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HWY. No. 11

LOCATION PICKEREL AND JACK LAKE RD  
UNDERPASS

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OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. \_\_\_\_\_

REMARKS: \_\_\_\_\_

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**DRAFT**

**DRAFT FOUNDATION INVESTIGATION AND DESIGN REPORT  
FOR  
PROPOSED PICKEREL AND JACK LAKE ROAD UNDERPASS  
STRUCTURE SITE NO. 44-399  
DISTRICT 52, HUNTSVILLE  
W.P. 473-93-00**

**Submitted To:**

**Delcan Corporation  
133 Wynford Drive  
North York, Ontario, M3C 1K1  
Canada**

**Submitted By:**

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**August 1999  
TT98820E**

**DRAFT**

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August 5, 1999.  
**Ref. No.: TT98820E**

Delcan Corporation  
133 Wynford Drive  
North York, Ontario, M3C 1K1  
Canada

**Attention: Mr. Khaled El-Dalati, P. Eng.**

Dear Sir:

**Re: DRAFT FOUNDATION INVESTIGATION AND DESIGN REPORT  
FOR  
PROPOSED PICKEREL AND JACK LAKE ROAD UNDERPASS  
STRUCTURE SITE NO. 44-399  
DISTRICT 52, HUNTSVILLE  
W.P. 473-93-00**

We take pleasure in enclosing six (6) copies of our Draft Foundation Investigation and Design Report carried out for the above mentioned project and we will be glad to discuss any questions arising from this work.

Soil samples will be retained for a period of one year, and will thereafter be disposed of unless we are otherwise instructed.

We thank you for giving us this opportunity to be of service to you.

Sincerely,



Z.S. Ozden, P. Eng.,  
Principal Engineer.

ZSO/dee

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## 1.0 INTRODUCTION

AGRA, Consulting Geotechnical Engineers, was retained by Delcan Corporation to conduct a foundation investigation at the site of a proposed bridge that will carry Pickerel and Jack Lake Road over the proposed realigned southbound and northbound (existing) alignment of Highway 11. The site is located north of Burk's Falls, at the present intersection of Pickerel and Jack Lake Road and Highway 11, in the Township of Armour, Lot 14, Concession 11 in the District of Parry Sound. The proposed bridge will be an approximately 85 m long, two span, 2-lane structure.

The purpose of the investigation has been to obtain information about the subsurface conditions at the site of the proposed bridge and approach embankments by means of exploratory boreholes, and based on the findings, to provide recommendations for the geotechnical design of the foundations of the proposed structure and the approach fills.

## 2.0 SITE DESCRIPTION AND PHYSIOGRAPHY

The existing Highway 11 embankment to the south and north of the at-grade Pickerel and Jack Lake Road intersection is about 4 m and 6 m high, respectively. The grade at the intersection is about Elevation 327 m and drops easterly to about Elevation 325 m (at Borehole PL1 location) and westerly to Elevation 324 m (at Borehole PL2 location) but rises further west.

Houses are present about 50 ± m west of the proposed west abutment, and about 20 ± m east of the proposed east abutment. The ground on the west side of the present at-grade intersection, at the toe of the existing highway embankment is swampy. South-east of the intersection is a paved parking lot at an abandoned commercial property.

Based on available geologic information, the site is in an area of glaciolacustrine deposits. Generally after the last glacial withdrawal, ice-contact sediments (sands and gravels) followed by glaciofluvial sediments (ranging from deltaic, valley fill and nearshore sands and gravels) were deposited on top of the existing sandy glacial till or Precambrian bedrock. The area was then inundated by glacial Lake Algonquin, depositing sands, silts and clays in low lying areas. Organic soils (i.e. peat) were then deposited in poorly drained areas.

Based on published information, the bedrock generally consists of strongly foliated gneissic to migmatic rocks of the Central Gneiss Belt, which is part of the Grenville Province (a structural subdivision of the Canadian Shield).

### 3.0 INVESTIGATION PROCEDURES

The fieldwork for this project was performed during the periods of May 10 to 13, 15 and 16, 1999, and consisted of drilling and sampling five boreholes (Borehole Nos. PL1 through 5) and performing five dynamic cone penetration tests adjacent to the boreholes. The plan locations of the boreholes, along with stratigraphic sections are shown on Drawing No. 1.

Due to the presence of underground Bell and fibre-optic television cables along the east abutment location, the exact proposed abutment location was not accessible. The borehole was therefore drilled somewhat offset from the actual proposed foundation element, as close as practicable.

The boreholes were advanced using solid and hollow stem continuous flight augers with a track-mounted power auger drilling rig (BOA 6M) owned and operated by Groundworks Drilling Inc., under the full-time supervision of a soils engineer from AGRA.

Sampling in the boreholes was effected at frequent intervals of depth by the Standard Penetration Test Method (SPT), as specified in ASTM Method D 1586. This consists of freely dropping a 63.5 kg hammer a vertical distance of 0.76 m to drive a 51 mm diameter o.d. split barrel (split-spoon) sampler into the ground. The number of blows of the hammer 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 and this gives an indication of the consistency or the compactness condition of the soil deposit.

In addition, a dynamic cone penetration test was performed adjacent to each of the five boreholes. This test consists of driving a 60° point, 50 mm diameter cone attached to the drill rod continuously, into the undisturbed ground with a driving energy of 475 J (63.5 kg hammer falling freely a distance of 76 cm) per blow. The number of blows for each 30 cm of penetration is recorded and this provides an indication of the relative changes in the soil density with depth.

Due to the presence of cobbles and boulders within the glacial till deposits, the deeper boreholes (i.e. Boreholes PL3, 4 and 5) were either advanced by wash boring or rock coring methods through the cobbles and boulders in the overburden, utilizing NW and BW size casing to the bedrock surface and the bedrock was subsequently cored using NXL and BXL size core-barrels.

The borehole locations were established in the field by our engineering staff, in relation to the already staked out centre-line of Pickerel and Jack Lake Road (by Dearden and Stanton Limited). The borehole geodetic elevations and co-ordinates were later taken and provided to us by surveyors from Dearden and Stanton Limited.

The soil samples were shipped in sealed containers to our geotechnical laboratory in Toronto (Scarborough) for further examination and classification. A laboratory testing programme, consisting of natural moisture content determinations, Atterberg Limit tests and grain size analyses, was performed on selected representative soil samples. The results of the laboratory tests are presented on the appropriate Borehole Log Sheets and also in Figure Nos. 1 to 5.

The boreholes were left open until the end of each work day to enable us to take additional water level readings. Standpipe piezometers were installed in Boreholes PL2, 3, 4 and 5 to monitor the groundwater level over a prolonged period of time without interference from surface water.

#### 4.0 SUBSURFACE CONDITIONS

The subsurface conditions were explored at five borehole locations (Borehole Nos. PL1, 2, 3, 4 and 5), and were inferred at the locations of five dynamic cone penetration tests. The locations of the boreholes and cone penetration tests are shown on the Plan and Profile Drawing No. 1 and are also indicated on the individual Borehole Log Sheets. Cross-sections of inferred subsurface stratigraphy are given on Drawing No. 1.

The ground surface along the alignment of the Pickerel and Jack Lake Road is generally at a high at Highway 11, falls to the west of the west abutment location and then rises further to the west. East of Highway 11 the grade gently drops to the east. The road embankments are generally 4 to 6 m in height at the existing intersection and fall to a height of about 1 to 2 m at the proposed abutment locations.

In general, the boreholes have shown beneath a granular road fill, the presence of organic deposits on the west side of Highway 11. In general, underlying the organic deposits on the west side and the granular road fill on the east, the boreholes show the presence of sandy silt. The sandy silt is in turn underlain by a cohesionless glacial till deposit with frequent cobbles and boulders. Precambrian diorite to granodiorite bedrock was encountered at a depth of about 11 to 18 ± m (approximately Elevation 313 to 308 m). The groundwater table at the time of our investigation was encountered at depths of about 1 to 4 m below existing grade.

Details of the subsurface conditions encountered in the boreholes are presented on the Borehole Log Sheets. The following paragraphs are only meant to complement and summarize these data.

#### 4.1 SAND (FILL)

Below the surficial asphaltic concrete in Borehole PL5, and at the surface of the remaining boreholes, the road embankment was penetrated, which at the borehole locations, consists of sand with some gravel and traces of silt. The presence of cobbles and boulders was noted in the fill in Borehole PL4. The fill ranges in thickness from 0.6 m (at Borehole PL1) to 3.6 m (at Borehole PL4). Two grain size distribution analyses were conducted on samples of the fill and the results are presented in Figure No. 1. The results indicate 13-20% gravel, 71-82% sand and 5-9% silt and clay size particles.

Measured 'N'-values within the fill material generally range from 9 to 64 blows/0.3 m, indicating a compactive effort has been applied to the fill, although not uniformly.

Measured natural moisture contents range from 1 to 4%.

In our experience the thickness of fill frequently varies in between and beyond the borehole locations.

## 4.2 ORGANIC SOILS

Boreholes PL2, PL3 (west side) and PL4 (central borehole) encountered an organic-rich sand deposit underlain by peat at Boreholes PL3 and PL4, as described below.

### 4.2.1 Organic-rich Sand

Below the granular pavement fill in Boreholes PL2, 3 and 4 (west side and centre), an organic rich deposit was contacted, consisting of an irregular mixture of sand, silt and organics, to a depth of 2.1 m (about Elevation 321.7 m) to 4.3 m (Elevation 322.0 m) below the existing grade. The organic components within the deposit comprise of topsoil, rootlets, decomposed wood and peat. Measured natural moisture contents of samples from the material range from 26 to 37%. Standard penetration tests performed in this material yielded 'N'-values ranging from 6 to 12 blows/0.3 m, indicating a loose to compact condition, but generally loose.

### 4.2.2 Peat

Underlying this organic rich mixture in Boreholes PL3 and 4, a fibrous peat deposit was contacted to depths of 2.9 m or Elevation 320.9 m (Borehole PL3) to 5.2 m or Elevation 321.1 m (Borehole PL4). The thickness of the peat ranges from about 0.8 m at Borehole PL3 to 0.9 m at Borehole PL4. The moisture content of a sample from the peat was measured in the laboratory and a value of 140% was obtained. Occasional organic clay seams are present in this deposit in Borehole PL4. An Atterberg Limits test was conducted on a sample of the organic clay, and the results are presented in a plasticity chart in Figure No. 4. The results are also presented below.

Liquid Limit:	81%
Plastic Limit:	60%
Plasticity Index:	21%
Moisture Content:	97%

Measured 'N'-values of 3 and 4 blows/0.3 m were obtained within the peat.

The organic deposits, especially the peat, can be expected to be highly compressible.

In our experience the thickness of organic soils frequently varies in between and beyond the borehole locations.

#### 4.3 SANDY SILT

Below the fill in Boreholes PL1 and 5 and the organic deposits in Boreholes PL2 and 4, sandy silt was encountered. This cohesionless deposit extends to depths of 2.9 m (Elevation 321.0 m) to 6.8 m (Elevation 319.5 m) below existing grade. At the borehole locations the deposit is 0.8 to 3.3 m thick and contains traces of roots and decomposed organics. Measured 'N'-values range from 8 to 35 blows/0.3 m indicating a loose to dense condition, but generally compact. Measured natural moisture contents range from 17 to 35%.

Three grain size distribution analyses were conducted on samples from this deposit and the results are presented in Figure No. 2. The results indicate 0% gravel, 14-33% sand and 67-86% silt particles.

In Borehole PL5, a strong gasoline odour was noted within this deposit.

#### 4.4 SAND WITH GRAVEL

Below the sandy silt in Borehole PL4, a cohesionless sand with gravel deposit was contacted from a depth of 6.8 m (or Elevation 319.5 m) to 10.0 m (or Elevation 316.3 m) below existing grade. Two measured 'N'-values within this deposit are 36 and greater than 100 blows/0.3 m, indicating a dense to very dense condition. The natural moisture content of a sample from this deposit was measured to be 13%.

#### 4.5 HETEROGENEOUS MIXTURE OF SAND, SILT AND GRAVEL (GLACIAL TILL)

Underlying the above described soils, all five boreholes encountered a cohesionless glacial till deposit at depths ranging from 2.9 m (about Elevation 321.0 m) in Boreholes PL2 and 3 to 10.0 m (Elevation 316.3 m) at Borehole PL4. The glacial till deposit is a heterogeneous mixture of sand, silt and gravel size particles with traces of clay. In Borehole PL5 occasional sand layers were encountered at depth. Frequent cobbles and boulders were also encountered within this deposit. Boreholes PL1 and 2 were terminated within this deposit at depths of 7.9 m and 4.9 m, respectively, upon encountering refusal to further augering probably on boulders. Boreholes PL3, 4 and 5 also encountered refusal to further augering due to the presence of cobbles and boulders and in order to advance these boreholes through this deposit wash boring and rock coring methods were required. In these boreholes this basically granular glacial till deposit extended to depths ranging from 10.7 m (Elevation 313.1 m) to 18.3 m (Elevation 307.7 m).

Six grain size analyses were conducted on samples from the glacial till deposit, resulting in the following grain size measurements.

Gravel:	2-23%
Sand:	40-65%
Silt:	17-47%
Clay:	0-6%

The grain size analyses results are presented in Figure No. 3.

Measured 'N'-values within this deposit generally range from 8 to in excess of 50 blows/0.3 m, indicating a loose to very dense condition, but generally compact to dense in the upper portion becoming very dense with depth.

Measured natural moisture contents range from 4 to 21%.

Underlying the glacial till deposit in Borehole PL4, a cohesive clayey silt layer was contacted at a depth of 15.3 m (or Elevation 311.0 m) and extended to the surface of the bedrock at a depth of 15.8 m (or Elevation 310.5 m). An Atterberg Limits test was conducted on the sample obtained and the results are presented below:

Liquid Limit:	19%
Plastic Limit:	15%
Plasticity Index:	4%
Moisture Content:	19%

A measured 'N'-value within this 0.5 m thick cohesive layer resulted in a value in excess of 100 blows/0.3 m, indicating a hard consistency. However, it should be noted that the moisture content of the clayey soil is at the liquid limit.

#### 4.6 BEDROCK

Bedrock was encountered and cored in Boreholes PL3, 4 and 5 at depths of 10.7 m (Elevation 313.1 m), 15.8 m (Elevation 310.5 m) and 18.3 m (Elevation 307.7 m) below existing ground surface, respectively. Boreholes PL3 and 4 were advanced 3.1 and 3.2 m, respectively into the bedrock while Borehole PL5 could only be extended 1.5 m into the bedrock, due to the fact that the core bit was lost in this borehole. The recovered core samples show that the Precambrian bedrock consists of a massive, closely to moderately closely jointed, slightly metamorphosed diorite to granodiorite. The percentage of core recovery was 81 to 100%. Rock quality designation (R.Q.D.) values of 47 to 81% were measured in Boreholes PL3 and 4, with lower values in Borehole PL5 (16 to 17%). Based on these values together with a visual examination of the rock cores, the rock is considered to be of poor to good quality but generally fair to good in Boreholes PL3 and PL4. In Borehole PL5, based on RQD values, the rock is generally of very poor quality, but based on a visual examination of the cores, the rock can be generally described as poor quality.

Based on the borehole results, the bedrock surface dips from west to east.

*important!*

#### 4.7 GROUNDWATER CONDITIONS

Groundwater levels in the open boreholes were observed during the drilling and at the completion of each borehole. To enable us to measure water levels at the site over a prolonged period of time without interference from surface water, standpipe piezometers were installed in Boreholes PL2, 3, 4 and 5.

The recorded values, are shown on the individual Borehole Log Sheets. Based on the recorded values in the piezometers installed and moisture contents of recovered samples, the groundwater levels at the time of the investigation generally ranged from 1 to 4 ± m below the ground surface (approximately Elevation 324 to 321 m). It should, however, be pointed out that the groundwater at the site could fluctuate seasonally and can be expected to be somewhat higher during the early spring months and in response to major weather events.

#### 5.0 DISCUSSION AND RECOMMENDATIONS

The proposed Highway 11 realignment will consist of a four lane divided highway with an approximately 30 m wide median. The proposed bridge will carry Pickerel and Jack Lake Road over the proposed southbound and northbound (existing) alignment of Highway 11 (the proposed northbound lane will follow the existing Highway 11 alignment). The proposed bridge will be an approximately 85 m long, two span, 2-lane (12 m wide) structure. The ground surface along the alignment of the Pickerel and Jack Lake Road is generally at a high at Highway 11, falls to the west towards the west abutment location and then rises further to the west. East of Highway 11 the grade drops gently to the east. The road embankments are generally 4 to 6 m in height at the intersection and fall to a height of about 1 to 2 m at the proposed abutment locations.

In general, the existing ground elevation along the bridge alignment is 327 to 324 m. The proposed grade of Highway 11 at the bridge site is approximately Elevation 327 m. The proposed grade of the bridge deck will generally be about 335 ± m. Therefore the centre-line grade will be raised by about 8.5 and 11.5 m at the east and west abutment locations, respectively.

In general, the boreholes have shown beneath a pavement fill and organic deposits (on the west side of Highway 11), the presence of a generally compact sandy silt overburden overlying a compact to very dense cohesionless glacial till deposit with frequent cobbles and boulders. The Precambrian diorite to granodiorite bedrock was encountered at a depth of about 11 to 18 ± m (approximately Elevation 313 to 307 m). The groundwater table at the time of our investigation was encountered at depths of about 1 to 4 m below existing grade.

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## 5.1 FOUNDATIONS

It is our understanding that the abutments of the proposed bridge will be of the "integral" type and will be supported on driven steel H-piles. However, preliminary recommendations for shallow foundations are given later in this report.

The boreholes show that for the prevailing subsurface conditions the use of a low displacement pile, such as a steel H-pile with a heavy section, such as HP310X110 with reinforced tips as per MTO Specifications, would be better suited.

The piles would preferably be driven to the surface of the bedrock where uniformly high resistances can be utilized. The surface of the bedrock was recorded in Boreholes PL3, 4 and 5 at Elevations of 313.1, 310.5 and 307.7 m, respectively, although experience in the general area shows that in many cases the surface of the bedrock can frequently be uneven and unpredictable. Ideally, the piles would be driven to the surface of the bedrock where very high resistances can be used. However, the boreholes encountered frequent cobbles and boulders in the cohesionless glacial till deposit overlying the bedrock, and the boreholes had to be extended by wash boring and rock coring methods to reach the bedrock. Because of this it is likely that most of the piles will not reach the surface of the bedrock and will likely terminate in the bouldery glacial till overburden above the bedrock surface.

The following table summarizes the estimated approximate pile tip elevations and axial resistances that may be utilized for HP310X110 steel H-piles for design purposes.

TABLE 1

SUPPORT LOCATION	REFERENCE BOREHOLE	ESTIMATED DESIGN TIP ELEVATION (m)	FACTORED AXIAL RESISTANCE AT U.L.S.* (kPa)	AXIAL RESISTANCE AT S.L.S. (kPa)
West Abutment	PL3	316±	1,550	1,100
Central Pier	PL4	315±	1,360	950
East Abutment	PL5	316±	1,550	1,100

\* Resistance factor of 0.5 applied, as per the Ontario Highway Bridge Design Code, 3<sup>rd</sup> Edition (1991).

The minimum pile length below the pile caps (e.g. central pier) should not be less than 5 m. The minimum permissible pile length should also be discussed with the structural engineer. It is possible that depending on the frequency of cobbles and boulders, the piles may drive several meters below the tip elevations given above. We recommend that this aspect be taken into consideration when ordering the piles. It is also possible that due to the presence of cobbles and boulders, premature refusal may be encountered above the elevations given in Table 1. If



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difficulties are experienced in penetrating to the required pile tip elevation it may be necessary to preauger with a small diameter pilot hole (e.g. 200 mm) to a depth of about 1 to 2 m above the acceptable pile tip elevation, as directed by the Engineer.

*Do we actually need to do this?*

The piles should be driven with a suitably heavy hammer capable of delivering a rated capacity of at least 50 kJ/blow. The energy should however be restricted to not more than 60 kJ/blow.

The driving of the piles should be controlled by a recognized pile driving formula, such as the Hiley Formula. The estimated ultimate resistance of the piles (driven to practical refusal in the overburden) by the Hiley Formula is approximately 3,100 kN. This value was arrived by dividing the factored axial resistance at U.L.S. by a resistance factor of 0.5, as per current MTO convention. Because of the presence of frequent cobbles or boulders in the overburden, and the anticipated hard driving conditions, as mentioned before, the piles should be equipped with reinforced tips as per MTO Standards (O.P.S.D. 3301.00).

✓

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

✓

In cohesionless soils the coefficient of horizontal subgrade reaction may be estimated from;

$$k_s = n_h z/d$$

where  $k_s$  = coefficient of horizontal subgrade reaction  
 $z$  = depth  
 $d$  = pile width  
 $n_h$  = coefficient related to soil density as given in Table 2

*Mark lateral resistance of piles*

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

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TABLE 2

AREA/REFERENCE BOREHOLE NO.	APPLICABLE DEPTH FROM EXISTING GROUND SURFACE/ELEVATION	SOIL TYPE	BULK UNIT WEIGHT (kN/m <sup>3</sup> )	ANGLE OF INTERNAL FRICTION ( $\phi$ ) DEGREES	RECOMMENDED $n_h$ VALUE (MN/m <sup>2</sup> )
West Abutment					
PL3	3 - 6 m (321 - 318 m)	loose to compact glacial till	21	31	4.0
	6 - 10 m (318 - 314 m)	very dense glacial till	22	33	11.0
Central Pier					
PL4	5 - 7 m (321 - 319 m)	dense to loose sandy silt	19	27	3.0
	7 - 10 m (319 - 316 m)	dense to v. dense sand with gravel	21	33	9.0
	10 - 15 m (316 - 311 m)	v. dense glacial till	22	33	11.0
East Abutment					
PL5	2 - 5 m (324 - 321 m)	loose to compact sandy silt	19	27	3.0
	5 - 9 m (321 - 317 m)	compact glacial till	21	32	5.0
	9 - 15 m (317 - 311 m)	v. dense glacial till	22	33	11.0

The recommended horizontal resistances for the HP310X110 steel H-piles are as follows:

Factored Horizontal Resistance at U.L.S. = 110 kN  
Horizontal Resistance at S.L.S. = 50 kN

Unbalanced horizontal forces at the pier location (and also at the abutment locations, if an integral abutment type structure will not be built) could also be resisted by battered piles.

In order to minimize the effect of any downdrag we recommend that the approach embankment fills be placed to their final grade elevation at least four weeks prior to driving the piles, provided that all the organic soils were removed and replaced with granular, as is discussed in Section 5.3 of this report. After this waiting period, the fill can be re-excavated to the required elevation for the driving of the piles.

*check values*

*Is this correct?*

In accordance with MTO requirements (MTO Structural Office Standard), piles for integral abutments require a 3 m long flex zone. In essence the current MTO standard for the flex zone consists of an annular space in between two consecutive CSP's. One of the CSP's surrounds the H-pile (i.e. has a diameter slightly greater than the pile width, while the second CSP has a somewhat larger diameter; typically 0.6 m for a 310 mm H-pile). The annular space in between the CSP's is the 3 m long flex zone. After the pile is driven, the space between the H-pile and the inner CSP is filled with cement bentonite or a suitable uniform sand, as per MTO requirements.

As mentioned before, the bridge will be of an integral abutment construction and therefore the use of steel H-piles is recommended for foundation support. In addition, owing to the high water level, necessitating careful dewatering, it is our opinion that the use of spread footing foundations is not a good option, unless successful dewatering can be applied. If, however, spread footing foundations have to be used, they should be supported on engineered Granular 'A' fill and our comments regarding the use of spread footing foundations are as follows:

Abutment Footings: It is recommended that after dewatering the site, all the organic and otherwise unsuitable soils should be removed to the surface of the inorganic stratum. The excavation should then be extended within a horizontal distance of not less than 2 m beyond the proposed footing perimeter, to the surface of competent soil.

The proposed excavation depths at the borehole locations are as follows:

TABLE 3

GENERAL AREA	BOREHOLE NO.	EXISTING GROUND ELEVATION AT BOREHOLE LOCATION	RECOMMENDED SOIL REMOVAL DEPTH/ELEVATION	ANTICIPATED SOIL SUBGRADE
West Abutment	PL3	323.8 m	3.1 m/320.7 m	silty sand till
East Abutment	PL5	326.0 m	3.0 m/323.0 m	sandy silt

Prior to the excavation it should be ensured that the site was dewatered to at least 1 m below the proposed subgrade excavation level. After the excavation reaches the required elevation, the subgrade should be evaluated and approved by the Geotechnical Engineer. If necessary, the excavation may need to be deepened to the surface of sufficiently competent soil, as directed by the Engineer. In this case, however, deeper dewatering may be required. In any case, after its approval, the exposed subgrade may need to be compacted from the surface, if required by the Engineer, to achieve a density of not less than 98% of the material's Standard Proctor Maximum Dry Density (SPMDD).

The fill used to raise the grade inside the excavation should consist of Granular 'A' quality material placed when its moisture content is within  $\pm 2\%$  of its optimum moisture content. It should be placed in layers not exceeding 200 mm in thickness and should be uniformly compacted to not less than 100% of its SPMDD.

The grade should then be raised to the required footing elevation using compacted Granular 'A' fill core as per MTO convention, as shown in Appendix B. The Granular 'A' should be uniformly compacted to 100% of its SPMDD and placed at a moisture content within  $\pm 2\%$  of optimum in accordance with O.P.S.S. 501. The preparation of the subgrade and the Granular 'A' core should be under full-time geotechnical supervision.

For abutment footings constructed in this manner and provided that the thickness of the engineered Granular 'A' fill is not less than 3.0 m, the following resistances can be used for design purposes.

$$\begin{array}{lcl} \text{Factored Resistance at U.L.S.} & = & 900 \text{ kPa} \\ \text{Resistance at S.L.S.} & = & 300 \text{ kPa} \end{array}$$

*350 kPa*

Pier Footing: At the pier location the final grade elevation will be about 325 m. Here too after dewatering, all the organic and otherwise unsuitable and/or weak soils should be removed. The excavation should then be extended within a horizontal distance of not less than 2 m beyond the proposed footing perimeter, to the surface of the competent soil. It should however be ensured that the site was previously dewatered to at least 1 m below the proposed excavation depth. Based on the findings of Borehole PL4, the recommended excavation Elevation is 319.5 m. After the excavation has reached the required elevation, the subgrade should be evaluated and approved by the Geotechnical Engineer. If necessary, the excavation may need to be deepened to the surface of the sufficiently competent soil, in which case deeper dewatering may be required. After approval, the subgrade may need to be compacted, if required by the Engineer, to achieve a density of not less than about 98% of the material's SPMDD. The fill used to raise the grade to the bottom of the footing level should consist of Granular 'A' quality material. It should be placed in layers not exceeding 200 mm in thickness at a moisture content of  $\pm 2\%$  of its optimum and should be uniformly compacted to not less than 100% of its SPMDD. Provided that the thickness of the Granular 'A' layer beneath the footing is not less than 3.0 m, a Factored Bearing Resistance at U.L.S. of 900 kPa and a Bearing Resistance at S.L.S. equal to 300 kPa can be assigned to the soil prepared in this manner.

Under inclined loading conditions, the bearing resistance at U.L.S. should be reduced in accordance with Clause 6-8.4.2. of the O.H.B.D.C., 3<sup>rd</sup> Edition.

For frost protection, the footings should have a permanent earth cover of at least 1.8 m.

The unfactored horizontal resistance against sliding between concrete and Granular 'A' type material can be calculated using a friction angle of 35 degrees.

In the above quoted values for spread footing foundations the serviceability condition is based on the premise that the total and differential settlements will not exceed 25 mm and 20 mm, respectively, provided that proper dewatering will be applied and the engineered fill will be placed on undisturbed, competent subgrade. The settlements due to grade raise at the proposed abutment locations will be in addition to the above quoted values. This aspect should be further looked into if the use of spread footings is to be considered. In that case, we also recommend that

*???*

additional boreholes be drilled to confirm and/or re-evaluate the above recommendations.

## 5.2 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<sup>3</sup>

Coefficient of Lateral Earth Pressures:

$$K_a = 0.27$$

$$K_o = 0.43$$

### Compacted Granular 'B'

Unit Weight = 21 kN/m<sup>3</sup>

Coefficient of Lateral Earth Pressures:

$$K_a = 0.31$$

$$K_o = 0.47$$

### Rock Fill

Unit Weight = 18 kN/m<sup>3</sup>

Coefficient of Lateral Earth Pressures:

$$K_a = 0.27$$

$$K_o = 0.43$$

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., 3<sup>rd</sup> 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 of the O.H.B.D.C., 3<sup>rd</sup> Edition.

.../...

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Vibratory equipment for use behind abutments and retaining walls should be res per current MTO practice.

If rock fill is used for backfill, special care is required to prevent damage to the retainin. In such a case, a cushion of Granular 'A' material or finely graded rock fill (e.g. less th normal diameter) should be placed between the structure and the rock fill. This cushion at least 0.45 m wide and if Granular 'A' is used, proper filtering should be provided to pre loss of finer particles from the Granular 'A' cushion into the coarse rock fill.

As an alternative to conventional retaining walls, MTO's Retained Soil System may be used. T 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 moderate to high appearance.

### 5.3 APPROACH EMBANKMENTS

The grades at the proposed west and east approach embankments, at the existing centerline gra will be raised by about 11.5 m and 8.5 m, respectively.

Based on the borehole results, the strength of the foundation materials is such that deep-sea failures are not anticipated, provided all organic soils, weak or otherwise unsuitable materials removed as per MTO Standards before placing the fill.

All organic and other unsuitable soils should be removed within an envelope given by an imagin slope not steeper than 1:1 from the toe of the proposed embankment as depicted by the ske given in Appendix C. After stripping, the exposed subgrade should be inspected, approved where feasible properly compacted from the surface under the supervision of qualified person using a suitable compactor.

It should be pointed out that Boreholes PL2, 3 and 4, drilled at the proposed west embankm west abutment and central pier locations, encountered below the existing granular pavement 1.3 to 1.6 m thick organic rich and peat deposits, which we recommend should be removed should also be pointed out that the thickness of these unsuitable soils could vary in between beyond borehole locations and may be in excess of the measured values.

Assuming properly compacted, acceptable inorganic earth fill material, 2 horizontal in 1 vertical slopes can be used but in accordance with MTO Northern Region Policy, for embankment hei of greater than 6 m, a 2 m wide mid-height bench (berm) should be provided. The berm grac should be sloped (say 1V:20H) to drain away from the embankment. Proper erosion co

measures should be implemented both during the construction and permanently. This can be achieved by immediate seeding or sodding (O.P.S.S. 572).

Provided that all organic and otherwise unsuitable materials are removed and the subgrade is properly compacted from the surface as detailed above, the settlement of the foundation materials (i.e. not including the settlement of the embankment material under its own weight) should not exceed 60 mm and should be substantially completed within four weeks of placing the embankment fill to its full height. Such settlements are considered acceptable and will not necessitate preloading or surcharging.

Water levels at the time of our investigation were generally measured immediately below the granular road fill. Unless dewatering is implemented, therefore, the removal of the organic soils encountered on the west side at Boreholes PL2, 3 and 4 will likely have to be carried out by dredging methods.

The materials used for the construction of the embankment fills should consist of approved, clean earth fill (e.g. Select Subgrade Materials - O.P.S.S. 1010). The existing embankment fill, provided it is carefully removed and not contaminated with the underlying soils, could be re-used for this purpose. The earth 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 Maximum Dry Density. The degree of compaction within the top 0.6 m of the fill (i.e. the subgrade immediately beneath the granular sub-base) should be increased to 98%. The selection, placement and compaction of the fill should be carried out under geotechnical control. The settlement of the embankment earth fills prepared as described above should not exceed approximately 60 mm. The time rate of settlement will depend on the materials used for construction. For granular fills it should be mostly elastic (i.e. should be substantially completed during the construction and within a few weeks thereafter) while clayey fills will consolidate over a longer period of time. This quoted settlement would be in addition to the foundation settlements quoted earlier in this section of the report.

For embankment construction rock fill can also be used, if available. Side slopes of 1 1/4H:1V can be maintained for embankments constructed from rock fill. In conformance with MTO Northern Region practice, a 2 m wide mid-height berm should be provided for embankment heights in excess of 6 m. Rock fill should not be used immediately behind the embankments because this will interfere with the installation of the piles.

#### 5.4 CONSTRUCTION COMMENTS

At the time of our investigation the groundwater levels in the boreholes were generally recorded immediately below the granular road fill. Groundwater at the original ground surface can be expected since the west side of the Highway is swampy. This condition will necessitate dewatering (or dredging methods) for the removal of organic soils. Dewatering is preferred to dredging because the subsequent placement of the embankment fills immediately above the existing ground surface can then take place under better controlled conditions, rather than below water. If the

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construction is not carried out under sufficiently controlled conditions the magnitude of settlement for embankments could increase.

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

If spread footing foundations are to be used, more extensive dewatering will be required to effectively lower the water table to at least 1 m below the anticipated excavation levels, and this will necessitate methods such as well pointing. Well points are generally effective to only about 5.5 m below the header pipe level and this should be taken into consideration if well points are to be used. We recommend that dewatering be implemented by a contractor specializing in this field. If dewatering is ineffective and the subgrade is disturbed, excessive settlements could occur after the application of structural loads.

In general, the sides of temporary excavations above the water table should be stable at 1.5H:1V side slopes. This to a certain extent depends on the height of excavation, the length of time they will remain open, protective measures, etc. and as such somewhat steeper side slopes may be feasible (i.e. 1H:1V) for shallow, short duration excavations while elsewhere some local flattening may be required. Temporary excavations below the watertable within the organic soils should be stable at about 3H:1V. If excavation and backfilling are carried out simultaneously, however, within the peat deposit 1:1 side slopes, as shown in Appendix C, will likely be stable for short periods of time.

## 5.5 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 is required for frost protection of foundations.

## 6.0 CLOSURE

We recommend that once the details of the structure are finalized, our recommendations should be reviewed for their specific applicability.

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

Sincerely,



Andrew Drevininkas, P. Eng.

AD/dee



Z.S. Ozden, P. Eng.

.../...

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## APPENDIX A

## **AGRA**

### **LIMITATIONS OF REPORT**

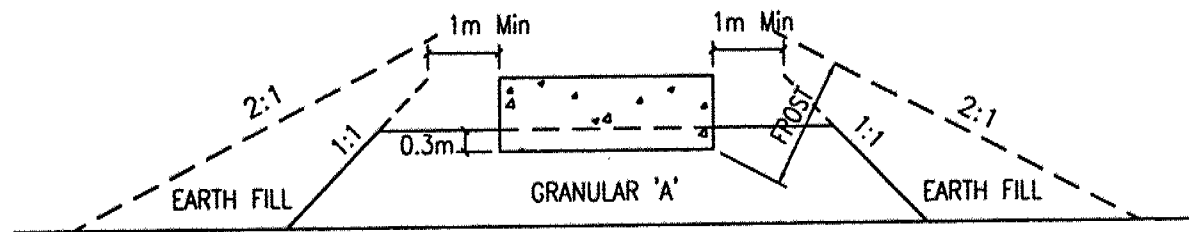
The conclusions and recommendations given in this report are based on information determined at the testhole locations. The information contained herein in no way reflects on the environmental aspects of the project, unless otherwise stated. Subsurface and groundwater conditions between and beyond the 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. It is recommended practice that the Geotechnical Engineer be retained during construction to confirm that the subsurface conditions throughout the site do not deviate materially from those encountered in testholes. 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. Since all details of the design may not be known, we recommend that we be retained during the final design stage to verify that the design is consistent with our recommendations, and that assumptions made in our analysis are valid.

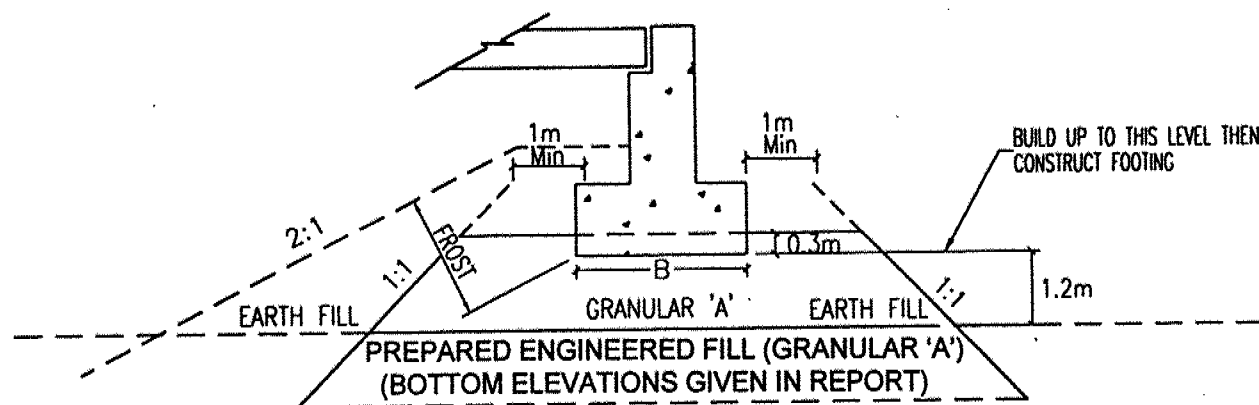
The comments made in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of testholes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of surficial 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. No other warranty is expressed or implied.

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. AGRA accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

## APPENDIX B



X SECTION



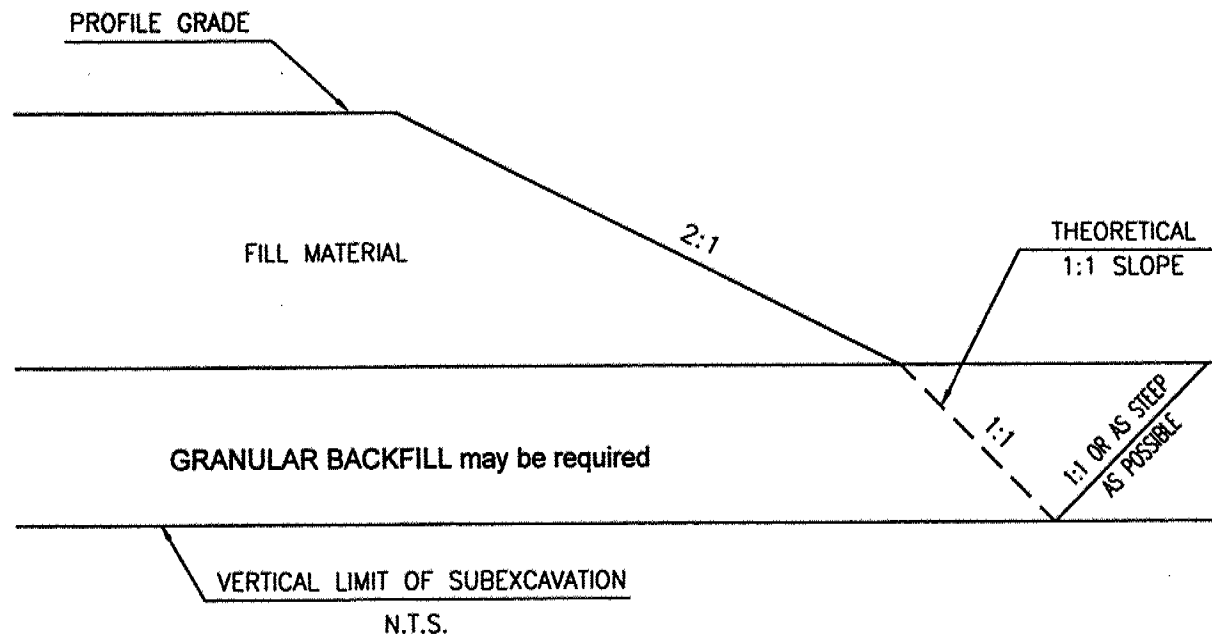
LONGITUDINAL SECTION

**NOTES:**

1. PREPARE SUBGRADE AS DISCUSSED IN REPORT.
2. PLACE GRANULAR 'A' & EARTH FILL TO BOTTOM OF FOOTING LEVEL COMPACTED ACCORDING TO CURRENT M.T.O. STANDARDS.
3. CONSTRUCT CONCRETE FOOTING.
4. PLACE REMAINDER OF GRANULAR 'A' & EARTH FILL AS REQUIRED.

ABUTMENT ON COMPACTED FILL  
SHOWING GRANULAR 'A' CORE  
 N.T.S.

## APPENDIX C



REMOVAL OF UNSUITABLE SOILS  
FROM BENEATH APPROACH FILLS  
N.T.S.

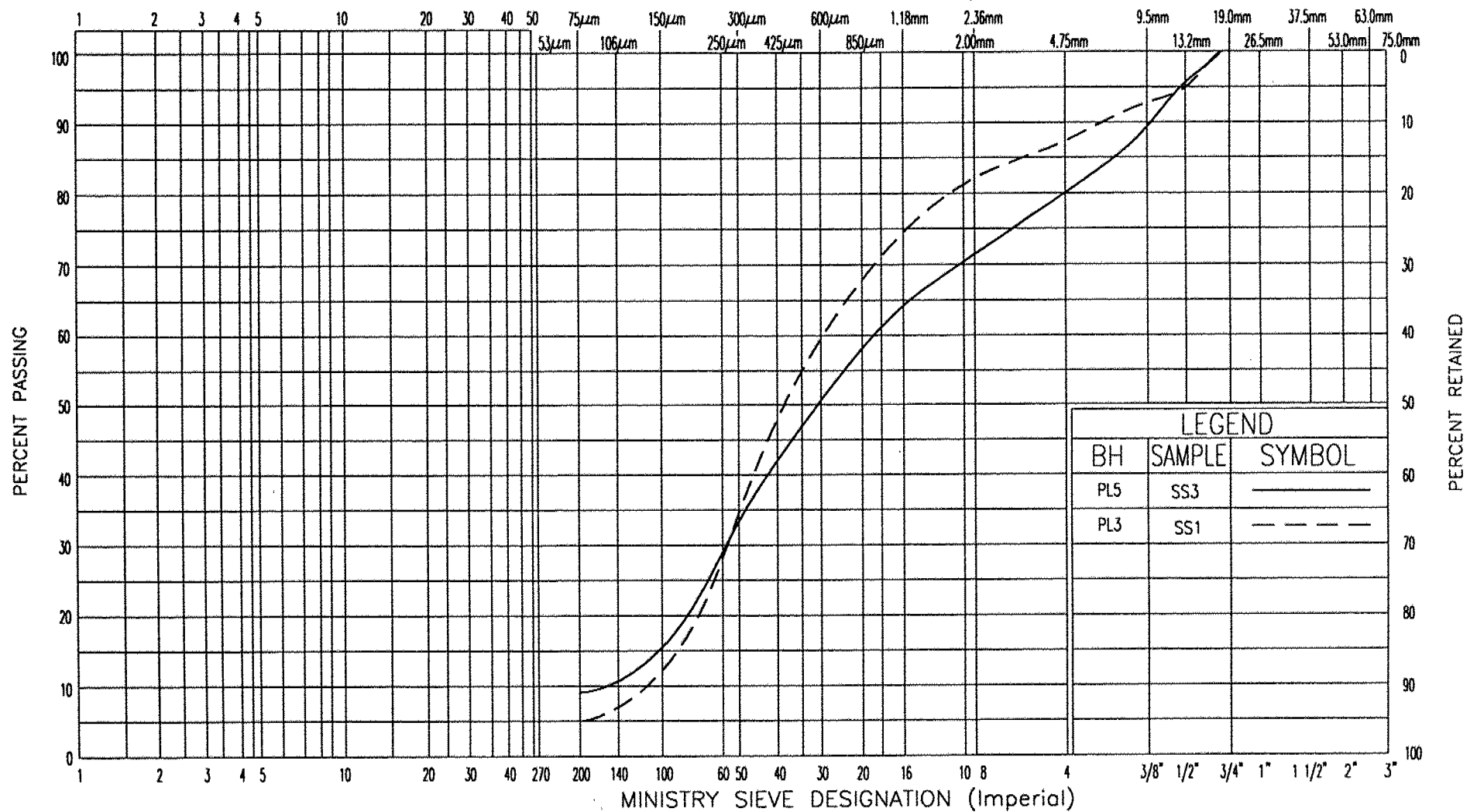
## FIGURES

# UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse

GRAIN SIZE IN MICROMETERS

MINISTRY SIEVE DESIGNATION (Metric)



GRAIN SIZE DISTRIBUTION  
SAND (FILL)

FIG No 1  
W P 473-93-00

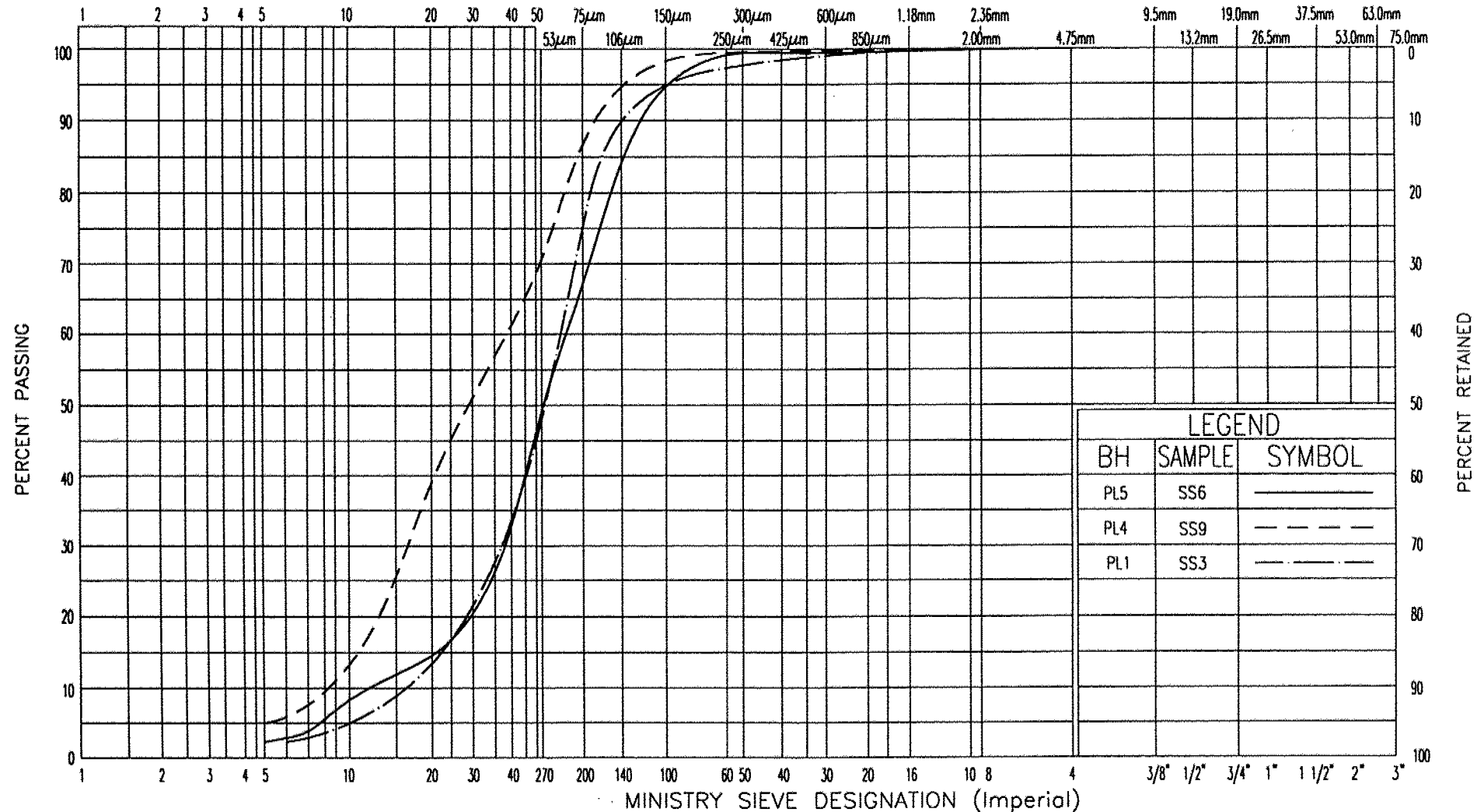


# UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse

GRAIN SIZE IN MICROMETERS

MINISTRY SIEVE DESIGNATION (Metric)



LEGEND		
BH	SAMPLE	SYMBOL
PL5	SS6	—————
PL4	SS9	- - - - -
PL1	SS3	—————



GRAIN SIZE DISTRIBUTION  
SANDY SILT

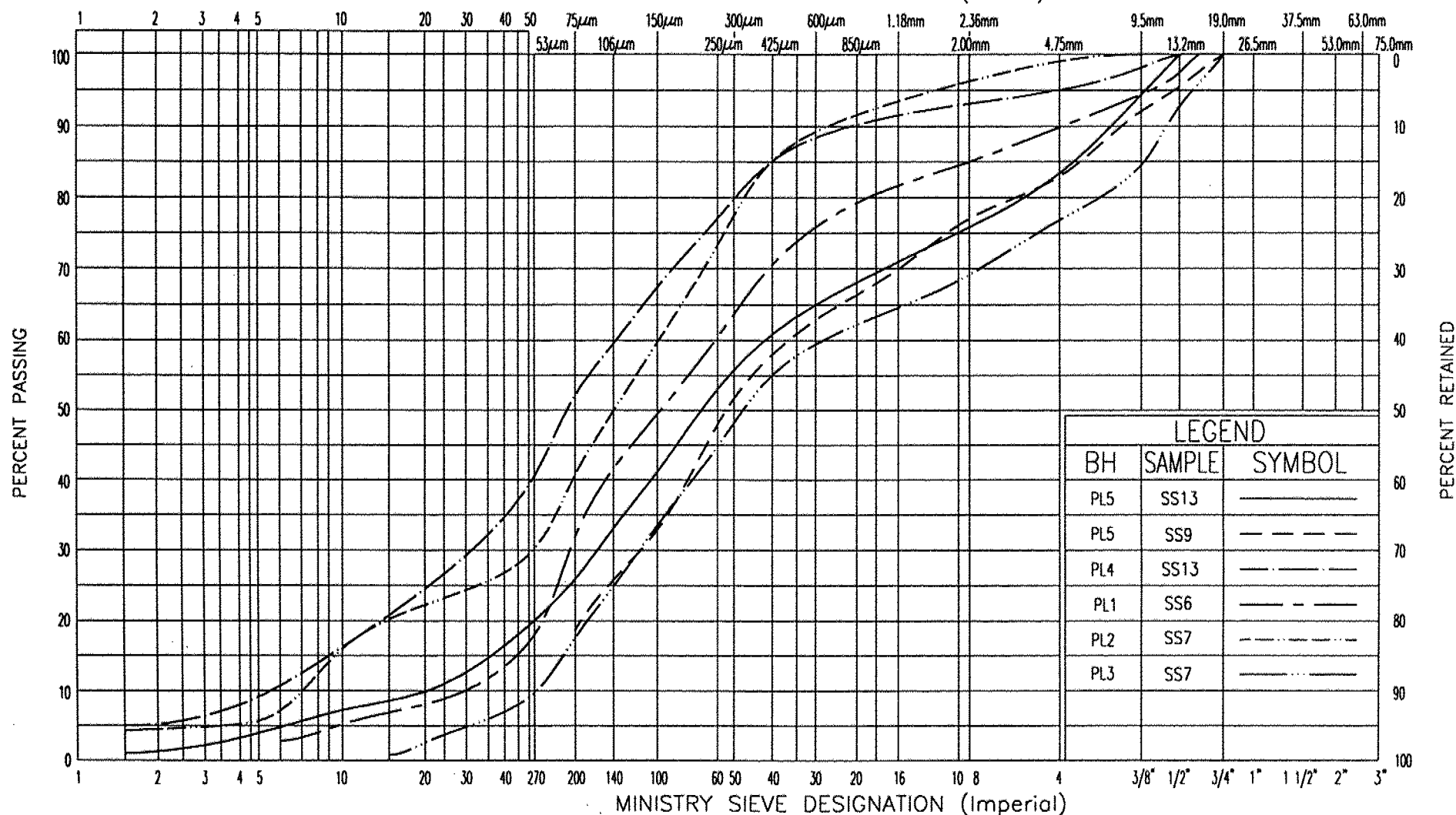
FIG No 2  
W P 473-93-00

# UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse

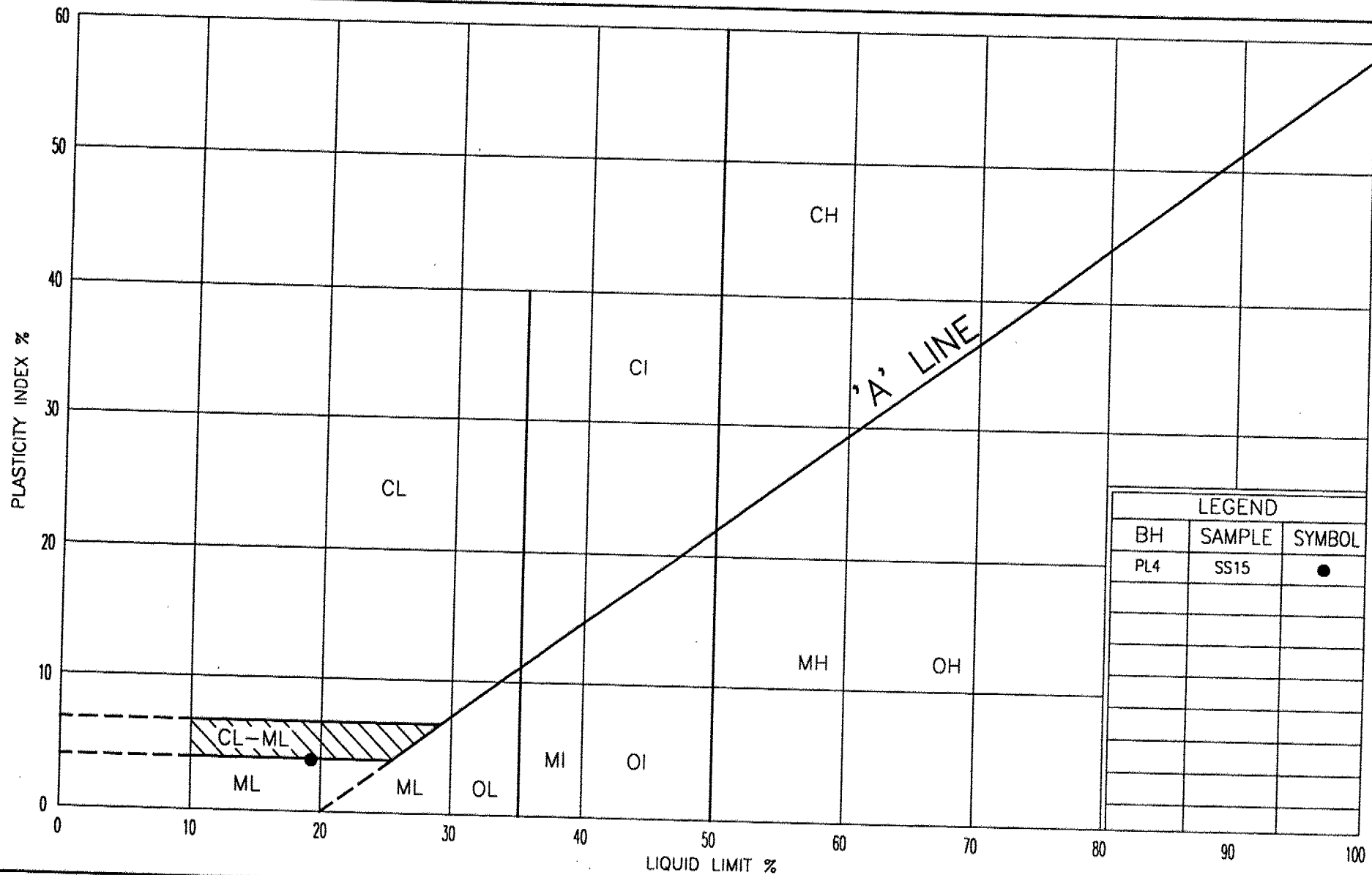
GRAIN SIZE IN MICROMETERS

MINISTRY SIEVE DESIGNATION (Metric)



GRAIN SIZE DISTRIBUTION  
HETEROGENIOUS MIXTURE OF SAND, SILT & GRAVEL  
(GLACIAL TILL)

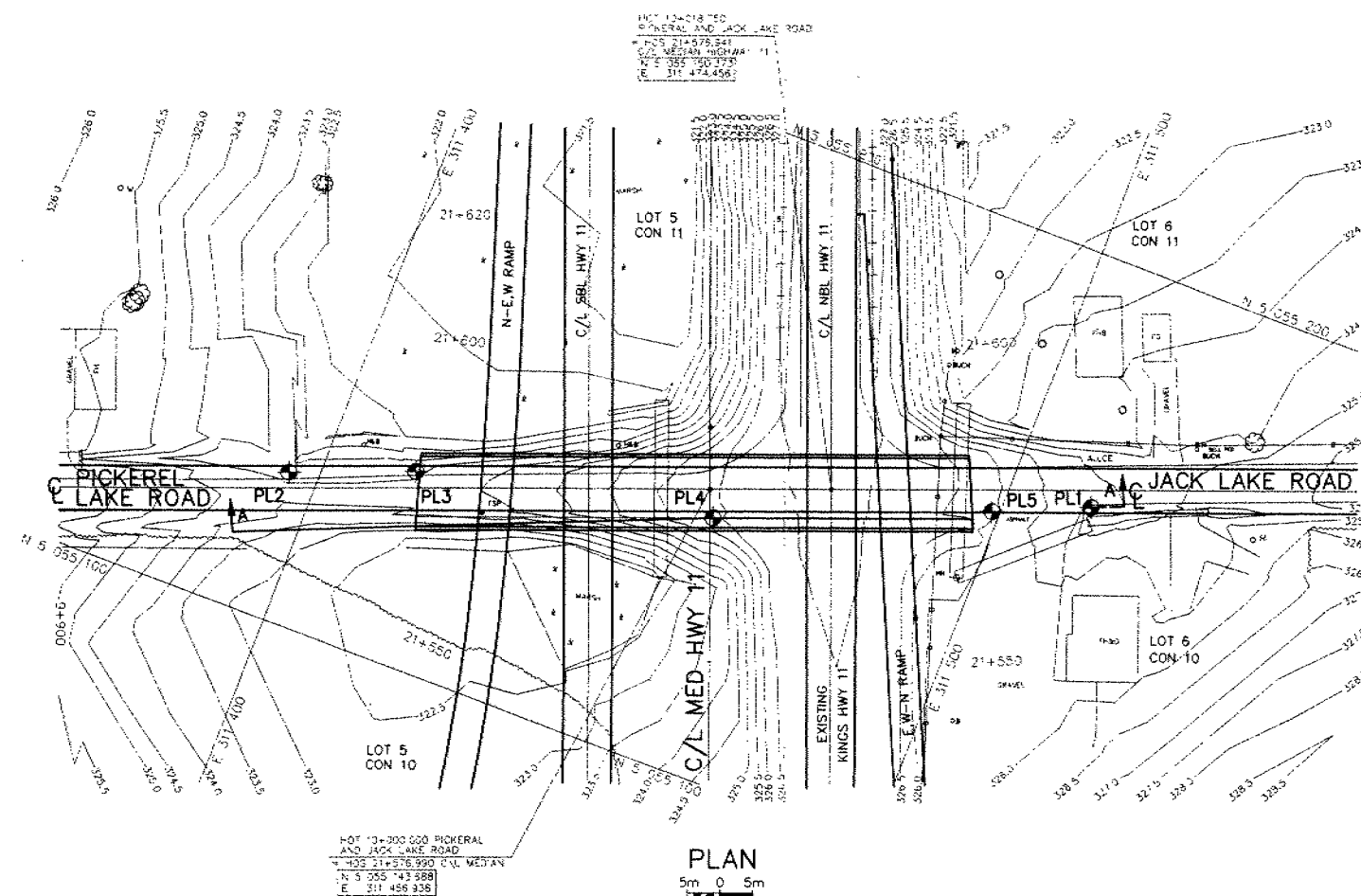
FIG No 3  
W P 473-93-00



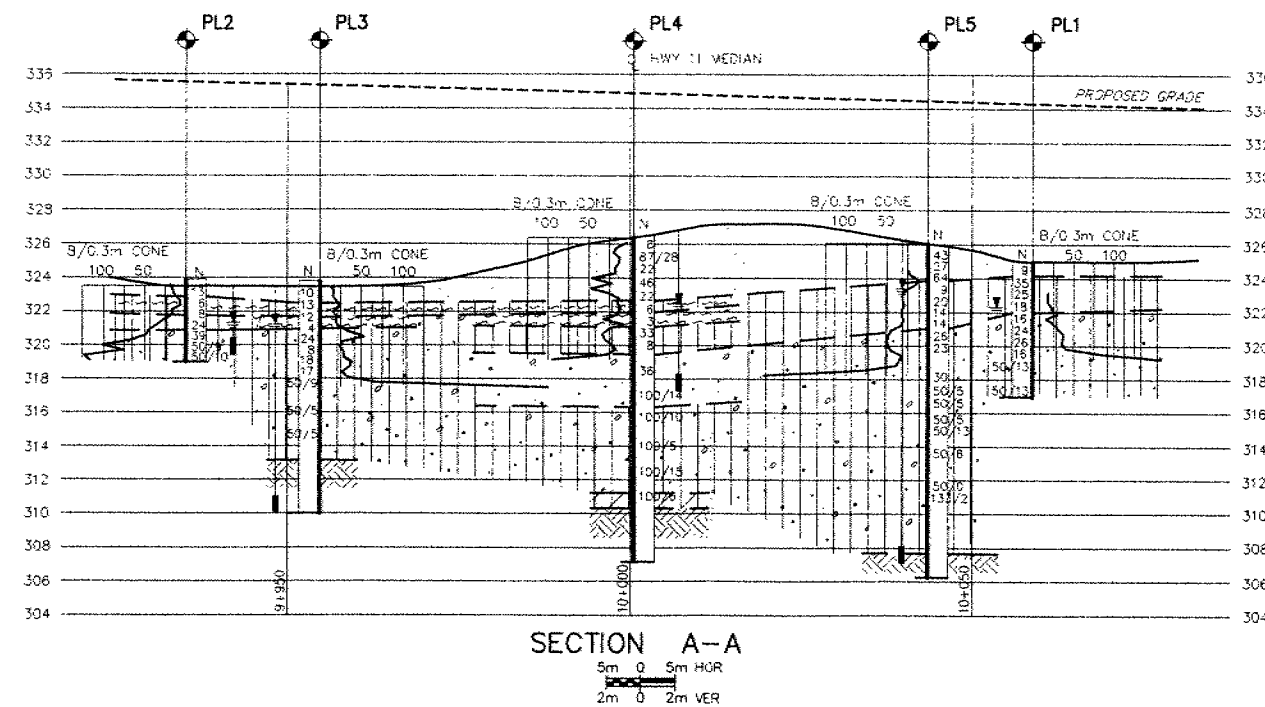
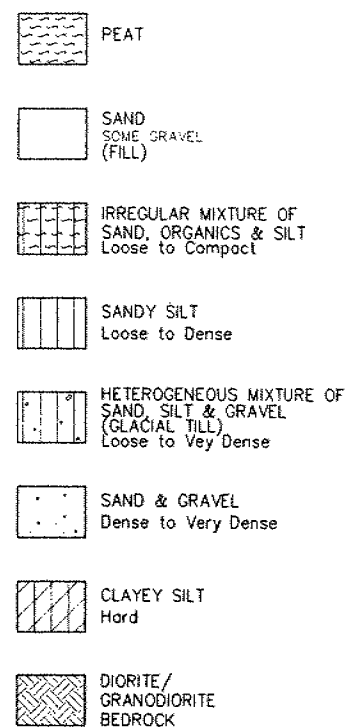
LEGEND		
BH	SAMPLE	SYMBOL
PL4	SS15	●



ENCLOSURES



SOIL STRATIGRAPHY LEGEND



METRIC

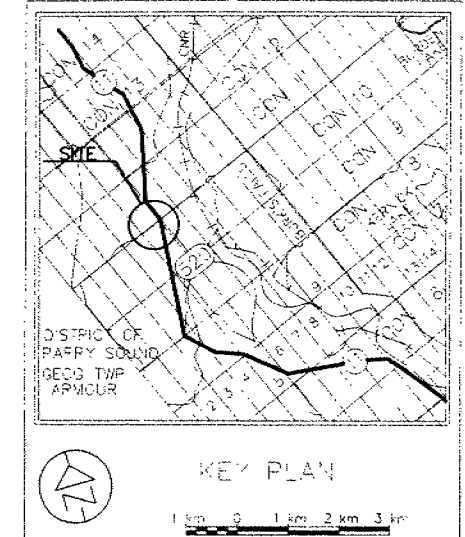
ALL DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES UNLESS  
OTHERWISE SHOWN. STATIONS  
IN KILOMETRES - METRES

CONT. No.  
W.P. No. 473-93-00

PICKEREL & JACK LAKE ROAD UNDERPASS  
BORE HOLE LOCATIONS & SOIL STRATA

**SHEET**

AGRA Earth &amp; Environmental Ltd.



LEGEND

- Bare Hole
- ⊕ Dynamic Cone Penetration Test (Cone)
- ⊕ Bare Hole & Cone
- W Slows/3.3m (Std Pen Test, 475 g/slow)
- CONE Slows/0.3m (60° Cone, 475 g/slow)
- ⊕ WL at time of investigation - May 99
- ⊕ WL in Piezometer
- ⊕ Piezometer

No	ELEVATION	CO-ORDINATES	
		NORTH	EAST
PL1	324.9	5 055 152	311 513
PL2	323.9	5 055 123	311 395
PL3	323.8	5 055 130	311 474
PL4	326.3	5 055 140	319 459
PL5	326.0	5 055 156	311 499

-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. Downstream 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
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REF. Hwy 11 Bridge Site Plan  
Dwg. by MTO, May, 1999

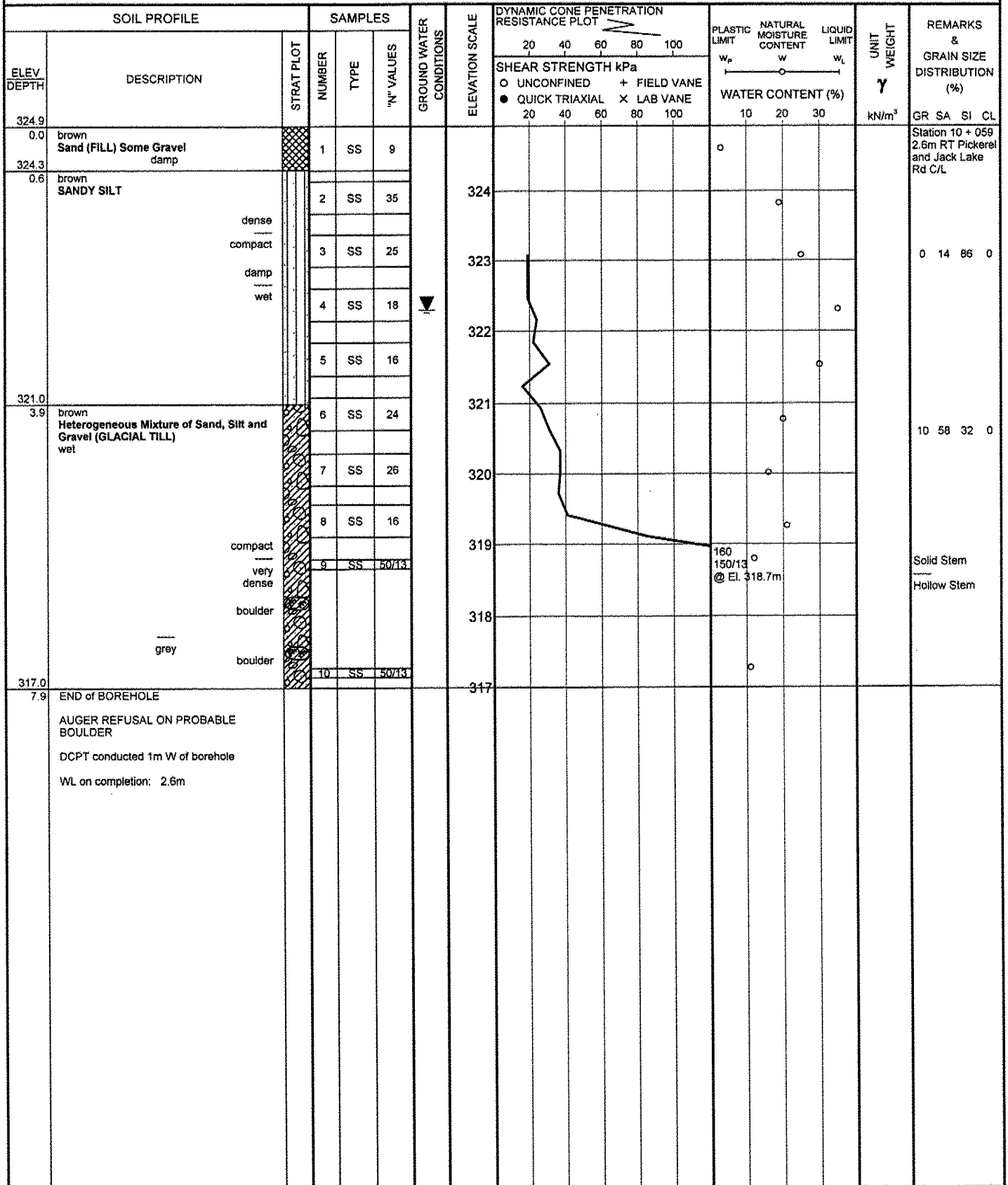
HWY No 11		0:57 PARRY SOUND	
SUBM'D TO	CHECKED AD	DATE July 1999	SFE 44-399
DRAWN MA		CHECKED	DWG 1

RECORD OF BOREHOLE No PL1

1 OF 1

METRIC

W.P. 473-93-00 LOCATION N 5055162.2 E 311512.8 ORIGINATED BY AD  
 DIST 52 HWY 11 BOREHOLE TYPE Solid Stem/Hollow Stem COMPILED BY CK  
 DATUM Geodetic DATE 10.5.99 CHECKED BY ZSO



RECORD OF BOREHOLE No PL2

1 OF 1

METRIC

W.P. 473-93-00 LOCATION N 5055122.9 E 311395.4 ORIGINATED BY AD  
 DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem COMPILED BY CK  
 DATUM Geodetic DATE 10 May 99 CHECKED BY ZSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × LAB VANE						
323.9															
0.0	brown Sand (FILL) Some Gravel damp		1	SS	13									Station 9 + 935 2.5m LT Pickerel and Jack Lake Rd C/L	
323.1	dark grey Irregular Mixture of Sand, Organics and Silt loose, damp		2	SS	6										
0.8			3	SS	8										
321.8	wet														
2.1	brown SANDY SILT compact, wet		4	SS	24										
321.0															
2.9	brown Heterogeneous Mixture of Sand, Silt & Gravel (GLACIAL TILL) dense to very dense, wet		5	SS	39										
	cobbles		6	SS	50/9										
	boulder														
	cobbles														
319.0	grey													2 56 38 4	
4.9	END of BOREHOLE													Auger probe 3m E of borehole Auger refusal at 4.9m.	
	AUGER REFUSAL POSSIBLY ON BOULDER														
	DCPT conducted 1m E of borehole														
	WL in OPEN BORE on completion: 2.4m  WL in PIEZOMETER May 26/99: 0.9m May 27/99: 0.8m														

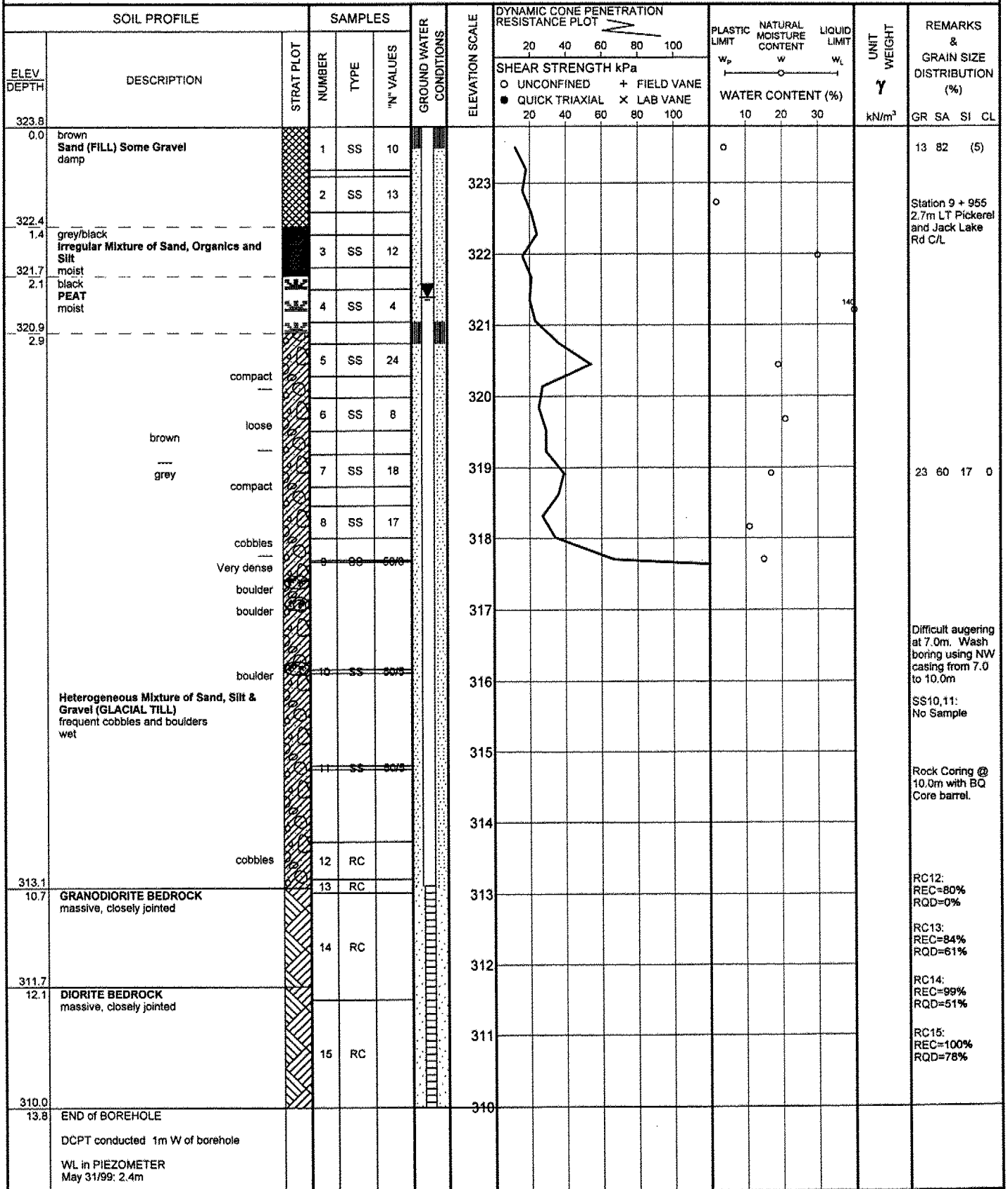


RECORD OF BOREHOLE No PL3

1 OF 1

METRIC

W.P. 473-93-00 LOCATION N 5055130.1 E 311413.7 ORIGINATED BY AD  
 DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem/Washboring COMPILED BY CK  
 DATUM Geodetic DATE 10 May 99 - 11 May 99 CHECKED BY ZSO

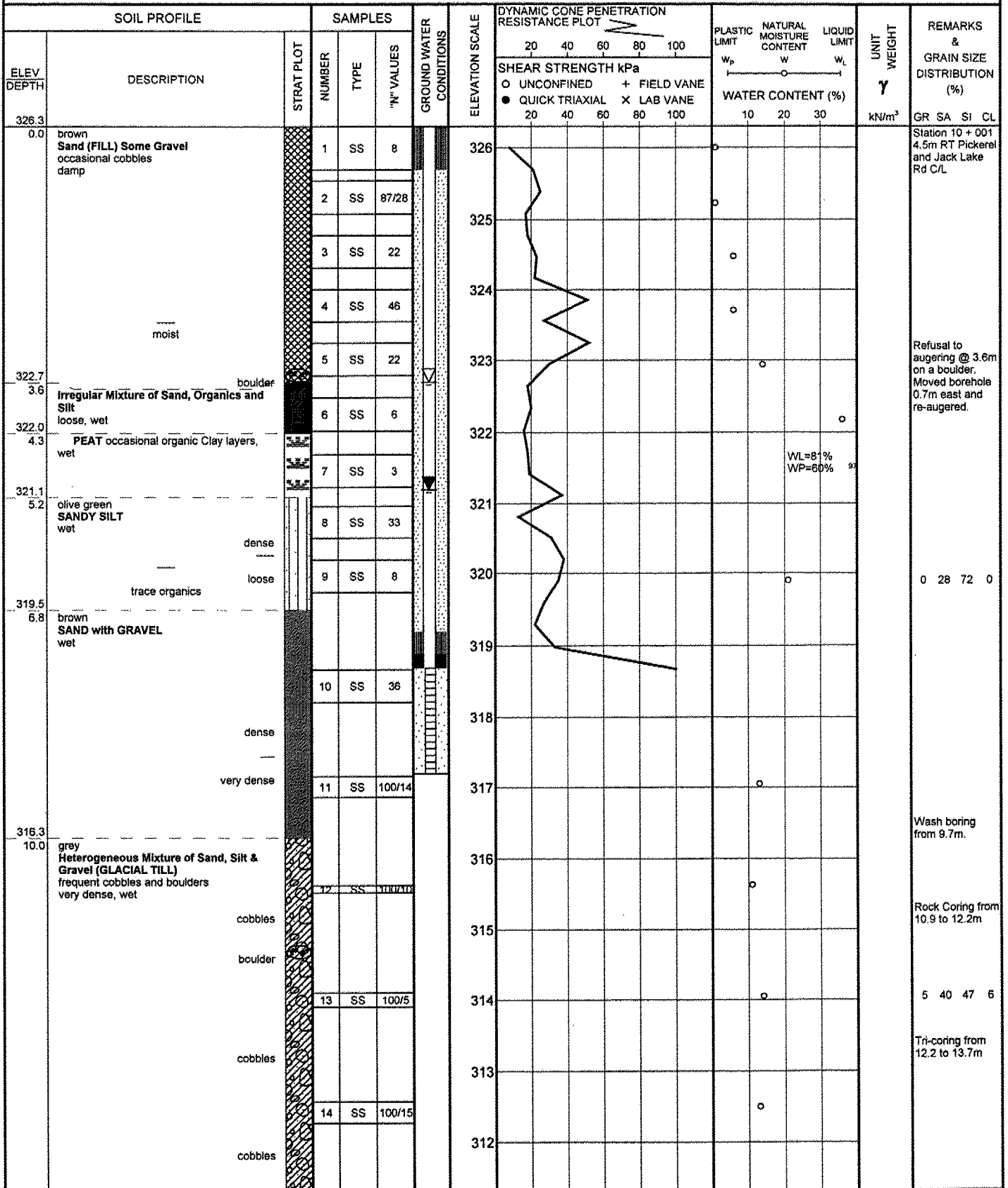


RECORD OF BOREHOLE No PL4

1 OF 2

METRIC

W.P. 473-93-00 LOCATION N 5055139.7 E 311459.0 ORIGINATED BY AD  
 DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem/Washboring COMPILED BY CK  
 DATUM Geodetic DATE 15 May 99 - 16 May 99 CHECKED BY ZSO



Continued Next Page

+ 3, x 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

## METRIC

[illegible]

+3, X3: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No PL5										1 OF 2		METRIC			
W.P. 473-93-00		LOCATION N 5055156.1 E 311498.8		ORIGINATED BY AD											
DIST 52 HWY 11		BOREHOLE TYPE Hollow Stem/Washboring		COMPILED BY CK											
DATUM Geodetic		DATE 11 May 99 - 13 May 99		CHECKED BY ZSO											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
326.0	50mm ASPHALTIC CONCRETE		1	SS	43										GR SA SI CL
0.0	Sand (FILL) Some Gravel damp		2	SS	27										Station 10 + 044 3.4m RT Pickrel and Jack Lake Rd C/L
323.9	grey-brown SANDY SILT loose to compact		3	SS	64										20 71 (9)
2.1			4	SS	9										
			5	SS	20										SS4.5: Strong Gasoline odour
	moist		6	SS	14										0 33 67 0
	wet		7	SS	14										
320.9	brown Heterogeneous Mixture of Sand, Silt & Gravel (GLACIAL TILL) frequent cobbles wet		8	SS	26										
5.1			9	SS	23										16 65 (19)
	cobbles		10	SS	30										
			11	SS	50/5										Auger refusal @ 8.7m. Advance by Wash boring and Rock coring with NW Casing
	grey		12	SS	50/5										
	very dense boulder		13	SS	50/5										22 52 24 2
			14	SS	50/13										
			15	SS	50/8										
	cobbles		16	RC											Advance with BW casing.
	cobbles		17	SS	50/0										
	cobbles		18	RC											

Continued Next Page

+ 3, X 3: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

**METRIC**

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3%</sup> STRAIN AT FAILURE

GEOCRES No. \_\_\_\_\_

DIST. 54 REGION \_\_\_\_\_W.P. No. 473-93-00

CONT. No. \_\_\_\_\_

W. O. No. \_\_\_\_\_

STR. SITE No. \_\_\_\_\_

HWY. No. 11LOCATION Proposed TCPL CrossingNo of PAGES -OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS: \_\_\_\_\_

\_\_\_\_\_

\_\_\_\_\_

\_\_\_\_\_

**DRAFT**

**FOUNDATION INVESTIGATION AND DESIGN REPORT  
FOR  
PROPOSED TCPL CROSSING, NBL  
DISTRICT 52, HUNTSVILLE  
W.P. 473-93-00**

**Submitted To:**

**Delcan Corporation  
133 Wynford Drive  
North York, Ontario, M3C 1K1  
Canada**

**Submitted By:**

**AGRA  
104 Crockford Blvd.  
Scarborough, Ontario, M1R 3C6  
Canada**

**August 1999  
TT98820S**

DRAFT

**AGRA Earth &  
Environmental Limited**  
104 Crockford Blvd.  
Scarborough, Ontario  
Canada M1R 3C6  
Tel (416) 751-6565  
Fax (416) 751-7592

August 4, 1999.  
**Ref. No.: TT98820S**

Delcan Corporation  
133 Wynford Drive  
North York, Ontario, M3C 1K1  
Canada

**Attention: Mr. Khaled El-Dalati, P, Eng.**

Dear Sir:

**Re: FOUNDATION INVESTIGATION AND DESIGN REPORT  
FOR  
PROPOSED TCPL CROSSING, NBL  
DISTRICT 52, HUNTSVILLE  
W.P. 473-93-00**

We take pleasure in enclosing six (6) Draft copies of our Geotechnical Investigation and Design Report carried out for the above mentioned project and we will be glad to discuss any questions arising from this work.

Soil samples will be retained for a period of three months, and will thereafter be disposed of unless we are otherwise instructed.

We thank you for giving us this opportunity to be of service to you.

Sincerely,



Z.S. Ozden, P. Eng.,  
Principal Engineer.

ZSO/dee



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## 1.0 INTRODUCTION

AGRA Consulting Geotechnical Engineers has been retained by DELCAN Corporation (DELCAN) to carry out a foundation investigation at the site of a proposed bridge which will carry the proposed northbound lane (NBL) of Highway 11 over two existing buried TransCanada PipeLines (TCPL) gas pipelines. The site is located at some 15 km south of Katrine, just east of the existing Highway 11 crossing of the pipelines, and is part of the Highway 11 Four Laning project from 0.7 km north of Highway 592N at Katrine, northerly 12.4 km (W.P. 473-93-00). This report addresses the foundation aspects of the proposed bridge and its approaches within 20 m of the structure. The single span bridge will have a span length of about 34 m and a width of about 13 m.

The purpose of this investigation is to determine the subsurface conditions at the site of the proposed bridge structure by means of a limited number of boreholes, in-situ tests and laboratory tests on selected samples. Based on our interpretation of the data obtained, recommendations are provided on the geotechnical aspects of foundation design. Comments are also provided on anticipated construction issues where they may affect the design of the proposed bridge and approach embankments from a geotechnical point of view.

## 2.0 SITE DESCRIPTION AND PHYSIOGRAPHY

The proposed bridge crossing of the TCPL pipelines is located approximately 15 km south of Katrine within MTO District 52, Huntsville, Ontario. The bridge will be located adjacent to the south slope of a ravine valley, some 50 m to 60 m to the southwest of the existing Beaver Dam. The ground surface varies from about Elevation 313 m at the south ravine slope crest to about Elevation 299 m to the north where the approach embankment will be up to 11 m in height. Vegetation cover on the south ravine slope, near the proposed south abutment location, consists of predominantly grass with some shrubs and small trees.

Based on available geologic information, the site is situated within an area of ice-contact sediments. In general, after the last glacial withdrawal, ice-contact sediments of sands and gravel, followed by glaciofluvial sediments of deltaic and nearshore sands and gravel, as well as lake bottoms silts and clays, were deposited on top of the existing sandy glacial till or Precambrian bedrock. The area was then inundated by glacial Lake Algonquin, depositing sands, silts and clays in low lying areas, such as at the proposed bridge site which is located within a ravine valley.

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### 3.0 INVESTIGATION PROCEDURES

The field work for this investigation was carried out on June 4 and 5, 1999 and between June 9 and 13, 1999, during which time four boreholes (Borehole Nos. TCPL1 to 4) were drilled and sampled. One borehole was put down at each of the proposed bridge abutments, and one borehole was put down at a location some 20 m from each of the abutments. Dynamic cone penetration testing (DCPT) was attempted at the bottom of Borehole TCPL1 which was located close to the proposed north abutment. The plan locations of the boreholes and selected stratigraphic sections are shown on Drawing No. 1. ✓

The investigation was carried out using a track-mounted power auger drill rig (BOA 6M) owned and operated by Groundworks Drilling Inc., under the full-time supervision of a member of AGRA's engineering staff. Hollow stem and solid stem augers were used to advance the boreholes within shallower depths. At greater depths, within the water-bearing sands and silts, wash-boring techniques, using tri-cones and/or casings, were required to further advance the boreholes. Rotary core drilling techniques were utilized to penetrate through cobbles and boulders encountered near the bottom of the two deep boreholes. ✓

In the boreholes, soil samples were obtained at regular intervals of depth using 50 mm outside diameter split barrel (split spoon) samplers in accordance with Standard Penetration Test (SPT) procedures, as specified by ASTM Standard D1586. The SPT consists of freely dropping a 63.5 kg hammer for a vertical distance of 0.76 m to drive the split spoon sampler into the ground. The number of blows of the hammer required to drive the sampler into the relatively undisturbed ground, for a vertical distance of 0.30 m, is recorded as the Standard Penetration Resistance or the 'N'-value of the soil. This value gives an indication of the relative state of compactness of cohesionless soils and the consistency of cohesive soils. Where the consistency of the soil permitted, in-situ vane shear tests using a standard MTO 'N'-size vane were attempted at regular intervals within the silty clay deposit. Thin walled Shelby tube samples were also obtained at selected locations within this deposit. ✓

A dynamic cone penetration test was performed at the bottom of Borehole TCPL1. This test consists of continuously driving a 60° point, 50 mm diameter cone attached to the drill rod, into the undisturbed ground with a driving energy of 475 kJ (63.5 kg hammer free falling for a distance of 76 cm) per blow. The number of blows for each 30 cm of penetration is recorded, providing an indication of the relative changes in the soil density with depth. ✓

Due to the presence of boulders and cobbles near the bottom of Boreholes TCPL1 and 3, rotary core drilling techniques were utilized to further advance the boreholes. Coring was carried out using a NXL size core barrel in conjunction with NW size casings. Relocate

Groundwater conditions in the open boreholes were observed throughout and immediately after the drilling operations. Standpipe piezometers were installed in Boreholes TCPL1, 2 and 3 to permit long term monitoring of groundwater levels. Two piezometers were sealed in different soil strata in each of Boreholes TCPL1 and 3. Borehole TCPL4 was grouted on completion of the field

work.

The drilling, sampling and in-situ testing operations were carried out under the full-time supervision of members of our engineering staff, who examined the samples and logged the boreholes. The soil samples were identified, placed in containers and transported back to our geotechnical laboratory in Toronto (Scarborough) for further examination and testing. Index and classification tests, including natural moisture contents, grain size distribution analysis and Atterberg limits tests, were carried out on selected representative soil samples. Laboratory oedometer tests, quick triaxial (UU) and consolidated, undrained triaxial compression with pore water pressure measurement (CU) tests were also carried out on selected silty clay samples to determine strength and deformation characteristics. The results of the laboratory tests are presented on the appropriate Record of Borehole sheets, on Figure Nos. 1 to 7, and in the text.

The drilling locations were initially established in the field by our field personnel based on the centreline of Highway 11 staked out by Dearden and Stanton Limited. The as-drilled borehole locations in terms of northing and easting co-ordinates, and elevations were surveyed by Dearden and Stanton Limited. We understand that these elevations are referenced to Geodetic datum. The locations and co-ordinates of the boreholes are shown on Drawing No. 1; the co-ordinates and elevations are indicated on the Record of Borehole sheets.

#### 4.0 SUBSURFACE CONDITIONS

The subsurface conditions were explored at four borehole locations (Borehole Nos. TCPL1 to TCPL4), and were inferred near the bottom of Borehole TCPL1 by a dynamic cone penetration test. The locations of the boreholes and the cone penetration test are shown on the Plan and Profile Drawing No. 1 and are also indicated on the Record of Borehole sheets. A cross-section of inferred subsurface stratigraphy is shown on Drawing No. 1.

The ground surface drops from about Elevation 313 m at the south ravine slope crest, some 30 m south of the proposed south abutment, to about Elevation 299 m at the lowest point of the ravine valley, some 40 m north of the proposed north abutment.

In general, the subsoils consist of a thin veneer of topsoil overlying surficial, loose to compact sand, some gravel, to silt, which are underlain by a stratum of typically stiff silty clay. Below the silty clay are extensive deposits of cohesionless soils with increasing grain sizes from silt, just below the clay, to sand with some gravel at depth. The relative state of compactness also increases with depth, ranging from loose to very dense. Frequent cobbles and/or boulders were inferred or encountered within the lower, very dense sand. Two groundwater levels were identified at this site; a perched water level at shallow depth above the silty clay and a lower water level associated with the underlying silts and sands.

Details of the subsurface conditions encountered in the boreholes are presented on the Record of Borehole sheets. The following paragraphs are only intended to complement and summarize these data.

*Do we need this stuff?*

#### 4.1 TOPSOIL

Topsoil was encountered at all of the boreholes, ranging in thickness from 0.1 m to 0.2 m.

It should be noted that the thickness of topsoil may vary in between and beyond the borehole locations.

✓

#### 4.2 SURFICIAL SAND TO SILT

In all four boreholes below a veneer of topsoil, surficial sand, some gravel, to silt deposits were encountered below the topsoil to depths of between 2.2 m and over 6.5 m. The thickness of these coarse to fine grained granular surficial deposits generally increases from north of the bridge, near the lowest elevation within the ravine valley, to south of the bridge, near the crest of the south ravine slope. These deposits were not fully penetrated by Borehole TCPL4. These soils contain rootlets and occasional decomposed organics in Borehole TCPL2. Measured SPT 'N'-values within this deposit generally increase with depth, ranging from 2 to 25 blows per 0.3 m penetration, indicating a very loose to compact state. Measured natural moisture contents range from 5 to 22%. Six grain size distribution analyses were carried out on samples of these soils with the following results:

*colour*

Gravel:	0-2%
Sand:	10-80%
Silt:	20-90%
Clay:	0%

The grain size distribution curves are shown on Figure No. 1.

#### 4.3 SILTY CLAY

*clayey silt to silty clay*

A stratum of silty clay was encountered at between about Elevations 300 m and 292 m in all but Borehole TCPL4 (terminated at Elevation 306.6 m). The thickness of the deposit ranges from 8.6 m in Borehole TCPL3 at the south abutment, to 4.2 m in Borehole TCPL2 at the north approach. This silty clay has occasional varved-like clayey silts and occasional sand seams. The thickness of these generally range from about 5 mm to 10 mm. Measured SPT 'N'-values within the silty clay range from 3 blows to 11 blows per 0.3 m penetration. Field vane tests gave in-situ undrained shear strengths typically 110 kPa or greater (vane could not be turned), but occasional values as low as 55 kPa were measured. These results indicate that the silty clay has a typically stiff to very stiff consistency.

Atterberg limits tests were carried out on representative samples of the silty clay and yielded the following results:

Liquid Limit:	30-53% (average 38%)
Plastic Limit:	19-28% (average 21%)
Plasticity Index:	9-19% (average 17%)

These values indicate that the soil may be classified as a clay of low to medium plasticity (group symbol CL - CI) as shown on Figure No. 7. Measured natural moisture contents vary from 29 to 52% throughout this deposit and, as such, they exceed or are close to the measured liquid limit values, where tested.

Three grain size distribution analyses were conducted on representative samples from this stratum, giving the following grain size measurements.

Gravel:	0%
Sand:	1-17%
Silt:	58-73%
Clay:	25-36%

The grain size analyses results are presented on Figure No. 2.

Three oedometer tests were carried out on relatively undisturbed Shelby tube samples of the silty clay recovered from Boreholes TCPL1 to TCPL3. Results of the tests indicate that the sample has the following properties: an  $m_v$  of  $2 \times 10^{-4} \text{ m}^2/\text{kN}$  within the anticipated field stress range under embankment loading, preconsolidation pressure  $\sigma'_p$  ranging from 140 kPa to 270 kPa, compression index  $C_c$  ranging from 0.22 to 0.59, and recompression index  $C_r$  ranging from 0.05 to 0.09.

Three isotropically consolidated, undrained triaxial (CU) tests and one unconsolidated, undrained (UU) test, were carried out on samples of the silty clay obtained from the Shelby tubes. Measured undrained shear strengths range between 50 kPa and 100 kPa.

#### 4.4 SILT TO SANDY SILT

Immediately underlying the silty clay, at about Elevation 293 to 292 m (or below depths of about 7 to 15 m below the ground surface), is a deposit of non-plastic silt with traces to some sand and occasional fine sand and thin clay seams. This silt grades to a sandy silt with depth at Boreholes TCPL1 and TCPL2. These deposits are fine grained granular materials. The thickness of this deposit is about 13 m at Borehole TCPL3 and about 11 m at Borehole TCPL1. This deposit is generally in a loose to compact state as indicated by SPT 'N'-values of between 5 blows and 16 blows per 0.3 m penetration, except at between Elevations 285 m and 278 m in Borehole TCPL3 where it is in a dense state ('N'-values of 32 blows to 47 blows). Measured natural moisture contents range from 20 to 30%.

Five grain size distribution analyses were carried out on samples of the silt with the following results:

Gravel:	0%
Sand:	1-13%
Silt and Clay:	87-99%

The grain size distribution curves for the silt is shown on Figure No. 3. Two grain size curves of silt layers within the underlying sand and silt are also included in this figure.

Two grain size distribution analysis were carried out on samples of sandy silt and both yielded the following results:

Gravel:	0%
Sand:	25%
Silt and Clay:	75%

The grain size distribution curves for the sandy silt is shown on Figure No. 4.

#### 4.5 SAND AND SILT

An extensive deposit of sand and silt, with some silt layers, was encountered between Elevations 282 and 259 m (23 m thick) at Borehole TCPL1 and between Elevations 278 and 265 m (13 m thick) at Borehole TCPL3. Standard Penetration tests conducted in this deposit in Borehole TCPL3 and below Elevation 276 m in Borehole TCPL1 gave 'N'-values generally ranging between 32 and 84 blows per 0.3 m, with occasional lower values (e.g. 16 and 19 blows per 0.3 m), indicating a generally dense to very dense condition with occasional compact conditions, probably associated with the more silty zones. In Borehole TPCL1, within the upper 4 m (i.e. between Elevation 282 and 278 m), the recorded 'N'-values are 5 to 6 blows per 0.3 m, indicating a loose to very loose relative density.

The measured natural moisture contents of samples recovered from the deposit range from 19 to 26%.

Three grain size distribution analyses were carried out on samples of the sand and silt, and yielded the following results:

Gravel:	0%
Sand:	47-57%
Silt:	43-53%

The grain size distribution envelope for the sand and silt is shown on Figure No. 5. Grain size distribution curves for samples of the silt zones are shown on Figure No. 3.

#### 4.6 SAND (some gravel, cobbles/boulders)

The sand and silt deposit is underlain by a sand deposit, with gravel and traces to some silt and clay, at Elevation 265 m at Borehole TCPL3, and at Elevation 259 m at Borehole TCPL1. Neither borehole fully penetrated this deposit, but rather they were terminated at Elevations 256.4 m and 254.6 m, after penetrating the deposit for about 5 and 8 m in Boreholes TCPL 1 and 3, respectively. This deposit is in a very dense state as indicated by SPT 'N'-values in excess of 50 blows per 0.3 m penetration throughout. Refusal to split spoon sampler and tri-cone advance were frequently encountered (indicating the possible presence of cobbles and boulders), and rotary core drilling was used to advance the boreholes through cobbles and boulders at several locations within the deposit. A dynamic cone penetration test was carried out near the bottom of Borehole TCPL1, and refusal was met at Elevation 254.6 m after a penetration of only 120 mm. Measured natural moisture contents range from 16 to 23%.

Three grain size distribution analyses were carried out on samples from the sand with gravel matrix and yielded the following results:

Gravel:	7-17%
Sand:	72-79%
Silt & Clay:	9-14%

The grain size distribution curves are shown on Figure No. 6.

#### 4.7 GROUNDWATER CONDITIONS

Groundwater levels in the open boreholes were observed during drilling and upon completion of each borehole. To permit long term monitoring of water levels at the site, standpipe piezometers were installed in Boreholes TCPL1, TCPL2 and TCPL3 (in Boreholes TCPL 1 and 3, two standpipe piezometers were installed in each borehole). The recorded values are shown on the individual Record of Borehole sheets.

The measured water level readings indicate that there is an upper, perched water level associated with the surficial sands and silts above the silty clay stratum. This upper water level, which appeared to be varying with the ground surface, was at the time of our investigation at Elevation 305 m in Borehole TCPL3 and at Elevation 301 m in Borehole TCPL1. These elevations correspond to about 1 m to 2 m below ground surface. The lower water level, which is associated with the sands and silts below the silty clay, was measured in all three boreholes (i.e. TCPL 1, 2 and 3) at about Elevation 296 m. This indicates a slight excess hydrostatic pressure in the basically granular soils underlying the silty clay.

It should, however, be pointed out that the groundwater at the site would fluctuate seasonally and can be expected to be somewhat higher during the spring months and in response to major weather events.



## 5.0 DISCUSSION AND RECOMMENDATIONS

The proposed Highway 11 realignment will consist of a four lane divided roadway with an approximately 30 m wide median. The work described in this report is associated with the proposed bridge to carry the proposed northbound lane (NBL) of Highway 11 over two existing TCPL pipelines, and the approach embankments within 20 m of the structure. The existing Highway 11 will become the southbound lane (SBL). It is understood that the proposed structure which will carry the new NBL, is a single span, two-lane (13 m wide) bridge, approximately 34 m in length.

The TCPL pipelines run in an approximately southwest-northeast orientation along the south slope of the ravine valley, which has an overall inclination of approximately 5 horizontal to 1 vertical. The north valley slope has a lesser inclination of approximately 10 horizontal to 1 vertical. The proposed horizontal and vertical alignment for Highway 11 and the location of the bridge was provided to us in plan and profile drawings by Delcan Corporation. Based on the drawings provided, the bridge will be located on the existing south valley slope and the existing grade at the proposed bridge location (centreline) is about Elevation 307.5 m at the south abutment location, dropping to about Elevation 302 m at the north abutment location. The proposed grade along the bridge alignment lies between approximately Elevations 311.5 m (on the south side) and 310.5 m (on the north side). The approach embankment at the south abutment location will therefore be about 4 m high whereas the approach embankment at the north abutment location will be about 9 m high. About 40 m north of the north abutment, the grade raise of the embankment will be up to 11 m high near Station 10+530.

relocate  
to  
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descriptive

Not  
abuse

To  
investigate  
requirements

In general, the subsoils at the bridge site typically consists of 2 m to 6 m of loose to compact sands and silts overlying a 5 m to 9 m thick stratum of silty clay of generally stiff consistency. Underlying the silty clay are extensive deposits of loose to compact silts and sands. These deposits become progressively coarser grained and denser with depth. Below about 22 m to 26 m depth, the sands and silts are typically dense to very dense. Below about 42 m (i.e. below Elevations 265 and 259 m), the sand deposit, which contains some gravel and frequent cobbles and boulders, is in a very dense state. Bedrock was not encountered in any of the boreholes put down at this site.

Piezometer readings at the boreholes indicate that there is an upper perched water level above the silty clay, and a lower water level associated with the underlying sands and silts. The perched water level was at the time of the investigation recorded at about Elevation 305 m at the south abutment location and about Elevation 301 m at the north abutment location; these readings correspond to 1 m to 2 m depth below existing ground surface. The lower water level was recorded at about Elevation 296 m.

## 5.1 FOUNDATIONS

At the south abutment, the surficial loose to compact sands and silts extend to 6 m depth and the groundwater level is at 2 m depth below ground surface. As such, shallow spread footings resting on native soils are not feasible for foundation support. At the north abutment, the underlying silty clay at 2 m depth, though stiff, would be subjected to long term consolidation settlement under embankment loading, and is therefore not suitable for foundation support. ✓

Alternatively, if perched abutments are considered in the design, spread footings founded on a compacted Granular 'A' core may be feasible. The consolidation settlement would be somewhat mitigated, but will still likely exceed the normally accepted value of 25 mm and, as such, the use of spread footing foundations in this instance too is not recommended. We will, however, be pleased to consider this option further if requested.

Timber friction piles embedded within the silty clay stratum would have a relatively low load carrying capacity and, therefore, a large number of piles would have to be used. We do not recommend this option, but we will be pleased to consider it further, if requested.

In view of the above and since an integral abutment bridge is the preferred option, consideration should be given to supporting the bridge on deep foundations in the form of steel H-piles, driven to practical refusal within the upper portion of the very dense sand, with gravel and frequent cobbles and boulders. In order to adequately penetrate the typically dense sands and silts immediately above the founding stratum, a heavier section such as HP310x110 with reinforced tips would be suitable for use.

It is considered likely that the driven H-piles will not be able to penetrate deep into the very dense sand with gravel and frequent cobbles and boulders. Based on the results of the boreholes, the following Table 1 summarizes the estimated average pile tip elevations that may be assumed for design purposes.

TABLE 1

SUPPORT LOCATION	REFERENCE BOREHOLE	ESTIMATED APPROXIMATE PILE TIP ELEVATION (m)
South Abutment	TCPL3	264±
North Abutment	TCPL1	258±

 ✓

The borehole results indicate that the founding sand deposit containing cobbles and/or boulders was encountered at higher elevation at the south abutment location compared with the north abutment location. Therefore, it may be expected that the piles will terminate at higher elevations at the south abutment.

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### 5.1.1 Resistance to Axial Loads

For HP310x110 steel H-piles driven to practical refusal within the very dense sand at or below the elevations shown in Table 1 above, the following axial resistances may be assumed for design.

Factored Axial Resistance at Ultimate Limit States (U.L.S.)	=	1,650 kN
Geotechnical Resistance at Serviceability Limit States (S.L.S.)	=	1,150 kN

*Axial*

The above values were selected in view of the fact that some premature pile refusals may be encountered at elevations higher than those shown in Table 1.

Negative skin friction can be induced on the piles due to consolidation settlement of the silty clay, and to some extent compression of the loose to compact sands and silts, beneath the approach embankments. As a result, down-drag forces would act on the pile, thus reducing its usable end-bearing capacity. The magnitude of the negative skin friction depends on many factors such as the relative movement between the surrounding compressible soils and the pile shaft, the elastic compression of the pile under the working load, the rate of consolidation of the silty clay, as well as the construction sequence and methodology of the piles and the embankment. At this site, it is considered that the induced settlements of the underlying compressible soils due to newly placed fill can result in substantial mobilization of negative skin friction on the piles. The effects of negative skin friction would be much more pronounced at the north abutment due to significantly higher embankment loading.

In order to minimize the effect of down-drag loads, we recommend that the approach fills be placed to their final grade elevation for at least one month prior to driving the piles. By preloading for a period of one month, it is anticipated that elastic settlement of the sands and silts would largely have been completed. By preloading for a few months, a significant portion of the consolidation settlement of the silty clay would have also taken place. For this reason the embankment fills should be placed as early as possible prior to driving the piles.

Assuming that the embankment fills will be placed to their final grade elevation (i.e. top of base course) at least one month prior to the start of pile driving and based on the results of Borehole TCPL1, the down-drag load acting on a HP310x110 steel H-pile is estimated at about 440 kN at the north abutment location. A suitable load factor should be applied to this as per Ontario Highway Bridge Design Code (O.H.B.D.C.), 3<sup>rd</sup> Edition. For the south abutment, where the height of the fill is less, the estimated unfactored down-drag load is 200 kN.

The piles should be driven with a suitably heavy hammer capable of delivering a rated capacity of at least 50 kJ per blow. The energy should, however, be restricted to not more than 60 kJ per blow. The driving of the piles should be controlled by a recognized pile driving formula, such as the Hiley Formula. The estimated ultimate resistance of the piles driven to practical refusal within the very dense sand at about the elevations quoted above, as given by the Hiley Formula, is approximately 3,300 kN. This value was arrived at by dividing the factored axial resistance at U.L.S. by a resistance factor of 0.5, as per current MTO practice.

.../...

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Cobbles and/or boulders were inferred or encountered within the very dense sand in the boreholes drilled at the abutment locations. In view of this and the hard driving conditions anticipated, the pile tips should be reinforced, as per MTO Standards (OPSD 3301.00) to minimize damage to the piles.

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

In accordance with MTO standard practice, the piles should be driven to about 2 to 3 m above the design elevations given in Table 1 and the driving should then be monitored and controlled by the Hiley Formula. If the driven pile encounters refusal above the recommended elevation, the engineer should be notified immediately.

During the driving process, piles which have already been driven should be monitored to determine if they are heaving due to effects of driving adjacent piles. If this phenomenon occurs, the affected piles should be re-driven. It is recommended that not less than 15% of the piles and at least three piles in each foundation support element be re-struck one to two days after initial installation, as a precaution against relaxation. If relaxation occurs, then all piles in that foundation element should be re-tapped.

use  
OPSD  
(see H-30  
and  
spec)

It is possible that some of the piles may penetrate to one to two meters below the estimated tip elevations and this aspect should be taken into consideration when ordering piles.

The geotechnical resistance at Serviceability Limit States (S.L.S.) is dependent on the settlement of the pile group and, therefore, is governed by the size of the pile group. The pile group configuration is currently not available to us. Provided that the piles are designed and installed as recommended above, it is considered that the quoted S.L.S. value corresponds to no more than 25 mm of settlement for the pile group. We will confirm the estimated settlement once information on the pile group configuration is known.

### 5.1.2 Resistance to Lateral Loads

Laterally applied loads on piles can be resisted geotechnically by the driven piles through passive pressure developed in the soil in which the piles are embedded. The pile tip elevations recommended above indicate that the piles will be in the order of 45 m in length. For conventional piled foundations, lateral resistance may be considered in accordance with Section 6-9.8.1 of the O.H.B.D.C., 3<sup>rd</sup> Edition.

The recommended horizontal resistances for a HP310x110 pile at this site are as follows:

Factored Horizontal Resistance at U.L.S.	=	120 kN
Horizontal Resistance at S.L.S.	=	50 kN

If an integral abutment type pile is not to be built, then unbalanced horizontal forces could be resisted by battered piles, but the degree of batter should be kept to a minimum in view of the anticipated consolidation settlements.

For lateral soil-pile interaction analysis, the horizontal subgrade reaction to the pile can be calculated from the expression:

$$k_s = n_h \times z/d$$

where

$k_s$  = coefficient of horizontal subgrade reaction  
 $n_h$  = coefficient related to soil density as given in Table 2  
 $d$  = pile width  
 $z$  = depth

*granular soils*  
*cohesive soils*  
*67 cm*  
*6*

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

TABLE 2

AREA/REFERENCE BOREHOLE NO.	APPLICABLE DEPTH FROM EXISTING GROUND SURFACE (ELEVATION)	SOIL TYPE	BULK UNIT WEIGHT (kN/m <sup>3</sup> )	ANGLE OF INTERNAL FRICTION ( $\phi$ ) DEGREES	RECOMMENDED $n_h$ VALUE (MN/m <sup>2</sup> )
North Abutment					
TCPL 1	0 - 2 m (301 - 299 m)	compact silty sand	20	30	5
	2 - 8 m (299 - 293 m)	stiff silty clay	18	—	5/2 (constant with depth)
	8 - 19 m (293 - 282 m)	loose to compact silt to sandy silt	19	29	4
	19 - 28 m (282 - 273 m)	loose to compact sand and silt	20	30	5
	28 - 39 m (273 - 262)	dense sand and silt	21	32	10 ✓

TABLE 2 (continued)

AREA/REFERENCE BOREHOLE NO.	APPLICABLE DEPTH FROM EXISTING GROUND SURFACE (ELEVATION)	SOIL TYPE	BULK UNIT WEIGHT (kN/m <sup>3</sup> )	ANGLE OF INTERNAL FRICTION (φ) DEGREES	RECOMMENDED n <sub>b</sub> VALUE (MN/m <sup>3</sup> )
<b>South Abutment</b>					
TCPL 3	0 - 3 m (307 - 304 m)	loose to compact silty sand	19	30	5
	3 - 6 m (304 - 301 m)	loose to compact silt	19	29	4
	6 - 15 m (301 - 292 m)	stiff silty clay	18	—	5/z (constant with depth)
	15 - 21 m (292 - 286 m)	compact silt	20	30	5
	21 - 32 m (286 - 275 m)	dense silt to sand and silt	21	32	9
	32 - 42 m (275 - 265 m)	very dense sand and silt	21	33	10

If the abutments are to be supported on conventional piled foundations (instead of using integral abutments), then there may be more than one row of piles. In this instance, group action for lateral loading should be considered when the pile spacing in the direction of loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction in the direction of loading by a reduction factor R as follows:

PILE SPACING IN DIRECTION OF LOADING d = PILE DIAMETER	SUBGRADE REACTION REDUCTION FACTOR R
8d	1.00
6d	0.70
4d	0.40
3d	0.25

## 5.2 LATERAL EARTH PRESSURES

Backfill behind abutments and retaining walls should consist of non-frost susceptible, free draining granular materials in accordance with MTO 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., 3<sup>rd</sup> Edition. For design purposes, the following

parameters (unfactored) can be used.

Compacted Granular 'A'

Unit Weight = 22 kN/m<sup>3</sup>

Coefficient of Lateral Earth Pressures:

$K_a = 0.27$  (active condition)

$K_o = 0.43$  (at-rest condition)

Compacted Granular 'B'

Unit Weight = 21 kN/m<sup>3</sup>

Coefficient of Lateral Earth Pressures:

$K_a = 0.31$  (active condition)

$K_o = 0.47$  (at-rest condition)

The above design parameters assume level ground surface and backfill behind the retaining structure.

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., 3<sup>rd</sup> 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 of the O.H.B.D.C., 3<sup>rd</sup> 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 moderate to high appearance.

### 5.3 APPROACH EMBANKMENTS

At the north abutment location, the proposed road grade is at about Elevation 310.5 m, which will require an approach embankment of about 9 m in height. The height of the embankment increases further north to a maximum of about 11 m at about Station 10+530. At the south abutment location, the proposed road grade is at about Elevation 311.5 m, which will require an approach embankment of up to 4 m in height. On this side, the height of the embankment fill decreases further south (i.e. to about zero at approximately 20 m south of the south abutment location).

The boreholes show that the subgrade consists of loose to compact sand to silt overlying typically stiff silty clay. Some organic materials are present within lower elevations of the ravine valley. Embankments of conventional fill of up to 11 m in height, with a slope inclination of 2 horizontal to 1 vertical, would be stable against deep-seated (i.e. foundation) failures, provided that the subgrade is properly prepared by removing all topsoil, organic and otherwise unsuitable materials as per MTO Standards before placing the fill.

Assuming properly compacted, acceptable inorganic earth fill material, 2 horizontal to 1 vertical (2H:1V) side slopes can be used throughout, except for embankment heights of greater than 6 m at the north approach where a 2 m wide mid-height berm should be provided to satisfy current requirements by MTO Northern Region. Mid-height berm is not required at the south approach where the embankment height is less than 6 m. The berm gradient should be sloped (say 1V:20H) to drain away from the embankment. 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 1H:1V from the toe of the proposed embankment as depicted by Figure No. 8 appended to this report. The average thickness of the unsuitable soils to be stripped can be assumed to be about 0.2 m. This thickness can be expected to increase to about 1 m in the vicinity of Borehole TCPL2. After stripping, the exposed subgrade should be inspected, approved and properly compacted from the surface under the supervision of qualified personnel, using a suitably heavy compactor.

Provided that all organic and otherwise unsuitable materials are removed and the subgrade is properly compacted from the surface as detailed above, the total settlement of the foundation materials (i.e. not including the settlement of the embankment material under its own weight) should not exceed 120 mm at the south approach and abutment locations. Half of this settlement (elastic) should be substantially complete during construction and about one month after placing the embankment fill to its full height. Post construction settlement due to clay consolidation is estimated to be in the order of 50 mm.



DRAFT

From the north abutment location to about Station 10+550 where embankment heights are between 9 m and 11 m, total foundation settlement induced as a consequence of embankment loading using normal earth fill is estimated to be up to about 300 mm at the abutment location (where the height of the fill will be about 9 m) increasing to about 400 mm further north at about Station 10+530 (where the height of fill reaches a maximum of 11 m). Post construction settlement due to clay consolidation would be in the order of 120 to 180 mm. After the completion of fill placement, it is anticipated that up to 50% of the anticipated total settlement would have taken place within one month of fill placement. If the fill is left in place for six months, 85% of the total settlement would have taken place. If scheduling permits, consideration should be given to preloading of the north abutment for up to six months, i.e. build up to profile grade and wait for up to six months prior to carrying out paving, to allow a substantial portion of the settlement to take place.

6-16  
85%  
x 400  
340

As discussed previously, preloading would minimize the effects of down-drag on the piles as a result of consolidation of the silty clay and elastic compression of the sands and silts. We, therefore, recommend that preloading be carried out at both abutments. In any case, the approach fills should be placed to their final grade elevation at least one month prior to driving the piles.

Reeds  
for  
monitoring

Water level measurements indicate that perched water levels are at about 1 m to 2 m below existing ground surface. We do not anticipate major problems due to groundwater seepage, but care should be exercised to minimize disturbance to the silty subgrade during subgrade preparation and backfilling for the construction of the embankments. Depending on the time of construction, some dewatering may be required to stabilize the silty soils.

The fill materials used for construction of the embankment should consist of approved, clean earth fill (e.g. Select Subgrade Materials - OPSS 1010). A majority of the fill will have to be imported for this purpose. 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 Maximum Dry Density. The degree of compaction within the top 0.6 m of the fill (i.e. the subgrade immediately beneath the granular sub-base) should be increased to 98%. The selection, placement and compaction of the fill should be carried out under geotechnical control. The settlement of the embankment fills prepared as described above should not exceed 85 mm for the north approach embankment. Fill settlements at the south approach should not exceed 30 mm. The time rate of settlement of the fill making up the embankment will depend on the material used for construction and for granular fills it should substantially be completed during the construction and within a few weeks thereafter (i.e. should be essentially elastic). Clayey fills can, on the other hand, be expected to consolidate over a longer period of time. It should also be pointed out that these quoted settlements will be in addition to the foundation settlements quoted earlier.

How  
about  
rockfill

## 5.4 CONSTRUCTION COMMENTS

All excavations should be carried out in accordance with the Ontario Occupational Health and Safety Act and its regulations.

Water level measurements in the boreholes indicate that perched groundwater is present at between 1 m and 2 m below existing grade. If an integral abutment type bridge is not constructed and if pile caps below existing grades are required, dewatering may be required for the excavations for pile cap construction. Care should be taken to minimize disturbance to the silty subgrade during excavation. Consideration may be given to an oversize excavation with perimeter drains. Temporary side slopes of 2 horizontal to 1 vertical should be stable provided that the groundwater is controlled by pumping from properly filtered sumps. If the underside of the pile caps are to be constructed at elevations more than about 1.5 m below the existing grades, more extensive dewatering or temporary sheeting would likely be required to provide groundwater control and to retain the soil during construction.

Precautions to ensure the integrity of the existing TCPL pipes during pile driving should be taken, especially near the west side of the south abutment where the horizontal distance from the south abutment to the centreline of the 914 mm diameter HPL 100-2 pipeline appears to approach 7 m. For this purpose you may wish to consult a vibration specialist and/or the pile driving contractor.

The construction activities and especially fill placement should also be undertaken so as to prevent damage to the existing pipelines. The geometry of any filling (particularly forward slopes) should be reviewed by the geotechnical engineer with this aspect in mind, both for permanent and temporary (i.e. preloading) fill configuration. For temporary fills, depending on the type of soil used and period of preloading, side slopes of the order of 1 to 1 1/4H:1V will likely be satisfactory. With this configuration the toe of forward slopes at certain locations can be expected to closely approach the existing pipes, and this aspect should be checked.

In addition, the soil under the fibre optic (FOTTS) cable can be expected to settle up to about 200 mm towards the east end of the proposed north abutment, due to the stresses induced by the proposed embankment fill. It would be prudent to check this aspect to ensure that the cable can withstand some stretching, due to the anticipated settlements, without significant damage.

## 5.5 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 is required for frost protection of foundations.

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## 6.0 CLOSURE

We recommend that once the details of the structure and approaches are finalized, our recommendations should be reviewed for their specific applicability.

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

Sincerely,



Sydney Pang, P. Eng.

SP/dee



Zuhtu S. Ozden, P. Eng.

*Signed & sealed by  
Engineer*

## APPENDIX A

## **AGRA**

### **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 environmental 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. It is recommended practice that the Geotechnical Engineer be retained during construction to confirm that the subsurface conditions throughout the site do not deviate materially from those encountered in testholes. 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.

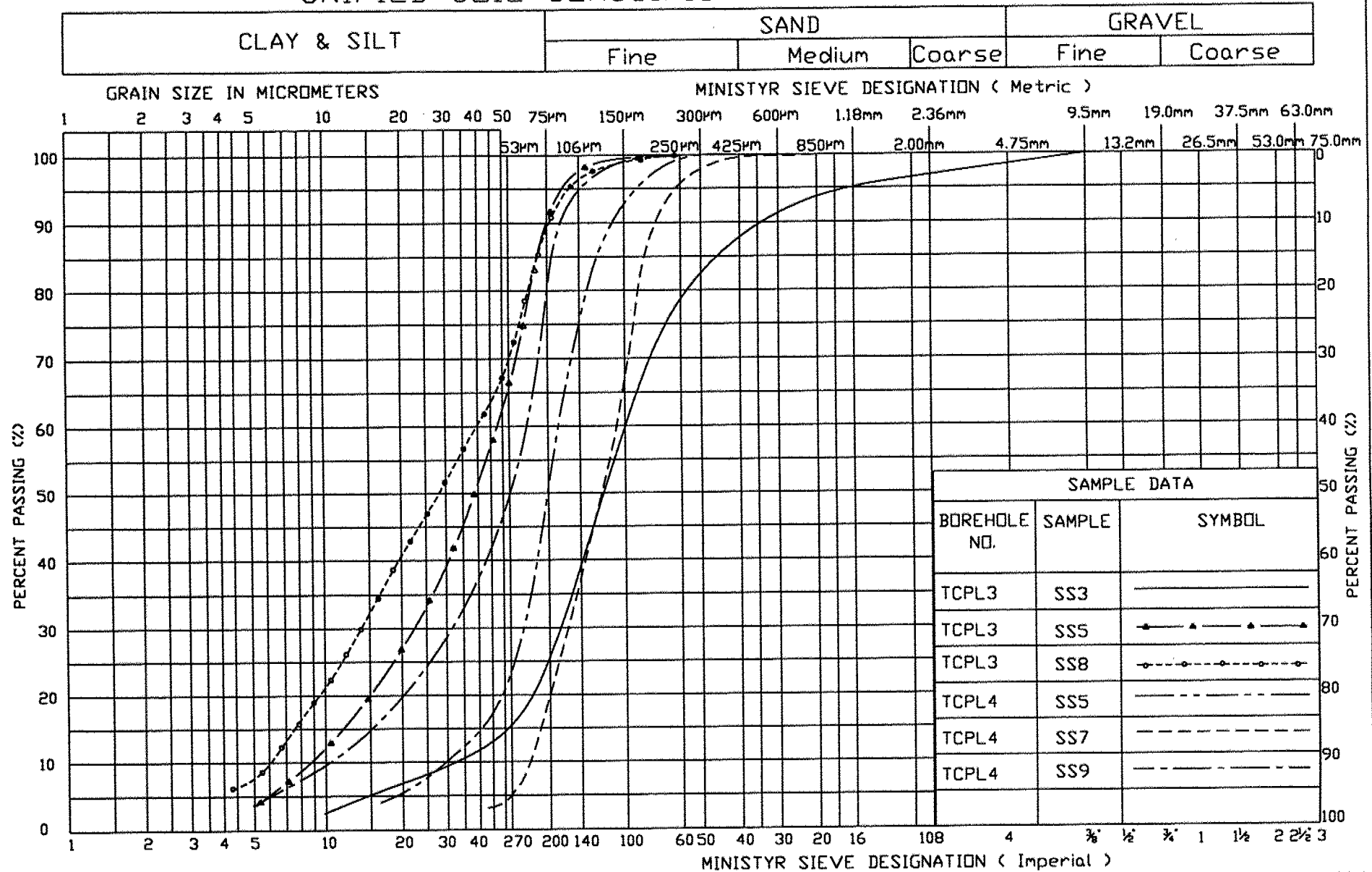
The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report. Since all details of the design may not be known, we recommend that we be retained during the final design stage to verify that the design is consistent with our recommendations, and that assumptions made in our analysis are valid.

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. No other warranty is expressed or implied.

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. AGRA accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

## FIGURES

# UNIFIED SOIL CLASSIFICATION SYSTEM



**AGRA**

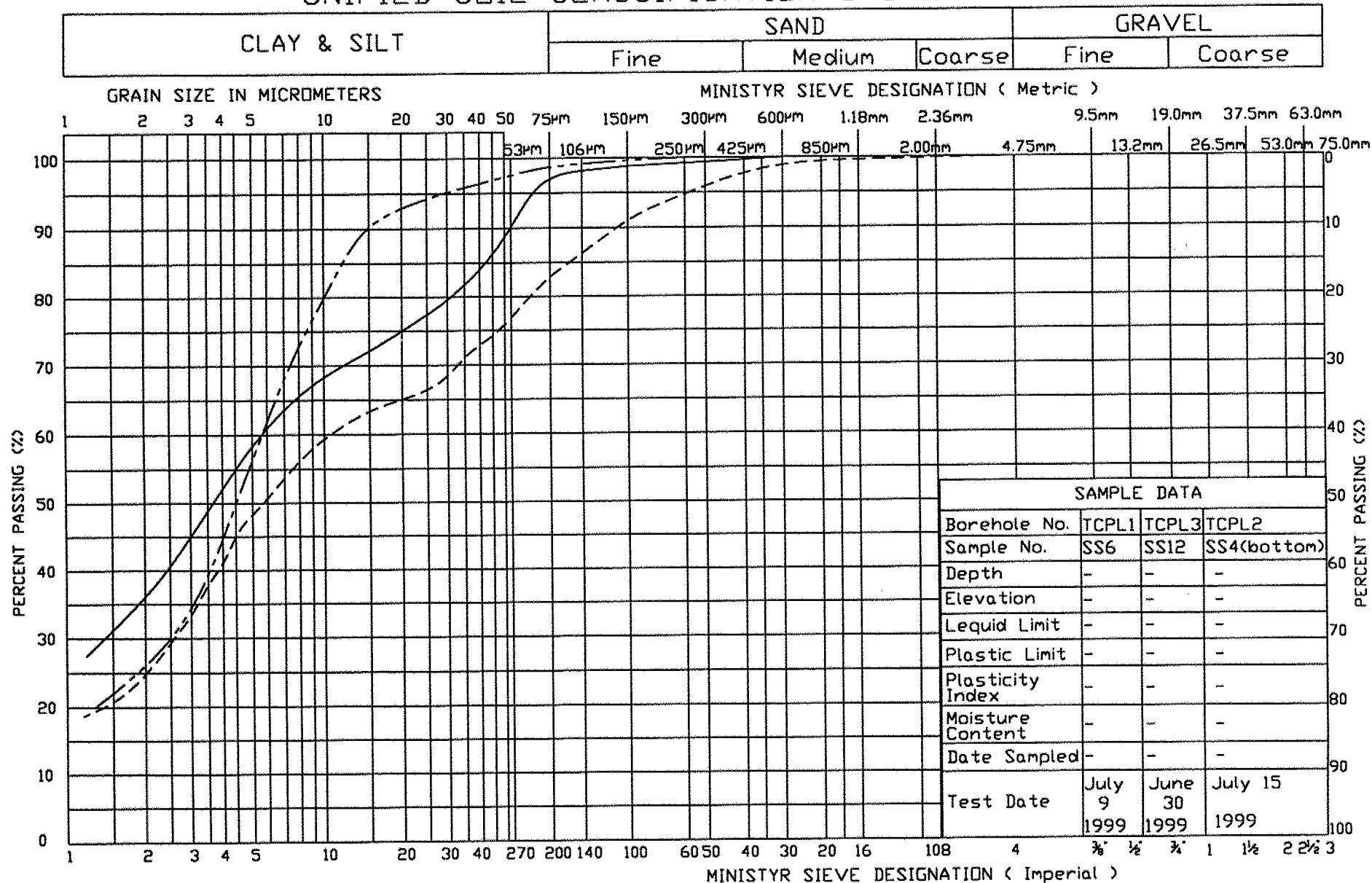
ENGINEERING GLOBAL SOLUTIONS

## GRAIN SIZE DISTRIBUTION

SURFICIAL SAND, SOME GRAVEL, TO SILT

CLIENT:	DELCAN
JOB NO.:	TT98820 W P 473-93-00
PROJECT:	HWY 11
LOCATION:	TCPL BRIDGE
DATE:	AUGUST 3, 1999
FIGURE:	1

# UNIFIED SOIL CLASSIFICATION SYSTEM



**AGRA**

ENGINEERING GLOBAL SOLUTIONS

## GRAIN SIZE DISTRIBUTION

SILTY CLAY

TCPL1: SS6  
 TCPL3: SS12  
 TCPL2: SS4(bottom)

CLIENT:	DELCAN	
JOB NO.:	TT98820	W P 473-93-00
PROJECT:	HWY 11	
LOCATION:	TCPL BRIDGE	
DATE:	AUGUST 3, 1999	FIGURE: 2

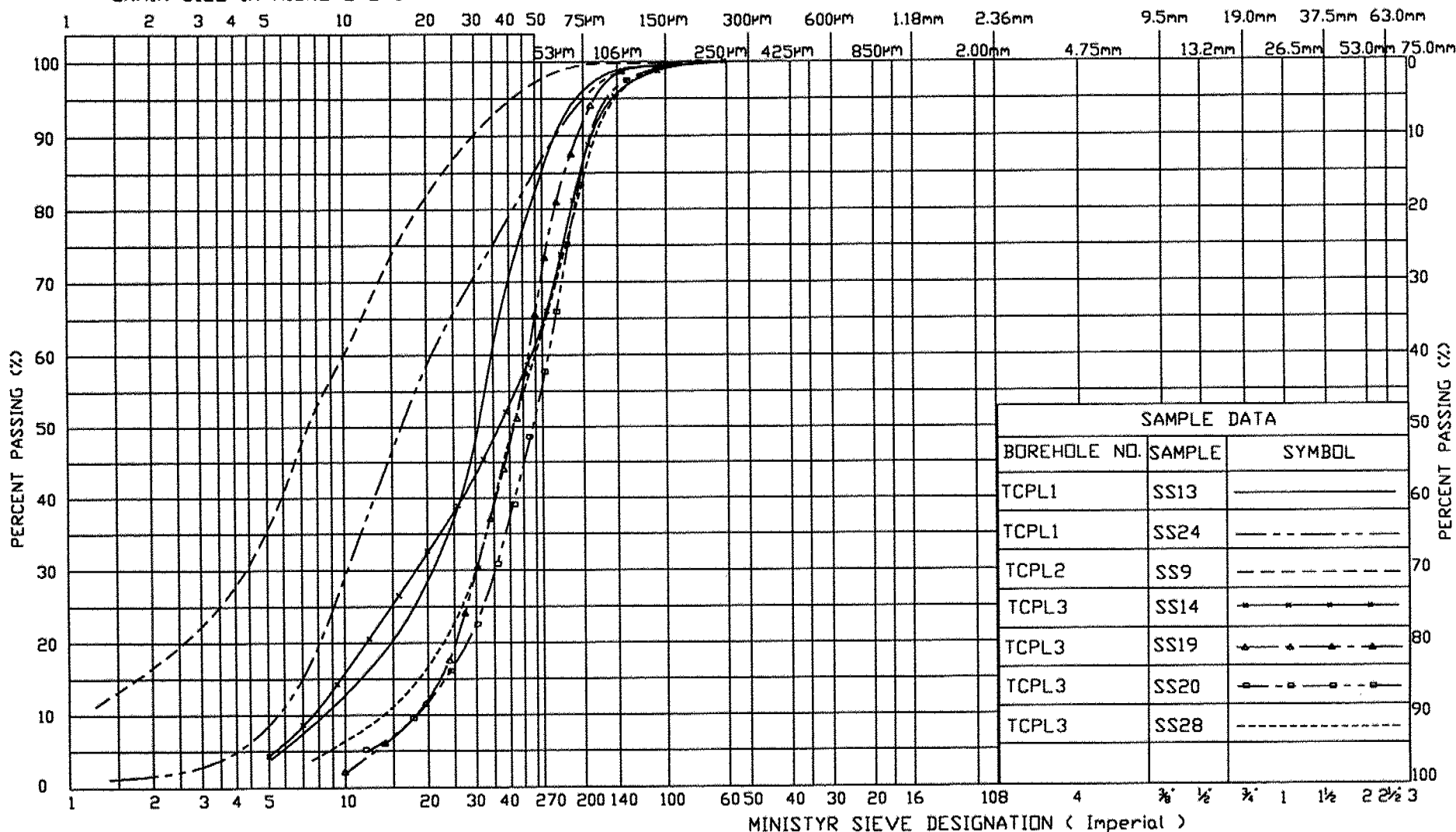


# UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse

GRAIN SIZE IN MICROMETERS

MINISTYR SIEVE DESIGNATION ( Metric )



SAMPLE DATA		
BOREHOLE NO.	SAMPLE	SYMBOL
TCPL1	SS13	—————
TCPL1	SS24	-----
TCPL2	SS9	-----
TCPL3	SS14	—•—•—•—•—
TCPL3	SS19	—•—•—•—•—
TCPL3	SS20	—•—•—•—•—
TCPL3	SS28	-----



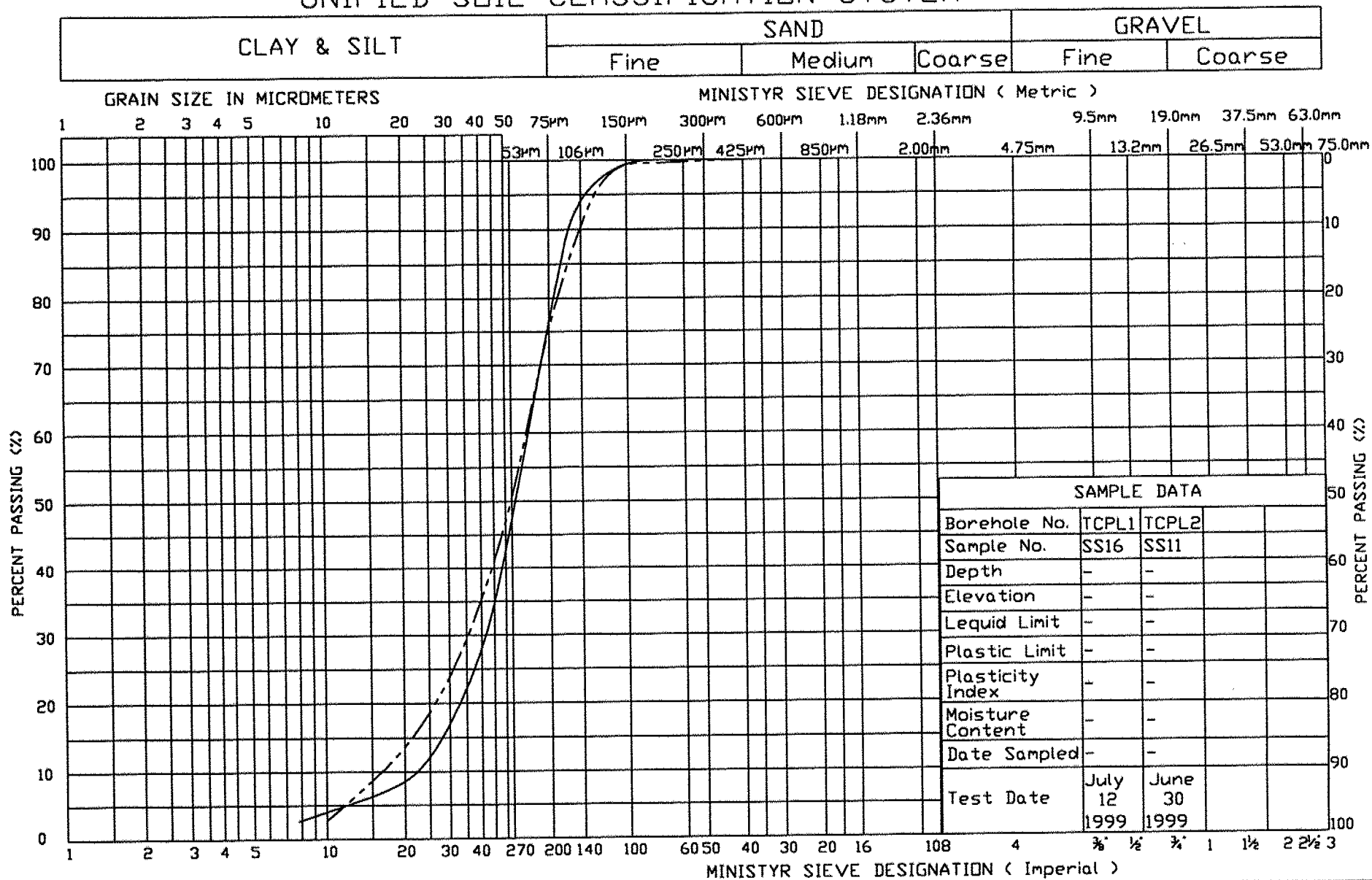
## GRAIN SIZE DISTRIBUTION

SILT

TRACE TO SOME SAND, OCCASIONAL CLAY

CLIENT:	DELCAN	
JOB NO.:	TT98820	W P 473-93-00
PROJECT:	HWY 11	
LOCATION:	TCPL BRIDGE	
DATE:	AUGUST 3, 1999	FIGURE: 3

# UNIFIED SOIL CLASSIFICATION SYSTEM



**AGRA**

ENGINEERING GLOBAL SOLUTIONS

## GRAIN SIZE DISTRIBUTION

SANDY SILT

TCPL1: SS16

TCPL2: SS11

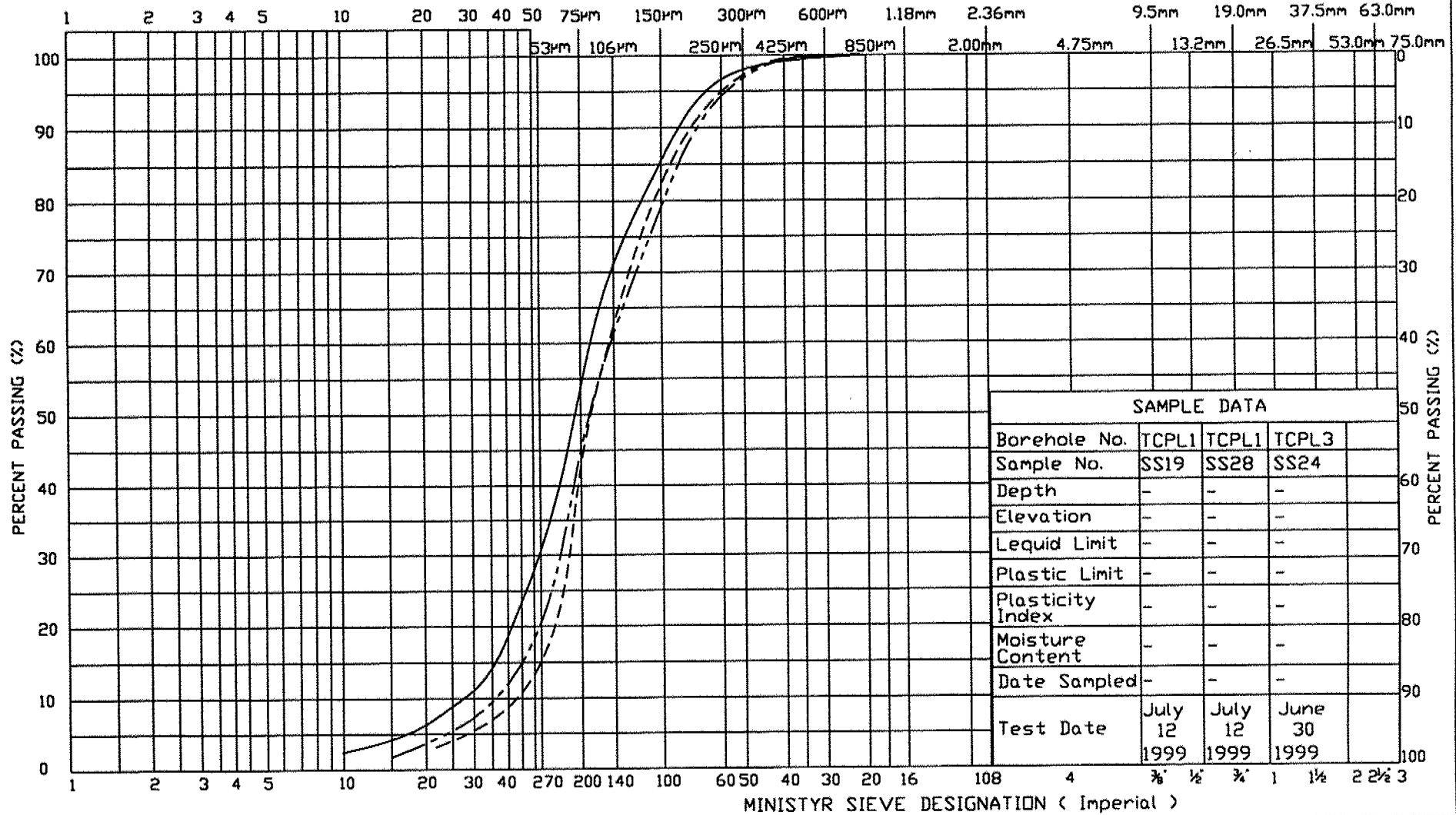
CLIENT:	DELCAN		
JOB NO.:	TT98820	W P 473-93-00	
PROJECT:	HWY 11		
LOCATION:	TCPL BRIDGE		
DATE:	AUGUST 3, 1999		FIGURE: 4

# UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse

GRAIN SIZE IN MICROMETERS

MINISTYR SIEVE DESIGNATION ( Metric )



## SAMPLE DATA

Borehole No.	TCPL1	TCPL1	TCPL3
Sample No.	SS19	SS28	SS24
Depth	-	-	-
Elevation	-	-	-
Liquid Limit	-	-	-
Plastic Limit	-	-	-
Plasticity Index	-	-	-
Moisture Content	-	-	-
Date Sampled	-	-	-
Test Date	July 12 1999	July 12 1999	June 30 1999



**AGRA**

ENGINEERING GLOBAL SOLUTIONS

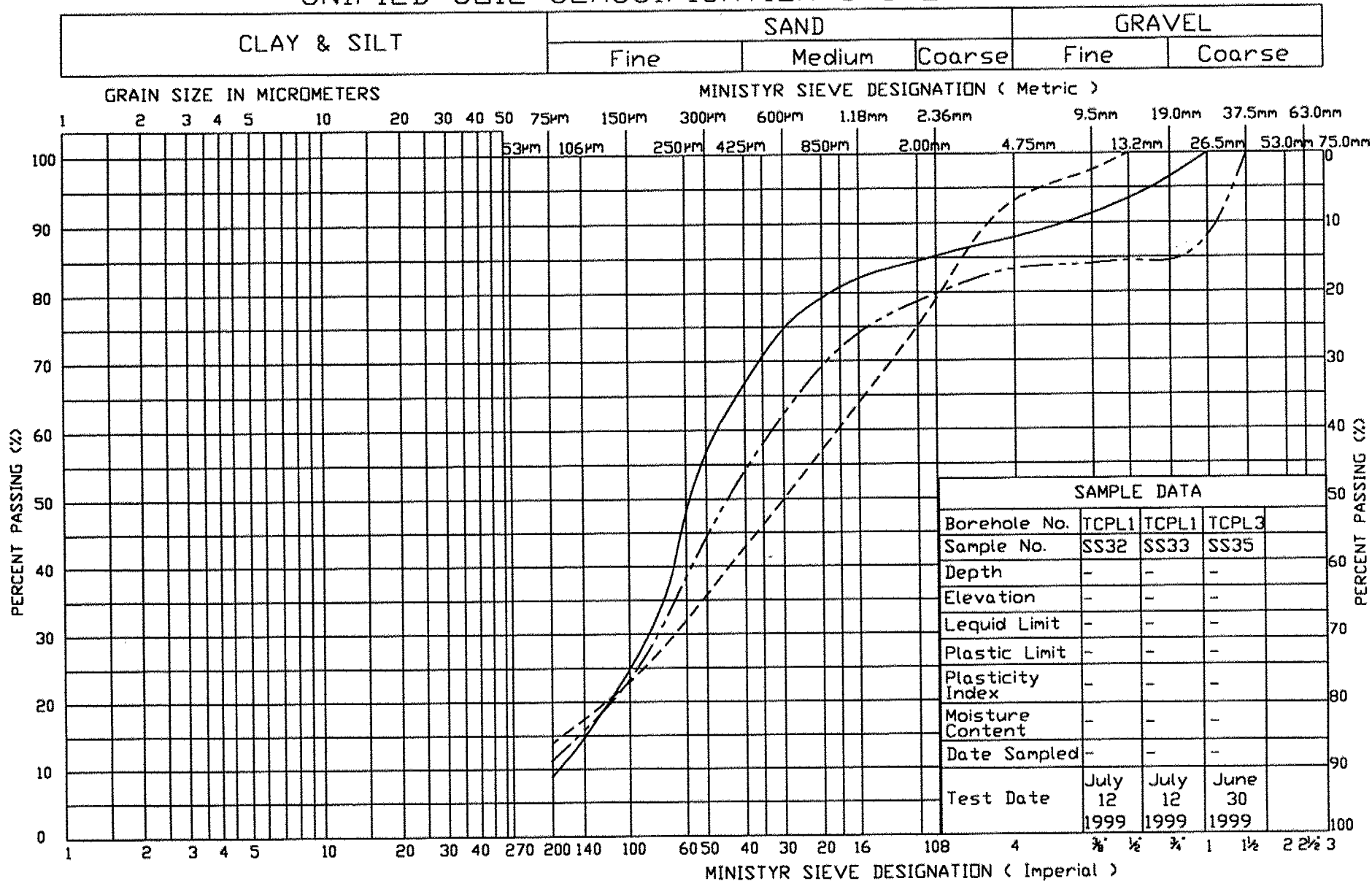
## GRAIN SIZE DISTRIBUTION

SAND AND SILT

TCPL1: SS19
TCPL1: SS28
TCPL3: SS24

CLIENT:	DELCAN
JOB NO.:	TT98820 W P 473-93-00
PROJECT:	HWY 11
LOCATION:	TCPL BRIDGE
DATE:	AUGUST 3, 1999
FIGURE:	5

# UNIFIED SOIL CLASSIFICATION SYSTEM



SAMPLE DATA			
Borehole No.	TCPL1	TCPL1	TCPL3
Sample No.	SS32	SS33	SS35
Depth	-	-	-
Elevation	-	-	-
Liquid Limit	-	-	-
Plastic Limit	-	-	-
Plasticity Index	-	-	-
Moisture Content	-	-	-
Date Sampled	-	-	-
Test Date	July 12 1999	July 12 1999	June 30 1999

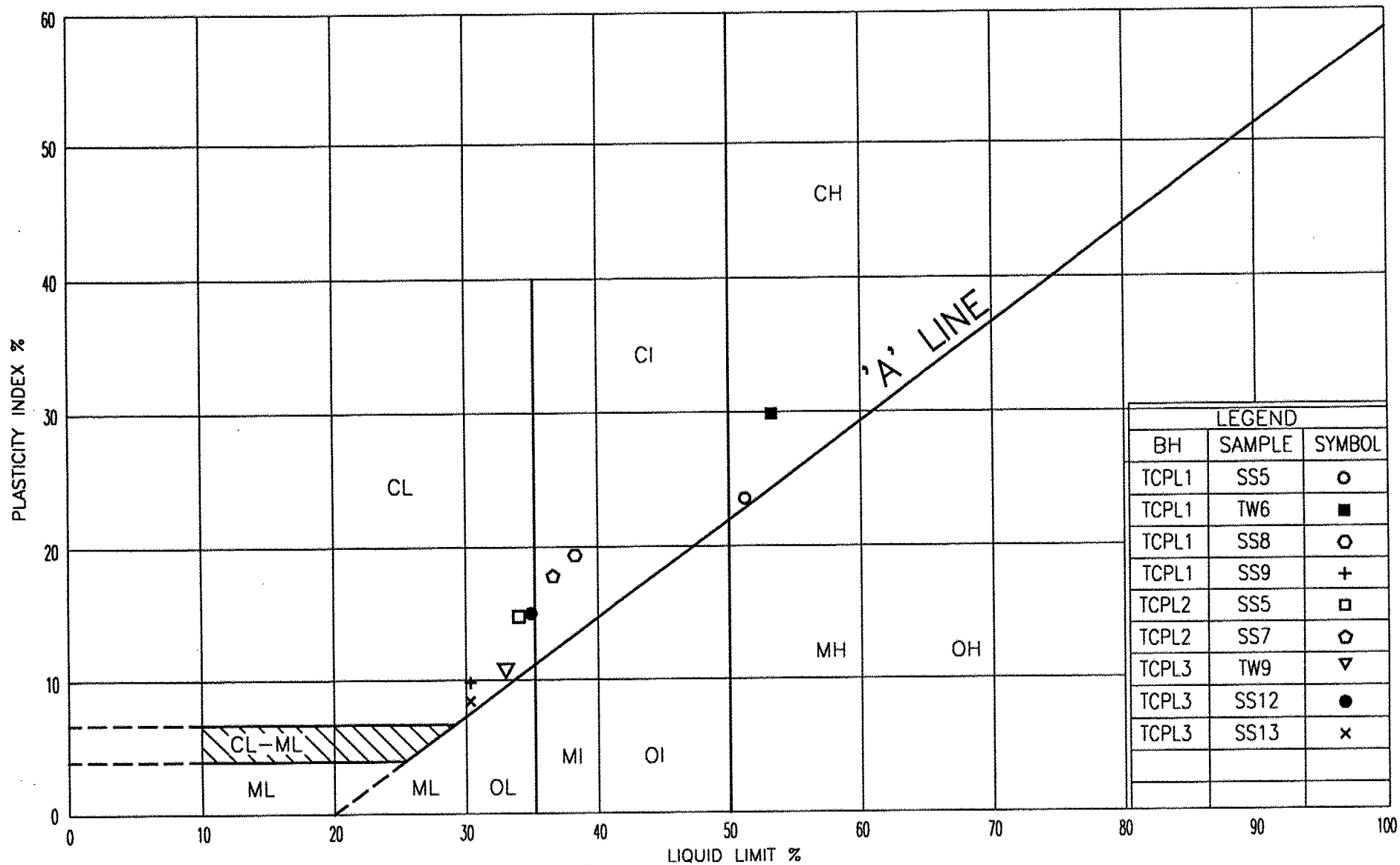


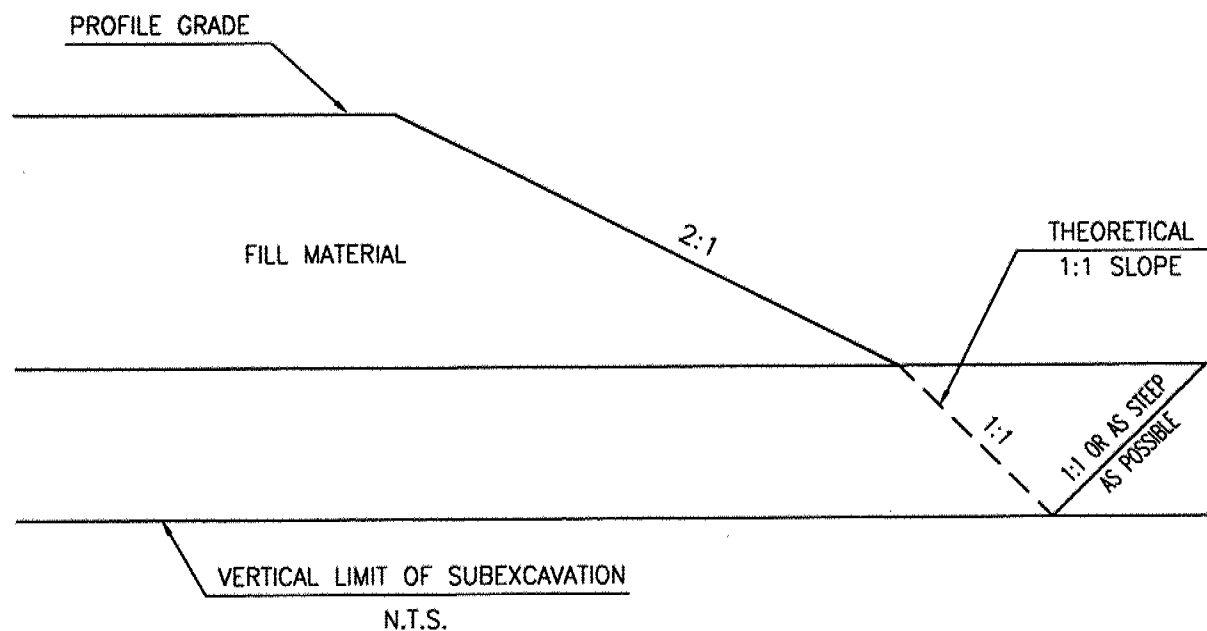
## GRAIN SIZE DISTRIBUTION

SAND  
TRACE TO  
SOME GRAVEL

TCPL1: SS32	_____
TCPL1: SS33	-----
TCPL3: SS35	-----

CLIENT:	DELCAN
JOB NO.:	TT98820 W P 473-93-00
PROJECT:	HWY 11
LOCATION:	TCPL BRIDGE
DATE:	AUGUST 3, 1999
FIGURE:	6





REMOVAL OF UNSUITABLE SOILS  
FROM BENEATH APPROACH FILLS  
( N.T.S. )

FIGURE No 8  
W P 473-93-00

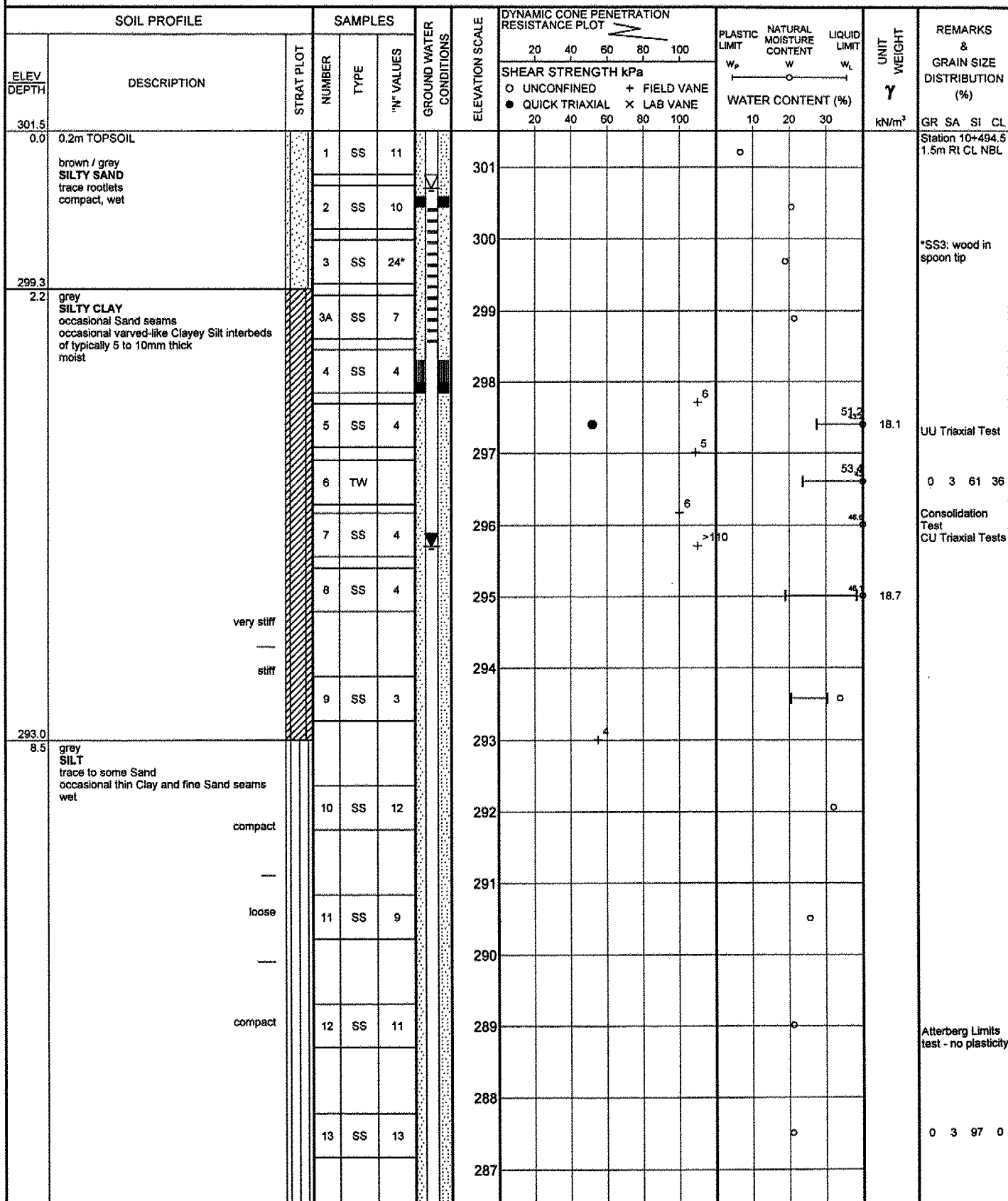
ENCLOSURES

RECORD OF BOREHOLE No TCPL1

1 OF 4

METRIC

W.P. 473-93-00 LOCATION N 5046910.3 E 316959.4 ORIGINATED BY AD  
 DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augering/Tri-coning COMPILED BY CK  
 DATUM Geodetic DATE 4 June 1999 - 5 June 1999 CHECKED BY ZSO



Continued Next Page

+ 3, X 3: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE



## METRIC

+3, X3: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

## METRIC

W.P.	473-93-00	LOCATION	N 5046910.3 E 316959.4	ORIGINATED BY	AD
DIST	52	HWY	11	BOREHOLE TYPE	Hollow Stem Augering/Tri-coning
DATUM	Geodetic	DATE	4 June 1999 - 5 June 1999	COMPILED BY	CK
				CHECKED BY	ZSO

Continued Next Page

+3, X3: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

**RECORD OF BOREHOLE No TCPL1**

4 OF 4

**METRIC**

W.P. 473-93-00 LOCATION N 5046910.3 E 316959.4 ORIGINATED BY AD  
DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augering/Tri-coning COMPILED BY CK  
DATUM Geodetic DATE 4 June 1999 - 5 June 1999 CHECKED BY ZSO

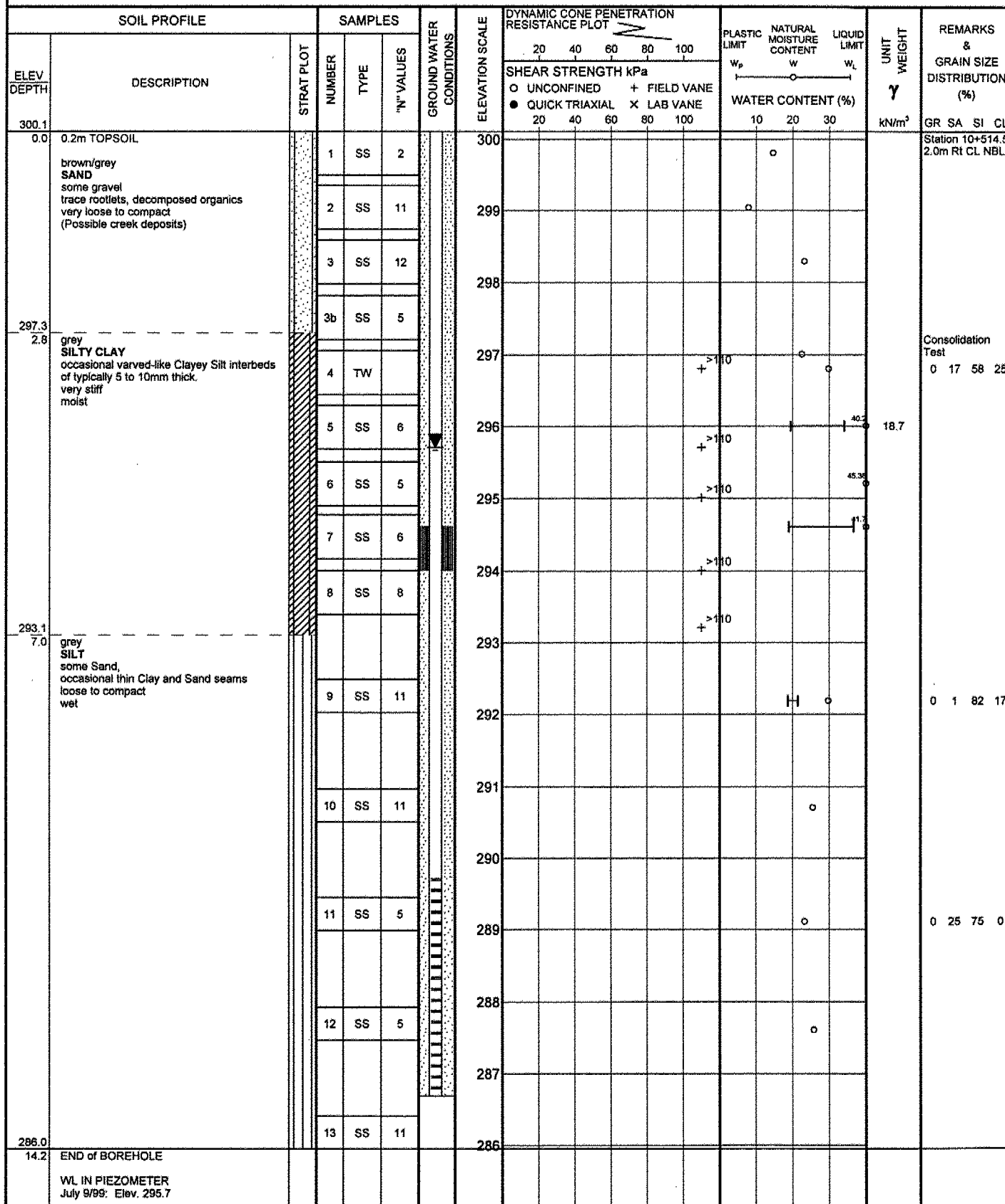
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
254.6	grey SAND some Silt, some Gravel frequent cobbles, some boulders, very dense, wet		34	SS	75/7.5												Coring through cobbles/boulders from 44.2m to 45.1m depths.
			35	SS	66/2.6												
			36	SS	100/5.0												
46.9	END of BOREHOLE  Refusal to split spoon sampler advance at 45.8m and 46.4m depths.  Dynamic Cone Penetration Test conducted @ 46.8m depth. Refusal @ 46.9m depth.  WL in open borehole on completion: 6.2m  Lower piezometer tip at Elev. 256m and upper piezometer tip at Elev. 298.5m  WL IN PIEZOMETER  July 9/98: Elev. 295.7m depth (lower) Elev. 300.7m depth (upper)																

RECORD OF BOREHOLE No TCPL2

1 OF 1

METRIC

W.P. 473-93-00 LOCATION N 5046929.9 E 316958.1 ORIGINATED BY AD  
 DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augering COMPILED BY CK  
 DATUM Geodetic DATE 9 June 1999 CHECKED BY ZSO



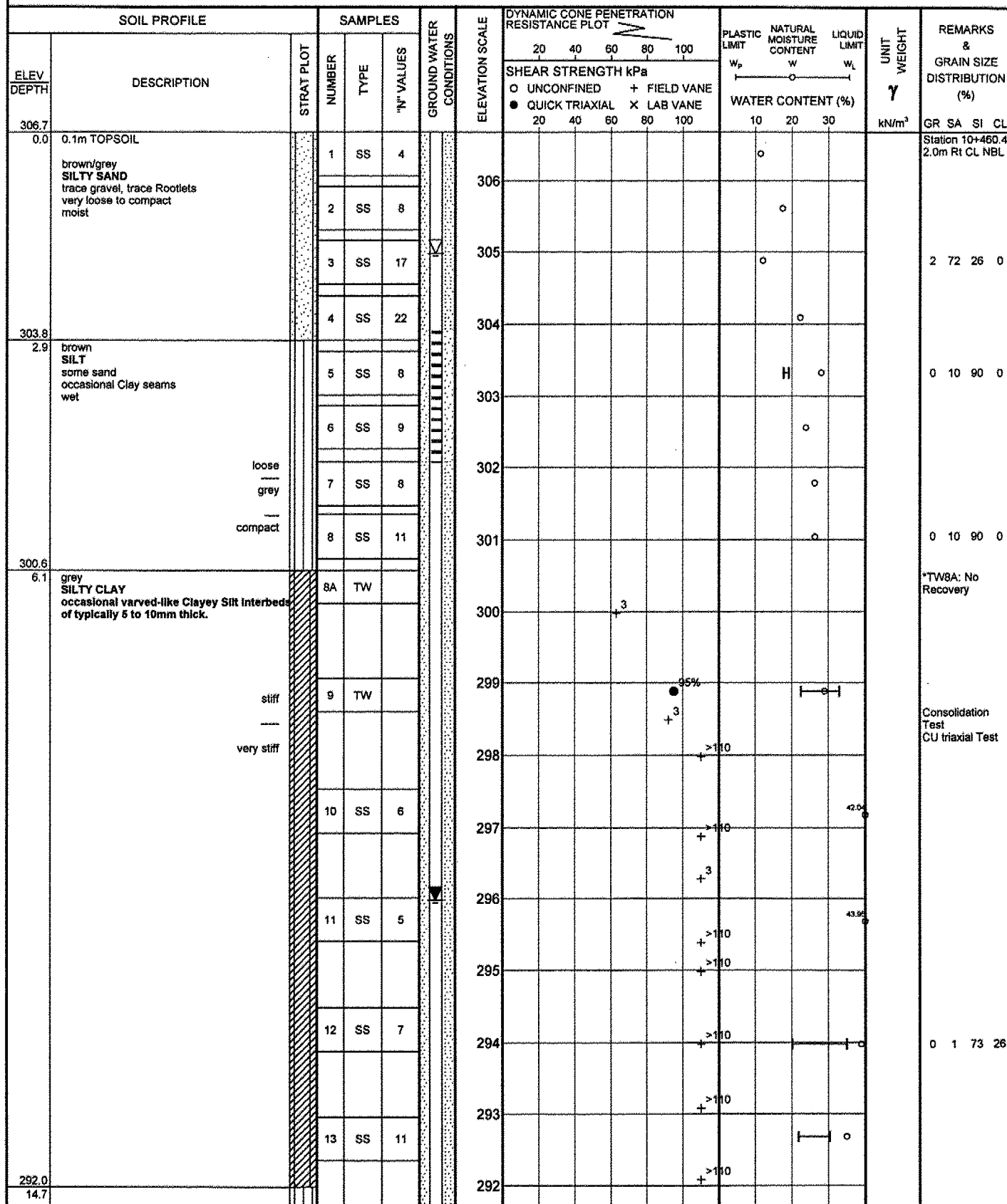
+ 3, X 3: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No TCPL3

1 OF 4

METRIC

W.P. 473-93-00 LOCATION N 5046876.5 E 316963.3 ORIGINATED BY AD  
 DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augering / Wash boring / Rock coring COMPILED BY CK  
 DATUM Geodetic DATE 9 June 1999 - 13 June 1999 CHECKED BY ZSO



Continued Next Page

+ 3, X 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

## METRIC

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3%</sup> STRAIN AT FAILURE

**RECORD OF BOREHOLE No TCPL3**

3 OF 4

**METRIC**

W.P. 473-93-00 LOCATION N 5048876.5 E 316963.3 ORIGINATED BY AD  
DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augering / Wash boring / Rock coring COMPILED BY CK  
DATUM Geodetic DATE 9 June 1999 - 13 June 1999 CHECKED BY ZSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	
								SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE	WATER CONTENT (%) 10 20 30			GR SA SI CL
	grey SAND and SILT wet		24	SS	45		276					0 57 43 0
	dense											
	very dense						275					
			25	SS	63		274					
			26	SS	59		273					
							272					
	dense		27	SS	48		271					
	compact		28	SS	19		270					0 16 84 0
							269					
	very dense		29	SS	67		268					
			30	SS	62		267					
							266					
			31	SS	66		265					
264.5												
42.2	grey SAND with GRAVEL frequent cobbles very dense, wet		32	SS	100/18		264					
							263					
			33	SS	100/19		262					

Continued Next Page

+ 3, x 3, Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE





**RECORD OF BOREHOLE No TCPL4**

1 OF 1

**METRIC**

W.P. 473-93-00 LOCATION N 5046853.6 E 316965.0 ORIGINATED BY AD  
DIST 52 HWY 11 BOREHOLE TYPE Solid Stem Augering COMPILED BY CK  
DATUM Geodetic DATE 13 June 1999 CHECKED BY ZSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
313.1								20	40	60	80	100					
0.0	0.15m TOPSOIL	loose	1	SS	7		313										GR SA SI CL Station 10+437.4 CL NBL
	brown SAND fine, some Silt layers compact, moist		2	SS	13		312										
			3	SS	13		311										
			4	SS	15		310										
	sand and silt		5	SS	14		309										0 48 52 0
			6	SS	23		308										0 80 20 0
308.7			7	SS	23		307										0 22 78 0
4.4	grey SILTY SAND some sandy silt layers fine to medium compact, moist		8	SS	22												
			9	SS	25												
306.6	sandy silt																
6.6	END of BOREHOLE																
	No free water in open borehole on completion.																

+<sup>3</sup>, X<sup>3</sup>

Numbers refer to  
Sensitivity

○ 3% STRAIN AT FAILURE

GEOCRES No. \_\_\_\_\_

DIST. 52 REGION \_\_\_\_\_W.P. No. 473-93-00

CONT. No. \_\_\_\_\_

W. O. No. \_\_\_\_\_

STR. SITE No. \_\_\_\_\_

HWY. No. 11LOCATION Magnetawan River  
Bridge NBLNo. of PAGES - 1

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OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. \_\_\_\_\_REMARKS: \_\_\_\_\_  
\_\_\_\_\_  
\_\_\_\_\_  
\_\_\_\_\_

**DRAFT  
FOUNDATION INVESTIGATION AND DESIGN REPORT  
FOR PROPOSED MAGNETAWAN RIVER/  
HIGHWAY 520 OVERPASS STRUCTURE  
AT BURKS FALLS  
HIGHWAY 11  
DISTRICT 52, HUNTSVILLE  
W.P. 473-93-00  
SITE 44-188**

**Submitted To:**

**Delcan Corporation  
133 Wynford Drive  
North York, Ontario, M3C 1K1  
Canada**

**Submitted By:**

**AGRA  
104 Crockford Blvd.  
Scarborough, Ontario, M1R 3C6  
Canada**

**December 1999  
TT98820**

January 12, 2000.  
**Ref. No.: TT98820**

Delcan Corporation  
133 Wynford Drive  
North York, Ontario, M3C 1K1  
Canada

**Attention: Mr. Khaled El-Dalati, P. Eng.**

Dear Sir:

**Re: DRAFT  
FOUNDATION INVESTIGATION AND DESIGN REPORT  
FOR PROPOSED MAGNETAWAN RIVER/  
HIGHWAY 520 OVERPASS STRUCTURE  
AT BURKS FALLS  
HIGHWAY 11  
DISTRICT 52, HUNTSVILLE  
W.P. 473-93-00  
SITE 44-188**

We take pleasure in enclosing seven (7) copies of our Draft Foundation Investigation and Design Report for the above mentioned project and we will be glad to discuss any questions arising from this work.

Soil samples will be retained for a period of one year, and will thereafter be disposed of unless we are otherwise instructed.

We thank you for giving us this opportunity to be of service to you.

Sincerely,

George S.W. Chow, P. Eng.,  
Designated MTO Contact.

GSWC/dma

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## 1.0 INTRODUCTION

AGRA Earth & Environmental Limited (AGRA) has been retained by DELCAN Corporation (DELCAN) to carry out a foundation investigation at the site of a proposed 7-span overpass bridge which will carry the proposed northbound lanes (NBL) of Highway 11 over the Magnetawan River and the existing Highway 520. The site is located at Burks Falls, just east of the existing Highway 11 bridge crossings of the river and the highway, as part of the Highway 11 Four Laning project from 0.7 km north of Highway 592N at Katrine, northerly 12.4 km (W.P. 473-93-00).

The purpose of this investigation is to determine the subsurface conditions at the site of the proposed bridge structure by means of a number of boreholes, in-situ tests and laboratory tests on selected samples. Based on our interpretation of the data obtained, recommendations are provided on the geotechnical aspects of foundation design. Comments are also provided on anticipated construction issues where they may affect the geotechnical design of the proposed bridge and approach embankments.

The preliminary plan and profile for the proposed bridge and approaches were provided to us by DELCAN.

## 2.0 SITE DESCRIPTION AND PHYSIOGRAPHY

The existing ground surface at the south approach is at about Elevation 289 m and the existing ground surface at the north approach is at about Elevation 288 m. The lowest point within the valley is at about Elevation 280 m, which is at the middle of the river flow channel. The "normal" river level is at about Elevation 283, or up to 3 m of water. The "100-year" water level is at about Elevation 286.5, which corresponds to up to 6.5 m of water. Moderate vegetation cover, consisting of predominantly grass, shrubs and small trees, exists at both approach locations.

The existing bridge crossing the Magnetawan River is a 4-span structure with approach embankments of up to approximately 7 m in height. Some toe erosion along the river banks was evident, but there was no sign of significant active erosion on the embankment slope faces at the time of our investigation. Further north, the existing bridge crossing Highway 520 is a 3-span structure with approach earth embankments of up to about 7 m in height. There was no sign of significant erosion on the embankment slope face at the time of our investigation.

The Magnetawan River flows in a north-easterly direction in the vicinity of the bridge site. At the location of the proposed south abutment, the existing terrain slopes in a northerly direction with a steepest inclination of 4 horizontal to 1 vertical (4H:1V). At the north abutment, the steepest slope inclination is in the order of 8H:1V.

Based on available geological information, the site is situated within an area of ice-contact sediments. In general, after the last glacial withdrawal, ice-contact sediments of sands and gravel, followed by glaciofluvial sediments of deltaic and nearshore sands and gravel, as well as lake bottoms silts and clays, were deposited on top of the existing sandy glacial till or Precambrian bedrock. The area was then inundated by glacial Lake Algonquin, depositing sands, silts and clays in low lying areas.

### 3.0 INVESTIGATION PROCEDURES

The field work for this investigation was carried out within several time periods, from May 27 to 31, June 2 and 3, August 19, September 2, 7 to 10, October 4 and 5, 1999, during which time twenty-one (21) boreholes (Borehole Nos. 1 to 2, 4 to 12, 14 to 22 and 24) were drilled and sampled. Boreholes 3 and 13 were advanced as Dynamic Cone Penetration Tests (DCPT). Two boreholes were put down at each of the proposed bridge abutments and piers, and one borehole was put down for each approach to the abutments. The remaining boreholes were put down at locations close to the proposed foundation footprints of an alternate 5-span structure. The 5-span alternative is now abandoned. The plan locations of the boreholes and selected stratigraphic sections are shown on Drawing Nos. 1 and 2. The originally proposed Borehole 23 was not drilled.

The investigation was carried out using a track-mounted power auger drill rig owned and operated by Groundworks Drilling Inc., under the full-time supervision of a member of AGRA's engineering staff. Hollow stem and solid stem augers were used to advance the boreholes within shallower depths. At greater depths, wash-boring techniques were required to further advance the boreholes. Rotary core drilling techniques were utilized to penetrate through cobbles and boulders within the glacial till deposit encountered near the bottom of the deeper boreholes.

In the boreholes, soil samples were obtained at regular intervals of depth using 51 mm outside diameter split barrel (split spoon) samplers in accordance with Standard Penetration Test (SPT) procedures, as specified by ASTM Standard D1586. The SPT consists of freely dropping a 63.5 kg hammer for a vertical distance of 0.76 m to drive the split spoon sampler into the ground. The number of blows of the hammer required to drive the sampler into the relatively undisturbed ground, for a vertical distance of 0.30 m, is recorded as the Standard Penetration Resistance or the 'N'-value of the soil. This value gives an indication of the relative state of compactness of cohesionless soils and the consistency of cohesive soils.

Boreholes 3 and 13 were advanced by carrying out Dynamic Cone Penetration Tests (DCPT). This test consists of continuously driving a 60° point, 50 mm diameter cone attached to the drill rod, into the undisturbed ground with a driving energy of 475 kJ (63.5 kg hammer free falling for a distance of 76 cm) per blow. The number of blows for each 30 cm of penetration is recorded, providing an indication of the relative changes in the soil density with depth. The DCPT was also carried out in all but Boreholes 14 and 16.

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Due to the presence of cobbles and boulders within the glacial till deposits, wash boring techniques in conjunction with casings and/or tri-coning were utilized to further advance the boreholes. Coring of cobbles, boulders and bedrock were carried out using a NQ size core barrel.

Groundwater conditions in the open boreholes were observed throughout and immediately after the drilling operations. Standpipe piezometers were installed in Boreholes 1, 2, 4 to 12, 15, 17, and 19, to permit long term monitoring of groundwater levels at the site. The piezometer installation at Boreholes 11 and 17 were subsequently found to have been buried due to re-grading operations of the area. All boreholes were adequately backfilled with a bentonite mix on completion of the field work.

The drilling, sampling and in-situ testing operations were carried out under the full-time supervision of members of our engineering staff, who examined the samples and logged the boreholes. The soil samples were identified and placed in containers; the rock core samples were securely stored in wooden core boxes, and transported back to our geotechnical laboratory in Toronto (Scarborough) for further examination and testing. Index and classification tests, including natural moisture content determination, grain size distribution analysis and Atterberg Limits tests, were carried out on selected representative soil samples. Unit weight determinations were carried out for several samples. The results of the laboratory tests are presented on the relevant Record of Borehole sheets and on Figure Nos. 1 to 8.

The drilling locations were initially established in the field by our field personnel based on the centreline of Highway 11 staked out by Dearden and Stanton Limited. The as-drilled borehole locations in terms of northing and easting co-ordinates, and elevations were surveyed by Dearden and Stanton Limited. We understand that these elevations are referenced to Geodetic datum. The locations and co-ordinates of the boreholes are shown on Drawing Nos. 1 and 2; the co-ordinates and elevations are also indicated on the Record of Borehole sheets.

#### **4.0 SUBSURFACE CONDITIONS**

The subsurface conditions were investigated at twenty-three (23) borehole locations, namely Boreholes 1 to 22, and 24. The locations of the boreholes are shown on plan and profile on Drawing Nos. 1 and 2, and are also indicated on the Record of Borehole sheets. Sections of inferred subsurface stratigraphy are also shown on the two drawings.

In general, the subsoils along the proposed bridge alignment consist of a thin veneer of topsoil overlying fill of variable thickness and compactness or consistency. Below the fill are deposits of predominantly sand, silty sand to sandy silt, overlying a silty sand glacial till with frequent cobbles and boulders. Organic deposits such as organic silts, peat and sands with organics were contacted below the fill at locations adjacent to the river. The top surface of the till slopes gently upwards and away from the river, from about Elevations 280 m to 285 m. The entire site is underlain by granite bedrock which was contacted between Elevations 275 m and 281 m. The bedrock surface

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appears to slope gently upwards and away from the river on the north side, and it remains relatively level on the south side of the river except at the south bank where it appears to be higher.

Details of the subsurface conditions encountered in the boreholes are presented on the Record of Borehole sheets. Descriptions of the subsurface conditions encountered in the boreholes are provided in the following sections. Boreholes 3 and 13 are DCPT holes with no sampling and testing.

#### 4.1 Topsoil and Peat

Topsoil was encountered at ground surface in all sampled boreholes except Boreholes 3, 7, 13, 17, 20. The typical thickness varies from 0.1 m to 0.3 m. In Boreholes 12, 16 and 18, a lower stratum of topsoil was encountered within or below the fill, and is up to 0.6 to 0.7 m thick in Boreholes 12 and 18. Within the area immediately to the north of the river where there was once a railway marshalling yard, the topsoil ranges from 0.6 m to 0.9 m in thickness (Boreholes 5, 6 and 21) with evidence of coal fragments present at two locations.

Peat of 0.3 m and 0.4 m in thickness was encountered in Boreholes 7 and 8 at ground surface and below the fill at 1.8 m depth, respectively.

It should be noted that the thickness of the topsoil and occasional peat layers, may vary in between and beyond the borehole locations.

#### 4.2 Fill

Fill was encountered below the topsoil or at ground surface in all sampled boreholes except Boreholes 14 and 19. The fill extends to between 0.7 m and 3.5 m depths. The fill generally consists of sandy silt to silty sand with gravel overlying clayey silt to silty clay, except in Borehole 6 where the sandy and clayey fills are interlayered. There are also organic inclusions within the fill. Within the cohesionless portions of the fill, SPT 'N'-values range from 1 to 35 blows per 0.3 m penetration, indicating very loose to dense conditions, with most values lying within the compact range. 'N'-values obtained within the cohesive portions of the fill typically varies from 4 to 40 blows per 0.3 m penetration, indicating firm to very stiff consistency. These 'N'-values indicate that the fill had experienced variable compactive effort. Figure 7 shows liquid limits and plasticity indices for samples of the cohesive fill.

Four grain size distribution analyses were carried out on representative samples of different types of cohesionless fill and yielded the following results:

Gravel:	0 - 6%
Sand:	25 - 58%
Silt:	35 - 69%
Clay:	1 - 6%

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The grain size distribution curves are shown on Figure 1.

Measured natural moisture contents typically range from approximately 15 to 30%, thus confirming the wetness observed in the fill samples.

#### 4.3 Clayey Silt to Silty Clay

Surficial clayey silt to silty clay was encountered below the fill in Boreholes 21, 22 and 24 at 2.1 m depth, extending to between 2.9 m and 4.1 m depths. In Borehole 14, the clayey silt underlies a sandy silt from 0.7 m to 1.2 m depths. A lower silty clay layer was encountered from 4.6 m to 5.2 m depth in Borehole 24. Measured SPT 'N'-values range from 23 blows to 37 blows per 0.3 m penetration, indicating a very stiff to hard consistency. Atterberg Limits test results indicate that these cohesive deposits have liquid limits of 22 to 30% and plasticity indices of 6 to 10% (see Figure 8). Measured moisture contents vary from 17 to 22%.

#### 4.4 Sand, Silty Sand to Sandy Silt

Deposits of sand, silty sand to occasional sandy silt were encountered below the fill and surficial cohesive soils in all sampled boreholes. The thickness of these soils are typically between 1 m and 3 m. In Boreholes 7, 8 and 12, the sand to silty sand contains organics and occasional gravel. On the south side of the river, SPT 'N'-values obtained within these deposits range from 11 blows to 36 blows per 0.3 m penetration, indicating a typically compact condition with occasional dense layers. On the north side of the river, the 'N'-values are significantly lower generally ranging between 1 blow to 16 blows per 0.3 m penetration. These values indicate that these deposits are in a very loose to compact condition.

Grain size analyses were carried out on three samples of the sand to silty sand with organics and yielded the following results:

Gravel:	0 - 31%
Sand:	49 - 52%
Silt:	18 - 48%
Clay:	0 - 2%

Figure 2 shows the relevant grain size distribution curves.

Grain size analyses were carried out on eight samples of the sandy silt to silty sand with the following results:

Gravel:	0 - 18%
Sand:	22 - 61%
Silt:	13 - 64%
Clay:	0 - 2%

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Figure 3 shows the relevant grain size distribution curves.

One grain size distribution analysis was carried out on a sample of sand with the following results:

Gravel:	0%
Sand:	95%
Silt and Clay:	5%

The grain size distribution curves for the sand is shown on Figure 4.

The above curves indicate that these soils have a broad range of gradation.

#### **4.5 Organic Silt**

A deposit of organic silt was encountered in two boreholes located on the south banks of the river. This soil has a thickness of about 0.9 m in Borehole 10 and about 1.0 m in Borehole 9, and contains decomposed organics. Measured SPT 'N'-values within this deposit range from 3 blows to 12 blows per 0.3 m penetration, indicating soft to stiff consistency. Measured natural moisture contents typically range from 20 to 41%, with the higher values likely associated with zones of higher organic contents.

#### **4.6 Sand and Gravel to Gravelly Sand**

Deposits of sand and gravel to gravelly sand was encountered at several locations. In Boreholes 15, 17 and 19, the sand and gravel was less than 0.5 m in thickness and was contacted immediately below the fill or surficial soils.

In Borehole 2, the sand and gravel was found overlying the till and has a thickness of about 1.5 m. SPT 'N'-values of 39 blows and 48 blows per 0.3 m penetration were measured at this location, indicating a dense state of compactness. Figure 5 shows the grain size distribution of a sample of the sand and gravel, which has 19% gravel, 61% sand, 18% silt and 2% clay.

A gravelly sand layer extending to the termination depth of the Borehole 18 was encountered below the till. 'N'-values range between 47 blows per 0.3 m penetration to greater than 50 blows per 0.14 m penetration, thus indicating a dense to very dense condition. Figure 5 also shows the grain size distribution of a sample of the gravelly sand, which has 59% gravel, 33% sand, 8% silt and clay.

Measured moisture contents generally range from 7 to 13%.

#### 4.7 Heterogeneous Mixture of Silt, Sand and Gravel (Glacial Till)

An extensive stratum of a heterogeneous mixture of sand, silt and gravel (glacial till) was encountered in all sampled boreholes except Boreholes 14 and 2. Frequent cobbles and boulders were encountered or inferred at variable depths within this till. Borehole 14 was put down for the approach embankment, and was terminated upon auger refusal on possible boulders. In Borehole 2, the till could only be inferred by coring through boulders and cobbles which were believed to have resulted in auger refusal.

Where fully penetrated, the thickness of this deposit ranges from about 8 m in Boreholes 17 and 19 located to the south of the river to as little as 2 m in Boreholes 2, 6 and 7. Boreholes 1, 5, 8, 10, 11, 12, 16, 20 and 22 did not fully penetrate the glacial till. The top surface of the till rises from approximately Elevation 281 m at the south river bank to about 285.5 near the south limit of the project area. The change in elevation is from 279.5 m to 284 m, from the north river bank to the north limit of the project area.

Measured SPT 'N'-values within the glacial till vary from 11 blows to greater than 50 blows per 0.3 m penetration, with most values greater than 30 blows. These 'N'-values indicate that the till is in a typically dense to very dense state with some compact layers. In the vicinity immediately to the north of the river, the 'N'-values range between 21 blows to 31 blows per 0.3 m penetration indicating a compact to dense condition.

Grain size distribution analyses were conducted on twenty-four representative samples from this stratum, giving the following grain size measurements.

Gravel:	0 - 48%
Sand:	24 - 40%
Silt:	8 - 56%
Clay:	1 - 6%

The range of grain size distribution are presented on Figure 6.

Measured natural moisture contents vary from 5 to 22% with most values ranging between 10 and 15%.

Attempts were made to carry out DCPT's in all (including Boreholes 3 and 13 where sampling and other in-situ testing was not carried out) but Boreholes 14, 16 and 24. Refusal to cone advance was met at most locations within the upper 1 m to 3 m of the glacial till. In Boreholes 2 and 12, cone refusal was met within the sands and gravel above the till. In Borehole 7, however, the cone fully penetrated the till to encounter refusal on the bedrock.

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#### **4.8 Bedrock**

Bedrock was cored and proven in Boreholes 2, 4, 6, 7, 9, 11, 15, 17, 19, 21 and 24. The cored lengths range between 2 m and 3.5 m. On the south side of the river, the bedrock surface remained practically level at Elevation 275 m except at the location of Borehole 15 where it rises to about Elevation 281 m. On the north side of the river, the bedrock surface rises gently in a northerly direction from about Elevation 277.5 m in Borehole 7 to about Elevation 280 m in Borehole 24.

The bedrock consists of grey to grey-black granite in a massive state with close to moderately close joints. Occasional mica zones were also observed in the rock cores. The joints are largely horizontal to sub-horizontal. In Borehole 2, the rock may be described as a pegmatite which is characterized by its large crystals of up to several centimetres.

The rock core recovery was typically between 80% and 100%. The Rock Quality Designation (R.Q.D.) values, however, generally vary from about 60% up to 100%, indicating fair to excellent rock quality. Occasional values of less than 50% may be attributed to the coring operations and/or the presence of mica zones (i.e. Borehole 19).

#### **4.9 Groundwater Conditions**

Groundwater levels in the open boreholes were observed during drilling and upon completion of each borehole. To permit long term monitoring of groundwater levels at the site, standpipe piezometers were installed in Boreholes 9, 10, 11, 12, 15, 17 and 19 on the south side of the river, and in Boreholes 1, 2, 4, 5, 6, 7, 8 and 24 on the north side of the river. No water level could be measured in Boreholes 11 and 17 as the pipes were buried due to site re-grading carried out after installation. The recorded values are shown on the respective Record of Borehole sheets.

It appears that the groundwater level is largely governed by the "normal" river level of about Elevation 283 m in Magnetawan River. The measured piezometric levels rise slightly from about Elevation 283 m at the river banks to about Elevations 284.5 m to 285 m near the northerly and southerly limits of the project.

It should, however, be pointed out that the groundwater at the site would fluctuate seasonally and can be expected to be somewhat higher during the spring months and in response to major weather events, which would have a direct impact on the river level. The 100-year river level is estimated to be near Elevation 286.5 m.

## 5.0 DISCUSSION AND RECOMMENDATIONS

The proposed Highway 11 realignment will consist of a four lane divided roadway. The median width varies from approximately 50 m near the south abutment to 30 m near the north abutment. The distance between the proposed northbound lane and southbound lane centrelines will range from 45 m to 65 m. The work described in this report is associated with the proposed bridge to carry the proposed northbound lane (NBL) of Highway 11 over the Magnetawan River and Highway 520, and the approach embankments within 20 m of the structure. This bridge is located within the municipal boundaries of the Village of Burks Falls. The existing Highway 11 will become the southbound lane (SBL). It is understood that the proposed structure, which will carry the new NBL, is a 7-span, two-lane (13 m wide) bridge, with span lengths ranging between 31 m and 42 m.

Horizontal and vertical alignments of the proposed NBL of Highway 11 at the bridge site were provided to us in plan and profile drawings by DELCAN. Based on the drawings provided, the bridge will span the river floodplain which encompasses the river flow channel and Highway 520.

For the purpose of this report, the foundation units are labelled from south to north as follows : south abutment, Pier 1, Pier 2, Pier 3, Pier 4, Pier 5, Pier 6 and north abutment (see Drawing Nos. 1 and 2).

In general, the subsoils along the proposed bridge alignment consist of variable fill overlying surficial deposits of silts, sands and gravel, which overlie a heterogeneous mixture of sand, silt and gravel (glacial till). This till contains frequent cobbles and boulders. The project area is underlain by granite bedrock. Near the banks of the river, surficial deposits of sand with organics as well as organic silts are encountered. Immediately adjacent to the north bank of the river where an abandoned rail marshalling yard was located, surficial soils contain coal fragments. The surface of the cohesionless till slopes gently upwards and away from the river, from about Elevations 280 m to 285 m. Refusal to augering and dynamic cone advance were frequently encountered in the cobbles and boulders within the cohesionless till. The bedrock surface to the south of the river remains essentially level at about Elevation 275 m, except at the location of the proposed south abutment where it slopes, or steps, steeply upwards towards south to up to Elevation 281 m. On the north side of the river, the bedrock surface slopes gently upwards in a northerly direction, from about Elevations 275 m to 280 m.

Piezometer readings at the boreholes indicate that the groundwater level within the floodplain varies between Elevations 283 m and 285 m, and is considered to be strongly influenced by the "normal" river level.

## **5.1 Foundations**

### **5.1.1 Spread Footings**

On the south side of the river, competent glacial till is typically present at between 1.5 m and 3.5 m depths except in the vicinity of the northeast portion of Pier 3 (Boreholes 9 and 10, adjacent to the river) where the till is at about 4.4 m depth. The groundwater table is, however, relatively shallow at 2 m depth or less. If spread footings are to be used, extensive dewatering such as the use of pump wells would be required to provide groundwater control during construction. Steel sheet pile cut-off walls are considered impractical since the sheetings would not be able to adequately penetrate the underlying cohesionless glacial till where cobbles and boulders are present. As such, shallow spread footings resting on native soils are not considered feasible for foundation support.

On the north side of the river, competent till is present at depths in excess of 5 m. In view of the extensive excavation and groundwater control requirements, spread footings resting on native soils are also not considered feasible.

If perched abutments are considered in the design, spread footings founded on a compacted Granular 'A' core may be feasible at the north abutment only. Based on existing information, the geometry at the proposed south abutment (maximum grade raise of only 5 m or less) does not appear to allow consideration of spread footings founded on a compacted Granular 'A' core.

### **5.1.2 Driven H-Piles**

In view of the above and since an integral abutment bridge is the preferred option, consideration should be given to supporting the bridge on deep foundations in the form of steel H-piles, driven to practical refusal within the typically dense to very dense glacial till containing frequent cobbles and boulders. At some locations, the driven piles could penetrate through the till into the underlying bedrock. In order to adequately penetrate the typically dense to very dense till with frequent cobbles and boulders, and to seat into bedrock at some locations, a heavier section such as HP 310x110 equipped with appropriate rock points would be suitable for use.

Based on the results of the boreholes, the following Table 1 summarizes the estimated average pile tip elevations that may be assumed for design purposes.

**TABLE 1**

SUPPORT LOCATION	REFERENCE BOREHOLES	ESTIMATED APPROXIMATE PILE TIP ELEVATION (m)
South Abutment	15 16*	281 ± (west side, possibly to bedrock) 281 ± (east side)*
Pier 1	18 17	279.5 ± (west side) 278 ± (east side)
Pier 2	19 20	278 ± (west side) 279 ± (east side)
Pier 3	10 9	277.5 ± (west side) 278 ± (east side)
Pier 4	7 8	277 ± (west side) 276.5 ± (east side)
Pier 5	21* 22*	276 ± (west side)* 278 ± (east side)*
Pier 6	4 3	279 ± (west side) 279 ± (east side)
North Abutment	24 2	280 ± (west side) 280 ± (east side, possibly to bedrock)

**Note: \*Borehole elevations were not available to us at time of report preparation. Elevations are estimated based on available plans and profiles.**

The above estimated pile tip elevations are based on the assumption that the piles would penetrate the compact to dense sands and silts into the very dense zones of the glacial till. Some piles could also fully penetrate the till (with cobbles or boulders) into the bedrock.

### 5.1.3 Resistance to Axial Loads

For HP 310x110 steel H-piles driven to practical refusal within the till or bedrock at or below the elevations shown in Table 1 above, the following axial resistances may be assumed for design.

Factored Axial Resistance at Ultimate Limit States (U.L.S.)	=	1,600 kN
Geotechnical Resistance at Serviceability Limit States (S.L.S.)	=	1,150 kN

The above values were selected in view of the fact that some premature pile refusals may be encountered at elevations higher than those shown in Table 1.



In accordance with MTO standard practice, the piles should be driven to about 2 m to 3 m above the elevations recommended in Table 1 above, after which driving should be monitored and controlled by a recognized pile driving formula, such as the Hiley formula. The estimated ultimate resistance of the piles driven to practical refusal within the very dense cohesionless glacial till, at about the elevations quoted above, is approximately 3,200 kN. The piles should be driven with a suitably heavy hammer capable of delivering a rated capacity of at least 50 kJ per blow. The energy should, however, be restricted to not more than 60 kJ per blow.

Cobbles and/or boulders were inferred or encountered within the very dense glacial till in most boreholes. In view of this and the possibility that the piles will penetrate through the till, the pile tips should be equipped with driving shoe such as reinforcing plates (OPSD 3301.00) and/or Oslo points as per MTO Standards (OPSD 3304.00) in order to be adequately seated into bedrock. Consideration may be given to pre-augering along the pile vertical alignment to an elevation of about 2 m above the pile tip elevations (in Table 1) prior to driving the piles.

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

If the driven pile encounters refusal above the recommended elevation, the Geotechnical Engineer familiar with the findings of this report and appointed by the Contract Administrator should be notified immediately.

During the driving process, piles which have already been driven should be monitored to determine if they are heaving due to effects of driving adjacent piles. If this phenomenon occurs, the affected piles should be re-driven. It is recommended that not less than 15% of the piles and at least three piles in each foundation support element be re-struck one to two days after initial installation, as a precaution against relaxation. If relaxation occurs, then all piles in that foundation element should be re-tapped.

It is possible that some of the piles may penetrate to one to two metres below the estimated tip elevations and this aspect should be taken into consideration when ordering piles.

The geotechnical resistance at Serviceability Limit States (S.L.S.) is dependent on the settlement of the pile group and, therefore, is governed by the size of the pile group. The pile group configuration is currently not available to us. Provided that the piles are designed and installed as recommended above, it is considered that the quoted S.L.S. value corresponds to no more than 25 mm of settlement for the pile group. We will confirm the estimated settlement once information on the pile group configuration is known.

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#### 5.1.4 Resistance to Lateral Loads

Laterally applied loads on piles can be resisted geotechnically by the driven piles through passive pressure developed in the soil in which the piles are embedded. The pile tip elevations recommended above indicate that the piles will be in the order of 7 to 10 m in length. Lateral pile resistance may be considered in accordance with Section 6-9.8.1 of the O.H.B.D.C., 3<sup>rd</sup> Edition.

The recommended horizontal resistances for a HP310x110 pile at this site are as follows:

Factored Horizontal Resistance at U.L.S.	=	120 kN
Horizontal Resistance at S.L.S.	=	50 kN

In accordance with MTO requirements (MTO Structural Office Standard), piles for integral abutments require a 3 m long flexible zone. In essence the current MTO standard for the flexible zone consists of an annular space in between two concentric CSP's. One of the CSP's surrounds the H-pile (i.e. has a diameter slightly greater than the pile width), while the second CSP has a somewhat larger diameter; typically 0.6 m for a 310 mm H-pile. After the pile is driven, the space between the H-pile and the inner CSP is filled with cement bentonite or coarse sand.

If conventional abutments on pile groups are to be built instead of integral abutments, then the unbalanced horizontal forces could be partially resisted by battered piles. Conventional pile groups at the piers should also have battered piles to resist horizontal forces.

For lateral soil-pile interaction analysis, the horizontal subgrade reaction to the pile can be calculated from the expression:

$$k_s = n_h \times z/d$$

where  $k_s$  = coefficient of horizontal subgrade reaction  
 $n_h$  = coefficient related to soil density as given in Table 2  
 $d$  = pile width  
 $z$  = depth

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

**TABLE 2**

AREA/REFERENCE BOREHOLE NO.	APPLICABLE DEPTH FROM EXISTING GROUND SURFACE (ELEVATION)	SOIL TYPE	BULK UNIT WEIGHT (kN/m <sup>3</sup> )	ANGLE OF INTERNAL FRICTION ( $\phi$ ) DEGREES	RECOMMENDED $n_h$ VALUE (MN/m <sup>3</sup> )
<b>South Abutment</b>					
15, 16	1 - 2 m (287 - 286 m)	compact/stiff Sand/Clayey Silt FILL	20	29	6
	2 - 3 m (286 - 285 m)	compact SILTY SAND/ SAND AND GRAVEL	20	31	5
	3 - 7 m (285 - 281 m)	dense to very dense SILTY SAND GLACIAL TILL (cobbles & boulders)	22	38	11
<b>North Abutment</b>					
2, 24	1 - 3 m (287 - 285 m)	firm to very stiff Clayey Silt to Silty Clay FILL	20	29	6
	3 - 4 m (285 - 284 m)	compact SANDY SILT TO SILTY SAND	20	30	5
	4 - 8 m (284 - 280 m)	dense to very dense SAND & GRAVEL/ SILTY SAND GLACIAL TILL (cobbles & boulders)	22	38	11
<b>Pier 1</b>					
17, 18	1 - 2 m (285 - 284 m)	loose to compact/firm Sandy/Clayey Silt FILL	20	29	4
	2 - 10 m	dense to very dense SILTY SAND GLACIAL TILL/ GRAVELLY SAND (cobbles & boulders)	22	38	11
<b>Pier 2</b>					
19, 20	1 - 2 m (284 - 283 m)	firm/compact CLAYEY SILT/Sand FILL/SILTY SAND	20	29	5
	2 - 10 m	dense to very dense SILTY SAND GLACIAL TILL (cobbles & boulders)	22	38	11

**TABLE 2 (continued)**

AREA/REFERENCE BOREHOLE NO.	APPLICABLE DEPTH FROM EXISTING GROUND SURFACE (ELEVATION)	SOIL TYPE	BULK UNIT WEIGHT (kN/m <sup>3</sup> )	ANGLE OF INTERNAL FRICTION ( $\phi$ ) DEGREES	RECOMMENDED $n_h$ VALUE (MN/m <sup>2</sup> )
<b>Pier 3</b>					
9, 10	1 - 2 m (284 - 283 m)	very loose Sandy Silt FILL	20	28	2
	2 - 3 m (283 - 282 m)	soft ORGANIC SILT	17	--	--
	3 - 4 m (282 - 281 m)	compact SAND	20	30	5
	4 - 10 m (281 - 275 m)	dense to very dense SILTY SAND GLACIAL TILL (cobbles & boulders)	22	38	1
<b>Pier 4</b>					
7, 8	1 - 6 m (284 - 279 m)	very loose to loose Sand FILL/ SAND/SILTY SAND (organics)	19	27	1
	6 - 8 m (279 - 277 m)	compact to very dense SILTY SAND GLACIAL TILL (cobbles & boulders)	21	36	9
<b>Pier 5</b>					
21, 22	1 - 2 m	firm Clayey Silt FILL	20	29	4
	2 - 3 m	very stiff to hard CLAYEY SILT/SILTY CLAY	21	31	6
	3 - 6 m	very loose to loose SAND	18	28	2
	6 - 10 m	dense to very dense SILTY SAND GLACIAL TILL (cobbles & boulders)	22	38	11
<b>Pier 6</b>					
3, 4	1 - 2 m (286 - 285 m)	soft to firm Silty Clay FILL	20	29	4
	2 - 6 m (285 - 281 m)	dense to loose SAND	20	30	5
	6 - 8 m (281 - 279 m)	compact SILTY SAND GLACIAL TILL (cobbles & boulders)	21	34	7

**Notes:** It is anticipated that all surficial organics and other deleterious materials within the footprints of the abutments and approaches will be sub-excavated.

Where conventional piled foundations are used, there is likely more than one row of piles. In this instance, group action for lateral loading should be considered when the pile spacing in the direction of loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction in the direction of loading by a reduction factor R as follows:

PILE SPACING IN DIRECTION OF LOADING $d$ = PILE DIAMETER	SUBGRADE REACTION REDUCTION FACTOR R
8d	1.00
6d	0.70
4d	0.40
3d	0.25

## 5.2 Lateral Earth Pressures

Backfill behind abutments and retaining walls should consist of non-frost susceptible, free-draining granular materials in accordance with MTO 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., 3<sup>rd</sup> Edition. For design purposes, the following parameters (unfactored) can be used.

### Compacted Granular 'A'

Unit Weight = 22 kN/m<sup>3</sup>

Coefficient of Lateral Earth Pressures:

$K_a = 0.27$  (active condition)

$K_o = 0.43$  (at-rest condition)

### Compacted Granular 'B'

Unit Weight = 21 kN/m<sup>3</sup>

Coefficient of Lateral Earth Pressures:

$K_a = 0.31$  (active condition)

$K_o = 0.47$  (at-rest condition)

The above design parameters assume level ground surface and backfill behind the retaining structure.

.../...

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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., 3<sup>rd</sup> 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 of the O.H.B.D.C., 3<sup>rd</sup> 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 moderate to high appearance.

### **5.3 Approach Embankments**

The existing grade at the south abutment location is at about Elevation 289 m, increasing to Elevation 292 m some 20 m to the south. The proposed grade is at about Elevation 294 m at the south approach and abutment locations, resulting in a required south approach embankment of between 2 m and 5 m in height.

Boreholes 15 and 16 show that the surficial portion of the subgrade at the south abutment location (approximate Station 19+098) consists of loose to dense sandy and stiff clayey fill, which was not encountered in Borehole 14 located at approximate Station 19+080 some 18 m south of the abutment. Underlying the fill are deposits of compact sand to silty sand which is underlain by dense to very dense cohesionless glacial till at about 2 m depth.

The height of the south approach embankment will decrease from 5 m at the abutment to zero at about Station 19+068. Embankments of accepted earth fill of up to 5 m in height, with a side slope inclination of 2 horizontal to 1 vertical (2H:1V), would be stable against deep-seated (i.e. foundation) failures, provided that the subgrade is properly prepared by removing all surficial topsoil, loose existing fill, organic and otherwise unsuitable materials as per current MTO Standards before placing the new fill. Appropriate benching at the sloping subgrade level may be considered to allow the new fill to key into the existing slope.

.../...

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The existing south forward slopes with a steepest inclination of 2H:1V are stable against deep seated failures. Embankment loading currently anticipated at the south approach and abutment would not result in instability, provided that the subgrade preparation is carried out as outlined above.

The existing grade is at about Elevation 288 m between Station 19+357(north abutment) and Station 19+380, increasing to 289 m some 20 m further north. The proposed grade is at about 298 m at the north abutment and approach locations, resulting in a required north approach embankment of up to 10 m in height.

Boreholes 2 and 24 show that the surficial portion of the subgrade at the north abutment location (approximate Station 19+357) consists of loose to compact sandy fill overlying firm to stiff clayey fill; the clayey fill was not encountered in Borehole 1 located at approximate Station 19+376 some 20 m north of the abutment. Underlying the fill are variable deposits of compact to dense, sandy silt, sand to sand and gravel. These soils are underlain by dense to very dense cohesionless glacial till at below about 4.5 m depth.

The height of the north approach embankment will range from 8 m to 10 m between the north abutment and Station 19+388 ±. Beyond this location, the height reduces uniformly to zero some 25 m further north. Embankments of conventional fill of up to 10 m in height, with a side slope inclination of 2 horizontal to 1 vertical, would be stable against deep-seated (i.e. foundation) failures, provided that the subgrade is properly prepared by removing all surficial topsoil, loose fill, organic and otherwise unsuitable materials as per MTO Standards before placing the fill. A mid-height bench of 2 m in width should be provided as per current MTO practice, which requires provision of benches for slopes higher than 6 m.

The existing north forward slopes are stable against deep seated failures. Embankment loading currently anticipated at the north approach and abutment would not result in instability, provided that the subgrade preparation is carried out as outlined above.

Assuming properly compacted, acceptable inorganic earth fill material, 2 horizontal to 1 vertical (2H:1V) side slopes can be used throughout. The berm gradient should be sloped (say 1V:20H) to drain away from the embankment. 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 1H:1V from the toe of the proposed embankment as depicted by Figure 9. The average thickness of the unsuitable soils to be stripped can be assumed to be about 1.0 m for both approaches. After stripping, the exposed subgrade should be inspected, approved and properly compacted from the surface under the supervision of qualified personnel, using a suitably heavy compactor.

Provided that all surficial organic, loose existing fill and otherwise unsuitable materials are removed and the subgrade is properly compacted from the surface as detailed above, the total settlement of the foundation materials (i.e. not including the settlement of the embankment material under its own weight) should be in the order of 25 mm at the south approach and abutment locations, and 50 mm at the north abutment and approach locations. This settlement should be substantially completed during construction and within one month after placing the embankment fill to its full height. Post construction settlement should be negligible. These settlements are considered acceptable and will not necessitate preloading or surcharging.

Piezometric measurements indicate that water levels are currently at more than 1 m below existing ground surface. Once stripping is completed, however, the groundwater at some locations will be within less than 1 m below ground surface. We do not anticipate major problems due to groundwater seepage, but care should be exercised to minimize disturbance to the subgrade during subgrade preparation and fill placement for the construction of the embankments.

The fill materials used for construction of the embankment should consist of approved, clean earth fill (e.g. Select Subgrade Materials - OPSS 1010). A majority of the fill will have to be imported for this purpose. 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 Maximum Dry Density. The degree of compaction within the top 0.6 m of the fill (i.e. the subgrade immediately beneath the granular sub-base) should be increased to 98%. The selection, placement and compaction of the fill should be carried out under geotechnical control.

The settlement of the embankment fills prepared as described above should not exceed 30 mm and 50 mm for both the south and north approach embankments, respectively. The time rate of settlement of the fill making up the embankment will depend on the material used for construction and for granular fills it should substantially be completed during the construction and within a few weeks thereafter (i.e. should be essentially elastic). Clayey fills can, on the other hand, be expected to consolidate over a longer period of time. It should also be pointed out that these estimated settlements will be in addition to the foundation settlements estimated earlier.



## 5.4 Excavation and Groundwater Control

All excavations should be carried out in accordance with the latest edition of the Ontario Occupational Health and Safety Act and its regulations.

Excavation for pile cap construction at the piers and flexible zone construction at the south integral abutment would be extended through fill into the native silts, sands, gravel and glacial till below the groundwater table. Temporary unsupported excavation side slopes not steeper than 1 horizontal to 1 vertical (1H:1V) through the fill would be stable provided that the excavation base is above the groundwater table. Slopes forming through the surficial fill will require flatter inclinations, say 1.5H:1V. Pumping from properly filtered sumps will be required to control water seepage due to perched water and surface runoff. Below the groundwater table, substantial inflow into the temporary excavations through cohesionless soils should be expected. Sump pumping alone will not be sufficient to maintain a reasonably dry excavation to facilitate pile driving and pile cap construction. Local dewatering by means of eductors and/or deep wells may be required. Interlocking sheet pile walls may not be a feasible option for groundwater control as the driven sheetings may encounter difficulties penetrating the cobbles or boulders within the glacial till.

Due to the close proximity of the river flow channel to the proposed Piers 3 and 4, excavation below the groundwater (river) level, if required, will be difficult and will likely require dewatering as well as interlocking steel sheetpile cutoff walls founded well within the cohesionless glacial till, or within the bedrock on the north bank of the river.

Care should be exercised to minimize disturbance to the silty subgrade during excavation.

## 5.5 Construction Comments

Borehole 7 reveals that the thickness of the glacial till is only about 2 m and that the till is in a dense state. At this location, it is anticipated that the sheetings would fully penetrate the till into the underlying bedrock. Relatively hard driving should be expected to penetrate the dense to very dense till and appropriate weight of sheet piling equipped with driving shoes should be used. The installation of the sheetings may have to involve trenching within the till where cobbles or boulders are present. Due to the close proximity of the river and the fact that the cohesionless till is not impervious, it is anticipated that the interface between the sheetings and the till/rock have to be sealed with mass concrete to minimize seepage.

Erosion and scour protection should be provided to the embankment and river slopes, both during construction and permanently. Such measures of protection may include suitably graded rip-rap and/or armour stone, and should be designed by a qualified professional.

## 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 is required for frost protection of foundations.

## 6.0 CLOSURE

We recommend that once the details of the structure and approaches are finalized, our recommendations should be reviewed for their specific applicability.

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

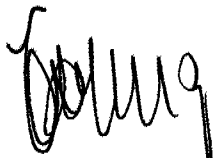
Sincerely,



Sydney Pang, P. Eng.



Ramon Miranda, P. Eng.



Eric Chung, P. Eng.,  
Designated MTO Contact.

SP/dma

## APPENDIX A

## **AGRA**

### **LIMITATIONS OF REPORT**

This report contains the findings of our geotechnical investigation, together with our recommendations and comments. These recommendations and comments are based on factual information and are intended only for use of the design engineers. The number of testholes may not be sufficient to determine all the factors that may affect construction methods and costs. 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 anticipated construction conditions are also discussed, but only to the extent that they may influence design decisions. Construction methods discussed, however, express our opinion only and are not intended to direct the contractors on how to carry out the construction. Contractors should also be aware that the data and their interpretation presented in this report may not be sufficient to assess all the factors that may have an effect upon the construction.

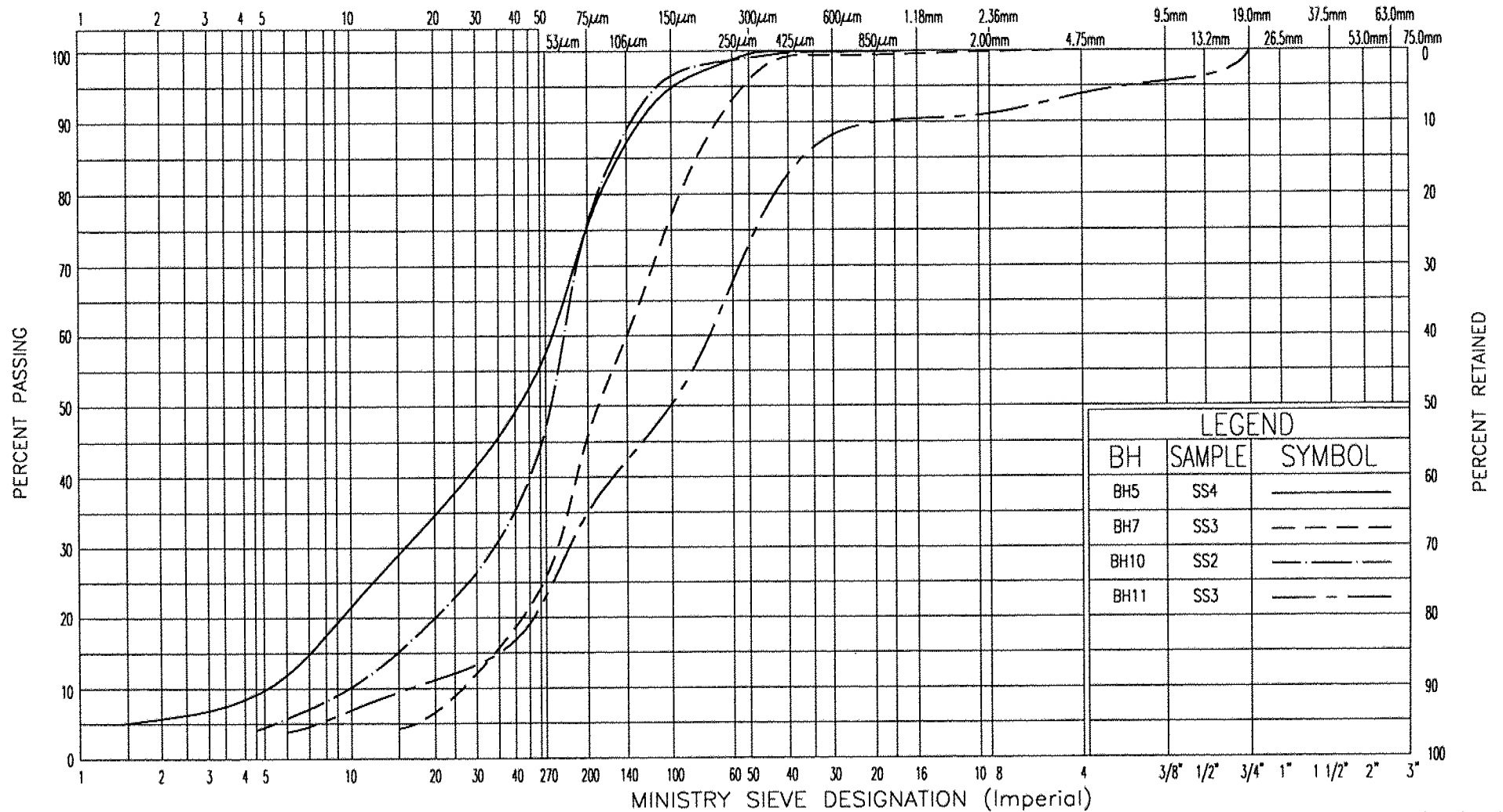
## FIGURES

# UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse

GRAIN SIZE IN MICROMETERS

MINISTRY SIEVE DESIGNATION (Metric)



AGRA

GRAIN SIZE DISTRIBUTION  
FILL

FIG No 1

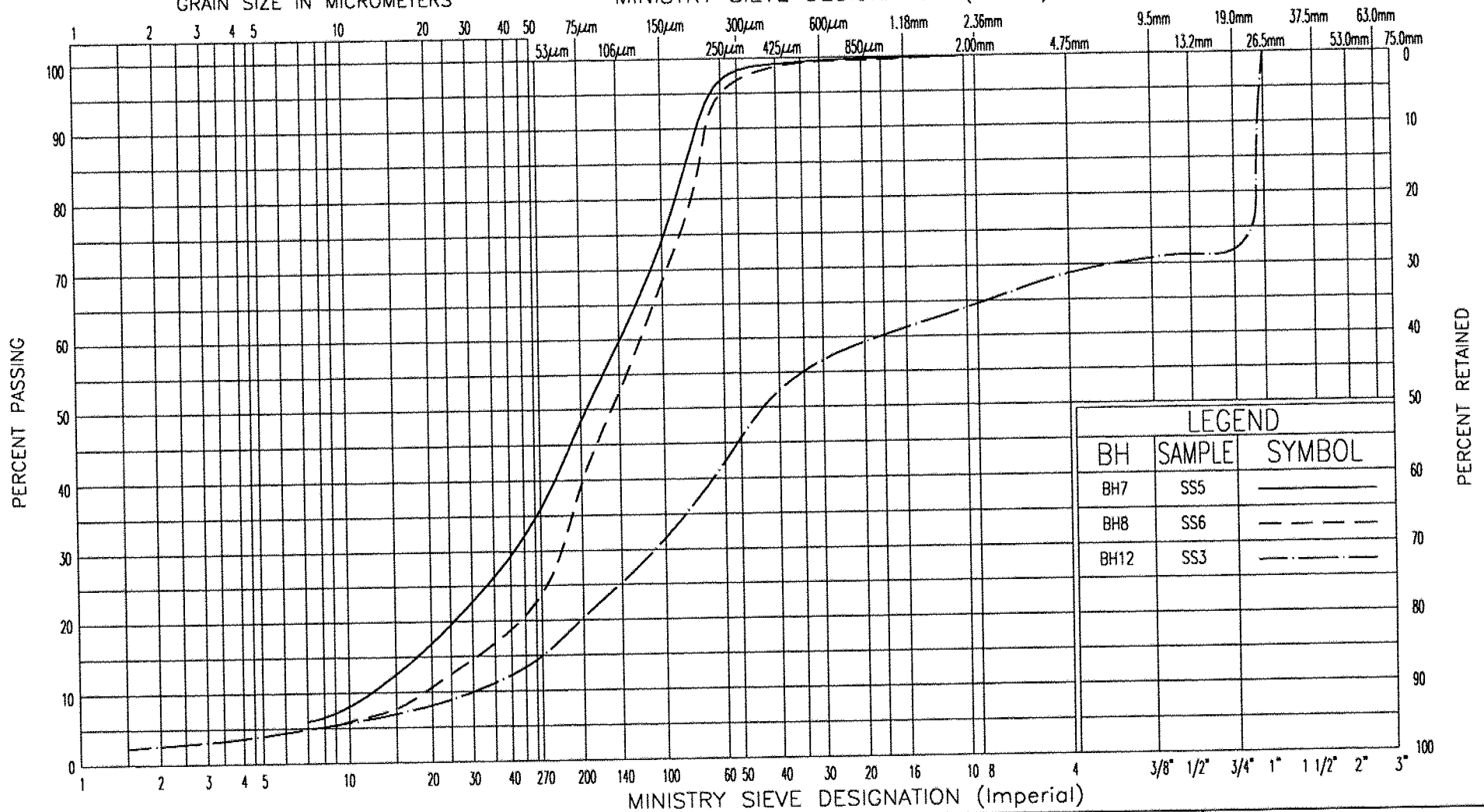
W P 473-93-00

# 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
BH7	SS5	————
BH8	SS6	- - - - -
BH12	SS3	- . - . -



GRAIN SIZE DISTRIBUTION  
SAND TO SILTY SAND with Organics

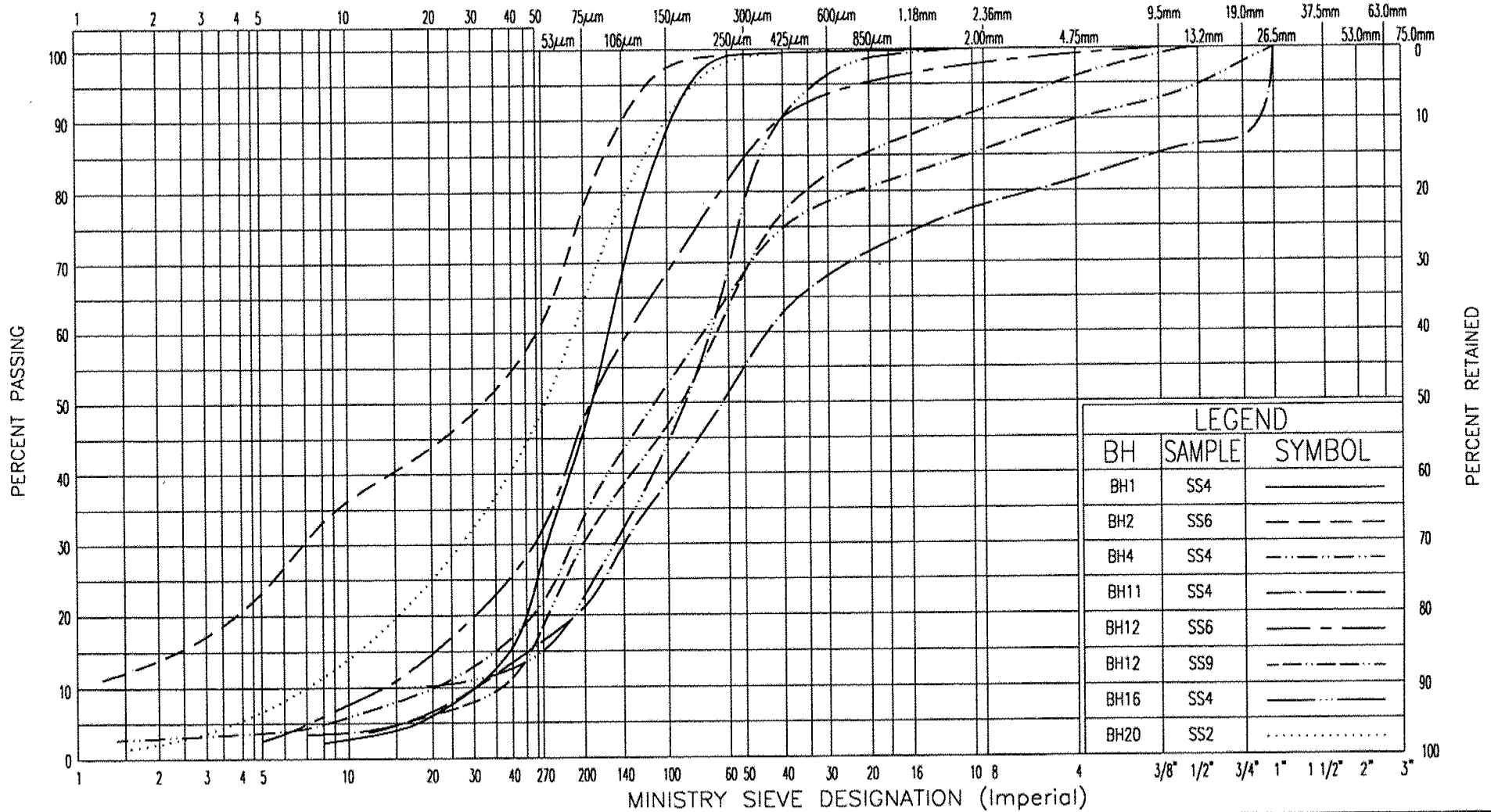
FIG No 2  
W P 473-93-00

# UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse

GRAIN SIZE IN MICROMETERS

MINISTRY SIEVE DESIGNATION (Metric)



GRAIN SIZE DISTRIBUTION  
SANDY SILT TO SILTY SAND

FIG No 3

W P 473-93-00



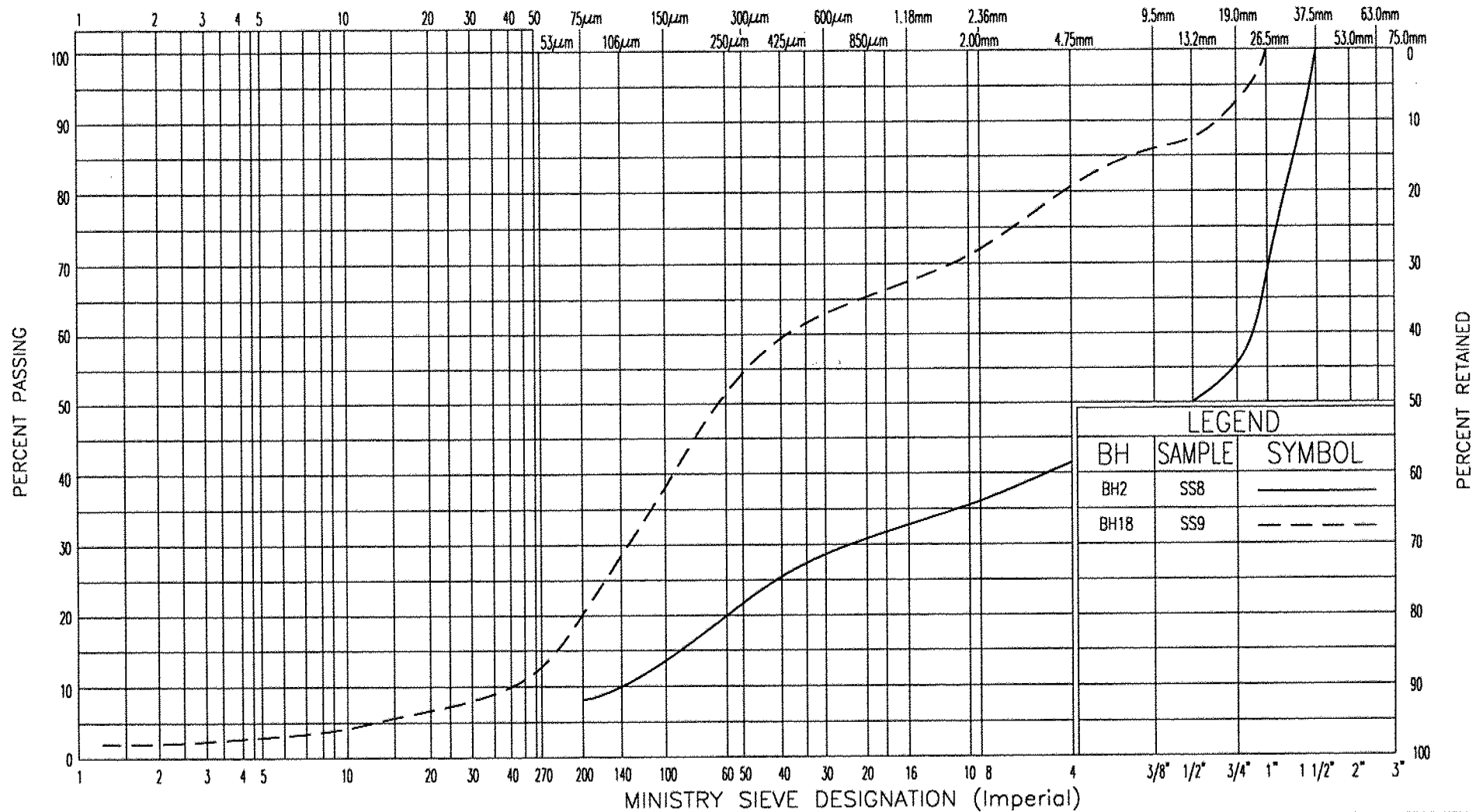


# UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse

GRAIN SIZE IN MICROMETERS

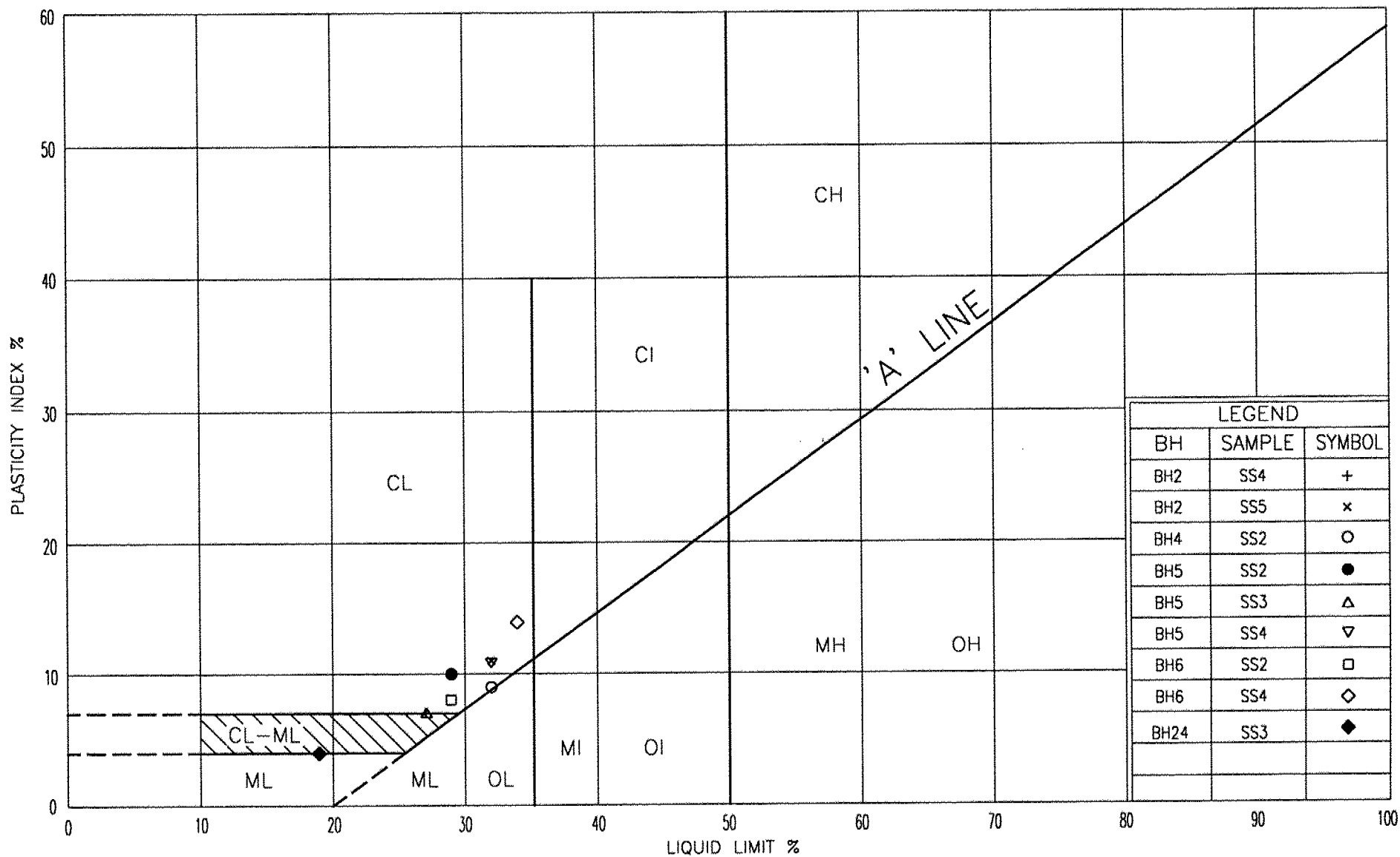
MINISTRY SIEVE DESIGNATION (Metric)



GRAIN SIZE DISTRIBUTION  
SAND & GRAVEL TO GRAVELLY SAND

FIG No 5  
W P 473-93-00



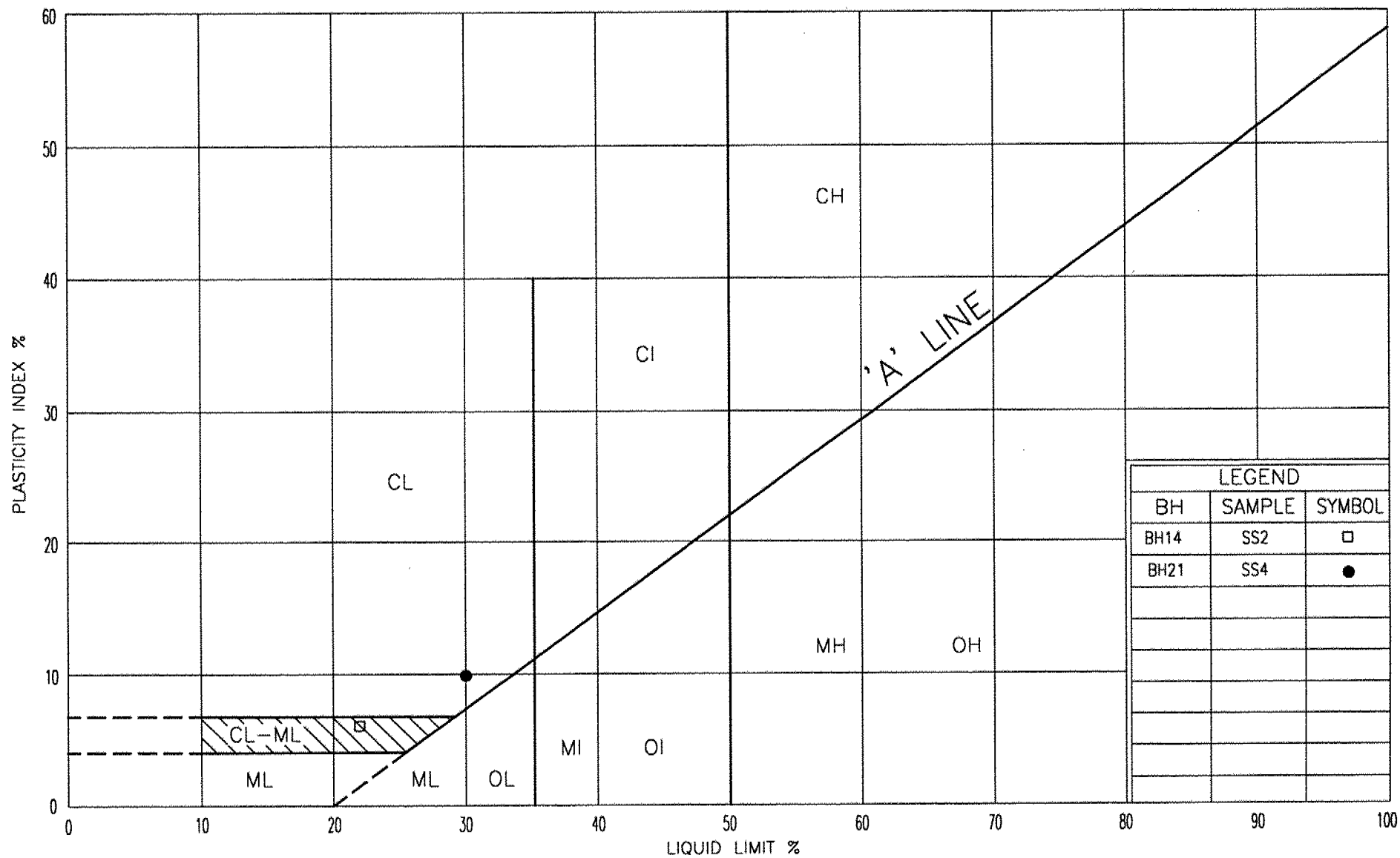


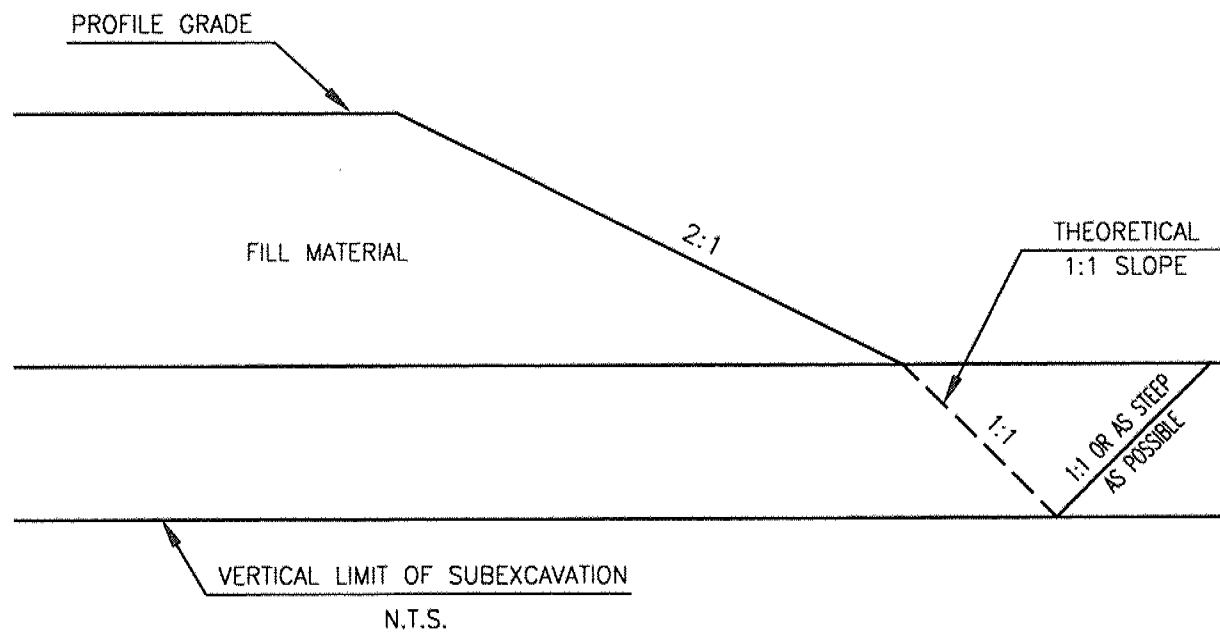
AGRA

PLASTICITY CHART  
Silty Clay to Clayey Silt FILL

FIG No 7

W P 473-93-00





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REMOVAL OF UNSUITABLE SOILS  
FROM BENEATH APPROACH FILLS  
( N.T.S. )

FIGURE No 9

W P 473-93-00

**ENCLOSURES**

# RECORD OF BOREHOLE No 1

1 OF 1

METRIC

W.P. 473-93-00 LOCATION N 5053018.1 E311419.7 ORIGINATED BY MA  
DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augering COMPILED BY AD  
DATUM Geodetic DATE 28 May 1999 CHECKED BY SP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
288.6								20 40 60 80 100						
0.0	0.15m TOPSOIL		1	SS	7		288	○ UNCONFINED + FIELD VANE						
	brown Silty Sand FILL trace Gravel, rootlets loose to compact moist		2	SS	19		287	● QUICK TRIAXIAL x LAB VANE						
287.2								20 40 60 80 100						
1.4	brown SANDY SILT with occasional Clay & Silt seams compact wet		3	SS	12		286							
			4	SS	16		285							
285.3			5	SS	16		284							
3.3	brown to grey SAND with Gravel loose wet		6	SS	7									
284.2														
4.4	grey HETEROGENEOUS MIXTURE of SAND, SILT&GRAVEL (GLACIAL TILL) very dense, wet		7	SS	SUB									
283.7														
4.9	END of BOREHOLE													
	AUGER REFUSAL ON BOULDER													
280.1														
8.5	END of DCPT													
	DCPT conducted 1.0m south  Water Level in Piezometer: July 9/99: 4.1m depth Sept 3/99: 4.3m depth Elev. 284.3m													



RECORD OF BOREHOLE No 2

1 OF 1

METRIC

W.P. 473-93-00 LOCATION N 5053001.8 E311426.4 ORIGINATED BY MA  
 DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augering COMPILED BY AD  
 DATUM Geodetic DATE 28 May 1999 CHECKED BY SP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE 20 40 60 80 100	PLASTIC LIMIT w <sub>p</sub> NATURAL MOISTURE CONTENT w LIQUID LIMIT w <sub>L</sub> WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
288.0	0.15m TOPSOIL brown to grey Organic stained Silty Sand FILL trace Gravel, rootlets Clay pockets, decomposed Organics stiff to firm damp		1	SS	11		287				
286.6			2	SS	6		286				
1.4			3	SS	4		285				
	brown to grey Silty Clay FILL trace to some Organics trace Gravel firm to hard damp		4	SS	40		284				
284.7			5	SS	37						0 22 63 15
3.3			6	SS	13						
283.6	brown SANDY SILT trace Clay compact wet		7	SS	48						
4.4			8	SS	39						59 33 (8)
282.0			9	RC			282				Auger Refusal @5.8m on boulders
6.0	HETEROGENEOUS MIXTURE of SAND, SILT & GRAVEL (GLACIAL TILL) frequent Cobbles & Boulders (INFERRED)		10	RC			281				
280.2			11	RC			280				RC11: REC=100% R.Q.D.=75%
7.8			12	RC			279				RC12: REC=96% R.Q.D.=31%
	GRANITE BEDROCK (PEGMATITE) massive, closely to moderately closely jointed		13	RC			278				RC13: REC=92% R.Q.D.=17%
277.2											
10.8	END of BOREHOLE  Water Level in Piezometer: July 9/99: 0.9m depth Sept 3/99: 1.3m depth Elev. 286.7m										

RECORD OF BOREHOLE No 3

1 OF 1

METRIC

W.P. 473-93-00 LOCATION N 5052964.8 E 311426.5 ORIGINATED BY MA  
 DIST 52 HWY 11 BOREHOLE TYPE Dynamic Cone Penetration Test (DCPT) COMPILED BY AD  
 DATUM Geodetic DATE 28 May 1999 CHECKED BY SP

SOIL PROFILE		SAMPLES				GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES									
287.5 0.0	NO SAMPLING & TESTING						287						GR SA SI CL	
							286							
							285							
							284							
							283							
							282							
281.5 6.1	END of DCPT													

RECORD OF BOREHOLE No 4

1 OF 1

METRIC

W.P. 473-93-00 LOCATION N 5052951.7 E311416.0 ORIGINATED BY MA  
 DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augering COMPILED BY AD  
 DATUM Geodetic DATE 27 May 1999 CHECKED BY SP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
287.1	0.1m TOPSOIL		1	SS	12		287	20	40	60	80	100		
0.0	gray/brown Silty Clay FILL Organic stained stiff to firm moist		2	SS	13		286							
284.9	with Sand		3	SS	7		285							
2.2	Silty trace Gravel		4	SS	36		284							10 57 31 2
	brown SAND wet		5	SS	34		283							
	dense		6	SS	11		282							
	compact		7	SS	5		281							
	loose		8	SS	8		280							
281.2	gray HETEROGENEOUS MIXTURE of SAND, SILT&GRAVEL (GLACIAL TILL) compact to very dense wet		9	SS	24		279							12 71 16 1
5.9			10	SS	50/14		278							Auger Refusal @7.6m on cobble & boulders
279.0	Cobbles		11	RC			277							RC11: REC=100% R.Q.D.=100%
8.1	GRANITE BEDROCK massive, closely to moderately closely jointed		12	RC			276							RC12: REC=100% R.Q.D.=88%
			13	RC										RC13: REC=100% R.Q.D.=79%
275.9	END of BOREHOLE													
11.2	DCPT conducted 1.0m north  Water Level in Piezometer: July 9/99: 2.8m depth Sept 3/99: 3.1m depth Elev. 284.0m													

RECORD OF BOREHOLE No 5

1 OF 1

METRIC

W.P. 473-93-00 LOCATION N 5052933.2 E311423.3 ORIGINATED BY MA  
 DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augering COMPILED BY AD  
 DATUM Geodetic DATE 27 May 1999 CHECKED BY SP

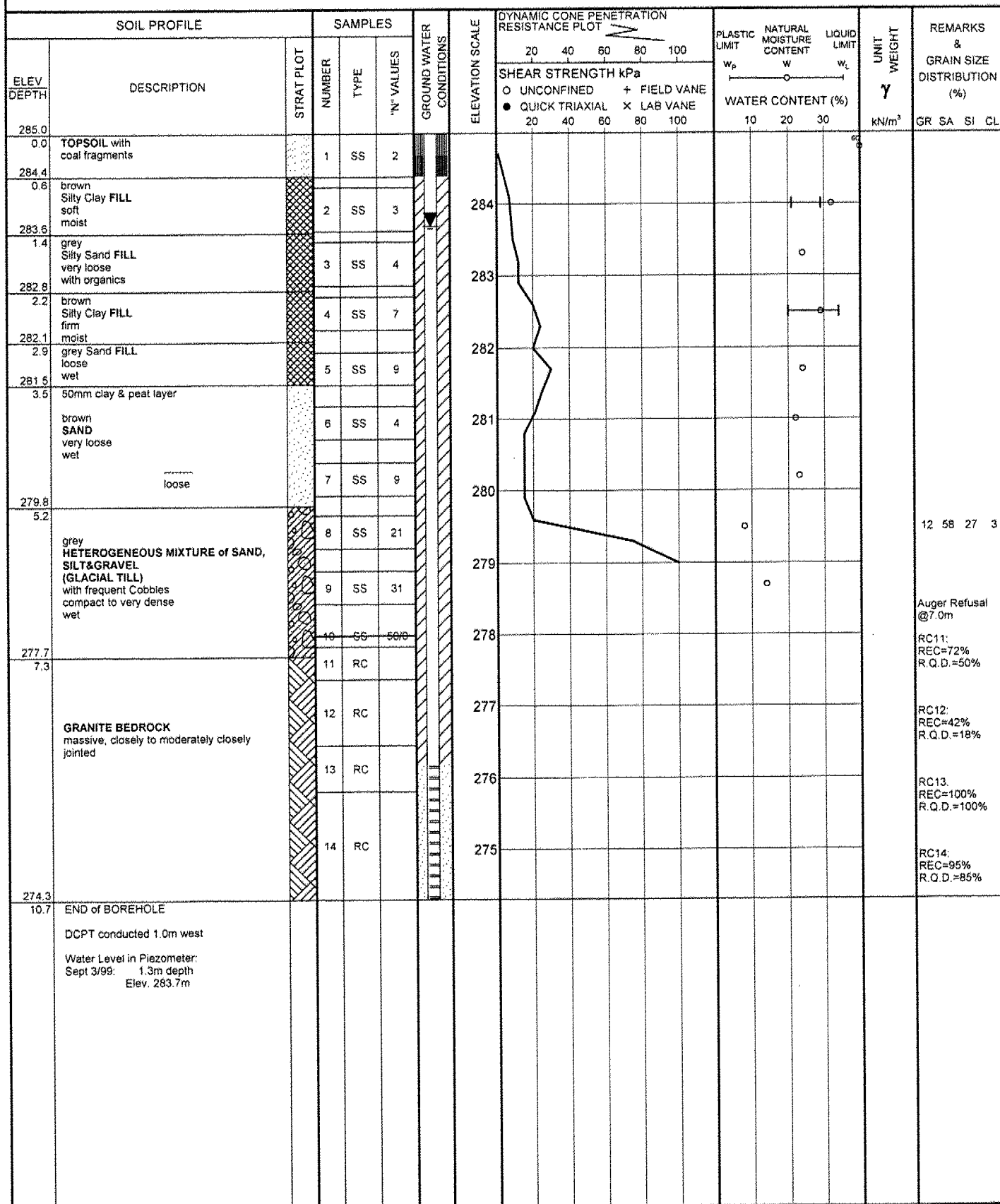
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE 20 40 60 80 100	PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES									
285.9 0.0	TOPSOIL		1	SS	4									
285.3 0.6	grey Clayey Silt FILL Organic stained stiff to hard moist		2	SS	9		285							
			3	SS	40		284							
	Sandy		4	SS	14		283							0 24 70 6
283.0 2.9	grey SILTY SAND trace decomposed Organics		5	SS	11		282							
282.3 3.6	compact, wet		6	SS	6		281							
	brown SAND loose wet		7	SS	5									
280.5 5.4	grey HETEROGENEOUS MIXTURE of SAND, SILT & GRAVEL		8	SS	60/23									18 53 29 0
280.2 5.7	(GLACIAL TILL) very dense END of BOREHOLE													
	AUGER REFUSAL ON BOULDER													
277.7 8.2	END of DCPT													
	DCPT test conducted 1.0m north													
	Water Level in Piezometer: July 9/99: 2.0m depth Sept 3/99: 2.4m depth Elev. 283.5m													

**RECORD OF BOREHOLE No 6**

1 OF 1

**METRIC**

W.P. 473-93-00 LOCATION N 5052911.9 E311432.6 ORIGINATED BY MA  
DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augering COMPILED BY AD  
DATUM Geodetic DATE 29 May 1999 CHECKED BY SP

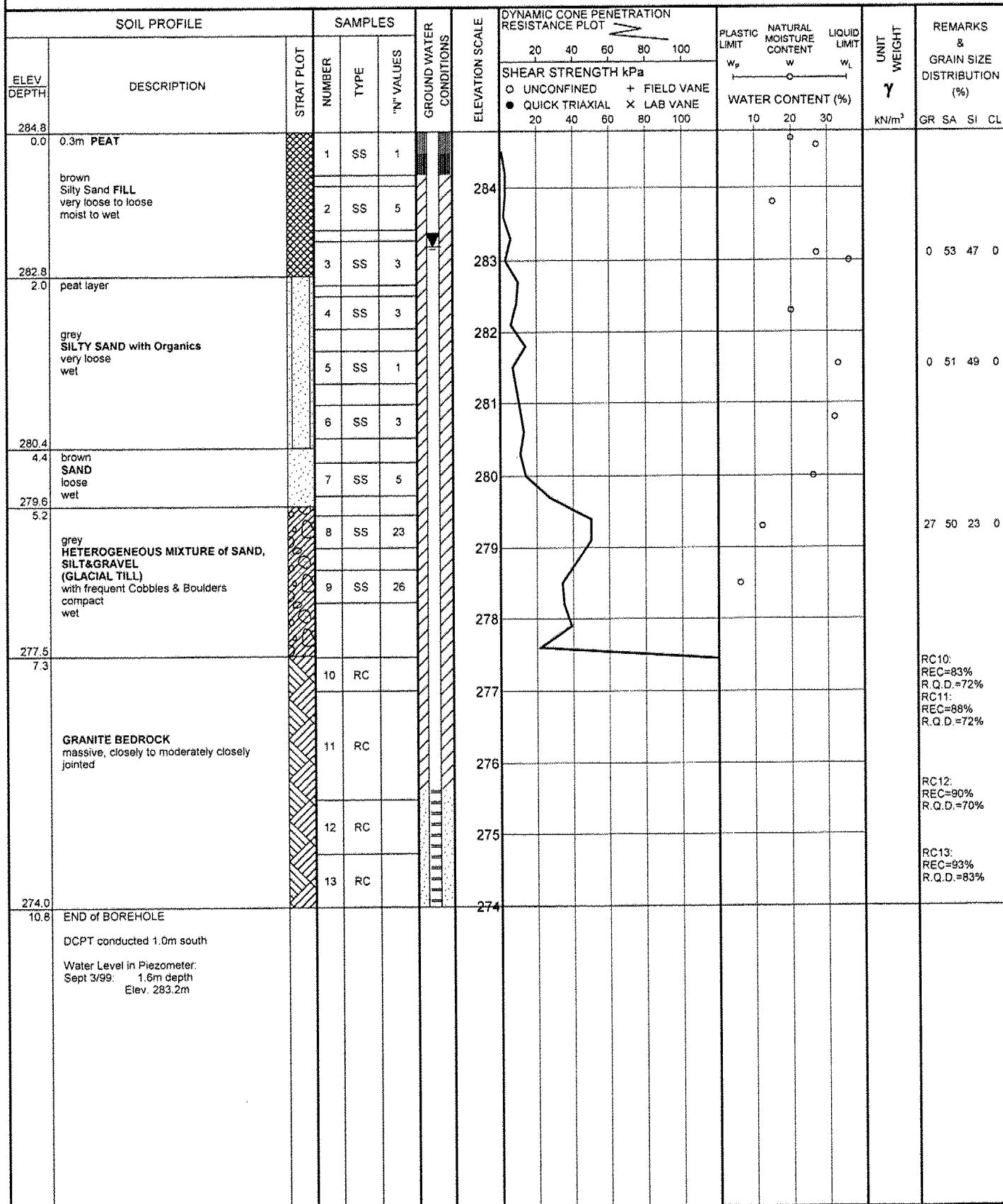


RECORD OF BOREHOLE No 7

1 OF 1

METRIC

W.P. 473-93-00 LOCATION N 5052883.4 E311421.7 ORIGINATED BY MA  
 DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augering COMPILED BY AD  
 DATUM Geodetic DATE 30 May 1999 CHECKED BY SP



RECORD OF BOREHOLE No 8

1 OF 1

METRIC

W.P. 473-93-00 LOCATION N 5052893.1 E311433.0 ORIGINATED BY MA  
 DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augering COMPILED BY AD  
 DATUM Geodetic DATE 30 May 1999 CHECKED BY SP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
284.7								20 40 60 80 100						
0.0	TOPSOIL with coal fragments		1	SS	2			○ UNCONFINED + FIELD VANE						
284.1								● QUICK TRIAXIAL × LAB VANE						
0.6	grey Sand FILL some Organics, Gravel very loose wet		2	SS	4			20 40 60 80 100						
282.9			3	SS	2									
1.8	PEAT													
282.5			4	SS	2									
2.2	grey SAND with Organics very loose wet		5	SS	3									
281.1														
3.6	grey SILTY SAND occasional Organic layers loose wet		6	SS	5									0 60 40 0
			7	SS	9									
279.5			8	SS	38									19 56 25 0
5.2	grey HETEROGENEOUS MIXTURE of SAND, SILT & GRAVEL (GLACIAL TILL) with frequent Cobbles occasional Sand layers compact to very dense wet		9	SS	50									
			10	SS	89									16 56 28 0
276.5	END of BOREHOLE													
8.3	AUGER REFUSAL ON PROBABLE BEDROCK  DCPT conducted 1.0m east  Water Level in Piezometer: Sept 3/99: 1.7m depth Elev. 283.0m													

RECORD OF BOREHOLE No 9

1 OF 1

METRIC

W.P. 473-93-00 LOCATION N 5052843.8 E 311441.4 ORIGINATED BY MA  
 DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augering COMPILED BY AD  
 DATUM Geodetic DATE 31 May 1999 CHECKED BY SP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE						
285.2	0.2m TOPSOIL		1	SS	5									
0.0	SAND with Organics FILL loose													
284.5			2	SS	3									
0.7	brown Sandy Silt FILL trace Organics very loose wet		3	SS	2									
282.6	Clayey		4	SS	4									
2.6	ORGANIC SILT loose wet		5	SS	10									
281.6			6	SS	25									
3.6	brown SAND compact wet		7	SS	50/15									
280.8			8	SS	50/5									
4.4			9	SS	50/13									
	grey HETEROGENEOUS MIXTURE of SAND, SILT & GRAVEL (GLACIAL TILL) with frequent Cobbles very dense wet		10	SS	50/11									
													</	



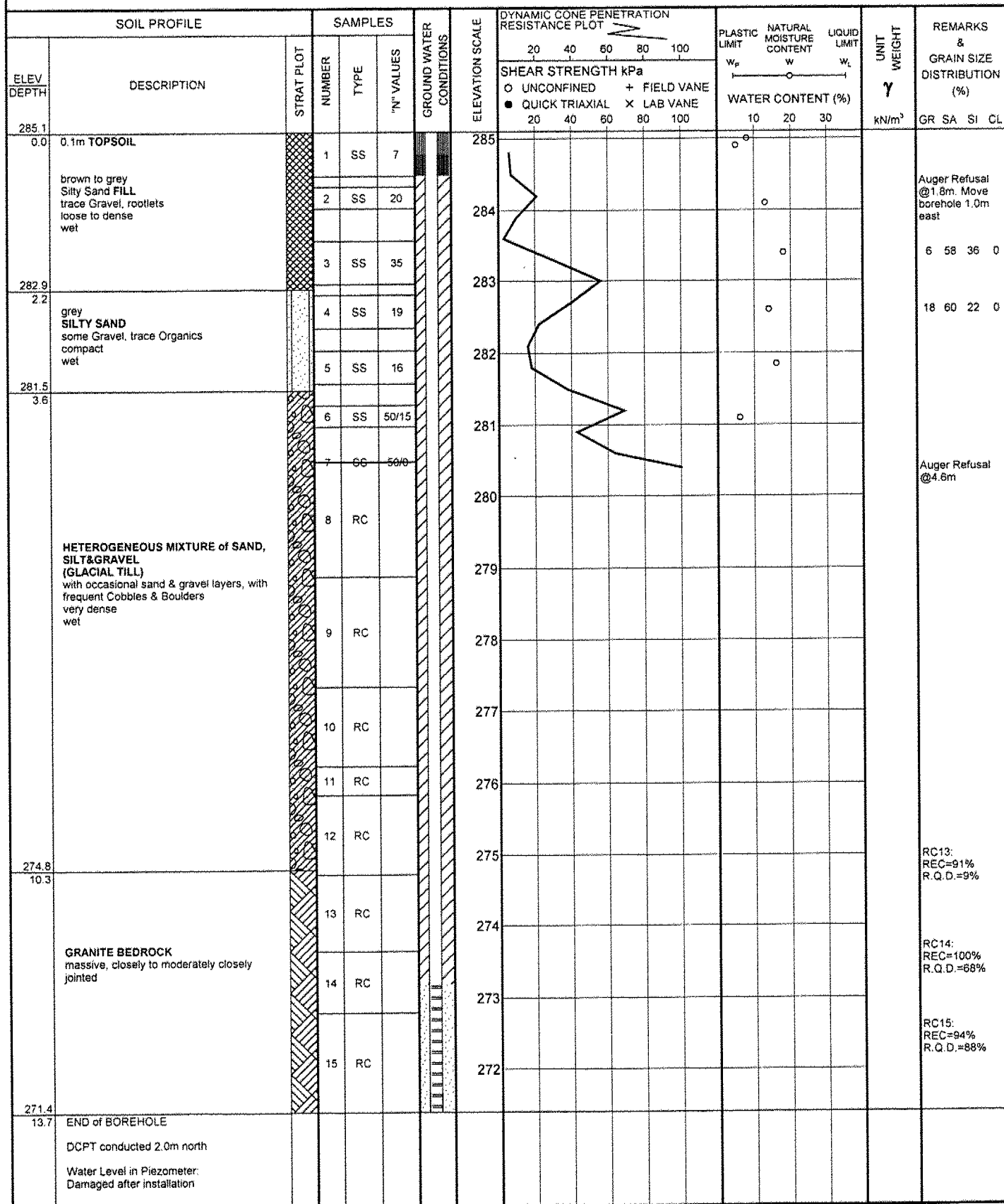
RECORD OF BOREHOLE No 10										1 OF 1		METRIC		
W.P. 473-93-00		LOCATION N 5052833.2 E311430.2				ORIGINATED BY MA								
DIST 52 HWY 11		BOREHOLE TYPE Hollow Stem Augering				COMPILED BY AD								
DATUM Geodetic		DATE 2 June 1999				CHECKED BY SP								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X LAB VANE	PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ KN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES									
285.1 0.0	0.1m TOPSOIL  brown Sandy Silt FILL trace Organics very loose moist to wet		1	SS	4		285							
283.7			2	SS	2		284							0 25 75 0
1.4 282.8	grey ORGANIC SILT trace decomposed Organics very loose wet		3	SS	4		283							
2.3 281.6	brown SAND loose to compact wet		4	SS	3		282						17.3	
			5	SS	12		281							
3.5			6	SS	37		280							
	dense very dense		7	SS	65		279							
	grey HETEROGENEOUS MIXTURE of SAND, SILT & GRAVEL (GLACIAL TILL) with frequent Cobbles wet		8	SS	89		278							
			9	SS	65									5 52 43 0
277.5 7.7	END of BOREHOLE  AUGER REFUSAL ON PROBABLE BOULDER  DCPT conducted 1.0m north  Water Level in Piezometer: Sept 3/99: 2.3m depth Elev. 282.8m		10	SS	50/0									

RECORD OF BOREHOLE No 11

1 OF 1

METRIC

W.P. 473-93-00 LOCATION N 5052812.2 E311443.2 ORIGINATED BY MA  
 DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augering COMPILED BY AD  
 DATUM Geodetic DATE 31 May 1999 CHECKED BY SP



# RECORD OF BOREHOLE No 12

1 OF 1

METRIC

W.P. 473-93-00 LOCATION N 5052791.1 E311445.9 ORIGINATED BY MA  
DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augering COMPILED BY AD  
DATUM Geodetic DATE 2 June 1999 CHECKED BY SP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ KN/m <sup>3</sup>	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						
285.8							20 40 60 80 100	20 40 60 80 100	10 20 30					GR SA SI CL	
0.0	0.2m TOPSOIL brown Sand FILL with Gravel loose damp		1	SS	7										
285.1															
0.7	TOPSOIL damp		2	SS	7										
284.4															
1.4	brown SAND with Organics with Gravel, some Silt, frequent Cobbles compact wet		3	SS	71/15									31 48 18 3	
283.0															
2.8	grey SILTY SAND frequent Cobbles trace Gravel, rootlets dense wet		4	SS	18										
281.4															
4.4			5	SS	31									1 51 48 0	
			6	SS	36										
			7	SS	25										
	compact very dense														
			8	SS	50/00										
	HETEROGENEOUS MIXTURE of SAND, SILT & GRAVEL (GLACIAL TILL) frequent Cobbles wet														
			9	SS	50/5									4 66 30 0	
279.3															
6.6	END of BOREHOLE														
	AUGER REFUSAL ON PROBABLE BOULDER														
	DCPT REFUSAL														
	Water Level in Piezometer: Sept 3/99: 1.7m depth Elev. 284.1m														

RECORD OF BOREHOLE No 13

1 OF 1

METRIC

W.P. 473-93-00 LOCATION N 5052986.3 E311417.2 ORIGINATED BY MA  
 DIST 52 HWY 11 BOREHOLE TYPE Dynamic Cone Penetration Test (DCPT) COMPILED BY AD  
 DATUM Geodetic DATE 3 June 1999 CHECKED BY SP

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					
287.5 0.0	NO SAMPLING & TESTING												
281.5 6.1	END of DCPT												

**RECORD OF BOREHOLE No 14**

1 OF 1

**METRIC**

W.P. 473-93-00 LOCATION N 5052725.6 E311463.3 ORIGINATED BY MA  
DIST 52 HWY 11 BOREHOLE TYPE Solid Stem Augering COMPILED BY AD  
DATUM Geodetic DATE 7 September 1999 CHECKED BY SP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
291.9								20 40 60 80 100						
0.0	0.15m TOPSOIL brown SANDY SILT trace Organics damp	very loose	1	SS	3			○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X LAB VANE						
291.2														
0.7														
290.7	brown CLAYEY SILT hard		2	SS	31		291							
1.2														
	brown SAND frequent Cobbles compact wet		3	SS	19		290							
289.2														
2.7	END OF BOREHOLE  Auger Refusal on Possible Boulders  Water Level on completion: none													

**RECORD OF BOREHOLE No 15**

1 OF 1

**METRIC**

W.P. 473-93-00 LOCATION N 5052738.1 E311453.2 ORIGINATED BY MA  
DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augering/Washboring/Rock coring COMPILED BY AD  
DATUM Geodetic DATE 2 September 1999 CHECKED BY SP





SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
287.9	0.35m TOPSOIL		1	SS	6									
0.0	Sand FILL with Silt, trace Clay, rootlets damp		2	SS	30							287	286	285
286.1	brown SAND & GRAVEL some Silt, dense, moist		3	SS	32									
1.8	grey HETEROGENEOUS MIXTURE of SAND, SILT & GRAVEL (GLACIAL TILL) frequent Cobbles & Boulders moist to wet		4	SS	34							285	284	283
285.7			5	SS	49									
2.2			6	SS	35							284	283	282
			7	SS	50/11									
			8	RC								283	282	281
281.0	GRANITE BEDROCK massive, closely to moderately closely jointed		9	RC										
6.9			10	RC								282	281	280
277.8	END of BOREHOLE													
10.1	Water Level in Piezometer: Sept 10/99: 2.8m depth Elev. 285.1m											281	280	279

**RECORD OF BOREHOLE No 16**

1 OF 1

**METRIC**

W.P. 473-93-00 LOCATION 19+102 6.5Rt NBL C/L ORIGINATED BY MA  
DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augering/Washboring COMPILED BY AD  
DATUM Geodetic DATE 18 August 1999 CHECKED BY SP

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 $\gamma$  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)			
								<div><div><div>○ UNCONFINED</div><div>● QUICK TRIAXIAL</div></div><div><div>+ FIELD VANE</div><div>× LAB VANE</div></div></div>										
288.4							20	40	60	80	100							
0.0	0.1m TOPSOIL grey Sandy Silt FILL trace rootlets very loose, moist		1	SS	4													
287.5																		
0.9	0.15m TOPSOIL grey-brown Clayey Silt FILL trace rootlets stiff, damp		2	SS	8													
286.6																		
1.8	brown-grey SILTY SAND trace Clay  compact, moist  dense, wet		3	SS	13													
			4	SS	16													
285.0			5	SS	37													
3.4	grey HETEROGENEOUS MIXTURE of SAND, SILT & GRAVEL (GLACIAL TILL) frequent Cobbles dense to very dense moist to wet		6	SS	74													
			7	SS	50/10													
			8	SS	59													
			9	SS	45													
281.4			10	SS	50/13													
7.0	END of BOREHOLE																	

## RECORD OF BOREHOLE No 17

1 OF 1

METRIC

W.P. 473-93-00 LOCATION N 5052778.7 E311453.3 ORIGINATED BY MA  
 DIST 52 HWY 11 BOREHOLE TYPE Solid Stem Augering / Wash boring COMPILED BY AD  
 DATUM Geodetic DATE 9 September 1999 - 10 September 1999 CHECKED BY SP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ KN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
286.4								20 40 60 80 100	20 40 60 80 100	10 20 30				
0.0	light brown SAND with GRAVEL (FILL) compact damp		1	SS	26									
285.7														
0.7	grey SANDY SILT to CLAYEY SILT (FILL) trace Organics loose to firm damp to moist		2	SS	6									
284.6														
1.8	brown SAND with GRAVEL, trace Silt		3	SS	50/13									
284.3	very dense													
2.1			4	SS	47									
	Silt some Sand		5	SS	42									0 10 83 7
	grey HETEROGENEOUS MIXTURE of SAND, SILT & GRAVEL (GLACIAL TILL) frequent Cobbles and Boulders moist to wet		6	SS	35									Auger refusal @ 4.4 m depth. Advance using wash boring.
	dense		7	SS	30									
			8	SS	33									
	very dense		9	SS	50/10									
	Sand, some Silt & Gravel		10	SS	50/8									22 58 20 0
			11	RC										
			12	RC										
			13	RC										
			14	RC										
			15	RC										
			16	RC										
			17	RC										
276.6														
9.8	brown SAND wet													
275.3			18	RC										
11.1														
	GRANITE BEDROCK massive, closely to moderately closely jointed		19	RC										RC18: REC=96% R.Q.D=68% RC19: REC=83% R.Q.D=83%
			20	RC										
			21	RC										RC20: REC=100% R.Q.D=37%
273.4	END OF BOREHOLE													
13.1	Water Level on completion: Not stabilized likely due to water used for coring.  Cave on completion: 8.5m Water Level in Piezometer: Damaged after installation													



# RECORD OF BOREHOLE No 18

1 OF 1

METRIC

W.P. 473-93-00 LOCATION N 5052767.7 E311444.7 ORIGINATED BY MA  
DIST 52 HWY 11 BOREHOLE TYPE Solid Stem Augering / Casing COMPILED BY AD  
DATUM Geodetic DATE 7 September 1999 CHECKED BY SP

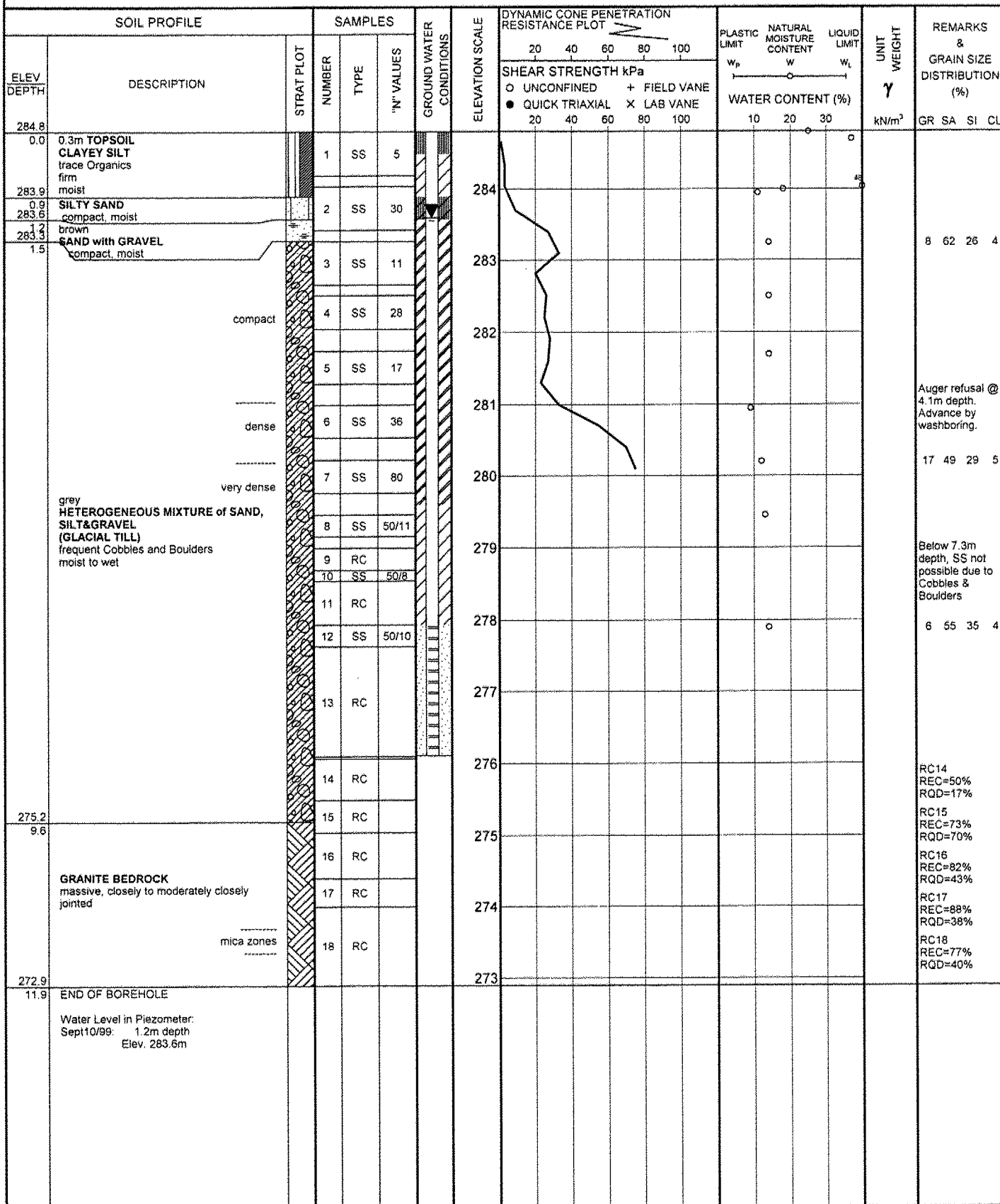
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
286.1								20 40 60 80 100	20 40 60 80 100					
0.0	0.1m TOPSOIL brown		1	SS	9		286	○ UNCONFINED + FIELD VANE						
285.3	SAND with GRAVEL (FILL) loose, damp							● QUICK TRIAXIAL x LAB VANE						
0.8	dark brown TOPSOIL		2	SS	7		285							
284.7	clayey damp													
1.4	grey													
284.4	SANDY SILT. Sand lenses, wet		3	SS	31									
1.7														
		dense					284							8 66 21 5
		compact	4	SS	22									
	grey-brown HETEROGENEOUS MIXTURE of SAND, SILT & GRAVEL (GLACIAL TILL) frequent Cobbles & Boulders occasional pockets of Sand moist		5	SS	14		283							
			6	SS	56		282							40 47 10 3
281.7		very dense												
4.4			7	SS	50/14		281							Auger refusal @ 4.4m depth, started using NW casing @ 4.4m depth to 4.9m, started using NQ casing @ 4.9m depth to 6.6m.
	brown GRAVELLY SAND frequent Cobbles and Boulders wet													
		very dense	8	SS	50/14									
		dense					280							20 60 17 3
			9	SS	47									
279.5	END OF BOREHOLE													
6.6	Water Level on completion: 1.5m (Not stabilized likely due to water used for coring)  Cave on completion: 3.5m													

# RECORD OF BOREHOLE No 19

1 OF 1

METRIC

W.P. 473-93-00 LOCATION N 5052798.9 E311438.0 ORIGINATED BY MA  
 DIST 52 HWY 11 BOREHOLE TYPE Solid Stem Augering / Wash boring COMPILED BY AD  
 DATUM Geodetic DATE 7 September 1999 CHECKED BY SP



RECORD OF BOREHOLE No 20

1 OF 1

METRIC

W.P. 473-93-00 LOCATION N 5052807.1 E311445.9 ORIGINATED BY MA  
 DIST 52 HWY 11 BOREHOLE TYPE Solid Stem Augering / Wash boring COMPILED BY AD  
 DATUM Geodetic DATE 8 September 1999 CHECKED BY SP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT $\gamma$	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		$w_p$	w	$w_L$		
285.0								20 40 60 80 100						
0.0	0.15m dark grey SAND with GRAVEL (FILL), trace Asphalt, compact, moist		1	SS	23			○ UNCONFINED + FIELD VANE						
284.2	light brown							● QUICK TRIAXIAL x LAB VANE						
0.8	grey SANDY SILT compact wet		2	SS	11		284	20 40 60 80 100						0 35 63 2
283.5														
1.5			3	SS	31		283							
	dense		4	SS	11									
			5	SS	29		282							
	compact		6	SS	26		281							
	grey HETEROGENEOUS MIXTURE of SAND, SILT & GRAVEL (GLACIAL TILL) frequent Cobbles moist to wet		7	SS	50/10									35 47 17 1
			8	SS	50/10		280							Auger refusal @ 4.5m depth. Advance by washboring. REC=8% RQD=0%
	v. dense		9	SS	50/10		279							
278.6	END OF BOREHOLE													
6.4	Water Level on completion: Not stabilized likely due to water used for coring													

**RECORD OF BOREHOLE No 21**

1 OF 1

**METRIC**

W.P. 473-93-00 LOCATION \_\_\_\_\_ ORIGINATED BY MA  
DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augering COMPILED BY AD  
DATUM Geodetic DATE 4 October 1999 CHECKED BY SP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)			
								○ UNCONFINED	+ FIELD VANE							● QUICK TRIAXIAL	x LAB VANE	
285.9 0.0	TOPSOIL		1	SS	14													
285.0 0.9	brown-grey MIXTURE of SAND, SILT, CLAY & ORGANICS (FILL) loose, wet grey Clayey Silt FILL trace organics firm, moist		2	SS	10								18.6					
284.7 1.2																		
283.8 2.1				3	SS	7								19.9				
	mottled SILTY CLAY trace Organics hard damp		4	SS	33								20.7					
282.9 3.0																		
282.3 3.6				5	SS	10												
	loose, wet brown SAND occasional thin Clay seams very loose to loose wet		6	SS	8													
				7	SS	4								0 95 (5)				
				8	SS	6												
279.9 6.0	grey HETEROGENEOUS MIXTURE of SAND, SILT & GRAVEL (GLACIAL TILL) with frequent Cobbles compact, wet		9	SS	29								0 23 76 1					
				10	SS	46								14 61 23 2				
				11	SS	50/11												
				12	SS	60												
	dense very dense		13	SS	50/14									25 58 16 1				
				14	SS	50/15												
275.6 10.3		GRANITE BEDROCK massive, closely to moderately closely jointed		15	RC									RC15: REC=100% R.Q.D.=100%				
				16	RC									RC16: REC=89% R.Q.D.=79%				
				17	RC									RC17: REC=100% R.Q.D.=100%				
				18	RC									RC18: REC=96% R.Q.D.=86%				
272.3 13.6	END OF BOREHOLE																	

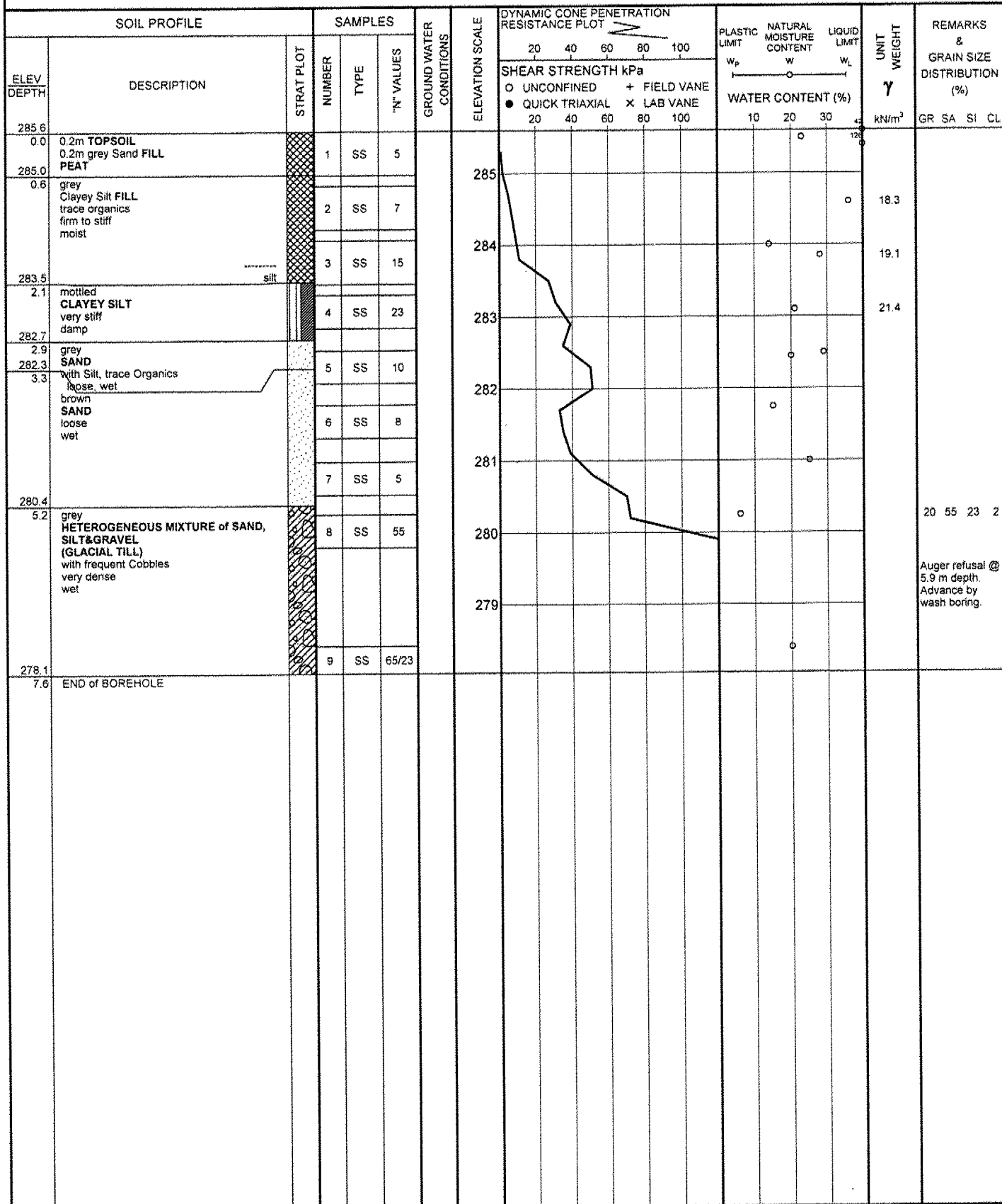
+ 3, x 3, Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 22

1 OF 1

METRIC

W.P. 473-93-00 LOCATION \_\_\_\_\_ ORIGINATED BY MA  
 DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augering COMPILED BY AD  
 DATUM Geodetic DATE 5 October 1999 CHECKED BY SP



# RECORD OF BOREHOLE No 24

1 OF 1

METRIC

W.P. 473-93-00 LOCATION N 5052995.4 E 311415.2 ORIGINATED BY MA  
DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augering COMPILED BY AD  
DATUM Geodetic DATE 18 August 1999 CHECKED BY SP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
288.6	0.15m TOPSOIL brown-grey Sand FILL with silt, some organics trace rootlets loose, damp		1	SS	7									
287.8	brown CLAYEY SILT FILL trace rootlets firm to very stiff moist		2	SS	4								20.8	
286.5			3	SS	17									
284.5	brown CLAYEY SILT very stiff to hard damp		4	SS	37									
			5	SS	25									
			6	SS	19									
284.0	red-brown SILTY SAND compact, wet													
283.4	grey-brown SILTY CLAY interbedded with SAND very stiff, wet		7	SS	17									
279.8	grey HETEROGENEOUS MIXTURE of SAND, SILT & GRAVEL (GLACIAL TILL) frequent Cobbles dense to very dense moist		8	SS	62									
			9	SS	31									
			10	SS	54									21 55 21 3
			11	SS	29									
			12	SS	50/13									48 40 11 1
277.1	GRANITE BEDROCK (PEGMATITE) massive, closely to moderately closely jointed		13	RC										RC13: REC=100% R.Q.D.=96%
			14	RC										RC14: REC=100% R.Q.D.=78%
			15	RC										RC15: REC=93% R.Q.D.=71%
11.6	END OF BOREHOLE													
	Water Level in Piezometer: Sept 3/99: 4.6m depth Elev. 284.0m													

**DRAFT**

**DRAFT  
FOUNDATION INVESTIGATION AND DESIGN REPORT  
FOR PROPOSED MAGNETAWAN RIVER BRIDGE  
AT KATRINE  
HIGHWAY 11, NBL  
DISTRICT 52, HUNTSVILLE  
W.P. 473-93-00**

**Submitted To:**

**Delcan Corporation  
133 Wynford Drive  
North York, Ontario, M3C 1K1  
Canada**

**Submitted By:**

**AGRA  
104 Crockford Blvd.  
Scarborough, Ontario, M1R 3C6  
Canada**

**October 1999  
TT98820M**

DRAFT

**AGRA Earth &  
Environmental Limited**  
104 Crockford Blvd.  
Scarborough, Ontario  
Canada M1R 3C6  
Tel (416) 751-6565  
Fax (416) 751-7592

October 5, 1999.  
**Ref. No.: TT98820M**

Delcan Corporation  
133 Wynford Drive  
North York, Ontario, M3C 1K1  
Canada

**Attention: Mr. Khaled El-Dalati, P. Eng.**

Dear Sir:


**Re: DRAFT  
FOUNDATION INVESTIGATION AND DESIGN REPORT  
FOR PROPOSED MAGNETAWAN RIVER BRIDGE  
AT KATRINE  
HIGHWAY 11, NBL  
DISTRICT 52, HUNTSVILLE  
W.P. 473-93-00**

We take pleasure in enclosing four (4) Draft copies of our Foundation Investigation and Design Report carried out for the above mentioned project and we will be glad to discuss any questions arising from this work.

Soil samples will be retained for a period of three months, and will thereafter be disposed of unless we are otherwise instructed.

We thank you for giving us this opportunity to be of service to you.

Sincerely,

  
George S.W. Chow, P. Eng.,  
Designated MTO Contact.

GSWC/dee



DRAFT

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## APPENDICES

Appendix A: Limitations of Report

Appendix B: Forward Slope Stability Analyses

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Figure 7

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Figure 8

## ENCLOSURES

BOREHOLE LOCATIONS AND SOIL STRATA ..... DWG. NO. 1  
RECORD OF BOREHOLE SHEETS

DRAFT

*specific  
no comment  
on  
planned  
construction  
in 1/2m*

## 1.0 INTRODUCTION

AGRA, Consulting Geotechnical Engineers, has been retained by DELCAN Corporation (DELCAN) to carry out a foundation investigation at the site of a proposed bridge which will carry the proposed northbound lane (NBL) of Highway 11 over the Magnetawan River. The site is located at Katrine, just east of the existing Highway 11 bridge crossing of the river, and is part of the Highway 11 Four Laning project from 0.7 km north of Highway 592N at Katrine, northerly 12.4 km (W.P. 473-93-00). This report addresses the foundation aspects of the proposed bridge and its approaches within 20 m of the structure. The single span bridge will have a span length of about 52 m and a width of about 13 m.

The purpose of this investigation is to determine the subsurface conditions at the site of the proposed bridge structure by means of a limited number of boreholes, in-situ tests and laboratory tests on selected samples. Based on our interpretation of the data obtained, recommendations are provided on the geotechnical aspects of foundation design. Comments are also provided on anticipated construction issues where they may affect the design of the proposed bridge and approach embankments from a geotechnical point of view.

The preliminary plan and profile for the proposed bridge and approaches were provided to us by DELCAN. The following document has also been referenced during the preparation of this report.

- Department of Highways of Ontario report titled "Foundation Investigation, Magnetawan River Bridge, Highway 11, Katrine, Ontario", W.P. 104-57, report no. 58-F-257C prepared by Trow Soderman & Associates, dated July 3, 1958 (GEOCREs No. 31E-40).

## 2.0 SITE DESCRIPTION AND PHYSIOGRAPHY

*groundwater concerns?*

The proposed bridge crossing of the Magnetawan River is located immediately to the south of Katrine, within MTO District 52 Huntsville, Ontario. This bridge which is to carry the proposed Highway 11 northbound lane (NBL) will span the width of the Magnetawan River at a location some 37 m to the east, centreline to centreline, of the existing Highway 11 bridge over the river. The existing Highway 11 at this location will become the southbound lane (SBL). The existing ground surface at the south approach is at about Elevation 295 m with a required embankment of up to 5 m in height. The existing ground surface at the north approach varies from about Elevation 296 m at the abutment location to about Elevation 299.5 m some 20 m north of the abutment, with corresponding required embankment of between 4 m and 1.5 m in height. The lowest point within the valley is at about Elevation 291 m. Moderate to heavy vegetation cover, consisting of predominantly grass, shrubs and small trees, exists at both approach locations.

*?*  
*drainage*  
*condition of*  
*existing bridge*  
*description of*  
*existing bridge*

Based on available geologic information, the site is situated within an area of ice-contact sediments. In general, after the last glacial withdrawal, ice-contact sediments of sands and gravel, followed by glaciofluvial sediments of deltaic and nearshore sands and gravel, as well as lake bottoms silts and clays, were deposited on top of the existing sandy glacial till or Precambrian

bedrock. The area was then inundated by glacial Lake Algonquin, depositing sands, silts and clays in low lying areas.

### 3.0 INVESTIGATION PROCEDURES

The field work for this investigation was carried out on March 30 and 31, 1999, April 7, 1999, April 14 to 16, 1999 and on April 27, 1999, during which time four boreholes (Borehole Nos. M1 to M4) were drilled and sampled. One borehole was put down at each of the proposed bridge abutments and one borehole was put down at a location some 20 m from each of the abutments. The planned locations of the boreholes and a stratigraphic section parallel to the proposed Highway 11 (NBL) centreline are shown on Drawing No. 1. *does it need to be of reference*

The investigation was carried out using a track-mounted power auger drill rig (BOA 6M) owned and operated by Groundworks Drilling Inc., under the full-time supervision of a member of AGRA's engineering staff. Hollow stem and solid stem augers were used to advance the boreholes within shallower depths. At greater depths, within the water-bearing sands and silts, wash-boring techniques using tri-cones and/or casings were required to further advance the boreholes. Rotary core drilling techniques were utilized to penetrate through cobbles and boulders encountered near the bottom of the two deep boreholes.

In the boreholes, soil samples were obtained at regular intervals of depth using 50 mm outside diameter split barrel (split spoon) samplers in accordance with Standard Penetration Test (SPT) procedures, as specified by ASTM Standard D1586. The SPT consists of freely dropping a 63.5 kg hammer for a vertical distance of 0.76 m to drive the split spoon sampler into the ground. The number of blows of the hammer required to drive the sampler into the relatively undisturbed ground, for a vertical distance of 0.30 m, is recorded as the Standard Penetration Resistance or the 'N'-value of the soil. This value gives an indication of the relative state of compactness of cohesionless soils and the consistency of cohesive soils. Where the consistency of the soil permitted, in-situ vane shear tests using a standard MTO 'N'-size vane were attempted at regular intervals within the silty clay deposit. Thin walled Shelby tube samples were also obtained at selected locations within this deposit. *1811*

Several attempts were made to carry out Dynamic Cone Penetration Tests (DCPT) within the sand and gravel deposit in Borehole M1. This test consists of continuously driving a 60° point, 50 mm diameter cone attached to the drill rod, into the undisturbed ground with a driving energy of 475 kJ (63.5 kg hammer free falling for a distance of 76 cm) per blow. The number of blows for each 30 cm of penetration is recorded, providing an indication of the relative changes in the soil density with depth.

Due to the presence of cobbles and boulders near the bottom of Boreholes M1 and M3, rotary core drilling techniques were utilized to further advance the boreholes below Elevations 268 m and 258 m, respectively. Coring was carried out using a NQ size core barrel.

Groundwater conditions in the open boreholes were observed throughout and immediately after the drilling operations. Standpipe piezometers were installed in Boreholes M1, M3 and M4 to permit long term monitoring of groundwater levels at the site. Two piezometers were sealed in different soil strata in Borehole M1. Borehole M2 was adequately backfilled with suitable materials on completion of the field work. *when read how checked*

The drilling, sampling and in-situ testing operations were carried out under the full-time supervision of members of our engineering staff, who examined the samples and logged the boreholes. The soil samples were identified, placed in containers and transported back to our geotechnical laboratory in Toronto (Scarborough) for further examination and testing. Index and classification tests, including natural moisture content determination, grain size distribution analysis and Atterberg limits tests, were carried out on selected representative soil samples. Organic content tests were carried out on samples of the organic silty sand. Laboratory quick triaxial (UU) and oedometer tests were also carried out on selected silty clay samples to determine strength and deformation characteristics. The results of the laboratory tests are presented on the relevant Record of Borehole sheets, on Figure Nos. 1 to 7, and in the text.

The drilling locations were initially established in the field by our field personnel based on the centreline of Highway 11 staked out by Dearden and Stanton Limited. The as-drilled borehole locations in terms of northing and easting co-ordinates, and elevations were surveyed by Dearden and Stanton Limited. We understand that these elevations are referenced to Geodetic datum. The locations and co-ordinates of the boreholes are shown on Drawing No. 1; the co-ordinates and elevations are indicated on the Record of Borehole sheets.

#### 4.0 SUBSURFACE CONDITIONS

The subsurface conditions were investigated at four borehole locations, Boreholes M1 to M4, inclusive. The locations of the boreholes are shown on plan and profile on Drawing No. 1 and are also indicated on the Record of Borehole sheets. A cross-section of inferred subsurface stratigraphy is shown on Drawing No. 1.

At the south approach within 20 m of the proposed south abutment location, the ground surface is generally flat-lying at about Elevation 295 m. At the north approach, the ground surface slopes upwards from about Elevation 296 m at the proposed north abutment location to about Elevation 299.5 m, some 20 m to the north.

In general, the subsoils at the site consist of a thin veneer of peat or topsoil overlying a layer of very loose silty sand with organics at the south approach; the peat is underlain by strata of loose to compact sand and silt at the north approach. Encountered below the surficial sands and silts is a layer of silty clay with some varved-like interbeds and of typically stiff to very stiff consistency. Extensive deposits of cohesionless soils, with increasing grain sizes with depth ranging from silt to sand and gravel, underlie the silty clay. The relative state of compactness also increases with depth, ranging from compact to very dense. Frequent cobbles and/or boulders were inferred or

encountered within the lower, very dense sand and gravel. The groundwater levels measured at different elevations within the underlying sands and silts generally correspond to the "normal" river level.

Details of the subsurface conditions encountered in the boreholes are presented on the Record of Borehole sheets. Descriptions of the subsurface conditions encountered in the boreholes are provided in the following sections.

#### 4.1 PEAT AND TOPSOIL

Peat of thickness varying between 0.2 m and 0.5 m was encountered in Boreholes M1, M2 and M3, but was not found in Boreholes M4. The two recorded SPT 'N'-values of 2 blows per 0.3 m penetration indicate a very soft to soft consistency.

Topsoil of about 0.2 m thick was encountered at ground surface in Borehole M4.

It should be noted that the thickness of the peat and topsoil may vary in between and beyond the borehole locations.

#### 4.2 SILTY SAND (with Organics)

A layer of silty sand with organics was encountered at three of the four borehole locations. The silty sand has a thickness of 4.8 m and 5.5 m in Boreholes M3 and M4, respectively, in the vicinity of the south approach, but it is only about 0.3 m thick in Borehole M2 at the proposed north abutment. This silty sand was not encountered in Borehole M1. This cohesionless soil contains layers and inclusions of rootlets, wood fibres and decomposed organics. Measured SPT 'N'-values within this deposit range from 1 blow to 4 blows per 0.3 m penetration, indicating a very loose state of compactness. Organic contents of between 1.5% and 4.5% were determined for four silty sand samples. Measured natural moisture contents typically range from 28 to 36%, with an occasional value as high as 74%. This high value is associated with Sample 6 in Borehole M3 where the organic content is measured to be 3.7%.

Four grain size distribution analyses were carried out on samples of this silty sand (same samples where organic contents were determined) with the following results:

Gravel:	0%
Sand:	60 - 82%
Silt:	18 - 40%
Clay:	3%

The grain size distribution curves are shown on Figure 1.

#### 4.3 SILTY CLAY

A stratum of silty clay was encountered at between about Elevations 290 m and 294 m in all four boreholes. The thickness of the deposit ranges from about 11 m in Borehole M3 to as little as 1.5 m in Borehole M4. Borehole M2 did not fully penetrate the silty clay. Based on the four boreholes drilled as part of this investigation and the boreholes put down during a previous investigation for the existing Highway 11 bridge, this silty clay appeared to be thickest on the south bank of the river (proposed south abutment location), gradually thinning towards the north bank (proposed north abutment location); the thickness of the silty clay also reduces in a southerly direction from the south abutment location. This silty clay has some varved-like clayey silt interbeds and occasional sand seams. The thickness of the clayey silts is in the order of 5 mm. Measured SPT 'N'-values within the silty clay range from 2 blows to 15 blows per 0.3 m penetration. Field vane tests indicated in-situ undrained shear strengths generally ranging from 55 kPa to greater than 110 kPa, i.e. vane could not be turned. A value as low as 22 kPa was recorded at the upper portion of the stratum in Borehole M1 and is likely attributed to disturbance at the bottom of the borehole prior to testing. Five unconsolidated, undrained triaxial (UU) tests were carried out on samples of the silty clay giving undrained shear strengths ranging between 50 kPa and 110 kPa. A low value of 28 kPa was determined for Sample 9 in Borehole M3; this may be attributed to sample disturbance. The field and laboratory results indicate that the silty clay has a typically stiff to very stiff consistency.

Atterberg limits tests were carried out on seven samples of the silty clay and yielded the following results:

Liquid Limit:	32 - 44% (average 40%)
Plastic Limit:	20 - 25% (average 22%)
Plasticity Index:	11 - 21% (average 17%)

These values indicate that the soil may be classified as a clay of medium to occasionally low plasticity (group symbol CI - CL) as shown on Figure 7. Measured natural moisture contents vary from 32 to 50% throughout this deposit and, as such, they exceed or are close to the measured liquid limit values, where tested.

One grain size distribution analysis was conducted on representative samples from this stratum, giving the following grain size measurements.

Gravel:	0%
Sand:	1%
Silt:	59%
Clay:	40%

The grain size analyses results are presented on Figure 2.

One oedometer test was carried out on a relatively undisturbed Shelby tube sample of the silty clay recovered from Borehole M3. Results of the tests indicate that the sample has the following properties: an average  $m_v$  of about  $2 \times 10^{-4} \text{ m}^2/\text{kN}$  within the anticipated field stress range under embankment loading, preconsolidation pressure  $\sigma'_p$  of about 240 kPa, compression index  $C_c$  of about 0.56, and recompression index  $C_r$  of about 0.07.

#### 4.4 SILT

Immediately lying above and/or below the silty clay are deposits of a non-plastic silt with traces of sand and occasional fine sand and thin clay seams. This soil was encountered below the silty clay at about Elevations 279 m and 288.5 m in Boreholes M3 and M4, respectively. This fine grained granular material was encountered at about Elevations 297 m and 298 m in Boreholes M1 and M2, respectively, and again at Elevation 287 m in Borehole M1. The thickness of this soil is about 4 m to 5 m in Boreholes M1 and M2, and about 3 m in Borehole M3. Borehole M4 did not fully penetrate this silt. Below the silty clay, this silt is generally in a compact state as indicated by SPT 'N'-values of between 10 blows and 22 blows per 0.3 m penetration, except at about Elevation 278 m in Borehole M3 where it is in a dense state ('N'-values of 38 blows). Above the silty clay, this silt is loose to compact as indicated by 'N'-values of 5 blows to 13 blows. Measured natural moisture contents range from 22% to 35%.

Four grain size distribution analyses were carried out on samples of the silt with the following results:

Gravel:	0%
Sand:	1 - 6%
Silt:	91 - 97%
Clay:	2 - 4%

The grain size distribution curves for the silt are shown on Figure 3.

#### 4.5 SAND AND SILT

A deposit of cohesionless sand and silt, with occasional sandy silt layers of up to about 1 m thick, was encountered between Elevations 283 and 275 m (8 m thick) in Borehole M1 and between about Elevations 276 and 268 m (8 m thick) in Borehole M3. Standard Penetration tests conducted in this deposit gave 'N'-values ranging between 13 blows and 30 blows per 0.3 m in Boreholes M1 and M3, indicating a typically compact condition. A higher value of 35 blows per 0.3 m indicates an occasional dense condition. The measured natural moisture contents of samples recovered from the deposit range from 22% to 25%.

Two grain size distribution analyses were carried out on samples of the sand and silt, and yielded the following results:

Gravel:	0%
Sand:	45 - 50%
Silt:	50 - 55%

The grain size distribution curves for the sand and silt is shown on Figure 4.

One grain size distribution analysis was carried out on a sample of the sandy silt, and yielded the following results:

Gravel:	0%
Sand:	35%
Silt:	65%
Clay:	0%

The grain size distribution curve for the sandy silt is shown on Figure 5. This figure also shows the curve for the sandy silt encountered within the underlying sand.

#### 4.6 SAND

A deposit of cohesionless fine to medium sand with traces of silt and occasional sandy silt layers was encountered below the sand and silt, at about Elevation 275 m (2.5 m thick) in Borehole M1 and at about Elevation 268 m (7 m thick) in Borehole M3. Standard Penetration tests conducted in this deposit gave 'N'-values ranging between 14 blows and 40 blows per 0.3 m, indicating a compact to dense condition. An 'N'-value of 36 blows was measured for the sandy silt zone of about 1 m thick at the bottom of this deposit in Borehole M3. The measured natural moisture contents of samples recovered from the deposit range from 18% to 26%.

One grain size distribution analysis was carried out on a sample of the sandy silt encountered within the sand, and yielded the following results:

Gravel:	1%
Sand:	34%
Silt:	65%
Clay:	0%

The grain size distribution curve for the sandy silt is also shown on Figure 5.

#### 4.7 SAND AND GRAVEL TO GRAVELLY SAND

The sand and silt deposit is underlain by a sand and gravel to gravelly sand deposit at Elevation 272 m in Borehole M1 and at Elevation 261 m in Borehole M3. Neither borehole fully penetrated this deposit, but they were terminated at Elevations 260.4 m and 252.3 m, after penetrating the deposit for about 11 and 8 m in Boreholes M1 and M3, respectively. This deposit is typically in a very dense state as indicated by SPT 'N'-values of greater than 50 blows per 0.3 m penetration,



except between Elevations 272 m and 269 m in Borehole M1 where the deposit is in a compact state as indicated by 'N'-values of 24 blows and 27 blows. Refusal to split spoon sampler advance were frequently encountered, and rotary core drilling techniques were used to advance the boreholes through cobbles and boulders below Elevations 268 m and 258 m in Boreholes M1 and M3, respectively. Attempts were made to carry out DCPT's at three elevations near the bottom of Borehole M1; refusal to cone advance was met at all three elevations. Measured natural moisture contents range from 10% to 15%.

Four grain size distribution analyses were carried out on samples from the sand and gravel to gravelly sand matrix of this deposit and yielded the following results:

Gravel:	22 - 70 %
Sand:	20 - 73 %
Silt & Clay:	5 - 25 %

The grain size distribution curves are shown on Figure 6.

#### 4.8 GROUNDWATER CONDITIONS

Groundwater levels in the open boreholes were observed during drilling and upon completion of each borehole. To permit long term monitoring of groundwater levels at the site, two standpipe piezometers were installed in Borehole M1, and one in each of Boreholes M3 and M4. The recorded values are shown on the respective Record of Borehole sheets.

The measured piezometer readings indicate that the groundwater level, at about Elevation 295 m, of the sands and silts is largely governed by the "normal" water level in Magnetawan River. This elevation corresponds to the ground surface (floodplain level) at the south approach, but about 2 m to 4 m depth below existing ground surface at the north approach.

It should, however, be pointed out that the groundwater at the site would fluctuate seasonally and can be expected to be somewhat higher during the spring months and in response to major weather events, which would have a direct impact on the river level.

*Sign & seal*

DRAFT

## 5.0 DISCUSSION AND RECOMMENDATIONS

The proposed Highway 11 realignment will consist of a four lane divided roadway with an approximately 30 m wide median. The work described in this report is associated with the proposed bridge to carry the proposed northbound lane (NBL) of Highway 11 over the Magnetawan River near Katrine, Ontario, and the approach embankments within 20 m of the structure. The existing Highway 11 will become the southbound lane (SBL). It is understood that the proposed structure, which will carry the new NBL, is a single span, two-lane (13 m wide) bridge, approximately 52 m in length.

The existing bridge is a single span structure with perched abutments, and approach embankments of up to 2 m in height. Visual observations by our field staff did not reveal signs of significant active erosion at the forward slopes at the time of our investigation.

*Site description*

The Magnetawan River flows in a westerly direction in the vicinity of the bridge site. The south slope of the river bank flow channel at the site has a relatively uniform inclination of about 3.5 horizontal to 1 vertical. The upper portion of the north slope has a steeper inclination of up to 2 horizontal to 1 vertical and the lower portion is at 4.5 horizontal to 1 vertical. The proposed horizontal and vertical alignments for Highway 11 and the location of the bridge were provided to us in plan and profile drawings by Delcan Corporation. Based on the drawings provided, the bridge will span the river channel, and the existing grade at the proposed bridge location (centreline) is about Elevation 295 m at the south approach and abutment locations, increasing to about Elevation 296 m at the north abutment location and up to Elevation 299.5 m at the north approach. The proposed grade along the bridge alignment lies between approximately Elevations 300 m (on the south side) and 300.5 m (on the north side). The embankment at the south approach and abutment locations will therefore be about 5 m high, whereas the embankment at the north abutment location will be about 4.5 m high. About 20 m north of the north abutment, the grade raise of the embankment will be about 1.5 m high near Station 11+210.

*Subsoil description*

In general, the subsoils at the south approach consist of a thin veneer of peat and about 5 m of very loose silty sand with organics overlying up to 11 m of silty clay of generally stiff consistency. At the north approach, the silty sand with organics is much thinner (about 0.3 m); instead a stratum of loose to compact silt exists between the silty sand with organics and the underlying silty clay. Underlying the silty clay are extensive deposits of compact to very dense silts, sands and gravel. These deposits become progressively coarser grained and denser with depth. Below about 30 m to 35 m, i.e. below Elevations 260 and 269 m, the sand and gravel deposit, which contains frequent cobbles and boulders, is in a very dense state. Refusal to split spoon sampler and dynamic cone advance was frequently encountered in the cobbles and boulders. Bedrock was not encountered in any of the boreholes put down at this site.

Piezometer readings at the boreholes indicate that a water level associated with the underlying sands and silts is at Elevation 295 m. This level corresponds to ground surface and between 2 m to 4 m depth at the south and north approaches, respectively.

## 5.1 FOUNDATIONS

At the south abutment location, the surficial very loose silty sand with organics extends to 5 m depth and the groundwater level is at ground surface. At the north abutment location, the loose to compact silt extends to 4 m depth with a groundwater level at 1 m depth below ground surface (on a slope). As such, shallow spread footings resting on native soils are not considered practical for foundation support at both locations.

Alternatively, if perched abutments are considered in the design, spread footings founded on a compacted Granular 'A' core may be feasible at the south abutment only. The consolidation settlement would be somewhat mitigated, but will still likely exceed the normally acceptable value of 25 mm and, as such, the use of spread footing foundations in this situation is also not considered. Based on existing information, the geometry at the proposed north abutment does not appear to allow consideration of spread footings founded on a compacted Granular 'A' core.

In view of the above and since an integral abutment bridge is the preferred option, consideration should be given to supporting the bridge on deep foundations in the form of steel H-piles, driven to practical refusal within the upper portion of the very dense sand and gravel with frequent cobbles and boulders. In order to adequately penetrate the typically compact to dense sand immediately above the founding stratum, a heavier section such as HP310x110 with reinforced tips would be suitable for use.

It is considered likely that the driven H-piles will not be able to penetrate deep into the very dense sand and gravel with frequent cobbles and boulders. Based on the results of the boreholes, the following Table 1 summarizes the estimated average pile tip elevations that may be assumed for design purposes.

TABLE 1

SUPPORT LOCATION	REFERENCE BOREHOLE	ESTIMATED APPROXIMATE PILE TIP ELEVATION (m)
South Abutment	M3	256±
North Abutment	M1	265±

The borehole results indicate that the founding sand and gravel deposit containing cobbles and/or boulders was encountered at a higher elevation at the north abutment location compared with the south abutment location. Therefore, it may be expected that the piles will terminate at higher elevations at the north abutment location.

### 5.1.1 Resistance to Axial Loads

For HP310x110 steel H-piles driven to practical refusal within the very dense sand and gravel at or below the elevations shown in Table 1 above, the following axial resistances may be assumed for design.

Factored Axial Resistance at Ultimate Limit States (U.L.S.)	=	1,650 kN
Geotechnical Resistance at Serviceability Limit States (S.L.S.)	=	1,150 kN

The above values were selected in view of the fact that some premature pile refusals may be encountered at elevations higher than those shown in Table 1.

Negative skin friction can be induced on the piles due to consolidation settlement of the silty clay, compression of the surficial silty sand with organics, and to a lesser extent compression of the underlying sands and silts, beneath the approach embankments. As a result, down-drag forces would act on the pile, thus reducing its usable end-bearing capacity. The magnitude of the negative skin friction depends on many factors such as the relative movement between the surrounding compressible soils and the pile shaft, the elastic compression of the pile under the working load, the rate of consolidation of the silty clay, compressibility of the silty sand with organics, as well as the construction sequence and methodology of the piles and the embankment. It is anticipated that the surficial peat will be removed as part of the conventional embankment construction procedures. At this site, it is considered that the anticipated settlements of the underlying compressible soils due to newly placed fill may result in some mobilization of negative skin friction on the piles. Although the height of the proposed embankment is only up to 5 m at the south approach, compression of the organic materials within the silty sand will occur. It is also anticipated that the down-drag effect would be more pronounced at the south abutment due to the organics and larger thickness of silty clay.

In order to minimize the effect of down-drag loads, we recommend that the approach fills be placed to their final grade elevation for at least one month prior to driving the piles. By preloading for a period of one month, it is anticipated that the elastic settlement of the sands and silts would largely have been completed. Substantial compression of the organic materials within the silty sand, as well as clay consolidation within the recompression range of stresses, would also occur in the same period of time. It is noted that excavation through the embankment fill to the underside elevation of the pile caps will be required prior to driving the piles.

Assuming that the embankment fills will be placed to their final grade elevation (i.e. top of base course) at least one month prior to the start of pile driving and based on the results of Borehole M1, the unfactored down-drag load acting on a HP310x110 steel H-pile is estimated at about 480 kN at the south abutment location. For the north abutment, where the thickness of compressible soils is smaller, the estimated unfactored down-drag load is 240 kN. A suitable load factor should be applied to these values as per the Ontario Highway Bridge Design Code (O.H.B.D.C.), 3<sup>rd</sup> Edition.

In accordance with MTO standard practice, the piles should be driven to about 2 m to 3 m above the elevations recommended in Table 1 above, after which driving should be monitored and controlled by a recognized pile driving formula, such as the Hiley formula. The estimated ultimate resistance of the piles driven to practical refusal within the very dense sand and gravel, at about the elevations quoted above, is approximately 3,500 kN. The piles should be driven with a suitably heavy hammer capable of delivering a rated capacity of at least 50 kJ per blow. The energy should, however, be restricted to not more than 60 kJ per blow. A final set of 8 blows per 12 mm of penetration should be obtained at the above rated energy.

Cobbles and/or boulders were inferred or encountered within the very dense sand and gravel in the boreholes drilled at the abutment locations. In view of this and the hard driving conditions anticipated, the pile tips should be reinforced, as per MTO Standards (OPSD 3301.00), to minimize damage to the piles.

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

If the driven pile encounters refusal above the recommended elevation, the Geotechnical Engineer familiar with the findings of this report and appointed by the Contract Administrator should be notified immediately.

During the driving process, piles which have already been driven should be monitored to determine if they are heaving due to effects of driving adjacent piles. If this phenomenon occurs, the affected piles should be re-driven. It is recommended that not less than 15% of the piles and at least three piles in each foundation support element be re-struck one to two days after initial installation, as a precaution against relaxation. If relaxation occurs, then all piles in that foundation element should be re-tapped.

It is possible that some of the piles may penetrate to one to two metres below the estimated tip elevations and this aspect should be taken into consideration when ordering piles.

The geotechnical resistance at Serviceability Limit States (S.L.S.) is dependent on the settlement of the pile group and, therefore, is governed by the size of the pile group. The pile group configuration is currently not available to us. Provided that the piles are designed and installed as recommended above, it is considered that the quoted S.L.S. value corresponds to no more than 25 mm of settlement for the pile group. We will confirm the estimated settlement once information on the pile group configuration is known.

### 5.1.2 Resistance to Lateral Loads

Laterally applied loads on piles can be resisted geotechnically by the driven piles through passive pressure developed in the soil in which the piles are embedded. The pile tip elevations recommended above indicate that the piles will be in the order of 30 to 35 m in length. Lateral pile resistance may be considered in accordance with Section 6-9.8.1 of the O.H.B.D.C., 3<sup>rd</sup> Edition.

The recommended horizontal resistances for a HP310x110 pile at this site are as follows:

Factored Horizontal Resistance at U.L.S.	=	120 kN
Horizontal Resistance at S.L.S.	=	50 kN

In accordance with MTO requirements (MTO Structural Office Standard), piles for integral abutments require a 3 m long flexible zone. In essence the current MTO standard for the flexible zone consists of an annular space in between two concentric CSP's. One of the CSP's surrounds the H-pile (i.e. has a diameter slightly greater than the pile width), while the second CSP has a somewhat larger diameter; typically 0.6 m for a 310 mm H-pile. After the pile is driven, the space between the H-pile and the inner CSP is filled with cement bentonite or coarse sand.

If conventional abutments on pile groups are to be built instead of integral abutments, then the unbalanced horizontal forces could be resisted by battered piles, but the degree of batter should be kept to a minimum in view of the anticipated consolidation settlements.

For lateral soil-pile interaction analysis, the horizontal subgrade reaction to the pile can be calculated from the expression:

$$k_s = n_h \times z/d$$

where  $k_s$  = coefficient of horizontal subgrade reaction  
 $n_h$  = coefficient related to soil density as given in Table 2  
 $d$  = pile width  
 $z$  = depth

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

**TABLE 2**

AREA/REFERENCE BOREHOLE NO.	APPLICABLE DEPTH FROM EXISTING GROUND SURFACE (ELEVATION)	SOIL TYPE	BULK UNIT WEIGHT (kN/m <sup>3</sup> )	ANGLE OF INTERNAL FRICTION ( $\phi$ ) DEGREES	RECOMMENDED $n_h$ VALUE (MN/m <sup>3</sup> )
<b>North Abutment</b>					
M1	0.5 - 2 m (299 - 297 m)	compact sand	20	30	5
	2 - 7 m (297 - 292 m)	loose to compact silt	19	29	4
	7 - 12 m (292 - 287 m)	stiff silty clay	18	---	5/z (constant with depth)
	12 - 24 m (287 - 275 m)	compact silt to sand and silt	19	29	4
	24 - 30 m (275 - 269 m)	compact sand to sand and gravel	21	32	6
<b>South Abutment</b>					
M3	0.5 - 5 m (295 - 290 m)	very loose silty sand with organics	18	26	1.5
	5 - 16 m (290 - 279 m)	stiff silty clay	18	---	5/z (constant with depth)
	16 - 27 m (279 - 268 m)	compact silt to sand and silt	19	29	4
	27 - 34 m (268 - 261 m)	compact to dense sand	21	33	7

Note: It is anticipated that all surficial peat within the footprints of the abutments and approaches will be sub-excavated.

If the abutments are to be supported on conventional piled foundations (instead of using integral abutments), then there may be more than one row of piles. In this instance, group action for lateral loading should be considered when the pile spacing in the direction of loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction in the direction of loading by a reduction factor R as follows:

PILE SPACING IN DIRECTION OF LOADING $d$ = PILE DIAMETER	SUBGRADE REACTION REDUCTION FACTOR $R$
8d	1.00
6d	0.70
4d	0.40
3d	0.25

## 5.2 LATERAL EARTH PRESSURES

Backfill behind abutments and retaining walls should consist of non-frost susceptible, free-draining granular materials in accordance with MTO 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., 3<sup>rd</sup> Edition. For design purposes, the following parameters (unfactored) can be used.

### Compacted Granular 'A'

Unit Weight = 22 kN/m<sup>3</sup>

Coefficient of Lateral Earth Pressures:

$K_a = 0.27$  (active condition)

$K_o = 0.43$  (at-rest condition)

### Compacted Granular 'B'

Unit Weight = 21 kN/m<sup>3</sup>

Coefficient of Lateral Earth Pressures:

$K_a = 0.31$  (active condition)

$K_o = 0.47$  (at-rest condition)

The above design parameters assume level ground surface and backfill behind the retaining structure.

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., 3<sup>rd</sup> 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 of the O.H.B.D.C., 3<sup>rd</sup> 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

*medium*

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

### 5.3 APPROACH EMBANKMENTS

At the south abutment and approach locations, the proposed road grade is at about Elevation 300 m, which will require an approach embankment of about 5 m in height. At the north abutment location, the proposed road grade is at about Elevation 300.5 m, which will require an approach embankment of up to 4.5 m in height. The height of the north embankment will decrease towards the north to about 1.5 m at approximately 20 m north of the abutment location (at about Station 11+210).

The boreholes show that the subgrade at the south abutment and approach consists of very loose silty sand with organics overlying typically stiff silty clay. Embankments of conventional fill of up to 5 m in height, with a side slope inclination of 2 horizontal to 1 vertical, would be stable against deep-seated (i.e. foundation) failures, provided that the subgrade is properly prepared by removing all surficial peat, topsoil, organic and otherwise unsuitable materials as per MTO Standards before placing the fill. It is considered impractical to remove organic materials embedded within the silty sand with organics unless such materials are encountered at or near ground surface.

*Sandstone*

Stability analysis indicated that the existing forward slopes are marginally stable, i.e. Factor of Safety of just above 1.0. Embankment loading currently anticipated at the abutments, which are to be located on the slope or at the crest, will cause instability. Consideration could be given to lengthening the proposed bridge by about 8 m to an overall length of 60 m (5 m at the north abutment and 3 m at the south abutment).

*not single span*

Preliminary analysis of the above structural configuration, in conjunction with slope flattening and benching (at the north abutment), would ensure both long term and short term stability (see Appendix B). If the option of lengthening the bridge is adopted and once the new abutment locations and configurations are available to us, we will perform more detailed analysis to confirm stability.

Consideration could also be given to other alternatives for enhancing forward slope stability, such as the use of lighter weight materials (blast furnace slag and styrofoam) as embankment fill, or lowering of the proposed Highway 11 grade to minimize embankment loading.

*assumed not*

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Delcan Corporation  
Proposed Magnetawan River Bridge at Katrine, NBL  
Highway 11, District 52, Huntsville, Ontario

TT98820M

October 1999

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*berm gradient and construction*  
*Signe*

Assuming properly compacted, acceptable inorganic earth fill material, 2 horizontal to 1 vertical (2H:1V) side slopes can be used throughout. The berm gradient should be sloped (say 1V:20H) to drain away from the embankment. 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 1H:1V from the toe of the proposed embankment as depicted by Figure 8. The average thickness of the unsuitable soils to be stripped can be assumed to be about 0.5 m. After stripping, the exposed subgrade should be inspected, approved and properly compacted from the surface under the supervision of qualified personnel, using a suitably heavy compactor.

Provided that all surficial organic and otherwise unsuitable materials are removed and the subgrade is properly compacted from the surface as detailed above, the total settlement of the foundation materials (i.e. not including the settlement of the embankment material under its own weight) should be in the order of 150 mm at the south approach and abutment locations. This settlement should be substantially completed during construction and within one month after placing the embankment fill to its full height. Post construction settlement due largely to secondary consolidation of the organic materials within the silty sand is estimated to be not more than 25 mm.

On the north side of the river, total foundation settlement induced as a consequence of embankment loading using normal earth fill is estimated to be in the order of 75 mm at the abutment location (where the height of the fill will be about 4 m) decreasing to about 40 mm further north at about Station 11+210 (where the height of fill is only about 1.5 m). Within one month after the completion of fill placement, it is anticipated that settlement would have substantially completed. Post construction settlement due to secondary consolidation of the clay would be less than 15 mm.

Preloading would minimize the effects of down-drag on the piles as a result of consolidation of the silty clay and the organic materials within the silty sand, as well as elastic compression of the sands and silts. We, therefore, recommend that the approach fills be placed to their final grade elevation at least one month prior to driving the piles. If scheduling permits, paving should be delayed for as long as possible to allow more settlement to take place.

Piezometric measurements indicate that water levels are at the existing ground surface (floodplain level) at the south approach. We do not anticipate major problems due to groundwater seepage, but care should be exercised to minimize disturbance to the subgrade during subgrade preparation and backfilling for the construction of the south embankment. Depending on the time of construction, some dewatering may be required to stabilize the sandy and silty soils. It is anticipated that dewatering should not be required for construction of the north approach embankment. Instead consideration may be given to steel sheetpile cofferdams for soil retention and groundwater control.

*for what purpose  
is it this dewatering?*

*is NSSD required?*

.../...

M:\reports\1999\magnetawanbridge1.wpd

The fill materials used for construction of the embankment should consist of approved, clean earth fill (e.g. Select Subgrade Materials - OPSS 1010). A majority of the fill will have to be imported for this purpose. 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 Maximum Dry Density. The degree of compaction within the top 0.6 m of the fill (i.e. the subgrade immediately beneath the granular sub-base) should be increased to 98%. The selection, placement and compaction of the fill should be carried out under geotechnical control. The settlement of the embankment fills prepared as described above should not exceed 30 mm for both the north and south approach embankments. The time rate of settlement of the fill making up the embankment will depend on the material used for construction and for granular fills it should substantially be completed during the construction and within a few weeks thereafter (i.e. should be essentially elastic). Clayey fills can, on the other hand, be expected to consolidate over a longer period of time. It should also be pointed out that these quoted settlements will be in addition to the foundation settlements quoted earlier.

#### 5.4 CONSTRUCTION COMMENTS

All excavations should be carried out in accordance with the latest edition of the Ontario Occupational Health and Safety Act and its regulations.

Water level measurements in the boreholes indicate that the groundwater coincides with the "normal" river level, which is at ground surface (floodplain level) at the south abutment and at between 2 m to 4 m depth at the north abutment. At the south abutment, it is anticipated that integral abutment piles can be installed from the existing ground surface (floodplain and river level) and therefore excavation requirements will be minimal. At the north abutment, excavation for slope flattening and benching may be carried out in open cut above the groundwater table, with temporary side slopes not steeper than 2 horizontal to 1 vertical. Pumping from properly filtered sumps will be required to control water seepage due to perched water and surface runoff. Due to the close proximity of the river flow channel to the proposed abutments, excavation below the groundwater (river) level, if required, will be difficult, and will likely require interlocking steel sheetpile cutoff walls driven into the underlying, relatively impervious silty clay. Care should be taken to minimize disturbance to the silty subgrade during excavation.

The permanent fill configuration affects the forward slope stability and should, therefore, be further analyzed during design, once the structural configuration and/or fill material selection are finalized, as discussed previously. The temporary, i.e. preloading fill, configuration as it relates to forward slope stability should also be checked by a Geotechnical Engineer familiar with the findings of this report and appointed by the Contract Administrator. Based on plans and profiles currently available to us, it is possible that the toe of the preloading fill slopes may slightly encroach into the river flow channel. Relevant authorities must be consulted to ensure that this is acceptable.

Erosion and scour protection should be provided to the embankment and river slopes, both during construction and permanently. Such measures of protection may include suitably graded rip-rap and/or armour stone, and should be designed by a qualified professional.

## 5.5 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 is required for frost protection of foundations.

## 6.0 CLOSURE

We recommend that once the details of the structure and approaches are finalized, our recommendations should be reviewed for their specific applicability.

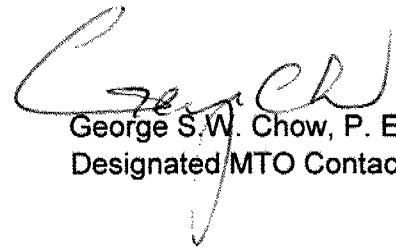
The Limitations of Report, as quoted in Appendix A, is an integral part of this report.

Sincerely,



Sydney Pang, P. Eng.

SP/dee



George S.W. Chow, P. Eng.,  
Designated MTO Contact.

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## APPENDIX A

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## AGRA

### 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 environmental 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. It is recommended practice that the Geotechnical Engineer be retained during construction to confirm that the subsurface conditions throughout the site do not deviate materially from those encountered in testholes. 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. Since all details of the design may not be known, we recommend that we be retained during the final design stage to verify that the design is consistent with our recommendations, and that assumptions made in our analysis are valid.

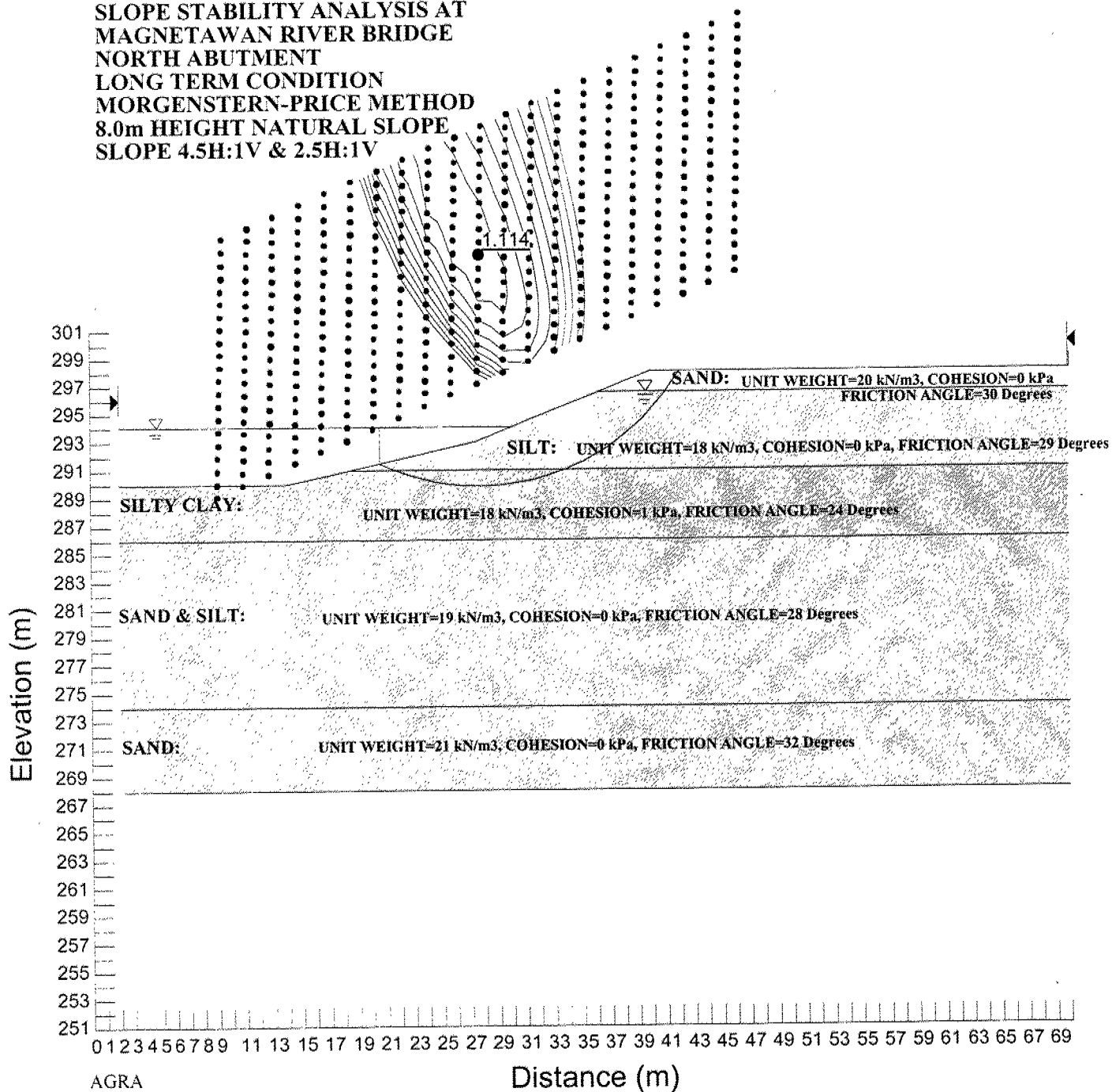
The comments made in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of testholes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of surficial 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. No other warranty is expressed or implied.

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. AGRA accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

## APPENDIX B

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FIGURE-B1 EXISTING CONDITION  
HWY 11, KATRINE  
SLOPE STABILITY ANALYSIS AT  
MAGNETAWAN RIVER BRIDGE  
NORTH ABUTMENT  
LONG TERM CONDITION  
MORGENSTERN-PRICE METHOD  
8.0m HEIGHT NATURAL SLOPE  
SLOPE 4.5H:1V & 2.5H:1V



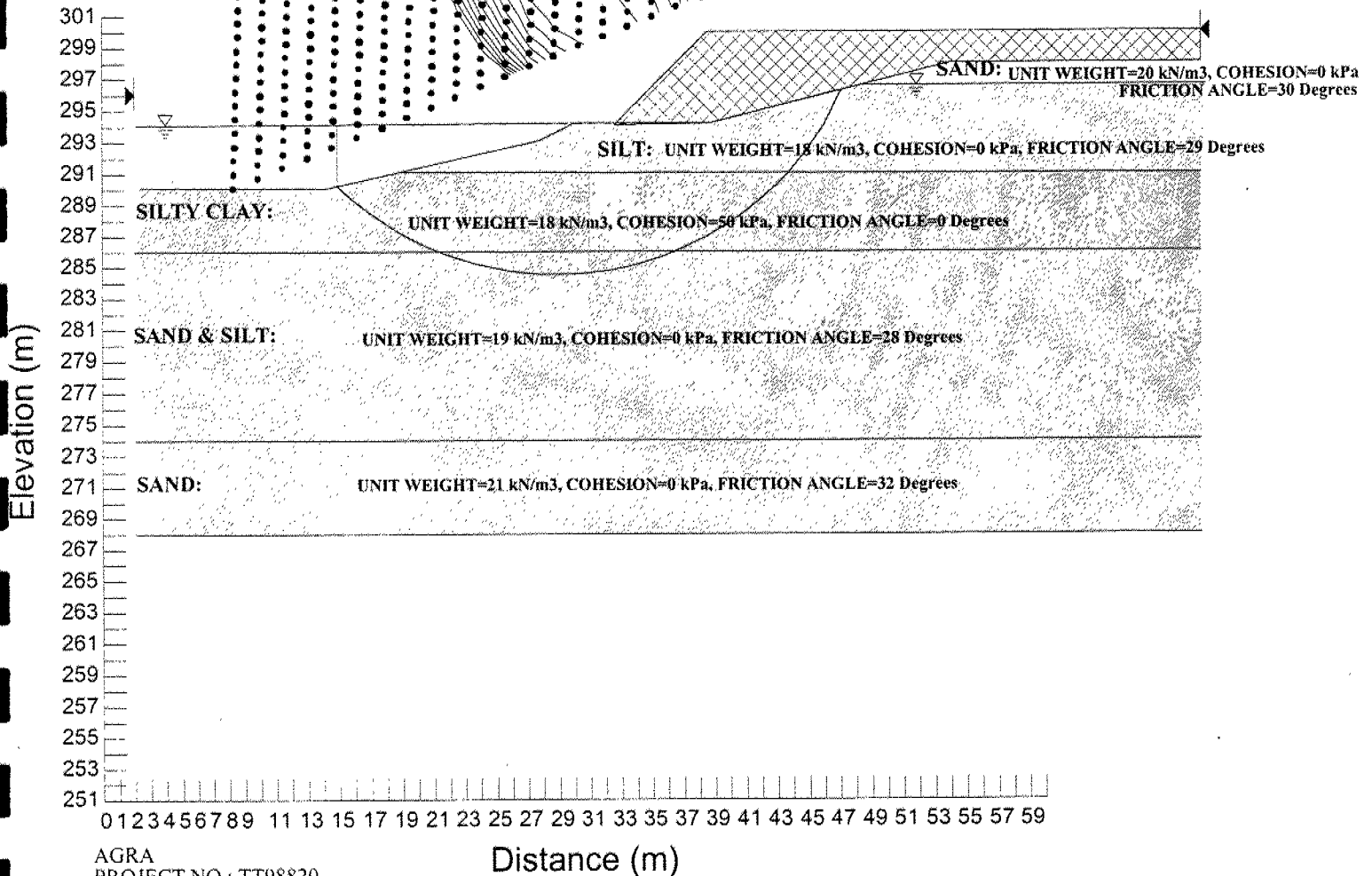
AGRA  
PROJECT NO.: TT98820  
PROJECT: HWY 11 FOUR LANING, KATRINE  
(FILE NAME: TT98HWY11MAGNETAWAN-NA2AA1P) / DATE: OCTOBER 3, 1999



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FIGURE-B2 DURING PRELOADING  
HWY 11, KATRINE  
SLOPE STABILITY ANALYSIS AT  
MAGNETAWAN RIVER BRIDGE  
NORTH ABUTMENT  
SHORT TERM CONDITION  
MORGENSTERN-PRICE METHOD  
8.0m HEIGHT NATURAL SLOPE  
SLOPE 4.5H:1V, 2H:1V  
WITH 8.8m WIDE BENCH  
SURCHARGE FORWARD SLOPE 1H:1V

NOTE: STRUCTURE MOVED 5m TOWARDS NORTH  
FROM THE DESIGNED LOCATION

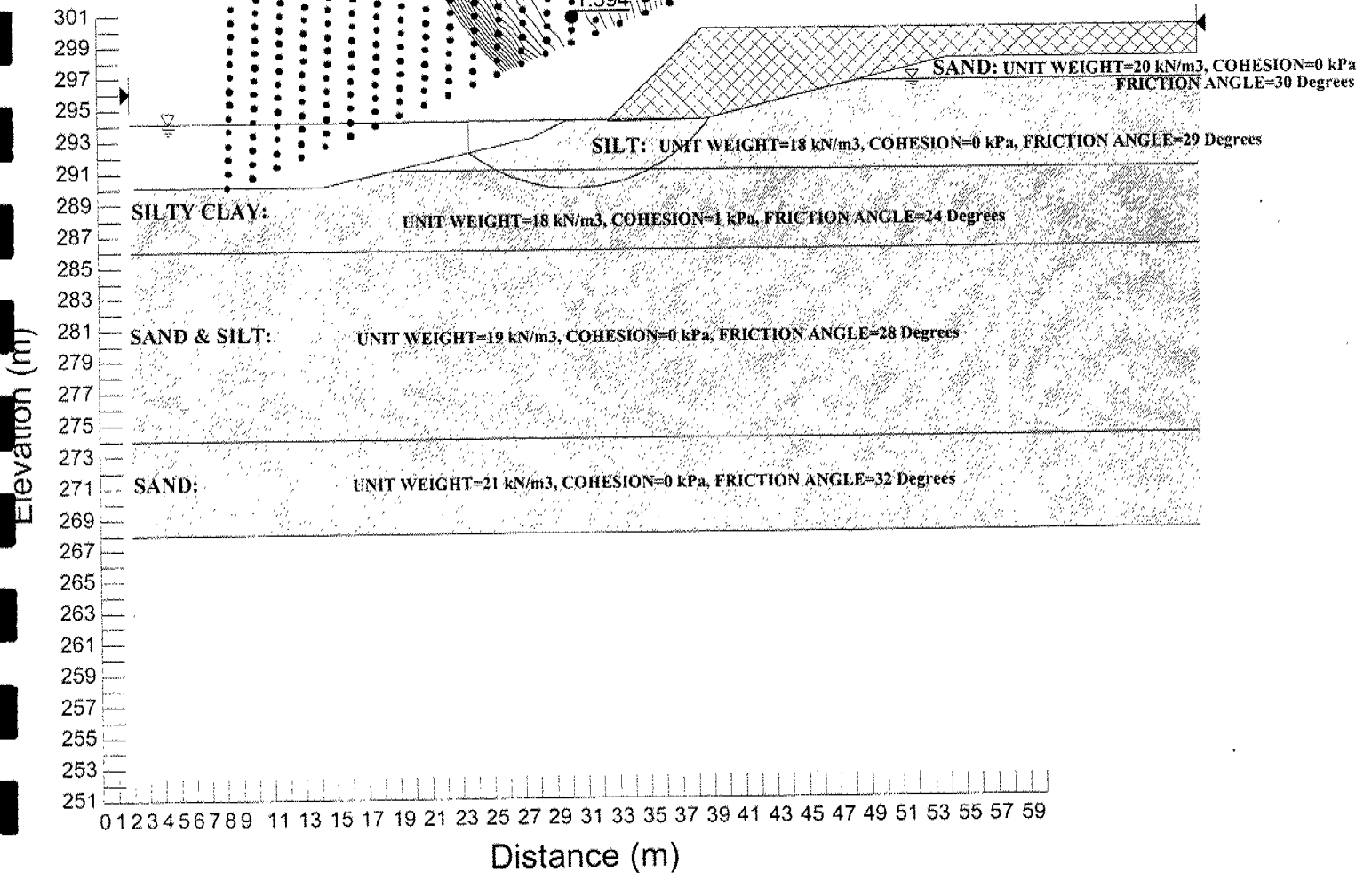


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PROJECT NO.: TT98820  
PROJECT: HWY 11 FOUR LANEING, KATRINE  
(FILE NAME: TT98HWY11MAGNETAWAN-N2A-1AP) / DATE: OCTOBER 3, 1999

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**FIGURE-B3 DURING PRELOADING  
HWY 11, KATRINE  
SLOPE STABILITY ANALYSIS AT  
MAGNETAWAN RIVER BRIDGE  
NORTH ABUTMENT  
LONG TERM CONDITION  
MORGENSTERN-PRICE METHOD  
8.0m HEIGHT NATURAL SLOPE  
SLOPE 4.5H:1V , 2H:1V  
WITH 8.8m WIDE BENCH  
SURCHARGE FORWARD SLOPE 1H:1V**

NOTE: STRUCTURE MOVED 5m TOWARDS NORTH  
FROM THE DESIGNED LOCATION

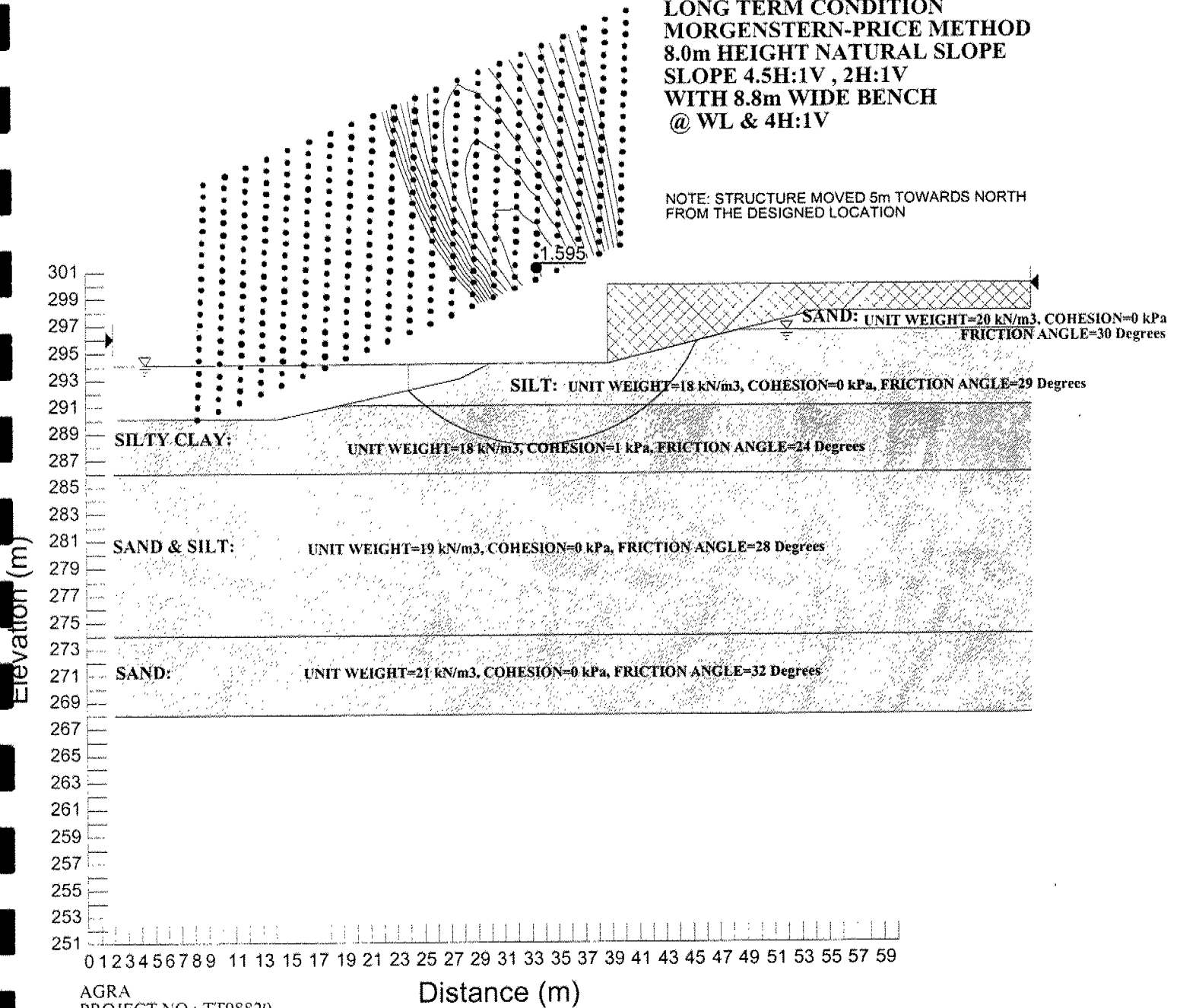


AGRA  
PROJECT NO.: TT98820  
PROJECT: HWY 11 FOUR LANING, KATRINE  
(FILE NAME: TT98HWY11MAGNETAWAN-N2A-1BP) / DATE: OCTOBER 3, 1999

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**FIGURE-B4**  
**HWY 11, KATRINE**  
**SLOPE STABILITY ANALYSIS AT**  
**MAGNETAWAN RIVER BRIDGE**  
**NORTH ABUTMENT**  
**LONG TERM CONDITION**  
**MORGENSTERN-PRICE METHOD**  
**8.0m HEIGHT NATURAL SLOPE**  
**SLOPE 4.5H:1V , 2H:1V**  
**WITH 8.8m WIDE BENCH**  
**@ WL & 4H:1V**

NOTE: STRUCTURE MOVED 5m TOWARDS NORTH  
 FROM THE DESIGNED LOCATION

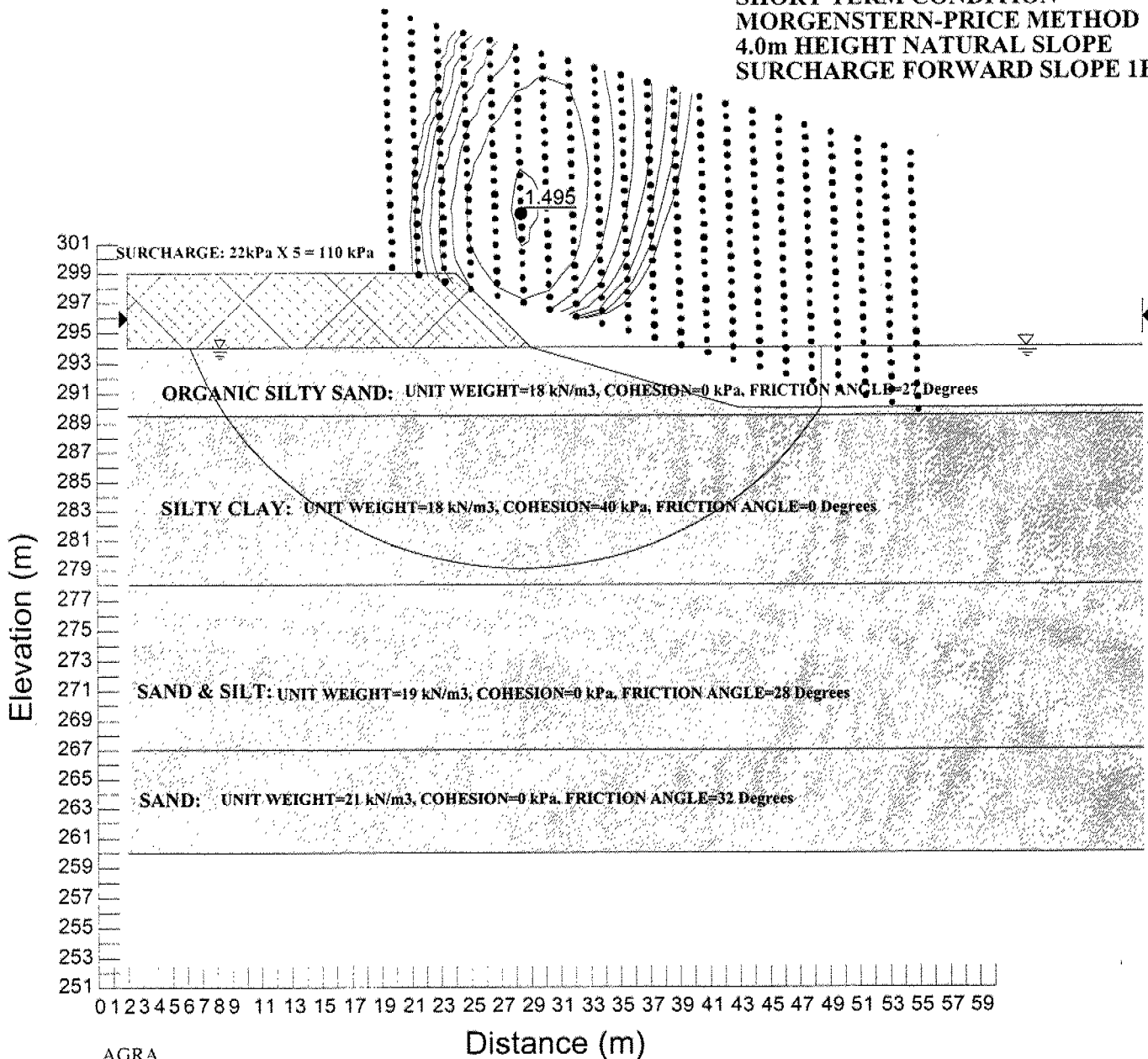


AGRA  
 PROJECT NO.: TT98820  
 PROJECT: HWY 11 FOUR LANING, KATRINE  
 (FILE NAME: TT98HWY11MAGNETAWAN-2A-1AP) / DATE: OCTOBER 3, 1999

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NOTE: STRUCTURE MOVED 3m TOWARDS SOUTH  
FROM THE DESIGNED LOCATION

FIGURE-B5 DURING PRELOADING  
HWY 11, KATRINE  
SLOPE STABILITY ANALYSIS AT  
MAGNETAWAN RIVER BRIDGE  
SOUTH ABUTMENT  
SHORT TERM CONDITION  
MORGENSTERN-PRICE METHOD  
4.0m HEIGHT NATURAL SLOPE  
SURCHARGE FORWARD SLOPE 1H:1V

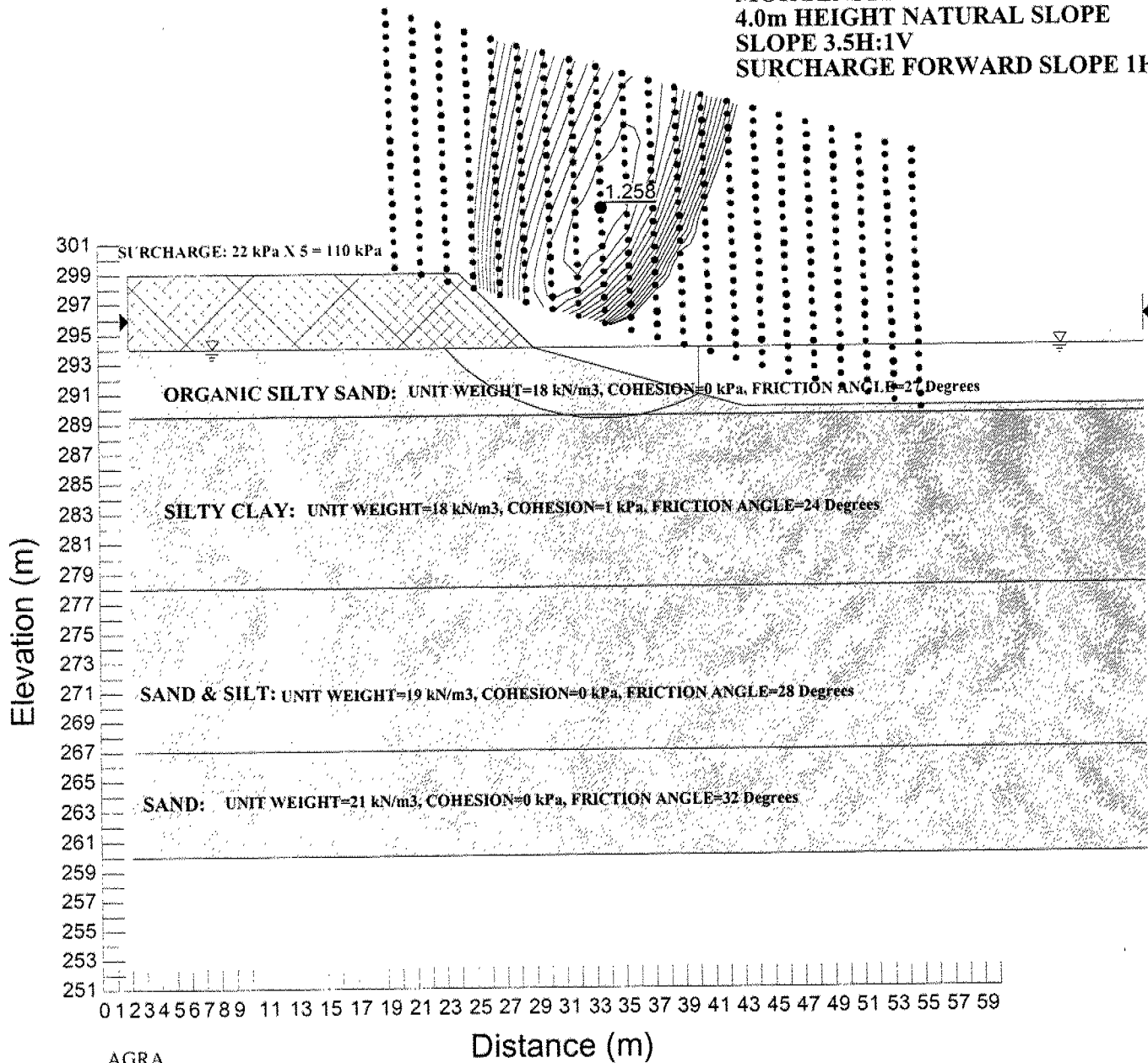


AGRA  
PROJECT NO.: TT98820  
PROJECT: HWY 11 FOUR LANE, KATRINE  
(FILE NAME: TT98HWY11MAGNETAWAN-3-IAP) / DATE: AUGUST 18, 1999

DRAFT

FIGURE-B6 DURING PRELOADING  
HWY 11, KATRINE  
SLOPE STABILITY ANALYSIS AT  
MAGNETAWAN RIVER BRIDGE  
SOUTH ABUTMENT  
LONG TERM CONDITION  
MORGENSTERN-PRICE METHOD  
4.0m HEIGHT NATURAL SLOPE  
SLOPE 3.5H:1V  
SURCHARGE FORWARD SLOPE 1H:1V

NOTE: STRUCTURE MOVED 3m TOWARDS SOUTH  
FROM THE DESIGNED LOCATION

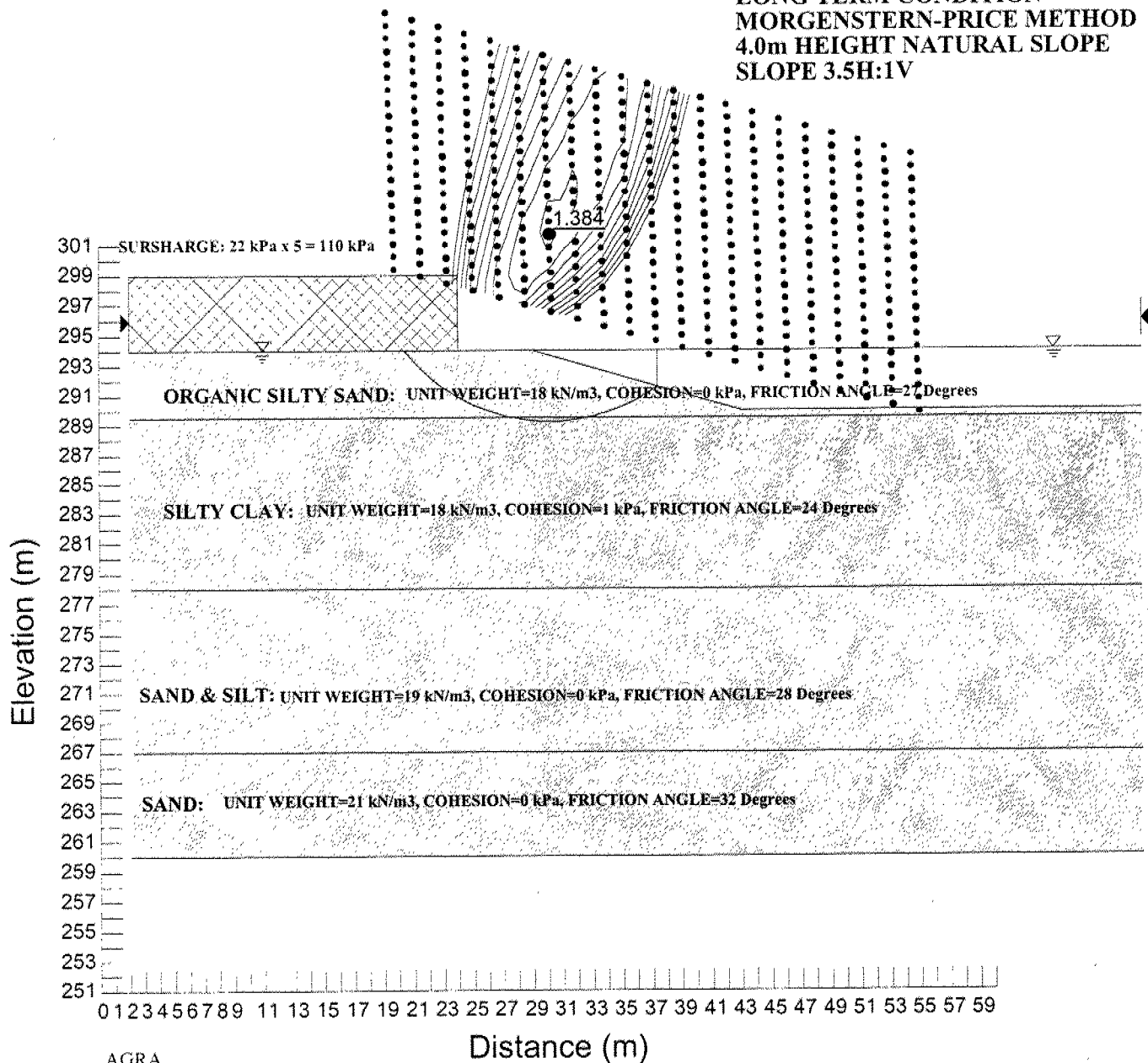


AGRA  
PROJECT NO.: TT98820  
PROJECT: HWY 11 FOUR LANING, KATRINE  
(FILE NAME: TT98HWY11MAGNETAWAN-3-IBP) / DATE: OCTOBER 3, 1999

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NOTE: STRUCTURE MOVED 3m TOWARDS SOUTH  
FROM THE DESIGNED LOCATION

FIGURE-B7  
HWY 11, KATRINE  
SLOPE STABILITY ANALYSIS AT  
MAGNETAWAN RIVER BRIDGE  
SOUTH ABUTMENT  
LONG TERM CONDITION  
MORGENSTERN-PRICE METHOD  
4.0m HEIGHT NATURAL SLOPE  
SLOPE 3.5H:1V



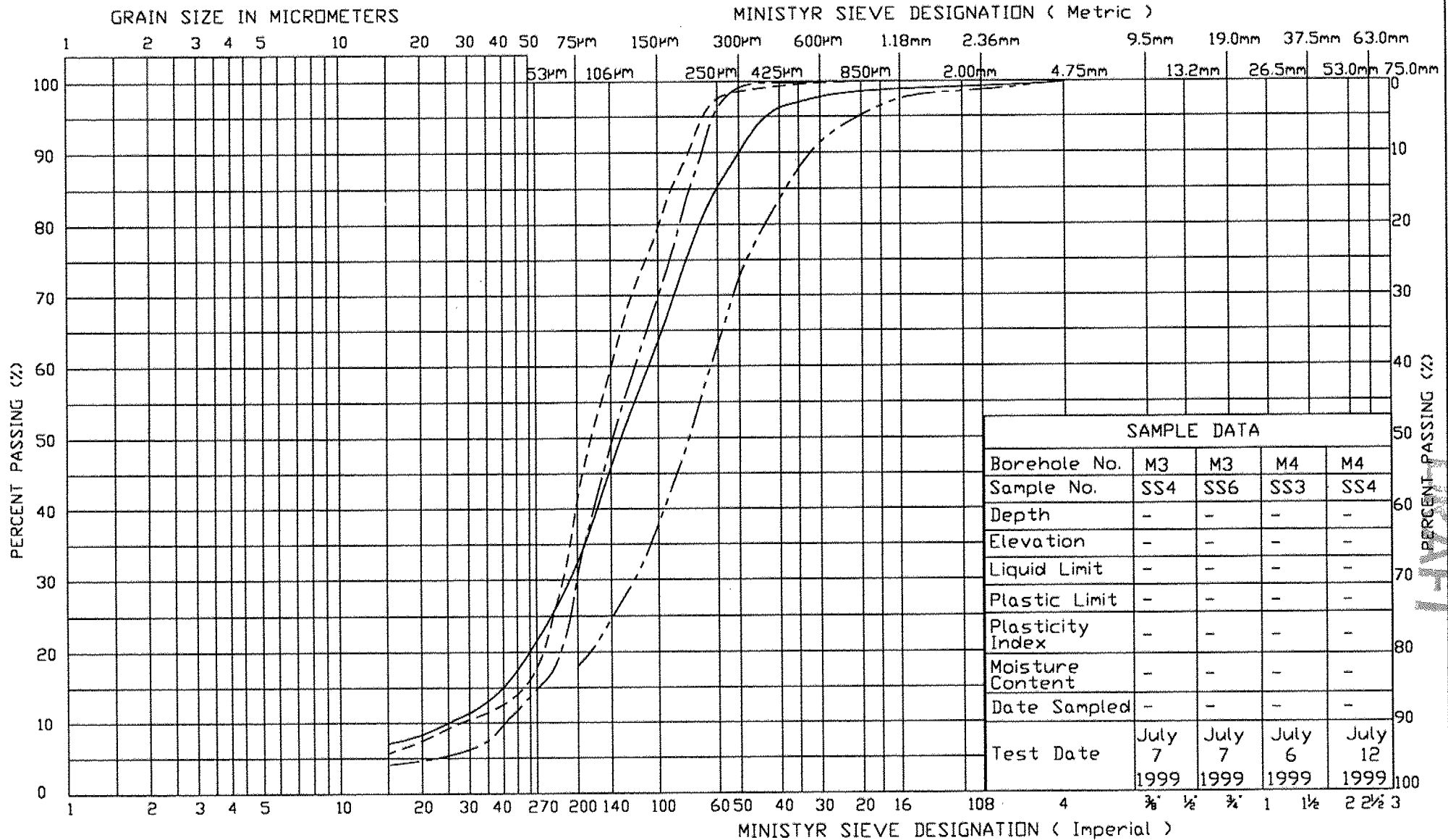
AGRA  
PROJECT NO.: TT98820  
PROJECT: HWY 11 FOUR LANING, KATRINE  
(FILE NAME: TT98HWY11MAGNETAWAN-3-1CP) / DATE: OCTOBER 3, 1999

DRAFT

## FIGURES

# UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse



**AGRA**

ENGINEERING GLOBAL SOLUTIONS

## GRAIN SIZE DISTRIBUTION

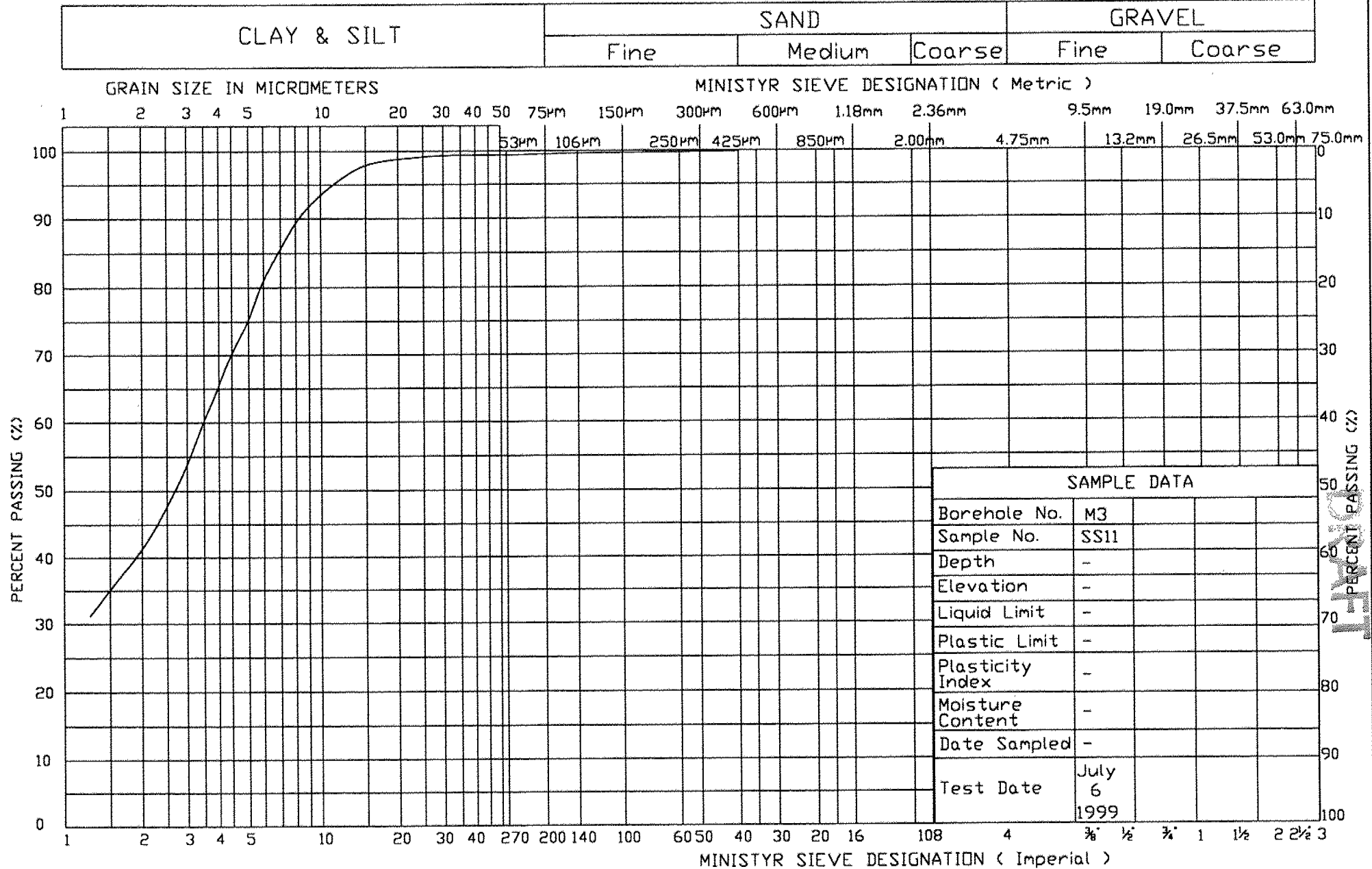
SILTY SAND WITH ORGANICS

M3: SS4	_____
M3: SS6	_____
M4: SS3	_____
M4: SS4	_____

CLIENT:	DELCAN
JOB NO.:	TT98820    W P 473-93-00
PROJECT:	HWY 11, KATRINE
LOCATION:	MAGNETAWAN RIVER BRIDGE
DATE:	AUGUST 18, 1999    FIGURE: 1



# UNIFIED SOIL CLASSIFICATION SYSTEM



SAMPLE DATA				
Borehole No.	M3			
Sample No.	SS11			
Depth	-			
Elevation	-			
Liquid Limit	-			
Plastic Limit	-			
Plasticity Index	-			
Moisture Content	-			
Date Sampled	-			
Test Date	July 6 1999			

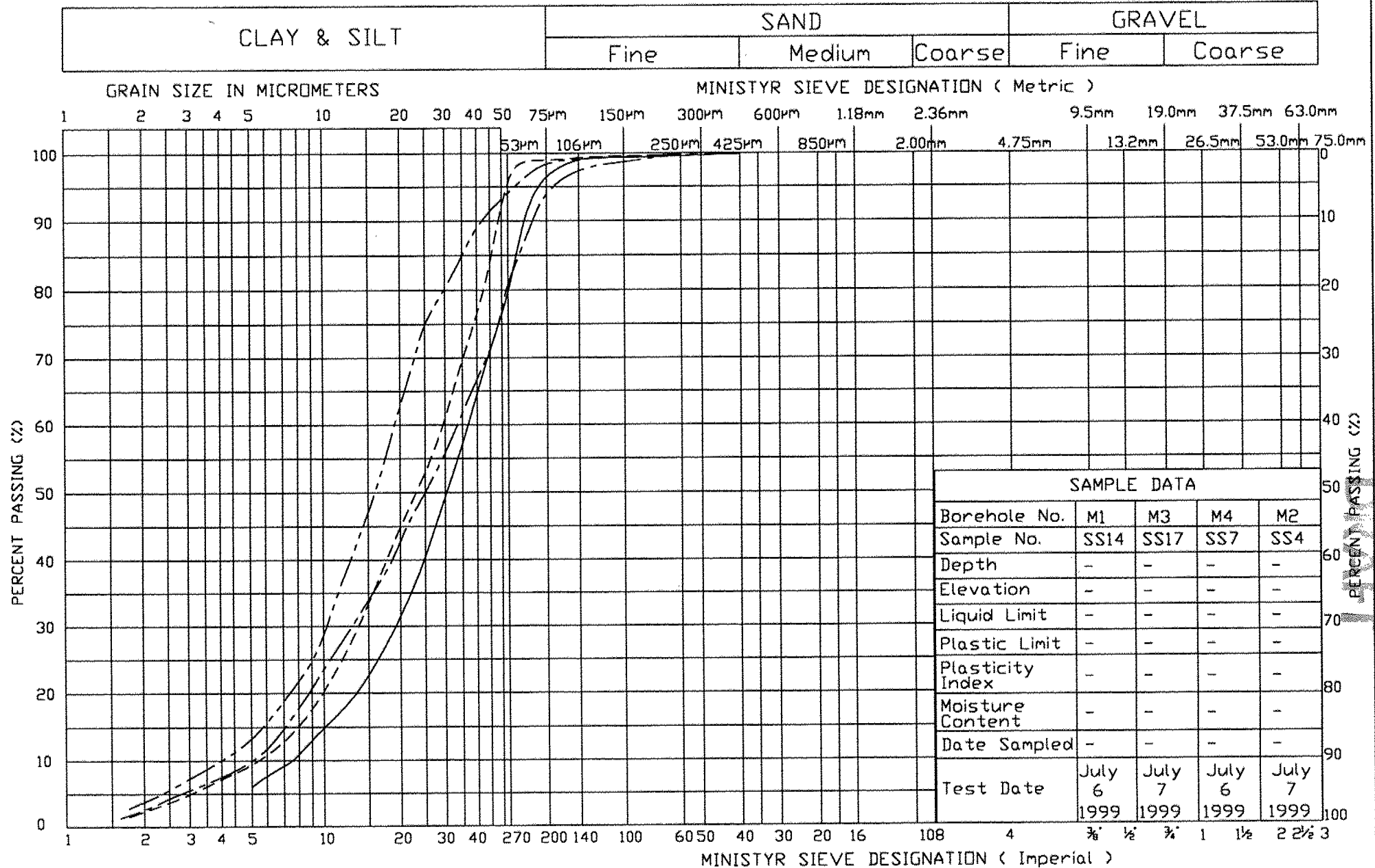
## GRAIN SIZE DISTRIBUTION

SILTY CLAY

M3: SS11

CLIENT:	DELCAN
JOB NO.:	TT98820 W P 473-93-00
PROJECT:	HWY 11, KATRINE
LOCATION:	MAGNETAWAN RIVER BRIDGE
DATE:	AUGUST 18, 1999
FIGURE:	2

# UNIFIED SOIL CLASSIFICATION SYSTEM



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## GRAIN SIZE DISTRIBUTION

SILT  
TRACE SAND

M1: SS14	_____
M3: SS17	-----
M4: SS7	-----
M2: SS4	-----

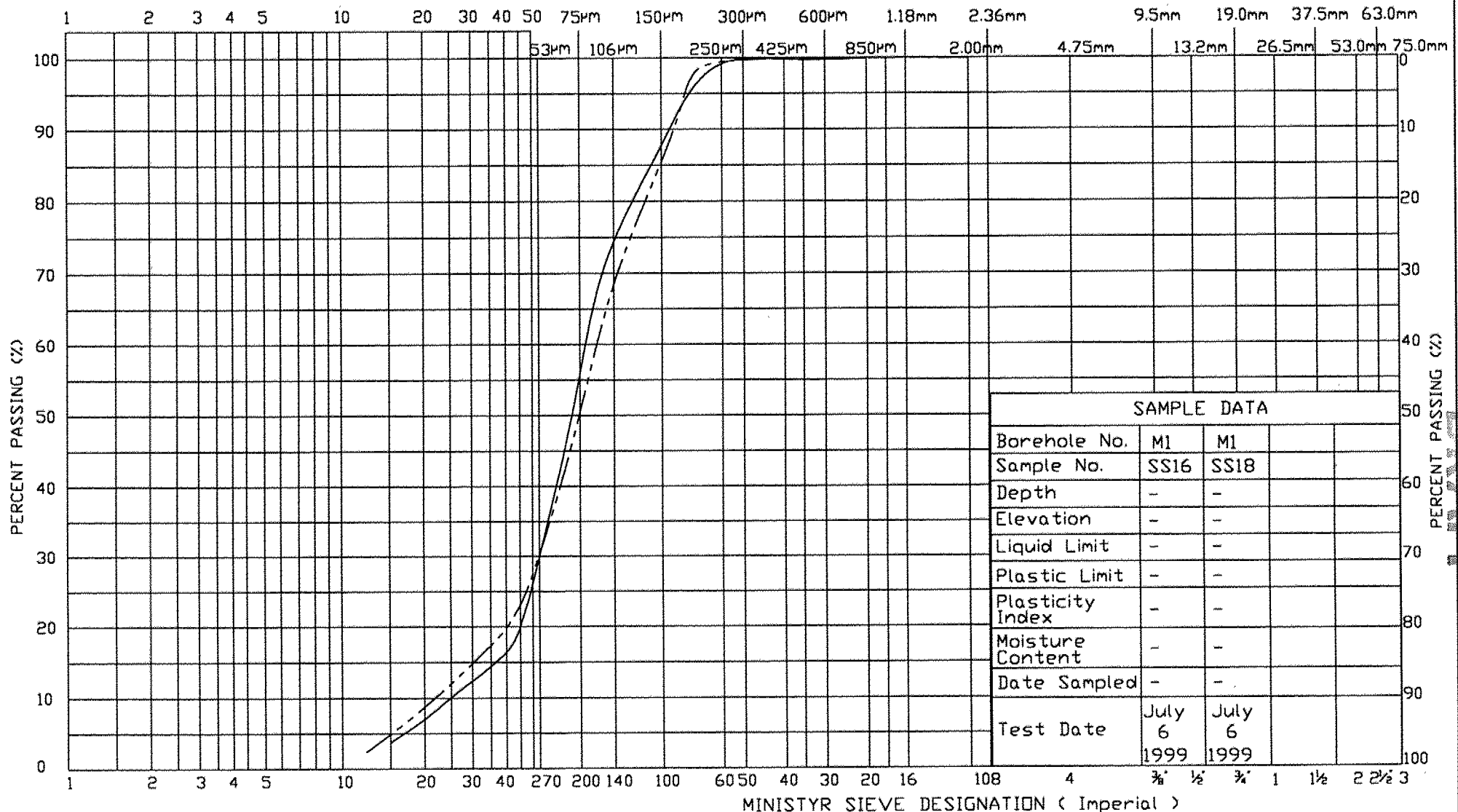
CLIENT:	DELCAN		
JOB NO.:	TT98820	W P 473-93-00	
PROJECT:	HWY 11, KATRINE		
LOCATION:	MAGNETAWAN RIVER BRIDGE		
DATE:	AUGUST 18, 1999		FIGURE: 3

# UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse

GRAIN SIZE IN MICROMETERS

MINISTYR SIEVE DESIGNATION ( Metric )



**AGRA**

ENGINEERING GLOBAL SOLUTIONS

## GRAIN SIZE DISTRIBUTION

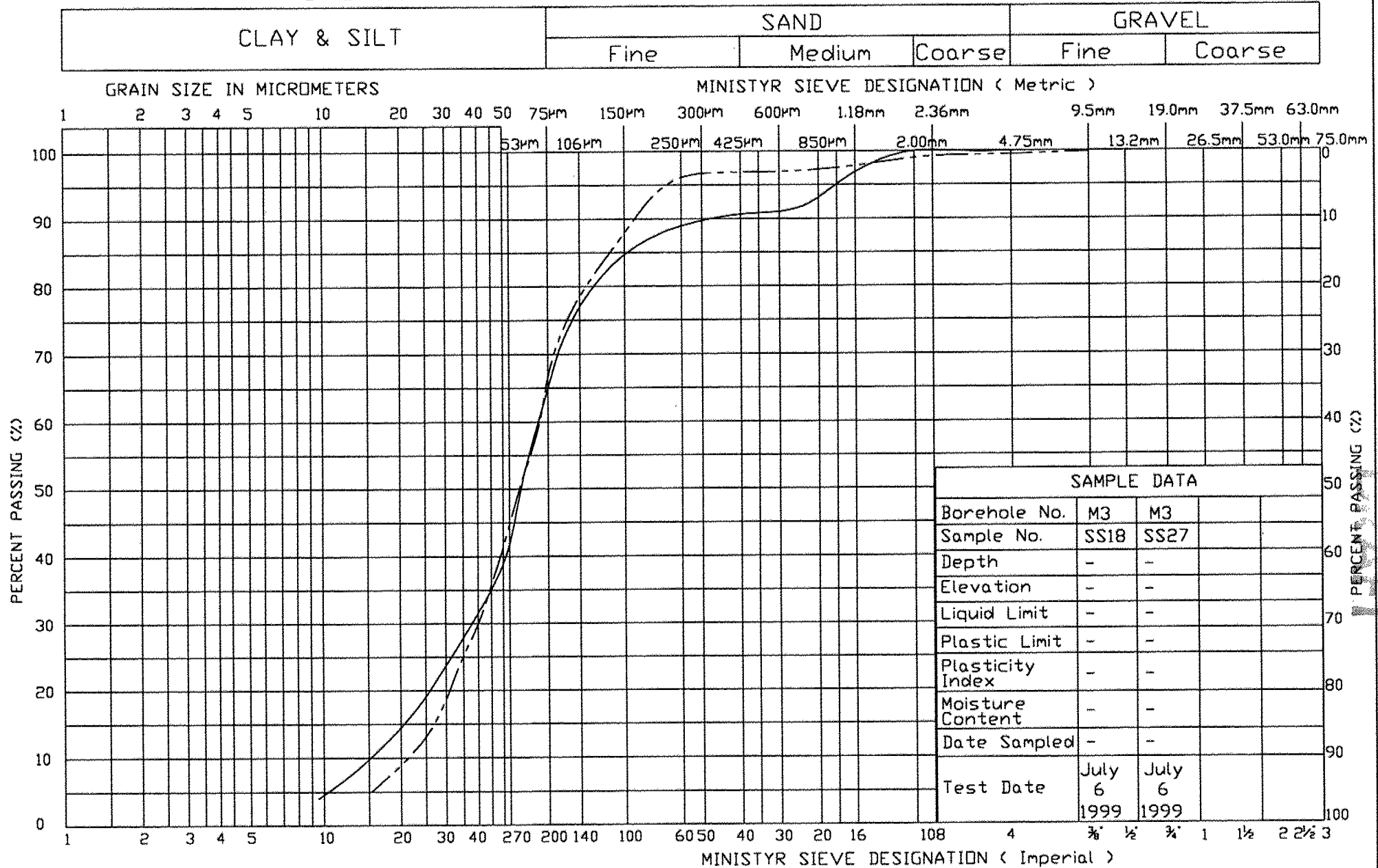
SILT and SAND

M1: SS16

M3: SS18

CLIENT:	DELCAN	
JOB NO.:	TT98820	W P 473-93-00
PROJECT:	HWY 11, KATRINE	
LOCATION:	MAGNETAWAN RIVER BRIDGE	
DATE:	AUGUST 18, 1999	FIGURE: 4

# UNIFIED SOIL CLASSIFICATION SYSTEM



**ENGINEERING GLOBAL SOLUTIONS**

## GRAIN SIZE DISTRIBUTION

SANDY SILT

M3: SS18

M3: SS27

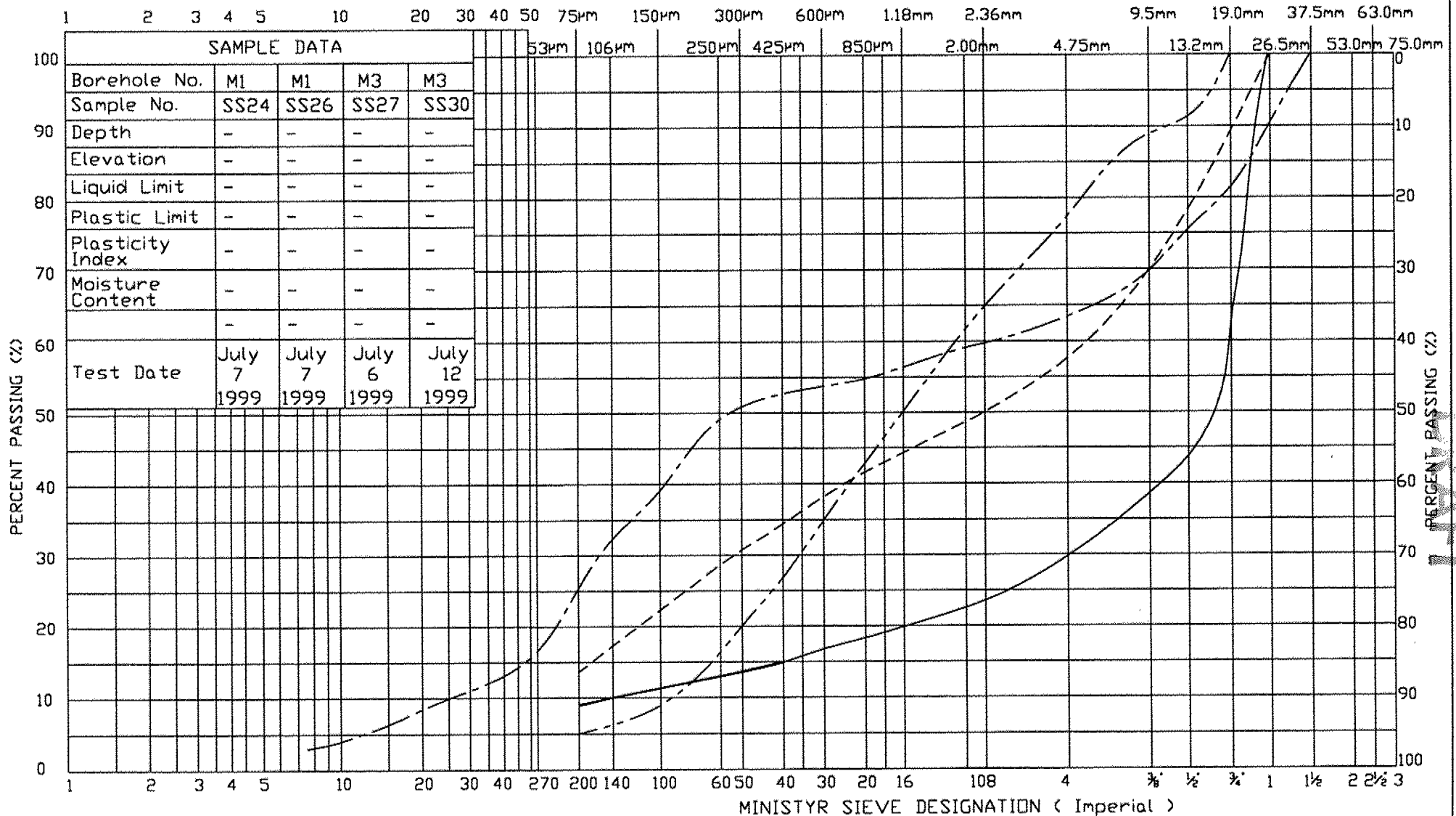
CLIENT:	DELCAN	
JOB NO.:	TT98820	W P 473-93-00
PROJECT:	HWY 11, KATRINE	
LOCATION:	MAGNETAWAN RIVER BRIDGE	
DATE:	AUGUST 18, 1999	FIGURE: 5

# UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse

GRAIN SIZE IN MICROMETERS

MINISTYR SIEVE DESIGNATION ( Metric )

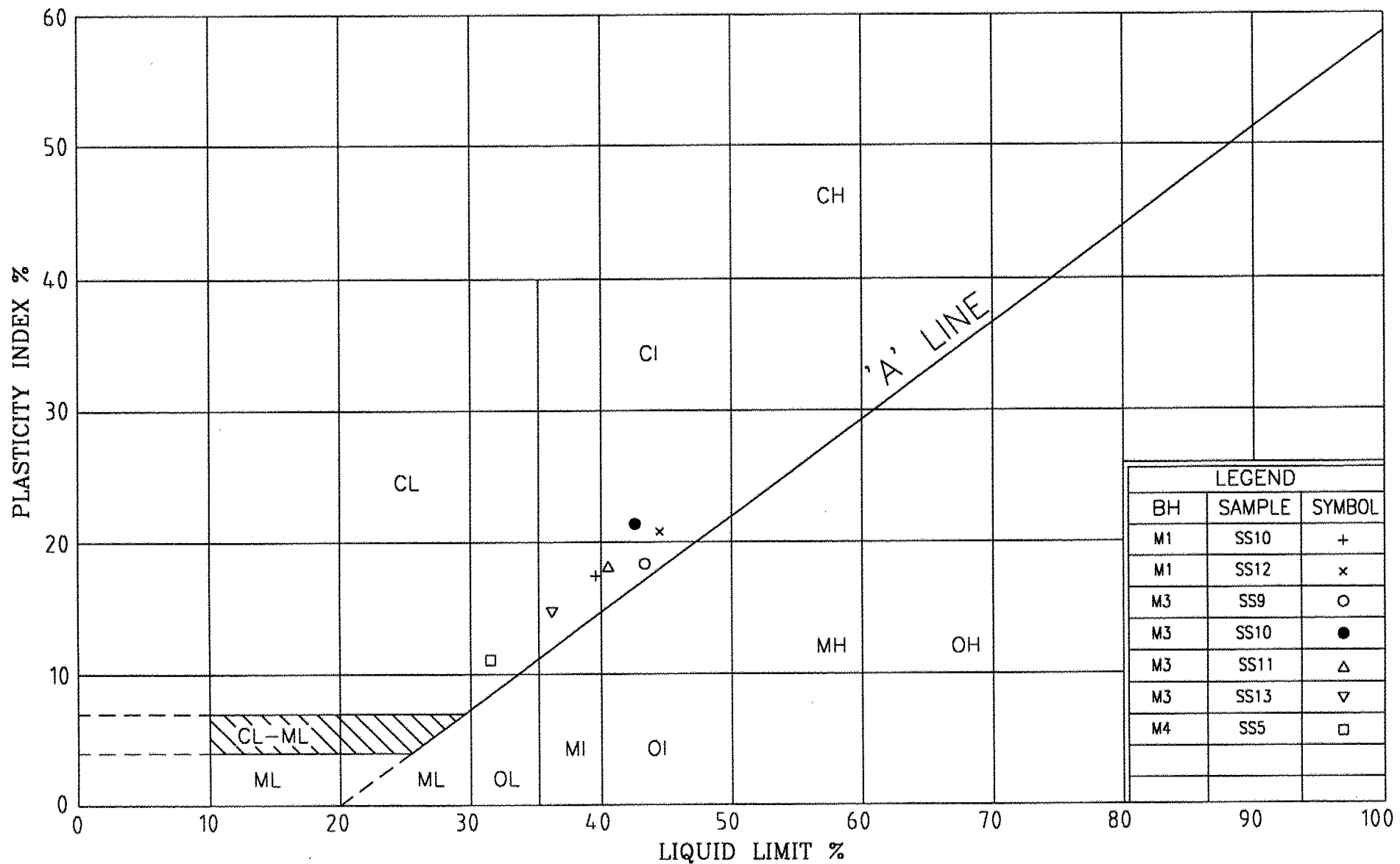


## GRAIN SIZE DISTRIBUTION

SAND and GRAVEL  
to  
GRAVELLY SAND

M1: SS24	_____
M1: SS26	_____
M3: SS27	_____
M3: SS30	_____

CLIENT:	DELCAN		
JOB NO.:	TT98820	W P 473-93-00	
PROJECT:	HWY 11, KATRINE		
LOCATION:	MAGNETAWAN RIVER BRIDGE		
DATE:	AUGUST 18, 1999	FIGURE: 6	



DRAFT

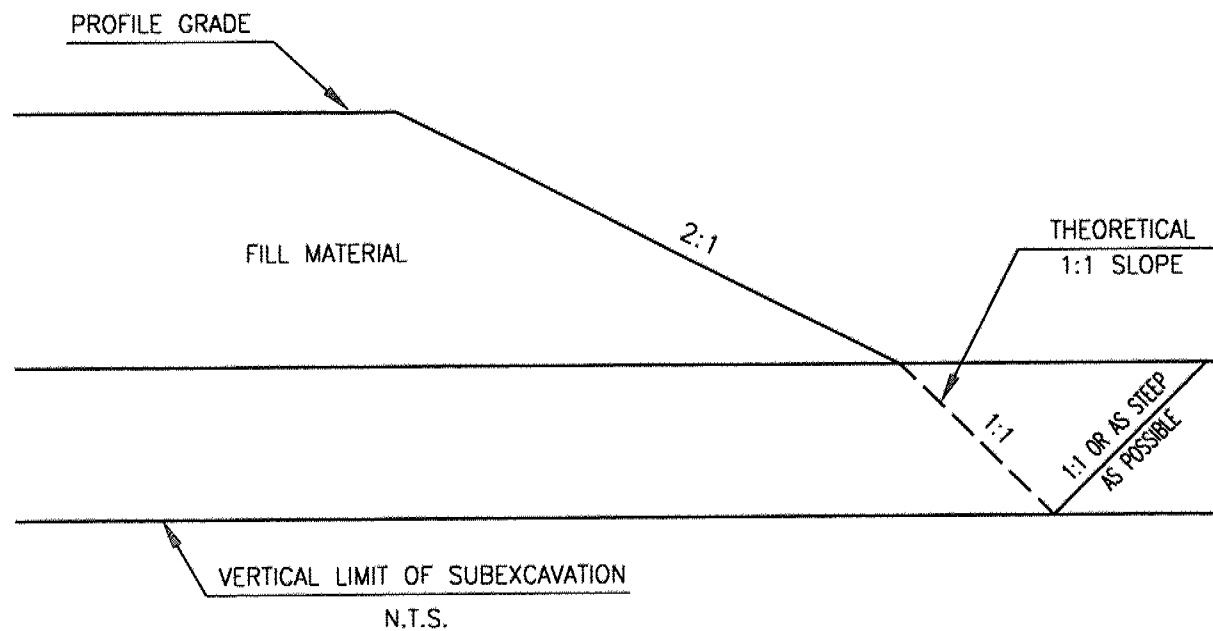


**AGRA**

ENGINEERING GLOBAL SOLUTIONS

# PLASTICITY CHART SILTY CLAY

FIGURE NO. 7  
W P 473-93-00



DRAFT



REMOVAL OF UNSUITABLE SOILS  
FROM BENEATH APPROACH FILLS  
( N.T.S. )

FIGURE No 8  
W P 473-93-00

DRAFT

ENCLOSURES



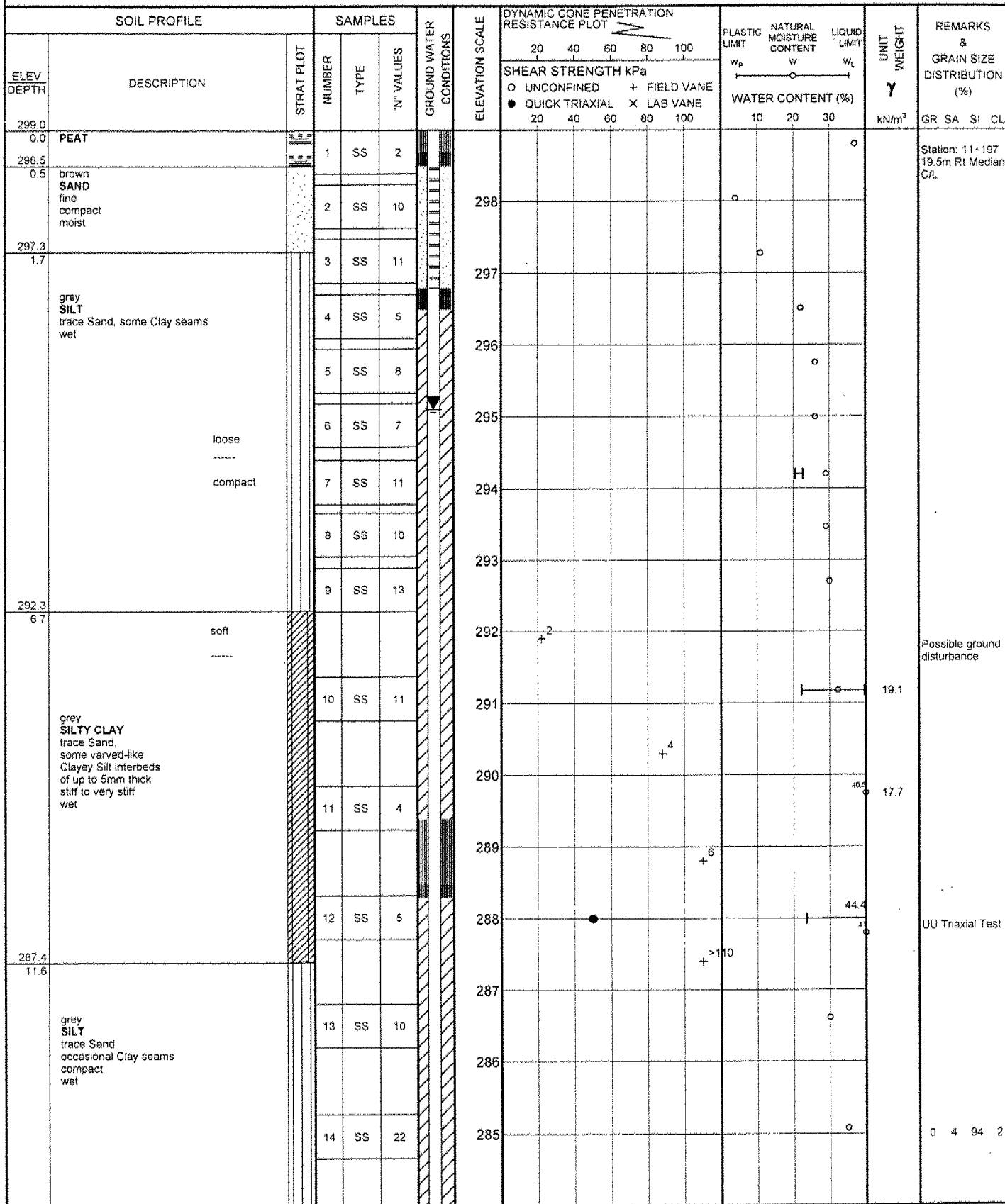
DRAFT

RECORD OF BOREHOLE No M1

1 OF 3

METRIC

W.P. 473-93-00 LOCATION NORTHING 5047566.8 EASTING 316708.5 ORIGINATED BY AD  
 DIST 52 HWY 11 BOREHOLE TYPE Solid Stem Augering/ Wash boring / Coring COMPILED BY SP  
 DATUM Geodetic DATE 30 March 1999 - 31 March 1999 CHECKED BY SP



Possible ground disturbance

UU Triaxial Test

0 4 94 2

# RECORD OF BOREHOLE No M1

2 OF 3

## METRIC

W.P.	473-93-00	LOCATION	NORTHING 5047566.8 EASTING 316708.5	ORIGINATED BY	AD
DIST	52	HWY	11	BOREHOLE TYPE	Solid Stem Augering / Wash boring / Coring
DATUM	Geodetic	DATE	30 March 1999 - 31 March 1999	COMPILED BY	SP
				CHECKED BY	SP

[illegible]

Continued Next Page

+ 3, X 3: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

# RECORD OF BOREHOLE No M1

3 OF 3

**METRIC**

W.P.	473-93-00	LOCATION	NORTHING 5047566.8 EASTING 316708.5	ORIGINATED BY	AD
DIST	52	HWY	11	BOREHOLE TYPE	Solid Stem Augering/ Wash boring / Coring
DATUM	Geodetic	DATE	30 March 1999 - 31 March 1999	COMPILED BY	SP
				CHECKED BY	SP

[illegible]

+ 3, × 3; Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No M2

1 OF 1

METRIC

W.P. 473-93-00 LOCATION NORTHING 5047576.8 EASTING 316702.9 ORIGINATED BY AD  
 DIST 52 HWY 11 BOREHOLE TYPE Solid Stem Augering COMPILED BY SP  
 DATUM Geodetic DATE 7 April 1999 CHECKED BY SP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
299.4								20 40 60 80 100						
299.0	PEAT							20 40 60 80 100						
0.2	0.3m brown SILTY SAND with Organics		1	SS	5		299							Station: 11+209 19.7m Rt Median C/L
	SAND fine compact moist		2	SS	12									
298.1			3	SS	13		298							
1.3			4	SS	9		297							0 6 92 2
	brown SILT trace Sand occasional Sand and Clay seams loose to compact wet													
	grey		5	SS	11		296							
			6	SS	9		295							
			7	TW	-		294							
293.5														
5.9	grey SILTY CLAY some varved-like Clay Silt interbeds of up to 5mm thick very stiff wet		8	SS	15		293							
292.2														
7.2	END OF BOREHOLE													
	Water level in open borehole at Elev. 297.6m on completion of drilling.													

# RECORD OF BOREHOLE No M3

1 OF 3

## METRIC

W.P.	473-93-00	LOCATION	NORTHING 5047514.0 EASTING 316734.5	ORIGINATED BY	AD
DIST	52	HWY	11	BOREHOLE TYPE	Hollow Stem Augering / Wash boring / Coring
DATUM	Geodetic	DATE	14 April 1999 - 16 April 1999	CHECKED BY	SP

[illegible]

Continued Next Page

+ 3, X 3: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

# RECORD OF BOREHOLE No M3

2 OF 3

METRIC

W.P.	473-93-00	LOCATION	NORTHING 5047514.0 EASTING 316734.5	ORIGINATED BY	AD
DIST	52	HWY	11	BOREHOLE TYPE	Hollow Stem Augering / Wash boring / Conng
DATUM	Geodetic	DATE	14 April 1999 - 16 April 1999	COMPILED BY	SP
				CHECKED BY	SP

[illegible]

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No M3

3 OF 3

METRIC

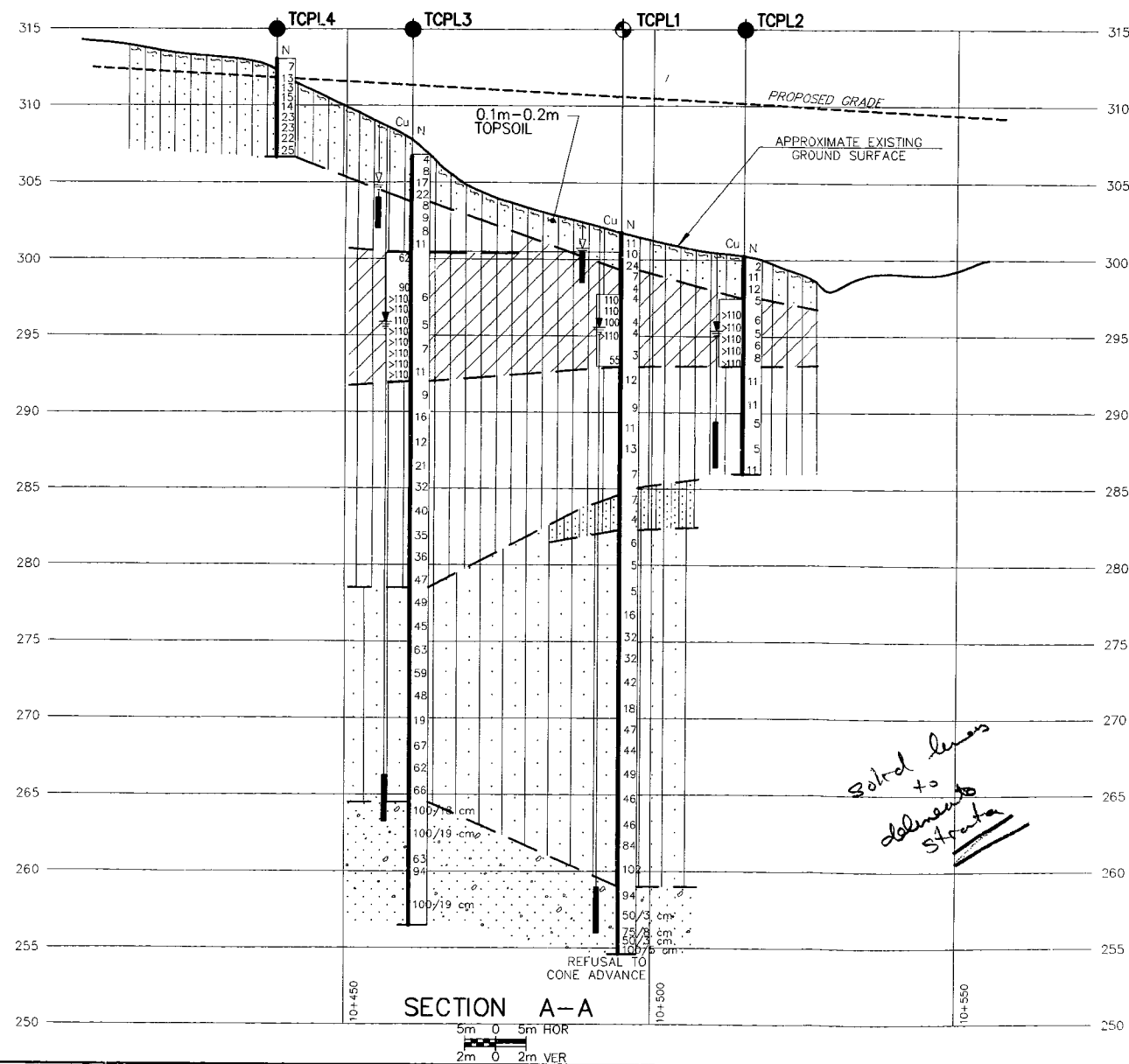
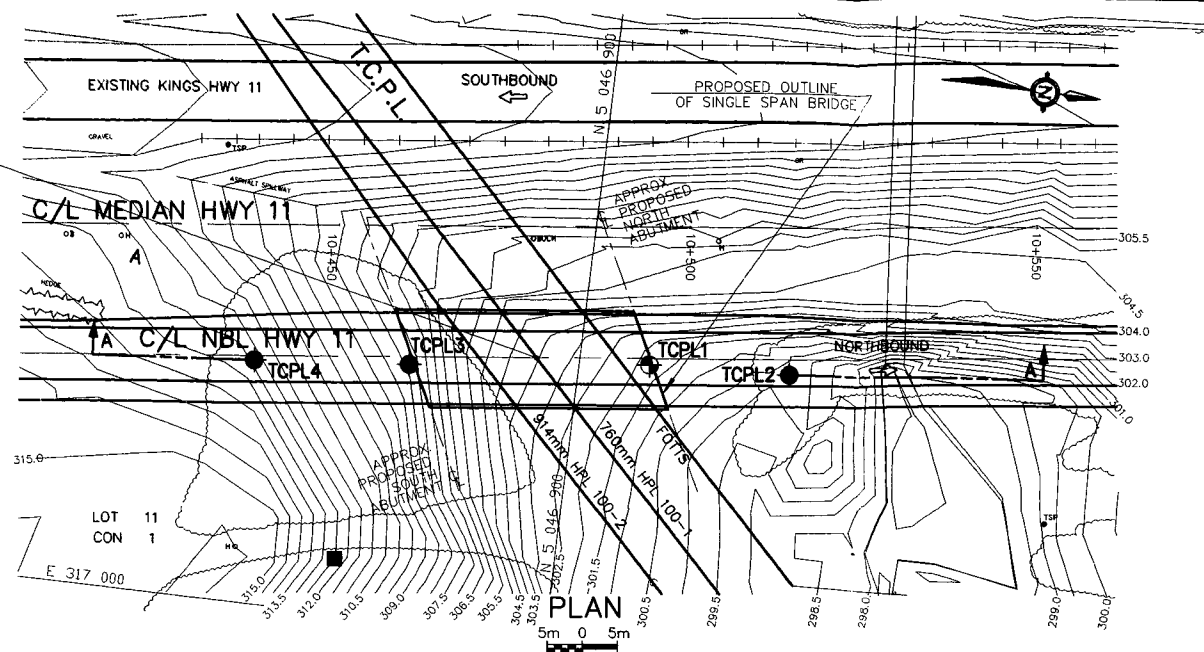
W.P. 473-93-00 LOCATION NORTHING 5047514.0 EASTING 316734.5 ORIGINATED BY AD  
 DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augering / Wash boring / Coring COMPILED BY SP  
 DATUM Geodetic DATE 14 April 1999 - 16 April 1999 CHECKED BY SP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100					
		</												

RECORD OF BOREHOLE No M4										1 OF 1	METRIC			
W.P. 473-93-00		LOCATION		NORTHING 5047496.4 EASTING 316748.0				ORIGINATED BY AD						
DIST 52 HWY 11		BOREHOLE TYPE		Solid Stem Augering/ Wash boring				COMPILED BY SP						
DATUM Geodetic		DATE		27 April 1999				CHECKED BY SP						
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
295.6 0.0	0.2m TOPSOIL		1	SS	7		20 40 60 80 100	20 40 60 80 100	10 20 30					Station: 11+116 19.1m Rt Median C/L
	SILTY SAND layers & inclusions of Wood chips & decomposed Organics very loose wet		2	SS	2									
			3	SS	4									Organic Content = 2.4%
			4	SS	1									Organic Content = 4.5%
289.9 5.7		grey SILTY CLAY varved Clayey Silt interbeds of up to 5mm thick very stiff wet		5	SS	4								0 60 40 0
288.3 7.3	grey SILT trace Sand, some Clay seams compact wet		6	SS	15									0 70 30 0
286.0 9.6	END OF BOREHOLE		7	SS	15									0 1 97 2
	Piezometer tip at Elevation 286.5m. Water level in piezometers: July 9/99: Elev. 294.8m													



HOT 10+78.221 C/L MEDIAN  
ELEV. 311.023 (18 750 m RT.)  
= 10+000.000 C/L HPL 100-1  
N 5 046 894.059  
E 316 959.894



#### SOIL STRATIGRAPHY LEGEND

- SURFICIAL SAND TO SILT  
Very Loose to Compact
- SILTY CLAY  
Stiff to Very Stiff
- SILT  
SOME SAND  
Loose to Dense
- SANDY SILT  
Very Loose to Loose
- SAND & SILT  
Loose to Very Dense
- SAND  
TRACE TO SOME GRAVEL  
FREQUENT COBBLES, SOME BOULDERS  
Very Dense

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

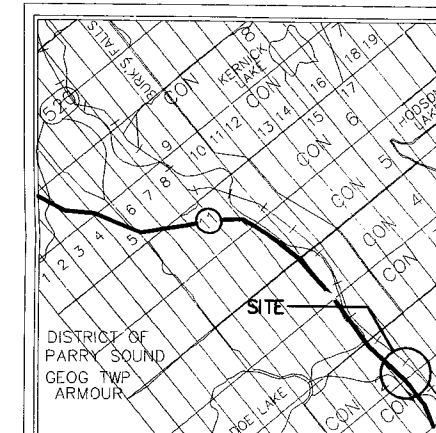
DIST. 52 HWY 11  
W.P. No. 473-93-00

TRANS-CANADA GAS PIPE LINE CROSSING  
BORE HOLE LOCATIONS & SOIL STRATA



SHEET  
1

AGRA Earth & Environmental Ltd.



#### LEGEND

- Bore Hole
- Dynamic Cone Penetration Test (Cone)
- Bore Hole & Cone
- Blows/0.3m (Std Pen Test, 475 J/blow)
- Field Vane Shear Strength (kPa)
- Blows/0.3m (60° Cone, 475 J/blow)
- WL in Piezometer (Upper) on July 9, 1999
- WL in Piezometer (Lower) on July 9, 1999
- Piezometer

No	ELEVATION	CO-ORDINATES NORTHING	EASTING
TCPL1	301.5	5 046 910	316 959
TCPL2	300.1	5 046 930	316 958
TCPL3	306.7	5 046 876	316 963
TCPL4	313.1	5 046 854	316 965

#### -NOTE-

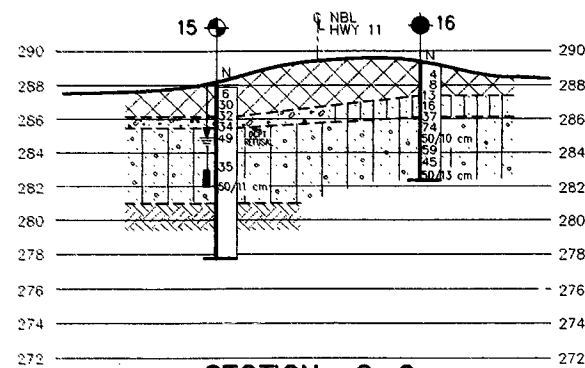
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

NOTE: The complete foundation investigation and design report for this project and other related documents must 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 30.2.3 of OPS Gen. Cond.

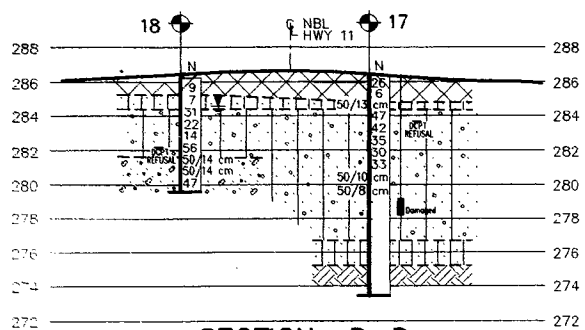
REV	DATE	BY	DESCRIPTION
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REF. Hwy 11 Bridge Site Plan  
Dwg. by MTO, June, 1999

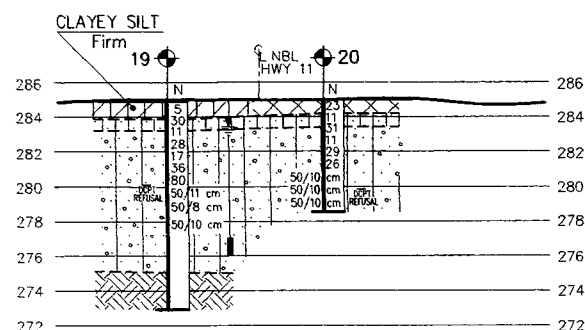
HWY No. 11	SUBMITTAL	CHECKED	DATE	DIST. PARRY SOUND
11	SP	SP	Jul, 1999	SITE
DRAWN	MA	CHECKED	MA	DATE



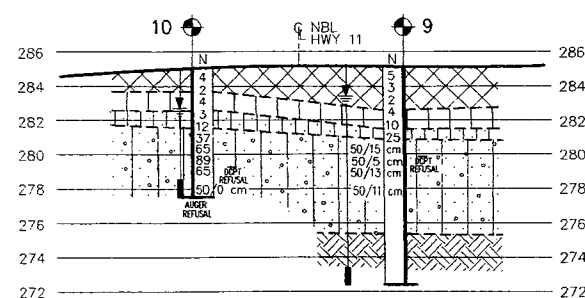
SECTION C-C



SECTION D-D



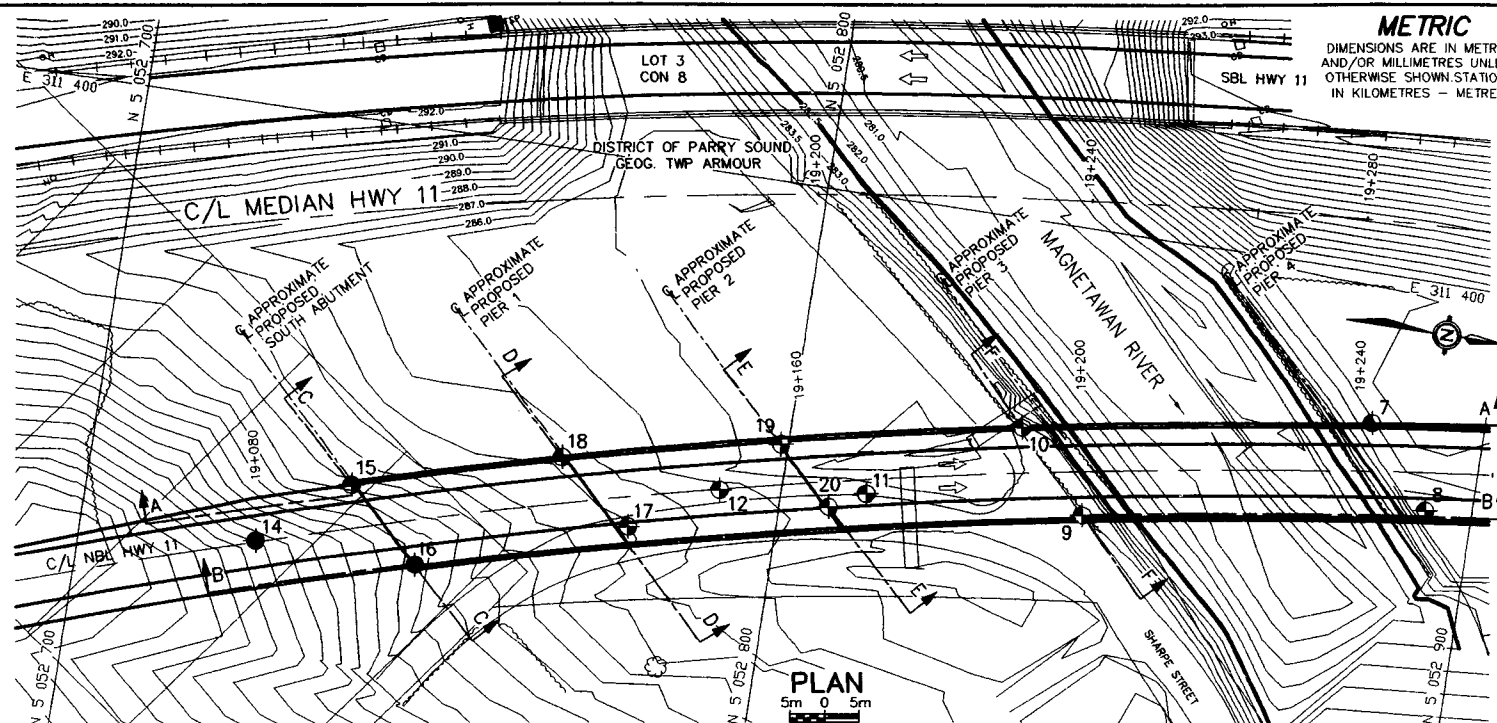
SECTION E-E



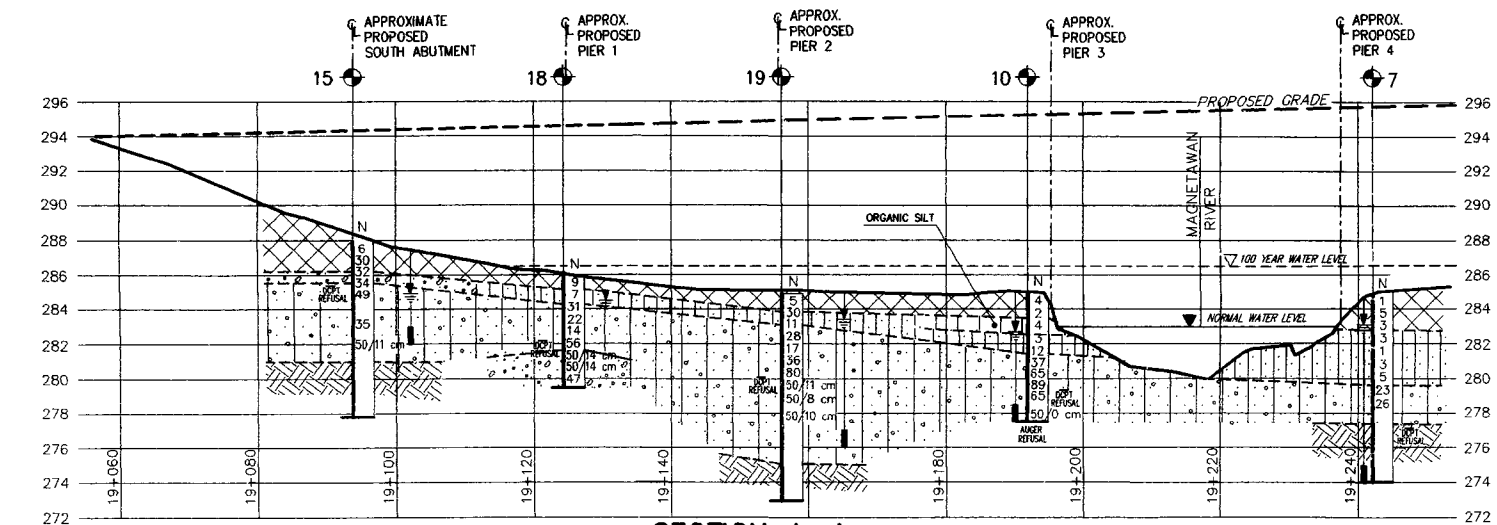
SECTION F-F

SOIL STRATIGRAPHY LEGEND

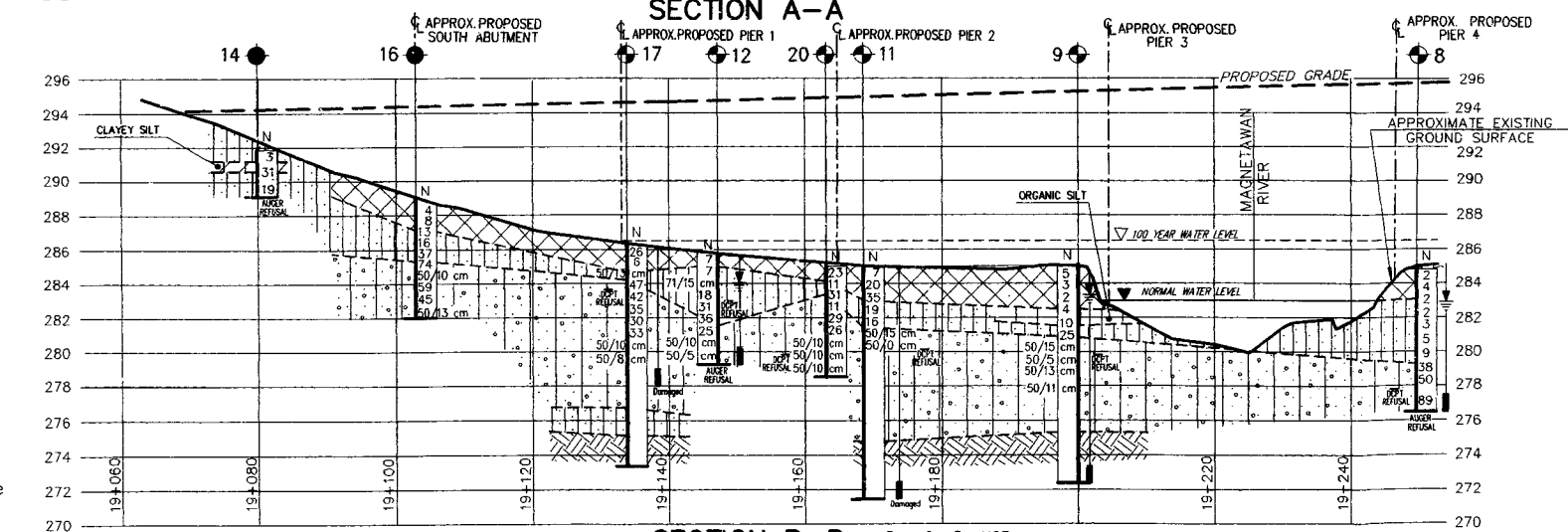
	FILL		GRAVELLY SAND TO SAND & GRAVEL
	Very Loose to Compact		Dense to Very Dense
	SAND, SILTY SAND & SANDY SILT		HETEROGENEOUS MIXTURE OF SAND, SILT & GRAVEL (GLACIAL TILL)
	Very Loose to Dense		Compact to Very Dense
	ORGANIC SILT		GRANITE BEDROCK
	Very Loose to Loose		



PLAN



SECTION A-A



SECTION B-B

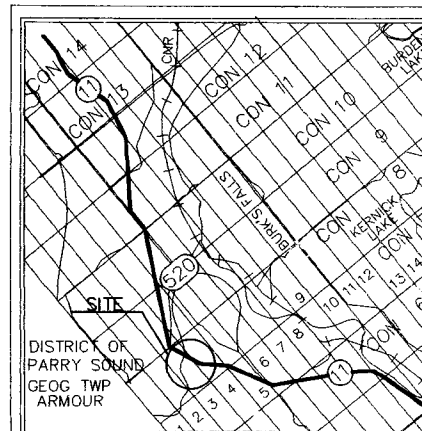
REF. Hwy 11 Bridge Site Plan  
Dwg. by MTO, Aug. 1999

SITE. No. 44-188  
W.P. No. 473-93-00

PROPOSED HIGHWAY 11 NBL  
MAGNETAWAN RIVER/HWY 520 OVERPASS  
BORE HOLE LOCATIONS & SOIL STRATA

SHEET  
1

AGRA Earth & Environmental Ltd.



KEY PLAN

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—Sept, 1999
- WL in Piezometer (stabilized)
- Piezometer

No	ELEVATION	CO-ORDINATES NORTH	EAST
7	284.8	5 052 883	311 422
8	284.7	5 052 893	311 433
9	285.2	5 052 844	311 441
10	285.1	5 052 833	311 430
11	285.1	5 052 812	311 443
12	285.8	5 052 791	311 446
14	291.9	5 052 726	311 463
15	287.9	5 052 738	311 453
16	TO BE PROVIDED		
17	286.4	5 052 779	311 453
18	286.1	5 052 768	311 445
19	284.8	5 052 799	311 438
20	285.0	5 052 807	311 446

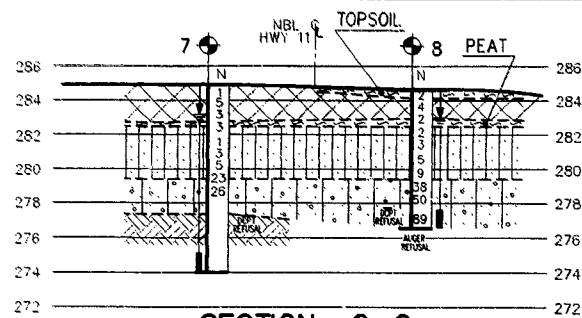
—NOTE—

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

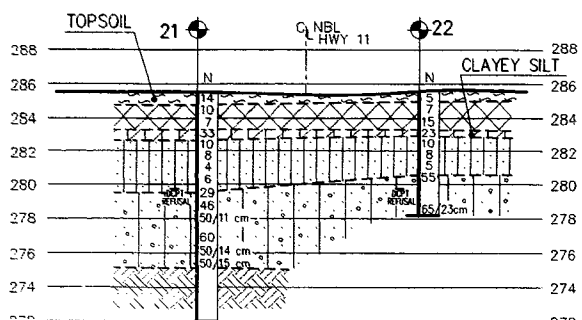
NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section GC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

REV	DATE	BY	DESCRIPTION
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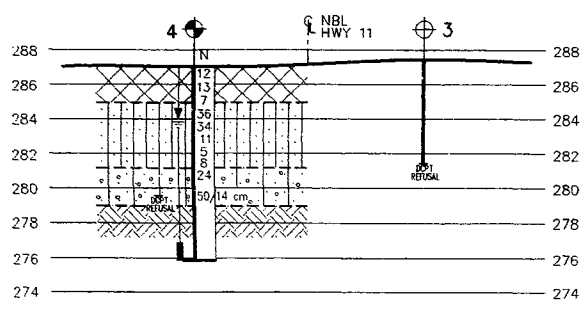
HWY No 11	SUBM'D SP	CHECKED SP	DATE November, 1999	DIST 52
DRAWN MA	CHECKED	APPROVED		SITE 44-188
				DWG 1



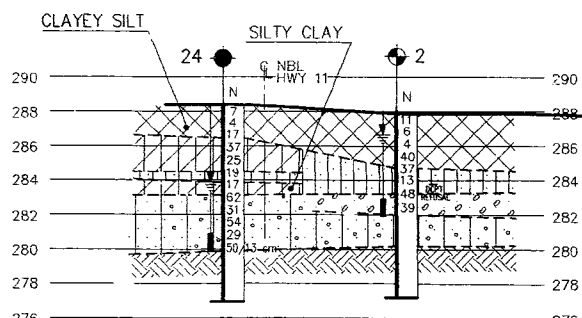
SECTION C-C



SECTION D-D



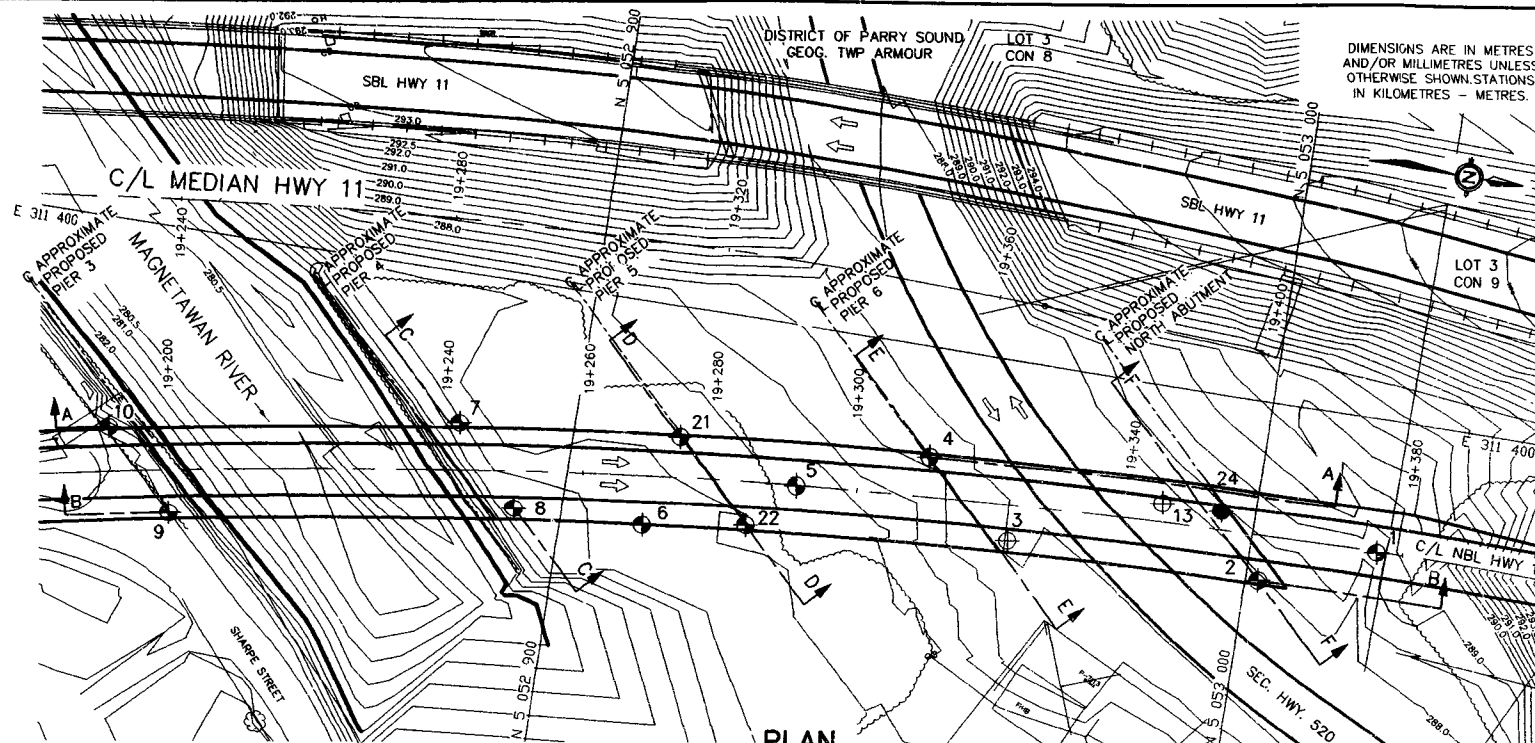
SECTION E-E



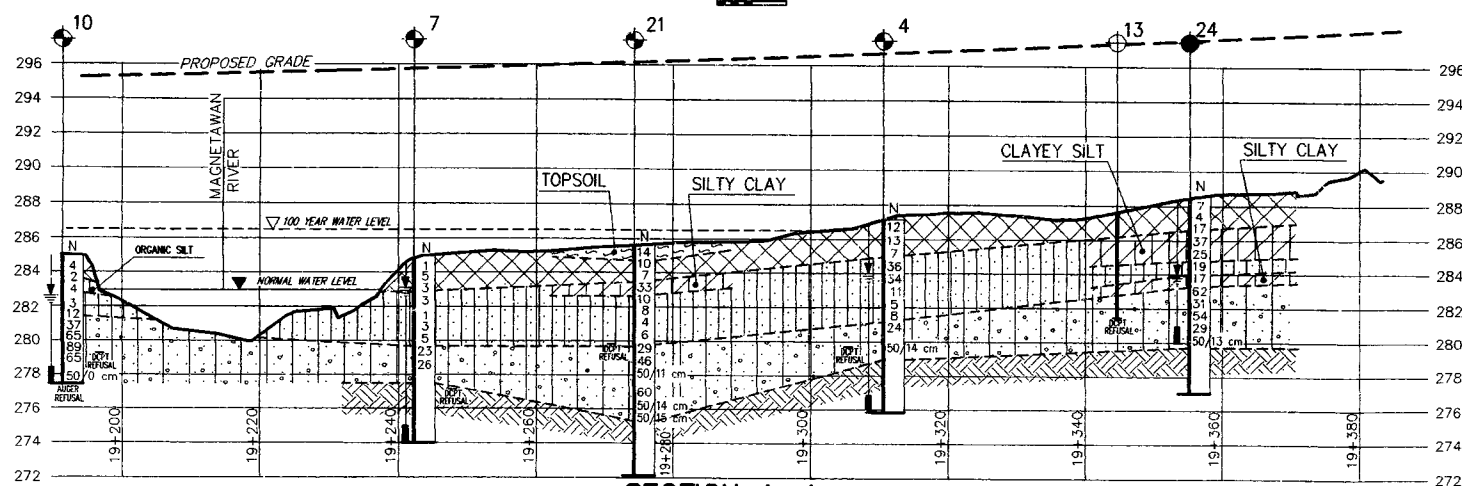
SECTION F-F

SOIL STRATIGRAPHY LEGEND

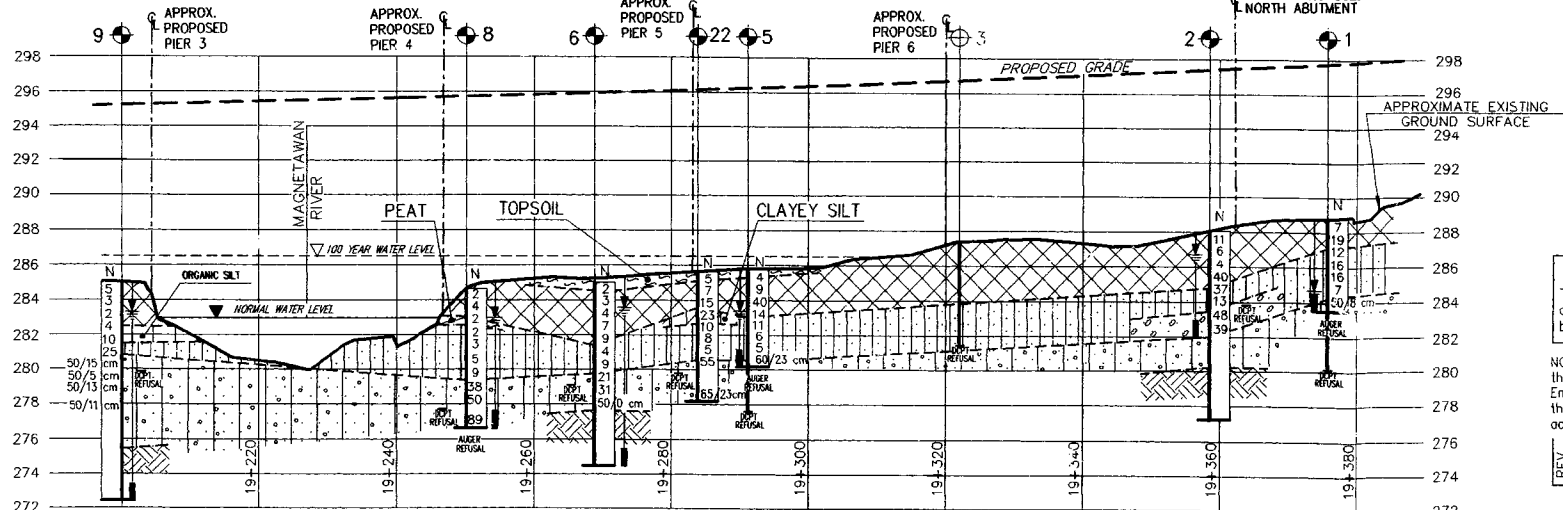
	FILL Very Loose to Compact		GRAVELLY SAND TO SAND & GRAVEL Dense to Very Dense
	SAND, SILTY SAND & SANDY SILT Very Loose to Dense		HETEROGENEOUS MIXTURE OF SAND, SILT & GRAVEL (GLACIAL TILL)
	ORGANIC SILT Very Loose to Loose		GRANITE BEDROCK



PLAN



SECTION A-A



SECTION B-B

5m 0 5m HOR  
2m 0 2m VER

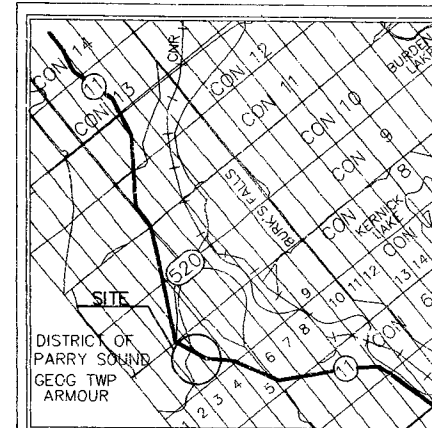
REF. Hwy 11 Bridge Site Plan  
Dwg. by MTO: 1999

SITE. No. 44-188  
W.P. No. 473-93-00

PROPOSED HIGHWAY 11 NBL  
MAGNETAWAN RIVER/HWY 520 OVERPASS  
BORE HOLE LOCATIONS & SOIL STRATA

SHEET  
2

AGRA Earth & Environmental Ltd.



KEY PLAN

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 - Sept. 1999
- WL in Piezometer (stabilized)
- Piezometer

No	ELEVATION	CO-ORDINATES NORTH	EAST
1	288.6	5 053 018	311 420
2	288.0	5 053 002	311 426
3	287.5	5 052 965	311 426
4	287.1	5 052 952	311 416
5	285.9	5 052 933	311 423
6	285.0	5 052 912	311 433
7	284.8	5 052 883	311 422
8	284.7	5 052 893	311 433
9	285.2	5 052 844	311 441
10	285.1	5 052 833	311 430
13	287.5	5 052 986	311 417
21	TO BE PROVIDED		
22	TO BE PROVIDED		
24	288.6	5 052 995	311 415

-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
-----	------	----	-------------

HWY No 11	SUBM'D SP	CHECKED SP	DATE November, 1999	DIST 52
	DRAWN MA	CHECKED	APPROVED	SITE 44-188
				DWG 2