



# Terraprobe

*Consulting Geotechnical & Environmental Engineering  
Construction Materials Inspection & Testing*

**FOUNDATION INVESTIGATION & DESIGN REPORT  
PORT ROBINSON ROAD UNDERPASS  
HIGHWAY 406 TWINNING  
PORT ROBINSON ROAD TO EAST MAIN STREET  
AGREEMENT No. 2008-E-0016, W.P. 280-99-00, SITE: 34-462  
GEOCRES No. 30M3-262**

**PREPARED FOR:** Giffels Associates Ltd./IBI Group  
30 International Blvd.  
Toronto, Ontario

**Attention:** Mr. Stephen Chiu, P.Eng.  
Manager, Transportation Engineering

File No. 1-09-4135  
September 24, 2010

**Terraprobe Inc.**  
10 Bram Court  
Brampton, Ontario  
L6W 3R6  
Phone: (905) 796 2650  
Fax: (905) 796 2250

**Distribution:**

2 Copies	-	MTO Project Manager (Central Region)
1 Copy	-	MTO Pavements and Foundations Section
1 Copy	-	Giffels Associates Limited
1 Copy	-	Terraprobe Inc., Brampton

---

**Terraprobe Inc.**

**Greater Toronto**  
10 Bram Court  
Brampton, Ontario L6W 3R6  
(905) 796-2650 Fax 796-2250  
brampton@terraprobe.ca

**Hamilton - Niagara**  
903 Barton Street, Unit 22  
Stoney Creek, Ontario L8E 5P5  
(905) 643-7560 Fax 643-7559  
stoneycreek@terraprobe.ca

**Central Ontario**  
220 Bayview Drive, Unit 25  
Barrie, Ontario L4N 4Y8  
(705) 739-8355 Fax 739-8369  
barrie@terraprobe.ca

**Northern Ontario**  
1012 Kelly Lake Rd.  
Sudbury, Ontario P3E 5P4  
(705) 670-0460 Fax 670-0558  
sudbury@terraprobe.ca

[www.terraprobe.ca](http://www.terraprobe.ca)

## TABLE OF CONTENTS

### Part 1

1	INTRODUCTION.....	1
2	SITE DESCRIPTION & PHYSIOGRAPHY.....	1
3	SITE INVESTIGATION AND FIELD TESTING .....	2
4	LABORATORY TESTING .....	3
5	DESCRIPTION OF SUBSURFACE CONDITIONS.....	3
5.1	Topsoil .....	4
5.2	Fill – Gravelly Sand.....	4
5.3	Fill – Silty Sand .....	4
5.4	Fill – Silty Clay.....	4
5.5	Silty Clay .....	5
5.6	Silt.....	6
5.7	Silty Clay to Clayey Silt .....	6
5.8	Clayey Silt Till.....	7
5.9	Bedrock (Guelph Formation).....	8
5.10	Water Levels.....	8
5.11	Miscellaneous .....	9
6	GENERAL .....	10

### Part 2

7	STRUCTURE FOUNDATIONS .....	10
7.1	Spread Footings .....	11
7.2	Augered Caissons (Drilled Shafts) .....	12
7.3	Driven Piles .....	12
7.3.1	Axial Resistance .....	13
7.3.2	Downdrag .....	13
7.3.3	Integral Abutment Considerations.....	14
7.3.4	Lateral Resistance.....	14
7.3.5	Pile Tips.....	16
7.3.6	Pile Installation.....	16
7.3.7	Pile Driving .....	16
7.4	Recommended Foundation .....	17
7.5	Frost Cover .....	17
8	EXCAVATION AND BACKFILL.....	17
8.1	General.....	17



8.2	Foundations .....	17
9	GROUND WATER CONTROL .....	17
10	APPROACH EMBANKMENTS .....	18
10.1	Stability .....	18
10.2	Settlement .....	19
10.3	Embankment Construction .....	22
11	BACKFILL TO ABUTMENTS .....	22
12	LATERAL EARTH PRESSURE .....	23
13	SEISMIC CONSIDERATIONS .....	24
13.1	Seismic Design Parameters .....	24
13.2	Liquefaction Potential .....	25
13.3	Retaining Wall Dynamic Earth Pressures .....	25
14	CONSTRUCTION CONCERNS .....	26

### Table

Table 1	List of Standard Specifications in Report
---------	---

### Appendices

Appendix A	Record of Borehole Sheets, Core Logs and Core Photos
Appendix B	Laboratory Test Results
Appendix C	Drawings titled “Borehole Locations and Soil Strata”
Appendix D	Foundation Comparison
Appendix E	Settlement Parameters and Results
Appendix F	Suggested NSSP wording
Appendix G	Settlement Monitoring Programme
Appendix H	Figures



## DESIGN SUMMARY

This project (W.P. 280-99-00) is the Ministry of Transportation of Ontario undertaking to twin Highway 406 from 0.2 km north of Port Robinson Road to its current terminus at East Main Street.

Terraprobe carried out the investigation as a sub-consultant to Giffels Associates Limited/IBI Group (Giffels), under the Ministry of Transportation Ontario (MTO) Agreement Number 2008-E-0016.

The project is located in the Regional Municipality of Niagara, City of Thorold and City of Welland, Ontario. Approximately 6.8 km of two lane staged freeway will be twinned from Sta. 9+970 to Sta. 6+545. Within the project limits Highway 406 has signalized intersections at Merritt Road, Woodlawn Road and East Main Street and one unsignalized intersection at Port Robinson Road.

A multi span underpass is proposed to carry Port Robinson Road traffic over Highway 406 NBL and SBL.

The main design recommendations are:

- The bridge should be supported on pile foundations.
- Downdrag loads are high and the approach fills should be placed in advance of the pile driving operations. Downdrag loads on the piles can be eliminated by using EPS Geofoam in the approach fill.
- The soils at this site are settlement sensitive and the estimated settlement due to approach fill placement is expected to range between 115 mm and 200 mm depending on the material used.
- This data shows that after 12 months, the post-construction consolidation settlement will be 25 mm or less (required performance) for all embankment material types.
- If an accelerated construction schedule is required (target of 6 months), approach embankments should be surcharged to accelerate settlement and to ensure full consolidation within the target 6 months after embankment construction.

Notwithstanding the foregoing the designer is advised to review this report in its entirety to ensure that the geotechnical recommendations provided herein are adequately addressed in the designs and contract documents.



**FOUNDATION INVESTIGATION REPORT**  
**PORT ROBINSON ROAD UNDERPASS**  
**HIGHWAY 406 TWINNING**  
**ONTARIO**  
**AGREEMENT No. 2008-E-0016, W.P. 280-99-00, SITE: 34-462**  
**GEOCRES No. 30M3-262**  
**PART 1: FACTUAL INFORMATION**

## **1 INTRODUCTION**

This report presents the factual findings obtained from a foundation investigation conducted at the Port Robinson Road Underpass bridge site in the City of Thorold, Ontario.

The purpose of this investigation was to explore the subsurface conditions at the site and based on the data obtained, to provide a borehole location plan, records of boreholes, stratigraphic profile and cross-sections, laboratory test results and a written description of the subsurface conditions. A model of the subsurface conditions was developed from the data obtained.

Terraprobe conducted the investigation as a sub-consultant to Giffels Associates Ltd./IBI Group, under the Ministry of Transportation Ontario (MTO) Agreement Number 2008-E-0016.

The following documents are referenced in the preparation of this report:

- Peto MacCallum Ltd., “Preliminary Foundation Investigation and Design Report for Port Robinson Road Underpass”, Highway 406 Four-Laning, G.W.P. 280-99-00, City of Thorold, Ontario, GEOCRES 30M03-237, dated January 20, 2009.

## **2 SITE DESCRIPTION & PHYSIOGRAPHY**

The site is located approximately 25 m south of the existing at grade intersection of Highway 406 and Port Robinson Road in the City of Thorold, Regional Municipality of Niagara, Ontario. At this location Highway 406 is a two-lane highway with gravel shoulders carrying both north and south bound traffic.

The topography is flat and vegetation at this site consists primarily of deciduous trees and wild bush. Areas of groomed grass can be found at some locations along the existing roadways.

The site is located between the Niagara Escarpment and Lake Erie in the physiographic region of Southern Ontario referred to as the Haldimand Clay Plain. The Haldimand Clay Plain is best described as falling into a series of parallel belts with the highest ground adjacent to the Escarpment. Generally this region is flat and poorly drained although it includes several distinctive



landforms such as dunes, cobble, clay and sand beaches, limestone pavements and back-shore wetland basins<sup>1</sup>.

The Niagara Region is underlain by a sequence of very gently south-dipping dolostones, limestones, shales and sandstones overlying Precambrian basement rock. The key elements in the bedrock geology of the region are the multiple layers of softer sedimentary limestones, shale, sandstone and dolostone.

The bedrock unit at this site is the Guelph Formation of Upper Silurian Age<sup>2</sup>. This unit consists essentially of unweathered, grey, laminated argillaceous dolostone.

### **3 SITE INVESTIGATION AND FIELD TESTING**

The site investigation and field testing for this project were carried out between December 21, 2009 and February 3, 2010 and consisted of drilling and sampling five boreholes to depths ranging from 30.5 m to 38.0 m. The boreholes were numbered PR1, PR2, PR3, PR4, and PR5 and their approximate locations are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix C.

The borehole locations were marked in the field by surveyors from Callon Dietz Inc. who also provided Terraprobe with their coordinates and geodetic elevations. Access to Borehole PR3 was difficult due to locally steep slopes and this borehole was relocated to be as close as feasible to the staked out location while allowing safe operation of the drill rig. Terraprobe obtained utility clearances and permits prior to drilling.

Samples of the overburden soils were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT), as specified in ASTM Method D1586. In the cohesive (clayey) deposits the undrained shear strength of the soil was measured in-situ by means of field vane tests using an MTO type field vane. Relatively undisturbed soil samples were also collected with thin-walled Shelby Tube samplers. The boreholes at the abutments were also advanced into bedrock by NQ size diamond coring techniques.

Ground water conditions in the open boreholes were observed throughout the drilling operations and standpipe piezometers consisting of 19 mm diameter PVC pipe with a slotted screen enclosed in sand were installed in selected boreholes to permit longer term ground water level monitoring. The remaining boreholes were abandoned in accordance with MOE Regulation 903 by sealing/grouting with a bentonite slurry mixture after drilling was complete.

---

<sup>1</sup> Chapman and Putnam, "The Physiography of South Ontario", 3<sup>rd</sup> Edition, 1984.

<sup>2</sup> Ontario Division of Mines, "Quaternary Geology Of The Welland Area", Preliminary Map P.796, 1972.



The locations and completion details of the piezometers are shown in Table 3.1.

**Table 3.1 – Piezometer Installation Details**

Piezometer Location	Piezometer Details	
	Tip Depth/ Elevation (m)	Completion Details
PR1	32.0/149.7	Piezometer with 1.5 m slotted screen installed with filter sand to 29.9 m, bentonite seal from 29.9 m to 29.0 m, drill cuttings from 29.0 m to 1.5 m, bentonite seal from 1.5 m to ground surface.
PR3	32.0/149.3	Hole sealed to 32.0 m with bentonite, piezometer with 1.5 m slotted screen installed with filter sand to 29.9 m, bentonite seal from 29.9 m to ground surface.
PR4	14.6/167.6	Hole sealed to 14.8 m with bentonite, piezometer with 1.5 m slotted screen installed with filter sand to 12.8 m, bentonite seal from 12.8 m to ground surface.
PR5	30.5/150.7	Piezometer with 1.5 m slotted screen installed with filter sand to 28.3 m, bentonite seal from 28.3 m to 27.1 m, drill cuttings from 27.1 m to 0.6 m, bentonite seal from 0.6 m to ground surface.

The drilling, sampling and coring operations were observed on a full time basis by members of Terraprobe's technical staff who logged the boreholes and rock cores and processed the recovered soil and rock samples for transport to Terraprobe's Brampton laboratory for further examination and testing.

#### **4 LABORATORY TESTING**

The recovered soil samples were subjected to Visual Identification (VI) and natural moisture content determination. Select samples were also subjected to a laboratory testing programme consisting of gradation analysis, Atterberg Limits tests, consolidation tests, unit weight and undrained shear strength testing with a laboratory vane. The results of this testing program are shown on the Record of Borehole sheets in Appendix A and the figures in Appendix B.

#### **5 DESCRIPTION OF SUBSURFACE CONDITIONS**

Reference is made to the Record of Borehole sheets in Appendix A. Details of the encountered soil and rock stratigraphy are presented in this appendix and on the "Borehole Locations and Soil Strata" drawings in Appendix C. An overall description of the stratigraphy is given in the following paragraphs. However, the factual data presented in the Record of Borehole Sheets governs any interpretation of the site conditions.

In general, the site is underlain by topsoil, gravelly sand fill, silty sand fill, silty clay fill and native overburden deposits of silty clay, silt, silty clay to clayey silt, and clayey silt till. These soils are underlain by bedrock consisting primarily of dolostone of the Guelph formation.



## **5.1 Topsoil**

Topsoil ranging from 180 mm to 300 mm thick was encountered on the site. Topsoil thickness may vary between and beyond the boreholes.

## **5.2 Fill – Gravelly Sand**

Borehole PR4 was drilled on the median between the existing Highway 406 NBL and the dedicated right turn lane that conveys Highway 406 northbound traffic to Port Robinson Road east. This borehole encountered a layer of gravelly sand fill (approximately 500 mm thick) that extends to a depth of 0.7 m (Elev. 181.5 m) below ground surface.

A sample of this fill material was subjected to grain size analysis and the results are presented in Figure B1. These results show a grain size distribution consisting of 22% gravel, 50% sand, 20% silt and 8% clay size particles.

A Standard Penetration test in the gravelly sand fill gave an 'N' value of 12 blows for 0.3 m penetration. Based on this result the fill is considered to have a compact relative density. The moisture content of a sample of this fill was 8% by weight.

## **5.3 Fill – Silty Sand**

Borehole PR3 was drilled near to the west shoulder of the existing Highway 406. This borehole encountered a discontinuous layer of silty sand fill (approximately 1.1 m thick) that extends to a depth of 1.4 m (Elev. 179.9 m) below ground surface. Based on visual and tactile examinations of the retrieved samples, the fill is essentially a cohesionless soil with frequent cohesive silty clay inclusions.

A sample of this fill material was subjected to a grain size analysis and the results are presented in Figure B2. These results show a grain size distribution consisting of 0% gravel, 48% sand, 34% silt and 18% clay size particles.

Standard Penetration tests in the fill gave 'N' values that ranged from 4 to 8 blows for 0.3 m penetration. Based on these results the fill is considered to have a loose relative density. The moisture content of samples of this fill ranged from 15% to 20% by weight.

## **5.4 Fill – Silty Clay**

Silty clay fill material was encountered at this site extending to depths ranging from 0.7 m (Elev. 180.5 m) to 2.1 m (Elev. 180.1 m) below ground surface.

Samples of this fill were subjected to grain size analysis and the results are presented in Figure B3. These results show a grain size distribution consisting of 0% gravel, 8-12% sand, 35-44% silt and 44-57% clay size particles.





The fill material was also subjected to an Atterberg Limits test and the results are presented in Figure B4. The index values from these tests are summarized below:

Liquid Limit:	40%
Plastic Limit:	20%
Plasticity Index:	20%
Natural Moisture Content:	19%

These values are characteristic of clayey soils of intermediate plasticity.

Standard Penetration tests in the silty clay fill gave 'N' values that ranged from 4 to 22 blows for 0.3 m penetration but generally the recorded 'N' values ranged from 4 to 11 blows for 0.3 m penetration. Based on these results the fill is considered to have a generally firm to stiff consistency with occasional very stiff zones. The moisture content of samples of this fill ranged from 17% to 26% by weight.

### **5.5 Silty Clay**

A major silty clay deposit exists at this site. This deposit was fully penetrated in all of the boreholes where it was found to extend to depths ranging from 14.7 m (Elev. 167.5 m) to 15.7 m (Elev. 166.0 m) below ground surface.

The grain size distribution plots of tested samples of the silty clay are presented in Figures B5 to B9 inclusive. These results show a grain size distribution consisting of 0-7% gravel, 0-4% sand, 16-77% silt and 23-83% clay size particles.

Samples of the silty clay were also subjected to Atterberg Limits tests and the results are illustrated on the plasticity charts, Figures B10 to B14 inclusive. The index values from these tests are summarized below:

Liquid Limit:	25-61%
Plastic Limit:	16-27%
Plasticity Index:	8-34%
Natural Moisture Content:	19-47%

These values indicate that the silty clay has a generally low to intermediate plasticity with occasional zones of high plasticity.

Standard Penetration tests in this stratum gave 'N' values that ranged from 0 to 27 blows for 0.3 m penetration but generally the recorded 'N' values ranged from 3 to 14 blows for 0.3 m penetration. Field vane tests gave in-situ undrained shear strengths ranging from 24 kPa to in excess of 100 kPa and laboratory vane tests on relatively undisturbed Shelby tube samples gave undrained shear strengths ranging from 38 kPa to 78 kPa. These values indicate that the consistency of the silty clay is generally firm to very stiff with infrequent soft zones. Moisture content of samples of the silty clay range from 19% to 47% by weight and the unit weight of selected samples ranged from 17.4 to 20.3 kN/m<sup>3</sup>.



The variation of undrained shear strength with elevation is depicted in Figure B22 (Elev. 178 m to Elev. 166 m). The plot generally illustrates a trend of decreasing shear strength with depth within this deposit. The upper portion of this deposit up to about Elev. 177.5 m is estimated to have relatively high undrained shear strength i.e. in excess of 100 kPa. Below Elev. 177.5 m the undrained shear strength decreases with depth and is about 25 kPa between Elev. 171.0 m and Elev. 170.0 m. Below Elev. 170.0 m the trend indicates increasing undrained shear strength with depth.

The Atterberg Limits tests results are also plotted against elevation, Figure B23 (Elev. 180 m to Elev. 167 m). These results illustrate that the natural moisture contents of this deposit are generally at or below the plastic limit up to about Elev. 177.5 m. Below Elev. 177.5 m the natural moisture content increases and is between the plastic and liquid limits.

Consolidation tests were also performed on Shelby tube samples retrieved from Boreholes PR1 and PR5 and the results are presented in Figures B24 to B29 inclusive. These results indicate estimated preconsolidation pressures ranging between 200 kPa and 360 kPa.

## **5.6 Silt**

A native silt deposit was encountered at this site in all of the boreholes. The deposit is approximately 2.1 m to 3.1 m thick and extends to depths ranging from 16.8 m (Elev. 164.5 m) to 17.8 m (Elev. 163.9 m) below ground surface. Based on visual and tactile examinations of the retrieved samples, the unit is essentially a cohesionless silt with frequent cohesive silty clay seams and partings.

The grain size distribution plots of tested samples of this silt deposit are presented in Figure B15. These results show a grain size distribution consisting of 0% gravel, 0-1% sand, 75-96% silt and 3-24% clay size particles.

The deposit is considered to have a very loose to compact relative density based on SPT 'N' values that ranged from 0 to 26 blows for 0.3 m penetration. SPT 'N' values of 0 are likely attributed to sample disturbance. The moisture content of samples from this deposit ranged from 16% to 29% by weight.

## **5.7 Silty Clay to Clayey Silt**

A native deposit of silty clay to clayey silt was encountered across this site. This stratum extends to depths ranging from 26.9 m (Elev. 154.8 m) to 29.9 m (Elev. 152.3 m) below ground surface.

The grain size distribution plots of tested samples from this stratum are depicted in Figure B16 and B17. These results show a grain size distribution consisting of 0-3% gravel, 0-11% sand, 63-81% silt and 16-37% clay size particles.



Samples were also subjected to Atterberg Limits tests and the results are plotted on the plasticity charts, Figure B18 and B19. The index values from these tests are summarized below:

Liquid Limit:	23-39%
Plastic Limit:	16-20%
Plasticity Index:	4-19%
Natural Moisture Content:	16-31%

These values indicate that the silty clay to clayey silt is of low to intermediate plasticity.

Standard Penetration tests in this deposit yielded 'N' values ranging from 6 to 43 blows for 0.3 m penetration. Field vane tests were also performed in this deposit and the results indicate undrained shear strengths ranging from 80 kPa to in excess of 100 kPa. Based on these results the silty clay to clayey silt is considered to have a stiff to hard consistency with occasional firm zones. The moisture content of samples from these deposits varies from 9% to 34% by weight.

The variation of undrained shear strength with elevation is depicted in Figure B22 (Elev. 164 m to Elev. 153 m). The plot illustrates a slight decrease in shear strength with depth. The undrained shear strength decreases from about 100 kPa at Elev. 164.0 m to about 75 kPa at Elev. 157.0 m. Below Elev. 157.0 m the trend indicates increasing undrained shear strength with depth.

The Atterberg Limits tests results are also plotted against elevation, Figure B23 (Elev. 164 m to Elev. 153 m). These results illustrate that the natural moisture content of the upper portion of this deposit is generally at or below the plastic limit up to about Elev. 158.0 m. Below Elev. 158.0 m the natural moisture content increases and is generally between the plastic and liquid limits up to about Elev. 154.0 m. Below Elev. 154.0 m the natural moisture content is below the plastic limit.

## **5.8 Clayey Silt Till**

A native deposit of clayey silt till was encountered across the site extending to depths ranging from 30.5 m to 34.3 m below ground surface or to elevations ranging from 150.7 m to 147.7 m. Boreholes PR1 and PR5 were terminated in this deposit at depths of 32.0 m (Elev. 149.7 m) and 30.5 m (Elev. 150.7 m) respectively. The lower 1.5 m to 1.8 m of this stratum overlying bedrock contains frequent cobbles and in Borehole PR3 a boulder was encountered above the bedrock.

The grain size distribution plot of a tested sample from this till deposit is depicted in Figure B20. These result shows a grain size distribution consisting of 3% gravel, 18% sand, 64% silt and 15% clay size particles.

A sample was also subjected to an Atterberg Limits test and the results are plotted on the plasticity chart, Figure B21. The index values from these tests are summarized below:

Liquid Limit:	20%
Plastic Limit:	14%
Plasticity Index:	6%
Natural Moisture Content:	11%

This value is typical of a low plasticity clayey silt soil.



Standard Penetration tests in this stratum deposits yielded 'N' values ranging from 16 to more than 100 blows per 0.3 m penetration but generally the recorded 'N' values ranged from 30 to more than 100 blows for 0.3 m penetration. Based on these results the clayey silt till is considered to have a hard consistency with occasional very stiff zones. The moisture content of samples from this deposit varies from 3% to 23% by weight.

### 5.9 Bedrock (Guelph Formation)

The overburden soils described above are underlain by the Guelph Formation. Bedrock was proved by coring at the abutment and pier locations. Table 5.1 summarizes the bedrock depth and the elevations to the top of bedrock.

**Table 5.1 – Depth to Bedrock**

Location	BH Number	Depth to Bedrock (m)	Top of Bedrock Elevation (m)
West Abutment	PR2	33.5	148.2
Pier	PR3	33.6	147.7
East Abutment	PR4	34.3	147.9

The bedrock is described as unweathered dolostone and its colour is light to medium brownish grey. Total core recovery in the bedrock generally ranged from 52% to 100%. The RQD values ranged widely from 0% to 76%, but generally most of the RQD values were between 24% and 69%. The core data also reveals that there is no trend of improving rock quality with depth. Based on these results the rock quality is considered to be very poor to fair with infrequent zones of good quality rock.

### 5.10 Water Levels

A standpipe piezometer was installed in selected boreholes. The water level readings measured on separate visits made after the completion of drilling are presented in Table 5.2.

**Table 5.2 – Water Level Measurements**

Borehole	Date	Water Levels	
		Depth (m)	Elevation (m)
PR1	January 11, 2010	7.0	174.7
	January 19, 2010	7.2	174.5
	January 27, 2010	7.1	174.6
	February 08, 2010	7.2	174.5
	February 19, 2010	7.1	174.6
PR3	January 19, 2010	8.2	173.1
	January 27, 2010	6.6	174.7
	February 08, 2010	0.6	180.7
	February 19, 2010	0.5	180.8
PR4	February 08, 2010	5.0	177.2
	February 19, 2010	4.5	177.7
	April 16, 2010	4.3	177.9
PR5	January 19, 2010	6.4	174.8
	January 27, 2010	6.2	175.0
	February 08, 2010	6.3	174.9
	February 19, 2010	6.2	175.0



The ground water table was estimated based on the recorded water levels in the standpipe piezometers and our review of the moisture contents of the retrieved samples. Based on these observations, the local ground water level is estimated to be about Elev.  $\pm 179.5$  m. At Borehole PR3, perched water exists in the silty sand fill at Elev.  $\pm 180.8$  m.

All ground water observations at this site are short term and the levels are expected to fluctuate seasonally and after severe weather events.

#### 5.11 Miscellaneous

The drilling, sampling and in-situ testing operations were conducted with track mounted drill rigs owned and operated by Determination Drilling & Soil Investigations of Hamilton, Ontario.

The boreholes were advanced using hollow-stem augers and rock cores were retrieved by NQ size diamond coring techniques.

Messrs. Alexander Winkelmann, E.I.T., and Phil Khuu, B.A.T carried out the field work and the laboratory testing was performed at Terraprobe's Brampton laboratory and the Mississauga laboratory of Golder Associates. The report was written by Rehman Abdul, P.Eng. and reviewed by Michael Tanos, P.Eng.

*Rehman Abdul*

Prepared by:  
R. Abdul, P.Eng.,  
Senior Geotechnical Engineer



*Michael Tanos*

Report Reviewed by:  
Michael Tanos, P.Eng.,  
Review Principal



**FOUNDATION DESIGN REPORT  
PORT ROBINSON ROAD UNDERPASS  
HIGHWAY 406 TWINNING**

**ONTARIO**

**AGREEMENT No. 2008-E-0016, W.P. 280-99-00, SITE: 34-462**

**GEOCRES No. 30M3-262**

**PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS**

**6 GENERAL**

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 approaches for the proposed structure.

A two span underpass is required to carry Port Robinson Road over the proposed north bound and south bound lanes of Highway 406. The structure will span 36.0 m between abutments and pier and will be approximately 11.7 m wide. The proposed finished grades at the structure will be about Elev. 189.4± m at the north and south abutments and Elev. 189.6± m at the pier. At the abutments the approach fill is estimated to range in height between ±7.5 m and ±8.5 m.

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 investigations.

**7 STRUCTURE FOUNDATIONS**

The proposed bridge is a multi span structure with two abutments and a pier as foundation elements.

The stratigraphy encountered at the abutment locations consist of 33.5 m to 34.3 m of overburden soils overlying bedrock of the Guelph Formation. The overburden includes compressible soils consisting of about 0.5 m to 1.4 m of silty clay fill material (extending to Elev. 180.5 m to Elev. 180.1 m), 12.6 m to 14.3 m of native silty clay (extending to Elev. 167.5 m to Elev. 166.0 m) and 9.1 m to 12.2 m of silty clay to clayey silt (extending to Elev. 154.8 m to Elev. 152.3 m). The ground water level at this site is about Elev. ±179.5 m.

Consideration was given to the following foundation types:

- Spread footings
- Augered Caissons (drilled shafts)
- Driven piles

A comparison of the foundation alternatives based on advantages and disadvantages of each is included in Appendix D.



## 7.1 Spread Footings

Based on the proposed geometry conventional abutments supported on spread footings bearing on the native silty clay are not considered to be feasible and practical. The bearing capacity of the silty clay is low requiring relatively large footings and the abutment stems will be about  $\pm 9$  m high. The loads imposed by the abutment fills will also trigger time dependent consolidation settlement of the silty clay and the magnitude of these settlements will exceed 25 mm. Consequently, spread footings on native soils are not considered to be a feasible alternative.

At the pier location (Borehole PR3) the bearing capacity of the underlying silty clay is relatively low and the footing will be relatively large. The silty clay is also settlement sensitive. Therefore, a spread footing bearing on the native silty clay is not considered to be a feasible option for supporting the bridge pier.

If it is beneficial to the overall design, spread footings may be founded on an engineered fill pad provided the foundation footprint is preloaded to eliminate undesirable settlements. If an engineered fill pad is used at this site, all topsoil and fill should be stripped from below the footprint of the footing and the native soil should be stripped at least to the following elevations, and deeper if required to achieve the minimum thickness of engineered fill.

**Table 7.1 – Recommended Base Elevations of Engineered Fill Pad**

Borehole No.	Foundation Element	Elevation
PR2	West Abutment	180.3
PR3	Pier	179.9
PR4	East Abutment	180.1

It is recommended that the thickness of the fill pad is equal to or greater than the footing width, and should not be less than 2 m. The engineered fill should be placed directly on prepared native silty clay and should consist of OPSS Granular “A” compacted to 100% of its Standard Proctor maximum dry density (SPMDD) at  $\pm 2\%$  of optimum moisture content (OPSS 501, Section 501.08.02, Method A) and generally conforming to the geometry illustrated in Figure H1, Appendix H.

Provided a minimum footing width of 2 m is maintained, a footing bearing on a compacted Granular ‘A’ pad may be designed for the following concentric, vertical geotechnical resistances:

Factored Geotechnical Resistance at ULS – 900 kPa

Factored Geotechnical Resistance at SLS – 350 kPa

The geotechnical resistances quoted above are for concentric, vertical loads only. In the case of eccentric or inclined loading, the geotechnical resistance must be calculated as illustrated in 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 expected to not exceed 25 mm provided the soils are preloaded to eliminate undesirable settlement.



At the pier footing the soils underlying the Granular “A” pad can be preloaded for 1 year with 3 m high local earth fill placed on the engineered fill pad or the preload period can be reduced to 6 months by surcharging with 3 m high Granular “A” placed on the engineered fill pad. At the abutments the soils underlying the Granular “A” pad can be preloaded for 6 months by building the approach embankments and surcharging with 2 m of embankment fill.

The sliding resistance of mass concrete poured on a compacted Granular “A” pad may be computed on the basis of an ultimate coefficient of friction of 0.7.

## **7.2 Augered Caissons (Drilled Shafts)**

Augered caisson foundations were also considered for supporting the structure. However, the overburden is not considered to be suitable for this scheme and the caissons must be founded on the bedrock at depths in the order of  $\pm 33.5$  to  $\pm 34.3$  m below original ground surface or to elevations ranging from 147.7 m to 148.2 m. Fractured zones exist within the bedrock and there is no evidence of increasing rock quality within the investigated depths. Rock sockets made in this fractured bedrock zone may not provide reliable performance. The matrix of the clayey silt till overlying bedrock also contains cobbles and boulders and boulder removals from the base of the excavation could be difficult.

It would also be difficult to seal the bottom of the liner to exclude ground water due to the presence of cobbles and boulders above the bedrock and the fractured zones within the bedrock. Unwatering the caisson and maintaining a sufficiently dry excavation to permit cleaning, inspection and high quality construction on the bedrock would also be difficult.

For these reasons, caisson foundations are not recommended.

## **7.3 Driven Piles**

The subsurface conditions at the site are considered suitable for the design of foundations supported on steel H-piles. Steel tube piles were considered but were excluded. These high displacement piles will temporarily alter the pore water pressure of the silty clay deposits during driving. A substantial increase in penetration resistance will occur and it may be impossible to drive piles to the required penetration and capacity especially as more piles are driven in the group. Heave will also occur and damage is likely due to overdriving and overstressing. H-pile sections are low displacement sections that have a higher probability of being installed successfully on bedrock.

Steel H-piles are likely to be driven to practical refusal on bedrock at the foundation elements. However, the lower 1.7 m to 1.9 m of the clayey silt till overlying bedrock contains cobbles and in Borehole PR3 a boulder was also encountered. Therefore, at some locations piles may encounter effective refusal in the clayey silt till without reaching bedrock.





### 7.3.1 Axial Resistance

Two steel pile sections believed to be currently available have been considered for use in the proposed foundations. Since the piles will be bearing on bedrock it is recommended that the foundation elements be designed based on the concentric, axial geotechnical resistances given in Table 7.2. These reduced ULS resistances are recommended because of the very poor to poor rock quality. The bedrock is considered to be “unyielding” and the SLS condition will not govern for piles founded on bedrock. The actual pile tip elevations will be controlled as described in Section 7.3.7. The structural resistance of the pile should be checked by the structural designer.

**Table 7.2 – Axial Resistance of Various Pile Sections**

Location	PILE TYPE - HP 310x110			
	Reference Borehole	Estimated Pile Tip Elevation (m)	Founding Stratum	Factored Axial Resistance U.L.S (kN)
West Abutment	PR2	148.2±	Bedrock	1600
Pier	PR3	147.7±	Bedrock	
East Abutment	PR4	147.9±	Bedrock	

Location	PILE TYPE – HP 360X132			
	Reference Borehole	Estimated Pile Tip Elevation (m)	Founding Stratum	Factored Axial Resistance U.L.S (kN)
West Abutment	PR2	148.2±	Bedrock	1800
Pier	PR3	147.7±	Bedrock	
East Abutment	PR4	147.9±	Bedrock	

The H-piles for the recommended foundation scheme will be driven to effective refusal either in the overburden soil or on bedrock. Piles will penetrate through till layers that contain cobbles and boulders. Given these aggressive driving conditions it is recommended that the pile tips be fitted with rock points to provide increased cutting ability, reinforcement to the pile section and effective contact with bedrock.

The contract documents should contain a NSSP alerting the contractor to the fact that cobbles and boulders may be encountered in the overburden soils. Furthermore, it may not be possible to drive some piles to bedrock because of the frequent cobbles and boulder in the lower portion of the clayey silt till overlying bedrock. The NSSP should require the QVE to terminate driving before the pile is damaged by overdriving. Suggested wording for the NSSP is included in Appendix F.

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

### 7.3.2 Downdrag

The construction of approximately ±7.5 m to ±8.5 m high embankments at the bridge abutments will cause settlement of the underlying soils which will in turn impart downdrag loads on piles that are installed before fill placement.



The downdrag forces imparted on the two pile sections were estimated based on settlement of the upper 12.6 m to 14.3 m of silty clay and the lower 9.1m to 12.3 m of silty clay to clayey silt. The unfactored loads recommended for design are:

- HP 310 x 110 – 1150 kN/pile
- HP 360 x 132 – 1350 kN/pile

We recommend that the approach fills be constructed and preloaded with a 2 m high surcharge of embankment fill approximately 6 months in advance of the pile driving operations. The approach fill will have to be monitored to ensure that settlement is essentially complete prior to pile driving operations. By adopting this construction approach the downdrag forces will be significantly reduced and the potential for lateral squeeze (due to unbalanced fill) will be minimized.

### 7.3.3 Integral Abutment Considerations

The ground conditions at this site are considered suitable for an integral abutment design. The integral abutment design requires that the piles possess flexibility in the upper 3 m of the pile length.

At the abutment locations the upper 3 m of pile will lie wholly in the embankment fill. Therefore, to provide the required flexibility in the piles, the upper 3 m of the piles should be surrounded by a 600 mm diameter CSP as specified by the integral abutment design procedures.

The space between the pile and the CSP should be filled with sand. A NSSP should be included in the contract drawings specifying the gradation of the sand according to Table 7.3.

**Table 7.3 – 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%

### 7.3.4 Lateral Resistance

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:

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

$$k_s = 67 S_u / D \text{ [cohesive soils] (kN/m}^3\text{)}$$

$$p_{ult} = 3 \cdot \gamma \cdot z \cdot K_p \text{ [cohesionless soils] (kPa)}$$

$$p_{ult} = 9 S_u \text{ [cohesive soils] (kPa)}$$



where  $z$  = depth of embedment of pile (m)  
 $D$  = pile width (m)  
 $S_u$  = undrained shear strength (Table 7.4) (kPa)  
 $n_h$  = coefficient of horizontal subgrade reaction (Table 7.4) ( $\text{kN/m}^3$ )  
 $\gamma$  = unit weight (Table 7.4) ( $\text{kN/m}^3$ )  
 $K_p$  = passive earth pressure coefficient

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 must not exceed the ultimate lateral resistance or the factored structural flexural resistance of the pile. For design purposes a maximum horizontal passive resistance of 120 kN (ULS) is recommended.

The spring constant,  $K$ , for analysis may be obtained by the expression,  $K = k_s \times L \times D$  ( $\text{kN/m}$ ), where  $k_s$  is the coefficient of horizontal subgrade reaction ( $\text{kN/m}^3$ ),  $D$  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 7.4 – Recommended Soil Parameters**

Area Reference Borehole No	Applicable Elevation	Soil Type	Bulk Unit Weight ( $\text{kN/m}^3$ )	Angle of Internal Friction ( $\phi$ ) Degrees	Undrained Shear Strength ( $S_u$ ) (kPa)	Recommended $n_h$ Value ( $\text{kN/m}^3$ )*
West Abutment PR2	181.4 – 180.3	Fill – Silty Clay	18	0	50	–
	180.3 – 177.0	Silty Clay	20	0	100	–
	177.0 – 172.0	Silty Clay	20	0	80	–
	172.0 – 168.0	Silty Clay	20	0	30	–
	168.0 – 166.0	Silty Clay	20	0	60	–
	166.0 – 163.9	Silt	18	25	–	1000
	163.9 – 154.8	Silty Clay/Clayey Silt	20	0	100	–
	154.8 – 148.2	Clayey Silt Till	21	0	200	–
East Abutment PR4	182.0 – 181.5	Fill - Sand	19	25	–	8000
	181.5 – 180.1	Fill – Silty Clay	18	0	50	–
	180.1 – 178.0	Silty Clay	20	0	100	–
	178.0 – 176.5	Silty Clay	20	0	30	–
	176.5 – 171.5	Silty Clay	20	0	80	–
	171.5 – 167.5	Silty Clay	20	0	40	–
	167.5 – 164.5	Silt	18	25	–	1000
	164.5 – 152.3	Silty Clay/Clayey Silt	20	0	100	–
	152.3 – 147.9	Clayey Silt Till	21	0	200	–

\* Values estimated based on Table 20.3 data, Canadian Foundation Engineering Manual, 3<sup>rd</sup> edition, 1992

Since the piles are end bearing, the vertical resistance will not be significantly affected by the pile spacing. Pile interaction should be considered with reference to CHBDC Clause 6.8.9.2.

For lateral soil/pile group interaction analysis, the equation for  $k_s$  quoted in this section may be used in conjunction with appropriate reduction factors.



Where a pile group is oriented **perpendicular** to the direction of loading, group action may be considered by reducing values for  $k_s$  by a reduction factor R as follows:

Pile Spacing Perpendicular to Direction of Loading	Horizontal Subgrade Reaction Reduction Factor, R
4 D*	1.00
1 D*	0.50

\* D is the width of the pile, and spacing is measured centre to centre

Where a pile group is oriented **parallel** to the direction of loading, group action may be considered by reducing values for  $k_s$  by a reduction factor R as follows:

Pile Spacing Parallel to Direction of Loading	Horizontal Subgrade Reaction Reduction Factor, R
8 D*	1.00
6 D*	0.70
4 D*	0.40
3 D*	0.25

\* D is the width of the pile, and spacing is measured centre to centre

Intermediate values may be obtained by interpolation. For conventional abutments and at the pier location the lateral resistance may be provided by battered piles.

### 7.3.5 Pile Tips

Due to the presence of cobbles and boulders in the clayey silt till overlying bedrock, the tips of all piles should be fitted with H-section rock points from an approved manufacturer such as Titus Steel Company (Standard “H” bearing pile point) or Associated Pile & Fitting Corp (APF Hard Bite).

The use of rock points is recommended for the following reasons:

- The piles will be penetrating into soil containing cobbles and boulders, which requires a higher level of protection.
- This requirement will assist the piles to fully penetrate zones of cobbles to achieve effective refusal on bedrock.

### 7.3.6 Pile Installation

Pile installation should be in accordance with OPSS 903, November 2009. The Contract Documents should contain a NSSP alerting the Contractor to the presence of cobbles and boulders in the till soils. The recommended maximum inclination of battered piles should not be greater than 3V:1H.

### 7.3.7 Pile Driving

Note No. 6 from Article 3.3.3 Pile Driving Notes in the MTO Structural Manual should be used on the Foundation Drawing, i.e. “Piles to be fitted with rock points and driven into bedrock in accordance with OPSS 903, November 2009”.

Hammers used for pile installation must be capable of installing the piles to the depths specified in the contract documents. Since the piles will be seated on bedrock the hammer used must also be



capable of delivering a controlled blow in 10% increments ranging in energy from zero to the maximum hammer energy. A typical hammer energy of 60 kilojoules/blow, with an energy transfer (efficiency) of 40% is recommended.

#### **7.4 Recommended Foundation**

The use of H-piles at the abutments allows for the design of an integral or semi-integral abutment structure. From a geotechnical point of view, it is recommended that all foundations for this bridge be supported on steel H-piles.

#### **7.5 Frost Cover**

Pile caps and footings should be provided with a minimum of 1.2 m of earth cover over the footing base (founding elevation).

### **8 EXCAVATION AND BACKFILL**

#### **8.1 General**

All excavations 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 classified as Type 2 soils above the water table and Type 4 soils below the water table. The fill material may be classified as Type 3 soils. Excavations may be sloped at 2H:1V.

#### **8.2 Foundations**

The excavation and backfilling for foundations must be carried out in accordance with OPSS 902, November 2009.

### **9 GROUND WATER CONTROL**

The local ground water level at this site exists at about Elev.  $\pm 179.5$  m and it is unlikely that excavations will extend below the ground water level.

If dewatering is required the design of the unwatering system should be the responsibility of the Contractor. A suitable system that might be employed can include gravity drainage and pumping from strategically placed filtered sumps.

Any accumulation of water from the base of excavations should be removed prior to placing concrete or compacting fill. Placement of concrete or compacting engineered fill must be done in the dry.



## 10 APPROACH EMBANKMENTS

### 10.1 Stability

Embankments constructed at conventional 2H:1V slopes in the Niagara area have historically performed below par. Shallow surficial failures usually occur on the face of these slopes thereby requiring frequent maintenance in order to prevent more significant deep-seated failures.

Recent studies conducted by the Ministry of Transport indicate that these shallow surficial failures occur because of the mineralogy of the local soils and its inherent effect on the effective shear strength of the local clay fill. Poor performance was also attributed to climatic effects including precipitation, wetting and drying cycles, snow melt and freezing and thawing cycles.

The historical performance of existing embankments in this area was considered when selecting approach embankment alternatives for this bridge. The options that were considered are:

- Embankments constructed with local earth borrow.
- Embankments constructed with rock fill.
- Embankments constructed with SSM imported from a designated source.
- Reinforced Earth Embankments.
- Embankments constructed with light weight and ultra light weight fill.

The global, internal and surficial stability of the embankments will depend on their slope geometries and also to a large degree on the material used to construct the embankment. For the purpose of embankment stability analyses, the commercially available slope stability program Slide 5.0 developed by Rocscience Inc. was used. The Janbu, Morgenstern-Price and Bishop's simplified method for stability analysis were employed and a minimum target factor of safety of 1.3 was established. The soil parameters used for the slope stability analyses are presented in Table 10.1 and the minimum factors of safety obtained for the various embankment options are included in Table 10.2.

**Table 10.1 – Soil Parameters**

Material Type	Short-Term Analysis			Long-Term Analysis		
	$\phi$ (degrees)	c (kPa)	$\gamma$ (kN/m <sup>3</sup> )	$\phi'$ (degrees)	c' (kPa)	$\gamma$ (kN/m <sup>3</sup> )
Local Earth Fill	31	0	19.0	31	0	19.0
Rock Fill	42	0	19.0	42	0	19.0
Select Subgrade Material	32	0	20.0	32	0	20.0
Light Weight Fill*	35	0	14.5	35	0	14.5
Ultra Light Weight Fill*	35	0	11.5	35	0	11.5
Fill – Silty Clay	0	50	18.0	29	7	18.0
Silty Clay	0	25 – 100	20.0 – 20.5	27 – 29	5 – 7	20.0 – 20.5
Silt	25	0	18.0	25	0	18.0
Silty Clay Till	0	200	21.0	29	5	21.0

\* Pelletized Blast Furnace Slag – Reference: Lafarge Canada Inc.



**Table 10.2 – Factors of Safety**

<b>Embankment Composition</b>	<b>Design Side Slope</b>	<b>Minimum Factor of Safety Short-Term</b>	<b>Minimum Factor of Safety Long-Term</b>
Local Earth Fill	3H:1V	1.4	2.1
Rock Fill	1.25H:1V	1.4	1.5
SSM	2H:1V	1.3	1.5
RSS Embankment	2H:1V	1.5	1.7
Light Weight Fill	2H:1V	1.7	1.7
Ultra Light Weight Fill	2H:1V	1.7	1.7

The analysis indicates that the factors of safety with respect to shallow surficial failures in the embankment fill and deep seated failures in the underlying soils will be equal to or greater than the target factor of safety of 1.3 for the recommended design side slopes.

The forward slopes at the abutments were also assessed for stability. A target Factor of Safety of 1.5 was established and the slope stability analyses yielded the following factors of safety:

- Local Earth Fill (3H:1V) – Factors of safety ranging from 1.7 to 1.8.
- Rock Fill (1.25H:1V) – Factors of safety of 1.5.
- SSM (2H:1V) – Factors of safety ranging from 1.6 to 2.3.
- Light Weight Fill (2H:1V) – Factors of safety of 1.6.
- Ultra Light Weight Fill (2H:1V) – Factors of safety of 1.6.

Where earth fill, SSM or light weight fill embankments are higher than 8 m, mid-height berms should be incorporated in the design. The berms should:

- extend for the length through which the embankment height exceeds 8 m
- be at least 2 m wide
- have 2% positive drainage to shed run-off water.

Where rock fill embankments are higher than 10 m, mid-height berms should be incorporated in the design. The berms should:

- extend for the length through which the embankment height exceeds 10 m
- be at least 2 m wide
- have 2% positive drainage to shed run-off water.

## **10.2 Settlement**

To predict the magnitude and time rate of settlement of the underlying silty clay soils the commercially available program Settle 3D developed by Rocscience Inc. was used. The highest embankment sections (next to the bridge approaches) were selected as critical sections.

The deformation parameters used for the analyses were established from data obtained from consolidation tests as well as from predictions based on undrained shear strengths, laboratory index tests and soil moisture contents.



Pre-consolidation pressures were estimated from the consolidation test e-log p curves and the Strain-Energy method proposed by Becker (1987). The empirical correlation suggested in the literature by Skempton (1957) was also used to estimate preconsolidation pressures. Profiles of the preconsolidation pressure design lines versus elevation are illustrated in Figure E1. The vertical effective overburden stress is also plotted on this figure.

Values of the compression index ( $C_c$ ) and recompression index ( $C_r$ ) were estimated from the consolidation tests as well as from laboratory index test data using empirical correlations proposed in literature by Terzaghi & Peck (1967), Nagaraj & Murty (1985), Lav & Ansal (2001), Kulhawy & Mayne (1990) and Das (1993). Profiles of the design lines versus elevation are shown on Figures E2 and E3.

Initial void ratio ( $e_o$ ) values were estimated from the consolidation tests as well as from empirical correlations proposed in the literature by Cozzolino (1961), Azzouz et al. (1976) and Lav & Ansal (2001). A profile of the design line versus elevation is shown in Figure E4.

The data indicates an over-consolidated silty clay stratum. The parameters used for the settlement calculations are tabulated below. There is a wide scatter in the data. The two rows of data for  $P_c$  represent the range of values for the upper and lower half of the two strata.

**Table 10.3 – Settlement Parameters**

Parameter	Upper Silty Clay	Lower Silty Clay	Lower Silty Clay/Clayey Silt
Preconsolidation Pressure Range $P_c$ (kPa)	600 to 450 500 to 360	450 to 360	450 to 360
Coefficient of Compressibility - $C_c$	0.30 to 0.35	0.22 to 0.26	0.14 to 0.17
Recompression Index - $C_r$	0.04 to 0.045	0.030 to 0.040	0.019 to 0.025
Initial Void Ratio - $e_o$	0.95 to 1.0	0.80 to 0.90	0.55 to 0.65

Settlement analyses were undertaken for various embankment compositions and geometries and the estimated range of total settlements below the embankment centre line are provided in Table 10.4 and in Figure E5. Where the loads induced by the embankments do not exceed the estimated preconsolidation pressure the recompression index ( $C_r$ ) was used for settlement calculations. Where the embankment loads exceed the preconsolidation pressure the analysis was based on soil recompression and consolidation and both the recompression index ( $C_r$ ) and the coefficient of consolidation ( $C_c$ ) were used.

**Table 10.4 – Approach Embankments Estimated Consolidation Settlements**

Type of Fill	Unit Weight of Fill ( $kN/m^3$ )	Side Slope Geometry	Settlement (mm)
Local Earth Fill	19.0	3H:1V	155 – 200
Rock Fill	19.0	1.25H:1V	155 – 200
Select Subgrade Material	20.0	2H:1V	155 – 200
Lightweight Fill*	14.5	2H:1V	130 – 170
Ultra Lightweight Fill*	11.5	2H:1V	115 – 145
EPS Geofoam	0.31	2H:1V	Negligible

\* Pelletized Blast Furnace Slag – Reference: Lafarge Canada Inc.





Embankments comprised of local earth fill or select subgrade material will also settle during construction (fill compression) and this settlement is expected to be about 1% of the fill height. This settlement should be immediate in nature and essentially be complete shortly after construction is complete. The lightweight and ultra lightweight fill are expected to settle about 0.5% of the fill height. For rock fill, compression is expected to be:

- 0.5% of fill height for embankments up to 5 m high,
- 0.75% of fill height for embankments of 5 to 10 m high,
- 1% of fill height for embankments of 10 to 15 m high.

The length of time required to complete consolidation settlement of the underlying soils is a function of the value of the coefficient of consolidation of the native silty clay strata and the assumed depth of drainage path. Given the heavily over-consolidated and likely fractured nature of the silty clay soils above the water table, it is reasonable to assume that consolidation/recompression will occur quickly in this layer and that the rate of consolidation will be primarily controlled by the coefficient of consolidation and thickness of the underlying silty clay stratum below the water table. The coefficient of consolidation was estimated to range between  $2.0 \times 10^{-3} \text{ cm}^2/\text{s}$  and  $3.5 \times 10^{-3} \text{ cm}^2/\text{s}$ . Tabulated below are the ranges of predicted settlements (embankment centreline) at various time periods.

Port Robinson Road Underpass					
Embankment Type	Settlement At Various Time Periods (mm)				Total Settlement (mm)
	6 months	12 months	18 months	24 months	
Local Earth Fill	120 – 155	135 – 175	145 – 185	150 – 190	155 – 200
Rock Fill	120 – 155	135 – 175	145 – 185	150 – 190	155 – 200
Select Subgrade Material	120 – 155	135 – 175	145 – 185	150 – 190	155 – 200
Lightweight Fill	105 – 130	115 – 145	120 – 155	125 – 160	130 – 165
Ultra Lightweight Fill	90 – 115	100 – 125	105 – 135	110 – 140	115 – 145

It is understood that a maximum allowable post-construction settlement of about 25 mm would be considered acceptable. This data shows that after 12 months, the post-construction consolidation settlement will be 25 mm or less (required performance) for all embankment material types.

If an accelerated construction schedule is required (target of 6 months) we recommend conventional temporary surcharging be carried out (2 m of additional fill height) to accelerate the settlement and ensure full consolidation within the target 6 months after embankment construction (Figures H2 to H4).

Based on the foregoing, other means/methods (wick drains) of accelerating the settlement are not warranted. However, a settlement monitoring program must be conducted to confirm the anticipated settlement performance. A recommended settlement monitoring programme is provided in Appendix G.

Surcharged embankments were analysed for stability in accordance with the recommended side slopes and the analyses yielded factors of safety greater than a target factor of safety of 1.3.



### 10.3 Embankment Construction

It is recommended that the topsoil, any deleterious material and soft/loose and other unsuitable soils be removed within an envelope given by an imaginary slope not steeper than 1H:1V from the toe of the proposed embankment. Borrow material must meet the requirements of OPSS 212, (2008). Grading shall be undertaken in accordance with OPSS 201, (2007) and OPSS 206, (2009).

The recommended stripping depths and elevations of the proposed embankments are:

Location	Average Stripping Depth (m)	Recommended Stripping Elevation (m)
West Approach	±0.2	±181.5
East Approach	±0.7	±180.5

After stripping, the exposed soils should be inspected, approved and properly compacted from the surface in accordance with OPSS 501. Embankment construction should be in accordance with OPSS 501 and OPSS 206.

Wet silty clay soils at this site will be weakened when subjected to construction traffic. To facilitate construction operations in inclement weather (when stripping to the recommended elevation) surface water runoff should be controlled by gravity drainage and a system of interceptor trenches. In wet weather an approximately 200 mm thick free draining granular layer would also be required to minimize disturbance and maintain trafficability of construction equipment.

Proper erosion control measures should be implemented both during construction and permanently. Temporary erosion and sediment control must be provided in accordance with OPSS 577. Fill slopes must be provided with permanent erosion protection in accordance with OPSS 571 and/or OPSS 572.

It is also imperative that the designs include provisions for preventing the flow of surface water down the face of slopes. Surface water must be directed to armoured outfalls/outlets designed to drain into roadside ditches.

### 11 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 250 mm and should include 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 should be in accordance with OPSS 902. Granular backfill should be placed to the extents shown in OPSD 3101.150, and rock backfill should be placed to the extents shown in OPSD 3101.200.



All granular material should meet the specifications of Special Provision 110S13 “Amendment to OPSS 1010, April 2004”.

Compaction equipment to be used adjacent to retaining structures should be restricted in accordance with Special Provision 105S10 “Amendment to OPSS 501, February 1996”.

The design of the abutment should incorporate a subdrain as shown in OPSD 3101.150 or OPSD 3101.200, as applicable.

## **12 LATERAL EARTH PRESSURE**

For cases where backfill to the abutment is placed in accordance with OPSD 3101.150 or OPSD 3101.200, as recommended, the lateral earth pressure will be governed by the properties of the material within the backfill limits shown in the respective OPSD, i.e. a line projected up at 1.5H:1V for granular backfill and 1.25H:1V for rock backfill.

If the support system allows yielding of the wall (unrestrained system), active horizontal earth pressure may be used in the geotechnical design of the structure. If the support system does not allow yielding (restrained system), at-rest horizontal earth pressures should be used. The amount of wall movement required for the development of active, passive and at-rest earth pressures may be interpreted using Figure C6.9.1(a) in the Commentary to the CHBDC.

Earth pressures acting on the structure should be computed in accordance with Clause 6.9 of the CHBDC but generally are given by the expression:

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

$P_h$  = horizontal pressure on the wall (kPa)

$K$  = earth pressure coefficient (see table 12.1)

$\gamma$  = unit weight of retained soil (see table 12.1)

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

$q$  = value of any surcharge (kPa)

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 1.7 m for Granular B Type I or at a depth of 2.0 m for Granular A or Granular B Type II.



Earth pressure coefficients for backfill to the abutment wall are dependent on the material used as backfill. Typical values are given in Table 12.1.

**Table 12.1 – Earth Pressure Coefficients**

Wall 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 $\phi = 42^\circ; \gamma = 19.0 \text{ kN/m}^3$	
	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*	0.30	0.48*	0.20	0.28*
At Rest (Restrained Wall)	0.43	-	0.47	-	0.33	-
Passive (Movement Towards Soil Mass)	3.70	-	3.30	-	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 or semi-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. However, the use of Granular B Type I may be restricted if the approach embankment consists of rock fill.

The factors in the table 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 CHBDC, 2006.

## 13 SEISMIC CONSIDERATIONS

### 13.1 Seismic Design Parameters

The site is treated as lying in Seismic Zone 0. The following seismic parameters (Welland) should be used for design:

- Velocity Related Seismic Zone 0
- Zonal Velocity Ratio 0
- 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. Therefore, according to Table 4.4.6.1 of the CHBDC, a Site Coefficient “S” (ground motion amplification factor) of 1.0 should be used in seismic design.

### 13.2 Liquefaction Potential

The potential for liquefaction of the deep silt layer encountered at this site was assessed using the Seed and Idriss (1971) method<sup>1</sup>. The silt is prone to liquefaction at the foundation elements. However, the silt is overlain by a relatively thick non-liquefiable layer of silty clay which will prevent any observable effects of this in-depth liquefaction from reaching ground surface. Furthermore, since the foundation loads will be transferred by steel piles bearing on hard clayey silt till and/or bedrock, the vertical geotechnical resistance of the piles will not be compromised.

The silt layer located below the immediate approach embankments is not susceptible to liquefaction. The embankments will bear on stiff to very stiff silty clay soils above the ground water level therefore there is negligible potential for soil liquefaction below the embankments. Some toe failure may occur but is expected to be limited and readily repairable.

### 13.3 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.

In calculating the active, passive and at rest earth pressure coefficients the angle of friction between the wall and backfill material is assumed to be  $0.5 \phi$ . For the design of retaining walls, the coefficients of horizontal earth pressure in Table 13.1 may be used:

**Table 13.1 – Earth Pressure Coefficient for Earthquake Loading**

Earth Pressure Coefficient (K) for Earthquake Loading						
Wall Condition	Granular A or Granular B Type II $\phi = 35^\circ$ ; $\delta = 17.5^\circ$ $\gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I $\phi = 32^\circ$ ; $\delta = 16^\circ$ $\gamma = 21.2 \text{ kN/m}^3$		Rock Fill $\phi = 42^\circ$ ; $\delta = 21^\circ$ $\gamma = 19.0 \text{ kN/m}^3$	
	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 ( $K_{AE}$ )*	0.28	0.45	0.32	0.55	0.21	0.30
Passive ( $K_{PE}$ )	3.69	-	3.26	-	5.05	-
At Rest ( $K_{OE}$ )**	0.53	-	0.58	-	0.44	-

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

\*\* After Woods

<sup>1</sup> 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, September, pp. 1249-1273.

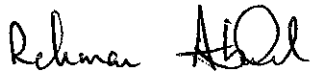


#### 14 CONSTRUCTION CONCERNS

During construction, the Contract Administrator should employ experienced geotechnical staff to observe construction activities related to foundation construction.

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

- the possibility of piles encountering cobbles and boulders and "hanging up" before reaching effective refusal at the design tip elevations.
- the nature and geotechnical properties of the local earth fill used in the approach fills.



Engineering Analysis and Report Preparation by:  
R. Abdul, P.Eng.,  
Senior Geotechnical Engineer

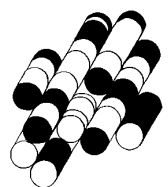


Report Reviewed by:  
Michael Tanos, P.Eng.,  
Review Principal



# TABLE

**TERRAPROBE INC.**



**TABLE 1**

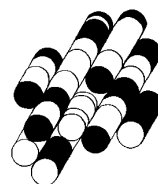
<b>DOCUMENT</b>	<b>TITLE</b>
OPSS 201	Construction Specification for Clearing, Close Cut Clearing, Grubbing and Removal of Surface and Piled Boulders.
OPSS 206	Construction Specification for Grading.
OPSS 212	Construction Specification of Borrow.
OPSS 501	Construction Specification for Compacting.
OPSS 571	Construction Specification for Sodding.
OPSS 572	Construction Specification for Seed and Cover.
OPSS 577	Construction Specification for Temporary Erosion and Sediment Control Measures.
OPSS 902	Construction Specification for Excavation & Backfilling of Structures
OPSS 903	Construction Specification for Deep Foundations
OPSS 1010	Material Specifications for Aggregates, Select Subgrade, Backfill
OPSD 3101.150	Walls, Abutment Backfill – Min. Granular Requirement
OPSD 3101.200	Walls, Abutment Backfill – Rock





# APPENDICES

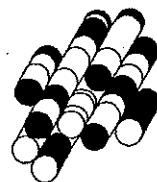
**TERRAPROBE INC.**



# **APPENDIX A**

## **Record of Borehole Sheets, Core Logs and Core Photos**

**Terraprobe Inc.**



## LIMITATIONS AND RISK

### Procedures

The soil conditions were confirmed at the borehole and test pit locations only and conditions may vary between and beyond the boreholes. The boundaries between the various strata as shown on the logs are based on non-continuous sampling. These boundaries represent an inferred transition between the various strata, rather than a precise plane of stratigraphic change.

This investigation has been carried out using investigation techniques and engineering analysis methods consistent with those ordinarily exercised by Terraprobe and other engineering practitioners, working under similar conditions and subject to the time, financial and physical constraints applicable to this project. The discussions and recommendations that have been presented are based on the factual data obtained.

It must be recognized that there are special risks whenever engineering or related disciplines are applied to identify subsurface conditions. Even a comprehensive sampling and testing programme implemented in accordance with the most stringent level of care may fail to detect certain conditions. Terraprobe has assumed for the purposes of providing design parameters and advice, that the conditions that exist between sampling points are similar to those found at the sample locations. The conditions that Terraprobe has interpreted to exist between sampling points can differ from those that actually exist.

It may not be possible to drill a sufficient number of boreholes or sample and report them in a way that would provide all the subsurface information that could affect construction costs, techniques, equipment and scheduling. Contractors bidding on or undertaking work on the project should be directed to draw their own conclusions as to how the subsurface conditions may affect them, based on their own investigations and their own interpretations of the factual investigation results, cognizant of the risks implicit in the subsurface investigation activities.

### Changes In Site And Scope

It must be recognized that the passage of time, natural occurrences, and direct or indirect human intervention at or near the site have the potential to alter subsurface conditions. Groundwater levels are particularly susceptible to seasonal fluctuations.

The design advice is based on the factual data obtained from this investigation made at the site by Terraprobe and are intended for use by the owner and its retained designers in the design phase of the project. If there are changes to the project scope and development features, or there is any additional information relevant to the interpretations made of the subsurface information, the geotechnical design parameters and comments relating to constructibility issues and quality control may not be relevant or complete for the revised project. Terraprobe should be retained to review the implications of such changes with respect to the contents of this report.

This report was prepared for the express use of the Ministry of Transportation, its retained design consultants and Giffels Associates Ltd./IBI Group. It is not for use by others. This report is copyright of Terraprobe Inc. and no part of this report may be reproduced by any means, in any form, without the prior written permission of Terraprobe Inc. The Ministry of Transportation, its retained design consultants and Giffels Associates Ltd./IBI Group, are authorized users.

## EXPLANATION OF TERMS USED IN REPORT

**N VALUE:** THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg. FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS  $\bar{N}$ .

**DYNAMIC CONE PENETRATION TEST:** CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

**CONSISTENCY:** COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH ( $c_u$ ) AS FOLLOWS:

$c_u$ (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	> 200
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

**DENSENESS:** COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 - 5	5 - 10	10 - 30	30 - 50	> 50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND/OR STRENGTH.

**RECOVERY:** SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

**MODIFIED RECOVERY:** SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY IS:

RQD (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

**JOINTING AND BEDDING:**

SPACING	50mm	50 - 300mm	0.3m - 1m	1m - 3m	> 3m
JOINTING	VERY CLOSE	CLOSE	MOD. CLOSE	WIDE	VERY WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK	VERY THICK

## ABBREVIATIONS AND SYMBOLS

### FIELD SAMPLING

SS	SPLIT SPOON	TP	THINWALL PISTON
WS	WASH SAMPLE	OS	OSTERBERG SAMPLE
ST	SLOTTED TUBE SAMPLE	RC	ROCK CORE
BS	BLOCK SAMPLE	PH	TW ADVANCED HYDRAULICALLY
CS	CHUNK SAMPLE	PM	TW ADVANCED MANUALLY
TW	THINWALL OPEN	FS	FOIL SAMPLE

### STRESS AND STRAIN

$u$	kPa	PORE WATER PRESSURE
$r_u$	1	PORE PRESSURE RATIO
$\sigma$	kPa	TOTAL NORMAL STRESS
$\sigma'$	kPa	EFFECTIVE NORMAL STRESS
$\tau$	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
$\epsilon$	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
$E$	kPa	MODULUS OF LINEAR DEFORMATION
$G$	kPa	MODULUS OF SHEAR DEFORMATION
$\mu$	1	COEFFICIENT OF FRICTION

### MECHANICAL PROPERTIES OF SOIL

$m_v$	kPa <sup>-1</sup>	COEFFICIENT OF VOLUME CHANGE
$C_c$	1	COMPRESSION INDEX
$C_s$	1	SWELLING INDEX
$C_\alpha$	1	RATE OF SECONDARY CONSOLIDATION
$C_v$	m <sup>2</sup> /s	COEFFICIENT OF CONSOLIDATION
$H$	m	DRAINAGE PATH
$T_v$	1	TIME FACTOR
$U$	%	DEGREE OF CONSOLIDATION
$\sigma'_{vo}$	kPa	EFFECTIVE OVERBURDEN PRESSURE
$\sigma'_p$	kPa	PRECONSOLIDATION PRESSURE
$\tau_f$	kPa	SHEAR STRENGTH
$c'$	kPa	EFFECTIVE COHESION INTERCEPT
$\phi'$	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
$c_u$	kPa	APPARENT COHESION INTERCEPT
$\phi_u$	-°	APPARENT ANGLE OF INTERNAL FRICTION
$\tau_R$	kPa	RESIDUAL SHEAR STRENGTH
$\tau_r$	kPa	REMOULDED SHEAR STRENGTH
$S_r$	1	SENSITIVITY = $c_u / \tau_r$

## PHYSICAL PROPERTIES OF SOIL

$\rho_s$	kg/m <sup>3</sup>	DENSITY OF SOLID PARTICLES	$e$	1%	VOID RATIO	$e_{min}$	1%	VOID RATIO IN DENSEST STATE
$\gamma_s$	kN/m <sup>3</sup>	UNIT WEIGHT OF SOLID PARTICLES	$n$	1%	POROSITY	$I_D$	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
$\rho_w$	kg/m <sup>3</sup>	DENSITY OF WATER	$w$	1%	WATER CONTENT	$D$	mm	GRAIN DIAMETER
$\gamma_w$	kN/m <sup>3</sup>	UNIT WEIGHT OF WATER	$S_r$	%	DEGREE OF SATURATION	$D_n$	mm	n PERCENT - DIAMETER
$\rho$	kg/m <sup>3</sup>	DENSITY OF SOIL	$w_L$	%	LIQUID LIMIT	$C_u$	1	UNIFORMITY COEFFICIENT
$\gamma$	kN/m <sup>3</sup>	UNIT WEIGHT OF SOIL	$w_p$	%	PLASTIC LIMIT	$h$	m	HYDRAULIC HEAD OR POTENTIAL
$\rho_d$	kg/m <sup>3</sup>	DENSITY OF DRY SOIL	$w_s$	%	SHRINKAGE LIMIT	$q$	m <sup>2</sup> /s	RATE OF DISCHARGE
$\gamma_d$	kN/m <sup>3</sup>	UNIT WEIGHT OF DRY SOIL	$I_p$	%	PLASTICITY INDEX = $(w_L - w_p)$	$v$	m/s	DISCHARGE VELOCITY
$\rho_{sat}$	kg/m <sup>3</sup>	DENSITY OF SATURATED SOIL	$I_L$	1	LIQUIDITY INDEX = $(w - w_p)/I_p$	$i$	1	HYDRAULIC GRADIENT
$\gamma_{sat}$	kN/m <sup>3</sup>	UNIT WEIGHT OF SATURATED SOIL	$I_c$	1	CONSISTENCY INDEX = $(w_L - w)/I_p$	$k$	m/s	HYDRAULIC CONDUCTIVITY
$\rho'$	kg/m <sup>3</sup>	DENSITY OF SUBMERGED SOIL	$e_{max}$	1%	VOID RATIO IN LOOSEST STATE	$j$	kN/m <sup>2</sup>	SEEPAGE FORCE
$\gamma'$	kN/m <sup>3</sup>	UNIT WEIGHT OF SUBMERGED SOIL						

## EXPLANATORY SHEET FOR CORE LOG

### Column Number

1. Elevation of borehole collar.
2. Depth of geotechnical boundary in borehole
3. Geologic symbol for rock or soil material
4. General description of geotechnical unit - qualitative description, including rock type(s), percentage rock types, frequency and sizes of interbeds, colour, texture.

### Joint (discontinuity) Characteristics

5. Number of joint sets: a rock mass can be intersected by a number of joint sets of varying orientations.
6. Joint type: B = Bedding joint C = Cross joint
7. Orientation: only variations in dip can be identified in core; dip direction is from field mapping or oriented core:  
F = Flat = 0 - 20° D = Dipping = 20 - 50° V = Vertical = 50 - 90°
8. Joint spacing: this is an approximate measure of spacing between joints in specific joint sets.

SPACING	> 3 m	1 m - 3 m	0.3 m - 1 m	50 mm - 300 mm	< 50 mm
	VERY WIDE	WIDE	MODERATE	CLOSE	VERY CLOSE

9. Roughness:

RU = Rough Undulating  
SU = Smooth Undulating  
LU = Slickensided Undulating

RP = Rough Planar  
SP = Smooth Planar  
LP = Slickensided Planar

10. Filling:

T = Tight, hard, non-softened  
O = Oxidation surface staining only  
SA = Slightly altered; clay-free  
S = Sandy particles; clay-free  
Si = Sandy and silty, minor clay  
NC = Non-softening Clays; 5mm  
SC = Swelling Clay fillings; 5mm

#### Approximate $\phi$

25 - 35  
25 - 30  
25 - 30  
20 - 25  
16 - 24  
6 - 12

11. Aperture: estimated size of joint opening.

12. Degree of weathered rock material:

DEGREE	DESCRIPTION				
UNWEATHERED	NO SIGNS OF DISCOLOURATION OR OXIDIZATION				
SLIGHTLY WEATHERED	PARTIAL DISCOLOURATION; FRACTURES (JOINTS), TYPICALLY OXIDIZED				
MODERATELY WEATHERED	TOTAL DISCOLOURATION				
HIGHLY WEATHERED	TOTAL DISCOLOURATION; TYPICALLY FRIABLE AND PITTED				
COMPLETELY WEATHERED	RESEMBLE A SOIL; ROCK STRUCTURE - USUALLY PRESERVED				

13. Strength of rock material:

		MPa			
VERY HIGH STRENGTH	SPECIMEN CAN ONLY BE CHIPPED BY GEOLOGICAL HAMMER	> 200			
HIGH STRENGTH	SPECIMEN REQUIRES A NUMBER OF BLOWS OF A GEOLOGICAL HAMMER TO FRACTURE IT; CANNOT BE SCRAPPED WITH POCKET KNIFE	50 - 200			
MEDIUM STRENGTH	SPECIMEN CANNOT BE FRACTURED BY A SINGLE, FIRM BLOW OF GEOLOGICAL HAMMER; CAN BE SCRAPPED WITH POCKET KNIFE, NOT PEELED	15 - 50			
LOW STRENGTH	SHALLOW INDENTATIONS MADE BY FIRM BLOW WITH POINT OF GEOLOGICAL HAMMER; CAN BE PEELED WITH POCKET KNIFE WITH DIFFICULTY	4 - 15			
VERY LOW STRENGTH	CRUMBLES UNDER FIRM BLOW WITH POINT OF GEOLOGICAL HAMMER; CAN BE PEELED	1 - 4			

14. Fracture frequency: number of natural joints occurring over a meter length of core. All natural joints are counted irrespective of the number of joint sets.

FRACTURE FREQUENCY	JOINT SPACING	LENGTH			
0.3 m	VERY WIDE	> 3 m			
0.3 - 1 m	WIDE	1 m - 3 m			
1 - 3 m	MODERATE	0.03 m - 1 m			
3 - 20 m	CLOSE	0.005 m - 0.03 m			
20 m	VERY CLOSE	< 0.005 m			

15. Run number and Core Recovery

- (i) Drill run number

- (ii) Total Core Recovery is the total length of core pieces, irrespective of their individual lengths obtained in a core run, and expressed as a percentage of the length of that core run.

16. Rock Quantity Designation (RQD): The total length of those pieces of sound core which are 0.01 metres or greater in length in a core run, expressed as a percentage of the total length of that core run. Sound pieces of rock are those pieces separated by natural breaks and not machine breaks or subsequent artificial breaks.

Rock Mass Classification (after Deare)					
RQD (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
DESCRIPTION	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

17. Core and Casing sizes: changes of core and casing sizes are indicated.

18. Water recovery, level and tests:

- (i) percentage drill water recovery

- (ii) water level depth

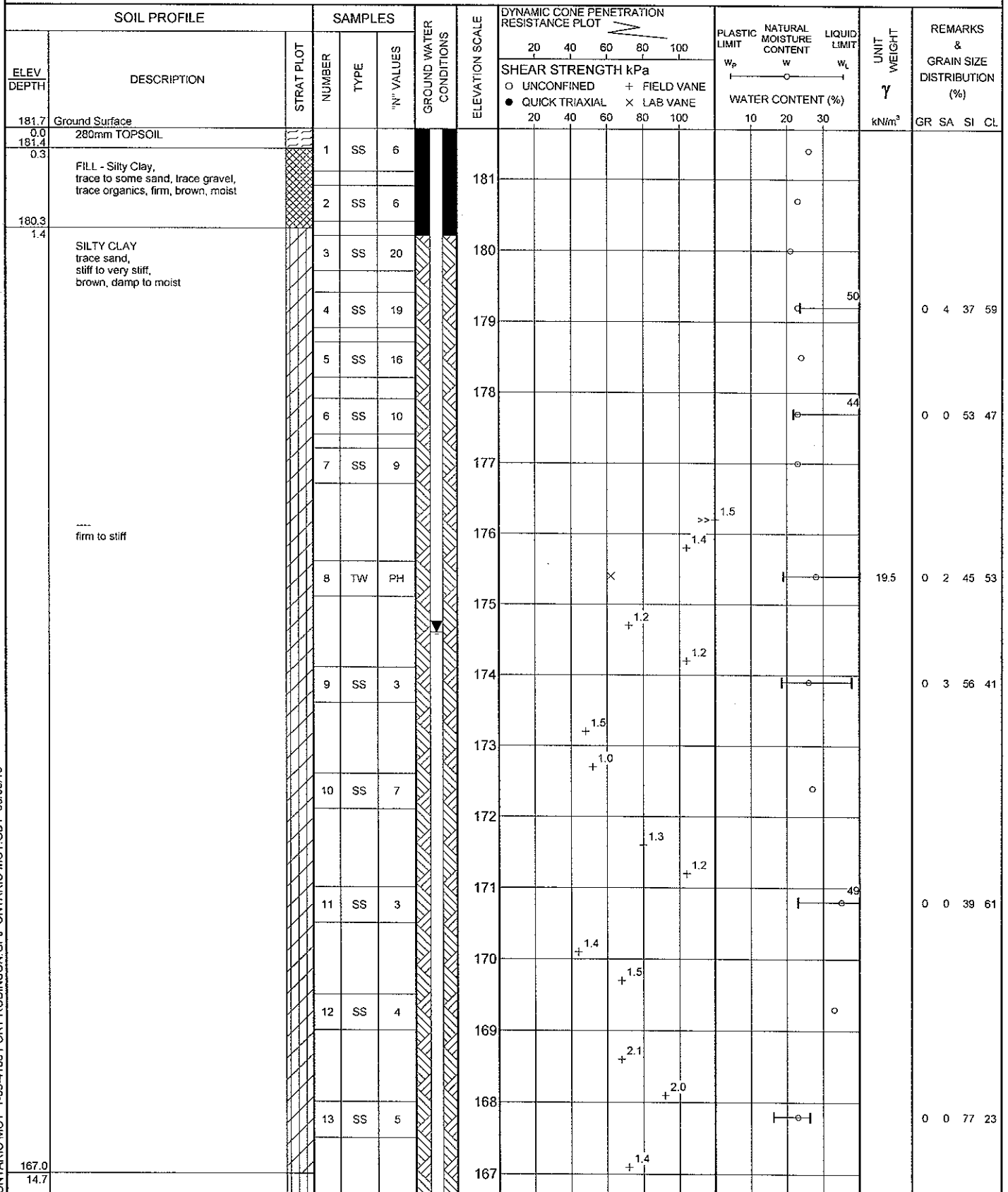
- (iii) positions and results of tests, e.g., permeability and packer tests

# RECORD OF BOREHOLE No PR1

1 OF 3

METRIC

W.P. 280-99-00 LOCATION Coords: N:4766747.4 E:326297.5 ORIGINATED BY PK  
DIST HWY 406 BOREHOLE TYPE Hollow Stem Augers COMPILED BY DB  
DATUM Geodetic DATE 01.04.10 - 01.06.10 CHECKED BY RA



Continued Next Page

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

## METRIC

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

RECORD OF BOREHOLE No PR1

3 OF 3

METRIC

W.P. 280-99-00 LOCATION Coords: N:4766747.4 E:326297.5 ORIGINATED BY PK  
DIST HWY 406 BOREHOLE TYPE Hollow Stem Augers COMPILED BY DB  
DATUM Geodetic DATE 01.04.10 - 01.06.10 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100	W <sub>p</sub>	W	W <sub>L</sub>		
149.7	frequent cobbles.		24	SS	182/ 28cm	151											
32.0			25	SS	100/ 2.5cm	150											
	End of Borehole																
	Resistance to augering at 10.5m, 12.8m, 31.4m, and 31.7m.																
	Borehole was dry (not stabilized) and hole open to full depth on completion.																
	Consolidation test performed on TW 8.																
	Piezometer installation consists of a 19mm diameter, Schedule 40 PVC pipe with a 1.52m slotted screen.																
	Water Level Readings:																
	Date Depth(m) Elevation(m)																
	Jan.11.10 7.0 174.7																
	Jan.19.10 7.2 174.5																
	Jan.27.10 7.1 174.6																
	Feb.08.10 7.2 174.5																
	Feb.19.10 7.1 174.6																

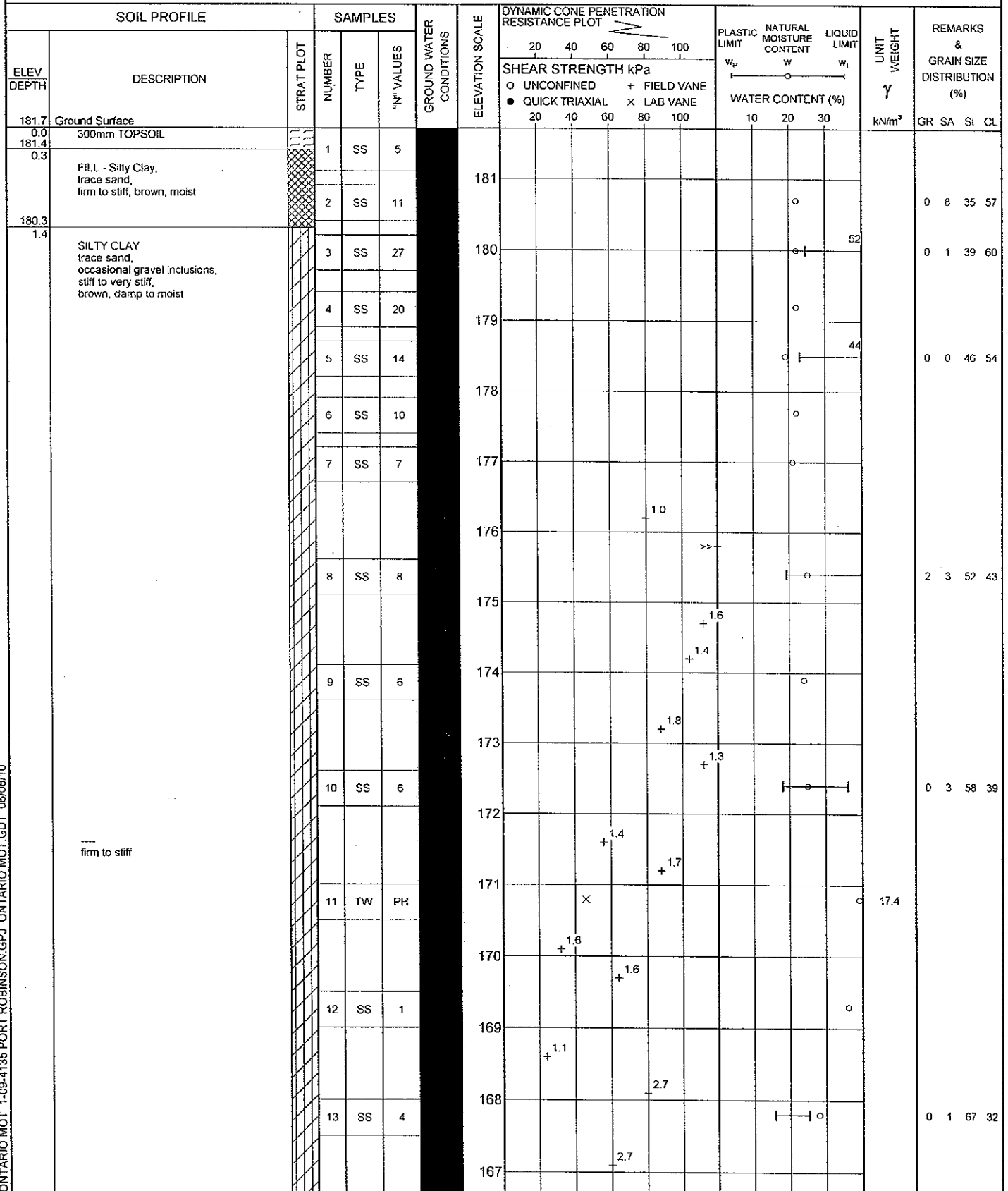


RECORD OF BOREHOLE No PR2

1 OF 3

METRIC

W.P. 280-99-00 LOCATION Coords: N:4766747.3 E:326311.5 ORIGINATED BY PK  
DIST HWY 406 BOREHOLE TYPE Hollow Stem Augers / NQ Rock Coring COMPILED BY DB  
DATUM Geodetic DATE 12.29.09 - 12.30.09 CHECKED BY RA



Continued Next Page

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

## 2 OF 3

## METRIC

SOIL PROFILE						SAMPLES
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	
166.0 15.7	SILT trace clay, trace sand, very loose, brown, wet	[Symbol]	14	SS	3	
		[Symbol]	15	SS	0	
163.9 17.8		SILTY CLAY TO CLAYEY SILT trace sand, stiff to hard, brown / reddish brown, damp to moist	[Symbol]	16	SS	15
			[Symbol]	17	SS	12
			[Symbol]	18	SS	23
			[Symbol]	19	SS	43
			[Symbol]	20	SS	17
			[Symbol]	21	SS	16
154.8 26.9	CLAYEY SILT trace to some sand, trace gravel, very stiff to hard, brown, moist  (GLACIAL TILL)		[Symbol]	22	SS	16
			[Symbol]	23	SS	41
			[Symbol]			

DYNAMIC CONE PENETRATION RESISTANCE PLOT

SHEAR STRENGTH kPa

- O UNCONFINED + FIELD VANE
- QUICK TRIAXIAL x LAB VANE

WATER CONTENT (%)

NATURAL MOISTURE CONTENT W<sub>p</sub>, W, W<sub>L</sub>

UNIT WEIGHT γ KN/m³

REMARKS &  
GRAIN SIZE DISTRIBUTION (%) GR SA SI C

Dec.29  
Dec.30

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

## 3 OF 3

METRIC

[illegible]

ONTARIO MOT 1-09-4135 PORT ROBINSON.GPJ ONTARIO MOT.GDT 06/08/10

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

# CORE LOG



**Terraprobe**

Project Highway 406 Twinning				Orientation Vertical		Ground Elevation 181.7m		Datum Geodetic		Borehole No. PR2	
Location Welland, Ontario				Date Started December 30, 2009		Completed December 30, 2009		Logged By AW		Sheet 1 of 1	
W.P.: 280-99-00				Drilling Agency DDSI		Drill Type Track-Mount		Core Barrel & Bit Design NQ		Project No. 1-09-4135	

ELEVATION (m)	DEPTH (m)	SYMBOL	GENERAL DESCRIPTION	Joint Characteristics								WEATHERING	STRENGTH	FRACTURE FREQUENCY	RUN NO.	CORE RECOVERY %	R Q D %	CORE SIZE/CASING	MPa	UNIT WEIGHT (kN/m³)
				No. OF SETS	JOINT TYPE	ORIENTATION	SPACING	ROUGHNESS	FILLING	APERTURE										
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19		
149.7	32.0		Overburden, refer to Borehole Log PR2																	
149.2	32.5		Clayey Silt TILL, refer to Borehole Log PR2																	
148.7	33.0																			
148.2	33.5		GUELPH FORMATION BEDROCK																	
147.7	34.0		DOLOSTONE Unweathered, thinly laminated, light to medium brownish-grey, medium strength, argillaceous.	1	B	F	C	SP	T	0 to 1										
147.2	34.5																			
146.7	35.0																			
146.2	35.5																			
145.7	36.0																			
145.2	36.5																			
144.7	37.0																			
144.2	37.5		End of Core Log																	
143.7	38.0																			

Remarks:

**LEGEND:**

Dolostone

Clayey Silt TILL

# RECORD OF BOREHOLE No PR3

1 OF 3

METRIC

W.P. 280-99-00 LOCATION Coords: N:4766747.0 E:326343.5 ORIGINATED BY PK  
 DIST HWY 406 BOREHOLE TYPE Hollow Stem Augers / NQ Rock Coring COMPILED BY DB  
 DATUM Geodetic DATE 01.07.10 - 01.08.10 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × LAB VANE						
181.3	Ground Surface							20 40 60 80 100							
0.0 181.0	280mm TOPSOIL							20 40 60 80 100							
0.3	FILL - Silty Sand, frequent clayey inclusions, loose, grey, wet		1	SS	8									0 48 34 18	
			2	SS	4										
179.9			3	SS	6										
1.4	SILTY CLAY trace sand, occasional gravel inclusions, firm to stiff, grey / brown, moist		4	SS	12									0 1 39 60	
			5	SS	14										
			6	SS	9									1 1 65 33	
	soft		7	SS	4										
			8	SS	4									0 4 55 41	
			9	SS	7									0 3 54 43	
			10	SS	4									0 1 57 42	
			11	TW	PH								18.3		
			12	SS	1									0 0 65 35	
			13	SS	3										
166.6															
14.7															

Continued Next Page

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


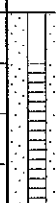
ONTARIO MOT 1-09-4135 PORT ROBINSON.GPJ ONTARIO MOT.GDT 06/08/10

2 OF 3

METRIC

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

## METRIC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa	W <sub>P</sub>	W	W <sub>L</sub>			
147.7 33.6	frequent cobbles  boulder  BEDROCK - DOLOSTONE Unweathered, thinly laminated, light to medium brownish grey, medium strength, argillaceous.		24	SS	172		151							
			25	SS	33		150							Jan.07
			1	RUN	NQ		149							Jan.08
			2	RUN	NQ		148							RUN#1 TCR=0% SCR=0% RQD=0%
			3	RUN	NQ		147							RUN#2 TCR=71% SCR=69% RQD=29%
			4	RUN	NQ		146							RUN#3 TCR=100% SCR=100% RQD=76%
144.2 37.1	End of Borehole  Unable to push vane beyond 5.5m and 14.7m.  Borehole filled with drill water on completion.  Piezometer installation consists of a 19mm diameter, Schedule 40 PVC pipe with a 1.52m slotted screen.  Water Level Readings: Date Depth(m) Elevation(m) Jan.19.10 8.2 173.1 Jan.27.10 6.6 174.7 Feb.08.10 0.6 180.7 Feb.19.10 0.5 180.8						145							RUN#4 TCR=96% SCR=94% RQD=25%


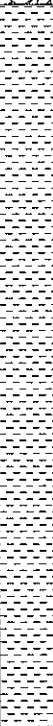
+ 3, × 3: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

# CORE LOG



**Terraprobe**

Project	Highway 406 Twinning	Orientation	Vertical	Ground Elevation	181.3m	Datum	Geodetic	Borehole No.	PR3
Location	Welland, Ontario	Date Started	January 8, 2010	Completed	January 8, 2010	Logged By	AW	Sheet	1 of 1
W.P.:	280-99-00	Drilling Agency	DDSI	Drill Type	Track-Mount	Core Barrel & Bit Design	NQ	Project No.	1-09-4135

ELEVATION (m)	DEPTH (m)	SYMBOL	GENERAL DESCRIPTION	Joint Characteristics							WEATHERING	STRENGTH	FRACTURE FREQUENCY	RUN NO.	CORE RECOVERY %	R Q D %	CORE SIZE/CASING	MPa UNCONFINED COMPRESSIVE STRENGTH	UNIT WEIGHT (kN/m <sup>3</sup> )
				No. OF SETS	JOINT TYPE	ORIENTATION	SPACING	ROUGHNESS	FILLING	APERTURE									
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	
149.3	32.0		Overburden, refer to Borehole Log PR3																
148.8	32.5		Clayey Silt TILL, (Boulder 32.90m to 33.60m), refer to Borehole Log PR3												#1 TCR 0 SCR 0	0	NQ		
148.3	33.0																		
147.8	33.5		GUELPH FORMATION BEDROCK																
147.3	34.0		DOLOSTONE Unweathered, thinly laminated, light to medium brownish grey, medium strength, argillaceous.	1	B	F	C	SP	T	0 5 1					#2 TCR 71 SCR 69	29	NQ		
146.8	34.5																		
146.3	35.0			1	B	F	C	SP	T	0 5 2					#3 TCR 100 SCR 100	76	NQ		
145.8	35.5																		
145.3	36.0																		
144.8	36.5																		
144.3	37.0																		
			End of Core Log																
143.8	37.5																		
143.3	38.0																		

Remarks:

LEGEND:



Dolostone



Clayey Silt Till



# RECORD OF BOREHOLE No PR4

1 OF 3

METRIC

W.P. 280-99-00 LOCATION Coords: N:4768752.2 E:326382.2 ORIGINATED BY PK  
 DIST HWY 406 BOREHOLE TYPE Hollow Stem Augers / NQ Rock Coring COMPILED BY DB  
 DATUM Geodetic DATE 01.28.10 - 02.03.10 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								20 40 60 80 100									
182.2	Ground Surface																
182.0	180mm TOPSOIL																
0.2	FILL - Sand, gravelly, some silt, trace clay, trace organics, compact, brown, moist		1	SS	12									22 50 20 8			
181.5																	
0.7	FILL - Silty Clay, some sand, trace gravel, stiff to very stiff, brown, moist		2	SS	10												
			3	SS	22												
180.1																	
2.1	SILTY CLAY trace sand, firm to very stiff, brown, damp to moist		4	SS	12									0 12 44 44			
			5	SS	12									0 1 35 64			
			6	SS	13									0 0 47 53			
			7	SS	4												
			8	TW	PH								20.3				
			9	SS	7									0 2 59 39			
			10	SS	8												
			11	SS	1									0 1 16 83			
			12	SS	4												
			13	SS	3												
167.5																	
14.7																	

Continued Next Page

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

ONTARIO MOT 1-09-4135 PORT ROBINSON.GPJ ONTARIO MOT.GDT 06/08/10

2 OF 3

METRIC

○ 3% STRAIN AT FAILURE

## METRIC



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

# CORE LOG



**Terraprobe**

Project	Highway 406 Twinning	Orientation	Vertical	Ground Elevation	182.2m	Datum	Geodetic	Borehole No.	PR4
Location	Welland, Ontario	Date Started	February 3, 2010	Completed	February 3, 2010	Logged By	AW	Sheet	1 of 1
W.P.:	280-99-00	Drilling Agency	DDSI	Drill Type	Truck Mount	Core Barrel & Bit Design	NQ	Project No.	1-09-4135

ELEVATION (m)	DEPTH (m)	SYMBOL	GENERAL DESCRIPTION	Joint Characteristics								WEATHERING	STRENGTH	FRACTURE FREQUENCY	CORE RECOVERY %		CORE SIZE/CASING	MPa UNCONFINED COMPRESSIVE STRENGTH	UNIT WEIGHT (KN/m³)
				NO. OF SETS	JOINT TYPE	ORIENTATION	SPACING	ROUGHNESS	FILLING	APERTURE	R				Q D %				
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	
149.7	32.5		Overburden, refer to Borehole Log PR4																
			Clayey Silt Till, refer to Borehole Log PR4																
149.2	33.0														#1 TCR 23 SCR 18	0	NQ		
148.7	33.5																		
148.2	34.0																		
			GUELPH FORMATION BEDROCK												#2 TCR 54 SCR 40	0	NQ		
147.7	34.5			2	BC	FV	VC	SP	T	0 to 1									
			DOLOSTONE																
			Unweathered, thinly laminated, light to medium brownish grey, medium strength, argillaceous.	1	B	F	C	VC	SP	T	0 to 1								
147.2	35.0			1	B	F	C	VC	SU	T	0 to 1				#3 TCR 100 SCR 100	41	NQ		
				2	BC	FV	VC	SU	T										
146.7	35.5			1	B	F	C	SP	T	0 to 1					#4 TCR 56 SCR 34	0	NQ		
146.2	36.0																		
145.7	36.5			1	B	F	C	SP	T	0 to 1									
														#5 TCR 100 SCR 100	62	NQ			
145.2	37.0																		
			<u>Rubblelized zones at:</u> 34.95-35.00m; 35.05-35.20m; 37.20-37.25m.	1	B	F	VC	SP	T										
144.7	37.5		Rubble indicated by 'c'.	1	B	F	C	SP	T	0 to 1				#6 TCR 100 SCR 100	14	NQ			
			<u>Highly fractured zone at:</u> 34.30-34.80m.																
144.2	38.0		End of Core Log																
143.7	38.5																		

Remarks:

## LEGEND:

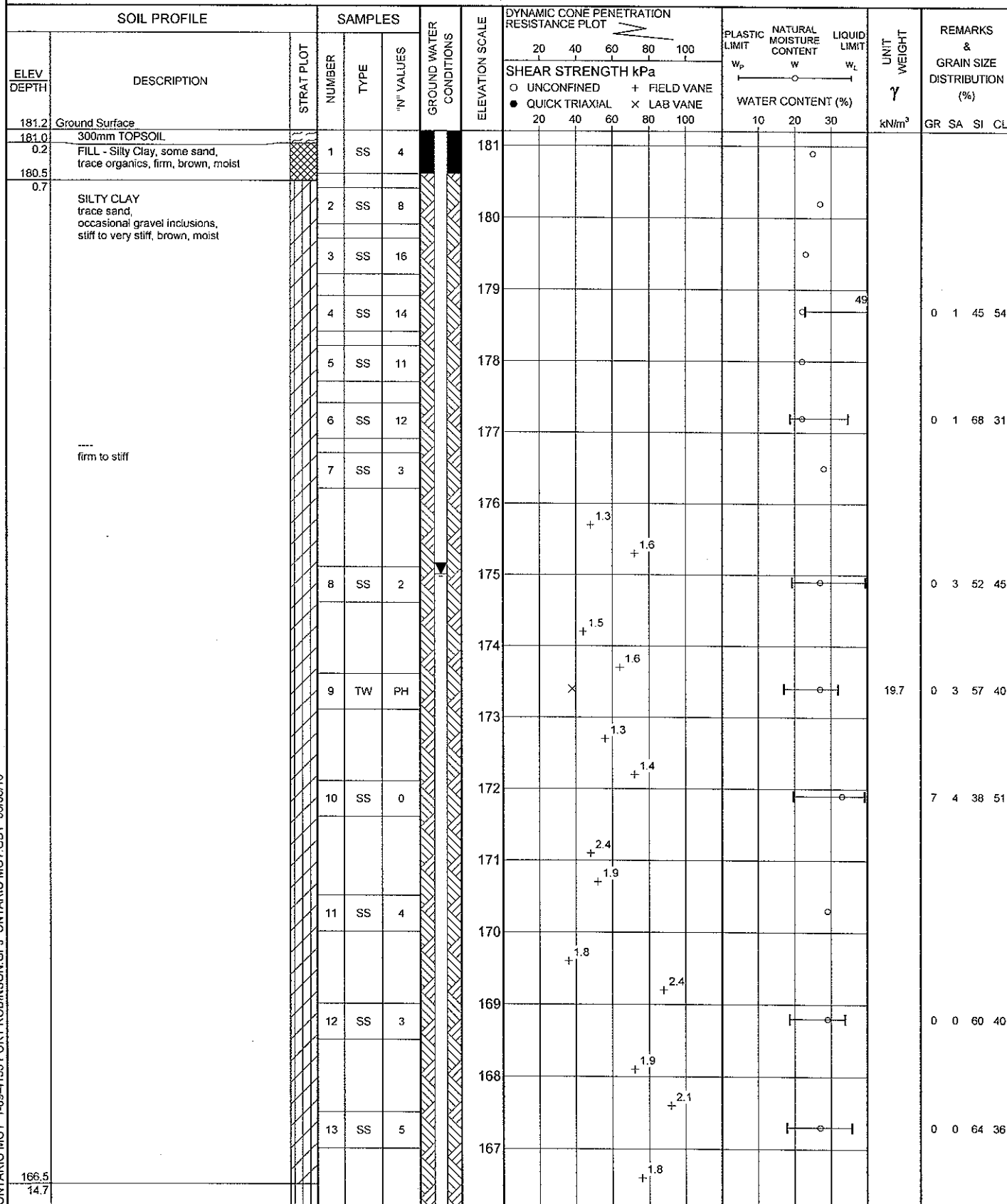
	Dolostone
	Rubble
	Clayey Silt TILL

RECORD OF BOREHOLE No PR5

1 OF 3

METRIC

W.P. 280-99-00 LOCATION Coords: N:4766743.3 E:326398.5 ORIGINATED BY PK  
DIST HWY 406 BOREHOLE TYPE Hollow Stem Augers COMPILED BY DB  
DATUM Geodetic DATE 12.21.09 - 12.22.09 CHECKED BY RA



ONTARIO MOT 1-09-4135 PORT ROBINSON.GPJ ONTARIO MOT.GDT 06/08/10

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

# RECORD OF BOREHOLE No PR5

2 OF 3

METRIC

W.P. 280-99-00 LOCATION Coords: N:4766743.3 E:326398.5 ORIGINATED BY PK  
DIST HWY 406 BOREHOLE TYPE Hollow Stem Augers COMPILED BY DB  
DATUM Geodetic DATE 12.21.09 - 12.22.09 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × LAB VANE						
	SILT trace clay, loose to compact, brown, wet		14	SS	6		166							0 0 93 7	
							165								
163.9			15	SS	26		164								
17.3	SILTY CLAY TO CLAYEY SILT trace to some sand, trace gravel, stiff to very stiff, brown, damp to moist						163								
			16	SS	8		162			1.7					
			17	SS	17		161								
							160								
			18	SS	16		159								
							158								
			19	SS	15		157								
							156								
			20	SS	9		155								
							154								
			21	SS	9		153								
							152								
153.8	CLAYEY SILT trace to some sand, trace gravel, very stiff to hard, brown, damp  (GLACIAL TILL)		22	SS	25										
27.4															
			23	SS	31										

Continued Next Page

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

ONTARIO MOT 1-09-4135 PORT ROBINSON.GPJ ONTARIO MOT.GDT 06/08/10

RECORD OF BOREHOLE No PR5

3 OF 3

METRIC

W.P. 280-99-00 LOCATION Coords: N:4766743.3 E:326398.5 ORIGINATED BY PK  
DIST HWY 406 BOREHOLE TYPE Hollow Stem Augers COMPILED BY DB  
DATUM Geodetic DATE 12.21.09 - 12.22.09 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
150.7			24	SS	92		151										
30.5	End of Borehole																
	Unable to push vane beyond 19.2m.																
	Resistance to augering at 28.9m.																
	No sample recovery at SS5 and SS23. Sampler redriven and disturbed sample collected.																
	Sampler wet at 6.1m.																
	Borehole was dry (not stabilized) and hole open to full depth on completion.																
	Consolidation test performed on TW 9.																
	Piezometer installation consists of a 19mm diameter, Schedule 40 PVC pipe with a 1.52m slotted screen.																
	Water Level Readings:																
	Date Depth(m) Elevation(m)																
	Jan.19.10 6.4 174.8																
	Jan.27.10 6.2 175.0																
	Feb.08.10 6.3 174.9																
	Feb.19.10 6.2 175.0																

ONTARIO MOT 1-09-4135 PORT ROBINSON.GPJ ONTARIO MOT.GDT 06/08/10

+ 3, X 3. Numbers refer to  
Sensitivity

○ 3% STRAIN AT FAILURE

Foundation Investigation Report  
Highway 406 Twinning - Port Robinson Road to East Main Street  
Agreement No.: 2008-E-0016; W.P. 280-99-00

---



**Bedrock Core Sample**

Borehole: PR2

Runs: 1, 2, 3 & 4

Depth: 32.0m – 37.3m





Foundation Investigation Report  
Highway 406 Twinning - Port Robinson Road to East Main Street  
Agreement No.: 2008-E-0016; W.P. 280-99-00

---



**Bedrock Core Sample**

Borehole: PR3

Runs 1, 2, 3 & 4

Depth: 32.0m – 37.1m



Foundation Investigation Report  
Highway 406 Twinning - Port Robinson Road to East Main Street  
Agreement No.: 2008-E-0016; W.P. 280-99-00

---



**Bedrock Core Sample**

Borehole: PR4

Runs: 1, 2, 3, 4, 5 & 6

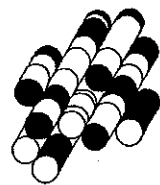
Depth: 32.5m – 38.0m



# **APPENDIX B**

## **Laboratory Test Results**

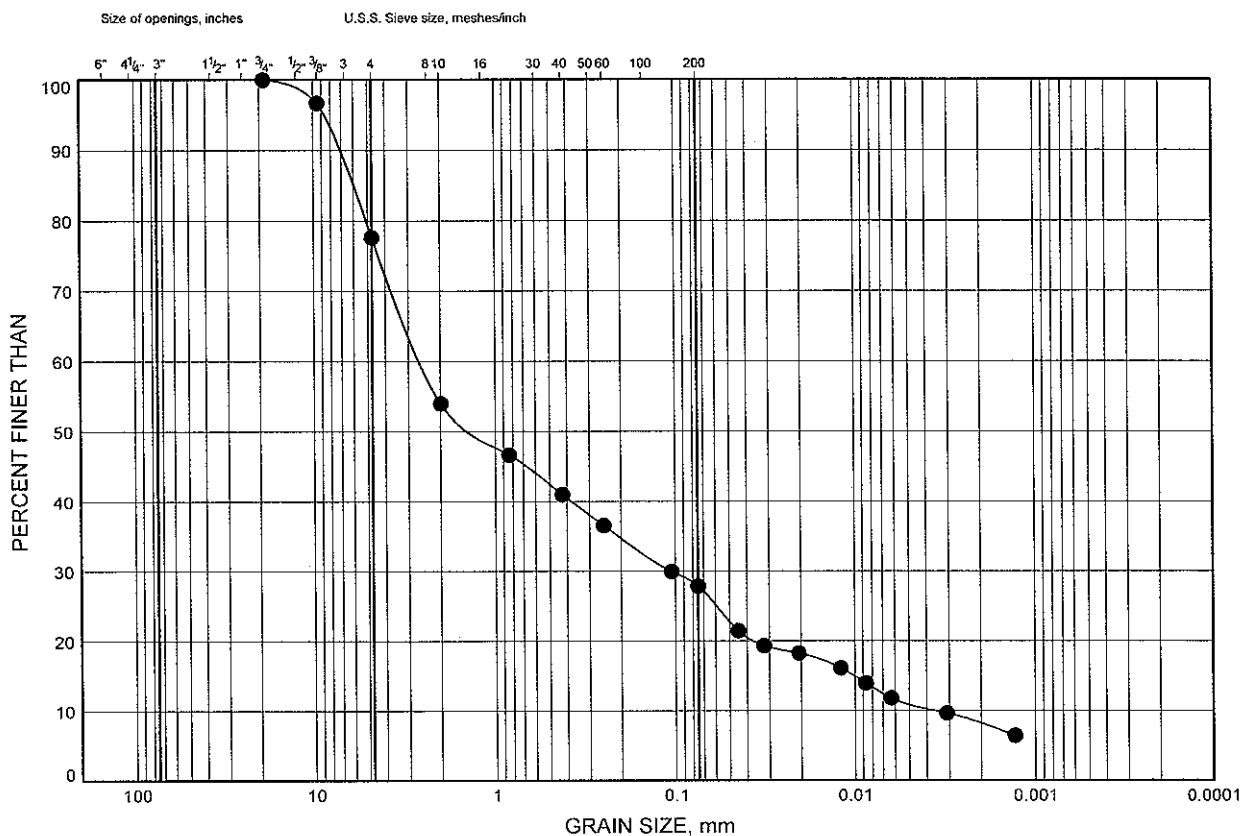
**Terraprobe Inc.**



# GRAIN SIZE DISTRIBUTION

FIGURE B1

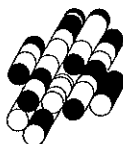
## FILL - Gravelly Sand



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

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	PR4	0.3	181.9

Date June 2010  
Project 1-09-4135

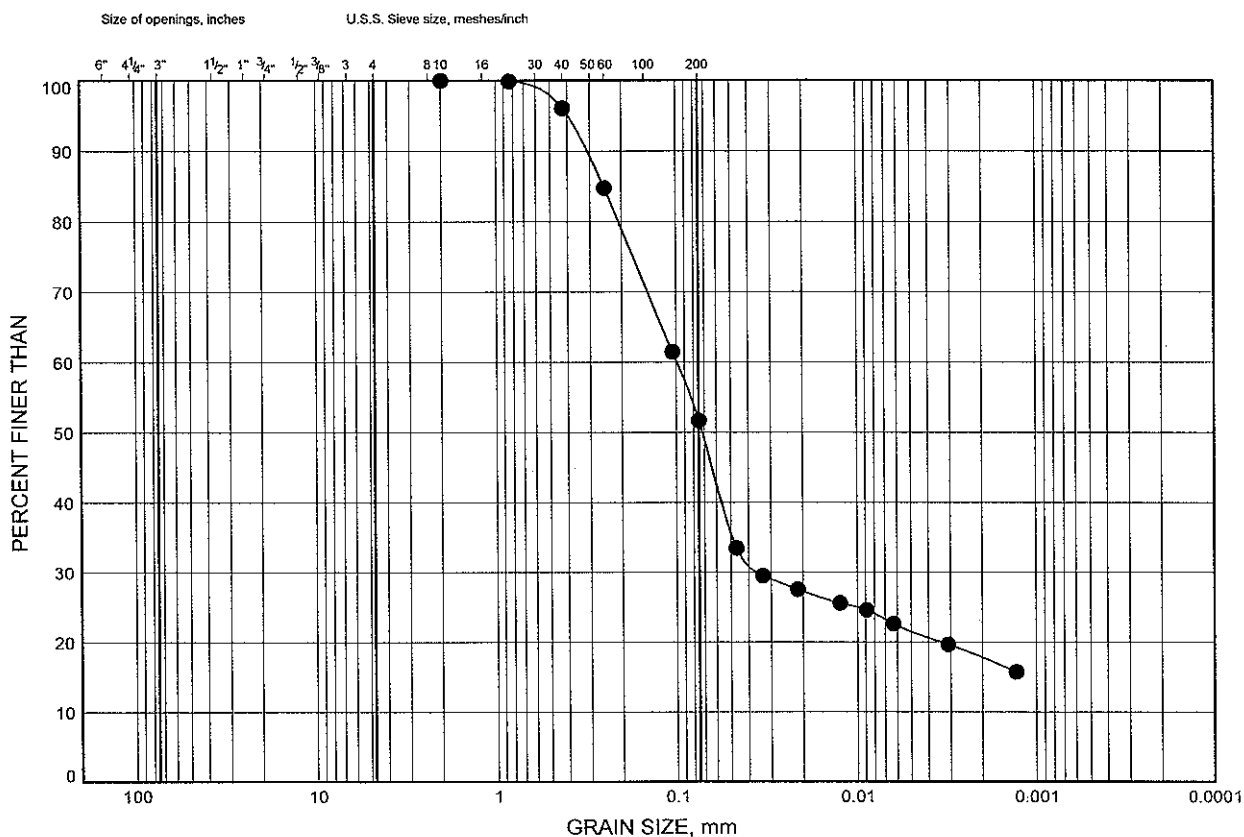


Prep'd DB  
Chkd. MP

# GRAIN SIZE DISTRIBUTION

FIGURE B2

## FILL - Silty Sand

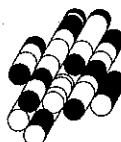


COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	PR3	0.3	181.0

Date June 2010

Project 1-09-4135



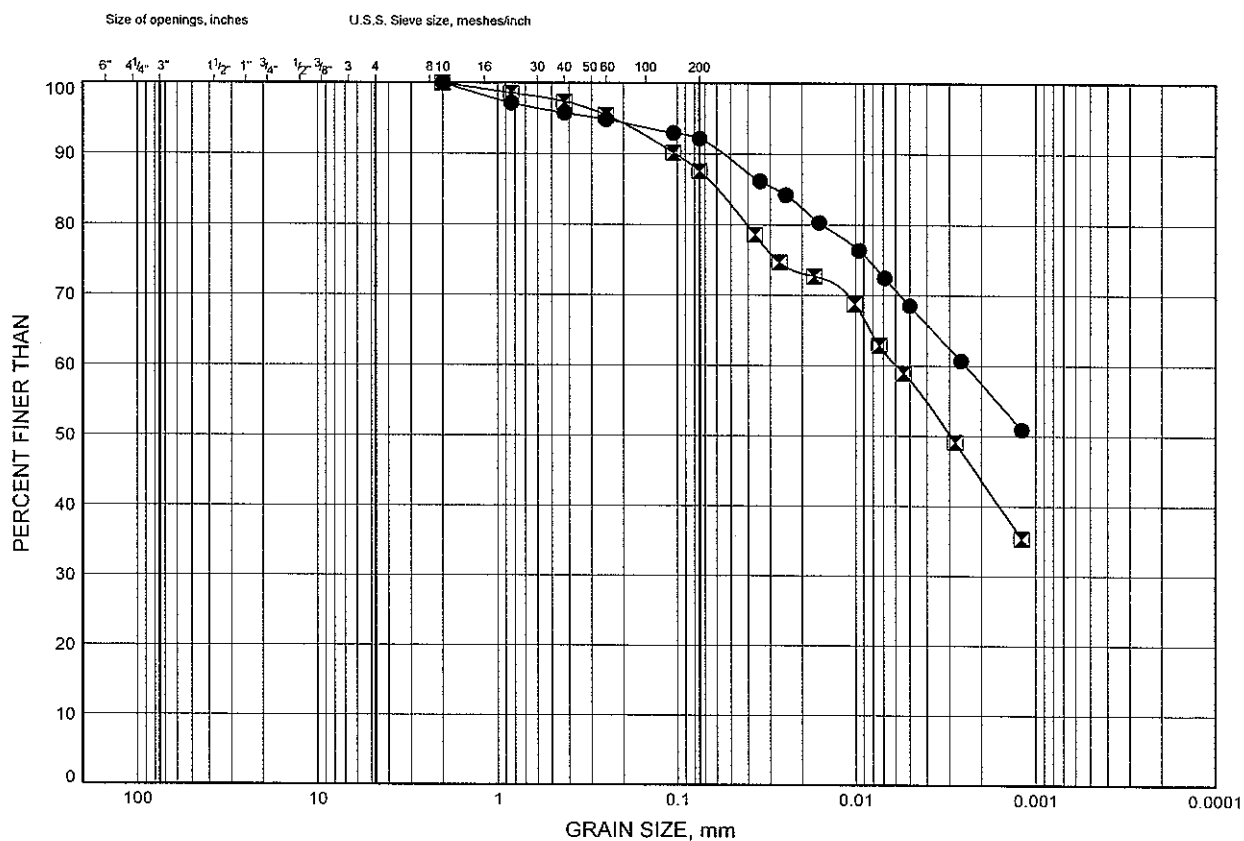
Prep'd DB

Chkd. MP

# GRAIN SIZE DISTRIBUTION

FIGURE B3

## FILL - Silty Clay

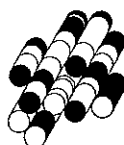


COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	PR2	1.0	180.7
☒	PR4	1.7	180.5

Date June 2010

Project 1-09-4135



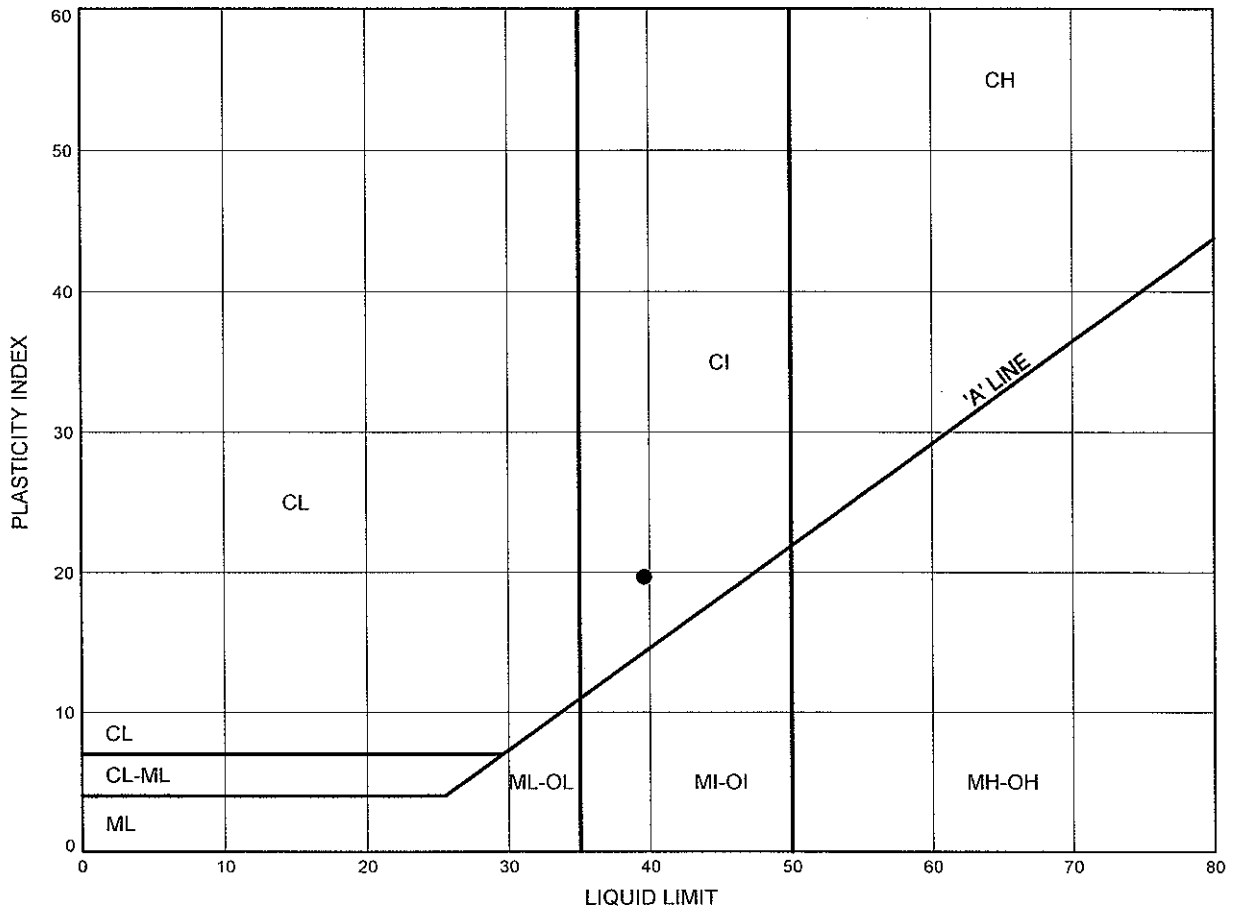
Prep'd DB

Chkd. MP

# ATTERBERG LIMITS TEST RESULTS

FIGURE B4

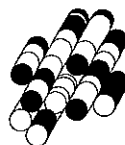
## FILL - Silty Clay



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	PR4	1.7	180.5

Date June 2010

Project 1-09-4135



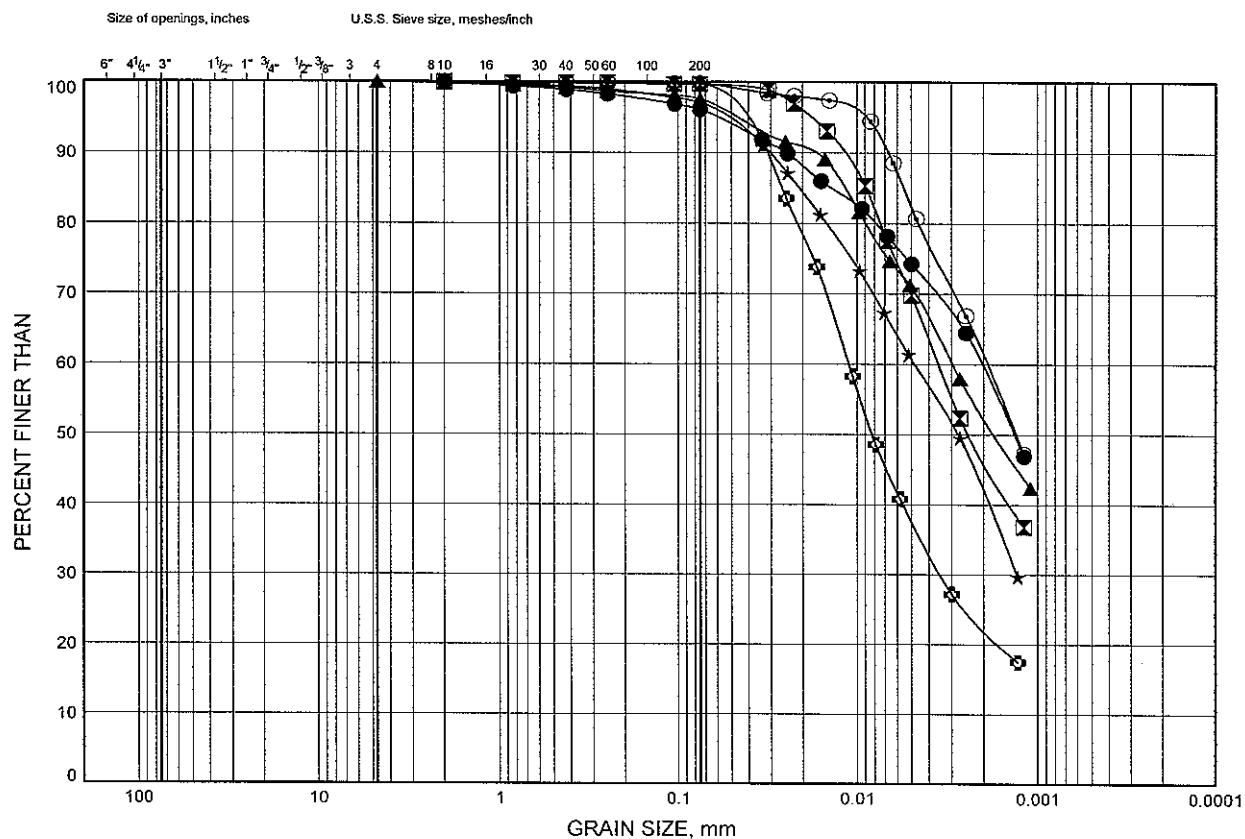
Prep'd DB

Chkd. MP

# GRAIN SIZE DISTRIBUTION

FIGURE B5

## SILTY CLAY



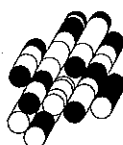
COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL    BOREHOLE    DEPTH (m)    ELEVATION (m)

●	PR1	2.5	179.2
⊠	PR1	4.0	177.7
▲	PR1	6.3	175.4
★	PR1	7.8	173.9
⊙	PR1	10.9	170.8
⊛	PR1	13.9	167.8

Date June 2010

Project 1-09-4135



Prep'd DB

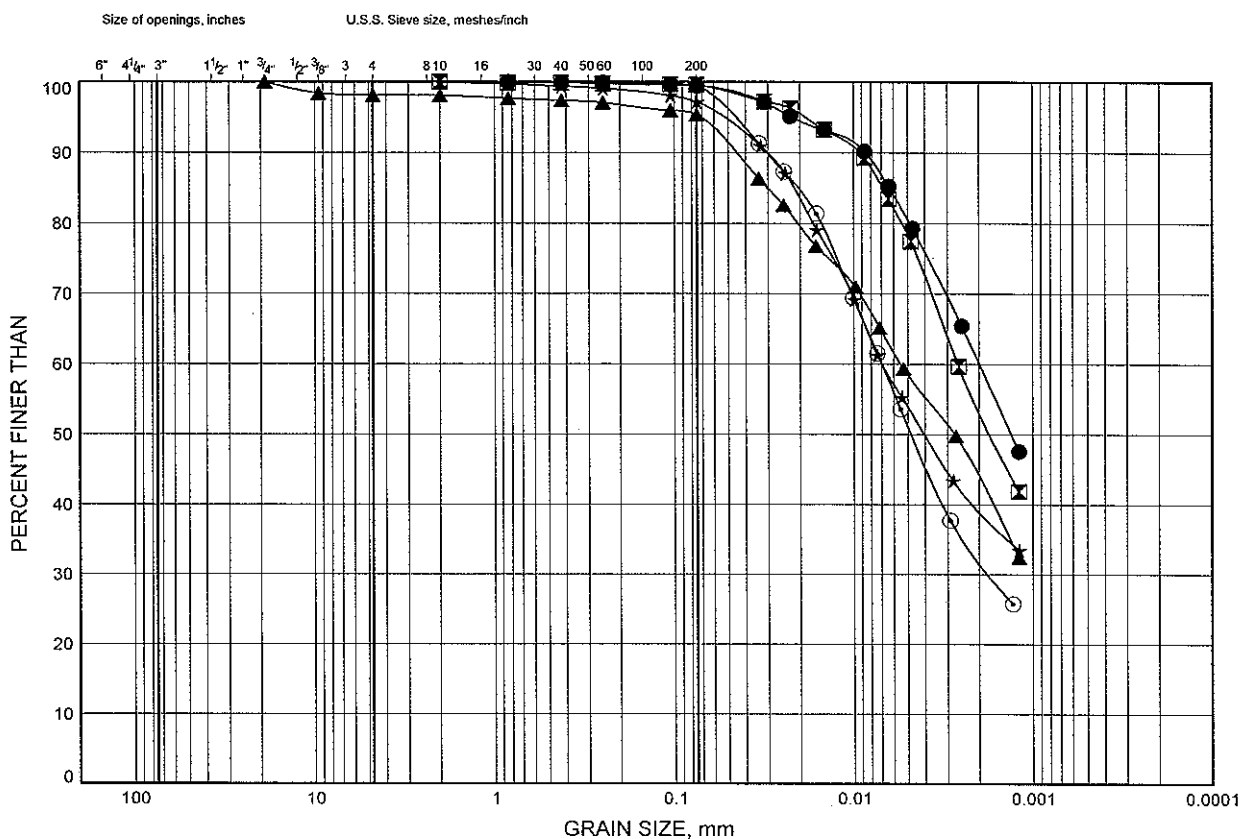
Chkd. MP



# GRAIN SIZE DISTRIBUTION

FIGURE B6

## SILTY CLAY

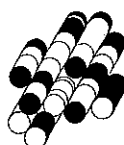


COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	PR2	1.7	180.0
⊠	PR2	3.2	178.5
▲	PR2	6.3	175.4
★	PR2	9.3	172.4
⊙	PR2	13.9	167.8

Date June 2010

Project 1-09-4135



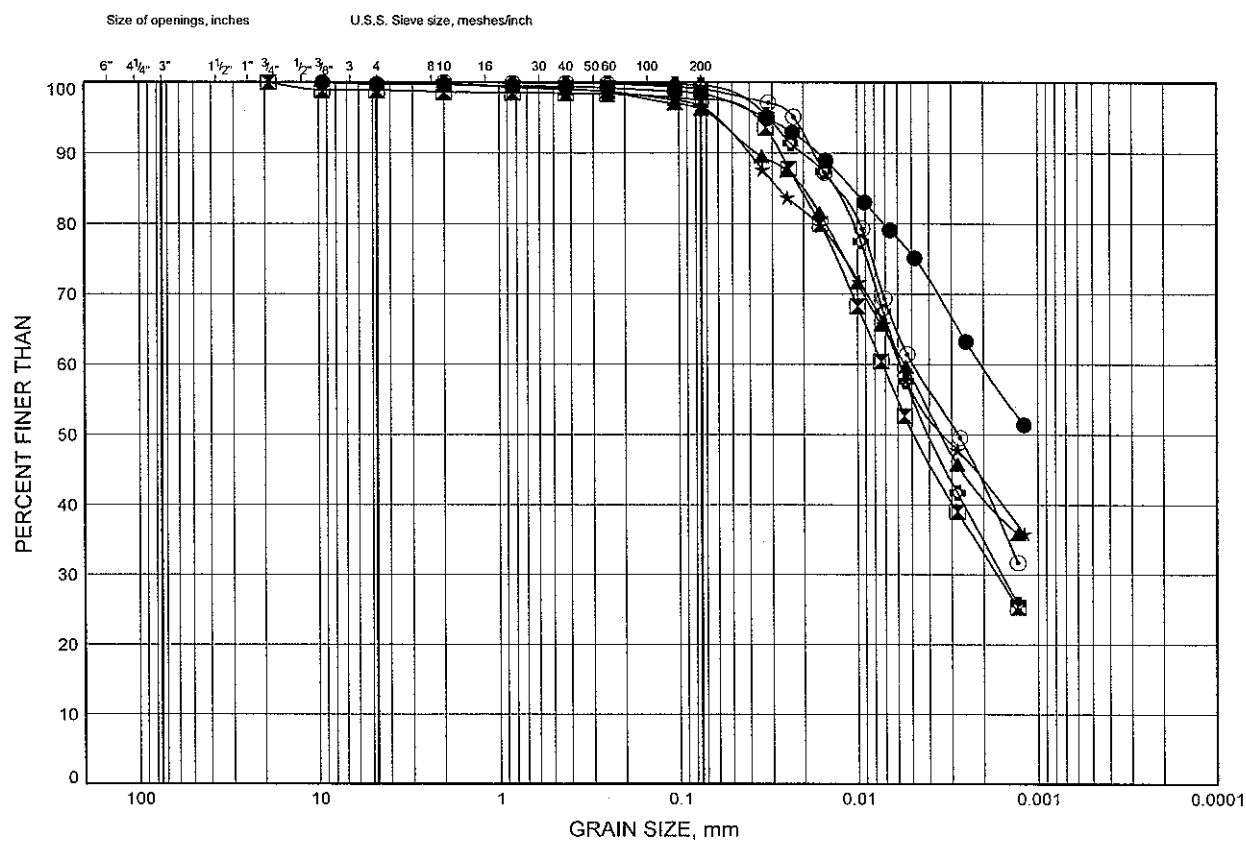
Prep'd DB

Chkd. MP

# GRAIN SIZE DISTRIBUTION

FIGURE B7

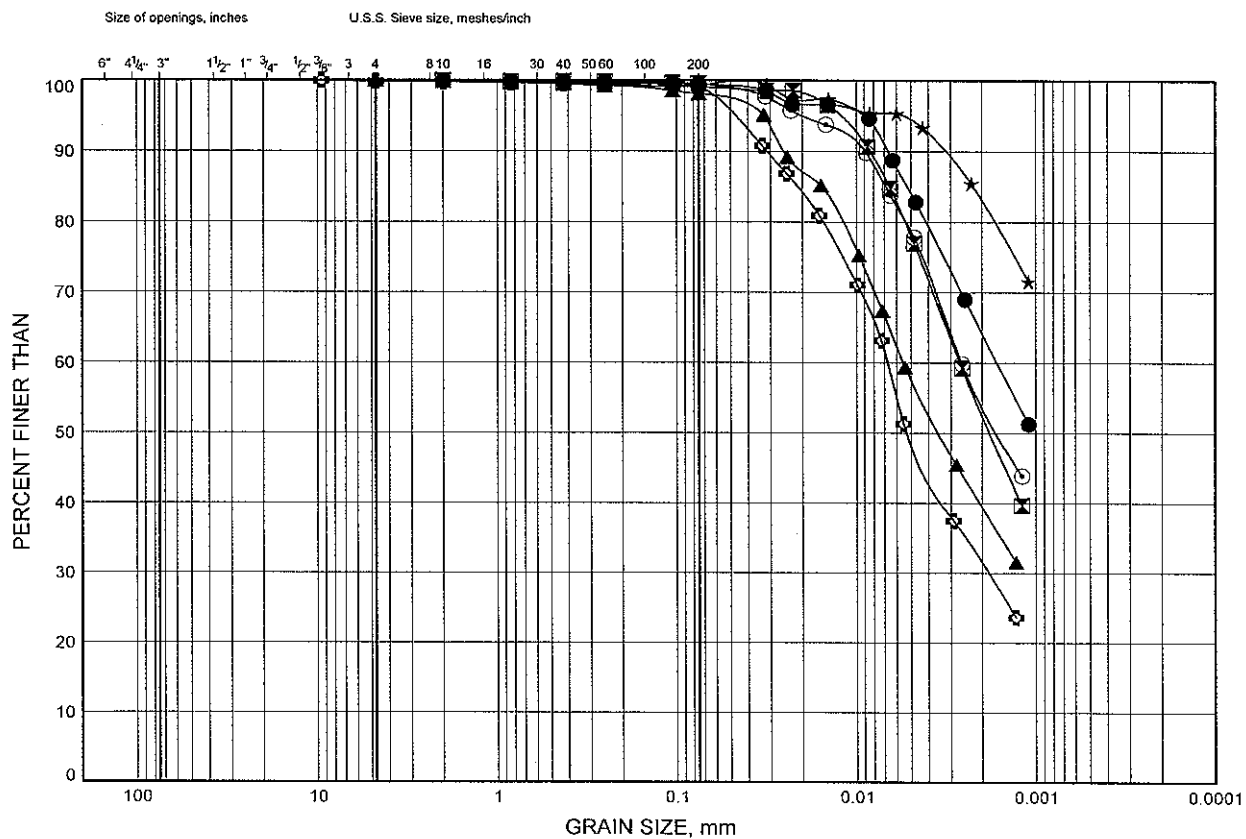
## SILTY CLAY



# GRAIN SIZE DISTRIBUTION

FIGURE B8

## SILTY CLAY

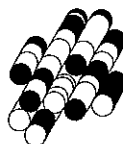


COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	PR4	2.5	179.7
⊠	PR4	4.0	178.2
▲	PR4	7.8	174.4
★	PR4	10.9	171.3
⊙	PR5	2.5	178.7
⊗	PR5	4.0	177.2

Date June 2010

Project 1-09-4135



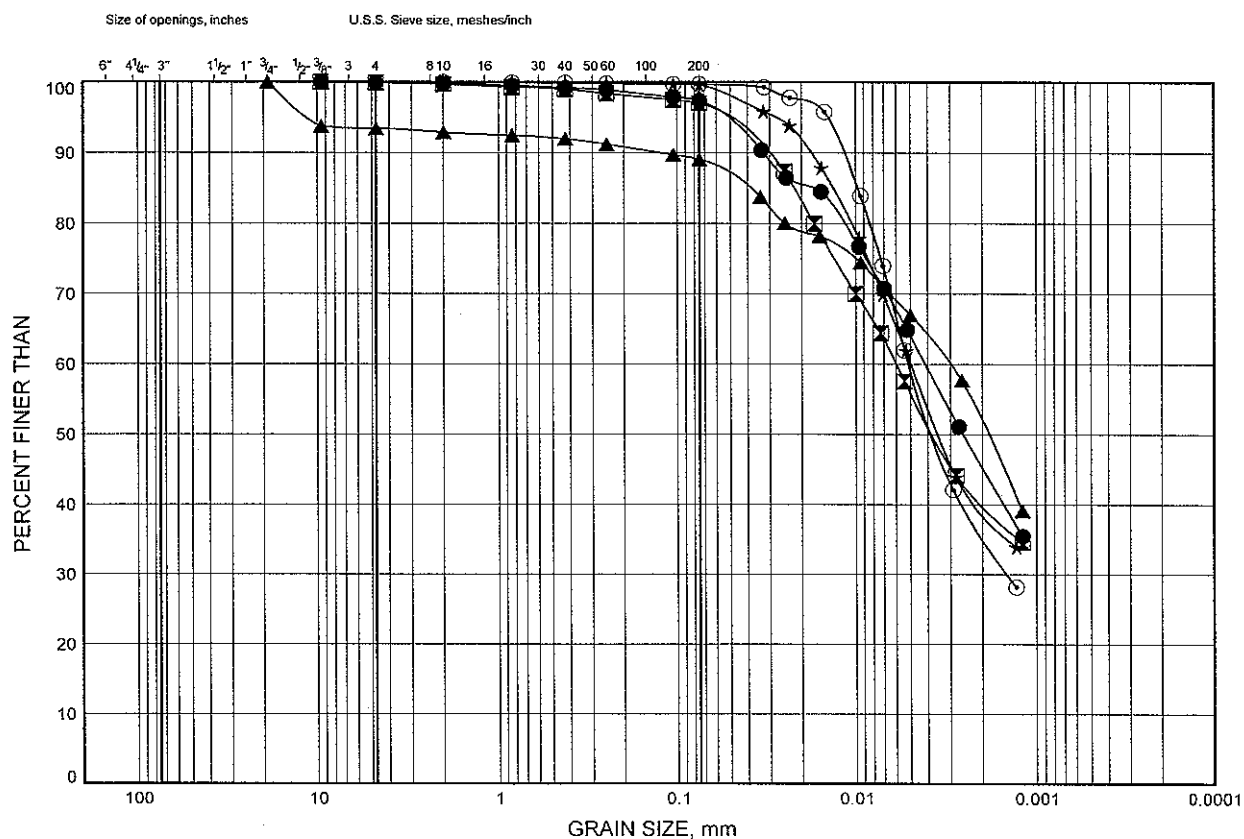
Prep'd DB

Chkd. MP

# GRAIN SIZE DISTRIBUTION

FIGURE B9

## SILTY CLAY



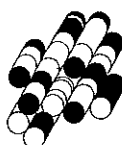
COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL    BOREHOLE    DEPTH (m)    ELEVATION (m)

●	PR5	6.3	174.9
⊠	PR5	7.8	173.4
▲	PR5	9.3	171.9
★	PR5	12.4	168.8
⊙	PR5	13.9	167.3

Date June 2010

Project 1-09-4135



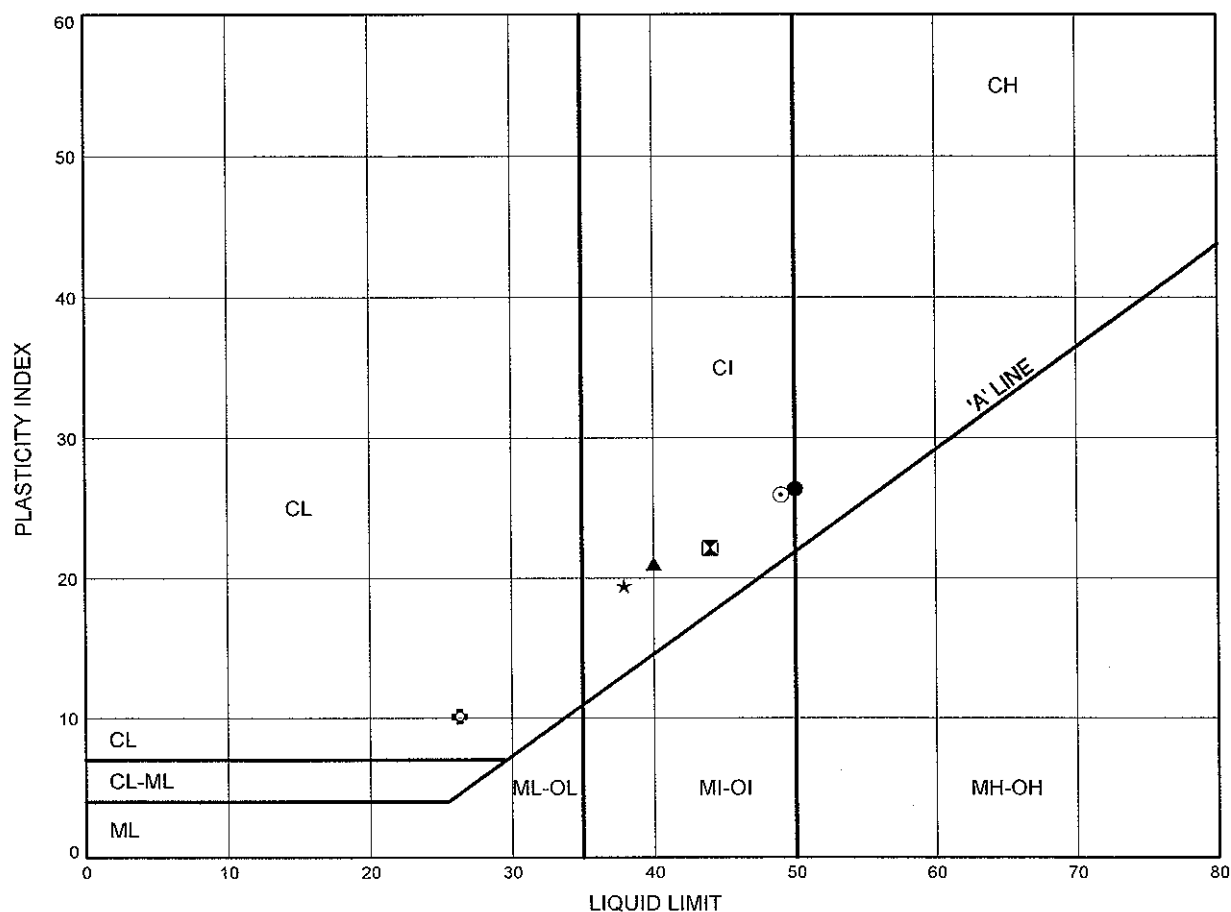
Prep'd DB

Chkd. MP

# ATTERBERG LIMITS TEST RESULTS

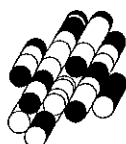
FIGURE B10

## SILTY CLAY



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	PR1	2.5	179.2
⊠	PR1	4.0	177.7
▲	PR1	6.3	175.4
★	PR1	7.8	173.9
⊙	PR1	10.9	170.8
⊛	PR1	13.9	167.8

Date June 2010  
Project 1-09-4135

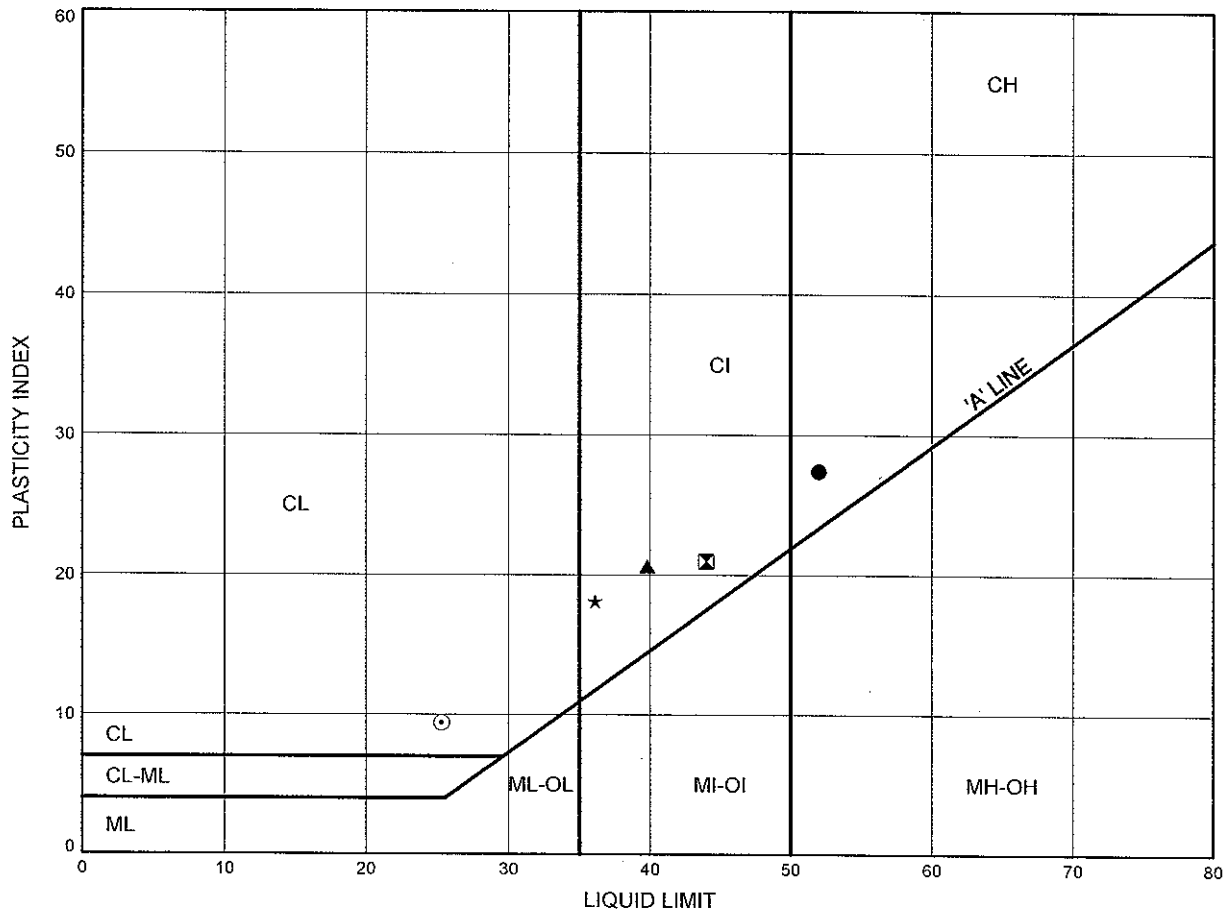


Prep'd DB  
Chkd. MP

# ATTERBERG LIMITS TEST RESULTS

FIGURE B11

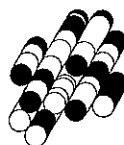
## SILTY CLAY



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	PR2	1.7	180.0
⊠	PR2	3.2	178.5
▲	PR2	6.3	175.4
★	PR2	9.3	172.4
⊙	PR2	13.9	167.8

Date June 2010

Project 1-09-4135



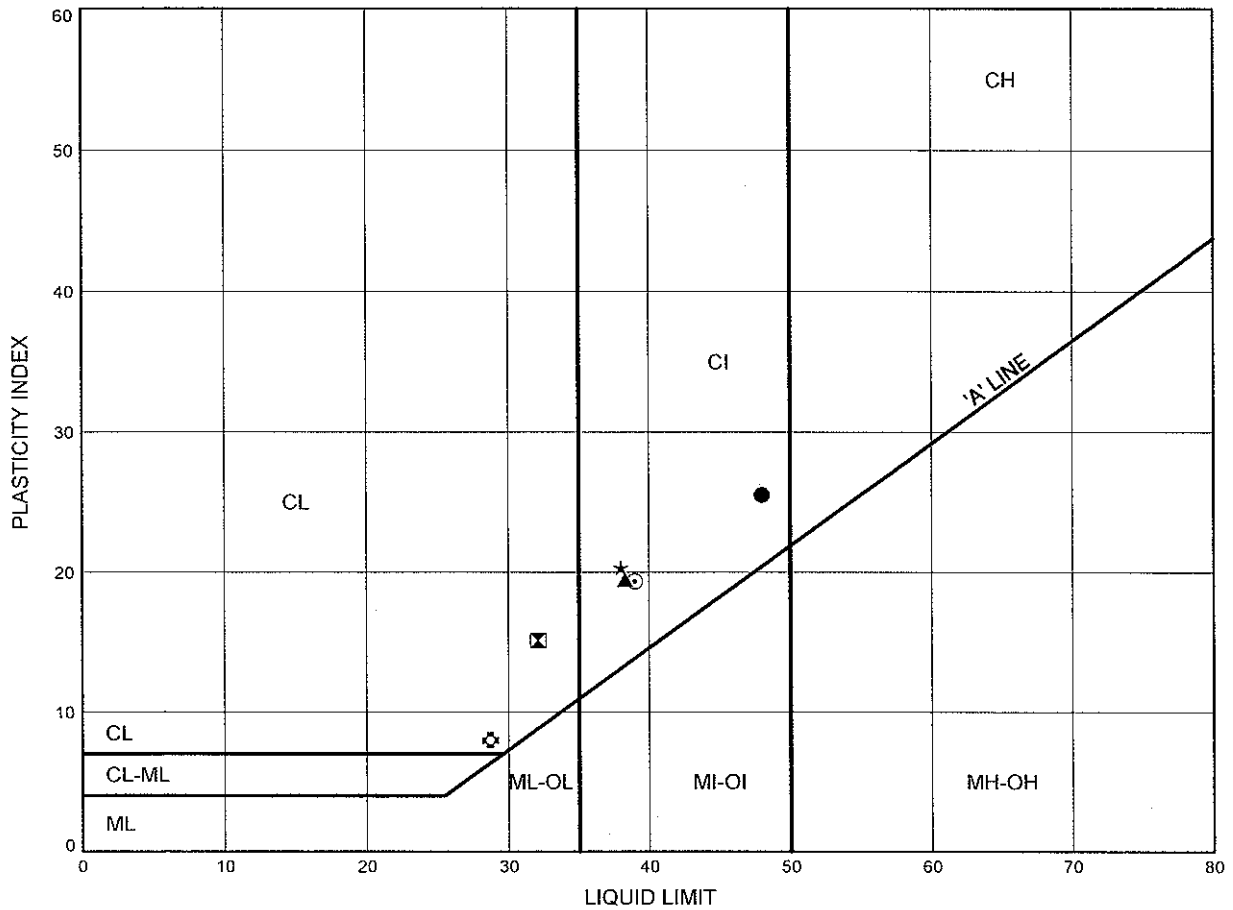
Prep'd DB

Chkd. MP

# ATTERBERG LIMITS TEST RESULTS

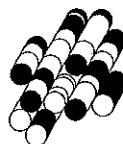
FIGURE B12

## SILTY CLAY



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	PR3	2.5	178.8
⊠	PR3	4.0	177.3
▲	PR3	6.3	175.0
★	PR3	7.8	173.5
⊙	PR3	9.3	172.0
⊛	PR3	12.4	168.9

Date June 2010  
Project 1-09-4135

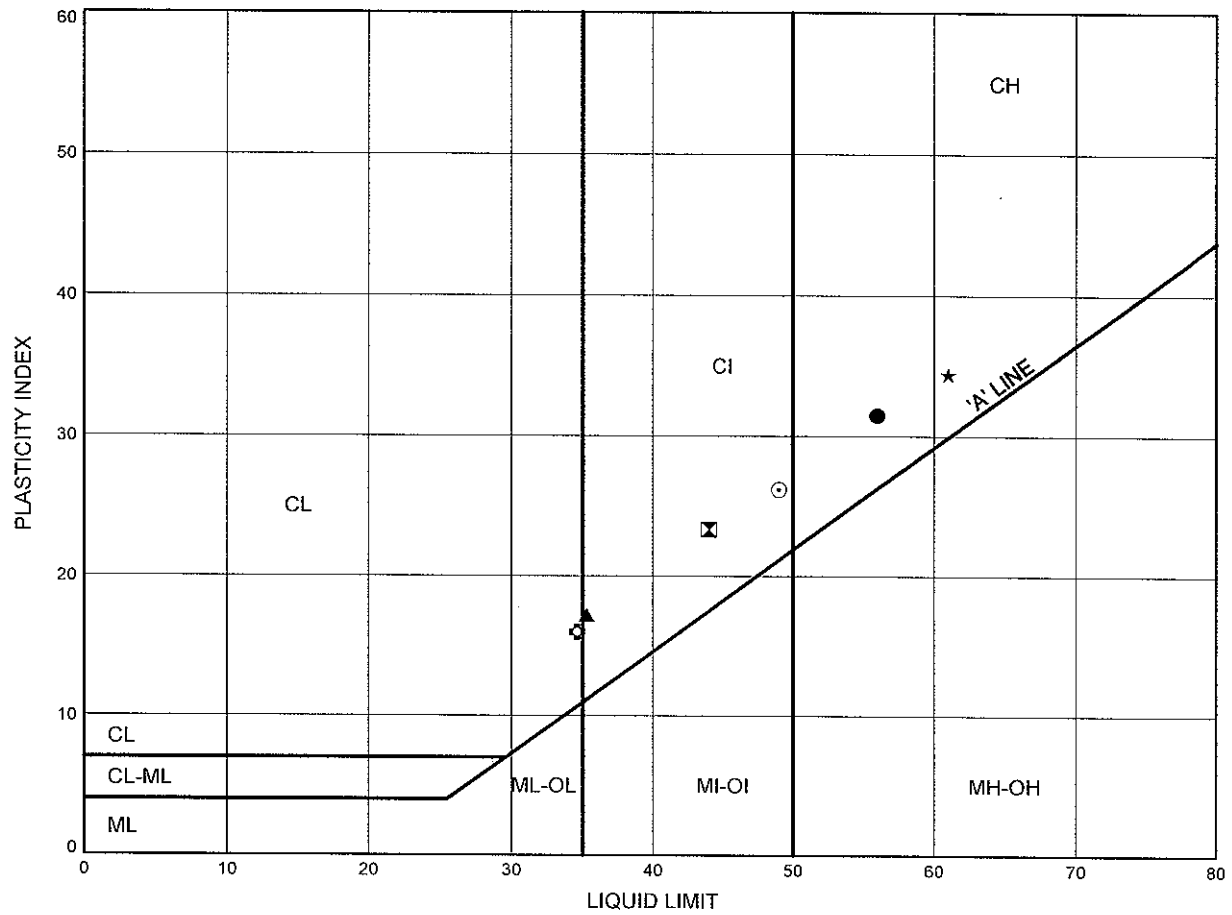


Prep'd DB  
Chkd. MP

# ATTERBERG LIMITS TEST RESULTS

FIGURE B13

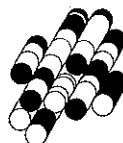
## SILTY CLAY



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	PR4	2.5	179.7
⊠	PR4	4.0	178.2
▲	PR4	7.8	174.4
★	PR4	10.9	171.3
⊙	PR5	2.5	178.7
⊛	PR5	4.0	177.2

Date June 2010

Project 1-09-4135



Prep'd DB

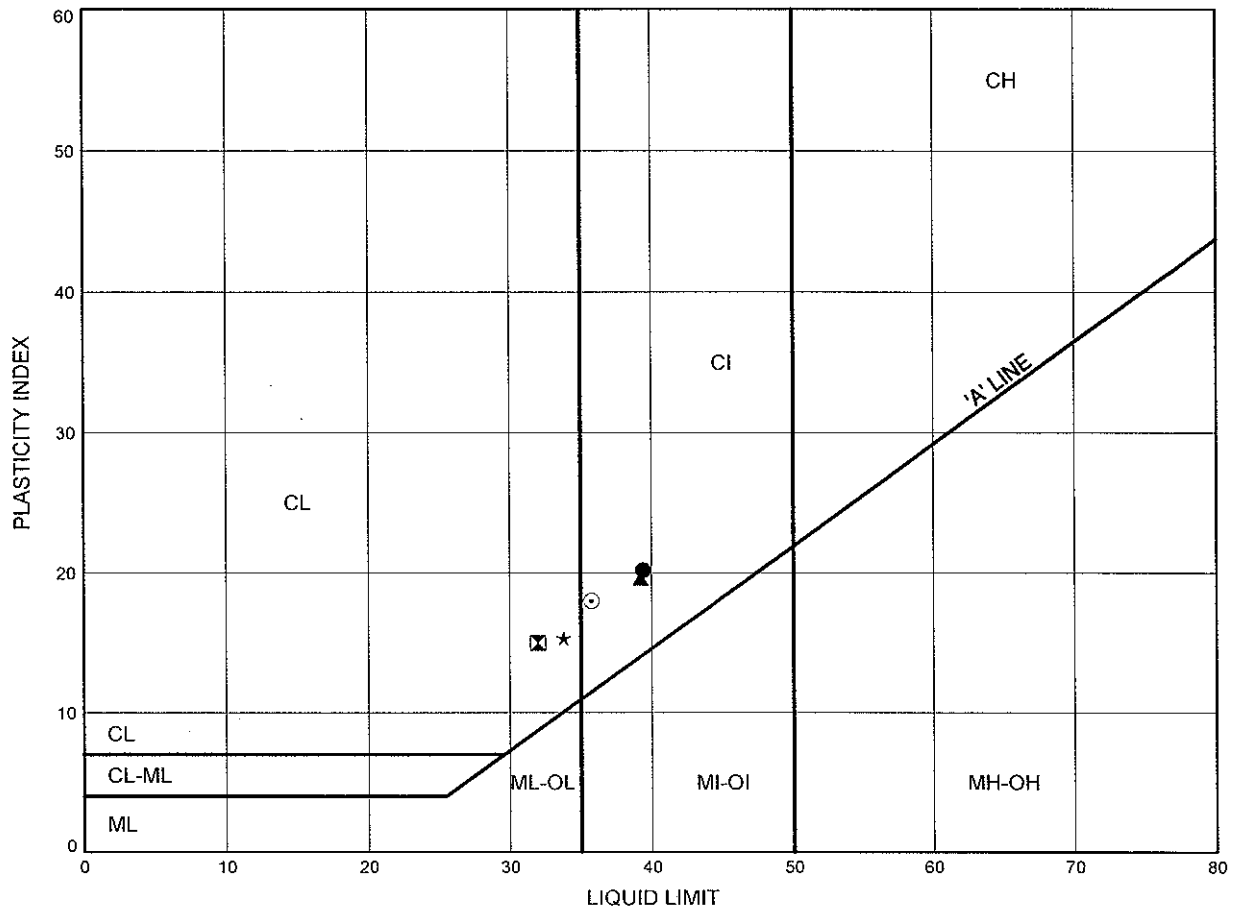
Chkd. MP



# ATTERBERG LIMITS TEST RESULTS

FIGURE B14

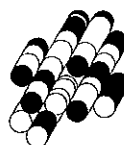
## SILTY CLAY



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	PR5	6.3	174.9
⊠	PR5	7.8	173.4
▲	PR5	9.3	171.9
★	PR5	12.4	168.8
⊙	PR5	13.9	167.3

Date June 2010

Project 1-09-4135



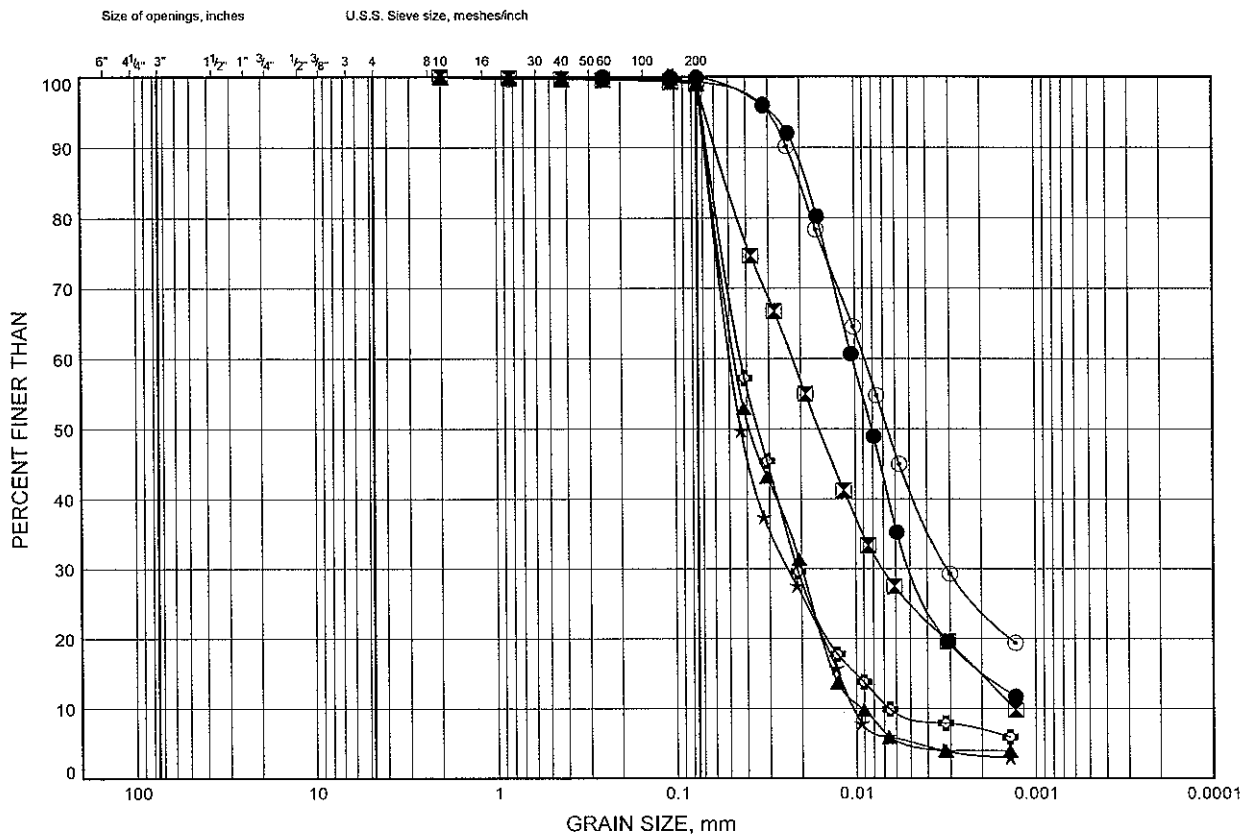
Prep'd DB

Chkd. MP

# GRAIN SIZE DISTRIBUTION

FIGURE B15

## SILT



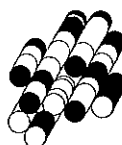
COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL    BOREHOLE    DEPTH (m)    ELEVATION (m)

●	PR1	15.4	166.3
⊠	PR1	17.0	164.7
▲	PR2	17.0	164.7
★	PR3	15.4	165.9
⊙	PR4	15.4	166.8
⊛	PR5	15.4	165.8

Date June 2010

Project 1-09-4135



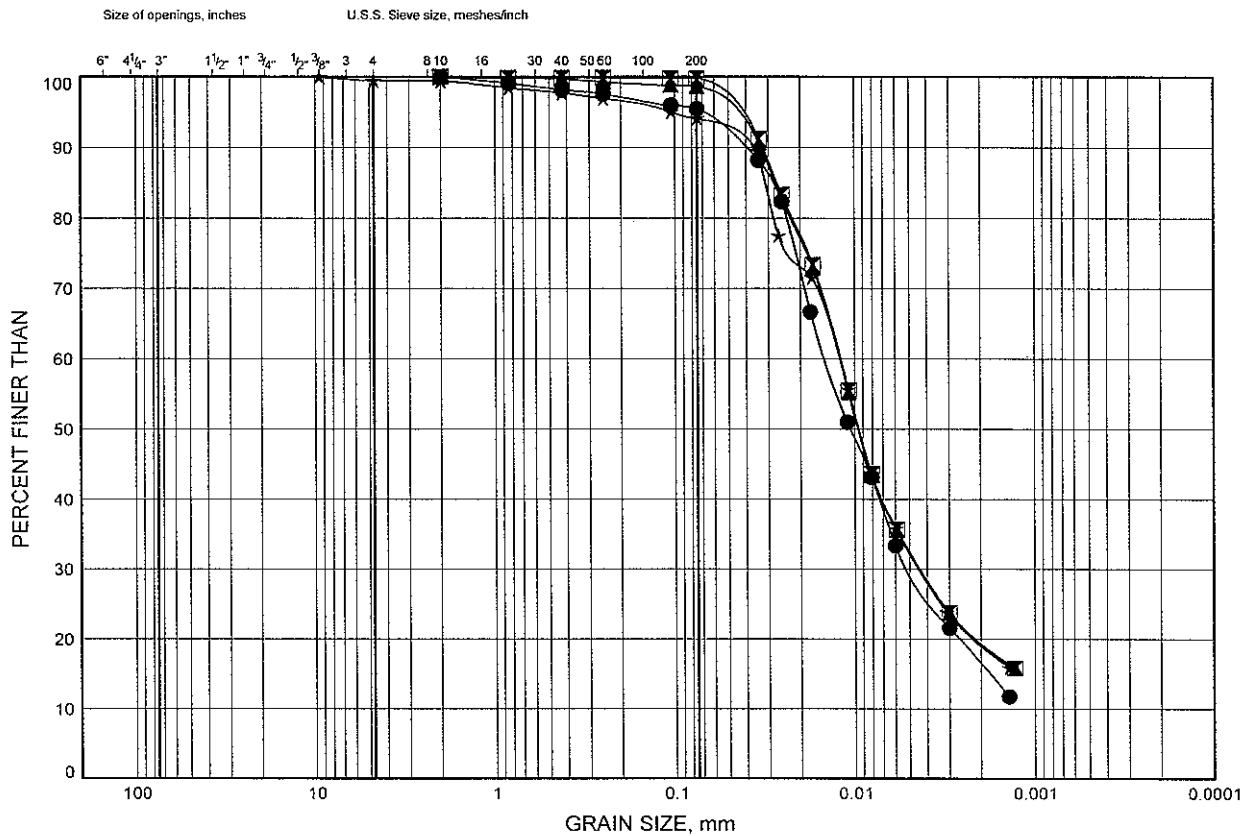
Prep'd DB

Chkd. MP

# GRAIN SIZE DISTRIBUTION

FIGURE B16

## SILTY CLAY TO CLAYEY SILT



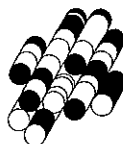
COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
--------	----------	-----------	---------------

●	PR2	20.0	161.7
⊠	PR2	23.1	158.6
▲	PR2	26.1	155.6
★	PR3	18.5	162.8

Date June 2010

Project 1-09-4135



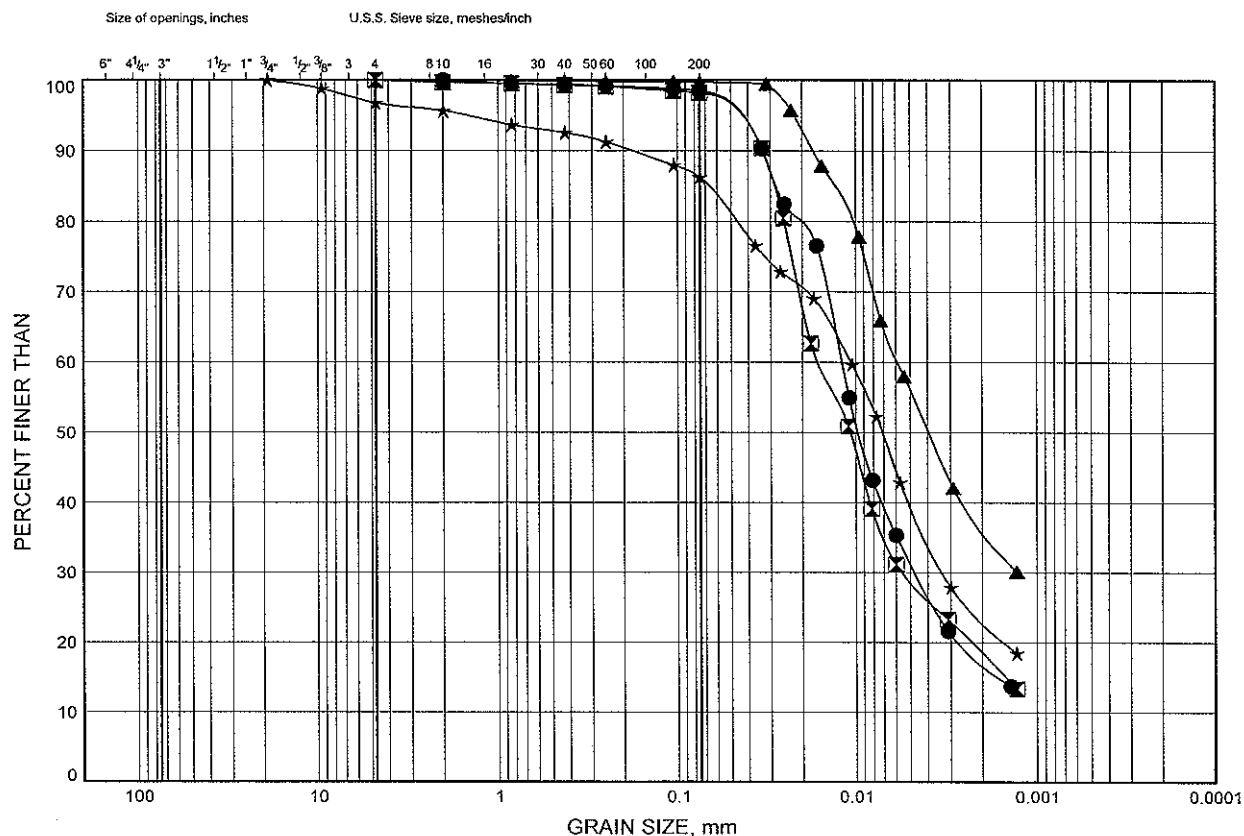
Prep'd DB

Chkd. MP

# GRAIN SIZE DISTRIBUTION

FIGURE B17

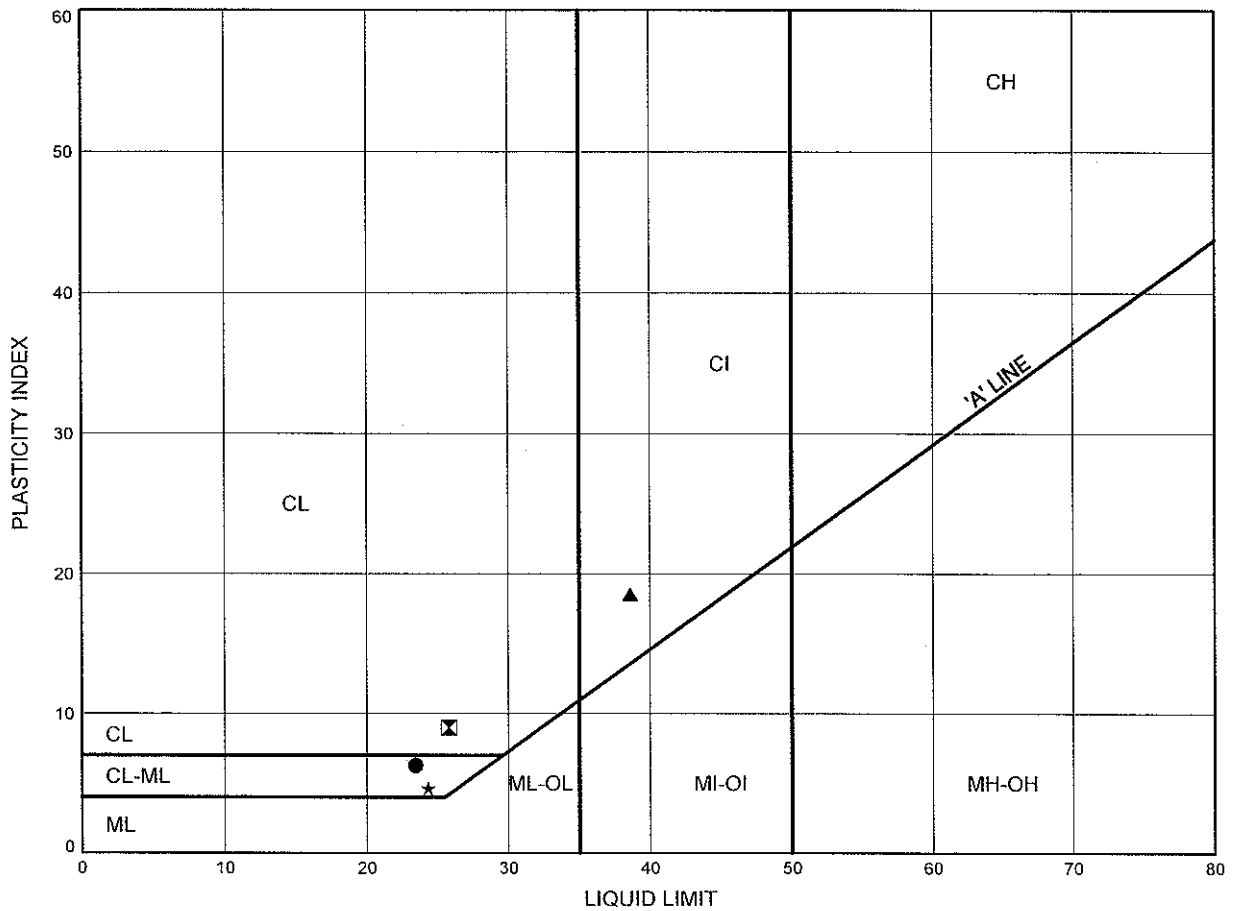
## SILTY CLAY TO CLAYEY SILT



# ATTERBERG LIMITS TEST RESULTS

FIGURE B18

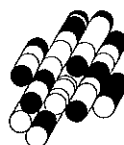
## SILTY CLAY TO CLAYEY SILT



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	PR2	20.0	161.7
⊠	PR2	23.1	158.6
▲	PR2	26.1	155.6
★	PR3	18.5	162.8

Date June 2010

Project 1-09-4135



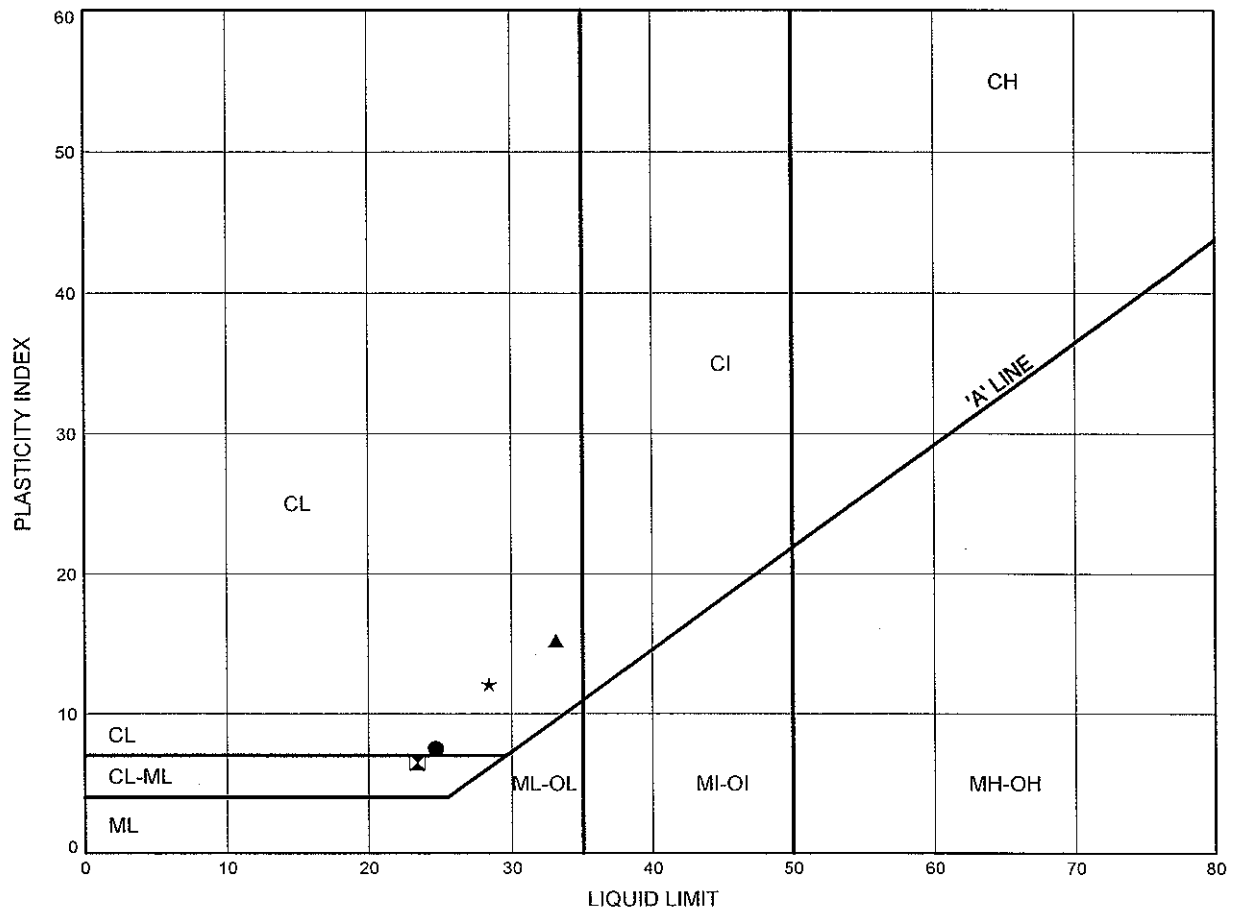
Prep'd DB

Chkd. MP

# ATTERBERG LIMITS TEST RESULTS

FIGURE B19

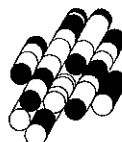
## SILTY CLAY TO CLAYEY SILT



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	PR3	23.1	158.2
⊠	PR4	21.5	160.7
▲	PR4	24.6	157.6
★	PR4	27.6	154.6

Date June 2010

Project 1-09-4135



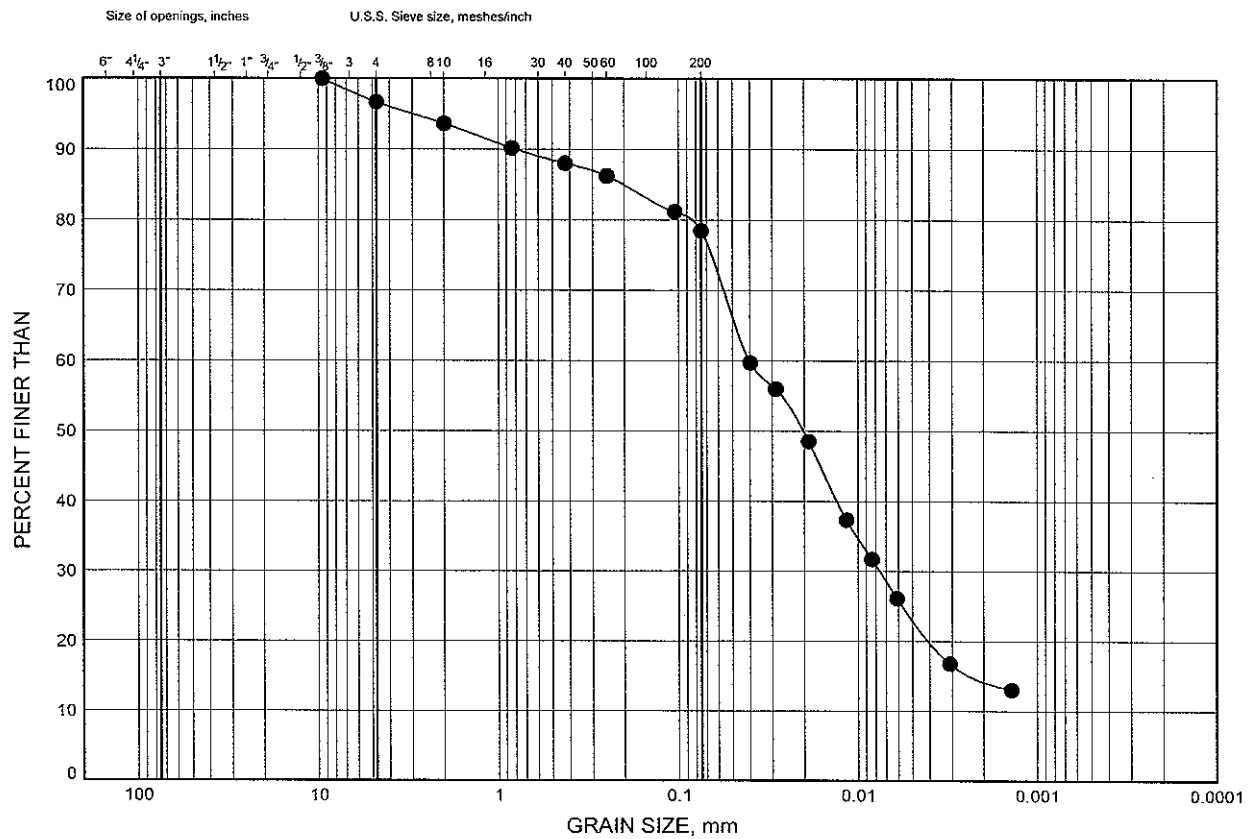
Prep'd DB

Chkd. MP

# GRAIN SIZE DISTRIBUTION

FIGURE B20

## CLAYEY SILT TILL

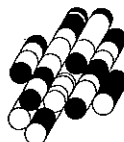


COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	PR3	27.6	153.7

Date June 2010

Project 1-09-4135



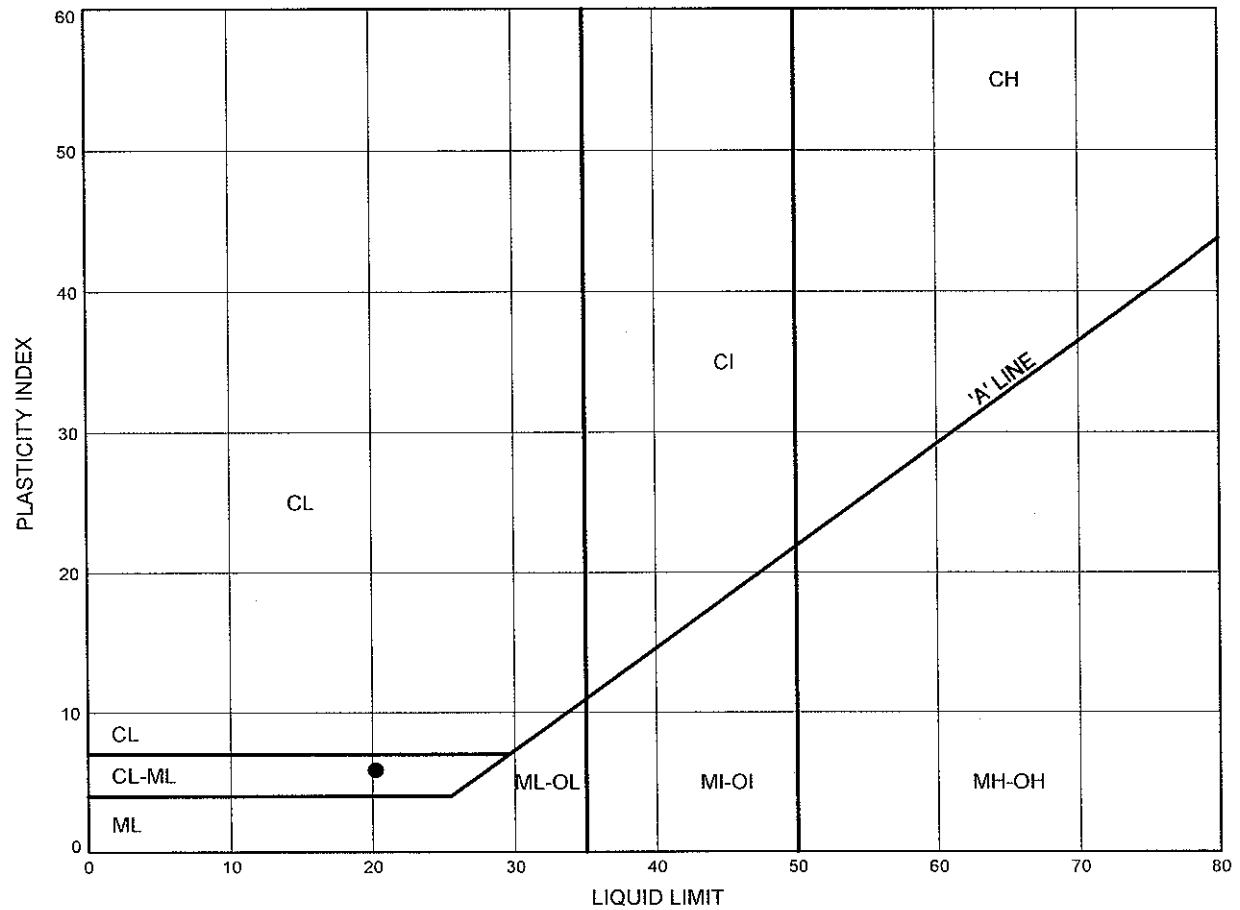
Prep'd DB

Chkd. MP

# ATTERBERG LIMITS TEST RESULTS

FIGURE B21

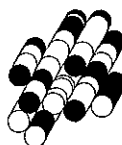
## CLAYEY SILT TILL



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	PR3	27.6	153.7

Date June 2010

Project 1-09-4135



Prep'd DB

Chkd. MP



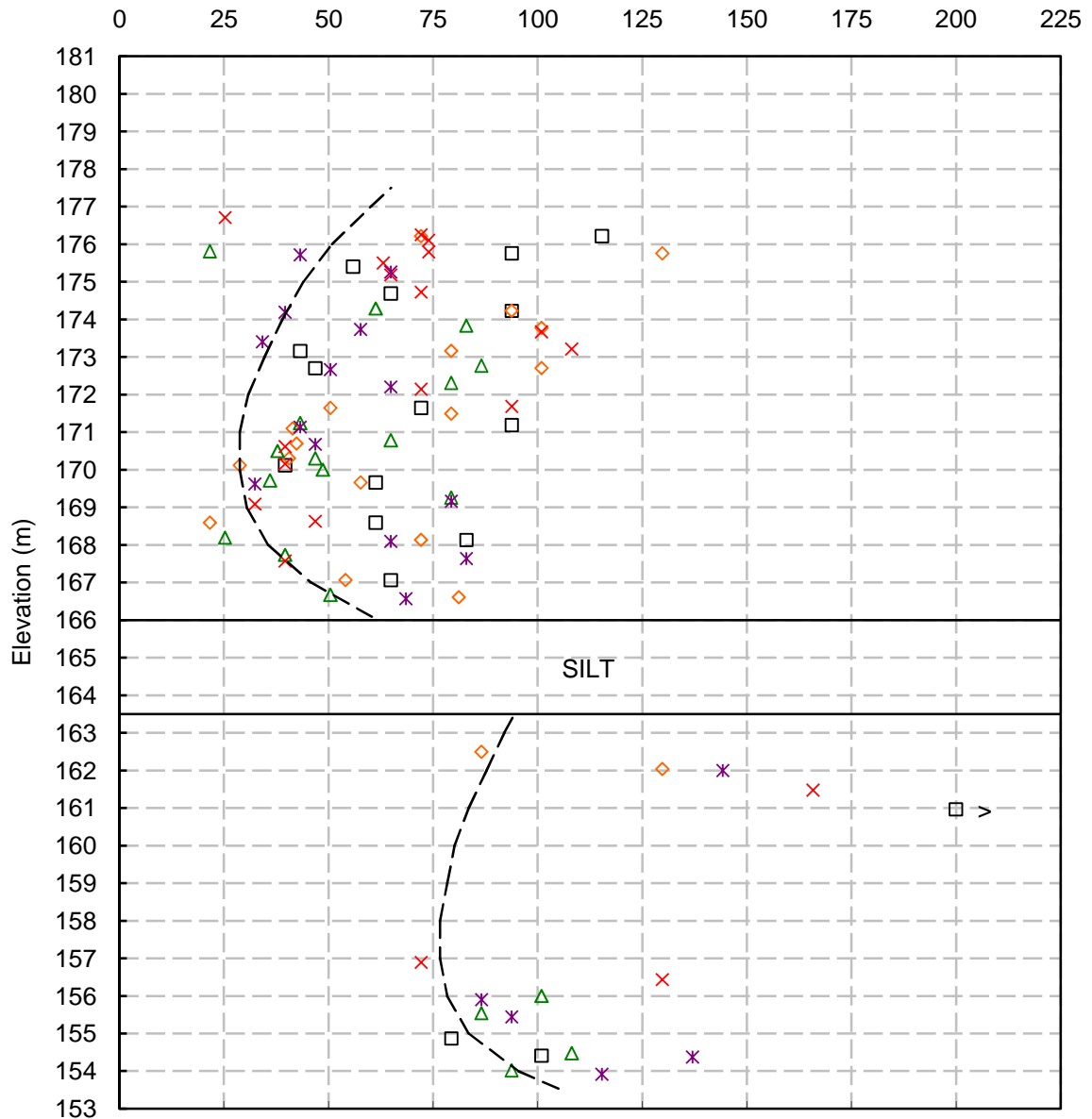
# CORRECTED UNDRAINED SHEAR STRENGTH

FIGURE B22

HWY 406 TWINNING - PORT ROBINSON ROAD

Silty Clay

Corrected Cu (kPa)



□PR1

◇PR2

△PR3

×PR4

\*PR5

## Field Shear Vane Correction

Morris & Williams (1994)

$(\mu = 1.18 \text{ EXP}(-0.08 \text{ Ip}) + 0.57)$

## Applied Correction Factors

0.72 (Elev.>177.5m)

0.90 (Elev.<177.5m)

Project No. : 1-09-4135

Date : September, 2010



**Terraprobe Inc.**

Prepared By : HW

Checked By : RA

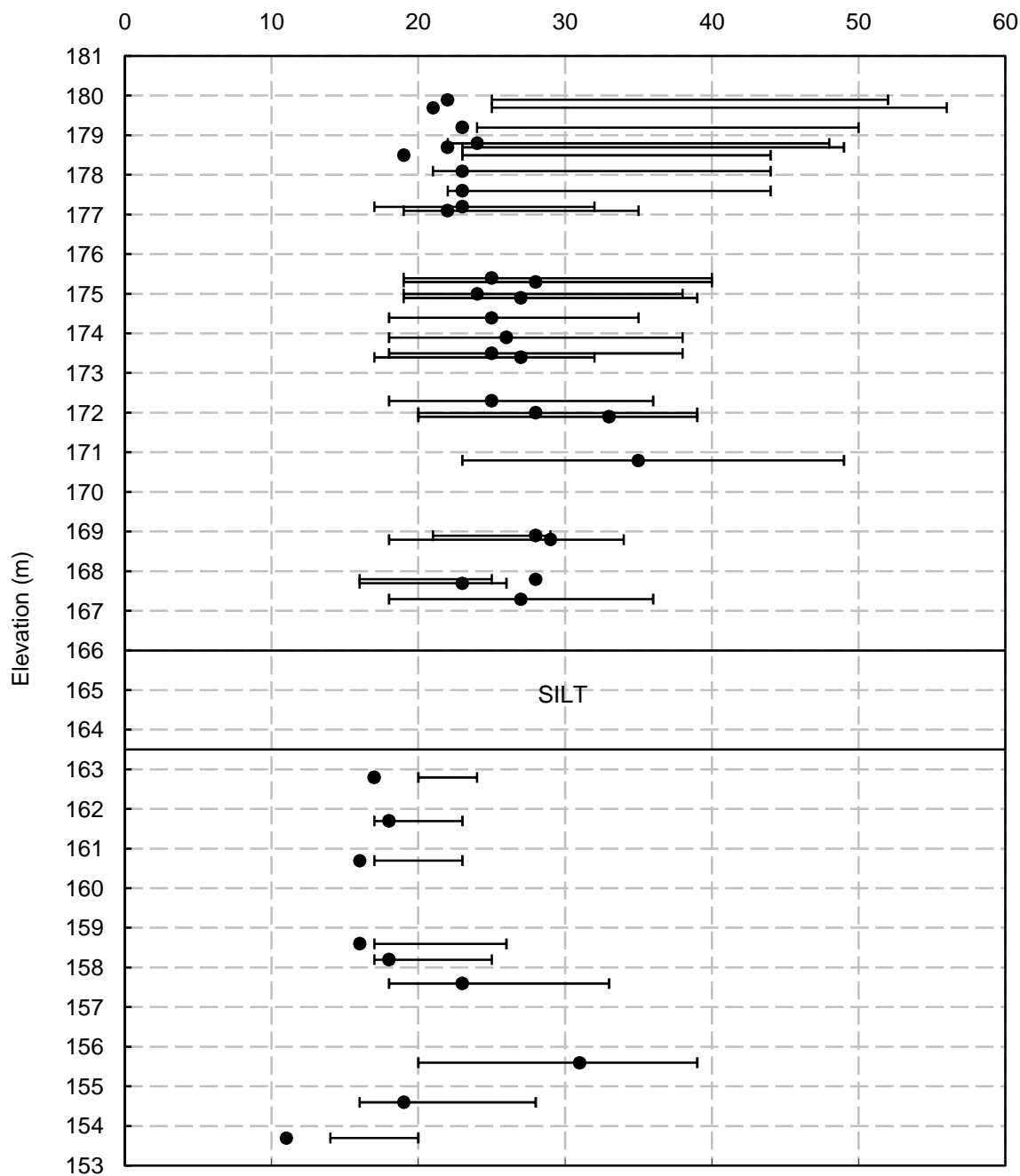
# ATTERBERG LIMITS AND WATER CONTENTS

FIGURE B23

HWY 406 TWINNING - PORT ROBINSON ROAD

Silty Clay

Atterberg Limits & Water Contents (%)



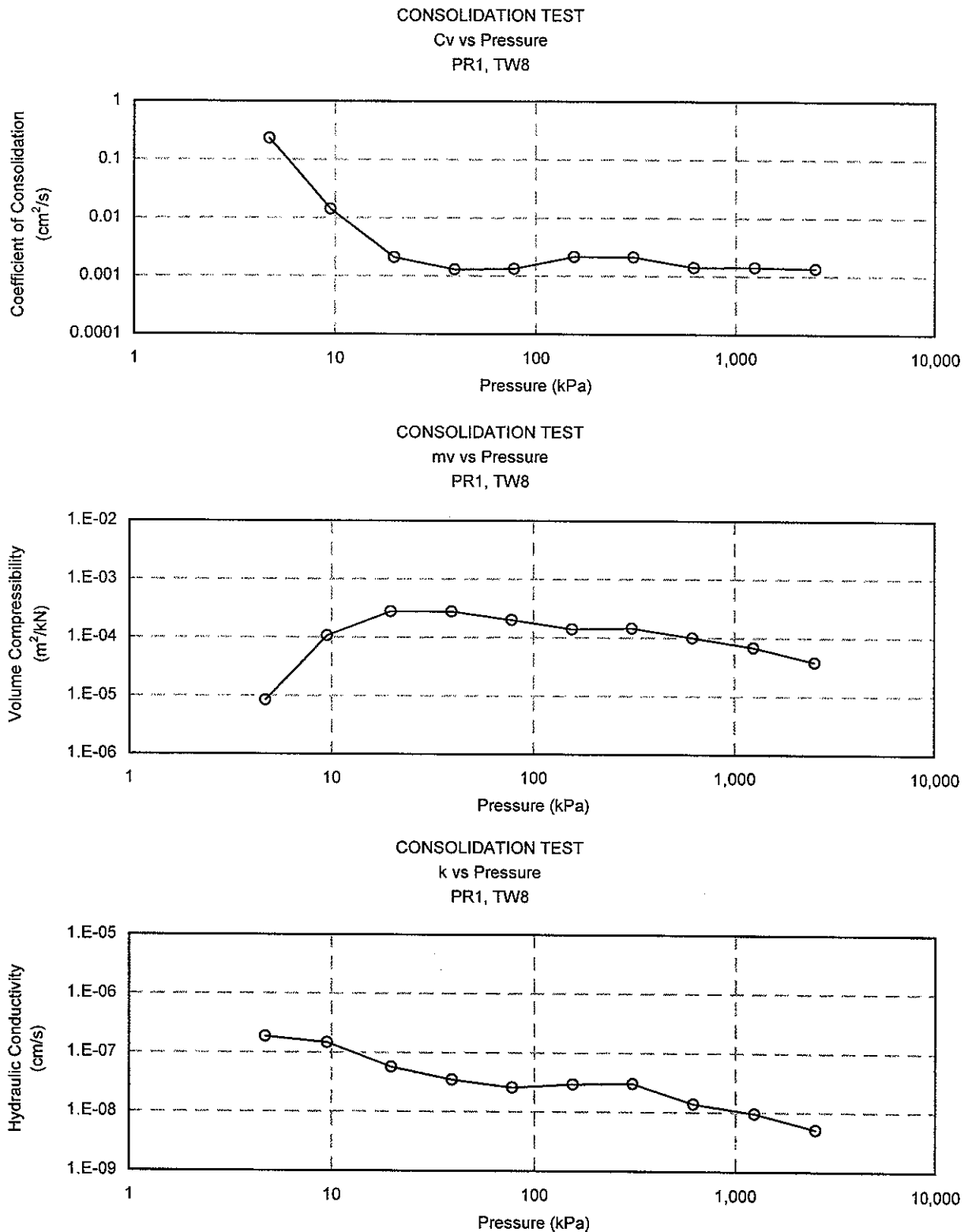
Project No. : 1-09-4135

Date : September, 2010

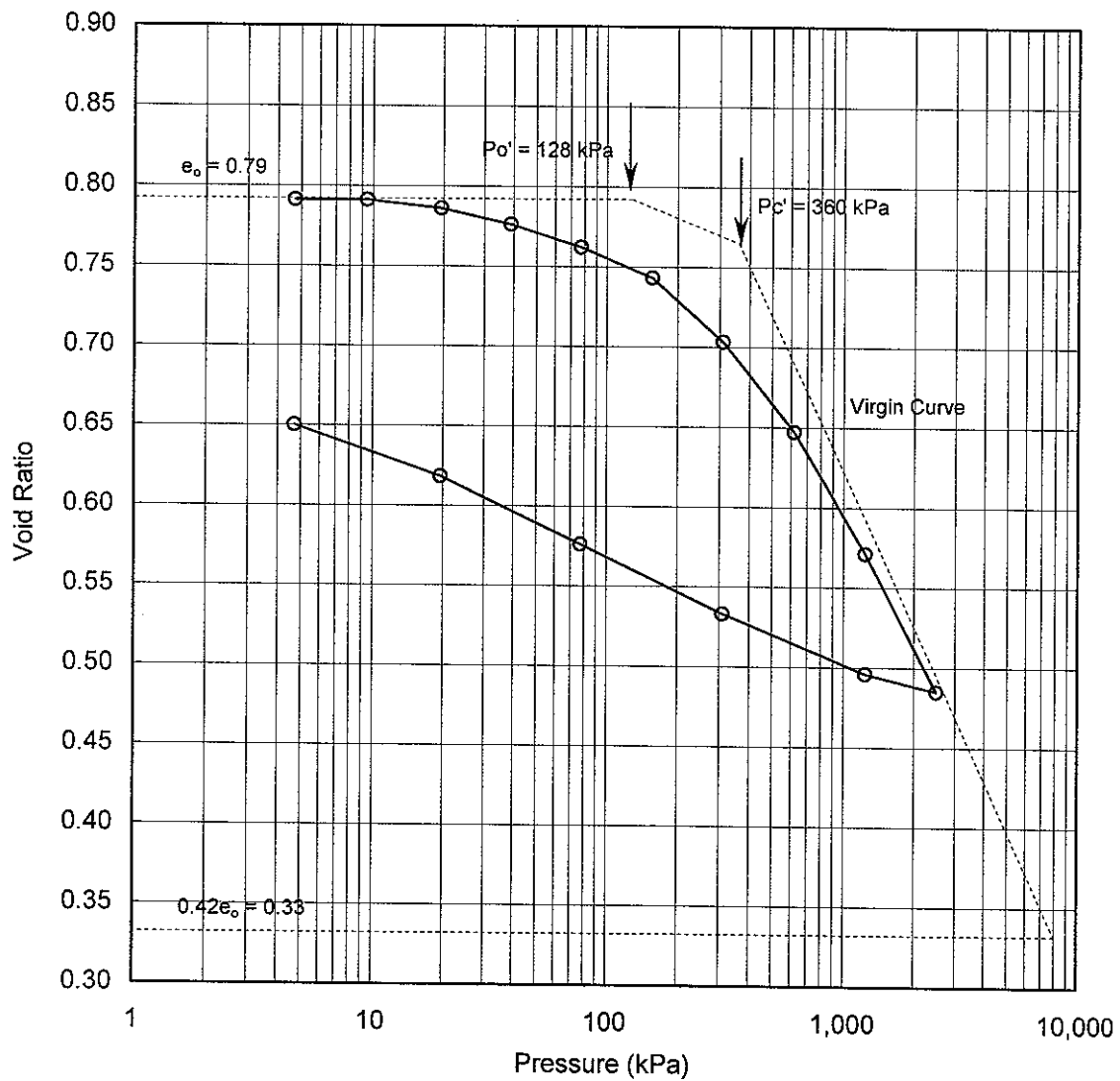


Prepared By : HW

Checked By : RA



CONSOLIDATION TEST  
e vs Pressure  
PR1, TW8



Soil Type : Silty Clay

$e_o =$	0.79	$\omega_L =$	40%	$P_o' =$	128 kPa
$\omega =$	28%	$\omega_p =$	19%	$P_c' =$	360 kPa
$\gamma =$	19.5 kN/m <sup>3</sup>	PI =	21%	Cc =	0.321
Gs =	2.78			Cr =	0.060

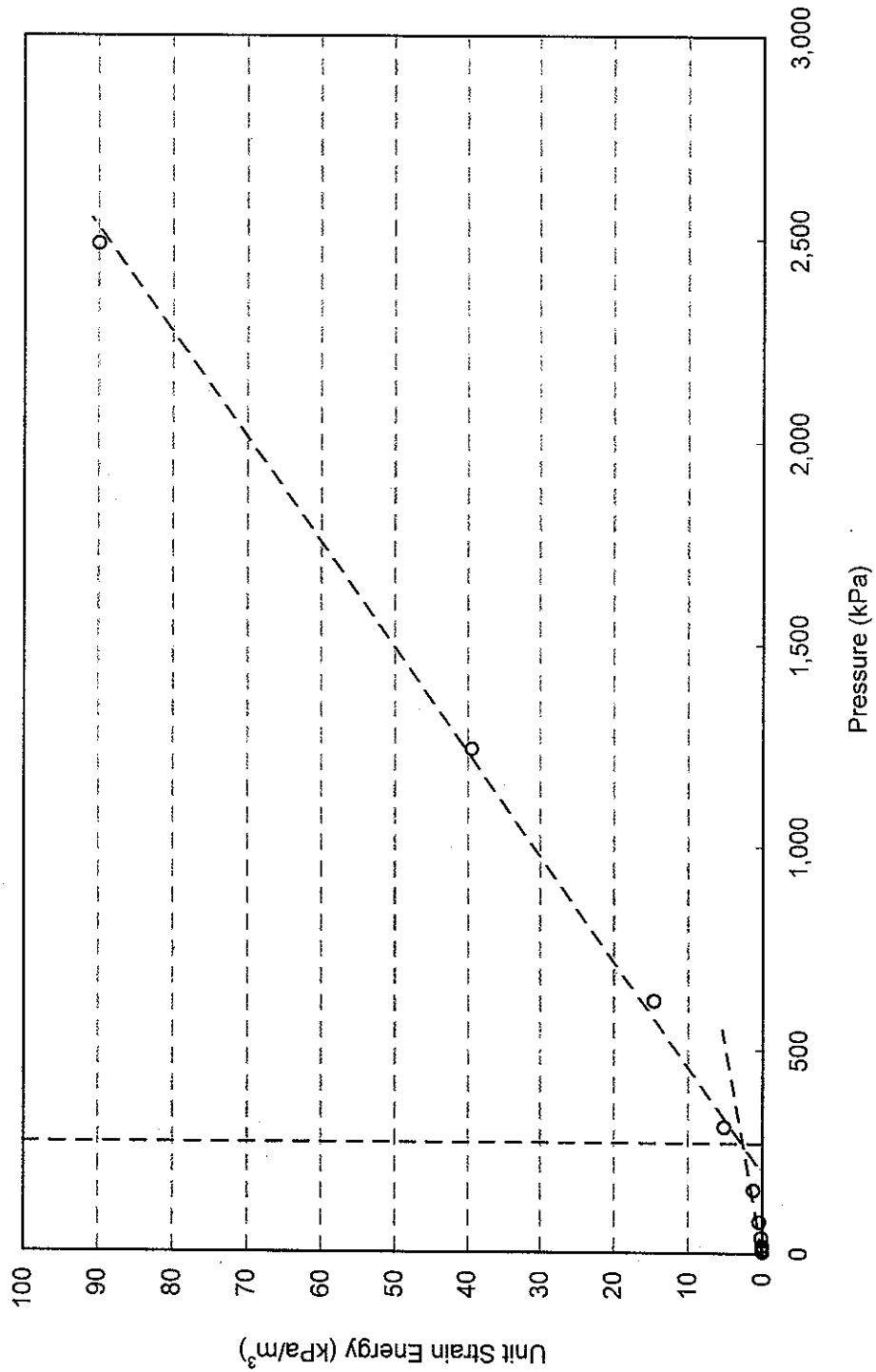
Project No. : 1-09-4135  
Date : June 2010



**Terraprobe Inc.**

Prepared By : HW  
Checked By : RA

CONSOLIDATION TEST  
Unit Strain Energy vs Pressure  
PR1, TW8

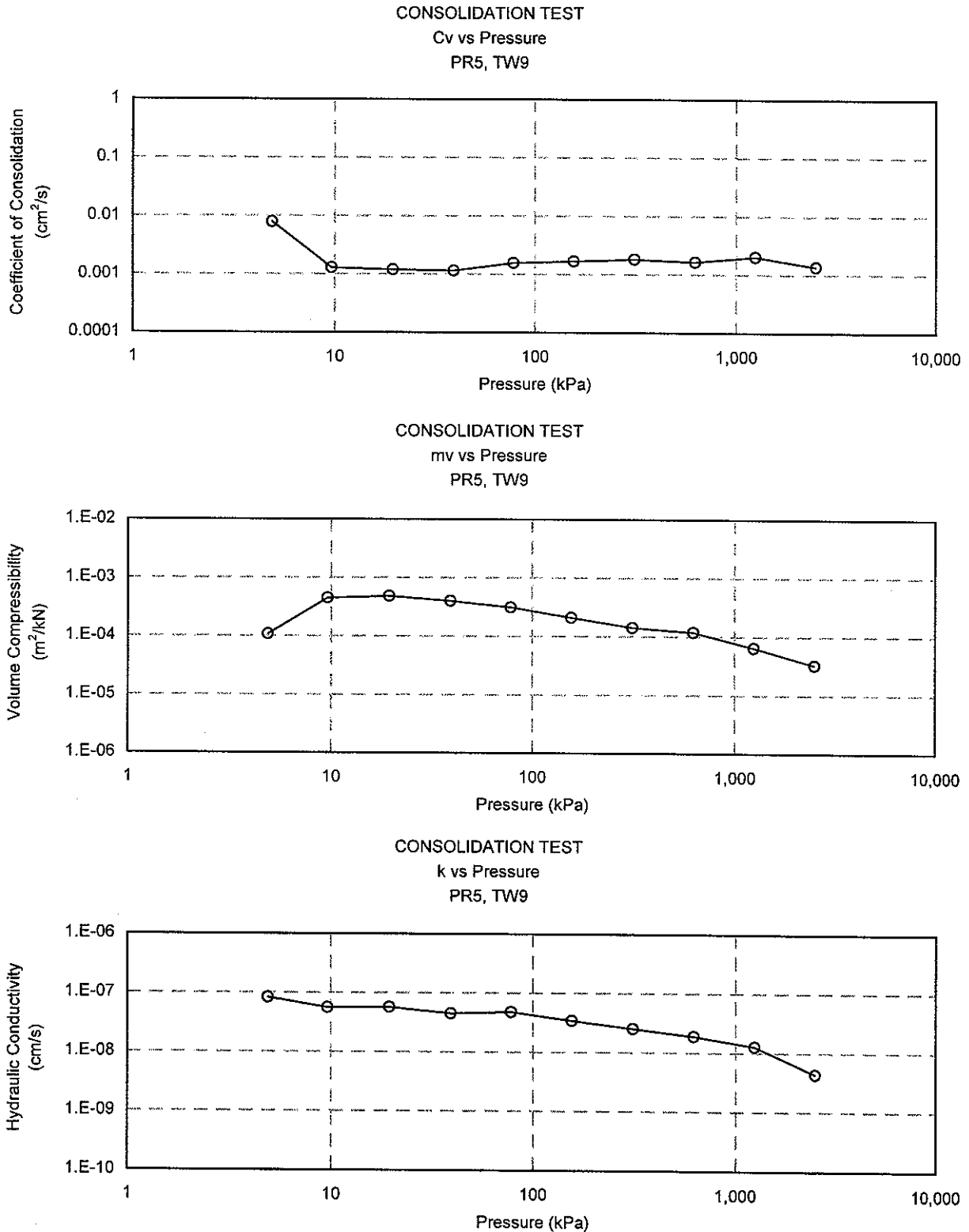


Project No. : 1-09-4135  
Date : June 2010

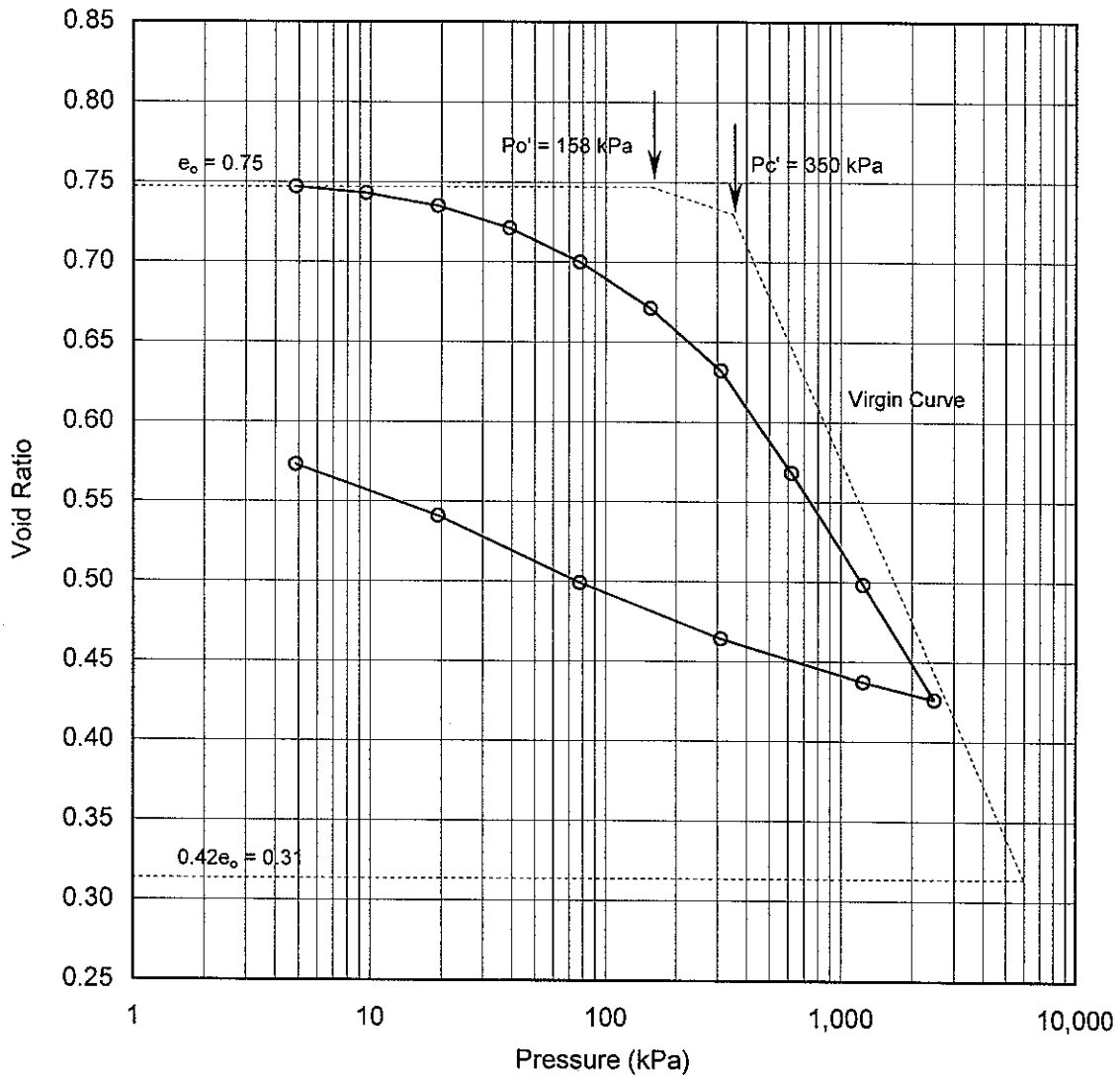


**Terraprobe Inc.**

Prepared By : HW  
Checked By : RA



CONSOLIDATION TEST  
e vs Pressure  
PR5, TW9



Soil Type : Silty Clay

$e_o =$	0.75	$\omega_L =$	32%	$P_o' =$	158 kPa
$\omega =$	27%	$\omega_p =$	16%	$P_c' =$	350 kPa
$\gamma =$	19.7 kN/m <sup>3</sup>	PI =	15%	Cc =	0.337
Gs =	2.76			Cr =	0.049

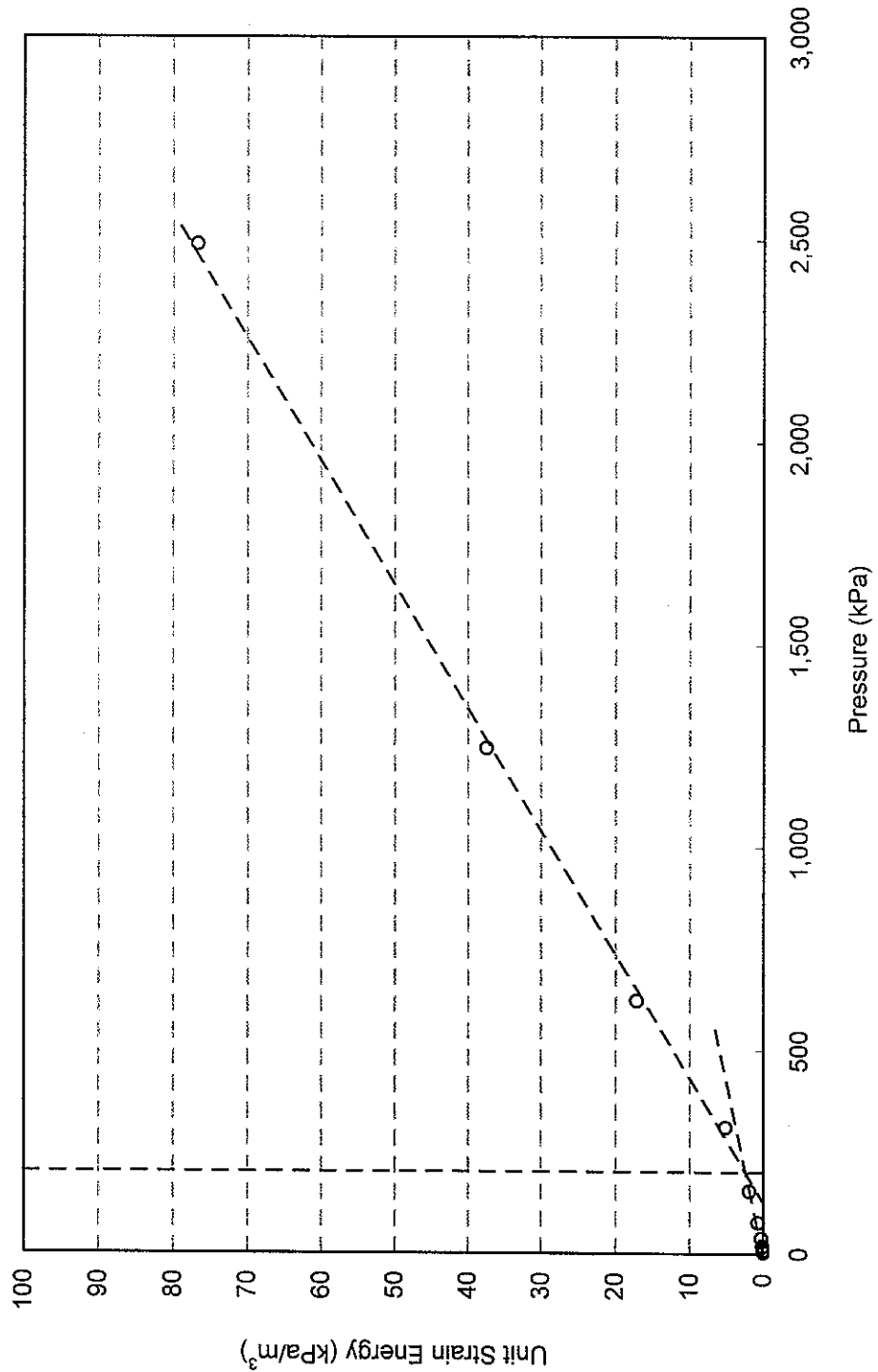
Project No. : 1-09-4135  
Date : June 2010



**Terraprobe Inc.**

Prepared By : HW  
Checked By : RA

CONSOLIDATION TEST  
Unit Strain Energy vs Pressure  
PR5, TW9



$P_c = 200 \text{ kPa}$

Project No. : 1-09-4135

Date : June 2010



**Terraprobe Inc.**

Prepared By : HW

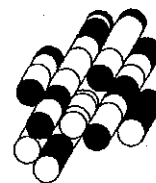
Checked By : RA

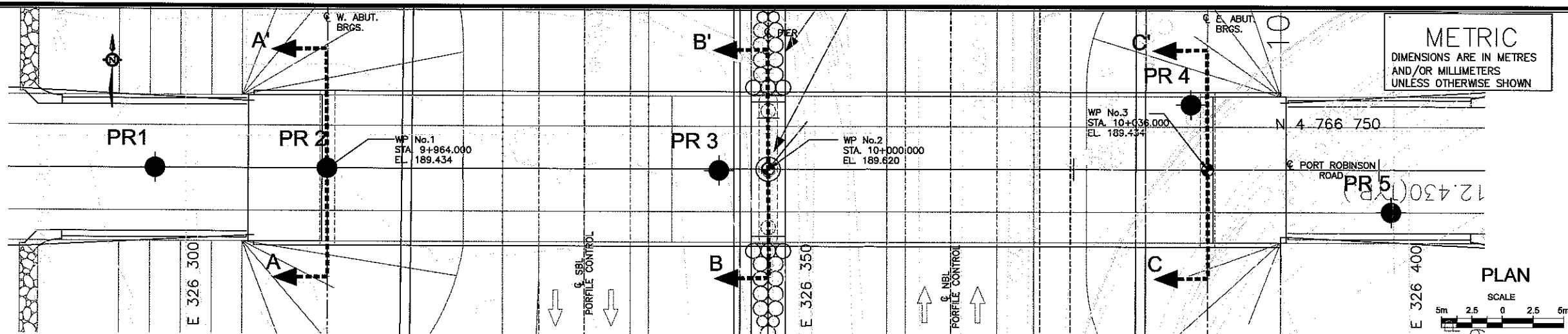


# **APPENDIX C**

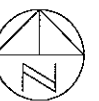
## **Drawings titled “Borehole Locations and Soil Strata”**

**Terraprobe Inc.**





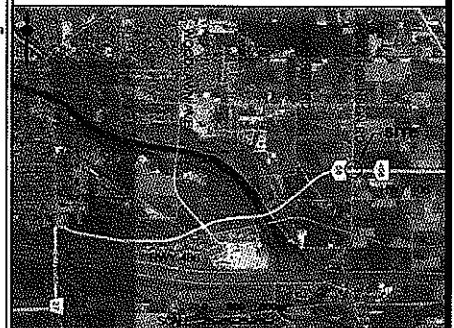
CONT No  
WP No 280-99-00



HIGHWAY 406  
PORT ROBINSON ROAD UNDERPASS  
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET  
1 OF

Giffels Associates Limited  
Consulting Engineers and Architects  
An IBI Group Company



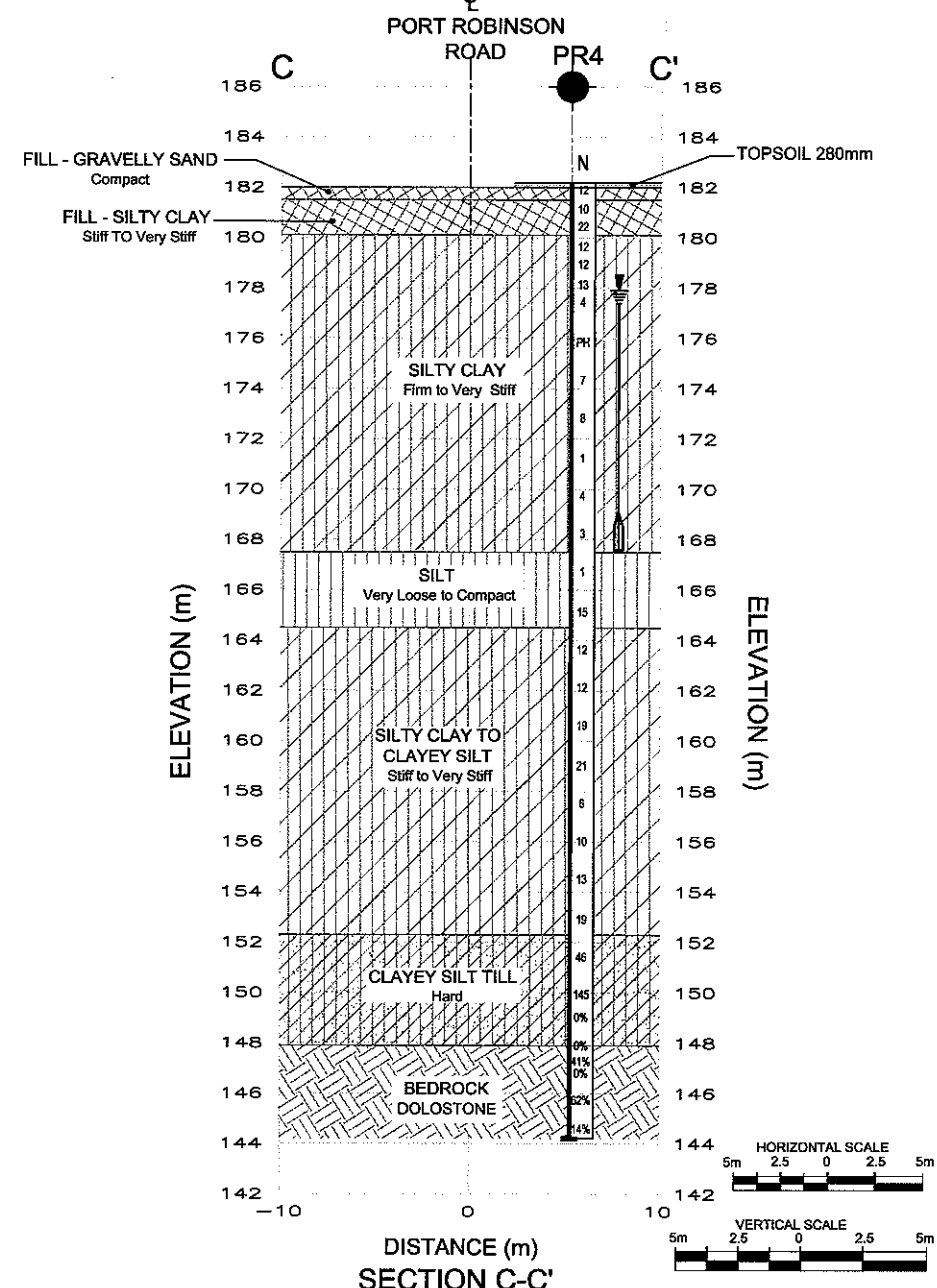
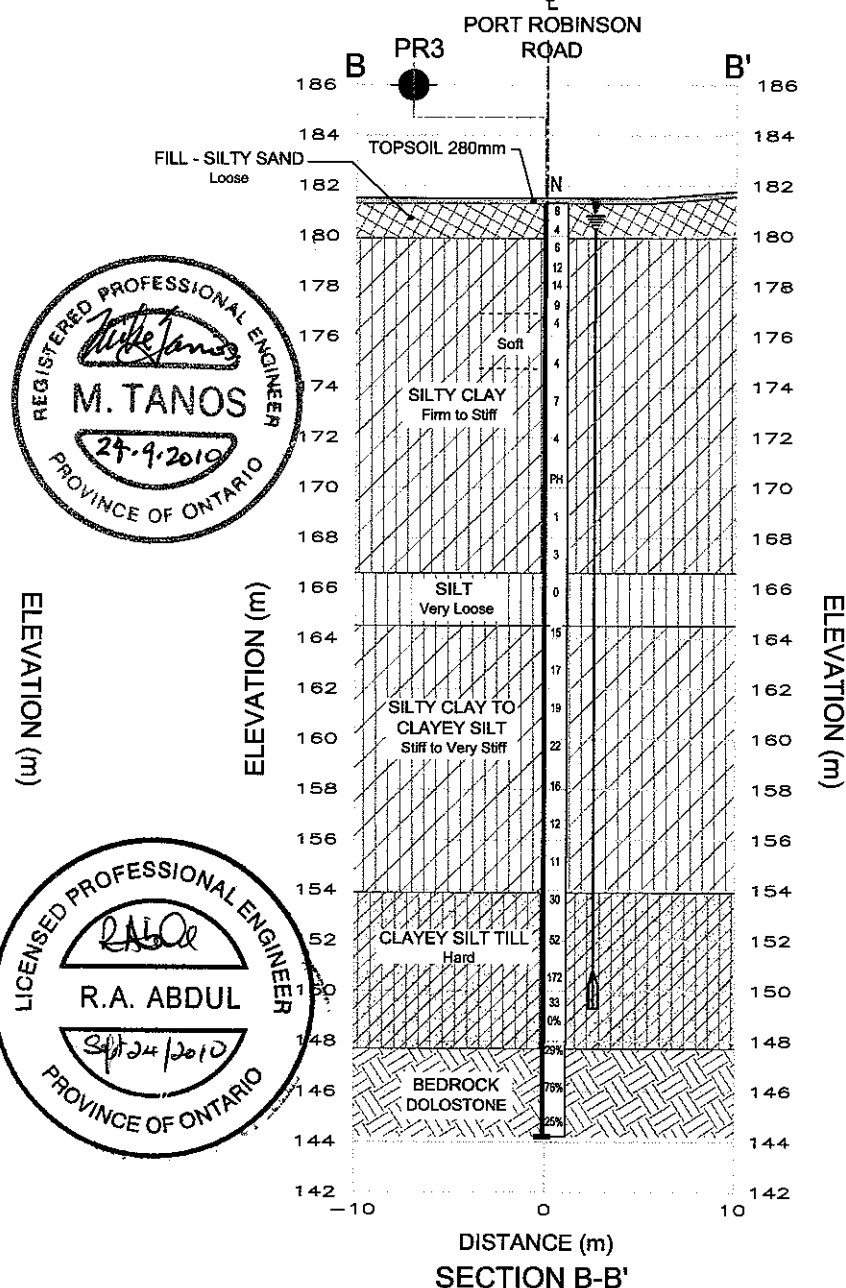
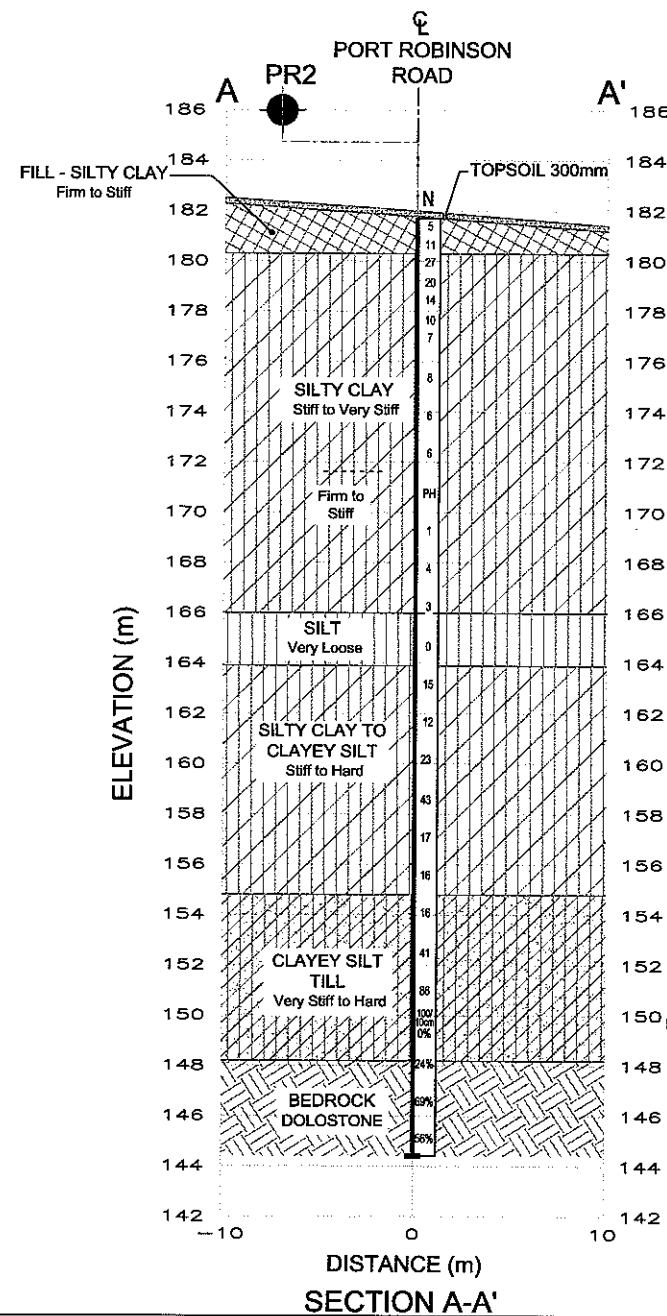
KEY PLAN

LEGEND	
	Bore Hole
	Dynamic Cone Penetration Test
	Bore Hole And Cone
	Blows/0.3m (Std Pen Test, 475 J/blow)
	Blows/0.3m (60° Cone, 475 J/blow)
	WL at Time of Investigation
	WL in Piezometer (MAY 2010)
	Rock Quality Designation
	Auger Refusal

No	ELEV.	COORDINATES	
		NORTHING	EASTING
PR1	181.7	4 766 747.4	326 297.5
PR2	181.7	4 766 747.3	326 311.5
PR3	181.3	4 766 747.0	326 343.5
PR4	182.2	4 766 752.2	326 362.2
PR5	181.2	4 766 743.3	326 398.5

**NOTE**  
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.  
This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

REVISIONS			
DATE	BY	DESCRIPTION	
DESIGN R.A.	CODE CHBDC2006	LOAD	DATE SEPT. 2010
DRAWN K.C.	CHK R.A.	STRUCT 34-462	GEOCRE 30M3-262

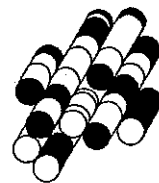




# **APPENDIX D**

## **Foundation Comparison**

**Terraprobe Inc.**



**COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT**

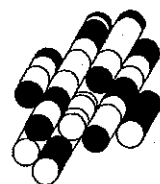
Foundation Element	Driven Piles	Augered Caissons	Footing on Native Soil	Footing on Engineered Fill
East and West Abutments	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. High geotechnical resistances available by driving piles to bedrock.</li> <li>ii. Readily installed.</li> <li>iii. Reliable performance and low risk.</li> <li>iv. Allows for the design of an integral or semi-integral abutment.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Construction concerns related to the possibility of piles being obstructed by a boulder during driving.</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. High geotechnical resistances available by founding caissons on bedrock.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Relatively high construction effort required to install caissons compared to driven piles.</li> <li>ii. Higher risk of encountering potential construction problems compared to driven piles.</li> <li>iii. Precludes consideration of a semi-integral abutment structure.</li> </ul>	<p><b>Advantages:</b></p> <p>None</p> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Uneconomically large footings due to low geotechnical resistance of soils.</li> <li>ii. Unreliable performance and high risk due to settlement sensitive soils. Potential for unacceptable settlements and differential settlements.</li> <li>iii. Relatively long abutment stems required.</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. Possibility of shortening the abutment height.</li> <li>ii. Allows for the design of a semi-integral abutment.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Lower geotechnical resistance than bedrock.</li> <li>ii. Requires preloading to eliminate undesirable settlement prior to footing construction.</li> </ul>
Pier	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. High geotechnical resistances available by driving piles to bedrock.</li> <li>ii. Readily installed.</li> <li>iii. Reliable performance and low risk.</li> <li>iv. Allows for the design of an integral or semi-integral abutment.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Construction concerns related to the possibility of piles being obstructed by a boulder during driving.</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. High geotechnical resistances available by founding caissons on bedrock.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Relatively high construction effort required to install caissons compared to driven piles.</li> <li>ii. Higher risk of encountering potential construction problems compared to driven piles.</li> </ul>	<p><b>Advantages:</b></p> <p>None</p> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Uneconomically large footings due to low geotechnical resistance of soils.</li> <li>ii. Unreliable performance and high risk due to settlement sensitive soils. Potential for unacceptable settlements and differential settlements.</li> </ul>	<p><b>Advantages:</b></p> <p>None</p> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Lower geotechnical resistance than bedrock.</li> <li>ii. Requires preloading to eliminate undesirable settlement prior to footing construction.</li> </ul>



# **APPENDIX E**

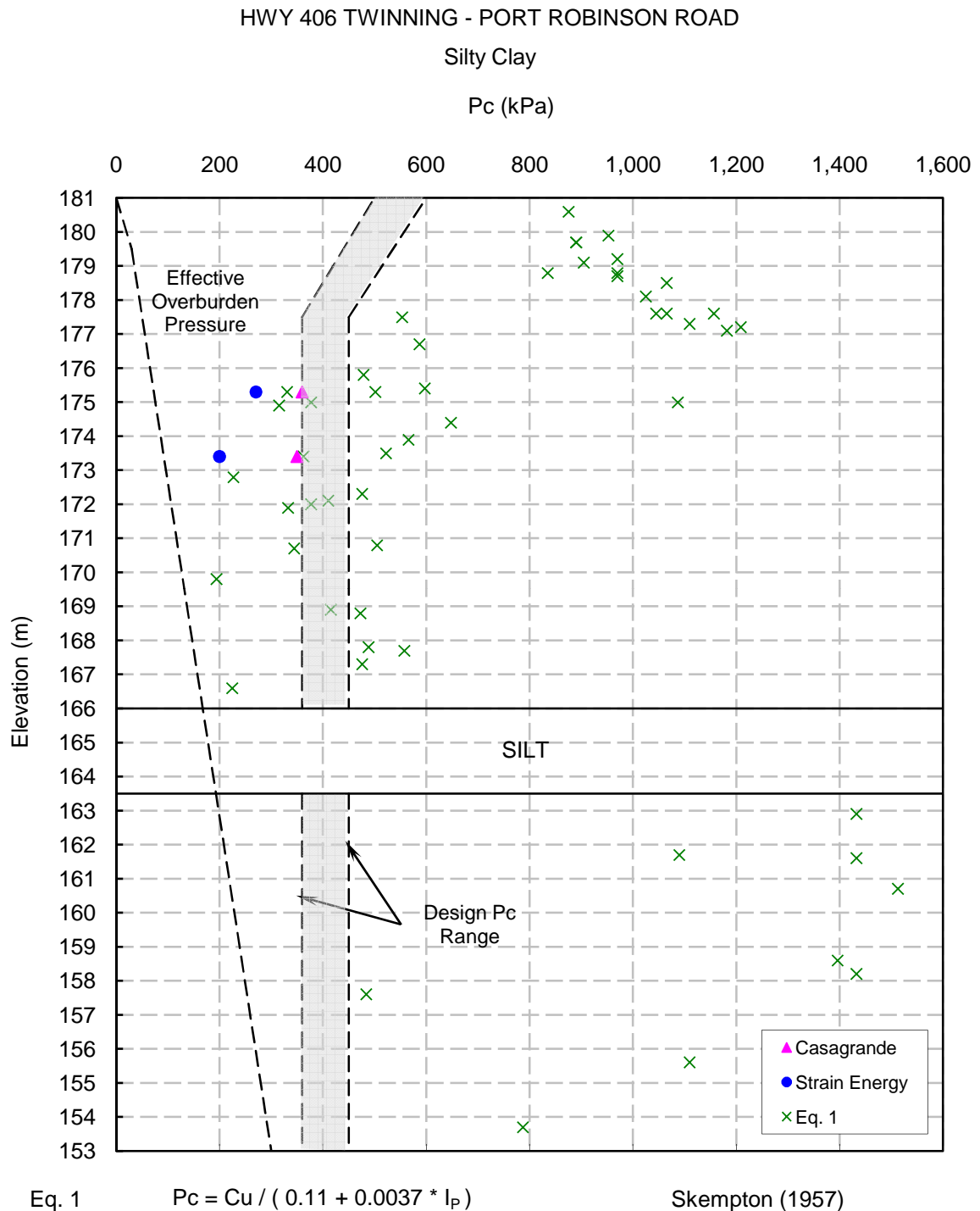
## **Settlement Parameters and Results**

**Terraprobe Inc.**



# PREDICTED AND MEASURED PRECONSOLIDATION STRESS

FIGURE E1



Project No. : 1-09-4135

Date : September, 2010



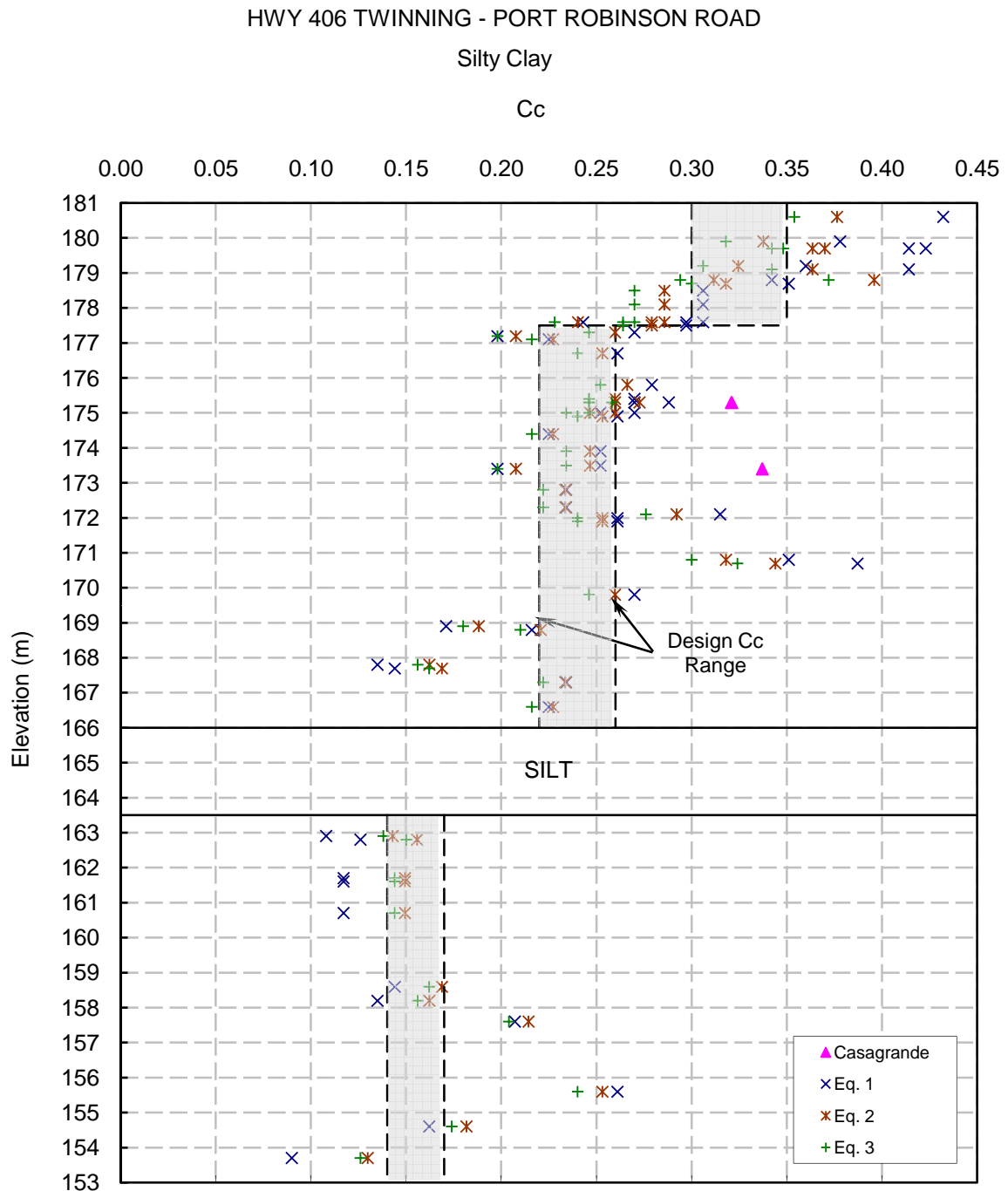
**Terraprobe Inc.**

Prepared By : HW

Checked By : RA

# PREDICTED AND MEASURED COMPRESSION INDEX

FIGURE E2



Eq. 1  $Cc = 0.009 * (LL - 10)$

Terzaghi & Peck (1967)

Eq. 2  $Cc = 0.002343 * LL * Gs$

Nagaraj & Murty (1985)

Eq. 3  $Cc = 0.006 * (LL + 1)$

Lav & Ansal (2001)

Project No. : 1-09-4135

Date : September, 2010



**Terraprobe Inc.**

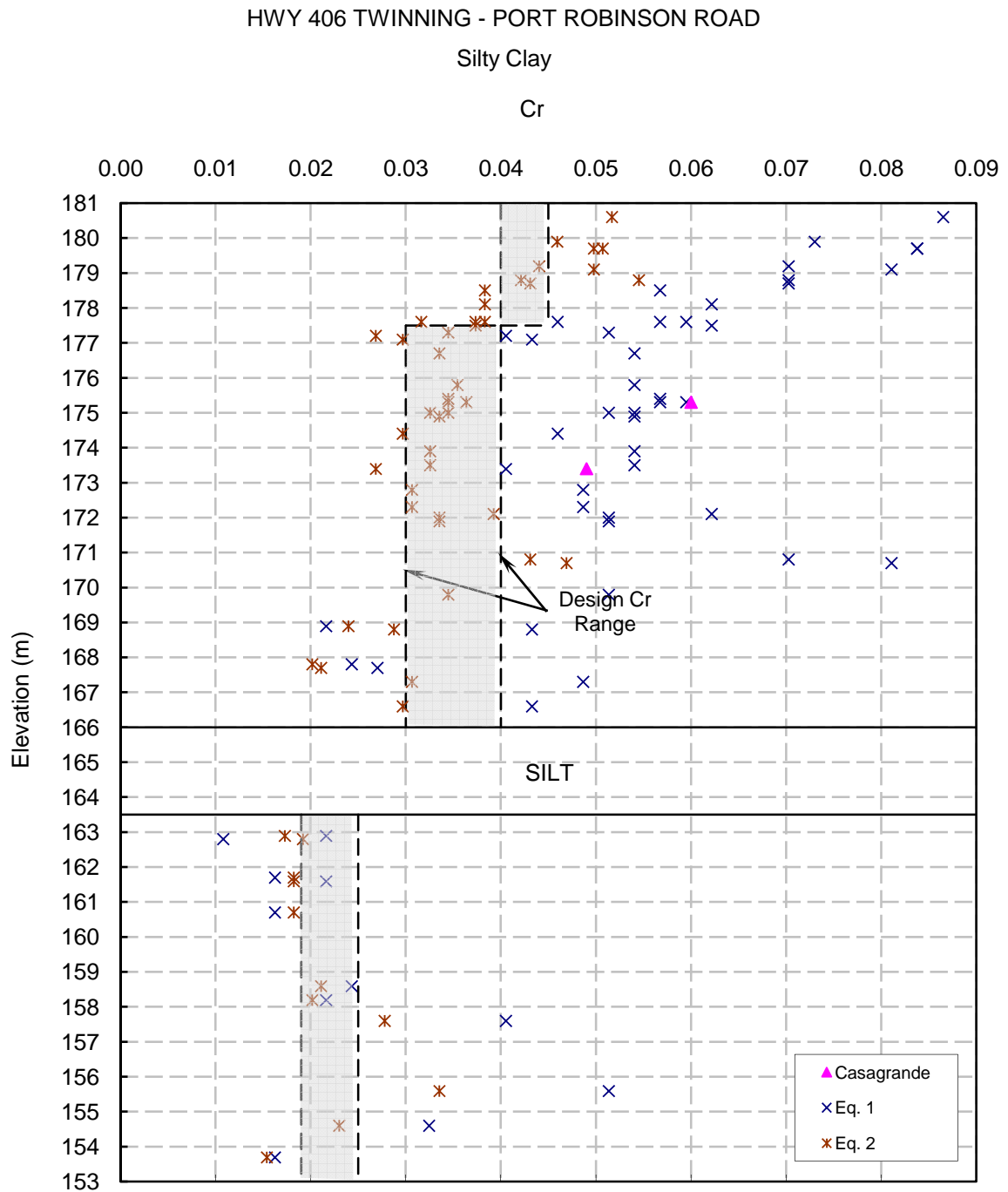
Prepared By : HW

Checked By : RA



# PREDICTED AND MEASURED RECOMPRESSION INDEX

FIGURE E3



Eq. 1  $Cr = I_p / 370$

Kulhawy & Mayne (1990)

Eq. 2  $Cr = C_c / 5 \sim C_c / 10$

Das (1993)

Project No. : 1-09-4135

Date : September, 2010

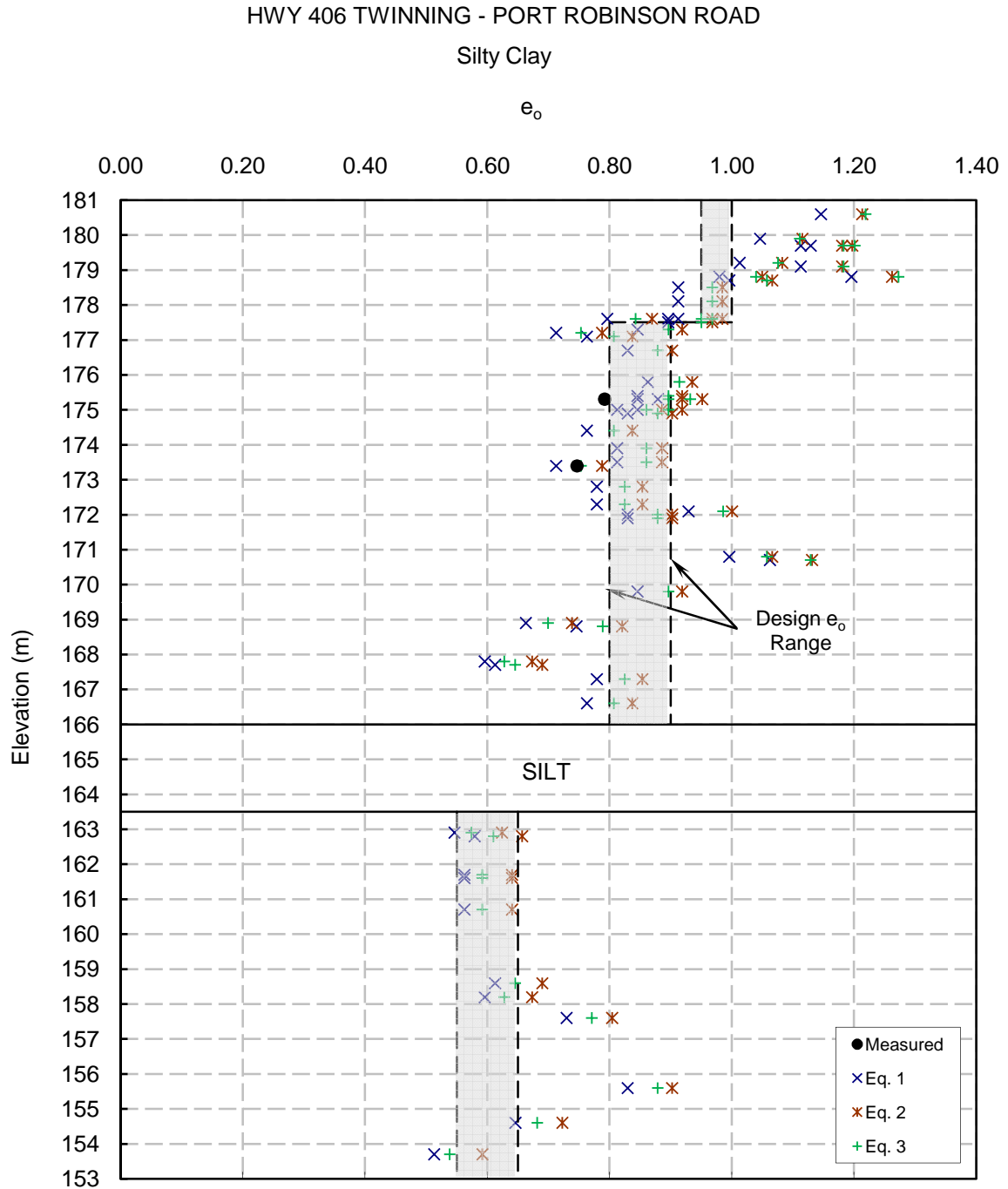


Prepared By : HW

Checked By : RA

# PREDICTED AND MEASURED VOID RATIO

FIGURE E4



Eq. 1       $e_o = ( Cc - 0.256 ) / 0.43 + 0.84$

derived from Cozzolino (1961)

Eq. 2       $e_o = Cc / 0.37 - 0.003 * LL + 0.34$

derived from Azzouz et al. (1976)

Eq. 3       $e_o = ( Cc + 0.10 ) / 0.40$

derived from Lav & Ansal (2001)

Project No. : 1-09-4135

Date :      September, 2010



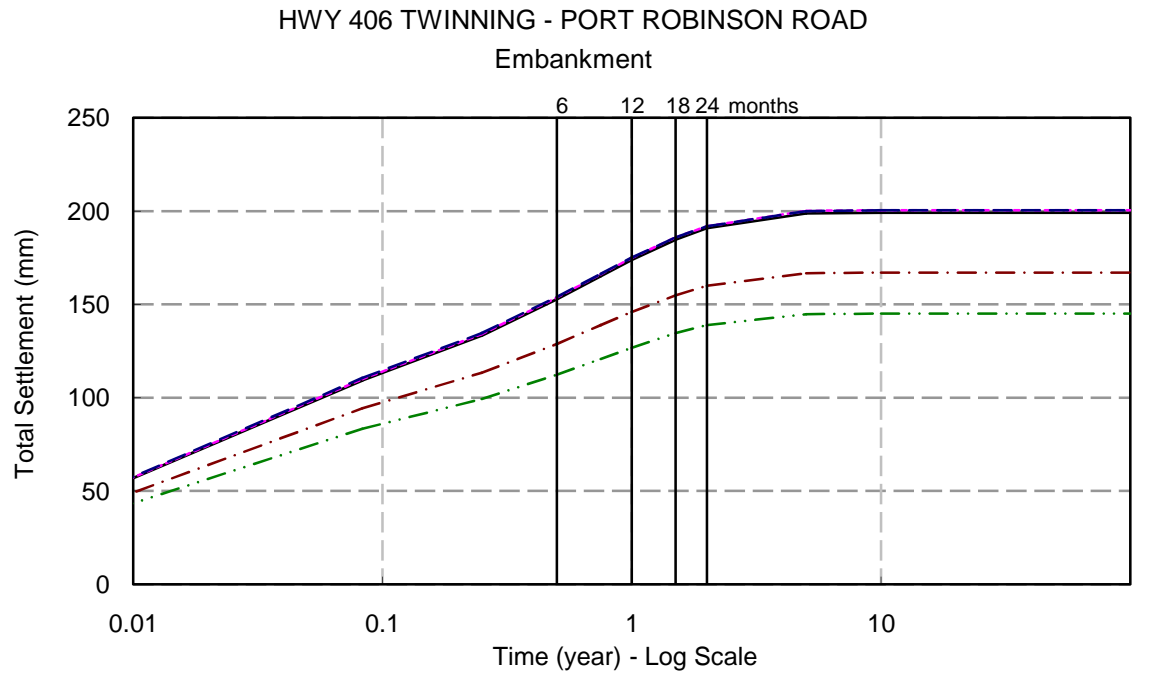
**Terraprobe Inc.**

Prepared By : HW

Checked By : RA

# SETTLEMENT VS. LOG TIME

FIGURE E5



— Local Earth Fill    - - - Rock Fill    - - - SSM    - . - Light Weight Fill    - . . Ultra Light Weight Fill

Project No. : 1-09-4135

Date : September, 2010



**Terraprobe Inc.**

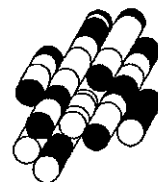
Prepared By : HW

Checked By : RA

# **APPENDIX F**

## **Suggested NSSP Wording**

**Terraprobe Inc.**



In this report reference is made to the following Provincial Standard:

- OPSS 903, November 2009

The contract documents should contain a NSSP containing the following wording:

**Cobbles and Boulders**

“The Contractor is informed that the soils at this site may contain cobbles and boulders that could impede the progress of pile driving operations. The soil conditions are described in the Foundation Investigation Report prepared for this site”.

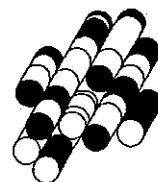
If a pile encounters refusal on cobbles and boulders the QVE should terminate driving before the pile is damaged by overdriving. If a pile has not been driven to effective refusal at the recommended design tip elevations and further driving is likely to cause damage to the pile, pile driving should cease and the contract administrator should be notified.



# **APPENDIX G**

## **Settlement Monitoring Programme**

**Terraprobe Inc.**



**SUPPLY AND INSTALLATION OF EMBANKMENT MONITORING EQUIPMENT –**  
**Item No.**

---

Special Provision

---

**1.0 GENERAL**

**1.1 Scope**

This special provision contains the requirements for the supply and installation of the following geotechnical instruments:

- Settlement Plates (SP)
- Vibrating Wire Piezometers (VWP)
- Standpipe Piezometers (SSP)
- Survey Benchmark/s (BM)

**1.2 Purpose**

The purpose of these instruments is to monitor settlements and pore water pressures in the foundation soils under the new approach embankments. The data will be used for planning the commencement of pile driving operations, construction scheduling, and final paving operations. Settlements will be measured by level surveying of the top of the settlement rods.

The piling at the foundation elements, the fill placement, timing for the removal of the preload, and final paving operations shall be controlled by the instrumentation readings.

**1.3 Personnel**

The Contractor shall retain a Geotechnical Consultant with MTO classification of “Geotechnical (Structures and Embankments) – High Complexity”, to undertake the supply and installation of geotechnical instruments.

The Contractor (as referenced herein) shall be understood to refer to the Contractor and their Geotechnical Consultant.

**1.4 Or equal**

The term “or equal” shall be understood to indicate that the equal product is the same or better than the specified product in function, performance, reliability, quality and general configuration. Only one supplier shall be selected for the supply of data acquisition system and vibrating wire instruments (piezometers).

**1.5 Notification**

The Contract Administrator shall be notified a minimum of 15 working days in advance of commencing the installation of instruments.

## **1.6 Submission Requirements**

The Contractor shall submit details of proposed installations including:

- Design and construction drawings, including equipment layout;
- Installation methodology and timing; monitoring shed;
- Equipment and material specifications, data sheets;
- Location and types of survey benchmarks; and
- Installation schedule.

Submissions shall be made to the Contract Administrator a minimum of 15 days before the start of the instrument installation.

## **1.7 Subsurface Conditions**

The subsurface conditions at the site(s) are described in the report:

- Foundation Investigation Report – Port Robinson Road Underpass, Highway 406 Twinning, Port Robinson Road to East Main Street, Ontario. Agreement No. 2008-E-0016, W.P. 280-99-00, Site No. 34-462, Geocres. No. 30M3-262, dated September 24, 2010, by Terraprobe Inc.

The owner warrants that the information provided in the report can be relied upon with the following exceptions.

1. Any interpretations of the data or opinions expressed in the report are not warranted; and
2. Although the raw measured data presented is warranted, the Contractor must satisfy himself as to the sufficiency of the information presented and obtain any updated or additional information, and perform any studies, analysis or investigations the Contractor deems necessary in order to prepare his design, at no additional cost to the Owner.

## **1.8 Equipment Operation and Weather Conditions**

All installations and monitoring equipment and associated materials shall be capable of withstanding the range of temperatures possible for their location within the ground or on the surface. The instruments shall be capable of operating within the manufacturer's stated accuracy throughout the temperature range. Monitoring shall be conducted year round and the Contractor is advised that the equipment should be accessible for monitoring throughout the duration of the Contract.



## 2.0 INSTALLATION

A summary of instrumentation requirements is given in Table 2.1. Details and specific material requirements are presented elsewhere in this special provision.

**Table 2.1 – Instrument & Benchmark Quantities and Locations**

INSTRUMENT I.D.	STATION	OFFSET FROM CENTRELINE	NO. OF INSTRUMENTS			
			SP	VWP	SSP	BM
West Approach						
SP1	9+935	5.5 m Lt	1			
SP2	9+935	5.5 m Rt	1			
VWP1	9+945	0		1		
SSP1	9+945	Outside of construction area			1	
SP3	9+950	0	1			
SP4	9+960	5.5 m Lt	1			
SP5	9+960	5.5 m Rt	1			
BM1	N/A	N/A				1
East Approach						
SP6	10+040	5.5 m Lt	1			
SP7	10+040	5.5 m Rt	1			
SP8	10+050	0	1			
VWP2	10+055	0		1		
SSP1	10+055	Outside of construction area			1	
SP9	10+065	5.5 m Lt	1			
SP10	10+065	5.5 m Rt	1			
BM2	N/A	N/A				1
Total Instruments			10	2	2	2

### 2.1 Instrument Location

Prior to the installation of instruments, the Contractor shall accurately survey and stake the location of each instrument and obtain a ground surface elevation at each instrument location.

### 2.2 Survey Benchmarks (BM)

The Contractor shall provide a minimum of one non-yielding deep seated survey benchmark (BM) at the site. Alternatively the contractor may select stable non-settling points on existing structures within the area subject to approval by the contract administrator.

The number and locations(s) of benchmark(s) shall be such that direct sighting is possible from all settlement rods to at least one benchmark.

### **2.3 Accuracy of Surveying for Elevations**

Elevations shall be surveyed referenced to Geodetic datum to an accuracy of  $\pm 2$  mm or better.

### **2.4 Monitoring Instrument Location**

All monitoring instruments shall be located in MTM NAD83 northing and easting coordinates.

### **2.5 Materials and Equipment**

The Contractor shall supply all materials and equipment required for the installation of instrumentation unless noted otherwise.

### **2.6 Underground Utilities**

The Contractor shall be responsible for locating and protecting all underground utilities prior to drilling boreholes for installing instruments. Any damage to underground utilities caused by the Contractor's work shall be repaired by the Contractor, at no cost to the Ministry.

### **2.7 Marking and Labelling**

The location of any above ground monitoring fixture shall be made clearly visible to nearby traffic before, during and after embankment construction. Marking shall be of sufficient size to be visible from a reversing vehicle and after heavy snow falls.

Instruments or their data cables shall be clearly labelled in the field, each instrument having a unique identifier. The labelling shall remain legible for at least 1 year.

### **2.8 Protection of Instruments**

All instruments shall be adequately protected by the Contractor such that they are not damaged during construction. Any instrument damaged by the Contractor's work shall be immediately replaced at no cost to the Ministry.

### **2.9 Boreholes**

The Contractor shall make a basic stratigraphic log of boreholes as they are being drilled. In situ or laboratory testing is not required.

Boreholes shall be advanced using conventional drilling methods and shall be as straight and vertical as practical.

### **2.10 Installation Program**

Instrument installation shall be completed before the start of any piling installation and before any embankment construction. Table 2.2 provides a summary of the installation schedule requirements.

**Table 2.2 – Installation Program**

<b>TYPE</b>	<b>START INSTALLATION</b>	<b>FINISH INSTALLATION</b>
SP	After excavating to recommended stripping elevation of embankment	On completion of embankment construction
VWP	Before Piling and Embankment Construction	Before Piling and Embankment Construction
SSP	Before Piling and Embankment Construction	Before Piling and Embankment Construction
BM	Before commencement of embankment construction	Before commencement of embankment construction

### **3.0 BENCHMARK (BM) – SUPPLY & INSTALLATION**

#### **3.1 GENERAL**

##### **3.1.1 Scope**

This Section contains the requirements for the supply and installation of benchmark/s (BM).

The purpose of the benchmark is to provide non-settling references for the surveying of settlement rods.

##### **3.1.2 General Procedure**

The benchmark consists of a steel rod anchored to the bottom of a borehole. The benchmark shall be installed prior to embankment construction. The number and locations of benchmarks shall be such that direct sighting is possible from all settlement rods to at least one benchmark. Elevations shall be surveyed to an accuracy of  $\pm 2\text{mm}$  or better.

Prior to the installation of instruments, the Contractor shall accurately survey and stake the locations of each instrument and obtain a ground elevation at each instrument location.

##### **3.1.3 Location**

Benchmarks shall be located and installed outside of the area of construction activity. Notwithstanding the installation details provided herein the contractor may select stable non-settling points on existing structures within the area subject to approval by the contract administrator.

**Table 3 – Approximate Bench Mark Locations**

<b>Station</b>	<b>Offset (m)</b>	<b>No. of BM</b>	<b>Estimated Rod Anchor Elevation (m)</b>
Outside of Construction Area	N/A	BM1	152.0
Outside of Construction Area	N/A	BM2	152.0

#### **3.2 MATERIALS**

##### **3.2.1 General**

The Contractor shall supply all materials and equipment required for the installation of the benchmark.

##### **3.2.2 Rod**

The Contractor shall supply a steel pipe Schedule 40 with an outside diameter not less than 25.4 mm (1”), supplied in lengths as required to complete the installation as described.

The top end of each length of rod shall be threaded to receive a cap. A rounded cap shall be installed at the top of the rod in such a way that a single survey point can be clearly identified and returned to.

### **3.2.3 Sand**

The Contractor shall supply clean washed sand. The sand shall be Sakcrete washed general-purpose sand – or equal.

### **3.2.4 Grout**

The Contractor shall supply cement-bentonite grout. A suitable grout mix design consists of 23 kg of bentonite (OPSS 1205), 143 litres of water and 40 kg of cement (Type G.U. – OPSS 1301).

### **3.2.5 Rod Anchor Grout**

The Contractor shall supply cement-bentonite grout. A suitable grout mix design consists of 14 kg of bentonite (OPSS 1205), 49 litres of water and 40 kg of cement (Type G.U. – OPSS 1301).

### **3.2.6 Friction Reducing Sleeve**

The Contractor shall supply a friction reducing sleeve consisting of Schedule 50 – 50.8 mm (2”) O.D. PVC pipe cut perpendicular to the axis of the pipe.

## **3.3 INSTALLATION**

### **3.3.1 General**

The Contractor shall install the benchmark in accordance with the information below.

### **3.3.2 Borehole Installation**

The borehole shall be advanced to the rod anchor elevation provided in Table 3 using suitable drilling techniques. The diameter of the borehole shall be sufficient to fit the rod, friction reducing sleeve and rod anchor. The sides of the borehole shall be stable and the borehole shall be free of drilling mud and debris.

### **3.3.3 Rod**

The coupling of the rods shall be such that all sections have the same axis and no separation or contraction will occur at the couplings.

#### **3.3.4 Rod Anchor**

The rod shall be installed vertically in the borehole with its bottom end resting at the bottom of the borehole. The bottom portion of the rod shall be fixed against the surrounding native soil by grouting the bottom 0.5 m of the borehole to form a concrete/soil anchor.

Once grouting is completed and the rod anchor grout has set, the Contractor shall pour 0.5 m of clean sand in the borehole above the concrete/soil anchor to create a base for the end of the friction reducing sleeve to rest on.

The elevation of the bottom of the rod anchor shall be determined by measuring the length of the rod to the ground surface elevation.

#### **3.3.5 Friction Reducing Sleeve**

The friction reducing sleeve shall be over the entire length of the rod above the rod anchor and sand.

#### **3.3.6 Installation Details**

The elevation, easting and northing of the top of the benchmark rod shall be surveyed.

### **3.4 COORDINATION WITH MONITORING**

#### **3.4.1 Notification**

The Contractor shall notify the Contract Administrator no later than 3 days after installing a benchmark. At this time the Contractor shall also supply the following information to the Contract Administrator.

- Location of the rod anchor and elevation top of rod;
- Dates of installation;
- Stratigraphic log of subsurface conditions at the benchmark, including drilling method notes;
- Installation notes/sketches; and
- Description of benchmarks, sleeve and rod anchor.

#### **3.4.2 Monitoring**

Monitoring of settlements with reference to the benchmark shall be done by others. Monitoring shall be conducted during and following the embankment construction at the north and south approaches. The Contractor shall provide installation information as specified above and provide access to the benchmark for monitoring including, but not limited to snow clearing in the winter. The Contractor shall provide electric power and general area lighting as needed.

### **3.5 REPORTING**

The Contractor shall record and report relevant installation details to the Contract Administrator. These include, but are not limited to:

- Benchmark easting, northing in MTM NAD83 coordinates;
- Elevation of bottom of rod anchor and top of rod relative to Geodetic datum;
- Dates of installation; and
- Installation notes/sketches.

## **4.0 SETTLEMENT PLATES (SP) – SUPPLY & INSTALLATION**

### **4.1 GENERAL**

#### **4.1.1 Scope**

This Section contains the requirements for the supply and installation of settlement plates.

The purpose of the settlement plates is to monitor settlements of the foundation soils below the embankment base. The settlement readings shall help to establish the timing for the removal of preload fill, the commencement of pile driving operations, as well as final paving operations. Settlement is measured by survey of the top of the rod with reference to stable, non-settling benchmarks.

#### **4.1.2 General Procedure**

The settlement rods shall be attached to a plate at the existing ground surface. As embankment construction proceeds the rods shall be extended above the new top of embankment.

Sleeves around the rods shall be installed to reduce friction and allow uninhibited movement of the rod with the plate.

A protective surround shall be extended with the rods as embankment construction proceeds.

#### **4.1.3 Location**

The locations of the settlement plates are shown on the Contract Drawings and are given in Table 4.

**Table 4 – Approximate Settlement Plate Locations**

<b>Station</b>	<b>Offset (m)</b>	<b>No. of SP</b>	<b>Estimated Fill Thickness (m)*</b>
<b>West Approach</b>			
9+935	5.5 m Lt	1	8.0
9+935	5.5 m Rt	1	8.0
9+950	0	1	8.0
9+960	5.5 m Lt	1	8.0
9+960	5.5 m Rt	1	8.0
<b>East Approach</b>			
10+040	5.5 m Lt	1	9.0
10+040	5.5 m Rt	1	9.0
10+050	0	1	9.0
10+065	5.5 m Lt	1	9.0
10+065	5.5 m Rt	1	9.0

Notes:\* Embankment thickness based on surface elevation of removal levels/stripping depths.



## **4.2 MATERIALS**

### **4.2.1 General**

The Contractor shall supply all materials and equipment required for the installation of the settlement plates.

### **4.2.2 Plate**

The Contractor shall supply a steel plate with thickness of at least 6.35 mm. The plate shall be at least 0.5 m by 0.5 m.

### **4.2.3 Rod**

The Contractor shall supply a steel pipe Schedule 40 with an outside diameter not less than 25.4 mm (1"), supplied in lengths as required to complete the installation as described in Section 4.3.

The top end of each length of rod shall be threaded to receive a cap. A rounded cap shall be installed at the top of the rod in such a way that a single survey point can be clearly identified and returned to.

### **4.2.4 Friction Reducing Sleeve**

The Contractor shall supply a friction reducing sleeve consisting of Schedule 40 – 50.8mm (2") O.D. PVC pipe cut perpendicular to the axis of the pipe.

### **4.2.5 Protective Surround**

The Contractor shall supply a protective surround for the portion of the rod within the embankment. The surround shall consist of 300 mm diameter corrugated steel pipe (CSP – OPSS 1801) with the ends cut perpendicular to the axis of the pipe and free of burrs and sharp edges. The space between the CSP and the Friction Reduction Sleeve (PVC pipe) shall be filled with medium to coarse sand.

## **4.3 INSTALLATION**

### **4.3.1 General**

The Contractor shall install settlement rods as per the Contract Drawings provided in addition to what is stated or emphasized below.

### **4.3.2 Settlement Plate**

The settlement plate shall be installed horizontally after subgrade preparation is completed and prior to fill placement.

The elevation of the base of the plate shall be surveyed before backfilling.

#### **4.3.3 Rod**

The rod shall be fixed to the center of the plate and installed perpendicular to the plate.

The coupling of the rods shall be such that all sections have the same axis and no separation or contraction will occur at the couplings.

#### **4.3.4 Friction Reducing Sleeve**

The friction reducing sleeve shall be over the entire length of the rod that is below ground and within the embankment fill except that the cap on top of the settlement rod shall extend 25 mm above the top of the friction sleeve at all times.

### **4.4 EXTENSION OF ROD**

The settlement rods shall be extended upwards as the embankment is constructed so that the top of the rod is always at least 0.3 m but not more than 2 m above the surrounding fill.

#### **4.4.1 Protective Surround**

The CSP, Friction Reducing Sleeve and sand protective surround shall be extended with the rods.

The settlement rod shall be in the center of the CSP and friction-reducing sleeve.

The annulus between the CSP and the friction-reducing sleeve shall be filled with sand to a level not higher than the top of the sleeve.

#### **4.4.2 Installation Details**

The elevation, easting and northing of the center of the base of the plate shall be surveyed.

The elevation, easting and northing of the top of the rod shall be surveyed.

The total distance from the base of the plate to the top of the rod shall be measured to an accuracy of  $\pm 2$  mm or better.

### **4.5 COORDINATION WITH MONITORING**

#### **4.5.1 Notification**

The Contractor shall notify the Contract Administrator no later than 3 days after installing a settlement rod. At this time the Contractor shall also supply the following information to the Contract Administrator.

- Elevation of plate and rod referenced to Geodetic datum;
- Dates of installation;
- Installation notes/sketches; and
- Description of settlement rods, sleeve and plate.

Adjustments in the length of any settlement rod shall be coordinated with the Contract Administrator to allow surveying by others of the elevation of the top of the rod immediately before and immediately after adjustment. This surveying is necessary to accurately track the settlement data.

#### **4.5.2 Monitoring**

Monitoring of the settlement plates shall be done by others. Monitoring shall be conducted during the embankment construction and preload period. A target settlement of 175 mm is specified. A minimum preload period of 6 months is required. The Contractor shall provide installation information as specified above and provide access to the settlement rods for monitoring including, but not limited to a level scaffolding platform and ladder, if required and snow clearing in the winter. The Contractor shall provide electric power and general area lighting as needed for reading the instruments.

#### **4.6 REPORTING**

The Contractor shall record and report relevant installation details to the Contract Administrator. These include, but are not limited to:

- Settlement rod easting, northing referenced to MTM NAD83 coordinates;
- Elevation of the plate and the top of the rod referenced to Geodetic datum;
- Distance between base of plate and top of rod;
- Dates of installation; and
- Installation notes/sketches.

## **5.0 VIBRATING WIRE PIEZOMETER (VWP) – SUPPLY & INSTALLATION**

### **5.1 GENERAL**

#### **5.1.1 Scope**

This Section contains the requirements for the supply and installation of vibrating wire (VW) piezometers.

The purpose of the piezometers is to monitor piezometric head at depth within the foundation soil below the embankments. The piezometer readings shall help to establish the timing and sequence of the piling at the foundation elements, the removal of embankment preload, and final paving operations.

#### **5.1.2 General Procedure**

The piezometers shall be installed in boreholes prior to the start of any embankment construction, any preload fill construction, and any piling. Prior to installation of instruments adjacent to new construction features (including limit of pile cap, edge of unwatering system, extent of sub-excavation and backfilling), the construction features shall be laid out in the field to ensure there are no conflicts with the instruments.

The VW signal cables for the VWPs shall be extended out of the embankment and preload footprint area (where applicable) and away from the piling area through a metal or plastic conduit buried in trenches, as shown in the Contract Drawings.

The conduits for the VW signal cables for the VWPs may be routed so that they may be connected to a single data acquisition system (data-logger).

#### **5.1.3 Locations**

The Contractor shall install VW sensors at the locations and depths given in Table 5.

**Table 5 – VW Piezometer Locations**

<b>Station</b>	<b>Offset (m)*</b>	<b>No. of VWP</b>	<b>Approximate Elevation of Ground Surface (m)</b>	<b>Tip Elevations (m)</b>
West Approach 9+945	0	1	188.5	159.0
East Approach 10+055	0	1	188.5	174.0

Notes: \* Offset from centerline of Port Robinson Road.

## **5.2 MATERIALS**

### **5.2.1 VW Piezometers**

The Contractor shall supply VW borehole piezometers by Slope Indicator model 52611020 (-5 to 50 psi), RST model VW2100-0.35 – or equal; compatible with the Slope Indicator CR1000 data-logger, RST model ELGL1200 – or equal. All VW piezometers (and Settlement Cells) shall be of the same make.

All piezometers shall be calibrated prior to installation and the calibration data for each piezometer shall be provided for the Contract Administrator.

### **5.2.2 Signal Cable**

The Contractor shall supply Slope Indicator model 50613524 cable, RST model EL380004 cable – or equal. The length of cable for each piezometer shall be carefully estimated from the construction Contract Drawings to ensure that there is enough signal cable for each piezometer to provide enough slack in the borehole and along the trenches until each cable is out of the construction area where they shall be protected from earthmoving equipment.

### **5.2.3 Bentonite**

The Contractor shall supply bentonite (OPSS 1205) in pellet form in sufficient quantity to form borehole plugs as required.

### **5.2.4 Filter Sand**

The Contractor shall supply clean washed sand for filter around VWP sensors. The sand shall be Sakcrete washed general-purpose sand – or equal.

### **5.2.5 Grout**

The Contractor shall supply cement-bentonite grout. A suitable grout mix design consists of 23 kg of bentonite (OPSS 1205), 143 litres of water and 40 kg of cement (Type G.U. – OPSS 1301).

### **5.2.6 Trench Burial and Conduit**

The signal cable for each piezometer shall be buried in a shallow trench and taken out of the construction area. The Contractor shall supply suitable conduits (e.g. Schedule 40 – 75 mm (3”) – steel pipe or Schedule 80 - 75 mm (3”) – rigid PVC pipe) to protect the signal cables in the trenches and above ground surface. If appropriate, several signal cables may be housed in a single conduit and laid in a common trench.

The signal cables and conduits shall be routed such that future grading works do not interfere with the cables or conduits.

### **5.2.7 Data Acquisition System (Data-Logger)**

The signal cables from the vibrating wire piezometers shall be connected to a data-logger (to be located away from the proposed approach embankment), Slope Indicator model 56701000 (CR1000), RST model ELGL1200 – or equal. The data-logger shall consist of the following:

- ENC 16/18 Water-proof Enclosure model 56705020, model ELF0638 – or equal;
- SC32A Serial Interface (with RS232 transfer cable) model 56704010, model CS-SC32A – or equal;
- VW Interface model 56701510 or 56701500, model CS-AVW200 – or equal;
- AM16/32 Multiplexer model 56702110, model ELGL2042 – or equal;
- A suitable power supply which shall be able to last for 2 years (i.e. large capacity rechargeable battery coupled with solar panel); and
- LoggerNet Software model 56708020, model CS-Loggernet – or equal.

A minimum of one data logger shall be installed. The Contractor shall submit a detailed proposal on the setup of the data-logging system (i.e. number and location of the data-logging unit(s) to the Contract Administrator for review, prior to ordering the data-logger(s). The Contractor shall program the data-logger according to the following:

- Recording Software: VWP data shall be recorded at 5 minutes intervals during piling and four times a day (one reading every 6 hours) when not piling
- Test Software: once this program is transferred to the data-logger, one shall be able to test the system and record data manually on site

The real-time data shall be retrieved on site by direct wire (i.e. RS232 Cable) with a portable laptop computer as specified in the next section.

### **5.2.8 Portable Laptop Computer**

The Contractor shall supply:

- A New Portable Laptop Computer (with a Three year warranty): Intel Pentium M or IV or better (1.6 GHz or above) with Windows 7 Professional Operating System, minimum 1GB memory, Network Card: 10/100 Integrated Ethernet LAN, a minimum of 80GB hard drive storage, a DVD/CD-RW ROM and Microsoft Office Standard 2007, to retrieve, read and store the VW piezometer readings.
- Extra battery pack and cigarette lighter charger.

The portable laptop computer will become property of the MTO and shall be handed to the Contract Administrator after the installation of instruments for the Monitoring program.

The calibration factors for all vibrating wire instruments shall be entered in the portable laptop computer by the Contractor for initialization of the instruments.

### **5.2.8 Wooden Posts**

Wooden posts: 100 mm x 100 mm (4"x4"), minimum 3 m (10") long, if required.

## **5.3 INSTALLATION**

### **5.3.1 General**

Installation of the VW piezometers shall be as per the manufacturer's recommendations in addition to what is stated or emphasized below.

The VWP's shall not be installed closer than 1.5 m to the nearest adjacent edge of shoring or unwatering system.

The exact location of the VWP installations shall be determined in the field after sub-excavation and backfilling to original ground surface.

### **5.3.2 Protection for Long-term Monitoring (Monitoring Shed)**

The Data-logger shall be installed in a walk-in Monitoring Shed to prevent vandalism and prolonged wear-out of the data-loggers against extreme weather. The Monitoring Shed shall be a lockable and weather proof enclosure surrounded by 2 m high chainlink fence and a lockable gate. The Monitoring Shed shall also be seating on a gravel pad and securely tied down to the ground. The location of the Monitoring Shed shall not be susceptible to ground settlement. The Contractor shall submit a detailed proposal of the Monitoring Shed (i.e. materials and location(s) etc.) to the Contract Administrator for review, prior to construction.

The Contractor shall ensure access to the Monitoring Shed at all times, including but not limited to snow clearing in the winter.

### **5.3.3 Completion of Installation**

It is known that the process of installing VW piezometers can temporarily alter the pore water pressure acting on the piezometer tip. The installation of a VW piezometer shall not be considered to be complete until the pore pressure acting on the piezometer has returned to and stabilized at the value prevailing in the surrounding, unaffected soil mass. The Contractor shall take daily reading of the pore pressures until the value has stabilized. Stabilization shall be deemed to have occurred:

- a) When no change in the measured value has occurred over a period of 5 days and the measured value is within 10% of the anticipated hydrostatic value.
- b) When the daily rate of change is less than four (4) kPa per day for three consecutive days and the measured value is within 5% of the anticipated hydrostatic value.
- c) Failing either of the two above conditions, as determined by the Contract Administrator.

The Contractor shall be prepared to wait for a period of 10 to 15 days after completion of installation of instruments for the baseline readings to stabilize prior to the commencement of the construction works.

## **5.4 COORDINATION WITH MONITORING**

### **5.4.1 Notification**

The Contractor shall notify the Contract Administrator no later than 3 days after installing a VW piezometer. At this time, the Contractor shall also supply the following information to the Contract Administrator.

- VW piezometer location, easting, northing referenced to MTM NAD83 coordinates;
- Elevations of VW sensor referenced to Geodetic datum;
- Stratigraphic log of subsurface conditions, including drilling method notes;
- Dates of installation;
- Installation notes/sketches;
- Model, make and serial numbers of VW sensors, readout unit and signal cable; and
- Calibration details of VW sensors.

### **5.4.2 Monitoring**

Monitoring of the VW piezometers shall be done by others. Monitoring shall be conducted during and after piling at the abutments, during embankment fill construction and during the preload period. The Contractor shall provide installation information as specified above and provide access to the data-loggers for monitoring.

The Contractor shall transfer the Portable Laptop Computer to the Contract Administrator, including all the data-logging softwares and hardware, operation instructions and calibration constants. The Contractor shall also transfer the keys for the locks of the Monitoring Shed(s). The Contractor shall be available for one site meeting with the Contract Administrator to transfer and explain about any questions from the Contract Administrator regarding the data-logging system.



## **6.0 STANDPIPE PIEZOMETER (SSP) – SUPPLY & INSTALLATION**

### **6.1 General**

#### **6.1.1 Scope**

This Section contains the requirements for the supply and installation of standpipe piezometers.

The purpose of the standpipe piezometer is to provide bench mark data by monitoring the hydrostatic piezometric head at depth outside of the construction area of the approach embankment fill.

#### **6.1.2 General Procedure**

The standpipes shall be installed prior to any piling and embankment fill construction.

Standpipes shall be installed in vertical boreholes.

#### **6.1.3 Location**

The locations of the standpipes shall be outside of the construction area near the given Station. The depths of the standpipes are given in Table 6

**Table 6 – Standpipe Piezometer Locations and Depths**

<b>Station</b>	<b>Offset* (m)</b>	<b>No. of SSP</b>	<b>Tip Elevations (m)</b>
West Approach 9+945	30	1	159.0
East Approach 10+055	30	1	174.0

Note: \* Approx. offset from centerline of Port Robinson Road

## **6.2 MATERIALS**

### **6.2.1 General**

The Contractor shall supply material and equipment, required for installation of the standpipe piezometers.

### **6.2.2 Pipe and Couplings**

The Contractor shall supply Schedule 40 flush jointed – 19 mm (3/4”) PVC pipe (e.g. 75x5R or 75x10R – Canadian Pipe Supply Ltd.).

### **6.2.3 Perforated Section**

The Contractor shall supply one 1.5 m long slotted Schedule 40 flush-jointed – 19 mm (3/4”) PVC slotted pipe (e.g. 75x5S Slot 10 Sch 40 – F/J – PVC – Canadian Pipe Supply Ltd.) for each SSP.

### **6.2.4 Bottom Cap**

The Contractor shall supply bottom caps Schedule 40 flush-jointed – 19 mm (3/4”) PVC (e.g. 448–007FJ – Canadian Pipe Supply Ltd.) to fit the perforated section.

### **6.2.5 Top Caps**

The Contractor shall supply vented top caps Schedule 40 – 19 mm (3/4”) PVC (e.g. 448-007FJ-perforated – Canadian Pipe Supply Ltd.) to fit the pipe.

### **6.2.6 Filter Sand**

The Contractor shall supply clean washed sand for backfilling around perforated section. The sand shall be Sakcrete washed general purpose sand – or equal.

### **6.2.7 Bentonite**

The Contractor shall supply bentonite (OPSS 1205) in pellet form for backfilling above the filter sand.

### **6.2.8 Grout**

The Contractor shall supply cement-bentonite grout for general backfilling. A suitable grout mix design consists of 23 kg of bentonite (OPSS 1205), 143 litres of water and 40 kg of cement (Type G.U. – OPSS 1301).

### **6.2.9 Protective Housing**

The Contractor shall supply a protective housing consisting of 100 mm minimum diameter galvanized steel pipe with a locking cap.

## **6.3 INSTALLATION**

### **6.3.1 General**

Installation of the standpipe shall be as per the Contract Drawings provided in addition to what is stated or emphasized below.

The borehole shall be advanced to 300 mm below the tip elevation using suitable drilling techniques. The sides of the borehole shall be stable and the borehole shall be free of debris.

The standpipe piezometers must be of sufficient length above the ground surface to accommodate the piezometric head and to allow for snow accumulation.

The standpipe piezometer location shall be at sections indicated on the Contract Drawings.

## **6.4 COORDINATING WITH MONITORING**

### **6.4.1 Notification**

The Contractor shall notify the Contract Administrator no later than 3 days after installing a standpipe. At this time, the Contractor shall also supply the following information to the Contract Administrator.

- Standpipe piezometer location, easting, northing referenced to MTM NAD83 coordinates;
- Elevation of ground level referenced to Geodetic datum;
- Stratigraphic log of subsurface conditions at the standpipe;
- Dates of installation;
- Depth of pipe, stick-up; and
- Installation notes/backfilling notes.

### **6.4.2 Monitoring**

Monitoring of standpipe piezometers shall be done by others. Monitoring shall be conducted during and after piling at the abutments, embankment fill construction and preload period. The Contractor shall provide installation information as specified above and provide access to the standpipe piezometers for monitoring including, but not necessarily limited to snow clearing in the winter. The Contractor shall provide electric power and general area lighting as needed for reading the instruments.

## **7.0 DECOMMISSING OF INSTRUMENTS**

### **7.1 General**

The Contractor shall decommission all the Settlement Plates (SP), VW piezometers (VWP), and Standpipe Piezometers (SSP) at the end of the monitoring program following construction unless advised otherwise by the Contract Administrator. Decommissioning of instrumentation shall be carried out according to the Ontario Water Resources act, R.R.O. 1990, Regulation 903 (as amended by Ontario Reg. 372).

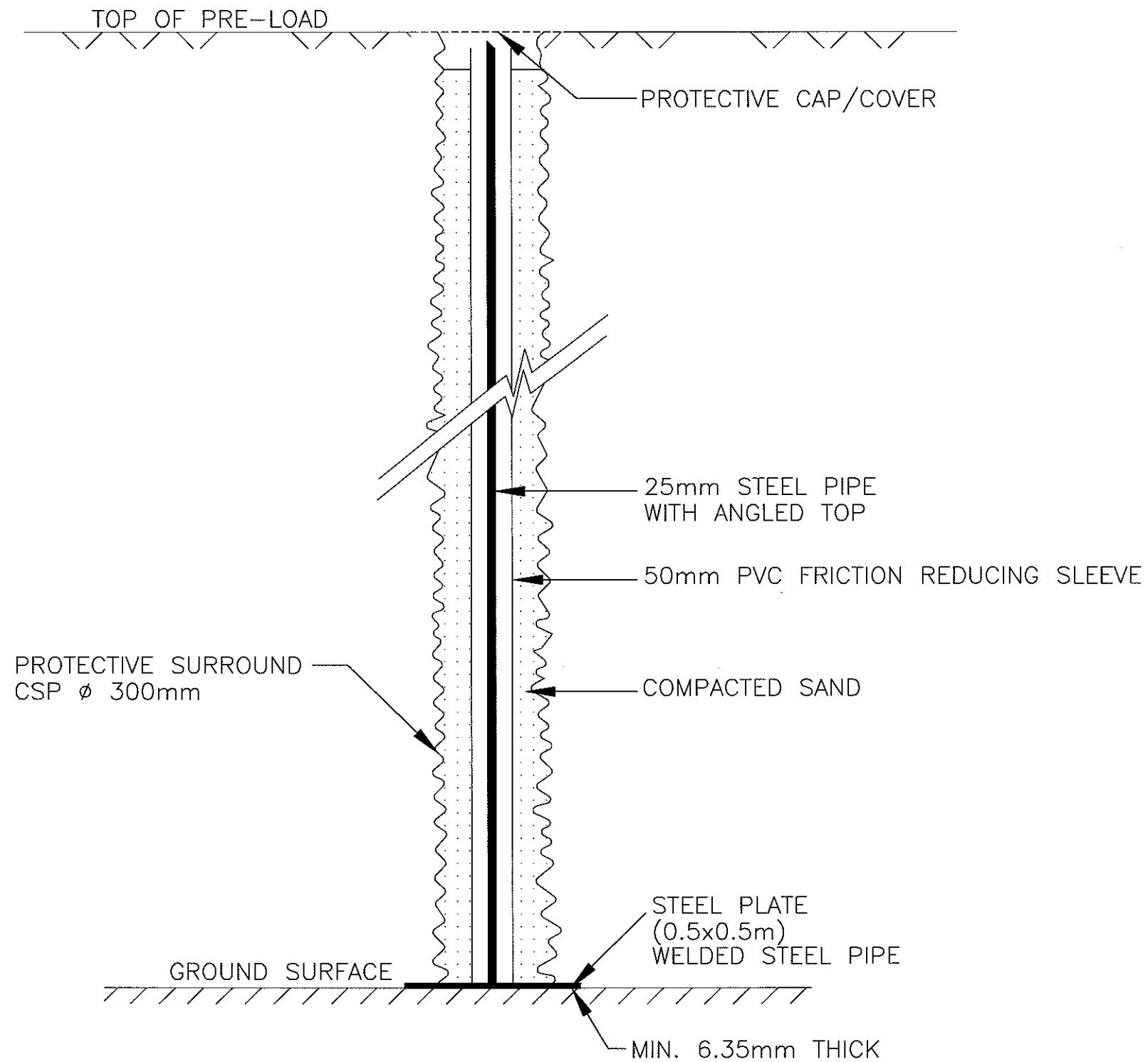
## **8.0 PAYMENT**

### **8.1 Basis Of Payment**

Payment at the Lump Sum price for this tender item shall be full compensation for all labour, monitoring equipment and material to do the work.



C:\Users\jw\Documents\1-09-4135 HWY 406 PORT ROBINSON ROAD\SETTLEMENT PLATE\09-4135 HWY 406 SETTLEMENT PLATE DETAIL.dwg, KAWAL



SETTLEMENT PLATE (SP)  
N.T.S.

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

CONT No  
WP No 280-99-00

HWY 406  
PORT ROBINSON UNDERPASS  
SETTLEMENT MONITORING  
INSTRUMENT DETAILS

 **Terraprobe Inc.**  
Consulting Geotechnical & Environmental Engineering  
Construction Materials, Inspection & Testing  
10 Bram Court - Brampton Ontario L6W 3R6 (905) 796-2650

GENERAL NOTES:

- THIS DRAWING TO BE READ IN CONJUNCTION  
WITH THE SETTLEMENT MONITORING  
INSTRUMENT LAYOUT DWG.

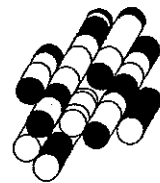


REVISIONS						
	DATE	BY	DESCRIPTION	DESIGN	CODE	LOAD
				R.A.	CHBDC2006	SEPT. 2010
				K.C.	CHK	STRUCT

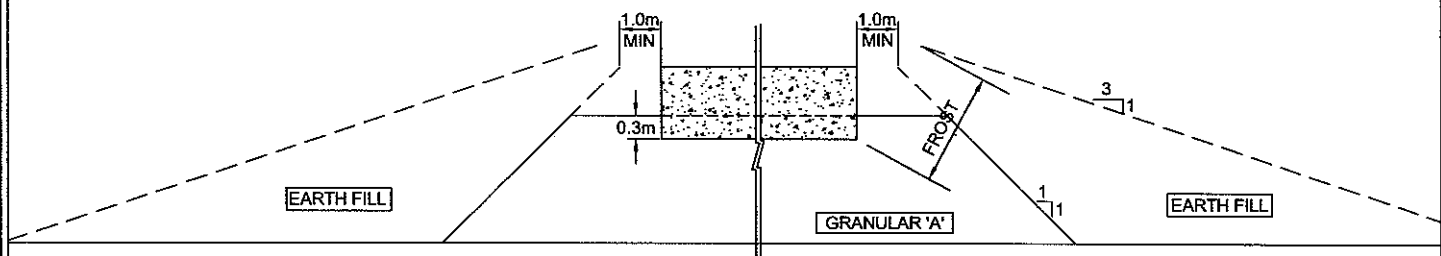
# APPENDIX H

## Figures

**Terraprobe Inc.**

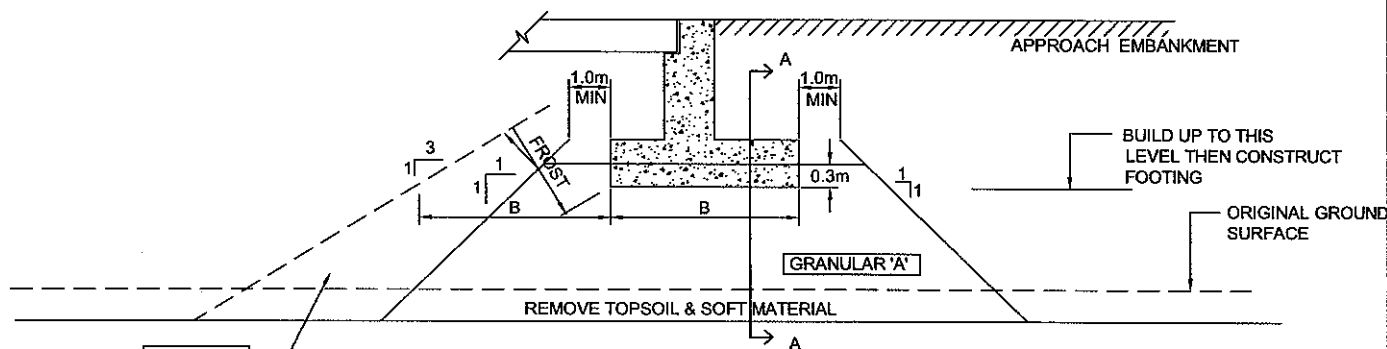






CROSS-SECTION A-A

NOT TO SCALE



LONGITUDINAL SECTION

NOT TO SCALE

NOTES:

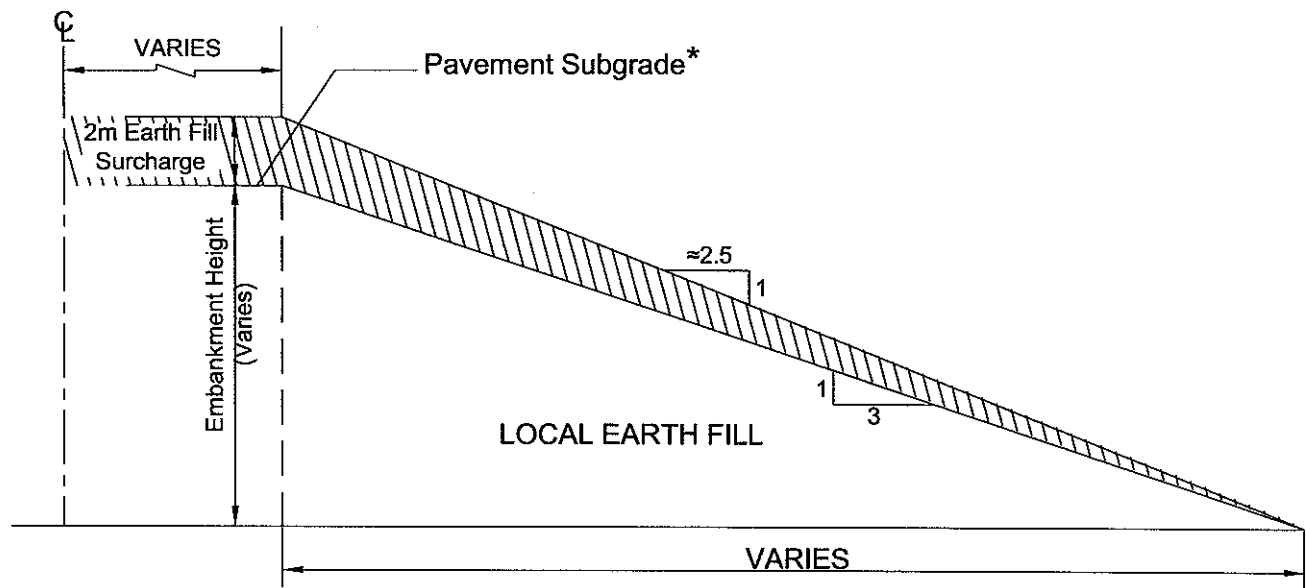
1. REMOVE TOPSOIL AND OR SOFT SUBSOIL UNDER AREA OF COMPACTED GRANULAR 'A' AND EARTH FILL.
2. PLACE GRANULAR 'A' AND EARTH FILL TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO O.P.S.S.501.
3. CONSTRUCT CONCRETE FOOTING.
4. PLACE REMAINDER OF GRANULAR 'A' AND EARTH FILL AS REQUIRED.
5. SOURCE M.T.C. 1982.

ENGINEER	R.A.
DRAWN	K.C.
DATE	SEPT. 2010
APPROVED	M.T.
SCALE	N.T.S.

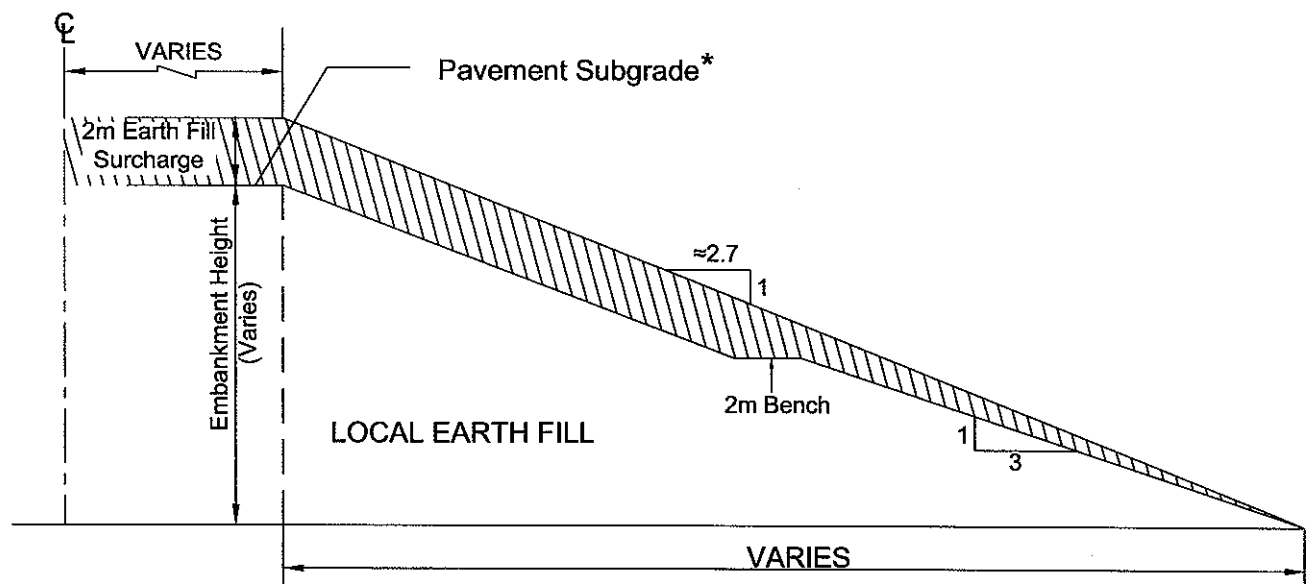
# ABUTMENT ON COMPACTED FILL SHOWING GRANULAR A CORE

**Terraprobe Inc.**  
Consulting Geotechnical & Environmental Engineering  
Construction Materials, Inspection & Testing  
10 Brian Court - Brampton Ontario L6Y 3N2 (905) 756-2525

FIGURE H1



Local Earth Fill Embankment < 8m

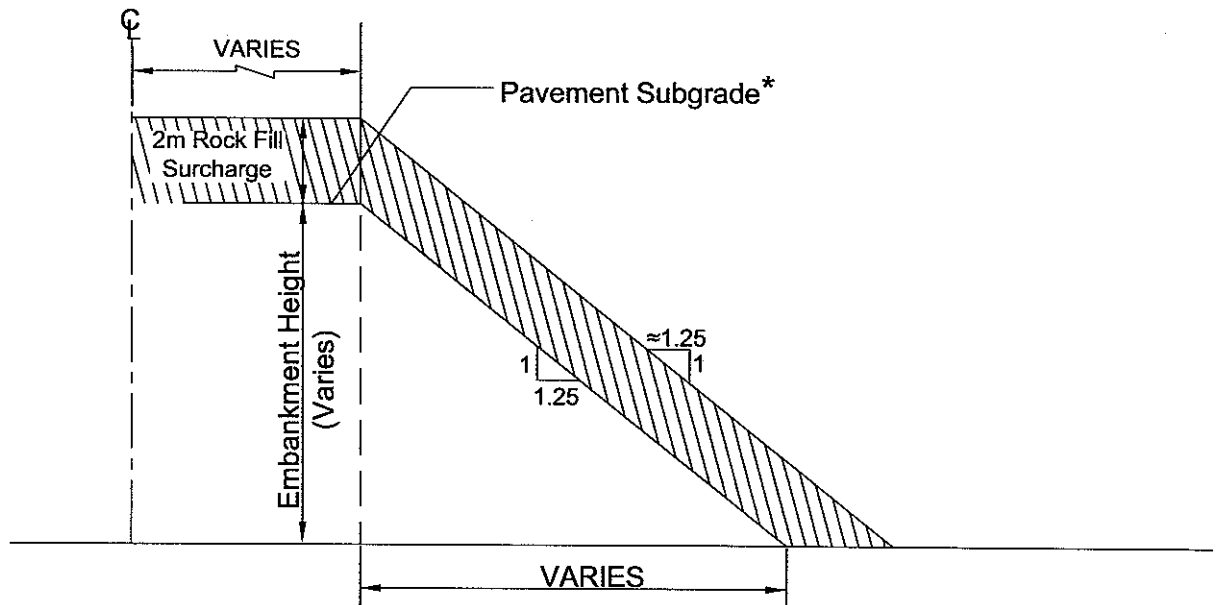


Local Earth Fill Embankment 8m  $\geq$  12m

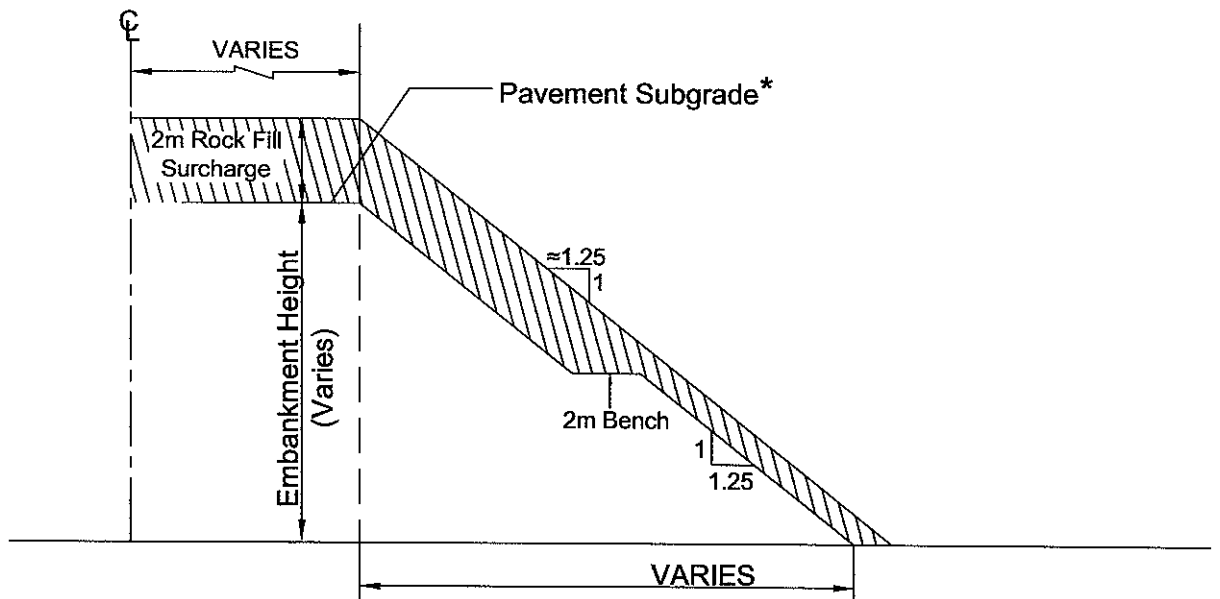
\* Notes- Pavement subgrade to be established after removal of surcharge

N.T.S

## SURCHARGE ARRANGEMENT



Rock Fill Embankment <10m



Rock Fill Embankment 10m  $\geq$  12m

\* Notes- Pavement subgrade to be established after removal of surcharge.

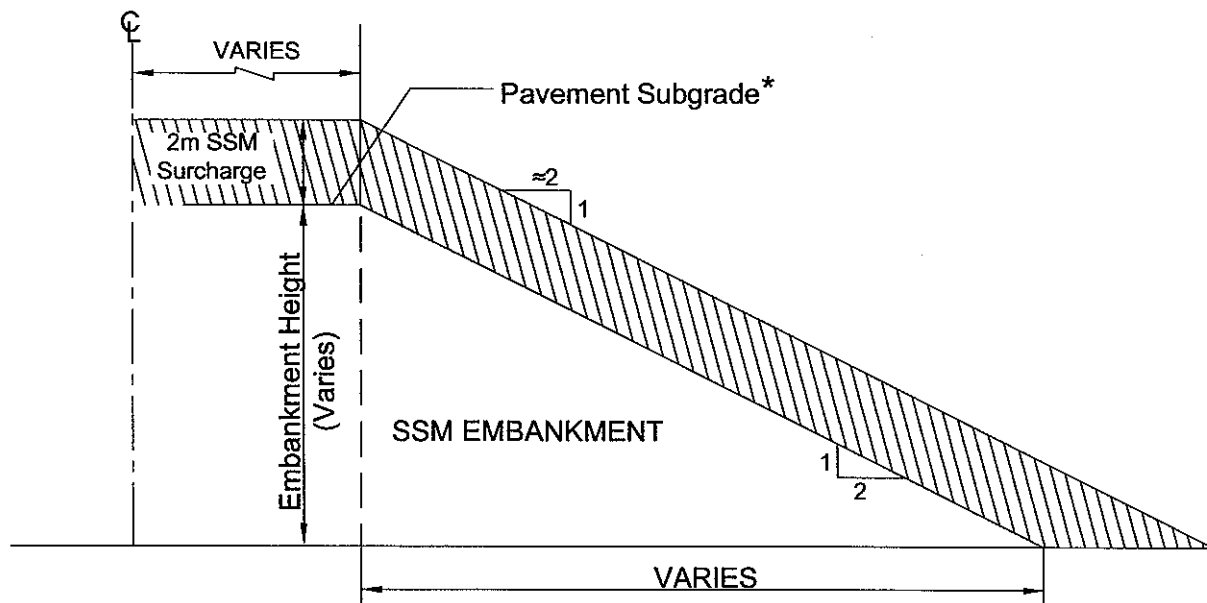
N.T.S

## SURCHARGE ARRANGEMENT

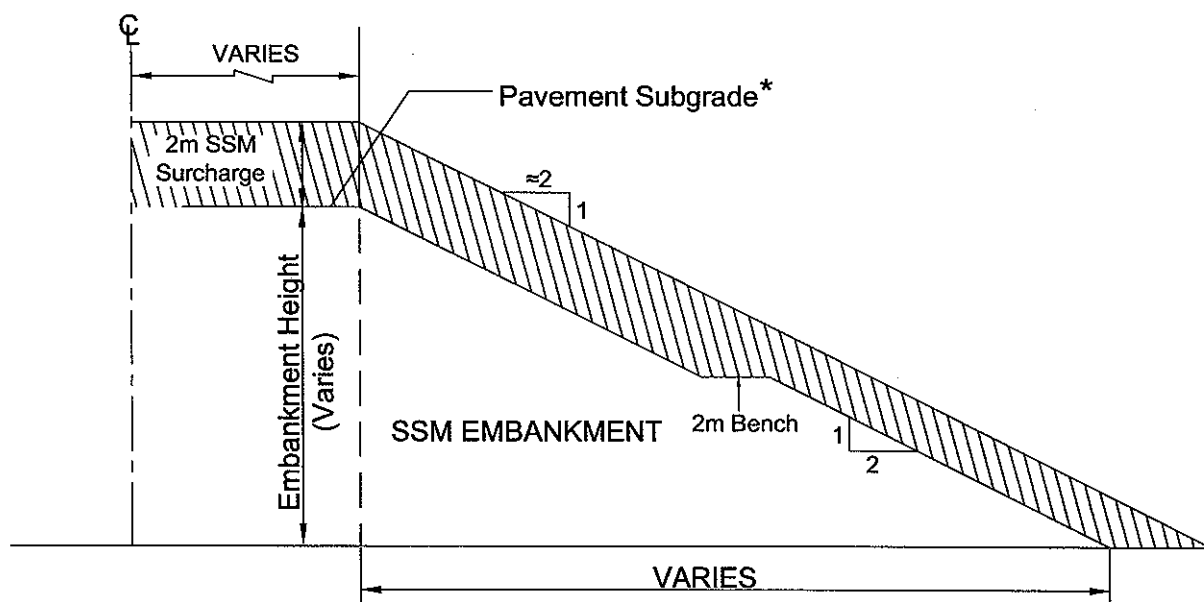
TERRAPROBE

File No. 1-09-4135

FIGURE H3



SSM Embankment <8m



SSM Embankment 8m  $\geq$  12m

\* Notes- Pavement subgrade to be established after removal of surcharge.  
Only SSM surcharge recommended in order to minimize handling/sorting and compaction of dissimilar materials.

N.T.S

## SURCHARGE ARRANGEMENT