



Terraprobe

Consulting Geotechnical & Environmental Engineering
Construction Materials Inspection & Testing

**FOUNDATION INVESTIGATION & DESIGN REPORT
WOODLAWN ROAD OVERPASS, HIGHWAY 406 SBL
HIGHWAY 406 TWINNING
PORT ROBINSON ROAD TO EAST MAIN STREET
AGREEMENT No. 2008-E-0016, W.P. 280-99-00, SITE: 34-463/2
GEOCRES NO. 30M3-260**

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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 new interchange is proposed at Woodlawn Road and a single span overpass is required to carry Highway 406 SBL traffic over the proposed Woodlawn Road.

The main design recommendations are:

- The bridge should be supported on pile foundations.
- Downdrag loads should be considered in the foundation design if piles are installed prior to settlement being complete. Downdrag loads on the piles can be eliminated by preloading using a temporary retaining system or by using EPS Geofoam.
- The soils at this site are settlement sensitive and the estimated settlement due to approach fill placement is expected to range between 90 mm and 130 mm depending on the material used. Detailed analyses indicate that after the first six months of embankment construction the remaining post-construction settlement will be equal to or less than the acceptable maximum of 25 mm. Therefore other means/methods (wick drains) of accelerating the settlement are not warranted.
- Given the uncertainty in accurately predicting the time rate of settlement we recommended that conventional temporary surcharging be carried out (2 m of additional earth fill height) to accelerate the settlement and ensure full consolidation within 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.



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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted at the Woodlawn Overpass bridge site on the proposed Highway 406 SBL in the City of Welland, 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.

2 SITE DESCRIPTION & PHYSIOGRAPHY

The site is located approximately 50 m north of the existing at grade intersection of Highway 406 and Woodlawn Road/Daimler Parkway in the City of Welland, 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 generally 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 basins¹.

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

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



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 Salina Formation of Upper Silurian Age². This unit consists essentially of easily weathered, grey, very finely crystalline, laminated argillaceous dolostone with grey, calcareous shale partings and gypsum veins and lenses of varying thicknesses.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project were carried out between November 10, 2009 and April 29, 2010 and consisted of drilling and sampling six boreholes to depths ranging from 11.3 m to 32.2 m. The boreholes were numbered SBL 12+685CL, SBL 12+750CL, WS1, WS2, WS3 and WS4 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 some specific borehole locations was difficult due to locally steep slopes. The locations of these boreholes were selected 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.

² 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
SBL 12+685CL	10.7/172.0	Piezometer with 3.0 m slotted screen installed with filter sand to 7.0 m, bentonite seal from 7.0 m to 6.4 m, drill cuttings from 6.4 m to 0.3 m, bentonite seal from 0.3 m to 0.2 m and a flush mounted casing installation from 0.2 m to ground surface.
SBL 12+750CL	10.5/172.4	Piezometer with 3.0 m slotted screen installed with filter sand to 6.9 m, bentonite seal from 6.9 m to 6.6 m, drill cuttings from 6.6 m to 0.3 m and bentonite seal from 0.3 m to ground surface.
WS1	22.9/159.8	Hole sealed to 22.9 m with bentonite, piezometer with 1.5 m slotted screen installed with filter sand to 20.4 m, bentonite seal from 20.4 m to ground surface.
WS4	24.4/158.3	Hole sealed to 24.4 m with bentonite, piezometer with 1.5 m slotted screen installed with filter sand to 21.9 m, bentonite seal from 21.9 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, unconfined compression tests 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 and about 25.9 m to 27.5 m of overburden soils consisting of topsoil, fill material (sand and gravel and silty clay) and native deposits of silty clay, silt, silty clay till, sandy silt to sand and silt till, and clayey silt till. These soils are underlain by bedrock of the Salina Formation.



5.1 Topsoil

Topsoil ranging from 25 mm to 200 mm in thickness was encountered at this site. Topsoil thickness may vary between and beyond the boreholes.

5.2 Fill – Sand and Gravel

Borehole SBL 12+685CL was drilled on the gravel shoulder of the dedicated right turn lane that carries Highway 406 north bound traffic to Woodlawn Road. The sand and gravel fill is approximately 470 mm thick and extends to a depth of 0.5 m (Elev. 182.2 m) below ground surface.

A sample of this fill material was subjected to a grain size analysis and the results are presented in Figure B1. These results show a grain size distribution consisting of 41% gravel, 42% sand, 13% silt and 4% clay size particles.

A Standard Penetration test in the sand and gravel fill gave an 'N' value of 11 blows for 0.3 m penetration indicating a compact relative density. The moisture content of a sample of this fill was 4% by weight.

5.3 Fill – Silty Clay

Silty clay fill material was encountered at this site extending to depths ranging from 0.7 m (Elev. 182.2) to 2.9 m (Elev. 180.1) below ground surface. Silty clay fill was not encountered in Boreholes SBL12+685CL and WS2.

Samples of this fill material were subjected to grain size analysis and the grain size distribution curves are illustrated in Figure B2. These results show a grain size distribution consisting of 0% gravel, 2-3% sand, 36% silt and 61-62% clay size particles.

Samples of the fill were also subjected to Atterberg Limits tests and the results are plotted on the plasticity chart, Figure B3. The index values from this test are summarized below:

Liquid Limit:	49-53%
Plastic Limit:	23-24%
Plasticity Index:	26-29%
Natural Moisture Content:	18-22%

These values are characteristic of clayey soils of intermediate to high plasticity.

Standard Penetration tests in the silty clay fill gave 'N' values that ranged from 7 to 43 blows for 0.3 m penetration but generally, the recorded 'N' values ranged from 17 to 43 blows for 0.3 m penetration. Based on these results the fill is considered to have a generally very stiff to hard consistency with occasional firm to stiff zones. The moisture content of samples of this fill ranged from 18% to 31% by weight.



5.4 Silty Clay

A major silty clay deposit was encountered across the site. This deposit was fully penetrated in some of the boreholes where it was found to extend to depths of 14.7 m below ground surface or to elevations ranging from 168.4 m to 168.0 m. The approach boreholes were terminated in this deposit at depths of 11.3 m (Elev. 171.6 m) and 12.2 m (Elev. 170.5 m).

The grain size distribution plots of tested samples of the silty clay are presented in Figures B4 to B8 inclusive. These results show a grain size distribution consisting of 0-8% gravel, 0-6% sand, 37-83% silt and 12-58% 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 B9 to B13 inclusive. The index values from these tests are summarized below:

Liquid Limit:	25-47%
Plastic Limit:	15-23%
Plasticity Index:	9-24%
Natural Moisture Content:	16-24%

These values indicate that the silty clay has a generally low to intermediate plasticity.

Standard Penetration tests in this stratum gave 'N' values that ranged from 7 to 61 blows for 0.3 m penetration but generally the recorded 'N' values ranged from 15 to 46 blows for 0.3 m penetration. Field vane tests gave in-situ undrained shear strengths ranging from 64 kPa to in excess of 100 kPa. An unconfined compression test gave an undrained shear strength of 77 kPa and laboratory vane tests on relatively undisturbed Shelby tube samples gave undrained shear strengths ranging from 49 kPa to 126 kPa. These values indicate that the consistency of the silty clay is generally stiff to hard with infrequent firm zones. The moisture content of samples of the silty clay range from 12% to 27% by weight and the unit weight of selected samples ranged from 20.7 to 21.1 kN/m³.

The variation of undrained shear strength with elevation is depicted in Figure B20. The plot illustrates a wide scatter in the data with no obvious trend with depth. An interpreted dashed line is shown representing a lower bound trend with depth, for the data. The upper portion of this deposit up to about Elev. 176.0 m is estimated to have relatively high undrained shear strength i.e. in excess of 125 kPa. Below Elev. 176.0 m the undrained shear strength decreases with depth and is about 50 kPa at Elev. 172.0 m. Below Elev. 172.0 m the trend indicates increasing undrained shear strength with depth.

The Atterberg Limits tests results are also plotted against elevation, Figure B21. These results illustrate that the natural moisture contents are generally at or below the plastic limit up to Elev. 177.0 m. Below Elev. 177.0 m the moisture content is generally slightly above the plastic limit.



Consolidation tests were also performed on Shelby tube samples retrieved from Boreholes SBL 12+685CL and SBL 12+750CL and the results are presented in Figures B22 to B27 inclusive. These results indicate estimated preconsolidation pressures ranging between 370 kPa and 550 kPa.

5.5 Silt

A native discontinuous silt deposit was encountered at this site in Boreholes WS1, WS2 and WS4. The deposit is approximately 0.9 m to 1.5 m thick and extends to depths of 5.9 m below ground surface or to elevations ranging from 177.2 m to 176.8 m. 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 curves of tested samples of the silt deposit are presented in Figure B14. These results show a grain size distribution consisting of 0-1% gravel, 1-2% sand, 75-79% silt and 20-22% clay size particles.

The deposit is considered to have a dense to very dense relative density based on SPT 'N' values that ranged from 36 to 64 blows for 0.3 m penetration. The moisture content of samples from this deposit ranged from 16% to 22% by weight.

5.6 Silty Clay Till

A deposit of silty clay till was encountered across the site extending to depths ranging from 18.0 m to 18.7 m below ground surface or to elevations ranging from 165.1 m to 164.0 m.

The grain size distribution curves of tested samples from this unit are depicted in Figure B15. These results show a grain size distribution consisting of 4-17% gravel, 17-24% sand, 48-55% silt and 17-18% clay size particles. Till soils will also contain random cobble and boulder inclusions.

Samples were also subjected to Atterberg Limits tests and the results are plotted on the plasticity chart, Figure B16. The index values from these tests are summarized below:

Liquid Limit:	21-23%
Plastic Limit:	14%
Plasticity Index:	7-9%
Natural Moisture Content:	10-13%

These values are characteristic of clayey soils of low plasticity.

Standard Penetration tests in this deposit yielded 'N' values ranging from 40 to more than 100 blows per 0.3 m penetration. A field vane test was also attempted in this deposit and the results (no-turn on vane) indicate undrained shear strengths more than 100 kPa. Based on these results the silty clay till is considered to have a hard consistency. The moisture content of samples retrieved from this stratum varies from 7% to 16% by weight.



5.7 Sandy Silt to Sand and Silt Till

The site is underlain by discontinuous deposits of sandy silt till and a deposit of sand and silt till. These units extend to depths ranging from 25.9 m (Elev. 157.1 m) to 27.5 m (Elev. 155.2 m) below ground surface.

The grain size distribution plots of tested samples from these till deposits are depicted in Figure B17. These results show a grain size distribution consisting of 2-26% gravel, 10-38% sand, 36-72% silt and 8-16% clay size particles. Till soils will also contain random cobble and boulder inclusions.

Standard Penetration tests in these deposits yielded 'N' values ranging from 65 to more than 100 blows per 0.3 m penetration. Based on these results these units are considered to have a very dense relative density. The moisture content of samples from these deposits varies from 3% to 13% by weight.

5.8 Clayey Silt Till

A deposit of clayey silt till was encountered at this site in Boreholes WS1, WS2 and WS3. The deposit is approximately 1.6 m to 2.5 m thick and extends to a depth of 22.3 m below ground surface or to elevations ranging from 160.8 m to 160.4 m.

The grain size distribution plots of tested samples from this till deposit are depicted in Figure B18. These results show a grain size distribution consisting of 2-15% gravel, 31-35% sand, 35-48% silt and 15-19% clay size particles. Till soils will also contain random cobble and boulder inclusions.

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

Liquid Limit:	15-17%
Plastic Limit:	11-12%
Plasticity Index:	4-5%
Natural Moisture Content:	7-15%

These values indicate a clayey silt matrix of low plasticity.

Standard Penetration tests conducted in this stratum yielded 'N' values ranging from 44 to more than 100 blows per 0.3 m penetration. Based on these results the silty clay till is considered to have a hard consistency. The moisture content of samples of this till varies from 7% to 15% by weight.



5.9 Bedrock (Salina Formation)

The overburden soils described above are underlain by the Salina Formation. Bedrock was proved by coring at the abutment 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)
South Abutment	WS1	27.5	155.2
	WS2	27.3	155.8
North Abutment	WS3	25.9	157.1
	WS4	26.3	156.4

The bedrock is described as unweathered dolostone and shale and its colour is generally grey. It is thinly laminated with white unweathered gypsum and calcite veins. Total core recovery in the bedrock generally ranged from 58% to 100% and one recorded TCR of 19% was obtained in Run 1 of Borehole WS1. The RQD values ranged widely from 12% to 49% and an RQD of 0% was obtained in Run 1 of Borehole WS1. Rubble and highly fractured zones were observed in the rock cores which contributed to the relatively low RQD values. The core data reveals that there is generally no trend of improving rock quality with depth. Based on these results the rock quality is considered to be very poor to poor.

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)
SBL 12+685CL	Piezometer destroyed	-	-
SBL 12+750CL	November 19, 2009	4.6	178.3
	November 30, 2009	7.8	175.1
	December 08, 2009	4.1	178.8
	December 15, 2009	3.2	179.7
	January 04, 2010	2.7	180.2
	January 11, 2010	2.7	180.2
	January 19, 2010	2.6	180.3
WS1	January 27, 2010	6.8	175.9
	February 08, 2010	6.8	175.9
WS4	February 08, 2010	4.7	178.0
	April 16, 2010	2.1	180.6
	April 29, 2010	5.1	177.6
	May 04, 2010	5.7	177.0
	May 06, 2010	4.1	178.6
	May 18, 2010	5.9	176.8

The ground water table was estimated based on the recorded water levels in the standpipe piezometers and our review of moisture contents of the retrieved samples. Based on these observations, the local ground water level is estimated at approximately Elev. ± 181.0 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 and truck mounted drill rigs owned and operated by Groundworks Drilling Limited of Toronto, Ontario, DBW Drilling Limited of Ajax, Ontario and Determination Drilling & Soil Investigations of Hamilton, Ontario.

A combination of hollow-stem auger and solid stem auger drilling techniques and casing and washboring methods were used to advance the boreholes. NQ size rock cores of the bedrock were obtained using diamond drilling techniques.

Messrs. Alexander Winkelmann, E.I.T., Marc Paoliello, E.I.T, and Phil Khuu, B.A.T, carried out the field work. 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.

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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.

Highway 406 SBL will cross above the proposed Woodlawn Road via a single span structure approximately ± 13.2 m wide and measuring ± 31 m in length between abutments. The proposed finished grades at the structure will be about Elev. ± 190.8 m at the south abutment and Elev. ± 190.1 m at the north abutment. At the abutments the approach fill is estimated to range in height between ± 7 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 single span structure with two abutments as foundation elements.

The stratigraphy encountered at the abutment locations consist of 25.9 m to 27.5 m of overburden soils overlying bedrock of the Salina Formation. The overburden includes compressible soils consisting of about 2.0 m to 2.8 m of silty clay fill material and a native silty clay deposit that ranges in thickness from 11.1 m to 13.2 m. The ground water level at this site is about ± 181.0 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 a conventional abutment supported on spread footings is not considered to be a feasible and practical option. The soils at this site are settlement sensitive and the abutment fills will trigger time dependent consolidation settlements that are estimated to exceed 25 mm total settlement. Consequently, spread footings on native soils are not considered to be a feasible alternative.

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 settlement. 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. 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
WS1	South Abutment	180.5
WS2		182.5
WS3	North Abutment	180.0
WS4		180.5

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. This settlement is expected to be substantially complete by the end of construction.

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. The base of the caissons will have to be made in relatively permeable sandy silt till soils containing cobbles and boulders below the ground water table. Cobbles and boulders (if encountered) will cause construction challenges.

It would also be difficult to seal the bottom of the liner to exclude ground water due to the permeable nature of the overburden soils and the presence of cobbles and boulders. Unwatering the caisson, maintaining a sufficiently dry excavation and constructing a relatively undisturbed base would also be challenging and impractical.

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 H-piles are likely to encounter practical refusal in the very dense sandy silt till and sand and silt till at both foundation elements. The single span bridge is most likely to be designed with integral or semi-integral abutments and therefore H-piles are the most suitable. Steel tube piles are much less suitable and not likely to be considered.

7.3.1 Axial Resistance

Two steel pile sections believed to be currently available have been considered for use in the proposed foundations. Piles driven at the abutment locations and encountering effective refusal in the very dense sandy silt till and sand and silt till should be designed on the basis of the concentric, axial geotechnical resistances given in Table 7.2. The actual pile tip elevations will be controlled as described in Section 7.3.6 and 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)	SLS (25 mm Settlement) (kN)
South Abutment	WS1	160.0±	Sandy Silt Till	1600	1200
	WS2	160.0±	Sandy Silt Till		
North Abutment	WS3	158.0±	Sandy Silt Till		
	WS4	158.0±	Sand and Silt Till		

Table 7.2 – Axial Resistance of Various Pile Sections

Location	PILE TYPE – HP 360X132				
	Reference Borehole	Estimated Pile Tip Elevation (m)	Founding Stratum	Factored Axial Resistance U.L.S (kN)	SLS (25 mm Settlement) (kN)
South Abutment	WS1	159.0±	Sandy Silt Till	2100	1600
	WS2	159.0±	Sandy Silt Till		
North Abutment	WS3	158.0±	Sandy Silt Till		
	WS4	157.0±	Sand and Silt Till		



The H-piles for the recommended foundation scheme will be driven to effective refusal in very dense till soils. 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 and reinforcement to the pile section.

Cobbles exist within the matrix of the underlying till soils and some piles may encounter effective refusal above the estimated pile tip elevations. The contract documents should contain a NSSP alerting the contractor to the fact that cobbles and boulders may be encountered in the overburden soils. The NSSP should require the QVE to terminate driving before the pile is damaged by overdriving. If the required resistance according to the Hiley Formula is not achieved and further driving is likely to cause damage to the pile, pile driving should cease and the contract administrator should be notified. 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 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 a compressible silty clay stratum that extends to Elev. 168.0 m. The unfactored loads recommended for design are

- HP 310 x 110 – 800 kN/pile
- HP 360 x 132 – 925kN/pile

Constructing the approach fills approximately 1 year in advance of the pile driving operations (to preload the underlying soils) can significantly reduce downdrag forces on the piles. Settlement monitoring is required to ensure that settlement is essentially complete prior to pile driving operations. A permanent RSS wall can also be constructed with suitably sized CSP's installed to facilitate pile installations (within the CSP) after settlement is complete.

Alternatively, EPS Geofoam can be used (estimated volume of 20,000 m³) to reduce the magnitude of downdrag forces to a negligible amount provided that the geofoam extends a lateral distance of 30 m from the bridge abutments. Conventional earth fill can be used beyond this limit.

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 in the RSS core. In order to provide the upper 3 m of the pile with the required flexibility, and to ensure that superstructure movement does



not damage the RSS wall a 2-CSP system is recommended as per MTO SO-96-01. An outer CSP is placed around an inner sand filled CSP (about 600 mm in diameter).

The space between the pile and the inner 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) (kN/m³)

γ = unit weight (Table 7.4) (kN/m³)

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



Table 7.4 – Recommended Soil Parameters

Area Reference Borehole No	Applicable Elevation	Soil Type	Bulk Unit Weight (kN/m ³)	Angle of Internal Friction (φ) Degrees	Undrained Shear Strength (S _u) (kPa)	Recommended n _h Value (kN/m ³)*
South Abutment WS1	182.6 – 180.6	Fill – Silty Clay	18	0	50	–
	180.6 – 178.3	Silty Clay	20	0	150	–
	178.3 – 176.8	Silt	19	30	–	8500
	176.8 – 168.0	Silty Clay	20	0	150	–
	168.0 – 164.1	Silty Clay Till	21	0	225	–
	164.1 – 162.1	Sandy Silt Till	20	35	–	10000
	162.1 – 160.4	Clayey Silt Till	21	0	225	–
	160.4 – 155.2	Sandy Silt Till	21	35	–	10000
South Abutment WS2	183.1 – 182.4	Silty Clay	20	0	50	–
	182.4 – 178.7	Silty Clay	20	0	150	–
	178.7 – 177.2	Silt	19	30	–	8500
	177.2 – 168.4	Silty Clay	20	0	80	–
	168.4 – 165.1	Silty Clay Till	21	0	225	–
	165.1 – 162.4	Sandy Silt Till	20	35	–	10000
	162.4 – 160.8	Clayey Silt Till	21	0	225	–
	160.8 – 155.8	Sandy Silt Till	21	35	–	10000
North Abutment WS3	182.9 – 180.1	Fill – Silty Clay	18	0	50	–
	180.1 – 168.3	Silty Clay	20	0	100	–
	168.3 – 164.8	Silty Clay Till	21	0	225	–
	164.8 – 163.2	Sandy Silt Till	20	35	–	10000
	163.2 – 160.7	Clayey Silt Till	21	0	225	–
	160.7 – 157.1	Sandy Silt Till	20	35	–	10000
North Abutment WS4	182.6 – 180.6	Fill – Silty Clay	18	0	50	–
	180.6 – 177.7	Silty Clay	20	0	150	–
	177.7 – 176.8	Silt	19	30	–	8500
	176.8 – 168.0	Silty Clay	20	0	90	–
	168.0 – 164.0	Silty Clay Till	21	0	225	–
	164.0 – 156.4	Sand and Silt Till	20	35	–	10000

* Values estimated based on Table 20.3 data, Canadian Foundation Engineering Manual, 3rd 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, the lateral resistance may be provided by battered piles.

7.3.5 Pile Tips

Due to the presence of till layers that may contain cobbles and boulders, 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 in the overburden soils.

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.

7.3.7 Pile Driving

Pile driving should be controlled by the Hiley Formula and an ultimate pile resistance to be specified by the designer in accordance with Clause 3.3.2 (b) Construction Stage of the Structural Manual. The appropriate pile driving note is “Piles to be driven in accordance with Standard SS 103-11 using an ultimate resistance of “R” kN per pile”. Piles should be driven with a suitable hammer capable of delivering a rated energy of at least 60 kJ/blow, but not more than 70 kJ/blow.

“R” must have the minimum values shown in Table 7.5.

Table 7.5 – Ultimate Geotechnical Resistance of Piles

Pile	Ultimate Resistance (R) (kN)
HP 310X110	3,200 kN
HP 360X132	4,200 kN



Hiley formula calculations need not be carried out until the pile has been driven below Elev. 161 m at the south abutment and Elev. 159 m at the north abutment.

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. 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. ± 181.0 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 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 was considered when selecting approach embankment alternatives for this bridge and 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		
	ϕ (degrees)	c (kPa)	γ (kN/m ³)	ϕ' (degrees)	c' (kPa)	γ (kN/m ³)
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	27	5	18.0
Silty Clay	0	60 – 150	20.0 – 20.5	27 – 29	5 – 7	20.0 – 20.5
Silt	30	0	19.0	30	0	19.0
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

Embankment Composition	Design Side Slope	Minimum Factor of Safety Short-Term	Minimum Factor of Safety Long-Term
Local Earth Fill	3H:1V	2.0	2.0
Rock Fill	1.25H:1V	1.6	1.6
SSM	2H:1V	2.0	1.8
RSS Embankment	2H:1V	1.5	1.4
Light Weight Fill	2H:1V	2.2	2.2
Ultra Light Weight Fill	2H:1V	2.2	2.2

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 greater than the target factor of safety of 1.3 for the recommended design side slopes.

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 these figures.

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 Kulhawy & Mayne (1990), Terzaghi & Peck (1967), Nagaraj & Murty (1985) 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 that an over-consolidated desiccated upper crust exists within the 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
Preconsolidation Pressure Range - P_c (kPa)	600 to 450 500 to 400	450 to 300 400 to 300
Coefficient of Compressibility - C_c	0.20 to 0.22	0.15 to 0.18
Recompression Index - C_r	0.04	0.020 to 0.029
Initial Void Ratio - e_o	0.9	0.6

Settlement analyses were undertaken for various embankment compositions and geometries and the estimated range of total settlements 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 Centreline - 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	120 – 130
Rock Fill	19.0	1.25H:1V	120 – 130
Select Subgrade Material	20.0	2H:1V	120 – 130
Lightweight Fill*	14.5	2H:1V	100 – 110
Ultra Lightweight Fill*	11.5	2H:1V	90 – 100
EPS Geofoam	0.31	2H:1V	Negligible

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

The approach 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 is 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 $3.3 \times 10^{-3} \text{ cm}^2/\text{s}$ and $5 \times 10^{-3} \text{ cm}^2/\text{s}$.

Tabulated below is the range of predicted settlements (below embankment centreline) at various time periods after embankment construction.

Woodlawn Overpass Hwy 406 SBL					
Embankment Type	Settlement At Various Time Periods (mm)				Total Settlement (mm)
	6 months	12 months	18 months	24 months	
Local Earth Fill	105 – 110	110 – 120	115 – 125	120 – 130	120 – 130
Rock Fill	105 – 110	110 – 120	115 – 125	120 – 130	120 – 130
Select Subgrade Material	105 – 110	110 – 120	115 – 125	120 – 130	120 – 130
Lightweight Fill	90 – 95	95 – 105	95 – 105	100 – 110	100 – 110
Ultra Lightweight Fill	80 – 85	85 – 90	85 – 90	90 – 100	90 – 100

It is understood that a maximum allowable post-construction settlement of about 25 mm would be considered acceptable. This data shows that after 6 months, the post-construction consolidation settlement will be 25 mm or less (required performance) for all embankment material types. Therefore other means/methods (wick drains) of accelerating the settlement are not warranted. However, a settlement monitoring program (Appendix G) must be conducted to confirm the anticipated settlement performance.

Given the uncertainty in accurately predicting the time rate of settlement, we recommend that conventional temporary surcharging be carried out (2 m of additional earth fill height) to accelerate the settlement and ensure full consolidation within the target 6 months after embankment construction (Figures H2 to H4). 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.

Temporary surcharging will require a temporary retaining system to retain the forward slopes of the fill at the abutments if Woodlawn Road is constructed in advance of the structure. This retaining system can consist of either gabion baskets or stacked concrete blocks installed with a 1H:1V inclination (Figure H5).

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)
North Approach	±0.2	±182.7
South Approach	±0.0	±182.7

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 RETAINED SOIL SYSTEMS

The general arrangement drawings indicate that retaining walls will be used at both abutments to retain the forward slope of the approach fills. The soils at this site are settlement sensitive and this aspect must be taken into account when designing RSS walls for this site.

A standard one-stage MSE wall is not recommended because the facing panels cannot accommodate the estimated settlement. However, a two-stage RSS system can be used at this site for a false abutment as well as for wing walls. This RSS system has provisions for constructing a cast-in-place facing after settlement is complete

RSS walls should be specified to be “High Performance” and “High Appearance”. The contract drawings should include information on the longitudinal alignment of the wall in plan, the top and base elevations of the wall in profile, cross-sectional space constraints and a NSSP for the RSS wall. The design, supply, and construction of RSS should be in accordance with SP 599S22. Materials quality control and quality assurance testing and acceptance criteria for precast concrete facing elements should be in accordance with SP 599S23.



11.1 Foundation

The performance of an RSS is dependent, among other factors, on the characteristics of its foundation. Failure to provide an adequate foundation may lead to settlement and distortion of the RSS and, in severe cases, to possible failure of the system.

At this site, it is recommended that the levelling pad be centred on top of a pad of engineered fill consisting of OPSS Granular A compacted in accordance with the OPSS 501, Method A. The engineered fill pad should be at least 500 mm thick and should be at least twice as wide as the levelling pad.

In addition to the requirements for the levelling pad, the RSS can be founded on the native silty clay or well compacted fill. All topsoil, organics and soft soils should be removed from the subgrade of the RSS wall prior to placement of the fill. The founding subgrades should be inspected and evaluated by the Quality Verification Engineer (QVE), at the time of construction.

The recommended geotechnical resistances and construction details for the proposed RSS walls are tabulated below.

RSS Location	RSS Wall Base Elev. (m)	Max. Wall Height (m)	Required SLS Bearing Resistance (kPa)	Recommended Geotechnical Resistances (kPa)		Relevant Borehole & Subgrade Soil	Additional Requirements
				ULS **	SLS		
North Abutment Face	±182.5	4.5	135	425	N/A*	WS3/WS4 Silty Clay Fill	Settlement monitoring required.
South Abutment Face	±182.0	5.0	150	425	N/A*	WS1/WS2 Silty Clay, Silty Clay Fill	Settlement monitoring required.

* Settlement will be greater than 25 mm and a geotechnical resistance for 25 mm settlement is not provided.

** ULS values are factored.

The settlement of RSS walls at this site will depend on the material used, the foundation soils and the quality of construction. Time dependent consolidation settlement will occur in the underlying silty clay deposit and the total settlement at the abutment locations (where a vertical wall is proposed) is expected to range between 90 mm and 130 mm. The facing should be designed to accommodate the settlements outlined in Table 10.4 and a 12 month target time for facing installation is recommended. However, the timing for installation will be contingent upon approval by the geotechnical engineer and the contract documents should contain an NSSP for this aspect of the work.

The following parameters may be used for the design of the RSS:

- Bearing resistance for the levelling pad on engineered fill:
 - Factored ULS 150 kPa, SLS 100 kPa
- Ultimate coefficient of sliding resistance of cast in-situ concrete levelling pad on Granular A = 0.70
- Ultimate coefficient of sliding resistance of RSS mass on Granular A = 0.70
- Ultimate coefficient of sliding resistance of RSS mass on silty clay soils = 0.5



The entire block of reinforced earth must also be designed against various modes of failure including sliding and overturning.

11.2 Global Stability

The global stability of the RSS wall is dependent on the characteristics of the embankment fill and the foundation soils, the geometry of the embankment and location of the RSS within the embankment. The RSS wall will be in the form of a rectangular block from ground surface extending to the full height, or in the form of a triangular wedge resting on a lower embankment slope. RSS walls are likely to be used for a false abutment and could also be used as wing walls at the abutments.

Stability analyses on selected configurations were carried out considering the following variables:

- RSS founded at the base of the embankment on stiff to hard silty clay soils.
- Fill behind the RSS is horizontal.
- RSS block with full retained height and block width (length of RSS reinforcement) equal to 70% of the height.
- Water Level at Elev. ± 181.0 m.

Analysis carried out on RSS walls located at the base of the embankment next to the abutment locations indicates that a minimum anchor length equivalent to 70% of the wall height is required in order to obtain a target factor of safety of 1.3 or greater.

Consequently, it may be assumed that RSS walls founded at the base of the embankment will be stable against global failure. For an RSS wall founded in the embankment slope, the specific geometry and soil conditions must be analyzed to determine the requirements for global stability. The actual design configuration must be checked for global stability prior to finalization.

The internal stability of the RSS wall should be analyzed by the supplier/designer of the proprietary product selected for this site.

12 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.

13 LATERAL EARTH PRESSURE

For cases where backfill to the abutment is placed in accordance with OPSD 3101.150 or OPSD 3102.200, 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 13.1)

γ = unit weight of retained soil (see table 13.1)

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 13.1.

Table 13.1 – 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.

14 SEISMIC CONSIDERATIONS

14.1 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.

14.2 Liquefaction Potential

The potential for liquefaction of the foundations soils was assessed using the Seed and Idriss (1971) method¹. There is no potential for liquefaction of the foundation soils below the abutments.

Furthermore, since the foundation loads will be transferred by steel piles to very dense till, the vertical geotechnical resistance of the piles will not be compromised.

The immediate approach embankments will bear on stiff to hard silty clay soils above the ground water level and 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.

14.3 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 0.5ϕ . For the design of retaining walls, the coefficients of horizontal earth pressure in Table 14.1 may be used:

Table 14.1 – 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

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



15 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.

Rehman A.O.

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



Michael Tanos.

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



TABLE

TERRAPROBE INC.

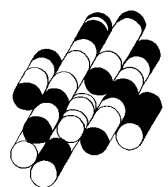


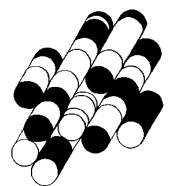
TABLE 1

DOCUMENT	TITLE
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 208.010	Benching of Earth Slopes.
OPSD 3101.150	Walls, Abutment Backfill – Min. Granular Requirement
OPSD 3101.200	Walls, Abutment Backfill – Rock



APPENDICES

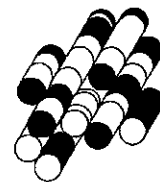
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APPENDIX A

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_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa ⁻¹	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
C_r	m ² /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_r	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_r	1	SENSITIVITY = c_u / τ_r

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m ³	DENSITY OF SOLID PARTICLES	e	1.0	VOID RATIO	e_{min}	1.0	VOID RATIO IN DENSEST STATE
γ_s	kN/m ³	UNIT WEIGHT OF SOLID PARTICLES	n	1.0	POROSITY	I_c	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
ρ_w	kg/m ³	DENSITY OF WATER	w	1.0	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kN/m ³	UNIT WEIGHT OF WATER	S_r	%	DEGREE OF SATURATION	D_u	mm	n PERCENT - DIAMETER
ρ	kg/m ³	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ	kN/m ³	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
ρ_d	kg/m ³	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m ² /s	RATE OF DISCHARGE
γ_d	kN/m ³	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $(w_L - w_p)$	v	m/s	DISCHARGE VELOCITY
ρ_{sat}	kg/m ³	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $(w - w_p) / (w_L - w_p)$	i	1	HYDRAULIC GRADIENT
γ_{sat}	kN/m ³	UNIT WEIGHT OF SATURATED SOIL	I_c	1	CONSISTENCY INDEX = $(w_L - w) / (w_L - w_p)$	k	m/s	HYDRAULIC CONDUCTIVITY
ρ'	kg/m ³	DENSITY OF SUBMERGED SOIL	e_{max}	1.0	VOID RATIO IN LOOSEST STATE	j	kN/m ²	SEEPAGE FORCE
γ'	kN/m ³	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:

Approximate g

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

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 SBL 12+685CL

1 OF 1

METRIC

W.P. 280-99-00 LOCATION Coords: N:4764160.4 E:327319.7 ORIGINATED BY MP
DIST HWY 406 BOREHOLE TYPE Hollow Stem Augers COMPILED BY DB
DATUM Geodetic DATE 11.17.09 CHECKED BY RA

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		20	40	60	80	100					
182.7	Ground Surface															
0.0	470mm FILL - Sand and Gravel, some silt, trace clay, compact, grey, damp		1	SS	11											41 42 13 4
182.2																
0.5	SILTY CLAY trace sand, occasional gravel inclusions, very stiff, brown, moist		2	SS	18											
			3	SS	15											
			4	SS	16											0 1 50 49
			5	SS	19											
			6	SS	15											
			7	SS	13											0 2 68 30
			8	SS	11											
			9	SS	8											1 5 68 26
			10	TW	PH											20.7 0 3 70 27
170.5	End of Borehole															
12.2	Borehole was dry (not stabilized) and hole open to full depth on completion. Piezometer installation consists of a 19mm diameter, Schedule 40 PVC pipe with a 3.0m slotted screen. Water Level Readings: Date Depth(m) Elevation(m) Piezometer destroyed after drilling. Consolidation test performed on TW 10.															

ONTARIO MOT 1-09-4135 WS BRIDGE.GPJ ONTARIO MOT.GDT 05/26/10

RECORD OF BOREHOLE No WS1

1 OF 3

METRIC

W.P. 280-99-00 LOCATION Coords: N:4764163.6 E:327303.9 ORIGINATED BY MP
DIST HWY 406 BOREHOLE TYPE Solid Stem Augers / Casing and Washboring / NQ Rock Coring COMPILED BY DB
DATUM Geodetic DATE 01.19.10 - 04.29.10 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
182.7	Ground Surface							20 40 60 80 100	20 40 60 80 100	10 20 30				GR SA SI CL
182.6	80mm TOPSOIL		1	SS	7		182							0 2 36 62
	FILL - Silty Clay, trace sand, firm to hard, brown, damp		2	SS	28								53	
			3	SS	37		181							
180.6														
2.1	SILTY CLAY trace sand, trace gravel, hard, brown, damp		4	SS	32		180							
			5	SS	33									
			6	SS	52		179							
178.3														
4.4	SILT trace sand, trace gravel, frequent silty clay seams and partings, dense, brown, damp		7	SS	47		178							1 2 75 22
			8	SS	39									
176.8														
5.9	SILTY CLAY trace sand, trace gravel, very stiff to hard, brown, damp to moist		9	SS	15		177							
							176			3.3				
			10	SS	23		175							0 5 83 12
							174			1.8				
			11	SS	25									0 4 62 34
							173			1.4				
			12	TW	PH		172							
							171							
			13	SS	41		170							
							169							1 3 72 24
			14	SS	28									
168.0														
14.7							168							

Continued Next Page

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

ONTARIO MOT 1-09-4135 WS BRIDGE GPJ ONTARIO MOT.GDT 05/26/10

2 OF 3

METRIC

SOIL PROFILE	SAMPLES			DYNAMIC CONE PENETRATION			
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Continued Next Page

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

ONTARIO MOT 1-09-4135 WS BRIDGE, GPJ ONTARIO MOT.GDT 05/26/10

3 OF 3

METRIC

[illegible]

CORE LOG



Terraprobe

Project	Highway 406 Twinning	Orientation	Vertical	Ground Elevation	182.7m	Datum	Geodetic	Borehole No.	WS1
Location	Welland, Ontario	Date Started	April 29, 2010	Completed	April 29, 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 ³)
				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		
155.7	27.0																			
			Overburden, refer to Borehole Log WS1																	
155.2	27.5		SALINA FORMATION BEDROCK	1	B	F	VC	RP	T	0 to 1					#1 TCR 19 SCR 11	0	NQ			
154.7	28.0		INTERBEDDED DOLOSTONE AND SHALE Unweathered, thinly laminated, grey, medium strength, argillaceous with unweathered, laminated, white, very low strength gypsum and calcite layers / veins and frequent unweathered, white, low strength, coarse grained calcitic vugs.	1	B	F	VC	RP	T											
154.2	28.5			1	B	F	VC	RP	T						#2 TCR 62 SCR 34	18	NQ			
153.7	29.0			1	B	F	C	SP	T	0 to 1										
153.2	29.5			2	BC	FV	VC	SP	T											
152.7	30.0			1	B	F	C	SP	T	0 to 5					#3 TCR 91 SCR 69	43	NQ			
152.2	30.5			1	B	F	VC	SP	T	0 to 1										
				1	B	F	C	SP	T	0 to 1										
151.7	31.0			1	B	F	VC	SP	T	0 to 1										
				1	B	F	VC	SP	T	0 to 1										
151.2	31.5			2	BC	FV	C	SP	T	0 to 1					#4 TCR 94 SCR 80	30	NQ			
				2	BC	FV	VC	SP	T	0 to 1										
150.7	32.0			1	B	F	C	SP	T	0 to 2										
			End of Core Log																	
150.5	32.5		Rubblelized zones at: 27.50-29.00m; 29.50-29.70m; 30.70-30.80m; 31.00-31.30m. Rubble indicated by 'a'.																	
			Highly fractured zone at: 31.80-32.00m.																	
150.0	33.0																			

Remarks:

LEGEND:

	Interbedded Dolostone and Shale
	Rubble

RECORD OF BOREHOLE No WS2

1 OF 3

METRIC

W.P. 280-99-00 LOCATION Coords: N:4764174.7 E:327313.4 ORIGINATED BY AW
 DIST HWY 406 BOREHOLE TYPE Hollow Stem Augers / NQ Rock Coring COMPILED BY DB
 DATUM Geodetic DATE 01.28.10 - 02.01.10 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
								○ UNCONFINED	+ FIELD VANE						
								● QUICK TRIAXIAL	× LAB VANE						
183.1	Ground Surface						20 40 60 80 100	20 40 60 80 100	10 20 30				GR SA SI CL		
0.0	firm		1	SS	7		183								
			2	SS	38		182							2 3 37 58	
	SILTY CLAY trace sand, trace gravel, hard, brown, damp		3	SS	43		181								
			4	SS	36		180							0 1 51 48	
			5	SS	29		179								
			6	SS	24		178								
178.7			7	SS	37		177							0 1 79 20	
4.4	SILT trace sand, frequent silty clay seams and partings, dense, brown, damp		8	SS	36		176								
			9	SS	21		175								
177.2			10	SS	22		174							0 5 68 27	
5.9	SILTY CLAY trace sand, trace gravel, stiff to very stiff, brown, damp to moist		11	TW	PH		173								
			12	SS	10		172							0 3 70 27	
			13	SS	15		171								
			14	SS	28		170							0 2 72 26	
168.4							169								
14.7															

Continued Next Page

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

ONTARIO MOT 1-09-4136 WS BRIDGE GPJ ONTARIO MOT.GDT 05/28/10

RECORD OF BOREHOLE No WS2

2 OF 3

METRIC

W.P. 280-99-00 LOCATION Coords. N:4764174.7 E:327313.4 ORIGINATED BY AW
DIST HWY 406 BOREHOLE TYPE Hollow Stem Augers / NQ Rock Coring COMPILED BY DB
DATUM Geodetic DATE 01.28.10 - 02.01.10 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	W _p W W _L	WATER CONTENT (%)				
								SHEAR STRENGTH kPa							
								○ UNCONFINED + FIELD VANE							
								● QUICK TRIAXIAL × LAB VANE							
								20 40 60 80 100		10 20 30			GR SA SI CL		
	SILTY CLAY trace sand, trace gravel, hard, brown, damp (GLACIAL TILL)		15	SS	40		168								
			16	SS	52		167								
							166								Jan.28
165.1															Jan.29
18.0	SANDY SILT trace to some clay, trace gravel to gravelly, very dense, brown, damp to moist (GLACIAL TILL)		17	SS	111		165								26 17 45 12
							164								
			18	SS	100/ 13cm		163								
162.4															
20.7	CLAYEY SILT and sand, some gravel, hard, brown, damp (GLACIAL TILL)		19	SS	87		162								15 35 35 15
							161								
160.8															
22.3	SANDY SILT and gravel, trace clay, very dense, brown, moist (GLACIAL TILL)		20	SS	100/ 10cm		160								
							159								
			21	SS	100/ 10cm		158								
			22	SS	100/ 5cm		157								Jan 29
															Feb.01
	---- frequent cobbles						156								RUN#1 TCR=71% SCR=57% RQD=18%
155.8															
27.3	BEDROCK - INTERBEDDED DOLOSTONE AND SHALE Unweathered, thinly laminated, grey, medium strength, argillaceous with unweathered, laminated, white, very low strength gypsum and calcite layers / veins and frequent unweathered, white, low strength, coarse grained calcitic vugs.		1	RUN	NQ		155								RUN#2 TCR=89% SCR=85% RQD=30%
			2	RUN	NQ		154								
153.2															

Continued Next Page

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

ONTARIO MOT 1-09-4135 WS BRIDGE.GPJ ONTARIO MOT.GDT 05/28/10

3 OF 3

METRIC

[illegible]

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

CORE LOG



Terraprobe

Project	Highway 406 Twinning	Orientation	Vertical	Ground Elevation	183.1m	Datum	Geodetic	Borehole No.	WS2
Location	Welland, Ontario	Date Started	February 1, 2010	Completed	February 1, 2010	Logged By	AW	Sheet	1 of 1
W.P.:	280-99-00	Drilling Agency	GW	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 ³)
				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	
156.6	26.5		Overburden, refer to Borehole Log WS2																
156.1	27.0		Sandy Silt TILL, refer to Borehole Log WS2																
155.6	27.5		SALINA FORMATION BEDROCK	1	B	F	C	SU	T	0.63				#1 TCR 71 SCR 57	18	NQ			
			INTERBEDDED DOLOSTONE AND SHALE Unweathered, thinly laminated, grey, medium strength, argillaceous with unweathered, laminated, white, very low strength gypsum and calcite layers / veins and frequent unweathered, white, low strength, coarse grained calcitic vugs.	1	B	F	VC	SP	T	0.61									
155.1	28.0			1	B	F	C	SP	T	0.61									
154.6	28.5			1	B	F	VC	SP	T	0.61				#2 TCR 89 SCR 85	30	NQ			
154.1	29.0																		
153.6	29.5			1	B	F	C	SP	T	0.61									
153.1	30.0		End of Core Log																
			<u>Rubble zones at:</u> 27.80-27.90m; 28.80-28.90m.																
			Rubble indicated by 'a'.																
			<u>Highly fractured zone at:</u> 28.30-28.80m.																
152.6	30.5																		
152.1	31.0																		
151.6	31.5																		
151.1	32.0																		
150.6	32.5																		

Remarks:

LEGEND:

- Interbedded Dolostone and Shale
- Rubble
- Sandy Silt Till

RECORD OF BOREHOLE No WS3

1 OF 2

METRIC

W.P. 280-99-00 LOCATION Coords: N:4764192.6 E:327292.7 ORIGINATED BY MP
 DIST HWY 406 BOREHOLE TYPE Solid Stem Augers / NQ Rock Coring COMPILED BY DB
 DATUM Geodetic DATE 01.20.10 - 01.22.10 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100					
183.0	Ground Surface													
182.9	50mm TOPSOIL		1	SS	17									
	FILL - Silty Clay, trace sand, very stiff to hard, brown, damp		2	SS	23		182							
			3	SS	36		181						49	0 3 36 61
			4	SS	43		180						43	0 1 53 46
180.1	SILTY CLAY trace sand, trace gravel, stiff to hard, brown, damp		5	SS	39		179							
2.9			6	SS	30		178							0 1 65 34
			7	SS	24		177							
			8	SS	20		176							
			9	TW	PH		175							
			10	SS	9		174							
			11	SS	13		173							
			12	SS	17		172							
			13	SS	23		171							
			14	SS	23		170							
168.3							169							
14.7														

Continued Next Page

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

ONTARIO MOT 1-09-4135 WS BRIDGE.GPJ, ONTARIO MOT.GDT, 05/26/10

RECORD OF BOREHOLE No WS3

2 OF 2

METRIC

W.P. 280-99-00 LOCATION Coords: N:4764192.6 E:327292.7 ORIGINATED BY MP
DIST HWY 406 BOREHOLE TYPE Solid Stem Augers / NQ Rock Coring COMPILED BY DB
DATUM Geodetic DATE 01.20.10 - 01.22.10 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL					
20 40 60 80 100			20 40 60 80 100			10 20 30									

164.8	SILTY CLAY trace sand to sandy, trace gravel, hard, brown, damp (GLACIAL TILL)		15	SS	46											
18.2																
163.2	SANDY SILT some clay, trace gravel, dense to very dense, brown, damp to moist (GLACIAL TILL)		16	SS	79											
19.8																
160.7	CLAYEY SILT sandy, trace to some gravel, hard, brown, damp (GLACIAL TILL)		17	SS	65											
22.3																
157.1	SANDY SILT trace to some gravel, trace clay, very dense, brown, damp to moist (GLACIAL TILL)		20	SS	100/ 5cm											
25.9																
154.8	BEDROCK - INTERBEDDED DOLOSTONE AND SHALE Unweathered, thinly laminated, grey, medium strength, argillaceous with unweathered, laminated, white, very low strength gypsum and calcite layers / veins and frequent unweathered, white, low strength, coarse grained calcitic vugs.		21	SS	100/ 2.5cm											
28.2																
	boulder		1	RUN	NQ											
	End of Borehole		2	RUN	NQ											
	Unable to push vane beyond 12m. Resistance to augering at 24.3m. Borehole open to full depth and filled with drill water upon completion of drilling. Borehole sealed with bentonite slurry to ground surface.															

CORE LOG



Terraprobe

Project	Highway 406 Twinning	Orientation	Vertical	Ground Elevation	183.0m	Datum	Geodetic	Borehole No.	WS3
Location	Welland, Ontario	Date Started	January 22, 2010	Completed	January 22, 2010	Logged By	AW	Sheet	1 of 1
W.P.:	280-99-00	Drilling Agency	DBW	Drill Type	Track-Mount	Core Barrel & Bit Design	NQ	Project No.	1-09-4135

ELEVATION (m)	DEPTH (m)	SYMBOL	GENERAL DESCRIPTION	Joint Characteristics								STRENGTH	FRACTURE FREQUENCY	RUN NO. CORE RECOVERY %	R Q D %	CORE SIZE/CASING	MPa UNCONFINED COMPRESSIVE STRENGTH	UNIT WEIGHT (kN/m³)
				No. OF SETS	JOINT TYPE	ORIENTATION	SPACING	ROUGHNESS	FILLING	APERTURE	WEATHERING							
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
158.5	24.5																	
158.0	25.0		Overburden, refer to Borehole Log WS3															
157.5	25.5		Sandy Silt TILL, (Boulder 25.07 to 25.90m), refer to Borehole Log WS3															
157.0	26.0		SALINA FORMATION BEDROCK	1	B	F	M	SP	T									
156.5	26.5		INTERBEDDED DOLOSTONE AND SHALE Unweathered, thinly laminated, grey, medium strength, argillaceous with unweathered, laminated, white, very low strength gypsum and calcite layers / veins and frequent unweathered, white, low strength, coarse grained calcitic vugs.	1	B	F	VC	SP	T	0 to 1								
156.0	27.0																	
155.5	27.5			1	B	F	M	SP	T	0 to 1								
155.0	28.0																	
154.5	28.5		End of Core Log <u>Rubble zones at:</u> 26.10-26.20m. Rubble indicated by 'a'.															
154.0	29.0																	
153.5	29.5																	
153.0	30.0																	
152.5	30.5																	

Remarks: 0.5m boulder present in recovered rock core at top of Run No.1

LEGEND:

- Interbedded Dolostone and Shale
- Rubble
- Sandy Silt TILL

RECORD OF BOREHOLE No WS4

1 OF 3

METRIC

W.P. 280-99-00 LOCATION Coords: N:4764202.3 E:327305.2 ORIGINATED BY MP
 DIST HWY 406 BOREHOLE TYPE Solid Stem Augers / NQ Rock Coring COMPILED BY DB
 DATUM Geodetic DATE 01.28.10 - 02.01.10 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
182.7	Ground Surface						20 40 60 80 100							
	25mm TOPSOIL		1	SS	13									
	FILL - Silty Clay, trace sand, trace gravel, stiff to very stiff, brown, damp to moist		2	SS	17									
			3	SS	13									
180.6			4	SS	45									
2.1	SILTY CLAY trace sand, trace gravel, hard, brown, damp		5	SS	45									
			6	SS	43									
			7	SS	61									
177.7	SILT trace clay, trace sand, frequent silty clay seams and partings, very dense, brown, damp		8	SS	64									
5.0			9	SS	37									
176.8			10	SS	37									
5.9	SILTY CLAY trace sand, trace gravel, stiff to hard, brown, damp		11	SS	20									
			12	TW	PH									
			13	SS	25									
			14	SS	23									
168.0														
14.7														

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

ONTARIO MOT 1-09-4135 WS BRIDGE.GPJ ONTARIO MOT.GDT 05/26/10

RECORD OF BOREHOLE No WS4

2 OF 3

METRIC

W.P. 280-99-00 LOCATION Coords: N:4764202.3 E:327305.2 ORIGINATED BY MP
DIST HWY 405 BOREHOLE TYPE Solid Stem Augers / NQ Rock Coring COMPILED BY DB
DATUM Geodetic DATE 01.28.10 - 02.01.10 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
								20 40 60 80 100						
								○ UNCONFINED + FIELD VANE						
								● QUICK TRIAXIAL × LAB VANE						
								20 40 60 80 100						
									10 20 30					

ONTARIO MOT 1-09-4135 WS BRIDGE.GPJ ONTARIO MOT.GDT 05/26/10

Continued Next Page

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

CORE LOG



Terraprobe

Project	Highway 406 Twinning	Orientation	Vertical	Ground Elevation	182.7m	Datum	Geodetic	Borehole No.	WS4
Location	Welland, Ontario	Date Started	February 1, 2010	Completed	February 1, 2010	Logged By	AW	Sheet	1 of 1
W.P.:	280-99-00	Drilling Agency	DBW	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 O D %	CORE SIZE/CASING	MPa UNCONFINED COMPRESSIVE STRENGTH	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		
158.0	25.0																			
157.5	25.5		Overburden, refer to Borehole Log WS4																	
157.0	26.0		Sand and Silt TILL, refer to Borehole Log WS4											#1 TCR 59 SCR 27	12	NQ				
156.5	26.5		SALINA FORMATION BEDROCK	1	B	F	C	SP	T	0 to 1										
			INTERBEDDED DOLOSTONE AND SHALE Unweathered, thinly laminated, grey, medium strength, argilloceous with unweathered, laminated, white, very low strength gypsum and calcite layers / veins and frequent unweathered, white, low strength, coarse grained calcitic vugs.	1	B	F	VC	SP	T											
156.0	27.0																			
155.5	27.5			1	B	F	C	SP	T	0 to 1				#2 TCR 100 SCR 92	26	NQ				
155.0	28.0																			
154.5	28.5			1	B	F	C	SP	T	0 to 1				#3 TCR 58 SCR 47	22	NQ				
154.0	29.0																			
			End of Core Log																	
153.5	29.5		<u>Rubblelized zones at:</u> 26.50-26.80m. Rubble indicated by 'a'.																	
153.0	30.0																			
152.5	30.5																			
152.0	31.0																			

Remarks:

LEGEND:

- Interbedded Dolostone and Shale
- Rubble
- Sand and Silt TILL

RECORD OF BOREHOLE No SBL 12+750CL

1 OF 1

METRIC

W.P. 280-99-00 LOCATION Coords: N:4764219.9 E:327296.2 ORIGINATED BY PK
DIST HWY 406 BOREHOLE TYPE Hollow Stem Augers COMPILED BY DB
DATUM Geodetic DATE 11.10.09 CHECKED BY RA

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

182.9	Ground Surface													
182.7	200mm TOPSOIL													
0.2	FILL - Silty Clay, trace sand, trace organics, stiff, dark brown, moist		1	SS	8									
182.2														
0.7	SILTY CLAY trace sand, very stiff to hard, brown, moist		2	SS	24									
			3	SS	37									
			4	SS	46									
			5	SS	33									
			6	SS	20									
			7	SS	27									
			8	SS	34									
			9	SS	21									
			10	TW	PH									
171.6	End of Borehole													
11.3	Water level at 10.4m (not stabilized) and hole open to full depth on completion. Piezometer installation consists of a 19mm diameter, Schedule 40 PVC pipe with a 3.0m slotted screen. Water Level Readings: Date Depth(m) Elevation(m) Nov.19.09 4.6 178.3 Nov.30.09 7.8 175.1 Dec.08.09 4.1 178.8 Dec.15.09 3.2 179.7 Jan.04.10 2.7 180.2 Jan.11.10 2.7 180.2 Jan.19.10 2.6 180.3 Consolidation test performed on TW 10.													

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

ONTARIO MOT 1-09-4135 WS BRIDGE.GPJ ONTARIO MOT.GDT 05/26/10

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: WS1

Runs: 1, 2, 3 & 4

Depth: 27.5m – 32.2m



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: WS2

Runs: 1 & 2

Depth: 26.8m – 29.9m



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: WS3
Runs 1 & 2
Depth: 25.1m – 28.2m



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



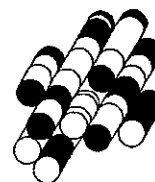
Bedrock Core Sample
Borehole: WS4
Runs: 1, 2 & 3
Depth: 25.8m – 29.2m



APPENDIX B

Laboratory Test Results

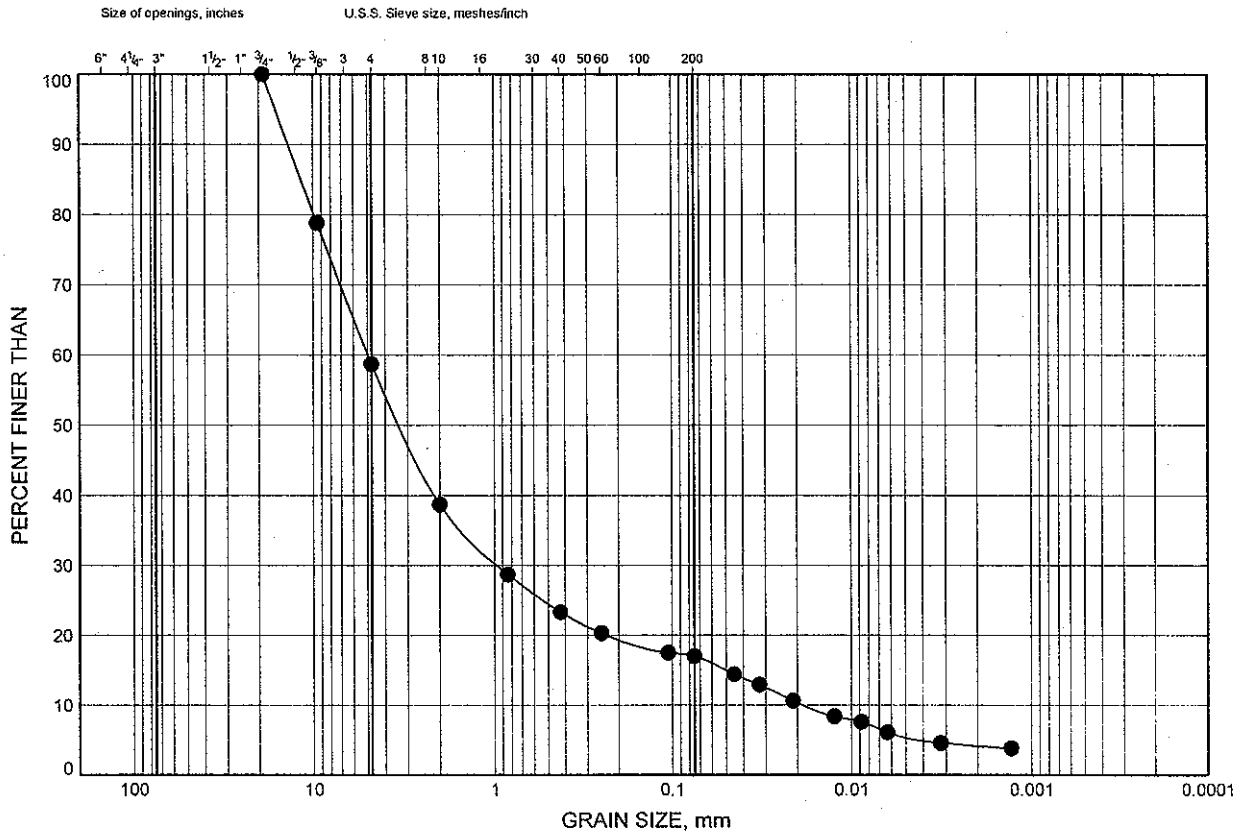
Terraprobe Inc.



GRAIN SIZE DISTRIBUTION

FIGURE B1

FILL - Sand and Gravel



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

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	SBL 12+685CL	0.3	182.4

Date May 2010

Project 1-09-4135



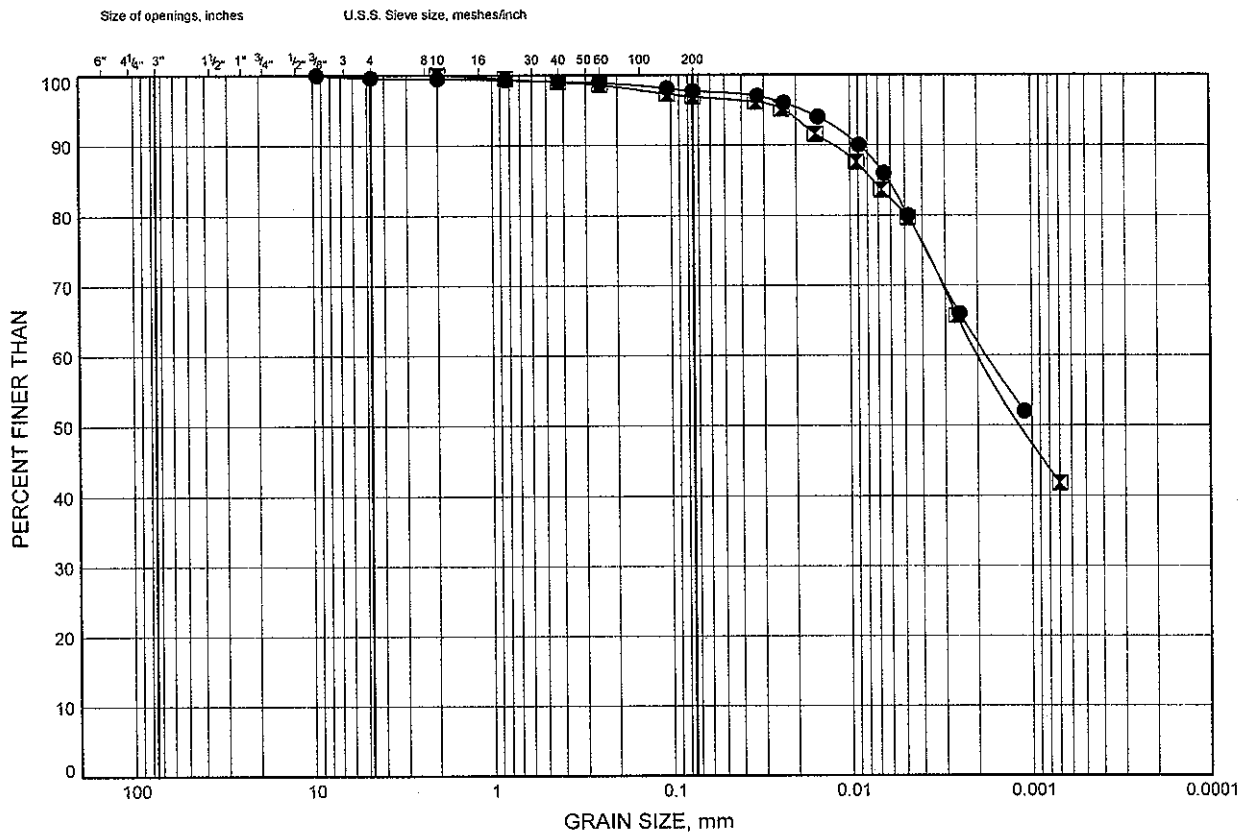
Prep'd DB

Chkd. MP

GRAIN SIZE DISTRIBUTION

FIGURE B2

FILL - Silty Clay

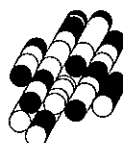


COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	WS1	1.0	181.7
◻	WS3	1.7	181.3

Date May 2010

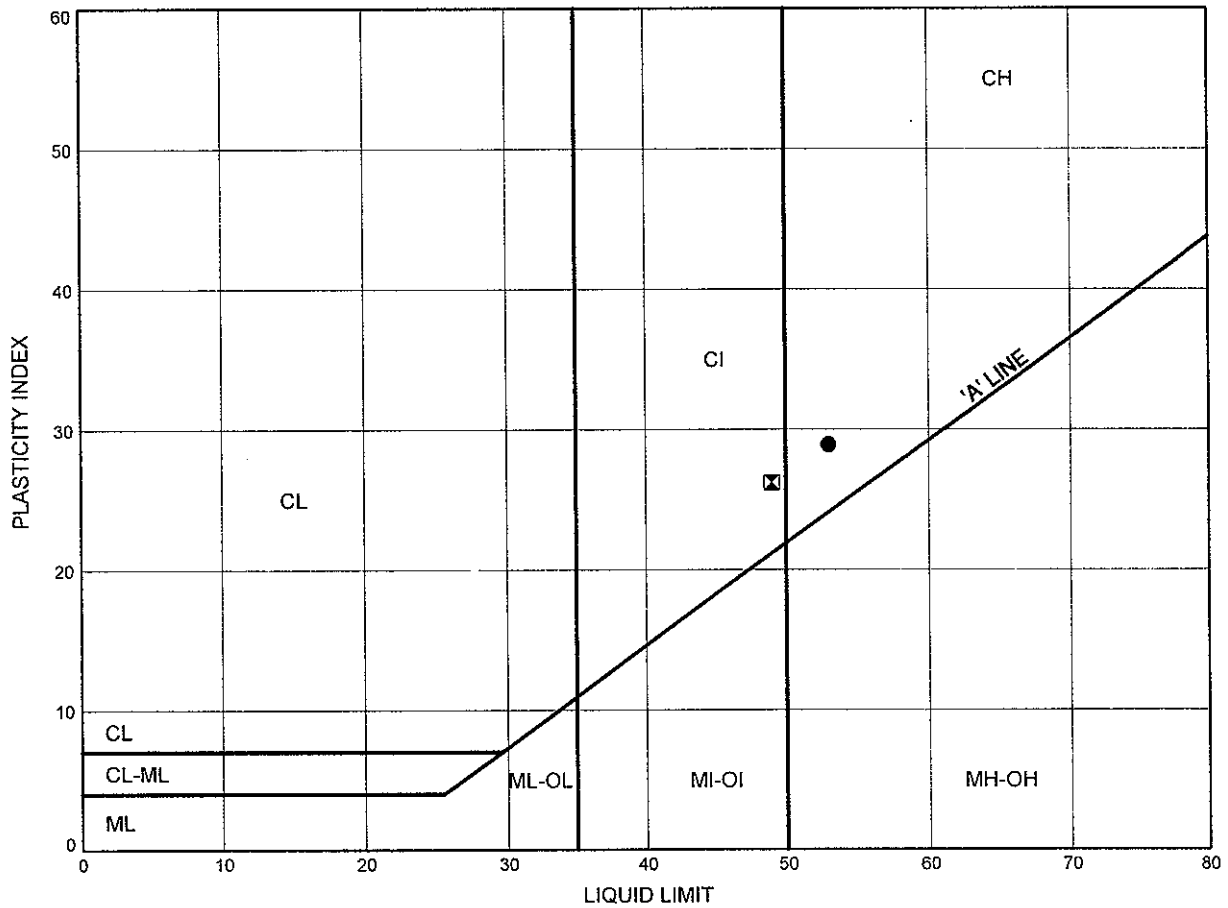
Project 1-09-4135



Prep'd DB

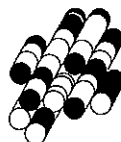
Chkd. MP

FIGURE B3



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	WS1	1.0	181.7
⊠	WS3	1.7	181.3

Date May 2010
Project 1-09-4135

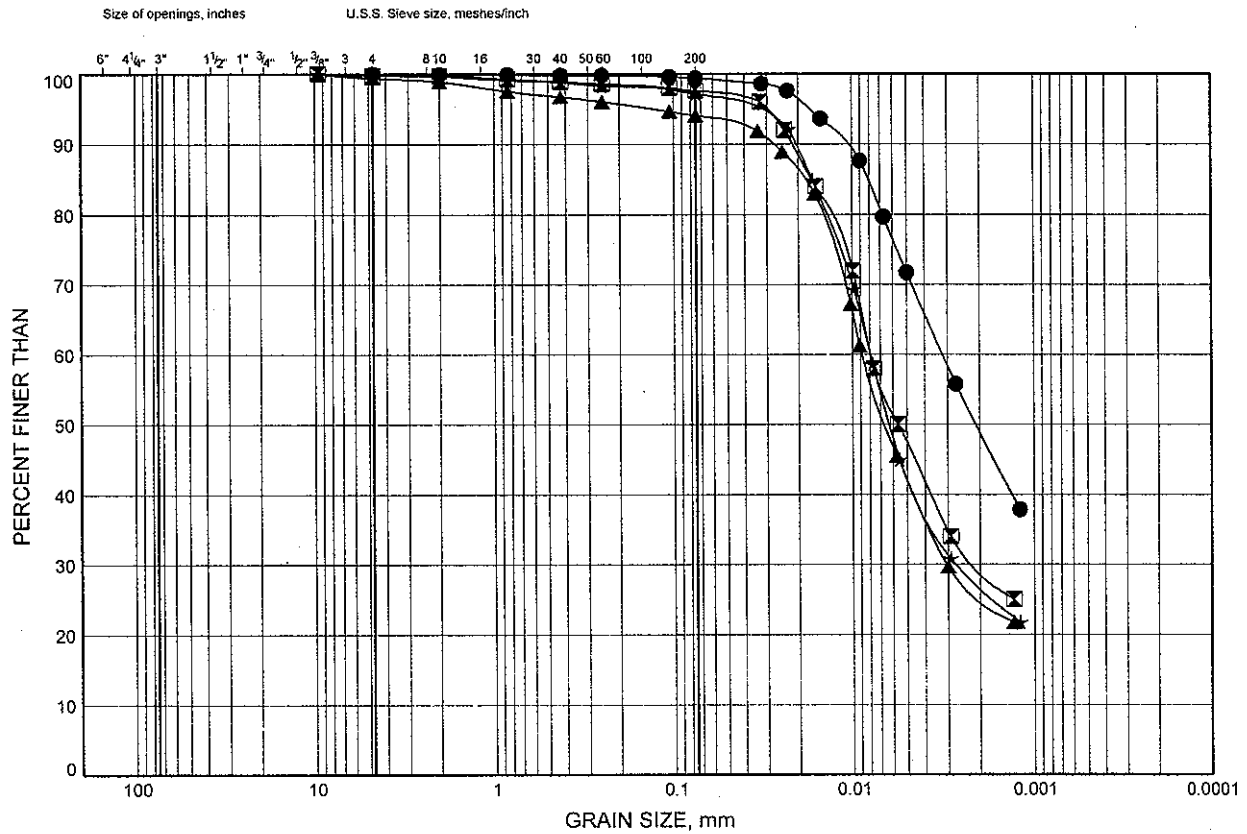


Prep'd DB
Chkd MP

GRAIN SIZE DISTRIBUTION

FIGURE B4

SILTY CLAY

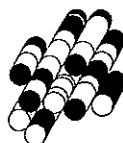


COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	SBL 12+685CL	2.5	180.2
⊠	SBL 12+685CL	6.3	176.4
▲	SBL 12+685CL	9.3	173.4
★	SBL 12+685CL	10.9	171.8

Date May 2010

Project 1-09-4135



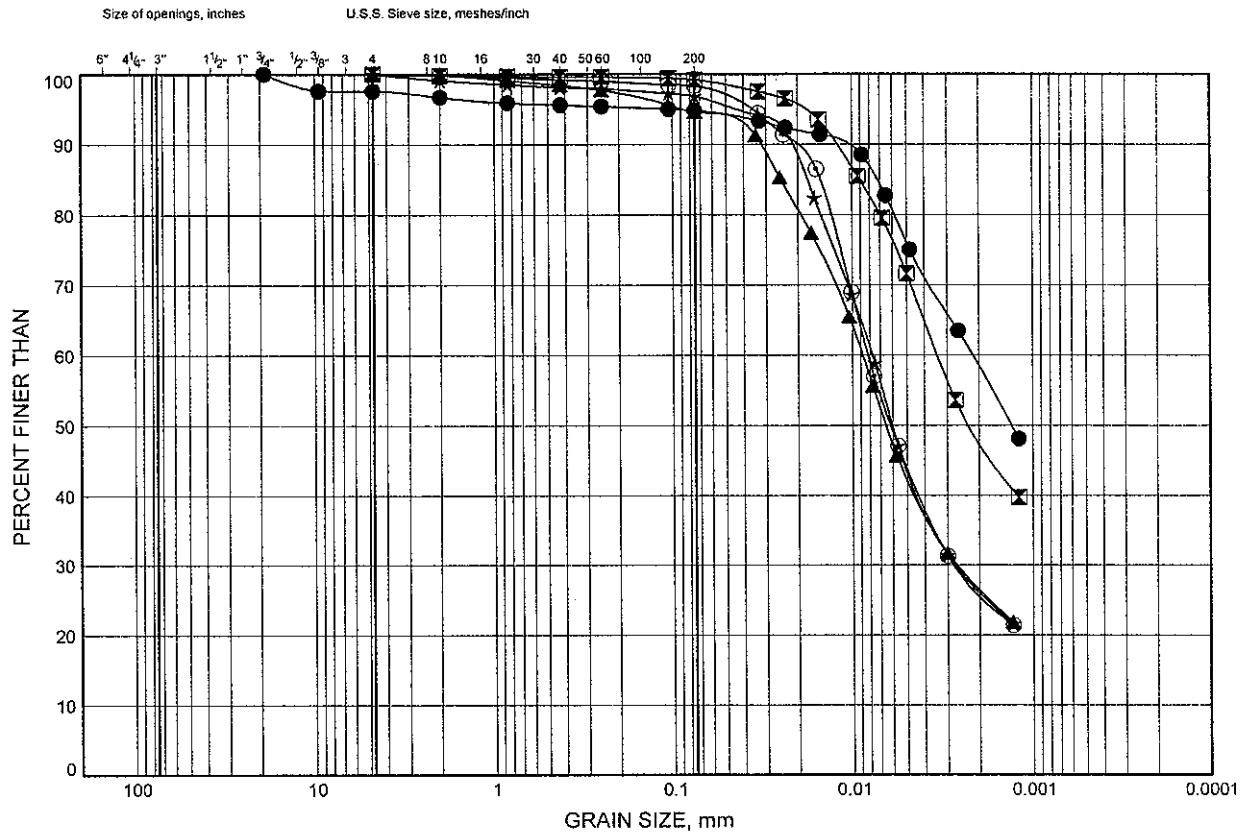
Prep'd DB

Chkd. MP

GRAIN SIZE DISTRIBUTION

FIGURE B6

SILTY CLAY

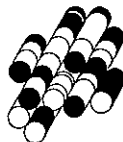


COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	WS2	1.0	182.1
⊠	WS2	2.5	180.6
▲	WS2	6.3	176.8
★	WS2	10.9	172.2
⊙	WS2	12.4	170.7

Date May 2010

Project 1-09-4135



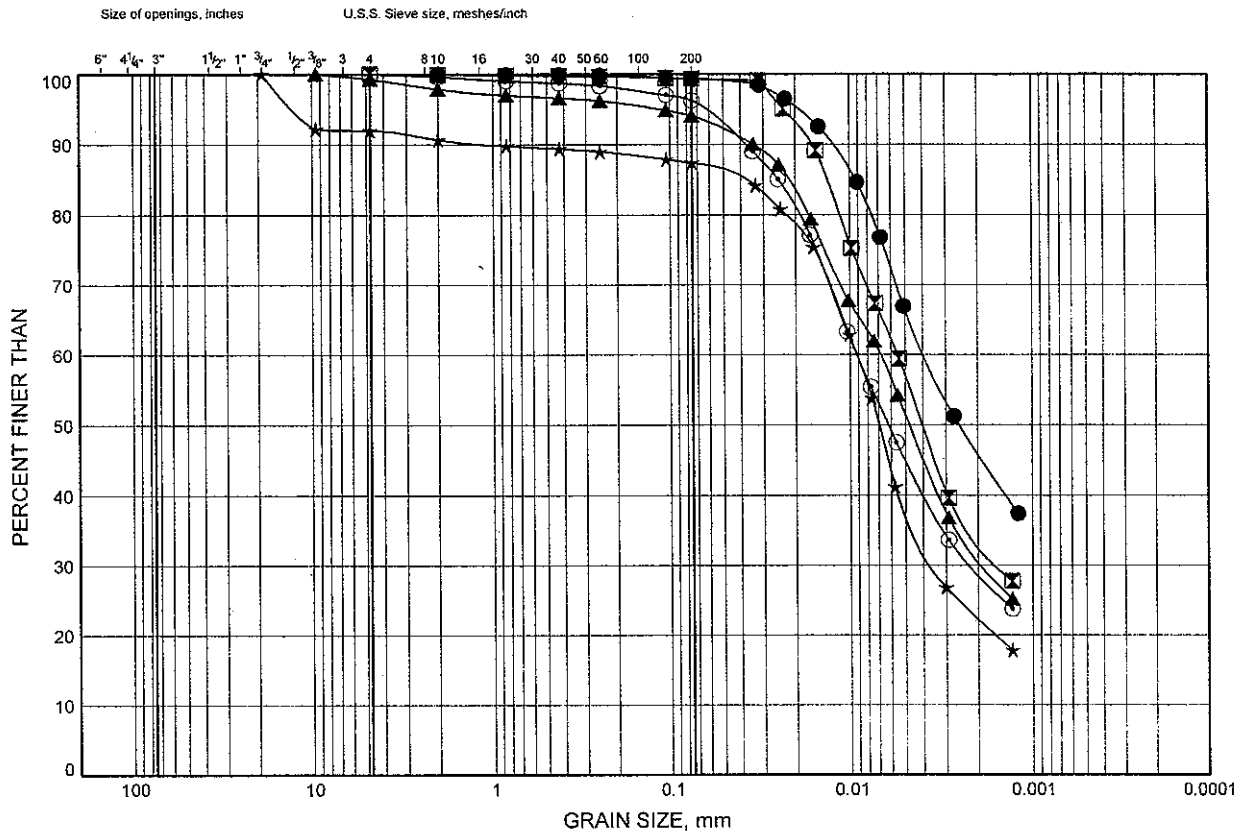
Prep'd DB

Chkd. MP

GRAIN SIZE DISTRIBUTION

FIGURE B7

SILTY CLAY



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

SYMBOL BOREHOLE DEPTH (m) ELEVATION (m)

●	WS3	3.2	179.8
⊠	WS3	4.7	178.3
▲	WS3	7.8	175.2
★	WS3	10.9	172.1
⊙	WS3	13.9	169.1

Date May 2010

Project 1-09-4135



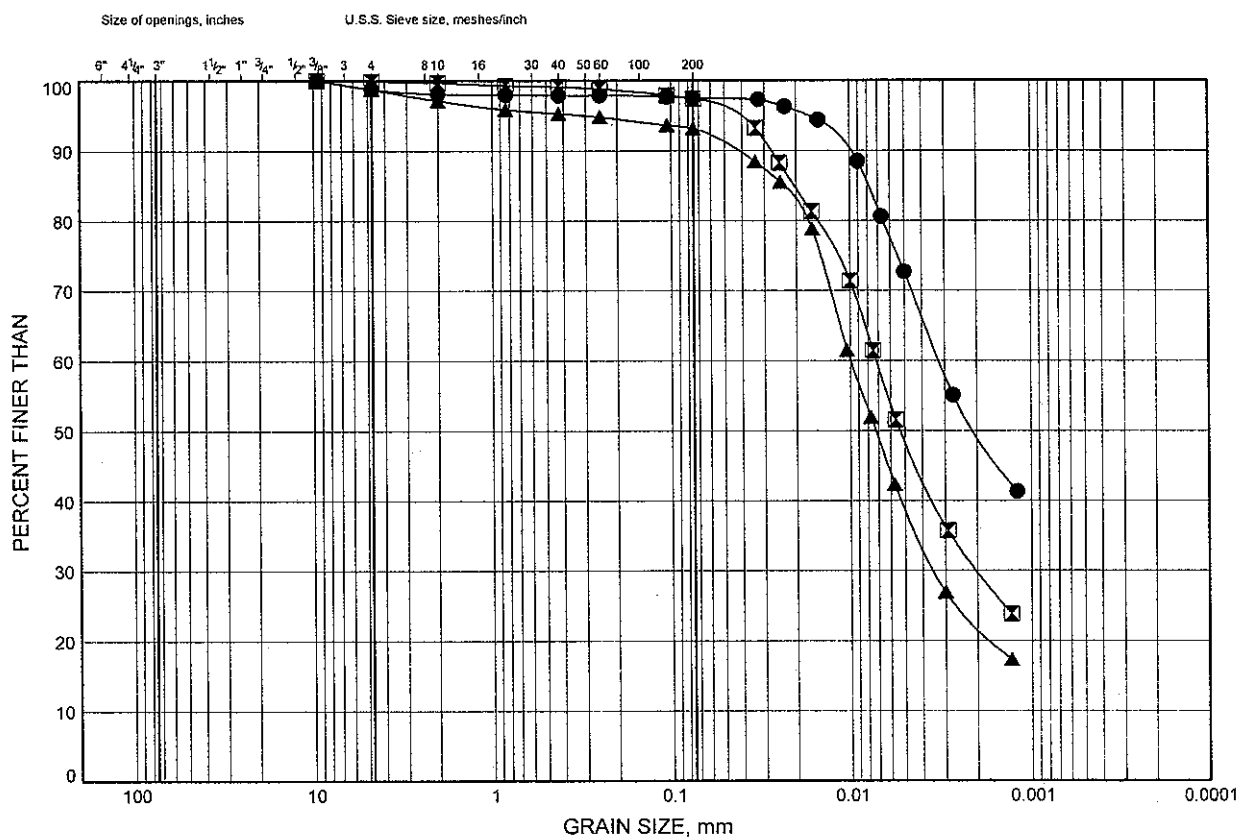
Prep'd DB

Chkd. MP

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)
--------	----------	-----------	---------------

●	WS4	2.5	180.2
⊠	WS4	9.3	173.4
▲	WS4	12.4	170.3

Date May 2010

Project 1-09-4135



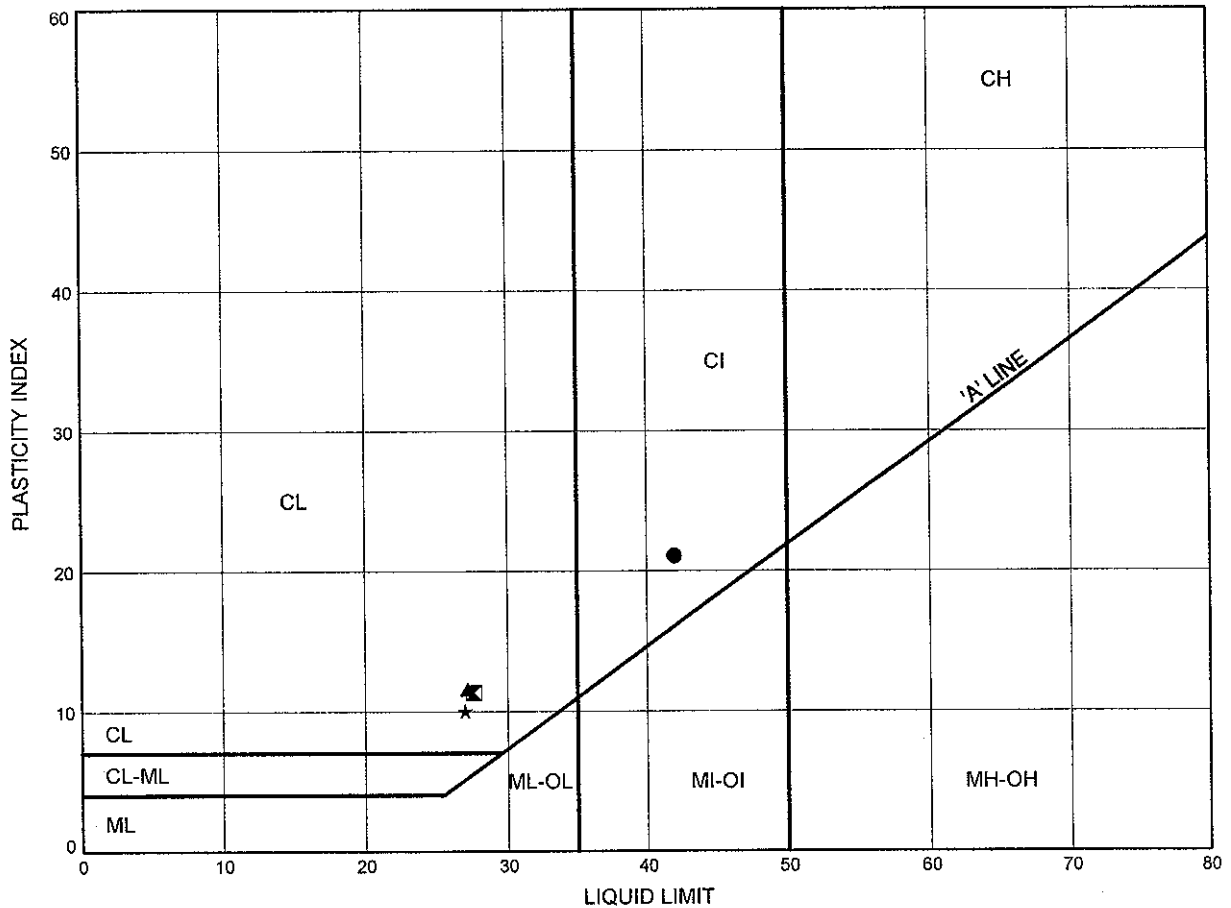
Prep'd DB

Chkd. MP

ATTERBERG LIMITS TEST RESULTS

FIGURE B9

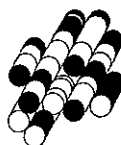
SILTY CLAY



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	SBL 12+685CL	2.5	180.2
⊠	SBL 12+685CL	6.3	176.4
▲	SBL 12+685CL	9.3	173.4
★	SBL 12+685CL	10.9	171.8

Date May 2010

Project 1-09-4135



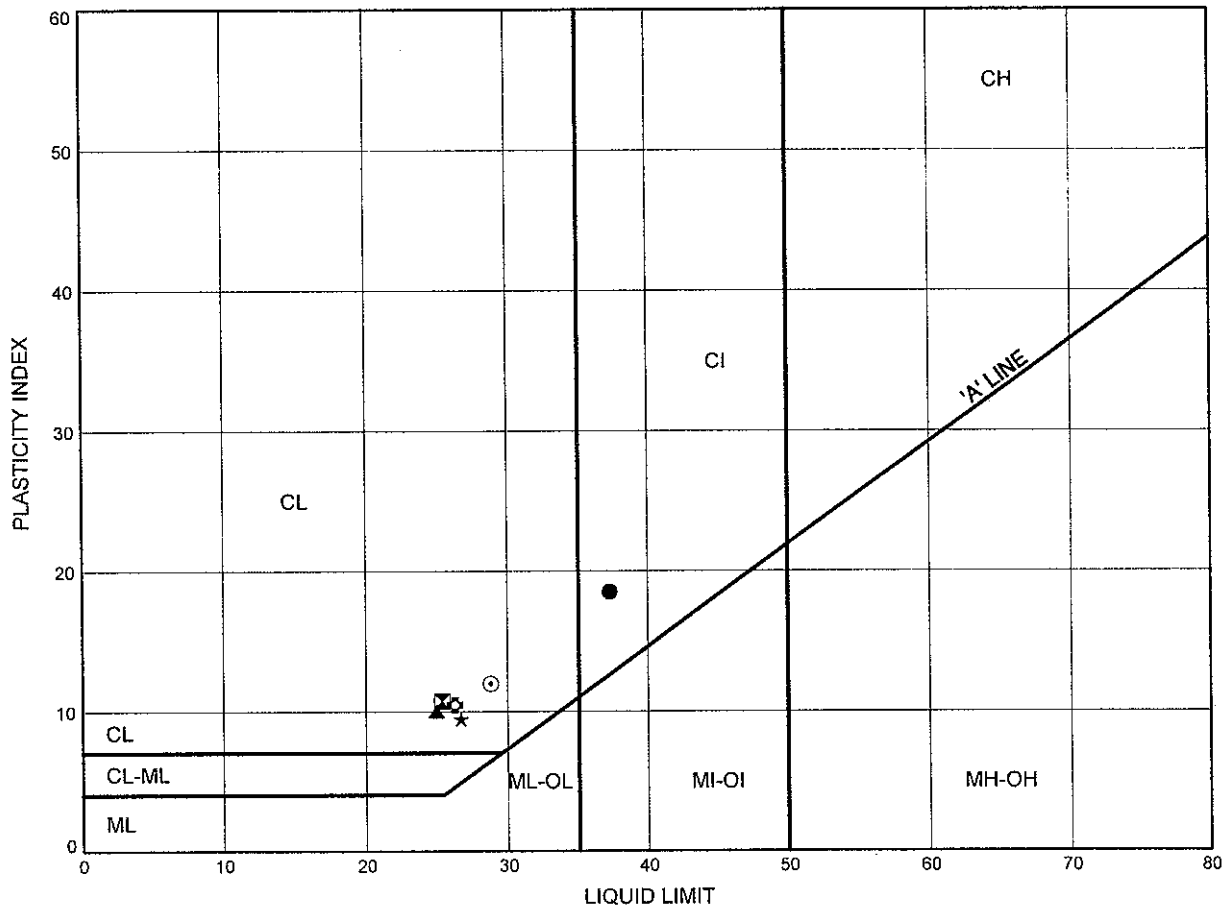
Prep'd DB

Chkd. MP

ATTERBERG LIMITS TEST RESULTS

FIGURE B10

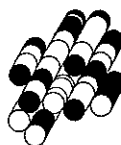
SILTY CLAY



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	SBL 12+750CL	3.2	179.7
⊠	SBL 12+750CL	7.8	175.1
▲	SBL 12+750CL	10.1	172.8
★	WS1	7.8	174.9
⊙	WS1	9.3	173.4
⊛	WS1	13.9	168.8

Date May 2010

Project 1-09-4135



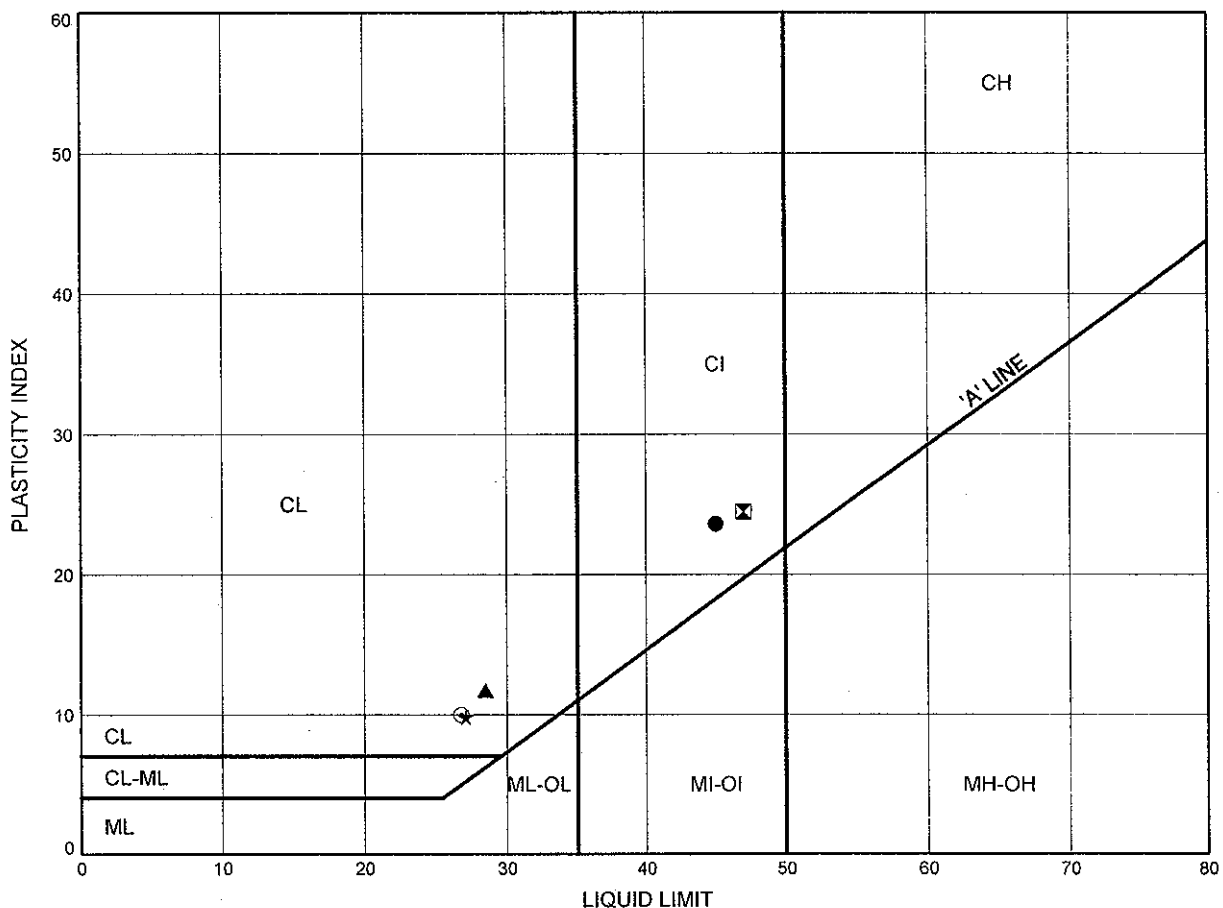
Prep'd DB

Chkd. MP

ATTERBERG LIMITS TEST RESULTS

FIGURE B11

SILTY CLAY



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	WS2	1.0	182.1
⊠	WS2	2.5	180.6
▲	WS2	6.3	176.8
★	WS2	10.9	172.2
⊙	WS2	12.4	170.7

Date May 2010

Project 1-09-4135



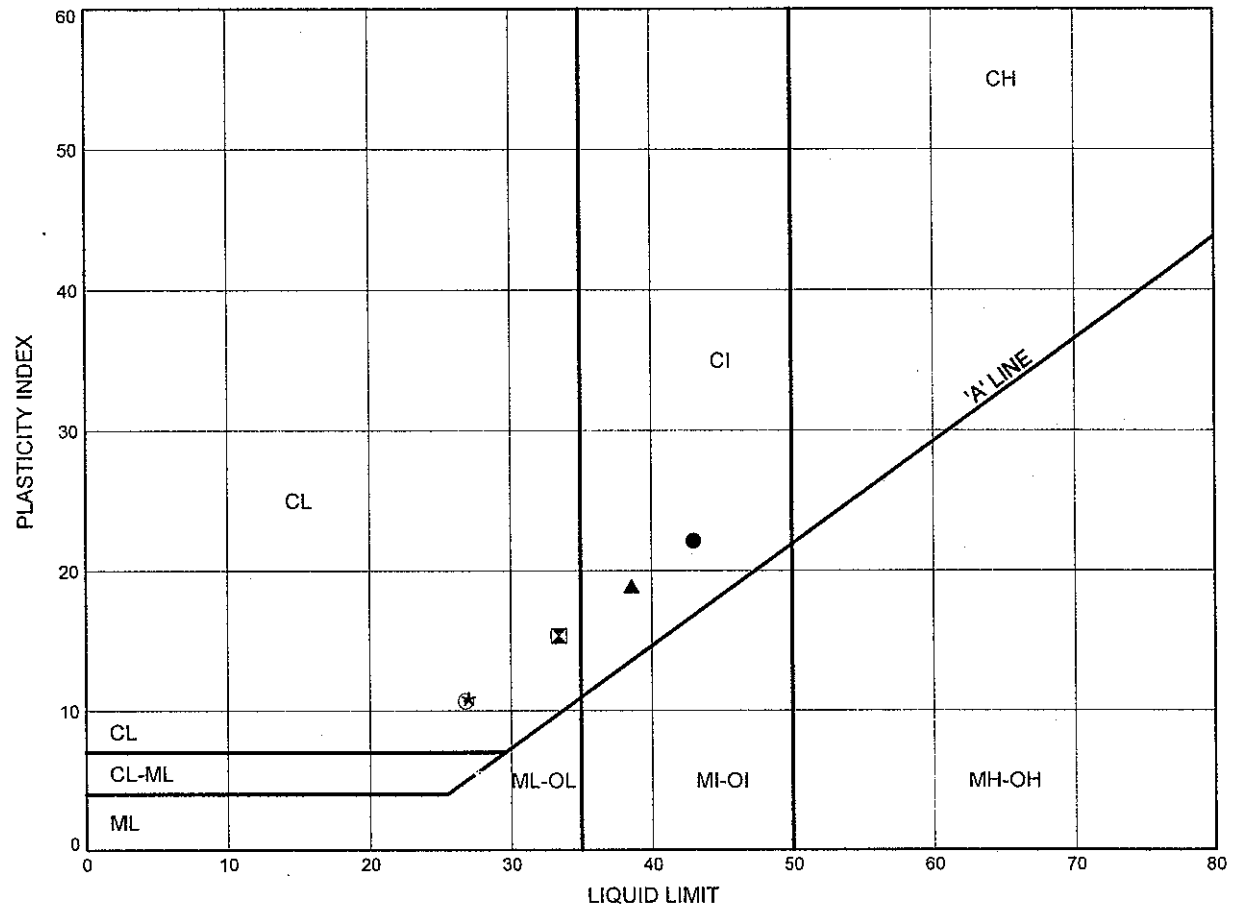
Prep'd DB

Chkd. MP

ATTERBERG LIMITS TEST RESULTS

FIGURE B12

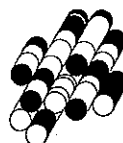
SILTY CLAY



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	WS3	3.2	179.8
⊠	WS3	4.7	178.3
▲	WS3	7.8	175.2
★	WS3	10.9	172.1
⊙	WS3	13.9	169.1

Date May 2010

Project 1-09-4135



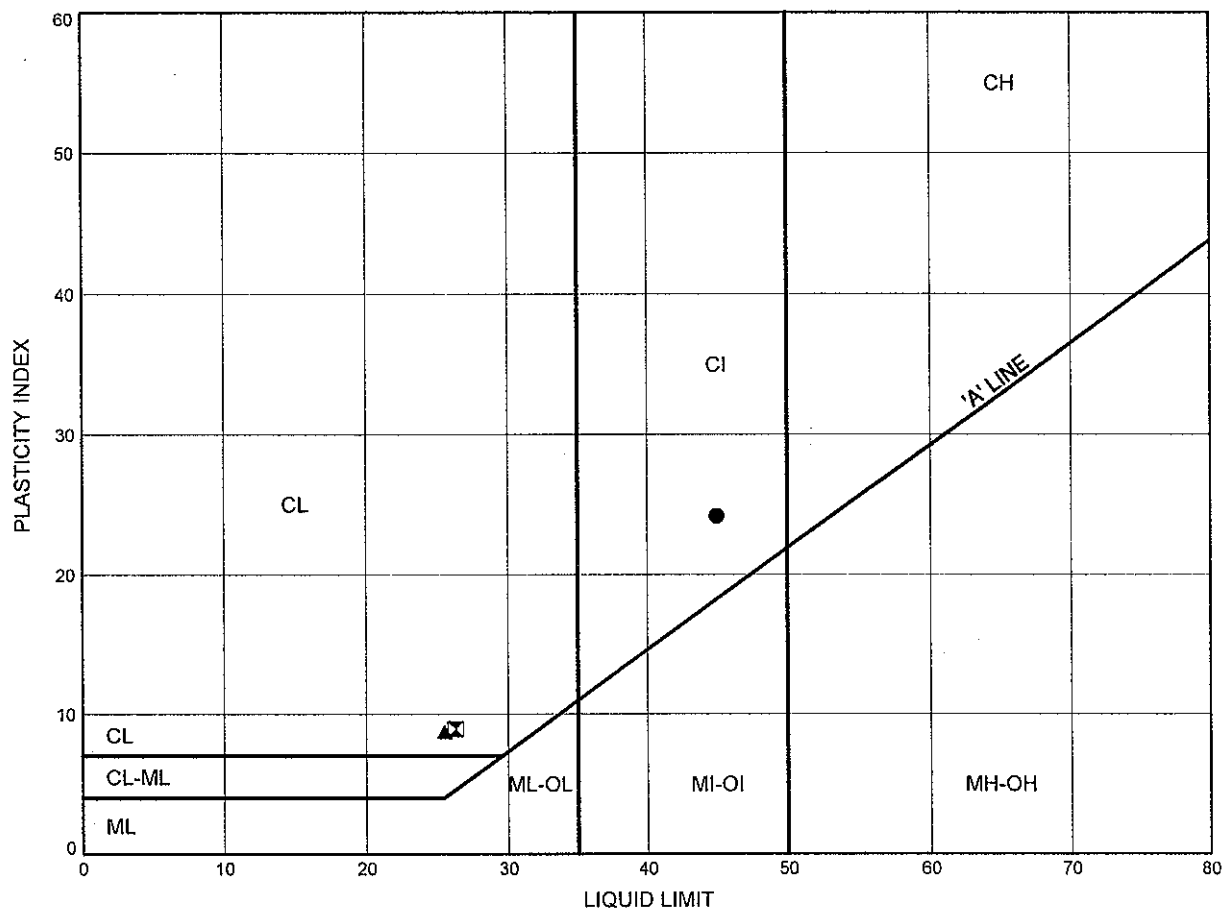
Prep'd DB

Chkd. MP

ATTERBERG LIMITS TEST RESULTS

FIGURE B13

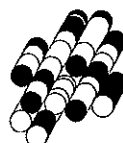
SILTY CLAY



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	WS4	2.5	180.2
⊠	WS4	9.3	173.4
▲	WS4	12.4	170.3

Date May 2010

Project 1-09-4135



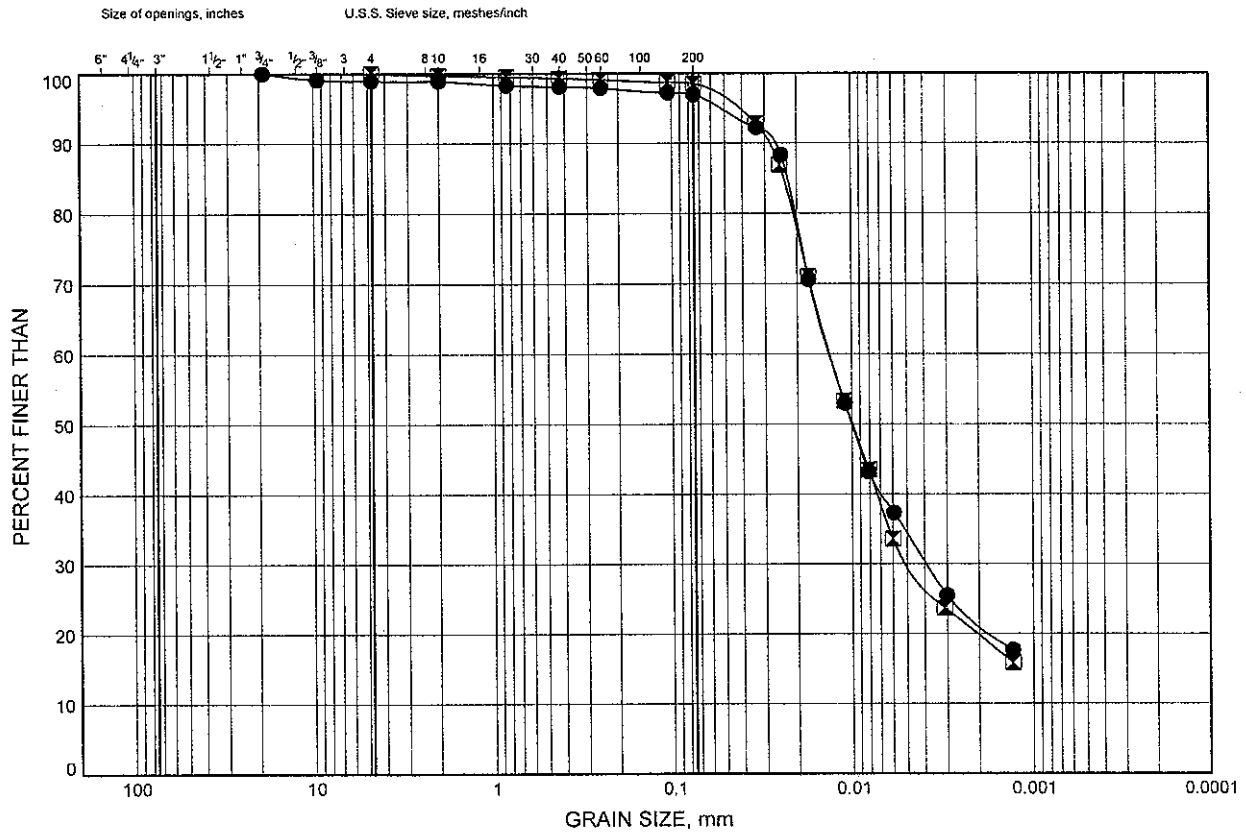
Prep'd DB

Chkd. MP

GRAIN SIZE DISTRIBUTION

FIGURE B14

SILT

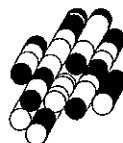


COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	WS1	4.7	178.0
⊠	WS2	4.7	178.4

Date May 2010

Project 1-09-4135



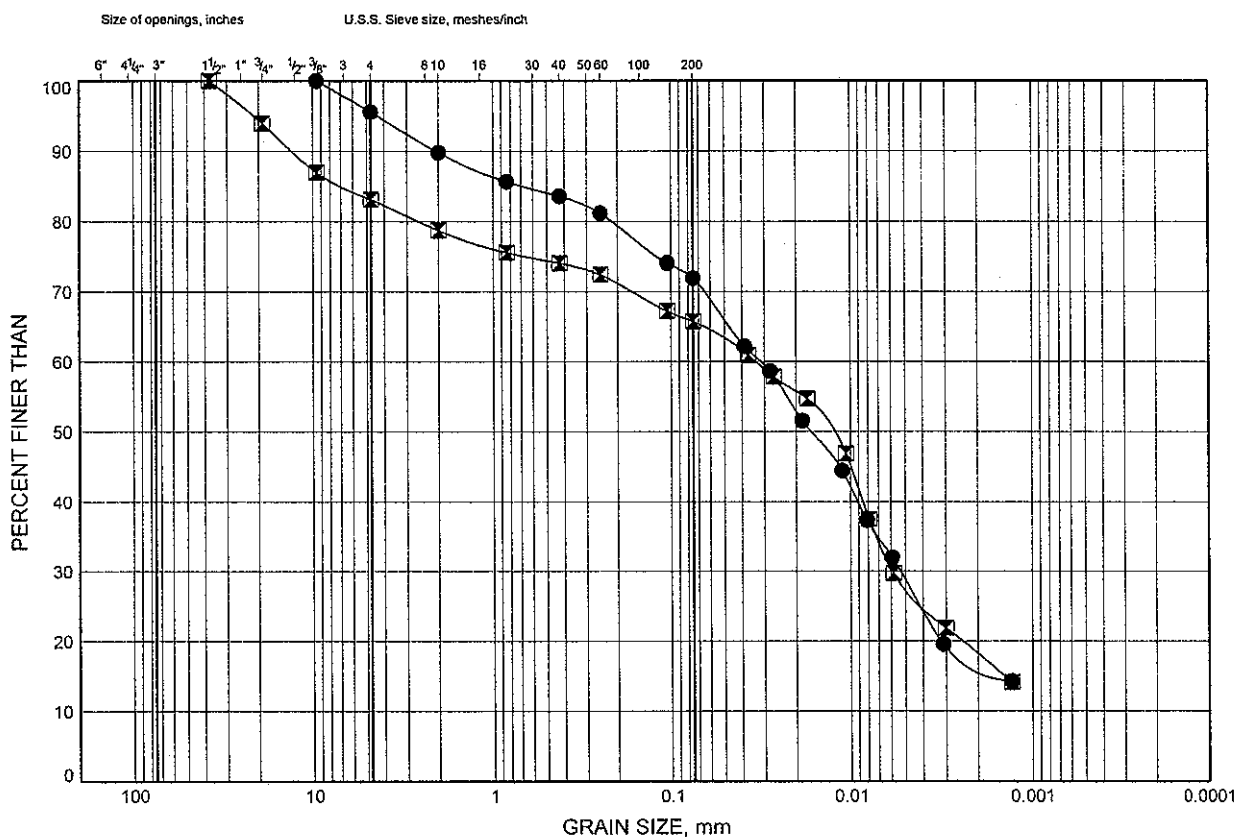
Prep'd DB

Chkd. MP

GRAIN SIZE DISTRIBUTION

FIGURE B15

SILTY CLAY TILL



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

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	WS1	15.4	167.3
⊠	WS4	15.4	167.3

Date May 2010

Project 1-09-4135



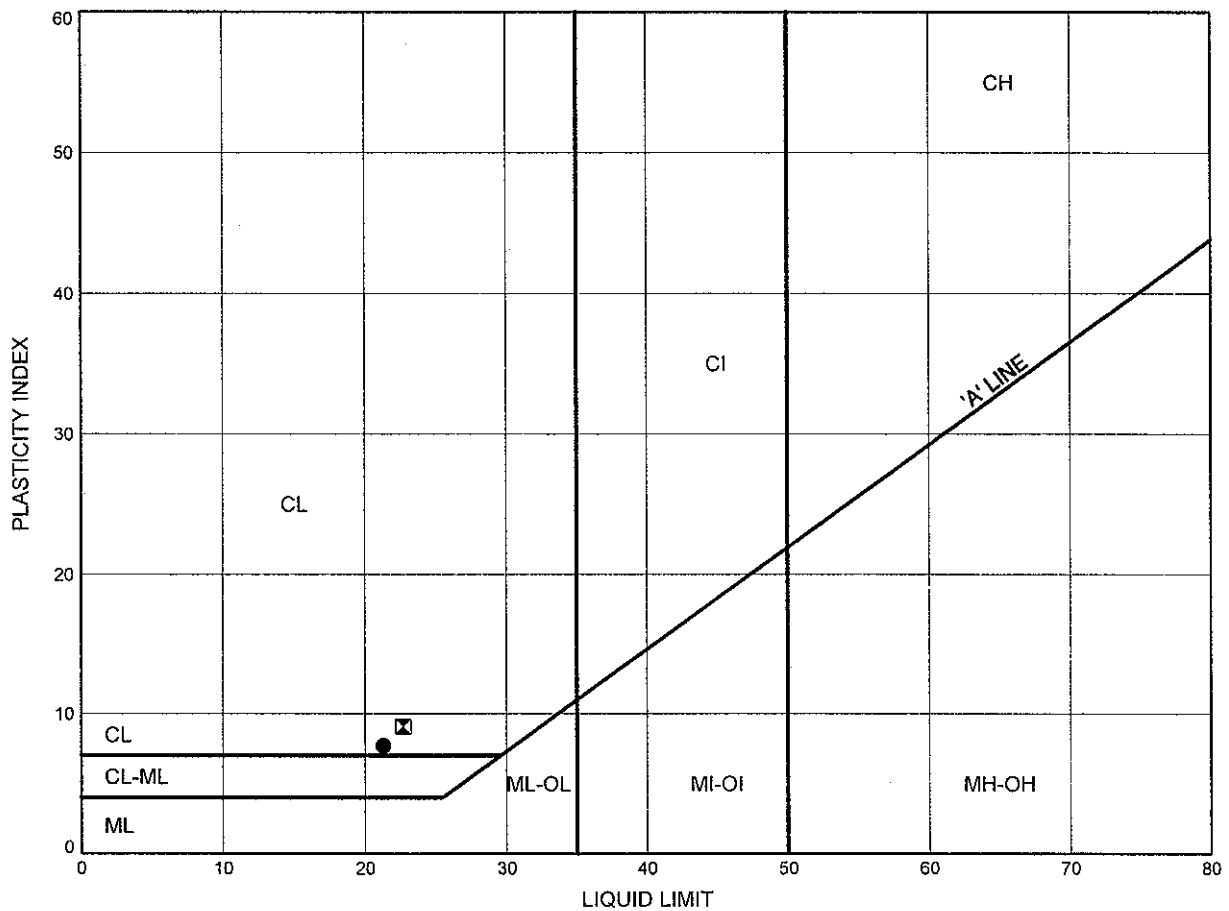
Prep'd DB

Chkd. MP

ATTERBERG LIMITS TEST RESULTS

FIGURE B16

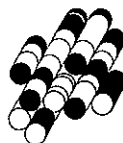
SILTY CLAY TILL



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	WS1	15.4	167.3
⊠	WS4	15.4	167.3

Date May 2010

Project 1-09-4135



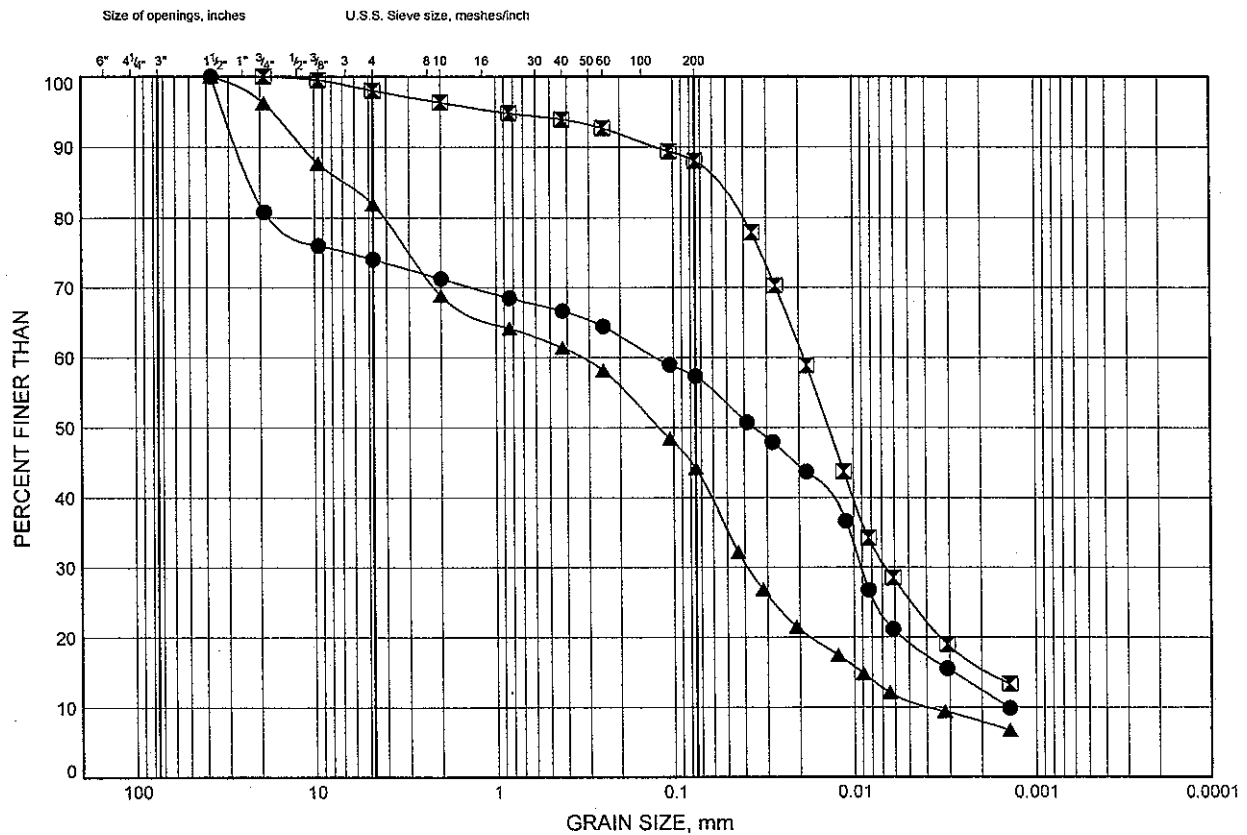
Prep'd DB

Chkd. MP

GRAIN SIZE DISTRIBUTION

FIGURE B17

SANDY SILT TO SAND AND SILT TILL

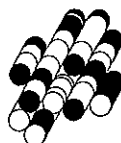


COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	WS2	18.5	164.6
⊠	WS3	18.5	164.5
▲	WS4	23.1	159.6

Date May 2010

Project 1-09-4135



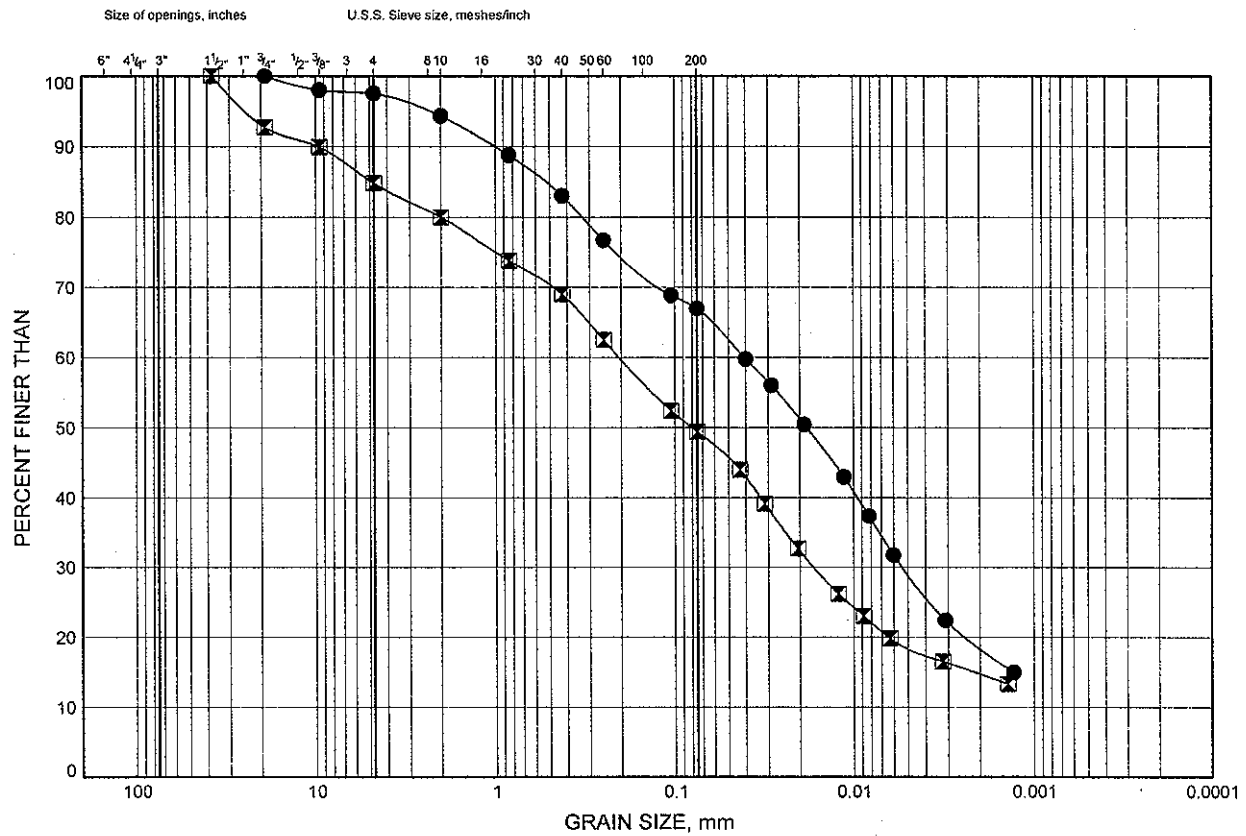
Prep'd DB

Chkd. MP

GRAIN SIZE DISTRIBUTION

FIGURE B18

CLAYEY SILT TILL

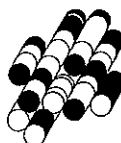


COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	WS1	21.5	161.2
⊠	WS2	21.5	161.6

Date May 2010

Project 1-09-4135



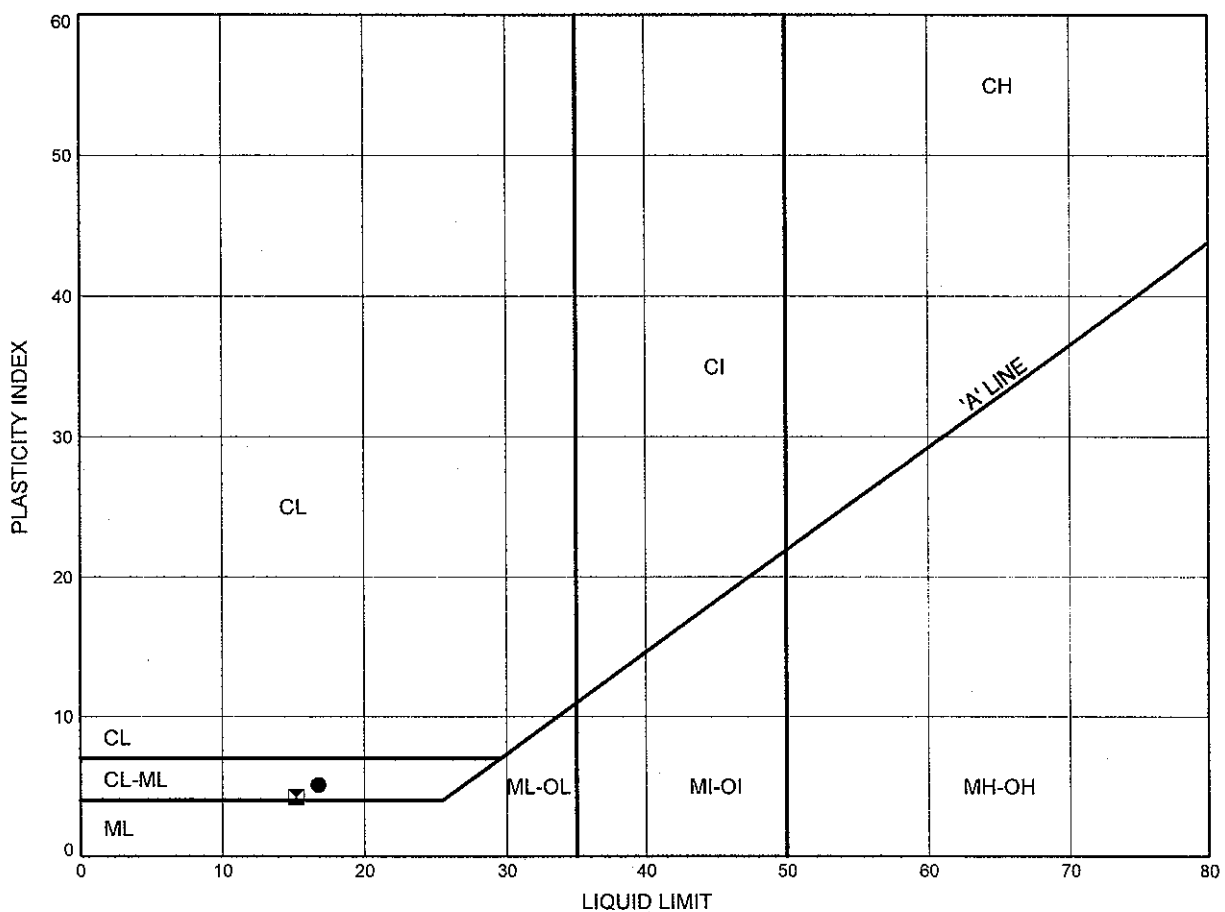
Prep'd DB

Chkd. MP

ATTERBERG LIMITS TEST RESULTS

FIGURE B19

CLAYEY SILT TILL



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	WS1	21.5	161.2
⊠	WS2	21.5	161.6

Date May 2010
Project 1-09-4135



Prep'd DB
Chkd. MP

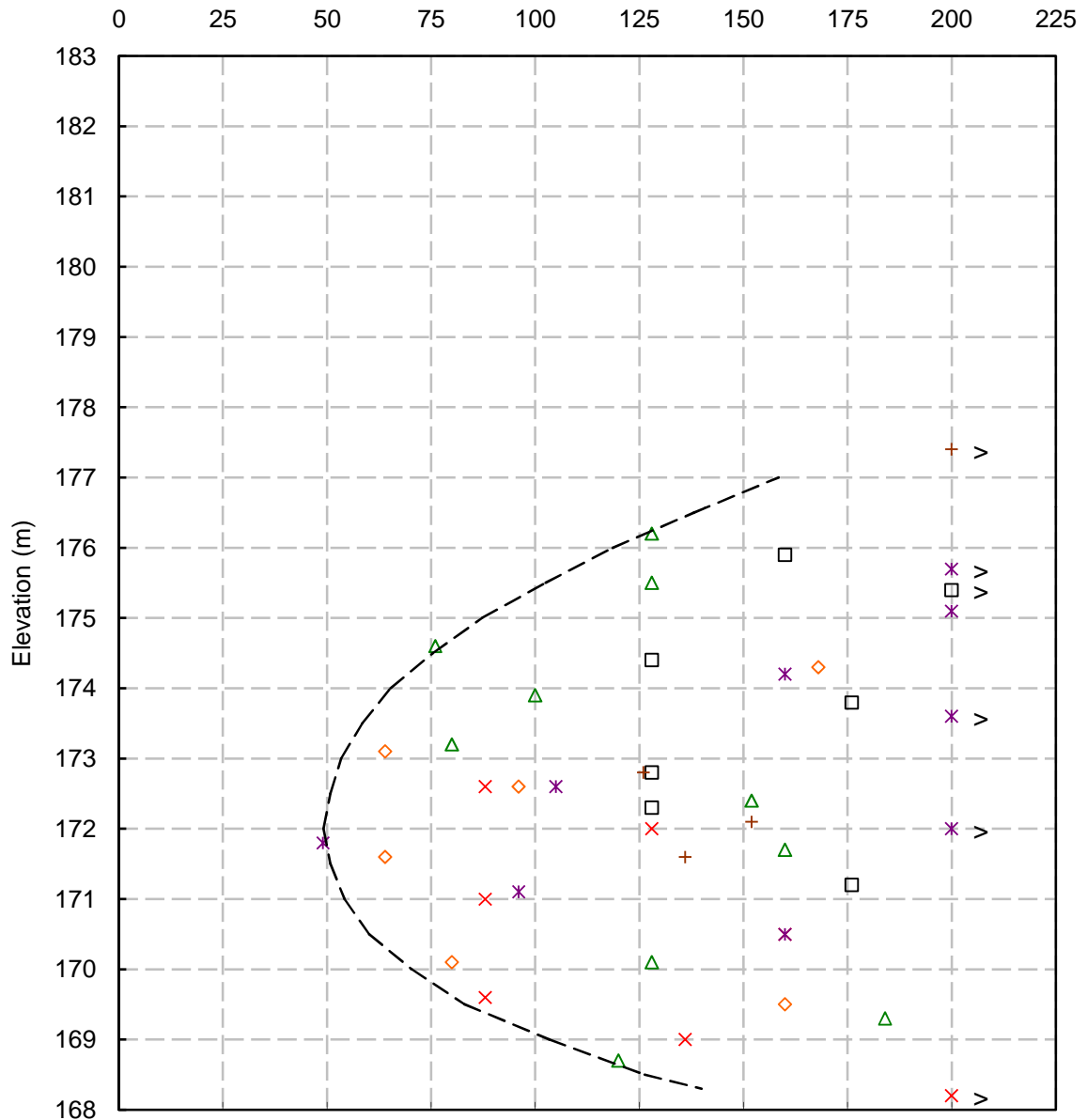
CORRECTED UNDRAINED SHEAR STRENGTH

FIGURE B20

HWY 406 TWINNING - WOODLAWN OVERPASS (SBL)

Silty Clay

Corrected Cu (kPa)



□ WS1 ◇ WS2 △ WS3 × WS4 * SBL 12+685 CL + SBL 12+750 CL

Field Shear Vane Correction

Morris & Williams (1994)

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

Applied Correction Factors

0.79 (Elev.>177m)

1.00 (Elev.<177m)

Project No. : 1-09-4135

Date : September, 2010



Terraprobe Inc.

Prepared By : HW

Checked By : RA

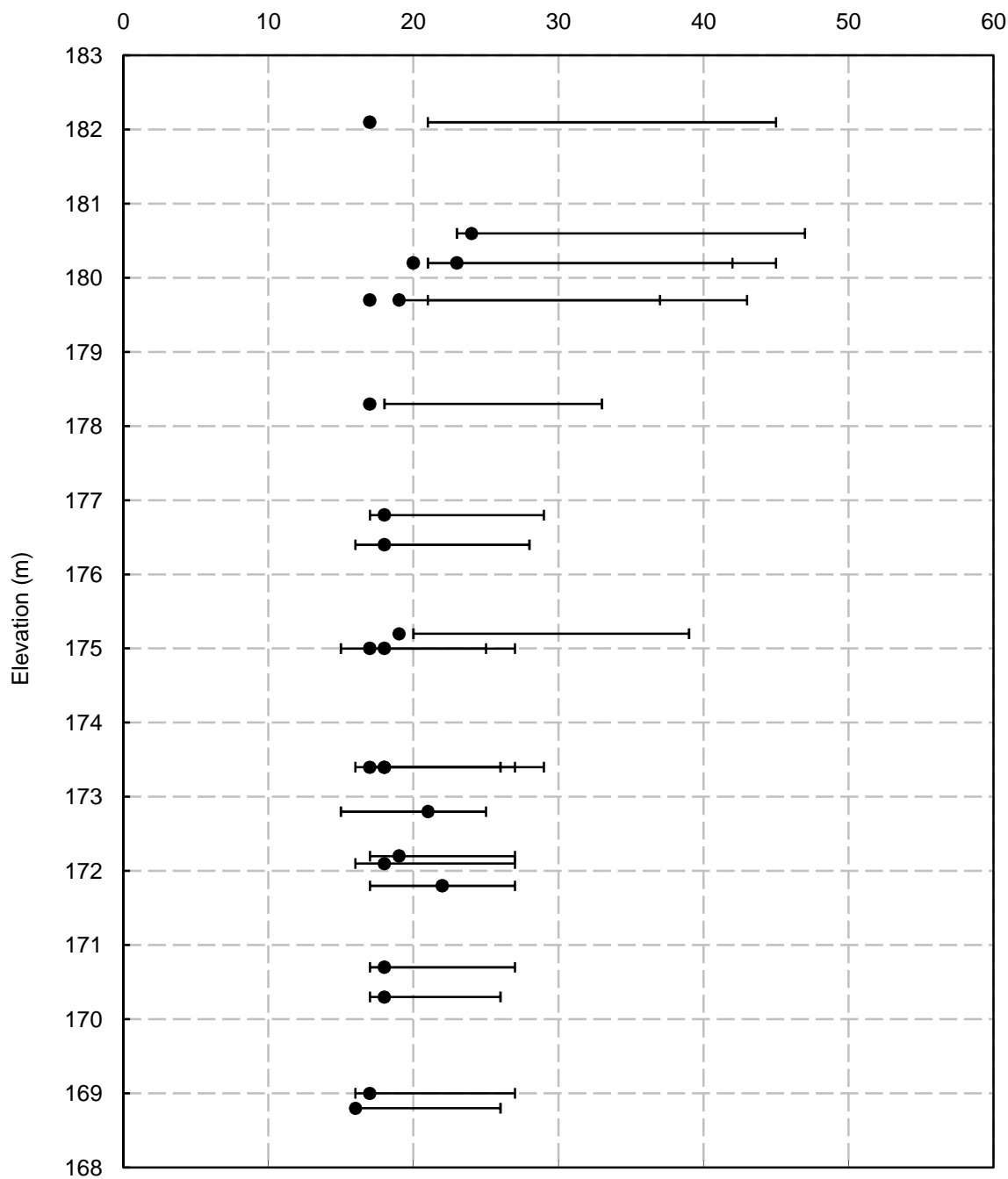
ATTERBERG LIMITS AND WATER CONTENTS

FIGURE B21

HWY 406 TWINNING - WOODLAWN OVERPASS (SBL)

Silty Clay

Atterberg Limits & Water Contents (%)



Project No. : 1-09-4135

Date : September, 2010



Prepared By : HW

Checked By : RA

C:\Documents and Settings\Hongliu\My Documents\Project 2009\1-09-4135 - HWY 406 Foundations\Bridges\1-09-4135 Soil Parameter Estimation-WS1.xls

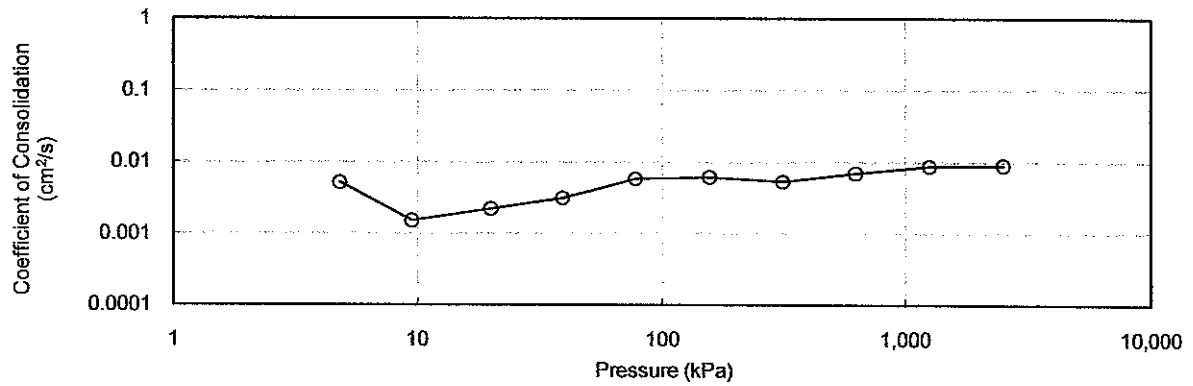
HWY 406 TWINNING - WOODLAWN OVERPASS (SBL)

FIGURE B22

CONSOLIDATION TEST

Cv vs Pressure

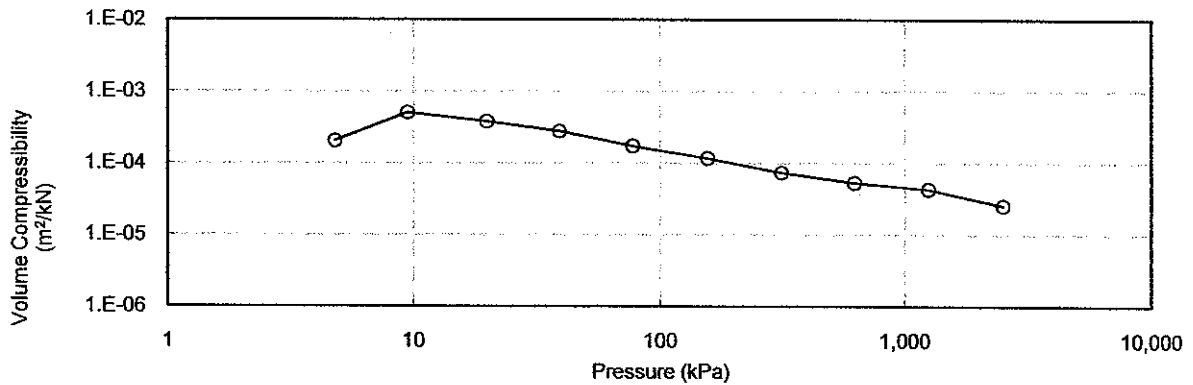
SBL 12+685 CL, TW10



CONSOLIDATION TEST

mv vs Pressure

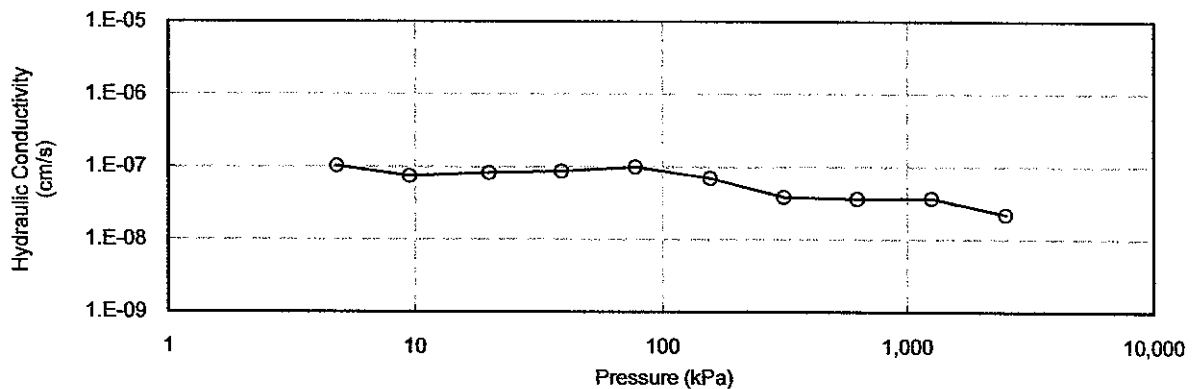
SBL 12+685 CL, TW10



CONSOLIDATION TEST

k vs Pressure

SBL 12+685 CL, TW10



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



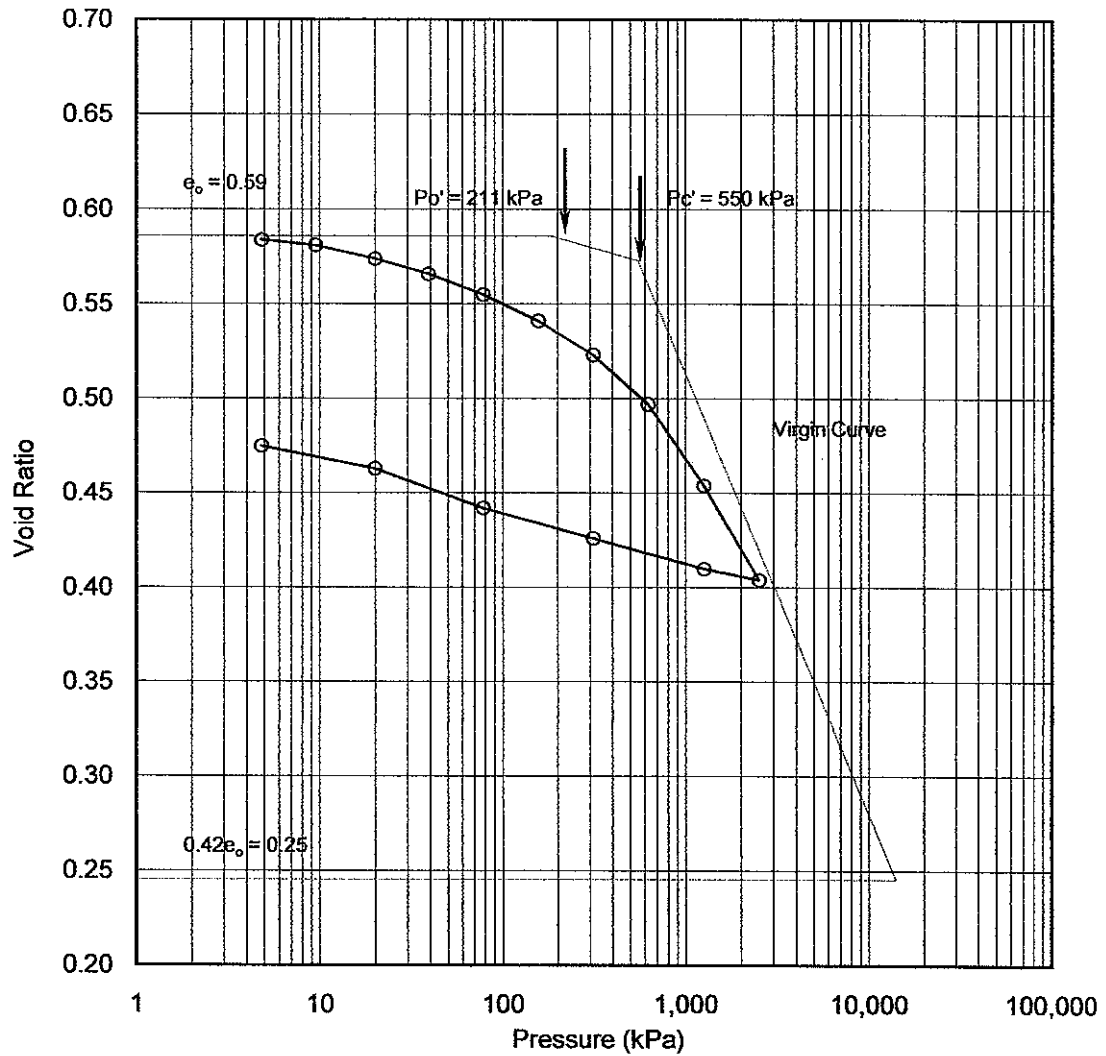
Terraprobe Inc.

Prepared By : HW
Checked By : RA

CONSOLIDATION TEST

e vs Pressure

SBL 12+685 CL, TW10



Soil Type : Silty Clay

$e_o =$	0.59	$\omega_L =$	27%	$P_{o'} =$	211 kPa
$\omega =$	22%	$\omega_P =$	17%	$P_{c'} =$	550 kPa
$\gamma =$	20.7 kN/m ³	PI =	10%	Cc =	0.233
Gs =	2.75			Cr =	0.027

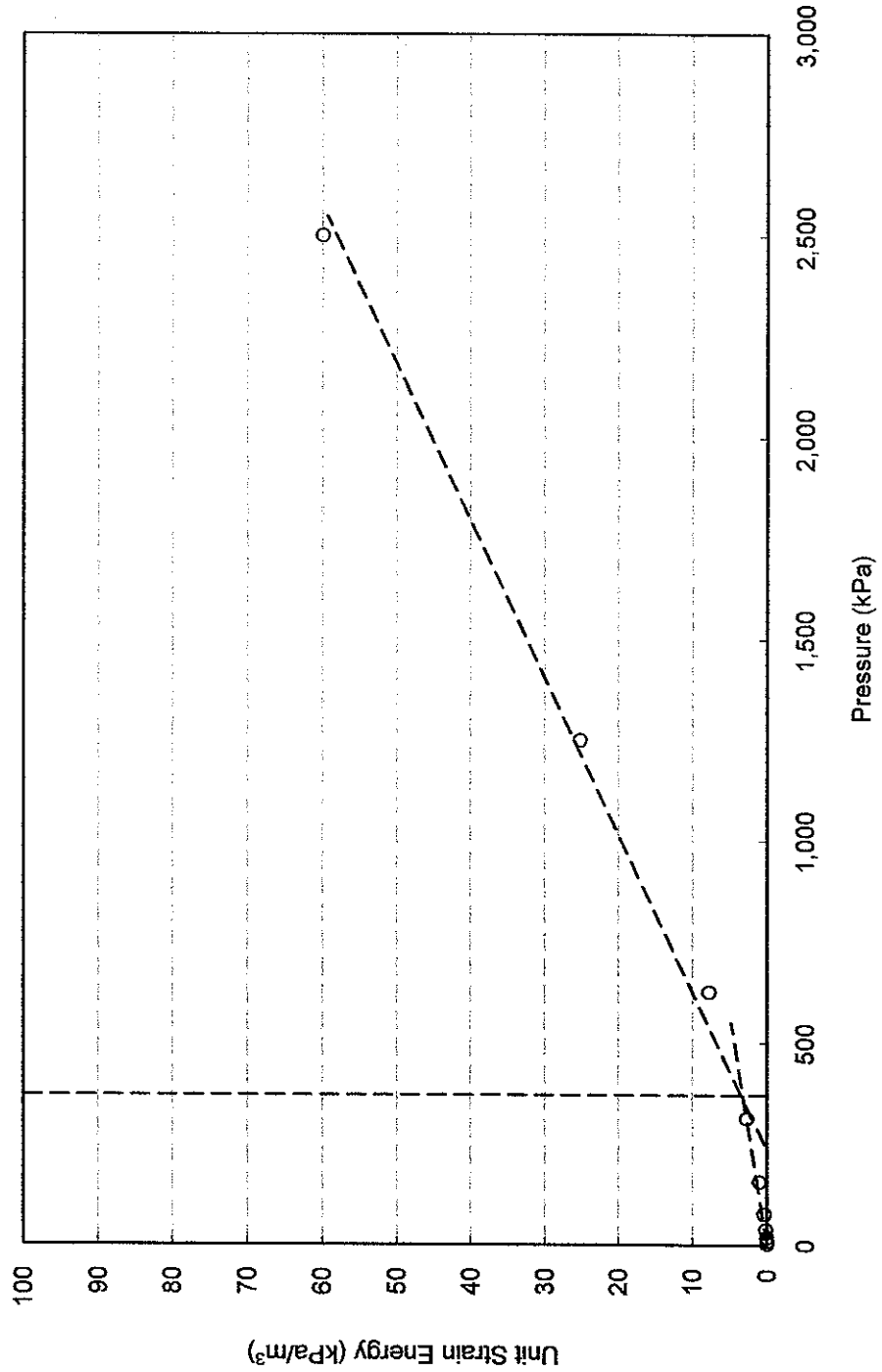
Project No. : 1-09-4135
 Date : September 2010



Terraprobe Inc.

Prepared By : HW
 Checked By : RA

CONSOLIDATION TEST
Unit Strain Energy vs Pressure
SBL 12+685 CL, TW10



$P_c = 370 \text{ kPa}$

Project No. : 1-09-4135

Date : September 2010



Terraprobe Inc.

Prepared By : HW

Checked By : RA

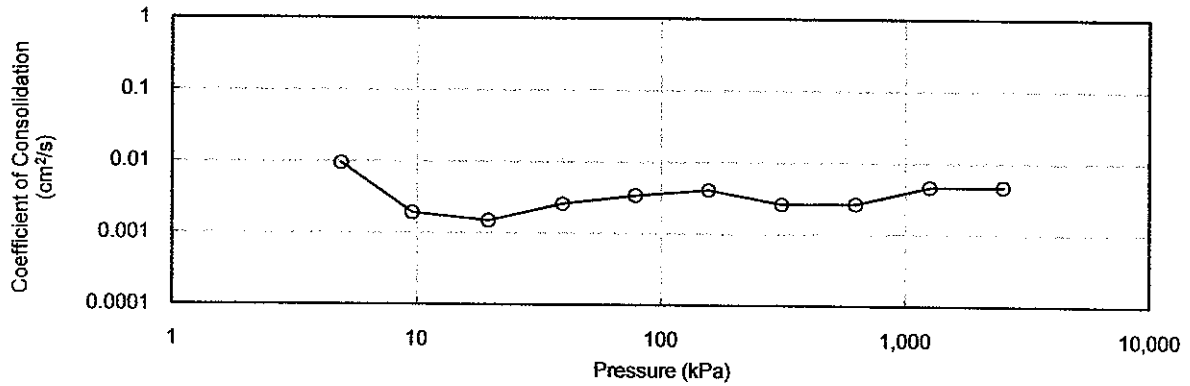
HWY 406 TWINNING - WOODLAWN OVERPASS (SBL)

FIGURE B25

CONSOLIDATION TEST

Cv vs Pressure

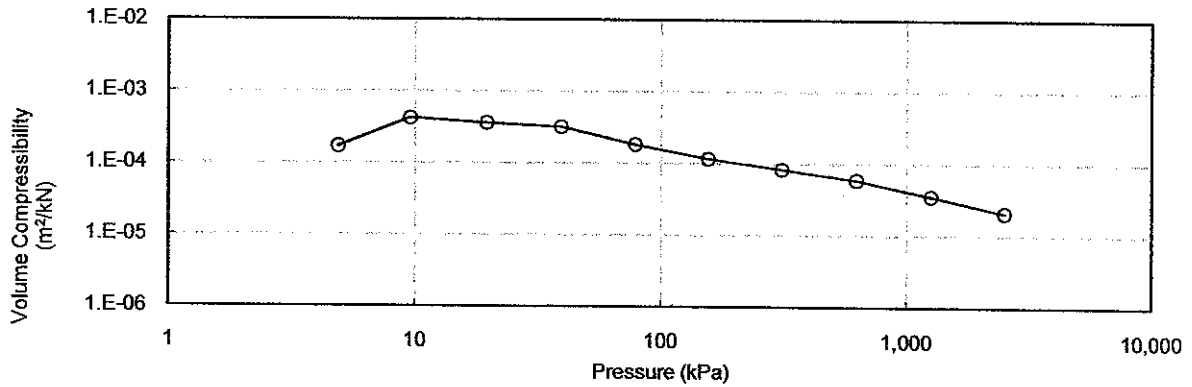
SBL 12+750 CL, TW10



CONSOLIDATION TEST

mv vs Pressure

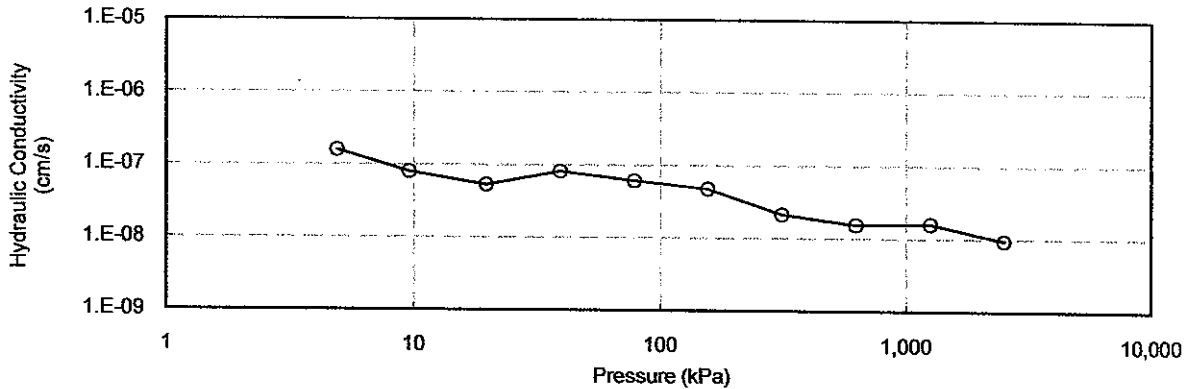
SBL 12+750 CL, TW10



CONSOLIDATION TEST

k vs Pressure

SBL 12+750 CL, TW10



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



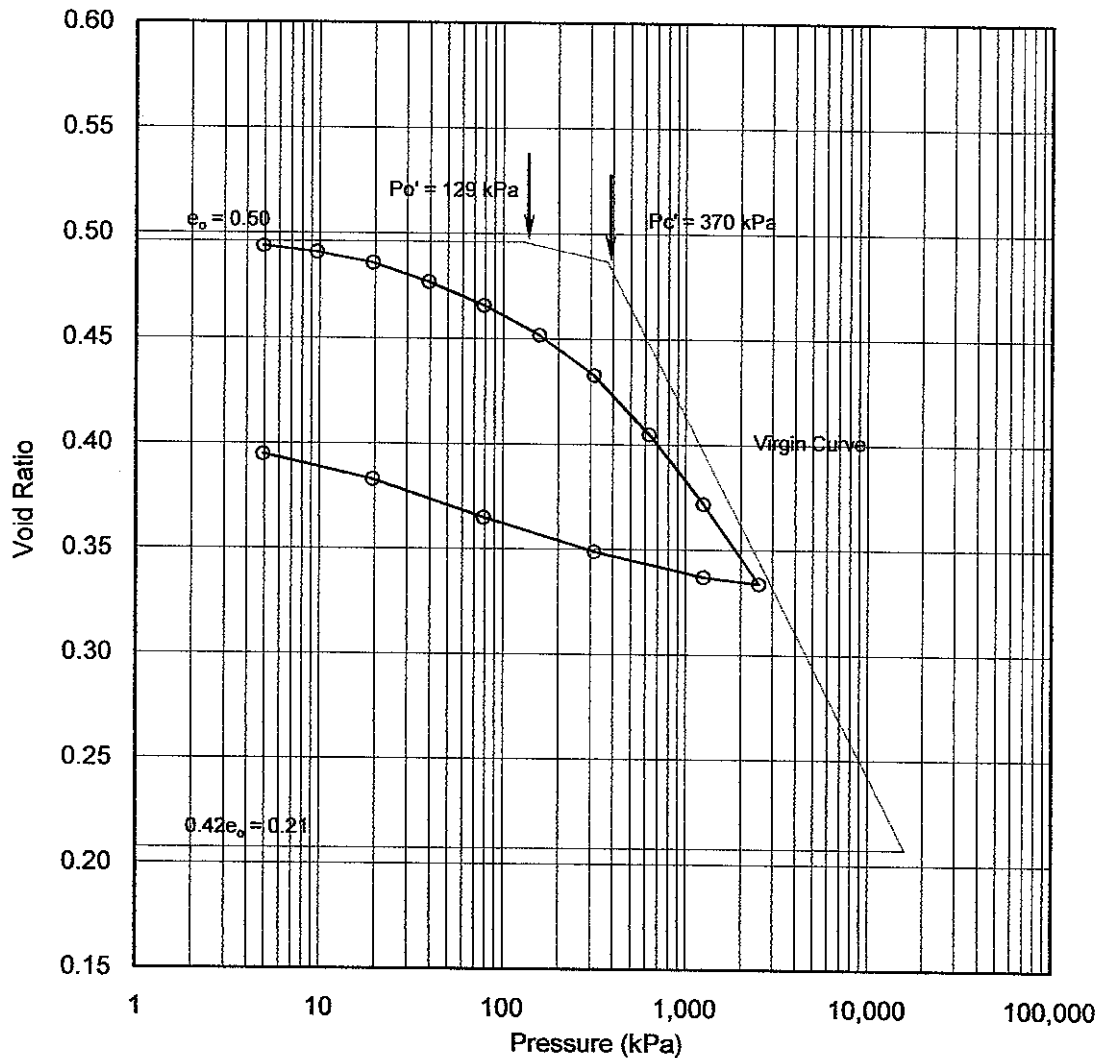
Terraprobe Inc.

Prepared By : HW
Checked By : RA

CONSOLIDATION TEST

e vs Pressure

SBL 12+750 CL, TW10



Soil Type : Silty Clay

$e_0 =$	0.50	$\omega_L =$	25%	$P_{o'} =$	129 kPa
$\omega =$	19%	$\omega_P =$	15%	$P_{c'} =$	370 kPa
$\gamma =$	21.1 kN/m ³	PI =	10%	Cc =	0.171
Gs =	2.70			Cr =	0.020

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



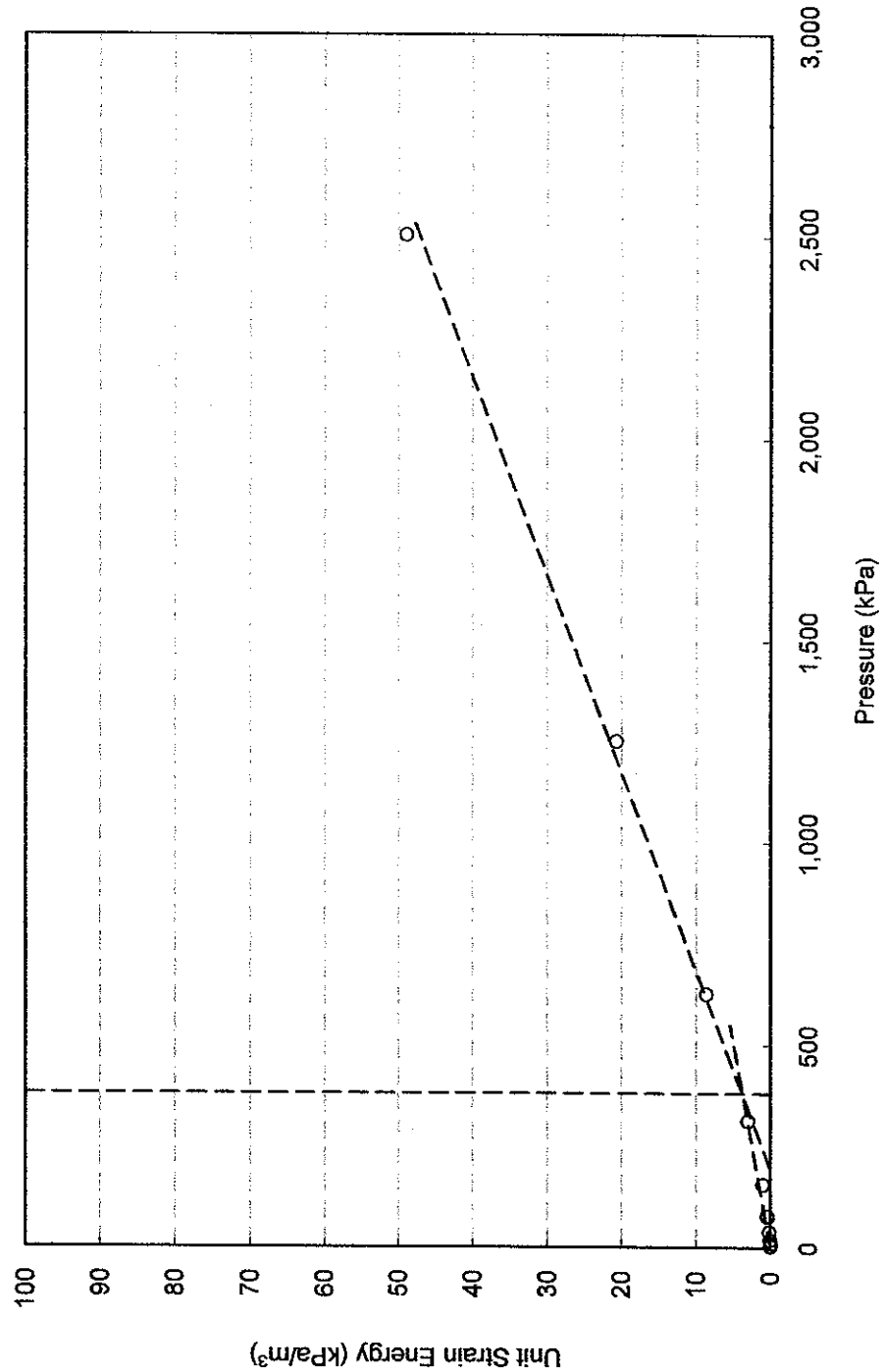
Terraprobe Inc.

Prepared By : HW
 Checked By : RA

HWY 406 TWINNING - WOODLAWN OVERPASS (SBL)

FIGURE B27

CONSOLIDATION TEST Unit Strain Energy vs Pressure SBL 12+750 CL, TW10



Project No. : 1-09-4135

Date : September 2010



Terraprobe Inc.

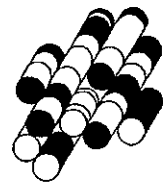
Prepared By : HW

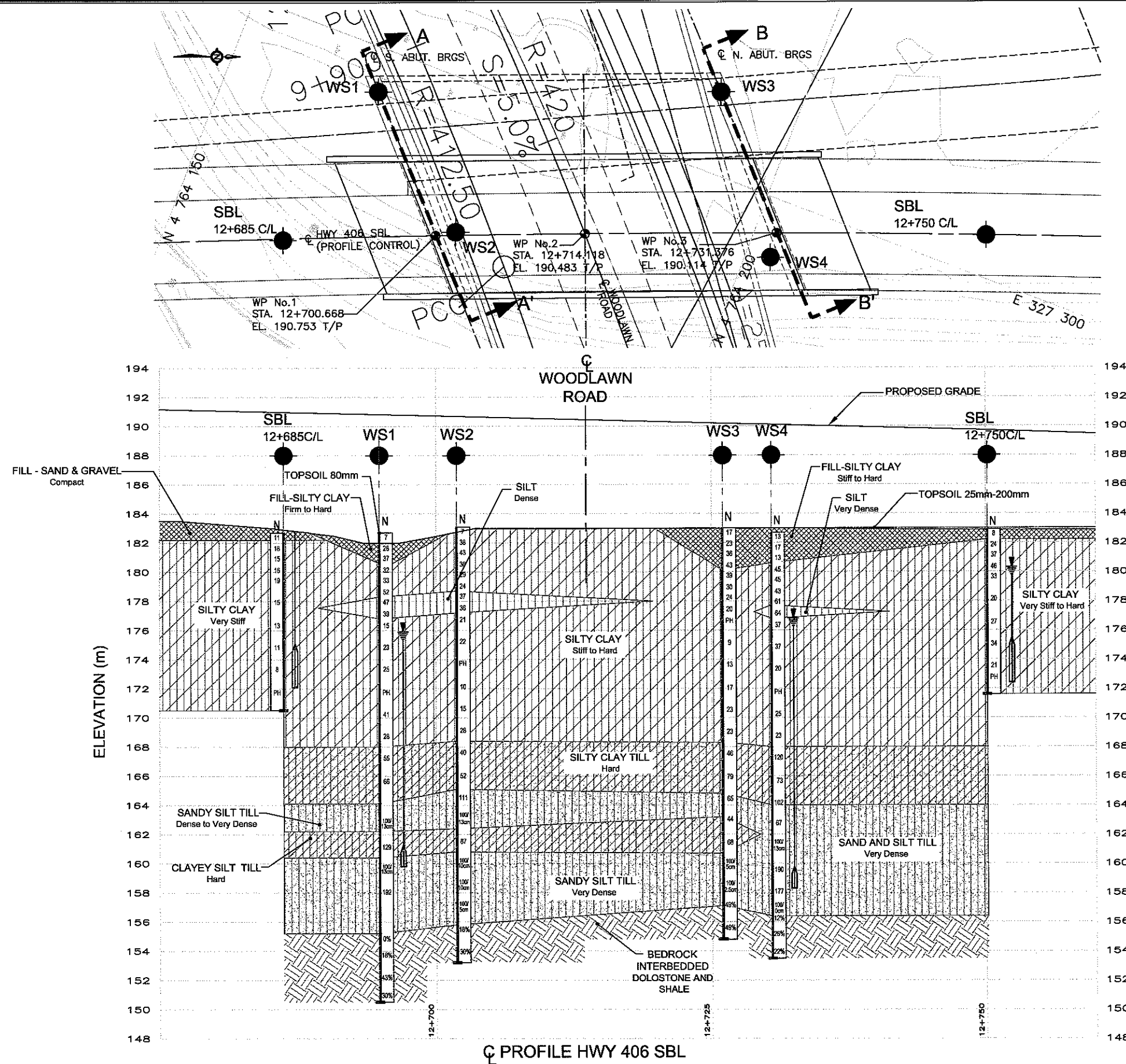
Checked By : RA

APPENDIX C

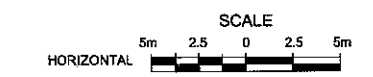
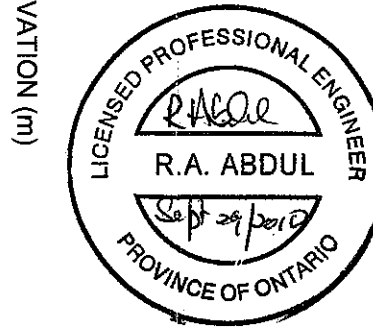
**Drawings titled “Borehole
Locations and Soil Strata”**

Terraprobe Inc.





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

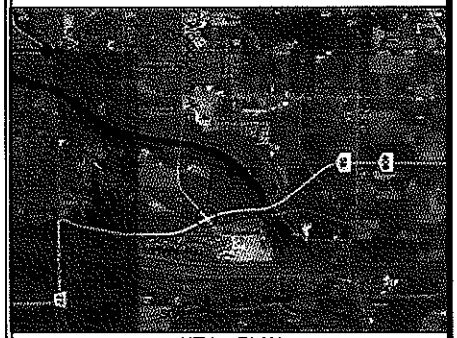


CONT No
WP No 280-99-00

HIGHWAY 406
HIGHWAY 406 SBL
WOODLAWN ROAD OVERPASS
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)
- Piezometer
- Rock Quality Designation
- Auger Refusal

No	ELEV.	COORDINATES	
		NORTHING	EASTING
WS1	182.7	4 764 163.6	327 303.9
WS2	183.1	4 764 174.7	327 313.4
WS3	183.0	4 764 192.6	327 292.7
WS4	182.7	4 764 202.3	327 305.2
SBL 12+685C/L	182.7	4 764 160.4	327 319.7
SBL 12+750C/L	182.9	4 764 219.9	327 296.2

NOTE

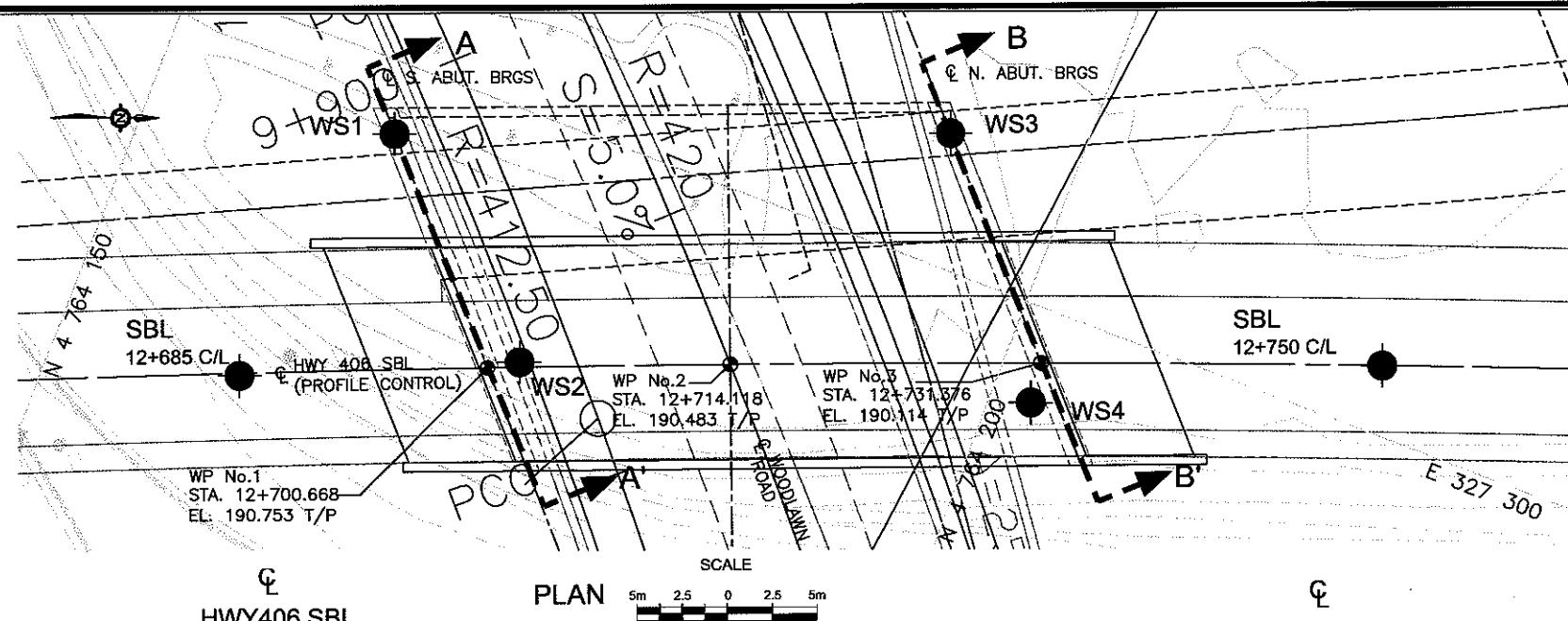
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

REVISIONS

DATE	BY	DESCRIPTION
DESIGN R.A.	CODE CHB0C2006	LOAD
DRAWN K.C.	CHK R.A.	STRUCT 34-463/2
		GEODRES 30M3-260

DATE SEPT. 2010



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

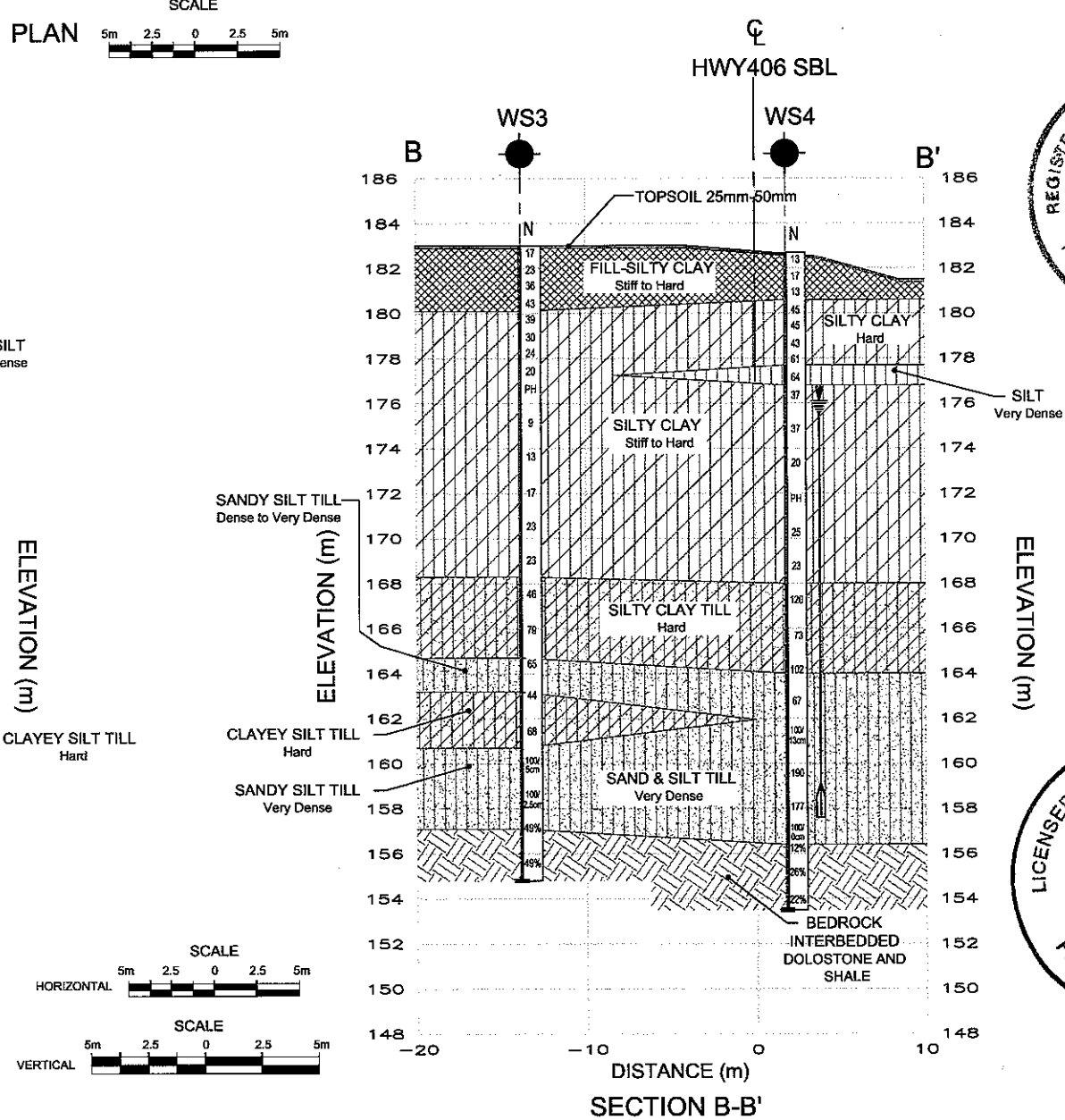
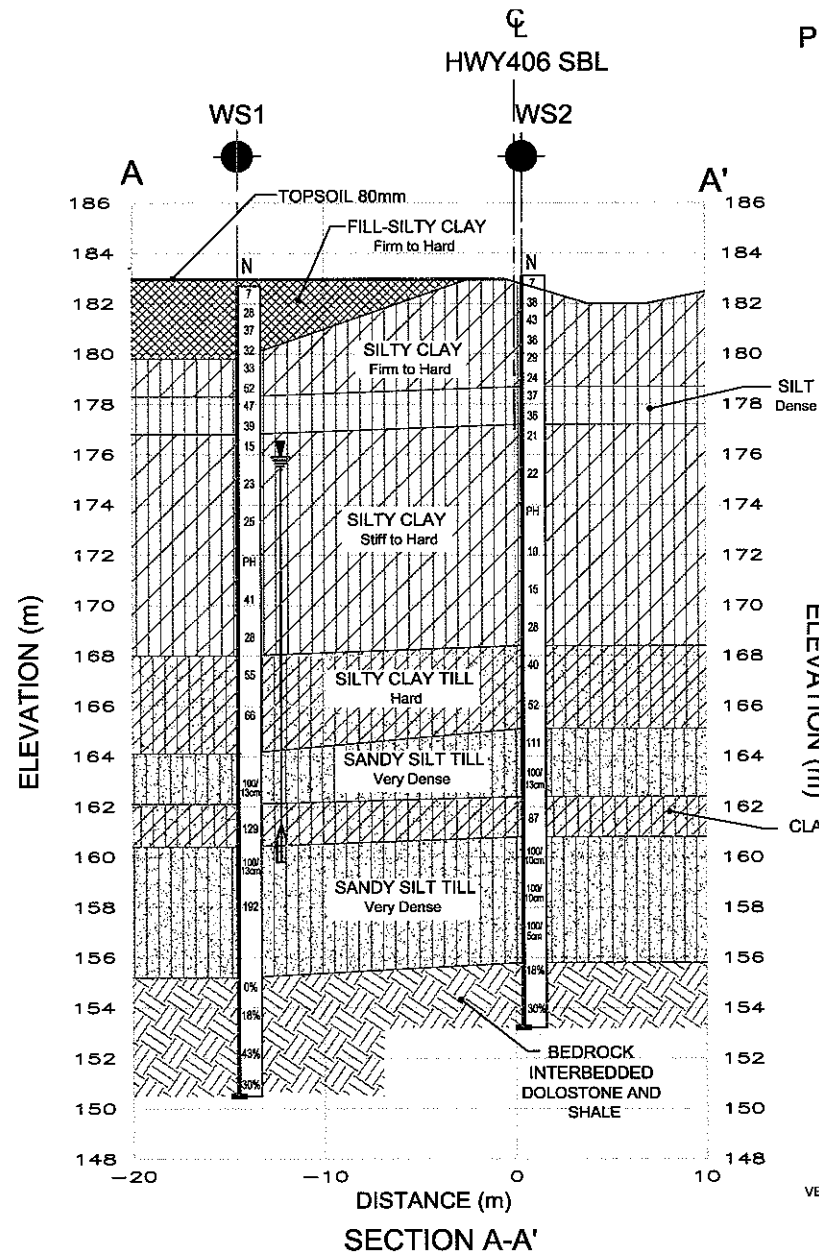
CONT No
WP No 280-99-00



HIGHWAY 406
HIGHWAY 406 SBL
WOODLAWN ROAD OVERPASS
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
- 'N'
- Blows/0.3m (Std Pen Test, 475 J/blow)
- CONE
- Blows/0.3m (60° Cone, 475 J/blow)
- WL at Time of Investigation
- WL in Piezometer (MAY 2010)
- Piezometer
- 90% Rock Quality Designation
- A/R Auger Refusal

No	ELEV.	COORDINATES	
		NORTHING	EASTING
WS1	182.7	4 764 163.6	327 303.9
WS2	183.1	4 764 174.7	327 313.4
WS3	183.0	4 764 192.6	327 292.7
WS4	182.7	4 764 202.3	327 305.2
SBL 12+685C/L	182.7	4 764 160.4	327 319.7
SBL 12+750C/L	182.9	4 764 219.9	327 296.2

NOTE

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

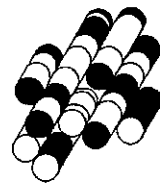
This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

REVISIONS	DATE	BY	DESCRIPTION
DESIGN R.A.	CODE CHBDC2006	LOAD	DATE SEPT. 2010
DRAWN K.C.	CHK R.A.	STRUCT 34-483/2	GEOPRES 30M3-280

APPENDIX D

Foundation Comparison

Terraprobe Inc.



COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

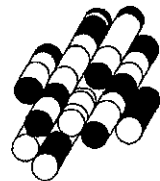
Foundation Element	Driven Piles	Augered Caissons	Footing on Native Soil	Footing on Engineered Fill
North and South Abutments	<p>Advantages:</p> <ul style="list-style-type: none"> i. High geotechnical resistances available by driving piles to effective refusal. ii. Readily installed. iii. Reliable performance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



APPENDIX E

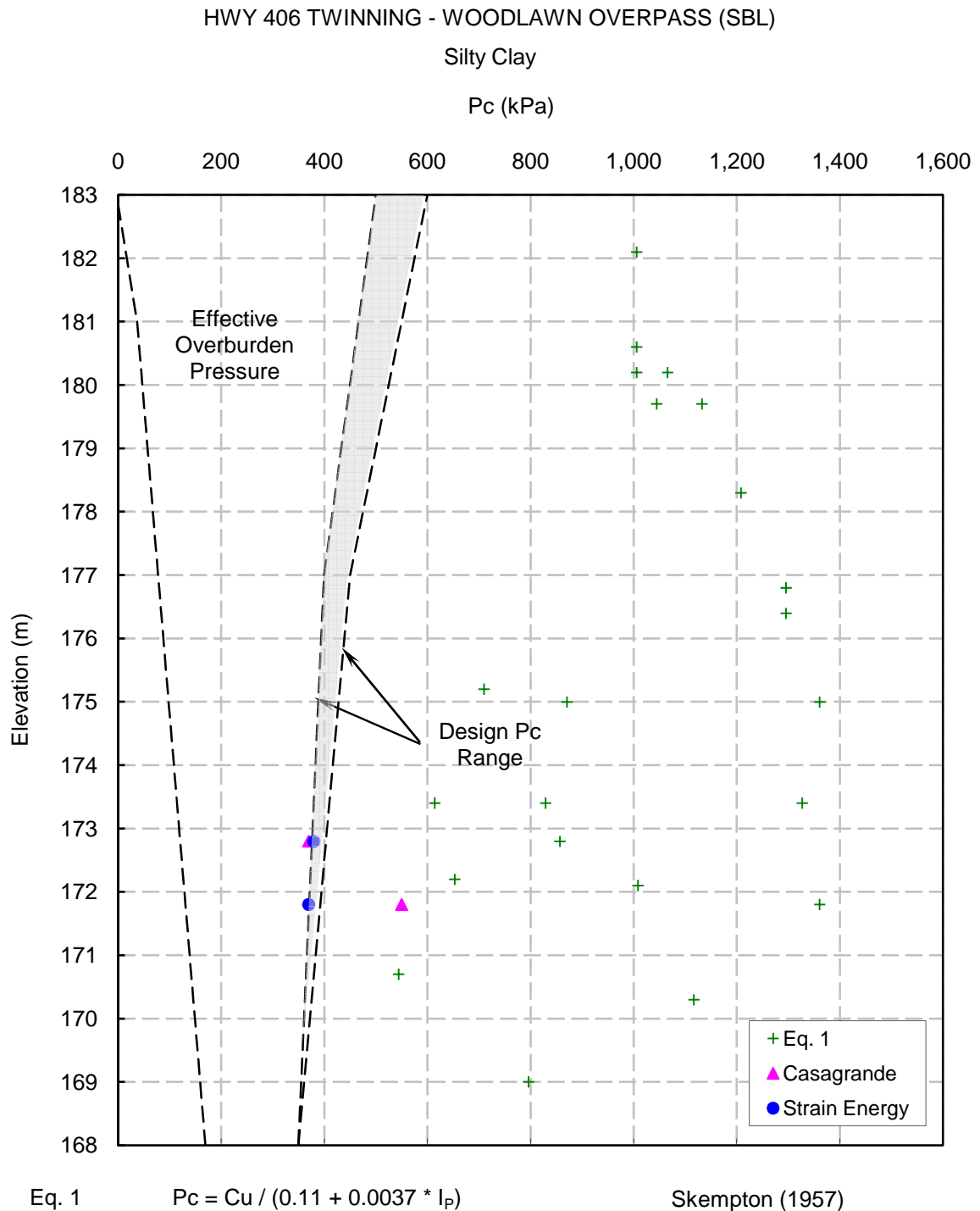
Settlement Parameters and Results

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

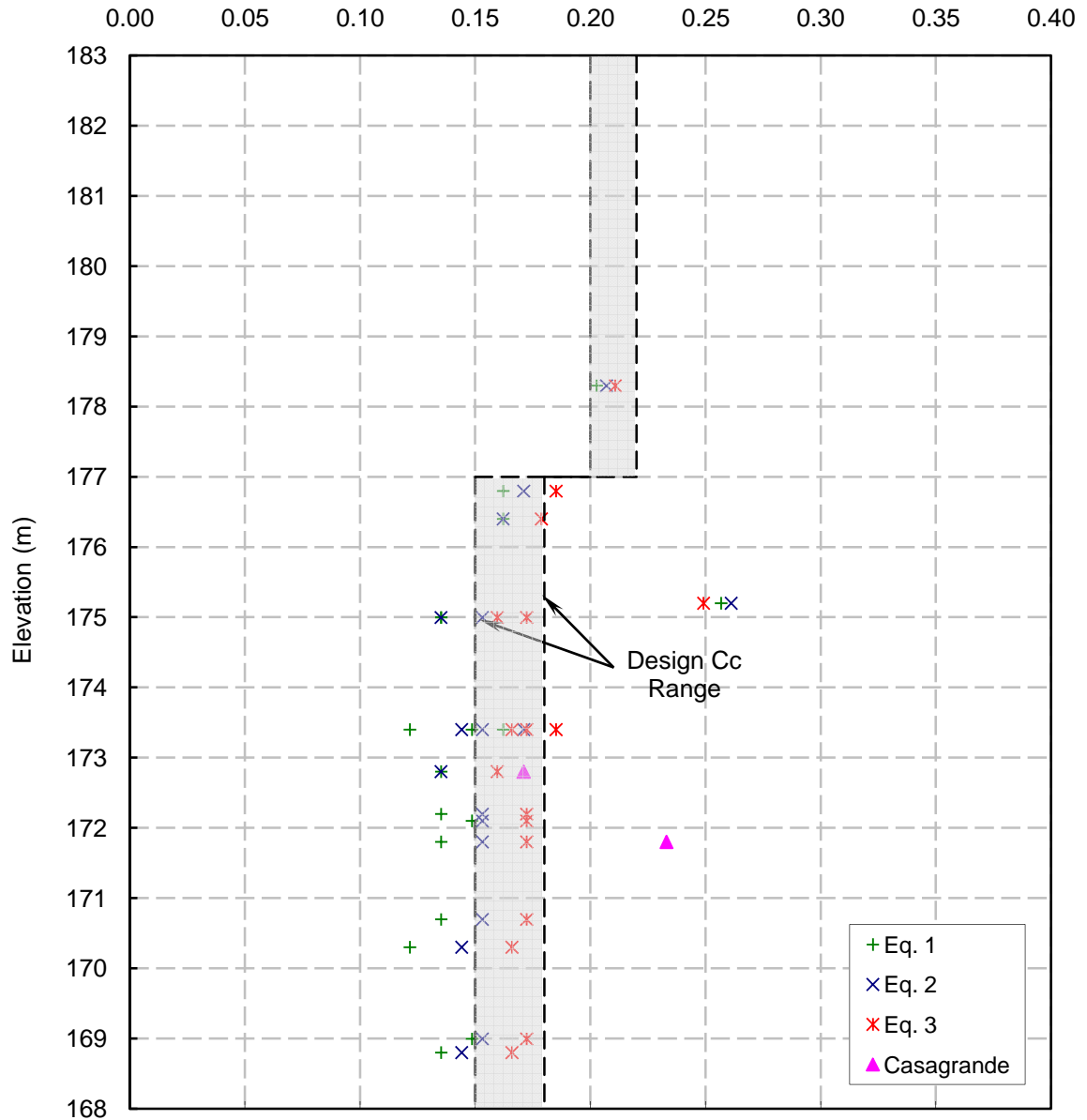
PREDICTED AND MEASURED COMPRESSION INDEX

FIGURE E2

HWY 406 TWINNING - WOODLAWN OVERPASS (SBL)

Silty Clay

Cc



Eq. 1 $Cc = Ip / 74$

Kulhawy & Mayne (1990)

Eq. 2 $Cc = 0.009 * (LL - 10)$

Terzaghi & Peck (1967)

Eq. 3 $Cc = 0.2343 * LL * Gs$

Nagaraj & Murty (1985)

Project No. : 1-09-4135

Date : September, 2010



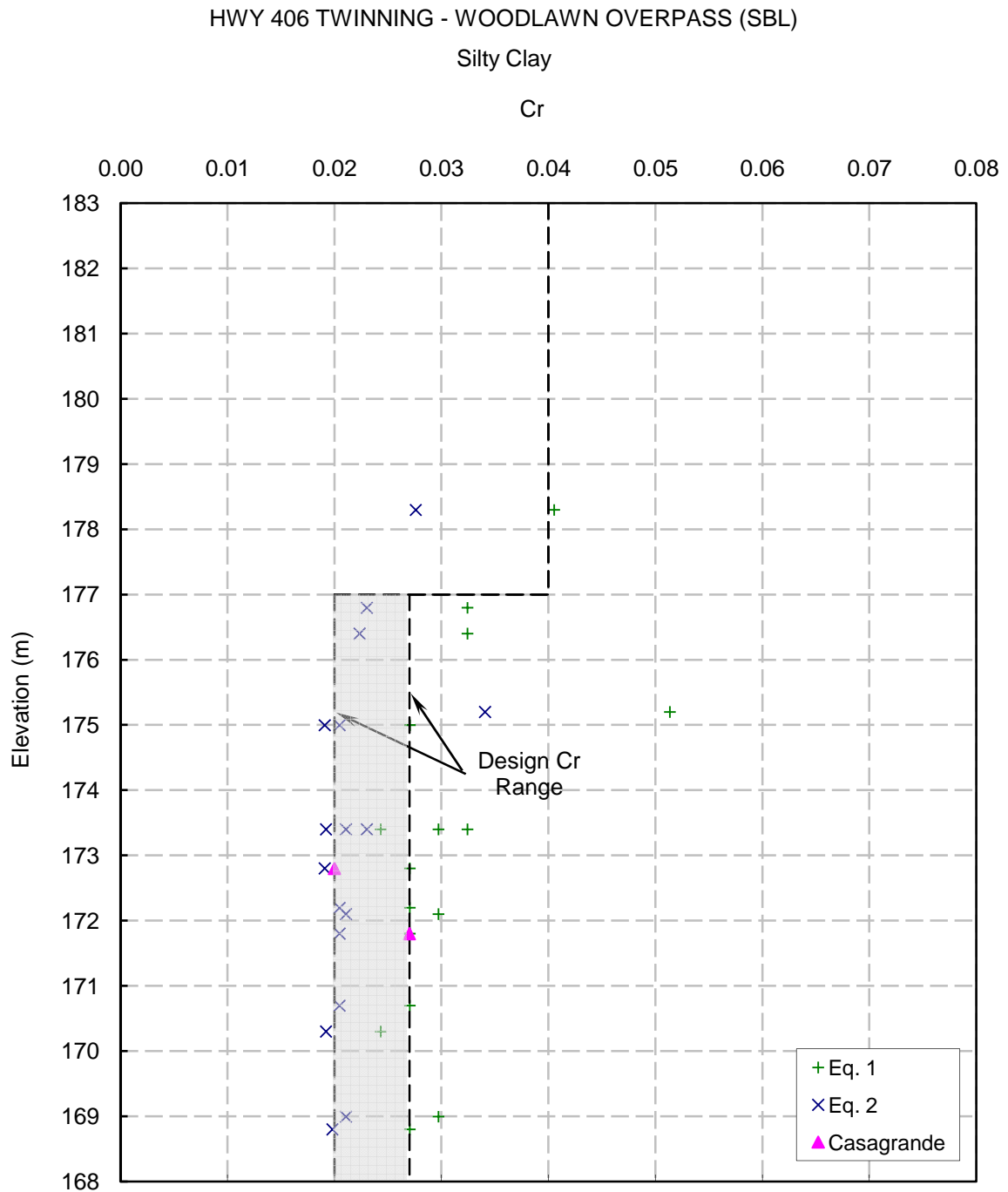
Terraprobe Inc.

Prepared By : HW

Checked By : RA

PREDICTED AND MEASURED RECOMPRESSION INDEX

FIGURE E3



Eq. 1 $Cr = Ip / 370$

Kulhawy & Mayne (1990)

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

Das (1993)

Project No. : 1-09-4135

Date : September, 2010



Terraprobe Inc.

Prepared By : HW

Checked By : RA

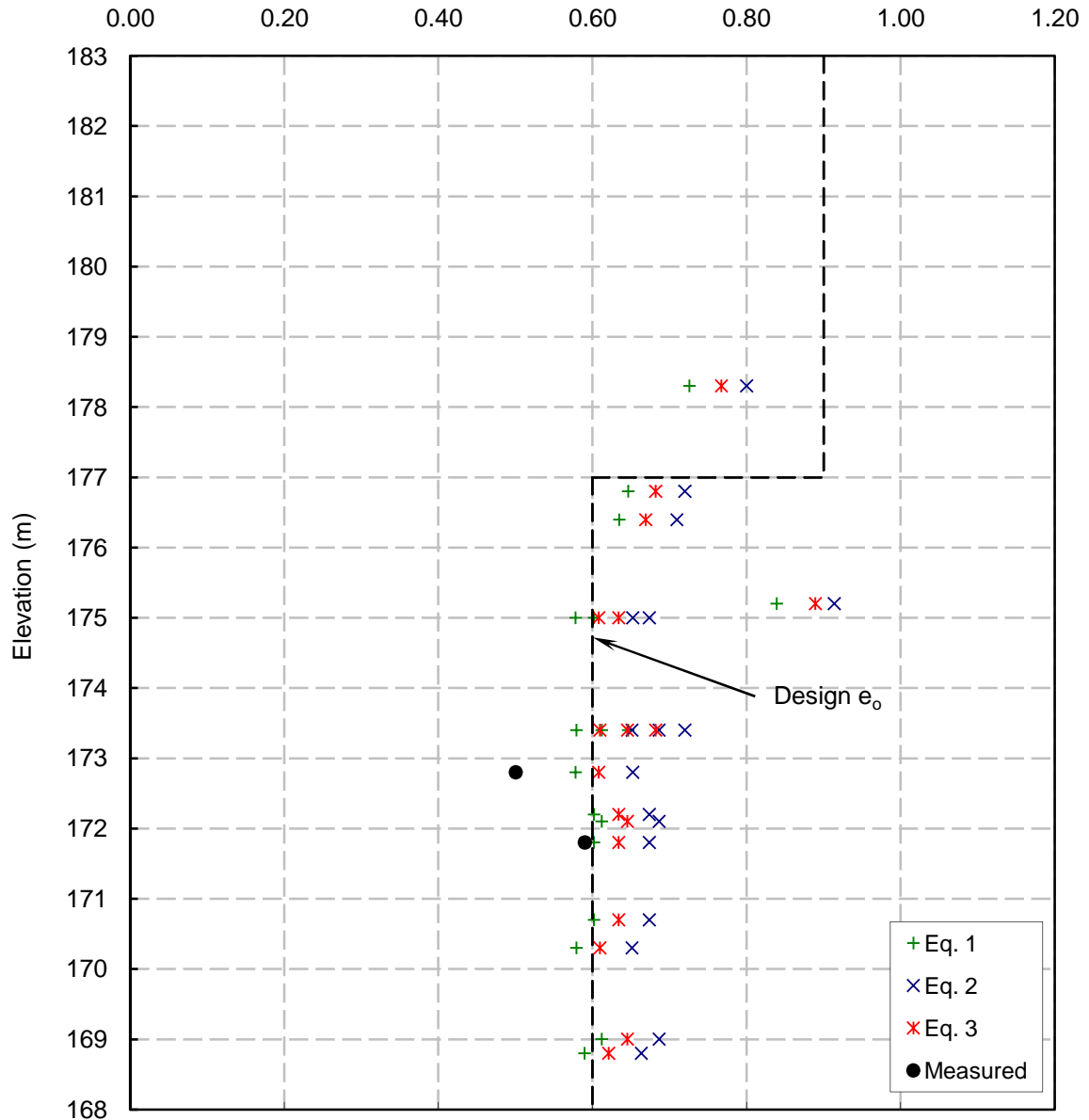
PREDICTED AND MEASURED VOID RATIO

FIGURE E4

HWY 406 TWINNING - WOODLAWN OVERPASS (SBL)

Silty Clay

e_o



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.

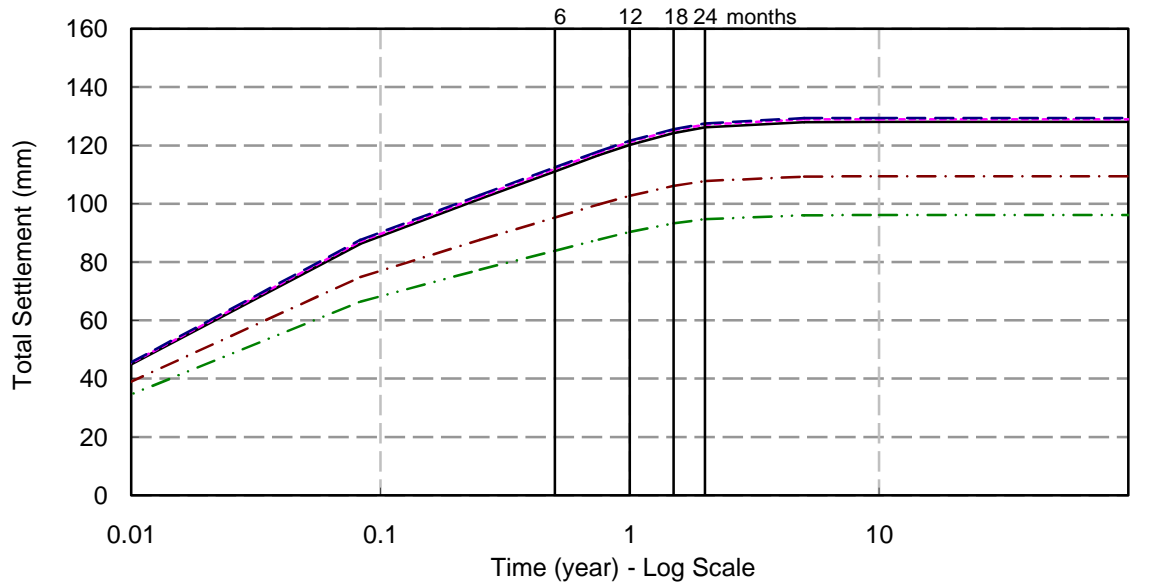
Prepared By : HW

Checked By : RA

SETTLEMENT VS. LOG TIME

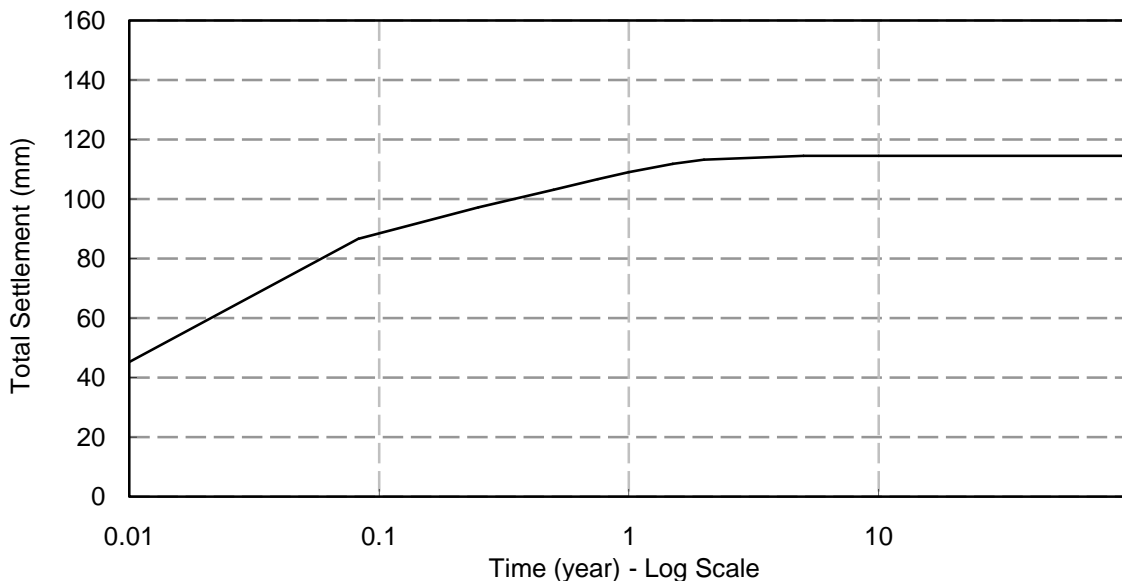
FIGURE E5

HWY 406 TWINNING - WOODLAWN OVERPASS (SBL) Embankment



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

RSS Block



— RSS Block

Project No. : 1-09-4135

Date : September, 2010



Terraprobe Inc.

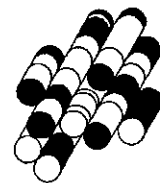
Prepared By : HW

Checked By : RA

APPENDIX F

Suggested NSSP Wording

Terraprobe Inc.



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

- OPSSS 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”.

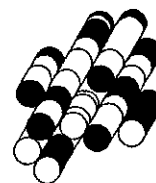
If a pile encounters refusal on cobbles and boulders the QVE should terminate driving before the pile is damaged by overdriving. If a pile has not been driven to effective refusal at the recommended design tip elevations and further driving is likely to cause damage to the pile, pile driving should cease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



APPENDIX G

Settlement Monitoring Guidelines

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 plates and 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.

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;
- 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 – Woodlawn Road Overpass, Highway 406 SBL, Highway 406 Twinning, Ontario. Site No., W.P. 280-99-00, Structure No. 34-463/2, Geocres No. 30M3-260, dated September 29, 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 – Instrumentation & Benchmark Quantities and Locations

INSTRUMENT I.D.	STATION	OFFSET FROM CENTRELINE	NO. OF INSTRUMENTS		
			SP	VWP	SSP
North Approach					
SP1	406 SBL 12+736	2.5 m Lt.	1		
SP2	406 SBL 12+736	2.5 m Rt.	1		
SP3	406 SBL 12+755	CL	1		
VWP1	406 SBL 12+745	CL		1	
SSP1	406 SBL 12+745	Outside of construction area			1
SP4	406 SBL 12+760	2.5 m Lt.	1		
SP5	406 SBL 12+760	2.5 m Rt.	1		
BM1	N/A	N/A			
South Approach					
SP6	406 SBL 12+695	2.5 m Lt.	1		
SP7	406 SBL 12+695	2.5 m Rt.	1		
SP8	406 SBL 12+680	CL	1		
VWP2	406 SBL 12+690	CL		1	
SSP2	406 SBL 12+690	Outside of construction area			1
SP9	406 SBL 12+670	2.5 m Lt.	1		
SP10	406 SBL 12+670	2.5 m Rt.	1		
Total Instruments			10	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. The number and locations(s) of benchmark(s) shall be such that direct sighting is possible from all settlement plates (SP) to at least one benchmark.

2.3 Accuracy of Surveying for Elevations

Elevations shall be surveyed referenced to Geodetic datum to an accuracy of ± 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 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 Installation Program

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

Table 2.2 – Installation Program

TYPE	START INSTALLATION	FINISH INSTALLATION
SP	After excavating to recommended stripping elevation of embankment	On completion of embankment/RSS 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/RSS construction	Before commencement of embankment/RSS 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 (SR).

3.1.2 General Procedure

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 (SR) 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.

Table 3 – Approximate Bench Mark Locations

Station	Offset (m)	No. of BM	Estimated Rod Anchor Elevation (m)
Outside of Construction Area	N/A	BM1	166.0

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 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 rods are shown on the Contract Drawings and are given in Table 4.

Table 4 – Approximate Settlement Plate Locations

Station	Offset (m)	No. of SP	Estimated Fill Thickness (m)*
North Approach			
406 SBL 12+736	2.5 m Lt.	1	8.0
406 SBL 12+736	2.5 m Rt.	1	8.0
406 SBL 12+755	CL	1	8.0
406 SBL 12+760	2.5 m Lt.	1	7.5
406 SBL 12+760	2.5 m Rt.	1	7.5
South Approach			
406 SBL 12+695	2.5 m Lt.	1	9.0
406 SBL 12+695	2.5 m Rt.	1	9.0
406 SBL 12+680	CL	1	9.0
406 SBL 12+670	2.5 m Lt.	1	9.5
406 SBL 12+670	2.5 m Rt.	1	9.5

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 rods.

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 ± 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 rods shall be done by others. Monitoring shall be conducted during the embankment construction and preload period. A target settlement of 110 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

Station	Offset (m)*	No. of VWP	Approximate Elevation of Ground Surface (m)	Tip Elevations (m)
406 SBL 12+745	CL	1	182.7	174.0
406 SBL 12+690	CL	1	182.5	174.0

Notes: * Offset from centerline of Hwy 406.

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 weathered proof enclosure surrounded by 2 m high chainlink fence and a lockable gate. The Monitoring Shed shall also be seated 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 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 benchmark 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

Station	Offset* (m)	No. of SSP	Tip Elevations (m)
406 SBL 12+745	30 LT	1	174.0
406 SBL 12+690	30 LT	1	174.0

Note: * Approx. offset from centerline of Hwy 406.

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 the piling and embankment fill construction and preload period at the approaches. 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.

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

CONT No
WP No 280-99-00



HWY 406 SBL
SETTLEMENT MONITORING
INSTRUMENT LAYOUT
WOODLAWN ROAD OVERPASS

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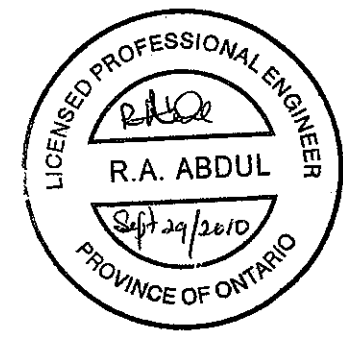
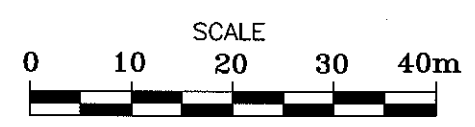
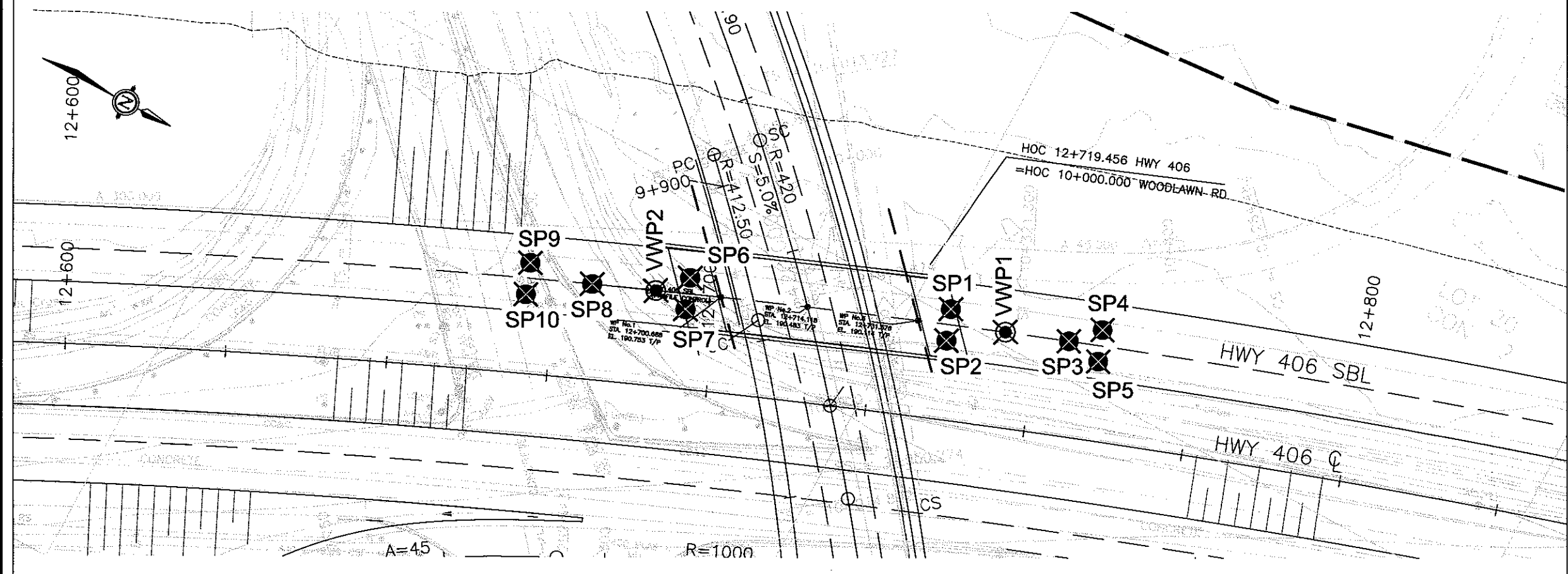
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- 1. THIS DRAWING TO BE READ IN CONJUNCTION WITH INSTRUMENT DETAILS DRAWING.

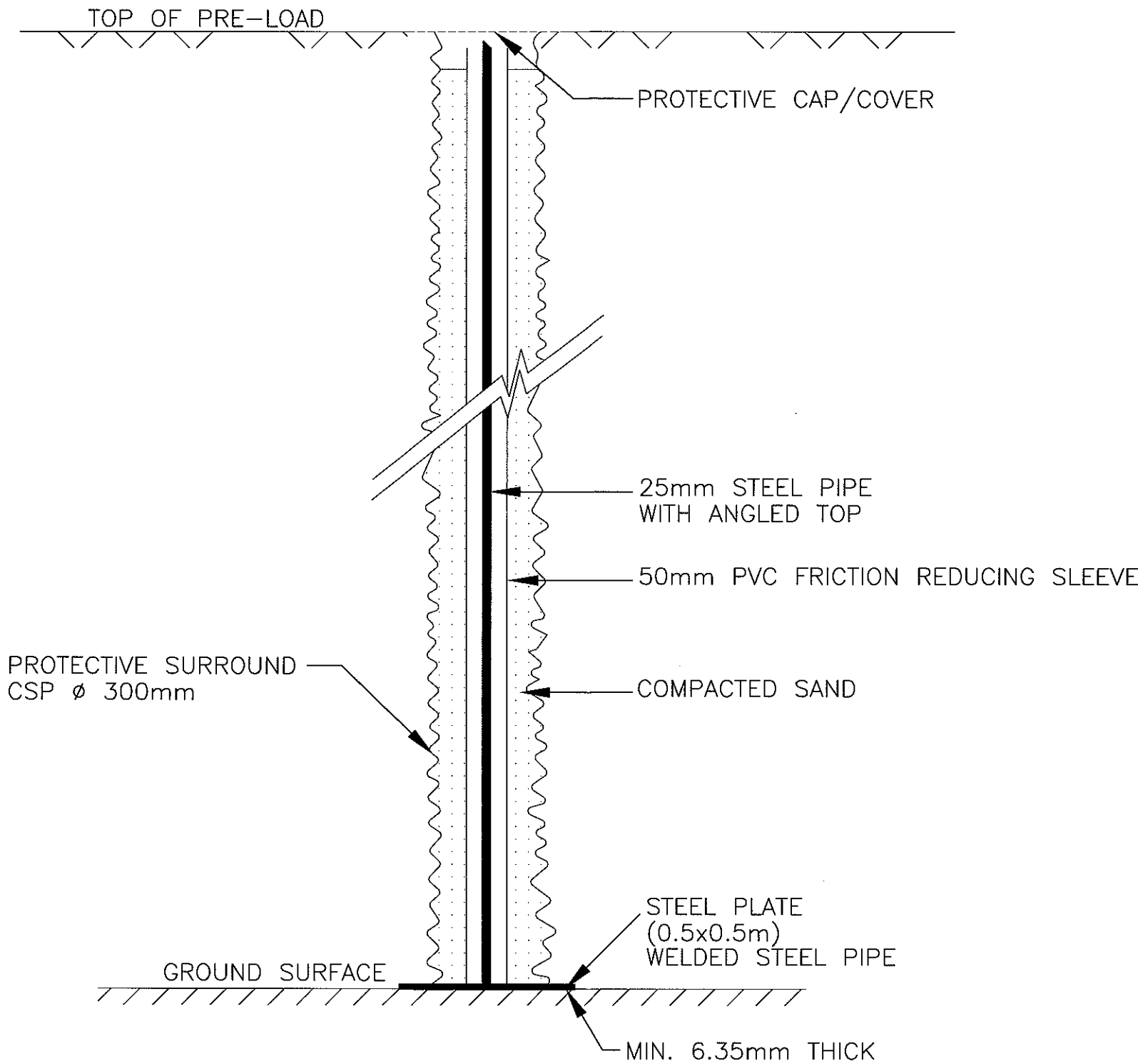
LEGEND

- SP1 APPROXIMATE LOCATION OF SETTLEMENT PLATE (SP)
- VWP1 APPROXIMATE LOCATION OF VIBRATING WIRE PIEZOMETER (VWP)

INSTRUMENT LOCATIONS			
I.D.	LOCATION	STATION	OFFSET FROM CENTRELINE(m)
WOODLAWN SBL			
SP1	406 SBL	12+736	2.5 Lt
SP2	406 SBL	12+736	2.5 Rt
SP3	406 SBL	12+755	0
SP4	406 SBL	12+760	2.5 Lt
SP5	406 SBL	12+760	2.5 Rt
SP6	406 SBL	12+695	2.5 Lt
SP7	406 SBL	12+695	2.5 Rt
SP8	406 SBL	12+680	0
SP9	406 SBL	12+670	2.5 Lt
SP10	406 SBL	12+670	2.5 Rt
VWP1	406 SBL	12+745	0
VWP2	406 SBL	12+690	0



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



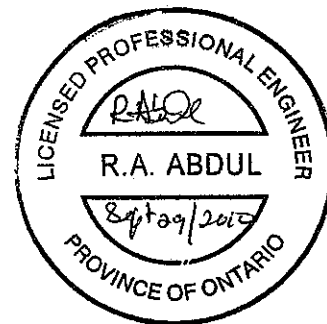
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETERS
UNLESS OTHERWISE SHOWN

CONT No
WP No 280-99-00

HWY 406 SBL
SETTLEMENT MONITORING
INSTRUMENT DETAILS
WOODLAWN ROAD OVERPASS

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GENERAL NOTES:
THIS DRAWING TO BE READ IN CONJUNCTION
1. WITH THE SETTLEMENT MONITORING
INSTRUMENT LAYOUT DWG.



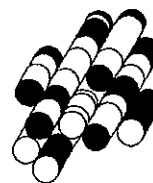
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N.T.S.

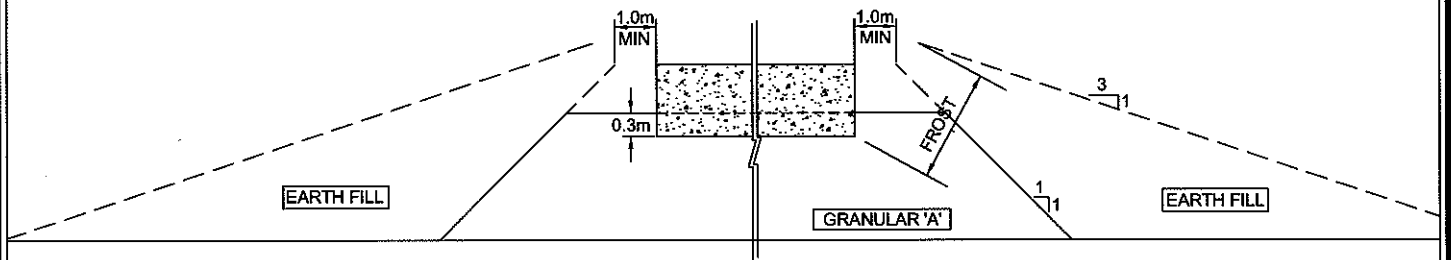
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				K.C.	CHK	STRUCT

APPENDIX H

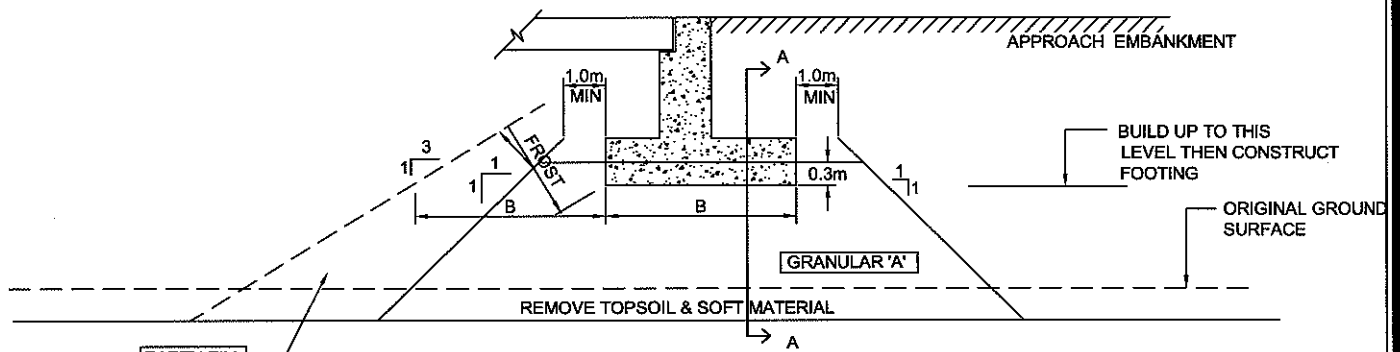
Figures

Terraprobe Inc.





CROSS-SECTION A-A
NOT TO SCALE



LONGITUDINAL SECTION
NOT TO SCALE

NOTES:

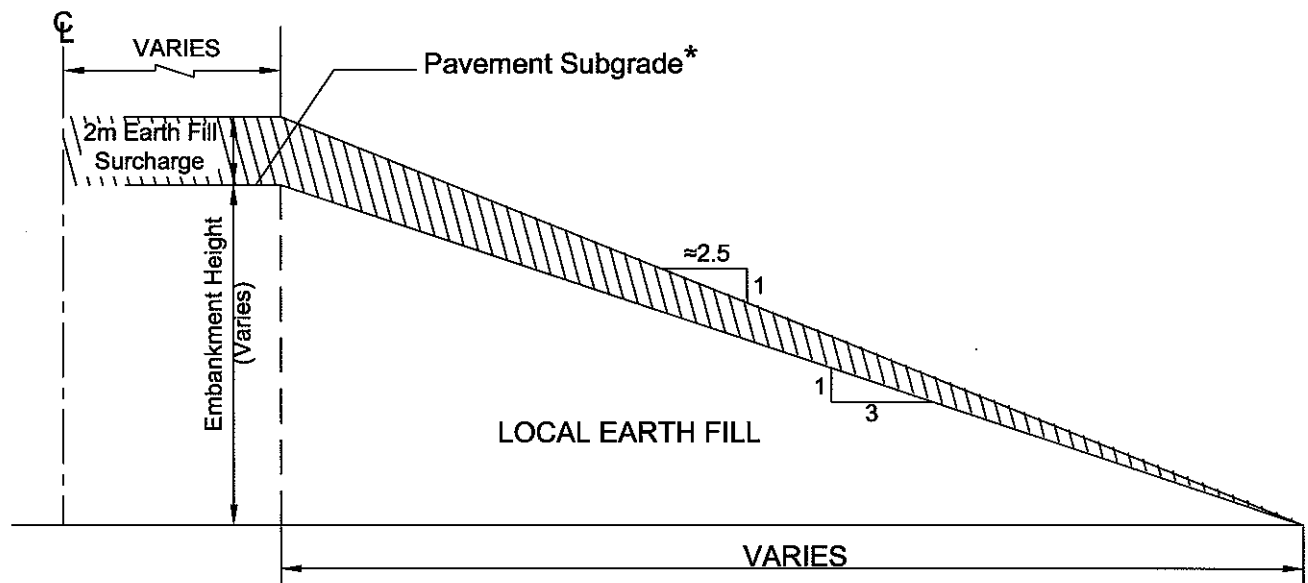
1. REMOVE TOPSOIL AND OR SOFT SUBSOIL UNDER AREA OF COMPACTED GRANULAR 'A' AND EARTH FILL.
2. PLACE GRANULAR 'A' AND EARTH FILL TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO O.P.S.S.501.
3. CONSTRUCT CONCRETE FOOTING.
4. PLACE REMAINDER OF GRANULAR 'A' AND EARTH FILL AS REQUIRED.
5. SOURCE M.T.C.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

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

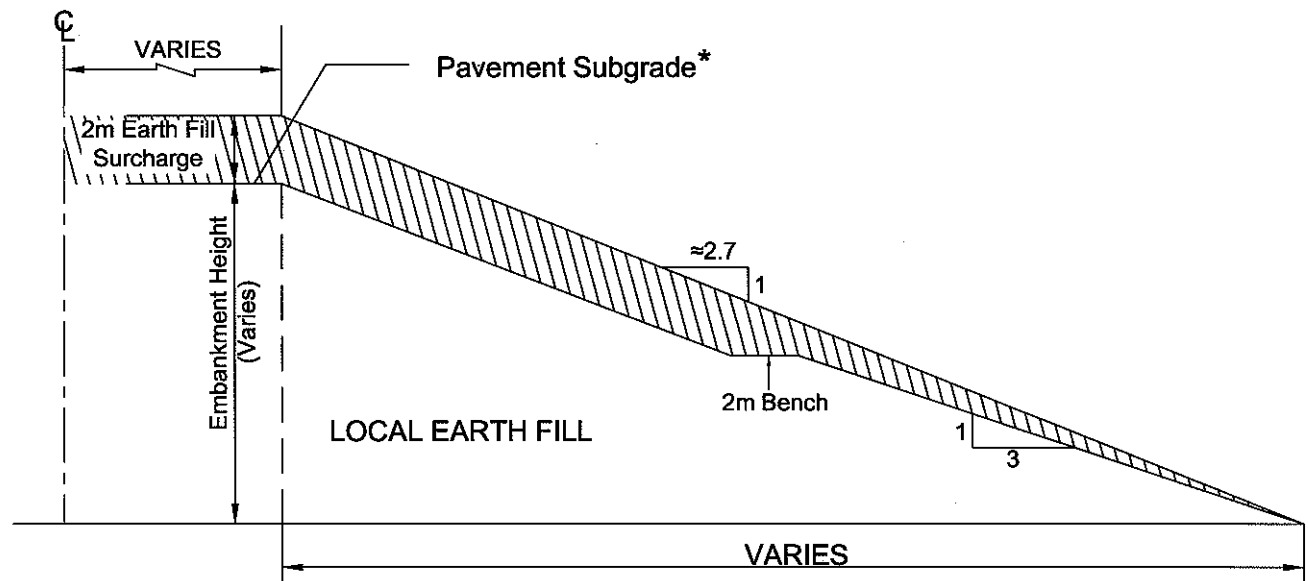
ABUTMENT ON COMPACTED FILL SHOWING GRANULAR A CORE

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FIGURE H1



Local Earth Fill Embankment < 8m

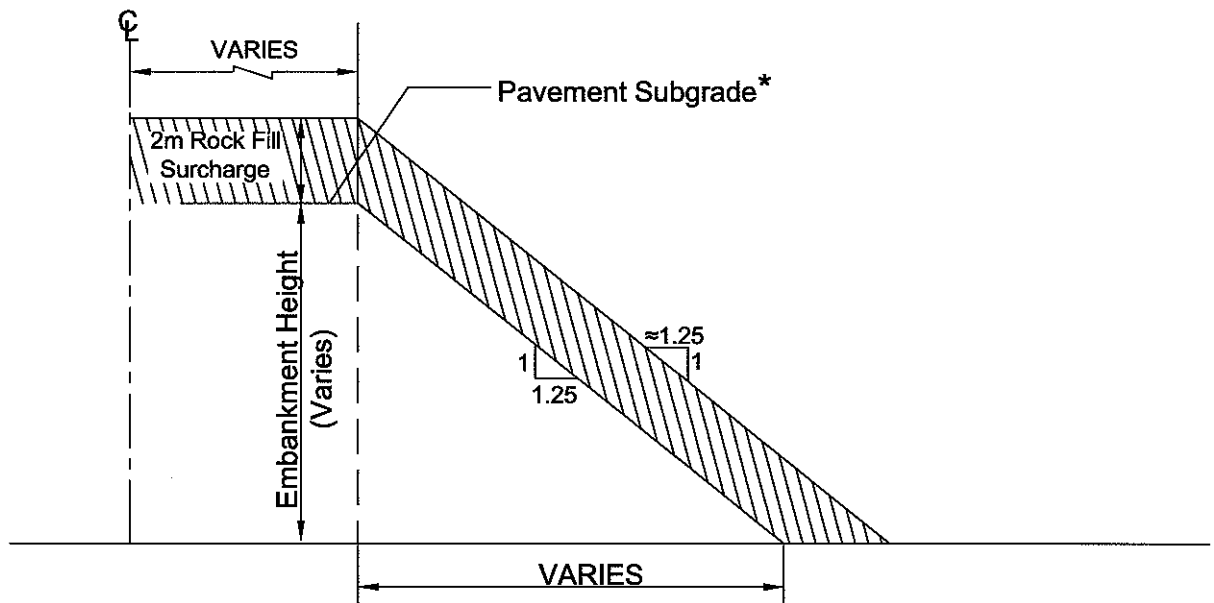


Local Earth Fill Embankment $8\text{m} \geq 12\text{m}$

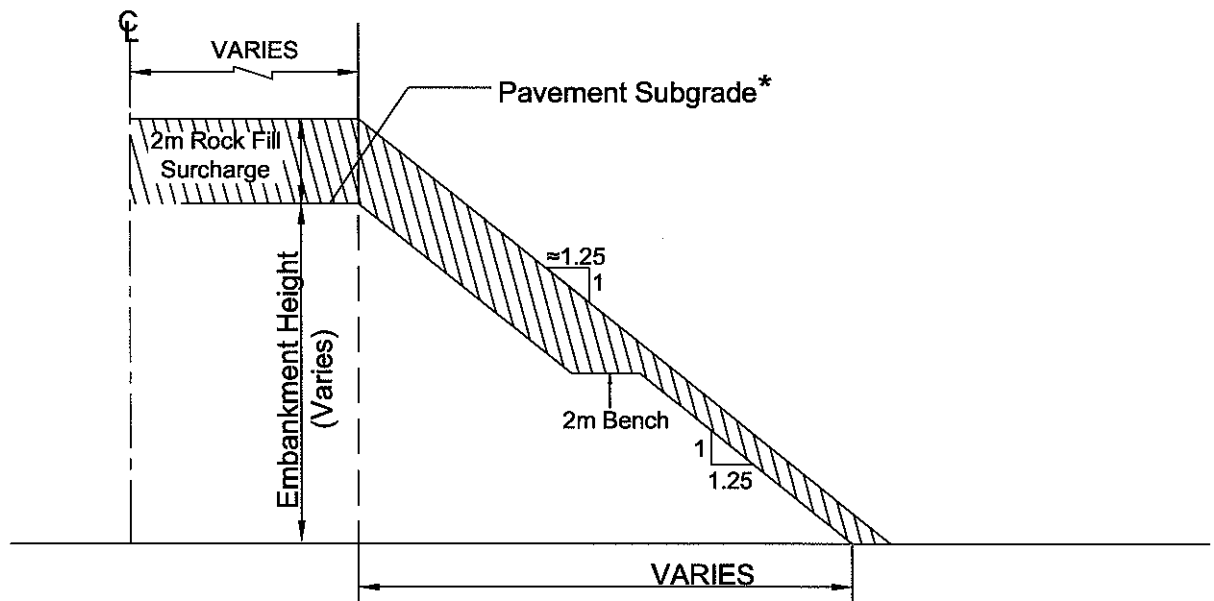
* Notes- Pavement subgrade to be established after removal of surcharge

N.T.S

SURCHARGE ARRANGEMENT



Rock Fill Embankment <10m

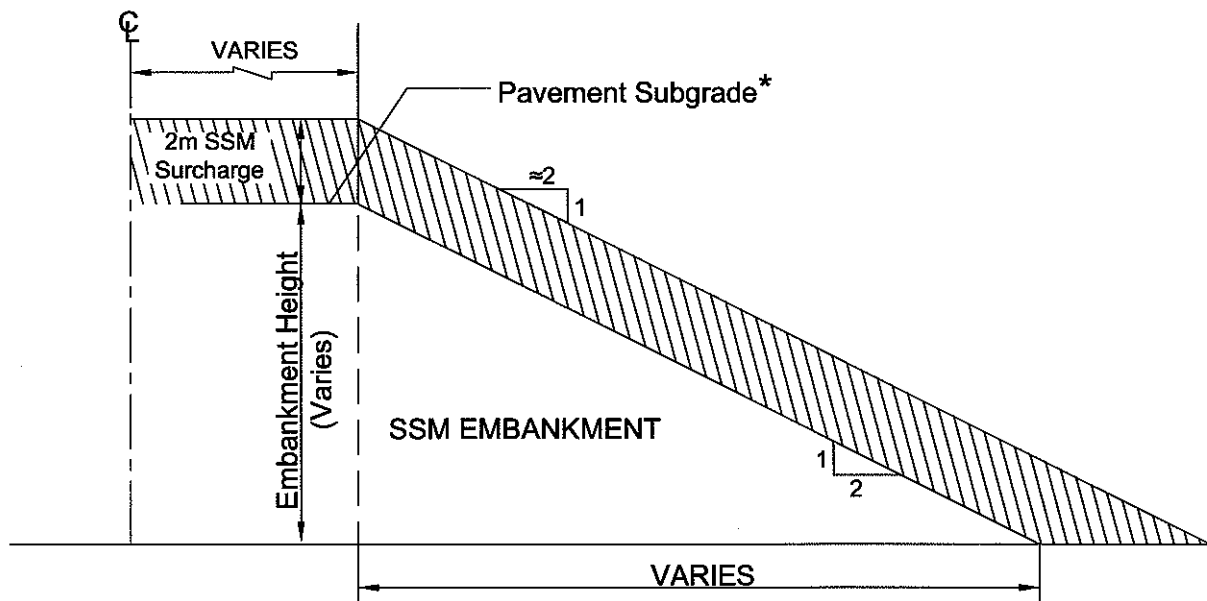


Rock Fill Embankment 10m \geq 12m

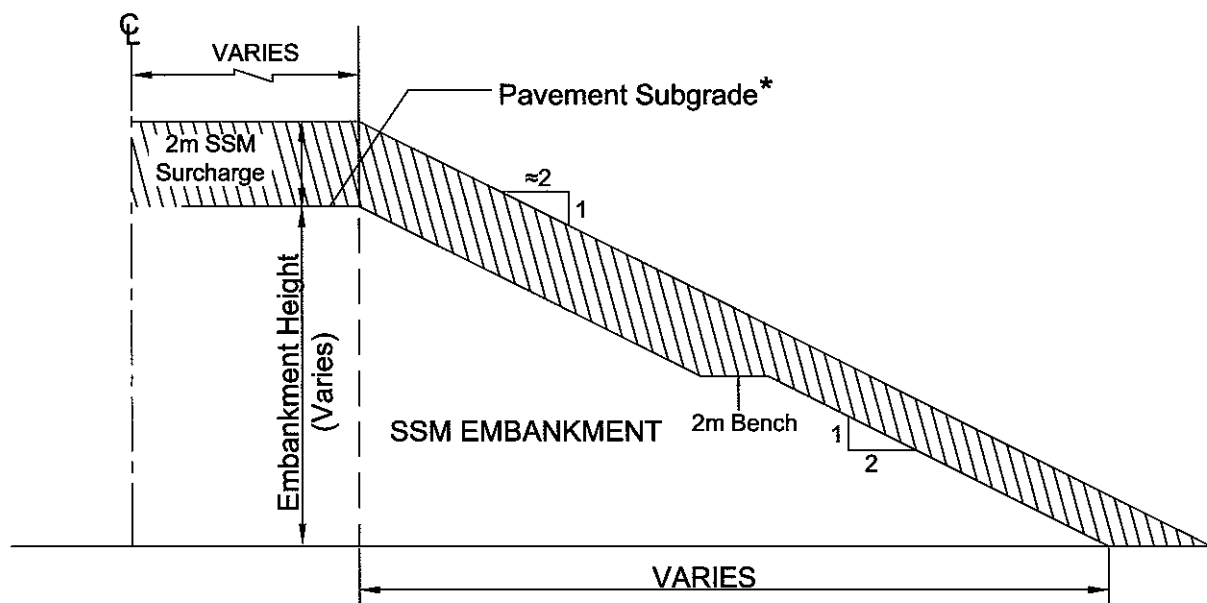
* Notes- Pavement subgrade to be established after removal of surcharge.

N.T.S

SURCHARGE ARRANGEMENT



SSM Embankment <8m

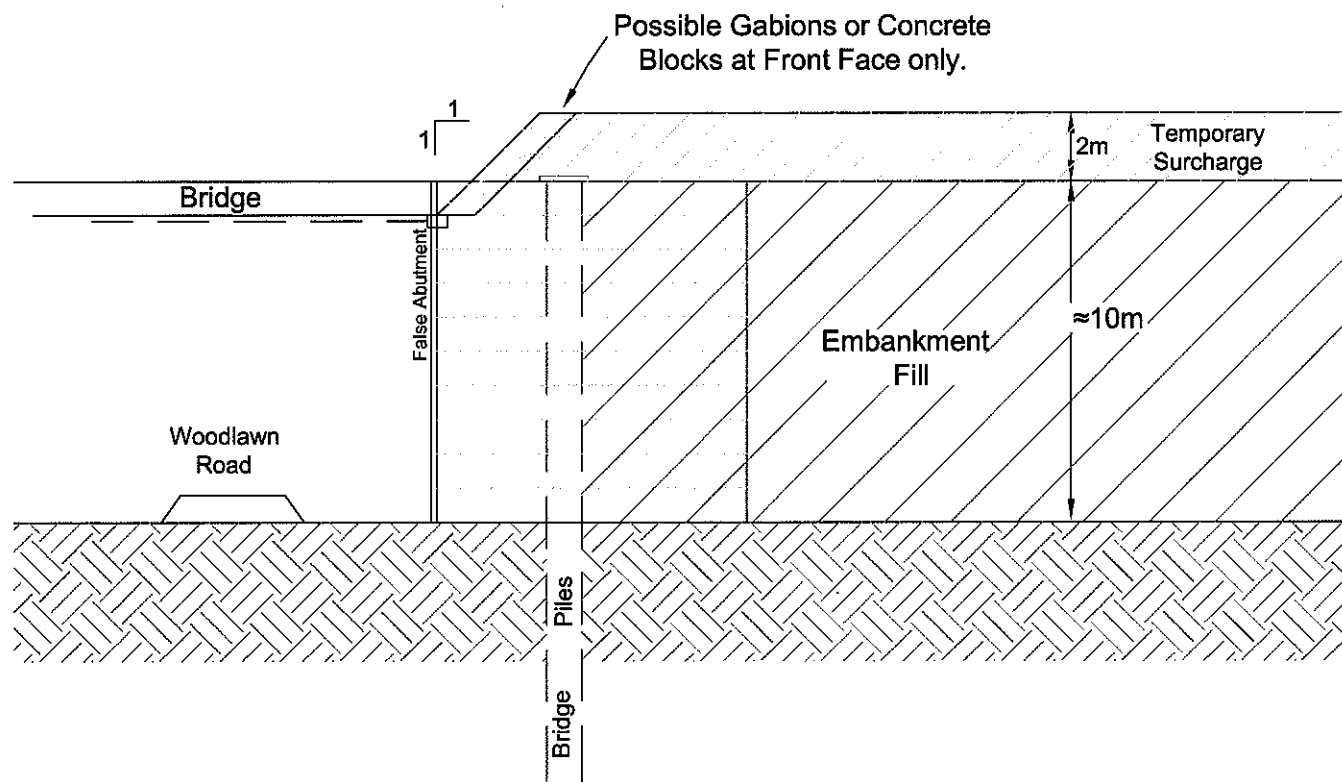


SSM Embankment 8m ≥12m

* 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



N.T.S

TEMPORARY RETAINING WALL ARRANGEMENT

TERRAPROBE

File No. 1-09-4135

FIGURE H5