

**FOUNDATION INVESTIGATION AND DESIGN REPORT
HIGHWAY 592 / SUNSET PASS DRIVE OVER HIGHWAY 11
HIGHWAY 11, HIGHWAY 518 WEST TO HIGHWAY 520
G.W.P. 480-93-00, W.P. 5404-04-01, SITE 44-394**

Geocres Number: 31E-240

Report to

Marshall Macklin Monaghan

Thurber Engineering Ltd.
2010 Winston Park Drive, Suite 103
Oakville, Ontario
L6H 5R7
Phone: (905) 829 8666
Fax: (905) 829 1166

February 1, 2006

File: 19-1423-16

C:\Thurber\Projects\19\1423 MMM\16 Hwy 11\Foundations\Hwy 592 Sunset
FIDR FINAL.doc

TABLE OF CONTENTS**PART 1: FACTUAL INFORMATION**

1	INTRODUCTION	1
2	SITE DESCRIPTION	1
3	SITE INVESTIGATION AND FIELD TESTING	2
4	LABORATORY TESTING	3
5	DESCRIPTION OF SUBSURFACE CONDITIONS	3
5.1	General	3
5.2	Topsoil	4
5.3	Fill	4
5.4	Upper Silt/Sand	4
5.5	Silty Clay	4
5.6	Fine Sand	5
5.7	Sand and Gravel with Cobbles and Boulders	5
5.8	Lower Silt	6
5.9	Bedrock	6
5.10	Depths to Refusal	6
5.11	Water Levels	7
6	MISCELLANEOUS	8

PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

7	INTRODUCTION	9
8	STRUCTURE FOUNDATIONS	10
8.1	Spread Footings	10
8.1.1	Footings on Native Soil	10
8.1.2	Footings on Engineered Fill	11
8.2	Driven Steel Piles	12
8.2.1	Pile Tips	12
8.2.2	Pile Installation	12
8.2.3	Pile Driving	13
8.2.4	Downdrag	13
8.2.5	Lateral Resistance of Piles	14

8.3	Caissons.....	15
8.4	Recommended Foundation.....	16
8.5	Abutment Type.....	16
8.6	Frost Protection	16
9	EXCAVATION	17
10	UNWATERING	17
11	APPROACH EMBANKMENTS	17
12	RETAINED SOIL SYSTEMS.....	19
13	BACKFILL TO ABUTMENTS	20
14	EARTH PRESSURE COEFFICIENTS (ABUTMENTS).....	20
15	SEISMIC CONSIDERATIONS	21
15.1	Seismic Design Parameters	21
15.2	Liquefaction Potential	22
15.3	Retaining Wall Dynamic Earth Pressures	22
15.4	Slope Stability Considerations	23
16	CONSTRUCTION CONCERNS	23
17	CLOSURE	23

Appendices

Appendix A	Record of Borehole Sheets
Appendix B	Laboratory Test Results
Appendix C	Factual Data from Shaheen & Peaker Report
Appendix D	Foundation Comparison
Appendix E	Special Provisions
Appendix F	Selected Slope Stability Output
Appendix G	Borehole Locations and Soil Strata

**FOUNDATION INVESTIGATION AND DESIGN REPORT
HIGHWAY 592 / SUNSET PASS DRIVE OVER HIGHWAY 11
HIGHWAY 11, HIGHWAY 518 WEST TO HIGHWAY 520
G.W.P. 480-93-00, W.P. 5404-04-01, SITE 44-394**

Geocres Number: 31E-240

PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted at the site of a proposed underpass structure to carry Highway 592/Sunset Pass Drive over the four-laned Highway 11 south of the village of Katrine, Ontario. A previous, preliminary investigation had been carried out at the structure location by Shaheen & Peaker Limited (S&P) and the factual data from that investigation has been incorporated in the current assignment.

The purpose of the investigation was to explore the subsurface conditions at the site and, based on the data obtained, to provide a borehole location plan, borehole logs, stratigraphic profile and cross-sections, and a written description of the subsurface conditions. A model of the subsurface conditions was developed through considering a combination of the data from the previous S&P investigation and the data obtained in the course of the present investigation. This model describes the geotechnical conditions influencing design and construction of the foundations and approach embankments for the underpass structure.

Thurber carried out the investigation as a sub-consultant to Marshall Macklin Monaghan, under the Ministry of Transportation Ontario (MTO) Agreement Number 5005-A-000285.

2 SITE DESCRIPTION

The site is located along existing Highway 11 approximately 60 m south of the present intersection with Highway 592 and Sunset Pass Drive. The site lies south of the Village of Katrine, in Perry Township immediately south of the boundary with Armour Township.

The general site area is located within the physiographic region known as the Canadian Shield, characterized by Pre-Cambrian bedrock typically occurring as rounded knobs and ridges where exposed. Locally however, the site lies between the bedrock ridges in an area infilled with relatively deep deposits of ice-contact and glacio-fluvial sands and gravels, overlain by glacio-lacustrine soils.

The area on the east side of Highway 11 is fairly open and sparsely treed. To the west of Highway 11 adjacent to Sunset Pass Drive, the land is wooded. Drainage is generally towards the

Magnetawan River located some 700 m to the east and north. Scattered dwellings and businesses are present along Highway 11 north of the site and on Sunset Pass Drive.

Road grades along existing Highway 11 at the site generally fall towards the north, from approximate elevation 345 m some 700 m south of the site to elevation 300 m at the Magnetawan River crossing 1200 m to the north. Existing road grade at the site is near elevation 324 m, and appears to have been built up several metres above the original ground surface.

3 SITE INVESTIGATION AND FIELD TESTING

Thurber carried out site investigation and field testing for this project at the proposed location of the west abutment and east approach between October 15 and 19, 2004. Additional investigation was conducted at the abutments and pier during the period February 25 to March 18, 2005. Preliminary investigation was carried out by S&P at the foundation units and approaches between April 19 and June 7, 2001.

The 2004 and 2005 site investigations consisted of drilling and sampling four boreholes (Boreholes 394-1 to 394-4) to depths of 24.8 to 38.6 m at the foundation unit locations and one borehole (borehole 394-5) to 11.1 m at the east approach. Borehole 394-4 was supplemented by dynamic cone penetration testing. The approximate locations of the boreholes are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix G.

Prior to the start of drilling, the borehole locations were staked in the field and utility clearances were obtained.

Hollow stem augers were used to advance the deep boreholes to depths of 6.1 to 18.3 m and the approach hole to the full depth. Mud rotary tricone and coring procedures with casing were used to advance through cobbly/bouldery material below these depths. Samples were obtained using a split spoon sampler in conjunction with Standard Penetration Testing (SPT).

The positions of the principal boreholes considered in the preparation of this report, relative to the structure site are as shown in Table 3.1.

Table 3.1 – Borehole Locations Relative to Structure

Location on Structure	Boreholes Considered in Design
East Approach	RA1*, 394-5
East Abutment	RA2*, 394-1
Pier	RA3*, 394-2
West Abutment	RA4*, 394-3, 394-4
West Approach	RA5*

* Boreholes drilled by S&P in 2001

The coordinates and elevations of the boreholes are given on the Borehole Locations and Soil Strata Drawing and on the individual Record of Borehole Sheets in Appendix A.

Standpipe piezometers, consisting of 19 mm PVC pipe with slotted tip, were installed in the boreholes at the abutments to monitor the groundwater level. A shallow piezometer was installed at the west abutment in the course of the preliminary investigation.

The completion details for the piezometer are shown in Table 3.2.

Table 3.2 – Piezometer Details

Piezometer Location	Piezometer Details	
	Tip Depth/ Elevation	Completion Details
BH 394-1	29.6/290.4	Piezometer with 1.5 m tip installed at bottom of borehole. Sand filter to 27.7, bentonite seal to 26.8, grout to the surface.
BH 394-4	25.9/296.7	Piezometer with 1.5 m tip installed at bottom of borehole. Sand filter to 23.8, bentonite seal to 22.9, grout to 0.9 and bentonite seal to the surface.

A member of Thurber's engineering staff supervised the drilling and sampling operations on a full time basis. The inspector logged the boreholes and the recovered samples and processed them for transport to Thurber's Oakville office.

4 LABORATORY TESTING

All recovered soil samples were subjected to visual identification and to natural moisture content determination. The results of this testing are shown on the Record of Borehole sheets in Appendix A.

Selected samples were subjected to gradation analysis (sieve and hydrometer) and the results are shown on the Record of Borehole sheets in Appendix A and on the charts in Appendix B. A total of 21 samples were selected for this testing

5 DESCRIPTION OF SUBSURFACE CONDITIONS

5.1 General

Reference is made to the Record of Borehole sheets in Appendix A and to the Record of Borehole sheets prepared during the preliminary investigation included in Appendix C. Details of the encountered soil stratigraphy are presented in these appendices and on the attached Borehole Locations and Soil Strata Drawing. An overall description of the stratigraphy is given in the following paragraphs however the factual data presented in the borehole logs governs any interpretation of the site conditions.

The soil stratigraphy encountered at this site is consistent with that encountered in much of the Highway 11 corridor between Huntsville and North Bay. The bedrock is mantled by sand and gravel containing cobbles and boulders which is overlain by glacial outwash soils deposited in glacio-fluvial and glacio-lacustrine environments.

In general terms, the site was found to be underlain by a thin veneer of topsoil or fill; discontinuous layers of silt and silty clay; fine sand; and sand and gravel with cobbles and boulders.

More detailed descriptions of the individual strata are presented below.

5.2 Topsoil

Topsoil was identified surficially in all boreholes except boreholes RA3, 394-2 and 394-3 where fill was encountered. The topsoil thickness was established only at the borehole locations and was generally 50 to 150 mm. The topsoil thickness may vary between and beyond the borehole locations and the data is not intended for the purpose of estimating quantities.

5.3 Fill

Fill was encountered in borehole RA3 drilled from the west shoulder of existing Highway 592. The fill extended to a depth of 2.9 m (elevation 318.9 m) and consisted of non-cohesive fine sand. SPT N-values obtained in the fill ranged from 12 to 22 blows per 300 mm penetration, indicating a compact condition. Moisture contents ranged from 13 to 23%.

Sand and gravel fill was encountered in borehole 394-2 drilled near the east toe of the existing Highway 11 embankment. The fill was dense (N-value of 41 blows/0.3 m) and 0.9 m thick. A 0.9 m thick layer of mixed snow and soil was placed in the west ditch of Highway 11 to facilitate drilling of borehole 394-3.

5.4 Upper Silt/Sand

Below the fill and topsoil layers, a unit of silt to fine sand was encountered. This deposit was typically non-cohesive with some cohesive zones and silty clay seams. SPT N-values ranged from 2 to 6 blows/300 mm penetration in the upper very loose silt immediately below the topsoil, and from 11 to 28 blows/300 mm penetration in compact material below this depth. The silt/sand layer extended to depths of 1.4 to 4.6 m (elevation 319.9 to 316.0 m).

Grain size distribution curves for the silt are reported on the Record of Borehole sheets and plotted in Figure B1 of Appendix B. Grain size results from the preliminary investigation are included in Appendix C. Moisture contents ranged from 10 to 32% with the higher values recorded in the upper very loose zone.

5.5 Silty Clay

A stratum of cohesive silty clay was encountered below the fill and sand in boreholes RA3 and 394-2 at the pier and below the silt/sand in boreholes RA4 and 394-4 at the west abutment. The clayey deposit was contacted at depths of 2.3 to 4.6 m (elevation 317.6 to 319.6 m) and was 1.8 to 3.3 m thick, 6.1 m in borehole 394-2.

Standard Penetration Tests conducted in this deposit yielded N-values of 12 to 27 (stiff to very stiff consistency) at the pier (boreholes RA3 and 394-2) and 6 to 12 (firm to stiff consistency) at the west abutment (boreholes RA4 and 394-4). The undrained shear strength measured by in situ vane testing exceeded 100 kPa in the boreholes drilled during

the previous investigation, and was 75 kPa with a sensitivity of 2.1 in borehole 394-2 drilled at the pier.

Grain size distributions for this soil are provided on the Record of Borehole sheets and in Figure B2 of Appendix B. Grain size results from the preliminary study are included in Appendix C as well. The results of Atterberg Limits testing (Figure B6 of Appendix B and Figure 3 of Appendix C) classify the soil as low to medium plastic silty clay (CL to CI). Moisture contents ranged between 24 and 40%.

5.6 Fine Sand

The silt and silty clay layers are underlain by a stratum of fine-grained sand that varies substantially in thickness across the site. In boreholes RA3 to RA5 (pier to west approach), the thickness ranged from 0.8 to 4.7 m with a lower boundary between elevation 310.1 and 317.6 m. In borehole 394-2 at the pier, a corresponding 3.0 m thick layer of silt was encountered with a lower boundary at elevation 308.4 m. This layer was not encountered in borehole 394-4.

In borehole RA2 at the east abutment, this unit was 32.1 m thick with a lower boundary at elevation 284.8. Borehole 394-1, also at the east abutment, was terminated in the sand deposit at 29.6 m depth (elevation 290.4 m). Boreholes RA1 and 394-5 at the east approach were terminated in sand at 8.1 and 11.1 m depth; the sand in borehole 394-5 varied from fine-grained to coarse-grained.

SPT values measured in the sand ranged from 14 to greater than 100 blows per 0.3 m of penetration, indicating a compact to very dense condition. The measured natural moisture contents ranged from 2 to 23%. The soil is generally described as brown in colour, becoming grey below approximate elevation 315 m in borehole RA2.

Grain size distributions for this soil are reported on the Record of Borehole sheets and are plotted in Figure B3 in Appendix B. Grain size results from the preliminary study are included in Appendix C as well.

5.7 Sand and Gravel with Cobbles and Boulders

A deposit of sand and gravel was encountered below the fine sand in all boreholes except boreholes RA1, 394-1 and 394-5. The upper contact with this deposit ranged from depths of 3.7 to 13.7 m (elevation 317.8 to 308.4 m) in the boreholes at the west abutment and pier, and a depth of 35.8 m (elevation 284.8 m) in borehole RA2 at the east abutment. At the west abutment and pier (boreholes 394-2 to 394-4), this deposit included zones of sand, fine sand and gravelly sand.

Frequent cobbles and boulders were encountered in this deposit at the pier and west abutment. Rock coring and wash boring methods were necessary to advance through these zones at several depths.

SPT N-values obtained in the sand and gravel ranged from 14 to greater than 100 blows per 300 mm of penetration. It is likely however that the sampler was driving on cobbles and boulders in many cases, and the resulting SPT values may be unrepresentative. Based on the SPT and dynamic cone penetration testing, the sand and gravel is considered to be dense to very dense with occasional compact zones.

The results of grain size distribution analyses conducted on samples of the sand and gravel deposit and sand zones are presented on Figures B5 and B4, respectively, of Appendix B, and in Appendix C. The samples excluded particle sizes greater than about 30 mm. Moisture contents ranged from 1 to 22%, presumably depending upon the particle size of the individual sample tested. A moisture content of 29% was determined on one isolated sample.

Boreholes RA2, RA4, RA5 and 394-2 to 394-4 were terminated in the sand and gravel deposit at depths of 9.6 to 42.6 m (elevation 311.7 to 278.0 m). A silt stratum was contacted below the sand and gravel at 39.1 m depth (elevation 282.7 m) in borehole RA3 at the pier.

5.8 Lower Silt

A layer of very dense silt was encountered below the sand and gravel in borehole RA3 at the pier only. This layer was revealed in one split spoon sample only, and the borehole was terminated upon a possible boulder at 39.7 m depth (elevation 282.1). An SPT N-value of 60 blows for 50 mm of penetration was obtained in the silt.

5.9 Bedrock

Bedrock was not contacted within the exploration depths of 8.1 to 42.6 m during the investigation.

5.10 Depths to Refusal

The depths at which effective refusal was encountered, defined as an SPT value exceeding 100 blows for 0.3 m of penetration or bedrock, are shown in Table 5.1.

Table 5.1 – Refusal Depths (Elevations)

Location	Borehole	Refusal Depth (Elevation), m	Material
East Abutment	RA2	32.3 (288.3)	Fine Sand
	394-1	24.4 (295.6)	Fine to Medium Sand
Pier	RA3	35.4 (286.4)	Sand and Gravel
	394-2	35.1 (287.0)	Sand and Gravel
West Abutment	394-3	21.3 (299.2)	Sand and Gravel
	394-4	20.0 (301.9)	Sand

5.11 Water Levels

The initial and final groundwater depths and elevations are shown in Table 5.2.

Table 5.2 – Groundwater Depths and Elevations

Location	Borehole	Date	Water Level (m)		Comment
			Depth	Elevation	
East Approach	RA1 394-5	24-Apr-01	>8.1	<312.7	Upon completion
		19-Oct-04	>11.1	<309.9	Upon completion
East Abutment	RA2 394-1	24-Apr-01	1.5	319.1	Not stabilized
		09-Mar-05	20.9	299.1	In piezometer
Pier	RA3	26-Apr-01	3.4	318.4	Not stabilized
West Abutment	RA4 394-4 394-4	16-Jul-01	>7.6	<313.8	In piezometer
		19-Oct-04	15.2	306.7	Upon completion
		21-Oct-04	15.6	306.3	In piezometer
West Approach	RA5	27-Apr-01	>9.6	<311.7	Upon completion

The water levels observed in boreholes RA2 and RA3 are believed to reflect a perched condition above the relatively impermeable clayey layers within/below the upper silt/sand deposits.

The above values are short-term readings and seasonal fluctuations of the groundwater level are to be expected. In particular, the groundwater level may be at a higher elevation after the spring snowmelt or after periods of heavy rainfall.

6 MISCELLANEOUS

Marshall Macklin Monaghan completed field layout for the site investigation and provided borehole coordinates and ground surface elevations.

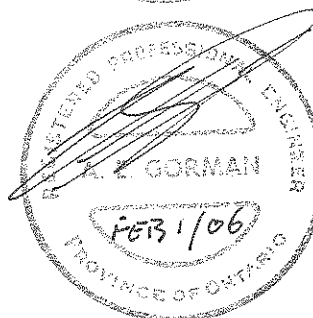
George Downing Estate Drilling Limited and All-Terrain Drilling Limited supplied and operated the drilling and sampling equipment used for the current investigation. Full time supervision of the field activities, including obtaining utility clearances, was carried out by Mr. George Azzopardi and Mr. Warren Wunderlick of Thurber.

Interpretation of the field data and preparation of the investigation report was conducted by Mr. Murray Anderson, P.Eng. Overall supervision of the field program and review of the report was performed by Mr. Alastair E. Gorman, P.Eng. The report was also reviewed by Dr. P.K. Chatterji, Ph.D., a Designated Principal Contact for MTO Foundations Projects.

Thurber Engineering Ltd.
Murray R. Anderson, P.Eng., M.Eng.
Senior Geotechnical Engineer



Alastair E. Gorman, P.Eng., M.Sc.
Senior Foundations Engineer



P.K. Chatterji, P.Eng., Ph.D.
Review Principal.



**FOUNDATION INVESTIGATION AND DESIGN REPORT
HIGHWAY 592 / SUNSET PASS DRIVE OVER HIGHWAY 11
HIGHWAY 11, HIGHWAY 518 WEST TO HIGHWAY 520
G.W.P. 480-93-00, W.P. 5404-04-01, SITE 44-394**

Geocres Number: 31E-240

PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

7 INTRODUCTION

This report presents interpretation of the geotechnical data in the factual report and presents geotechnical design recommendations to assist the design team to select and design a suitable foundation system and approach fills for the proposed structure.

Highway 11 will be widened from a two-lane undivided roadway to a four-lane divided highway. The new southbound lanes will follow the existing roadway alignment and new northbound lanes will be constructed to the east, separated by a 30 m wide median.

A two-span, 84.0 m long underpass structure with a post-tensioned trapezoidal voided deck is proposed to carry Highway 592/Sunset Pass Drive over the widened Highway 11 at this site. Integral abutments are under consideration.

At the structure location, road grade for the new southbound lanes will be near the existing Highway 11 grade. Finished road grade on the new northbound lanes will be some 2 to 3 m above the existing ground surface.

At the east abutment, the finished grade of Highway 592 will be about Elevation 333.4 and the original ground lies at approximate Elevation 321.0, resulting in an approach fill approximately 12.5 m high.

At the west abutment, the finished grade of Sunset Pass Drive will be about Elevation 332.0 and the original ground lies at approximate Elevation 323.0, resulting in an approach fill approximately 9 m high.

The discussion and recommendations presented in this report are based on our understanding of the project and on the factual data obtained in the course of the investigation, together with the factual data from the previous investigation by S&P.

8 STRUCTURE FOUNDATIONS

Foundation alternatives are presented in the following sections together with the corresponding geotechnical design parameters. A preferred foundation scheme is recommended.

Based on the results of the exploratory boreholes drilled at the proposed abutment locations, the stratigraphy consists of a surficial fill or topsoil layer overlying strata of very loose to compact silt/sand and firm to very stiff silty clay extending to depths of 1.4 to 10.7 m, underlain by compact to very dense fine sand. Dense to very dense sand and gravel with cobbles and boulders was contacted below the fine sand at depths of 3.7 to 13.7 m at the west abutment and pier, and at 35.8 m at the east abutment.

Initial consideration was given to the following foundation types:

- Spread footings on native soil
- Spread footings on engineered fill
- Driven steel H-piles
- Caissons (drilled shaft piles)

Appendix D contains a table presenting a comparison of the technical advantages and disadvantages of the different foundation schemes at this site.

8.1 Spread Footings

8.1.1 Footings on Native Soil

The near surface silts, sands and clays at the abutment locations are considered too variable to provide adequate support to spread footings due to the low bearing resistance available and the potential for comparatively large settlements. Extending footings down through these materials to bear on the underlying compact to dense sand could be considered.

Provided a minimum footing width of 2 m is maintained, footings bearing on the compact to dense sand may be designed for a concentric, vertical geotechnical resistance of 700 kPa at factored ULS and a resistance of 300 kPa at SLS. Footings designed using these resistances should be founded at or below the following elevations:

Table 8.1 – Maximum Elevation of Footings on Native Sand

Location	Borehole Number	Maximum Founding Elevation
East Abutment	RA2, 394-1	313.5
Pier	RA3, 394-2	311.5
West Abutment	RA4, 394-3, 394-4	314.7

The resistance values are for vertical, concentric loads. Where eccentric or inclined loads are applied, the resistance used in design must be reduced in accordance with the CHBDC Clause 6.7.3 and Clause 6.7.4.

For footings designed on the basis of the geotechnical resistance values given above, total settlement under a footing is not expected to exceed 25 mm. Differential settlements are not expected to exceed 15 mm across the width of the structure.

The lateral resistance of the footings founded on fine sand may be computed using an unfactored friction coefficient of 0.55. This is an “ultimate” value and requires a degree of sliding movement to occur to fully mobilize the resistance.

In view of the depth of subexcavation (4 to 11 m), the presence of a perched water condition, and the potential for further variability in the sand deposit below the founding level, construction of spread footings on the native soil is not the favoured option.

8.1.2 Footings on Engineered Fill

Construction of spread footings on engineered fill replacing the upper silts, sands and clay may be considered. The upper deposits are considered unsuitable for the support of an engineered fill pad due to the low bearing resistance available and the potential for comparatively large settlements. Subexcavation of these materials before fill placement is therefore recommended.

The underside of the engineered fill pad should extend down to or below the elevations given in Table 8.2. The fill should be at least 1.5 m thick below the footing.

Table 8.2 – Maximum Elevation of Underside of Engineered Fill

Location	Borehole Number	Maximum Elevation of Underside of Engineered Fill Pad
East Abutment	RA2, 394-1	316.9
Pier	RA3, 394-2	315.5
West Abutment	RA4, 394-3, 394-4	315.5

The engineered fill must consist of OPSS Granular A placed in 150 mm lifts and compacted to 100% of its SPMDD at $\pm 2\%$ of optimum moisture content and generally conforming to the geometry illustrated in Figure 1, Appendix E.

Provided a minimum footing width of 2 m is maintained, a footing bearing on the engineered fill may be designed for a concentric, vertical geotechnical resistance of 900 kPa at factored ULS and a resistance of 350 kPa at SLS.

These resistance values are for vertical, concentric loads. Where eccentric or inclined loads are applied, the resistance used in design must be reduced in accordance with the CHBDC Clause 6.7.3 and Clause 6.7.4.

For footings designed on the basis of the geotechnical resistance values given above, total settlement under a footing is not expected to exceed 20 mm. Differential settlements are not expected to exceed 10 mm across the width of the structure.

The lateral resistance of the footings founded on engineered fill may be computed using an unfactored friction coefficient of 0.7. This is an “ultimate” value and requires a degree of sliding movement to occur to fully mobilize the resistance.

It is noted that the sub-excavation depths extend up to 7 m below existing grade at the boreholes. Considering the fill thickness, excavation shoring requirements adjacent to existing Highway 11, and perched water condition needing dewatering, supporting the structure on engineered fill is not the favoured option from a geotechnical perspective.

8.2 Driven Steel Piles

The geotechnical conditions encountered at this site are considered suitable for driven steel H-pile foundations.

The piles are expected to develop resistance through shaft resistance and end-bearing within the fine sand and sand and gravel deposits. The piles should be designed on the basis of the axial geotechnical resistances given in Table 8.3.

Table 8.3 – Pile Geotechnical Resistance

Pile Section	Piles Driven Into Sand with Cobbles and Boulders				
	ULS (Factored)	SLS (25 mm Settlement)	Estimated Pile Tip Elevation		
			East Abutment	Pier	West Abutment
HP 310 X 110	1,800 kN	1,600 kN	288	286	300
HP 360 X 132	2,100 kN	1,800 kN	288	286	300

The estimated pile tip elevations are also provided in Table 8.3. The elevation at which the design capacity is achieved is expected to vary and in some cases a pile may encounter refusal on cobbles and boulders. The pile tip elevations shown in Table 8.3 should be used for cost estimating purposes only. The actual pile tip elevations will be controlled as described in Section 8.2.2 Pile Installation.

8.2.1 Pile Tips

The pile tips should be fitted with cast steel, H-section rock points from an approved manufacturer such as Titus Steel (Standard H-point), APF Hard Bite, or approved equivalent. Use of rock points is recommended to provide a higher level of protection against pile damage when driving through the dense to very dense soils to achieve the required resistance and in case cobbles and boulders are encountered.

8.2.2 Pile Installation

Pile installation should be in accordance with Special Provision No. 903S01.

The Contract Documents should contain a NSSP alerting the Bidders to:

- The presence of cobbles and boulders in the sand and gravel.

- The possibility of piles within a group achieving the specified resistance at different elevations.
- The possibility of some piles meeting refusal on a large boulder.

The NSSP should require the QVE to terminate driving before the pile is damaged by overdriving.

To facilitate pile installation, embankment fill through which piles will be driven must not contain oversize material, i.e. no particles exceeding 75 mm in size.

8.2.3 Pile Driving

Pile driving within 2 m of the estimated pile tip elevation (Table 8.3) should be controlled by the Hiley Formula and an ultimate pile resistance to be specified by the designer in accordance with Clause 3.3.2 (b) Construction Stage of the Structural Manual. The appropriate pile driving note is "Piles to be driven in accordance with Standard SS 103-11 using an ultimate resistance of "R" kN per pile". "R" must have the minimum values shown in Table 8.4.

Table 8.4 – Ultimate Geotechnical Resistance of Piles

Pile	Ultimate Resistance, R (kN)
HP 310X110	3,600 kN
HP360X132	4,200 kN

The Contractor should be alerted to the fact that the piles may meet refusal on a large boulder in the sand and gravel deposit. If this happens, the Hiley formula is not applicable, and a site decision must be made that refusal has been encountered and that further pile driving must be controlled to adequately seat the pile on the boulder. To avoid overdriving and damaging the tip, a limiting criterion of 10 blows at full energy for 12 mm penetration for two consecutive sets of 10 blows should be established to control pile driving on a boulder. The geotechnical resistances given in Table 8.3 remain valid in this situation.

8.2.4 Downdrag

The soils at the east abutment are non-cohesive. In general, settlements induced in the native soils around the piles by construction of the approach embankments will be substantially complete as construction of the embankment is completed.

At the west abutment, a clay layer of 1.8 to 2.2 m thickness was encountered at depths of 2.3 and 3.7 m. Consolidation of this layer following construction of the embankment is expected to result in post-construction settlement in the order of 50 mm and be essentially complete within three months (Section 11). Embankment construction in advance of pile driving is recommended to reduce potential downdrag effects.

It is recommended that the approach embankments be constructed at least three months in advance of pile driving. The embankment should be constructed up to the level of the

abutment from the forward slope to a distance back sufficiently far to allow access and operation of construction equipment. Beyond that distance, the embankment should be constructed to full height. Provided the embankments are constructed in this manner, downdrag forces are not considered to be an issue at this site.

8.2.5 Lateral Resistance of Piles

The lateral resistance of the piles may be calculated using a value for the coefficient of horizontal subgrade reaction (k_s) and ultimate lateral resistance (p_{ult}) as follows:

$$k_s = n_h \cdot z / D \quad (\text{kN/m}^3) \quad \text{for cohesionless soils}$$

$$= 67 \cdot c_u / D \quad (\text{kN/m}^3) \quad \text{for cohesive soils}$$

$$p_{ult} = 3 \cdot \gamma \cdot z \cdot K_p \quad (\text{kPa})$$

where	z	=	depth of embedment of pile in metres
	D	=	pile width in metres
	n_h	=	coefficient of horizontal subgrade reaction (Table 8.5)
	c_u	=	undrained shear strength (Table 8.5)
	γ	=	unit weight (Table 8.5), use submerged weight below water table
	K_p	=	passive earth pressure coefficient (Table 8.5)

The above equations and recommended parameters may be used to analyze the interaction between a pile and the surrounding soil. The lateral pressures obtained from the analysis should not exceed the ultimate lateral resistance.

The spring constant, K , for analysis may be obtained by the expression, $K = k_s \times L \times D$ (kN/m), where k_s is the coefficient of horizontal subgrade reaction (kN/m³), D is the pile width (m) and L is the length (m) of the pile segment or element used in the analysis. The ultimate lateral resistance, P_{ult} , may be obtained from the expression, $P_{ult} = p_{ult} \times L \times D$.

Table 8.5 – Parameters for Lateral Pile Resistance

Location	Elevation	n_h (kN/m ³)	c_u (kPa)	K_p	Unit Weight (kN/m ³)
East Abutment	OGL to 315	3,000		3.0	20
	315 to 290	6,000		3.3	21
	290 to 278	8,000		3.5	21
Pier	OGL to 318	3,000		3.0	20
	318 to 313	-	100	2.7	19
	313 to 310	6,000		3.3	21
	310 to 282	8,000		3.5	21
West Abutment	OGL to 320	3,000		3.0	20
	320 to 318	-	100	2.7	19
	318 to 305	6,000		3.3	21
	305 to 296	8,000		3.5	21

The total horizontal passive resistance of a single pile used in design should not exceed the following values:

Table 8.6 – Maximum Horizontal Passive Resistance of Piles

Pile	Maximum Passive Resistance	
	Factored ULS	SLS
HP 310X110	110 kN	40 kN
HP360X132	150 kN	50 kN

The modulus of subgrade reaction may have to be reduced, based on the pile spacing. The reduction factors to be used for a pile group oriented perpendicular or parallel to the direction of loading are provided in Table 8.7. Intermediate values may be obtained by linear interpolation.

Table 8.7 – Subgrade Reaction Reduction Factors for Pile Spacing

Condition	Pile Spacing, Centre to Centre*	Reduction Factor
Pile group oriented <i>perpendicular</i> to direction of loading	4D	1.0
	1D	0.5
Pile group oriented <i>parallel</i> to direction of loading	8D	1.0
	6D	0.7
	4D	0.4
	3D	0.25

* where D is the breadth of pile

In the case of conventional abutments, i.e. not integral, horizontal loads may be resisted by means of battered piles.

8.3 Caissons

The soil and groundwater conditions at this site are not considered to be suitable for caisson foundations. To achieve the high resistance necessary to justify the construction costs, the caissons would have to be founded in the very dense sand and gravel near elevation 285 to 300. This level is some 7 m below the water level measured in the piezometer installed in borehole 394-4. It would be impossible to achieve a liner seal at the caisson base to allow unwatering of the caisson, and slurry excavation with tremie concreting would be necessary.

Further, caisson excavation would be relatively slow and problematic due to the numerous cobbles and boulders encountered in the sand and gravel deposit.

Caissons are also not considered to be suitable for construction on a batter to resist horizontal loads.

On the basis of the installation difficulties and risks assessed for this site, caissons are not recommended.

8.4 Recommended Foundation

The recommended foundation system for the pier and both abutments at this site is steel H-piles driven to a prescribed ultimate resistance as controlled by application of the Hiley formula.

8.5 Abutment Type

From a geotechnical perspective, the subsurface conditions at this site are considered to be suitable for the construction of conventional, semi-integral or integral abutments. However, the recommended foundation system of H-piles makes integral abutments a feasible option.

The integral abutment design requires that the piles possess flexibility in the upper 3 m of the pile length. At this site, the upper 3 m of the pile length will lie in approach fill or very loose to compact silt and sand. Accordingly, to provide the required flexibility in the piles, the upper 3 m of the piles should be surrounded by a 600 mm diameter CSP filled with sand (for a “true abutment” supported on piles) or by concentric CSPs in accordance with standard integral abutment design procedures (for a “false abutment”).

Backfill sand placed in the CSP should meet the gradation shown in Table 8.8 and must be placed after pile driving to minimize the potential for densification.

Table 8.8 – Integral Abutment Sand Grading

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

8.6 Frost Protection

The depth of earth cover required to provide frost protection for footings and pile caps at this site is 1.8 m.

It is possible to reduce the thickness of earth cover by the substitution of synthetic insulation. A 25 mm thickness of rigid, extruded polystyrene insulation is equivalent to 600 mm of earth cover. Synthetic insulation must be covered to provide protection where it is used.

Rock fill is not equivalent to earth fill in terms of thermal resistance. Frost may penetrate deeper through rock fill than earth fill and the possibility exists for freezing conditions to

develop below the pile cap. Therefore, non-frost susceptible free-draining granular fill with less than 5% particles by mass finer than 75 μm should be specified for the pile driving pad within the rock fill.

9 EXCAVATION

All excavation must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the native soils at this site may be classed as Type 3 soils above the water table. This classification is based on the lack of cohesion in the soils and the resulting possibility that excavation slopes will slough if excavated vertically for the lower 1.2 m. Excavation slopes should not exceed 1V:1H above the groundwater level.

Where 1V:1H sidewalls cannot be accommodated due to spatial restrictions, such as along existing Highway 11, a shoring system should be provided to support the excavation sidewalls.

Excavation is not expected to extend below the groundwater level on site, measured near elevation 306 in the piezometer installed in borehole 394-4. However, a perched water table may exist within the upper silt and sand deposits underlain by less permeable silty clay layers. Some groundwater seepage and sloughing of excavation sidewalls in the upper silts/sands should be anticipated.

Selection of the appropriate excavation procedures and dewatering system is the responsibility of the Contractor. The Contract Documents should alert him to the requirement to maintain a stable excavation and a dry, sound base on which to work. Any shoring system should be designed by a shoring specialist, taking account of the potential for groundwater seepage and sloughing conditions.

10 UNWATERING

Based on the preliminary GA for the bridge structure, it is not expected that work at the abutments will require excavation below the perched groundwater level. However, the Contractor should be prepared to pump from sumps to remove any seepage water or surface water collecting in an excavation.

The design of any dewatering system that may be required is the responsibility of the Contractor and the Contract Documents should alert him to this responsibility. While the responsibility for dewatering should remain with the Contractor, a suitable system that might be employed involves pumping from filtered sumps.

11 APPROACH EMBANKMENTS

The global, internal and surficial stability of the approach embankment fill will depend on the slope geometry but also to a large degree on the material used to construct the embankment. If the embankment is constructed of blast rock fill, it may be assumed that the side slopes will be stable at inclinations up to 1.25H:1V. Embankments constructed using granular material, select subgrade material and most earth materials will have stable side slopes at inclinations of up to 2H:1V.

Global stability analyses were conducted for 2H:1V earth fill embankments and for 1.25H:1V rock fill embankments. The computer output for the stability analyses is shown in Figures F1 to F4 in Appendix F. In each case the factor of safety against global failure was greater than 1.5.

Settlement analysis involved computation of the immediate settlement of the foundation soils under the imposed embankment loading using elastic theory, and calculation of long-term consolidation settlement of the silty clay layer using Terzaghi one-dimensional consolidation theory. The engineering properties of the soils used in the analyses were selected based on laboratory oedometer testing conducted on similar materials during other investigations on the Highway 11 project as well as on correlations developed between moisture/index/strength properties and compression parameters from previous testing.

The soils under the east approach embankment are regarded as behaving as cohesionless materials and settlements are expected to be immediate in nature. It is estimated that the settlement under the embankment loading will be in the order of 30 to 40 mm.

A 1.8 to 2.2 m thick layer of silty clay was encountered below the upper silt/sand unit in boreholes RA4 and 394-4 drilled at the west abutment. This layer was not identified in the approach borehole RA5. Consolidation settlements in the order of 50 mm are anticipated due to the loading imposed by a 9 m high embankment constructed over this deposit. This settlement is expected to be more than 90% complete after 3 months. It is therefore recommended that the approach embankment be in place at least 3 months prior to road construction to reduce the post-construction settlement experienced by the road surface.

The embankments discussed above are considered to be stable under earthquake loading on the assumption of a stable foundation. This topic is dealt with more completely in Section 15: Seismic Considerations.

All topsoil and organic soils should be stripped from the footprint of the immediate approach fills.

Embankment construction should be in accordance with OPSS 206, as amended by Special Provision "Amendment to OPSS 206, December 1993", dated November 2002 and referenced in Appendix E.

Where embankments are higher than 6 m, mid-height berms should be incorporated in the design. The berms should:

- extend for the length through which the embankment height exceeds 6 m
- be 2 m wide
- have 2% positive drainage to shed run-off water.

Earth fill embankment slopes must be provided with erosion protection in accordance with OPSS 572.

12 RETAINED SOIL SYSTEMS

RSS walls used in conjunction with bridge abutments must be “High Performance” and “High Appearance”. Therefore it is critical that the RSS walls are not subject to settlement due to compression of the foundation soils and embankment fill.

Provided the approach fill is placed at least 3 months prior to RSS wall construction and proper ground preparation is carried out prior to construction of the walls, RSS systems are considered suitable for the subsurface conditions at this site and are expected to meet the aesthetic and structural requirements. The following minimum preparation of the base below the RSS is recommended:

- The topsoil and loose to very loose silt/sand (approximate depth of 1.0 m indicated by the boreholes) should be stripped from the footprint of the RSS.
- The RSS mass must be founded on an engineered fill pad at least 2 m thick. The engineered fill must consist of OPSS Granular A or Granular B in conformance with SP110F13 and compacted according to OPSS 501 and SP105S10.
- The engineered fill pad must extend at least 500 mm beyond the limits of the RSS mass and levelling strip.
- The highest permitted founding level for the underside of the engineered fill is Elevation 319.0 at both the east and west abutments. Lower founding elevations may be required to accommodate the required thickness of engineered fill.
- For a RSS wall bearing on rock fill, the rock fill subgrade must be blinded with spall material and covered by a minimum 600 mm thickness of OPSS Granular B Type II fill.
- Approach fill should be in place for at least 3 months prior to construction of the RSS to reduce post-construction consolidation settlement of the foundation soils.

For a wall constructed on a prepared subgrade as outlined above, the geotechnical parameters to be used for the design of the RSS walls are presented in Table 12.1.

Table 12.1 – RSS Design Parameters

Parameter	East Abutment	West Abutment
Factored ULS Bearing Resistance at Contact of RSS Wall and Granular Pad	500 kPa	400 kPa
SLS Bearing Resistance at Contact of RSS Wall and Granular Pad	250 kPa	300 kPa
Coefficient of Sliding Resistance at Contact between RSS Wall and Granular Pad	0.55	0.55

Preliminary stability analysis of RSS walls with maximum heights of 12.5 and 9.0 m for the east and west abutments, respectively, and reinforcement extending a distance back from the wall face a

distance of two-thirds of the wall height was carried out. The analyses indicate a minimum factor of safety of 1.4 against global instability.

The design, supply and construction of RSS must be in accordance with SP 599S22. The supplier of the proprietary RSS system must demonstrate that it will meet the Ministry's specifications for performance and appearance. The RSS supplier/designer may specify more stringent criteria or other requirements related to the particular design.

13 BACKFILL TO ABUTMENTS

In the case of integral or semi-integral abutments, backfill to the abutment should be granular material.

In the case of a conventional abutment, granular backfill is recommended but rock backfill can be permitted. A NSSP is required to specify grading limits for the rock fill. The rock fill used as backfill to the abutment should be limited to fragments no greater than 75 mm and including adequate spalls to fill voids in the rock fill.

In all cases where the approach embankment consists of rock fill and granular backfill to the abutment wall is used, the granular backfill must consist of OPSS Granular "B" Type II.

The backfill to the abutment walls must be in accordance with OPSS 902 as amended by Special Provision 902S01. Granular backfill must be placed to the extents shown in OPSD 3501.000, and rock backfill must be placed to the extents shown in OPSD 3505.000.

Compaction equipment to be used adjacent to retaining structures must be restricted in accordance with SSP 105S10.

The design of the abutment must incorporate a subdrain as shown in OPSD 3501.000 or OPSD 3505.000, as applicable.

14 EARTH PRESSURE COEFFICIENTS (ABUTMENTS)

Earth pressures acting on the structure may be assumed to be triangular and to be governed by the characteristics of the abutment backfill. For a fully drained condition, the pressures should be computed in accordance with the CHBDC but generally are given by the expression:

$$p_h = K (\gamma h + q)$$

where: p_h = horizontal pressure on the wall at depth h (kPa)

K = earth pressure coefficient (see Table 14.1)

γ = unit weight of retained soil (see Table 14.1)

h = depth below top of fill where pressure is computed (m)

q = value of any surcharge (kPa)

Earth pressure coefficients for backfill to the abutment wall are dependent on the material used as backfill. Typical values are shown in Table 14.1.

Table 14.1 – Earth Pressure Coefficients (K)

Condition	Earth Pressure Coefficient (K)					
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$		Rock Fill (Limited to 300 mm size) $\phi = 42^\circ, \gamma = 19 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (Unrestrained Wall)	0.27	0.40*	0.31	0.43*	0.2	.30*
At rest (Restrained Wall)	0.43	-	0.47	-	.33	-
Passive (Movement Towards Soil Mass)	3.7	-	3.3	-	5.0	-

* For wing walls.

In conventional design, the use of a material with a high friction angle and low active pressure coefficient (e.g. Granular A, Granular B Type II) might be preferred as it results in lower earth pressures acting on the wall. In the case of integral abutments, material with a lower passive pressure coefficient (e.g. Granular B Type I) might be preferred as it results in lower forces acting on the ballast wall as the wall moves toward the soil mass.

The factors in Table 14.1 above are “ultimate” values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.9.1 (a) in the Commentary to the Canadian Highway Bridge Design Code.

In accordance with Clause 6.9.3 of the CHBDC, a compaction surcharge should be added. The magnitude should be 12 kPa at the top of fill and decreasing to 0 kPa at a depth of 2.0 m for Granular B Type I or 1.7 m for Granular A or Granular B Type II.

15 SEISMIC CONSIDERATIONS

For design purposes, the site is treated as lying in Seismic Zone 1.

15.1 Seismic Design Parameters

The following seismic parameters should be used for design:

- Velocity Related Seismic Zone 1
- Zonal Velocity Ratio 0.05
- Acceleration Related Seismic Zone 1
- Zonal Acceleration Ratio 0.05
- Peak Horizontal Acceleration 0.08

The Soil Profile Type at this site has been classified as Type I. Thus, according to Table 4.4.6.1 of the CHBDC, a Site Coefficient “S” of 1.0 should be used in seismic design.

15.2 Liquefaction Potential

The potential for liquefaction of the foundation soils has been assessed using the Seed and Idriss (1971) method¹.

Using this method, it was determined that the foundation soils at the abutments are not in danger of liquefaction under earthquake loading.

15.3 Retaining Wall Dynamic Earth Pressures

In accordance with Clause 4.6.4 of the CHBDC, retaining structures should be designed using active (K_{AE}) and passive (K_{PE}) earth pressure coefficients that incorporate the effects of earthquake loading. The seismic earth pressure coefficients to be used in design at this site are shown in Table 15.1.

Table 15.1 – Earth Pressure Coefficients (K) for Seismic Design

Condition	Earth Pressure Coefficient (K) for Earthquake Loading					
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ, \delta = 17^\circ$		OPSS Granular B Type I $\phi = 32^\circ, \delta = 16^\circ$		Rock Fill (Limited to 300 mm size) $\phi = 42^\circ, \delta = 21^\circ$	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active*, K_{AE} (Unrestrained Wall)	0.28	0.46	0.31	0.58	0.21	.30
At rest**, K_{OE} (Restrained Wall)	0.53	-	0.58	-	.44	-
Passive*, K_{PE} (Movement Towards Soil Mass)	7.0	-	5.5	-	14.1	-

* After Mononobe and Okabe, passive case assumes a horizontal surface in front of the wall.

** After Woods

In Table 15.1, the angle of friction between the wall and the backfill, δ , is taken as 50% of the angle of internal friction of the backfill, ϕ .

¹ Seed, H.B. and Idriss, I.M. 1971, "Simplified Procedure for Evaluating Soil Liquefaction Potential" *Journal of Soil Mechanics and Foundations Division*, ASCE, Vol. 101, No. SM9, pp. 1249 – 1273.

15.4 Slope Stability Considerations

Seismic effects were taken into account in the slope stability analyses conducted for this site using pseudo-static methods and assuming that the foundation soils would not be subject to liquefaction. Under these conditions, satisfactory factors of safety were obtained from the analysis, i.e. all values exceeded 1.0.

16 CONSTRUCTION CONCERNS

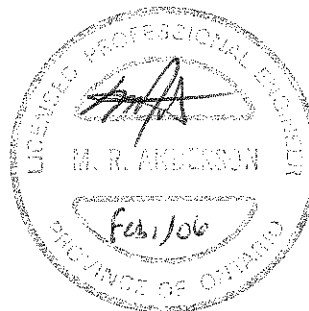
Potential construction concerns include, but are not necessarily limited to:

- The possibility of piles reaching refusal on large boulders. In this case, the Hiley formula is not appropriate and site staff must make a decision regarding pile resistance and the appropriateness of continued driving.
- The potential variability of pile lengths at refusal or set.
- Seepage and potential sloughing of excavation sidewalls due to a perched water condition.

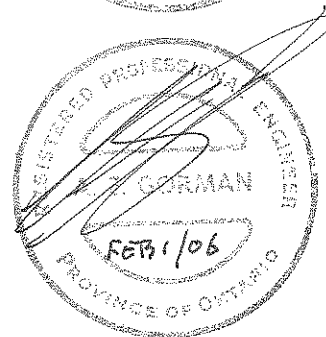
17 CLOSURE

Engineering analysis and preparation of the foundation design report was conducted by Mr. Murray Anderson, P.Eng. The report was reviewed by Mr. Alastair E. Gorman, P.Eng. The report was also reviewed by Dr. P.K. Chatterji, Ph.D., a Designated Principal Contact for MTO Foundations Projects.

Thurber Engineering Ltd.
Murray R. Anderson, P.Eng., M.Eng.
Senior Geotechnical Engineer



Alastair E. Gorman, P.Eng., M.Sc.
Senior Foundations Engineer



P.K. Chatterji, P.Eng., Ph.D.
Review Principal



Appendix A

Record of Borehole Sheets

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

CLASSIFICATION	PARTICLE SIZE	VISUAL IDENTIFICATION
Boulders	Greater than 200mm	same
Cobbles	75 to 200mm	same
Gravel	4.75 to 75mm	5 to 75mm
Sand	0.075 to 4.75mm	Not visible particles to 5mm
Silt	0.002 to 0.075mm	Non-plastic particles, not visible to the naked eye
Clay	Less than 0.002mm	Plastic particles, not visible to the naked eye

2. COARSE GRAIN SOIL DESCRIPTION (50% greater than 0.075mm)

TERMINOLOGY	PROPORTION
Trace or Occasional	Less than 10%
Some	10 to 20%
Adjective (e.g. silty or sandy)	20 to 35%
And (e.g. sand and gravel)	35 to 50%

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

DESCRIPTIVE TERM	UNDRAINED SHEAR STRENGTH (kPa)	APPROXIMATE SPT ⁽¹⁾ 'N' VALUE
Very Soft	12 or less	Less than 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	Greater than 200	Greater than 30

NOTE: Hierarchy of Soil Strength Prediction

- 1) Laboratory Triaxial Testing
- 2) Field Insitu Vane Testing
- 3) Laboratory Vane Testing
- 4) SPT value
- 5) Pocket Penetrometer



4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

DESCRIPTIVE TERM	SPT 'N' VALUE
Very Loose	Less than 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	Greater than 50

5. LEGEND FOR RECORDS OF BOREHOLES

SYMBOLS AND ABBREVIATIONS FOR SAMPLE TYPE	SS Split Spoon Sample	WS Wash Sample	AS Auger (Grab) Sample
	TW Thin Wall Shelby Tube Sample	TP Thin Wall Piston Sample	
	PH Sampler Advanced by Hydraulic Pressure	PM Sampler Advanced by Manual Pressure	
	WH Sampler Advanced by Self Static Weight	RC Rock Core	SC Soil Core

$$\text{Sensitivity} = \frac{\text{Undisturbed Shear Strength}}{\text{Remoulded Shear Strength}}$$


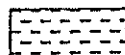



 Water Level
 Shear Strength Determination by Pocket Penetrometer

- (1) SPT 'N' Value Standard Penetration Test 'N' Value – refers to the number of blows from a 63.5kg hammer free falling a height of 0.76m to advance a standard 50 mm outside diameter split spoon sampler for 0.3 m depth into undisturbed ground.
- (2) DCPT Dynamic Cone Penetration Test – Continuous penetration of a 50 mm outside diameter, 60° conical steel point attached to "A" size rods driven by a 63.5 kg hammer free falling a height of 0.76 m. The resistance to cone penetration is the number of hammer blows required for each 0.3 m advance of the conical point into undisturbed ground.

UNIFIED SOILS CLASSIFICATION


MAJOR DIVISIONS		GROUP SYMBOL	TYPICAL DESCRIPTION
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW	Well-graded gravels or gravel-sand mixtures, little or no fines.
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines.
		GM	Silty gravels, gravel-sand-silt mixtures.
		GC	Clayey gravels, gravel-sand-clay mixtures.
	SAND AND SANDY SOILS	SW	Well-graded sands or gravelly sands, little or no fines.
		SP	Poorly-graded sands or gravelly sands, little or no fines.
		SM	Silty sands, sand-silt mixtures.
		SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS	SILTS AND CLAYS $W_L < 50\%$	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays. ($W_L < 30\%$).
		CI	Inorganic clays of medium plasticity, silty clays. ($30\% < W_L < 50\%$).
		OL	Organic silts and organic silty-clays of low plasticity.
	SILTS AND CLAYS $W_L > 50\%$	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
		CH	Inorganic clays of high plasticity, fat clays.
		OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils.
CLAY SHALE			
SANDSTONE			
SILTSTONE			
CLAYSTONE			
COAL			

EXPLANATION OF ROCK LOGGING TERMS

ROCK WEATHERING CLASSIFICATION		SYMBOLS	
Fresh (FR)	No visible signs of weathering.		
Fresh Jointed (FJ)	Weathering limited to the surface of major discontinuities.		CLAYSTONE
Slightly Weathered (SW)	Penetrative weathering developed on open discontinuity surfaces, but only slight weathering of rock material.		SILTSTONE
Moderately Weathered (MW)	Weathering extends throughout the rock mass, but the rock material is not friable.		SANDSTONE
Highly Weathered (HW)	Weathering extends throughout the rock mass and the rock is partly friable.		COAL
Completely Weathered (CW)	Rock is wholly decomposed and in a friable condition, but the rock texture and structure are preserved.		Bedrock (general)

DISCONTINUITY SPACING		STRENGTH CLASSIFICATION			
Bedding	Bedding Plane Spacing	Rock Strength	Approximate Uniaxial Compressive Strength (MPa)	(psi)	Field Estimation of Hardness*
Very thickly bedded	Greater than 2m	Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Thickly bedded	0.6 to 2m				
Medium bedded	0.2 to 0.6m	Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Thinly bedded	60mm to 0.2m				
Very thinly bedded	20 to 60mm	Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Laminated	6 to 20mm				
Thinly Laminated	Less than 6mm	Medium Strong	25.0 to 50.0	3,500 to 7,500	Breaks under single blow of geological hammer.
TERMS		Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.	Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.	Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail
Rock Quality Designation: (RQD)	Total length of sound core recovered in pieces 0.1m in length or larger as a percentage of total core run length.				
Uniaxial Compressive Strength (UCS)	Axial stress required to break the specimen				
Fracture Index: (FI)	Frequency of natural fractures per 0.3m of core run.				

METRIC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 	PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W _P W W _L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
320.0							SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100				

[illegible]

(%) STRAIN AT FAILURE

ONTMT4S 2316.GPJ 30/01/06

RECORD OF BOREHOLE No 394-1

2 OF 4

METRIC

W.P. 5404-04-01 LOCATION N 5 046 329.1 E 317 045 .7, Hwy 592 / Sunset Pass ORIGINATED BY GA
HWY 11 BOREHOLE TYPE Hollow Stem Augers and Mud Rotary Tricone COMPILED BY WM
DATUM Geodetic DATE 25.02.05 - 01.03.05 CHECKED BY MA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE							w _p w w _L		
								● QUICK TRIAXIAL × LAB VANE									
							20 40 60 80 100										

0 56 44
(SI+CL)

Continued Next Page

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

RECORD OF BOREHOLE No 394-1

3 OF 4

METRIC

W.P. 5404-04-01 LOCATION N 5 046 329.1 E 317 045 .7, Hwy 592 / Sunset Pass ORIGINATED BY GA
 HWY 11 BOREHOLE TYPE Hollow Stem Augers and Mud Rotary Tricone COMPILED BY WM
 DATUM Geodetic DATE 25.02.05 - 01.03.05 CHECKED BY MA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L		
								SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	WATER CONTENT (%)				
								20 40 60 80 100	20 40 60				
			8	SS	84		299						
							298						
							297						
							296						
			9	SS	110/ .275		295						0 85 15 (SI+CL)
							294						
			10	SS	100		293						
							292						
			11	SS	109		291						
			12	SS	107								0 93 7 (SI+CL)
290.4													
29.6	END OF BOREHOLE AT 29.57 m. Piezometer installation consists of 19 mm diameter Schedule 40 PVC pipe												

Continued Next Page

+ 3 . × 3 : Numbers refer to
Sensitivity 20
15 5
10 (%) STRAIN AT FAILURE

ONTMT4S 2316.GPJ 30/01/06

RECORD OF BOREHOLE No 394-1

4 OF 4

METRIC

W.P. 5404-04-01 LOCATION N 5 046 329.1 E 317 045 .7, Hwy 592 / Sunset Pass ORIGINATED BY GA
 HWY 11 BOREHOLE TYPE Hollow Stem Augers and Mud Rotary Tricone COMPILED BY WM
 DATUM Geodetic DATE 25.02.05 - 01.03.05 CHECKED BY MA

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					
	with a 1.52 m slotted screen.												
	WATER LEVEL READINGS: DATE DEPTH (m) 03.03.05 26.13 04.03.05 24.71 0.9.03.05 20.92												

METRIC

[illegible]

(%) STRAIN AT FAILURE

METRIC

+ 3, x 3: Numbers refer to Sensitivity

RECORD OF BOREHOLE No 394-2

3 OF 4

METRIC

W.P. 5404-04-01 LOCATION N 5 046 342.4 E 317 006 .2, Hwy 592 / Sunset Pass ORIGINATED BY GA
 HWY 11 BOREHOLE TYPE Hollow Stem Augers and Mud Rotary Tricone COMPILED BY WM
 DATUM Geodetic DATE 04.03.05 - 04.03.05 CHECKED BY MA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
								20 40 60 80 100						
								○ UNCONFINED + FIELD VANE						
								● QUICK TRIAXIAL × LAB VANE						
								20 40 60 80 100				20 40 60		
300.8							302							
21.3	SAND, fine to medium grained, some silt, trace gravel Very Dense Brown Moist to Wet		8	SS	117		301							
							300							
			9	SS	100		299							
							298							
			10	SS	64		297							
							296							
							295							
			11	SS	61		294							
							293							
293.2														
29.0	SAND, medium to coarse grained, trace silt, some gravel Very Dense Brown Wet		12	SS	75									

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

METRIC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						WATER CONTENT (%)
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						
320.5 0.0	FILL: Temporary drilling platform													
319.6 0.9	SILT, fine grained, trace sand													
			1	SS	11								0 3 88 8	
316.0 4.6	SAND and GRAVEL, coarse grained, frequent cobbles and boulders Very Dense													
			2	SS	50/ .000									
312.6 7.9	GRAVEL, trace sand Very Dense Brown Wet													
			3	SS	52									

+ 3, × 3: Numbers refer to Sensitivity

METRIC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLT.	NUMBER	TYPE	"N" VALUES			20 40 60 80 100					
								SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE					
							WATER CONTENT (%) 20 40 60				kN/m ³	GR SA SI CL	
308.3							310						
12.2	SAND, medium to coarse grained, trace gravel, trace silt Compact Brown Wet		4	SS	24		309						
307.1							308						
13.4	SAND and GRAVEL, trace silt, occasional cobbles Dense to Very Dense Grey						307						
			5	SS	100/ .150		306						
							305						
			6	SS	41		304						
							303						
			7	SS	58		302						
							301						

+ 3, × 3: Numbers refer to Sensitivity

METRIC

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					
						○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE							
			8	SS	106								
			9	SS	100/.075								
			10	SS	123								
295.7													
24.8	END OF BOREHOLE AT 24.84 m. BOREHOLE OPEN TO 24.38 m IN CASING. BOREHOLE GROUTED TO SURFACE.												

RECORD OF BOREHOLE No 394-4

1 OF 3

METRIC

W.P. 5404-04-01 LOCATION N 5 046 345.8 E 316 956.7, Hwy 592/Sunset Pass ORIGINATED BY WRW/GA
 HWY 11 BOREHOLE TYPE Hollow Stem Augers, Casing and Coring COMPILED BY WM
 DATUM Geodetic DATE 15.10.04 - 18.10.04 CHECKED BY MA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT		NATURAL MOISTURE CONTENT		LIQUID LIMIT		UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W P		W		W L			
321.9								20 40 60 80 100									
0.0	TOPSOIL (150 mm)																
0.2	SAND, fine grained, trace to some silt Compact Brown Moist to Wet		1	SS	27		321										
			2	SS	19		320										
319.6																	
2.3	Silty CLAY, laminated, trace sand Stiff to Firm Grey-Brown		3	SS	11		319										
			4	SS	7												0 3 55 42
317.8							318										
4.1	Gravelly SAND, trace silt, occasional cobbles and boulders Dense to Very Dense Brown Dry		5	SS	37		317										
			6	SS	68		316										
			7	SS	68		314										
			8	SS	99		313										

Continued Next Page

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

RECORD OF BOREHOLE No 394-4

2 OF 3

METRIC

W.P. 5404-04-01 LOCATION N 5 046 345.8 E 316 956.7, Hwy 592/Sunset Pass ORIGINATED BY WRW/GA
 HWY 11 BOREHOLE TYPE Hollow Stem Augers, Casing and Coring COMPILED BY WM
 DATUM Geodetic DATE 15.10.04 - 18.10.04 CHECKED BY MA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
311.7														
10.2	SAND, trace gravel, trace silt Dense Brown Dry		9	SS	49		311							5 90 5 (SI+CL)
309.7							310							
12.2	SAND, fine grained, with cobbles and boulders, trace silt to silty Very Dense Grey Dry		10	SS	100/ .225		309							
			11	SS	40/ .075		308							
							307							
			12	SS	63		306							0 71 29 (SI+CL)
305.1							305							
16.8	SAND and GRAVEL, with cobbles and boulders, trace silt Dense to Very Dense Grey Dry		13	SS	49		304							
							303							
303.0			14	SS	63		302							52 43 4 (SI+CL)
18.9	SAND, trace to some gravel, trace silt, occasional cobbles and boulders Very Dense Brown Wet		15	SS	81									

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 394-4

3 OF 3

METRIC

W.P. 5404-04-01 LOCATION N 5 046 345.8 E 316 956.7, Hwy 592/Sunset Pass ORIGINATED BY WRW/GA
 HWY 11 BOREHOLE TYPE Hollow Stem Augers, Casing and Coring COMPILED BY WM
 DATUM Geodetic DATE 15.10.04 - 18.10.04 CHECKED BY MA

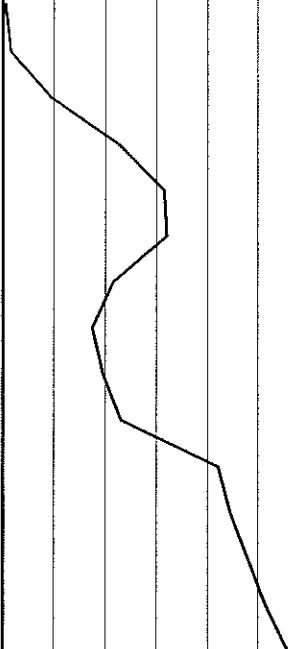
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
			16	SS	100/ .225												
			17	SS	105		301										
300.6			18	SS	103/ .125												
21.3	SAND and GRAVEL, with cobbles and boulders, trace silt Very Dense Brown Wet		19	SS	102/ .150		299										
			20	SS	100/ .100		298										
			21	SS	100/ .075		297										
295.9							296										
26.0	END OF BOREHOLE AT 25.98 m. BOREHOLE OPEN TO 25.98 m. Piezometer installation consists of 19 mm diameter Schedule 40 PVC pipe with a 1.52 m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) 19.10.04 15.2 21.10.04 15.6																

RECORD OF BOREHOLE No 394-4A

1 OF 1

METRIC

W.P. 5404-04-01 LOCATION N 5 046 346.9 E 316 955.6, Hwy 592/Sunset Pass ORIGINATED BY GA
 HWY 11 BOREHOLE TYPE Dynamic Cone Penetration Test (DCPT) COMPILED BY WM
 DATUM Geodetic DATE 19.10.04 - 19.10.04 CHECKED BY MA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						
0.0	DCPT started from surface.													
4.6	END OF DCPT AT 4.57 m.													

RECORD OF BOREHOLE No 394-5

1 OF 2

METRIC

W.P. 5404-04-01 LOCATION N 5 046 316.5 E 317 057.9, Hwy 592/Sunset Pass ORIGINATED BY GA
 HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY WM
 DATUM Geodetic DATE 19.10.04 - 19.10.04 CHECKED BY MA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE		WATER CONTENT (%) W _P W W _L				
321.0								20 40 60 80 100						
320.8	TOPSOIL (50 mm)													
0.1	Sandy SILT, occasional rootlets Loose to Compact Brown Dry		1	SS	6		321							
			2	SS	11		320							
	some sand													
			3	SS	26		319						0 16 79 5	
318.7														
2.3	SAND, fine to medium grained, trace silt Compact Grey to Brown Dry		4	SS	24		318							
			5	SS	24		317						0 98 2 (SI+CL)	
316.4														
4.6	SAND, fine to coarse grained, trace gravel, occasional cobbles Compact to Dense Brown Dry		6	SS	26		316							
			7	SS	29		315							
			8	SS	45		314							
							313							
			9	SS	66		312							

Continued Next Page

+ ³, × ³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 394-5

2 OF 2

METRIC

W.P. 5404-04-01 LOCATION N 5 046 316.5 E 317 057.9, Hwy 592/Sunset Pass ORIGINATED BY GA
 HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY WM
 DATUM Geodetic DATE 19.10.04 - 19.10.04 CHECKED BY MA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
								20 40 60 80 100	○ UNCONFINED + FIELD VANE						
								20 40 60 80 100	● QUICK TRIAXIAL × LAB VANE						
310.3							311								
10.7	SAND, fine grained, trace silt Dense		10	SS	47										
309.9	Brown														
11.1	Dry END OF BOREHOLE AT 11.13 m. BOREHOLE GROUTED TO SURFACE.						310								

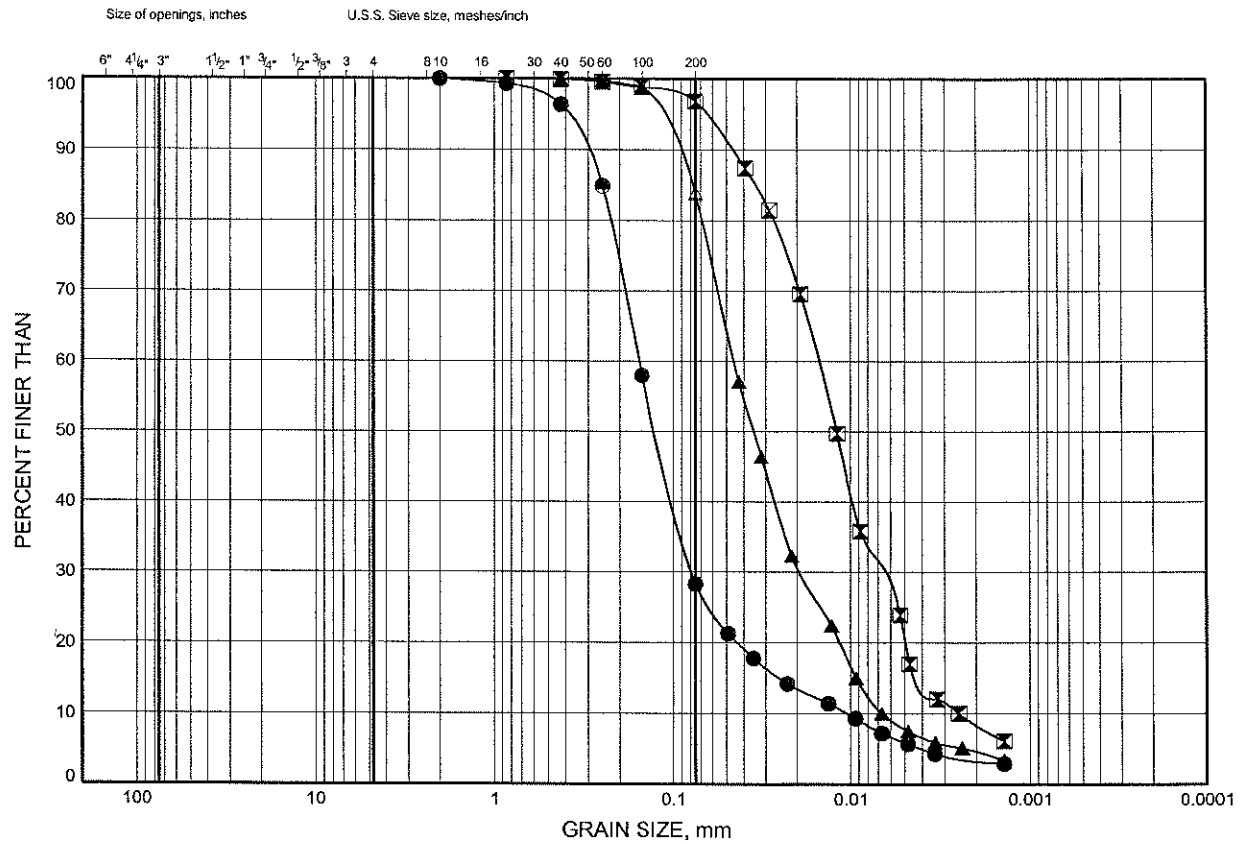
Appendix B

Laboratory Test Results

Hwy 11 Katrine GRAIN SIZE DISTRIBUTION

FIGURE B1

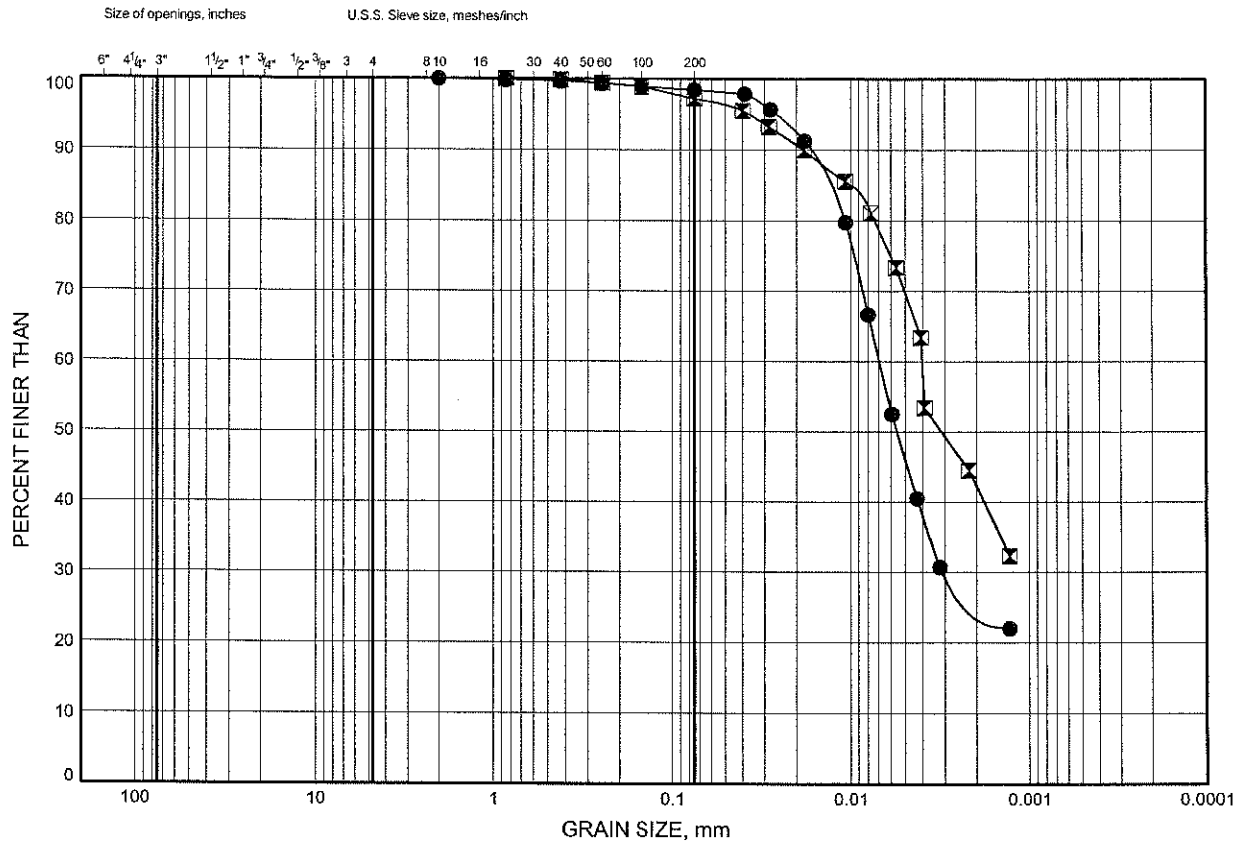
SILT to Silty SAND



Hwy 11 Katrine GRAIN SIZE DISTRIBUTION

FIGURE B2

Silty CLAY

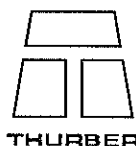


COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	394-2	9.45	312.68
◻	394-4	3.35	319.25

Date January 2006

Project 5404-04-01



THURBER

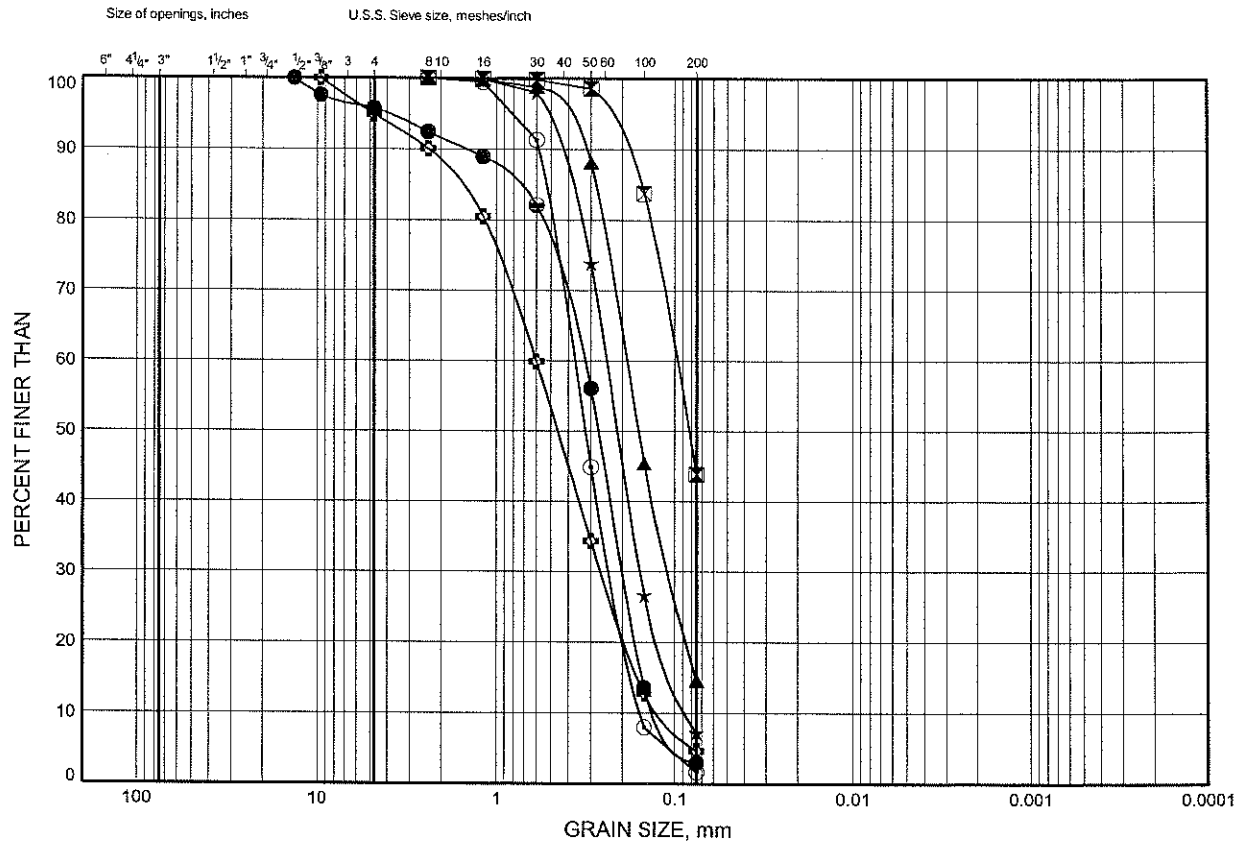
Prep'd WM

Chkd. MRA

Hwy 11 Katrine GRAIN SIZE DISTRIBUTION

FIGURE B3

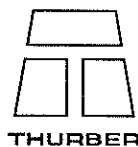
SAND



COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	394-1	3.35	316.63
⊠	394-1	15.54	304.44
▲	394-1	24.69	295.29
★	394-1	29.26	290.72
⊙	394-5	3.35	317.65
⊗	394-5	9.68	311.32

Date January 2006
Project 5404-04-01

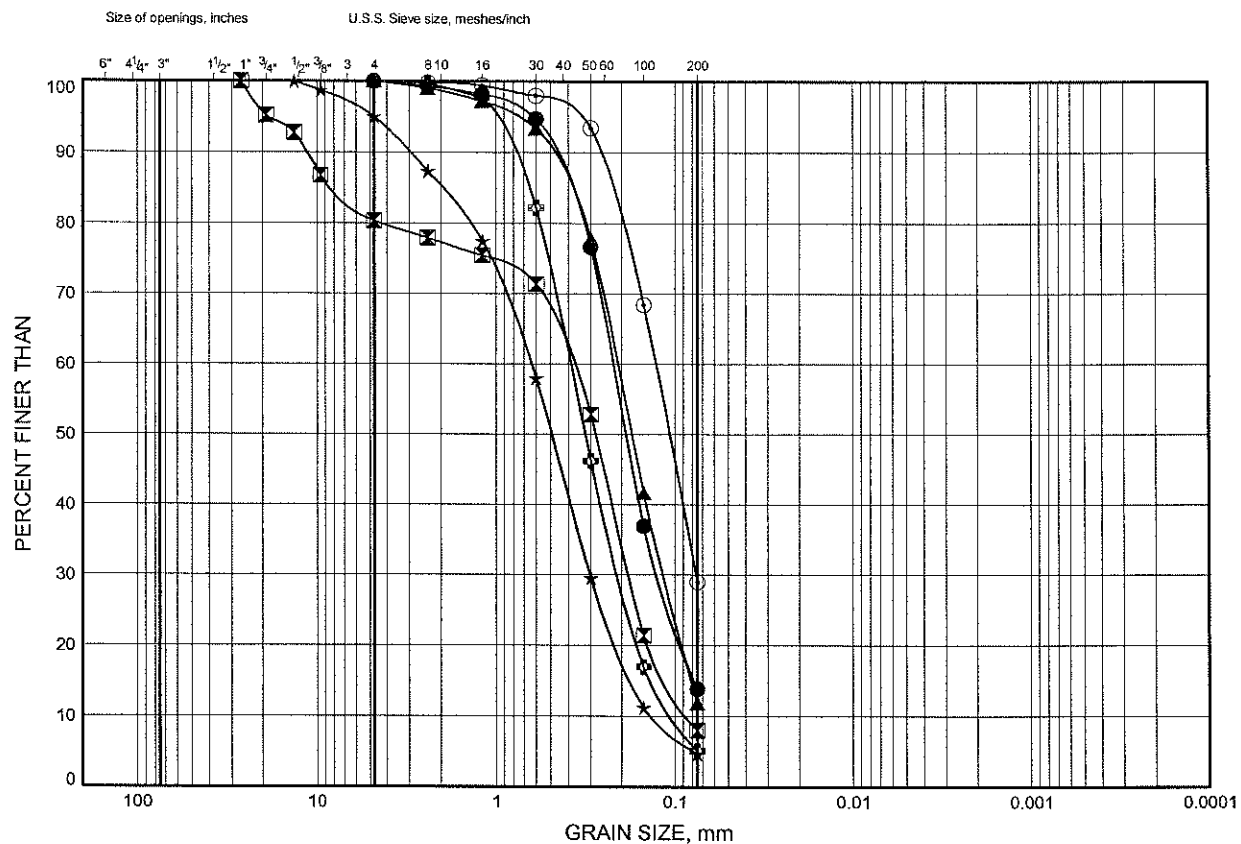


Prep'd WM
Chkd. MRA

Hwy 11 Katrina GRAIN SIZE DISTRIBUTION

FIGURE B4

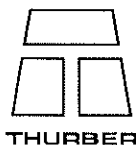
SAND



COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	394-2	24.69	297.44
⊠	394-2	30.78	291.35
▲	394-2	36.88	285.25
★	394-4	10.97	311.63
⊙	394-4	15.54	307.06
⊗	394-4	20.04	302.56

Date January 2006
Project 5404-04-01

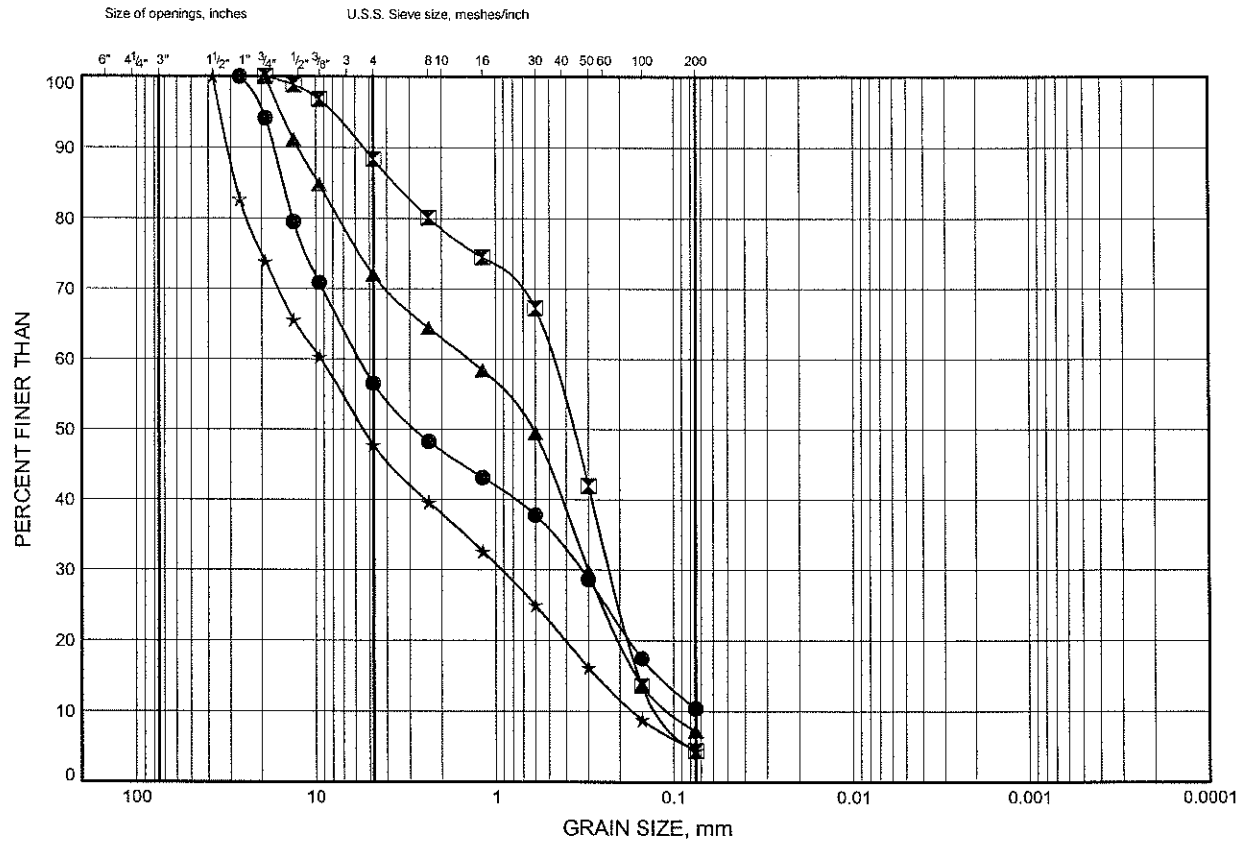


Prep'd WM
Chkd. MRA

Hwy 11 Katrine GRAIN SIZE DISTRIBUTION

FIGURE B5

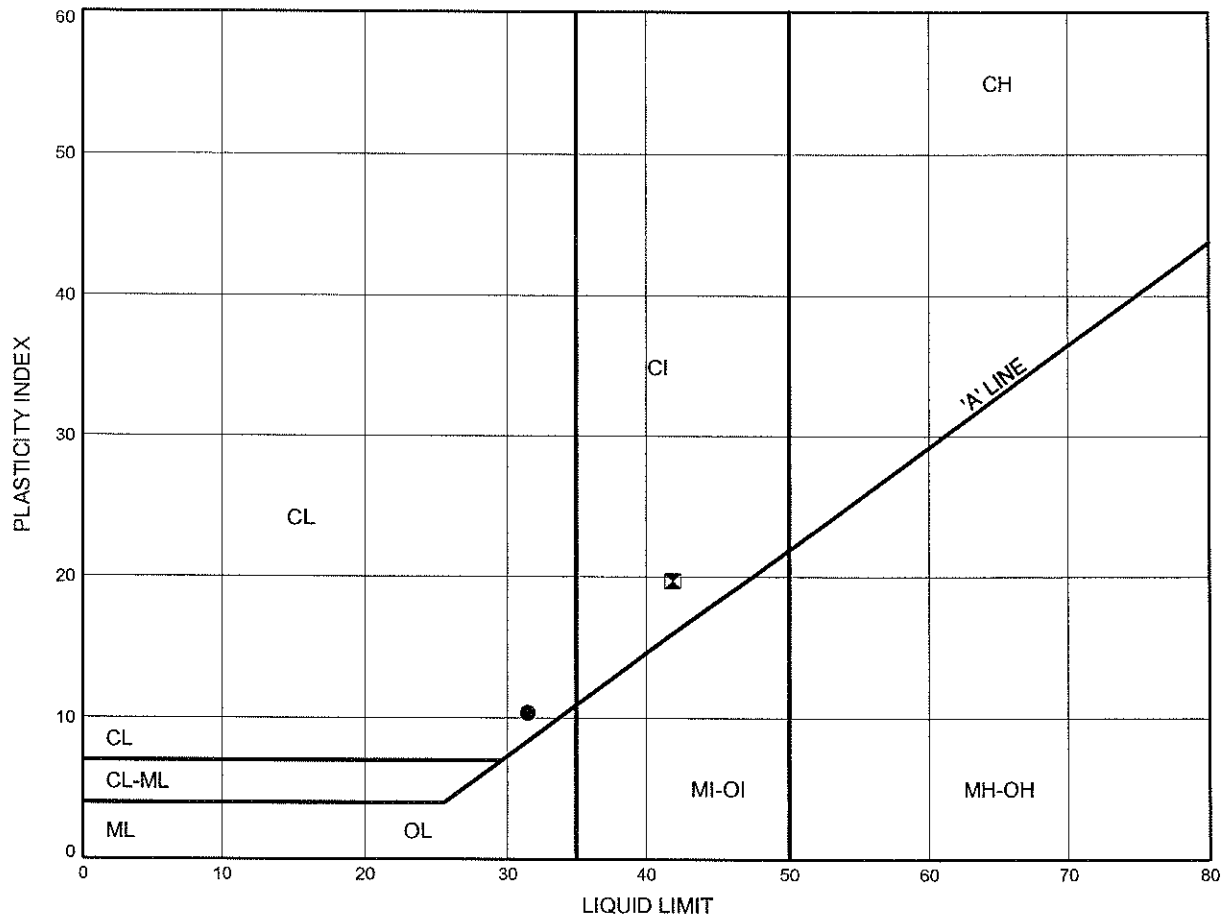
SAND and GRAVEL



Hwy 11 Katrine
ATTERBERG LIMITS TEST RESULTS

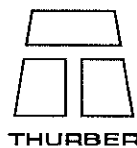
FIGURE B6

Silty CLAY



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	394-2	9.45	312.68
⊠	394-4	3.35	319.25

Date January 2006
 Project 5404-04-01



Prep'd WM
 Chkd. MRA

Appendix C

Data From Shaheen & Peaker Report

RECORD OF BOREHOLE No RA1

1 OF 1

METRIC

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

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
320.8	Ground Surface													
0.0	150 mm topsoil brown SILT very loose ----- with rootlets to 0.6 m. ----- grey damp compact		1	SS	4									
319.4			2	SS	24									
1.4			3	SS	20									
	FINE SAND trace to some silt, compact to dense, brown, damp		4	SS	19									
			5	SS	17									
			6	SS	19									
			7	SS	24									
			8	SS	29									
	some gravel		9	SS	27									
			10	SS	44									
			11	SS	27									
312.7	End of borehole *Borehole dry. (Water level not stabilized) and hole open to full depth on completion													
8.1														

RECORD OF BOREHOLE No RA2

1 OF 3

METRIC

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

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100					
320.6	Ground Surface													
0.0	150 mm Topsoil sandy	very loose	1	SS	2									
	SILT with silty clay seams to 2.1 m. compact, grey/brown, moist		2	SS	21									
			3	SS	15									
			4	SS	22									
			5	SS	16									
316.9														
3.7	FINE SAND trace to some silt, damp to moist		6	SS	19									
	brown		7	SS	14									
	grey		8	SS	16									
			9	SS	26									
	compact													
	dense to very dense		10	SS	43									
	some gravel		11	SS	48									
			12	SS	53									
			13	SS	48									
			14	SS	32									
305.6														

15.0

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No RA2

2 OF 3

METRIC

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

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa	WATER CONTENT (%)					
305.6														
15.0														
	compact		15	SS	23		305							
	dense to very dense						304							
			16	SS	32		303							0 96 (4)
	FINE SAND trace to some silt, occasional silt seams, grey						302							Commenced washboring
			17	SS	35		301							
	moist						300							
	wet		18	SS	79		299							
							298							
			19	SS	48		297							
							296							
			20	SS	61		295							
							294							
			21	SS	54		293							
							292							
			22	SS	62		291							0 93 (7)
			23	SS	53									April 20
														April 23
			24	SS	77									
290.6														

30.0

Continued Next Page

+ 3 x 3 : Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE




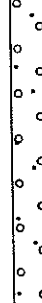
RECORD OF BOREHOLE No RA2										3 OF 3		METRIC			
W.P. 314-99-00		LOCATION Highway 11/Regional Road 592 - Katrine, ON - Coords: N 5 046 319.2; E 317 037.5				ORIGINATED BY R.A.									
DIST 52 HWY 11		BOREHOLE TYPE Solid & Hollow Stem Augers, Casing & Washboring				COMPILED BY G.T.									
DATUM Geodetic		DATE 19.04.01 to 24.04.01				CHECKED BY Z.O.									
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT		REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	20 40 60 80 100	W P W W L	WATER CONTENT (%)	20 40 60	γ	GR SA SI CL	
290.6															
30.0			25	SS	70		290								
							289								
			26	SS	99/29		288								
							287								
			27	SS	50/14		286								
							285								
			28	SS	97/29		284								
284.8							283								
35.8			29	SS	50/10		282								
							281								
			30	SS	99/20		280								
							279								
			31	SS	100/0		278								
			32	SS	50/5										
			33	SS	00/5										
278.0															
42.6	End of borehole *Water level at 1.5 m (not stabilized) and hole open to 18.3 m on completion														

RECORD OF BOREHOLE No RA3

1 OF 3

METRIC

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

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE					WATER CONTENT (%) W _p W W _L					
321.8 0.0	Ground Surface						20	40	60	80	100							
	FINE SAND compact, brown, moist (FILL)		1	SS	12													
			2	SS	16													
			3	SS	22													
			4	SS	16													
318.9 2.9	SAND: some silt, trace clay, laminated, compact, brown, moist	some silt	5	SS	15													
318.1 3.7	frequent silt seams, stiff		6	SS	12													
	SILTY CLAY layered, with silt seams, very stiff, grey/brown		7	SS	15													
			8	TW	PH													
	frequent silt seams		9	SS	27													
314.8 7.0	FINE SAND trace to some silt, dense, damp, brown		10	SS	35													
			11	SS	38													
	some gravel		12	SS	45													
310.1 11.7	SAND AND GRAVEL wet brown dense grey very dense		13	SS	37													
			14	SS	55													
306.8																		

15.0

Continued Next Page

+ 3, X 3: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

April 24

April 25
Commenced
washboring

RECORD OF BOREHOLE No RA3

2 OF 3

METRIC

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

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100					
306.8														
15.0														
	SAND AND GRAVEL dense to very dense grey, wet		15	SS	102/29	**								**102/29 denotes 102 blows/29 cm
			16	SS	116	***								***No recovery
			17	SS	57									
			18	SS	57									35 61 (4)
			19	SS	52									
	sand		20	SS	57									
			21	SS	60/14	****								****No recovery
	frequent cobble & boulders		22	RC	-									April 25
		boulder												
		cobble												April 26
291.8														
30.0														

Continued Next Page

+ 3 . x 3 : Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No RA3

3 OF 3

METRIC

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

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
							20	40	60	80	100	20	40	60	
291.8															
30.0	SAND AND GRAVEL grey, wet														
			25	SS	38										80 19 (1)
	compact to dense ----- very dense		26	SS	102										
			27	SS	60/14										
			28	SS	60/11										
282.7															
39.1	SILT														Unable to extend casing beyond 39.7 m
282.1	very dense, grey, wet		29	SS	60/5										
39.7	End of borehole *Water level at 3.4 m (not stabilized) and hole open to 7.9 m on completion Dynamic Cone Penetration Test performed from 23.4 m to 27.4 m & 29.5 m to 33.5 m soil stratigraphy inferred only. Borehole extend by coring from 28.9 m to 29.1 m & 29.7 m to 29.8 m.														

RECORD OF BOREHOLE No RA4

1 OF 2

METRIC

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

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80						100	20
321.4	Ground Surface																	
0.0	150 mm Topsoil		1	SS	6													
	SILT brown to 3.0 m, grey below loose to compact, moist		2	SS	13													
	sandy		3	SS	28													
	some silty clay seams, stiff		4	SS	15													
			5	SS	12													
317.7																		
3.7	SILTY CLAY grey moist to wet		6	SS	8													
	firm		7	SS	6													
	stiff		8	SS	12													
315.5																		
5.9	FINE SAND trace to some silt, compact, grey, damp		9	SS	20													
314.7																		
6.7	SAND AND GRAVEL grey		10	SS	41													
	dense		11	SS	39													
	compact		12	SS	14													
	very dense		13	SS	60/14													
	frequent cobbles																	
	damp		14	SS	60													
	wet																	
	boulder		15	RC	100%													
306.4																		

15.0

Continued Next Page

+ 3 . x 3: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

Commenced
washboring

June 04

June 05

RECORD OF BOREHOLE No RA5

1 OF 1

METRIC

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

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE									
321.3	Ground Surface						20	40	60	80	100						
0.0	150 mm Topsoil SILT some sand, rootlets to 0.6 m very loose, brown, moist		1	SS	3	*	321										
319.9			2	SS	-	**	320									**Spoon bouncing on tree roots	
1.4	FINE SAND: trace to some silt, dense, brown, moist		3	SS	50		319									0 87 (13)	
			4	SS	38		318										
			5	SS	38		317										
317.6			6	SS	41		316										
3.7	SAND AND GRAVEL: brown dense ----- very dense		7	SS	50	***	315									***Coarse gravel in spoon tip	
			8	SS	60/8		314										
			9	SS	111		313										
	damp ----- moist		10	SS	62		312										
311.7			11	SS	64												
9.6	End of borehole *Borehole dry (not stabilized) and open to 5.6 m on completion																

+ 3, X 3: Numbers refer to
Sensitivity

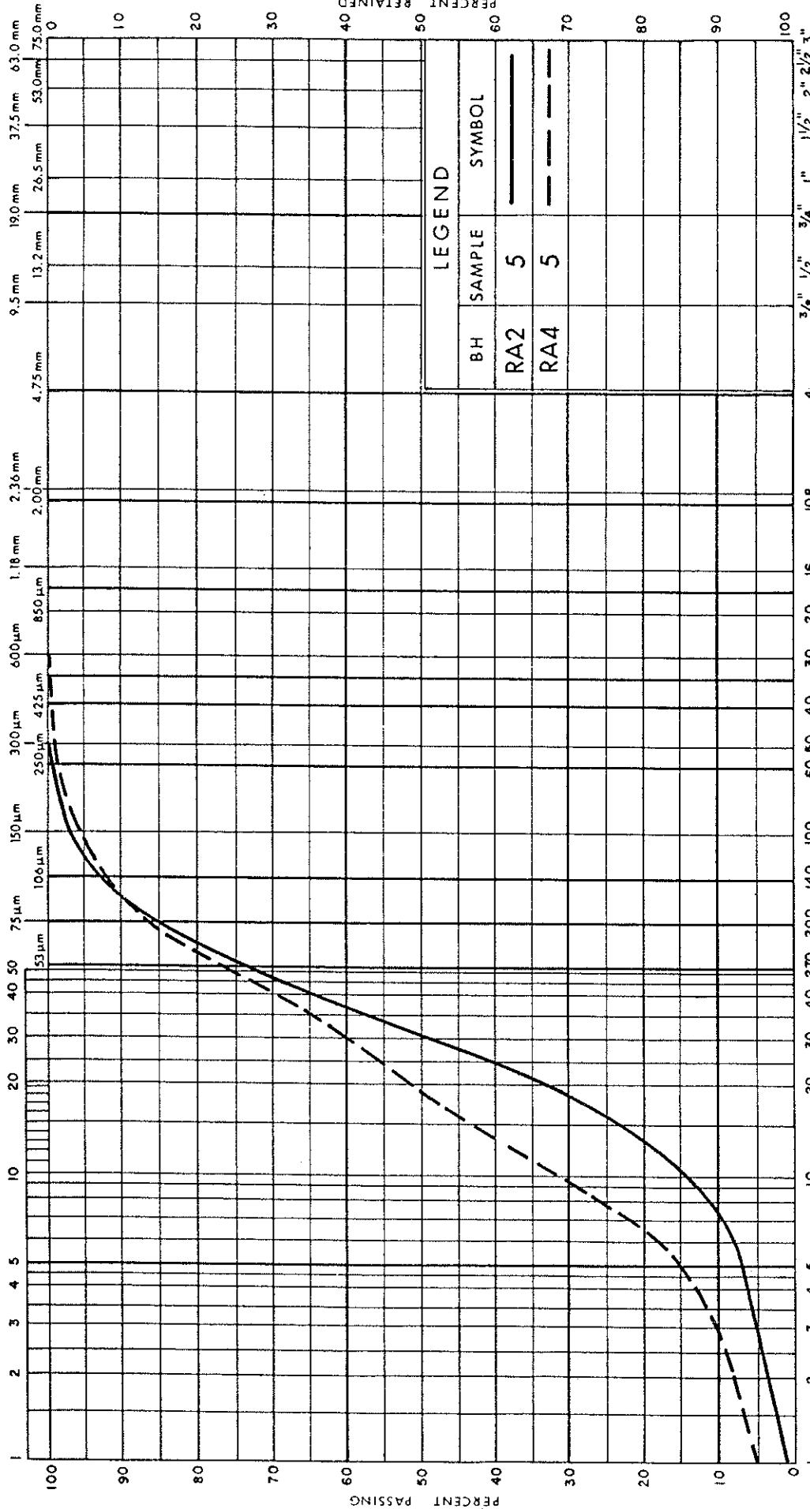
20
15 5
10 (%) STRAIN AT FAILURE

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT		SAND			GRAVEL		
		Fine	Medium	Coarse	Fine	Coarse	

GRAIN SIZE IN MICROMETERS

MINISTRY SIEVE DESIGNATION (Metric)



MINISTRY SIEVE DESIGNATION (Imperial)

Ministry of
Transportation



GRAIN SIZE DISTRIBUTION SILT

FIG No 1

W P 314-99-00

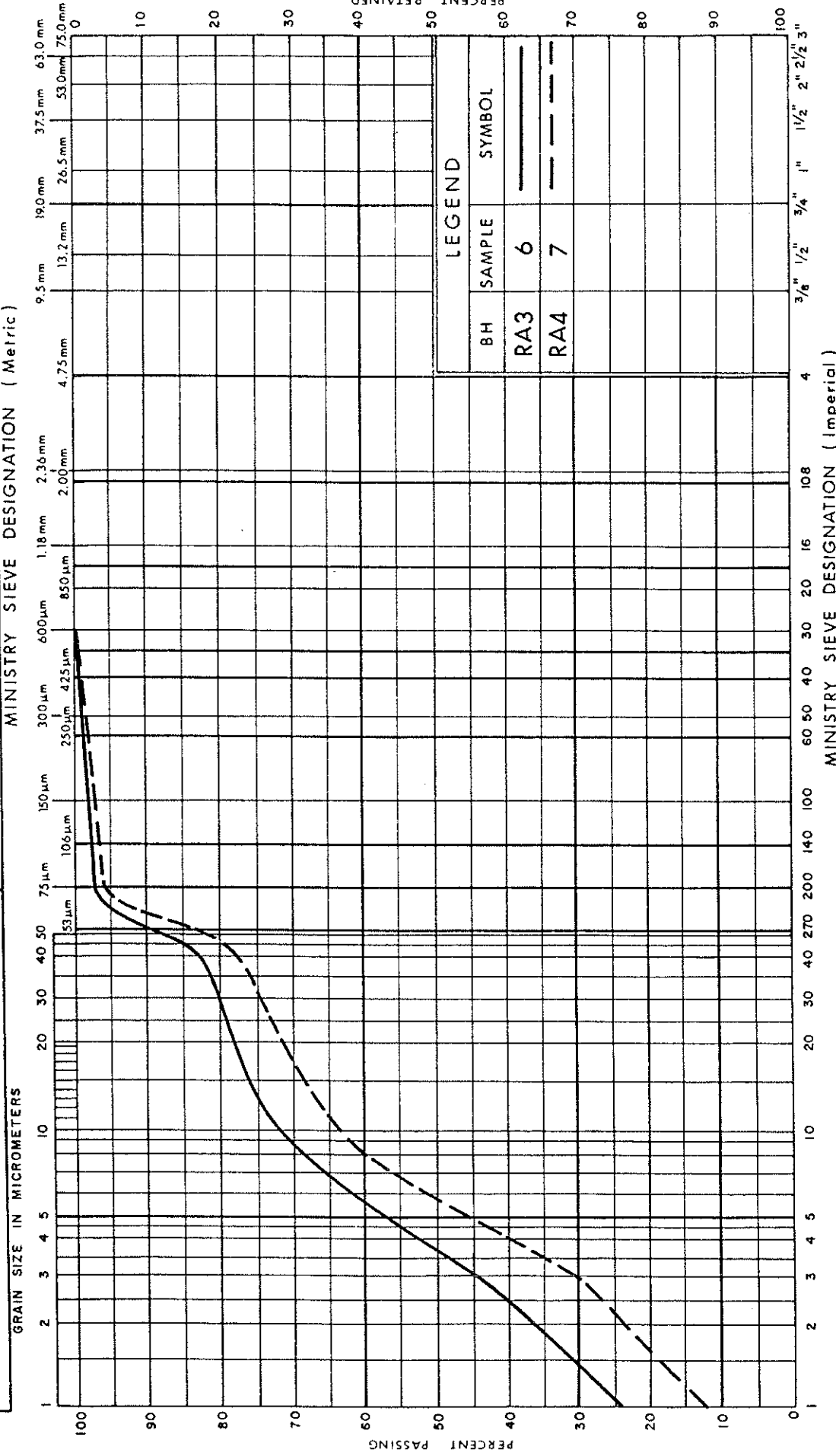
SPT 1010D

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT		SAND			GRAVEL	
		Fine	Medium	Coarse	Fine	Coarse

GRAIN SIZE IN MICROMETERS

MINISTRY SIEVE DESIGNATION (Metric)



GRAIN SIZE DISTRIBUTION SILTY CLAY

FIG No 2

W P 314-99-00

SPT 1010D

Ministry of
Transportation



Ontario

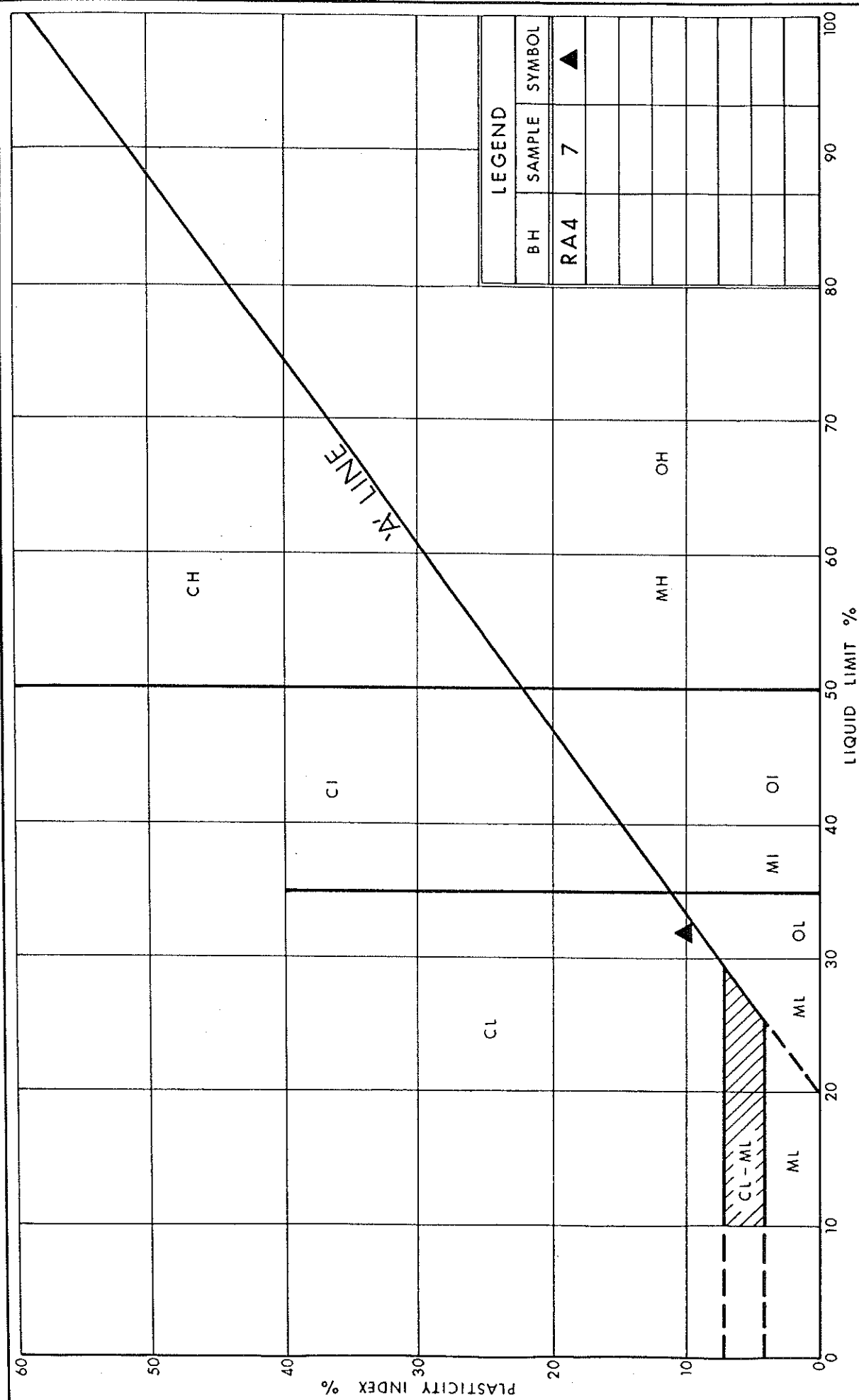


FIG No 3

WP 314-99-00

SPT 1010D

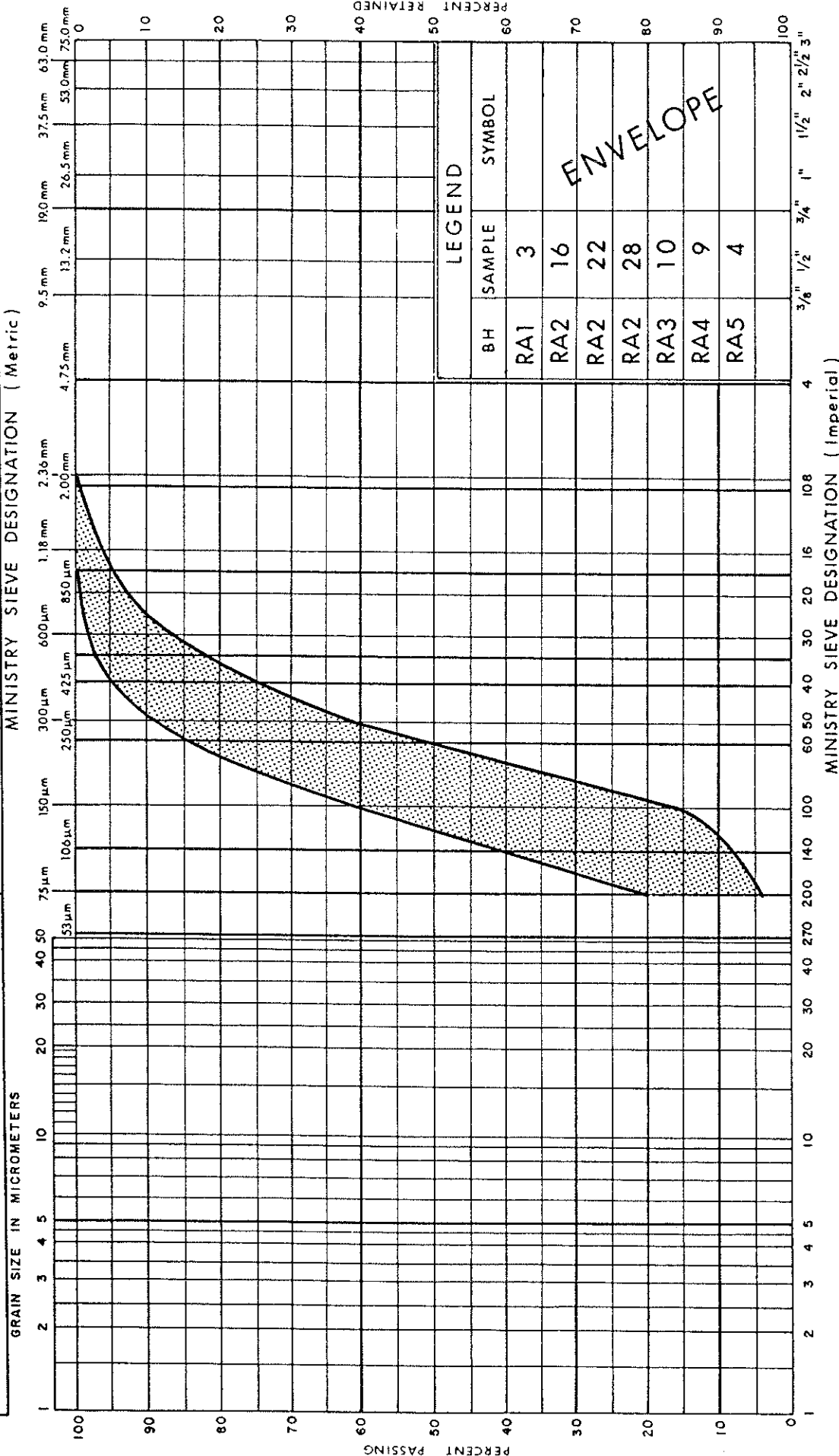
PLASTICITY CHART SILTY CLAY

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT		SAND			GRAVEL		
		Fine	Medium	Coarse	Fine	Coarse	

GRAIN SIZE IN MICROMETERS

MINISTRY SIEVE DESIGNATION (Metric)



Ministry of
Transportation

GRAIN SIZE DISTRIBUTION

FINE SAND

FIG No 4

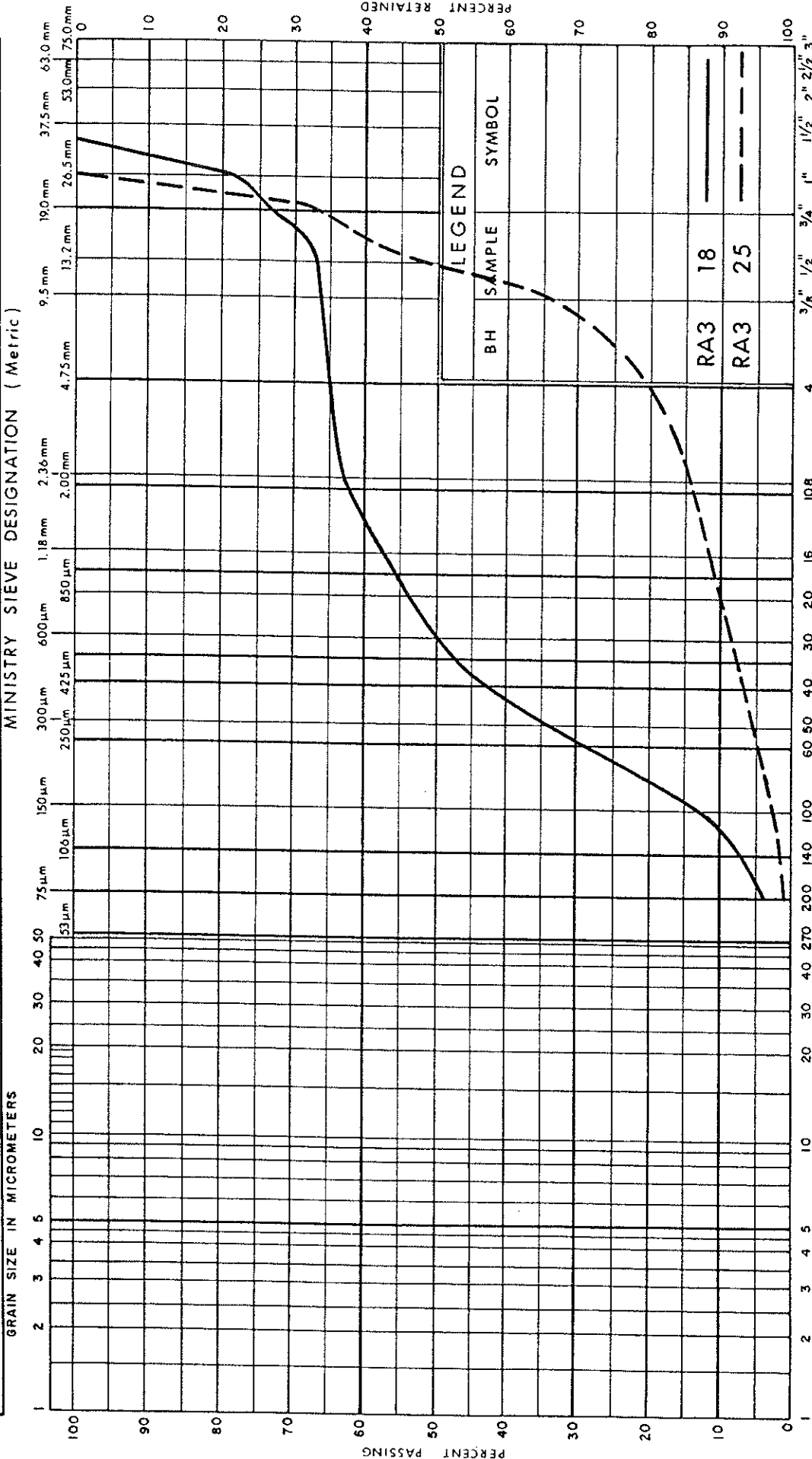
W P 314-99-00

SPT 1010D



UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT			SAND			GRAVEL		
Fine			Medium			Fine		
MINISTRY SIEVE DESIGNATION (Metric)			Coarse			Coarse		



Ministry of
Transportation



GRAIN SIZE DISTRIBUTION SAND AND GRAVEL

FIG No 5

W P 314-99-00

SPT 1010D

Appendix D

Foundation Comparison

COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

Driven Piles	Footings on Native Soil	Footings on Engineered Fill	Caissons
<p>Advantages:</p> <ul style="list-style-type: none"> i. Piles will develop high geotechnical resistance if driven to refusal in the very dense soil. ii. Allows choice of conventional, integral or semi-integral abutment design. iii. Readily installed. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher unit costs than footings. ii. Construction concerns related to the possibility of pile being obstructed by a boulder during driving. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. Ease of construction. ii. Allows choice of conventional or semi-integral abutment. iii. Lower cost than deep foundations. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Low geotechnical resistance is available in the highly variable upper soil deposits at this site. ii. Potential for unacceptable magnitude of settlement. iii. Subexcavation required to penetrate upper variable material, possibly within a perched water condition. <p>NOT RECOMMENDED</p>	<p>Advantages:</p> <ul style="list-style-type: none"> i. Would permit use of higher geotechnical resistance than is available on the native soil. ii. Allows choice of conventional or semi-integral abutment. iii. Allows use of perched abutments. iv. Lower cost than deep foundations. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Cost of constructing engineered fill. ii. Low geotechnical resistance is available in the highly variable upper soils at this site. iii. Potential for unacceptable magnitude of settlement. iv. Subexcavation required, including shoring, roadway protection and dewatering of perched water condition. <p>NOT RECOMMENDED</p>	<p>Advantages:</p> <ul style="list-style-type: none"> i. High resistance is available for caissons founded on very dense soil. ii. Construction of caissons could continue in freezing weather. iii. Choice of conventional or semi-integral abutment design. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Soil conditions encountered at this site are considered to be unsuitable. <p>NOT RECOMMENDED</p>

Appendix E

Special Provisions

The following Special Provisions are referenced in this report:

- Amendment to OPSS 206, December 1993
- Special Provision No. 902S01
- Special Provision No. 903S01

Suggested text for a NSSP on Pile Installation should contain the following:

“ The soil at this site contains cobbles and boulder. The presence of cobbles and boulders will potentially have an impact on the installation of piles. Some possible impacts that must be taken into consideration include, but are not necessarily limited to:

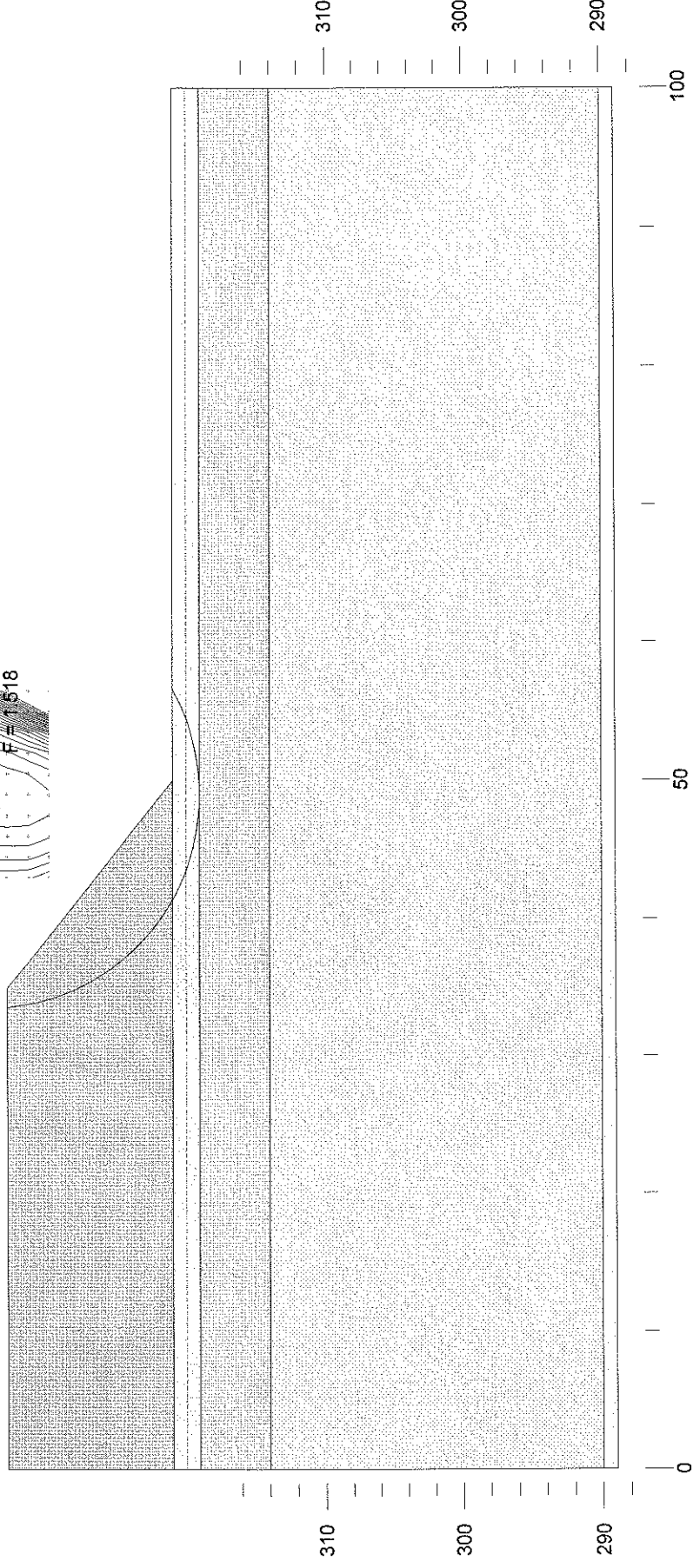
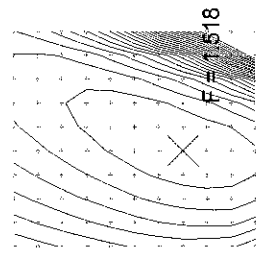
- *The need to provide protection to the pile tips in the form of driving shoes*
- *The cobbles and boulders may impede the driving of the piles resulting in more arduous driving*
- *Some piles may meet refusal on boulders that are large enough not to be dislodged or broken by the pile driving*
- *As a result of the presence of boulders, piles may meet refusal at varying depths*
- *Pile driving must be controlled according to the criteria specified for the site.”*

Appendix F

Selected Slope Stability Output

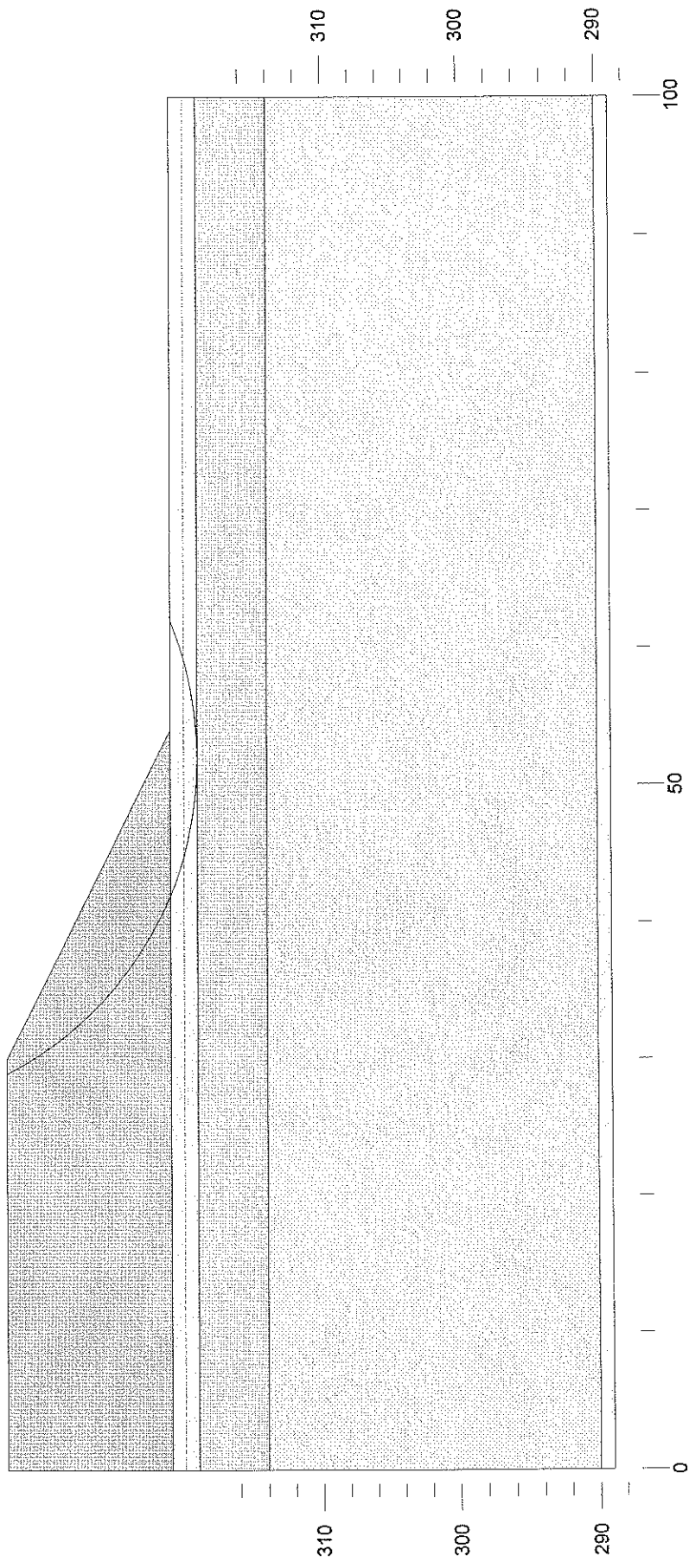
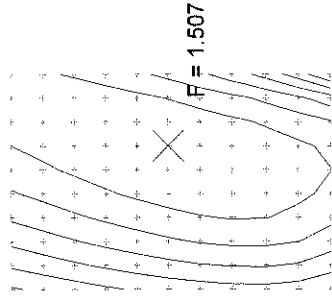
Thurber Engineering Ltd. - Toronto
 19-1423-16
 Hwy 11 - Katrine
 October 25, 2004
 Hwy 592 Sunset Pass
 east approach rockfill

	Gamma C	Phi	Piezo
	kN/m3	deg	Surf.
Rockfill	20	42	0
Silt	19	30	1
Compact Sand	20	31	0
Dense Sand	21	33	0
Sand & Gravel	22	38	0



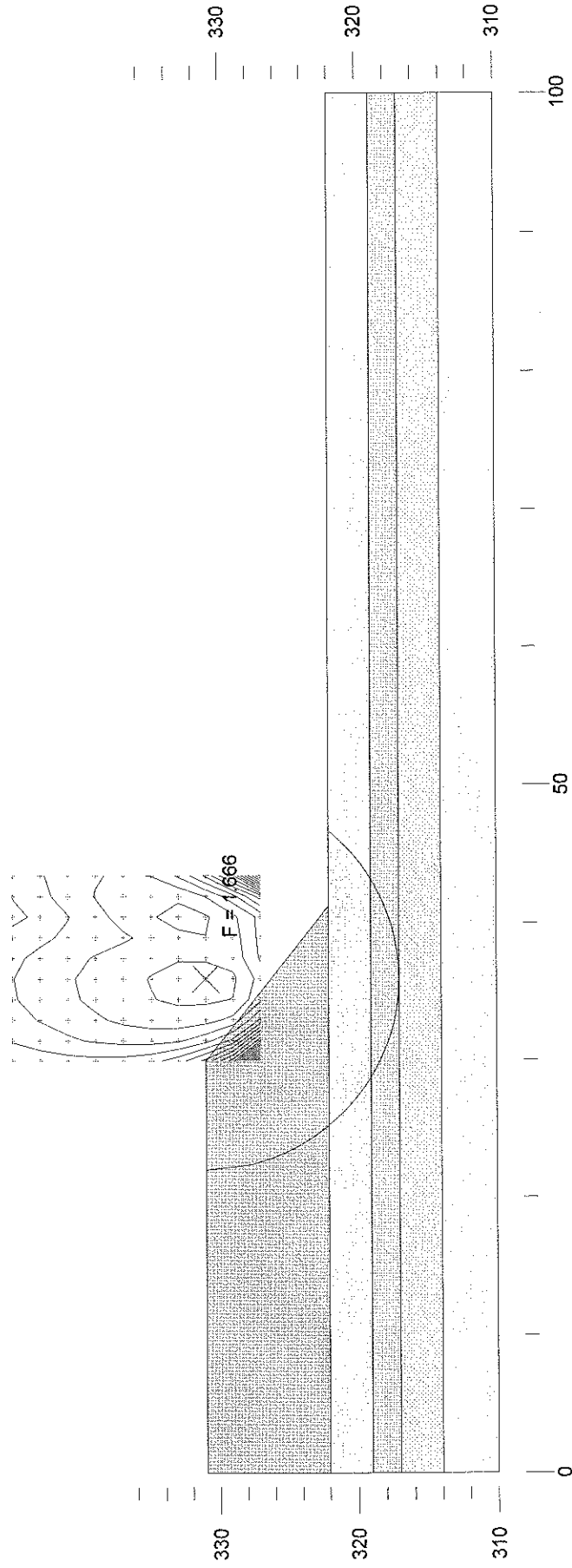
Thurber Engineering Ltd. - Toronto
 19-1423-16
 Hwy 11 - Katrine
 October 25, 2004
 Hwy 592 Sunset Pass
 east approach earth fill

	Gamma C	Phi	Piezo
	kN/m3	deg	Surf.
Earth Fill	20	30	0
Silt	19	30	1
Compact Sand	20	31	0
Dense Sand	21	33	0
Sand & Gravel	22	38	0



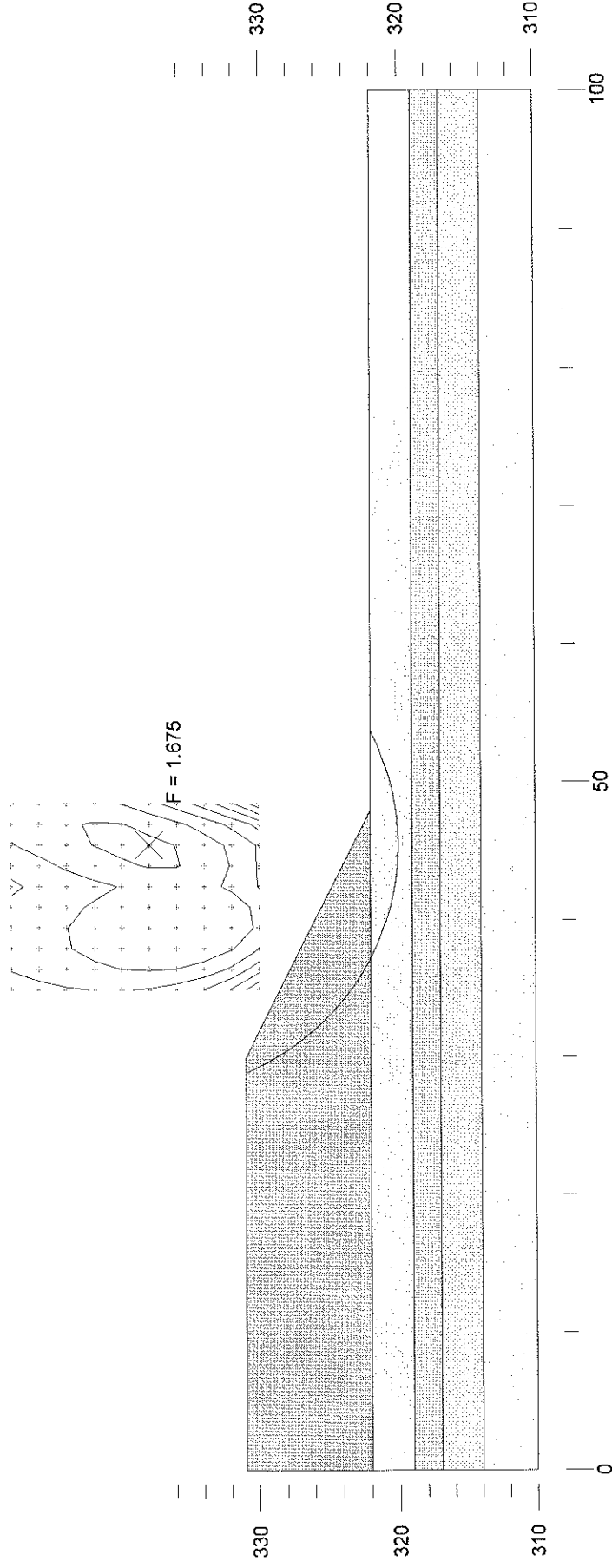
Thurber Engineering Ltd. - Toronto
 19-1423-16
 Hwy 11 - Katrine
 October 25, 2004
 Hwy 592 Sunset Pass
 west approach rockfill

	Gamma C	Phi	Piezo
	kN/m3	deg	Surf.
Rockfill	20	0	0
Silt	19	0	0
Clay	19	50	0
Sand	21	0	0
Sand & Gravel	22	0	0



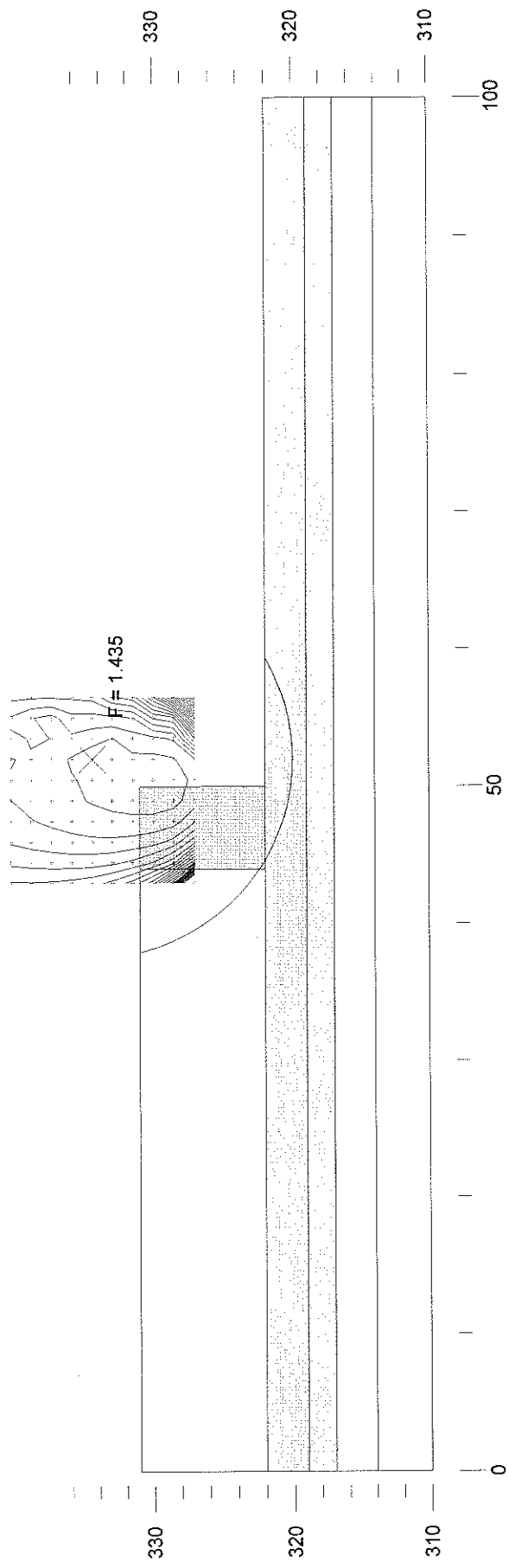
Thurber Engineering Ltd. - Toronto
 19-1423-16
 Hwy 11 - Katrine
 October 25, 2004
 Hwy 592 Sunset Pass
 west approach earth fill

	Gamma C	Phi	Piezo
	kN/m ³	deg	Surf.
Earth Fill	20	0	0
Silt	19	0	0
Clay	19	50	0
Sand	21	0	32
Sand & Gravel	22	0	38



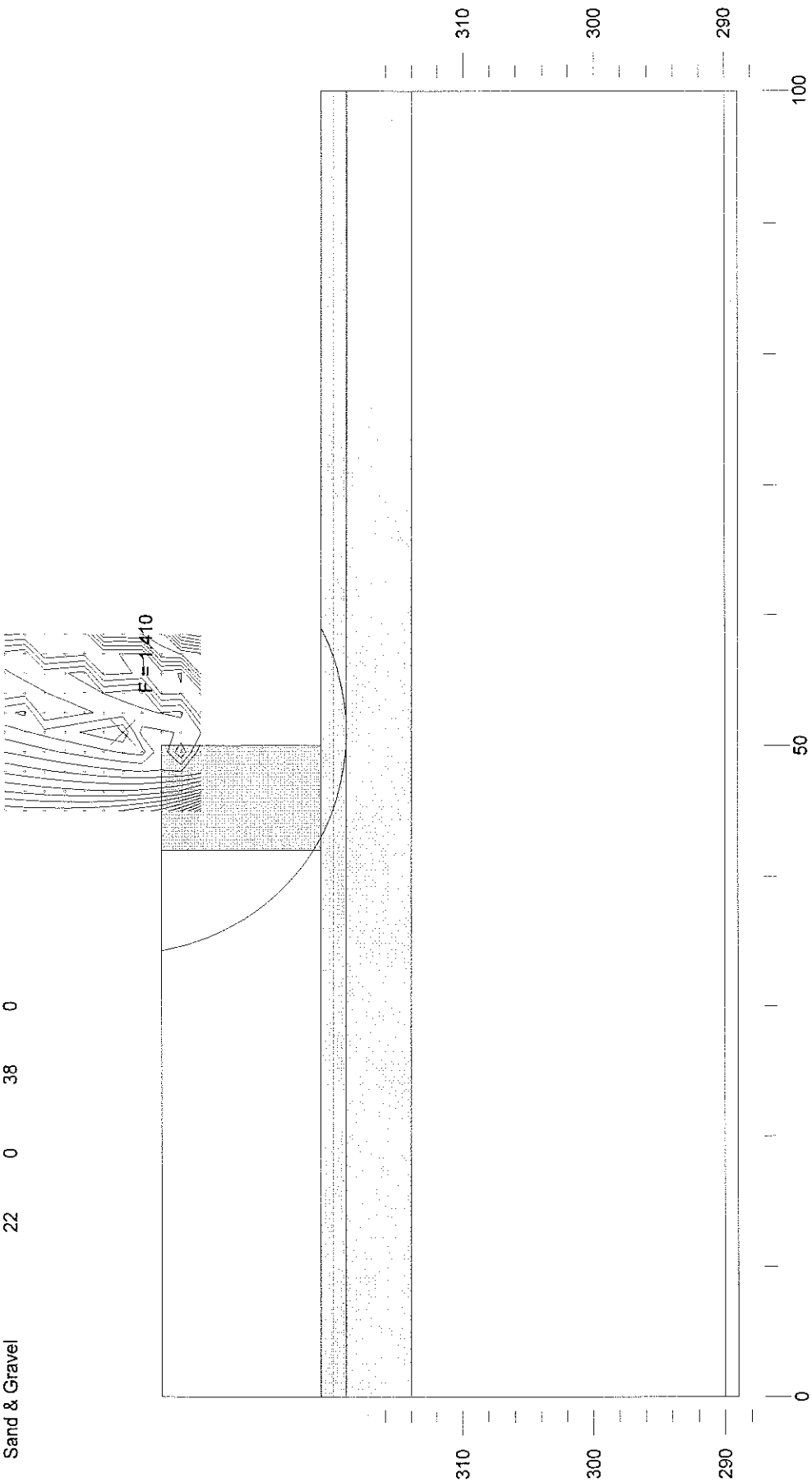
Thurber Engineering Ltd. - Toronto
 19-1423-16
 Hwy 11 - Katrine
 November 24, 2004
 Hwy 592 Sunset Pass
 west RSS wall

	Gamma C	Phi	Piezo
	kN/m ³	deg	Surf.
RSS	21	1000	0
Earth Fill	20	0	0
Silt	19	0	0
Clay	19	50	0
Sand	21	0	0
Sand & Gravel	22	0	0



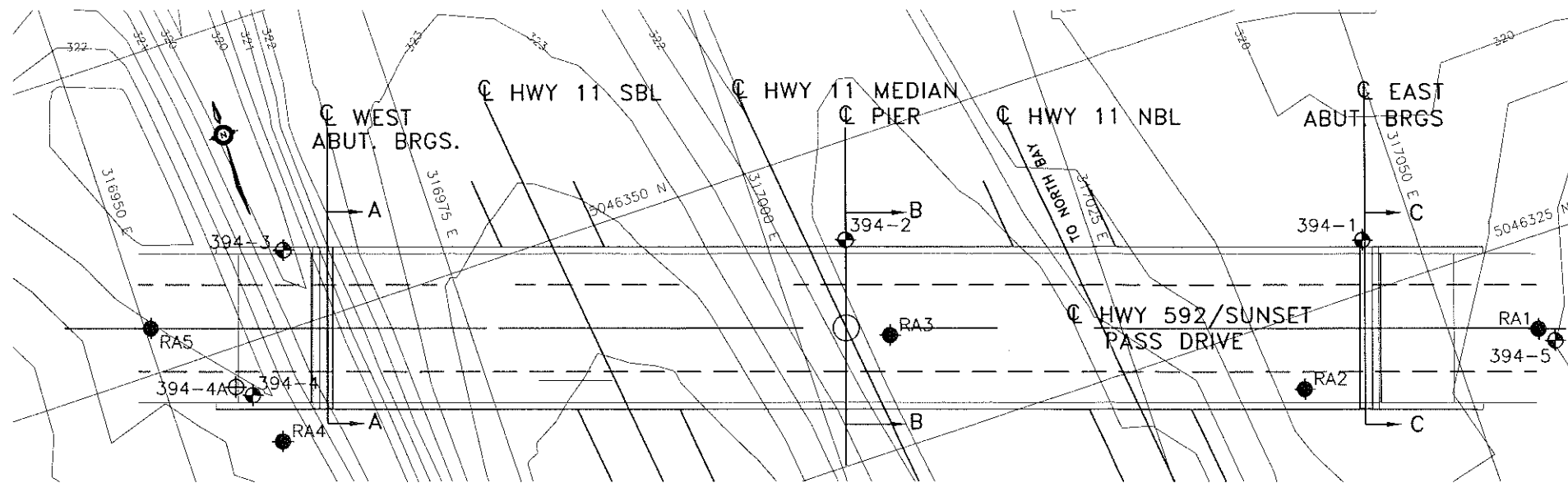
Thurber Engineering Ltd. - Toronto
 19-1423-16
 Hwy 11 - Katrine
 November 24, 2004
 Hwy 592 Sunset Pass
 east RSS wall

	Gamma C	Phi	Piezo
	kN/m ³	deg	Surf.
RSS	21	1000	0
Earth Fill	20	0	0
Silt	19	0	1
Compact Sand	20	0	0
Dense Sand	21	0	0
Sand & Gravel	22	0	0

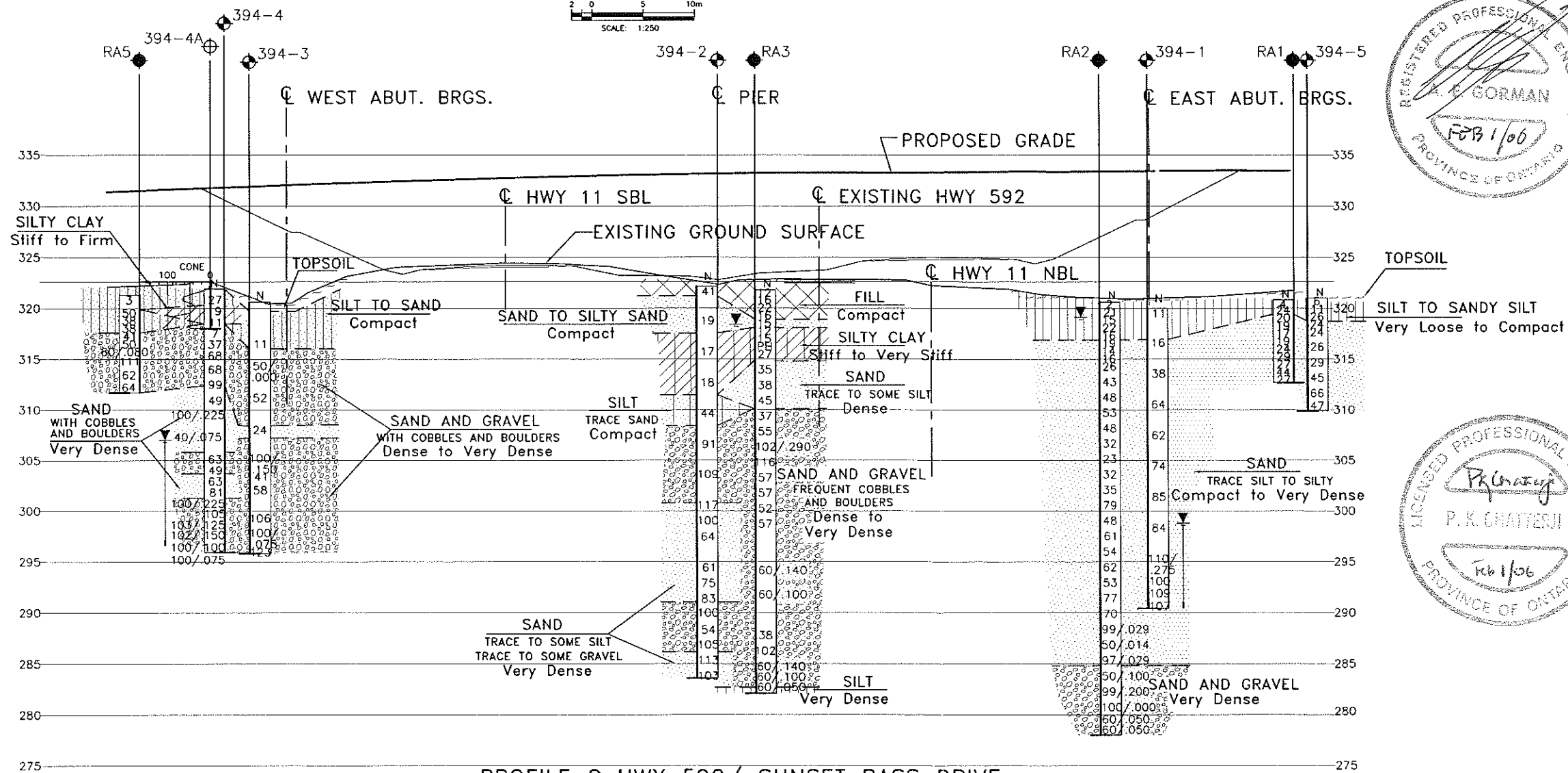


Appendix G

Drawings



PLAN
SCALE: 1:250

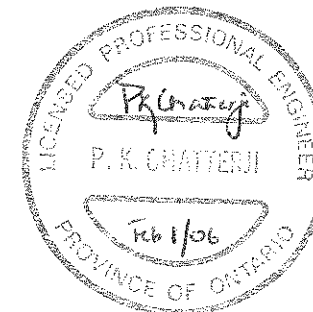
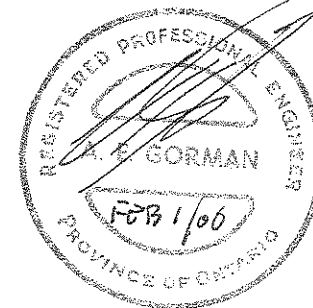
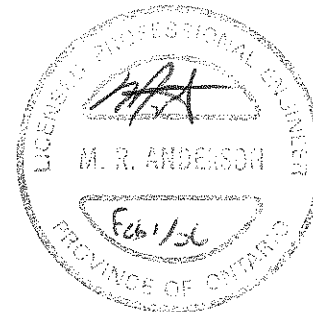


PROFILE @ HWY 592/ SUNSET PASS DRIVE
SCALE: 1:250

BENCH MARK 327.838
N & W IN N ROOT
OF 0.2 TWIN SPRUCE
6.4 LT 24+335.298

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN



HWY 11
CONT No 2006-5148
WP No 5404-04-01

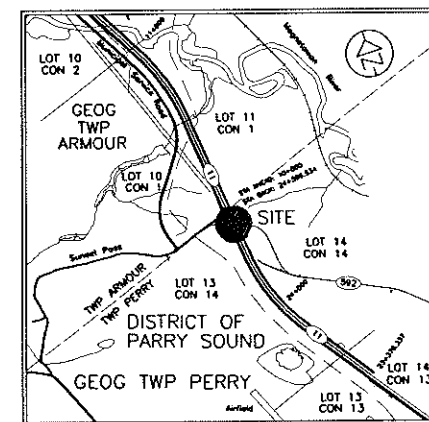
HWY 592/SUNSET PASS
DRIVE OVER HWY 11
BOREHOLE LOCATIONS AND SOIL STRATA

**Marshall
Macklin
Monaghan**
PROJECT MANAGERS • ENGINEERS • SURVEYORS • PLANNERS



SHEET
301

THURBER ENGINEERING LTD.
THURBER



KEY PLAN
0 500m 1km

LEGEND

- Bore Hole by THURBER
- ⊕ Dynamic Cone Penetration Test (DCPT)
- Bore Hole by SHAHEEN & PEAKER LIMITED
- N Blows/ 0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/ 0.3m (60' Cone, 475 J/blow)
- PH Pressure, Hydraulic
- WL at Time of Investigation
- Head Artesian Water
- Piezometer
- 90% Rock Quality Designation (RQD)
- A/R Auger Refusal

NO	ELEVATION	NORTHING	EASTING
394-1	320.0	5046329.1	317045.7
394-2	322.1	5046342.4	317006.2
394-3	320.5	5046356.0	316962.8
394-4A	322.6	5046346.9	316955.6
394-4	321.9	5046345.8	316956.7
394-5	321.0	5046316.5	317057.9
RA1	320.8	5046317.8	317056.9
RA2	320.6	5046319.2	317037.5
RA3	321.8	5046334.0	317007.1
RA4	321.4	5046341.5	316957.8
RA5	321.3	5046353.6	316950.5

NOTE

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

REVISIONS										
	DATE	BY	DESCRIPTION							
DESIGN	MA	CHK	AEG	CODE	CHBDC	2000	LOAD	CL-625-001	DATE	JAN. 2006
DRAWN	SS	CHK	MA	SITE	44-394	STRUCT	SCHEME		DWG	2

METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

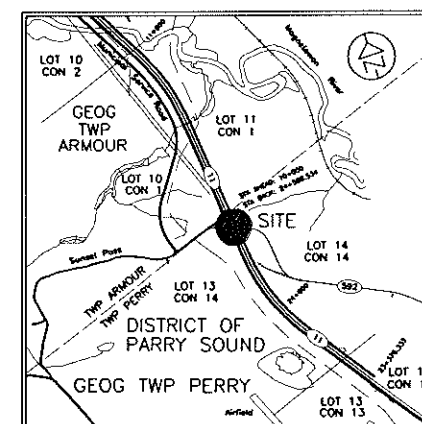
HWY 11
CONT No 2006-5148
WP No 5404-04-01

HWY 592/SUNSET PASS
DRIVE OVER HWY 11
SOIL STRATA

SHEET
302



THURBER ENGINEERING LTD.



KEY PLAN
0 500m 1km

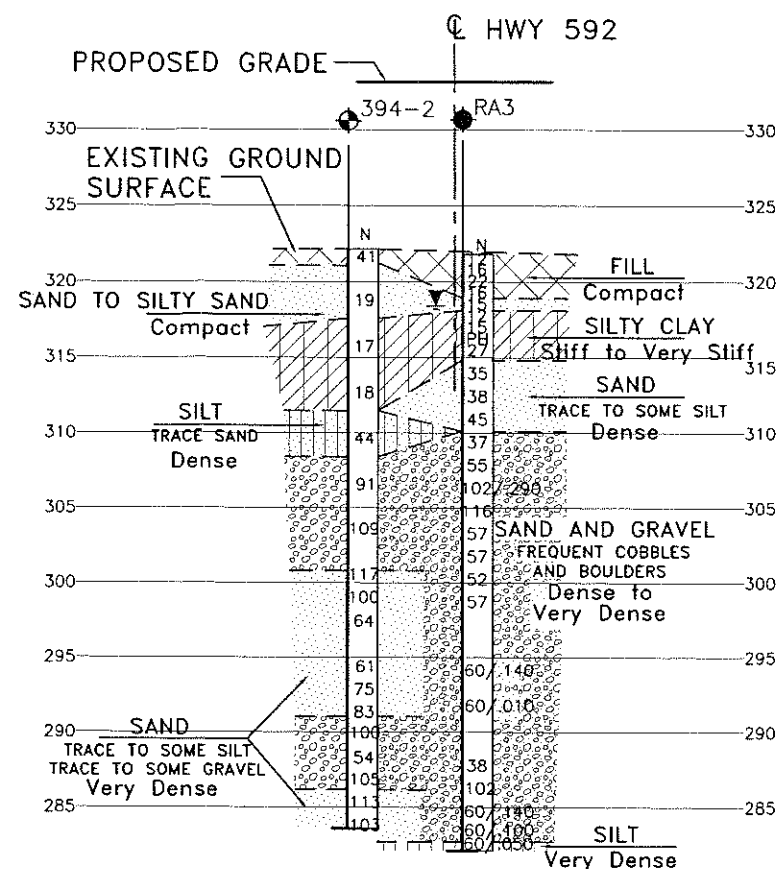
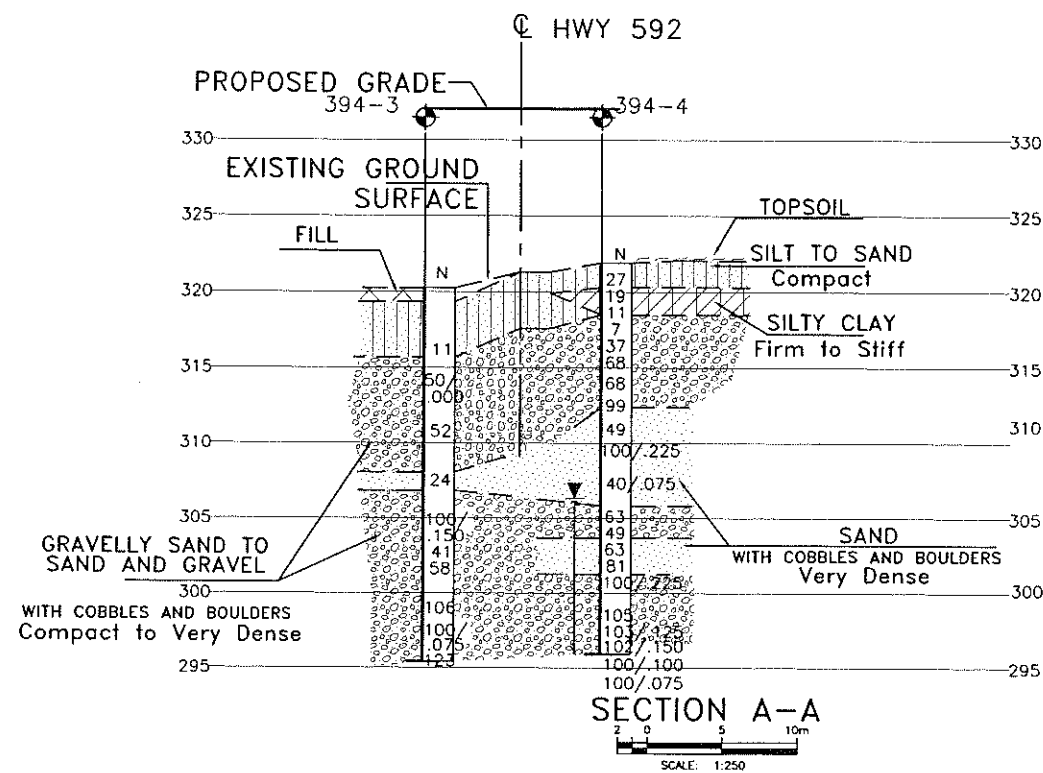
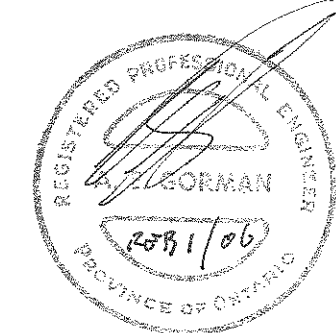
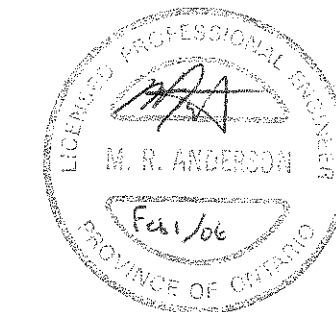
LEGEND

- Bore Hole by THURBER
- Dynamic Cone Penetration Test (DCPT)
- Bore Hole by SHAHEEN & PEAKER LIMITED
- N Blows/ 0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/ 0.3m (60' Cone, 475 J/blow)
- PH Pressure, Hydraulic
- WL at Time of Investigation
- Head Artesian Water
- Piezometer
- 90% Rock Quality Designation (RQD)
- A/R Auger Refusal

NO	ELEVATION	NORTHING	EASTING
394-1	320.0	5046329.1	317045.7
394-2	322.1	5046342.4	317006.2
394-3	320.5	5046356.0	316962.8
394-4A	322.6	5046346.9	316955.6
394-4	321.9	5046345.8	316956.7
394-5	321.0	5046316.5	317057.9
RA1	320.8	5046317.8	317056.9
RA2	320.6	5046319.2	317037.5
RA3	321.8	5046334.0	317007.1
RA4	321.4	5046341.5	316957.8
RA5	321.3	5046353.6	316950.5

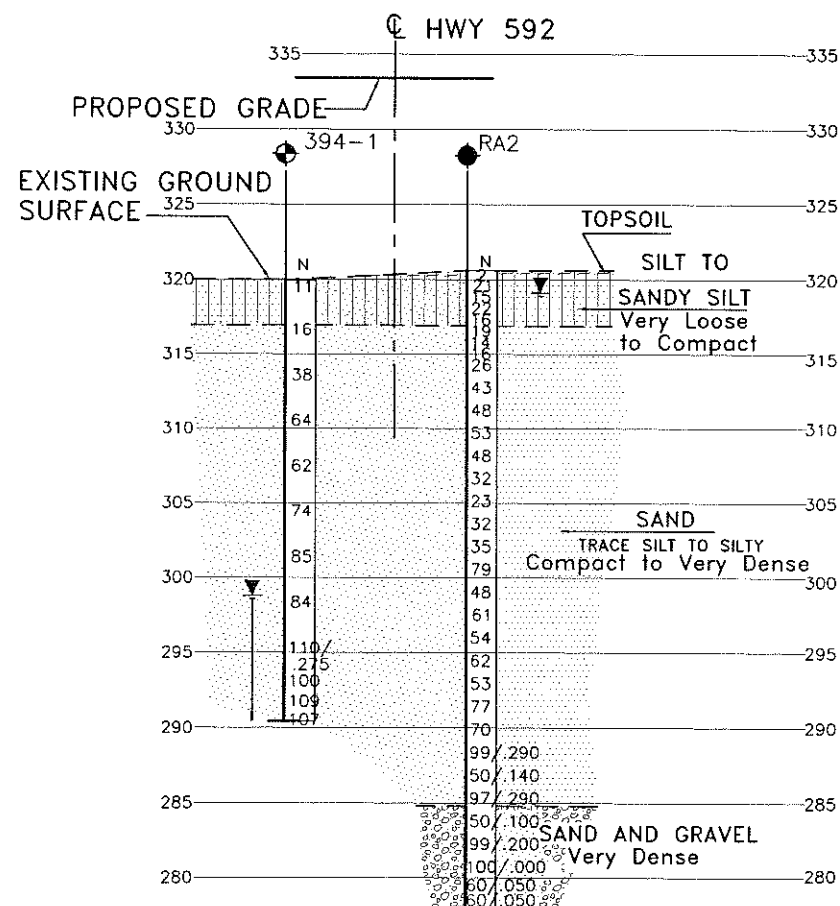
NOTE

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



SECTION B-B

SCALE: 1:250



SECTION C-C

SCALE: 1:250

BENCH MARK 327.638
N & W IN N ROOT
OF O.2 TWIN SPRUCE
6.4 LT 24+335.298

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS	DATE	BY	DESCRIPTION
DESIGN MA	CHK AEG	CODE CHBDC	2000/LOAD CL-625-0M/DATE JAN. 2006
DRAWN SS	CHK MA	SITE 44-394	STRUCT. SCHEME DWG 3