GEOCRES No: 31E-191

PRELIMINARY FOUNDATION INVESTIGATION REPORT
PROPOSED HIGHWAY 11, MUNICIPAL ROAD
UNDERPASS
KATRINE, ONTARIO
W.P. 314-99-00

GEOCRES No: 31E-191

Prepared For:

STANTEC CONSULTING LIMITED

Prepared by:

SHAHEEN & PEAKER LIMITED

Project: SPT1010E
November 7, 2001

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

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PRELIMINARY FOUNDATION INVESTIGATION REPORT PROPOSED HIGHWAY 11, MUNICIPAL ROAD UNDERPASS KATRINE, ONTARIO W.P NO. 314-99-00

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 the municipal service road over the realigned southbound and northbound lanes of Highway 11. The site is located just north of Katrine, Ontario, approximately 0.7 km north of Three Mile Lake Road and about 6 km south of the Village 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 (underpass) will be located to the immediate west of existing Highway 11, near the easterly end of an open field which is surrounded on all sides by heavily forested areas. The area is hummocky, and is probably utilized for grazing, and drains towards the Magnetawan River via a wide swale which slopes from west to east. The Magnetawan River meanders approximately 150 m east of the proposed underpass.

The existing Highway 11 is a two-lane asphalt paved road. After completing the four-lane highway, the existing highway will be utilized as a service road for accessing the near-by communities. The alignment of the reconstructed Highway 11 will be on the east side of the existing Highway 11 to the south of the bridge site and crosses to the west side immediately south of the proposed bridge site. The ground surface elevation in the immediate vicinity of the underpass is between El. 303 m and 313 m, generally rising towards the north and towards the west. To the south, the grade first rises and then drops to the Magnetawan River while to the north, the grade first rises and then drops sharply into the valley of a small watercourse.

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-flucial 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.

The geological information leads us to believe that the surficial swale mentioned earlier in this section of the report had a predecessor in the geological past: during and towards the end of the ice ages a wide channel was scoured into the bedrock. The channel was subsequently filled with coarser materials directly above the bedrock and later with fine sands and silts in the upper part of the overburden.

3. INVESTIGATION PROCEDURES

The fieldwork for the proposed overpass was performed during the period of April 6 through April 17, 2001, and consisted of drilling three deep boreholes (numbered BPR 2, BPR 3 and BPR 4), at the each of the planned bridge support location, and two shallow boreholes (numbered BPR 1 and BPR 5) below the approach embankments. Table 3.1 below summarizes the borehole locations, elevations and depths.

Table 3.1 Overview of Borehole Locations, Elevations and Depths

Borehole No.	BPR 1	BPR 2	BPR 3	BPR 4	BPR 5
Sta. and Offset (m)	9+935.0 0.00	9+950.0 – 5.00	10+000.0 – 3.00	10+048.9 4.00	10+070.0 0.00
Ground El.	312.5 m	312.2 m	308.6 m	307.1 m	309.4 m
Depth (m)	6.6	21.2	25.3	24.5	9.6
Location	South Approach	South Abutment	Central Pier	North Abutment	North Approach

Distance
between Boreholes: ♦----- ~ 15 m ---♦--- ~ 50 m----♦--- ~ 49 m ----♦---- ~ 21 m-----♦

The boreholes were located in the field by Shaheen & Peaker Limited using for reference stakes installed by Stantec at the planned abutment and pier locations. Geodetic elevations and horizontal control (using stations and offset distances, and Ontario grid coordinates) were determined by Stantec after 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. Generally, the drilling of each borehole began by means of solid stem continuous flight augers and when caving occurred continued with hollow-stem augers. Below 10 to 13 m depth in the deeper boreholes frequent boulders and cobbles impeded the advancement of the boreholes by augering therefore casing was installed in the hollow-stem auger and the boreholes were continued by washboring and diamond drilling. Also in the deeper boreholes drilling mud was used in the casing to counterbalance the hydrostatic head and the rods and the sampler were withdrawn slowly while pouring mud into the hole to minimize disturbance to the cohesionless sands. In spite of this measure, some inevitable disturbance may have reduced the recorded N-values.

Sampling in the boreholes was effected 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 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).

Where the consistency of the soil permitted in the cohesive (clayey) deposits, the undrained shear strength of the soil was measured in-situ by means of field vane tests using an MTO type field vane equipment.

In the deep boreholes the bedrock was explored by diamond drilling. Wire-line core barrel was used to obtain NQ size (63.5 mm dia.) core. From the recovered length of core pieces the total core recovery (REC) and Rock Quality Designation (RQD) values were obtained which are expressed as percentages and indicate the quality of the cored rock.

Since the groundwater level did not attain equilibrium condition in the open boreholes at the time of completion, the water levels were estimated from the condition of the recovered samples and from the laboratory moisture content test results. For long-term observation of the ground water conditions, a sealed piezometer was installed in Borehole BPR 3.

The soil profile and ground water level encountered in the boreholes, sampling depths, N-values and vane test results, together with the coring data 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 and bulk unit weight measurements, Atterberg (liquid and plastic) Limits tests, and grain-size analyses. The results of laboratory tests are presented on the appropriate Record of Borehole Sheets and also in Appendix B.

4. SUBSURFACE CONDITIONS

All the five boreholes were drilled in an open field within 5 m from the centreline of the proposed underpass and Table 4.1 below presents an overview of the borehole locations.

The ground surface at the bridge site falls from about Elevation 313 to 312 m at the south approach and south abutment locations to about 309 and 307± m at the central pier and north abutment locations, respectively, and then rises back to about Elevation 309 to 310 m at the north approach. In the general area, the grade also falls from west to east.

The boreholes indicate that, below a relatively shallow veneer of topsoil, an extensive laminated silt deposit is the principal surficial soil type. The thickness of this silt deposit increases towards the north from about 5 to 6 m at the south abutment (in Borehole BPR 2) and south approach (Borehole BPR 1) to 13 m below the north abutment (in Borehole BPR 4). Below the north approach embankment (in Borehole BPR 5) the silt deposit is overlain by a ~4 m thick silty clay layer. With depth the silt contains increasing percentage of fine sand, and grades to a silty sand stratum which contains gravel particles. This stratum is about 3 to 8 m thick and its surface is also sloping towards the north from about El. 307 m at the south abutment to about El. 294 m at the north abutment.

The deposit grades to coarser granular materials which consist of gravelly sand with silt and frequent cobbles and boulders. The surface of this unit was contacted at about El. 299 below the south abutment, and at about El. 291 at the north abutment, corresponding to about 13 and 16 m depth, respectively and is approximately 4 to 11 m thick. It is a competent and very dense formation overlying the gneiss bedrock, which was encountered at El. 295± m below the south abutment, and at Elevation 286± below the central pier and north abutment. Although stabilized groundwater readings could not be taken during the field work, at the time of drilling the groundwater level was probably between 1.5 m and 2 m depth in the lower areas and between 3 and 4 m in the upper boreholes.

An inferred stratigraphic profile is given in Drawing 1 while the details of the subsurface conditions encountered in the boreholes are presented on the Record of Borehole Sheets in Appendix A. The various strata encountered in the boreholes and their geotechnical properties are briefly described in the following subsections of this report.

4.1. TOPSOIL

At the borehole locations the ground surface elevations ranged from El. 312.5 m (Borehole BPR 1) to El. 307.1 m (Borehole BPR 4). In all the five boreholes drilled for the proposed structure topsoil was encountered extending to the average depth of 160 mm (range: 75 to 250 mm). As can be expected, the topsoil was thickest in the borehole drilled in the low area of the site (in Borehole BPR 4).

One water content measurement was over 50%, due to the presence of organic matter in the topsoil.

4.2 SILT

A major silt deposit was encountered in all boreholes; in Boreholes BPR 1 through BPR 4 the silt deposit is directly below the topsoil while in Borehole BPR 5 it was encountered at 4.3 m below ground surface (at El. 305.1 m), underlyinig a silty clay stratum.

The colour of the silt is brown to about 1.4 m to 3.8 m depth, corresponding to El. 305.2 m to 309.7 m. The colour change occurs at lower elevations in the deeper lying boreholes. In Borehole BPR 5 where the silt layer was encountered at a greater depth, its colour is grey.

The silt deposit is laminated and, in some samples, thin (1 to 2 mm thick) clay laminations were observed. The average thickness of the stratum is about 8 m, ranging from about 5 m to about 13 m, and possibly more because Borehole BPR 5 was terminated in the silt. The lower boundary of the silt deposit is around El. 307 m in the higher borings (BPR 1 and BPR 2) and as low as El. 294 ± m in the lowest borehole (BPR 4). The silt is considered a non-plastic and cohesionless (i.e. fine grained granular) deposit which is confirmed by an unsuccessful attempt to perform an Atterberg limits test on sample SS 7 taken from Borehole BPR 4. The clay laminations, however, could lend some plasticity to some zones of the silt deposit. Eleven grain-size distribution tests were performed on samples taken from the silt deposit (see Figure 1, in Appendix B). The summary of the results is presented in Table 4.2.1 below.

Table 4.2.1
Silt Deposit – Grain Size Distribution

SOIL COMPONENT	Percent by Weight		
	Average	Maximum	Minimum
Gravel	0	0	0
Sand	8	37	0
Silt	86	98	63
Clay	6	22	0

A trend of increasing fine sand content with depth was observed in the recovered soil samples, which can be best seen in the gradation results of three samples obtained from Borehole BPR 4: the sand content was only 2% at about 1 m depth but increased to 37% at about 12 m depth.

The silt deposit was generally wet except in Boreholes BPR 1 and 3, where the upper zones were in a moist condition. Attention is called to the fact that the boreholes were drilled in early spring when the upper zones of the soil had still high water content after the snow cover melted. The natural water content of the silt samples was found to range from 18 to 29%, with an average of about 23%. The average bulk density of six samples obtained in Borehole BPR 3 was 18.9 kN/m^3 (with a range between 18.5 and 19.8 kN/m^3).

Based on the standard Penetration test results (N-values) which range from 2 to 29 blows/0.3 m, the silt deposit is very loose to compact. By discarding three low N-values (4, 2 and 5 blows/0.3 m penetration) near the ground surface in Boreholes BPR 1, 2 and 3, and two very low N-values (2 and 3 blows) at greater depths in Boreholes BPR 4 and 5 (which were believed to have been caused by unbalanced groundwater effects), the average N-value in Boreholes BPR 1 2, 3 and 5 is about 11 indicating a compact to loose condition while in BPR 4, it is indicating a compact condition.

4.3 SILTY CLAY

In Borehole BPR 5 (most northerly borehole) a silty clay deposit was encountered below the topsoil. The deposit extends to 4.3 m depth (to El. 305.1 m) where it grades to the silt deposit described in the preceding Section 4.2 in detail. The colour of the silty clay is brown to about 1.4 m depth and grey below. The silty clay deposit is laminated, with thin silt layers and the silt content is increasing with depth which is indicated by the results of two grain size distribution tests summarized in Table 4.3.1. (For the grain size distribution curves see Appendix B, Figure 2.)

TABLE 4.3.1

Silty Clay Deposit – Grain Size Distribution

SOIL COMPONENT	Percent by Weight		
	Average	Upper Sample	Lower Sample
Gravel	0	0	0
Sand	2	4	0
Silt	62	46	77
Clay	36	50	23

The silty clay is a cohesive deposit and two Atterberg Limits tests yielded the following results (the plasticity chart is shown in Appendix B, Figure 3).

Liquid Limit: 40 and 27 %
Plastic Limit: 24 and 22 %
Plasticity Index: 16 and 5 %

The higher and lower Atterberg test results were obtained on samples taken from 0.9 m and 4.0 m depths, respectively. They indicate the decreasing plasticity of the deposit with depth.

The N-values ranged from 15 to 6 blows indicating that the upper portion of the silty clay has stiff consistency but the deposit becomes weaker, (i.e. firm), with depth. This suggests that the silty clay layer is overconsolidated to some extent, probably caused by desiccation of the upper zones of the deposit. (Immediately below the ground surface the N-value was 5 but this is not considered representative.) Also, a very low N-value of 2 blows was obtained at about 4 m depth. Since the silty clay contains silt layers and the silt content is increasing with depth, this N-value was probably caused by the presence of a saturated silt layer which was disturbed when the solid stem augers were withdrawn before sampling.

To determine the undrained shear strength of the silty clay depost, two vane tests were performed near its lower boundary. Both results were in excess of 100 kPa, indicating very stiff consistency. In our opinion and taking all findings into account, the actual shear strength of the silty clay deposit is probably less – about

75 kPa – at this depth, and the high vane test result is most likely due to the presence of silt layers.

4.4 SILTY SAND, SOME GRAVEL

This is a cohesionless and granular deposit which was encountered in four of the five boreholes. (Probably Borehole BPR 5 did not extend sufficiently deep to encounter the deposit.) The upper and lower boundaries are at an average depth of 8.2 m and 11.8 m, respectively, ranging from 5.0 to 13.2 m, and 6.6 to 16.2 m, again respectively, possibly deeper, because Borehole BPR 1 was terminated in this material. In the three deep boreholes (BPR 2, BPR 3 and BPR 4), the thickness of the deposit was found to range from 3.0 m to 8.2 m with an average of 4.6 m. The upper boundary of the silty sand is about El. 307 m in the higher borings (BPR 1 and BPR 2) and drops as low as El. 293.9 m in Borehole BPR 4 which was drilled at the lowest elevation. The lower boundary of the silty sand layer drops from El. 299.0 to El. 290.9 m in Boreholes BPR 2 and BPR 4, respectively.

Four grain size distribution curves are shown in Fig. 4, in Appendix B. It can be seen that the material consists of up to 38% gravel, 48 to 65% sand, 13 to 29% silt and negligible amount (maximum 2%) of clay size particles. There are cobbles and boulders in the deposit, through which diamond drilling was required in some instances to advance the boreholes.

The deposit is a cohesionless (granular) material and the N-values ranged from 22 to 95 blows per 0.3 m penetration indicating compact to very dense condition. Occasionally, the sampler could not be driven the full 0.3 m depth due to the high density of the silty sand, and due to obstructions caused by coarse gravel, particles, cobbles and boulders.

4.5 GRAVELLY SAND WITH SILT, COBBLES AND BOULDERS

The three deep borings (Nos. BPR 2, 3 and 4), encountered a glacial outwash deposit consisting of sand, gravel and silt in varying proportions, and numerous cobbles and boulders embedded in the granular matrix. This is a cohesionless deposit whose upper boundary is at an average depth of 13.6 m (range: 11.2 to 16.2 m) below ground suface. The deposit extends to the bedrock surface which was encountered at 17.3 to 22.4 m below ground surface at the three borehole locations. The upper boundaries drop from south to north from El. 299.0 m (in Borehole BPR 2) to El. 290.9 m (in Borehole BPR 4). The lower boundaries also slope northwards and follow the bedrock surface which also drops from El. 294.9 m to El. 285.7, respectively, in the above boreholes.

A grain size distribution test was performed on Sample 18 from Borehole BPR2. The gradation curve is shown in Figure 5 in Appendix B, and the results are summarized below.

Gravel:	37%
Sand:	47%
Silt:	16%

Although this grain size distribution curve is very similar to that of the overlying stratum (characterized as silty sand, some gravel), the principal difference between the two deposits is the presence of numerous cobbles and boulders in the lower one. The grain size distribution curve does not include the oversize particles; therefore, the curve cannot be considered as representative of the entire material but only of the matrix.

N-value recorded in this deposit range from 73 to more than 100 blows for 0.3 m penetration, indicating a very dense condition. In almost all cases the split spoon could only be driven to 5 to 15 cm because of the high density of the material or because of obstructions caused by boulders and cobbles.

Natural water content mesurement on samples from the deposit ranged from 8 to 18 per cent, depending on the composition of the deposit: where the samples contained more silt and fine sand, the water content of the samples was also higher.

4.6 BEDROCK

The bedrock surface was encountered in the three deep boreholes at the average depth of 20.4 m (range: 17.3 to 22.4 m), corresponding to El. 294.9 m in Borehole BPR 2 and to El. 286 ± m in Boreholes BPR 3 and BPR 4.

The bedrock consists mainly of light grey to dark grey gneiss with white quartz intrusions and occasional fracture zones. It is generally unweathered to moderately weathered but generally unweathered to slightly weathered, and was explored by diamond drilling to depths ranging from 2.9 m to 3.9 m. The recovery rates ranged from 73% to generally 100%, with Rock Quality Designation (RQD) values between 60% and 100 %. Most results were over 75 % indicating a good rock quality.

4.7 GROUNDWATER

The ground water did not stabilize by the time the fieldwork was completed, although the available information gives an indication of the probable ground water levels. In addition to the moisture condition of the samples, one such indication is the color change from brown to grey which occurred between about 2 and 4 m depth, corresponding to El. 309 ± and 306 ±m depth. The lower groundwater elevations were obtained in the lower boreholes.

Some water level observations were not too far below or at these levels (e.g. in Borehole BPR 1 at 5.9 m/El. 306.6 m, in Borehole BPR 4 at 1.5 m/El. 305.6 m, and in Borehole BPR 5 at 3m/El. 306.4m). This indicates that the probable ground water levels were at 1.5 to 2 m depth in the lower boreholes and 3 to 4 m in the higher boreholes. These depths correspond to El. 306± m in the higher borings and to El. 300.6± m in the lower ones. The groundwater table can be expected to fluctuate with the seasons and weather events.

Yours truly

SHAHEEN & PEAKER LIMITED

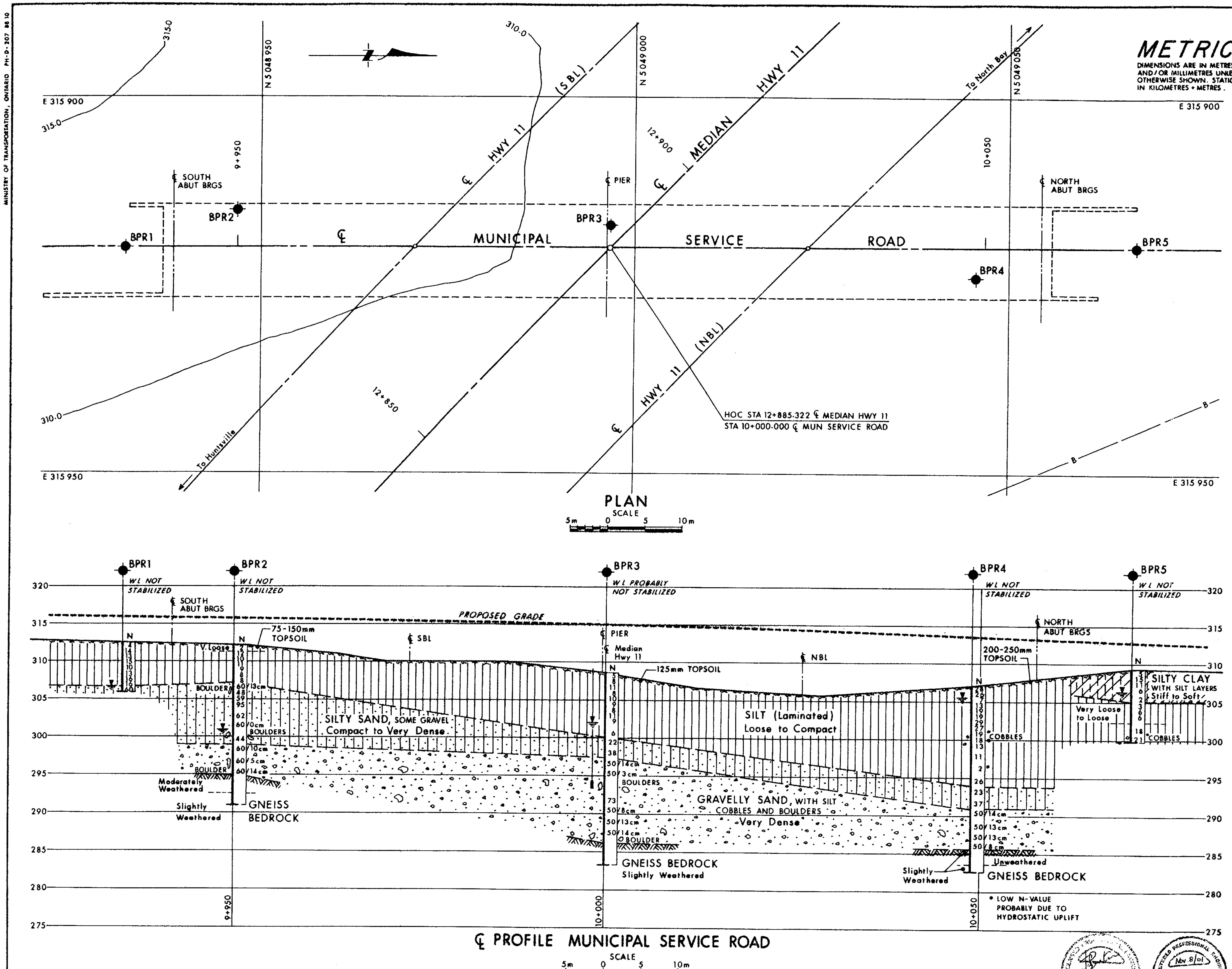
Z. S. Ozden, P.Eng

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

REGISTERED PROFESSIONAL ENGINEER
Z. S. OZDEN
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
K. R. PEAKER
PROVINCE OF ONTARIO

DRAWINGS



METRIC

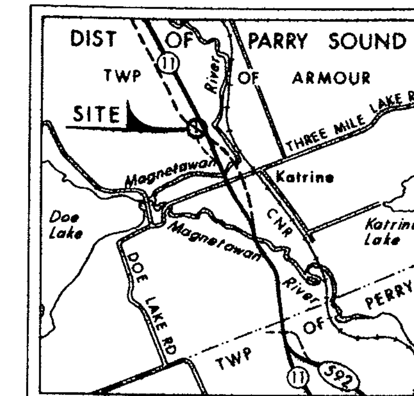
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No
WP No 314-99-00

HWY 11
MUNICIPAL SERVICE RD UNDERPASS
BORE HOLE LOCATIONS & SOIL STRATA

SHEET

Shaheen & Peaker Limited



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)
- WL at time of investigation Apr. 2001
- WL in Piezometer
- Piezometer

No	ELEVATION	CO-ORDINATES NORTH	EAST
BPR1	312.5	5048931.8	315920.1
BPR2	312.2	5048946.8	315915.0
BPR3	308.6	5048996.8	315917.0
BPR4	307.1	5049045.7	315924.0
BPR5	309.4	5049066.8	315920.0

=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 31E-191

HWY No 11			DIST 52
SUBM'D LSR	CHECKED LR	DATE Aug 7, 2001	SITE
DRAWN JP	CHECKED ZO	APPROVED	DWG 1

Z. S. OZDEN

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

APPENDIX A

Records of Boreholes

Ministry of Transportation
Ontario

Foundation Design

RECORD OF BOREHOLE No BPR1

METRIC

W.P. 314-99-00
LOCATION Municipal Service Road Underpass - Katrine, ON - Coords: N 5 048 931.8; E 315 920.1
ORIGINATED BY G.I
DIST 52 HWY 11
BOREHOLE TYPE Solid Stem Augering
COMPILED BY G.T
DATUM Geodetic
DATE 06.04.01
CHECKED BY LSR

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
312.5	Ground Surface										
0.0	75 mm **Topsoil**; **SILT** (laminated) moist to wet; loose	1	SS	4		312					
	compact	2	SS	14				0	2	98	0
		3	SS	13		311					
	increasing fine sand content	4	SS	15		310					
	brownish	5	SS	10		309					
	grey	6	SS	13				0	12	88	0
		7	SS	16		308					
306.6		8	SS	16	▼*	307					
5.9 / 305.9	**SILTY SAND** some gravel, very dense, grey, wet	9	SS	60		306					
6.6	End of borehole Refusal to augering at 6.6 m *Water level at 5.9 m (not stabilized) and hole open to 6.0 m on completion										

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

+³, ×³: Numbers refer to Sensitivity

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

RECORD OF BOREHOLE No BPR2

METRIC

W.P.	314-99-00	LOCATION	Municipal Service Road Underpass - Katrine, ON - Coords: N 5 048 946.8; E 315 915.0	ORIGINATED BY	G.I
DIST 52	HWY 11	BOREHOLE TYPE	Solid and Hollow Stem Augering, Washboring & NQ Rock Coring	COMPILED BY	G.T
DATUM	Geodetic	DATE	11.04.01 & 12.04.01	CHECKED BY	LSR

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
312.2	Ground Surface
0.0	150 mm **Topsoil**; **SILT** (laminated), wet; rootlets to 1.2 m; very loose / compact to loose; increasing fine sand content; brown / grey
307.2 / 5.0	**SILTY SAND** some gravel, dense to very dense, grey; BOULDER; moist to wet / wet; occasional sandy silt till lenses; BOULDERS
299.0 / 13.2	**GRAVELLY SAND** with silt, cobbles and boulders, very dense, grey, wet
297.2 / 15.0	Continued Next Page

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	2	
2	SS	12	0 4 92 4
3	SS	10	
4	SS	11	
5	SS	9	
6	SS	8	
7	SS	8	0 14 82 4; HST augering
8	SS	60/13	washboring
9	NQ		
10	SS	48	
11	SS	59	
12	SS	95	4 65 29 2
13	SS	62	
14	SS	60/0	
15	NQ		
16	NQ		
17	SS	44	38 48 (14)
18	SS	60/10	37 47 (16)

GROUND WATER CONDITIONS: water level at approx. elev. 301; April 11 / April 12

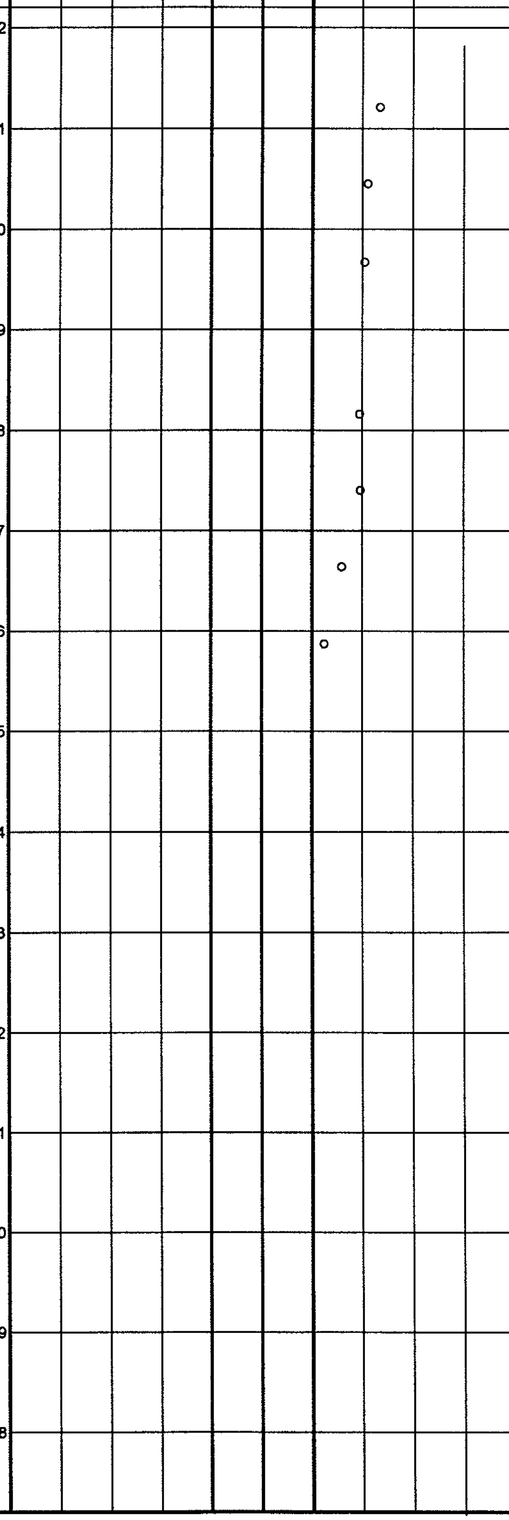
ELEVATION SCALE: 312, 311, 310, 309, 308, 307, 306, 305, 304, 303, 302, 301, 300, 299, 298

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, × 3: Numbers refer to Sensitivity

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

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No BPR2

METRIC

W.P. 314-99-00 LOCATION Municipal Service Road Underpass - Katrine, ON - Coords: N 5 048 946.8; E 315 915.0 ORIGINATED BY G.I

DIST 52 HWY 11 BOREHOLE TYPE Solid and Hollow Stem Augering, Washboring & NQ Rock Coring COMPILED BY G.T

DATUM Geodetic DATE 11.04.01 & 12.04.01 CHECKED BY LSR

ELEV / DEPTH (297.2)	SOIL PROFILE: DESCRIPTION	SAMPLES: NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%)
15.0	**GRAVELLY SAND** with silt, cobbles and boulders very dnese, grey, wet	19	SS	60/5		297	
						296	
		20	SS	60/14			
294.9	BOULDER	21	NQ			295	
17.3	BEDROCK (Gneiss) grey, moderately weathered	22	NQ RC	Rec. 73%			RQD=60%
						294	
		23	NQ RC	Rec. 85%		293	RQD=65%
	------- slightly weathered					292	
		24	NQ RC	Rec. 90%			RQD=77%
291.0 / 21.2	End of borehole *Water level at 11.2 m (not stabilized in hollow stem augers). Hole open to 1.7 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

GR SA SI CL

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

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

RECORD OF BOREHOLE No BPR3

METRIC

W.P. 314-99-00	LOCATION Municipal Service Road Underpass - Katrine, ON - Coords: N 5 048 996.8; E 315 917.0	ORIGINATED BY G.I
DIST 52 HWY 11	BOREHOLE TYPE Hollow Stem Augering, Washboring & NQ Rock Coring	COMPILED BY G.T
DATUM Geodetic	DATE 06.04.01 to 11.04.01	CHECKED BY LSR

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
308.6	Ground Surface
0.0	125 mm **Topsoil** **SILT** (laminated) loose to compact, wet clayey laminations brown / grey increasing fine sand content
300.1 / 8.5	**SILTY SAND** some gravel, grey, wet compact / dense
297.4 / 11.2	**GRAVELLY SAND** with silt, cobbles and boulders, very dense, grey wet BOULDERS
293.6 / 15.0	Continued Next Page

SAMPLES

NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL	REMARKS
1	SS	5	18.8					
2	SS	8		0	0	86	14	
3	SS	11	18.9					
4	SS	8	18.5	0	4	96	0	
5	SS	10	18.6					
6	SS	9	19.8					
7	SS	8	18.7					
8	SS	11						
9	SS	9						
10	SS	6						
11	SS	22		14	60	26	0	April 06 ----- April 09
12	SS	38						
13	SS	50/14						
14	SS	50/3						HST augering ------ washboring
14A	NQ	Rec. 26%						

ELEVATION SCALE: 308, 307, 306, 305, 304, 303, 302, 301, 300, 299, 298, 297, 296, 295, 294

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

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

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

Ministry of Transportation
Ontario

Foundation Design

RECORD OF BOREHOLE No BPR3

METRIC

W.P.	314-99-00	LOCATION	Municipal Service Road Underpass - Katrine, ON - Coords: N 5 048 996.8; E 315 917.0	ORIGINATED BY	G.I
DIST	52 HWY 11	BOREHOLE TYPE	Hollow Stem Augering, Washboring & NQ Rock Coring	COMPILED BY	G.T
DATUM	Geodetic	DATE	06.04.01 to 11.04.01	CHECKED BY	LSR

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%)
293.6						
15.0	**GRAVELLY SAND** with silt, cobbles and boulders, very dense, grey wet	14A	NQ		293	
		15	SS	73	292	April 09 - - - - - April 10
		16	SS	50/8	291, 290	
		17	SS	50/13	289, 288	
		18	SS	50/14	287	
286.2	BOULDER	19	NQ			
22.4	BEDROCK (Gneiss) grey, slightly weathered	20	NQ RC	Rec. 100%	286, 285	RQD=85%
		21	NQ RC	Rec. 95%	284	RQD=87%
283.3 25.3	End of borehole Hole caved at 3.2 m on completion *Water level in hollow stem auger at 8.3 m Piezometer installed on April 11/2001 to 15.2 m Water level in piezometer at 6.5 m depth on April 11/2001. Probably not stabilized					

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; GR SA SI CL

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

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

Ministry of Transportation Ontario

RECORD OF BOREHOLE No BPR4

METRIC

W.P. 314-99-00	LOCATION Municipal Service Road Underpass - Katrine, ON - Coords: N 5 049 045.7; E 315 924.0		ORIGINATED BY G.I
DIST 52	HWY 11	BOREHOLE TYPE Solid and Hollow Stem Augering, Washboring & NQ Rock Coring	COMPILED BY G.T
DATUM Geodetic		DATE 11.04.01 to 17.04.01	CHECKED BY LSR

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 (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)

Soil Profile

ELEV / DEPTH	DESCRIPTION
307.1	Ground Surface
0.0	250 mm **Topsoil**
	SILT (laminated) occasional thin clay layers; loose to compact, - - - - - - - compact; moist, brown - - - - - grey, wet; cobbles; fine sand content increasing with depth
293.9 / 13.2	**SILTY SAND** some gravel, compact, grey, wet
292.1 / 15.0	Continued Next Page

Samples

NUMBER	TYPE	"N" VALUES
1	SS	28
2	SS	26
3	SS	7
4	SS	13
5	SS	19
6	SS	19
7	SS	29
8	SS	27
9	SS	19
10	SS	18
11	SS	13
12	SS	11
13	SS	2 **
14	SS	26
15	SS	23

Elevation scale: 307, 306, 305, 304, 303, 302, 301, 300, 299, 298, 297, 296, 295, 294, 293

Remarks & Grain Size Distribution (%)

Location	GR	SA	SI	CL
Sample 2	0	2	76	22
Sample 7	0	9	91	0
Sample 14	0	37	(63)	

April 11
- - - - -
April 16

** SS13 Low N-value probably due to hydrostatic uplift

Hollow Stem Augering
- - - - - - -
Washboring

+ ³, × ³ : Numbers refer to Sensitivity

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

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RECORD OF BOREHOLE No BPR4

METRIC

W.P.	314-99-00	LOCATION	Municipal Service Road Underpass - Katrine, ON - Coords: N 5 049 045.7; E 315 924.0	ORIGINATED BY	G.I
DIST	52 HWY 11	BOREHOLE TYPE	Solid and Hollow Stem Augering, Washboring & NQ Rock Coring	COMPILED BY	G.T
DATUM	Geodetic	DATE	11.04.01 to 18.04.01	CHECKED BY	LSR

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
292.1					292		April 16 ----- April 17
15.0	**SILTY SAND** some gravel, grey, wet, dense	16	SS	37			28 58 14 0
290.9					291		
16.2	**GRAVELLY SAND** with silt, cobbles and boulders, very dense, grey, wet	17	SS	50/14	290		
		18	SS	50/13	289	o	
		19	SS	50/13	288, 287	o	
285.7		20	SS	50/8	286		
21.4	BEDROCK (Gneiss) grey — slightly weathered	21	NQ RC	Rec. 100%	285		RQD=93%
	— unweathered	22	NQ RC	Rec. 100%	284		RQD=100%
	— slightly weathered	23	NQ RC	Rec. 100%	283		RQD=72%
282.6							
24.5							Casing withdrawn on April 18

End of borehole
*Water level not stabilized on completion
Borehole open to 7.9 m depth
*Groundwater probably at 1.5 m depth from sample moisture condition

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 20 5 10 (%) STRAIN AT FAILURE

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RECORD OF BOREHOLE No BPR5

METRIC

W.P. 314-99-00 LOCATION Municipal Service Road Underpass - Katrine, ON - Coords: N 5 049 066.8; E 315 920.0 ORIGINATED BY G.I

DIST 52 HWY 11 BOREHOLE TYPE Solid Stem Augering COMPILED BY G.T

DATUM Geodetic DATE 06.04.01 CHECKED BY LSR

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
309.4	Ground Surface							
0.0	200 mm **Topsoil**		1	SS	5		309	
	SILTY CLAY with silt layers; brown ---- grey; damp to moist ----- moist ---- wet; silt content increasing with depth; stiff to soft		2	SS	15		308	0 4 46 50
			3	SS	11			
			4	SS	6		307	
			5	AS		▼*	306	High vane test results probably caused by silt layers
305.1			6	SS	2		305	0 0 77 23
4.3	**SILT** grey, wet; very loose to loose -------------- compact; ------ increasing fine sand content		7	SS	3		304	
			8	SS	6			0 0 86 14
			9	SS	6		303	
			10	SS	18		302, 301	0 4 92 4
299.8	COBBLES		11	SS	21		300	
9.6	End of borehole *Water level not stabilized; encountered at 3.0 m during drilling Borehole caved at 4.9 m on completion							

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

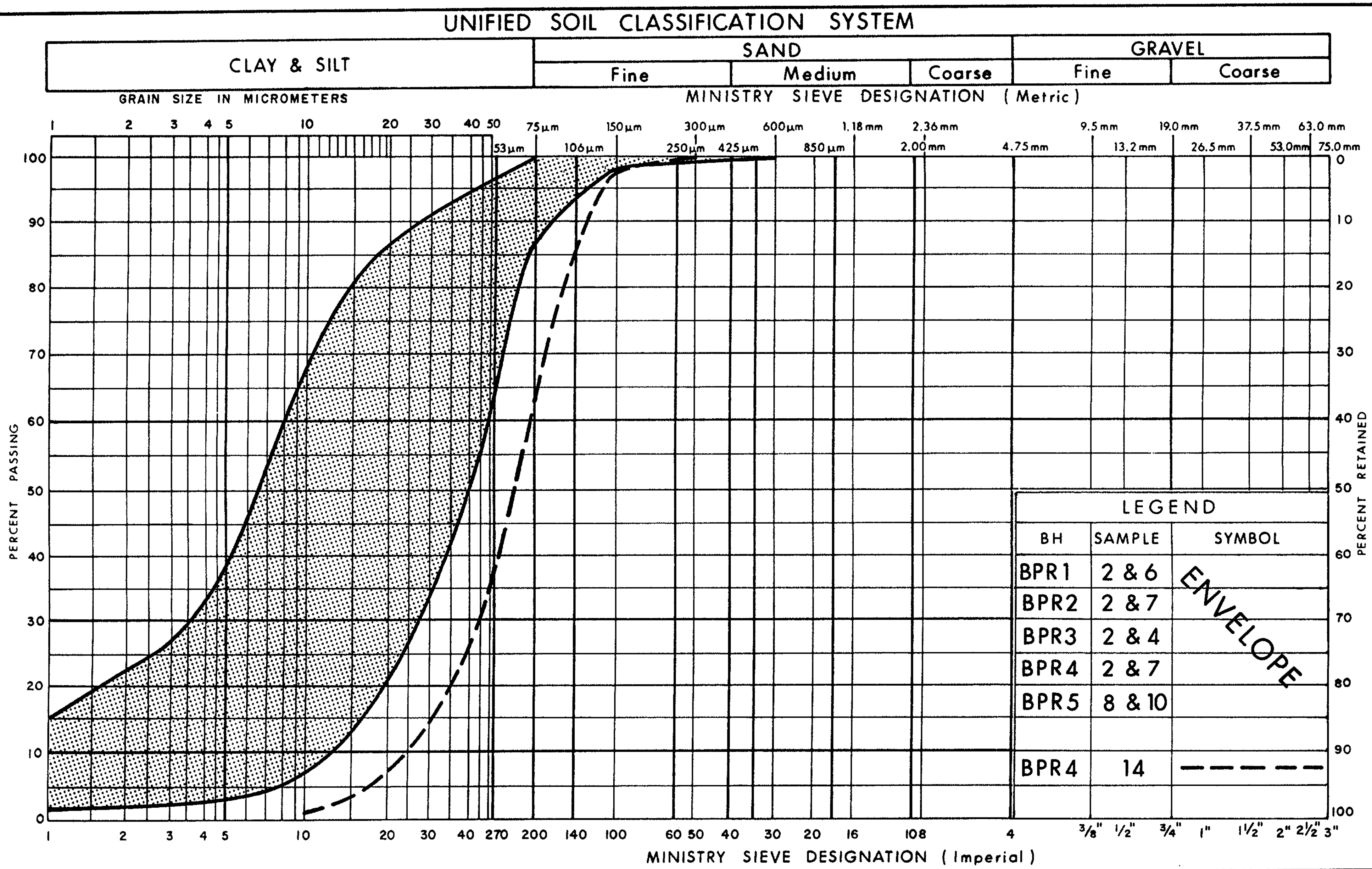
Continued Next Page

$+^3$, $\times^3$: 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

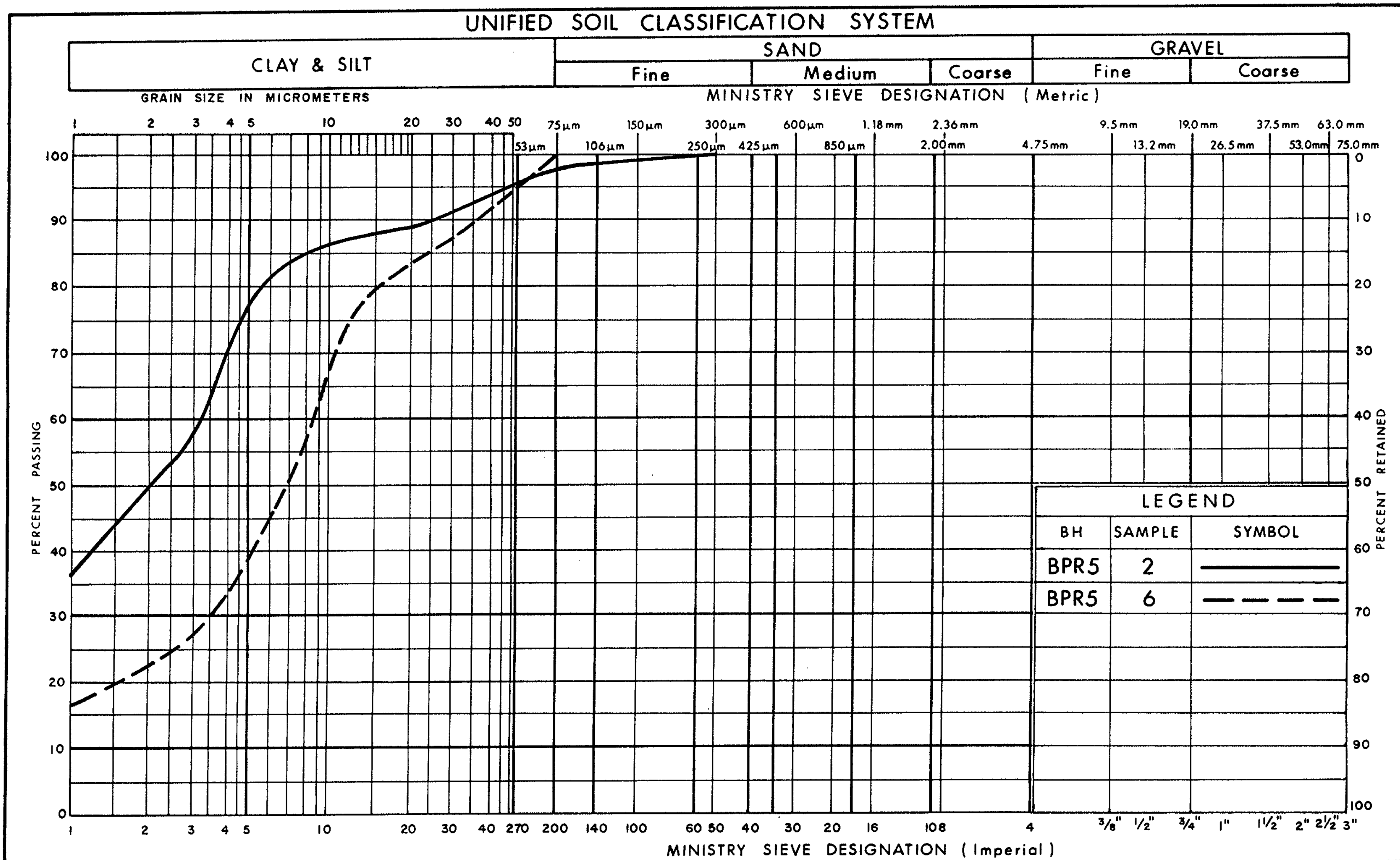
SILT

FIG No 1

W P 314-99-00

SPT 1010E

78 12 M



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GRAIN SIZE DISTRIBUTION

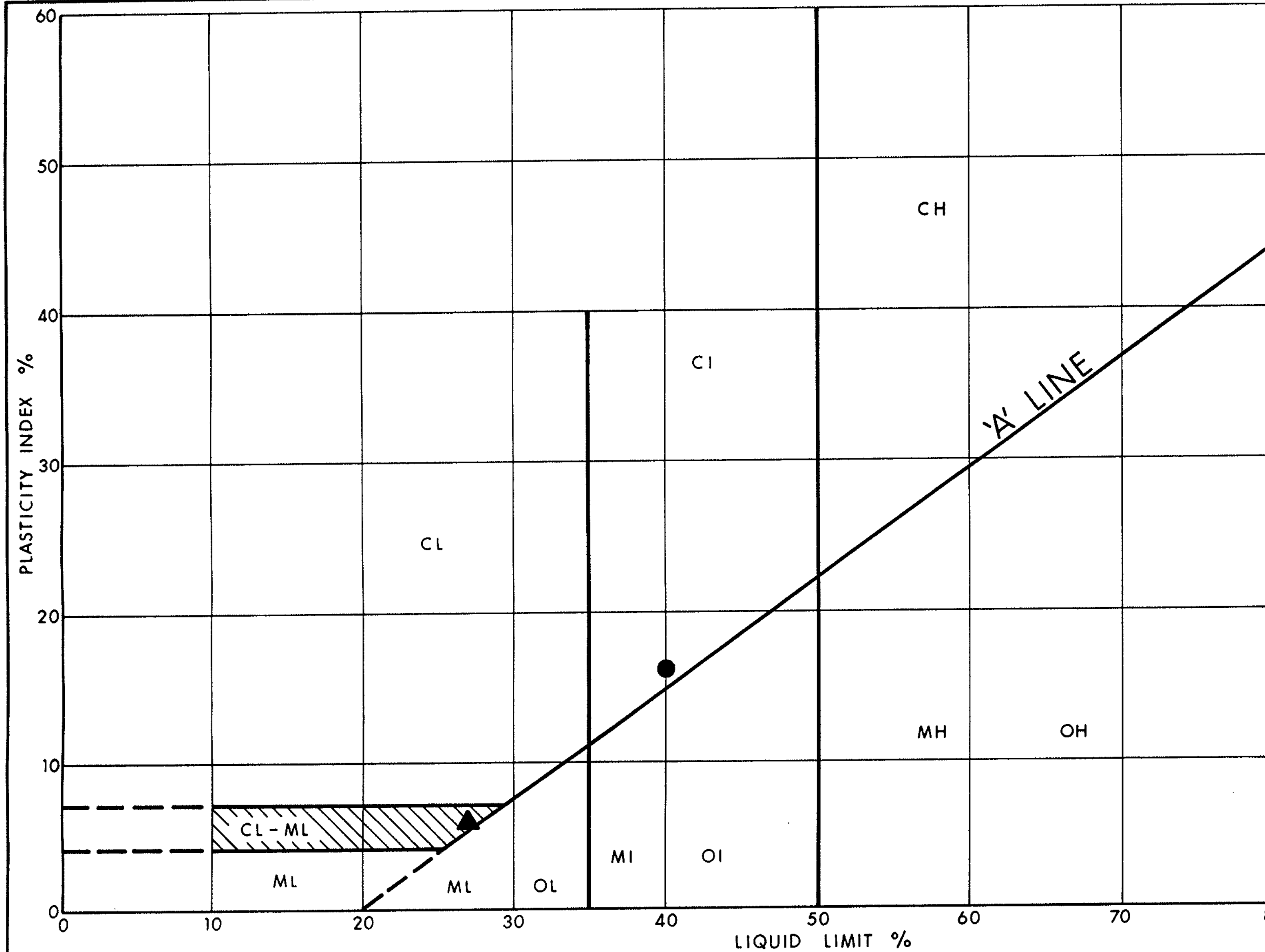
SILTY CLAY

FIG No 2

W P 314-99-00

SPT 1010E

Oct 75, FF-S-21



LEGEND		
BH	SAMPLE	SYMBOL
BPR5	2	●
BPR5	6	▲

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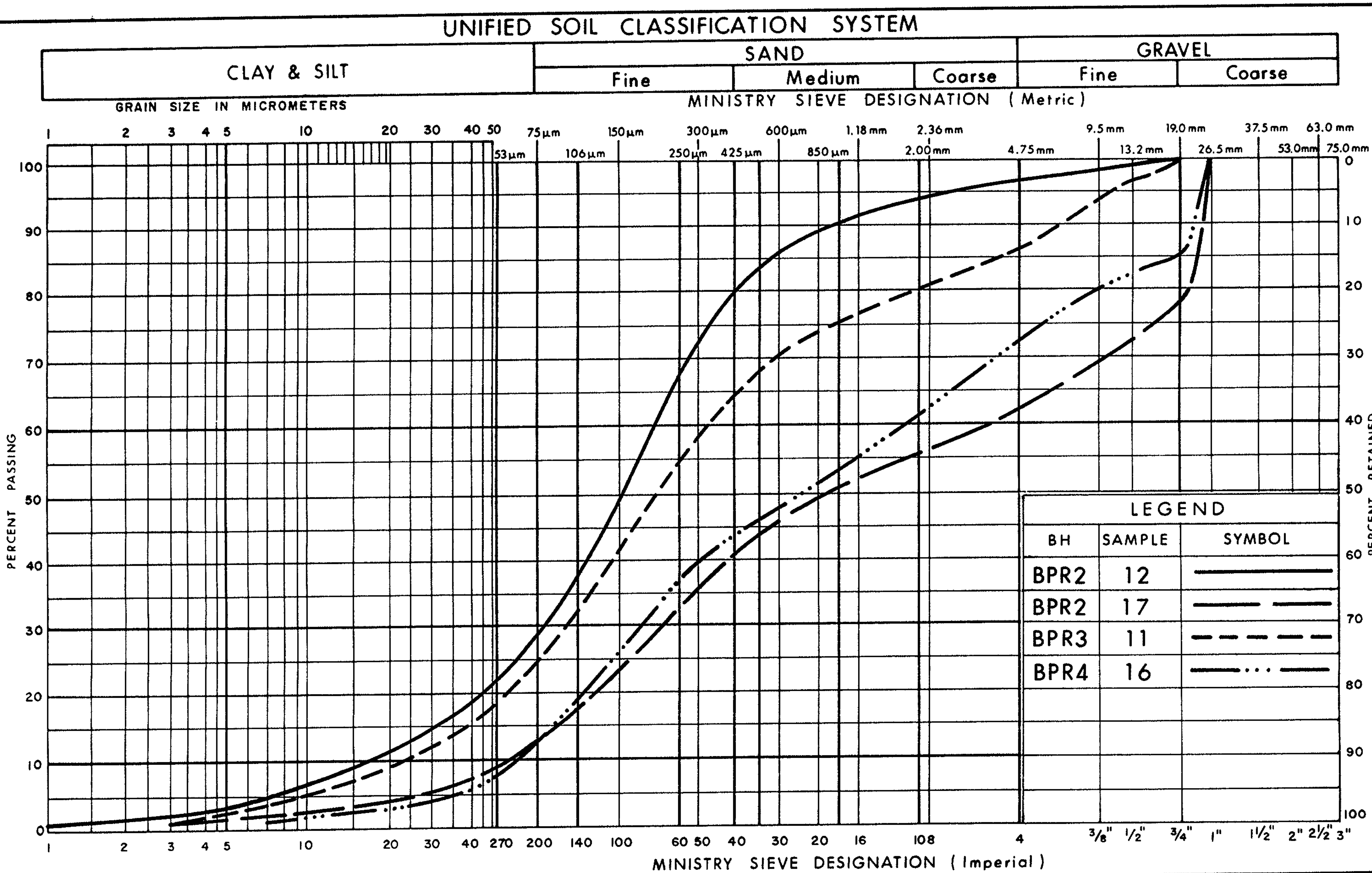
PLASTICITY CHART
SILTY CLAY

FIG No 3

W P 314-99-00

SPT 1010E

78 12 M



LEGEND		
BH	SAMPLE	SYMBOL
BPR2	12	
BPR2	17	
BPR3	11	
BPR4	16	

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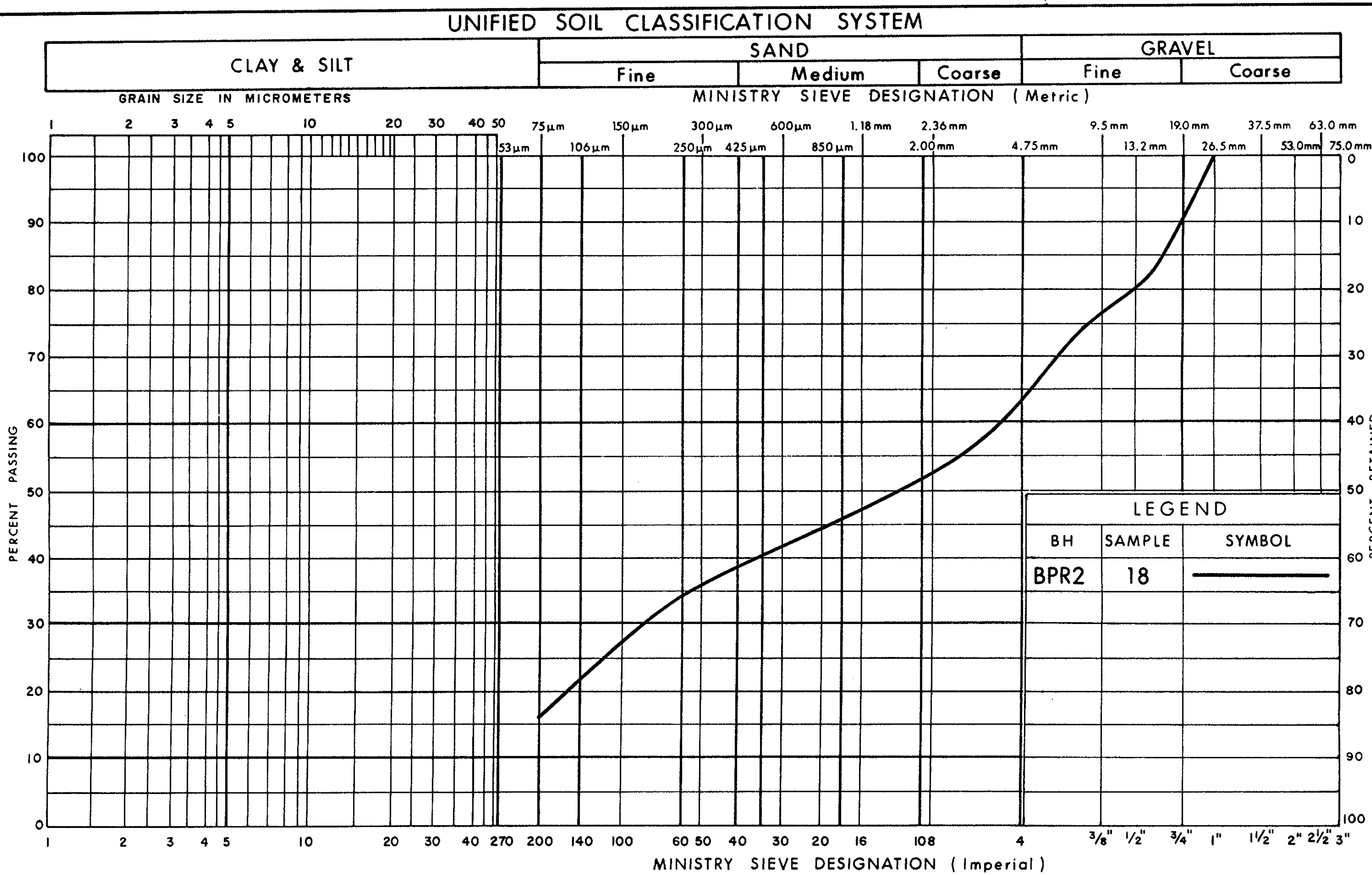
GRAIN SIZE DISTRIBUTION
SILTY SAND, SOME GRAVEL

FIG No 4
W P 314-99-00
SPT 1010E

78 12 M



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GRAIN SIZE DISTRIBUTION

GRAVELLY SAND WITH SILT, COBBLES & BOULDERS

FIG No 5

W P 314-99-00

SPT 1010E

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

SS	SPLIT SPOON	TP	THINWALL PISTON
WS	WASH SAMPLE	OS	OSTERBERG SAMPLE
ST	SLOTTED TUBE SAMPLE	RC	ROCK CORE
BS	BLOCK SAMPLE	PH	TW ADVANCED HYDRAULICALLY
CS	CHUNK SAMPLE	PM	TW ADVANCED MANUALLY
TW	THINWALL OPEN	FS	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 HIGHWAY 11, MUNICIPAL ROAD UNDERPASS
KATRINE, ONTARIO
W.P. 314-99-00

Prepared For:

STANTEC CONSULTANTS LIMITED

Prepared by:

SHAHEEN & PEAKER LIMITED

Project: SPT1010E
November 7, 2001

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

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PRELIMINARY FOUNDATION DESIGN REPORT
PROPOSED HIGHWAY 11, MUNICIPAL ROAD UNDERPASS
KATRINE, ONTARIO
W.P. 314-99-00

5. DISCUSSION AND RECOMMENDATIONS

5.1 GENERAL

The new alignment for Highway 11 across Katrine will consist of a four-lane divided roadway with a typically 30 m wide median. The exising two-lane highway will be used as a municipal service road for local access. According to preliminary arrangement drawings prepared by and received from Stantec, the municipal service road will pass above the four lane Highway 11 at Station 12 + 885.322 m (= centreline of the median of the reconstructed four-lane Highway 11). At this crossing point the centreline of the Service Road is defined as Station 10 + 000.000 m. The angle between the centerlines of the two roadways is 44^0 33' 16".

To avoid level crossing, an underpass (bridge) structure will be built to carry the municipal road over the north- and southbound lanes of Highway 11. The underpass is planned to be an approximately 12.5 m wide structure and will consist of two spans, each 58 m long, adding up to a total length of 116 m.

The top of pavement of the four-lane Highway 11 will be about El. 306.4 m and 307.5 m at the center of the S.B.L. and the N.B.L., respectively. The elevations of the top of bridge structure (i.e. pavement) at the supports of the municipal service road underpass are shown in Table 5.1.1.

Table 5.1.1
Underpass Elevations at Supports

SUPPORT	UNDERPASS (MUNICIPAL SERVICE ROAD) TOP OF BRIDGE PAVEMENT Station	Elevation
South Abutment	9 + 941.504 m	315.852 m
Centre Pier	10 + 000.000 m	314.952 m
North Abutment	10 + 057.504 m	313.211 m

The girders of the underpass will be made of reinforced concrete of a structural height of 2250 mm. The structure will be continuous, with fixed support at the centre pier. Due to the continuity, the structure will be sensitive to differential settlements.

The approach embankments will range in height between 3 and 5 metres along the centerline of the proposed road, being somewhat less along the west and somewhat more along the east sides. Our discussion and analysis will be based on these premises.

The borehole results give the indication that the proposed underpass will probably be built above a deep channel which was scoured into the bedrock and whose remnant is indicated by the surface topography of the site: a surface drainage channel collects runoff from the west and conveys it eastward towards the Magnetawan River. This surface drainage channel crosses the proposed underpass near the central pier location. (This aspect may need to be considered in the actual design of the roadway.)

At the borehole locations, below a shallow topsoil cover, there is a 5 to 13 m thick, loose to compact silt deposit, except at Borehole BPR 5 location where it is overlain by a 4 m thick silty clay layer. The silt deposit is underlain by a 3 to 5 m thick layer of compact to very dense silty sand with some gravel content. Finally, the lowest zones of the overburden consist of a very dense gravelly sand with silt, and frequent cobbles and boulders. This deposit of glacial origin was found to be 4 to

11 m thick, and overlies the gneiss bedrock which was encountered in the three deep boreholes at an average depth of 20.4 m (generally dipping from south down to north). The groundwater was probably at 1.5 to 4 m depth below surface in the lower and higher boreholes, respectively, at the time of the fieldwork (April, 2001.)

5.2 FOUNDATIONS

5.2.1 SPREAD FOOTING FOUNDATIONS

The findings in the boreholes indicate that, at the location of the proposed abutments and pier, the silt deposit is in a loose to compact state to about 5 to 12 m depth. As a result, the allowable bearing pressure would be 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 the continuous structure. For these reasons, the use of normal spread footings is not recommended, including the use of footings on engineered fill (i.e. on compacted Granular 'A' pad), based on reliability.

5.2.2 DEEP FOUNDATIONS

Deep foundations are feasible at the site. Such foundations would derive their geotechnical resistance from end bearing by penetrating several metres into the following strata:

At south abutment:	dense to very dense silty sand/very dense gravelly sand
At central pier :	very dense gravelly sand
At north abutment:	very dense gravelly sand

Drilled caissons would have to penetrate to 7 to 17 m below the existing grades in order to reach soil strata providing adequate geotechnical resistance. Since this depth would be under significant head of ground water, special dewatering measures would be required to construct the caissons, unless they can be socketed into bedrock. In this case, however, because of the presence of coarse granular soils with frequent cobbles and boulders overlying the bedrock, the construction costs will escalate. For these reasons, drilled caisson foundations are not considered to be economical. The use of augerpress piles can be considered but these too will be difficult to construct, have little resistance to lateral loads and will unlikely be cost-effective.

5.2.2.1 DRIVEN PILES

In our opinion, the use of driven piles is feasible.

5.2.2.1.1 TIMBER PILES

Although the anticipated length of timber piles (15 to 18 m) would be sufficient on this project, such piles may not be economical because of their comparatively low load carrying capacity and potential construction difficulties caused by boulders in the subsoil which could damage the piles resulting in a number of wasted units. Due to these considerations timber piles are not recommended on this site, based on reliability.

5.2.2.2.2 CONCRETE PILES

Concrete piles are not considered an economical solution on this site because of the presence of boulders and variable length of piles. Also, concrete piles are heavy, sensitive to handling stresses and are difficult to splice therefore such piles are not recommended.

5.2.2.2.3 STEEL PILES

Driven steel H-piles and steel tube piles are available options. If "an integral abutment" type bridge is to be considered then the use of H-piles is preferable. The boreholes indicate the presence of very dense and bouldery granular deposits overlying the bedrock and the piles are anticipated to develop satisfactory bearing resistance when penetrating 2 to 4 m into these materials.

A heavy pile section should be selected due to the anticipated tough driving conditions through the coarse grained deposits containing cobbles and boulders into the very dense bearing stratum. The anticipated pile tip depths and elevations and axial pile resistances are shown in Table 5.2.2.2.3.1. These values should be confirmed during the detailed foundation investigation.

Table 5.2.2.2.3.1
Recommended Preliminary Axial Resistance
for 310x110 Steel H-Piles

Support Location	Reference Borehole	Existing Ground Surface Elevation (m)	Estimated Depth of Pile Tip below Existing Ground Surface (m)	Estimated Pile Tip Elevation (m)	Recom-mended Factored Axial Resistance at U.L.S. (kN)	Recom-mended Axial Resistance at S.L.S. (kN)	Bearing Stratum
South Abutment	BPR 2	312.2	10.2-13.2	302.0--299.0	1700	1100	Silty sand/ Gravelly sand
Pier	BPR 3	308.6	13.6	295.0	1700	1100	Gravelly sand
North Abutment	BPR 4	307.1	18.1-19.1	289.0--288.0	1700	1100	Gravelly sand

Considering the length of the piles and in view of the fact that frequent cobbles and boulders were encountered in the boreholes below about 6 m to 14 m depth, the use of a heavy pile section (e.g. HP310x110) with reinforced tips as per MTO specifications (OPSD 3301.00) is recommended.

Steel tube piles may also be considered. Tube piles will provide a lower resistance, as they will not drive as deep in comparison with H-piles but the lower resistances may somewhat be compensated by the anticipated shorter pile lengths. As the upper zones of the soils encountered at the site are not in a dense condition, the required flex zone in the case of an integral abutment type bridge may not present a problem for the design of integral abutments. This should, however, be discussed with the structural engineer.

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 1500 kN and an Axial Resistance S.L.S. equal to 1000 kN per pile at depths ranging between 7 to 10 m (Elevation 305-302 m at Borehole BPR 2), 12.5 m or Elevation 296 m (at Borehole BPR 3 and about to 17 m or Elevation 290 m at Borehole BPR 4. Tube piles will need to be filled with concrete after their installation and examination (for possible damage). Again, as a protection against hard driving conditions and coarse soil (cobbles and boulders) particles relatively thick steel section should be selected.

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

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 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 at 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 3400 kN (i.e. 1700 divided by 0.5 = 3400) and for 300 mm nominal diameter steel tube piles it would be 1500÷0.5 = 3000 kN.

In accordance with the above criterion, the piles should be driven to about 3 m above the design elevation and driving should then 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 this phenomenon occurs, the affected piles will need to be re-driven. 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, as per OPSS-903S01, 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 ordering of the piles.

The minimum spacing between the piles should 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.

5.2.3 GENERAL COMMENTS ABOUT FOUNDATIONS

To accommodate the grade of the new underpass structure, approach embankments of about 3 to 5 m height will be required. Induced stresses due to the weight of the fill placed for the approach embankments will cause the 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 downdrag, and also to pre-induce the settlements for the performance of the paved highway near the bridges, and to minimize lateral loads on the piles from the lateral yield of the silts (at the south abutment) and silty clays (at the north abutment), the embankments should be placed to their final grades about six weeks ahead of pile driving at both abutments.

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:

$$k_s = n_h z/d \qquad \text{kN/m}^3$$

Where

k_s = coefficient of horizontal subgrade reaction
z = depth — m
d = pile width — m
n_h = coefficient related to soil density; see Table 5.2.3.1 — kN/m^3

Also presented in the same table are estimated values for angle of internal friction and bulk unit weights. Since the soils at the abutment locations are considered to be cohesionless, the undrained shear strength is not applicable on this site.

Table 5.2.3.1.
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 BPR 2	0.2– 5.0 5.0 – 13.2 13.2-17.3	312.0-307.2 307.2-299.0 299.0-294.9	Silt Silty sand Gravelly sand	18.5 20.5 21.5	30 34 36	1400 11000 11000	Not applicable
North Abutment Borehole BPR 4	0.3 – 13.2 13.2-16.2 16.2-21.4	306.8-293.9 293.9-290.9 290.9-285.7	Silt Silty sand Gravelly sand	18.5 20.5 21.5	31 34 36	1400 8000 11000	Not applicable

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

Factored Horizontal Resistance at U.L.S. = 120 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.

In accordance with MTO requirements (MTO Structural Office Standard), piles for integral abutments require a 3 m long flex zone. In essence where a false RSS type abutment is to be constructed, the current MTO standard for the flex zone consists of an annular space in between two concentric corrugated steel pipes (CSP's). One of the CSP's surrounds the H-pile (i.e. has a diameter of about 600 mm surrounding the pile, while the second CSP has a somewhat larger diameter; typically 800 m for a 310 mm H-pile). The annular space in between the CSP's is the 3 m long flex zone.

As the surficial soils are weak, it will likely be necessary to remove and replace the upper 0.6 to 0.8m of the existing subgrade with engineered fill in order to provide a suitable founding medium for the foundations of the panel facing of the RSS wall. Depending on the season of construction, the water table at the site could be high, requiring some dewatering during this operation.

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.

As an alternative to conventional retaining walls, MTO's Retained Soil System may be used. The following should be included in the Contract Documents:

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

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

5.4 APPROACH EMBANKMENTS

As mentioned before, in order to provide sufficient grade differential over the existing grade, approximately 3 to 5 m high embankments will have to be constructed for the proposed overpass.

Based on the limited borehole and laboratory data, preliminary calculations indicate that the subsurface deposits at the site can support a maximum 5 m high embankment 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. In cases the height of slope is more than 7 m (e.g. forward slopes for the abutment due to lowering of the existing grades for the S.B.L. of Highway 11) a detailed stability analysis should be conducted.

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).

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 (i.e. high water table and silty soils). In wet areas 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 granular materials.

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 30 mm. However, the underlying foundation soils can be expected to settle an additional 50 to 60 mm. As mentioned before, we recommend that the embankment fills be placed to their full height at least six weeks ahead of pile driving. If this recommendation is followed, about 90% of the estimated settlements would be completed before the highway is paved, which is considered acceptable.

5.5 CONSTRUCTION COMMENTS

The groundwater table at the site appears to be high. 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 below the groundwater table, dewatering will be necessary to stabilize the silt and to facilitate the construction. If the head of water is not more than about 0.5 m, the water can be collected in temporary filtered sumps and removed by pumping. Proper filtering is necessary to prevent the removal of soil fines. If the head of water is higher than about 0.5 m, more elaborate dewatering methods such as deep filtered sumps, filtered wells or more elaborate methods, such as vacuum well points, would be necessary.

If the existing grades will be lowered for the construction of the S.B.L. of Highway 11, then the presence of a high water table should be taken into consideration, including monitoring of the water table by means of piezometers during the detailed foundation investigation.

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

REGISTERED PROFESSIONAL ENGINEER
Z. S. OZDEN
PROVINCE OF ONTARIO

Zuhtu Ozden, P.Eng.

LICENSED PROFESSIONAL ENGINEER
K. R. PEAKER
PROVINCE OF ONTARIO

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.

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