior to placing concrete. The tube piles should have sufficient wall thickness and base plate thickness to minimize potential damage caused by the expected hard driving conditions.

Closed-end steel tube piles (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 1400 kN and an Axial Resistance S.L.S. equal to 900 kN. It may be possible to increase these values, depending on the findings of the detailed investigation.

The use of steel H-piles is the preferred alternative at this site, based on previous experience with similar projects.

5.2.2.1.1 DRIVING OF PILES

The piles will need to be driven using a suitably heavy hammer capable of delivering a rated energy of at least 55 kJ/blow, but not more than 70 kJ/blow. The driving of the piles in the field should be controlled by a recognized pile driving formula, such as the Hiley Formula. The estimated ultimate resistance of the piles based on 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. (For instance, using this criterion, the estimated ultimate axial resistance for steel H-piles as per the Hiley Formula is $1650 \div 0.5 = 3300$ kN. Again as per the Hiley Formula, the estimated ultimate axial resistance for 300 mm nominal diameter steel tube piles is $1400 \div 0.5 = 2800$ kN.)

In accordance with the above criterion, the piles should be driven to about 3 m above the design elevation and then the driving can be 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 heaving occurs, the heaved piles will have to be re-driven. As well, re-tapping, to check that relaxation has not occurred, will be necessary. In accordance with the Special Provision SP903S01 – Construction Specification for Piling – at least 10% of the piles (but not less than two piles) driven at each support element should be re-tapped not less than 24 hours after the driving of the pile, to check that relaxation has not occurred. If it has then all the piles should be re-tapped. To minimize the potentially adverse effect of driving piles on adjacent piles, 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 estimated values and therefore this aspect will have to be considered in the contract documents and when ordering the piles. In addition, the piles may 'hang-up' on cobbles and boulders which were generally encountered below about 24 m depth (Elevation 271 m).

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. Steel tube pile must be inspected for possible damage at the end of installation, before pouring the concrete.

5.2.2.1.2 BATTERED PILES

The general arrangement drawing indicates that lateral loads will be resisted by battered piles. For practical reasons and ease of driving the batter of piles should not be flatter than 1(H) to 4 (V).

5.2.2.1.3 GENERAL COMMENTS ABOUT STEEL TUBE AND H-PILES

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

To accommodate the grade of the new structure, approach embankments of about 5 to 6 m height will be required. Induced stresses, due to the weight of the fill placed for the approach embankments, will cause settlement of the underlying soils which will then transfer loads by negative skin friction to the piles, thus causing some down-drag on the piles. In order to minimize this down-drag, and to minimize lateral loads on the piles from the lateral yield of the silts and clayey silts at the abutments, the embankments should be placed to their subgrade elevations about three weeks ahead of pile driving at both abutments. This measure should also pre-consolidate the soil deposits below the approach embankments and improve the performance of the paved highway near the bridges.

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

In the event that the general arrangement of the bridge is changed and integral abutments are designed, the lateral resistance and deflection of the piles may need to be analyzed. In most cases, these depend on whether the soil surround is granular (i.e. cohesionless) or cohesive. For the analysis, the silty fine sand deposit and the silt layers could be considered as granular deposits while the clayey silt deposit should be considered as a cohesive material.

For calculating the lateral resistance/deflection of the piles in granular (i.e. cohesionless) soils (in the silty fine sand and in the silt), the coefficient of horizontal subgrade reaction can be obtained from the following expression:

$$k_s = n_h z / d$$

where k_s = coefficient of horizontal subgrade reaction kN/m^3
 z = depth m
 d = pile width m
 n_h = coefficient related to soil density as given in Table 5.2.2.1.3.1 kN/m^3

For calculating the lateral resistance/deflection of the piles in cohesive soils (i.e. in the clayey silt deposit) the coefficient of horizontal subgrade reaction can be obtained from the formula below.

$$k_s = 67 c_u / d$$

where k_s = coefficient of horizontal subgrade reaction kN/m^3
 c_u = undrained shear strength of soil kPa
 d = pile width m

For the appropriate values of n_h and c_u see Table 5.2.2.1.3.1. Also presented in the same table are estimated values for angle of internal friction and bulk unit weights.

Table 5.2.2.1.3.1
Geotechnical Properties of Soil Deposits

Area/ Reference Borehole No.	Applicable Depth Below Existing Ground Surface (m)	Applicable Elevation (m)	Soil Type	Bulk Unit Weight (kN/m^3)	Angle of Internal Friction (ϕ) Degrees	Recom- mended n_h Value (kN/m^3)	Recom- mended Undrained Shear Strength (kPa)
South Abutment Borehole RT 2	0.1 – 15.1	295.0 – 280.0	Silty Fine Sand	18.5	30	1300	
	15.1 – 24.8	280.0 – 270.3	Silty Fine Sand	19.5	31	4400	
	24.8 – 28.2	270.3 – 266.9	Gravelly sand	21.5	35	11000	
North Abutment Borehole RT 3	0.1 – 2.2	294.8 – 292.7	Silty Fine Sand	18.5	30	1300	50
	2.2 – 3.7	292.7 – 291.2	Clayey Silt	18.5			
	3.7 – 5.2	291.2 – 289.7	Silt	18.0	29	1200	
	5.2 – 10.0	289.7 – 284.9	Silty Fine Sand	18.5	30	1300	
	10.0 – 23.3	284.9 – 271.6	Silty Fine Sand	19.5	31	4400	
	23.3 – 30.2	271.6 – 264.7	Gravelly Sand	21.5	35	11000	

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

Factored Horizontal Resistance at U.L.S. = 110 kN/pile
Horizontal Resistance at S.L.S. = 40 kN/pile

5.2.2.2 TIMBER PILES, DRIVEN CONCRETE PILES AND CAISSON FOUNDATIONS

Timber piles are generally only 15 to 18 m in length. Because of this, after driving, their tips can be expected to be only about 12 to 15 m below the existing grades where the soil is loose to compact. These piles will, therefore, not provide adequate geotechnical resistance.

Driven concrete piles are generally not cost-effective. In addition, owing to the presence of cobbles and boulders below about 24 m they can be damaged. Furthermore, the lack of a definite competent stratum elevation (i.e. variable pile tip elevations) may necessitate splicing which is costly.

Drilled and cast-in-place concrete piles (caissons) will be very difficult to install in the water bearing granular deposits under upward hydrostatic pressure. The use of auger press piles can be considered but these have little resistance to lateral loads and will likely be uneconomical.

The use of timber, driven concrete and auger press piles and caisson foundations is therefore not recommended, based on cost and/or reliability.

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/m³
Coefficient of Lateral Earth Pressure:

$$K_a = 0.27$$

$$K_o = 0.43$$

Compacted Granular 'B' Type 1

Unit Weight = 21 kN/m³

Coefficient of Lateral Earth Pressure:

$$K_a = 0.31$$

$$K_o = 0.47$$

Rock Fill

Unit Weight = 18 kN/m³

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 the 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 will need to be restricted in size as per current MTO practice.

5.4 APPROACH EMBANKMENTS

The approach embankment will be about 5 m high at the south abutment and 6 m high at the north abutment.

Based on the limited borehole and laboratory data, our preliminary stability analyses indicate that the subsurface deposits at the site can support the

approximately 6 m high embankments with 2H:1V side slopes with an adequate margin of safety, provided that all organic, weak or otherwise unsuitable materials are removed as per MTO standards before placing the fill. This assessment, however, should be considered as preliminary and should be confirmed by more field and laboratory testing and stability analyses. In particular, for forward slopes under the abutments, a stability analysis should be carried out once the exact configuration is known in relation to the banks of the river.

Assuming properly compacted, acceptable inorganic earth fill material, 2 horizontal in 1 vertical side slopes can be used for the construction of the approach fills. 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 suitably heavy compactor. The existing site conditions (i.e. high water table and silty soils) could influence the choice of compaction equipment. In wet conditions some dewatering may be needed in order to achieve proper compaction and the first one or two lifts of the fill may need to consist of free-draining granular materials.

The materials used for the construction of the embankment fills should consist of approved, acceptable earth fill. As mentioned before, oversize materials (having a 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 40 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 their final subgrade level at least three weeks ahead of pile driving. If this recommendation is followed, about 80% of the estimated settlements would be completed before the highway is paved. The residual settlements would be considered acceptable.

5.5 CONSTRUCTION COMMENTS

The groundwater table at the site was recorded near the ground surface level. This aspect should be taken into consideration when carrying out construction of the pile caps, stripping and backfilling.

If the excavations for the pile caps extend to or below the groundwater level, dewatering will be required to stabilize the soil and to facilitate the construction. If the head of water is not more than about 0.5 m, the water can be collected by gravity drainage and removed by pumping from temporary sumps. Proper filtering of the sumps is necessary in order to prevent the removal of soil fines as a result of pumping action. If, however, the head of water is higher than about 0.5 m then more elaborate dewatering methods such as deep filtered sumps, filtered wells or vacuum well point, would be necessary.

From our experience in the general area, the presence of mild artesian conditions (i.e. slightly above ground surface) could occur emanating from the lower zones of the silty fine sand and especially the underlying coarse sand and gravel deposit. We recommend that consideration be given to this aspect when conducting the detailed foundation investigation for this bridge site. Artesian conditions may create problems if soil fines are carried from the foundation soils to the ground surface. Loss of fines may endanger the integrity of deep foundations or cause a reduction in axial and lateral resistances. In addition, environmental problems could occur due to the migration of fine-grained materials. Should evidence of an artesian condition be found, mitigating measures will need to be incorporated in the design (e.g. a filter blanket with subdrains at the base of the pile caps) in case they are

needed. In addition, careful observations may be required during and immediately after the installation of the foundation units.

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 recommendations given in this report are for preliminary design purposes only and should be reviewed when the detailed investigation is carried out.

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

Shaheen & Peaker Limited



Zuhtu Ozden, P.Eng



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

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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.

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.