



Terraprobe

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

**FOUNDATION INVESTIGATION & DESIGN REPORT
OLD WELLAND CANAL/WELLAND RIVER BRIDGE
HIGHWAY 406 TWINNING
PORT ROBINSION ROAD TO EAST MAIN STREET
AGREEMENT No. 2008-E-0016, W.P. 280-99-00, SITE: 34-304/1
GEOCRES No. 30M3-253**

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted at the Old Welland Canal/Welland River bridge site on the proposed two-lane Highway 406 NBL in the City of Thorold, Ontario. The Ministry of Transportation (MTO) conducted investigations at this site for the existing eleven span bridge to the west of this alignment, and the factual data from these investigations have been used as general reference for the preparation of this report.

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

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

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

- Ministry of Transportation, "Highway 406 Old Welland Canal Overpass and Causeway", W.P. 11-68-16, MTO District 4, GEOCRES 30M3-188, dated February 04, 1988.
- Ministry of Transportation, "Highway 406 Welland River and Welland Canal Bridge, Highway 406", W.P. 171-90-01, MTO District 4, GEOCRES 30M3-192, dated May, 1991.

2 SITE DESCRIPTION & PHYSIOGRAPHY

The site is located about 1.2 km south of Merritt Road and about 900 m north of Woodlawn Road in the City of Thorold, Regional Municipality of Niagara. The centre line of the proposed alignment is approximately 30 m east of the centre line of the present eleven span concrete bridge that currently carries both north and south bound traffic on Highway 406.

The alignment crosses the Welland River and the Old Welland Canal. The Welland River is located south of the Old Welland Canal and is separated from the canal by a narrow strip of parkland (Merritt Island). Merritt Island is man made and was created when the Welland Canal was constructed adjacent to the Welland River.



The topography is generally flat to undulating with scattered man-made high ground areas. Vegetation at this site consists primarily of deciduous trees and wild bush. Minor areas of groomed grass can be found on Merritt Island and the north shore of the Old Welland Canal. There is an asphalt paved roadway on Merritt Island and on the north shore of the Old Welland Canal both of which are oriented parallel to the existing bodies of water.

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 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 18 and December 08, 2009 and consisted of drilling and sampling sixteen boreholes to depths ranging from 12.0 m to 30.9 m. The boreholes were numbered W1 to W16 inclusive and their approximate locations are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix D.

The borehole locations were marked in the field by surveyors from Callon Dietz Inc. who also provided Terraprobe with their coordinates and geodetic elevations. Callon Dietz Inc. also completed topographic surveys of the Welland River and the Old Welland Canal beds. Utility clearances and permits were obtained by Terraprobe 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 and pier locations 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

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

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



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. This procedure was also used to seal Borehole W5 where artesian conditions were encountered.

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
W1	12.2/166.1	Piezometer with 1.5 m slotted screen installed with filter sand to 10.4 m, bentonite seal from 10.4 m to 9.8 m, drill cuttings from 9.8 m to 0.6 m and bentonite seal from 0.6 m to ground surface.
W2	10.7/167.7	Piezometer with 1.5 m slotted screen installed with filter sand to 8.8 m, bentonite seal from 8.8 m to 8.2 m, drill cuttings from 8.2 m to 0.6 m and bentonite seal from 0.6 m to ground surface.
W3	24.7/152.1	Hole sealed to 24.7 m with bentonite, piezometer with 1.5 m slotted screen installed with filter sand to 22.3 m and bentonite seal from 22.3 m to ground surface.
W6	21.3/154.3	Hole sealed to 21.3 m with bentonite, piezometer with 1.5 m slotted screen installed with filter sand to 19.2 m and bentonite seal from 19.2 m to ground surface.
W9	25.0/150.6	Piezometer with 1.5 m slotted screen installed with filter sand to 22.9 m and bentonite seal from 22.9 m to ground surface.
W14	22.7/152.2	Piezometer with 1.5 m slotted screen installed with filter sand to 19.3 m, bentonite seal from 19.3 m to ground surface.
W15	15.2/164.0	Piezometer with 1.5 m slotted screen installed with filter sand to 13.4 m, bentonite seal from 13.4 m to 13.1 m, drill cuttings from 13.1 m to 2.1 m and bentonite seal from 2.1 m to ground surface.
W16	12.2/170.7	Piezometer with 1.5 m slotted screen installed with filter sand to 10.4 m, bentonite seal from 10.4 m to 9.8 m, drill cuttings from 9.8 m to 0.6 m and bentonite seal from 0.6 m to ground surface.

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

4 LABORATORY TESTING

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



5 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets in Appendix A and the boreholes drilled by MTO in 1987 for the existing bridge (Appendix C). Details of the encountered soil and rock stratigraphy are presented in these appendices and on the “Borehole Locations and Soil Strata” drawings in Appendix D. 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 water, topsoil and about 21.3 m to 27.5 m of overburden soils consisting of fill material (sand and gravel, silty clay, cobbles and gravel) and native deposits of silty clay, silt, silty clay till, sandy silt to silt and sand till, sand and gravel and cobbles. These soils are underlain by bedrock of the Salina formation.

5.1 Topsoil

Topsoil ranging from 50 mm to 200 mm thick was encountered across the site. Topsoil thickness may vary between and beyond the boreholes.

5.2 Fill – Sand and Gravel

Sand and gravel fill containing trace to some silt and trace clay was encountered in Boreholes W3 and W13. The thickness of the fill is approximately 200 mm and this layer extends to Elev. 176.6 m and Elev. 174.4 m below ground surface. The moisture content (by weight) of a sample of this fill was 8%.

5.3 Fill – Silty Clay

Fill material consisting of silty clay, trace sand to sandy, trace to some gravel and occasional cobbles were encountered across this site at variable depths. On the south side of the Welland River the fill extends to depths ranging from 0.7 m to 0.9 m below ground surface or to elevations ranging from 177.7 m to 176.1 m. On Merritt Island the fill extends to depths ranging from 8.6 m (Elev. 167.3 m) to 10.8 m (Elev. 164.8 m) below ground surface. North of the Old Welland Canal this silty clay fill extends to depths ranging from 0.7 m to 7.0 m below ground surface i.e. elevations ranging from 175.9 m to 174.1 m.

The grain size distribution plots of tested samples of this fill are presented in Figures B1 and B2. These results show a grain size distribution consisting of 0-13% gravel, 0-17% sand, 43-73% silt and 25-54% clay size particles.

Samples were also subjected to Atterberg Limits tests and the results are presented in Figures B3 and B4. The index values from these tests are summarized below:

Liquid Limit:	29-50%
Plastic Limit:	17-25%
Plasticity Index:	12-25%
Natural Moisture Content:	9-27%



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

Standard Penetration tests in the silty clay fill gave 'N' values that ranged from 5 to 73 blows for 0.3 m penetration but generally 'N' values ranged from 5 to 28 blows for 0.3 m penetration. Based on these results the fill is considered to have a generally firm to very stiff consistency with occasional hard zones. The moisture content of samples of this fill ranged from 4% to 27% by weight.

5.4 Fill – Cobbles and Gravel

A 1.1 m thick layer of fill consisting of cobbles and gravel was encountered in Borehole W1 extending to a depth of 1.8 m (Elev. 176.5 m) below ground surface. SPT 'N' values ranged from 48 to 100 blows for less than 0.3 m penetration indicating a very dense relative density. The moisture content of samples of this fill varies from 18% to 20% by weight.

5.5 Silty Clay

A native silty clay deposit was encountered at this site. South of the Welland River the deposit extends to depths ranging from 2.9 m to 4.9 m below ground surface or to elevations ranging from 174.6 m to 173.5 m. In the Welland River this stratum extends to a depth of 7.3 m (Elev. 163.5 m) in Borehole W5. North of the Old Welland Canal the silty clay deposit extends to depths ranging from 0.7 m to 8.6 m below ground surface i.e. elevations ranging from 174.3 m to 172.7 m.

The grain size distribution plots of tested samples of the silty clay are presented in Figures B5 and B6. These results show a grain size distribution consisting of 0% gravel, 0-2% sand, 62-77% silt and 21-38% clay size particles.

Samples were also subjected to Atterberg Limits tests and the results are illustrated on the plasticity charts, Figures B7 and B8. The index values from these tests are summarized below:

Liquid Limit:	26-45%
Plastic Limit:	14-21%
Plasticity Index:	10-24%
Natural Moisture Content:	18-22%

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

Standard Penetration tests in this stratum gave 'N' values that ranged from 0 to 83 blows for 0.3 m penetration but generally 'N' values ranged from 6 to 26 blows for 0.3 m penetration. Based on these results the silty clay deposit is considered to have a generally firm to very stiff consistency with occasional hard zones. The moisture content of samples from this stratum ranged from 18% to 25% by weight.



5.6 Silt

A silt deposit was encountered at this site south of the Welland River and north of the Old Welland Canal. The deposit extends below ground surface to depths ranging from 4.4 m to 5.9 m (elevations ranging from 172.5 m to 172.4 m) south of the Welland River. North of the Old Welland Canal this deposit extends to depths ranging from 2.1 m to 10.3 m below ground surface i.e. elevations ranging from 172.6 m to 171.9 m.

The grain size distribution plots of tested samples of the silt are presented in Figure B9. These results show a grain size distribution consisting of 0% gravel, 0-2% sand, 89-96% silt and 4-10% clay size particles.

The deposit is considered to have a loose to very dense relative density based on SPT 'N' values that ranged from 8 to 52 blows for 0.3 m penetration. The moisture content of samples from this deposit ranged from 19% to 30% by weight.

5.7 Silty Clay Till

Discontinuous upper and lower layers of silty clay till were encountered across the site. Boreholes W1, W2, W15 and W16 were terminated in the upper silty clay deposit at depths ranging from 12.0 m to 19.7 m below ground surface i.e. elevations ranging from 166.4 m to 163.2 m. In the remaining boreholes the upper silty clay till stratum was fully explored and was found to extend to depths ranging from 12.2 m to 19.5 m below ground surface or to elevations ranging from 159.0 m to 156.1 m.

The grain size distribution plots of samples retrieved from the upper till deposit are presented in Figures B10 to B16 inclusive. These results show a grain size distribution consisting of 0-14% gravel, 0-26% sand, 42-83% silt and 12-57% clay size particles. Till soils will also contain random cobble and boulder inclusions.

Samples of the upper silty clay till were also subjected to Atterberg Limits tests and the results are presented in Figures B17 to B23 inclusive. The index values from these tests are summarized below:

Liquid Limit:	19-38%
Plastic Limit:	12-18%
Plasticity Index:	4-19%
Natural Moisture Content:	9-32%

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



Standard Penetration tests in this upper silty clay till yielded 'N' values ranging from 2 to more than 100 blows for 0.3 m penetration. Field vane tests gave in-situ undrained shear strengths ranging from 32 kPa to in excess of 100 kPa and laboratory vane tests on relatively undisturbed Shelby tube samples gave undrained shear strengths ranging from 25 kPa to in excess of 100 kPa. These values indicate that the consistency of the silty clay is generally firm to very stiff with occasional hard zones. Moisture content of samples of the upper silty clay till range from 9% to 35% by weight and the unit weight of selected samples ranged from 19.6 to 22.5 kN/m³

The variation of undrained shear strength with elevation is depicted in Figure B28. No apparent relationship is evident between these two variables. The Atterberg Limits tests results are also plotted against elevation, Figure B29. These results illustrate that the natural moisture content is generally between the plastic and liquid limits except for a zone from about Elev. 163 m to Elev. 160 m where the natural moisture content is at or close to the plastic limit.

Consolidation tests of the upper silty clay till were also performed on Shelby tube samples retrieved from Boreholes W2 and W15 and the results are presented in Figures B30 to B35. These results indicate an estimated preconsolidation pressure that ranges between 250 kPa and 375 kPa.

The lower silty clay till extends to depths ranging from 18.4 m to 25.3 m below ground surface or to elevations ranging from 153.5 m to 150.5 m.

Grain size distribution plots of tested samples of the lower silty clay till are illustrated in Figure B24. These results show a grain size distribution consisting of 0-3% gravel, 4-30% sand, 36-50% silt and 23-58% clay size particles. Random cobble and boulder inclusions can also be expected in till soils.

Samples of the lower silty clay till were also subjected to Atterberg Limits tests and the results are presented in Figures B25. The index values from these tests are summarized below:

Liquid Limit:	16-35%
Plastic Limit:	12-18%
Plasticity Index:	4-17%
Natural Moisture Content:	9-26%

These values indicate that the till is generally a low plasticity silty clay with occasional clayey silt zones.

Standard Penetration tests in the lower silty clay till yielded 'N' values ranging from 8 to more than 100 blows per 0.3 m penetration and field vane tests attempted in this deposit gave undrained shear strengths more than 100 kPa. Based on these results the lower silty clay till is considered to have a stiff to hard consistency. The moisture content of samples obtained this lower till deposit ranged from 9% to 34% by weight.



5.8 Sandy Silt to Silt and Sand Till

The site is underlain by upper and lower granular till deposits with a soil matrix that ranges from sandy silt to silt and sand. These deposits extend to depths ranging from 18.4 m to 26.5 m or to elevations ranging from 155.0 m to 150.0 m.

The results of grain size distribution tests conducted on samples obtained from the upper and lower deposits are illustrated in Figures B26 and B27. These results show grain size distributions consisting of 1-13% gravel, 28-46% sand, 37-62% silt and 5-11% clay size particles.

Standard Penetration tests in these deposits gave 'N' values that generally ranged from 11 to more than 100 blows per 0.3 m penetration indicating a compact to very dense relative density. Recorded 'N' values of 3 blows per 0.3 m penetration were obtained in boreholes W6, W7 and W8 in the upper portion of the stratum indicating a very loose relative density. The moisture content of samples from these strata ranged from 7% to 21% by weight.

5.9 Sand and Gravel

Layers of sand and gravel containing occasional cobble inclusions were encountered in Borehole W5 and W14. The deposits are approximately 1.1 m to 1.4 m thick and extend to depths ranging from 21.9 m (Elev. 152.9 m) to 21.3 m (Elev. 149.5 m) below ground surface.

Standard Penetration tests in these deposits gave 'N' values that ranged from 71 to more than 100 blows for 0.3 m penetration. Based on these results the deposits are considered to have a very dense relative density. The moisture content of samples from these strata ranged from 7% to 15% by weight.

5.10 Cobbles

A deposit of cobbles containing some gravel overlies the bedrock in some of the boreholes. Diamond drilling techniques had to be implemented in order to penetrate this deposit at some locations. The deposit ranges in thickness from 0.4 m to 1.5 m and extends to depths ranging from 21.4 m to 27.5 m below ground surface or to elevations ranging from 152.0 m to 150.0 m.

Based on recorded 'N' values of more than 100 blows for 0.3 m penetration, the deposit is considered to have a very dense relative density.



5.11 Bedrock (Salina Formation)

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

Table 5.1 – Depth to Bedrock

Location	BH Number	Depth to Bedrock (m)	Top of Bedrock Elevation (m)
South Abutment	W3	26.8	150.0
	W4	27.5	150.1
Pier 2	W5	21.3	149.5
Pier 4	W6	25.1	150.5
	W7	25.2	150.0
Pier 5	W8	25.3	150.6
	W9	25.2	150.4
Pier 7	W10	22.0	151.3
Pier 8	W11	21.4	152.0
Pier 9	W12	21.4	152.0
North Abutment	W13	21.4	153.2
	W14	21.9	152.9

The bedrock is described as slightly weathered and its colour is light grey to dark grey. It is thin to medium bedded with light grey to dark grey moderately weathered shale interbeds. Total core recovery in the bedrock generally ranged from 42% to 100% and a recorded TCR of 0% was obtained in the first run of Borehole W7.

The RQD values ranged widely from 0% to 79% but generally, most of the RQD values were below 50%. The core data also reveals that there is no trend of improving rock quality with depth. Based on these results the rock quality is considered to be very poor to poor with occasional zones of fair to good quality rock.



5.12 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)
W1	November 30, 2009	3.0	175.3
	December 08, 2009	2.7	175.6
	December 16, 2009	2.6	175.7
	January 04, 2010	2.5	175.8
	January 11, 2010	2.5	175.8
W2	November 30, 2009	6.3	172.1
	December 08, 2009	4.4	174.0
	December 16, 2009	4.7	173.7
	January 14, 2010	4.2	174.2
W3	December 08, 2009	4.3	172.5
	December 16, 2009	4.2	172.6
	January 04, 2010	4.2	172.6
W5*	December 08, 2009	0.9 (high)	Above 171.7
W6	November 30, 2009	2.4	173.2
	December 08, 2009	2.7	172.9
	December 16, 2009	2.4	173.2
	January 04, 2010	2.4	173.2
W9	December 08, 2009	2.5	173.1
	December 16, 2009	2.3	173.3
	January 11, 2010	2.4	173.2
	January 14, 2010	2.4	173.2
W14	December 08, 2009	1.5	173.3
	December 16, 2009	1.4	173.4
	January 11, 2010	1.5	173.3
	January 14, 2010	1.5	173.3
W15	November 30, 2009	4.1	175.1
	December 08, 2009	4.2	175.0
	December 16, 2009	4.2	175.0
	January 04, 2010	4.2	175.0
W16	November 30, 2009	7.8	175.1
	December 08, 2009	7.7	175.2
	January 04, 2010	7.5	175.4
	January 14, 2010	7.5	175.4

* Artesian Condition

Based on these observations, the local ground water level generally follows the contours of the land existing at approximately Elev. 175.8 m south of the Welland River falling to about Elev. 172.6 m near the south bank of the river. North of the Old Welland Canal the water table exists at about Elev. 175.4 m falling gradually to about Elev. 173.3 at the north bank of the canal. On Merritt Island the water table is estimated to be at about Elev. 173.2 m.

In the Welland River artesian conditions were encountered in Borehole W5 in the lower granular deposits overlying bedrock. The head of water is estimated to be about 0.9 m (Elev. 171.7 m) above the free water level in the river. Below Merritt Island excess hydrostatic pressure was encountered in the underlying sandy silt to silty sand till confined between the less permeable upper and lower silty clay deposits.



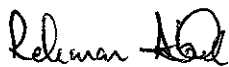
All ground water observations at this site are short term and the levels are expected to fluctuate seasonally and after severe weather events. The ground water level will also be controlled by the water levels in the Welland River and Old Welland Canal.

5.13 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 drilling techniques and casing and washboring methods were used to advance the boreholes. Coring also had to be resorted to in Boreholes W4 and W12 in order to penetrate frequent cobbles overlying the top of bedrock.

Messrs. Lucas Yu, E.I.T, Marc Paoliello, E.I.T, and Phil Khuu, B.A.T, observed and recorded the field work and the laboratory testing was performed at Terraprobe's Brampton laboratory. The report was written by Rehman Abdul, P.Eng. and reviewed by Michael Tanos, P.Eng.



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Senior Geotechnical Engineer



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



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AGREEMENT No. 2008-E-0016, W.P. 280-99-00, SITE: 34-304/1
GEOCRES No. 30M3-253

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 NBL will cross over the Old Welland Canal and the Welland River via an eleven span structure approximately 13 m wide and measuring about ± 295 m in length between abutments. The bridge will have a radius of curvature of 904.75 m, a 5.3% super elevated deck and the foundation elements will be skewed at ± 21.6 degrees to the proposed alignment centre line. The proposed finished grades at the structure will be about Elev. $181 \pm$ m at the south abutment rising gradually to Elev. $182 \pm$ m at Sta. 13+746 than gradually decreasing to Elev. $181 \pm$ m at the north bridge abutment.

Three piers (Pier 1, Pier 2 and Pier 3) will be built in the Welland River and five piers (Pier 6 to Pier 10 inclusive) will be constructed in the Old Welland Canal. Pier 4 and Pier 5 will be constructed on Merritt Island.

At the south abutment the approach fill will be about $8 \pm$ m high decreasing to about $3 \pm$ m at the south limit of this approach (Sta. 13+535). The approach fill at the north abutment will be about $8 \pm$ m high transitioning into a cut section further north. The cut section commences at about Sta. 13+900 and a $2.5 \pm$ m high cut will be required at the north limit of this approach (Sta. 13+910) to achieve the desired approach grade.

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 an eleven span structure with two abutments and ten piers as foundation elements.

The stratigraphy encountered at the abutment and pier locations consist of 21.3 m to 27.5 m of overburden soils overlying bedrock of the Salina Formation. The ground water level is about Elev. 175.8 m south of the Welland River falling to about Elev. 172.6 m near the south bank of the river. North of the Old Welland Canal the water table exists at about Elev. 175.4 m falling gradually to about Elev. 173.3 at the north bank of the canal. On Merrit Island the water table is estimated to be at about Elev. 173.2 m.

In the Welland River artesian conditions were encountered in Borehole W5 in the lower granular deposits overlying bedrock. The head of water is estimated to be about 0.9 m (Elev. 171.7 m) above the free water level in the river. Below Merritt Island excess hydrostatic pressure was encountered in the underlying sandy silt to silty sand till confined between the less permeable upper and lower silty clay deposits.

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 E.

7.1 Spread Footings

The proposed founding elevations of the abutments are Elev. ± 173 m and the soils at this elevation generally consist of compact to dense silt and firm silty clay underlain by firm to stiff silty clay till. Silt soils are not considered to be a suitable founding stratum for supporting foundations. The geotechnical resistance of the underlying silty clay till is also low thereby requiring relatively large footings. Furthermore, the abutment stems will be about 9 m high and the applied loads will trigger time dependent consolidation settlements. Consequently, spread footings on native ground are not considered to be a feasible foundation alternative and are not recommended.

Consideration was also given to placing spread footings on an engineered fill pad but the geotechnical resistance of the underlying soils are low and the applied loads will cause time dependent consolidation settlements. Therefore, this option is not recommended.

7.2 Augered Caissons (Drilled Shafts)

Augered caisson foundations were also considered for supporting the structure. However, the overburden is not considered to be suitable for this scheme and the caissons must be founded on the bedrock at depths in the order of ± 22 to ± 28 m below original ground surface. Highly fractured



zones exist within the bedrock and there is no evidence of increasing rock quality within the investigated depths. Rock sockets made in this fractured bedrock zone may not provide reliable performance.

The base of the caissons would be about ± 20 to ± 23 m below the ground water level, resulting in high hydrostatic heads at the base and in the Welland River artesian conditions will be encountered in the underlying permeable units. It would be difficult to seal the bottom of the liner to exclude ground water due to the permeable nature of the overburden soils, the presence of cobbles (and possibly boulders) above the bedrock and the highly fractured zones within the bedrock. Unwatering the caisson and maintaining a sufficiently dry excavation to permit cleaning, inspection and high quality construction within the bedrock would also be challenging and impractical.

Furthermore, the existing bridge at this site was constructed on pile foundations that have performed reliably. Therefore, a similar foundation scheme will have a high probability of providing reliable performance and the risk will be low.

Given the high construction effort required, the poor quality of the rock and the higher risk, caisson foundations are not recommended for supporting the structure.

7.3 Driven Piles

The subsurface conditions at the site are considered suitable for the design of foundations supported on steel H-piles. Furthermore, the existing bridge is supported on pile foundations that have provided reliable performance.

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

Steel H-piles are likely to be driven to practical refusal in/on bedrock at all foundation elements. However, there is a dense layer of cobbles that overlies bedrock at some locations and piles may encounter effective refusal in this stratum without reaching bedrock.

7.3.1 Axial Resistance

Two steel H-pile sections viz. HP 310 x 110 and HP 360 x 132 were considered. The recommended factored, vertical, concentric, geotechnical resistances of the HP 310 x 110 and HP 360 x 132 sections at ULS when driven to bedrock are 1600 kN and 1800 kN respectively. These reduced ULS resistances are recommended because of the very poor to poor rock quality. The bedrock is considered to be “unyielding” and the SLS condition will not govern for piles founded on bedrock. The approximate tip elevations of these pile sections are presented in Table 7.1.



Table 7.1 – Tip Elevations of Various Pile Sections Driven to Bedrock

Reference Borehole	Support Location	Estimated Pile Tip Elevation (m)
Pile Types – HP 310x110 & HP 360x132		
! W3	South Abutment	150.0±
! W4		150.1±
N/A	Pier 1	*149.8±
W5	Pier 2	149.5±
N/A	Pier 3	*149.8±
W6	Pier 4	150.5±
W7		150.0±
W8	Pier 5	150.6±
W9		150.4±
N/A	Pier 6	*150.9±
! W10	Pier 7	151.3±
! W11	Pier 8	152.0±
! W12	Pier 9	152.0±
N/A	Pier 10	*152.5±
W13	North Abutment	153.2±
W14		152.9±

* Tip elevation interpolated based on top of bedrock elevation of adjacent boreholes.

! Layer of cobbles encountered above bedrock.

The structural resistance of the pile should be checked by the structural designer. The recommended geotechnical resistances are based on the assumption that the piles will be driven into soils that provide full lateral support against buckling. Where piles are driven and left partially exposed or immersed in water the structural designer may need to modify these resistances to take into account the unsupported length.

The H-piles for the recommended foundation scheme will be driven to bedrock. Piles will be required to penetrate relatively thin layers of cobbles and till layers (that may contain boulders). It is therefore recommended that the pile tips of H-pile sections be fitted with rock points to provide reinforcement to the pile section and effective contact with bedrock.

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

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



7.3.2 Downdrag

The construction of approximately ± 8 m high embankments at the bridge abutments will cause settlement of the underlying soils. Downdrag forces will be imparted on piles if they are installed before the embankments are constructed. Downdrag forces will also cause additional bending forces to be imposed on batter piles. The downdrag forces imparted on piles at the abutment locations were estimated based on a compressible silty clay stratum that extends to Elev. 158.5 m and unfactored loads of 700 kN/pile (HP 310 x 110 section) and 825 kN/pile (HP 360 x 132 section) were obtained. However, we do not recommend designing for downdrag forces.

We recommend that the approach fills be constructed approximately 1 year in advance of the pile driving operations and monitored to ensure that settlement is essentially complete prior to pile driving operations. By adopting this construction approach the downdrag forces will be significantly reduced and the potential for lateral squeeze (due to unbalanced fill) will be minimized.

7.3.3 Integral Abutment Considerations

In Ontario integral abutment bridges are usually suitable where the total bridge span is less than 100 m and the skew is less than 20 degrees. This bridge is not a candidate for an integral abutment design since it is 295 m long and the skew is 21.6 degrees.

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 7.2) (kPa)

n_h = coefficient of horizontal subgrade reaction (Table 7.2) (kN/m³)

γ = unit weight (Table 7.2) (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 7.2 – 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 W3	176.8 – 176.6	Fill – Sand and Gravel	19	30	–	2000
	176.6 – 176.1	Fill – Silty Clay	18	0	50	-
	176.1 – 173.9	Silty Clay	19	0	100	-
	173.9 – 172.4	Silt	18.5	33	–	4400
	172.4 – 159.0	Silty Clay Till	21	0	80	-
	159.0 – 154.5	Sand and Silt Till	20	35	–	10000
	154.5 – 152.9	Silty Clay Till	21	0	225	-
	152.9 – 150.9	Silt and Sand Till	20	35	–	10000
South Abutment W4	150.9 – 150.0	Cobbles	19	33	–	9000
	177.5 – 176.7	Fill – Silty Clay	18	0	50	-
	176.7 – 173.9	Silty Clay	19	0	100	-
	173.9 – 172.4	Silt	18.5	33	–	4400
	172.4 – 158.3	Silty Clay Till	21	0	80	-
	158.3 – 155.3	Sand and Silt Till	20	35	–	8500
	155.3 – 152.3	Silty Clay Till	21	0	225	-
	152.3 – 151.1	Silt and Sand Till	20	35	–	10000
North Abutment W13	151.1 – 150.1	Cobbles	19	33	–	9000
	174.6 – 174.4	Fill – Sand and Gravel	19	30	–	2000
	174.4 – 173.9	Silty Clay	19	0	75	-
	173.9 – 172.5	Silt	19	33	–	4400
	172.5 – 158.4	Silty Clay Till	21	0	80	-
North Abutment W14	158.4 – 153.2	Sand and Silt Till	20	35	–	8500
	174.8 – 174.1	Fill – Silty Clay	18	0	75	-
	174.1 – 172.7	Silty Clay	19	0	50	-
	172.7 – 171.9	Silt	19	28	–	1300
	171.9 – 158.6	Silty Clay Till	21	0	80	-
	158.6 – 154.0	Sand and Silt Till	20	35	–	8500
	154.0 – 152.9	Sand and Gravel	20	33	–	9000

* 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 cobbles and boulders overlying bedrock and till layers that may contain cobbles and boulders, the tips of all 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 the zone of cobbles and boulders to achieve refusal and effective contact with the bedrock.

7.3.6 Pile Installation

Pile installation should be in accordance with OPSS 903, November 2009. The recommended maximum inclination of battered piles should not be greater than 3V:1H.

The Contract Documents should contain a NSSP alerting the Contractor to the presence of cobbles and boulders in the till soils as well as the layer of cobbles encountered above the bedrock in Boreholes W3, W4, W10, W11 and W12.

7.3.7 Pile Driving

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

Hammers used for pile installation must be capable of installing the piles to the depths specified in the contract documents. Since the piles will be seated on bedrock the hammer used must also be capable of delivering a controlled blow in 10% increments ranging in energy from zero to the maximum hammer energy. A typical hammer energy of 60 kilojoules/blow, with an energy transfer (efficiency) of 40% is recommended.



7.4 Recommended Foundation

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 TEMPORARY SHORING

Piers in the Welland River and Old Welland Canal will have to be constructed in dry conditions and a cofferdam will be required. Shoring operations may also be required on Merritt Island where construction operations are relatively close to the banks.

The shape of the soil pressure distribution diagram behind a shoring system depends upon the type of soil to be encountered and the amount of movement that can be permitted. The shoring system can be restrained, fixed or flexible. The sequence of work may also alter the shape of the pressure diagram during the various construction phases.

Earth pressure computations must also take into account the ground water level. Above the ground water level, earth pressure is computed using the bulk unit weight of the retained soil. Below the ground water level, the earth pressures are computed using the submerged unit weight of the soil. A hydrostatic pressure is also applied if the retained soil is not fully drained.

Flexible shoring should be designed on the basis of the active earth pressure coefficient (K_a). Where limited shoring movement (less than performance Level 1) is required the design should be based on the at rest earth pressure coefficient (K_o). For “kick out” design the lateral resistance should be computed on the basis of the passive earth pressure coefficient (K_p).

Decisions regarding shoring methods and sequencing are the responsibility of the Contractor. Shoring should be designed by a licensed Professional Engineer experienced in shoring design. Temporary shoring can be designed for a Performance Level 2, 25 mm maximum horizontal displacement.



The recommended unfactored values of the parameters for use in the design of structures subject to unbalanced earth pressures are given in Table 8.1.

Table 8.1 - Earth Pressure Coefficients

Soil	ϕ (deg)	γ (kN/m ³)	K_a	K_o	K_p
Fill – Silty Clay	29	18	0.35	0.52	2.90
Silty Clay	29	19	0.35	0.52	2.90
Silty Clay Till	27	21	0.38	0.55	2.70
Silt and Sand Till	33	20	0.29	0.45	3.40
Silty Clay Till	27	21	0.38	0.55	2.70
Silt and Sand Till and Sandy Silt Till	33	20	0.29	0.45	3.40
Sand and Gravel and Cobbles and Gravel	30	19	0.33	0.50	3.00

Braced cuts in clay may become unstable as a result of heaving of the bottom of the excavation. The base of the cofferdam at the pier locations were analysed for basal stability using the method proposed by Terzaghi (1943). A factor of safety of 1.5 is usually desirable.

The analysis was based on a 10 m to 12 m wide cofferdam founded in the silty clay till below the Welland River and Old Welland Canal. For these dimensions, the derived factors of safety were greater than 1.5. Basal stability must be reviewed and checked by a licensed Professional Engineer when the contractor finalizes the cofferdam dimensions. Temporary shoring should be in accordance with OPSS 539, November 2009.

9 EXCAVATION AND BACKFILL

9.1 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 3 soils above the water table and Type 4 soils below the water table. Excavation below the ground water level is not recommended without prior dewatering. Provided dewatering is carried out as described below, excavations may be sloped at 2H:1V

9.2 Foundations

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

10 GROUND WATER CONTROL

The local ground water level exists at about 175.8 m south of the Welland River falling to about Elev. 172.6 m near the south bank of the river. North of the Old Welland Canal the water table exists at about Elev. 175.4 m falling gradually to about Elev. 173.3 at the north bank of the canal. On Merrit Island the water table is estimated to be at about Elev. 173.2 m.



Artesian conditions exist in the lower granular deposits encountered below the Welland River. Below Merritt Island excess hydrostatic pressure was encountered in the underlying sandy silt to silty sand till confined between the less permeable upper and lower silty clay deposits. The contract documents should include an NSSP alerting the Contractor to the presence of artesian conditions. Suggested NSSP wording is provided in Appendix F.

The pile driving operations will cause significant remoulding of the clay soils around the pile shafts thereby forming a watertight barrier that will prevent the upward movement of ground water at the soil/pile interface. Therefore, an inverted granular filter below the pile caps is not required.

Excavations will extend below the ground water level and at the abutment locations a wet silt layer will be exposed. The silt will be easily disturbed by construction activity and will also yield water due to its relatively high permeability.

To alleviate construction related problems at the abutments we recommend that the silt layer be removed entirely (by excavating to about Elev. 172 m) to expose the underlying more impermeable silty clay soils. The excavation can be unwatered by installing a perimeter trench designed to drain to filtered sumps from which pumping can be undertaken. The design of the unwatering system should be the responsibility of the Contractor.

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

Cofferdams placed in the Welland River and Old Welland Canal will also have to be unwatered and relatively dry excavations will have to be maintained during construction. The excavations will be made in relatively impermeable silty clay soils that are not anticipated to yield significant volumes of water. However, the initial volumes will be large and the Contractor must ensure that suitably sized equipment is used. Adequately sized emergency “stand-by” equipment should also be considered in case of sudden failure of the primary equipment.

11 APPROACH EMBANKMENTS

11.1 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 necessitating remedial works 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.

Since there is no requirement to limit the magnitude of settlement of the soils underlying the approach embankments there is no advantage in using lightweight fill and this option was not considered.

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 1.3 was established. The soil parameters used for the slope stability analyses are presented in Table 11.1 and the minimum factors of safety obtained for the various embankment options are included in Table 11.2.

Table 11.1 – 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
Silty Clay	0	100-150	20.0	27	5	20.0
Silt	30	0	19.0-19.5	30	0	19.0-19.5
Silty Clay Till	0	75	20.0	27	5	20.0

Table 11.2 – 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 – 2.2	2.0 – 2.1
Rock Fill	1.25H:1V	1.4 – 1.8	1.4 – 1.7
SSM	2H:1V	1.6 – 1.9	1.6 – 1.8

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 1.4 or greater for the recommended design side slopes.

Where earth fill or SSM 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.

11.2 Stability of Forward Slopes

The forward slopes at the abutments were also assessed for stability based on normal operating conditions and rapid drawdown conditions during storm events. Static H.W.L's of Elev. 174.04 m (Welland River) and Elev. 173.7 m (Old Welland Canal) were used for the rapid drawdown analyses. A target Factor of Safety of 1.5 was established and the slope stability analyses yielded the following factors of safety:

- Local Earth Fill (3H:1V) – Factors of safety ranging from 1.8 to 1.9.
- Rock Fill (1.25H:1V) – Factors of safety ranging from 1.5 to 1.6.
- SSM (2H:1V) – Factors of safety ranging from 1.5 to 1.6.

11.3 Settlement

The underlying silty clay soils at this site will experience time dependent consolidation settlement due to the additional stress imposed by the embankments. Consolidation tests were conducted on two samples of the silty clay till. The preconsolidation pressure (P_c) ranges from about 250 kPa to 375 kPa and coefficient of consolidation (c_v) values ranged from 0.052 to 0.069 m²/day.

Consolidation settlement calculations of the underlying silty clay soils were undertaken for earth fill, rock fill and SSM embankments. The commercially available program Settle 3D developed by Rocscience Inc. was used for the settlement analyses. The estimated range of total settlements (embankment centreline) at the north and south approaches are provided in the following table.

Table 11.3 – Approach Embankments Estimated Consolidation Settlements

Type of Fill	Unit Weight of Fill (kN/m ³)	Side Slope Geometry	Settlement (mm)	
			North Approach	South Approach
Local Earth Fill	19.0	3H:1V	95 – 115	85 – 100
Rock Fill	19.0	1.25H:1V	95 – 115	85 – 100
Select Subgrade Material	20.0	2H:1V	100 – 120	90 – 115

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. 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. The coefficient of consolidation was estimated to range from



0.052 to 0.069 m²/day. Tabulated below is the range of predicted settlements (embankment centreline) at various time periods.

Embankment Type	Settlement At Various Time Periods (mm)				Total Settlement (mm)
	6 months	12 months	18 months	24 months	
Local Earth Fill	70 – 95	75 – 105	80 – 110	85 – 115	85 – 115
Rock Fill	70 – 95	75 – 105	80 – 110	85 – 115	85 – 115
Select Subgrade Material	70 – 100	80 – 110	85 – 115	85 – 120	90 – 120

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 must be conducted to confirm the anticipated settlement performance.

It is recommended that the approach fill be constructed 1 year in advance of the pile driving operations and settlement monitoring should be undertaken to ensure that the majority of the settlement is complete prior to constructing the abutments. The advantage of this construction approach (i.e. fill placement and settlement monitoring) is the certainty of negligible downdrag loads being imparted on the piles as well as reduced potential for lateral squeeze. A recommended settlement monitoring programme is provided in Appendix G.

However, if an accelerated construction schedule is required (and given the uncertainty in accurately predicting the time rate of settlement), we recommend that conventional temporary surcharging be carried out (2 m of additional fill height) to accelerate the settlement and ensure full consolidation within the target 6 months after embankment construction (Figures H1 to H3). 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. The temporary retaining system can consist of either gabion baskets or stacked concrete blocks at the toe of the fill as depicted in Figure H4.

11.4 Temporary Cuts

Approximately 5.5 m deep excavations will be required at the abutment locations in order to construct pile caps at a design elevation of Elev. 173± m. Global stability analyses were conducted on temporary cut slopes at both abutment locations and factors of safety against global failure of 1.4 and greater were obtained for temporary cut slopes inclined at 1 H:1 V in silty clay soils. The excavations will expose a dense silt layer at both approaches and temporary excavations made in this material should be at an inclination of 2H:1V or flatter.



The following requirements must be met:

- No traffic, construction equipment, stockpiles (including snow) or other construction supplies is permitted at the top of the cut slope within a distance of at least 1.5 m from the top of the cut.
- Exposed soil along the slope must be protected from surface erosion during construction.
- Construction activities should be scheduled so that the length of time the temporary cut slope is left open is reduced to the extent practical.
- Erosion control measures must be implemented as appropriate such that runoff from the site is reduced to the extent practical.
- Surface water must be diverted away from the excavation and from the top of the slope.

11.5 Embankment Construction

Embankment construction should be in accordance with OPSS 206, November 2009 and the approach fills should be constructed in advance of pile driving operations. Oversize materials (e.g. greater than 75 mm nominal diameter) should not be used in the embankment fills through which piles will be driven.

Earth fill embankment slopes and cut slopes must be provided with erosion protection in accordance with OPSS 571 and OPSS 572.

At the north approach the embankment fill will transition into an earth cut. Bonding between the embankment fill and the existing soils should be established by benching as per OPSD 208.010.

Removals at the north and south abutments should extend to Elev. 172 m and Elev. 172.5 m respectively over a horizontal distance of 15 m to remove the wet silt layer and to expose the underlying low permeability silty clay soils.

12 RETAINED SOIL SYSTEMS

The soils at this site are settlement sensitive and this aspect must be taken into account when designing RSS walls for this site. Proprietary “two stage” systems are normally used where settlement situations arise. These systems consist of a mechanically stabilized structure with a “stage one” flexible facing that accommodates settlement. After settlement is complete a permanent “stage two” facing is attached. RSS walls used at this site should be “two stage” systems and would be subject to the requirements presented in this section. A conventional concrete abutment will be required for the contemplated design but RSS could be used for wing walls.

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 an NSSP for the RSS wall. The design, supply, and construction of the 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.

12.1 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 for the RSS facing units 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 well compacted approach fill. All founding subgrades should be inspected and evaluated by the Quality Verification Engineer (QVE), at the time of construction.

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 approach fill = 0.6

All topsoil, organics and soft soils should be removed from the subgrade of the RSS wall prior to placement of the fill.

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

The settlement of a wall will depend on the fill material used, the foundation soils and the quality of construction. The flexible facing should be designed to accommodate the settlements outlined in Table 11.3.



12.2 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.

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

- RSS founded at the base of the embankment.
- 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 founded on a Granular 'A' pad at the abutments.
- Water Level at Elev. 172.5 m (south abutment) and Elev. 173.5 m (north abutment).

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.

If the RSS wall is 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.

13 BACKFILL TO ABUTMENTS

For 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.



14 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 14.1)

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

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

q = value of any surcharge (kPa)

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 14.1.

Table 14.1 – 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*	0.31	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.

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.

15 EROSION PROTECTION

The H.W.L's in the Welland River and Old Welland Canal are Elev. 174.04 m and Elev. 173.70 m respectively. These water levels will partially submerge the forward slopes at the bridge abutments and the banks of Merritt Island and will cause erosion if the slopes are not protected.

We recommend rock protection (rip-rap) be used to armour areas that are susceptible to erosion. During storm events surface water can cause erosion beneath the rip-rap and movement of fines through the rip-rap blanket will occur. Therefore, a properly designed granular and fabric filter blanket would be required. The sides/ends of the filter fabric must also be anchored by burying in an anchor trench. Erosion protection should be in accordance with OPSS 511, November 2008.

16 SEISMIC CONSIDERATIONS

16.1 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.

16.2 Liquefaction Potential

The potential for liquefaction of the silt layer below the footprint of the approach embankments was assessed using the Seed and Idriss (1971) method¹. The results indicate that there is negligible potential for soil liquefaction below the embankments. Some toe failure may occur but is expected to be limited and readily repairable.

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.



There is also negligible potential for soil liquefaction of the foundation soils below the abutments and piers.

16.3 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 16.1 may be used:

Table 16.1 – 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	0.31	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

17 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.
- unwatering.
- artesian conditions.
- pollution, siltation or disruption of environmentally sensitive areas.



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TABLE

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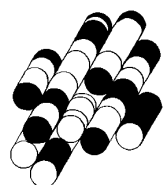


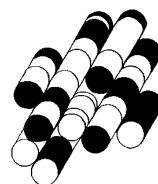
TABLE 1

DOCUMENT	TITLE
OPSS 206	Construction Specification for Grading.
OPSS 501	Construction Specification for Compacting.
OPSS 511	Construction Specification for Rip-Rap, Rock Protection and Granular Sheeting.
OPSS 539	Construction Specification for Temporary Protection Systems.
OPSS 571	Construction Specification for Sodding.
OPSS 572	Construction Specification for Seed and Cover.
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.
SP 105S10	Construction Specification for Compaction.
SP 110S13	Amendment to OPSS 1010, April 2004.
SP 599S22	Supply and Construction of Retained Soil Systems.
SP 599S23	Materials, quality control, quality assurance testing and acceptance criteria for precast concrete facing elements including panels.



APPENDICES

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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_v	m ² /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_r	1	SENSITIVITY = c_u / τ_r

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m ³	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	e_{min}	1, %	VOID RATIO IN DENSEST STATE
γ_s	kN/m ³	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
ρ_w	kg/m ³	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kN/m ³	UNIT WEIGHT OF WATER	S_r	%	DEGREE OF SATURATION	D_n	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 = $(w - w_p)/I_p$	i	1	HYDRAULIC GRADIENT
γ_{sat}	kN/m ³	UNIT WEIGHT OF SATURATED SOIL	I_c	1	CONSISTENCY INDEX = $(w_L - w)/I_p$	k	m/s	HYDRAULIC CONDUCTIVITY
ρ'	kg/m ³	DENSITY OF SUBMERGED SOIL	e_{max}	1, %	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:

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

Approximate ϕ

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

11. Aperture: estimated size of joint opening.

12. Degree of weathered rock material:

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

13. Strength of rock material:

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

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.

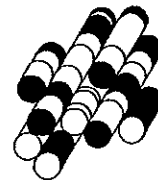
The design advice is based on the factual data obtained from this investigation made at the site by Terraprobe and are intended for use by the owner and its retained designers in the design phase of the project. If there are changes to the project scope and development features, or there is any additional 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

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APPENDIX A

Record of Borehole Sheets

Terraprobe Inc.



RECORD OF BOREHOLE No W1

1 OF 2

METRIC

W.P. 280-99-00 LOCATION Coords: N:4764997.1 E:327226.3 ORIGINATED BY PK
DIST HWY 406 BOREHOLE TYPE Hollow Stem Augers COMPILED BY DB
DATUM Geodetic DATE 11.20.09 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							WATER CONTENT (%)
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × LAB VANE						
178.3	Ground Surface						20 40 60 80 100	20 40 60 80 100	10 20 30						
178.1	200mm TOPSOIL														
0.2	FILL - Silty Clay, some gravel, trace sand, firm, brown, dry to damp		1	SS	7									13 4 44 39	
177.6															
0.7	FILL - Cobbles and Gravel, very dense, grey, moist to wet		2	SS	100/ 8cm										
176.5															
1.8	SILTY CLAY very stiff to hard, brown, damp to moist		3	SS	48										
			4	SS	30									0 0 62 38	
			5	SS	19										
174.6															
3.7	SILT trace clay, compact to dense, brown, wet		6	SS	48									0 0 96 4	
			7	SS	46										
			8	SS	21										
172.4															
5.9	SILTY CLAY trace sand, trace gravel, stiff to very stiff, grey, damp to moist (GLACIAL TILL)		9	SS	23										
			10	SS	20									2 3 69 26	
			11	SS	15									0 2 71 27	
			12	TW	PH								21.0	0 3 69 28	
			13	SS	17										
164.7															
13.6	End of Borehole														
	Borehole was dry (not stabilized) and hole open to full depth on completion. Sampler bouncing on probable obstruction at 0.7m														

Continued Next Page

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

ONTARIO MOT 1-09-4135 WELLAND CANAL GPJ ONTARIO MOT GDT 03/22/10

METRIC

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

Water Level Readings:		
Date	Depth(m)	Elevation(m)
Nov.30.09	3.0	175.3
Dec.08.09	2.7	175.6
Dec.16.09	2.6	175.7
Jan.04.10	2.5	175.8
Jan.11.10	2.5	175.8

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

RECORD OF BOREHOLE No W2

1 OF 1

METRIC

W.P. 280-99-00 LOCATION Coords: N:4765017.0 E:327225.3 ORIGINATED BY PK
DIST HWY 406 BOREHOLE TYPE Hollow Stem Augers COMPILED BY DB
DATUM Geodetic DATE 11.20.09 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								WATER CONTENT (%)		
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL						× LAB VANE		
178.4	Ground Surface																	
178.2	180mm TOPSOIL																	
0.2	FILL - Silty Clay, trace sand, trace gravel, trace rootlets, very stiff, brown, moist		1	SS	17													
177.7																		
0.7	SILTY CLAY very stiff to hard, brown, damp to moist		2	SS	83													
			3	SS	26										0 0 64 35			
			4	SS	58													
			5	SS	41										0 0 77 23			
			6	SS	52													
173.5			7	SS	53													
4.9	SILT trace clay, compact, brown, wet		8	SS	18													
172.5			9	SS	12													
5.9	SILTY CLAY trace sand, trace gravel, firm to very stiff, grey, damp to moist (GLACIAL TILL)		10	SS	15													
			11	TW	PH									21.0	0 3 71 26			
			12	SS	18													
166.4																		
12.0	End of Borehole Borehole was dry (not stabilized) and hole open to full depth on completion. No sample recovery at SS9 and SS10. Sampler redriven and disturbed sample collected. Piezometer installation consists of a 19mm diameter, Schedule 40 PVC pipe with a 1.52m slotted screen. Water Level Readings: Date Depth(m) Elevation(m) Nov.30.09 6.3 172.1 Dec.08.09 4.4 174.0 Dec.16.09 4.7 173.7 Jan.14.10 4.2 174.2																	

ONTARIO MOT 1-09-4135 WELLAND CANAL_GPJ ONTARIO MOT GDT 03/22/10

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

RECORD OF BOREHOLE No W3

1 OF 3

METRIC

W.P. 280-99-00 LOCATION Coords: N:4765031.1 E:327217.8 ORIGINATED BY MP
DIST HWY 406 BOREHOLE TYPE Hollow Stem Augers / Casing and Washboring / NQ Rock Coring COMPILED BY DB
DATUM Geodetic DATE 11.23.09 - 11.24.09 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			SHEAR STRENGTH kPa						
								20 40 60 80 100		10 20 30				
								○ UNCONFINED + FIELD VANE						
								● QUICK TRIAXIAL × LAB VANE						
								20 40 60 80 100		WATER CONTENT (%)				
176.8	Ground Surface													
176.6	FILL - Sand and Gravel, trace silt, grey / brown, damp to moist		1	SS	6						○			
0.2	FILL - Silty Clay, trace sand, trace gravel, firm, brown, damp													
176.1														
0.7	SILTY CLAY trace sand, very stiff, brown, damp to moist		2	SS	17		176				○			
			3	SS	25								45	0 1 63 36
			4	SS	21		175				○			
173.9														
2.9	SILT trace sand, trace clay, dense to very dense, brown, wet		5	SS	52		174				○			0 2 92 6
			6	SS	32		173				○			
172.4														
4.4	SILTY CLAY trace sand, trace gravel, stiff to very stiff, grey, damp to moist (GLACIAL TILL)		7	SS	14		172				○			
			8	SS	19		171	1.1	1.3					5 3 68 24
			9	SS	9		170	1.2		1.4				
			10	SS	10		169	1.4						1 2 72 25
			11	TW	PH		168	1.4	1.4					

Continued Next Page

+³, x³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No W3

3 OF 3

METRIC

W.P. 280-99-00 LOCATION Coords: N:4765031.1 E:327217.8 ORIGINATED BY MP
 DIST HWY 406 BOREHOLE TYPE Hollow Stem Augers / Casing and Washboring / NQ Rock Coring COMPILED BY DB
 DATUM Geodetic DATE 11.23.09 - 11.24.09 CHECKED BY RA

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20	40	60	80	100	W _p	W		
145.9			3	RUN	NQ											RUN#3 TCR=98% SCR=68% RQD=21%
30.9	End of Borehole No sample recovery at SS10. Sampler redriven and disturbed sample collected. Vane sinking under weight of rods at 11.6m. Resistance to augering at 13.7m and 14.3m. Piezometer installation consists of a 19mm diameter, Schedule 40 PVC pipe with a 1.52m slotted screen. Water Level Readings: Date Depth(m) Elevation(m) Dec.08.09 4.3 172.5 Dec.16.09 4.2 172.6 Jan.04.10 4.2 172.6															

CORE LOG



Terraprobe

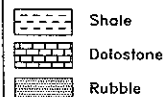
Project	Highway 406 Twinning	Orientation	Vertical	Ground Elevation	176.8m	Datum	Geodetic	Borehole No.	W3
Location	Welland, Ontario	Date Started	November 24, 2009	Completed	November 24, 2009	Logged By	B. Ripley	Sheet	1 of 1
W.P.:	280-99-00	Drilling Agency	Ground Works	Drill Type	Bombardier	Core Barrel & 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/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		
150.8	26.0																			
150.3	26.5		Overburden, see Borehole Log W3																	
150.0	26.8																			
149.8	27.0		SALINA FORMATION BEDROCK																	
			Dolostone (93.9%) Frequent shale seams to shaly, light grey, highly fractured, thinly to medium bedded, medium strength, slightly weathered.																	
149.3	27.5		Shale (6.1%) Moderately weathered, dark grey, low to medium strength.	1	B	F V	C	RP SP	T	0 to 1							NQ			
148.8	28.0																			
148.3	28.5																			
147.8	29.0			1	B	V	C	RP SP	T	0 to 1							NQ			
147.3	29.5																			
146.8	30.0			1	B	F D V	C	RP SP	T	0 to 1							NQ			
146.3	30.5																			
145.9	30.9																			
145.8	31.0		End of Core Log																	
			Highly fractured zones at: 26.8-27.0m; 27.3-27.34m; 30.7-30.8m; 30.3-30.5m.																	
145.3	31.5		Rubble at 26.8-26.9m.																	
144.8	32.0																			

Remarks:

Rubble indicated by '4'.

LEGEND:



RECORD OF BOREHOLE No W4

1 OF 3

METRIC

W.P. 280-99-00 LOCATION Coords: N:4765036.5 E:327230.2 ORIGINATED BY MP
 DIST HWY 406 BOREHOLE TYPE Hollow Stem Augers / Casing and Washboring / NQ Rock Coring COMPILED BY DB
 DATUM Geodetic DATE 11.25.09 - 11.27.09 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			SHEAR STRENGTH kPa							
							20 40 60 80 100			10 20 30					
							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE								
177.6	Ground Surface														
177.5	100mm TOPSOIL														
0.1	FILL - Silty Clay, trace sand, trace gravel, stiff, brown, damp to moist		1	SS	14		177								
176.7			2	SS	52		176								
0.9	SILTY CLAY trace sand, very stiff to hard, brown, damp to moist		3	SS	37		175								
			4	SS	27		174								
			5	SS	19		173								
173.9			6	SS	37		172								
3.7	SILT trace clay, trace sand, compact to dense, brown, wet		7	SS	26		171								
172.4			8	SS	18		170								
5.2	SILTY CLAY trace sand, stiff to very stiff, grey, damp to moist (GLACIAL TILL)		9	SS	17		169								
			10	TW	PH		168								
			11	SS	11		167								
			12	SS	15		166								
			13	SS	3		165								
			14	SS	17		164								
162.6	hard						163								

ONTARIO MOT 1-09-4135 WELLAND CANAL.GPJ ONTARIO MOT.GDT 03/22/10

Continued Next Page




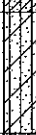
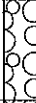

+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No W4

2 OF 3

METRIC

W.P. 280-99-00 LOCATION Coords: N:4765036.5 E:327230.2 ORIGINATED BY MP
DIST HWY 406 BOREHOLE TYPE Hollow Stem Augers / Casing and Washboring / NQ Rock Coring COMPILED BY DB
DATUM Geodetic DATE 11.25.09 - 11.27.09 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							WATER CONTENT (%)			
							20 40 60 80 100											
								○ UNCONFINED + FIELD VANE										
								● QUICK TRIAXIAL × LAB VANE										
							20 40 60 80 100											
15.0	SILTY CLAY trace sand, trace gravel, hard, grey, damp (GLACIAL TILL)		15	SS	50									6 9 64 21				
				16	SS	120/ 15cm												
				17	SS	44												
158.3 19.3	SAND AND SILT trace to some clay, trace gravel, compact to very dense, grey, moist to wet (GLACIAL TILL)		18	SS	27													
				19	SS	67								6 42 42 10				
155.3 22.3	SILTY CLAY trace sand to sandy, trace gravel, very stiff to hard, grey, damp to moist (GLACIAL TILL)																	
				20	SS	39												
				21	SS	28												
152.3 25.3	SILT AND SAND trace clay, trace to some gravel, very dense, grey, moist (GLACIAL TILL)		22	SS	70									Nov.26 Nov.27				
151.1 26.5	COBBLES some gravel, inferred very dense, grey, wet													RUN#1 TCR=89% SCR=53% RQD=22%				
150.1 27.5	DOLOSTONE BEDROCK Light grey, slightly weathered, thinly to medium bedded, medium strength, with interbeds of dark grey, moderately weathered, low to medium strength shale.			1	RUN	NQ												
														RUN#2 TCR=94% SCR=94% RQD=63%				
				2	RUN	NQ												
147.9 29.7	End of Borehole																	

Continued Next Page

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

ONTARIO MOT 1-09-4135 WELLAND CANAL GPJ ONTARIO MOT GDT 03/22/10

3 OF 3

METRIC

[illegible]

Unable to push vane to 15.2m.
Resistance to augering at 13.7m and 14.3m.
Borehole sealed with bentonite slurry to ground surface.

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

CORE LOG



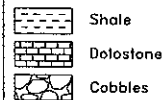
Terraprobe

Project	Highway 406 Twinning	Orientation	Vertical	Ground Elevation	177.6m	Datum	Geodetic	Borehole No.	W4
Location	Welland, Ontario	Date Started	November 27, 2009	Completed	November 27, 2009	Logged By	B. Ripley	Sheet	1 of 1
W.P.:	280-99-00	Drilling Agency	Ground Works	Drill Type	Bombardier	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/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	
151.6	26.0																		
151.1	26.5		Overburden, see Borehole Log W4																
150.9	26.7		Overburden,																
150.6	27.0		COBBLES some gravel, see Borehole Log W4																
150.1	27.5		SALINA FORMATION BEDROCK	1	B	F V	C	RP SP	T	0 to 1				#1 TCR 69 SCR 53	22	NQ			
149.6	28.0		Dolostone (91%) Frequent shale seams to shaly, light to medium grey, highly fractured, thinly to medium bedded, medium strength, slightly weathered.																
149.1	28.5		Shale (9%) Moderately weathered, medium to dark grey, low to medium strength.											#2 TCR 94 SCR 94	63	NQ			
148.6	29.0			1	B	V	C	RP SP	T	0 to 1									
148.1	29.5																		
147.9	29.7		End of Core Log																
147.6	30.0		Highly fractured zones at: 27.9-27.95m.																
147.1	30.5		Rubble at 27.5 and 27.8m.																
146.6	31.0																		
146.1	31.5																		
145.6	32.0																		

Remarks:

LEGEND:



1 OF 2

METRIC

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

RECORD OF BOREHOLE No W5

2 OF 2

METRIC

W.P. 280-99-00 LOCATION Coords: N:4765094.5 E:327222.4 ORIGINATED BY PK
DIST HWY 406 BOREHOLE TYPE Hollow Stem Augers / Casing and Washboring / NQ Rock Coring COMPILED BY DB
DATUM Geodetic DATE 12.08.09 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
155.5 15.3	SILTY CLAY some sand, trace gravel, very stiff to hard, reddish brown, moist to wet (GLACIAL TILL)		10	SS	34		155							2 18 50 30
152.4 18.4	SILT AND SAND some gravel, trace clay, compact, grey, wet (GLACIAL TILL)		12	SS	16		152							
150.9 19.9	SAND AND GRAVEL occasional cobbles, very dense, grey, wet		13	SS	100/ 5cm		150							
149.5 21.3	DOLOSTONE BEDROCK Light grey, slightly weathered, thinly to medium bedded, medium strength, with interbeds of light to dark grey, moderately weathered, low to medium strength shale.		1	RUN	NQ		149							RUN#1 TCR=100% SCR=62% RQD=13%
			2	RUN	NQ		148							RUN#2 TCR=100% SCR=81% RQD=41%
			3	RUN	NQ		147							RUN#3 TCR=100% SCR=92% RQD=34%
145.5 25.3	End of Borehole Artesian conditions encountered. Water flowing from augers. Top of auger stem is Elev. ~ 171.7m. Resistance to augering at 9.1m. Borehole sealed by pumping bentonite slurry into hole from 25.3m to 5.2m.						146							

ONTARIO MOT 1-09-4135 WELLAND CANAL GPJ ONTARIO MOT.GDT 03/22/10

CORE LOG



Terraprobe

Project	Highway 406 Twinning	Orientation	Vertical	Ground Elevation	170.8m	Datum	Geodetic	Borehole No.	W5
Location	Welland, Ontario	Date Started	December 8, 2009	Completed	December 9, 2009	Logged By	B. Ripley	Sheet	1 of 1
W.P.:	280-99-00	Drilling Agency	DDSI	Drill Type	Bombardier	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	UNCONFINED COMPRESSIVE STRENGTH MPa	UNIT WEIGHT (kN/m ³)
				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
150.3	20.5																	
149.8	21.0		Overburden, see Borehole Log W5															
149.5	21.3																	
149.3	21.5		SALINA FORMATION BEDROCK															
			Dolostone (97%)	1	B	V	C	RP	T	0 to 1				#1	13	NQ		
			Frequent shale seams to shaly, light to medium grey, highly fractured, thinly to medium bedded, medium strength, slightly weathered.											TCR 100				
148.8	22.0													SCR 62				
			Shale (3%)											#2	41	NQ		
			Moderately weathered, medium to dark grey, low to medium strength.											TCR 100				
148.3	22.5													SCR 81				
147.8	23.0			1	B	V	C	RP	T	0 to 1								
147.3	23.5																	
146.8	24.0													#3	34	NQ		
														TCR 100				
146.3	24.5			1	B	V	C	RP	T	0 to 1				SCR 92				
145.8	25.0																	
145.5	25.3																	
145.3	25.5		End of Core Log															
144.8	26.0		Highly fractured zones at:															
			21.4-21.5m; 21.7-21.9m; 22.8-23.1m;															
			23.6-23.7m; 24.8-24.9m.															
144.3	26.5																	

Remarks:

LEGEND:



Dolostone

RECORD OF BOREHOLE No W6

1 OF 3

METRIC

W.P. 280-99-00 LOCATION Coords: N:4765154.5 E:327213.4 ORIGINATED BY MP
DIST HWY 406 BOREHOLE TYPE Hollow Stem Augers / Casing and Washboring / NQ Rock Coring COMPILED BY DB
DATUM Geodetic DATE 11.19.09 - 11.20.09 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			SHEAR STRENGTH kPa		w _p	w	w _L		
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × LAB VANE					
175.6	Ground Surface							20 40 60 80 100						GR SA SI CL
175.6 0.1	50mm TOPSOIL		1	SS	5		175							
	FILL - Silty Clay, trace sand, trace gravel, firm to very stiff, brown / greyish brown, damp to moist		2	SS	8		174							0 0 50 50
			3	SS	5		173							
			4	SS	11		172							
			5	SS	7		171							
	---		6	SS	10		170							0 3 55 42
	frequent silt inclusions		7	SS	12		169							
	---		8	SS	15		168							
			9	SS	17		167							
			10	SS	13		166							
			11	SS	8		165							
			12	SS	3		164							
			13	SS	6		163							0 5 83 12
			14	TW	PH		162							
							161							
167.0														
8.6	SILTY CLAY trace sand, trace gravel, firm to stiff, grey, moist (GLACIAL TILL)													
											</			

Continued Next Page

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

ONTARIO MOT-1-09-4135 WELLAND CANAL.GPJ ONTARIO MOT.GDT 03/22/10

RECORD OF BOREHOLE No W6

2 OF 3

METRIC

W.P. 280-99-00 LOCATION Coords: N:4765154.5 E:327213.4 ORIGINATED BY MP
DIST HWY 406 BOREHOLE TYPE Hollow Stem Augers / Casing and Washboring / NQ Rock Coring COMPILED BY DB
DATUM Geodetic DATE 11.19.09 - 11.20.09 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			SHEAR STRENGTH kPa							WATER CONTENT (%)
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × LAB VANE						
							20 40 60 80 100								
15.0	SILTY CLAY trace sand, trace gravel, stiff to very stiff, grey, moist (GLACIAL TILL)		15	SS	26									1 8 69 22	
			16	SS	3										
157.1	very loose		17	SS	3										
18.5	SANDY SILT trace gravel, trace clay, compact to very dense, grey, wet (GLACIAL TILL)														
			18	SS	16									2 28 62 8	
			19	SS	37										
153.3	SILTY CLAY trace sand, trace gravel, stiff, grey, moist (GLACIAL TILL)														
22.3			20	SS	8									1 4 49 48	
			21	SS	10										
150.5	DOLOSTONE BEDROCK Light to dark grey, slightly weathered, thinly to medium bedded, medium strength, with interbeds of dark grey, moderately weathered, low to medium strength shale.		1	RUN	NQ									RUN#1 TCR=100% SCR=66% RQD=63%	
			2	RUN	NQ									RUN#2 TCR=99% SCR=99% RQD=32%	
			3	RUN	NQ									RUN#3 TCR=100% SCR=91% RQD=52%	
146.2	End of Borehole														
29.4															

ONTARIO MOT 1-09-4135 WELLAND CANAL.GPJ ONTARIO MOT GDT 03/22/10

Continued Next Page

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

RECORD OF BOREHOLE No W6

3 OF 3

METRIC

W.P. 280-99-00 LOCATION Coords: N:4765154.5 E:327213.4 ORIGINATED BY MP
 DIST HWY 406 BOREHOLE TYPE Hollow Stem Augers / Casing and Washboring / NQ Rock Coring COMPILED BY DB
 DATUM Geodetic DATE 11.19.09 - 11.20.09 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	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 x LAB VANE										
							20	40	60	80	100		10	20	30	kn/m³	GR SA SI CL
	Unable to push vane to 9.1m, 13.7m, 24.3m and 25.3m.																
	Soil rose from 20.2m to 14.3m in augers. Resorted to casing and washboring.																
	Piezometer installation consists of a 19mm diameter, Schedule 40 PVC pipe with a 1.52m slotted screen.																
	Water Level Readings:																
	Date Depth(m) Elevation(m)																
	Nov.30.09 2.4 173.2																
	Dec.08.09 2.7 172.9																
	Dec.16.09 2.4 173.2																
	Jan.04.10 2.4 173.2																

CORE LOG



Terraprobe

Project	Highway 406 Twinning	Orientation	Vertical	Ground Elevation	175.5m	Datum	Geodetic	Borehole No.	W6
Location	Welland, Ontario	Date Started	November 20, 2009	Completed	November 20, 2009	Logged By	B. Ripley	Sheet	1 of 1
W.P.:	280-99-00	Drilling Agency	DDSI	Drill Type	Bombardier	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	UNCONFINED COMPRESSIVE STRENGTH MPa	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
151.1	24.5		Overburden, see Borehole Log W6															
150.6	25.0																	
150.5	25.1																	
150.1	25.5		SALINA FORMATION BEDROCK															
			Dolostone (99%)															
			Occasional to frequent shale seams, light to medium grey, moderately to highly fractured, thinly to medium bedded, medium strength, slightly weathered. Calcite filled joints (occasional in RUN#2 and RUN#3).	1	B	V	C	RP SP	T	0 to 1				#1	63	NQ		
149.6	26.0													TCR 100				
														SCR 66				
149.1	26.5		Shale (1%)															
			Moderately weathered, medium to dark grey, low to medium strength.															
148.6	27.0			1	B	V	C	RP SP	T	0 to 1				#2	32	NQ		
														TCR 99				
														SCR 99				
148.1	27.5																	
147.6	28.0																	
147.1	28.5			1	B	V	C	RP SP	T	0 to 1				#3	52	NQ		
														TCR 100				
														SCR 91				
146.6	29.0																	
146.2	29.4																	
146.1	29.5		End of Core Log															
145.6	30.0		Highly fractured zones at:															
			25.2m; 25.4-25.6m; 28.0-28.2m.															
145.1	30.5																	

Remarks:

LEGEND:



Dolostone

RECORD OF BOREHOLE No W7

1 OF 2

METRIC

W.P. 280-99-00 LOCATION Coords: N:4765147.3 E:327203.1 ORIGINATED BY MP
 DIST HWY 406 BOREHOLE TYPE Hollow Stem Augers / Casing and Washboring / NQ Rock Coring COMPILED BY DB
 DATUM Geodetic DATE 11.23.09 - 11.24.09 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			SHEAR STRENGTH kPa								
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× LAB VANE					
175.2	Ground Surface					20	40	60	80	100	10	20	30			
175.1 0.1	80mm TOPSOIL		1	SS	9											
	FILL - Silty Clay, trace sand, trace gravel, firm to hard, brown / greyish brown, damp to moist		2	SS	42											
			3	SS	10											
			4	SS	11											
			5	SS	25											
	----- frequent silt inclusions -----		6	SS	19											
			7	SS	15											
			8	SS	15											
	----- frequent silt inclusions -----		9	SS	8											
			10	SS	24											
166.6 8.6	SILTY CLAY trace sand, trace gravel, stiff to very stiff, grey, damp to moist (GLACIAL TILL)		11	SS	5											
			12	SS	3											
			13	TW	PH											
			14	SS	21											
160.2																

ONTARIO MOT 1-09-4135 WELLAND CANAL GPJ ONTARIO MOT GDT 03/22/10

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	175.2m	Datum	Geodetic	Borehole No.	W7
Location	Welland, Ontario	Date Started	November 23, 2009	Completed	November 24, 2009	Logged By	B. Ripley	Sheet	1 of 1
W.P.:	280-99-00	Drilling Agency	DDSI	Drill Type	Bombardier	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/m ³)
				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
150.7	24.5																	
150.2	25.0		Overburden, see Borehole Log W7															
150.0	25.2																	
149.7	25.5		SALINA FORMATION BEDROCK	1	B	V	C	SP RP	T	0 to 1				#1 TCR 0	0	NQ		
149.2	26.0		Dolostone (99%) Occasional to frequent shale seams, light to medium grey, moderate to highly fractured, thinly to medium bedded, medium strength, slightly weathered.	1	B	V	C	SP RP	T	0 to 1				#2 TCR 97 SCR 91	57	NQ		
148.7	26.5		Shale (1%) Moderately weathered, medium to dark grey, low to medium strength.															
148.2	27.0													#3 TCR 95 SCR 88	48			
147.7	27.5			1	B	V	C	SP RP	T	0 to 1						NQ		
147.2	28.0																	
147.1	28.2		End of Core Log															
146.7	28.5		Highly fractured zones at: 25.2-25.5m; 27.9-28.1m.															
146.2	29.0																	
145.7	29.5																	
145.2	30.0																	
144.7	30.5																	

Remarks:

LEGEND:



Dolostone

RECORD OF BOREHOLE No W8

1 OF 3

METRIC

W.P. 280-99-00 LOCATION Coords: N:4765168.4 E:327205.7 ORIGINATED BY MP
 DIST HWY 406 BOREHOLE TYPE Hollow Stem Augers / Casing and Washboring / NQ Rock Coring COMPILED BY DB
 DATUM Geodetic DATE 11.30.09 - 12.01.09 CHECKED BY RA

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		20	40	60	80	100	W _p	W	W _L		
175.9	Ground Surface															
175.9 0.1	50mm TOPSOIL		1	SS	17											
	FILL - Silty Clay, trace sand, trace gravel, stiff to hard, brown / greyish brown, damp to moist		2	SS	12	175										
			3	SS	15	174										
			4	SS	26											
			5	SS	14	173										
			6	SS	28	172										
			7	SS	25	171										
			8	SS	22	170										
	----- frequent silt inclusions -----		9	SS	27	169										
			10	SS	73	168										
167.3 8.6	SILTY CLAY trace sand, trace gravel, occasional cobbles between 15.2m and 16.8m, stiff to very stiff, hard between 13.2m and 17.7m, grey, damp to moist (GLACIAL TILL)		11	SS	18	167										
			12	TW	PH	165									21.5	0 2 74 24
			13	SS	10	164										
			14	SS	41	162										2 8 66 24
160.9						161										

ONTARIO MOT 1-09-4135 WELLAND CANAL GPJ ONTARIO MOT GDT 03/22/10

Continued Next Page

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

RECORD OF BOREHOLE No W8

2 OF 3

METRIC

W.P. 280-99-00 LOCATION Coords: N:4765166.4 E:327205.7 ORIGINATED BY MP
DIST HWY 406 BOREHOLE TYPE Hollow Stem-Augers / Casing and Washboring / NQ Rock Coring COMPILED BY DB
DATUM Geodetic DATE 11.30.09 - 12.01.09 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									
								○ UNCONFINED	+ FIELD VANE						● QUICK TRIAXIAL	× LAB VANE	WATER CONTENT (%)
						20	40	60	80	100	10	20	30				
15.0	SILTY CLAY trace sand, trace gravel, very stiff to hard, grey, damp to moist (GLACIAL TILL)		15	SS	100/ 10cm												
			16	SS	30												
			17	SS	16												
156.6 19.3	very loose --- SILT AND SAND trace to some clay, trace to some gravel, occasional cobbles below 21.9m, very dense, grey, moist to wet (GLACIAL TILL)		18	SS	3												
			19	SS	60												
153.6 22.3	SILTY CLAY trace to some sand, trace gravel, very stiff to hard, grey, damp to moist (GLACIAL TILL)		20	SS	37												
			21	SS	19												
150.6 25.3	DOLOSTONE BEDROCK Light to dark grey, slightly weathered, thinly to medium bedded, medium strength, with interbeds of dark grey, moderately weathered, low to medium strength shale.		1	RUN	NQ										RUN#1 TCR=100% SCR=78% RQD=37%		
			2	RUN	NQ											RUN#2 TCR=97% SCR=90% RQD=26%	
			3	RUN	NQ											RUN#3 TCR=99% SCR=89% RQD=43%	
146.4 29.5	End of Borehole																

ONTARIO MOT 1-09-4135 WELLAND CANAL GPJ ONTARIO MOT GDT 03/22/10

Continued Next Page

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

METRIC

ELEV DEPTH	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			
	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40						60	80	100
								SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100									
								WATER CONTENT (%) 10 20 30									

[illegible]


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

CORE LOG



Terraprobe

Project	Highway 406 Twinning	Orientation	Vertical	Ground Elevation	175.9m	Datum	Geodetic	Borehole No.	W8
Location	Welland, Ontario	Date Started	December 1, 2009	Completed	December 1, 2009	Logged By	B. Ripley	Sheet	1 of 1
W.P.:	280-99-00	Drilling Agency	Ground Works	Drill Type	Bombardier	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	
151.4	24.5																		
150.9	25.0		Overburden, see Borehole Log W8																
150.6	25.3		SALINA FORMATION BEDROCK																
150.4	25.5		Dolostone (99%) Occasional to frequent shale seams, light to medium grey, moderately to highly fractured, thinly to medium bedded, medium strength, slightly weathered.	1	B	V	C	RP SP	T	0 to 1					#1 TCR 100 SCR 78	37	NQ		
149.9	26.0		Occasional calcite filled joints.																
149.4	26.5		Shale (1%) Moderately weathered, medium to dark grey, low to medium strength.																
148.9	27.0			1	B	V	C	RP SP	T	0 to 1					#2 TCR 97 SCR 90	26	NQ		
148.4	27.5																		
147.9	28.0																		
147.4	28.5				1	B	V	C	RP SP	T	0 to 1				#3 TCR 99 SCR 89	43	NQ		
146.9	29.0																		
146.4	29.5			End of Core Log															
			Highly fractured zones at:																
145.9	30.0		25.3m; 25.6-25.7m; 25.8m; 27.4m; 28.1m.																
145.4	30.5																		

Remarks:

LEGEND:



Dolostone

RECORD OF BOREHOLE No W9

1 OF 3

METRIC

W.P. 280-99-00 LOCATION Coords: N:4765163.6 E:327199.4 ORIGINATED BY MP
DIST HWY 406 BOREHOLE TYPE Hollow Stem Augers / Casing and Washboring / NQ Rock Coring COMPILED BY DB
DATUM Geodetic DATE 12.02.09 - 12.04.09 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			SHEAR STRENGTH kPa						
175.6	Ground Surface							20 40 60 80 100						
175.5	80mm TOPSOIL							○ UNCONFINED + FIELD VANE						
0.1								● QUICK TRIAXIAL × LAB VANE						
	FILL - Silty Clay, trace sand, trace gravel, firm to very stiff, brown / greyish brown, damp to moist		1	SS	7		175							
			2	SS	13		174							
			3	SS	20		174						47	0 3 43 54
			4	SS	14		173							
			5	SS	14		172							
			6	SS	23		171							
	----- frequent silt inclusions -----		7	SS	19		170							
			8	SS	19		169							
			9	SS	17		168							
			10	SS	28		167							
			11	SS	15		166							
164.8			12	SS	13		165							
10.8	SILTY CLAY trace to some sand, trace gravel, stiff to hard, grey, damp to moist (GLACIAL TILL)		13	SS	12		164							
			14	SS	33		163							1 15 62 22
							162							
							161							2 9 65 24
160.6														

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

ONTARIO MOT 1-09-4135 WELLAND CANAL.GPJ ONTARIO MOT.GDT 03/22/10

2 OF 3

METRIC

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

RECORD OF BOREHOLE No W9

3 OF 3

METRIC

W.P. 280-99-00 LOCATION Coords: N:4765163.6 E:327199.4 ORIGINATED BY MP
DIST HWY 406 BOREHOLE TYPE Hollow Stem Augers / Casing and Washboring / NQ Rock Coring COMPILED BY DB
DATUM Geodetic DATE 12.02.09 - 12.04.09 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																					
	<p>Piezometer installation consists of a 19mm diameter, Schedule 40 PVC pipe with a 1.52m slotted screen.</p> <p>Water Level Readings:</p> <table border="1"> <thead> <tr> <th>Date</th> <th>Depth(m)</th> <th>Elevation(m)</th> </tr> </thead> <tbody> <tr> <td>Dec.08.09</td> <td>2.5</td> <td>173.1</td> </tr> <tr> <td>Dec.16.09</td> <td>2.3</td> <td>173.3</td> </tr> <tr> <td>Jan.11.10</td> <td>2.4</td> <td>173.2</td> </tr> <tr> <td>Jan.14.10</td> <td>2.4</td> <td>173.2</td> </tr> </tbody> </table>	Date	Depth(m)	Elevation(m)	Dec.08.09	2.5	173.1	Dec.16.09	2.3	173.3	Jan.11.10	2.4	173.2	Jan.14.10	2.4	173.2															
Date	Depth(m)	Elevation(m)																													
Dec.08.09	2.5	173.1																													
Dec.16.09	2.3	173.3																													
Jan.11.10	2.4	173.2																													
Jan.14.10	2.4	173.2																													

CORE LOG



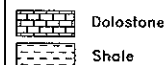
Terraprobe

Project	Highway 406 Twinning	Orientation	Vertical	Ground Elevation	175.6m	Datum	Geodetic	Borehole No.	W9
Location	Welland, Ontario	Date Started	December 4, 2009	Completed	December 4, 2009	Logged By	B. Ripley	Sheet	1 of 1
W.P.:	280-99-00	Drilling Agency	Ground Works	Drill Type	Bombardier	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	UNCONFINED COMPRESSIVE STRENGTH MPa	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
151.1	24.5																	
150.6	25.0		Overburden, see Borehole Log W9															
150.4	25.2																	
150.1	25.5		SALINA FORMATION BEDROCK															
			Dolostone (98%)															
			Occasional to frequent sand seams, light to medium grey, moderately to highly fractured, thinly to medium bedded, medium strength, slightly weathered.	1	B	V	C	RP SP	T	0 to 1				#1	24	NQ		
149.6	26.0		Occasional calcite filled joints / partings / bedding planes.															
149.1	26.5		Shale (2%)															
			Moderately weathered, medium to dark grey, low to medium strength, thinly laminated to fissile.															
148.6	27.0																	
148.1	27.5			1	B	V	C	RP SP	T	0 to 1				#2	22	NQ		
147.6	28.0																	
147.5	28.2		End of Core Log															
147.1	28.5																	
			Highly fractured zones at:															
			25.2-25.9m; 26.1-26.4m;															
146.6	29.0		Shale at 26.3-26.4m.															
146.1	29.5																	
145.6	30.0																	
145.1	30.5																	

Remarks:

LEGEND:



RECORD OF BOREHOLE No W10

1 OF 2

METRIC

W.P. 280-99-00 LOCATION Coords: N:4765222.6 E:327185.7 ORIGINATED BY PK
 DIST HWY 406 BOREHOLE TYPE Hollow Stem Augers / Casing and Washboring / NQ Rock Coring COMPILED BY DB
 DATUM Geodetic DATE 11.30.09 CHECKED BY RA

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			20	40	60	80	100		
173.3 0.0	Water Surface												
	WATER (Old Welland Canal)												
						173							
						172							
						171							
						170							
						169							
						168							
						167							
						166							
						165							
						164							
163.8 9.5	CANAL BED					163							
	Augered to 11.1m without sampling to stabilize augers. Inferred Silty Clay Till					162							
162.2 11.1	SILTY CLAY trace sand, trace gravel, very stiff to hard, soft below 15.4m, grey, damp to moist (GLACIAL TILL)		1	SS	18	161							
			2	SS	32	160							
			3	SS	33	159							
			4	SS	25								
			5	SS	17								
158.3													

Continued Next Page

+ 3, x 3. Numbers refer to
Sensitivity

○ 3% STRAIN AT FAILURE

ONTARIO MOT 1-09-4135 WELLAND CANAL.GPJ ONTARIO MOT.GDT 03/22/10

RECORD OF BOREHOLE No W10

2 OF 2

METRIC

W.P. 280-99-00 LOCATION Coords: N:4765222.6 E:327185.7 ORIGINATED BY PK
DIST HWY 406 BOREHOLE TYPE Hollow Stem Augers / Casing and Washboring / NQ Rock Coring COMPILED BY DB
DATUM Geodetic DATE 11.30.09 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			SHEAR STRENGTH kPa									
								20	40	60	80	100					
								20	40	60	80	100					
15.0	SILTY CLAY trace sand, trace gravel, soft, grey, damp to moist (GLACIAL TILL)		6	SS	3											0 2 68 30	
156.5																	
16.8	SILT AND SAND trace clay, trace gravel, compact to dense, grey, wet (GLACIAL TILL)		7	SS	33												1 37 57 5
			8	SS	17												
153.6																	
19.7	SILTY CLAY trace sand, very stiff, grey, damp to moist (GLACIAL TILL)		9	SS	16											0 6 36 58	
151.7																	
21.6	COBBLES - some gravel, very dense, grey, wet		10	SS	100/ 8cm												
151.3																	
22.0	DOLOSTONE BEDROCK Light grey, slightly weathered, thinly to medium bedded, medium strength, with interbeds of dark grey, moderately weathered, low to medium strength shale.		1	RUN	NQ											RUN#1 TCR=95% SCR=63% RQD=16%	
				2	RUN	NQ											RUN#2 TCR=100% SCR=91% RQD=43%
148.2	End of Borehole																
25.1	Borehole filled with drill water upon completion of coring. Borehole sealed with bentonite slurry from 25.1m to 11.1m.																

ONTARIO MOT 1-09-4135 WELLAND CANAL GPJ ONTARIO MOT GDT 03/22/10

CORE LOG



Terraprobe

Project	Highway 406 Twinning	Orientation	Vertical	Ground Elevation	173.3m	Datum	Geodetic	Borehole No.	W10
Location	Welland, Ontario	Date Started	November 30, 2009	Completed	November 30, 2009	Logged By	B. Ripley	Sheet	1 of 1
W.P.:	280-99-00	Drilling Agency	DDSI	Drill Type	Bombardier	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/m ³)
				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
151.8	21.5		Overburden, see Borehole Log W10															
151.3	22.0		SALINA FORMATION BEDROCK															
150.8	22.5		Dolostone (>99%) Occasional shale seams, light to medium grey, moderately to highly fractured, medium bedded, medium strength, slightly weathered.	1	B	V	C	RP	T	0 to 1				#1 TCR 95 SCR 63	16	NQ		
150.3	23.0		Occasional calcite filled joints poorly fossiliferous.															
149.8	23.5		Shale (<1%) Moderately weathered, medium to dark grey, low to medium strength.															
149.3	24.0			1	B	V	C	RP	T	0 to 1				#2 TCR 100 SCR 91	43	NQ		
148.8	24.5																	
148.3	25.0																	
148.2	25.1		End of Core Log															
147.8	25.5		Highly fractured zones at: 23.2-23.6m; 24.1-24.2m.															
147.3	26.0																	
146.8	26.5																	
146.3	27.0																	
145.8	27.5																	

Remarks:

LEGEND:



Dolostone

METRIC

ONTARIO MOT 1-09-4135 WELLAND CANAL.GPJ ONTARIO MOT.GDT 03/22/10

Continued Next Page

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

RECORD OF BOREHOLE No W11										2 OF 2		METRIC		
W.P. 280-99-00		LOCATION		Coords: N:4765254.3 E:327185.9				ORIGINATED BY PK						
DIST _____ HWY 406		BOREHOLE TYPE		Hollow Stem Augers				COMPILED BY DB						
DATUM Geodetic		DATE		11.25.09 - 11.26.09				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						
15.0	SILTY CLAY trace to some sand, trace to some gravel, sandy below 15.8m, firm, grey, damp to moist (GLACIAL TILL)													10 13 60 17
156.6														
16.8														
155.0	SILT AND SAND trace clay, trace gravel, very dense, grey, moist to wet. (GLACIAL TILL)		7	SS	51									
18.4														
152.4	SILTY CLAY trace to some sand, trace gravel, very stiff to hard, grey, damp (GLACIAL TILL)		8	SS	58									
21.0														
152.0														
21.4	COBBLES - some gravel, inferred very dense, grey, wet													
148.9														
24.5														
	DOLOSTONE BEDROCK Light to dark grey, slightly weathered, medium bedded, medium strength, with occasional dark grey, moderately weathered, low to medium strength shale seams.		1	RUN	NQ									
	End of Borehole Borehole sealed with bentonite slurry from 24.5m to 11.3m. No sample recovery at SS8.		2	RUN	NQ									
			3	RUN	NQ									

CORE LOG



Terraprobe

Project	Highway 406 Twinning	Orientation	Vertical	Ground Elevation	173.4m	Datum	Geodetic	Borehole No.	W11
Location	Welland, Ontario	Date Started	November 26, 2009	Completed	November 26, 2009	Logged By	B. Ripley	Sheet	1 of 1
W.P.:	280-99-00	Drilling Agency	DDSI	Drill Type	Bombardier	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/m ³)
				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
152.9	20.5																	
152.4	21.0		Overburden, see Borehole Log W11															
152.2	21.2		Overburden, COBBLES - some gravel, see Borehole Log W11															
152.0	21.4		SALINA FORMATION BEDROCK															
151.9	21.5																	
			Dolostone (99%) Occasional shale seams and partings, light to medium grey, moderately to highly fractured, medium bedded, medium strength, slightly weathered.	1	B	V	C	RP SP	T	0 to 1				#1 TCR 88 SCR 33	0	NQ		
151.4	22.0																	
150.9	22.5		Shale (1%) Moderately weathered, medium to dark grey, low to medium strength.											#2 TCR 100 SCR 16	79	NQ		
150.4	23.0			1	B	V	C	RP SP	T	0 to 1								
149.9	23.5																	
149.4	24.0			1	B	V	C	RP SP	T	0 to 1				#2 TCR 100 SCR 100	16	NQ		
148.9	24.5		End of Core Log															
148.4	25.0		Highly fractured zones at: 21.4-21.7m; 22.4-22.7m; 23.0-23.3m.															
147.9	25.5																	
147.4	26.0																	
146.9	26.5																	

Remarks:

LEGEND:



Dolostone
Cobbles

1 OF 2

METRIC

Continued Next Page

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

○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No W12

2 OF 2

METRIC

W.P. 280-99-00 LOCATION Coords: N:4765274.6 E:327168.7 ORIGINATED BY PK
DIST HWY 406 BOREHOLE TYPE Hollow Stem Augers / Casing and Washboring / NQ Rock Coring COMPILED BY DB
DATUM Geodetic DATE 11.24.09 - 11.25.09 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/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)					
								20 40 60 80 100	20 40 60 80 100	W _p W W _L					
						○ UNCONFINED + FIELD VANE									
						● QUICK TRIAXIAL × LAB VANE									
15.0	SILTY CLAY trace sand, trace gravel, soft to firm, grey, damp to moist (GLACIAL TILL)		4	SS	3									Nov.24	
			5	SS	7										Nov.25 0 1 83 16
156.7 16.7	SILT AND SAND trace clay, trace gravel, compact to dense, grey, wet (GLACIAL TILL)		6	SS	18									7 42 44 7	
			7	SS	39										
			8	SS	100/ 10cm										
153.9 19.5 153.5 19.9	SILTY CLAY - trace to some sand, trace gravel, hard, grey, damp (GLACIAL TILL)		1	RUN	NQ									RUN#1 TCR=26% SCR=26%	
	COBBLES some gravel, inferred very dense, grey, wet														
152.0 21.4	DOLOSTONE BEDROCK Light to dark grey, slightly weathered, medium bedded, medium strength, with occasional dark grey, moderately weathered, low to medium strength shale seams.		2	RUN	NQ									RUN#2 TCR=99% SCR=75% RQD=0%	
			3	RUN	NQ										RUN#3 TCR=100% SCR=89% RQD=32%
			4	RUN	NQ										RUN#4 TCR=71% SCR=63% RQD=24%
149.0 24.4	End of Borehole Borehole sealed with bentonite slurry from 24.4m to 12.6m.														

CORE LOG



Terraprobe

Project Highway 406 Twinning				Orientation Vertical		Ground Elevation 173.4m		Datum Geodetic		Borehole No. W12	
Location Welland, Ontario				Date Started November 25, 2009		Completed November 25, 2009		Logged By B. Ripley		Sheet 1 of 1	
W.P.: 280-99-00				Drilling Agency DDSI		Drill Type Bombardier		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
153.9	19.5		Overburden, see Borehole Log W12															
153.6	19.8		Overburden, SILTY CLAY TILL															
153.5	19.9		see Borehole Log W12															
153.4	20.0		Overburden,															
			COBBLES															
			some gravel,															
152.9	20.5		see Borehole Log W12															
152.4	21.0																	
152.0	21.4																	
151.9	21.5		SALINA FORMATION BEDROCK															
			Dolostone (>99%)	1	B	V	C	RP	T	0 to 1								
			Occasional to frequent shale seams and partings, light to medium grey to buff, moderately to highly fractured, medium bedded, medium strength, slightly weathered.															
151.4	22.0			1	B	V	C	RP	T	0 to 1								
150.9	22.5		Shale (<1%)															
			Moderately weathered, medium to dark grey, low to medium strength, fissile locally.															
150.4	23.0																	
149.9	23.5			1	B	V	C	RP	T	0 to 1								
149.4	24.0																	
149.0	24.4																	
148.9	24.5		End of Core Log															
			Highly fractured zones of:															
			22.8-23.6m;															
148.4	25.0																	
147.9	25.5																	

Remarks:

LEGEND:

Dolostone

Silty Clay Till

Cobbles

RECORD OF BOREHOLE No W13

1 OF 2

METRIC

W.P. 280-99-00 LOCATION Coords: N:4765319.2 E:327146.7 ORIGINATED BY PK
DIST HWY 406 BOREHOLE TYPE Hollow Stem Augers / Casing and Washboring / NQ Rock Coring COMPILED BY DB
DATUM Geodetic DATE 11.25.09 - 11.26.09 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	20 40 60 80 100					
174.6	Ground Surface													
174.4	200mm FILL - Sand and Gravel, trace to some silt, trace clay, grey, damp		1	SS	11		174							
0.2														
173.9	SILTY CLAY - trace sand, stiff, brown, damp		2	SS	33		173							0 1 92 7
0.7														
	SILT trace clay, trace sand, compact to dense, brown, wet		3	SS	24		172							
172.5														
2.1	SILTY CLAY trace sand, trace gravel, firm to stiff, grey, damp to moist (GLACIAL TILL)		4	SS	7		171							
			5	SS	10		170							0 3 72 25
			6	SS	5		169							
			7	TW	PH		168						21.3	1 3 71 25
			8	SS	8		167							1 2 68 29
			9	SS	10		166							
							165							
			10	SS	5		164							0 5 71 24
							163							
			11	SS	29		162							
			12	SS	30		161							5 7 66 22
159.6							160							

very stiff to hard

Continued Next Page

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

RECORD OF BOREHOLE No W13

2 OF 2

METRIC

W.P. 280-99-00 LOCATION Coords: N:4765319.2 E:327146.7 ORIGINATED BY PK
 DIST HWY 406 BOREHOLE TYPE Hollow Stem Augers / Casing and Washboring / NQ Rock Coring COMPILED BY OB
 DATUM Geodetic DATE 11.25.09 - 11.26.09 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			SHEAR STRENGTH kPa									
								20 40 60 80 100									
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE									
							20 40 60 80 100					WATER CONTENT (%)					
												10 20 30					
15.0	SILTY CLAY trace sand, trace gravel, hard, grey, damp to moist (GLACIAL TILL)		13	SS	33		159										
158.4							158										
16.2	SAND AND SILT trace to some gravel, trace clay, compact to very dense, grey, moist to wet (GLACIAL TILL)		14	SS	22		157										
			15	SS	54		156										12 46 37 5
			16	SS	56		155										
			17	SS	35/ 10cm NQ		154										Nov.25 Nov.26 RUN#1 TCR=42% SCR=48% RQD=0%
153.2	DOLOSTONE BEDROCK Light to dark grey, slightly weathered, medium bedded, medium strength, with occasional dark grey, moderately weathered, low to medium strength shale seams.		1	RUN	NQ		153										RUN#2 TCR=77% SCR=72% RQD=18%
21.4		2	RUN	NQ		152											
		3	RUN	NQ		151										RUN#3 TCR=93% SCR=90% RQD=77%	
149.9	End of Borehole					150											
24.7	Borehole filled with drill water on completion of coring. Unable to push vane to 10.5m. Resistance to augering at 14.6m. Borehole sealed with bentonite slurry to ground surface.																

ONTARIO MOT 1-09-4135 WELLAND CANAL.GPJ ONTARIO MOT.GDT 04/28/10

CORE LOG



Terraprobe

Project	Highway 406 Twinning	Orientation	Vertical	Ground Elevation	174.6m	Datum	Geodetic	Borehole No.	W13
Location	Welland, Ontario	Date Started	November 26, 2009	Completed	November 26, 2009	Logged By	B. Ripley	Sheet	1 of 1
W.P.:	280-99-00	Drilling Agency	DBW Drilling	Drill Type	Bombardier	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/m ³)
				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
154.1	20.5																	
153.6	21.0		Overburden, see Borehole Log W13															
153.3	21.3		Overburden, SAND AND SILT TILL															
153.2	21.4		see Borehole Log W13															
153.1	21.5																	
			SALINA FORMATION BEDROCK															
152.6	22.0		Dolostone (99%)															
			Occasional to frequent shale seams and partings, light to medium grey, moderately to highly fractured, medium bedded, medium strength, slightly weathered, poorly fossiliferous.															
152.1	22.5			1	B	V	C	RP	T	0 to 1								
151.6	23.0		Shale (1%)															
			Moderately weathered, medium to dark grey, low to medium strength.															
151.1	23.5																	
150.6	24.0			1	B	V	C	RP	T	0 to 1								
150.1	24.5																	
149.9	24.7		End of Core Log															
149.6	25.0																	
			Highly fractured zones at:															
149.1	25.5		22.3-22.4m; 22.5-22.6m; 23.0-23.1m.															
148.6	26.0																	
148.1	26.5																	

Remarks:

LEGEND:

	Dolostone
	SAND AND SILT TILL

RECORD OF BOREHOLE No W14

1 OF 2

METRIC

W.P. 280-99-00 LOCATION Coords: N:4765327.6 E:327158.2 ORIGINATED BY LY
DIST HWY 406 BOREHOLE TYPE Hollow Stem Augers / Casing and Washboring / NQ Rock Coring COMPILED BY DB
DATUM Geodetic DATE 11.23.09 - 11.24.09 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			SHEAR STRENGTH kPa						
174.8	Ground Surface							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						
0.0	FILL - Silty Clay, sandy, some gravel, stiff, brown, dry to damp		1	SS	13									
174.1														
0.7	SILTY CLAY trace sand, firm to stiff, brown, damp to moist		2	SS	8		174							0 1 71 28
			3	SS	8		173							
172.7														
2.1	SILT trace clay, trace sand, loose, brown, wet		4	SS	8		172							
171.9														
2.9	SILTY CLAY trace sand, trace gravel, firm to stiff, brown, damp to moist (GLACIAL TILL)		5	SS	8		171							
			6	SS	8		170							
			7	SS	7		169							
			8	SS	8		168							
			9	TW	PH		167							
			10	SS	11		166							
	</													

Continued Next Page

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

ONTARIO MOT 1-09-4135 WELLAND CANAL GPJ ONTARIO MOT.GDT 03/22/10

RECORD OF BOREHOLE No W14

2 OF 2

METRIC

W.P. 280-99-00 LOCATION Coords: N:4765327.6 E:327158.2 ORIGINATED BY LY
DIST HWY 406 BOREHOLE TYPE Hollow Stem Augers / Casing and Washboring / NQ Rock Coring COMPILED BY DB
DATUM Geodetic DATE 11.23.09 - 11.24.09 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									WATER CONTENT (%)		
								20	40	60	80						100	10	20
15.0	SILTY CLAY trace sand, trace gravel, very stiff, grey, damp to moist (GLACIAL TILL)		13	SS	16														
159.6																			
16.2	SAND AND SILT trace gravel, trace clay, compact to dense, grey, moist to wet (GLACIAL TILL)		14	SS	22														
			15	SS	37														
			16	SS	49														
154.0	SAND AND GRAVEL occasional cobbles, very dense, grey, wet		17	SS	71														
20.8																			
			18	SS	100/ 8cm														
152.9																			
21.9	DOLOSTONE BEDROCK Light to dark grey, slightly weathered, medium bedded, medium strength, with occasional dark grey, moderately weathered, low to medium strength shale seams.		1	RUN	NQ														
			2	RUN	NQ														

ONTARIO MOT 1-09-4135 WELLAND CANAL.GPJ ONTARIO MOT.GDT 03/22/10

CORE LOG



Terraprobe

Project Highway 406 Twinning				Orientation Vertical		Ground Elevation 174.8m		Datum Geodetic		Borehole No. W14	
Location Welland, Ontario				Date Started November 24, 2009		Completed November 24, 2009		Logged By B. Ripley		Sheet 1 of 1	
W.P.: 280-99-00				Drilling Agency DBW Drilling		Drill Type Bombardier		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/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
153.3	21.5		Overburden, see Borehole Log W14															
153.0	21.8		Overburden, SAND AND GRAVEL see Borehole Log W14															
152.9	21.9																	
152.8	22.0		SALINA FORMATION BEDROCK															
			Dolostone (>99%) Occasional shale seams and partings, light to medium grey, moderately to highly fractured, medium bedded, medium strength, slightly weathered, poorly fossiliferous.	1	B	V	C	RP SP	T	0 to 1							NQ	
152.3	22.5																	
151.8	23.0																	
			Shale (<1%) Moderately weathered, medium to dark grey, low to medium strength.															
151.3	23.5																	
150.8	24.0			1	B	V	C	RP SP	T	0 to 1							NQ	
150.3	24.5																	
150.0	24.9																	
149.8	25.0		End of Core Log															
			Highly fractured zones at: 21.9-21.95m; 22.6-22.7m.															
149.3	25.5																	
148.8	26.0																	
148.3	26.5																	
147.8	27.0																	
147.3	27.5																	

Remarks:

LEGEND:

Dolostone

Sand and Gravel

RECORD OF BOREHOLE No W15

1 OF 2

METRIC

W.P. 280-99-00 LOCATION Coords: N:4765346.7 E:327144.9 ORIGINATED BY LY
DIST HWY 406 BOREHOLE TYPE Hollow Stem Augers COMPILED BY DB
DATUM Geodetic DATE 11.26.09 - 11.27.09 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	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100					
179.2 0.0	Ground Surface													
	firm		1	SS	5		179							

	FILL - Silty Clay, trace sand, trace gravel, stiff to very stiff, brown, moist		2	SS	10		178							
			3	SS	9									1 8 49 42
			4	SS	25		177							
			5	SS	26		176							
175.5 3.7	SILTY CLAY trace sand, very stiff, brown, damp to moist		6	SS	19		175							
			7	SS	17									
174.0 5.2	SILT trace clay, trace sand, compact, grey, wet		8	SS	29		174							
			9	SS	19		173							0 1 89 10 Nov.26 Nov.27
172.2 7.0	SILTY CLAY trace sand, trace gravel, firm to stiff, hard below 14.8m, grey, damp to moist (GLACIAL TILL)		10	SS	6		172							1 3 64 32
			11	TW	PH		171							
							170						21.1	
			12	SS	8		169							
							168							
			13	SS	10		167							0 2 73 25
							166							
			14	SS	8		165							
164.2														

Continued Next Page

+ 3. X 3. Numbers refer to
Sensitivity

○ 3% STRAIN AT FAILURE



ONTARIO MOT 1-09-4135 WELLAND CANAL GPJ ONTARIO MOT.GDT 03/22/10

RECORD OF BOREHOLE No W15

2 OF 2

METRIC

W.P. 280-99-00 LOCATION Coords: N:4765346.7 E:327144.9 ORIGINATED BY LY
 DIST HWY 406 BOREHOLE TYPE Hollow Stem Augers COMPILED BY DB
 DATUM Geodetic DATE 11.26.09 - 11.27.09 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							
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE							
								20 40 60 80 100							
15.0	SILTY CLAY - sandy, some gravel, hard, grey, damp (GLACIAL TILL)		15	SS	31		164								
163.5															
15.7	End of Borehole														
	Unable to push vane to 15.0m.														
	Piezometer installation consists of a 19mm diameter, Schedule 40 PVC pipe with a 1.52m slotted screen.														
	Water Level Readings:														
	Date Depth(m) Elevation(m)														
	Nov.30.09 4.1 175.1														
	Dec.08.09 4.2 175.0														
	Dec.16.09 4.2 175.0														
	Jan.04.10 4.2 175.0														

RECORD OF BOREHOLE No W16

1 OF 2

METRIC

W.P. 280-99-00 LOCATION Coords: N:4765364.8 E:327139.5 ORIGINATED BY PK
 DIST HWY 406 BOREHOLE TYPE Hollow Stem Augers COMPILED BY DB
 DATUM Geodetic DATE 11.18.09 - 11.19.09 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			SHEAR STRENGTH kPa						
182.9	Ground Surface							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						GR SA SI CL
0.0	FILL - Silty Clay, some sand, trace gravel, frequent cobbles to 4.5m, firm to hard, damp to moist		1	SS	6						○			
			2	SS	17		182				○			
			3	SS	22		181			○			9 16 47 28	
			4	SS	31		180				○			
			5	SS	26		179						4 17 50 29	
	----- frequent gravel inclusions -----		6	SS	15		178			○				
	----- trace wood pieces -----		7	SS	13		177				○			
	----- topsoil stained -----		9	SS	9		176					○		
175.9 7.0	SILTY CLAY trace sand, very stiff, brown, moist		10	SS	20		175				○		0 1 65 34	
174.3 8.6	SILT trace clay, trace sand, dense, brown, wet		11	SS	36		174				○		sampler wet	
172.6 10.3	SILTY CLAY trace sand, trace gravel, firm to very stiff, grey, moist (GLACIAL TILL)		12	SS	7		173							
			13	SS	5		172				○			
							171		1.6				Nov.18 Nov.19	
							170		1.1				2 4 65 29	
			14	SS	11		169			1.7			22.5	
							168		2.0		○			

Continued Next Page

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

ONTARIO MOT 1-09-4135 WELLAND CANAL.GPJ ONTARIO MOT.GDT 03/22/10

RECORD OF BOREHOLE No W16

2 OF 2

METRIC

W.P. 280-99-00 LOCATION Coords: N:4765364.8 E:327139.5 ORIGINATED BY PK
 DIST HWY 406 BOREHOLE TYPE Hollow Stem Augers COMPILED BY DB
 DATUM Geodetic DATE 11.18.09 - 11.19.09 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										WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X LAB VANE												
15.0	SILTY CLAY trace sand, trace gravel, stiff to very stiff, grey, moist (GLACIAL TILL)		15	SS	17		167									20.9	0 1 68 31			
					16		SS	17												
					17		TW	PH												
163.2																				
19.7	End of Borehole																			
	Resistance to augering at 14.9m. Piezometer installation consists of a 19mm diameter, Schedule 40 PVC pipe with a 1.52m slotted screen. Water Level Readings: Date Depth(m) Elevation(m) Nov.30.09 7.8 175.1 Dec.08.09 7.7 175.2 Jan.04.10 7.5 175.4 Jan.14.10 7.5 175.4																			

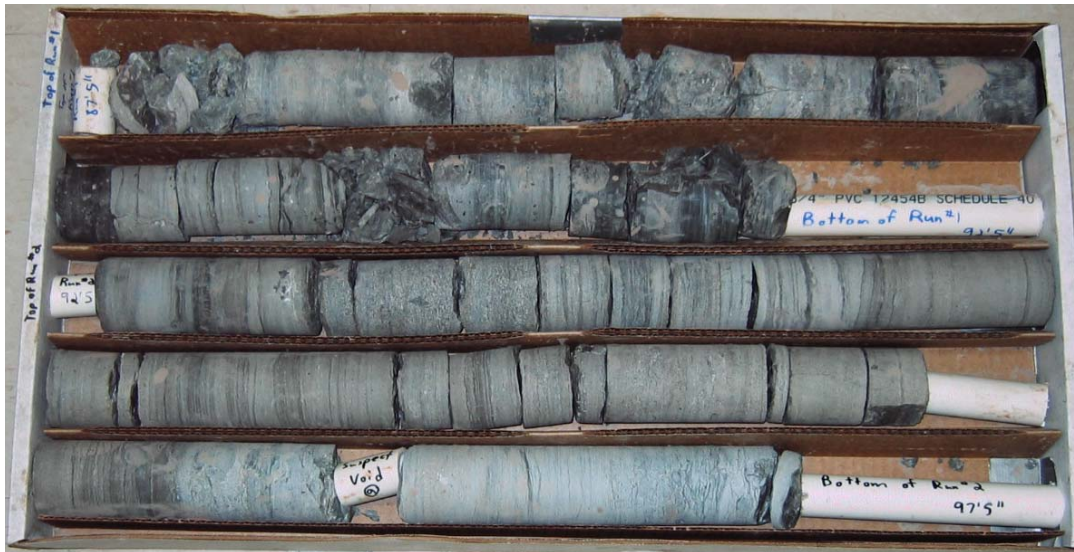
Foundation Investigation Report
Highway 406 Twinning – Port Robinson Road to East Main Street
Agreement No. 2008-E-0016; W.P. 280-99-00



Bedrock Core Sample
Borehole: W3
Runs: 1, 2 & 3
Depth: 26.8 m – 30.9 m



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: W4

Runs: 1 & 2

Depth: 26.6 m – 29.7 m



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: W5
Runs: 1, 2 & 3
Depth: 21.3 m – 25.3 m



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: W6
Runs: 1, 2 & 3
Depth: 25.1 m – 29.4 m



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: W7

Runs: 1, 2 & 3

Depth: 25.2 m – 28.2 m



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: W8
Runs: 1, 2 & 3
Depth: 25.3 m – 29.5 m



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: W9

Runs: 1 & 2

Depth: 25.2 m – 28.1 m



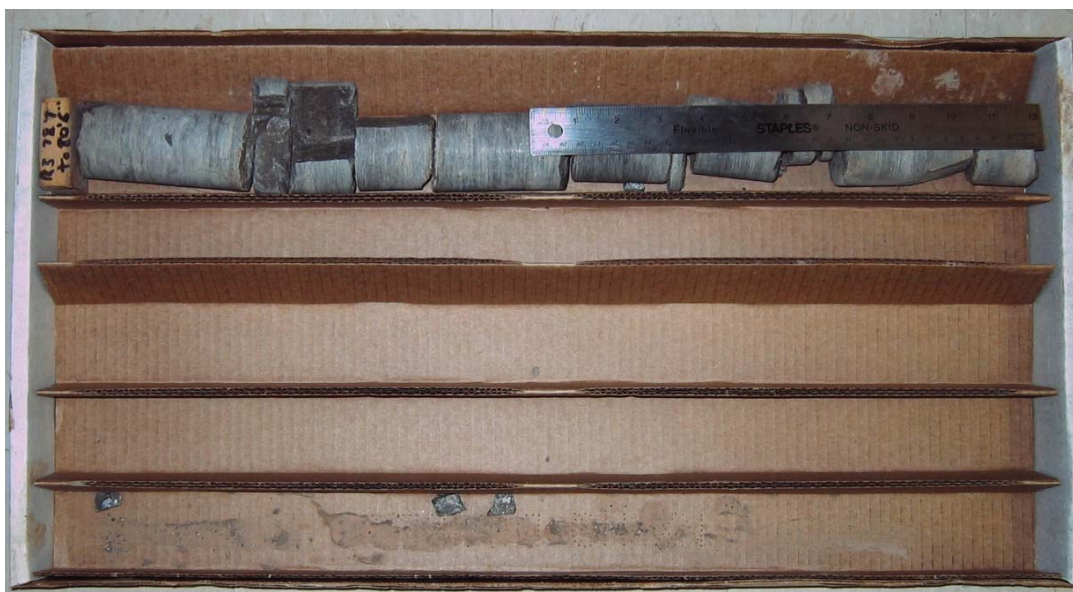
Foundation Investigation Report
Highway 406 Twinning – Port Robinson Road to East Main Street
Agreement No. 2008-E-0016; W.P. 280-99-00



Bedrock Core Sample
Borehole: W10
Runs: 1 & 2
Depth: 22.1 m – 25.1 m



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: W11
Runs: 1, 2 & 3
Depth: 21.2 m – 24.5 m



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: W12

Runs: 1, 2, 3 & 4

Depth: 19.8 m – 24.4 m



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: W13

Runs: 1, 2 & 3

Depth: 21.3 m – 24.9 m



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: W14

Runs: 1 & 2

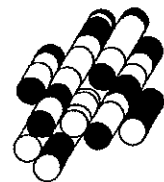
Depth: 21.8 m – 24.8 m



APPENDIX B

Laboratory Test Results

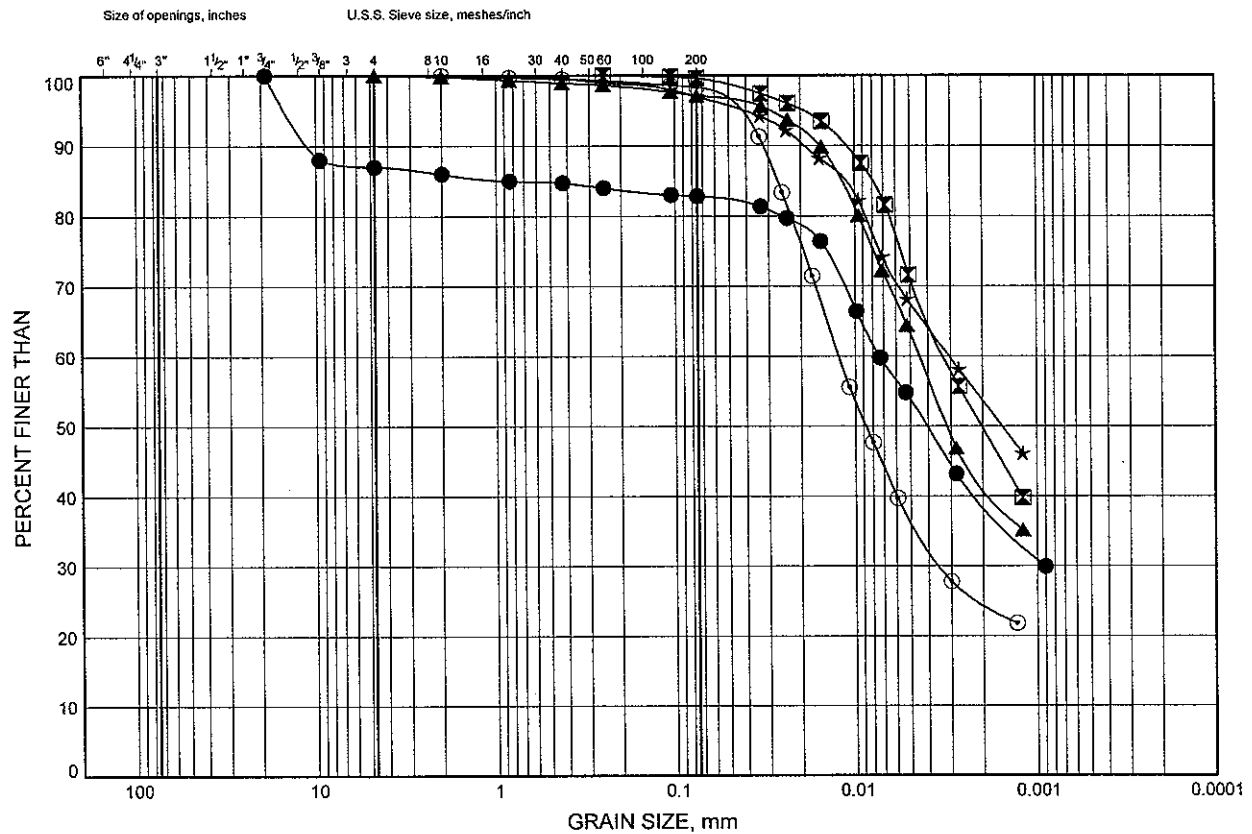
Terraprobe Inc.



GRAIN SIZE DISTRIBUTION

FIGURE B1

FILL - Silty Clay

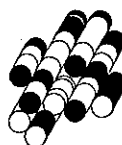


COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	W1	0.3	178.0
☒	W6	1.7	173.9
▲	W6	5.5	170.1
★	W9	1.7	173.9
⊙	W9	7.8	167.8

Date March 2010

Project 1-09-4135



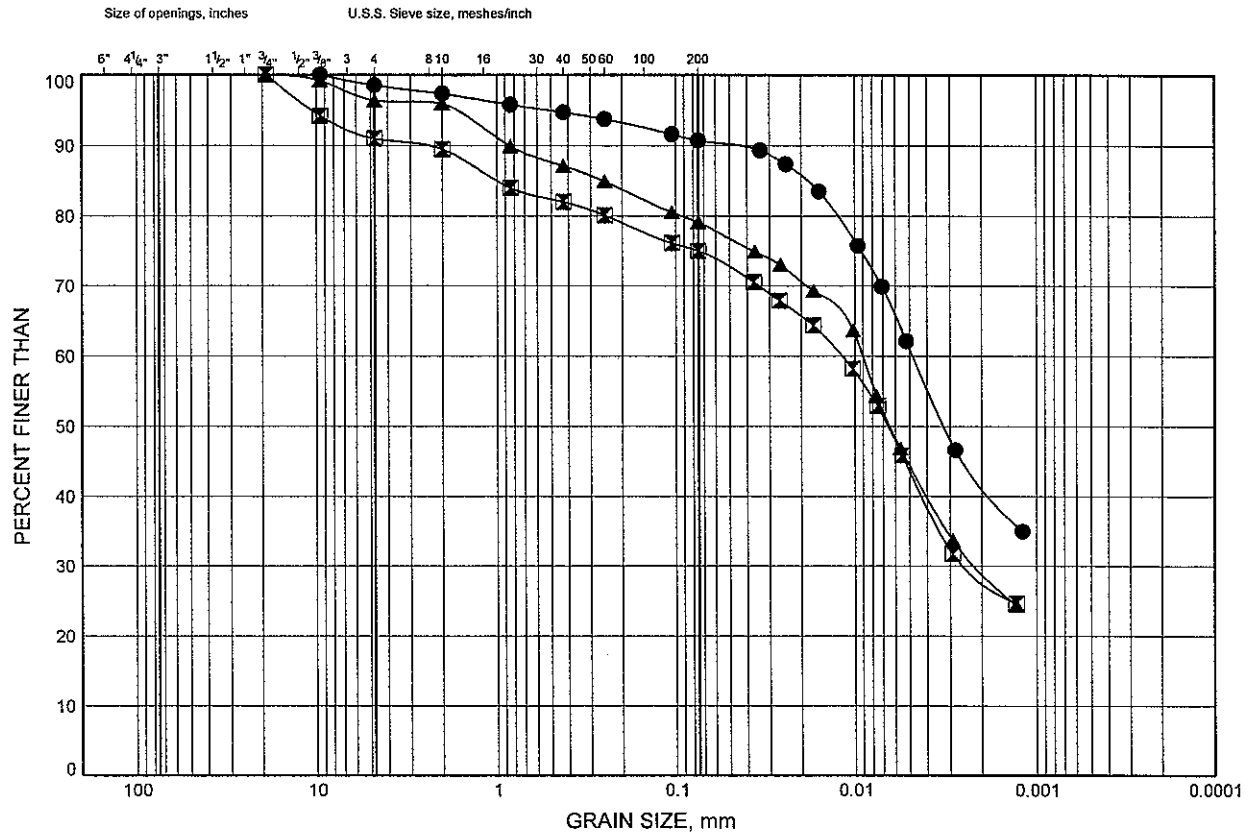
Prep'd DB

Chkd. RA

GRAIN SIZE DISTRIBUTION

FIGURE B2

FILL - Silty Clay



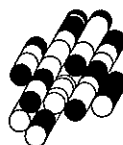
COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

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

●	W15	1.7	177.5
⊠	W16	1.7	181.2
▲	W16	3.2	179.7

Date March 2010

Project 1-09-4135



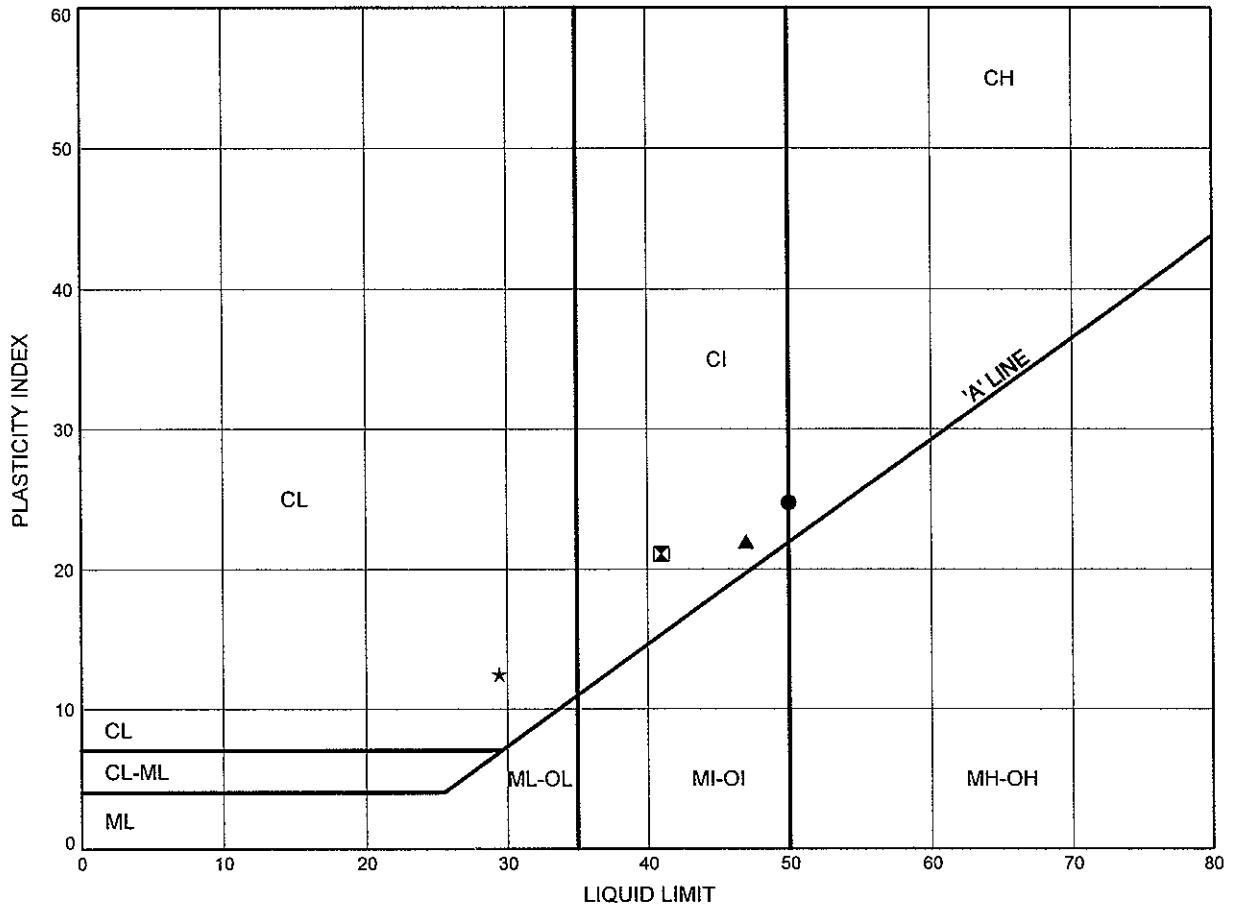
Prep'd DB

Chkd. RA

ATTERBERG LIMITS TEST RESULTS

FIGURE B3

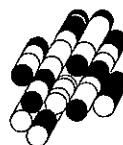
FILL - Silty Clay



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	W1	0.3	178.0
⊠	W6	1.7	173.9
▲	W9	1.7	173.9
★	W9	7.8	167.8

Date March 2010

Project 1-09-4135



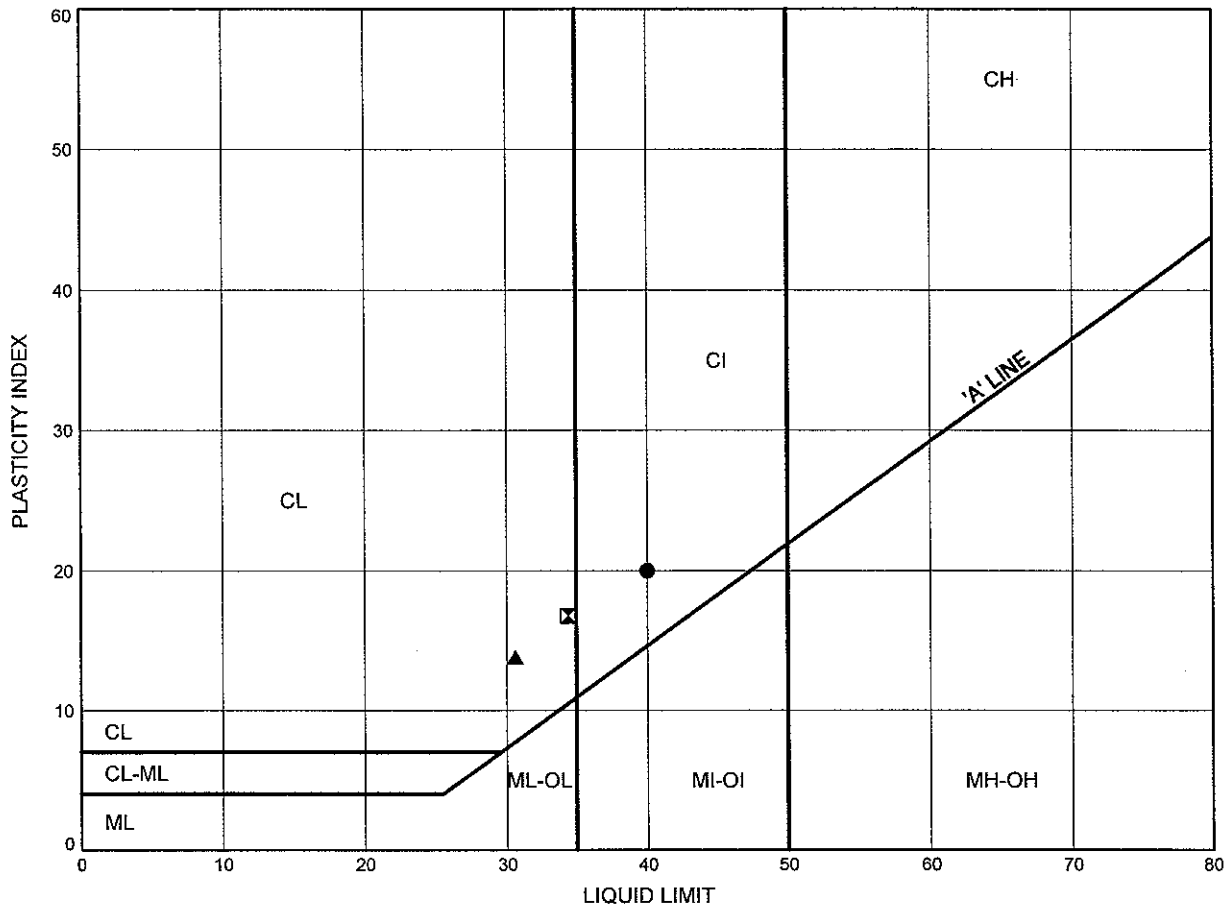
Prep'd DB

Chkd. RA

ATTERBERG LIMITS TEST RESULTS

FIGURE B4

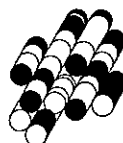
FILL - Silty Clay



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	W15	1.7	177.5
⊠	W16	1.7	181.2
▲	W16	3.2	179.7

Date March 2010

Project 1-09-4135



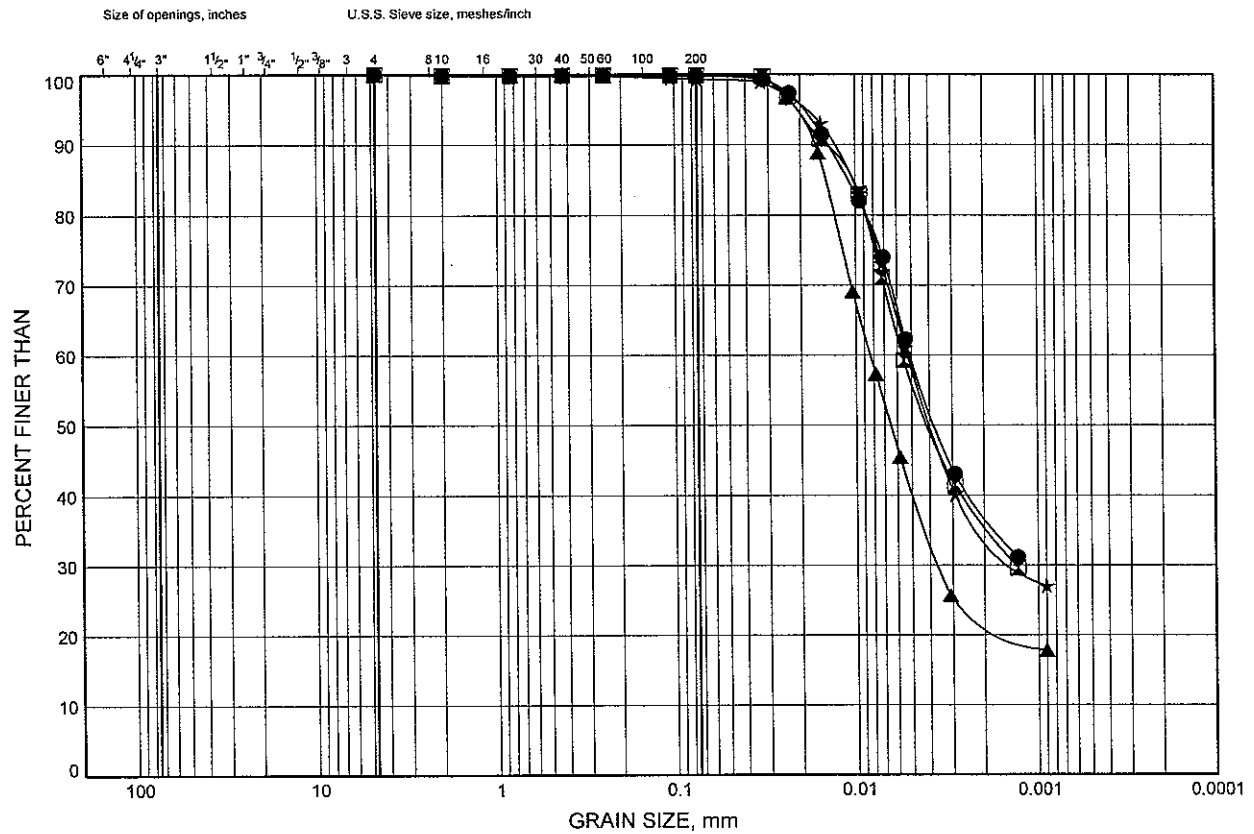
Prep'd DB

Chkd. RA

GRAIN SIZE DISTRIBUTION

FIGURE B5

SILTY CLAY



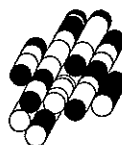
COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL BOREHOLE DEPTH (m) ELEVATION (m)

●	W1	2.5	175.8
⊠	W2	1.7	176.7
▲	W2	3.2	175.2
★	W3	1.7	175.1

Date March 2010

Project 1-09-4135



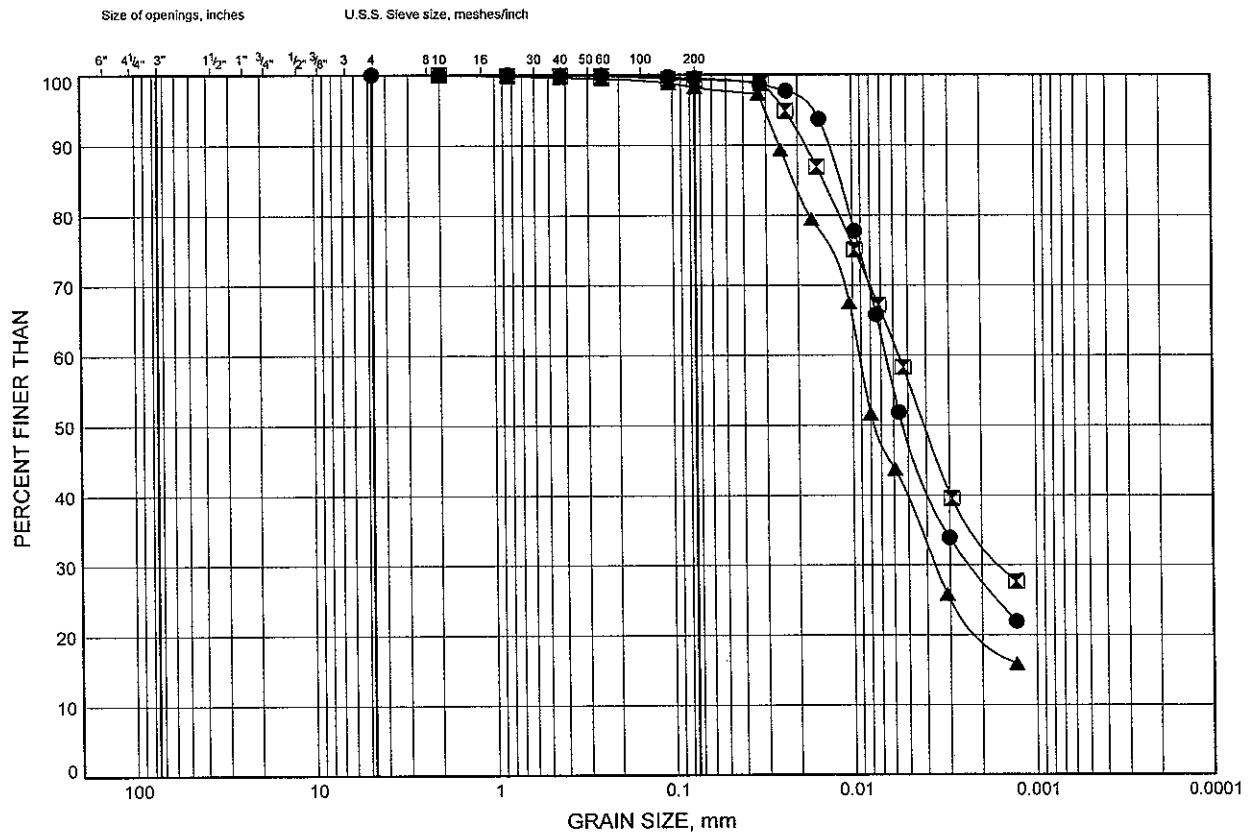
Prep'd DB

Chkd. RA

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)

▲	W5	6.3	164.5
●	W14	1.0	173.8
⊠	W16	7.8	175.1

Date March 2010

Project 1-09-4135



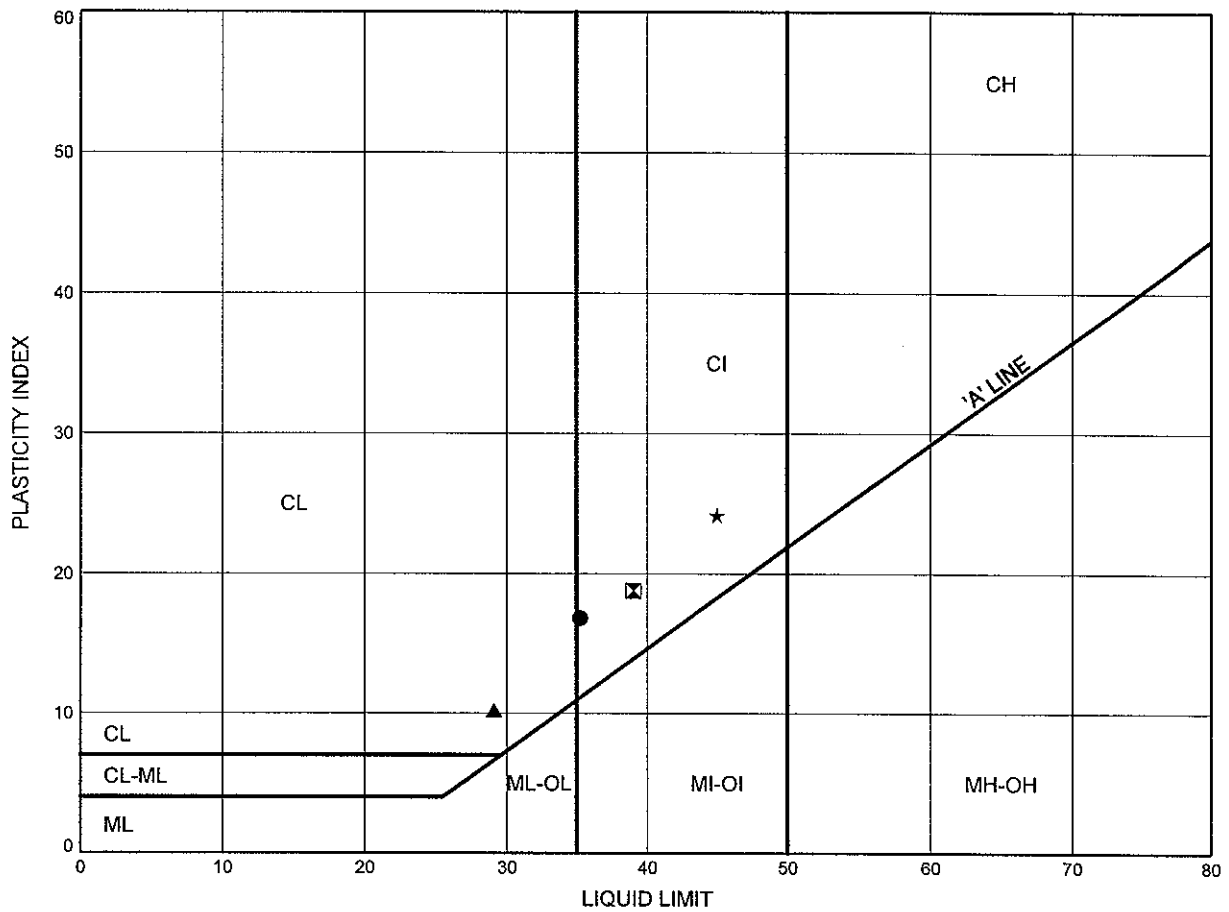
Prep'd DB

Chkd. RA

ATTERBERG LIMITS TEST RESULTS

FIGURE B7

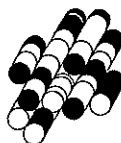
SILTY CLAY



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	W1	2.5	175.8
⊠	W2	1.7	176.7
▲	W2	3.2	175.2
★	W3	1.7	175.1

Date March 2010

Project 1-09-4135



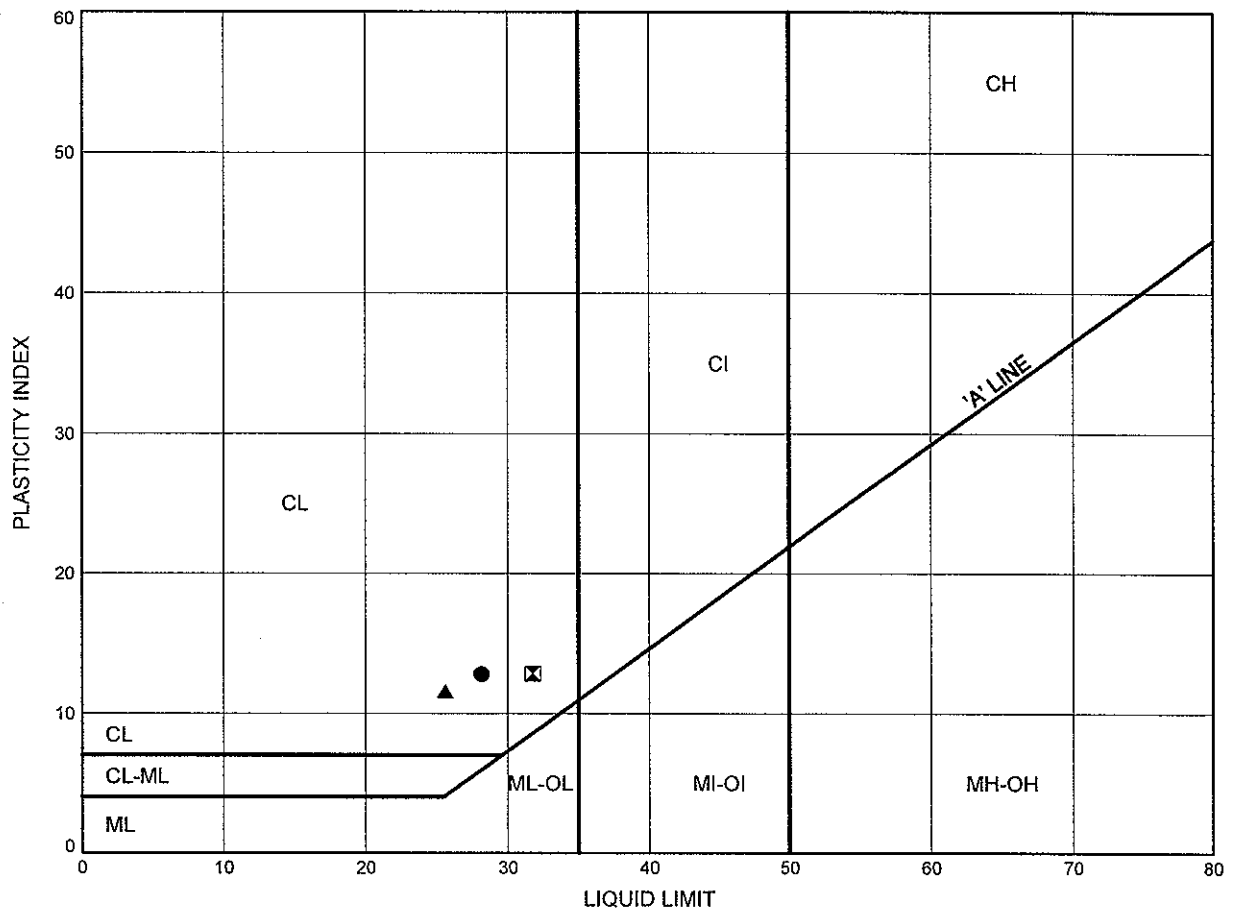
Prep'd DB

Chkd. RA

ATTERBERG LIMITS TEST RESULTS

FIGURE B8

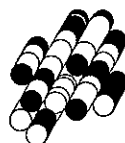
SILTY CLAY



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
▲	W5	6.3	164.5
●	W14	1.0	173.8
⊠	W16	7.8	175.1

Date March 2010

Project 1-09-4135



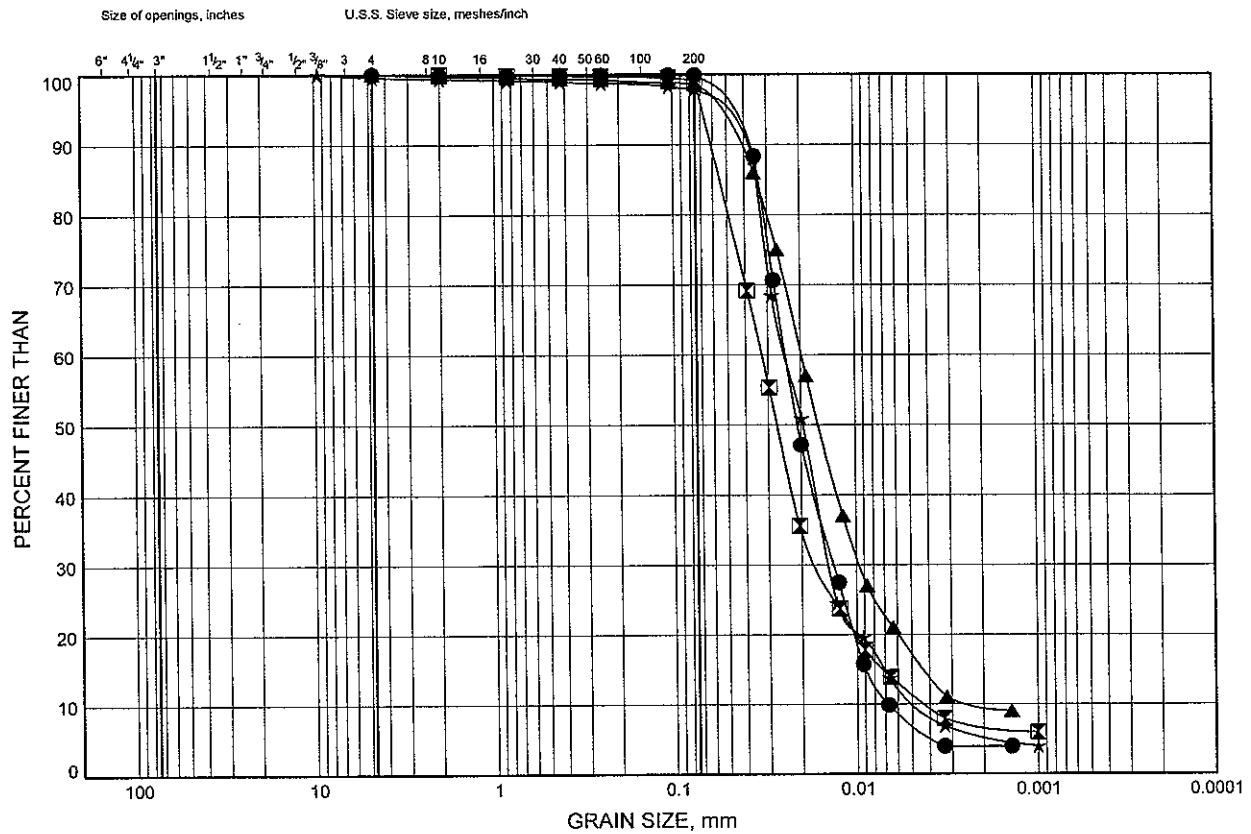
Prep'd DB

Chkd. RA

GRAIN SIZE DISTRIBUTION

FIGURE B9

SILT



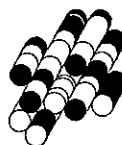
COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL BOREHOLE DEPTH (m) ELEVATION (m)

●	W1	4.7	173.6
★	W3	3.2	173.6
⊠	W13	1.7	172.9
▲	W15	6.3	172.9

Date March 2010

Project 1-09-4135



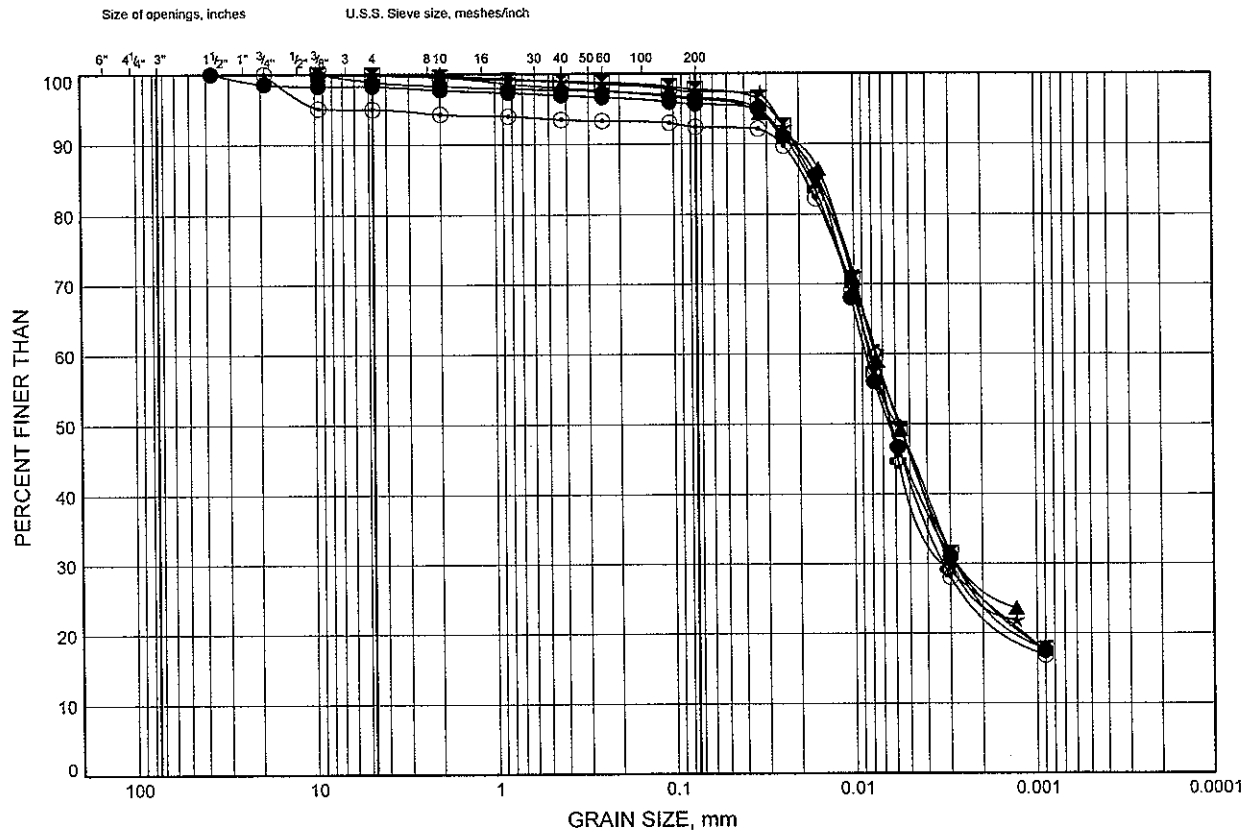
Prep'd DB

Chkd. RA

GRAIN SIZE DISTRIBUTION

FIGURE B10

UPPER SILTY CLAY TILL



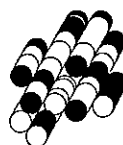
COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL BOREHOLE DEPTH (m) ELEVATION (m)

●	W1	7.8	170.5
⊠	W1	9.3	169.0
▲	W1	10.9	167.4
★	W2	9.3	169.1
⊙	W3	6.3	170.5
⊛	W3	9.3	167.5

Date March 2010

Project 1-09-4135



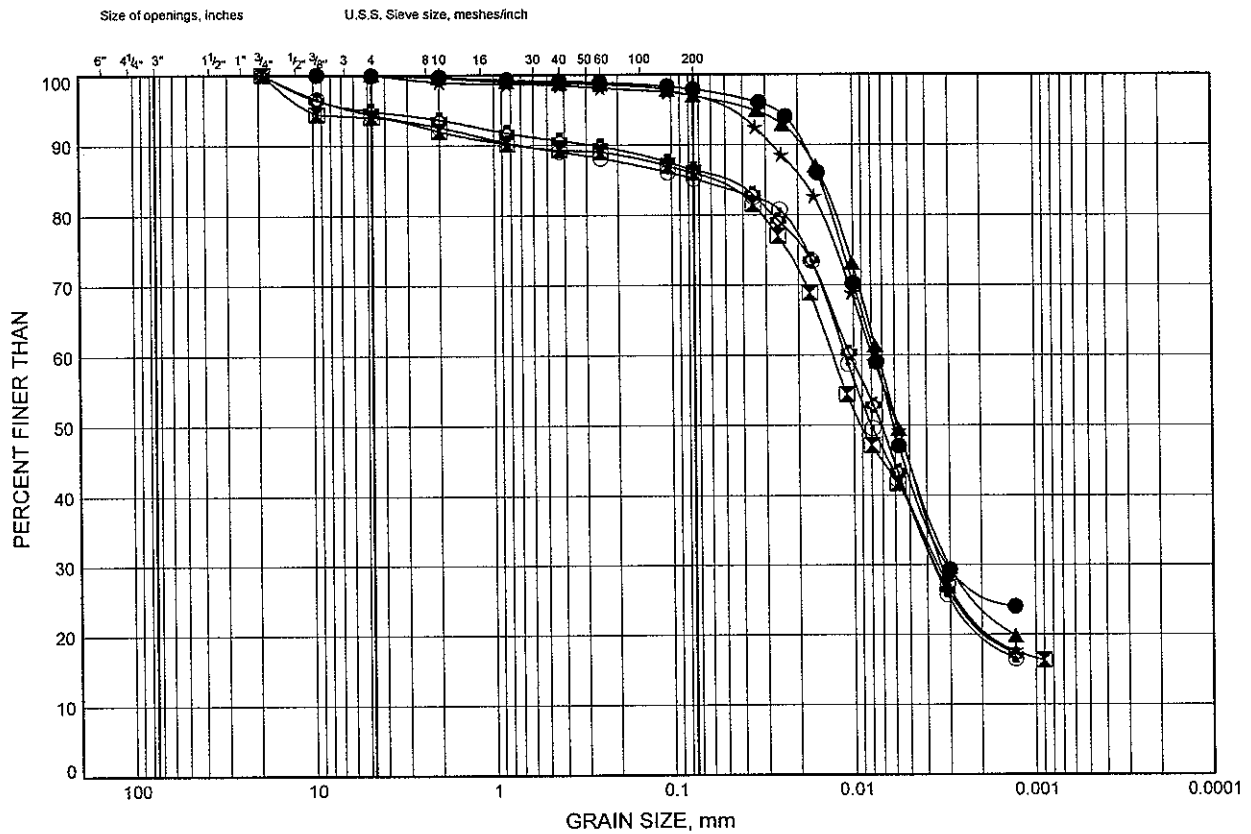
Prep'd DB

Chkd. RA

GRAIN SIZE DISTRIBUTION

FIGURE B11

UPPER SILTY CLAY TILL

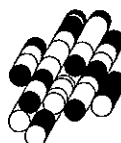


COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL BOREHOLE DEPTH (m) ELEVATION (m)

●	W3	10.9	165.9
⊠	W3	13.9	162.9
▲	W4	9.3	168.3
★	W4	12.4	165.2
⊙	W4	15.4	162.2
⊛	W5	8.4	162.4

Date March 2010
Project 1-09-4135

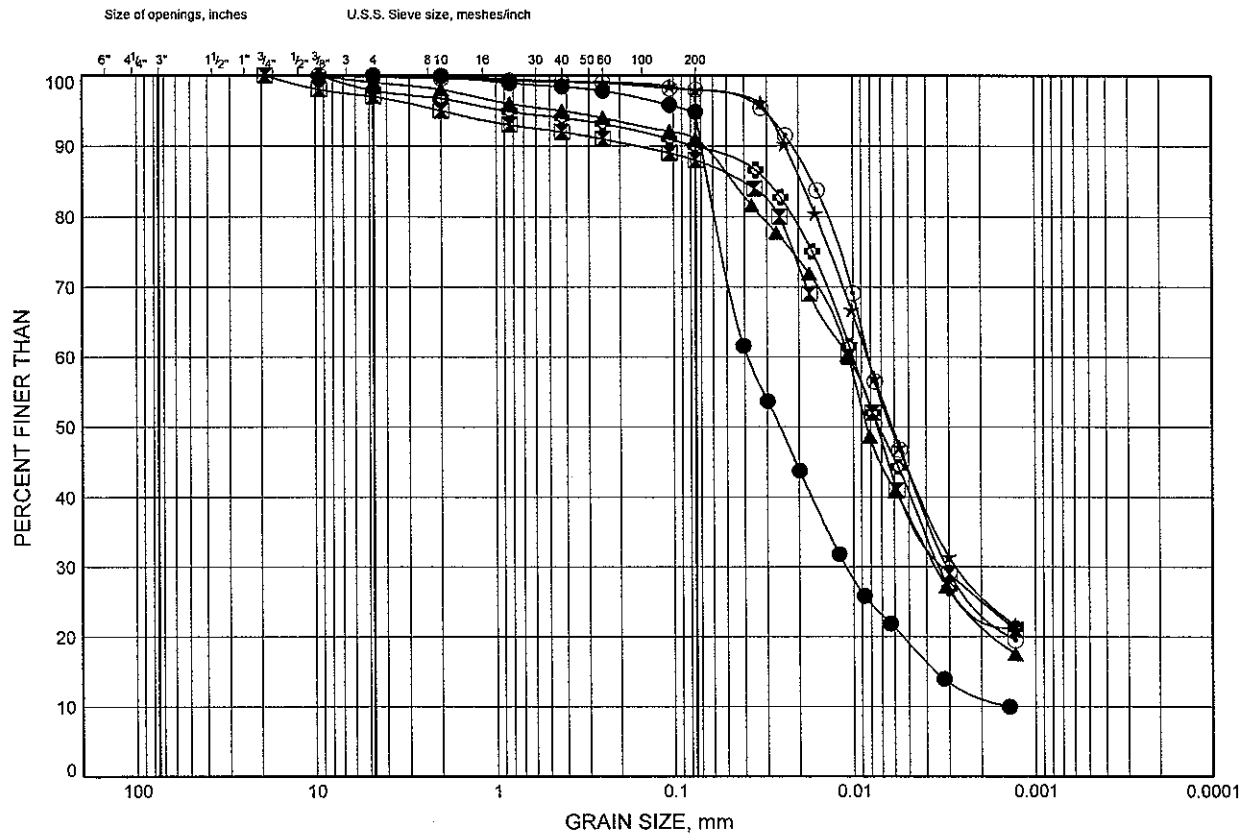


Prep'd DB
Chkd. RA

GRAIN SIZE DISTRIBUTION

FIGURE B12

UPPER SILTY CLAY TILL

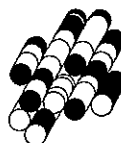


COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	W6	12.4	163.2
⊠	W6	13.9	161.7
▲	W6	15.4	160.2
★	W7	12.4	162.8
⊙	W8	10.9	165.0
⊕	W8	13.9	162.0

Date March 2010

Project 1-09-4135



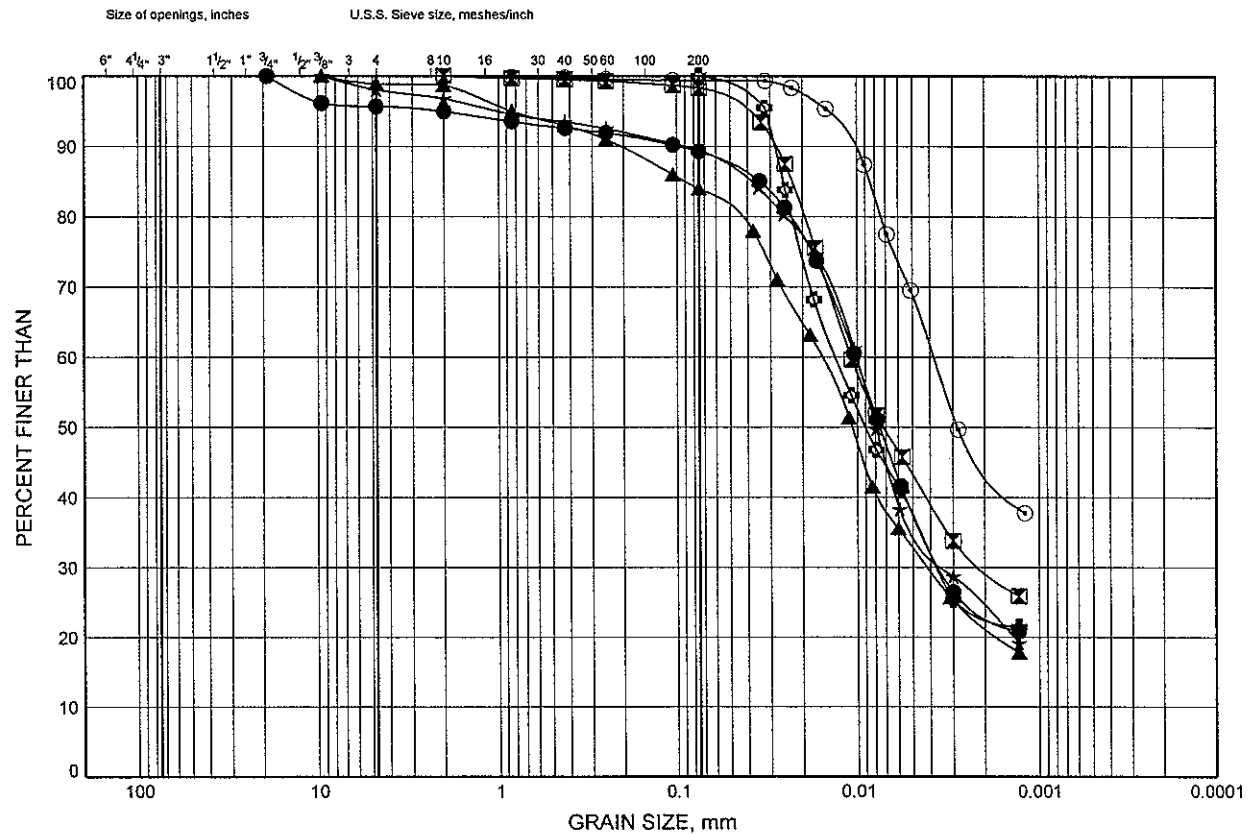
Prep'd DB

Chkd. RA

GRAIN SIZE DISTRIBUTION

FIGURE B13

UPPER SILTY CLAY TILL

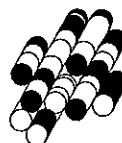


COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
▲	W9	12.4	163.2
★	W9	13.9	161.7
⊙	W9	17.0	158.6
⊕	W9	18.5	157.1
●	W10	12.8	160.5
⊠	W10	15.9	157.4

Date March 2010

Project 1-09-4135



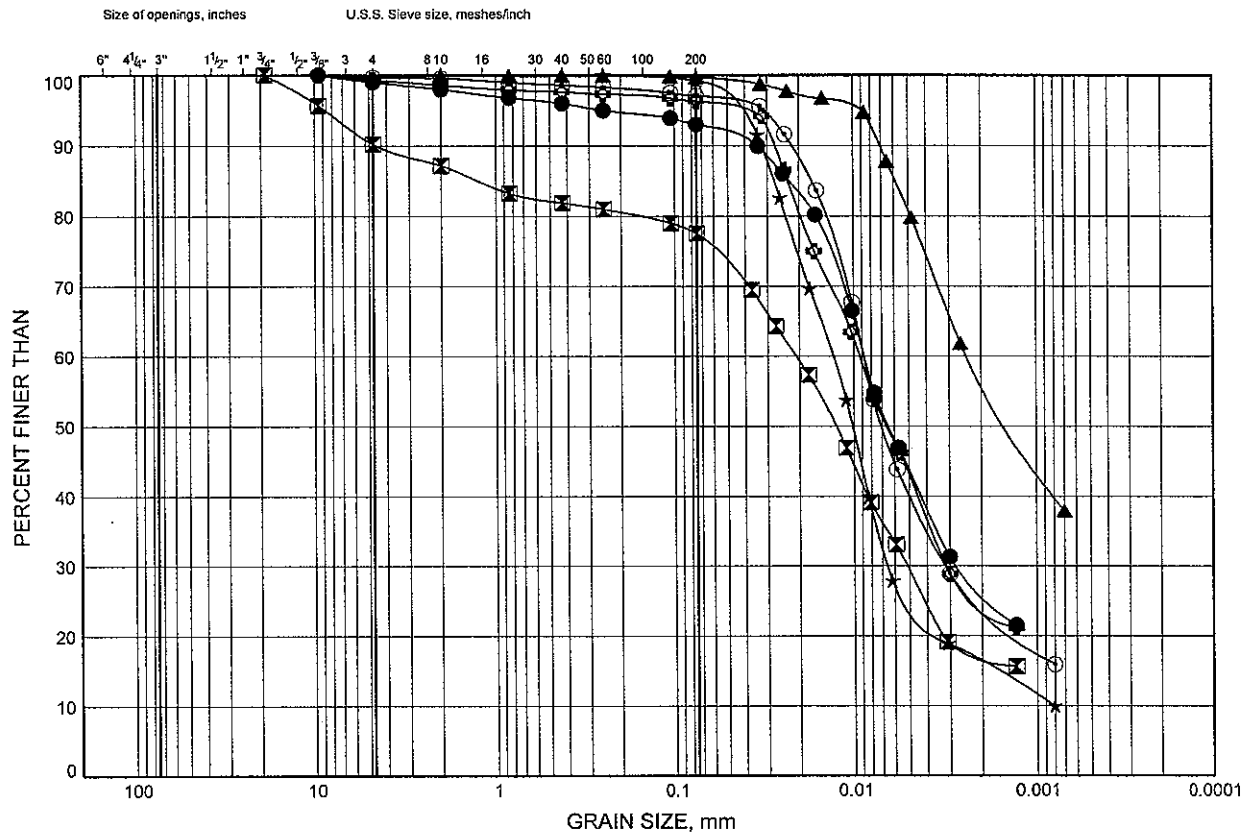
Prep'd DB

Chkd. RA

GRAIN SIZE DISTRIBUTION

FIGURE B14

UPPER SILTY CLAY TILL

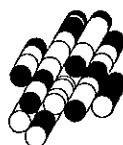


COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	W11	12.2	161.2
⊠	W11	16.0	157.4
▲	W12	14.4	159.0
★	W12	15.9	157.5
⊙	W13	4.0	170.6
⊞	W13	6.3	168.3

Date March 2010

Project 1-09-4135



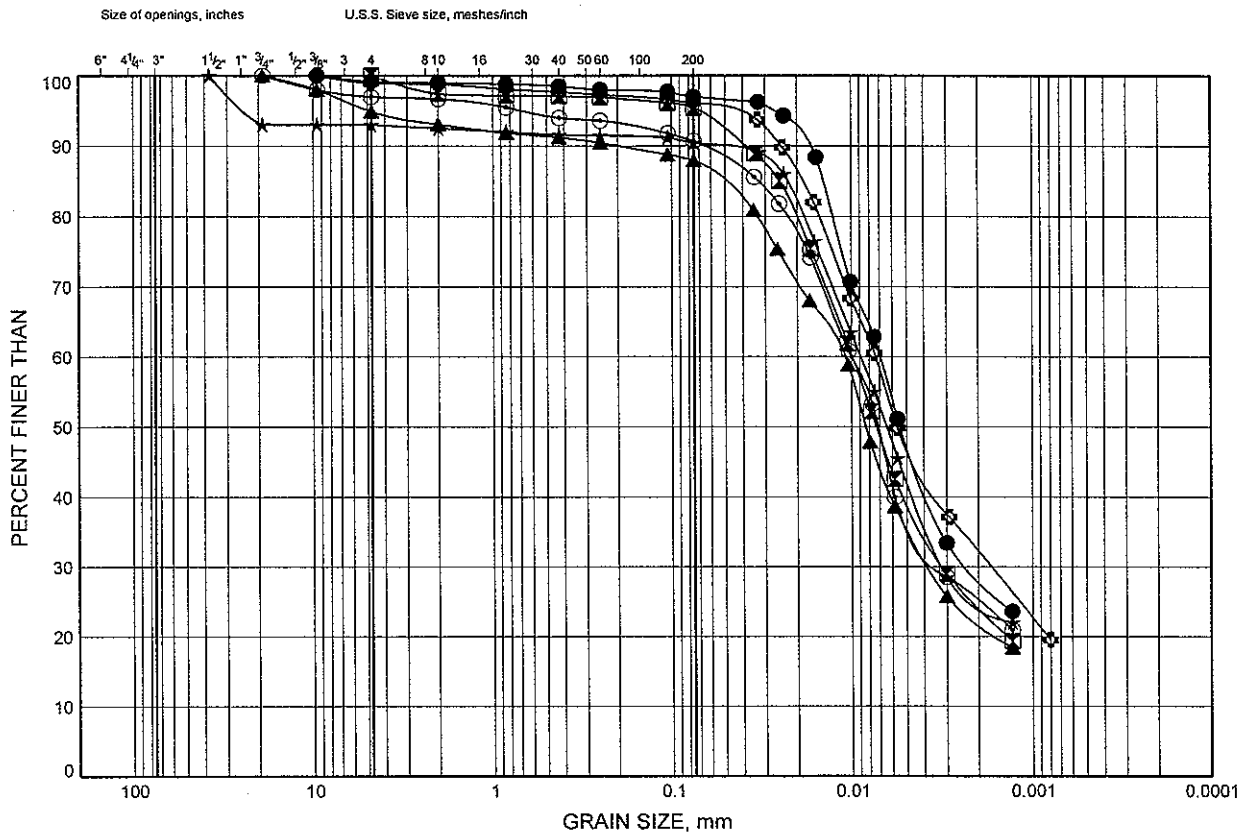
Prep'd DB

Chkd. RA

GRAIN SIZE DISTRIBUTION

FIGURE B15

UPPER SILTY CLAY TILL



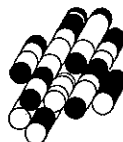
COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL BOREHOLE DEPTH (m) ELEVATION (m)

●	W13	7.8	166.8
⊠	W13	10.9	163.7
▲	W13	13.9	160.7
★	W14	9.3	165.5
⊙	W14	12.4	162.4
⊛	W15	7.8	171.4

Date March 2010

Project 1-09-4135



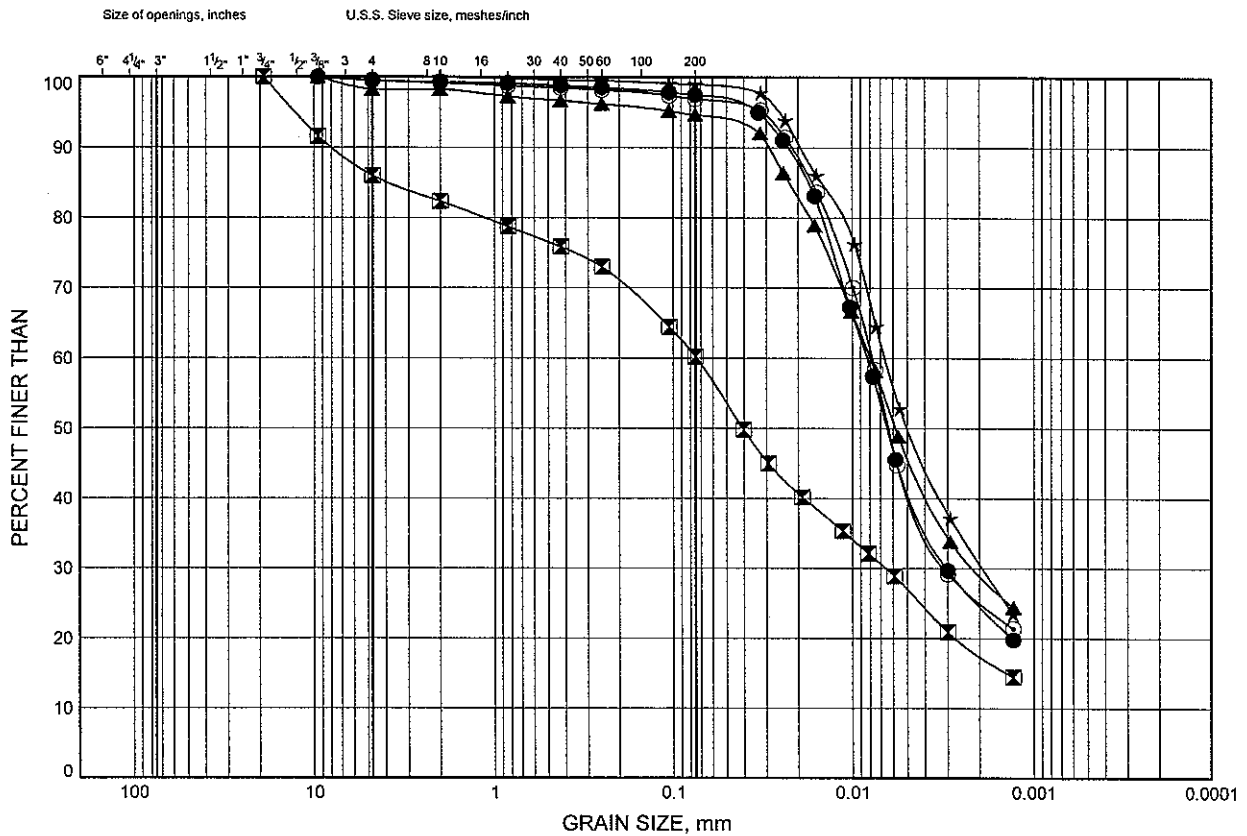
Prep'd DB

Chkd. RA

GRAIN SIZE DISTRIBUTION

FIGURE B16

UPPER SILTY CLAY TILL

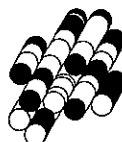


COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	W15	12.4	166.8
⊠	W15	15.4	163.8
▲	W16	12.4	170.5
★	W16	17.0	165.9
⊙	W16	18.5	164.4

Date March 2010

Project 1-09-4135



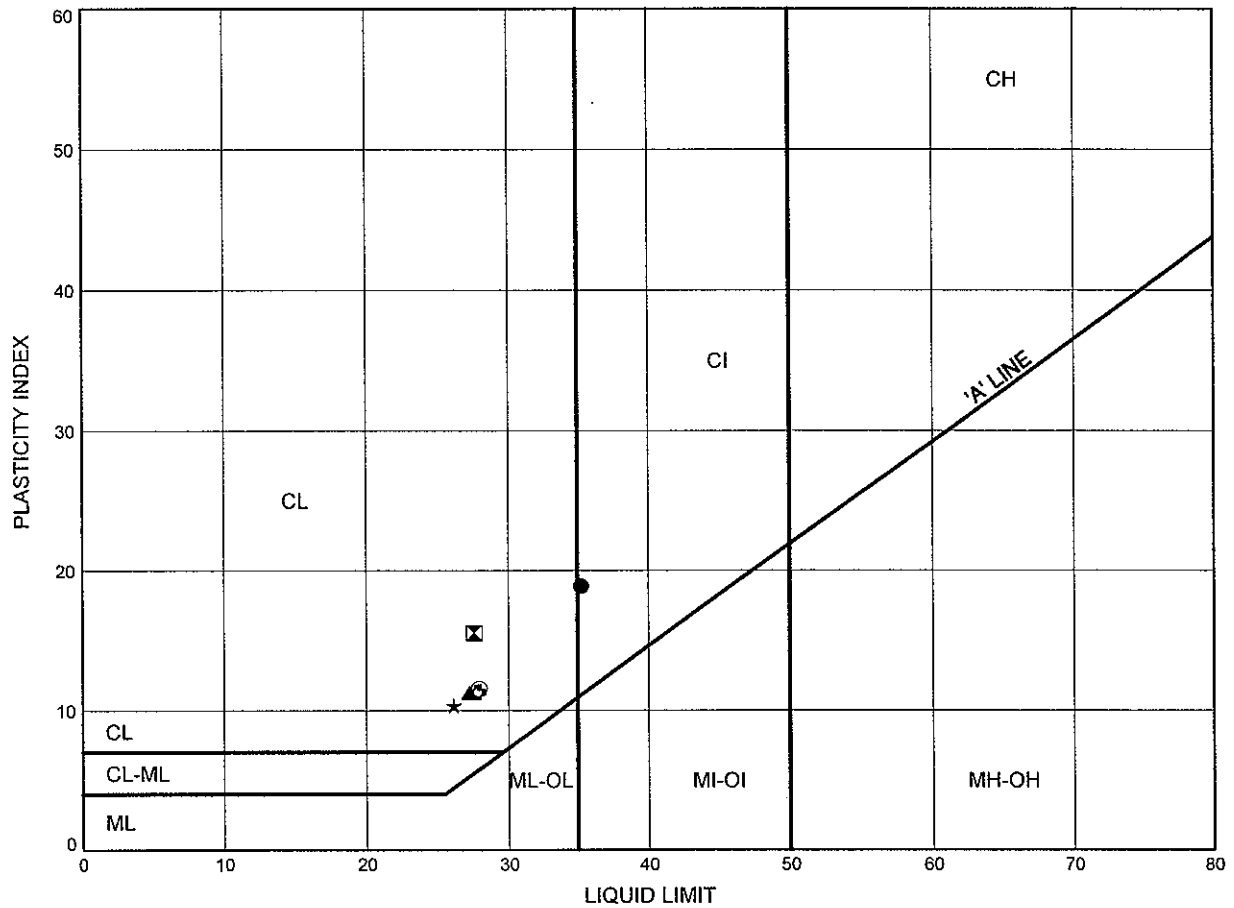
Prep'd DB

Chkd. RA

ATTERBERG LIMITS TEST RESULTS

FIGURE B17

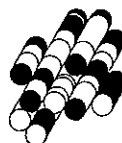
UPPER SILTY CLAY TILL



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	W1	7.8	170.5
⊠	W1	9.3	169.0
▲	W1	10.9	167.4
★	W2	9.3	169.1
⊙	W3	6.3	170.5
⊗	W3	9.3	167.5

Date March 2010

Project 1-09-4135



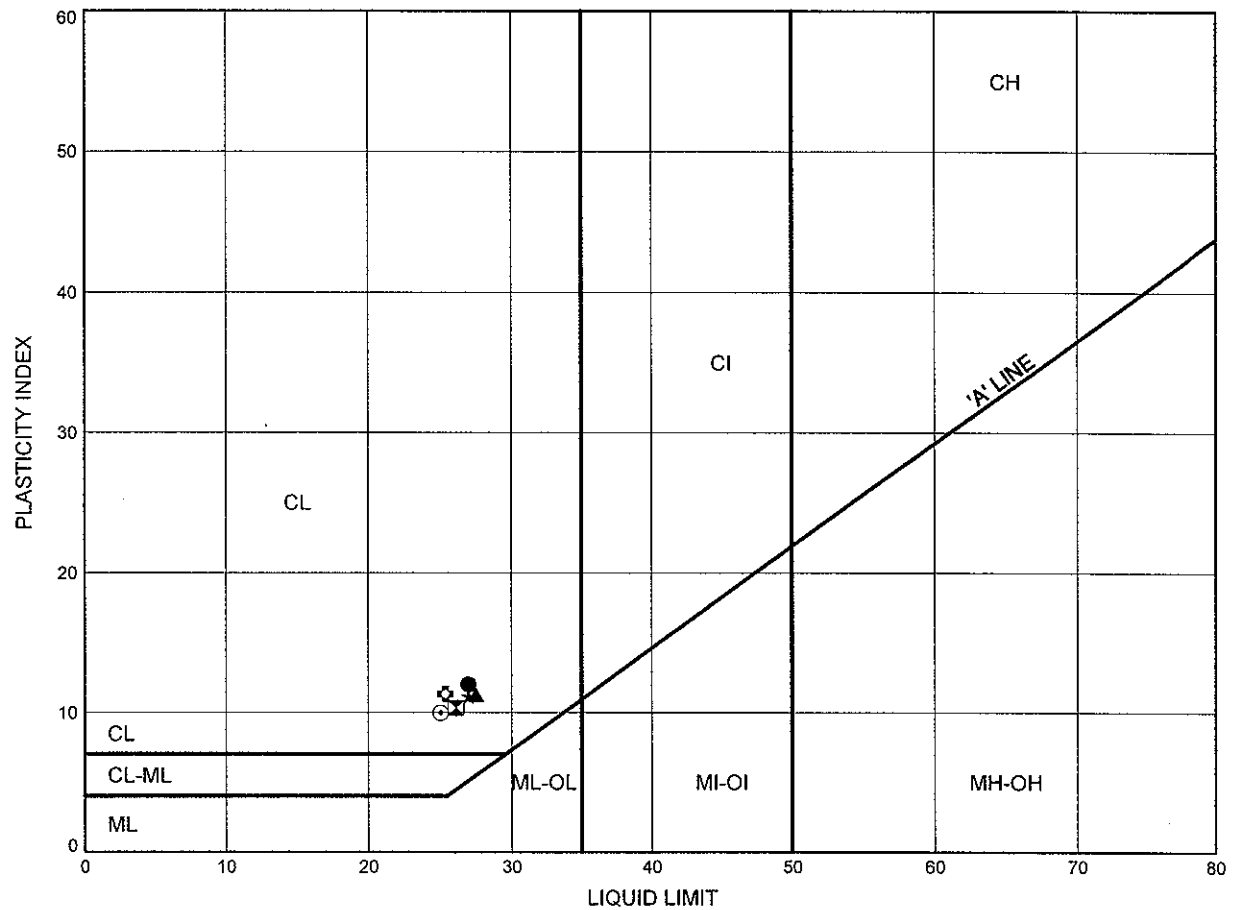
Prep'd DB

Chkd. RA

ATTERBERG LIMITS TEST RESULTS

FIGURE B18

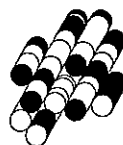
UPPER SILTY CLAY TILL



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	W3	10.9	165.9
⊠	W3	13.9	162.9
▲	W4	9.3	168.3
★	W4	12.4	165.2
⊙	W4	15.4	162.2
⊛	W5	8.4	162.4

Date March 2010

Project 1-09-4135



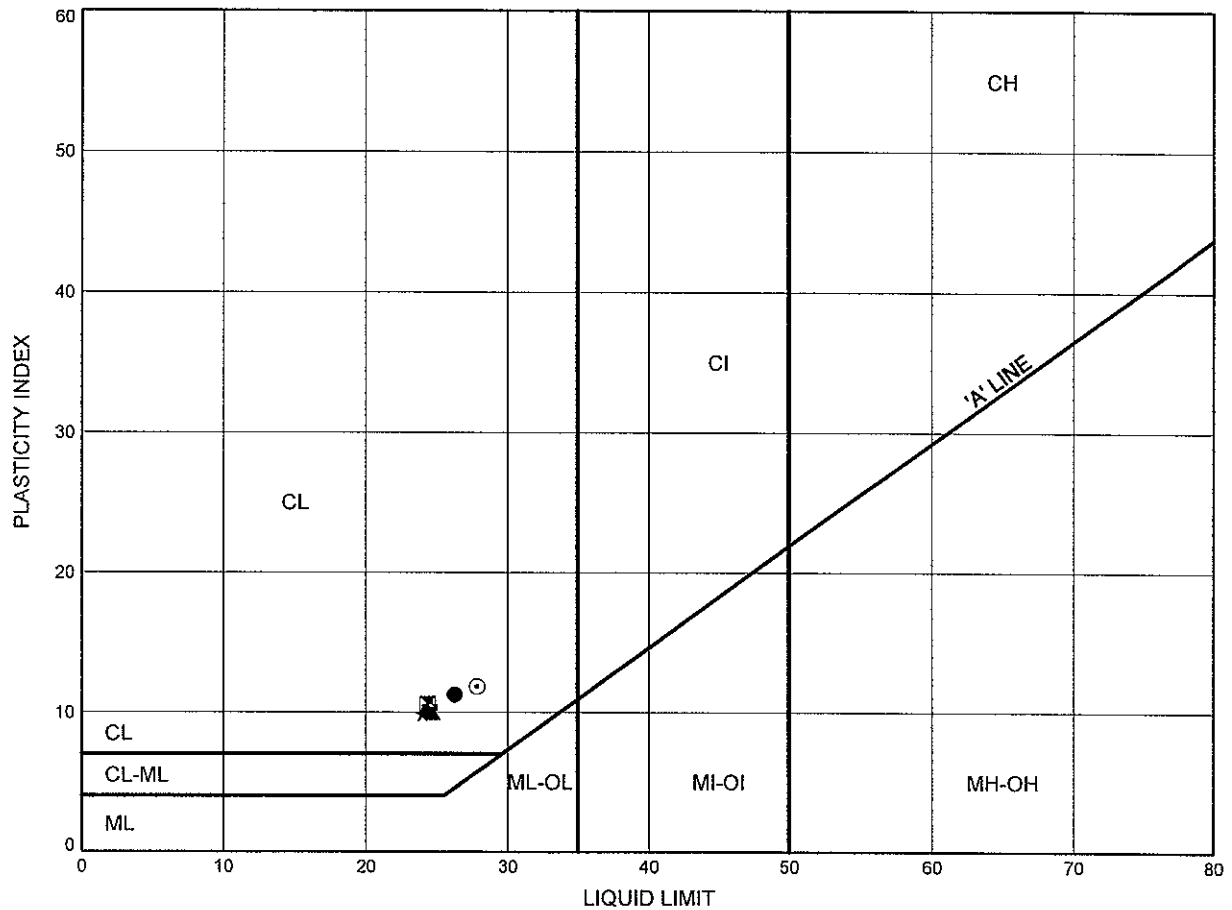
Prep'd DB

Chkd. RA

ATTERBERG LIMITS TEST RESULTS

FIGURE B19

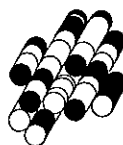
UPPER SILTY CLAY TILL



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	W6	12.4	163.2
⊠	W6	13.9	161.7
▲	W6	15.4	160.2
★	W7	12.4	162.8
⊙	W8	10.9	165.0
⊛	W8	13.9	162.0

Date March 2010

Project 1-09-4135



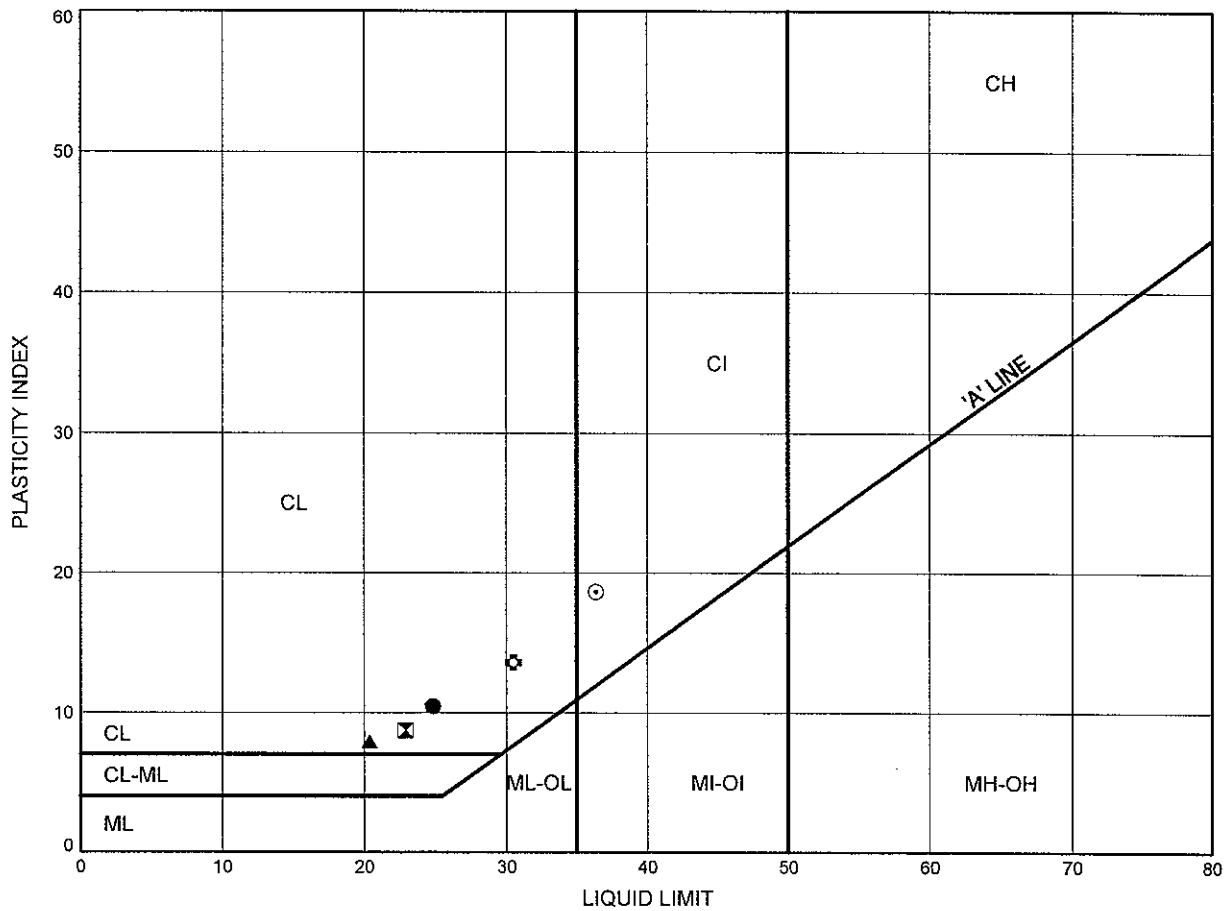
Prep'd DB

Chkd. RA

ATTERBERG LIMITS TEST RESULTS

FIGURE B20

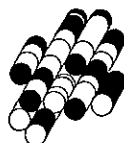
UPPER SILTY CLAY TILL



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
▲	W9	12.4	163.2
★	W9	13.9	161.7
⊙	W9	17.0	158.6
⊕	W9	18.5	157.1
●	W10	12.8	160.5
⊗	W10	15.9	157.4

Date March 2010

Project



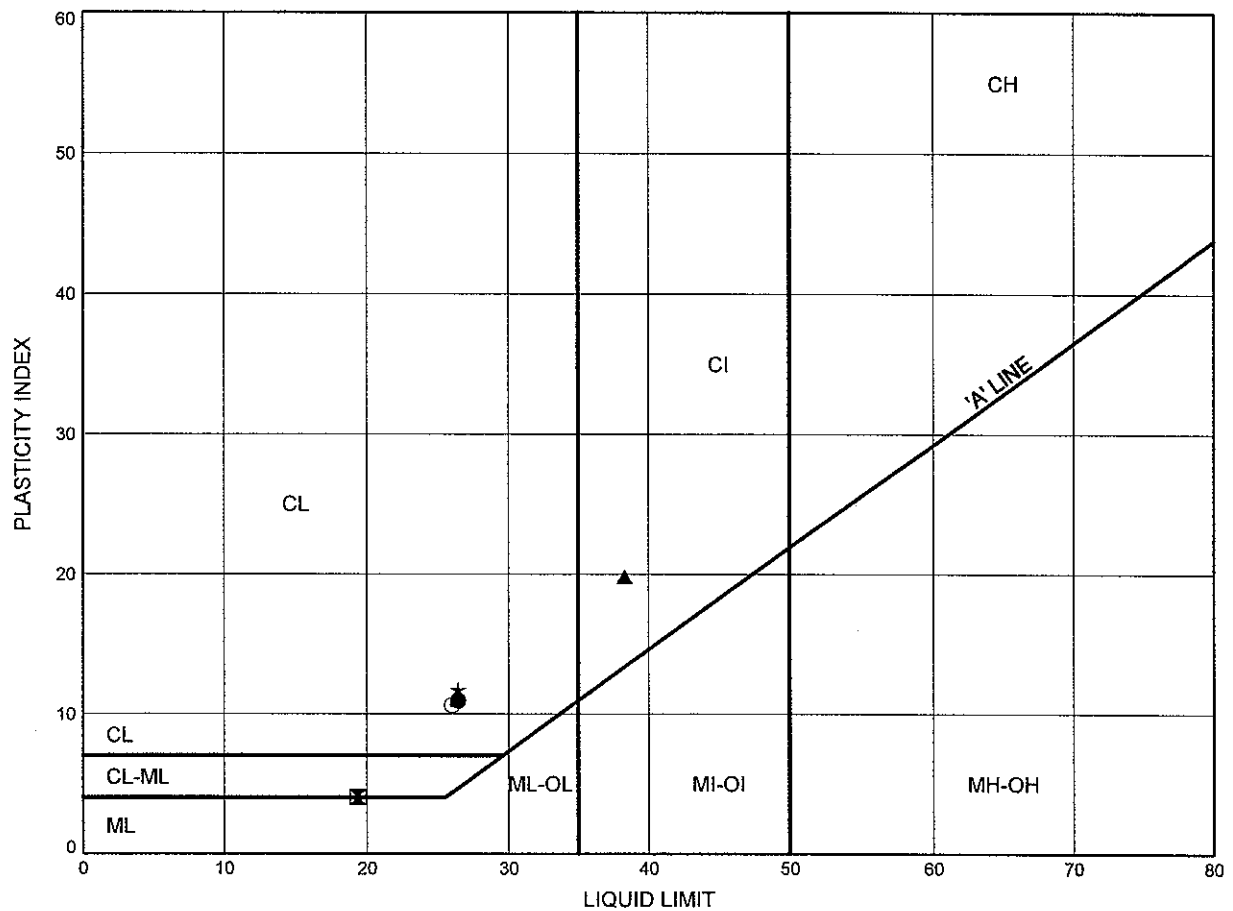
Prep'd DB

Chkd. RA

ATTERBERG LIMITS TEST RESULTS

FIGURE B21

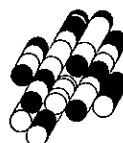
UPPER SILTY CLAY TILL



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	W11	12.2	161.2
⊠	W11	16.0	157.4
▲	W12	14.4	159.0
★	W13	4.0	170.6
⊙	W13	6.3	168.3

Date March 2010

Project 1-09-4135



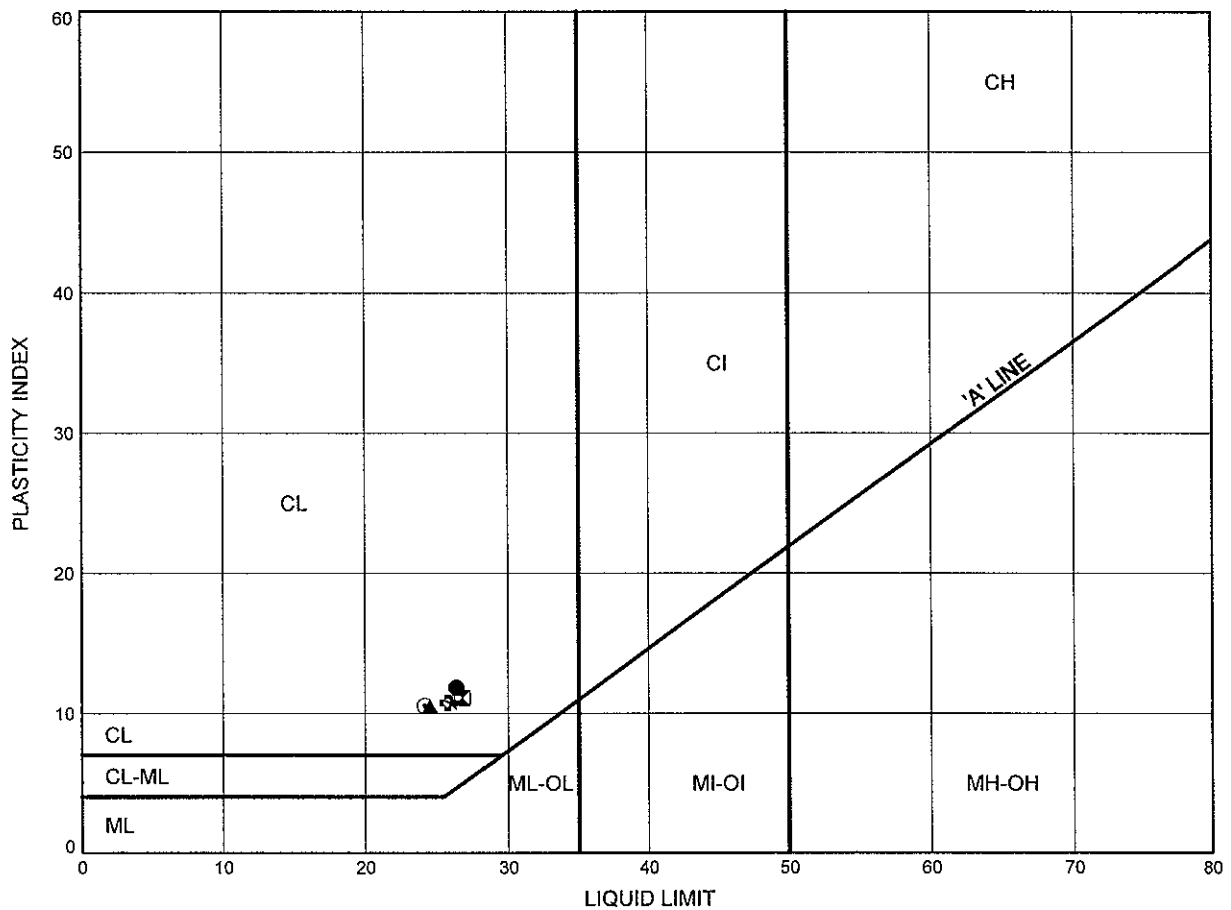
Prep'd DB

Chkd. RA

ATTERBERG LIMITS TEST RESULTS

FIGURE B22

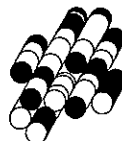
UPPER SILTY CLAY TILL



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	W13	7.8	166.8
⊠	W13	10.9	163.7
▲	W13	13.9	160.7
★	W14	9.3	165.5
⊙	W14	12.4	162.4
⊗	W15	7.8	171.4

Date March 2010

Project 1-09-4135



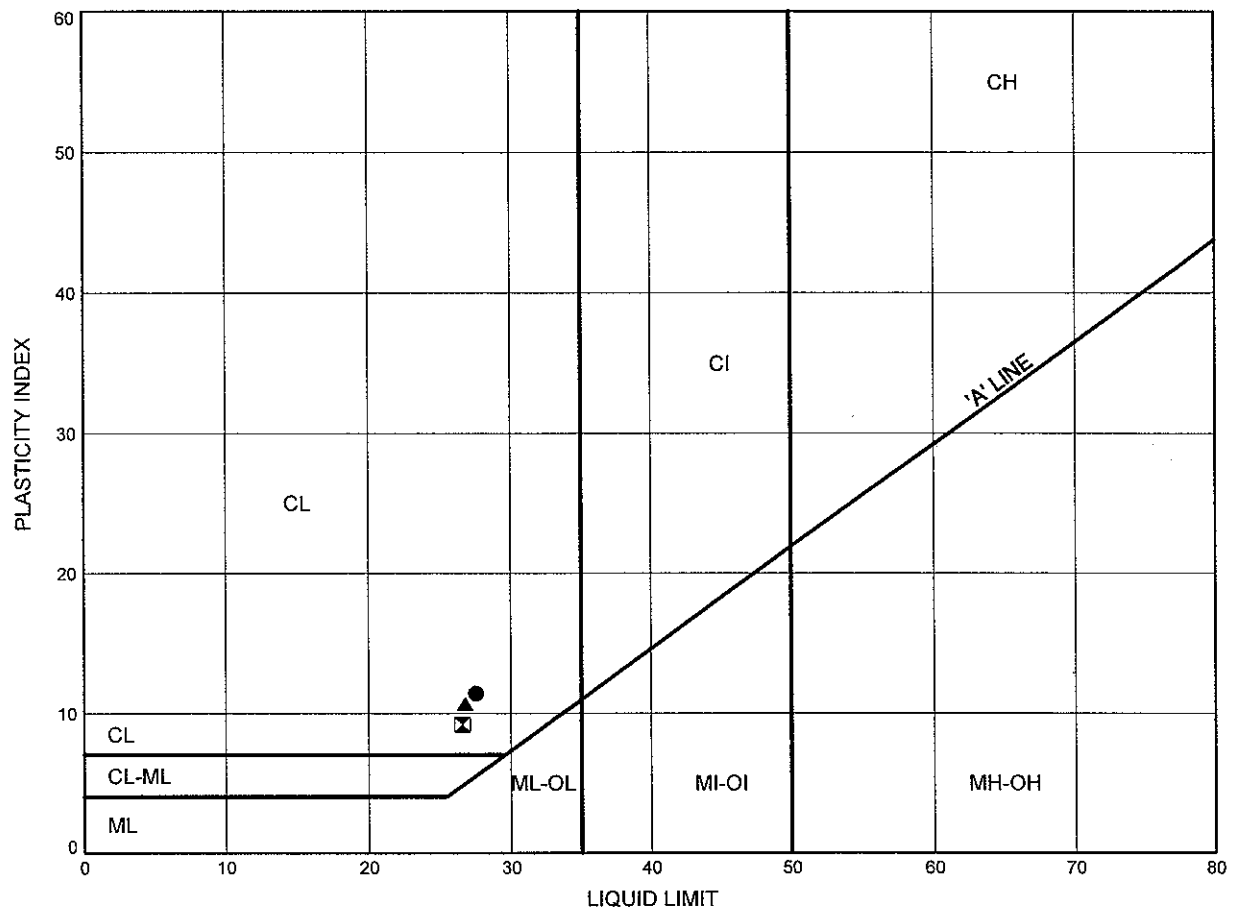
Prep'd DB

Chkd. RA

ATTERBERG LIMITS TEST RESULTS

FIGURE B23

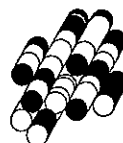
UPPER SILTY CLAY TILL



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	W16	12.4	170.5
⊠	W16	17.0	165.9
▲	W16	18.5	164.4

Date March 2010

Project 1-09-4135



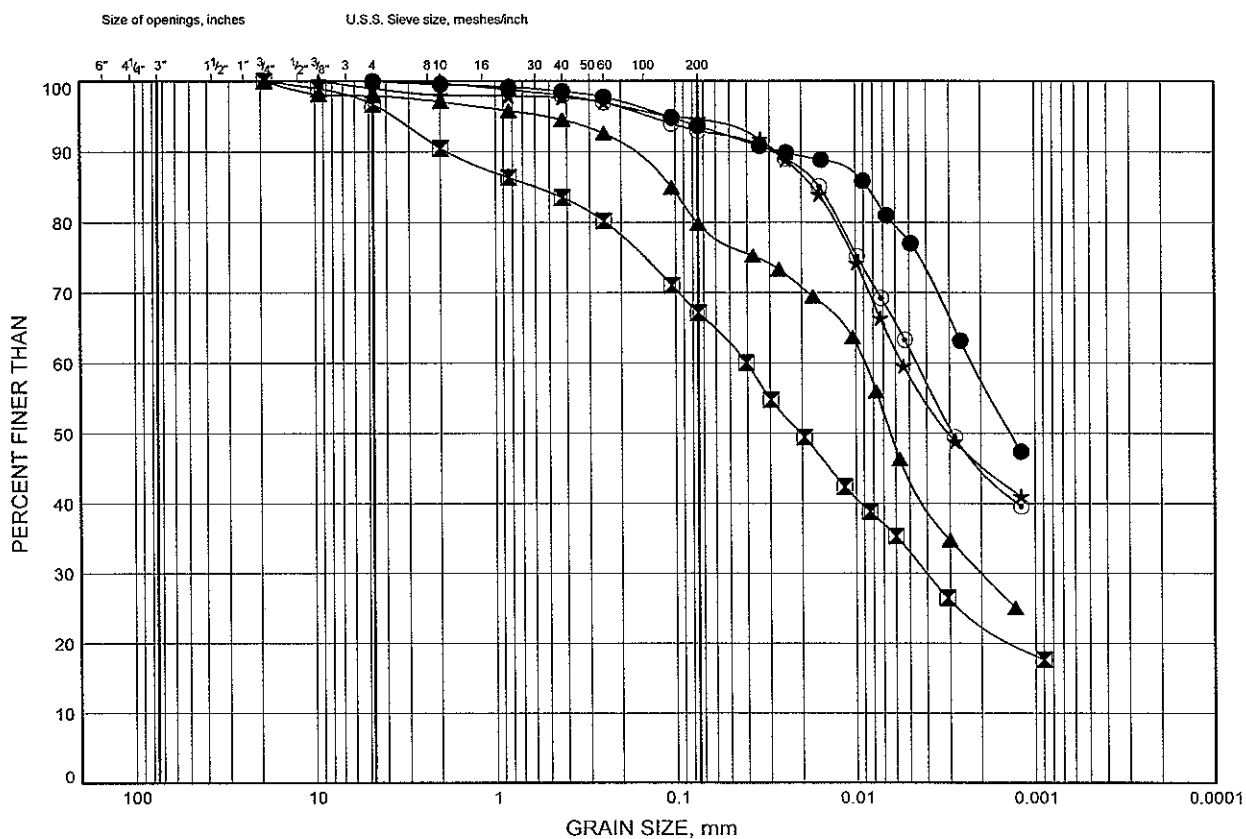
Prep'd DB

Chkd. RA

GRAIN SIZE DISTRIBUTION

FIGURE B24

LOWER SILTY CLAY TILL

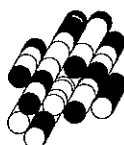


COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
☒	W3	23.1	153.7
▲	W5	16.0	154.8
★	W6	23.1	152.5
⊙	W9	23.1	152.5
●	W10	20.5	152.8

Date March 2010

Project 1-09-4135



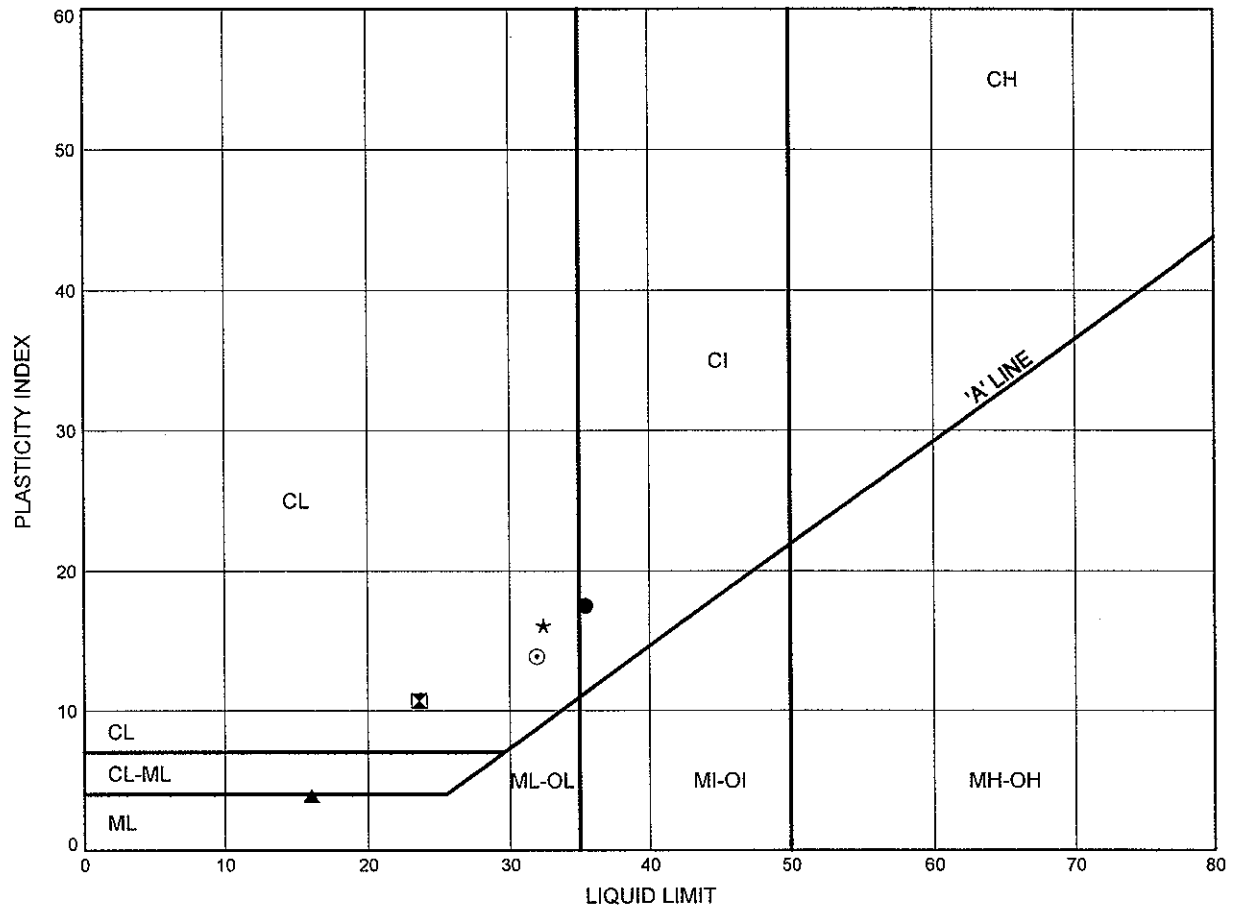
Prep'd DB

Chkd. RA

ATTERBERG LIMITS TEST RESULTS

FIGURE B25

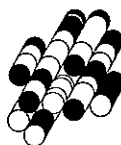
LOWER SILTY CLAY TILL



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
⊠	W3	23.1	153.7
▲	W5	16.0	154.8
★	W6	23.1	152.5
⊙	W9	23.1	152.5
●	W10	20.5	152.8

Date March 2010

Project 1-09-4135



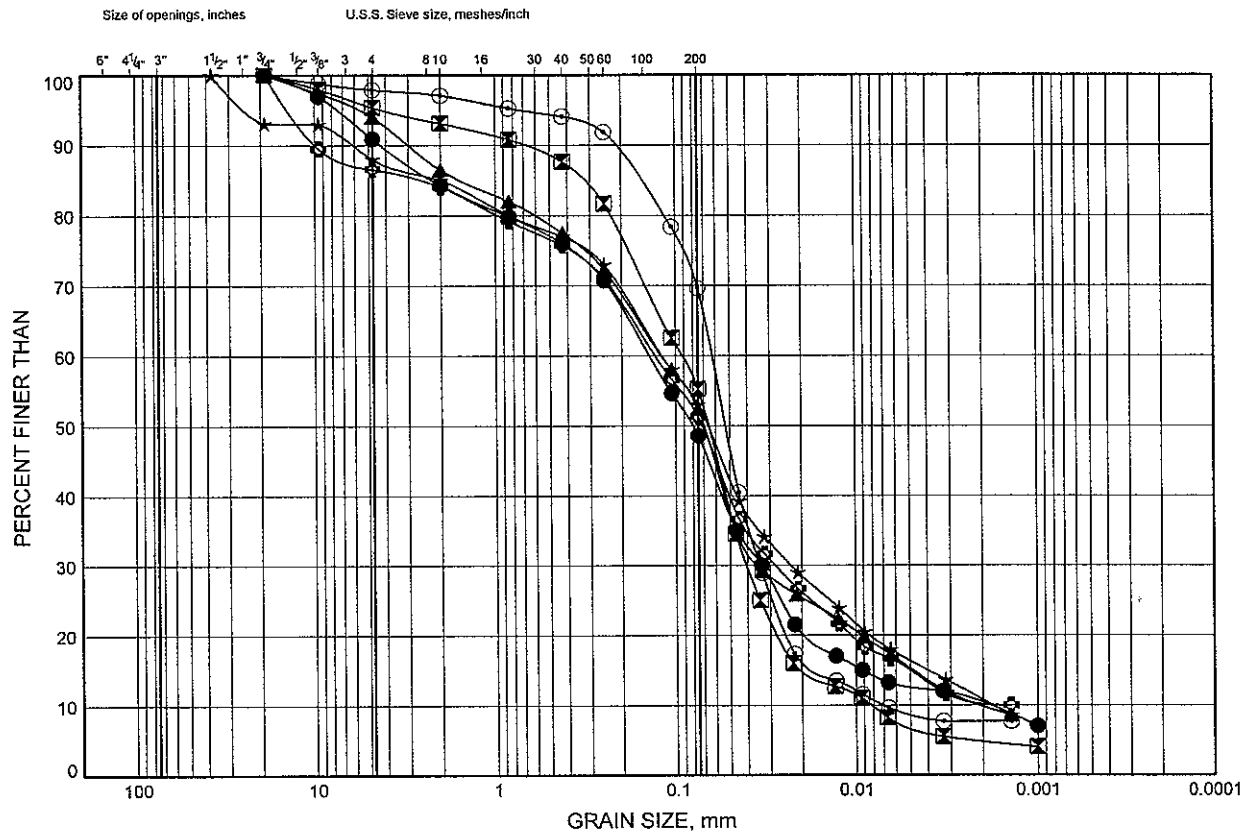
Prep'd DB

Chkd. RA

GRAIN SIZE DISTRIBUTION

FIGURE B26

SANDY SILT TO SILT AND SAND TILL



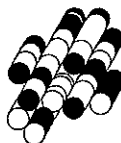
COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL BOREHOLE DEPTH (m) ELEVATION (m)

●	W3	20.0	156.8
⊠	W3	24.6	152.2
▲	W4	21.5	156.1
★	W5	13.0	157.8
⊙	W6	20.0	155.6
⊛	W9	20.0	155.6

Date March 2010

Project 1-09-4135



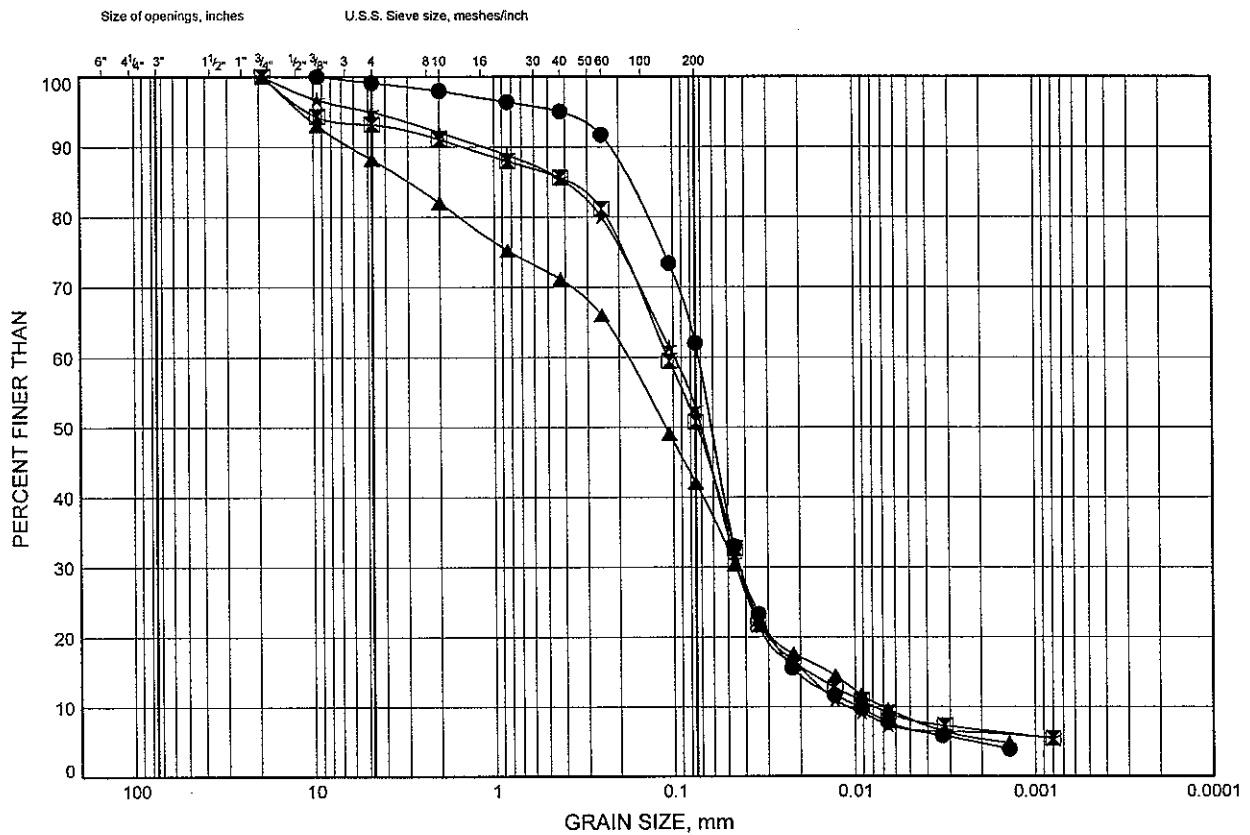
Prep'd DB

Chkd. RA

GRAIN SIZE DISTRIBUTION

FIGURE B27

SANDY SILT TO SILT AND SAND TILL



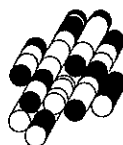
COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL BOREHOLE DEPTH (m) ELEVATION (m)

●	W10	17.4	155.9
⊠	W12	17.4	156.0
▲	W13	18.5	156.1
★	W14	18.5	156.3

Date March 2010

Project 1-09-4135



Prep'd DB

Chkd. RA

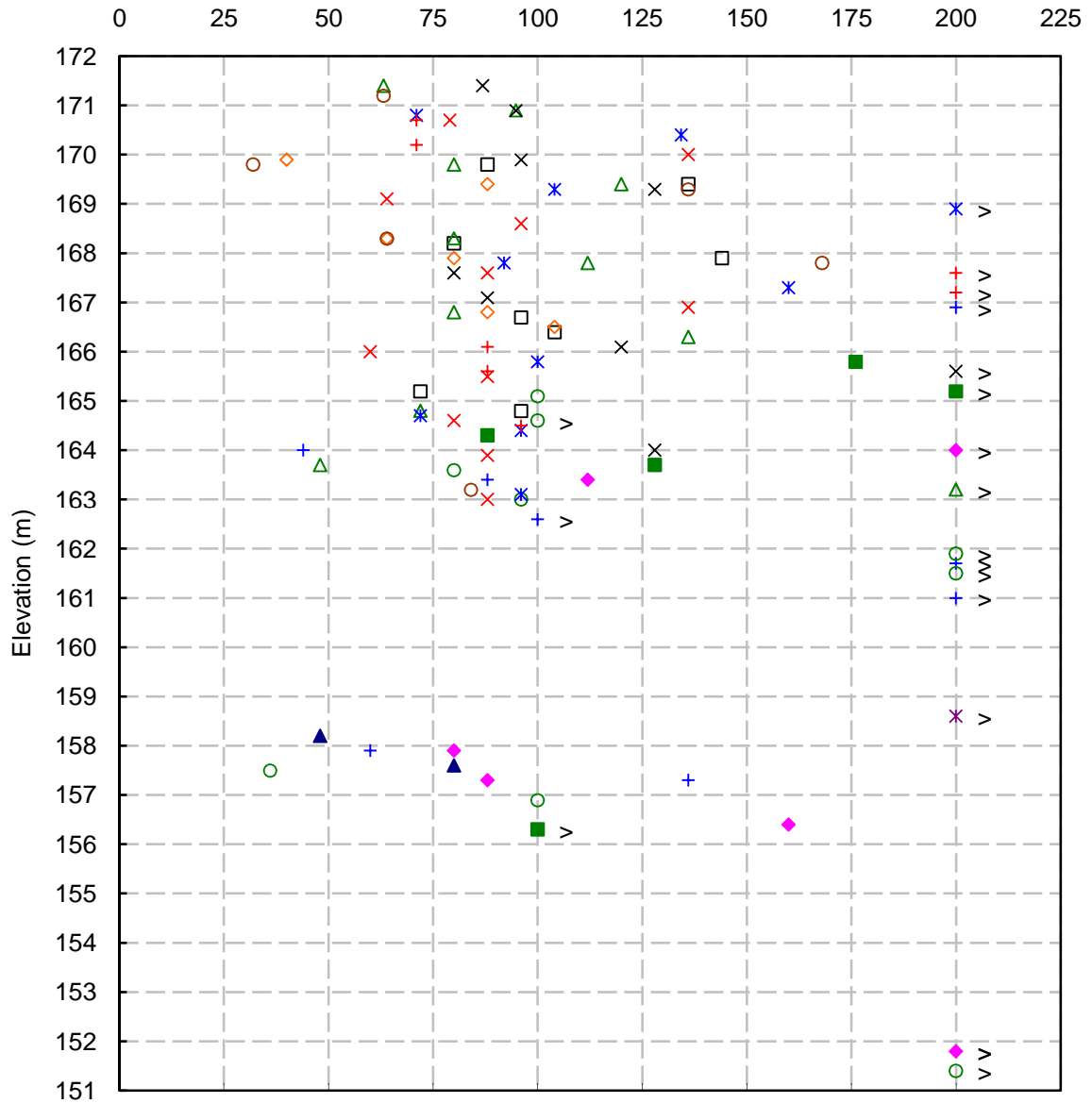
CORRECTED UNDRAINED SHEAR STRENGTH

FIGURE B28

OLD WELLAND CANAL / WELLAND RIVER BRIDGE

Silty Clay Till

Corrected Cu (kPa)



□ W1 ◇ W2 △ W3 × W4 * W5 + W6 ○ W7 ■ W8 ◆ W9 ▲ W11 × W13 * W14 + W15 ○ W16

Field Shear Vane Correction

Morris & Williams (1994)

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

Applied Correction Factors

0.99 (Elev.>170m)

1.00 (Elev.<170m)

Project No. : 1-09-4135

Date : September, 2010



Terraprobe Inc.

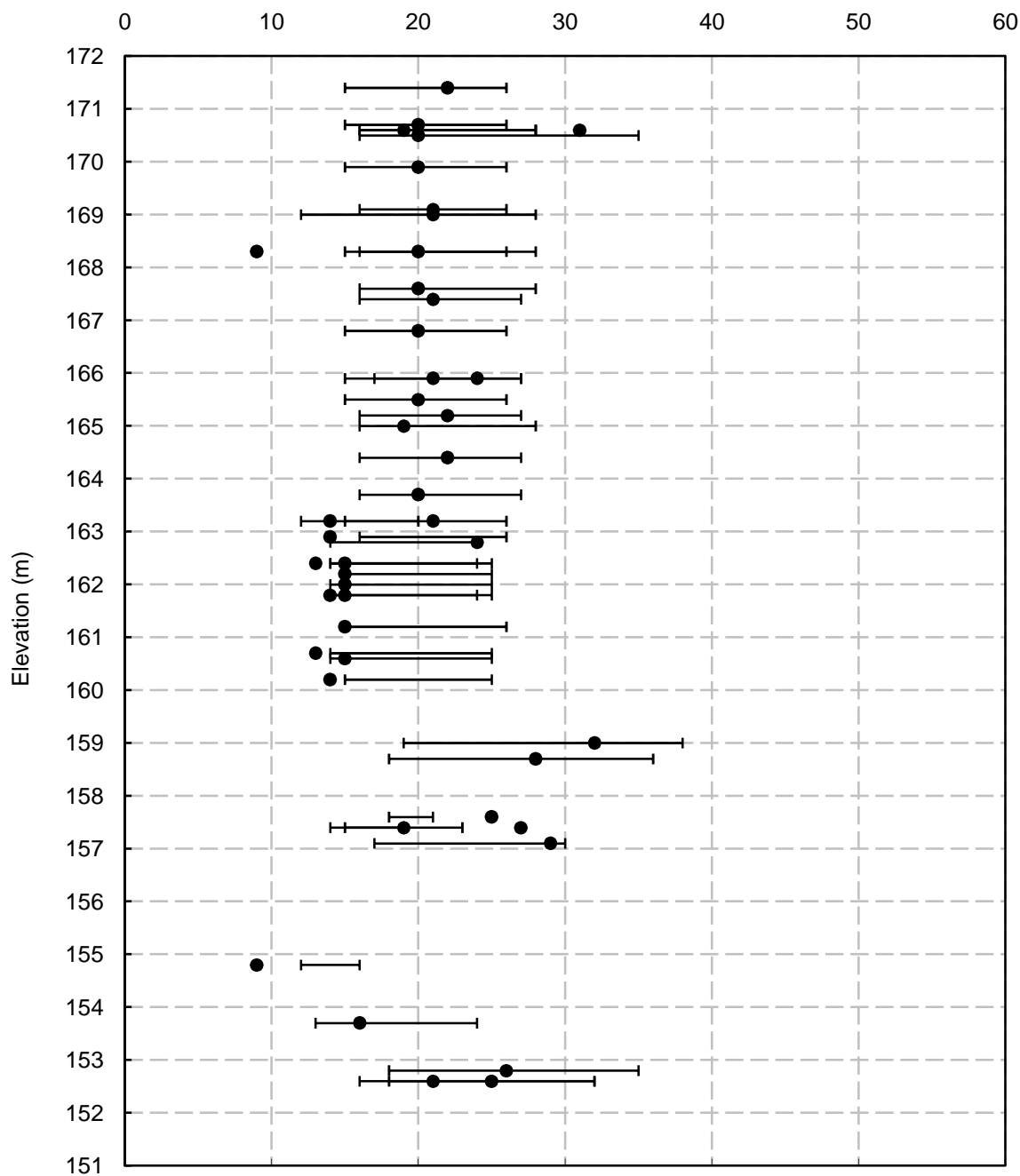
Prepared By : HW

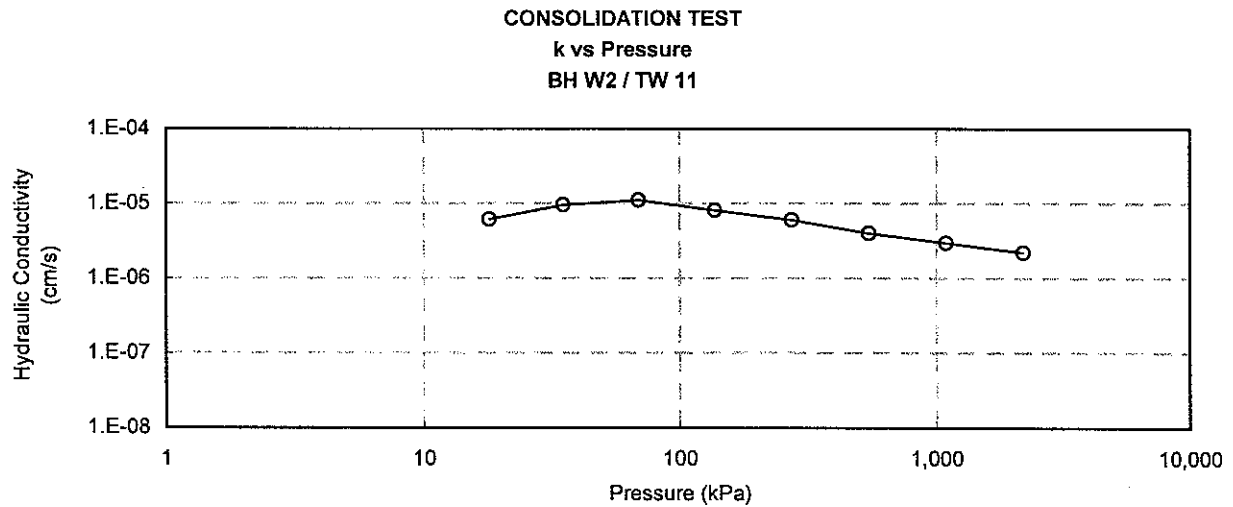
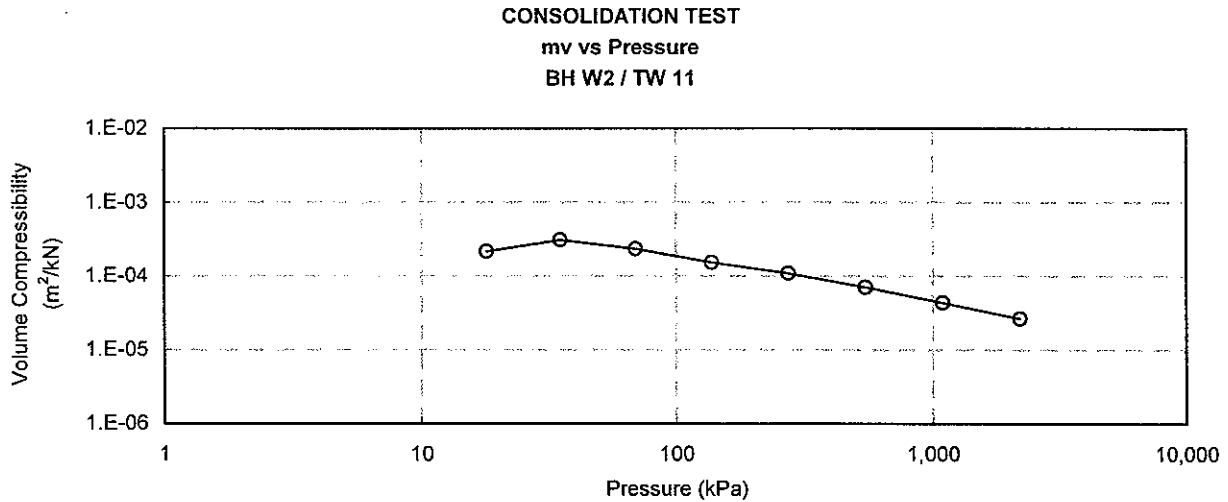
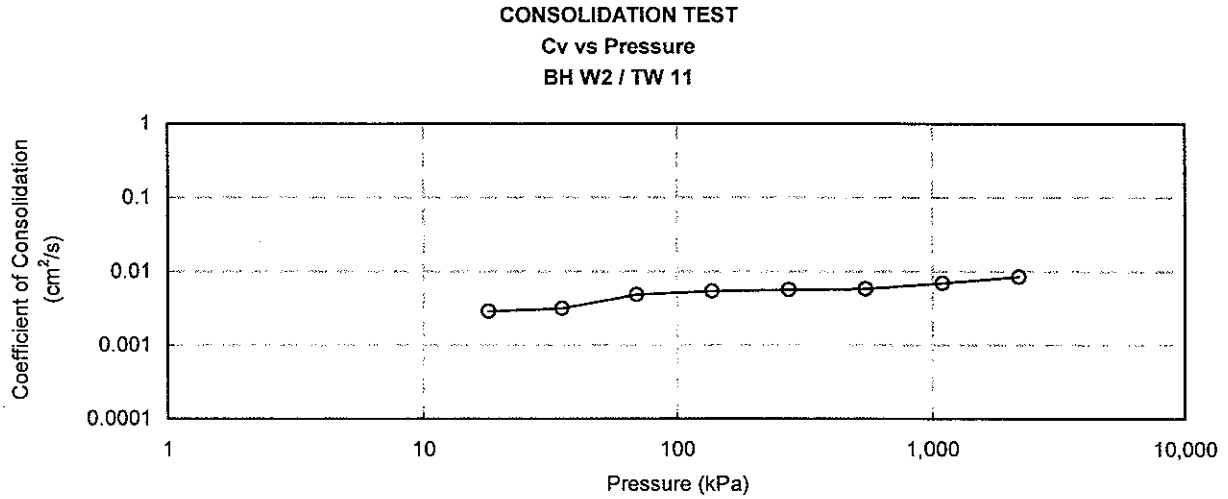
Checked By : RA

OLD WELLAND CANAL / WELLAND RIVER BRIDGE

Silty Clay Till

Atterberg Limits & Water Contents (%)

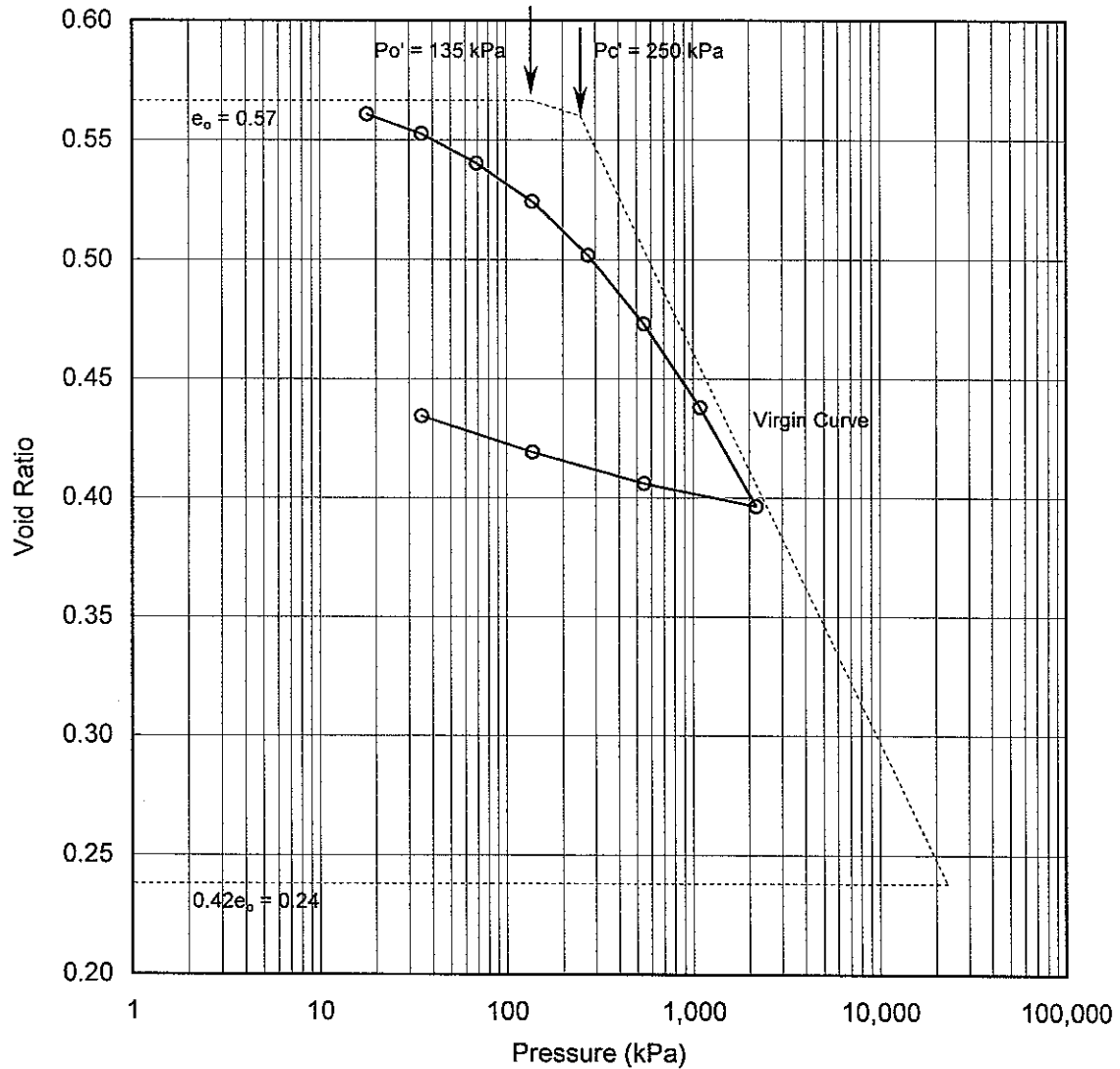




CONSOLIDATION TEST

e vs Pressure

BH W2 / TW 11



Soil Type : Silty Clay Till

$e_o =$	0.57	$\omega_L =$	26%	$Po' =$	135 kPa
$\omega =$	21%	$\omega_P =$	16%	$Pc' =$	250 kPa
$\gamma =$	20.6 kN/m ³	$PI =$	10%	$Cc =$	0.164
$G_s =$	2.72			$Cr =$	0.024

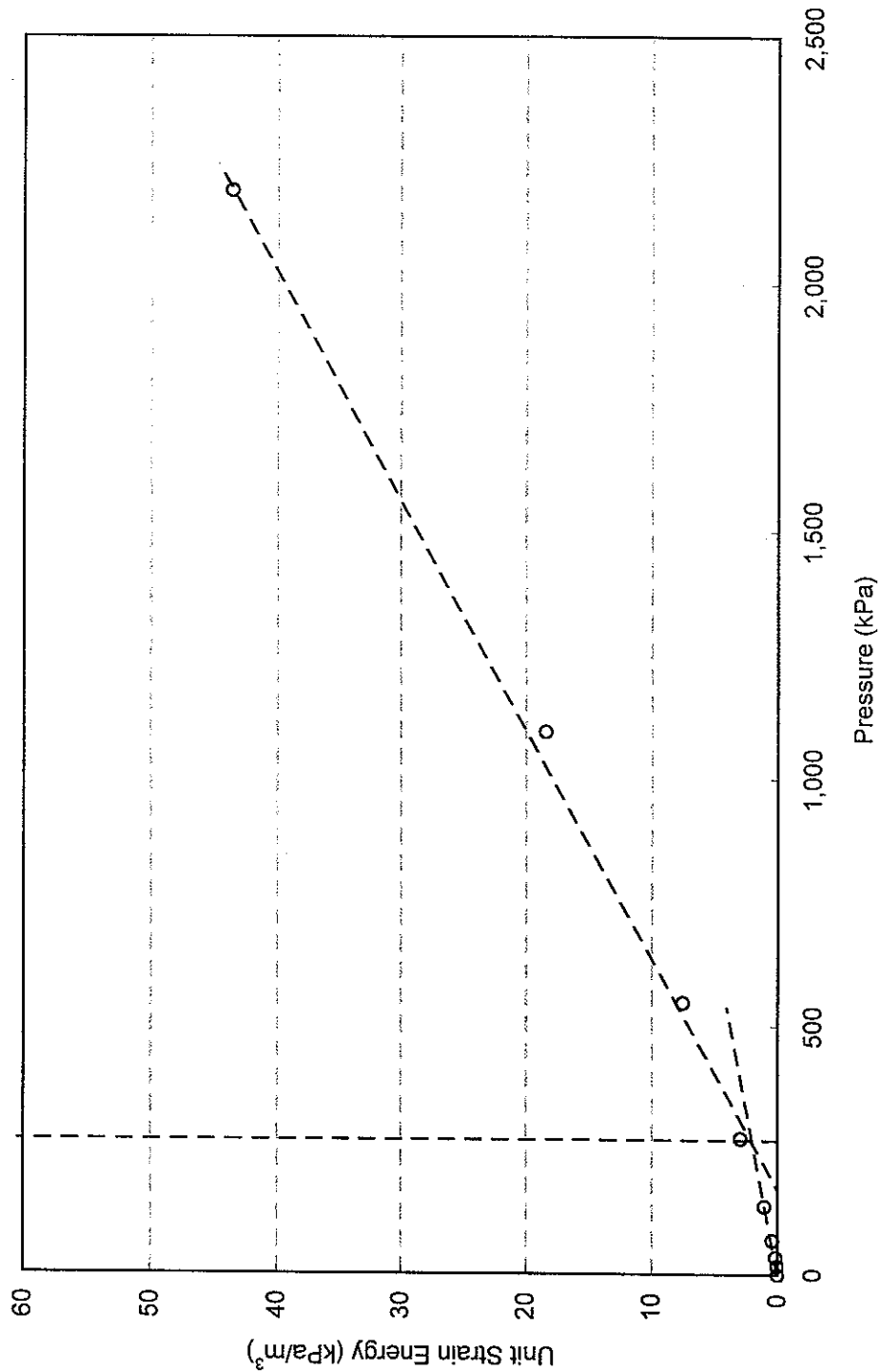
Project No. : 1-09-4135
Date : April 2010



Terraprobe Inc.

Prepared By : HW
Checked By : RA

CONSOLIDATION TEST
Unit Strain Energy vs Pressure
BH W2 / TW 11



$P_c = 270 \text{ kPa}$

Project No. : 1-09-4135

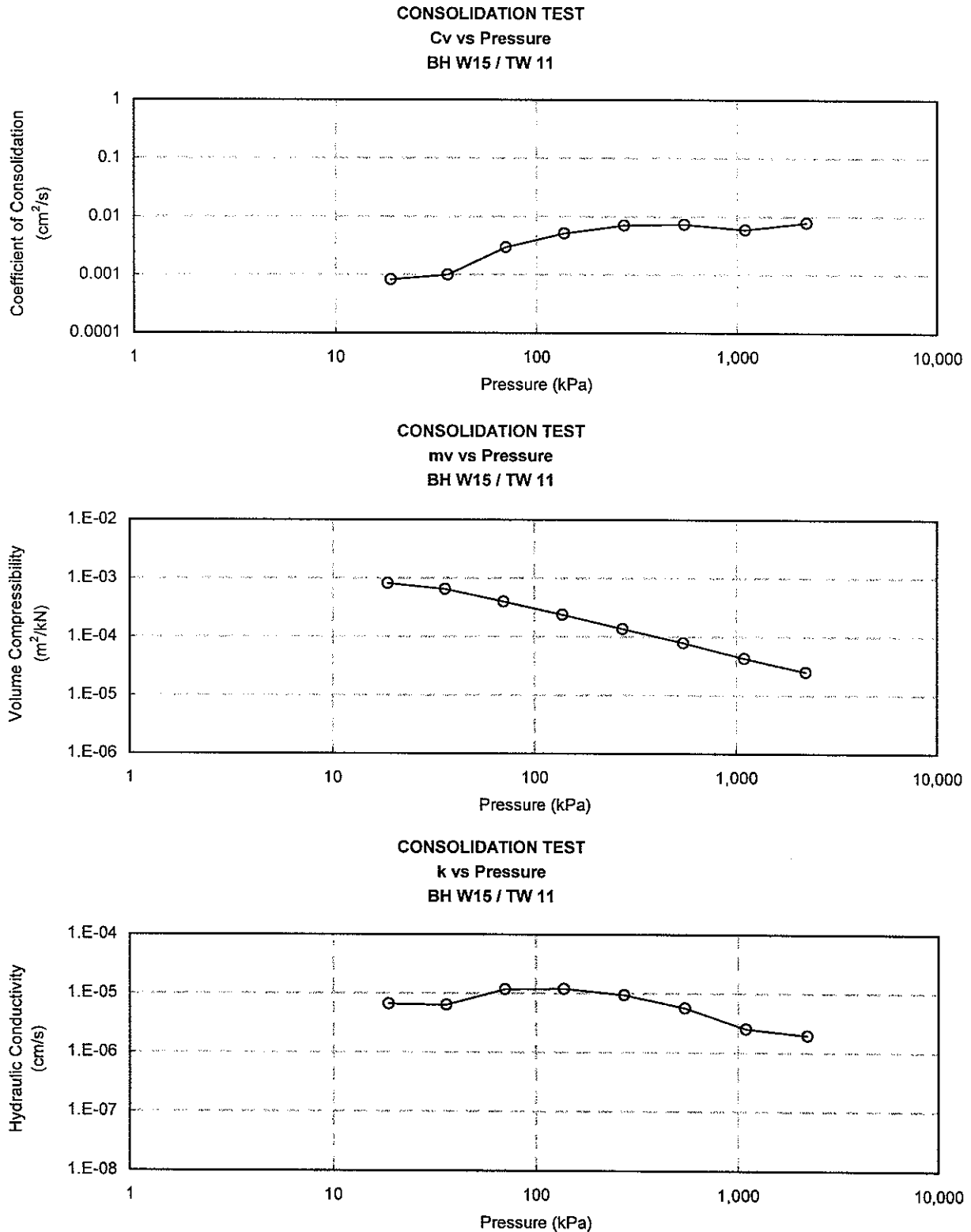
Date : April 2010



Terraprobe Inc.

Prepared By : HW

Checked By : RA



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



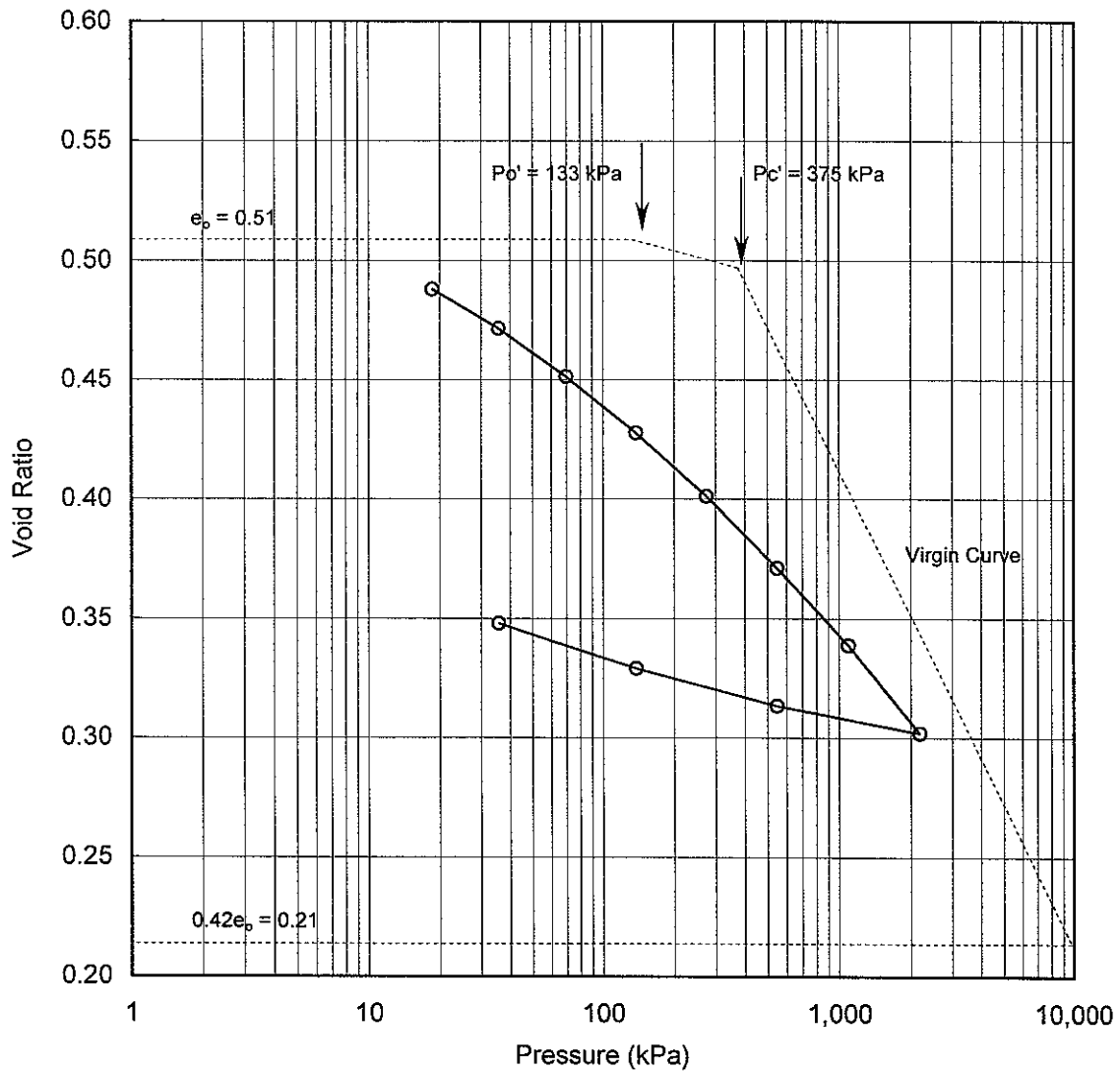
Terraprobe Inc.

Prepared By : HW
Checked By : RA

CONSOLIDATION TEST

e vs Pressure

BH W15 / TW 11



Soil Type : Silty Clay Till

$e_o =$	0.51	$\omega_L =$	26%	$P_o' =$	133 kPa
$\omega =$	22%	$\omega_p =$	15%	$P_c' =$	375 kPa
$\gamma =$	21.4 kN/m ³	PI =	11%	Cc =	0.200
Gs =	2.75			Cr =	0.026

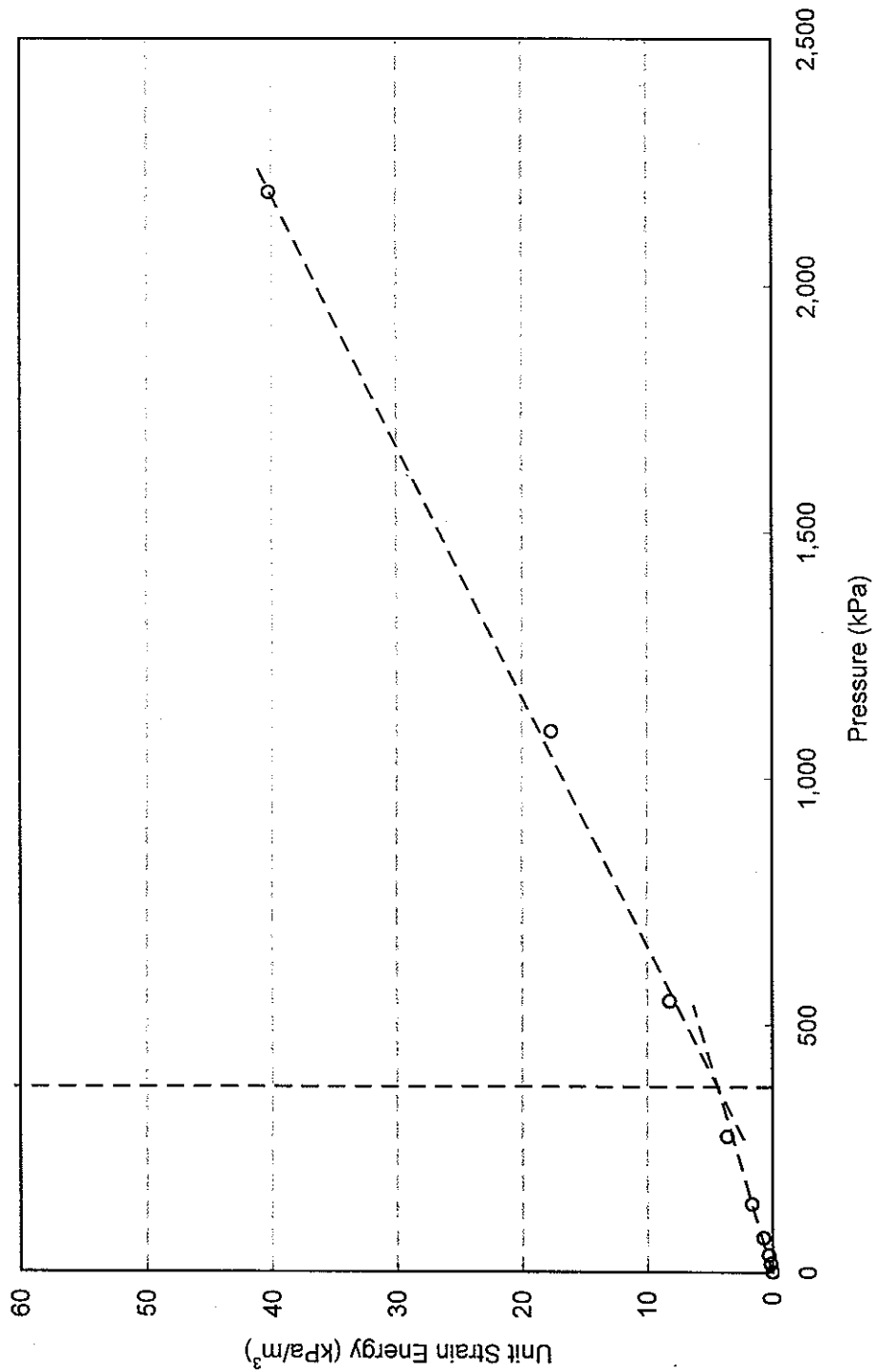
Project No. : 1-09-4135
Date : April 2010



Terraprobe Inc.

Prepared By : HW
Checked By : RA

CONSOLIDATION TEST
Unit Strain Energy vs Pressure
BH W15 / TW 11



$P_c = 375 \text{ kPa}$

Project No. : 1-09-4135

Date : April 2010



Terraprobe Inc.

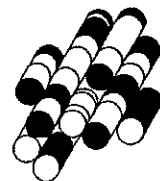
Prepared By : HW

Checked By : RA

APPENDIX C

Record of Borehole Sheets (Previous Investigations)

Terraprobe Inc.



RECORD OF BOREHOLE No 101

1 OF 1

METRIC

W.P. 171 - 90 - 01 LOCATION CO - ORDS. N 4 784 905.4; E 327 177.5 ORIGINATED BY P.V.
DIST 4 HWY 406 BOREHOLE TYPE CONTINUOUS FLIGHT AUGER (H.S.) & CONE TEST COMPILED BY M.S.
DATUM GEODETTIC DATE 87 10 19 to 87 10 21 CHECKED BY M.S.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		WATER CONTENT (%)	UNIT WEIGHT 7 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	20 40 60 80 100			
171.9	Ground Level											
0.0			1	SS	12							
			2	SS	17							
			3	SS	13							
			4	SS	15							
	CLAYEY SILT to SILTY CLAY. With Occasional Silt Seams, Trace of Sand, Stiff to Firm		5	SS	12							
			6	SS	14							
			7	SS	6							
			8	SS	5							
			9	TW	PH							
			10	SS	38							
			11	SS	37							
			12	SS	47							
155.8			13	SS	6							
16.1			14	SS	9							
	SANDY SILT, Trace of Gravel, Loose to Compact											
151.9			15	SS	29							
20.0	CLAYEY SILT to SILTY CLAY, Trace of Sand, Very Stiff											
150.3												
21.6			16	RC	REC							
	DOLOSTONE BEDROCK Sound		17	RC	REC							
147.9				BX	97%							
24.0	End of Borehole											
	• Note: Artesian Condition Water Level Rose to 0.5m Above Ground Level @: 172.4											

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

RECORD OF BOREHOLE No 102

1 OF 1

METRIC

W.P. 171 - 90 - 01 LOCATION CO - ORDS. N 4 784 893.6; E 327 185.0 ORIGINATED BY P.M.
DIST 4 HWY 408 BOREHOLE TYPE CONTINUOUS FLIGHT AUGER (H.S.) & CONE TEST COMPILED BY M.
DATUM GEODETIC DATE 87 10 22 to 87 10 23 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				WATER CONTENT (%)	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	N-VALUES			20 40 60 80 100	120 140 160 180 200	220 240 260 280 300	320 340 360 380 400			
171.7	Ground Level													
0.0	Boulders		1	SS	42									67 12 13 3
			2	SS	36									
	SANDY GRAVEL		3	SS	14									
	Some SILT, Trace of Clay		4	SS	16									
	Compact to Very Dense		5	SS	59									
	(Fill)		6	SS	80									
166.5			7	SS	74									
5.2			8	SS	20									
	CLAYEY SILT to SILTY CLAY		9	SS	13									
	Trace of Sand,		10	SW	PH									
	Stiff		11	SS	49									
			12	SS	60									
	Hard		13	SS	13									
156.3			14	SS	10									
15.4			15	SS	110									
	SANDY SILT, Trace of Gravel,													
	Compact to Very Dense													
152.2														
19.5	Presumed													
	CLAYEY SILT to SILTY CLAY,													
	Trace of Sand,													
150.4	Very Stiff													
21.3	End of Borehole													
	Presumed Bedrock													

+3, x3: Numbers refer to
Sensitivity

20
15-25 (X) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 103

1 OF 1

METRIC

W.P. 171 - 90 - 01 LOCATION CO - ORDS, N 4. 784 804.1; E 327 188.6 ORIGINATED BY .
DIST 4 HWY 405 BOREHOLE TYPE CONTINUOUS FLIGHT AUGER (H 5) & CONE TEST COMPILED BY .
DATUM GEODETIC DATE 87 10 24 to 87 10 27 CHECKED BY .

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT L	UNIT WEIGHT 7 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) CF SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			SHEAR STRENGTH xPo						
177.4	Ground Level							20 40 60 80 100						
0.0			1	SS	39									
			2	SS	40									
			3	SS	21									0 1 (99)
			4	SS	20									0 1 (99)
			5	SS	53									
			6	SS	20									
			7	SS	10									
			8	TW	PH									
			9	TW	PH									
			10	SS	8								20.7	1 2 (97)
			11	SS	10									
			12	TW	PH									
			13	SS	58									0 0 (100)
			14	SS	22									0 1 (99)
157.3			15	SS	73									20 36 (44)
20.1			16	SS	12									0 8 (92)
154.2			17	RC	28%									
23.2			18	RC BX	REC 78%									
150.5			19	RC BX	REC 100%									
28.9														
146.8														

+3, x5, Numbers refer to
Sensitivity

20
15-25 (x) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No. 104										1 OF 1		METRIC	
W.P. 171 - 90 - 01			LOCATION CO - ORDS. N 4 754 830.8; E 327 186.4			ORIGINATED BY: M							
DIST 4 HWY 408			BOREHOLE TYPE CONTINUOUS FLIGHT AUGER (H.S.) & CONE TEST			COMPILED BY: M							
DATUM GEOIDETIC			DATE 87 10 30 to 87 11 05			CHECKED BY: M							
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					
170.9	Water Level												
0.0													
168.5	Ground Level												
2.4	CLAYEY SILT to SILTY CLAY, Trace of Sand, Stiff to Very Stiff Hard		1	SS	10								
			2	SS	10								
			3	SS	9								
			4	SS	34								
			5	SS	45								
			6	SS	56								
			7	SS	22								
157.8	SANDY SILT, Some Gravel, Very Dense		8	SS	53								
13.1			9	SS	74								
155.4													
13.5	End of Borehole												
	• Note: Artesian Condition Water Level Rose to 1.4m Above Ground Level El: 172.3												

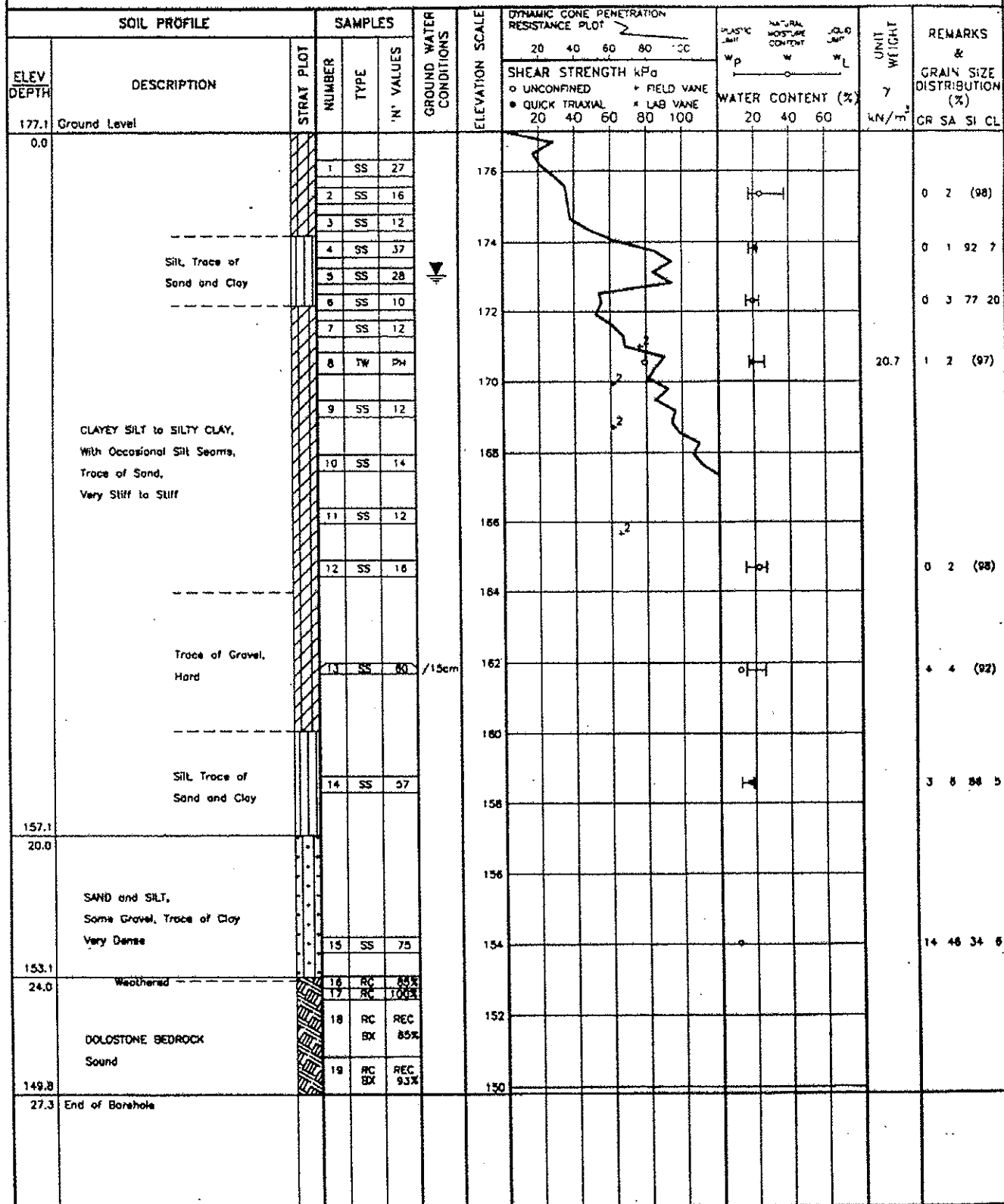
RECORD OF BOREHOLE No 105												1 OF 1		METRIC	
W.P. 171 - 90 - 01			LOCATION CO - ORDS. N 4 764 844.0; E 327 185.0			ORIGINATED BY									
DIST 4 HWY 406			BOREHOLE TYPE CONE TEST			COMPILED BY									
DATUM GEODETTIC			DATE 87 11 05			CHECKED BY									
SOIL PROFILE			SAMPLES			GROUND WATER 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	WATER CONTENT (%)						
170.9	Water Level							20 40 60 80 100							
0.0							170								
167.1	Ground Level						168								
3.8							166								
	Presumed CLAYEY SILT to SILTY CLAY. Trace of Sand, Stiff						164								
161.3							162								
9.4	End of Cone Test														

RECORD OF BOREHOLE No 201

1 OF 1

METRIC

W.P. 171 - 90 - 01 LOCATION CO - ORDS. N 4 765 090.2; E 327 121.8 ORIGINATED BY P.W.
DIST 4 HWY 405 BOREHOLE TYPE CONTINUOUS FLIGHT AUGER (H.S.) & CONE TEST COMPILED BY W.V.
DATUM GEODETIC DATE 87 10 14 to 87 10 16 CHECKED BY J.D.



+3, x3: Numbers refer to
Sensitivity

20
15-25 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 202

1 OF 1

METRIC

W.P. 171 - 90 - 01

LOCATION CO - QROS. N 4 785 073.8; E 327 128.3

ORIGINATED BY P

DIST 4 HWY 406

BOREHOLE TYPE SX CASING & CONE TEST

COMPILED BY M

DATUM GEODETIC

DATE 87 11 07 to 87 11 10

CHECKED BY P.P.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT			NATURAL MOISTURE CONTENT			LIQUID LIMIT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100	W _p	W	W _L	20	40	60			7	KN/m ³	GR
173.4	Water Level																						
0.0																							
171.4	Ground Level																						
2.0																							
			1	SS	10																		
			2	SS	10																		
			3	SS	10																		
			4	SS	11																		
			5	SS	11																		
			6	SS	11																		
			7	SS	11																		
			8	SS	67																		
			9	SS	94																		
			10	SS	61																		
			11	SS	63																		
157.4																							
16.0	SAND and SILT, Some Gravel, Trace of Clay, Dense		12	SS	33																		
156.0			13	SS	48																		
17.4	End of Borehole																						
154.8																							
18.6	End of Cone Test																						
	* Note: Artesian Condition Water Level Rose to 0.3m Above Ground Level Et: 173.7																						

+3, x3 Numbers refer to
Sensitivity

20
15-25 (%) STRAIN AT FAILING
16

RECORD OF BOREHOLE No 203

1 OF 1

METRIC

W.P. 171 - 80 - 01 LOCATION CO - QROD, N 4.784, 944.0; E 327.188.0 ORIGINATED BY
DIST 4 HWY 406 BOREHOLE TYPE CONE TEST COMPILED BY
DATUM GEODETIC DATE 87.11.12 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		WATER CONTENT (%)	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	20 40 60 80 100			
173.4	Water Level											
0.0												
170.7	Ground Level											
2.7												
	Presumed CLAYEY SILT to SILTY CLAY, Trace of Sand and Gravel, Stiff to Hard											
161.9												
11.5	End of Cone Test											

+3, x3: Numbers refer to
Sensitivity

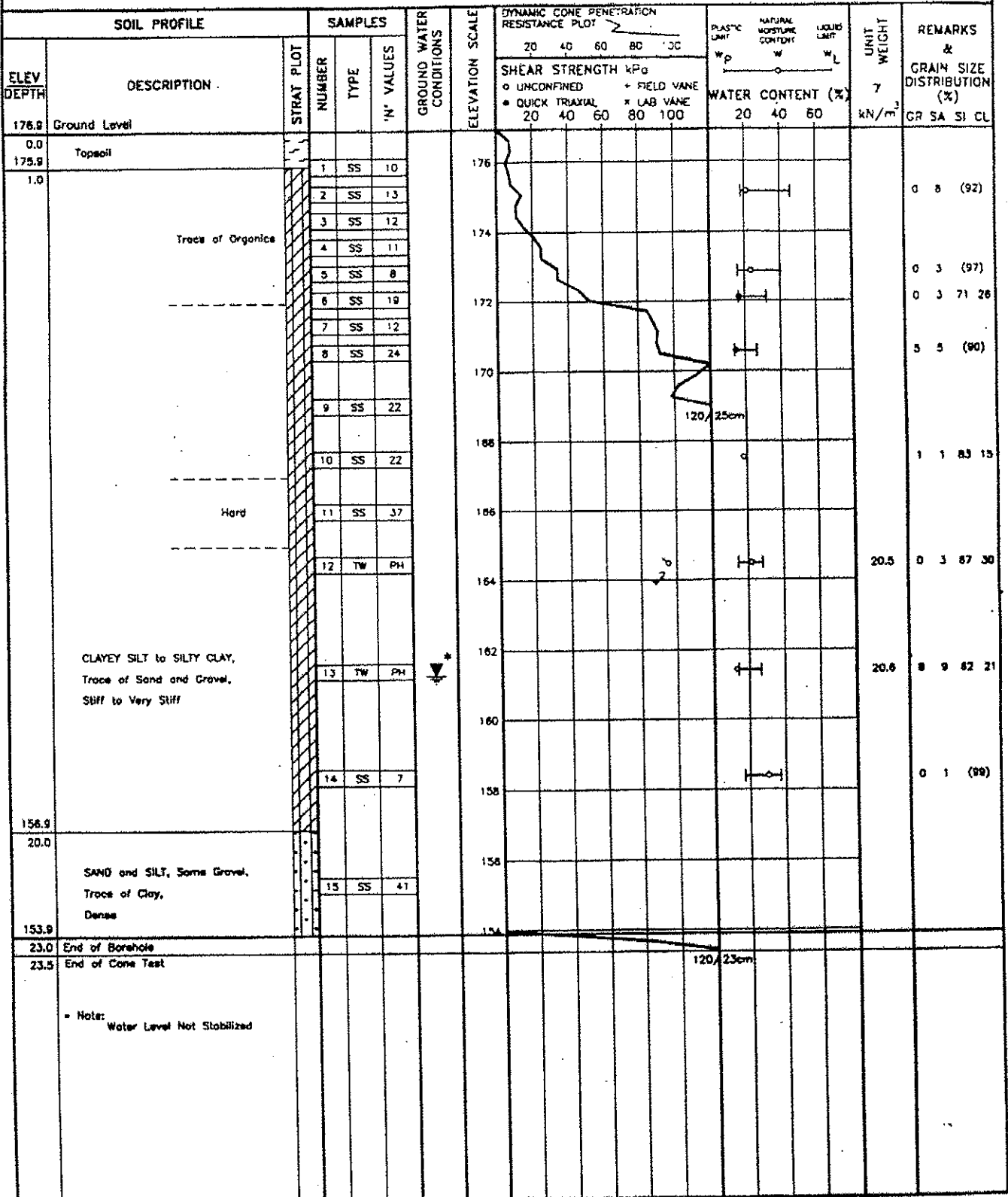
20
15-25 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 204

1 OF 1

METRIC

W.P. 171 - 90 - 01 LOCATION CO - ORDS. N 4 784 925.5; E 327 187.5 ORIGINATED BY P. V.
DIST 4 HWY 406 BOREHOLE TYPE CONTINUOUS FLIGHT AUGER (H. S.) & CONE TEST COMPILED BY M. Y.
DATUM GEODETIC DATE 87 11 17 to 87 11 18 CHECKED BY P. V.



+3, x3 Numbers refer to
Sensitivity

20
15-25 (X) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 205

1 OF 1

METRIC

W.P. 171 - 90 - 01 LOCATION CO - ORDS. N 4 765 038.5; E 327 121.0 ORIGINATED BY S.M. M.
DIST 4 HWY 406 BOREHOLE TYPE WASH BORE NX CASING, BXL ROCK CORE & CONE TEST COMPILED BY M.J.
DATUM GEODETIC DATE 88.11.16 to 88.11.23 CHECKED BY P.P.

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 7 KN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			20 40 60 80 100	20 40 60 80 100					
173.3	Water Level												
0.0													
164.0	Canal Bed												
9.3	Organic Silt		1	SS									0 14 60 28
			2	SS	97								29 14 41 16
			3	SS	71								
	CLAYEY SILT to SILTY CLAY. With Occasional Silt Seams. Some Sand and Gravel. Hard to Very Stiff		4	SS	28								
			5	SS	22								15 52 26 7
156.8			6	SS	38								
16.5			7	SS	69								
	SAND and SILT. Some Gravel. Trace of Clay. Very Dense												
151.9			8	SS	91								80 26 11 3
21.4			9	RC	83%								RC 0%
			10	RC	REC								RC 14%
			11	RC	REC								RC 0%
149.8			12	RC	50%								RC 22%
	DOLOSTONE BEDROCK												
23.5	End of Borehole												

+J, x, s; Numbers refer to
Sensitivity

20
15-0-5 (X) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 206

1 OF 1

METRIC

W.P. 121 - 90 - 01 LOCATION CO - ORDS. N 4 764 998.0; E 327 139.5 ORIGINATED BY S.W.
DIST 4 HWY 406 BOREHOLE TYPE WASH BORE NX CASING & CONE TEST COMPILED BY U.V.
DATUM GEOIDETIC DATE 88 11 23 CHECKED BY F.P.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PASTIC UNIT W _p	NATURAL MOISTURE CONTENT W	SOLID UNIT W _s	UNIT WEIGHT 7 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	20 40 60 80 100					
173.3	Water Level												
0.0													
163.3	Concl Bed												
10.0	Organic Silt		1	SS									
	CLAYEY SILT to SILTY CLAY, Some Sand and Gravel, Hard		2	SS	56								
			3	SS	48								
	Firm to Very Stiff		4	SS	6								
			5	SS	23								
154.8			6	SS	44								14 20 45 21
18.7	End of Borehole												

+3, x3, Numbers refer to
Sensitivity

20
15-20 (X) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 207

1 OF 1

METRIC

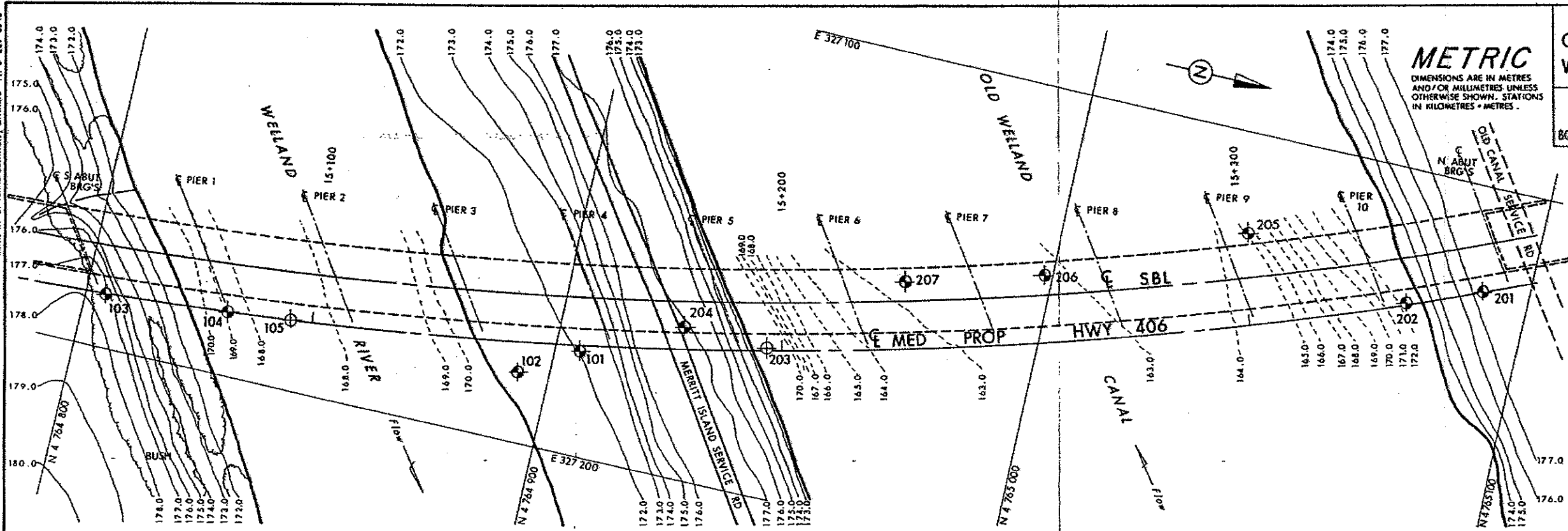
W.P. 171 - 90 - 01 LOCATION CO - ORDS. N 4 784 989.5; E 327 147.5 ORIGINATED BY S.M.
 DIST 4 HWY 406 BOREHOLE TYPE WASH BORE NX CASING & CONE TEST COMPILED BY M.L.
 DATUM GEODETTIC DATE 88.11.24 CHECKED BY J.D.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL MOISTURE CONTENT		UNIT WEIGHT γ KN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	20 40 60 80 100	20 40 60 80 100	W _p W W _L		
173.3	Water Level											
0.0												
182.8	Canal Bed											
10.4	Organic Silt		1	SS	38							0 12 85 23
	CLAYEY SILT to SILTY CLAY, Some Sand and Gravel, Hard		2	SS	47							
			3	SS	47							
	Firm		4	SS	7							0 20 80 20 0 38 52 10
135.8			5	SS								
17.5	End of Borehole											

+3, x⁵ Numbers refer to
Sensitivity

20
13-5 (X) STRAIN AT FAILURE
10

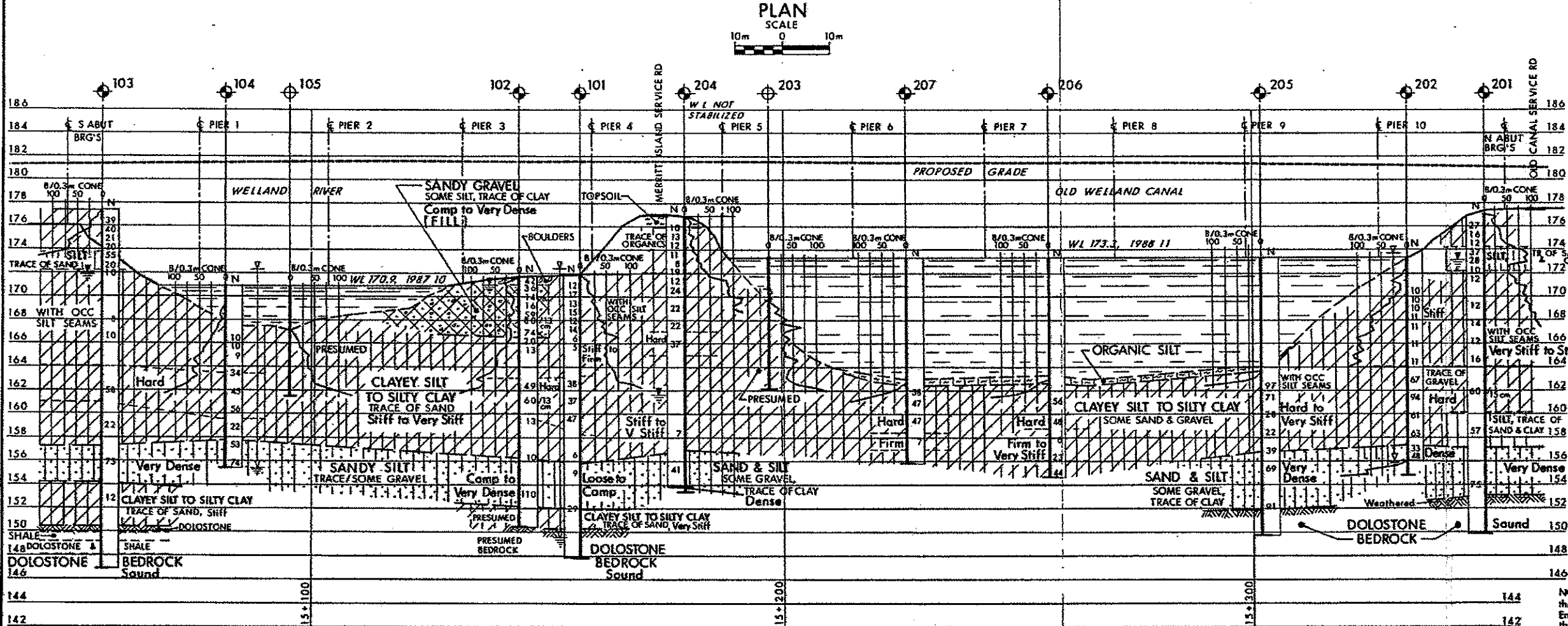
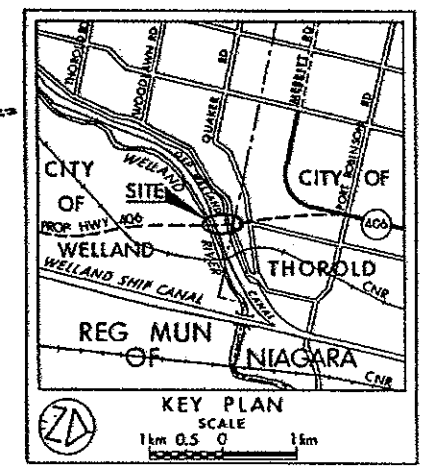
MINISTRY OF TRANSPORTATION, ONTARIO PH-0-207 4810



CONT No
WP No 171-90-01

WELLAND RIVER &
OLD WELLAND CANAL
BORE HOLE LOCATIONS & SOIL STRATA

SHEET



- LEGEND**
- Bore Hole
 - ⊕ Dynamic Cone Penetration Test (Cone)
 - ⊙ Bore Hole & Cone
 - N Blows/0.3m (Std Pen Test, 475 J/blow)
 - CONE Blows/0.3m (60° Cone, 475 J/blow)
 - W.L. at time of investigation
 - Head
 - ARTESIAN CONDITION
 - Encountered

No	ELEVATION	CO-ORDINATES	
		NORTH	EAST
101	171.9	4 764 905.4	327 177.5
102	171.7	4 764 893.6	327 185.0
103	177.4	4 764 804.1	327 188.6
104	170.9	4 764 830.6	327 186.4
105	170.9	4 764 844.0	327 185.0
201	177.1	4 765 090.2	327 121.8
202	173.4	4 765 073.8	327 128.3
203	173.4	4 764 944.0	327 168.0
204	176.9	4 764 925.5	327 167.5
205	173.3	4 765 038.5	327 121.0
206	173.3	4 764 998.0	327 139.5
207	173.3	4 764 969.5	327 147.5

NOTE

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

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section 102-2 of Form 100.

DATE	BY	DESCRIPTION
1991 05 06	DATE	1991 05 06
1991 05 06	CHECKED	1991 05 06
1991 05 06	DRAWN	1991 05 06

Geocres No 30M3-192

HWY No 406 SBL DIST 4

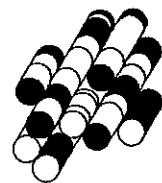
SUBAREA M.V. CHECKED BY DATE 1991 05 06 SITE 34-304

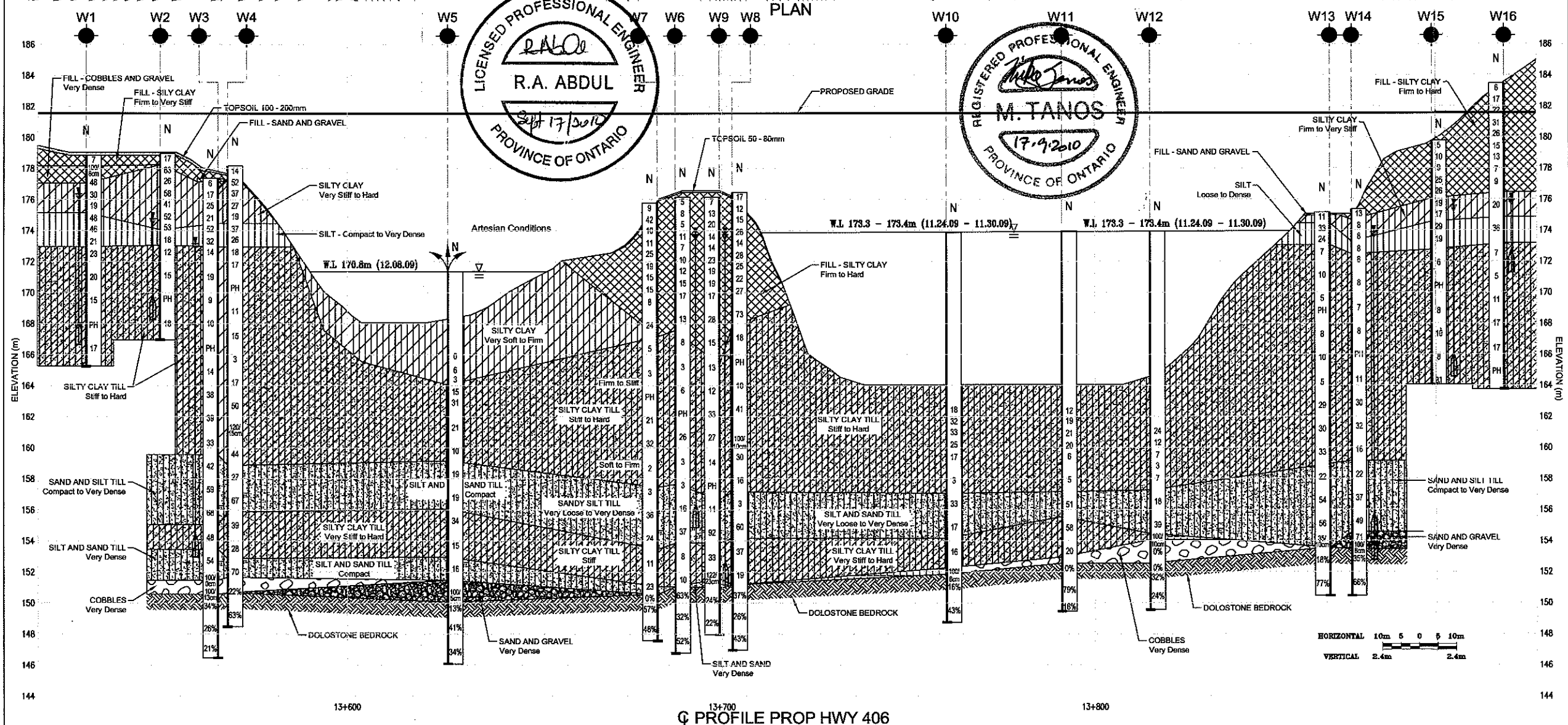
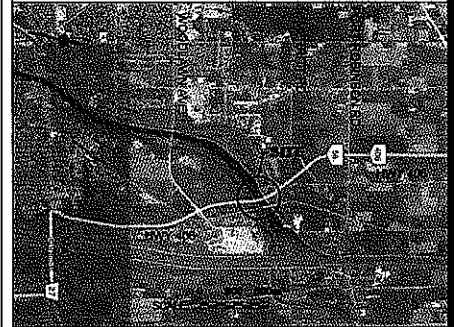
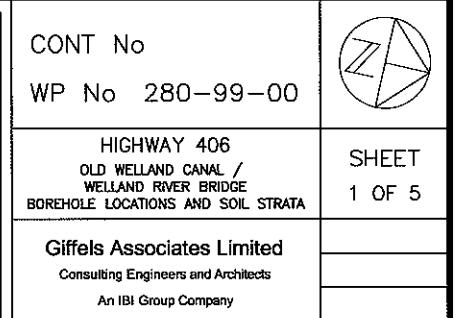
DRAWN BY CHECKED BY DATE 1991 05 06 DWG 2







APPENDIX D

Drawing

Terraprobe Inc.





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 (JAN. 2010)
	Piezometer
90%	Rock Quality Designation
A/R	Auger Refusal

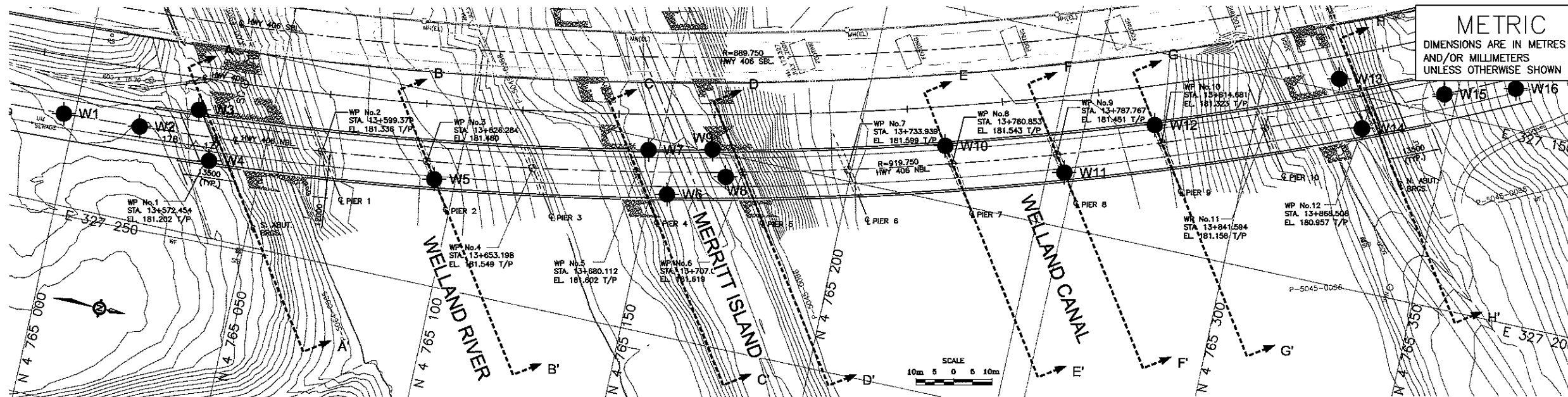
No	ELEV.	COORDINATES	
		NORTHING	EASTING
W1	178.3	4 764 997.1	327 226.3
W2	178.4	4 765 017.0	327225.3
W3	176.8	4 765 031.1	327 217.8
W4	177.6	4 765 036.5	327 230.2
W5	170.8	4 765 094.5	327 222.4
W6	175.6	4 765 154.5	327 213.4
W7	175.2	4 765 147.3	327 203.1
W8	175.9	4 765 168.4	327 205.7
W9	175.6	4 765 163.6	327 199.4
W10	173.3	4 765 222.6	327 185.7
W11	173.4	4 765 254.3	327 185.9
W12	173.4	4 765 274.6	327 168.7
W13	174.6	4 765 319.2	327 146.7
W14	174.8	4 765 327.6	327 158.2
W15	179.2	4 765 346.7	327 144.9
W16	182.9	4 765 364.8	327 139.5

NOTE

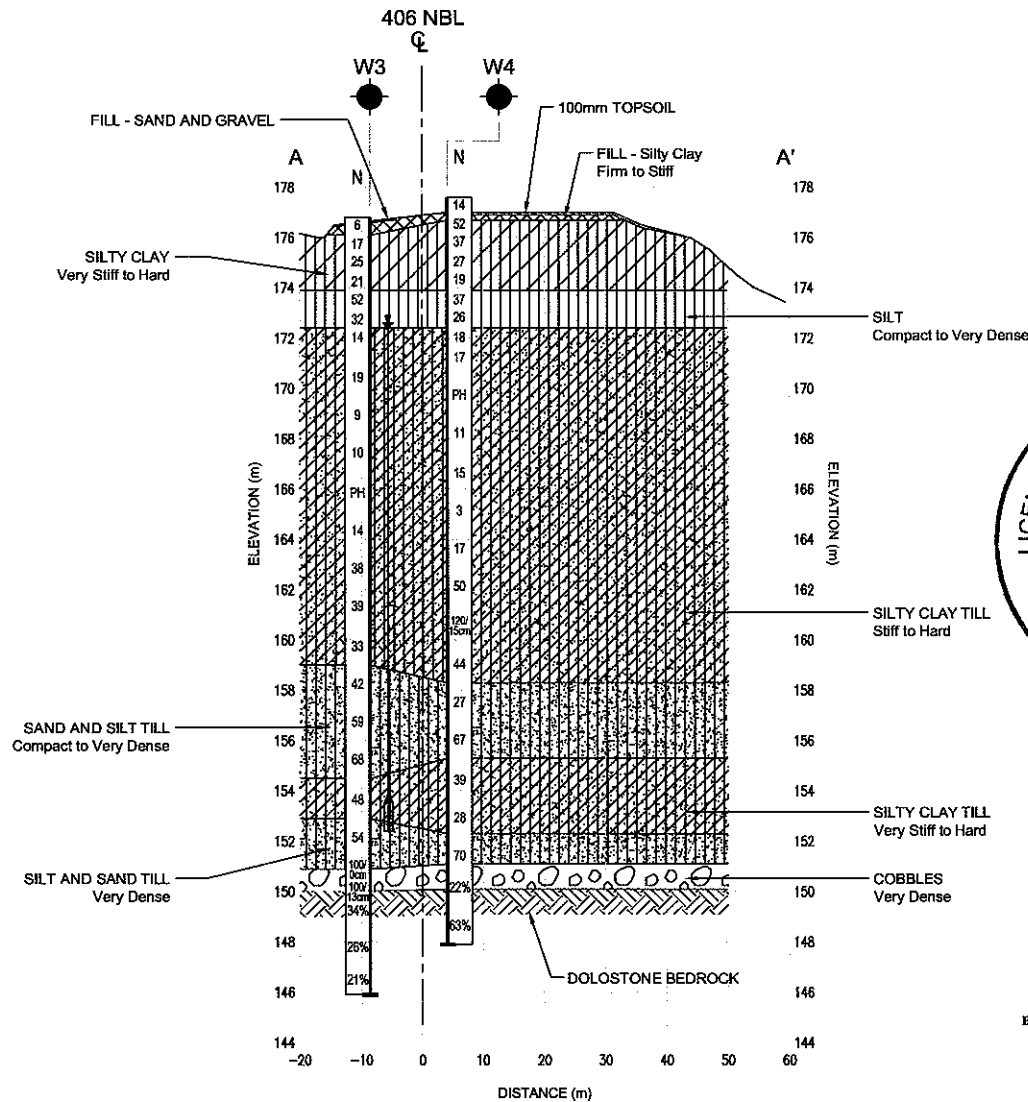
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

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

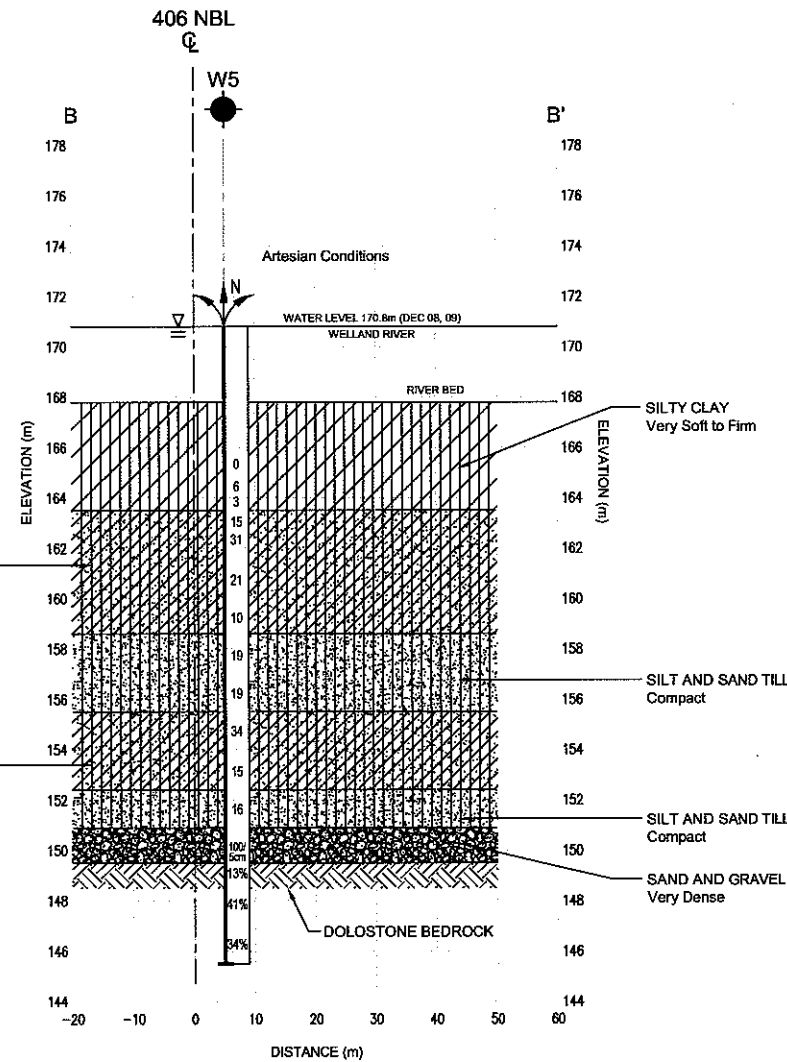
REVISIONS				
	DATE	BY	DESCRIPTION	
DESIGN	R.A	CODE	CHBDC2006	LOAD
				DATE SEPT. 2010
DRAWN	B.S	CHK	R.A	STRUCT
				34-304/1
				GEOCRETS 30M3-192



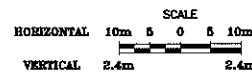
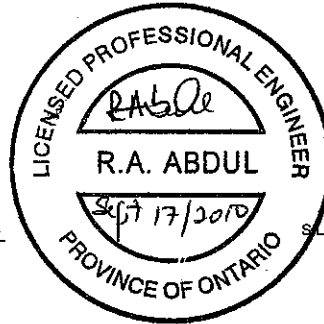
PLAN



SECTION A-A'



SECTION B-B'

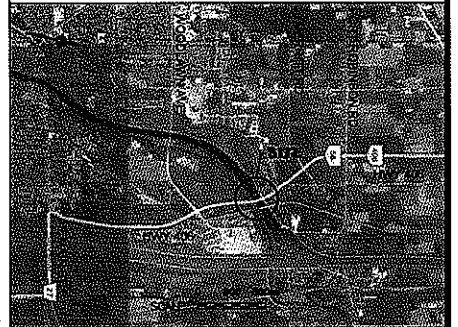


METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETERS
UNLESS OTHERWISE SHOWN

CONT No
WP No 280-99-00

HIGHWAY 406
OLD WELLAND CANAL /
WELLAND RIVER BRIDGE
BOREHOLE LOCATIONS AND SOIL STRATA

Giffels Associates Limited
Consulting Engineers and Architects
An IBI Group Company



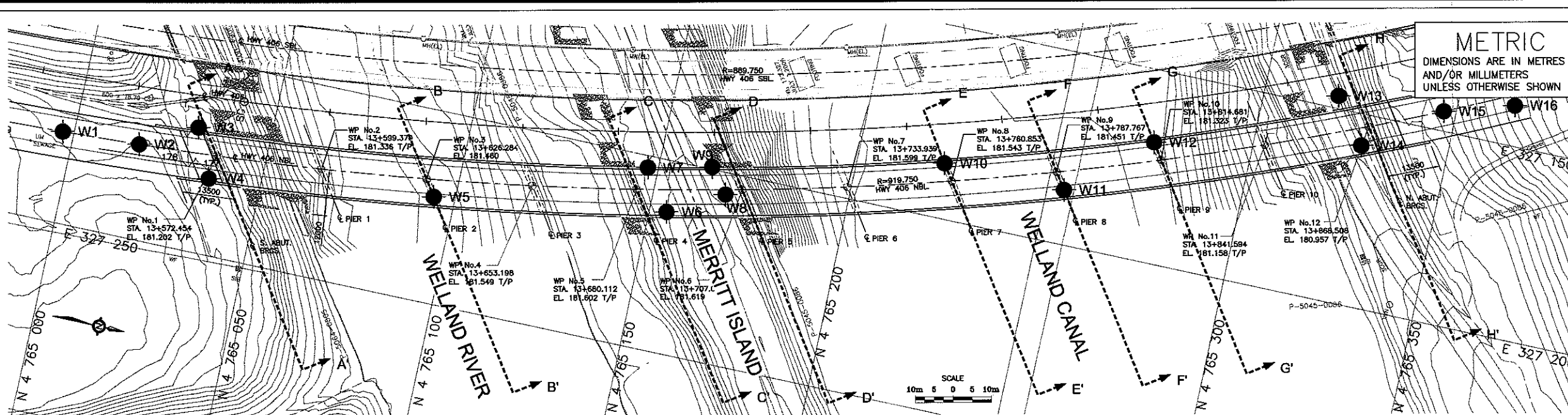
KEY PLAN

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 (JAN. 2010)
	Piezometer
	90% Rock Quality Designation
	A/R Auger Refusal

No	ELEV.	COORDINATES	
		NORTHING	EASTING
W1	178.3	4 764 997.1	327 226.3
W2	178.4	4 765 017.0	327 225.3
W3	176.8	4 765 031.1	327 217.8
W4	177.6	4 765 036.5	327 230.2
W5	170.8	4 765 094.5	327 222.4
W6	175.8	4 765 154.5	327 213.4
W7	175.2	4 765 147.3	327 203.1
W8	175.9	4 765 168.4	327 205.7
W9	175.6	4 765 163.6	327 199.4
W10	173.3	4 765 222.6	327 185.7
W11	173.4	4 765 254.3	327 185.9
W12	173.4	4 765 274.6	327 168.7
W13	174.6	4 765 319.2	327 146.7
W14	174.8	4 765 327.6	327 158.2
W15	179.2	4 765 346.7	327 144.9
W16	182.9	4 765 364.8	327 139.5

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

REVISIONS			
DATE	BY	DESCRIPTION	
DESIGN R.A.	CODE CHBDC2006	LOAD	DATE SEPT. 2010
DRAWN B.S.	CHK R.A.	STRUCT 34-304/1	GEOSCI 30M3-192

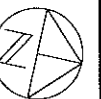


PLAN

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

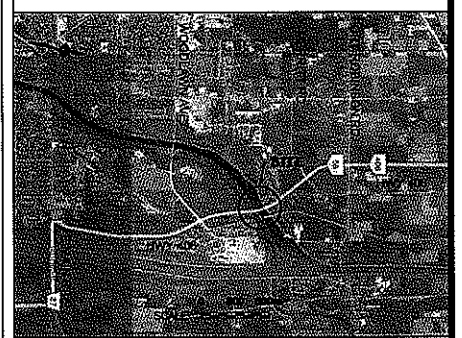
CONT No
WP No 280-99-00

HIGHWAY 406
OLD WELLAND CANAL /
WELLAND RIVER BRIDGE
BOREHOLE LOCATIONS AND SOIL STRATA



SHEET
3 OF 5

Giffels Associates Limited
Consulting Engineers and Architects
An IBI Group Company



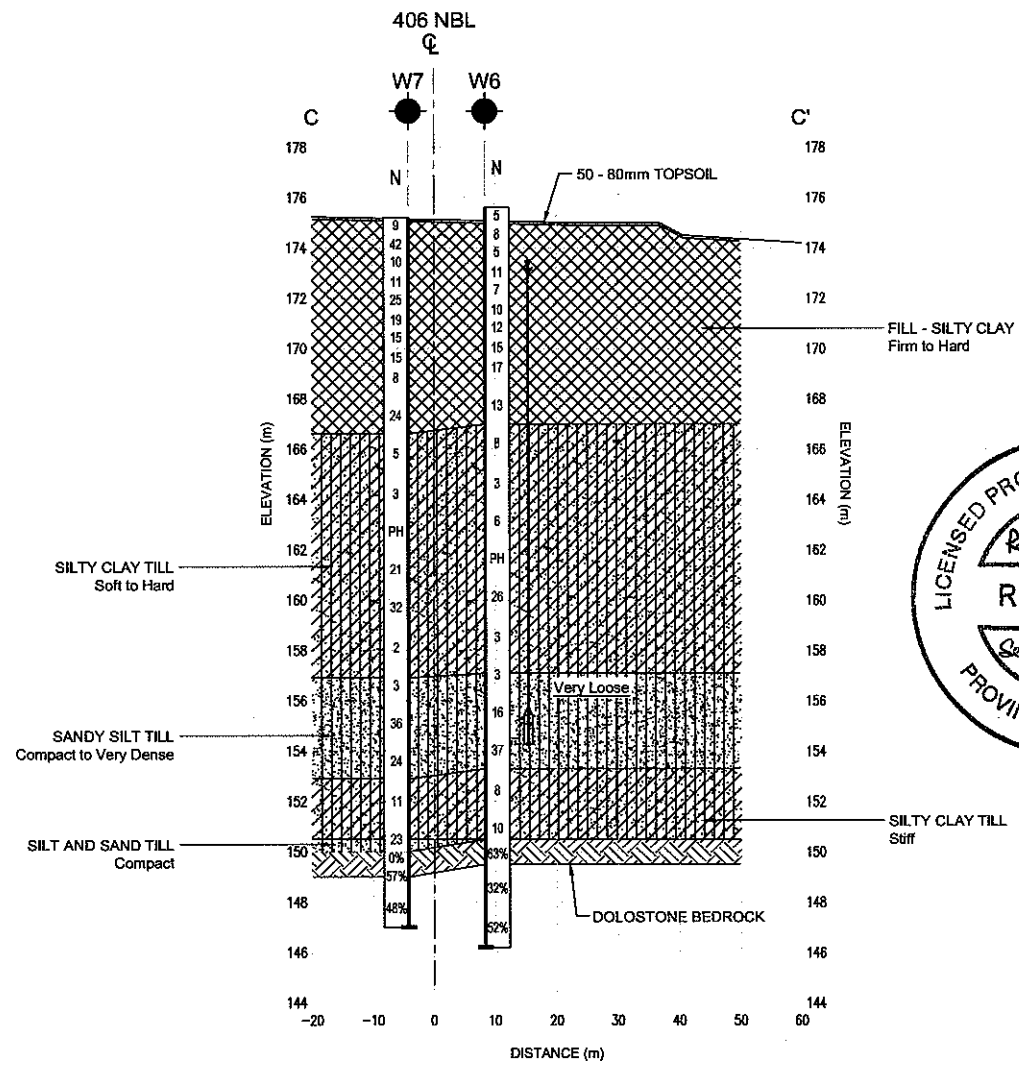
KEY PLAN

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 (JAN. 2010)
	Piezometer
	90% Rock Quality Designation
	A/R Auger Refusal

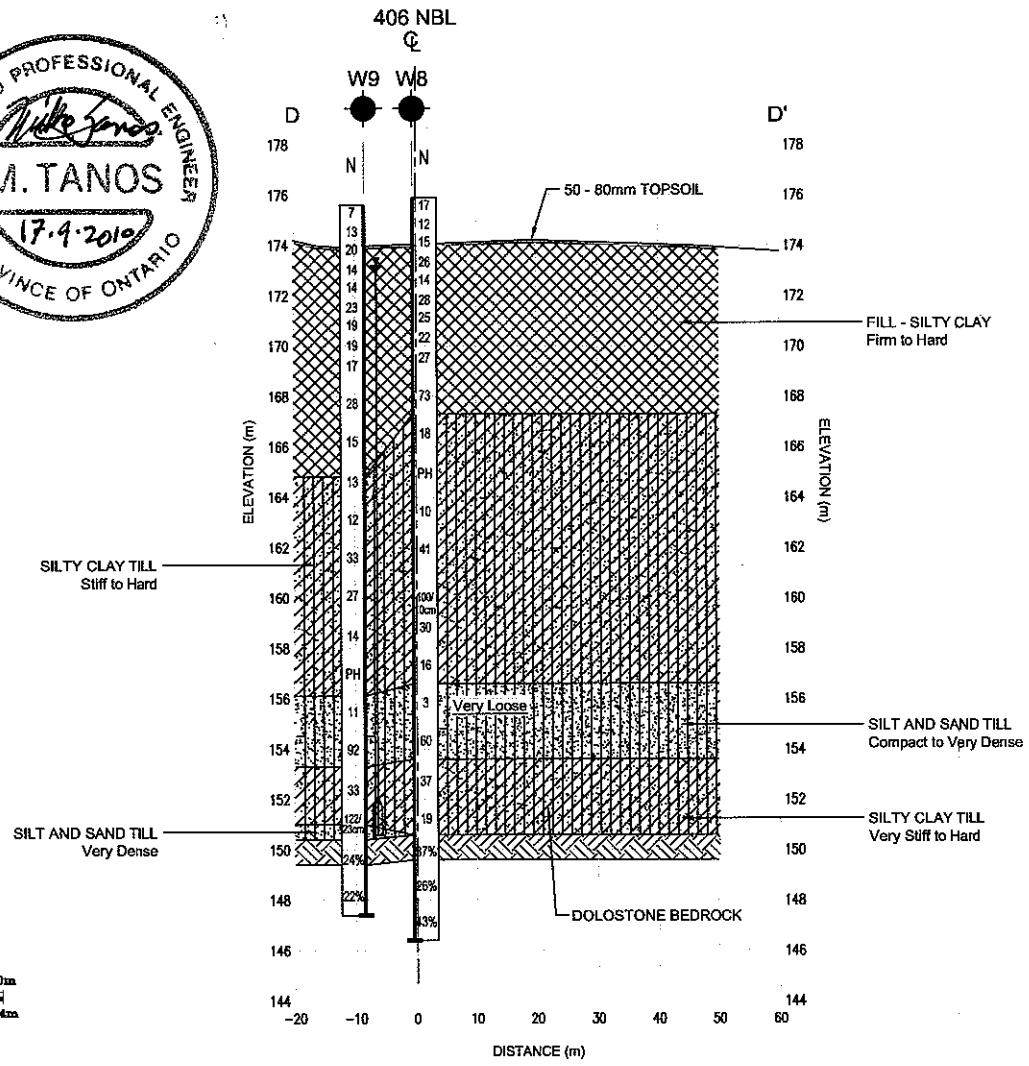
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		NORTHING	EASTING
W1	178.3	4 764 997.1	327 226.3
W2	178.4	4 765 017.0	327 225.3
W3	178.8	4 765 031.1	327 217.8
W4	177.6	4 765 036.5	327 230.2
W5	170.8	4 765 094.5	327 222.4
W6	175.6	4 765 154.5	327 213.4
W7	175.2	4 765 147.3	327 203.1
W8	175.9	4 765 168.4	327 205.7
W9	175.6	4 765 163.6	327 199.4
W10	173.3	4 765 222.6	327 185.7
W11	173.4	4 765 254.3	327 185.9
W12	173.4	4 765 274.6	327 168.7
W13	174.6	4 765 319.2	327 146.7
W14	174.8	4 765 327.6	327 158.2
W15	179.2	4 765 346.7	327 144.9
W16	182.9	4 765 364.8	327 139.5

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

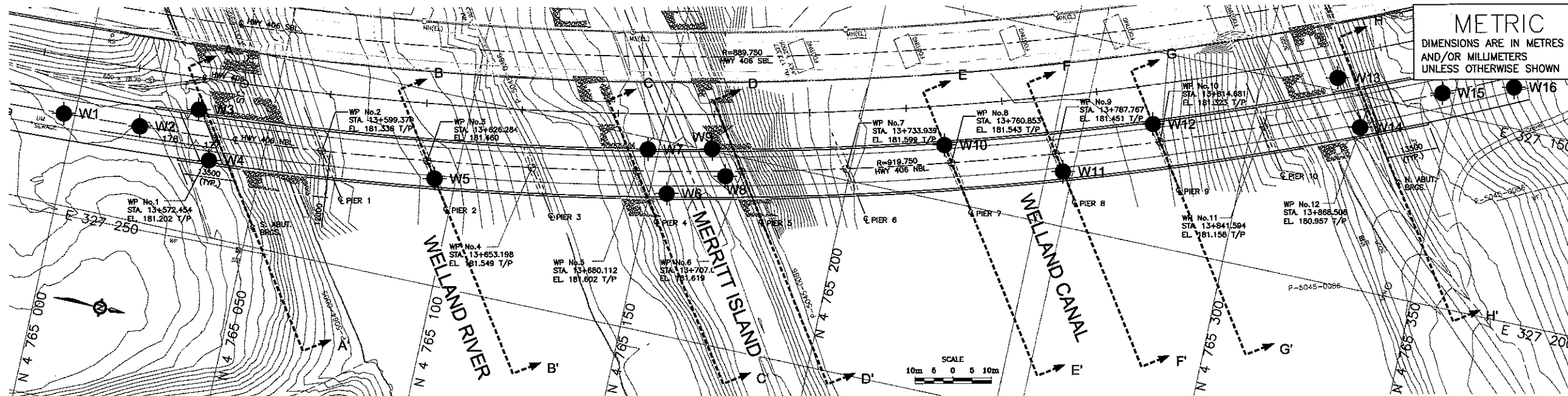
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DATE	BY	DESCRIPTION	
DESIGN R.A.	CODE CHBDC2006	LOAD	DATE SEPT. 2010
DRAWN B.S.	CHK R.A.	STRUCT 34-304/1	GEOTECH 30M3-192



SECTION C-C'



SECTION D-D'



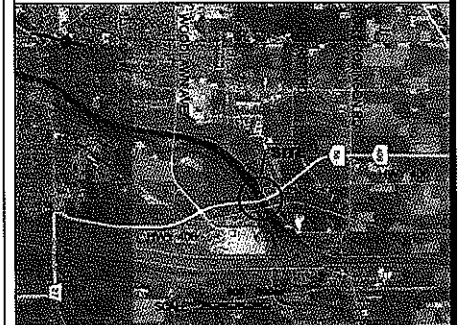
PLAN

CONT No
WP No 280-99-00

HIGHWAY 406
OLD WELLAND CANAL /
WELLAND RIVER BRIDGE
BOREHOLE LOCATIONS AND SOIL STRATA

Giffels Associates Limited
Consulting Engineers and Architects
An IBI Group Company

Terraprobe
Consulting Geotechnical & Environmental Engineering
Construction Materials Engineering, Inspection & Testing



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 (JAN. 2010)
- 90% Rock Quality Designation
- A/R Auger Refusal

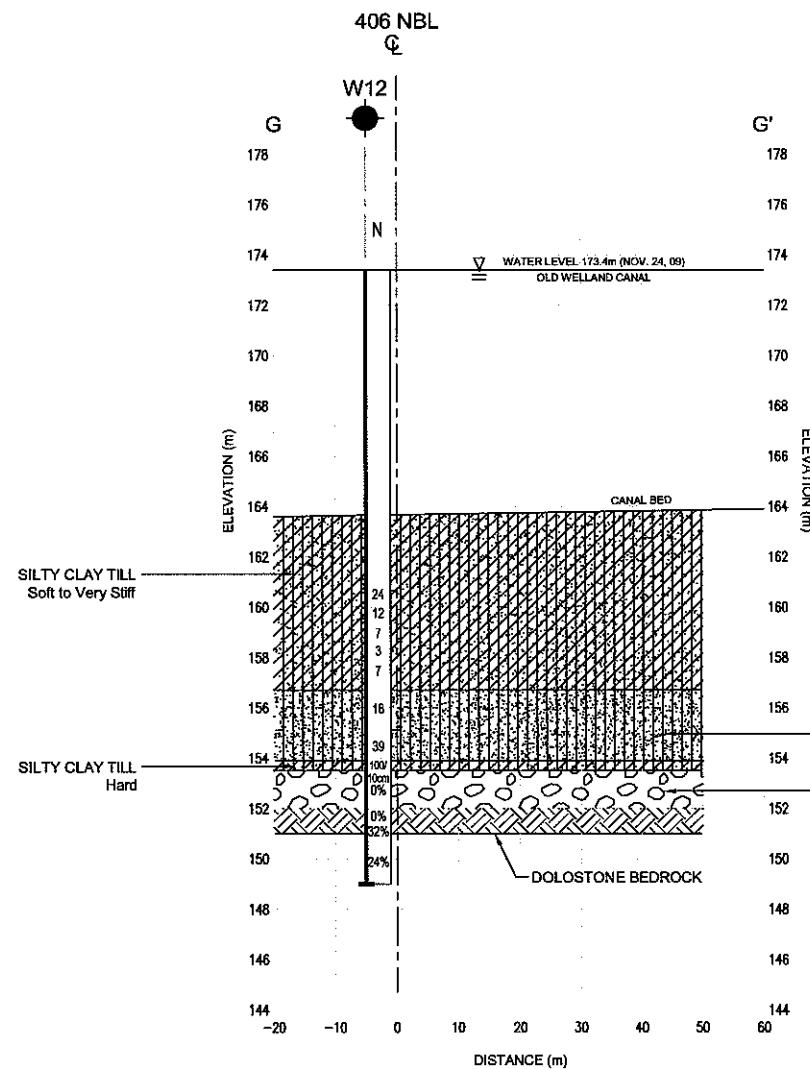
No	ELEV.	COORDINATES	
		NORTHING	EASTING
W1	178.3	4 764 997.1	327 226.3
W2	178.4	4 765 017.0	327 225.3
W3	176.8	4 765 031.1	327 217.8
W4	177.6	4 765 036.5	327 230.2
W5	170.8	4 765 094.5	327 222.4
W6	175.6	4 765 154.5	327 213.4
W7	175.2	4 765 147.3	327 203.1
W8	175.9	4 765 168.4	327 205.7
W9	175.6	4 765 163.6	327 199.4
W10	173.3	4 765 222.6	327 185.7
W11	173.4	4 765 254.3	327 185.9
W12	173.4	4 765 274.6	327 188.7
W13	174.6	4 765 319.2	327 146.7
W14	174.8	4 765 327.6	327 158.2
W15	179.2	4 765 346.7	327 144.9
W16	182.9	4 765 364.8	327 139.5

NOTE

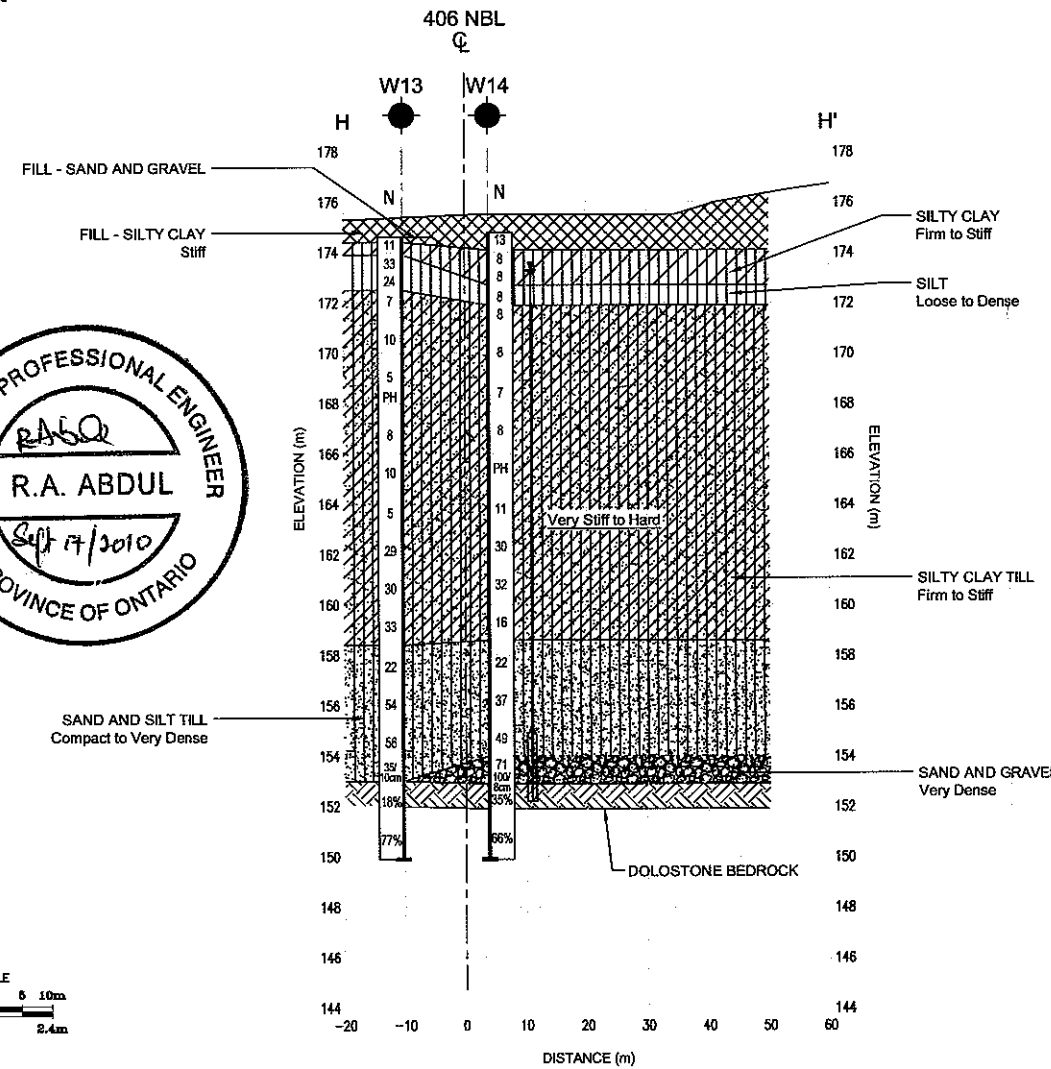
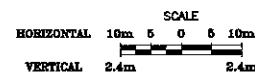
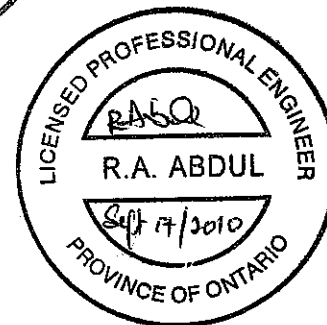
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

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

REVISIONS	DATE	BY	DESCRIPTION
DESIGN	R.A.	CODE	CHBDC2006
DRAWN	B.S.	CHK	R.A.
		STRUCT	34-304/1
			GEOCRES 3043-192



SECTION G-G'

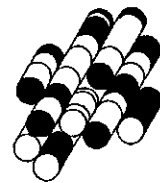


SECTION H-H'

APPENDIX E

Foundation Comparison

Terraprobe Inc.



COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

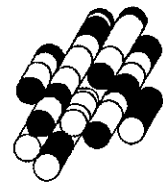
Foundation Element	Driven Piles	Augered Caissons	Footing on Native Soil	Footing on Engineered Fill
North and South Abutments and Piers on Merritt Island	<p>Advantages:</p> <ul style="list-style-type: none"> i. High geotechnical resistances available by driving piles to bedrock. ii. Readily installed. iii. Reliable performance and low risk based on performance of existing bridge. <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> <p>None</p> <p>Disadvantages:</p> <ul style="list-style-type: none"> i. High unit cost compared to other footing options such as driven piles. ii. Relatively high construction effort required to install caissons to bedrock compared to driven piles. iii. Higher risk of encountering potential construction problems compared to driven piles. 	<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> <p>None</p> <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Unreliable performance and high risk due to settlement sensitive soils. Potential for unacceptable settlements and differential settlements.
Piers (In Water)	<p>Advantages:</p> <ul style="list-style-type: none"> i. High geotechnical resistances available by driving piles to bedrock. ii. Readily installed iii. Reliable performance and low risk based on performance of existing bridge. iv. No problems associated with scour. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Cofferdam required to facilitate construction. ii. Construction concerns related to the possibility of piles being obstructed by a boulder during driving. iii. Requires proper corrosion protection. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. Cofferdam not required. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher unit cost compared to other footing options such as driven piles. ii. Relatively high construction effort required to install caissons to bedrock compared to driven piles iii. Higher risk of encountering potential construction problems compared to driven piles. 	<p>Advantages:</p> <p>Not applicable</p>	<p>Advantages:</p> <p>Not applicable</p>



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. A layer of layer of cobbles was also encountered above the bedrock in Boreholes W3, W4, W10, W11 and W12. 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 bedrock and further driving is likely to cause damage to the pile, pile driving should cease and the contract administrator should be notified.



Reference is made to the following Provincial Standard:

- OPSSS 517, November 2005

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

Artesian Conditions

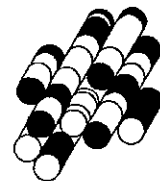
“The Contractor is informed that artesian conditions exist in the lower granular deposits encountered below the Welland River. Artesian conditions were encountered in Borehole W5 in the lower granular deposits overlying bedrock. Below Merritt Island excess hydrostatic pressure was encountered in the underlying sandy silt to silty sand till confined between the less permeable upper and lower silty clay deposits. The groundwater and soil conditions are described in the Foundation Investigation Report prepared for this site”.



APPENDIX G

Settlement Monitoring Programme

Terraprobe Inc.



SUPPLY AND INSTALLATION OF EMBANKMENT MONITORING EQUIPMENT –
Item No.

Special Provision

1.0 GENERAL

1.1 Scope

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

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

1.2 Purpose

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

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

1.3 Personnel

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

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

1.4 Or equal

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

1.5 Notification

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

1.6 Submission Requirements

The Contractor shall submit details of proposed installations including:

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

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

1.7 Subsurface Conditions

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

- Foundation Investigation Report – Old Welland Canal/Welland River Bridge, Highway 406 Twinning, Ontario. Site No. 34-304/1, W.P. 280-99-00, Geocres. No. 30M3-253, dated September 17, 2010, by Terraprobe Inc.

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

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

1.8 Equipment Operation and Weather Conditions

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

2.0 INSTALLATION

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

Table 2.1 – Instrument & Benchmark Quantities and Locations

INSTRUMENT I.D.	STATION	OFFSET FROM CENTRELINE	NO. OF INSTRUMENTS			
			SP	VWP	SSP	BM
North Approach						
SP1	406 NBL 13+875	0	1			
VWP1	406 NBL 13+885	0		1		
SSP1	406 NBL 13+885	Outside of construction area			1	
SP2	406 NBL 13+890	0	1			
BM1	N/A	N/A				1
South Approach						
SP3	406 NBL 13+560	0	1			
VWP2	406 NBL 13+550	0		1		
SSP2	406 NBL 13+550	Outside of construction area			1	
SP4	406 NBL 13+540	0	1			
SP5	406 NBL 13+520	0	1			
BM2	N/A	N/A				1
Total Instruments			5	2	2	2

2.1 Instrument Location

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

2.2 Survey Benchmarks (BM)

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

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

2.3 Accuracy of Surveying for Elevations

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

2.4 Monitoring Instrument Location

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

2.5 Materials and Equipment

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

2.6 Underground Utilities

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

2.7 Marking and Labelling

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

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

2.8 Protection of Instruments

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

2.9 Boreholes

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

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

2.10 Installation Program

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

Table 2.2 – Installation Program

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

3.0 BENCHMARK (BM) – SUPPLY & INSTALLATION

3.1 GENERAL

3.1.1 Scope

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

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

3.1.2 General Procedure

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

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

3.1.3 Location

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

Table 3 – Approximate Bench Mark Locations

Station	Offset (m)	No. of BM	Estimated Rod Anchor Elevation (m)
Outside of Construction Area	N/A	BM1	157.0
Outside of Construction Area	N/A	BM2	157.0

3.2 MATERIALS

3.2.1 General

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

3.2.2 Rod

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

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

3.2.3 Sand

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

3.2.4 Grout

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

3.2.5 Rod Anchor Grout

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

3.2.6 Friction Reducing Sleeve

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

3.3 INSTALLATION

3.3.1 General

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

3.3.2 Borehole Installation

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

3.3.3 Rod

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

3.3.4 Rod Anchor

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

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

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

3.3.5 Friction Reducing Sleeve

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

3.3.6 Installation Details

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

3.4 COORDINATION WITH MONITORING

3.4.1 Notification

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

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

3.4.2 Monitoring

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

3.5 REPORTING

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

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

4.0 SETTLEMENT PLATES (SP) – SUPPLY & INSTALLATION

4.1 GENERAL

4.1.1 Scope

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

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

4.1.2 General Procedure

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

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

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

4.1.3 Location

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

Table 4 – Approximate Settlement Plate Locations

Station	Offset (m)	No. of SP	Estimated Fill Thickness (m)*
North Approach			
406 NBL 13+875	0	1	9.0
406 NBL 13+890	0	1	3.5
South Approach			
406 NBL 13+560	0	1	9.5
406 NBL 13+540	0	1	3.0
406 NBL 13+520	0	1	3.0

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

4.2 MATERIALS

4.2.1 General

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

4.2.2 Plate

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

4.2.3 Rod

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

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

4.2.4 Friction Reducing Sleeve

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

4.2.5 Protective Surround

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

4.3 INSTALLATION

4.3.1 General

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

4.3.2 Settlement Plate

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

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

4.3.3 Rod

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

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

4.3.4 Friction Reducing Sleeve

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

4.4 EXTENSION OF ROD

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

4.4.1 Protective Surround

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

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

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

4.4.2 Installation Details

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

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

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

4.5 COORDINATION WITH MONITORING

4.5.1 Notification

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

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

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

4.5.2 Monitoring

Monitoring of the settlement plates shall be done by others. Monitoring shall be conducted during the embankment construction and preload period. A target settlement of 100 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)
North Abutment 406 NBL 13+885	0	1	178.5	169.0
South Abutment 406 NBL 13+550	0	1	178.5	169.0

Notes: * Offset from centerline of Highway 406 NBL.

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 VWPs shall not be installed closer than 1.5 m to the nearest adjacent edge of shoring or unwatering system.

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

5.3.2 Protection for Long-term Monitoring (Monitoring Shed)

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

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

5.3.3 Completion of Installation

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

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

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

5.4 COORDINATION WITH MONITORING

5.4.1 Notification

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

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

5.4.2 Monitoring

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

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

6.0 STANDPIPE PIEZOMETER (SSP) – SUPPLY & INSTALLATION

6.1 General

6.1.1 Scope

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

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

6.1.2 General Procedure

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

Standpipes shall be installed in vertical boreholes.

6.1.3 Location

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

Table 6 – Standpipe Piezometer Locations and Depths

Station	Offset* (m)	No. of SSP	Tip Elevations (m)
North Abutment 406 NBL 13+885	25	1	169.0
South Abutment 406 NBL 13+550	25	1	169.0

Note: * Approx. offset from centerline of Highway 406 NBL

6.2 MATERIALS

6.2.1 General

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

6.2.2 Pipe and Couplings

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

6.2.3 Perforated Section

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

6.2.4 Bottom Cap

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

6.2.5 Top Caps

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

6.2.6 Filter Sand

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

6.2.7 Bentonite

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

6.2.8 Grout

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

6.2.9 Protective Housing

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

6.3 INSTALLATION

6.3.1 General

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

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

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

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

6.4 COORDINATING WITH MONITORING

6.4.1 Notification

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

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

6.4.2 Monitoring

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

7.0 DECOMMISSING OF INSTRUMENTS

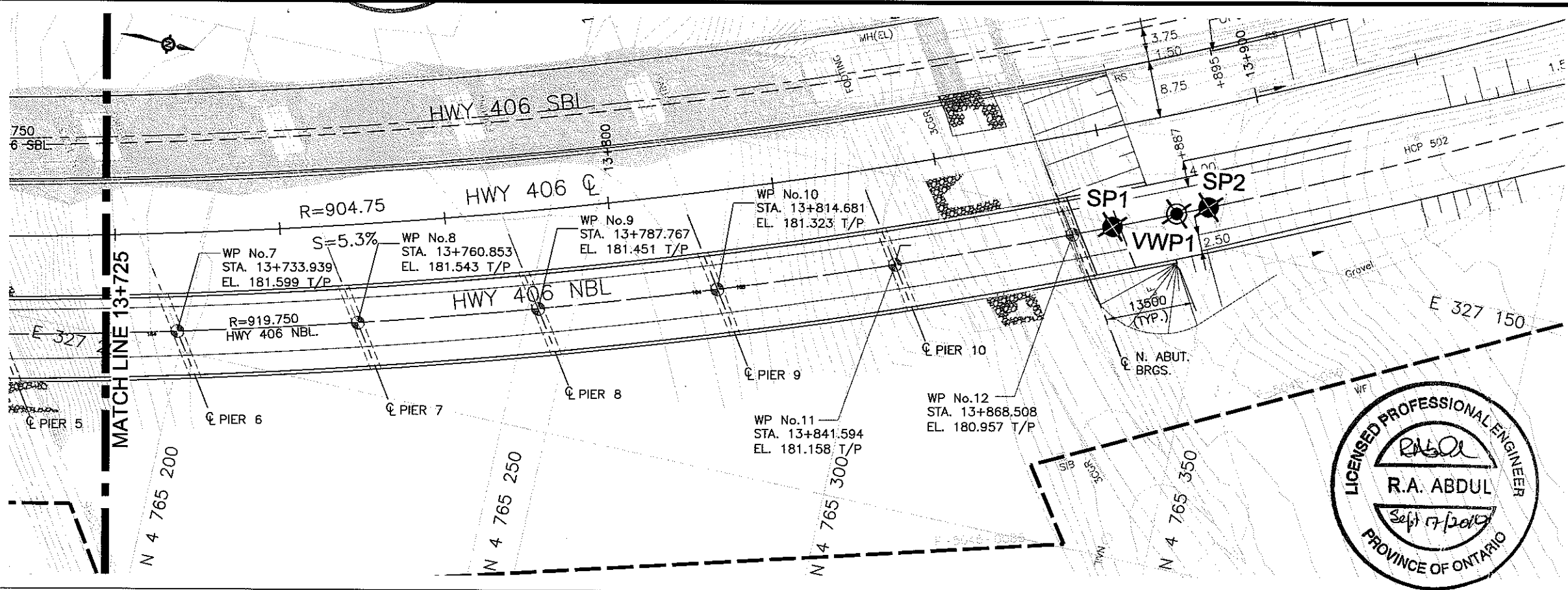
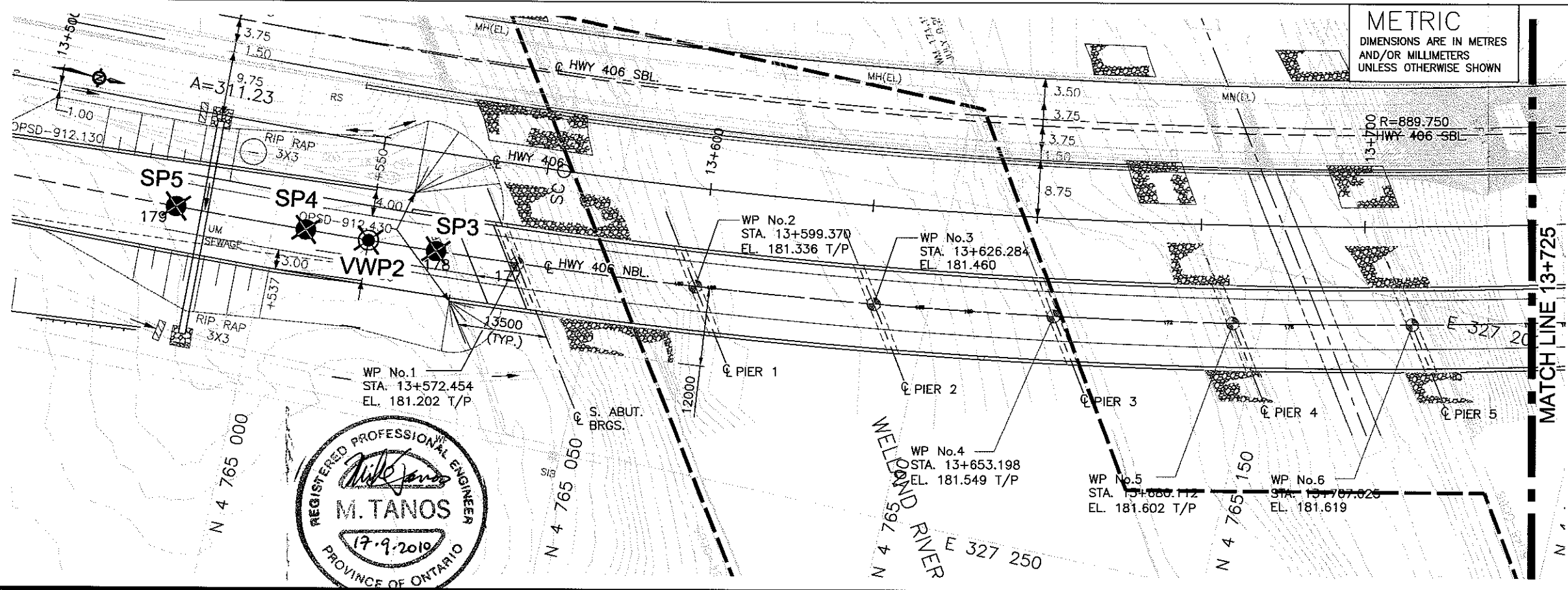
7.1 General

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

8.0 PAYMENT

8.1 Basis Of Payment

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



CONT No
WP No 280-99-00

HWY 406
OLD WELLAND CANAL/
WELLAND RIVER BRIDGE
SETTLEMENT MONITORING
INSTRUMENT LAYOUT



GENERAL NOTES:

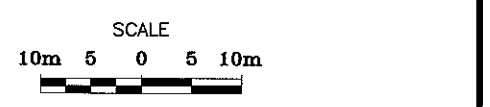
- THIS DRAWING TO BE READ IN CONJUNCTION WITH INSTRUMENT DETAILS DRAWING.

LEGEND

SP1 APPROXIMATE LOCATION OF SETTLEMENT PLATE (SP)

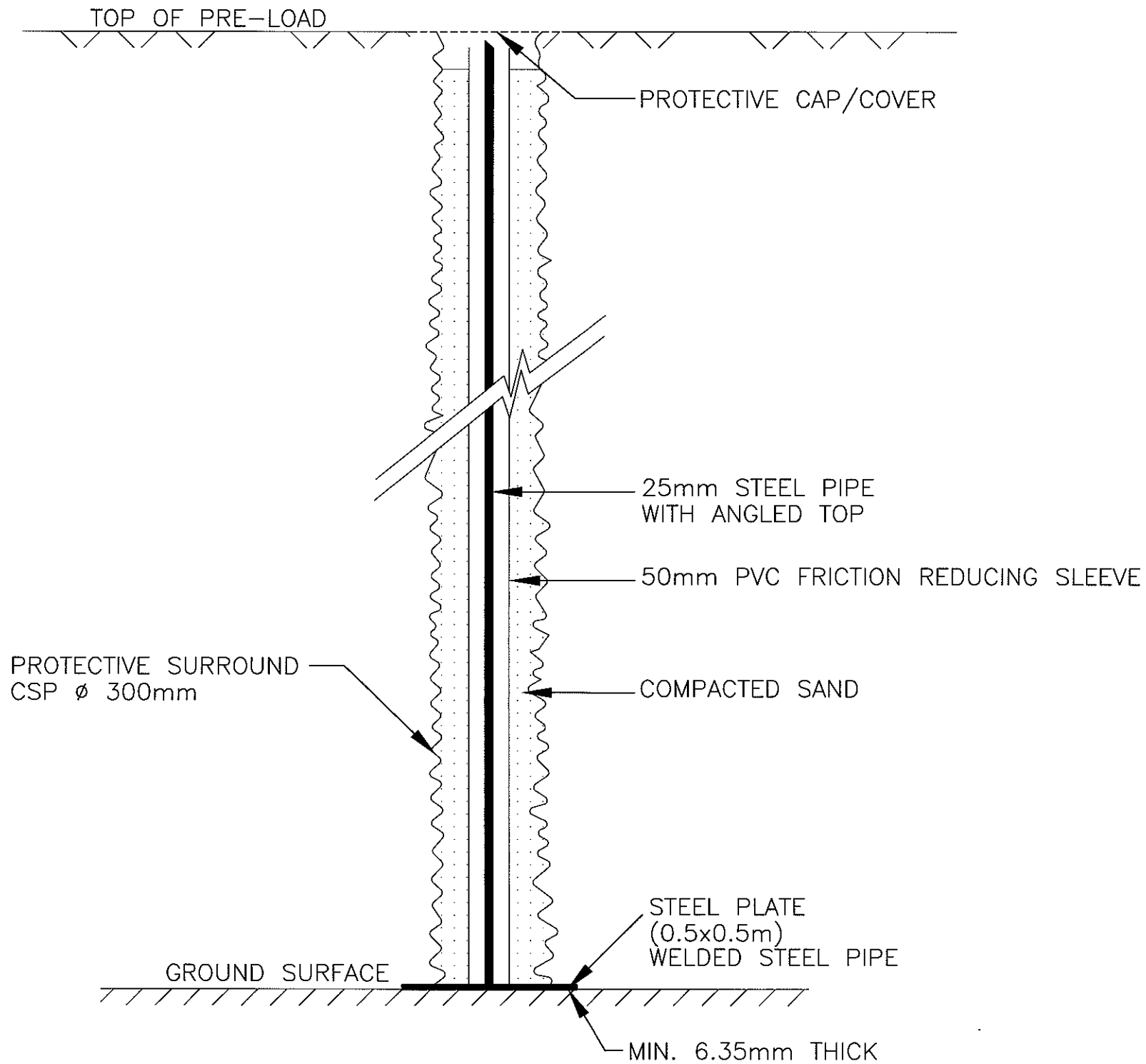
VWP1 APPROXIMATE LOCATION OF VIBRATING WIRE PIEZOMETER (VWP)

INSTRUMENT LOCATIONS			
I.D.	LOCATION	STATION	OFFSET FROM CENTRELINE(m)
NORTH APPROACH EMBANKMENT			
SP1	HWY406 NBL	13+875	0
SP2	HWY406 NBL	13+890	0
SOUTH APPROACH EMBANKMENT			
SP3	HWY406 NBL	13+560	0
SP4	HWY406 NBL	13+540	0
SP5	HWY406 NBL	13+520	0
VWP1	HWY406 NBL	13+885	0
VWP2	HWY406 NBL	13+550	0



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

C:\Users\jane\Documents\11-09-4135 HWY 406 - WELLAND BRIDGE\WELLAND SETTLEMENT PLATE\11-09-4135 HWY 406 SETTLEMENT PLATE DETAIL.DWG, KHAL



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

CONT No
WP No 280-99-00

HWY 406
OLD WELLAND CANAL/
WELLAND RIVER BRIDGE
SETTLEMENT MONITORING
INSTRUMENT DETAILS

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

GENERAL NOTES:

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

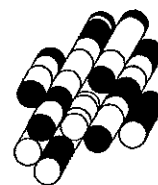


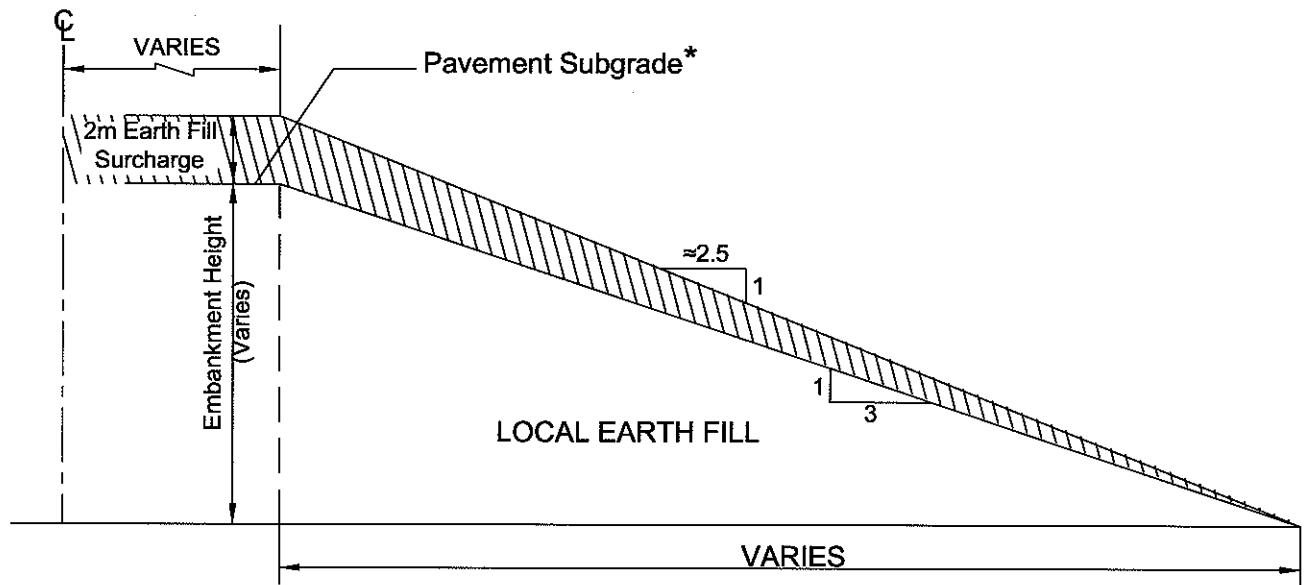
REVISIONS	DATE		BY		DESCRIPTION	
	DATE	BY	DESCRIPTION	DATE	DESCRIPTION	
DESIGN	R.A.	CODE	CHBDC2006	LOAD	DATE	SEPT. 2010
DRAWN	K.C.	CHK	R.A.	STRUCT		

APPENDIX H

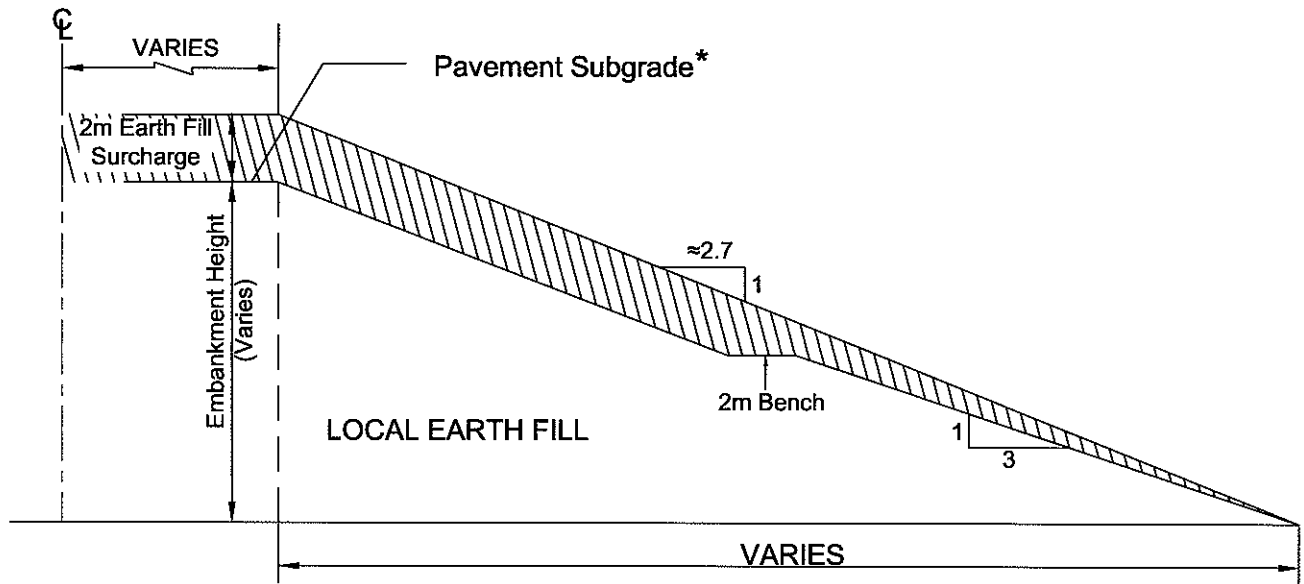
Figures

Terraprobe Inc.





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

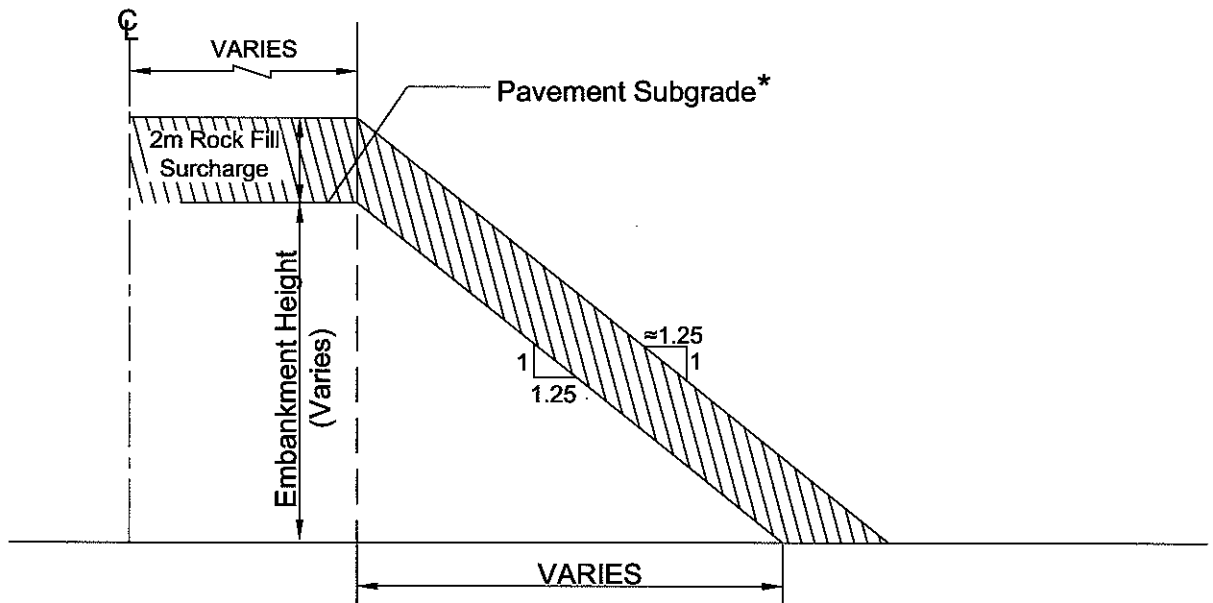
N.T.S

SURCHARGE ARRANGEMENT

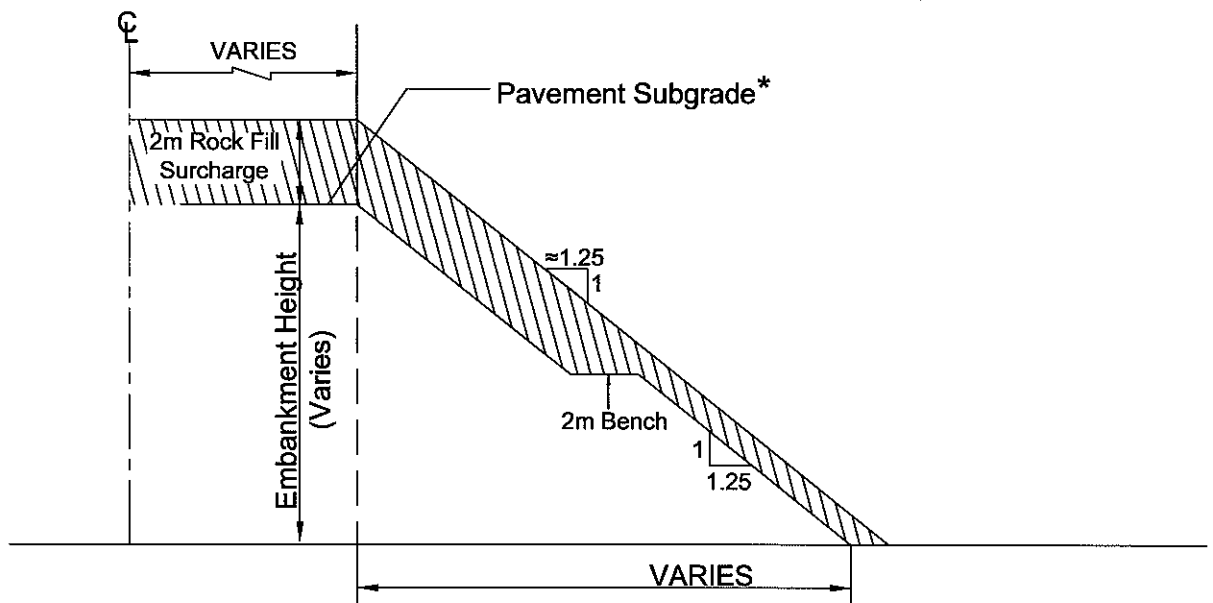
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File No. 1-09-4135

FIGURE H1



Rock Fill Embankment <10m



Rock Fill Embankment 10m \geq 12m

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

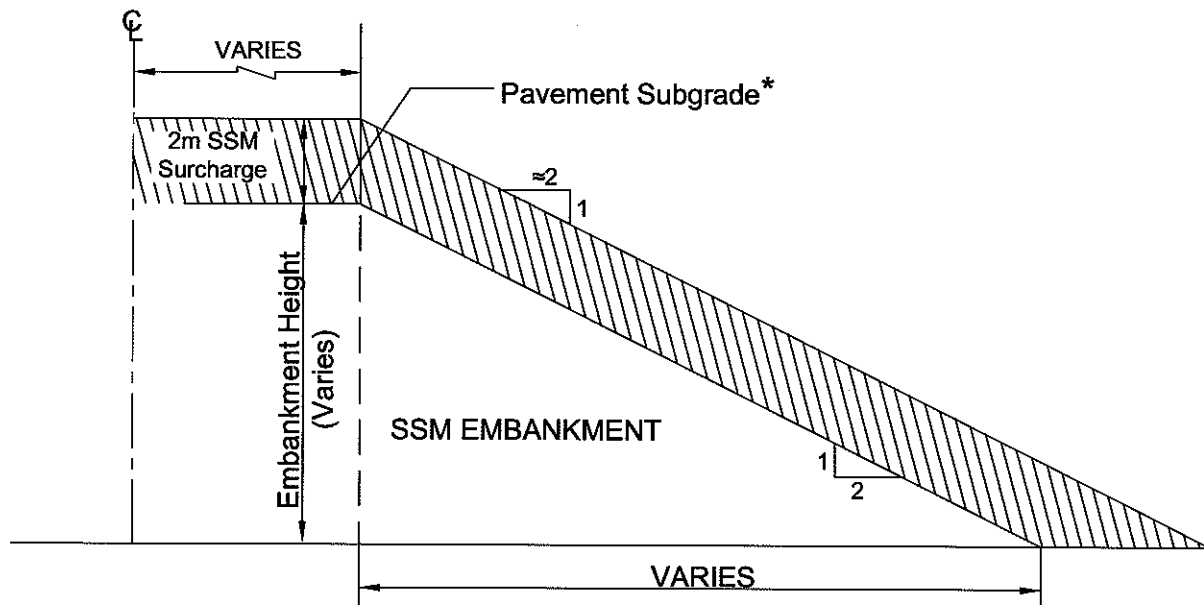
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SURCHARGE ARRANGEMENT

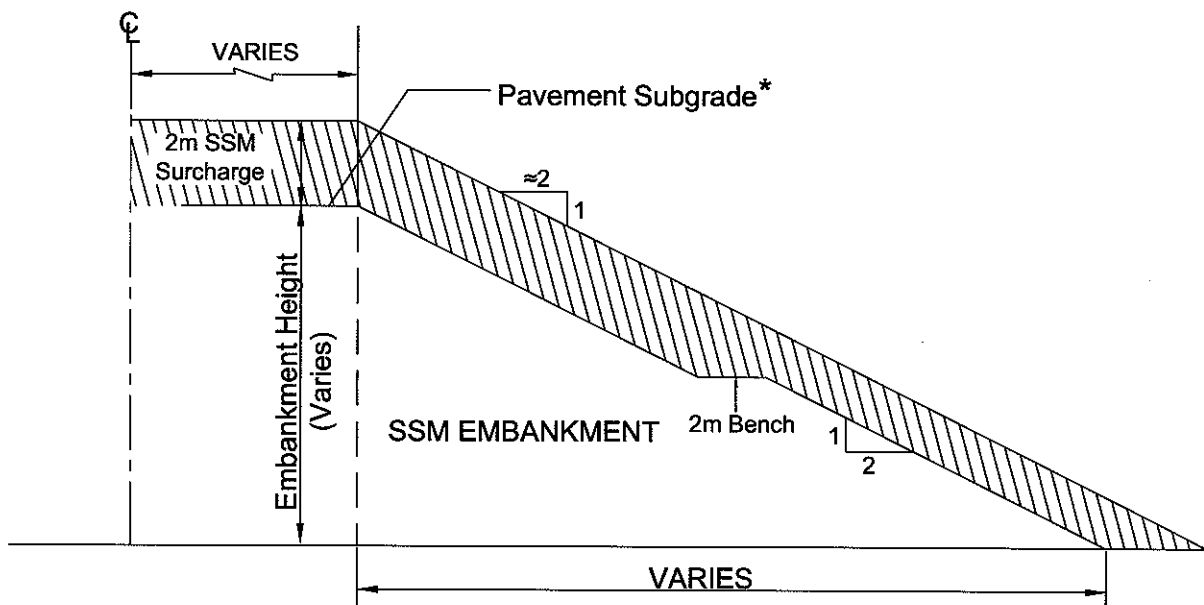
TERRAPROBE

File No. 1-09-4135

FIGURE H2



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.

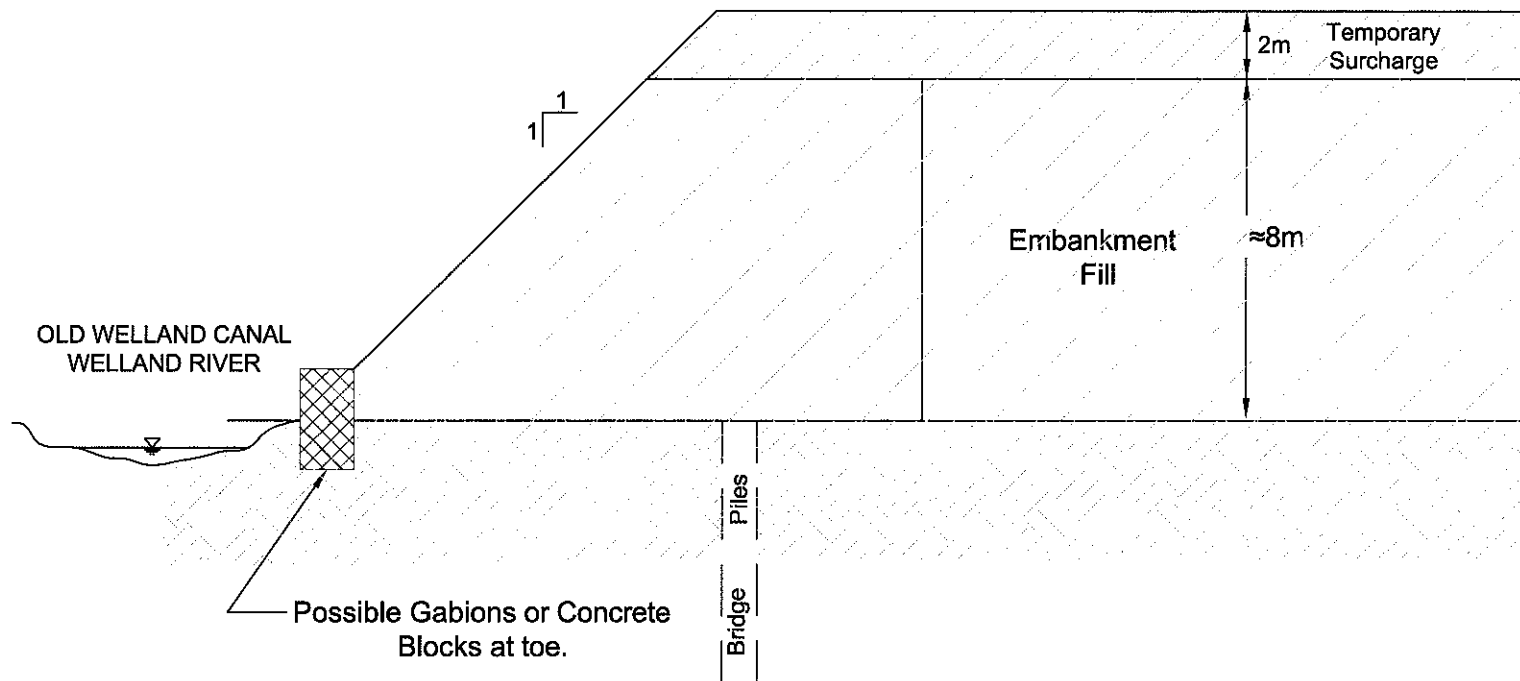
N.T.S

SURCHARGE ARRANGEMENT

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File No. 1-09-4135

FIGURE H3



N.T.S

TEMPORARY RETAINING WALL ARRANGEMENT

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FIGURE H4