FOUNDATION INVESTIGATION AND DESIGN REPORT
HIGHWAY 400 LINE 5 UNDERPASS
AND INTERCHANGE RECONSTRUCTION
BRADFORD WEST GWILLIMBURY, ONTARIO
TBWG WP P13-03
MTO GWP 2122-10-00

GEOCRES No. 31D-591

Report to

AECOM

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March 20, 2015
File: 19-4406-15
H:\19\4406\15 Foundation Engineering, Hwy.400 and 5th Line\Reports & Memos\19440615 Hwy 400 Line 5 Bridge and Interchange FIDR FINAL mar 15.doc

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted at the site of a proposed new underpass structure at Highway 400 and Line 5 in the Town of Bradford West Gwillimbury, Ontario. The proposed structure will replace the existing Line 5 bridge which crosses over Highway 400 along what appears to be a detoured alignment to the south of the original Line 5 alignment. The proposed underpass will be constructed as part of a new interchange proposed at the site.

The purpose of this investigation was to explore the subsurface conditions at the site and, based on the data obtained, to provide borehole location plans and soil strata drawings with stratigraphic profiles and cross-sections, records of boreholes, laboratory test results and written descriptions of the subsurface conditions. A model of the subsurface conditions was developed for the site based on the data obtained from the present investigation.

Thurber Engineering Ltd. (Thurber) carried out the investigation as a foundation sub-consultant to URS Canada Inc. (URS), ultimately for the Town of Bradford West Gwillimbury (TBWG).

During the preparation of this report and in addition to the boreholes drilled, general reference has been made to information on subsurface conditions contained in a previous foundation report for the area. The title of this report is listed as follows:

- Thurber Engineering Ltd. report titled "Preliminary Foundation Investigation and Design Report, 5th Line Underpass at Highway 400", Town of Bradford West Gwillimbury, Ontario", Geocres No. 31D-504, dated August 17, 2010 (Reference 1).

2 PROJECT AND SITE DESCRIPTION

The site of the proposed underpass structure lies approximately 40 m north of the existing bridge carrying Line 5 over Highway 400; and approximately 2.5 km south of Highway 400 and Simcoe Road 88 (former Highway 88) in the Town of Bradford West Gwillimbury, Ontario.

The entire project at this site involves reconstruction of the existing interchange. Other major components of the project requiring foundation engineering input include highway approach fills for the replacement underpass bridge, new interchange ramps, new bridge carrying the realigned Sideroad 5 over the North Schomberg River, two new culverts under Line 5 west of Highway 400, extension of two existing culverts under Highway 400 including the arch culvert (Sucker Creek Culvert) to the north of Line 5.

This report focuses on the foundation investigation for the new Highway 400 Line 5 underpass bridge and its immediate approaches. The new underpass bridge will replace the existing Line 5 bridge which crosses over Highway 400 along what appears to be a detoured alignment to the south of the original Line 5 alignment. The replacement bridge will assume the original Line 5 alignment.

The lands surrounding the interchange are relatively flat and primarily used for agricultural purposes. The existing 5th Sideroad and Coffey Road run alongside Highway 400 at the southwest and southeast quadrants, respectively. The North Schomberg River meanders on the west side of Highway 400 and flows under the highway through the Sucker Creek Culvert to the north of Line 5. Within the project area, vegetation cover largely consists of grass with some shrubs and small trees along the highway and Line 5, except for the west approach area of the new bridge where there are patches of medium to large trees.

It should be noted that current earthwork operations include fill placement for preloading and surcharging the immediate approaches of the proposed bridge. A monitoring and instrumentation program is also ongoing in conjunction with the earthworks as part of an advance contract.

Photographs in Appendix D show the general layout of the site and the existing structure prior to the commencement of the advance contract.

From published geological information, the site is located within the physiographic region known as the Schomberg Clay Plains which consists of deep deposits of stratified clay and silt overlying a drumlinized till plain. Depending on their sizes, the drumlins are completely or partially buried by the clay and silt deposits. The clay and silt deposits have average thicknesses of about 5 m although thicker deposits have also been identified.

3 SITE INVESTIGATION AND FIELD TESTING

A preliminary foundation investigation was carried out near the location of the proposed structure in March and April, 2010 (Reference 1). The preliminary site investigation consisted of two boreholes drilled (designated 10-01 and 10-02) and sampled to 32.4 m depth (Elevations 193.1 and 192.2 m). Borehole 10-01 was located near the north side of the proposed east abutment and drilled between March 24 and 26, 2010. Borehole 10-02 was located near the south side of the proposed west abutment and drilled between April 5 and 8, 2010.

The current site investigation and field testing for this underpass and its approaches were carried out on December 19, 2013 and from January 11 to 31, 2014 and consisted of drilling and sampling a total of eight boreholes (identified as 13-19 to 13-22 and 13-27 to 13-30). Four boreholes (Boreholes 13-19, 13-20, 13-29 and 13-30) were drilled at the proposed foundation elements (abutments and pier) to 35.7 m depth (Elevations 188.0 to 189.2). Four boreholes (Boreholes 13-21, 13-22, 13-27 and 13-28), were drilled at and near the immediate approaches. Termination depths for the approach boreholes ranged from 8.2 m to 11.3 m (Elevations 212.8 to 219.5 m), respectively.

The approximate locations of the boreholes drilled during the previous and current investigations are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix C. The coordinates and elevations of the boreholes are given on the drawing and on the individual Record of Borehole Sheets in Appendix A.

The borehole locations were marked in the field and utility clearances were obtained prior to drilling.

During the current investigation, a track mounted D52 drill rig was used in conjunction with hollow-stem augers to advance the boreholes. Soil samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT). In addition to the SPT samples, six thin wall Shelby tube samples were collected at selected depths from Boreholes 13-19 and 13-30 drilled at the proposed abutment locations. Four thin wall Shelby tube samples were collected during the previous investigation in 2010. The in situ shear strength of the cohesive soils was also assessed using an MTO ‘N’ size shear vane. A ’B’ size vane was also used at some locations to obtain values within the very stiff zones.

The drilling and sampling operations were supervised on a full time basis by a member of Thurber’s technical staff. The supervisor logged the boreholes and processed the recovered soil samples for transport to Thurber’s laboratory for further examination and testing.

Groundwater conditions were observed in the open boreholes during and upon completion of the drilling operations. Standpipe piezometers consisting of a 19 mm diameter Schedule 40 PVC pipe with a 3.0 m long slotted screen were installed within a column of filter sand in eight boreholes to permit longer term groundwater level monitoring. Two shallow piezometers, extending into the silty clay layer, were installed in separate holes adjacent to Boreholes 13-19 and 13-30. The completion details of the piezometers and boreholes are summarized in Table 3.1.

Table 3.1 – Piezometer and Borehole Completion Details

Foundation Unit	Borehole Number	Piezometer Tip Depth / Elevation (m)	Completion Details
West approach	13-27	9.1/212.9	Backfilled with filter sand from 9.8 m to 5.8 m, bentonite holeplug and auger cuttings from 5.8 m to ground surface.
	13-28	9.1/214.2	Backfilled with filter sand from 10.5 m to 5.8 m, bentonite holeplug and auger cuttings from 5.8 m to ground surface.
West abutment	13-19	33.5/190.2	*<u>Deep Piezometer</u>* Backfilled with filter sand from 35.7 m to 29.9 m, bentonite holeplug and auger cuttings from 29.9 m to ground surface.
		9.1/214.6	*<u>Shallow piezometer</u>* Backfilled with filter sand from 9.1 m to 5.8 m, bentonite holeplug and auger cuttings from 5.8 m to ground surface.
	10-02	9.1/215.5	Backfilled with filter sand to 6.2 m, bentonite from 6.2 m to 1.7 m, auger cuttings from 1.7m to ground surface.
Pier	13-20	None installed	Backfilled with bentonite and auger cuttings to 0.4 m, concrete from 0.4 m to 0.2 m, then asphalt patch to ground surface.
	13-29	None installed	Backfilled with bentonite and auger cuttings to 0.4 m, concrete from 0.4 m to 0.2 m, then asphalt patch to ground surface.
East Abutment	10-01	31.1/194.4	Backfilled with filter sand to 27.3 m, bentonite from 27.3 m to 2.6 m, auger cuttings from 2.6 m to ground surface.
	13-30	32.0/192.4	*<u>Deep Piezometer</u>* Backfilled with filter sand from 35.1 m to 28.3 m, bentonite holeplug and auger cuttings from 28.3 m to ground surface.
		9.1/215.3	*<u>Shallow piezometer</u>* Backfilled with filter sand from 9.1 m to 5.8 m, bentonite holeplug and auger cuttings from 5.8 m to ground surface.
East Approach	13-21	10.7/215.9	Backfilled with filter sand from 11.3 m to 7.3 m, bentonite holeplug and auger cuttings from 7.3 m to ground surface.
	13-22	7.6/220.1	Backfilled with filter sand from 8.2 m to 4.3 m, bentonite holeplug and auger cuttings from 4.3 m to ground surface.

4 LABORATORY TESTING

All recovered soil samples were subjected to visual identification and to natural moisture content determination. At least 25% of the recovered soil samples were subjected to grain size distribution analysis. Atterberg Limits tests were carried out on selected samples of native silty clay and silty clay till to determine the plasticity characteristics. The results of the laboratory testing are summarized on the Record of Borehole sheets included in Appendix A and are presented on the figures included in Appendix B.

One specimen was selected from each of four thin wall Shelby tube samples from Boreholes 13-19, 13-30, 10-01 and 10-02 for one-dimensional oedometer (consolidation) tests. The detailed results are shown in Appendix B.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

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

In general, the subsurface stratigraphy at the site consists of surficial fill or topsoil on firm to very stiff silty clay overlying very stiff to hard silty clay till. The till is underlain by dense to very dense layers of sands and silts. The groundwater level within the cohesive deposits is at, or within 0.5 m depth of, the existing ground surface.

In Boreholes 10-01 and 10-02 drilled during the previous investigation, it is noted that the cohesive soils are described as clayey silt and clayey silt till. However, based on geological records, geotechnical laboratory test results and soil descriptions used for similar deposits in recent MTO projects in the area, it is considered appropriate to describe the cohesive soils at this site as silty clay and silty clay till, respectively.

5.1 Topsoil

Topsoil was encountered surficially in all the boreholes drilled for the current investigation, except in Boreholes 13-20 and 13-29 which were located on the highway. The thickness of the topsoil ranged from 100 mm to 175 mm.

A 200 mm thick layer of topsoil was encountered surficially in Boreholes 10-01 and 10-02 drilled during the previous investigation.

The topsoil thickness may vary between and beyond the borehole locations, and the limited data presented in this report should not be used for quantity estimation purposes.

5.2 Pavement Structure

Pavement structure consisting of asphalt overlying granular fill materials (road base) was encountered in Boreholes 13-20 and 13-29, drilled on the southbound and northbound lanes of Highway 400.

The thickness of the asphalt ranged from 100 to 125 mm.

5.3 Fill

Two distinctive layers of fill were encountered at the site, granular fill (road base) below the asphalt on the highway and silty sand/silty clay below the topsoil in the open field west of the highway.

The granular fill encountered in Boreholes 13-20 and 13-29, drilled on Highway 400 lanes, consisted of brown gravelly sand and sand and gravel containing some silt. The thickness of the granular fill was 700 mm and 800 mm in Boreholes 13-20 and 13-29, respectively. The depths to the base of the granular fill varied from 0.8 m to 0.9 m (Elevations 224.1 to 224.0 m).

SPT ‘N’ values recorded in the granular fill were 27 and 34 blows for 0.3 m penetration, indicating a compact to dense state.

In Borehole 13-28, a 600-mm thick layer of silty clay fill with trace sand was contacted below the topsoil. Brown silty sand fill containing some clay and trace to some gravel was contacted below the topsoil in Boreholes 13-21 and 13-22 and below the silty clay fill in Borehole 13-28. These three boreholes were drilled at the location of the proposed approaches. The thickness of the silty sand fill layer ranged from 500 to 800 mm. The depth to the base of the silty sand/silty clay fill ranged from 0.6 to 1.5 m (Elevations 221.8 to 227.1 m).

SPT ‘N’ values recorded in the silty sand fill were 8 blows per 0.3 m of penetration, indicating a loose state in Boreholes 13-22 and 13-28. In Borehole 13-21, an ‘N’ value of 74 blows per 0.3 m of penetration indicated a very dense state. The ‘N’ value recorded in the silty clay fill in Borehole 13-28 was 8 blows per 0.3 m of penetration, indicating a stiff consistency.

The measured moisture contents of fill samples were 1% and 6% in the granular fill, 7% to 29% in the silty sand fill and 27% in the silty clay fill.

Two samples of the gravelly sand/sand and gravel fill and two samples of the silty sand fill were subjected to laboratory gradation analysis.

Grain size distribution curves for samples of the fill tested are presented on the Record of Borehole sheets included in Appendix A and on Figures B1 and B2 of Appendix B. The results of the laboratory test are summarized as follows:

Soil Particles	Percentage (%) Granular Fill	Percentage (%) Silty Sand Fill
Gravel	20 to 41	8 to 16
Sand	55 to 61	45 to 68
Silt	-	24
Clay	-	15
Silt and Clay	4 to 19	24

5.4 Silty Clay

An extensive deposit of native silty clay was encountered below the fill or topsoil in all boreholes. The silty clay typically contained silt and clay fractions with trace to some sand and trace gravel at some locations. The silty clay was brown within the upper 3 to 4 m becoming grey with depth. The thickness of this deposit varied from 21.2 to 28.2 m.

Boreholes 13-21, 13-22, 13-27 and 13-28 were terminated within the silty clay at depths ranging from 8.2 to 11.3 m (Elevations 212.2 to 219.5 m). Where the silty clay was fully penetrated in the remaining boreholes, the base of this soil was encountered at depths of 21.3 to 28.7 m (Elevations 196.2 to 202.3 m).

An over-consolidated surficial weathered crust, extending to approximately 2 to 10 m depths (base at approximate Elevations 213 to 223 m), was encountered in the boreholes. Within this crust, SPT ‘N’ values typically ranged from 10 to 19 blows for 0.3 m of penetration indicating a stiff to very stiff consistency, except for the surficial 1 to 2 m where the ground had a firm consistency as indicated by ‘N’ values of 4 to 8 blows per 0.3m penetration. An SPT ‘N’ value of 3 blows for 0.3 m of penetration, indicating a soft consistency, was encountered at about 7 m depth in Borehole 10-02.

Below the crust, a lightly over-consolidated, firm to stiff silty clay zone was encountered within approximate Elevations 223 and 210 m, with depths ranging from about 5 to 15 m. In situ vane testing indicated that the undrained shear strength ranges from 55 to 105 kPa which correspond to a typically firm to stiff and occasionally very stiff consistency. Using a ‘B’ size vane, a value of 140 kPa was measured in Borehole 13-19 near Elevation 216.5 m. SPT ‘N’ values typically ranged between 4 and 9 blows per 0.3 m penetration within this zone.

Below the firm to stiff zone, the silty clay generally becomes stiff to very stiff as indicated by ‘N’ values of 10 to 28 blows per 0.3 m penetration.

The measured moisture content of samples of the silty clay ranged from 12% to 32%. A moisture content value of 35% was measured for a sample from Borehole 13-27 near Elevation 220.2 m.

Forty one samples of silty clay were subjected to gradation analysis and thirty-seven samples underwent Atterberg Limits testing. Grain size distribution results are presented on the Record of Borehole sheets in Appendix A and on Figures B3 to B9 in Appendix B. Atterberg Limits test results are shown on the Records of Boreholes and also presented on Figures B12 to B18 of Appendix B. The results of the laboratory test are summarized as follows:

Soil Particles	Percentage (%)
Gravel	0 to 11
Sand	0 to 23
Silt	39 to 79
Clay	21 to 61

Soil Particles	Percentage (%)
Liquid Limit	18 to 49
Plasticity Index	11 to 29

The results indicate that the silty clay typically has low plasticity (CL), with zones of slight and medium plasticity (CL-ML and CI) respectively. One sample from 12.5 m depth in Borehole 10-01 indicated a borderline CI to MI classification.

The results of oedometer (consolidation) testing conducted on four samples of the silty clay obtained below the crust are included in Appendix B and are summarized in Table 5.1.

Table 5.1 – Consolidation Test Parameters

Borehole	Sample Depth (m)	Soil Type	w_o (%)	γ (kN/m^3)	e_o	p_o’ (kPa)	p_c’ (kPa)	OCR	C_c	C_r
13-19	7.6 – 8.2	CL	15	22.0	0.41	120	180	1.5	0.095	0.015
13-30	10.6-11.3	CL	16	21.4	0.44	150	180	1.2	0.091	0.017
10-01	12.2-12.8	CI-MI	24	20.0	0.65	170	170	1.0	0.18	0.021
10-02	6.1-6.7	CL	20	20.7	0.55	80	145	1.8	0.15	0.017

Comparison of the existing and preconsolidation pressures (p_o’ and p_c’) derived from the test results indicate that the silty clay below the crust is typically lightly over-consolidated to occasionally normally consolidated. The coefficient of consolidation, c_v, recorded during the test was generally in the order of 2 x 10^{-3} cm^2/s to 4 x 10^{-2} cm^2/s for the typical pressure range anticipated in the field. The compressibility characteristics of the silty clay vary with depth and are dependent on the moisture content and shear strength profiles.

5.5 Silty Clay Till

Grey silty clay till was encountered below the silty clay in Boreholes 13-19, 13-20, 13-30, 10-01 and 10-02 drilled near the proposed foundation units. The silty clay till generally contains trace to some sand and trace gravel. Occasional cobbles were encountered within the silty clay till in Borehole 10-01. The thickness of the silty clay till varied from 1.5 m to 6.7 m where fully penetrated. The base of the silty clay till was encountered at depths of 27.6 m to 34.4 m (Elevations 190.0 to 197.0). The silty clay till layer was not fully penetrated in Borehole 10-01 which was terminated at 32.4 m depth (Elevation 193.1 m).

SPT ‘N’ values recorded in the silty clay till ranged from 23 to 103 blows for 0.3 m penetration (typically between 23 and 44 blows), indicating a very stiff to hard consistency. A SPT ‘N’ value of 100 blows for less than 0.3 m of penetration was encountered at the base of Borehole 10-01. The measured moisture content of samples of the silty clay till ranged from 12% to 23%.

Five samples of silty clay till were subjected to gradation analysis and two samples also underwent Atterberg Limits testing. Grain size distribution curves are presented on the Record of Borehole sheets in Appendix A and on Figure B10 of Appendix B. Atterberg Limits test results are presented on the Records of Borehole sheets in Appendix A and on Figure B19 of Appendix B. The results of the laboratory test are summarized as follows:

Soil Particles	**Percentage (%)**
Gravel	0 to 3
Sand	0 to 18
Silt	49 to 62
Clay	30 to 50

Soil Particles	**Percentage (%)**
Liquid Limit	21 to 29
Plasticity Index	9 to 10

The results of the Atterberg Limits tests indicate that the silty clay till has a low plasticity (CL).

It should be noted that glacial tills inherently contain cobbles and boulders.

5.6 Sand, Silty Sand to Sand and Silt

Native cohesionless soils were contacted below the silty clay till. In Boreholes 13-19, 13-20, 13-30 and 10-02 layers of grey sand and silty sand containing trace clay were contacted at depths ranging from 27.6 to 34.4 m (elevations 190.0 to 197.0). A layer of grey sand and silt containing trace of clay was encountered at 28.7 m depth (elevation 196.2) in Borehole 13-29.

Boreholes 13-19, 13-20, 13-29, 13-30 and 10-02 were terminated within these cohesionless soils at depths ranging from 32.4 to 35.7 m (Elevations 188.0 to 192.2 m).

SPT 'N' values recorded in the sands and silts ranged from 40 to 115 blows for 0.3 m penetration, indicating a dense to very dense state. In Borehole 10-02, the SPT 'N' values were greater than 100 blows for less than 0.3 m of penetration, indicating a very dense state. These high values may be attributed to the presence of cobbles and/or boulders.

The measured moisture content of samples of the sand and silt layers ranged from 13 to 22%.

Five samples of sand, silty sand to sand and silt were subjected to gradation analysis testing. The results of these tests are summarized in the table below as well as on the Record of Borehole sheets included in Appendix A. Figure B11 in Appendix B presents the grain size distribution curves for these samples. A summary of the test results is as follows:

Soil Particles	Sand/Silt
Gravel	0 to 9
Sand	45 to 89
Silt	26 to 53
Clay	2 to 10
Silt & Clay	11 to 16

5.7 Groundwater Levels

Water levels were observed in the open boreholes upon completion of drilling operations. A total of twelve standpipe piezometers were installed in ten boreholes to monitor water levels after completion of drilling. The water levels measured in the piezometers are summarized in Table 5.2. The measurements recorded in the open boreholes upon completion of drilling are also included.

Table 5.2 – Water Level Measurements

Foundation Unit	Borehole Number	Date	Water Levels Depth (m)	Water Levels Elevation (m)	Comment
West approach	13-27	January 29, 2014 February 26, 2014 March 13, 2014	6.4 0.5 0.5	215.6 221.5 221.5	Open Borehole Piezometer Piezometer
	13-28	January 28, 2014 February 26, 2014 March 13, 2014	5.1 1.8 1.8	218.2 221.5 221.5	Open Borehole Piezometer Piezometer
West abutment	13-19*	January 28, 2014 February 26, 2014 March 13, 2014	19.4 7.7 7.3	204.3 216.0 216.4	Open Borehole Piezometer Piezometer
	13-19	February 26, 2014 March 13, 2014	2.5 2.6	221.2 221.1	Piezometer Piezometer
	10-02	April 9, 2010 April 20, 2010 May 3, 2010	9.1 1.5 1.0	215.5 223.1 223.5	Piezometer Piezometer Piezometer
Pier	13-20	January 11, 2014	1.8	223.1	Open Borehole
	13-29	January 15, 2014	26.5	198.4	Open Borehole
East Abutment	10-01	March 31, 2010 April 9, 2010 April 20, 2010 May 3, 2010	2.6 2.8 2.7 2.4	222.9 222.7 222.8 223.1	Piezometer Piezometer Piezometer Piezometer
	13-30*	January 23, 2014 February 26, 2014 March 13, 2014	3.4 1.0 0.8	221.0 223.4 223.6	Open Borehole Piezometer Piezometer
	13-30	February 26, 2014 March 13, 2014	1.2 0.7	223.2 223.7	Piezometer Piezometer
East Approach	13-21	February 26, 2014 March 13, 2014	1.6 1.7	225.0 224.9	Piezometer Piezometer
	13-22	February 26, 2014 March 13, 2014	2.1 2.2	225.6 225.5	Piezometer Piezometer

*Deep piezometer

The piezometric readings indicate that stabilized groundwater level within the silty clay and silty clay till generally ranges from 0.5 m to 2.8 m depths below ground surface, or between Elevations 221.1 and 223.7 m. This shows that the groundwater level rises gently from west to east across the site. A deep piezometer installed within the underlying native sand and silt layer in Borehole 13-19 indicated water levels at 7.3 to 7.7 m depths, or Elevations 216.0 and 216.4 m.

All groundwater observations at this site are short term and the levels are expected to fluctuate seasonally and after severe climatic events.

6 MISCELLANEOUS

Borehole locations were established in the field based on information provided by URS. The ground surface elevation and coordinates at all as-drilled borehole locations were established by Thurber upon completion of drilling. Underground utility clearances were obtained for the borehole locations prior to drilling.

Walker Drilling Inc. of Utopia, Ontario supplied a truck-mounted drill rig and conducted the drilling, sampling and in-situ testing operations.

The field investigation was supervised by Mr. George Azzopardi, C.E.T. of Thurber. Geotechnical laboratory testing was carried out in Thurber's Toronto Area laboratory.

Planning and co-ordination of the field program was conducted by Ms. Katrina Young, E.I.T. Overall direction of the program was provided by Mr. Sydney Pang, P.Eng. Interpretation of the data and preparation of this report was carried out by Mr. Sydney Pang, P.Eng. and Ms. R. Palomeque Reyna, P.Eng.

The report was reviewed by Mr. P.K. Chatterji, P.Eng., who is a Designated Principal Contact for MTO Foundations Projects.

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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

7 GENERAL

This report provides an interpretation of the geotechnical data in the factual report, and presents foundation design recommendations to assist the design team to select and design a suitable foundation system and approach embankments for the proposed underpass at Highway 400 and Line 5, located in the Town of Bradford West Gwillimbury, Ontario.

The proposed bridge location is approximately 40 m north of the existing bridge at Highway 400 and Line 5. The new bridge will be constructed along the original Line 5 alignment.

A preliminary General Arrangement (GA) drawing provided by URS, dated September 2014, indicates that the replacement underpass bridge has two spans each of 39.2 m in length and approximately 35 m in width, supported by two abutments and one pier. A 6 m long approach slab with sleeper slab is to be located behind each abutment. Each of the two integral abutments are designed to be supported by a single row of driven steel H-piles, and the centre pier is to be supported on driven steel H-piles arranged in groups. The west and east approach fills are up to 9 and 8 m high, respectively. The existing bridge will be removed as part of this project.

Retaining walls parallel to the Line 5 alignment will be constructed immediately beyond the east and west wingwalls. The length of the retaining walls will be 6.0 m and 7.0 m on the south and north sides, respectively.

Three technical memoranda have been issued by Thurber during the period of March to October 2014 providing subsurface conditions and preliminary geotechnical recommendations for foundation design and approach fills. These memoranda were issued upon the request of URS in order to facilitate preparation of the advance contract documents. The advance contract, which is currently underway, includes preloading and surcharging at the abutments and immediate approaches in conjunction with a geotechnical instrumentation and monitoring program developed by Thurber. The purpose of preloading and surcharging is to induce substantial foundation settlements within the embankment footprints during a waiting period such that post construction settlement would be limited to within tolerable limits and that potential downdrag forces on piles would not need to be considered in design. Foundation comments, engineering analysis results and recommendations contained in these memoranda are incorporated in this report.

The discussions and recommendations presented in this report are based on the factual data obtained during the course of the current investigation and selected data obtained from a previous investigation (Reference 1). The plans and profiles used for preparation of this report were provided by URS.

8 FOUNDATION CONDITIONS

In general, the subsurface conditions at this site consist of surficial asphalt and granular fill (road base) or topsoil overlying silty sand fill and silty clay fill. Below the fill, an extensive deposit of firm to very stiff silty clay was contacted. The thickness of the silty clay ranged from approximately 21 to 28 m. The silty clay grades into very stiff to hard silty clay till which is underlain by cohesionless layers of dense to very dense sands and silts. The groundwater level within the cohesive deposits is at, or within 2 m depth below, existing ground surface.

The presence of extensive compressible silty clay below the stiff crust presents challenges to the design and performance of shallow foundations. Accordingly, it is considered that deep foundations would be required to support the bridge on the underlying silty clay till and/or sands and silts. Further details on evaluation of foundation alternatives are as follows.

9 FOUNDATION DESIGN FOR REPLACEMENT UNDERPASS BRIDGE

9.1 Foundation Alternatives

Consideration was given to alternate foundation options taking into account the general layout of the site, subsurface stratigraphy and the proposed works. These options are listed below:

- Spread footings on native silty clay or engineered fill pad
- Driven steel H-piles
- Augered caissons (drilled shafts)

Spread footings, which will have limited capacity and subject to post construction settlement due to the compressible silty clay foundation, are not recommended at this site. Moreover, space constraints at the piers due to the narrow median and close proximity of the Highway 400 travelled lanes are expected to pose difficulties during spread footing construction.

Abutments

It is understood that integral abutments are proposed to be used for this bridge. Approach fills up to 9 m and 8 m high are to be placed behind the west and east abutments, respectively. From a foundation engineering perspective, it is considered feasible to use integral abutments founded on a single row of steel H-piles driven to practical refusal. Alternatively, augered caissons (drilled shafts) may also be considered if integral abutments are not to be used.

Pier

Both augered caissons (drilled shafts) and driven piles are technically feasible for providing foundation support to the pier. Driven steel H-piles require excavation for pile cap construction within roadway protection systems. Augered caissons can, however, be designed to be structurally connected to the superstructure without a pile cap. From a foundation engineering perspective and given that an end bearing stratum is not well defined at the pier location, augered caissons will provide higher shaft capacity due to their larger surface area in contact with the surrounding soils. A larger number of driven steel H-piles may be used to compensate for the smaller shaft capacity.

Augered H-piles, which have lower capacities than drilled shafts due to the smaller surface area, will have similar installation requirements and difficulties. This option is not further developed at this time.

More detailed comparison of the technical advantages and disadvantages of the alternative foundation schemes is presented in Appendix E.

9.2 Driven Steel H-Piles

Integral abutments are required to be supported on steel H-piles driven to practical refusal within the underlying very stiff to hard silty clay till or the dense to very dense sands and silts. A standard HP 310 x 110 section or a heavier HP 360 x 132 section may be used. Tills and other glacially derived soils inherently contain cobbles and/or boulders. The pile tips should, therefore, be reinforced to enhance driving (see Section 9.2.5).

For planning and design purposes, the recommended design founding elevations are as follows:

Table 9.1 – Design Pile Tip Elevations

Foundation Unit	Reference Borehole	Pile Tip Elevation (m)
West Abutment		
North Side	13-19	187 or lower
South Side	10-2	194 or lower
Pier		
North Side	13-20	191 or lower
South Side	13-29	191 or lower
East Abutment		
North Side	10-1	194 or lower
South Side	13-30	191 or lower

The pile tip elevations shown in Table 9.1 should be used for estimating purposes only. The actual pile tip elevations will be controlled as described in Section 9.2.5 Pile Installation.

9.2.1 Axial Resistance

For steel H-piles driven to practical refusal at the estimated elevations given above, the following axial design geotechnical resistances per pile may be used.

Table 9.2 – Pile Axial Resistances

Foundation Unit	Pile Section			
	HP 310 x 110		HP 360 x 132	
	Factored Geotechnical Resistance at ULS (kN)	Geotechnical Resistance at SLS (kN)	Factored Geotechnical Resistance at ULS (kN)	Geotechnical Resistance at SLS (kN)
Abutments	1,400	1,200	1,600	1,400
Pier	1,000	800	1,200	1,000

The SLS values correspond to a maximum pile settlement of 25 mm.

It is noted that the recommended resistances for the pier piles are lower than those for the abutment piles. This is because an end-bearing stratum was not identified in the boreholes advanced at the pier location.

The structural capacity of a pile must not be exceeded and should be confirmed by the structural designer.

9.2.2 Downdrag on Abutment Piles

Downdrag forces could be induced on piles embedded within the silty clay deposit due to consolidation of the silty clay under the weight of the new fill. These forces can be minimized by preloading and surcharging at the approach fills prior to pile installation. Reference should be made to the CHBDC (2010) Clauses 6.8.4 and C6.8.4 (commentary) for downdrag calculations.

The location of the neutral plane for a pile or pile group should be determined by using unfactored loads and unfactored geotechnical parameters. As a design check, based on the SLS (unfactored) loads quoted above and provided that preloading and surcharging is carried out as recommended, and using a load factor of 1.25 as per the CHBDC, it is estimated that factored downdrag loads in the order of 550 kN and 640 kN may act on each HP 310 x 110 and HP 360 x 132 pile, respectively. In accordance with the CHBDC, the sum of the factored downdrag load and the factored permanent loads acting on the pile should not exceed the structural resistance of the pile. In geotechnical analysis of downdrag, live load effects should not be considered.

Downdrag forces need not be considered in pile design at this site provided that pre-loading and surcharging is carried out, as discussed elsewhere in this report, prior to installing the new piles.

9.2.3 Lateral Resistance

For integral abutments, the flexibility of the upper portion of the pile may be provided by a single corrugated steel pipe (CSP) system. Reference should be made to the integral abutment manual for details of this system.

For pile lateral resistance design below the flexible zone, soil-pile interaction analyses may be carried out using the coefficient of horizontal subgrade reaction values provided in Table 9.3 below.

The lateral resistance of a pile may be calculated using values for the coefficient of horizontal subgrade reaction (k_s) and the pressures obtained from the analysis should not exceed the ultimate values given in the following relationships.

<u>Silty Clay / Silty Clay Till</u>

$$k_s = 67\ C_u / B \quad (\text{kN/m}^3)$$

$$p_{ult} = 9\ C_u \quad (\text{kPa})$$

where p_{ult} = ultimate lateral resistance mobilized by a pile, kPa
C_u = undrained shear strength of cohesive soils, kPa

γ	=	total unit weight of soil, kN/m^3
B	=	width of pile, m

For cohesive soils, the lateral resistance provided by the ground located between the final grade and a depth of 1.5B below that level should be neglected.

Table 9.3 – Recommended Geotechnical Parameters for Lateral Resistance Design

Location	Reference Boreholes	Approximate Elevation (m)	Undrained Shear Strength C_u (kPa)	Unit Weight γ (kN/m^3)	Soil Conditions
West Abutment	13-19 and 10-02	224 to 219	75	20	Silty Clay
		219 to 208	50	20	Silty Clay
		208 to 197	125	21	Silty Clay / Silty Clay Till
Pier	13-20 and 13-29	224 to 215	75	20	Silty Clay
		215 to 212	65	20	Silty Clay
		212 to 196	125	21	Silty Clay / Silty Clay Till
East Abutment	13-30 and 10-01	224 to 220	75	20	Silty Clay
		220 to 213	50	20	Silty Clay
		213 to 197	100	21	Silty Clay / Silty Clay Till

The spring constant, K, for analysis may be obtained by the expression, $K = k_s \times d_z \times B$ (kN/m), where k_s is the coefficient of horizontal subgrade reaction (kN/m^3), B is the pile width (m), d_z is the length (m) of the pile segment or element used in the analysis. The ultimate lateral resistance on any one segment of pile, P_{ult}, may be obtained from the expression, $P_{ult} = p_{ult} \times d_z \times B$. This represents the ultimate load at the contact between the soil and the pile above which additional load cannot be supported at greater displacements.

For lateral soil-pile group interaction analysis, the values for k_s should be reduced based on pile spacing.

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

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

where B is the diameter 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 of k_S using a reduction factor R as follows:

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

Intermediate values may be obtained by interpolation.

9.2.4 Frost Protection

Frost protection should be provided to all the pile caps and may take the form of 1.4 m of earth cover in any direction, or equivalent thermal insulation, over the underside of the footing.

9.2.5 Pile Installation

All piles shall be installed in accordance with OPSS 903.

The appropriate pile driving note to be shown on the contract drawing is "Piles to be driven in accordance with Standard Provision SS103-11 using an ultimate geotechnical resistance equal to two times the maximum factored design load at ULS, but must be driven below the elevations shown on the subsequent table.

Foundation Unit	Reference Borehole	Pile Tip Elevation (m)
West Abutment		
North Side	13-19	187
South Side	10-2	194
Pier		
North Side	13-20	191
South Side	13-29	191
East Abutment		
North Side	10-1	194
South Side	13-30	191

To facilitate pile installation, embankment fill through which piles will be driven must not contain any material with particle sizes greater than 75 mm.

Glacially derived soils inherently contain cobbles and boulders. In order to be able to penetrate boulders, cobbles and harder/dense zones to achieve the required tip elevations and soil resistance, it is recommended that the pile tips be reinforced with driving shoes such as the Titus Standard Points for H Piles or the conventional driving shoes as per OPSD 3000.100.

9.3 Augered Caissons (Drilled Shafts) for Pier

Augered caissons embedded within the stiff to very stiff silty clay are feasible to support the pier foundations. Augered caissons are unsuitable for supporting integral abutments. Table 9.4 presents the recommended founding depths and elevations for the caissons.

Table 9.4 – Founding Depths and Elevations for Augered Caissons

Foundation Element	Borehole	Founding Depth (m)	Founding Elevation (m)
Pier	13-20 and 13-29	25	200.0

9.3.1 Axial Resistance

The following Table 9.5 presents geotechnical resistances recommended for typical 1.2, 1.5 and 1.8 m diameter caissons associated with the founding depth given in Table 9.4.

Table 9.5 – Vertical Geotechnical Resistance for Augered Caissons

Caisson Diameter (m)	Axial Geotechnical Resistance	
	Factored ULS_f (kN)	SLS (up to 25 mm settlement) (kN)
1.2	2,500	2,000
1.5	3,500	2,800
1.8	4,500	3,600

If higher capacities are required, it is technically possible to extend the caissons to lower depths. However, there will be risks associated with basal stability as the augering progresses through the sands and silts under hydrostatic pressures.

9.3.2 Downdrag on Caissons

Downdrag on caissons is not a design issue at the pier since there is no fill placement in excess of the existing highway grade.

9.3.3 Lateral Resistance

Lateral bridge loadings can be geotechnically resisted by the caissons through passive pressure developed along the embedded portion of the shaft. The methodology outlined in Section 9.3 above may be used to estimate the lateral geotechnical resistance of the caisson by substituting the pile width, B, with the caisson diameter, D.

9.3.4 Caisson Installation

Caisson installation must be carried out in accordance with OPSS 903 where applicable.

The caisson installation equipment should be able to dislodge and remove any obstructions such as cobbles and boulders and to penetrate harder and denser layers within the silty clay till. An NSSP addressing this issue must be included in the contract documents to alert the bidders (see Appendix F).

The resistance values provided in Table 9.5 above are primarily based on shaft friction and assuming that the walls of each caisson are cleaned of loose material prior to placement of concrete. The groundwater conditions observed in the boreholes are high (relative close to ground surface). Soil sloughing and water seepage will occur in unsupported holes primarily from embedded sand and silt lenses and interlayers. Temporary liners must, therefore, be available on site to support the caisson sidewalls and to provide seepage cut-off where required. Any accumulated water may have to be pumped out from the hole prior to placing concrete. Concrete should be placed with a minimum delay after each caisson is drilled and cleaned. If accumulated water in the caisson hole cannot be removed, consideration should then be given to using the tremie technique to place concrete inside the caisson hole.

9.4 Recommended Foundation

From a foundations technical, constructability and cost-effectiveness perspective, the recommended foundations at the abutments and pier at this site are steel H-piles driven to achieve resistance in the lower, very stiff to hard silty clay till and/or dense to very dense sands and silts. Augered caissons embedded within the stiff to very stiff silty clay are feasible to support the pier foundations.

10 RETAINING WALLS

The current GA drawing dated September 2014 includes construction of four retaining walls on each corner/quadrant of the proposed underpass bridge behind the wingwalls. The retaining walls will extend parallel to the realigned Line 5 and will be 6.0 m long on the south side and 7.0 m long on the north side. It is understood that the current design calls for cantilever type, cast-in-place, concrete walls.

From a geotechnical perspective and based on the subsurface conditions, it is recommended that retaining wall footings be founded on a compacted Granular A pad resting on the underlying, undisturbed, native stiff weathered crust of the silty clay deposit.

Based on the GA drawing, the approximate design founding elevations are as presented in Table 10.1.

Table 10.1 – Founding Strata and Elevations

Foundation Unit	Boreholes	Founding Stratum and Elevation (m)
West Abutment		
Granular A pad base	13-19, 10-02	Top of native silty clay (≈ 223.5 to 224.5)*
Footing base (granular pad thickness ≥ one footing width)		≈ 228 (top of granular pad)
East Abutment		
Granular A pad base	13-30, 10-01	Top of native silty clay (≈ 224.0 to 225.0)*
Footing base (granular pad thickness ≥ one footing width)		≈ 228 (top of granular pad)

* Elevations are approximate since the subgrade level could have been altered during the advance contract work.

It is anticipated that the native silty clay will be exposed if all the preloading fill is removed. The exposed subgrade must be properly prepared as described below to avoid prolonged exposure. Placement and compaction of the Granular A pad must be carried out in the dry in general accordance with OPSS 501.

It is recommended that footings founded on a compacted Granular A pad with a minimum thickness of one footing width, at the founding elevations quoted in the table above, be designed for a factored geotechnical resistance at Ultimate Limit States (ULS) of 400 kPa and a geotechnical resistance at Serviceability Limit States (SLS) of 275 kPa (corresponding to maximum 25 mm settlement). These values are for vertical concentric loads only. Effects of load inclination and eccentricity need to be taken into account as per the Canadian Highway Bridge Design Code (CHBDC, 2010).

Resistance to lateral forces / sliding resistance between the concrete footings and compacted Granular A subgrade should be calculated in accordance with the CHBDC 2010 assuming an unfactored coefficient of friction, tan δ, of 0.55.

Frost protection should be provided to all retaining wall footings and may take the form of 1.4 m of earth cover in any direction, or equivalent thermal insulation, over the underside of the footings.

Once the desired founding subgrade level of the granular pad is reached, careful inspection should be carried out to delineate any loose/softened soils or other deleterious materials. The native subgrade should not be allowed to loosen or deteriorate by ponding water and construction traffic. The Granular A materials should then be placed and compacted to 100% standard Proctor maximum dry density (SPMDD) within ±2% of the optimum moisture content. Figure G1 in Appendix G shows typical details of an abutment on a compacted Granular A core that is applicable to the retaining walls.

11 LATERAL EARTH PRESSURES

Backfill to abutment walls and retaining walls should be in accordance with OPSS 902 and placed to the extents shown in OPSD 3101.150. Any backfill to these retaining structures should consist of Granular A or Granular B Type II material meeting the requirements of OPSS.PROV 1010.

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.

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

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

where:
- P_h = horizontal pressure on the wall at depth h (kPa)
- K = earth pressure coefficient (see Table 11.1)
- γ = unit weight of retained soil (see Table 11.1)
- 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 2.0 m for Granular B Type I or 1.7 m for Granular A or Granular B Type II.

Earth pressure coefficients for backfill to walls are dependent on the material used as backfill. Typical values are shown in Table 11.1 below.

Table 11.1 – Earth Pressure Coefficients (K)

Wall Condition	Earth Pressure Coefficient (K)					
	OPSS Granular A and Granular B Type II $\phi = 35°, \gamma = 22.8$ kN/m^3		OPSS Granular B Type I $\phi = 32°, \gamma = 21.2$ kN/m^3		Embankment Fill $\phi = 30°, \gamma = 20.0$ kN/m^3	
	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)
Active (Unrestrained Wall)	0.27	0.40	0.31	0.48	0.33	0.54
At rest (Restrained Wall)	0.43	-	0.47	-	0.50	-
Passive (Movement towards soil mass)	3.7	-	3.3	-	3.0	-

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 Table 11.1 are “ultimate” values and require certain movements for the respective conditions to be mobilized. The values to be used in design can be estimated from Figure C6.9.1 (a) in the Commentary to the Canadian Highway Bridge Design Code.

It is recommended that perforated sub-drains and/or weep holes be installed, where applicable, to provide positive drainage of the granular backfill behind the abutment and retaining walls.

12 HIGH FILLS AT IMMEDIATE APPROACHES

Design information provided by URS indicates that the new Highway 400 underpass bridge will require approach fills in the order of 8 to 9 m in height. This report focuses on the high fills at the immediate approaches within 20 m of the bridge abutments. Other high fill locations such as the ramp areas will be covered in a separate report.

The new fills within about 20 m behind the abutments will induce foundation settlement as a result of consolidation of the underlying silty clay deposit. This settlement will cause downdrag on the abutment piles and settlement under the approach slabs. In order to mitigate the adverse effects of this settlement, consideration was given to alternate options of fill construction. The feasibility of some common fill construction methods for this project was discussed in technical memoranda prepared in March and July 2014. The feasibility of these methods are summarized as follows:

- Preloading and surcharging is a feasible means to mitigate post construction ground settlement taking into consideration the subsurface conditions and the project requirements
- Sub-excavation is not feasible at this site since the silty clay deposit is extensive and the more compressible zones are present at depths below a weathered crust
- Wick drains are generally useful in accelerating consolidation within softer clays but are likely not cost effective for this site.
- Lightweight fill such as extruded polystyrene (EPS) can be used, but will create challenges in the design and operation of the integral abutments. Given that a waiting period is available for the preloading and surcharging, use of EPS is likely not cost effective for this project
- Another type of lightweight fill such as blast furnace slag does not have a sufficiently low unit weight to be effective in mitigating ground settlements.

The selected fill construction method is to preload and surcharge the approach fill areas (within 20m behind the abutments) to induce ground settlement within a waiting period. A program for earthworks (fill placement at the approaches) was prepared, as well as a instrumentation and monitoring program. It is noted that the preloading and surcharging program commenced in the Summer of 2014 and is ongoing at the time of preparation of this report.

12.1 Preloading and Surcharging

It is estimated that placement of up to 9 m of fill at the west abutment and 8 m of fill at the east abutment would result in immediate settlement in the order of 30 mm. Without preloading and surcharging, subsequent primary and secondary consolidation settlements are estimated to be up to 180 mm and 160 mm for the west and east approaches, respectively.

In order to mitigate post construction settlement to within tolerable limits and to reduce the downdrag force acting on the piles, it is recommended that the full height of fill plus 2 m of surcharge be placed in advance of bridge and abutment pile construction. Accordingly, the temporary preloading fill will be up to 11 m high at the west approach and up to 10 m high at the east approach. A waiting period of at least 6 months should be made available. The actual time lapse after which the surcharge may be removed can be determined by an instrumentation program to be implemented for the embankments. Once the surcharge is removed and provided that the preloading and surcharging is carried out as recommended, it is estimated that post construction settlement should not exceed approximately 20 mm in 10 years.

12.2 Stability Analyses

Based on the design preload and surcharge fill configurations provided by URS, limit equilibrium stability analyses were carried out for representative temporary and permanent cases. The stability analysis was carried out using the commercially available slope stability program GEO-SLOPE and employing the Morgenstern-Price method.

It is noted that there is space restriction in the vicinity of the west approach due to the presence of the North Schomberg River and Highway 400, resulting in the need to steepen the temporary fill slopes to 1.5H : 1V along some sections of the fill perimeter. For this slope inclination to remain stable during the preload/surcharge stage, compacted OPSS 1010 Granular A or B Type II is recommended to be used. Beyond this construction stage and after the river is realigned, Select Subgrade Material (SSM) may be considered as an alternate material for use in constructing the remainder of the embankments at a slope inclination not steeper than 2H : 1V. SSM or approved granular materials may be used at the east approach at all stages with a slope inclination not steeper than 2H : 1V.

As per current MTO practice, a mid-height bench is required for embankments in excess of 8 m in height. Based on the latest GA, mid-height berms each with a minimum width of 2m, have been incorporated for both the west and east approach fills.

As per typical MTO requirements, a Factor of Safety (F.S.) of 1.3 is acceptable for short term conditions (preload/surcharge) and for total stress (undrained) conditions. A F.S. of 1.5 is acceptable for long term (drained) conditions after excess pore pressures generated in the foundation soil caused by fill placement have dissipated. The results indicate that these acceptance criteria are generally satisfied for the cases analysed.

The computed factors of safety for temporary and permanent slope configurations with various combinations of geotechnical parameters are summarized in Table 12.1. Selected slope stability computation outputs are included in Figures H1 to H10 of Appendix H.

Table 12.1 – Computed Factors of Safety

Foundation Element	Side Slope	Case		Estimated Factors of Safety: Gran. A or B Type II ($\Phi' = 35°$)	Estimated Factors of Safety: SSM ($\Phi' = 30°$)	Figure
West Approach	1.5H : 1V	Temporary	Surcharge	1.28	-	H1
			Undrained	1.23		H2
	2.0H : 1V		Surcharge	1.53	-	H3
	2.0H:1V with 2 m bench	Permanent	Surcharge	1.53	-	-
			Drained	1.58	1.43	H4 & H5
			Undrained	1.47	1.46	H6 & H7
East Approach	2.0H : 1V	Temporary	Surcharge	-	1.46	H8
		Permanent	Drained	-	1.51	H9
			Undrained	-	1.80	H10

12.3 Embankment Design and Construction

It is recommended that MTO approved Select Subgrade Material (SSM) or granular materials satisfying OPSS 1010 requirements be used for constructing the approach embankments at this site. Based on the above analyses, the west permanent embankment constructed using these materials will be stable at a slope inclination not steeper than 2H : 1V with a mid-height bench of at least 2 m in width. For the east permanent embankment constructed with these materials, a slope inclination of 2H : 1V will be stable.

During the preloading stage at the west approach, OPSS 1010 Granular A or B Type II must be used to achieve stability for steeper sideslopes of 1.5H : 1V without a bench.

All embankment fill must be constructed with adequate quality control in accordance with OPSS 206 and 501 requirements. Silt or clay materials are not recommended for embankment construction at this site due to potentially higher post construction settlement, difficulties in achieving the specified compaction and potential embankment stability issues.

It is also recommended that all permanent and temporary slope surfaces be vegetated and seeded in accordance with current MTO practice with reference to OPSS 804. Erosion and sedimentation control should also be implemented with reference to OPSS 805. It is important to note that slopes steeper than 2H : 1V will be subject to surficial instability which may include sloughing and gullying. Surface runoff and precipitation must be prevented from flowing perpendicularly down any slope surface. Protection measures and remediation measures will have to be taken as necessary to avoid adverse impacts on the river and the highway.

An advance contract for preloading and surcharging at the underpass bridge abutments and immediate approaches is currently being implemented.

12.4 Instrumentation and Monitoring

The duration and time for substantial completion of settlement due to preloading and surcharging are to be confirmed by carrying out a geotechnical instrumentation and monitoring program as part of the advance contract. The proposed program includes settlement rods, structure points and pavement markers. The settlement rods are used to monitoring foundation settlement under the footprints of the new approach fills. The structure points and pavement markers are used to monitor potential settlement impact of the new fill on the existing Line 5 bridge and the adjacent Highway 400, respectively. The instrumentation and monitoring tasks are covered under the Contract Administrator (CA) assignment. Details on the type, location and number of instruments, procurement and installation procedures by the Contractor, monitoring frequency, data compilation, interpretation and reporting are included in Appendix I.

As of the date of issue of this draft report, the top of the approach fills has been reached for about three months, and the instrumentation and monitoring program is well underway.

13 ROADWAY PROTECTION

Roadway protection may be required during construction of the proposed underpass. An item titled "Protection System" as per OPSS 539 should be included in the contract documents. It is recommended that Performance Level 2 as per Clause 539.04.01.01 and the alignment of the shoring be specified on the contract drawings.

The design of roadway protection should be the responsibility of the Contractor. However, one option that is considered to be suitable for use as temporary shoring at this site is soldier pile and lagging wall. It is anticipated that the soldier piles will need to be extended to sufficient depth into the silty clay in order to develop the required toe resistance.

A temporary soldier pile and lagging wall may be designed using the parameters given below:

γ	=	20 kN/m^3
γ_w	=	10 kN/m^3
K_a	=	0.33 (approach fills)
	=	0.33 (silty clay)
K_p	=	3.0 (approach fills)
	=	3.0 (silty clay)

The designer of the roadway protection system should check whether the depth of pile is sufficient to provide base fixity.

The actual pressure distribution acting on the shoring system is a function of the construction sequence and the relative flexibility of the wall and these factors must be considered when designing the shoring system. All shoring systems should be designed by a Professional Engineer experienced in such designs.

14 EXCAVATION AND BACKFILL

Temporary excavations will be required during construction at this site. All temporary excavations must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purpose of OHSA, the native silty clay and existing fills at the site may be classified as Type 3 soils. Based on existing information, excavations will likely extend to or slightly below the groundwater level.

All excavations must be carried out in a manner that avoids undermining or destabilising the existing bridge foundations, existing approach slopes and the adjacent highway.

Earth excavation for pile cap construction at the pier will penetrate through the highway embankment fill and the underlying silty clay. Where space permits, temporary excavation may be formed with temporary sideslopes not steeper than 1H : 1V. Flatter slopes may be required at locations where the soils are less competent than what is assumed during design or where water seepage affects surficial stability.

Excavation and backfilling for foundation construction should be carried out with reference to the requirements in OPSS 902. Backfill to the abutments should consist of Granular A or Granular B Type II materials meeting the gradation and relevant requirements stipulated in OPSS 1010. Compaction procedures and equipment to be used adjacent to the existing structures must be in accordance with the relevant OPSS 501 requirements.

15 GROUNDWATER AND SURFACE WATER CONTROL

It is anticipated that the amount of perched water within the fills would be limited. Groundwater from water-bearing sand and silt interlayers within the silty clay should be expected. For temporary excavations for underpass construction at this site, groundwater control will likely be limited to diverting surface runoff and preventing precipitation from entering the excavations supplemented by sump pumping and use of perimeter ditches. Filtered sumps must be designed properly so that construction drainage water containing eroded soil and fines do not flow onto existing roadways.

The design of the dewatering systems that may be required is the responsibility of the Contractor. The Contract Documents must alert the Contractor of this responsibility and the need to engage a dewatering specialist.

16 SEISMIC CONSIDERATIONS

16.1 Seismic Design Parameters

The following seismic parameters should be used for design:

- Velocity Related Seismic Zone 0
- Zonal Velocity Ratio 0.05
- Acceleration Related Seismic Zone 1
- Zonal Acceleration Ratio 0.05
- Peak Horizontal Acceleration 0.167 g

The peak horizontal acceleration of 0.167 g is for a seismic event with 2% probability of exceedance in 50 years (2475-year return period) per NBCC 2010. The soil profile type for 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

Based on the CHBDC, the foundation silty clay at this site is assessed to be not prone to liquefaction.

The embankments composing of compacted granular materials will be constructed above the groundwater level and are not considered to be prone to liquefaction.

17 EXISTING UTILITY SERVICES AND ADJACENT STRUCTURES

Hydro lines were located on the north side of the proposed structure during the site investigation. It is understood that removal of the poles and hydro lines was scheduled to be completed in August 2014.

It is recommended that the exact locations of any existing utilities be established by the designer, and compared with the extent of the potential work zones related to the construction of the proposed structure and associated works. These utilities should not be damaged during construction of the new underpass and its immediate approaches. If necessary, relocation of, and/or special protective measures for affected utilities may be required.

Pile driving will be carried out for the new bridge at distances of about 30 m, or greater, from the existing bridge. Based on typical construction vibration guidelines, vibration resulting from driving piles for the new bridge is not expected to result in adverse effects on the existing bridge. Consideration may be given to carrying out pre-construction and post-construction condition survey of the existing bridge.

18 CONSTRUCTION CONCERNS

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

- The surficial soils at the site are susceptible to disturbance by construction traffic and particular attention/measures will be required to provide a stable trafficable base for movement of heavy equipment.
- Piles driven to the very stiff to hard silty clay till and dense to very dense sands and silts may achieve the required geotechnical resistance at varying elevations.
- Although there was little direct evidence of their presence during drilling, glacial deposits inherently contain cobbles and boulders, which may affect installation of H-piles, or caissons. The Contractor shall be prepared to remove, drill through and/or penetrate these obstructions and extend the piles/caissons to competent foundation level.
- The cohesionless sands and silts at depth would be susceptible to disturbance under conditions of unbalanced hydrostatic head. If caissons are employed, temporary steel liners should be used to support caisson sidewalls and provide seepage cut-off where required.
- The base of the caissons should be maintained higher than the top of the water-bearing sands and silts to minimize issues of basal instability.
- The monitoring program for preloading and surcharging of approach fills shall be continued as part of the advance contract to confirm the rate and magnitude of settlement, and to establish when ground settlement due to fill placement is practically stabilized prior to pile driving.

19 CLOSURE

Engineering analysis and preparation of the report were carried out by Ms. R. Palomeque Reyna, P.Eng.

The report was reviewed by Mr. Sydney Pang, P.Eng., and Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

THURBER ENGINEERING LTD.

Rocío Palomeque Reyna, P.Eng., M.Eng.
Geotechnical Engineer

Sydney Pang, P.Eng.,
Associate, Senior Foundations Engineer

Report Reviewed by:
P. K. Chatterji, P.Eng.,
Review Principal, Designated MTO Contact

Appendix A

Record of Borehole Sheets

(current and previous investigations)

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

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

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

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

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

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

NOTE: Hierarchy of Soil Strength Prediction

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

4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

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

5. LEGEND FOR RECORDS OF BOREHOLES

SYMBOLS AND ABBREVIATIONS FOR SAMPLE TYPE

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

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

▼ Water Level

C_{pen} Shear Strength Determination by Pocket Penetrometer

(1) SPT 'N' Value Standard Penetration Test 'N' Value – refers to the number of blows from a 63.5kg hammer free falling a height of 0.76m to advance a standard 50 mm outside diameter split spoon sampler for 0.3 m depth into undisturbed ground.

(2) DCPT Dynamic Cone Penetration Test – Continuous penetration of a 50 mm outside diameter, 60° conical steel point attached to "A" size rods driven by a 63.5 kg hammer free falling a height of 0.76 m. The resistance to cone penetration is the number of hammer blows required for each 0.3 m advance of the conical point into undisturbed ground.

UNIFIED SOILS CLASSIFICATION

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

EXPLANATION OF ROCK LOGGING TERMS

ROCK WEATHERING CLASSIFICATION

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

SYMBOLS

CLAYSTONE

SILTSTONE

SANDSTONE

COAL

Bedrock (general)

DISCONTINUITY SPACING

Bedding	**Bedding Plane Spacing**
Very thickly bedded	Greater than 2m
Thickly bedded	0.6 to 2m
Medium bedded	0.2 to 0.6m
Thinly bedded	60mm to 0.2m
Very thinly bedded	20 to 60mm
Laminated	6 to 20mm
Thinly Laminated	Less than 6mm

TERMS

Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.
Rock Quality Designation: (RQD)	Total length of sound core recovered in pieces 0.1m in length or larger as a percentage of total core run length.
Uniaxial Compressive Strength (UCS)	Axial stress required to break the specimen
Fracture Index: (FI)	Frequency of natural fractures per 0.3m of core run.

STRENGTH CLASSIFICATION

Rock Strength	**Approximate Uniaxial Compressive Strength (MPa)**	**(psi)**	**Field Estimation of Hardness***
Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Medium Strong	25.0 to 50.0	3,500 to 7,500	Breaks under single blow of geological hammer.
Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail

THURBER

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 13-19

METRIC

W.P. P-13-03 | LOCATION N 4 881 691.4 E 294 765.7 | ORIGINATED BY GA

HWY 400 | BOREHOLE TYPE Hollow Stem Augers | COMPILED BY MFA

DATUM Geodetic | DATE 2014.01.24 - 2014.01.28 | CHECKED BY KY

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)	GR	SA	SI	CL
223.7	GROUND SURFACE										
0.0	**TOPSOIL:** (100 mm)										
0.1	Silty **CLAY** Stiff to Very Stiff Brown Moist	1	SS	8							
					223						
		2	SS	8				0	0	51	49
					222						
		3	SS	17							
		4	SS	15	221						
		5	SS	13				0	0	52	48
					220						
		6	SS	17							
219.1 / 4.6	Trace to some sand, trace gravel Grey Moist to Wet				219						
		7	SS	14							
					218						
		8	SS	8							
					217	>>2.0					
					216			Oedometer Test			
		1	TW	PH				2	19	47	32
					215	3.0					
	Firm	9	SS	6							
					214	2.5					

Continued Next Page

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

UNIT WEIGHT γ kN/m^3

REMARKS & GRAIN SIZE DISTRIBUTION (%)

+ 3, × 3: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 13-19

METRIC

W.P.	P-13-03	LOCATION	N 4 881 691.4 E 294 765.7	ORIGINATED BY	GA
HWY	400	BOREHOLE TYPE	Hollow Stem Augers	COMPILED BY	MFA
DATUM	Geodetic	DATE	2014.01.24 - 2014.01.28	CHECKED BY	KY

Continued From Previous Page

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
	Silty **CLAY**, trace sand Firm to Stiff Grey Wet
207.4 / 16.3	Very Stiff

SAMPLES

NUMBER	TYPE	"N" VALUES	Field vane shear strength (sensitivity)	GR	SA	SI	CL
10	SS	5					
			+ 3.0 (elev. ~212)				
11	SS	4		0	0	48	52
			+ 3.0 (elev. ~210.4)				
2	TW	PH		0	0	62	38
3	TW	PH					
			+ 2.0 (elev. ~207.4)				
12	SS	17					
13	SS	19		0	5	55	40

ELEVATION SCALE: 213, 212, 211, 210, 209, 208, 207, 206, 205, 204

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m^3

REMARKS & GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

GROUND WATER CONDITIONS; STRAT PLOT

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 13-19

METRIC

W.P.	P-13-03	LOCATION	N 4 881 691.4 E 294 765.7	ORIGINATED BY	GA
HWY	400	BOREHOLE TYPE	Hollow Stem Augers	COMPILED BY	MFA
DATUM	Geodetic	DATE	2014.01.24 - 2014.01.28	CHECKED BY	KY

Continued From Previous Page

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
	Silty **CLAY**, trace sand, Very Stiff, Grey, Wet	14	SS	18				
202.3 / 21.3	Silty **CLAY**, trace to some sand, trace gravel, Very Stiff to Hard, Grey, Moist (TILL)	15	SS	28	0	14	54	32
		16	SS	35				
195.9 / 27.7	**SAND**, trace silt, Dense, Grey, Wet	17	SS	40				

Elevation scale: 203, 202, 201, 200, 199, 198, 197, 196, 195, 194

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m3

REMARKS & GRAIN SIZE DISTRIBUTION (%)

Continued Next Page

+3, ×3: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 13-19

METRIC

W.P.	P-13-03	LOCATION	N 4 881 691.4 E 294 765.7	ORIGINATED BY	GA
HWY	400	BOREHOLE TYPE	Hollow Stem Augers	COMPILED BY	MFA
DATUM	Geodetic	DATE	2014.01.24 - 2014.01.28	CHECKED BY	KY

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT kN/m³	GR	SA	SI	CL
	Continued From Previous Page											
	SAND, trace to some silt Very Dense Grey Wet					193						
						192						
			18	SS	63		○		0	84	16 (SI+CL)	
						191						
						190						
						189						
			19	SS	106		○					
188.0 35.7	END OF BOREHOLE AT 35.7 m. BOREHOLE OPEN TO 35.0m AND WATER LEVEL AT 19.4 m UPON COMPLETION. Piezometers installation consists of two 19 mm diameter Schedule 40 PVC pipes with a 3.0 m slotted screen.											

Column headings: SOIL PROFILE; SAMPLES; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

WATER LEVEL READINGS (DEEP PIEZOMETER):

DATE	DEPTH (m)	ELEV. (m)
Feb 26/ 14	7.7	216.0
Mar 13/ 14	7.3	216.4

WATER LEVEL READINGS (SHALLOW PIEZOMETER):

DATE	DEPTH (m)	ELEV. (m)
Feb 26/ 14	2.5	221.2
Mar 13/ 14	2.6	221.1

+ ³, × ³: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 13-20

METRIC

W.P.	P-13-03	LOCATION	N 4 881 705.9 E 294 805.2	ORIGINATED BY	GA
HWY	400	BOREHOLE TYPE	Hollow Stem Augers	COMPILED BY	AN
DATUM	Geodetic	DATE	2013.12.19 - 2014.01.11	CHECKED BY	KY

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
224.9	GROUND SURFACE										
224.9 / 0.0	**ASPHALT:** (100 mm)										
0.1	Gravelly **SAND**, some silt, Compact, Brown, Moist (FILL)	1	SS	27				20	61	19 (SI+CL)	
224.1 / 0.8	Silty **CLAY**, trace sand, Firm to Stiff, Brown, Moist	2	SS	11		224					
		3	SS	7	▽	223		0	0	49	51
		4	SS	6		222					
		5	SS	12		221					
		6	SS	14							
		7	SS	13		220					
						219					
		8	SS	15		218		0	3	50	47
217.7 / 7.2	Very Stiff										
		9	SS	16		217					
216.2 / 8.7	Grey					216					
		10	SS	13		215					

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100
PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60
UNIT WEIGHT γ kN/m3
REMARKS & GRAIN SIZE DISTRIBUTION (%)

Continued Next Page

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

RECORD OF BOREHOLE No 13-20

METRIC

W.P.	P-13-03	LOCATION	N 4 881 705.9 E 294 805.2	ORIGINATED BY	GA
HWY	400	BOREHOLE TYPE	Hollow Stem Augers	COMPILED BY	AN
DATUM	Geodetic	DATE	2013.12.19 - 2014.01.11	CHECKED BY	KY

Continued From Previous Page

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)	GR	SA	SI	CL
	Silty **CLAY** Firm to Stiff Grey Wet										
		11	SS	9	214		○				
					213	+ (Field Vane)					
		12	SS	6			├○┤	0	0	51	49
					212	+ 1.5 (Field Vane)					
		13	SS	10	211		○				
210.1 / 14.8	Very Stiff				210						
		14	SS	18			○				
208.6 / 16.3					209						
		15	SS	14	208		○				
					207						
		16	SS	15			├○┤	0	0	52	48
					206						
					205						

Continued Next Page

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100
PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60
UNIT WEIGHT γ kN/m3
REMARKS & GRAIN SIZE DISTRIBUTION (%)

+ 3, × 3: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 13-20

METRIC

W.P. P-13-03 LOCATION N 4 881 705.9 E 294 805.2 ORIGINATED BY GA

HWY 400 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN

DATUM Geodetic DATE 2013.12.19 - 2014.01.11 CHECKED BY KY

Continued From Previous Page

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
	Silty **CLAY** Stiff to Very Stiff Grey Wet	17	SS	10		~15				
					204					
					203					
					202					
		18	SS	22		~20				
					201					
199.9 / 25.0	Silty **CLAY**, trace to some sand, trace gravel Very Stiff Grey Wet (TILL)				200					
					199					
		19	SS	26		~23	0	4	62	34
					198					
					197					
					196					
		20	SS	23		~15				
					195					

Continued Next Page

Column headings: SOIL PROFILE; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

+³, ×³: Numbers refer to Sensitivity

20 / 15 ◐ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 13-20

METRIC

W.P.	P-13-03	LOCATION	N 4 881 705.9 E 294 805.2	ORIGINATED BY	GA
HWY	400	BOREHOLE TYPE	Hollow Stem Augers	COMPILED BY	AN
DATUM	Geodetic	DATE	2013.12.19 - 2014.01.11	CHECKED BY	KY

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m3	GR	SA	SI	CL
	Continued From Previous Page										
	Silty **CLAY**, trace to some sand, trace gravel Very Stiff Grey Wet (TILL)					194					
193.2 / 31.7	Silty **SAND**, trace clay Very Dense Grey Wet					193					
		21	SS	115							
						192					
						191					
						190					
		22	SS	63				0	64	26	10
189.2 / 35.7	END OF BOREHOLE AT 35.7 m. BOREHOLE OPEN TO 35.7 m AND WATER LEVEL AT 1.8 m. BOREHOLE BACKFILLED WITH BENTONITE, PARTIALLY MIXED WITH AUGER CUTTINGS TO 0.4 m, CONCRETE TO 0.2 m THEN ASPHALT PATCH TO SURFACE.										

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

REMARKS & GRAIN SIZE DISTRIBUTION (%)

+ 3, × 3 : Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 13-21

METRIC

W.P.	P-13-03	LOCATION	N 4 881 708.6 E 294 868.2
HWY	400	BOREHOLE TYPE	Solid Stem Augers
DATUM	Geodetic	DATE	2014.01.31 - 2014.01.31

ORIGINATED BY GA
COMPILED BY AN
CHECKED BY KY

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
226.6	GROUND SURFACE
0.0	**TOPSOIL:** (100 mm)
0.1	Silty **SAND**, some clay, some gravel, occasional rootlets, Very Dense, Brown, Moist (FILL)
225.8 / 0.8	Silty **CLAY**, occasional rootlets, Firm, Dark Brown, Moist
225.1 / 1.4	Stiff to Very Stiff
220.9 / 5.6	Trace sand, trace gravel, Grey, Wet

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
1	SS	74				
2	SS	6	0	0	56	44
3	SS	11				
4	SS	16				
5	SS	14				
6	SS	13	0	0	62	38
7	SS	14				
8	SS	16				
9	SS	10				
10	SS	9	8	9	41	42

GROUND WATER CONDITIONS

ELEVATION SCALE: 226, 225, 224, 223, 222, 221, 220, 219, 218, 217

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB VANE
20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m^3

Continued Next Page

$+^3$, $\times^3$: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 13-21

METRIC

W.P.	P-13-03	LOCATION	N 4 881 708.6 E 294 868.2	ORIGINATED BY	GA
HWY	400	BOREHOLE TYPE	Solid Stem Augers	COMPILED BY	AN
DATUM	Geodetic	DATE	2014.01.31 - 2014.01.31	CHECKED BY	KY

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page										
	Silty **CLAY**, trace sand, trace gravel Very Stiff Grey Wet						216				
			11	SS	18				○		
215.3 11.3	END OF BOREHOLE AT 11.3 m. BOREHOLE OPEN TO 11.3 m AND DRY UPON COMPLETION. Piezometer installation consists of 19 mm diameter Schedule 40 PVC pipe with a 3.0 m slotted screen.										

WATER LEVEL READINGS:

DATE	DEPTH (m)	ELEV. (m)
Feb 26/ 14	1.6	225.0
Mar 13/ 14	1.7	224.9

$+^3, \times^3$: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 13-22

METRIC

W.P.	P-13-03	LOCATION	N 4 881 720.5 E 294 907.9
HWY	400	BOREHOLE TYPE	Solid Stem Augers
DATUM	Geodetic	DATE	2014.01.30 - 2014.01.30

ORIGINATED BY	GA
COMPILED BY	AN
CHECKED BY	KY

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
227.7	GROUND SURFACE
0.0	**TOPSOIL:** (125 mm)
0.1	Silty **SAND**, some clay, some gravel, occasional rootlets Loose Brown Moist (FILL)
227.1 / 0.6	Silty **CLAY**, trace sand Stiff Brown Moist
225.5 / 2.2	Very Stiff
224.7 / 3.0	Grey
219.5 / 8.2	END OF BOREHOLE AT 8.2 m. BOREHOLE OPEN TO 8.2 m AND DRY UPON COMPLETION. Piezometer installation consists of 19 mm diameter Schedule 40 PVC pipe with a 3.0 m slotted screen.

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
1	SS	8	16	45	24	15
2	SS	10				
3	SS	14	0	9	43	48
4	SS	16				
5	SS	15				
6	SS	15				
7	SS	15				
8	SS	14				
9	SS	11	0	2	46	52

GROUND WATER CONDITIONS

ELEVATION SCALE: 227, 226, 225, 224, 223, 222, 221, 220

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20, 40, 60, 80, 100

SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB VANE
20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

WATER LEVEL READINGS:

DATE	DEPTH (m)	ELEV. (m)
Feb 26/ 14	2.1	225.6
Mar 13/ 14	2.2	225.5

+³, ×³: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 13-27

METRIC

W.P.	P-13-03	LOCATION	N 4 881 653.3 E 294 725.5	ORIGINATED BY	GA
HWY	400	BOREHOLE TYPE	Solid Stem Augers	COMPILED BY	AN
DATUM	Geodetic	DATE	2014.01.29 - 2014.01.29	CHECKED BY	KY

SOIL PROFILE | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100) | SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20, 40, 60, 80, 100) | PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (20, 40, 60) | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%)

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	GR	SA	SI	CL
222.0	GROUND SURFACE								
0.0	**TOPSOIL:** (100 mm)								
0.1	Silty **CLAY**, trace sand, occasional rootlets Firm to Stiff Brown Moist	1	SS	8					
		2	SS	6	221	0	8	60	32
	Occasional iron oxide staining	3	SS	8	220				
		4	SS	13					
219.0 3.0	Trace gravel Very Stiff Grey	5	SS	16	219				
218.3 3.7		6	SS	14	218	3	6	42	49
		7	SS	15	217				
					216				
		8	SS	8	215				
		9	SS	6	214	0	0	46	54
					213				
		10	SS	6					
212.2 9.8	END OF BOREHOLE AT 9.8 m.								

2.0

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 13-27

METRIC

W.P.	P-13-03	LOCATION	N 4 881 653.3 E 294 725.5	ORIGINATED BY	GA
HWY	400	BOREHOLE TYPE	Solid Stem Augers	COMPILED BY	AN
DATUM	Geodetic	DATE	2014.01.29 - 2014.01.29	CHECKED BY	KY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	Continued From Previous Page										
	BOREHOLE OPEN TO 9.8 m AND WATER LEVEL AT 6.4 m UPON COMPLETION. Piezometer installation consists of 19 mm diameter Schedule 40 PVC pipe with a 3.0 m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) Feb 26/ 14 0.5 221.5 Mar 13/ 14 0.5 221.5										

+³, ×³: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 13-28

METRIC

W.P. P-13-03	LOCATION N 4 881 664.6 E 294 748.7	ORIGINATED BY GA
HWY 400	BOREHOLE TYPE Solid Stem Augers	COMPILED BY AN
DATUM Geodetic	DATE 2014.01.28 - 2014.01.28	CHECKED BY KY

Shear strength kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE

Ground surface elevation: 223.3

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
223.3	GROUND SURFACE					
0.0 / 0.1	**TOPSOIL:** (100 mm)					
	Silty **CLAY**, trace sand, occasional rootlets. Stiff. Brown. Moist. (FILL)	1	SS	8	223	
222.6 / 0.7	Silty **SAND**, trace gravel, trace to some clay. Loose. Brown. Moist. (FILL)	2	SS	8	222	8 68 24 (SI+CL)
221.8 / 1.5	Silty **CLAY**. Stiff to Very Stiff. Brown. Moist	3	SS	10		
		4	SS	14	221	
		5	SS	17	220	0 0 50 50
219.6 / 3.7	Grey. Wet	6	SS	16	219	
		7	SS	19	218	
		8	SS	13	217	
		9	SS	15	216, 215	
214.6 / 8.7	Some sand	10	SS	14	214	0 12 49 39

Continued Next Page

$+^3$, $\times^3$: Numbers refer to Sensitivity

(%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 13-28

METRIC

W.P. P-13-03 | LOCATION N 4 881 664.6 E 294 748.7 | ORIGINATED BY GA

HWY 400 | BOREHOLE TYPE Solid Stem Augers | COMPILED BY AN

DATUM Geodetic | DATE 2014.01.28 - 2014.01.28 | CHECKED BY KY

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page										
212.8 10.5	Silty **CLAY**, some sand Stiff Grey Wet		11	SS	10		213		○ (~18)		

END OF BOREHOLE AT 10.5 m. BOREHOLE OPEN TO 9.9 m AND WATER LEVEL AT 5.1 m UPON COMPLETION.
Piezometer installation consists of 19 mm diameter Schedule 40 PVC pipe with a 3.0 m slotted screen.

WATER LEVEL READINGS:

DATE	DEPTH (m)	ELEV. (m)
Feb 26/ 14	1.8	221.5
Mar 13/ 14	1.8	221.5

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+ 3, × 3: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 13-29

METRIC

W.P.	P-13-03	LOCATION	N 4 881 655.2 E 294 807.7	ORIGINATED BY	GA
HWY	400	BOREHOLE TYPE	Hollow Stem Augers	COMPILED BY	AN
DATUM	Geodetic	DATE	2014.01.13 - 2014.01.15	CHECKED BY	KY

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
224.9	GROUND SURFACE
0.0	**ASPHALT:** (125 mm)
0.1	**SAND AND GRAVEL** Dense Brown Moist
224.0 / 0.9	(FILL)
	Silty **CLAY**, trace to some sand, trace gravel Stiff to Very Stiff Brown Moist
	Grey
217.7 / 7.2	Some sand
	Trace gravel

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	34	41 55 4 (SI+CL)
2	SS	16	
3	SS	15	
4	SS	16	
5	SS	17	0 0 39 61
6	SS	14	
7	SS	14	
8	SS	11	
9	SS	17	
10	SS	17	6 23 41 30

Column headings: GROUND WATER CONDITIONS; ELEVATION SCALE (224, 223, 222, 221, 220, 219, 218, 217, 216, 215); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (20 40 60); UNIT WEIGHT γ kN/m³.

Field vane at approx. elev. 217.7: 2.5

Continued Next Page

+ ³, × ³: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 13-29

METRIC

W.P.	P-13-03	LOCATION	N 4 881 655.2 E 294 807.7
HWY	400	BOREHOLE TYPE	Hollow Stem Augers
DATUM	Geodetic	DATE	2014.01.13 - 2014.01.15

ORIGINATED BY GA
COMPILED BY AN
CHECKED BY KY

Continued From Previous Page

SOIL PROFILE (ELEV / DEPTH, DESCRIPTION, STRAT PLOT)

Silty **CLAY**, trace gravel, trace to some sand
Stiff to Very Stiff
Brown
Moist

SAMPLE NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
11	SS	8				
12	SS	9	3	22	46	29
13	SS	15				
14	SS	16				
15	SS	14				
16	SS	16				

ELEVATION SCALE: 214, 213, 212, 211, 210, 209, 208, 207, 206, 205

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE: 20 40 60 80 100

Field vane sensitivity values: 2.5 (approx. El. 213.2), 2.5 (approx. El. 211.7)

PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m3

REMARKS & GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

GROUND WATER CONDITIONS

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 13-29

METRIC

W.P. P-13-03 | LOCATION N 4 881 655.2 E 294 807.7 | ORIGINATED BY GA

HWY 400 | BOREHOLE TYPE Hollow Stem Augers | COMPILED BY AN

DATUM Geodetic | DATE 2014.01.13 - 2014.01.15 | CHECKED BY KY

Continued From Previous Page

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
	Silty **CLAY**, trace sand Stiff to Very Stiff Grey Wet	17	SS	19			~ 19				
						204					
						203					
						202					
		18	SS	12			~ 20				
						201					
						200					
						199					
		19	SS	15			~ 22	0	0	72	28
					∇						
						198					
						197					
196.2 28.7	**SAND** and **SILT**, trace clay Dense Grey Wet										
						196					
		20	SS	31			~ 21	0	45	53	2
						195					

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DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100
PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60
UNIT WEIGHT γ kN/m³
REMARKS & GRAIN SIZE DISTRIBUTION (%)

+³, ×³: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

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Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 13-29

METRIC

W.P. P-13-03 LOCATION N 4 881 655.2 E 294 807.7 ORIGINATED BY GA

HWY 400 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN

DATUM Geodetic DATE 2014.01.13 - 2014.01.15 CHECKED BY KY

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS
	Continued From Previous Page						
	SAND and **SILT**, trace clay Dense Grey Wet				194		
					193		
		21	SS	37		○ (~19)	
					192		
					191		
					190		
189.2		22	SS	44		○ (~18)	
35.7	END OF BOREHOLE AT 35.7 m. BOREHOLE OPEN TO 35.1 m AND WATER LEVEL AT 26.5 m UPON COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND CUTTINGS TO 0.4 m, CONCRETE TO 0.2 m THEN ASPHALT PATCH TO SURFACE.						

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L, WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m3; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

+ 3, × 3: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 13-30

METRIC

W.P.	P-13-03	LOCATION	N 4 881 675.0 E 294 846.0	ORIGINATED BY	GA
HWY	400	BOREHOLE TYPE	Hollow Stem Augers	COMPILED BY	MFA
DATUM	Geodetic	DATE	2014.01.20 - 2014.01.23	CHECKED BY	KY

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
224.4	GROUND SURFACE
0.0	**TOPSOIL:** (175 mm)
0.2	Silty **CLAY** Firm to Stiff Brown Moist
218.8 / 5.6	Trace to some sand Grey
215.7 / 8.7	Trace gravel

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	6	
2	SS	11	0 0 54 46
3	SS	9	
4	SS	6	
5	SS	6	
6	SS	4	
1	TW	PH	Oedometer Test
7	SS	6	0 10 57 33
8	SS	7	
9	SS	6	

ELEVATION SCALE: 224, 223, 222, 221, 220, 219, 218, 217, 216, 215

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE: 20 40 60 80 100

Field vane sensitivity values: 2.5 (approx. elev. 218.7), 2.5 (approx. elev. 215.8)

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

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RECORD OF BOREHOLE No 13-30

METRIC

W.P. P-13-03	LOCATION N 4 881 675.0 E 294 846.0	ORIGINATED BY GA
HWY 400	BOREHOLE TYPE Hollow Stem Augers	COMPILED BY MFA
DATUM Geodetic	DATE 2014.01.20 - 2014.01.23	CHECKED BY KY

SOIL PROFILE — ELEV DEPTH; DESCRIPTION; STRAT PLOT

SAMPLES — NUMBER; TYPE; "N" VALUES

GROUND WATER CONDITIONS; ELEVATION SCALE

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100

PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

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

Continued From Previous Page

Silty **CLAY**, trace to some sand, trace to some gravel
Firm to Stiff
Grey
Moist

Number	Type	"N" Values	Approx. Elevation	Field Vane (kPa)	GR	SA	SI	CL
2	TW	PH	213.5		11	12	45	32
			212.8	+ (Sensitivity 2.0)				
10	SS	14	211.9					
11	SS	12	210.4					
12	SS	7	208.9		0	0	63	37
			208.1	+ (Sensitivity 2.0)				
13	SS	13	207.4					
14	SS	14	205.9					

Elevation scale: 214, 213, 212, 211, 210, 209, 208, 207, 206, 205

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

20 / 15 ◐ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 13-30

METRIC

W.P.	P-13-03	LOCATION	N 4 881 675.0 E 294 846.0	ORIGINATED BY	GA
HWY	400	BOREHOLE TYPE	Hollow Stem Augers	COMPILED BY	MFA
DATUM	Geodetic	DATE	2014.01.20 - 2014.01.23	CHECKED BY	KY

Continued From Previous Page

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT kN/m³	GR	SA	SI	CL
	Silty **CLAY**, trace to some sand, trace gravel Stiff Grey Moist	15	SS	8					2	17	52	29
					204							
						+ 2.5						
					203							
		3	TW	PH								
					202							
		16	SS	15								
					201							
					200							
199.7 / 24.7	Very Stiff											
					199							
		17	SS	18								
					198							
					197							
196.7 / 27.7	Silty **CLAY**, trace to some sand, trace gravel Hard Grey Moist (TILL)											
					196							
		18	SS	44					0	0	50	50
					195							

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DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

$+^3$, $\times^3$: Numbers refer to Sensitivity

20 / 15 / 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 13-30

METRIC

W.P.	P-13-03	LOCATION	N 4 881 675.0 E 294 846.0	ORIGINATED BY	GA
HWY	400	BOREHOLE TYPE	Hollow Stem Augers	COMPILED BY	MFA
DATUM	Geodetic	DATE	2014.01.20 - 2014.01.23	CHECKED BY	KY

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
	Continued From Previous Page									
	Silty **CLAY**, trace to some sand, trace gravel Hard Brown Moist (TILL)				194					
					193					
		19	SS	103	192	○ ⊢⊣	0	0	50	50
					191					
190.0 / 34.4	**SAND**, some silt, trace gravel Very Dense Grey Wet				190					
188.8 / 35.7		20	SS	107	189	○	9	79	12 (SI+CL)	

END OF BOREHOLE AT 35.7 m.
BOREHOLE OPEN TO 35.1 m AND WATER LEVEL AT 3.4 m UPON COMPLETION.
Piezometers installation consists of two 19 mm diameter Schedule 40 PVC pipes with a 1.52 m slotted screen.

WATER LEVEL READINGS (DEEP PIEZOMETER):

DATE	DEPTH (m)	ELEV. (m)
Feb 26/ 14	1.0	223.4
Mar 13/ 14	0.8	223.6

WATER LEVEL READINGS (SHALLOW PIEZOMETER):

DATE	DEPTH (m)	ELEV. (m)
Feb 26/ 14	1.2	223.2
Mar 13/ 14	0.7	223.7

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No BH10-01

METRIC

W.P. P-13-03 LOCATION N 4 881 710.7 E 294 841.6 ORIGINATED BY SL

HWY 400 BOREHOLE TYPE Hollow Stem Augers/TriCone Method COMPILED BY LRB

DATUM Geodetic DATE 2010.03.24 - 2010.03.26 CHECKED BY AEG

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	GR	SA	SI	CL
225.5	GROUND SURFACE								
0.0	**TOPSOIL**, with roots and rootlets								
0.2	Clayey **SILT**, trace to some sand, trace gravel Firm to Very Stiff Brown to Grey Moist (TILL)(CL-CI)	1	SS	4	225				
		2	SS	15		0	1	56	43
		3	SS	21	224				
		4	SS	19	223				
		5	SS	19	222	0	1	45	54
		6	SS	16	221				
		7	SS	9	220				
		1	TW		219				
		8	SS	10	218	2	15	47	36
	Firm from 8.5m to 9.9m	9	SS	6	217				
		10	SS	6	216				

Column headings: SOIL PROFILE; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20 40 60 80 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (20 40 60); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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+ 3, × 3 : Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No BH10-01

METRIC

W.P.	P-13-03	LOCATION	N 4 881 710.7 E 294 841.6	ORIGINATED BY	SL
HWY	400	BOREHOLE TYPE	Hollow Stem Augers/TriCone Method	COMPILED BY	LRB
DATUM	Geodetic	DATE	2010.03.24 - 2010.03.26	CHECKED BY	AEG

Continued From Previous Page

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)	GR	SA	SI	CL
	Clayey **SILT**, trace to some sand, trace gravel Stiff to Very Stiff Grey Moist (TILL)(CL)				215						
		11	SS	9							
					214						
		2	TW		213			0	5	48	47
						4.0					
					212						
		12	SS	10							
					211						
		13	SS	9	210						
					209						
		14	SS	20				1	6	61	32
					208						
		15	SS	27	207						
					206						

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

20 / 15 / 5 / 10 (%) STRAIN AT FAILURE

ONTMT4S 0615.GPJ 2012TEMPLATE(MTO).GDT 12/8/14

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No BH10-01

METRIC

W.P. P-13-03 | LOCATION N 4 881 710.7 E 294 841.6 | ORIGINATED BY SL

HWY 400 | BOREHOLE TYPE Hollow Stem Augers/TriCone Method | COMPILED BY LRB

DATUM Geodetic | DATE 2010.03.24 - 2010.03.26 | CHECKED BY AEG

Continued From Previous Page

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
	Clayey **SILT**, trace to some sand, trace gravel Stiff to Very Stiff Grey Moist to wet (TILL)(CL)	16	SS	14				
		17	SS	28				
		18	SS	11	0	1	56	43
197.1 28.4	Clayey **SILT**, some sand seams, trace gravel, occasional cobbles Hard Grey Moist (TILL)(CL)	19	SS	101	3	18	49	30

Elevation scale: 205, 204, 203, 202, 201, 200, 199, 198, 197, 196

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m3

REMARKS & GRAIN SIZE DISTRIBUTION (%)

Continued Next Page

+3, ×3: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

ONTMT4S 0615.GPJ 2012TEMPLATE(MTO).GDT 12/8/14

Ministry of Transportation
Ontario

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RECORD OF BOREHOLE No BH10-01

METRIC

W.P.	P-13-03	LOCATION	N 4 881 710.7 E 294 841.6	ORIGINATED BY	SL
HWY	400	BOREHOLE TYPE	Hollow Stem Augers/TriCone Method	COMPILED BY	LRB
DATUM	Geodetic	DATE	2010.03.24 - 2010.03.26	CHECKED BY	AEG

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE) 20 40 60 80 100	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page										
	Clayey **SILT**, some sand seams, trace gravel, occasional cobbles Hard Grey Moist (TILL)(CL)						195 194				
193.1			20	SS	100/.25				○		
32.4	END OF BOREHOLE AT 32.4m Piezometer installation consists of 25mm diameter Schedule 40 PVC pipe with a 3.05m slotted screen										

WATER LEVEL READINGS:

DATE	DEPTH (m)	ELEV. (m)
Mar. 31/10	2.59	222.9
Apr. 09/10	2.75	222.7
Apr. 20/10	2.72	222.8
May 03/10	2.41	223.1

ONTMT4S 0615.GPJ 2012TEMPLATE(MTO).GDT 12/8/14

$+^3, \times^3$: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

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Ontario

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RECORD OF BOREHOLE No BH10-02

METRIC

W.P.	P-13-03	LOCATION	N 4 881 654.2 E 294 774.3	ORIGINATED BY	SL
HWY	400	BOREHOLE TYPE	Hollow Stem Augers/TriCone Method	COMPILED BY	LRB
DATUM	Geodetic	DATE	2010.04.05 - 2010.04.08	CHECKED BY	AEG

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
224.6	GROUND SURFACE
0.0	**TOPSOIL**, some roots and rootlets
0.2	Clayey **SILT** Firm to Stiff Brown Moist (TILL)(CL)
	Trace gravel from 1.5m to 2.9m
	Soft and wet at 6.9m

SAMPLES

NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
1	SS	9				
2	SS	15				
3	SS	10	0	0	60	40
4	SS	11				
5	SS	10				
6	SS	6				
7	SS	8				
1	TW		1	3	53	43
8	SS	3				
9	SS	10				
2	TW					
10	SS	10	0	13	47	40

Column headings: STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (224, 223, 222, 221, 220, 219, 218, 217, 216, 215); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20, 40, 60, 80, 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (20, 40, 60); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued Next Page

+ ³, × ³ : Numbers refer to Sensitivity

20 / 15 / 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No BH10-02

METRIC

W.P.	P-13-03	LOCATION	N 4 881 654.2 E 294 774.3	ORIGINATED BY	SL
HWY	400	BOREHOLE TYPE	Hollow Stem Augers/TriCone Method	COMPILED BY	LRB
DATUM	Geodetic	DATE	2010.04.05 - 2010.04.08	CHECKED BY	AEG

SOIL PROFILE: ELEV DEPTH | DESCRIPTION | STRAT PLOT
SAMPLES: NUMBER | TYPE | "N" VALUES
GROUND WATER CONDITIONS | ELEVATION SCALE
DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE: 20 40 60 80 100
PLASTIC LIMIT w_P | NATURAL MOISTURE CONTENT w | LIQUID LIMIT w_L — WATER CONTENT (%): 20 40 60
UNIT WEIGHT γ kN/m³
REMARKS & GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

Continued From Previous Page

Clayey **SILT**, trace sand, trace gravel
Stiff to Very Stiff
Grey
Moist
(TILL)(CL-ML to CL)

Firm from 12.2m to 14.3m

Sample Number	Type	"N" Values	GR	SA	SI	CL
11	SS	12				
12	SS	4				
13	SS	5	2	10	65	23
14	SS	19				
15	SS	16				
16	SS	17	1	6	50	43

Elevation scale: 214, 213, 212, 211, 210, 209, 208, 207, 206, 205

Continued Next Page

+ ³, × ³ : Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No BH10-02

METRIC

W.P.	P-13-03	LOCATION	N 4 881 654.2 E 294 774.3	ORIGINATED BY	SL
HWY	400	BOREHOLE TYPE	Hollow Stem Augers/TriCone Method	COMPILED BY	LRB
DATUM	Geodetic	DATE	2010.04.05 - 2010.04.08	CHECKED BY	AEG

Continued From Previous Page

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	GR	SA	SI	CL
	Clayey **SILT**, trace sand, trace gravel Stiff to Very Stiff Grey Moist (TILL)	17	SS	23					
		18	SS	8		0	0	78	22
198.5 26.1	Clayey **SILT**, trace sand, trace gravel Hard Grey Moist (TILL)	19	SS	37					
197.0 27.6	SAND, trace to some silt Very Dense Grey Moist	20	SS	100 / 0.28					

Elevation scale: 204, 203, 202, 201, 200, 199, 198, 197, 196, 195

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No BH10-02

METRIC

W.P.	P-13-03	LOCATION	N 4 881 654.2 E 294 774.3	ORIGINATED BY	SL
HWY	400	BOREHOLE TYPE	Hollow Stem Augers/TriCone Method	COMPILED BY	LRB
DATUM	Geodetic	DATE	2010.04.05 - 2010.04.08	CHECKED BY	AEG

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
	Continued From Previous Page									
	SAND, trace to some silt Very Dense Grey Moist									
		21	SS	100 / 0.20	194	o	0	89	11 (SI+CL)	
					193					
192.2 32.4		22	SS	100 / 0.28		o				
	END OF BOREHOLE AT 32.4m Piezometer installation consists of 25mm diameter Schedule 40 PVC pipe with a 2.1m slotted screen									

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m^3; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

WATER LEVEL READINGS:

DATE	DEPTH (m)	ELEV. (m)
Apr. 09/10	9.06	215.5
Apr. 20/10	1.46	223.1
May 03/10	1.00	223.6

+ 3 , × 3 : Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

Appendix B

Laboratory Test Results

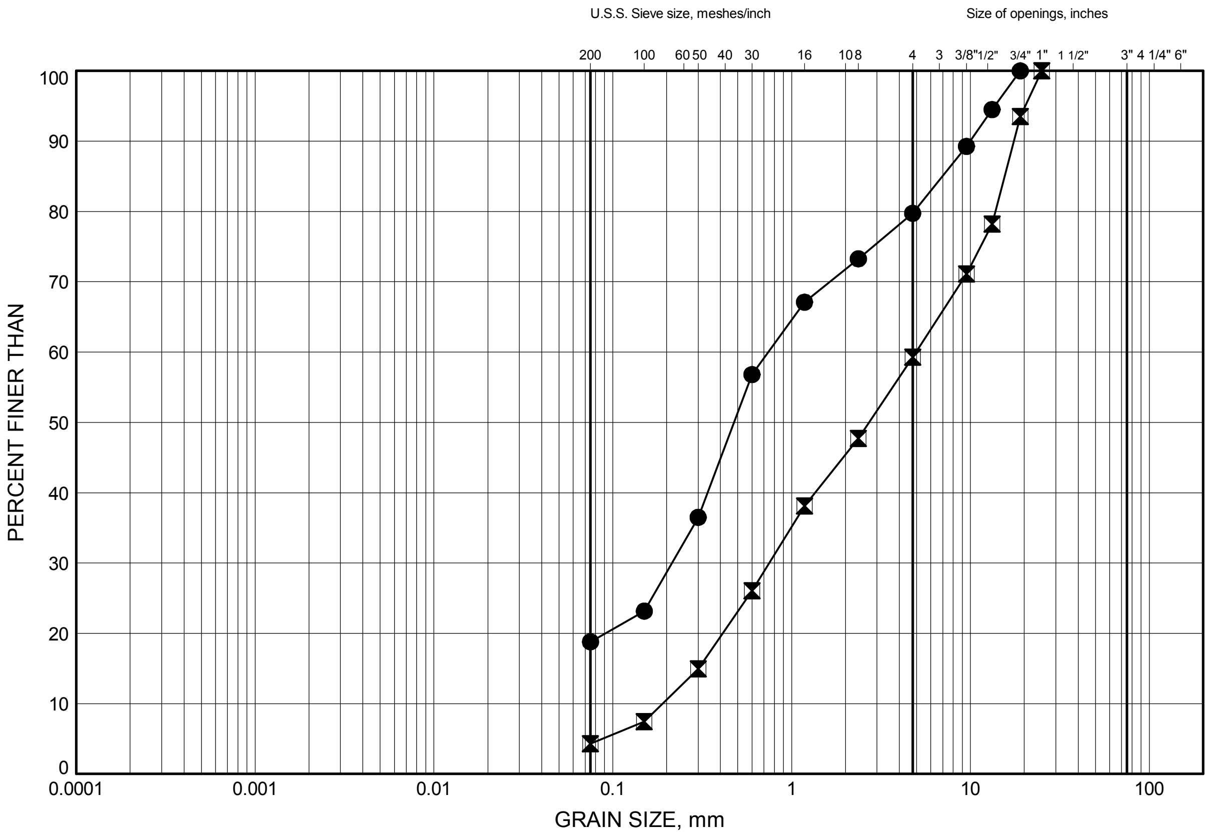
(current and previous investigations)

Foundation Engineering, Hwy. 400 and 5th Line

GRAIN SIZE DISTRIBUTION

FIGURE B1

SAND & GRAVEL/GRAVELLY SAND FILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	13-20	0.38	224.52
⧗	13-29	0.61	224.29

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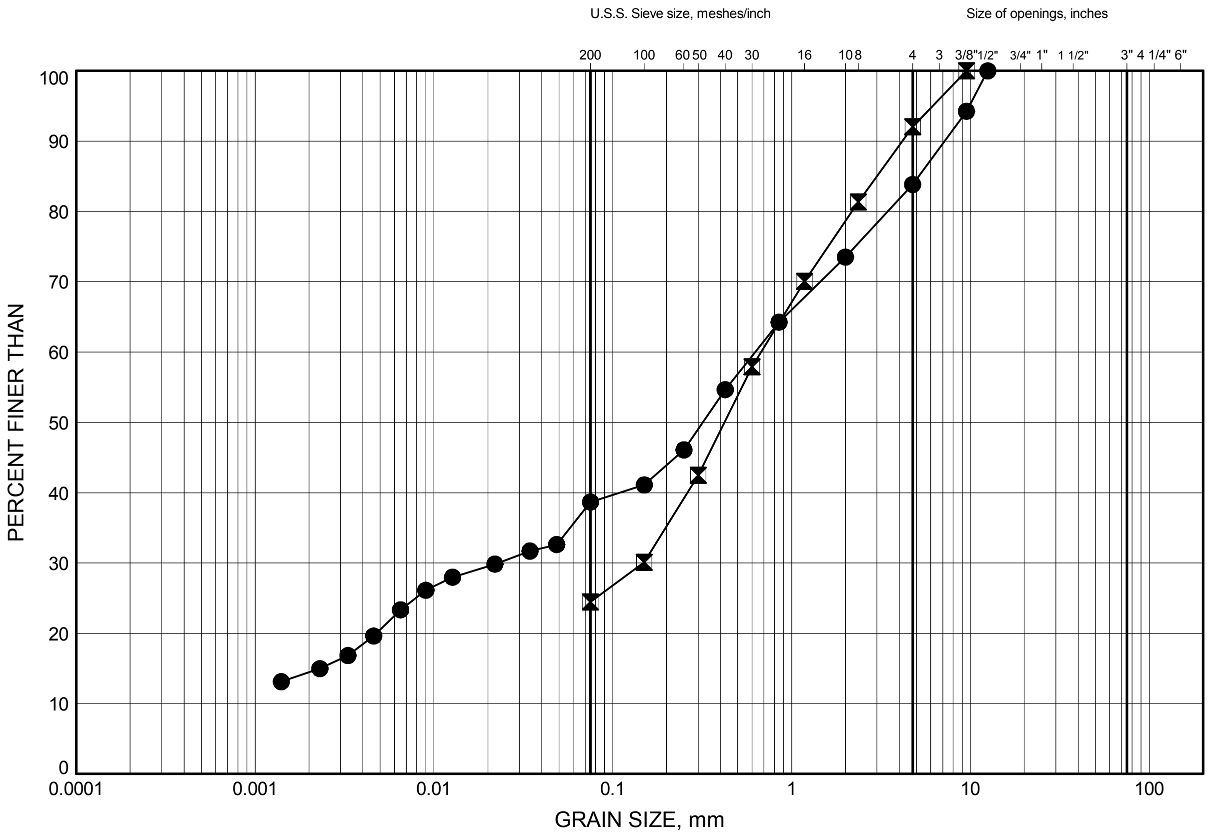
GRAIN SIZE DISTRIBUTION

FIGURE B2

SILTY SAND FILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	13-22	0.30	227.39
⧗	13-28	1.07	222.22

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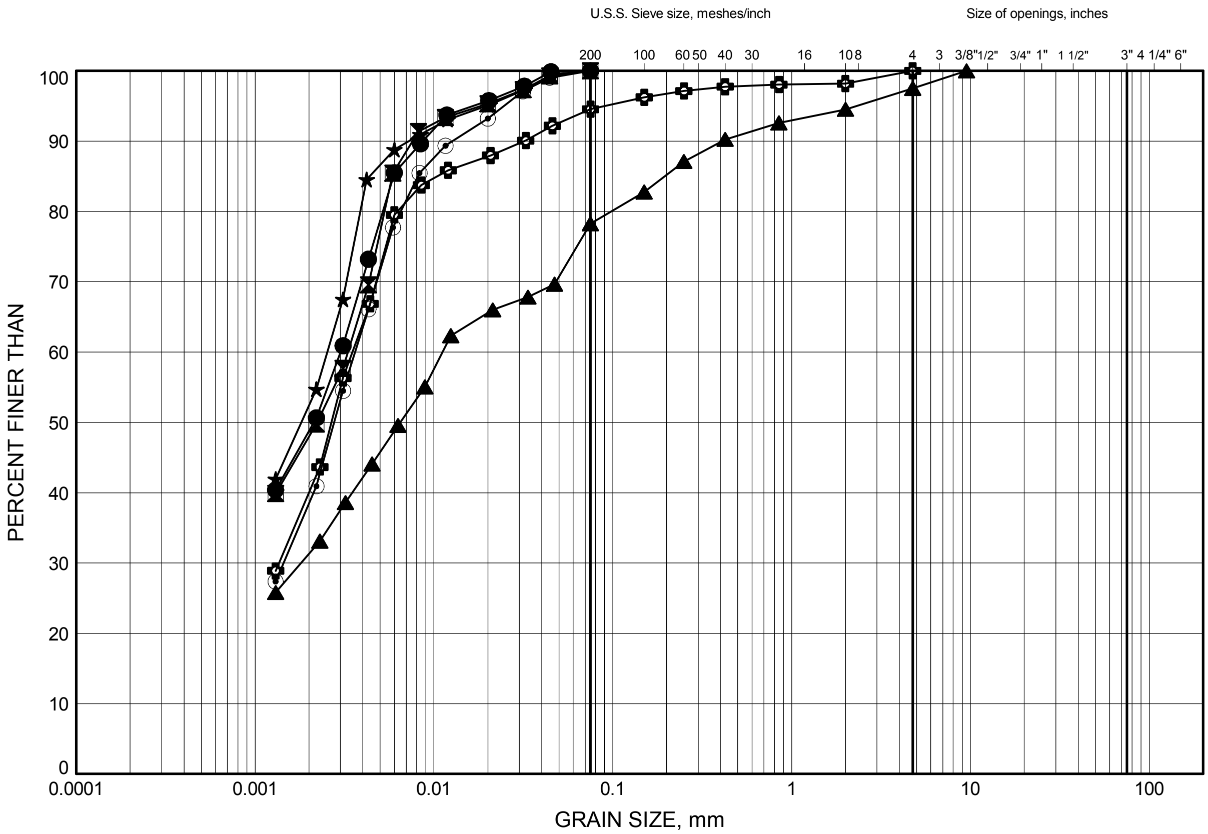
GRAIN SIZE DISTRIBUTION

FIGURE B3

SILTY CLAY



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	13-19	1.07	222.60
⧗	13-19	3.35	220.31
▲	13-19	7.92	215.74
★	13-19	12.50	211.17
⊙	13-19	14.02	209.64
✿	13-19	18.59	205.07

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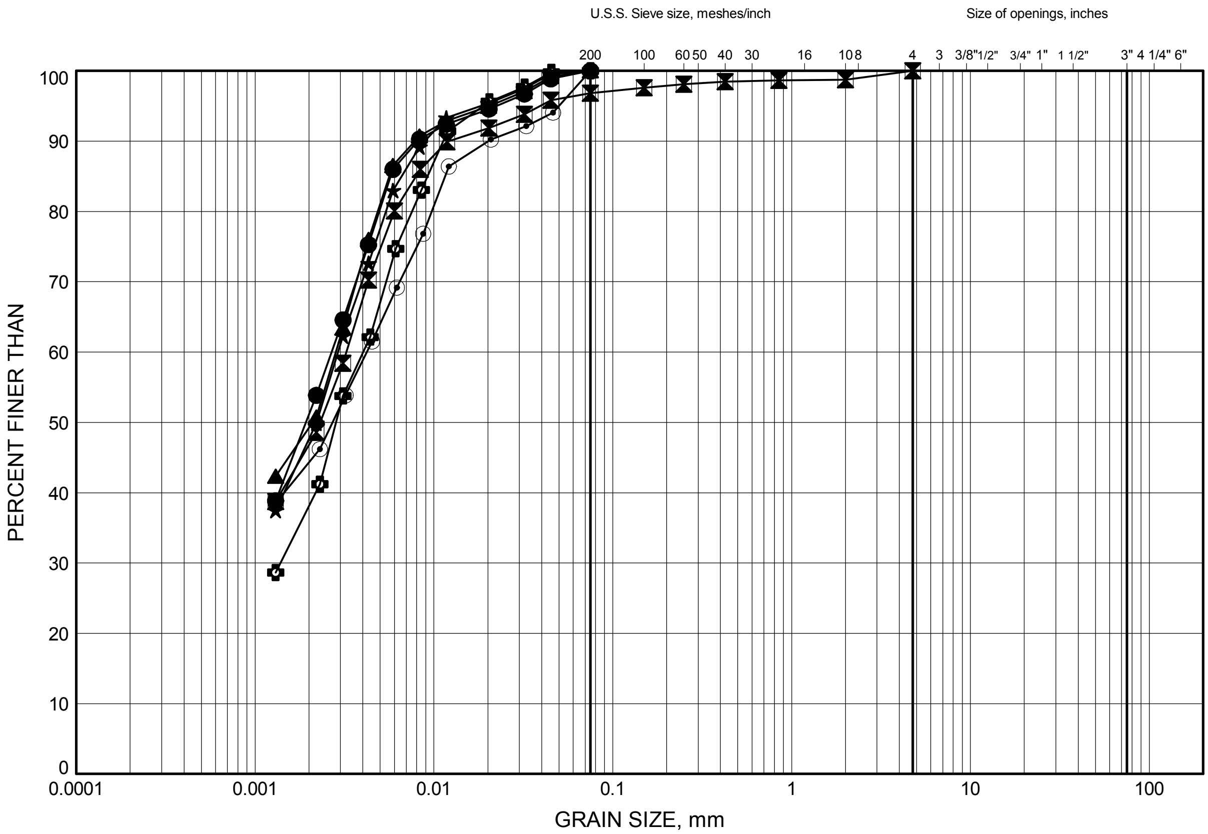
GRAIN SIZE DISTRIBUTION

FIGURE B4

SILTY CLAY



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	13-20	1.83	223.07
⧗	13-20	6.40	218.50
▲	13-20	12.50	212.40
★	13-20	18.59	206.31
⊙	13-21	1.07	225.51
✿	13-21	4.11	222.46

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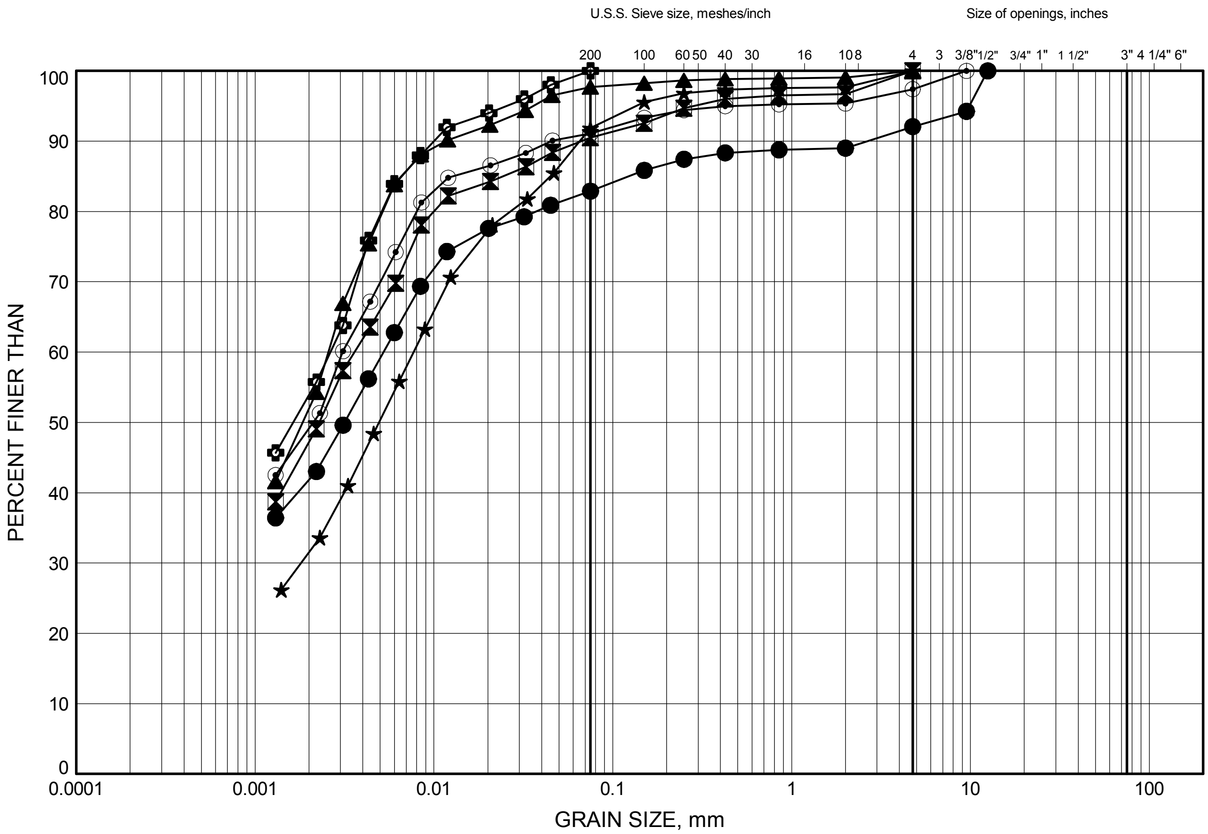
GRAIN SIZE DISTRIBUTION

FIGURE B5

SILTY CLAY



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	13-21	9.45	217.13
⧗	13-22	1.83	225.87
▲	13-22	7.92	219.77
★	13-27	1.07	220.92
⊙	13-27	4.11	217.88
✿	13-27	7.92	214.07

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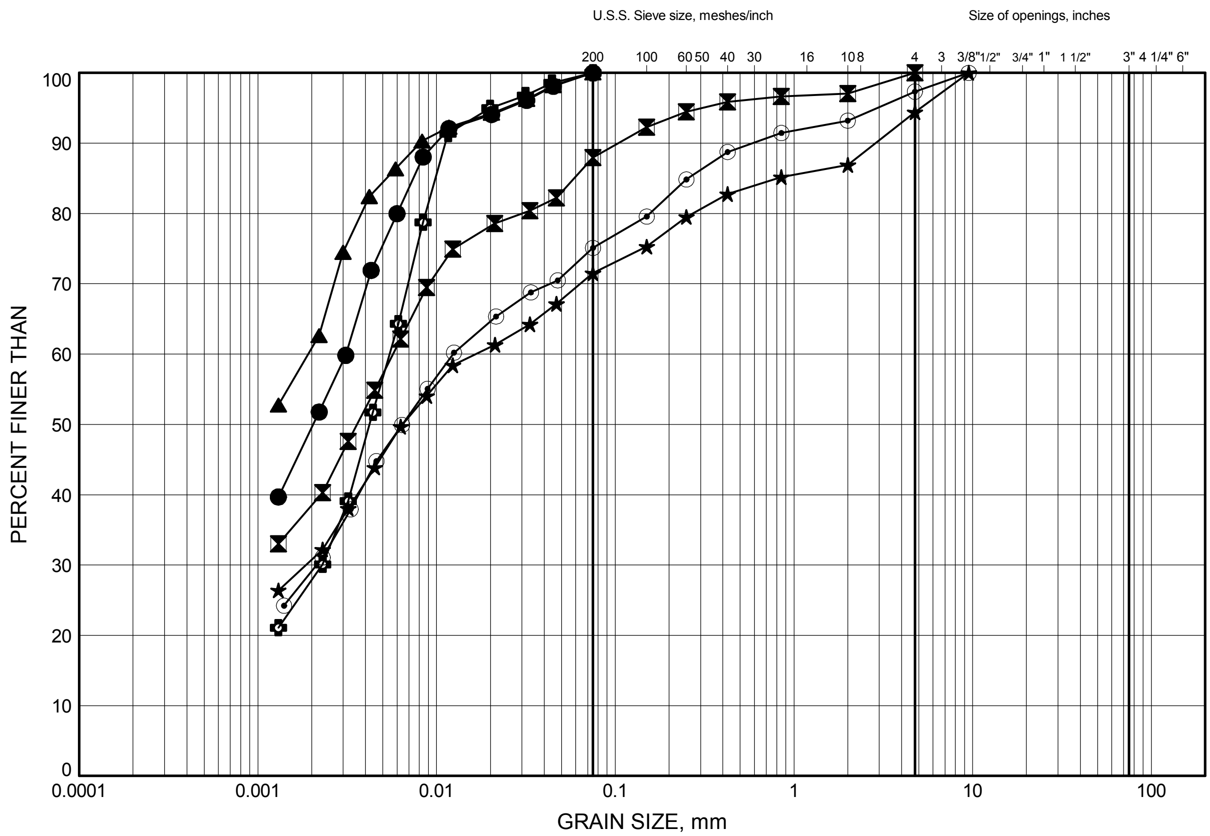
GRAIN SIZE DISTRIBUTION

FIGURE B6

SILTY CLAY



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	13-28	3.35	219.94
⧗	13-28	9.45	213.84
▲	13-29	3.35	221.55
★	13-29	9.45	215.45
⊙	13-29	12.50	212.40
✪	13-29	26.21	198.69

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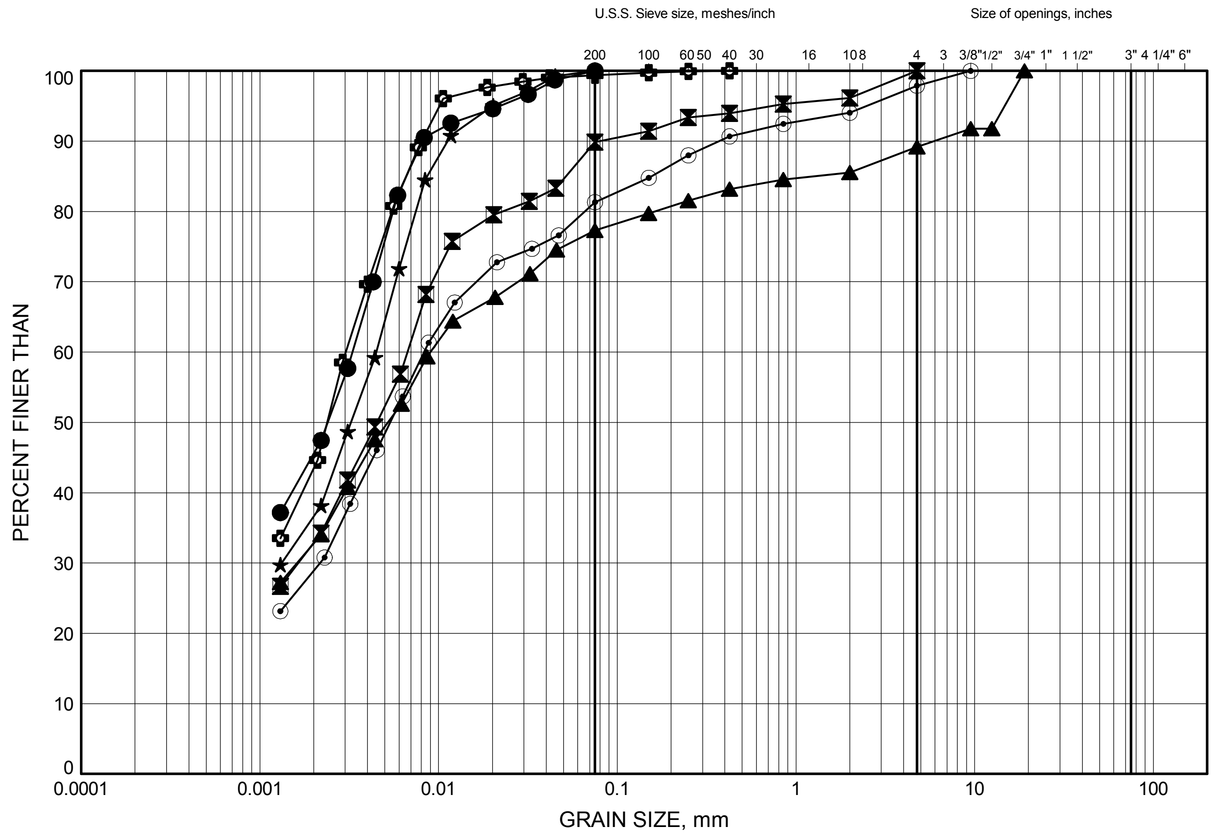
GRAIN SIZE DISTRIBUTION

FIGURE B7

SILTY CLAY



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	13-30	1.07	223.37
⧗	13-30	6.40	218.04
▲	13-30	10.97	213.46
★	13-30	15.54	208.89
⊙	13-30	20.12	204.32
✿	BH10-01	1.07	224.43

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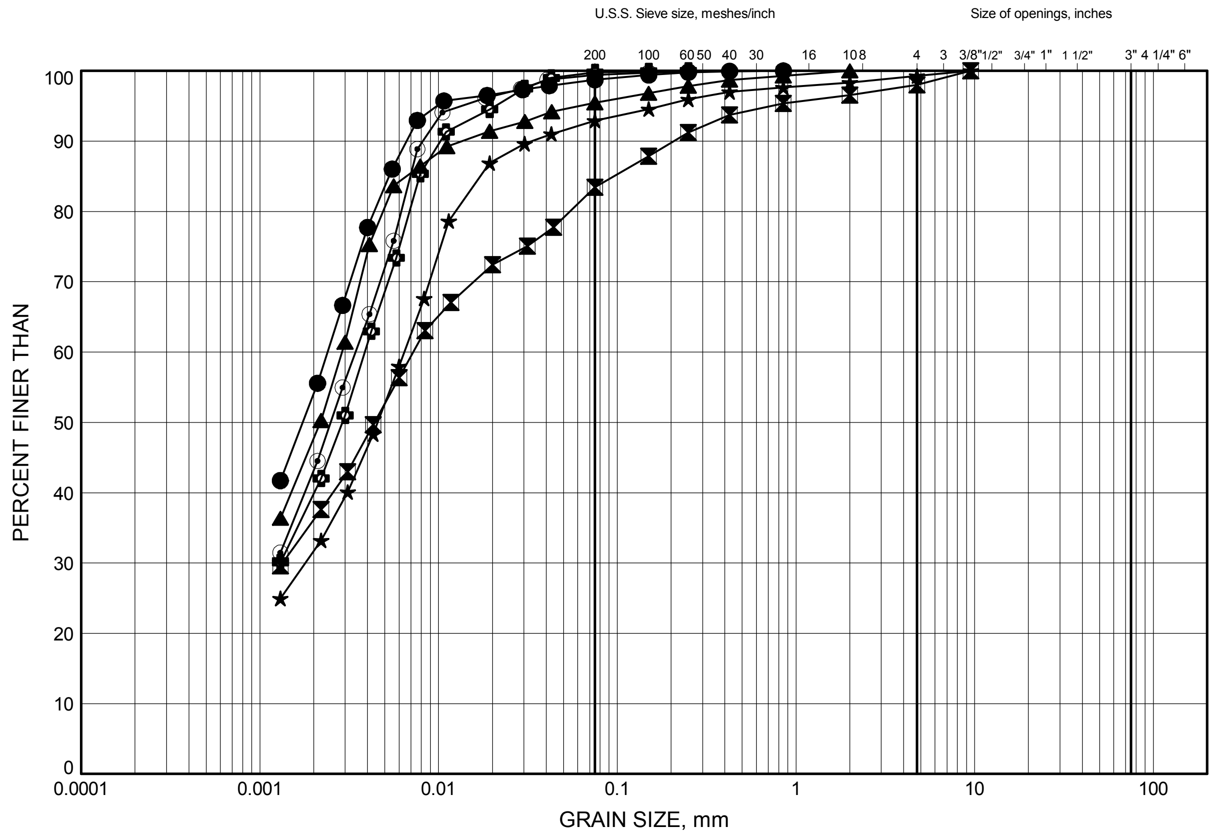
GRAIN SIZE DISTRIBUTION

FIGURE B8

SILTY CLAY



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	BH10-01	3.35	222.15
⧗	BH10-01	7.16	218.34
▲	BH10-01	12.50	213.00
★	BH10-01	17.07	208.43
⊙	BH10-01	26.21	199.29
✿	BH10-02	1.83	222.77

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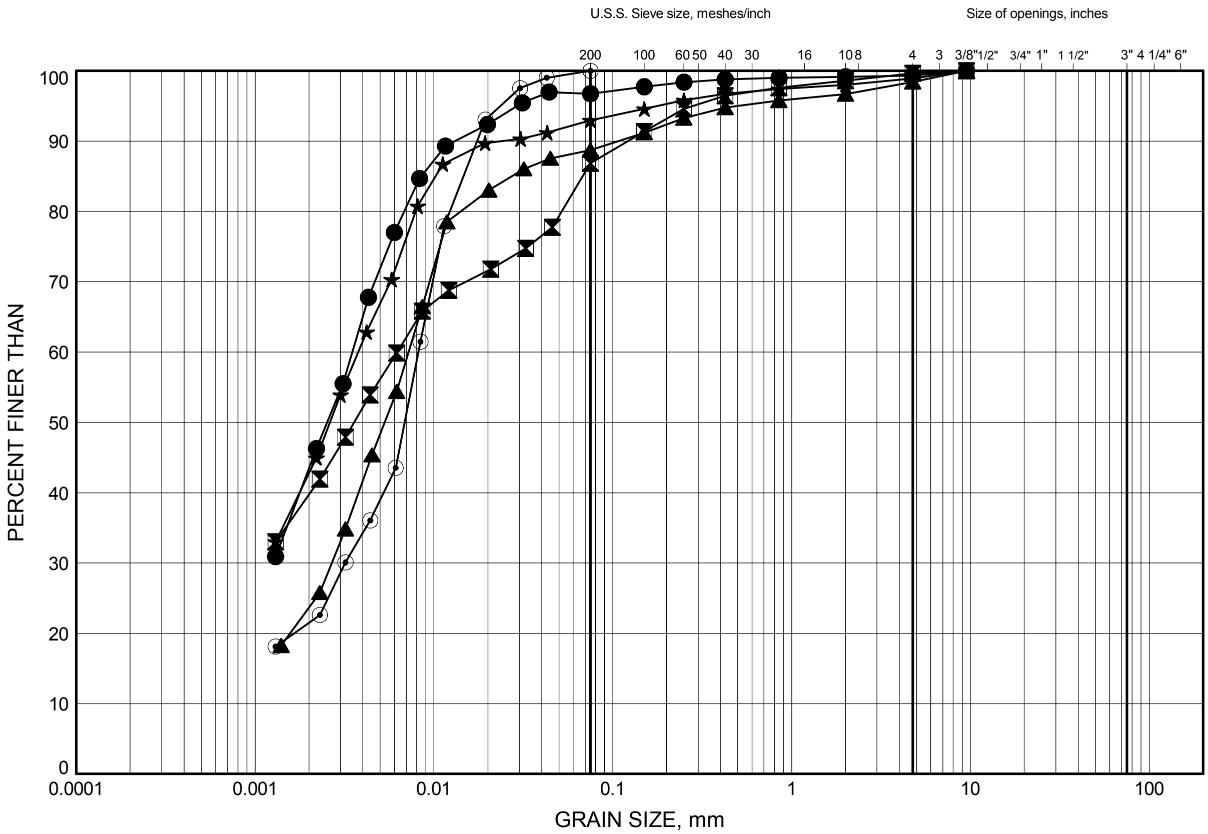
GRAIN SIZE DISTRIBUTION

FIGURE B9

SILTY CLAY



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	BH10-02	6.40	218.20
⧗	BH10-02	9.45	215.15
▲	BH10-02	14.02	210.58
★	BH10-02	18.59	206.01
⊙	BH10-02	23.16	201.44

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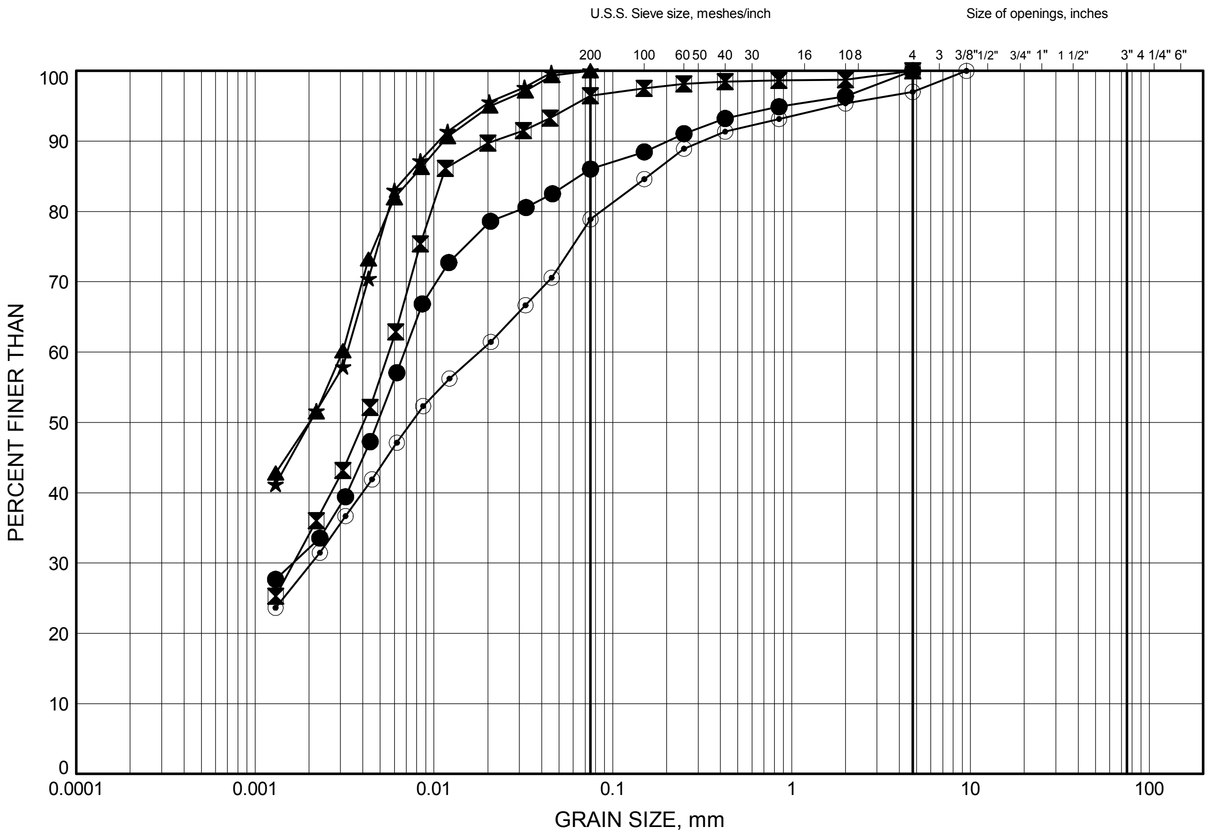
GRAIN SIZE DISTRIBUTION

FIGURE B10

SILTY CLAY TILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	13-19	23.16	200.50
⧗	13-20	26.21	198.69
▲	13-30	29.26	195.18
★	13-30	32.31	192.13
⊙	BH10-01	29.18	196.32

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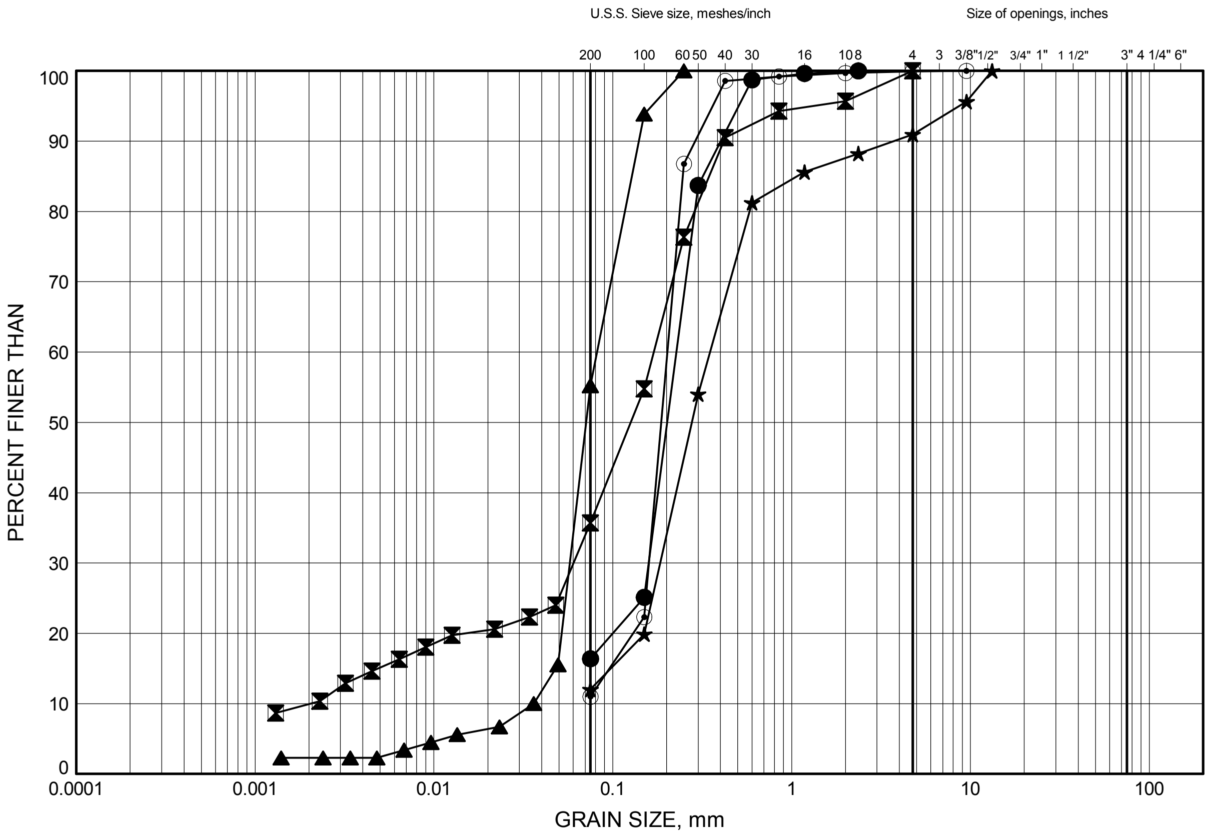
GRAIN SIZE DISTRIBUTION

FIGURE B11

SAND/SILTY SAND/SAND & SILT



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	13-19	32.31	191.35
⧗	13-20	35.36	189.54
▲	13-29	29.26	195.64
★	13-30	35.36	189.08
⊙	BH10-02	30.66	193.94

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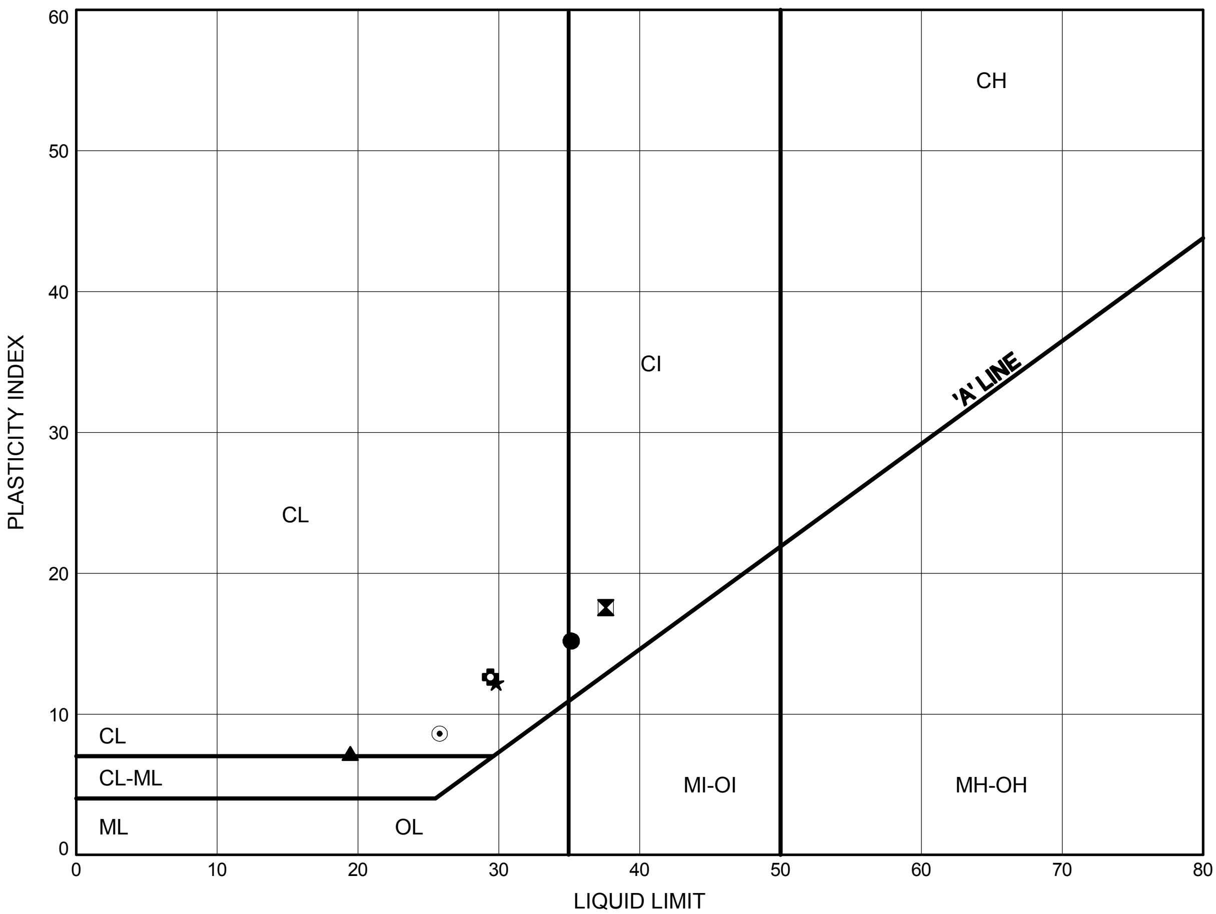
ATTERBERG LIMITS TEST RESULTS

FIGURE B12

SILTY CLAY



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	13-19	1.07	222.60
⧗	13-19	3.35	220.31
▲	13-19	7.92	215.74
★	13-19	12.50	211.17
⊙	13-19	14.02	209.64
✪	13-19	18.59	205.07

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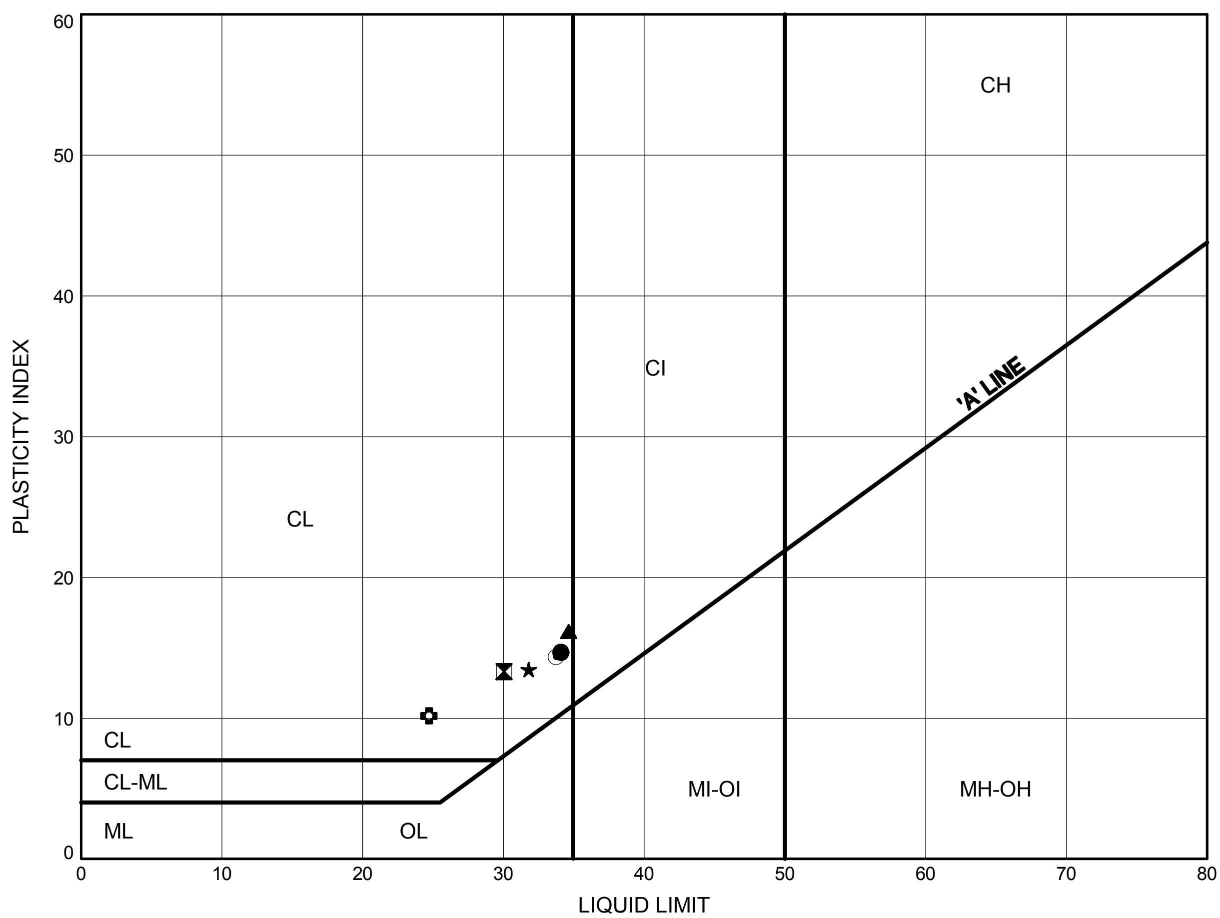
ATTERBERG LIMITS TEST RESULTS

FIGURE B13

SILTY CLAY



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	13-20	1.83	223.07
⧗	13-20	6.40	218.50
▲	13-20	12.50	212.40
★	13-20	18.59	206.31
⊙	13-21	4.11	222.46
✿	13-21	9.45	217.13

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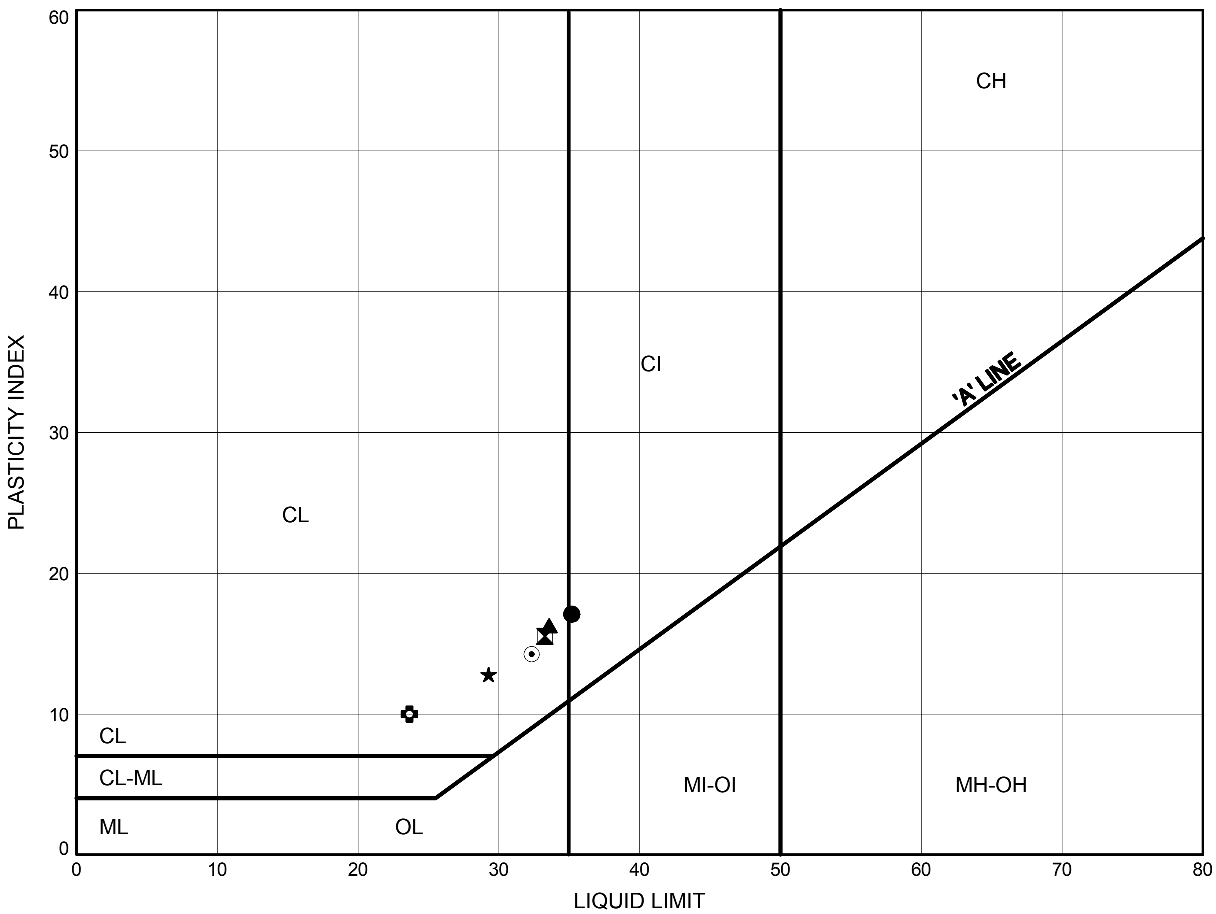
ATTERBERG LIMITS TEST RESULTS

FIGURE B14

SILTY CLAY



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	13-22	1.83	225.87
⧗	13-22	7.92	219.77
▲	13-27	4.11	217.88
★	13-27	7.92	214.07
⊙	13-28	3.35	219.94
✿	13-28	9.45	213.84

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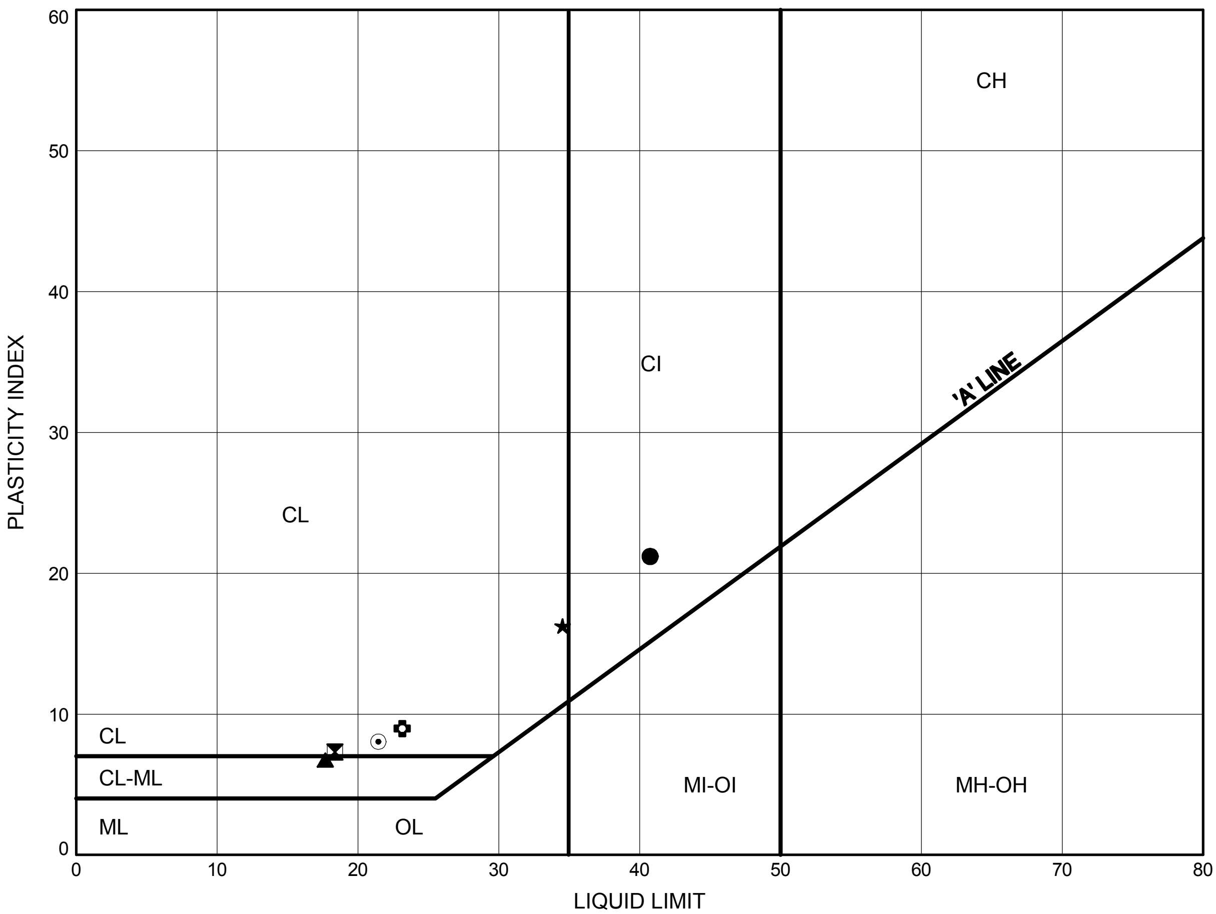
ATTERBERG LIMITS TEST RESULTS

FIGURE B15

SILTY CLAY



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	13-29	3.35	221.55
⧗	13-29	9.45	215.45
▲	13-29	12.50	212.40
★	13-30	1.07	223.37
⊙	13-30	6.40	218.04
✿	13-30	10.97	213.46

Date December 2014

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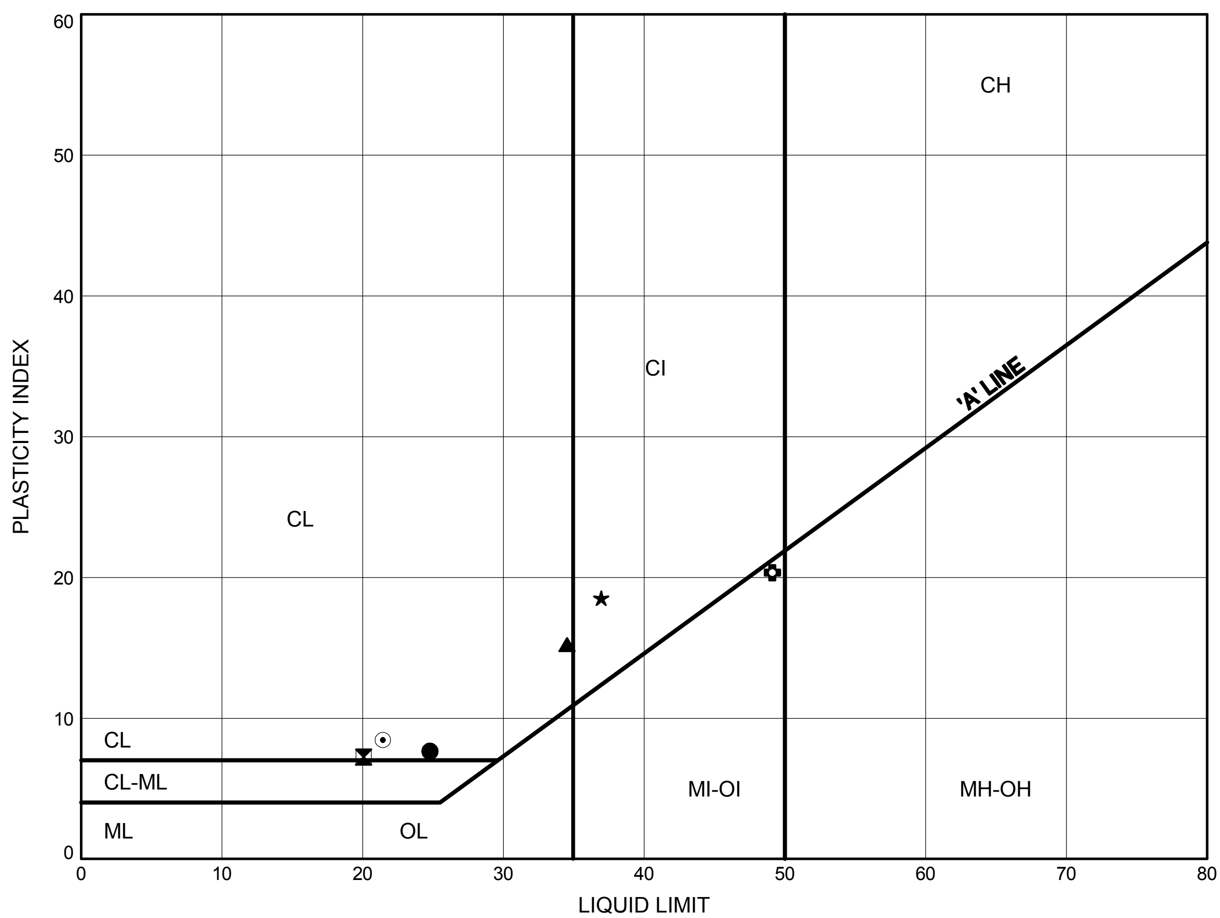
ATTERBERG LIMITS TEST RESULTS

FIGURE B16

SILTY CLAY



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	13-30	15.54	208.89
⧗	13-30	20.12	204.32
▲	BH10-01	1.07	224.43
★	BH10-01	3.35	222.15
⊙	BH10-01	7.16	218.34
✿	BH10-01	12.50	213.00

THURBALT 0615.GPJ 12/8/14

Date December 2014

W.P. P-13-03

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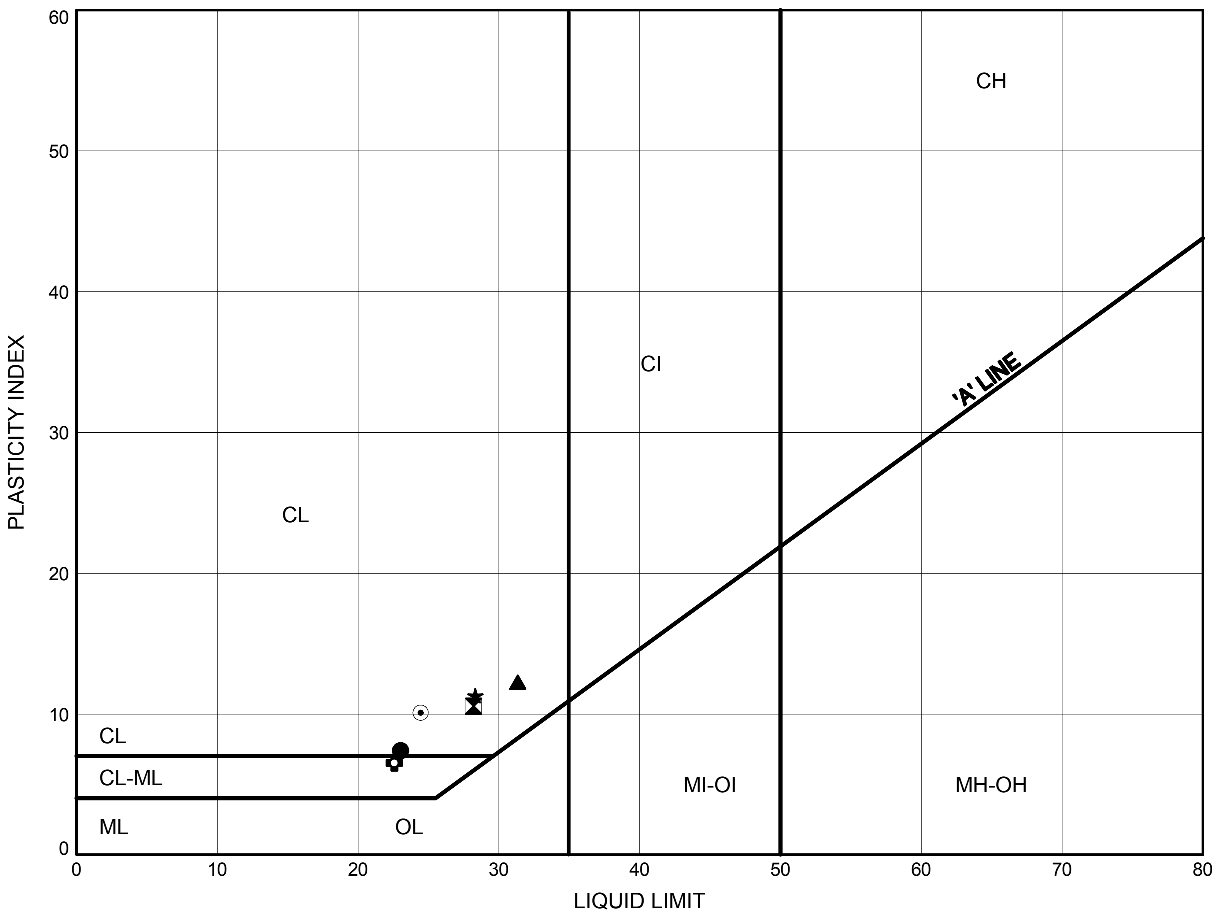
ATTERBERG LIMITS TEST RESULTS

FIGURE B17

SILTY CLAY



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	BH10-01	17.07	208.43
⧗	BH10-01	26.21	199.29
▲	BH10-02	1.83	222.77
★	BH10-02	6.40	218.20
⊙	BH10-02	9.45	215.15
✿	BH10-02	14.02	210.58

THURBALT 0615.GPJ 12/8/14

Date December 2014

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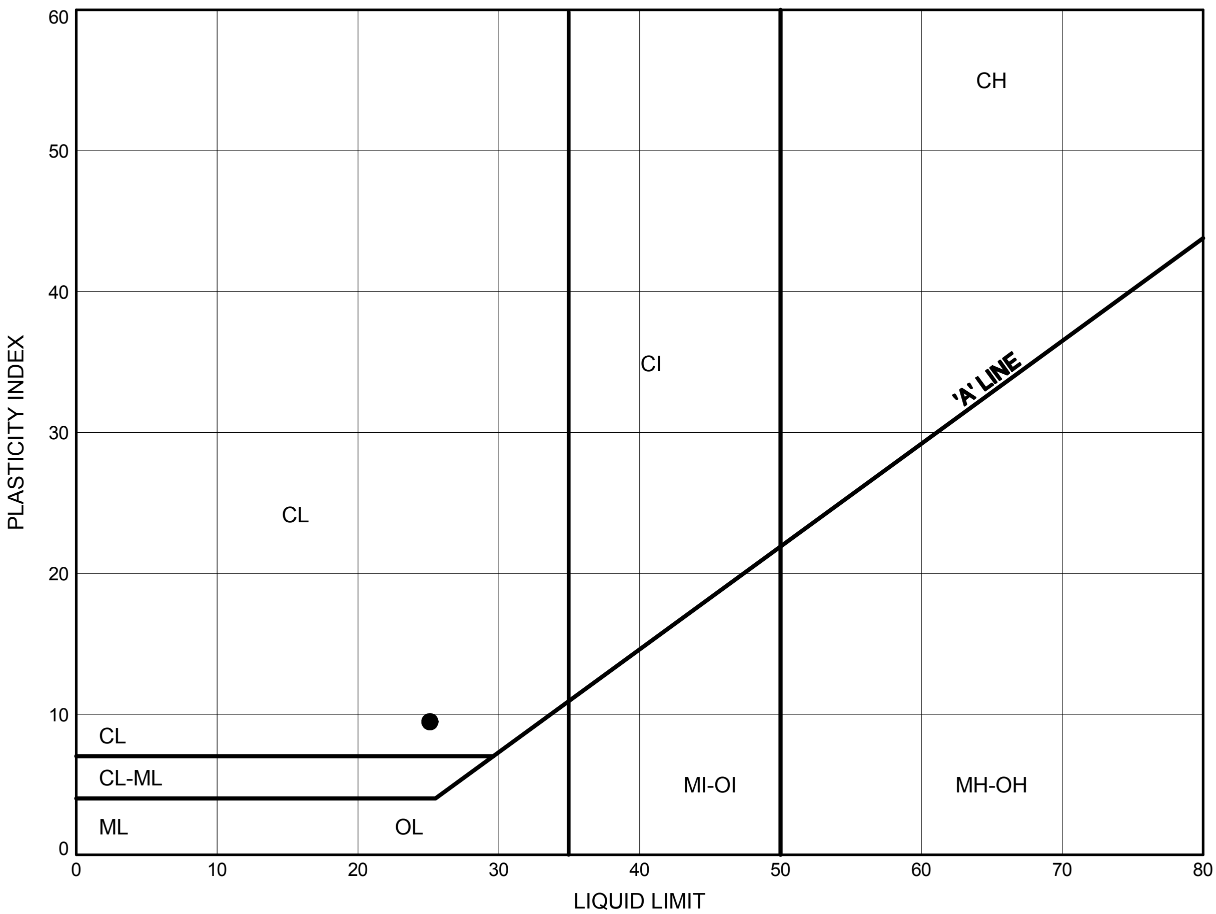
ATTERBERG LIMITS TEST RESULTS

FIGURE B18

SILTY CLAY



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	BH10-02	18.59	206.01

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Date December 2014

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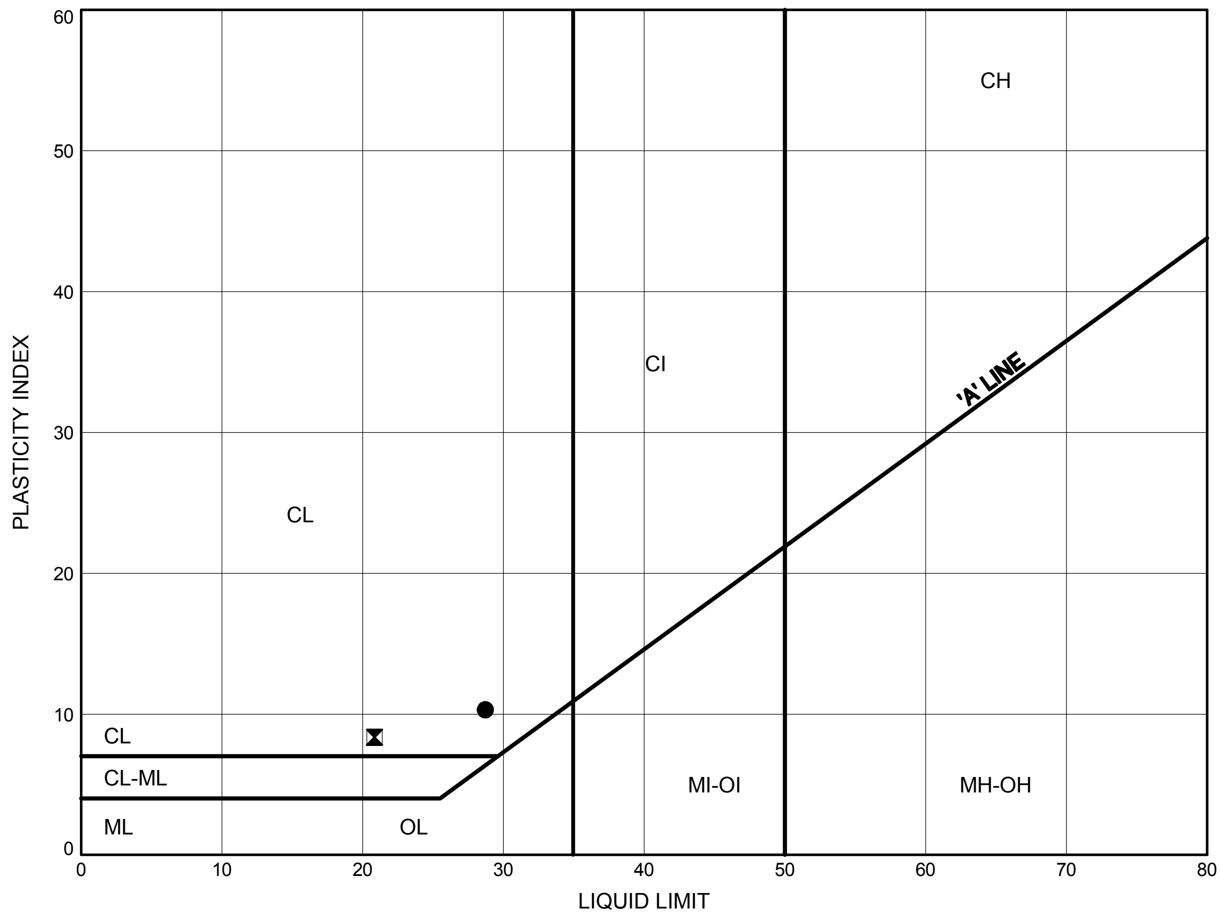
ATTERBERG LIMITS TEST RESULTS

FIGURE B19

SILTY CLAY TILL



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	13-30	32.31	192.13
⧗	BH10-01	29.18	196.32

Date December 2014

W.P. P-13-03

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Consolidation Test Report

CLIENT:	McCormick Rankin Corporation	FILE NUMBER:	19-1351-166
PROJECT:	Highway 400 & 5th Line EAS	REPORT DATE:	13-Apr-10

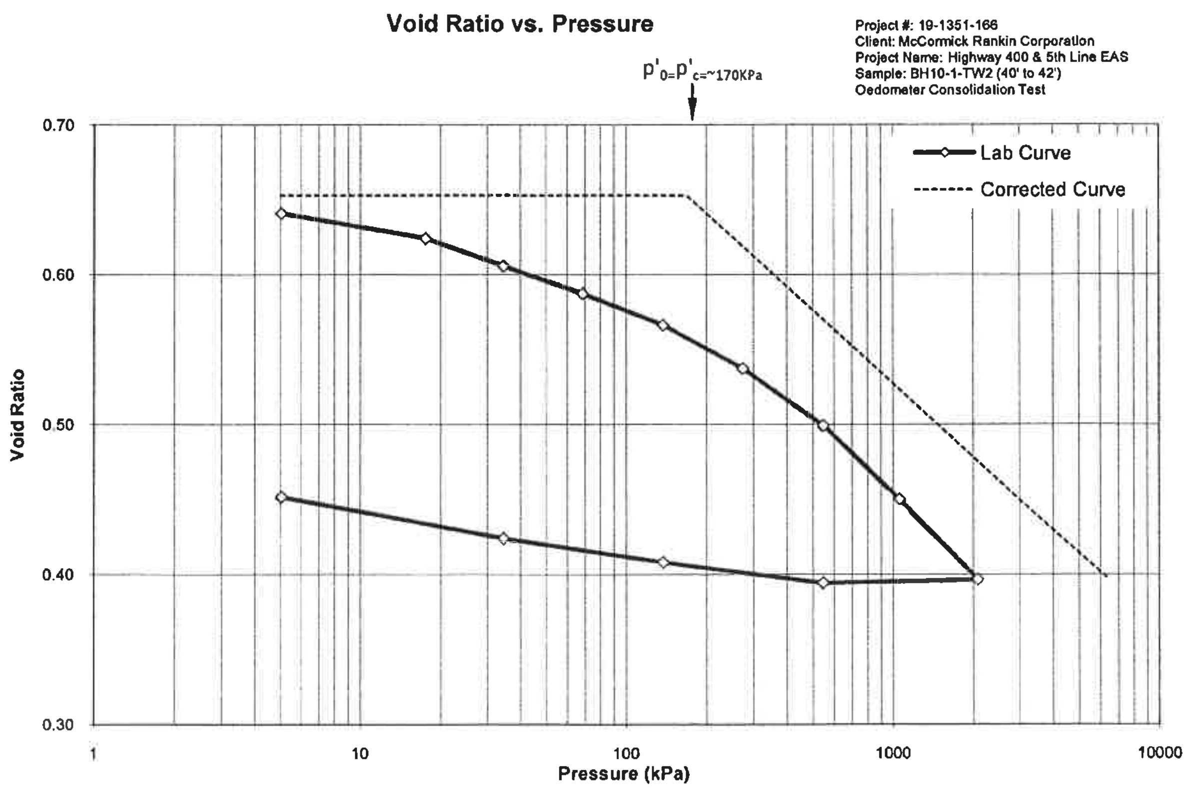
TEST DATES: March 30, 2010 - April 11, 2010

SAMPLE: BH10-1-TW2 (40' to 42')
Silty Clay, trace Gravel, grey, Grain Size: 47% Clay,48 % Silt, 5% Sand
Atterberg Limits: LL=49.2%, PI=20.4%

PROCEDURE: Test carried out in accordance with Standard Test Method for One-Dimensional Consolidation Properties of Soils, ASTM D 2435-04, method B

	Start of Test	End of Test
Wet Dens. (kg/m^3)	2035.1	2238.1
Dry Dens. (kg/m^3)	1639.6	1867.3
Moisture Cont. (%)	24.1	19.9
Void Ratio	0.653	0.451
Saturation (%)	100.0	

Note: A Specific Gravity of 2.71 was measured for the void ratio and saturation calculations.



TEST DONE BY: EA
REVIEWED BY: JPL

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Consolidation Test Report

Highway 400 & 5th Line EAS
19-1351-166

BH10-1-TW2 (40' to 42')

TRIMMING: The Specimen was manually trimmed to the size of consolidation ring, then mounted in a fixed ring consolidometer.

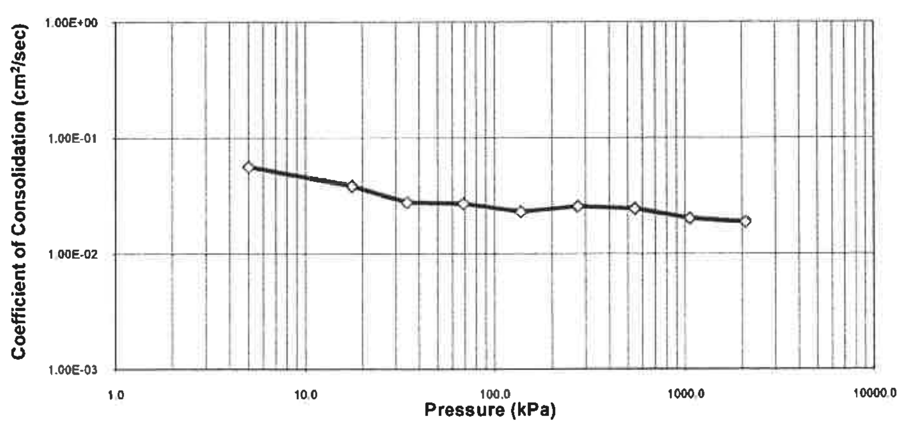
LOADING: A sealing load of 5 kPa was applied and the consolidometer was flooded with distilled water. Sample was monitored to ensure no swelling effect occurred before the start of the test. Subsequent loads were applied after 100% primary consolidation was reached.

CALCULATIONS: Coefficients of Consolidation were calculated by the square root time method.

Pressure (kPa)	Corr. H. (mm)	Avg. H. (mm)	d_{90} (mm)	t_{90} (min)	c_v (cm^2/s)	Void Ratio	m_v (m^2/kN)	k (cm/s)
0.0	20.000					0.653		
5.0	19.851	19.926	0.143	0.25	5.61E-02	0.641	1.48E-03	8.16E-06
17.6	19.648	19.750	0.142	0.36	3.83E-02	0.624	8.16E-04	3.06E-06
34.5	19.426	19.537	0.076	0.49	2.75E-02	0.606	6.66E-04	1.80E-06
68.5	19.202	19.314	0.097	0.49	2.69E-02	0.587	3.40E-04	8.96E-07
136.9	18.948	19.075	0.120	0.56	2.29E-02	0.566	1.93E-04	4.34E-07
273.2	18.597	18.773	0.125	0.49	2.54E-02	0.537	1.36E-04	3.39E-07
545.5	18.134	18.366	0.138	0.49	2.43E-02	0.499	9.14E-05	2.18E-07
1057.7	17.539	17.837	0.148	0.56	2.00E-02	0.450	6.40E-05	1.26E-07
2080.1	16.892	17.216	0.178	0.56	1.86E-02	0.396	3.61E-05	6.59E-08
545.5	16.865	16.879				0.394		
136.9	17.032	16.949				0.408		
34.5	17.228	17.130				0.424		
5.0	17.561	17.395				0.451		

Coefficient of Consolidation vs. Pressure

Project #: 19-1351-166
Client: McCormick Rankin Corporation
Project Name: Highway 400 & 5th Line EAS
Sample: BH10-1-TW2 (40' to 42')
Oedometer Consolidation Test



Notes: C_v and k calculated using t_{90} values

TEST DONE BY: EA
REVIEWED BY: JPL

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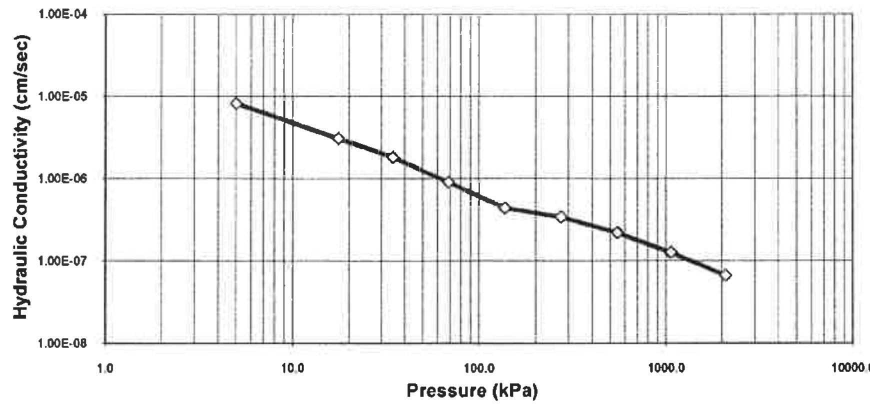
Consolidation Test Report

Highway 400 & 5th Line EAS
19-1351-166

BH10-1-TW2 (40' to 42')

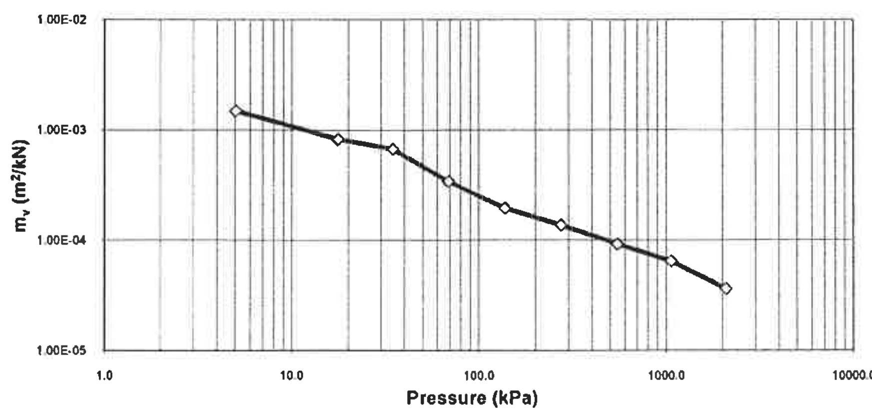
Hydraulic Conductivity vs. Pressure

Project #: 19-1351-166
Client: McCormick Rankin Corporation
Project Name: Highway 400 & 5th Line EAS
Sample: BH10-1-TW2 (40' to 42')
Oedometer Consolidation Test



m_v vs. Pressure

Project #: 19-1351-166
Client: McCormick Rankin Corporation
Project Name: Highway 400 & 5th Line EAS
Sample: BH10-1-TW2 (40' to 42')
Oedometer Consolidation Test



TEST DONE BY: EA
REVIEWED BY: JPL

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Consolidation Test Report

CLIENT:	McCormick Rankin Corporation	FILE NUMBER:	19-1351-166
PROJECT:	Highway 400 & 5th Line EAS	REPORT DATE:	30-Apr-10

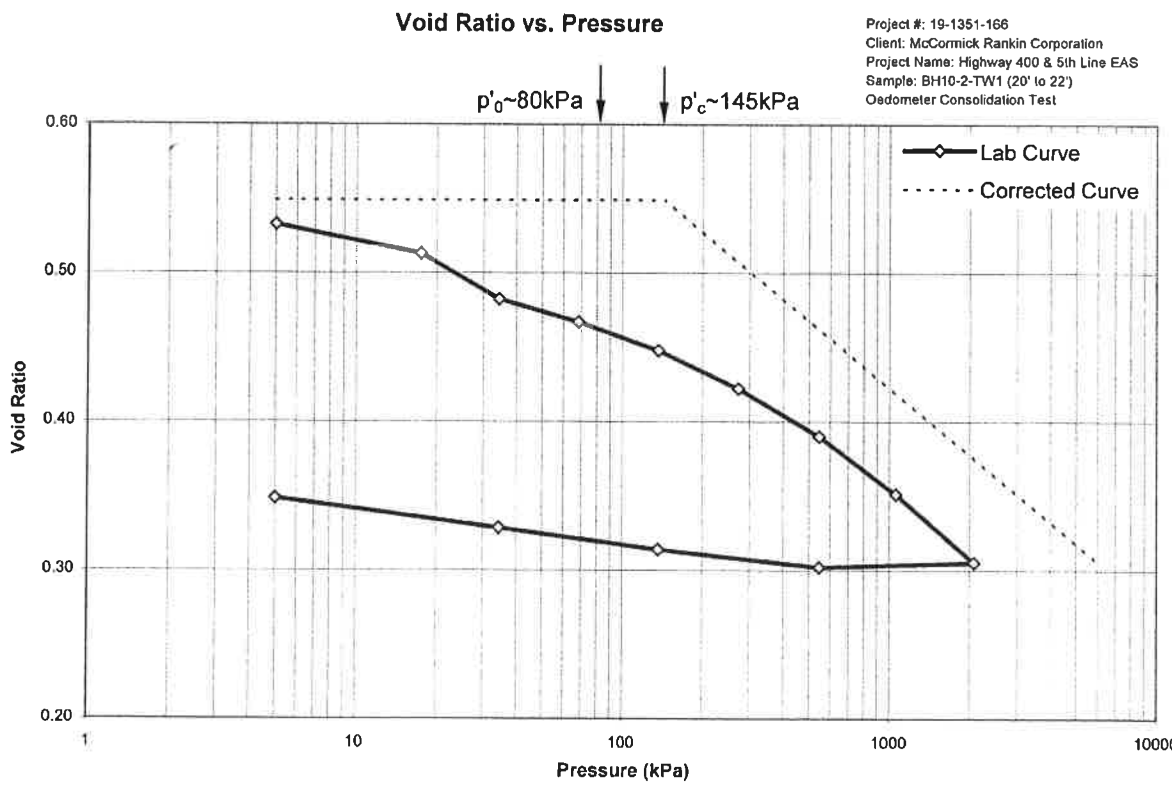
TEST DATES: April 13, 2010 - April 25, 2010

SAMPLE: BH10-2-TW1 (20' to 22')
Clay, Silty, trace Sand and Gravel, grey, (CL), Grain Size: 44 % Clay, 53% Silt, 3% Sand, 1% Gravel

PROCEDURE: Test carried out in accordance with Standard Test Method for One-Dimensional Consolidation Properties of Soils, ASTM D 2435-04, method B

	Start of Test	End of Test
Wet Dens. (kg/m^3)	2113.1	2393.3
Dry Dens. (kg/m^3)	1758.8	2020.2
Moisture Cont. (%)	20.1	18.5
Void Ratio	0.549	0.349
Saturation (%)	99.8%	

Note: A Specific Gravity of 2.73 was measured for the void ratio and saturation calculations.



TEST DONE BY: EA
REVIEWED BY: JPL

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Consolidation Test Report

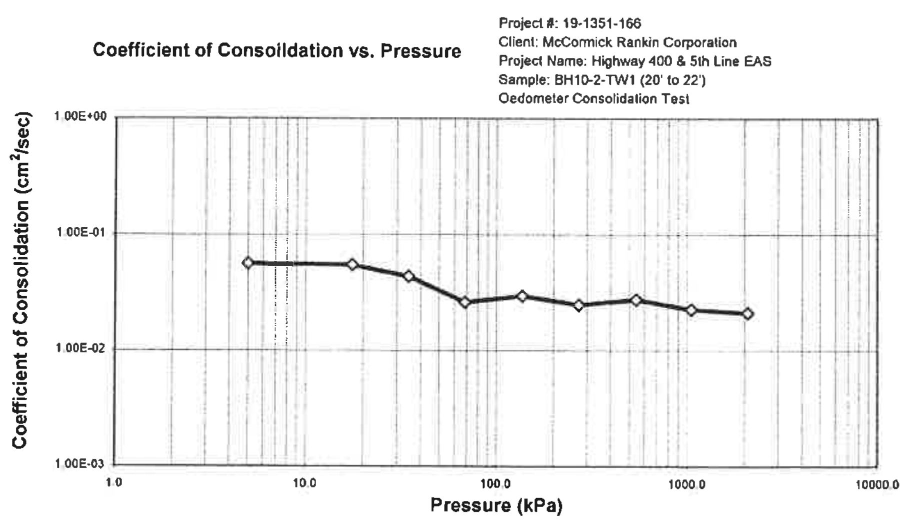
Highway 400 & 5th Line EAS
19-1351-166
BH10-2-TW1 (20' to 22')

TRIMMING: The Specimen was manually trimmed to the size of consolidation ring, then mounted in a fixed ring consolidometer.

LOADING: A seating load of 5 kPa was applied and the consolidometer was flooded with distilled water. Sample was monitored to ensure no swelling effect occurred before the start of the test. Subsequent loads were applied after 100% primary consolidation was reached.

CALCULATIONS: Coefficients of Consolidation were calculated by the square root time method.

Pressure (kPa)	Corr. H. (mm)	Avg. H. (mm)	d_{90} (mm)	t_{90} (min)	c_v (cm^2/s)	Void Ratio	m_v (m^2/kN)	k (cm/s)
0.0	20.000					0.549		
5.0	19.790	19.895	0.255	0.25	5.59E-02	0.533	2.09E-03	1.15E-05
17.6	19.530	19.660	0.272	0.25	5.46E-02	0.513	1.05E-03	5.62E-06
34.5	19.140	19.335	0.274	0.30	4.37E-02	0.482	1.18E-03	5.04E-06
68.5	18.940	19.040	0.084	0.49	2.61E-02	0.467	3.08E-04	7.89E-07
136.9	18.695	18.818	0.123	0.42	2.96E-02	0.448	1.89E-04	5.50E-07
273.2	18.364	18.530	0.154	0.49	2.48E-02	0.422	1.30E-04	3.15E-07
545.5	17.949	18.157	0.153	0.42	2.76E-02	0.390	8.30E-05	2.24E-07
1057.7	17.446	17.698	0.177	0.49	2.26E-02	0.351	5.47E-05	1.21E-07
2080.1	16.852	17.149	0.161	0.49	2.12E-02	0.305	3.33E-05	6.93E-08
545.5	16.813	16.833				0.302		
136.9	16.965	16.889				0.314		
34.5	17.153	17.059				0.328		
5.0	17.412	17.283				0.349		



Notes: C_v and k calculated using t_{90} values

TEST DONE BY: EA
REVIEWED BY: JPL

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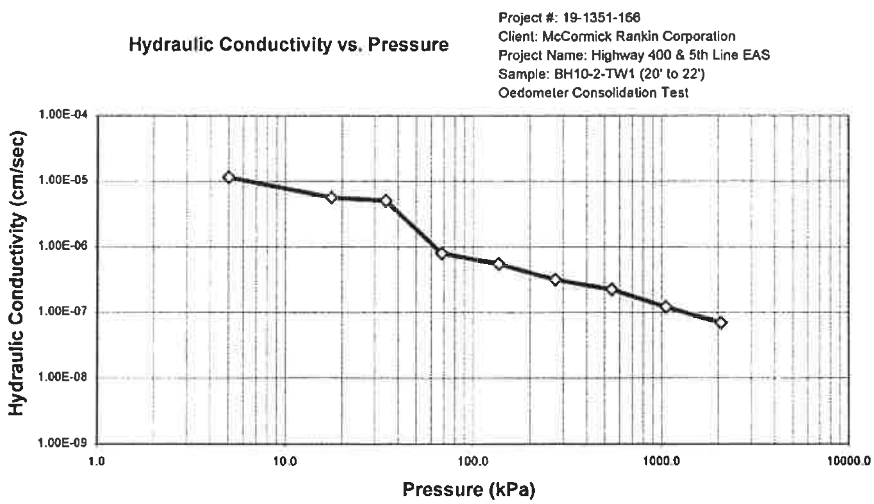
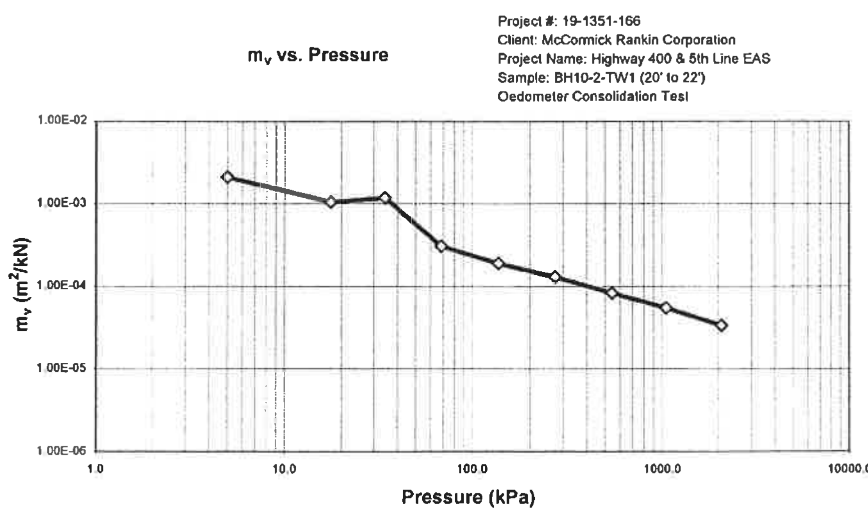
Consolidation Test Report

Highway 400 & 5th Line EAS
19-1351-166 BH10-2-TW1 (20' to 22')





TEST DONE BY: EA
REVIEWED BY: JPL

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FIGURE 1-1

Consolidation Test Report

CLIENT: URS Canada Inc.

FILE NUMBER: 19-4406-15

PROJECT: Hwy 400 / 5th Line

REPORT DATE: 23-Oct-14

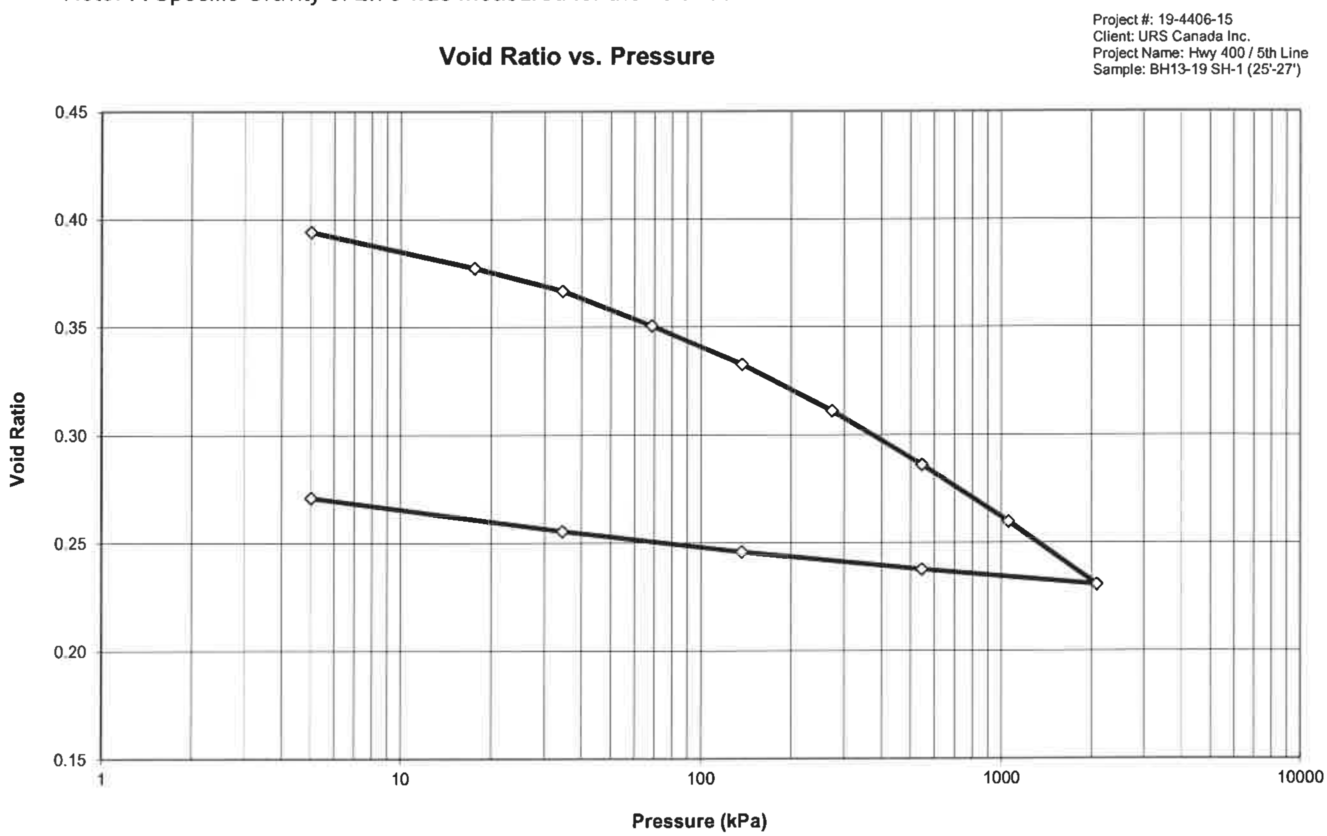
TEST DATES: February 03, 2014 - February 09, 2014

SAMPLE: BH13-19 SH-1 (25'-27')
Silty Clay, some sand, grey, contains 47% silt, 32% clay, and 19% sand; PL = 12%, LL=20%

PROCEDURE: Test carried out in accordance with Standard Test Method for One-Dimensional Consolidation Properties of Soils, ASTM D 2435-04, method A (constant load increment duration of 24 hrs)

	Start of Test	End of Test
Wet Dens. (kg/m^3)	2246.7	2415.8
Dry Dens. (kg/m^3)	1951.7	2160.9
Moisture Cont. (%)	15.1	11.8
Void Ratio	0.407	0.271

Note: A Specific Gravity of 2.75 was measured for the void ratio and saturation calculations.



TEST DONE BY: EA
REVIEWED BY:

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FIGURE 1-2

Consolidation Test Report

Hwy 400 / 5th Line
19-4406-15

BH13-19 SH-1 (25'-27')

TRIMMING: The Specimen was manually trimmed to the size of consolidation ring, then mounted in a fixed ring consolidometer.

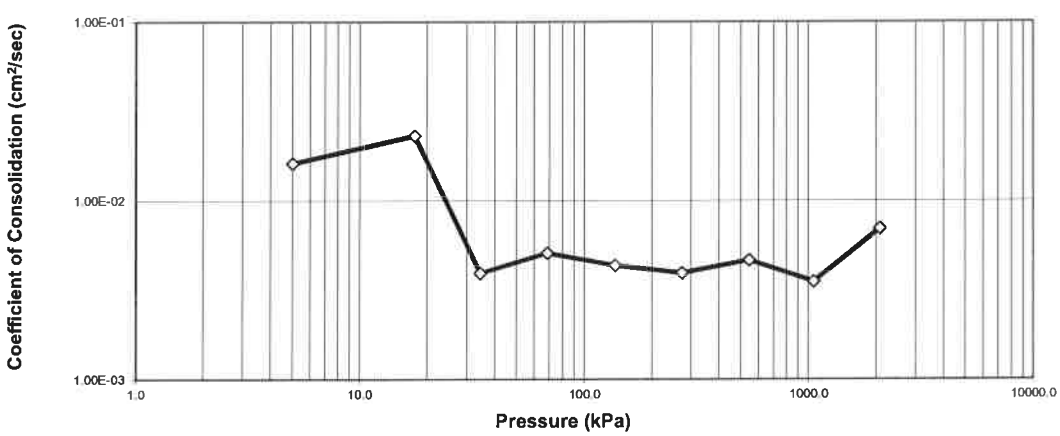
LOADING: A seating load of 5 kPa was applied and the consolidometer was flooded with distilled water. Sample was monitored to ensure no swelling effect occurred before the start of the test. Subsequent loads were applied after 100% primary consolidation was reached.

CALCULATIONS: Coefficients of Consolidation were calculated by the square root time method.

Pressure (kPa)	Corr. H. (mm)	Avg. H. (mm)	d_{90} (mm)	t_{90} (min)	C_V (cm^2/s)	Void Ratio	m_v (m^2/kN)	k (cm/s)
0.0	20.000					0.407		
5.0	19.817	19.909	-0.049	0.86	1.62E-02	0.394	1.82E-03	2.89E-06
17.6	19.579	19.698	-0.070	0.59	2.31E-02	0.377	9.58E-04	2.17E-06
34.5	19.428	19.504	-0.064	3.42	3.93E-03	0.367	4.54E-04	1.75E-07
68.5	19.198	19.313	-0.101	2.59	5.08E-03	0.351	3.49E-04	1.74E-07
136.9	18.944	19.071	-0.127	2.96	4.34E-03	0.333	1.94E-04	8.24E-08
273.2	18.636	18.790	-0.171	3.17	3.94E-03	0.311	1.19E-04	4.60E-08
545.5	18.281	18.459	-0.194	2.59	4.64E-03	0.286	7.00E-05	3.19E-08
1057.7	17.906	18.094	-0.205	3.28	3.53E-03	0.260	4.00E-05	1.39E-08
2080.1	17.493	17.700	-0.235	1.59	6.97E-03	0.231	2.26E-05	1.54E-08
545.5	17.591	17.542				0.238		
136.9	17.708	17.650				0.246		
34.5	17.845	17.777				0.255		
5.0	18.064	17.955				0.271		

Project #: 19-4406-15
Client: URS Canada Inc.
Project Name: Hwy 400 / 5th Line
Sample: BH13-19 SH-1 (25'-27')

Coefficient of Consoildation vs. Pressure



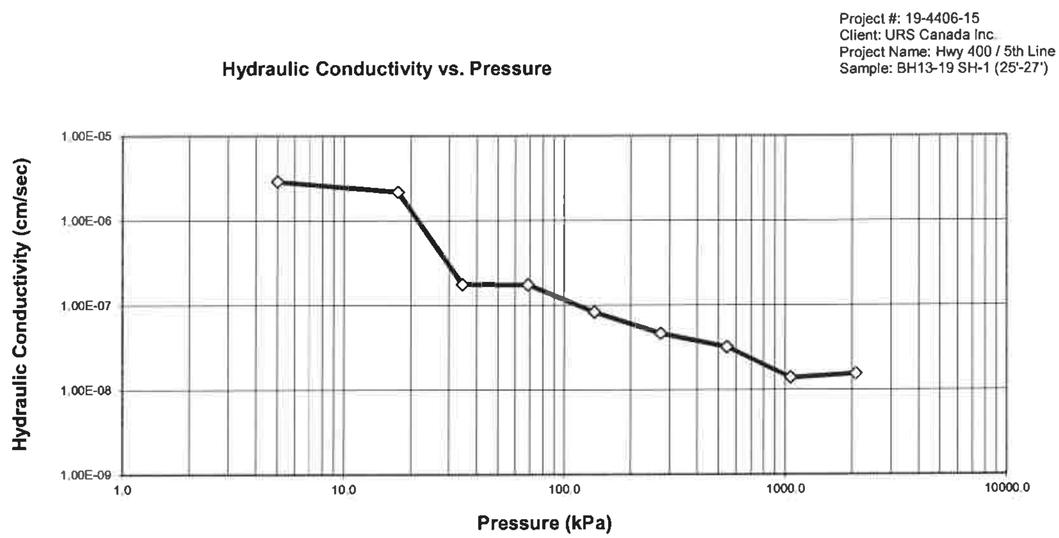
Notes: C_v and k calculated using t_{90} values

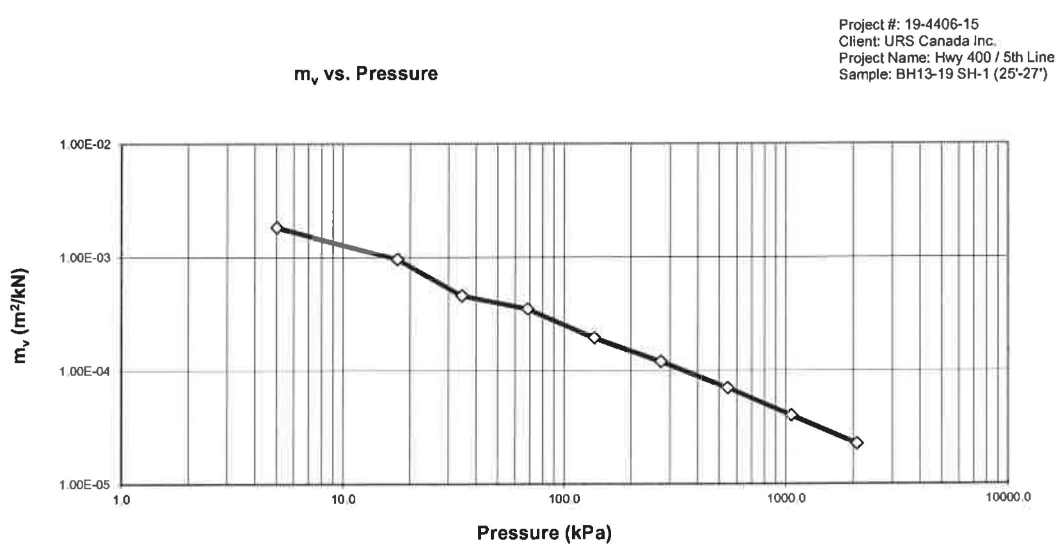
TEST DONE BY: EA
REVIEWED BY:

Consolidation Test Report

Hwy 400 / 5th Line
19-4406-15

BH13-19 SH-1 (25'-27')





TEST DONE BY: EA
REVIEWED BY:

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FIGURE 2-1

Consolidation Test Report

CLIENT: URS Canada Inc.

FILE NUMBER: 19-4406-15

PROJECT: Hwy 400 / 5th Line

REPORT DATE: 28-Oct-14

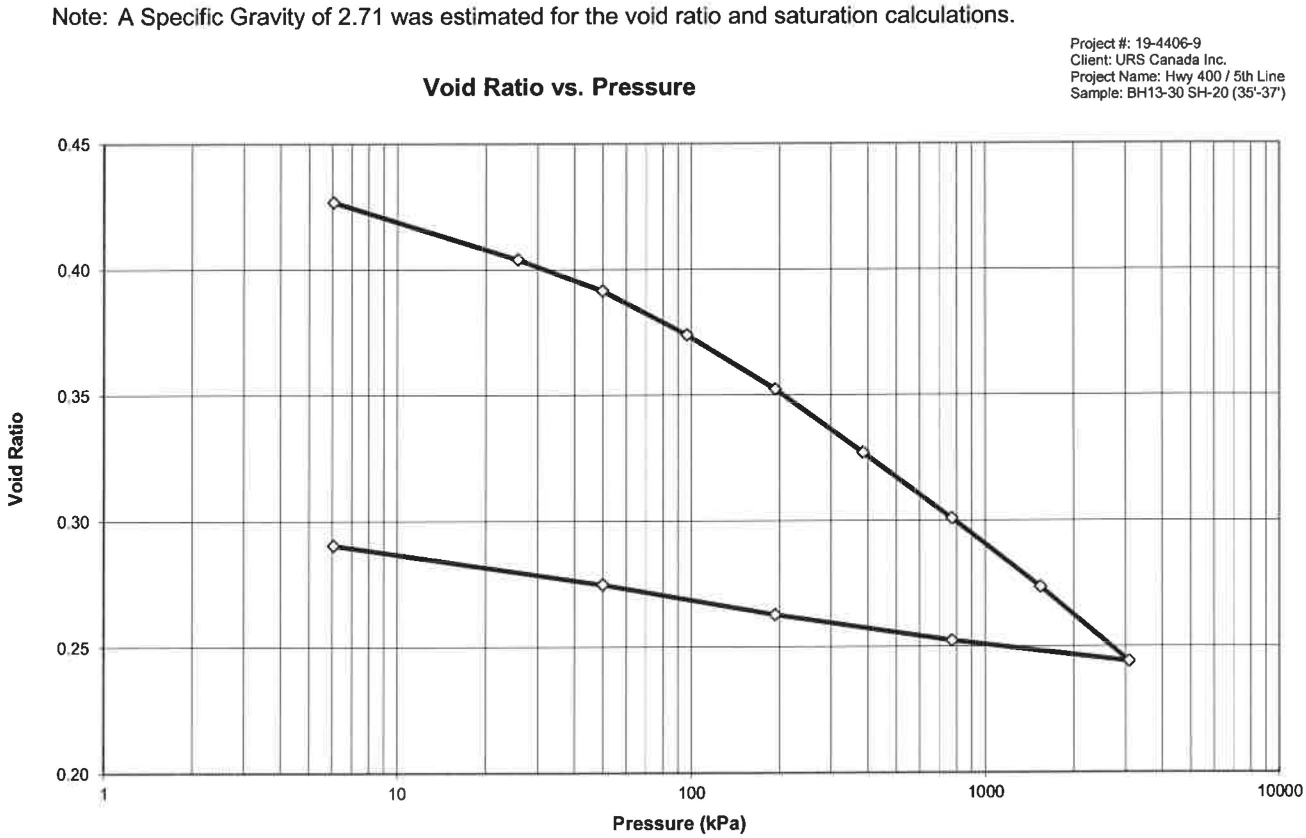
TEST DATES: February 03, 2014 - February 09, 2014

SAMPLE: BH13-30 SH-20 (35'-37')
Silty Clay, some sand, some gravel, grey, 45% silt, 32% clay, 12% sand, and 11% gravel; PL=14%, LL=23%

PROCEDURE: Test carried out in accordance with Standard Test Method for One-Dimensional Consolidation Properties of Soils, ASTM D 2435-04, method B

	Start of Test	End of Test
Wet Dens. (kg/m^3)	2181.2	2382.3
Dry Dens. (kg/m^3)	1884.7	2127.5
Moisture Cont. (%)	15.7	12.0
Void Ratio	0.439	0.275

Note: A Specific Gravity of 2.71 was estimated for the void ratio and saturation calculations.



TEST DONE BY: EA
REVIEWED BY:

THURBER ENGINEERING LTD.

Consolidation Test Report

Hwy 400 / 5th Line
19-4406-15

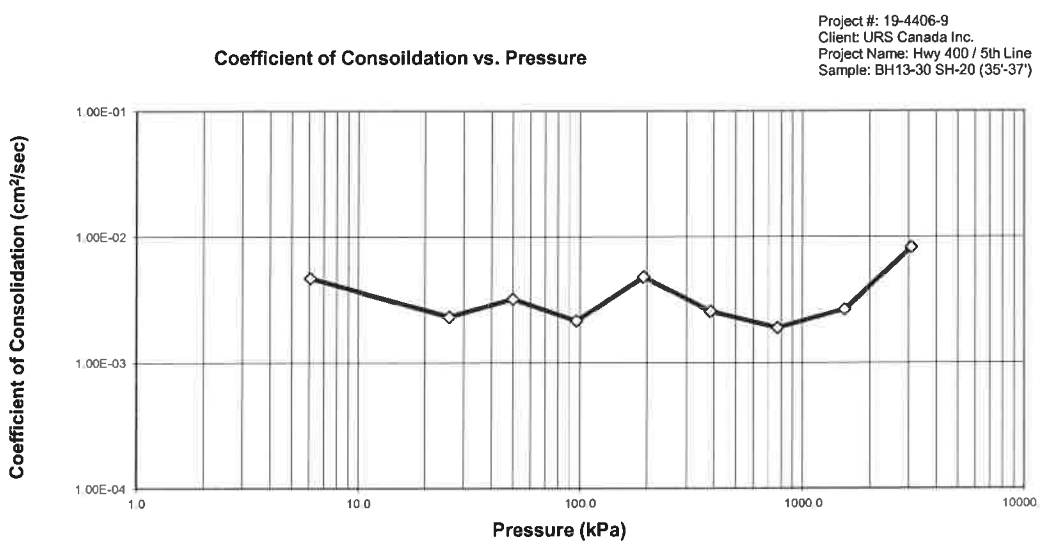
BH13-30 SH-20 (35'-37')

TRIMMING: The Specimen was manually trimmed to the size of consolidation ring, then mounted in a fixed ring consolidometer.

LOADING: A seating load of 6.1 kPa was applied and the consolidometer was flooded with distilled water. Sample was monitored to ensure no swelling effect occurred before the start of the test. Subsequent loads were applied after 100% primary consolidation was reached.

CALCULATIONS: Coefficients of Consolidation were calculated by the square root time method.

Pressure (kPa)	Corr. H. (mm)	Avg. H. (mm)	d_{90} (mm)	t_{90} (min)	c_v (cm^2/s)	Void Ratio	m_v (m^2/kN)	k (cm/s)
0.0	25.500					0.439		
6.1	25.284	25.392	-0.064	4.84	4.71E-03	0.427	1.40E-03	6.44E-07
25.7	24.879	25.082	-0.206	9.61	2.31E-03	0.404	8.17E-04	1.85E-07
49.9	24.658	24.769	-0.092	6.76	3.21E-03	0.391	3.67E-04	1.15E-07
96.6	24.348	24.503	-0.166	9.86	2.15E-03	0.374	2.69E-04	5.67E-08
193.2	23.964	24.156	-0.180	4.28	4.81E-03	0.352	1.63E-04	7.70E-08
385.7	23.515	23.740	-0.272	7.78	2.56E-03	0.327	9.73E-05	2.44E-08
770.7	23.053	23.284	-0.303	10.11	1.89E-03	0.301	5.10E-05	9.48E-09
1540.7	22.569	22.811	-0.309	6.92	2.66E-03	0.274	2.73E-05	7.11E-09
3081.4	22.045	22.307	-0.255	2.13	8.25E-03	0.244	1.51E-05	1.22E-08
770.7	22.194	22.120				0.252		
193.2	22.377	22.286				0.263		
49.9	22.590	22.484				0.275		
6.1	22.867	22.729				0.290		



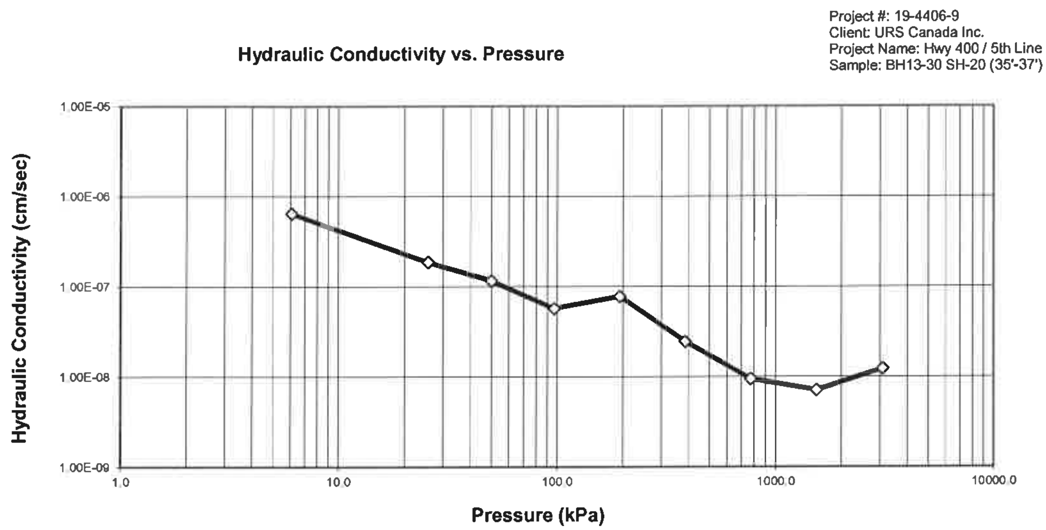
Notes: C_v and k calculated using t_{90} values

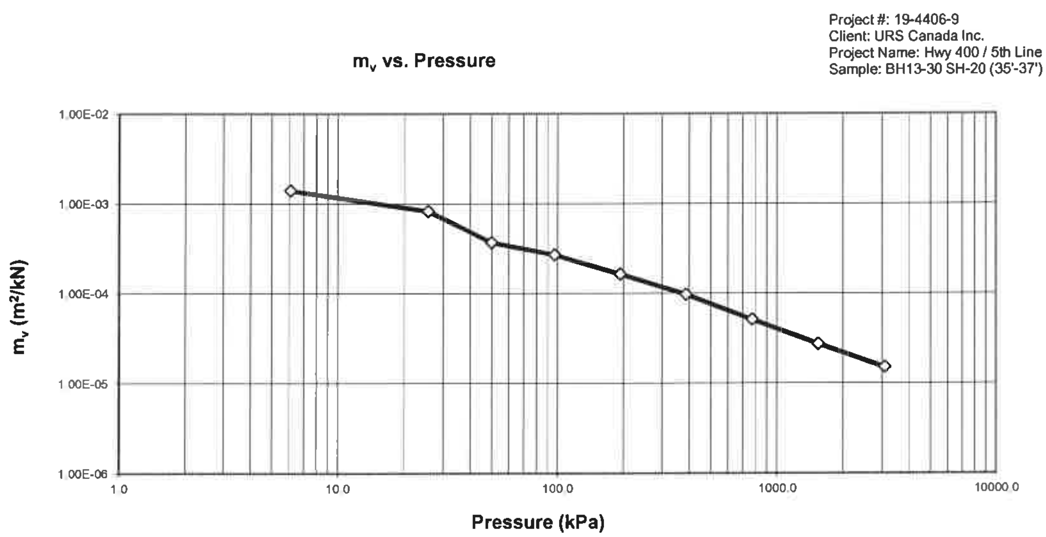
TEST DONE BY: EA
REVIEWED BY:

Consolidation Test Report

Hwy 400 / 5th Line
19-4406-15

BH13-30 SH-20 (35'-37')





TEST DONE BY: EA
REVIEWED BY:

Appendix C

Drawings titled "Borehole Locations and Soil Strata"

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

CONT No
WP No P-13-13

HIGHWAY 400
AT LINE 5
UNDERPASS BRIDGE
BOREHOLE LOCATIONS AND SOIL STRATA

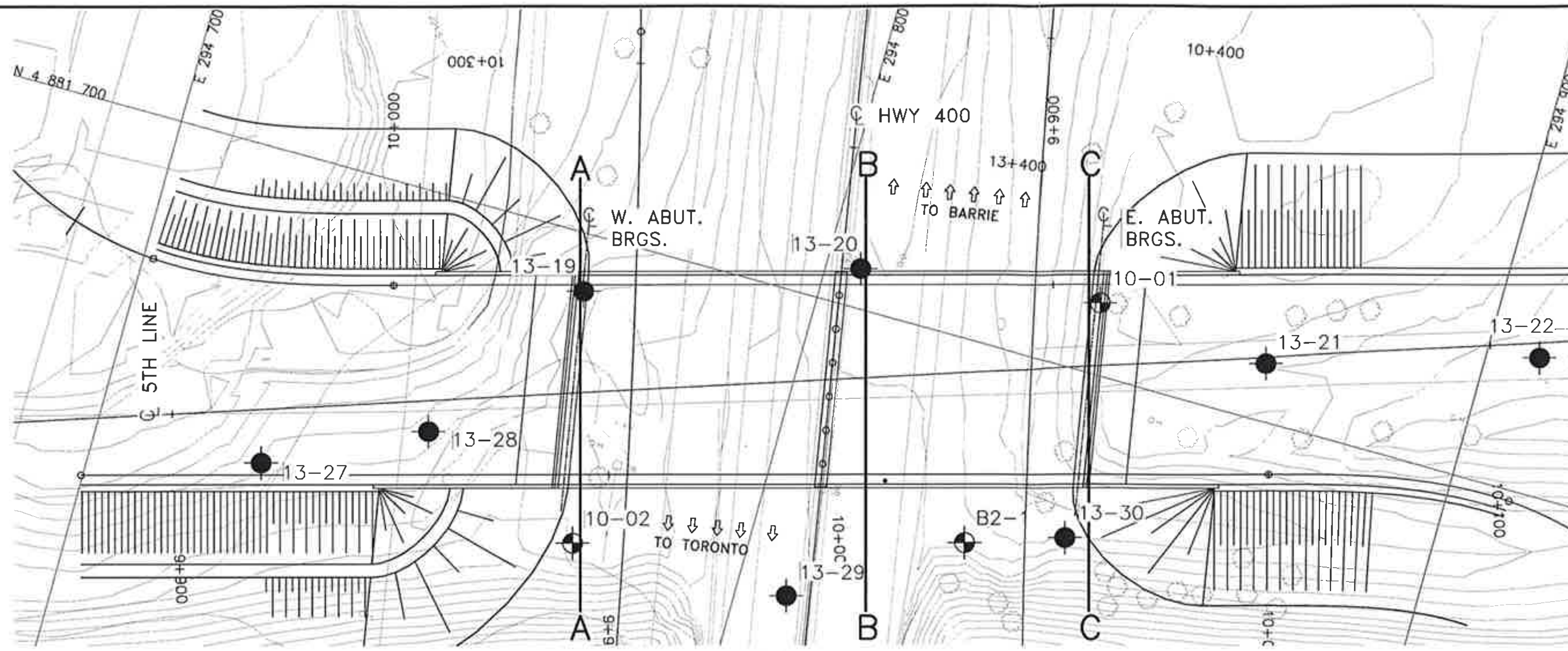
SHEET

AECOM

THURBER ENGINEERING LTD.

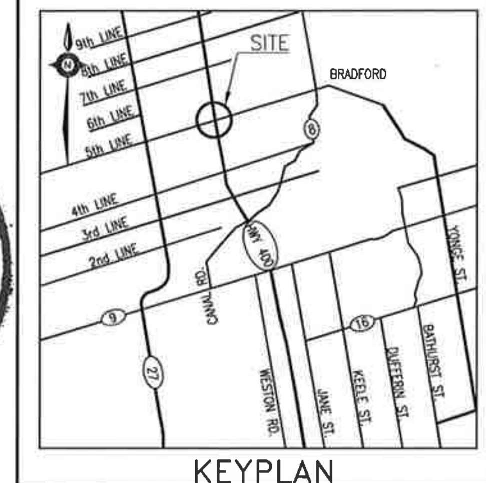
LICENSED PROFESSIONAL ENGINEER
S. PANG
Mar 20/15
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
Mar 20/15
PROVINCE OF ONTARIO

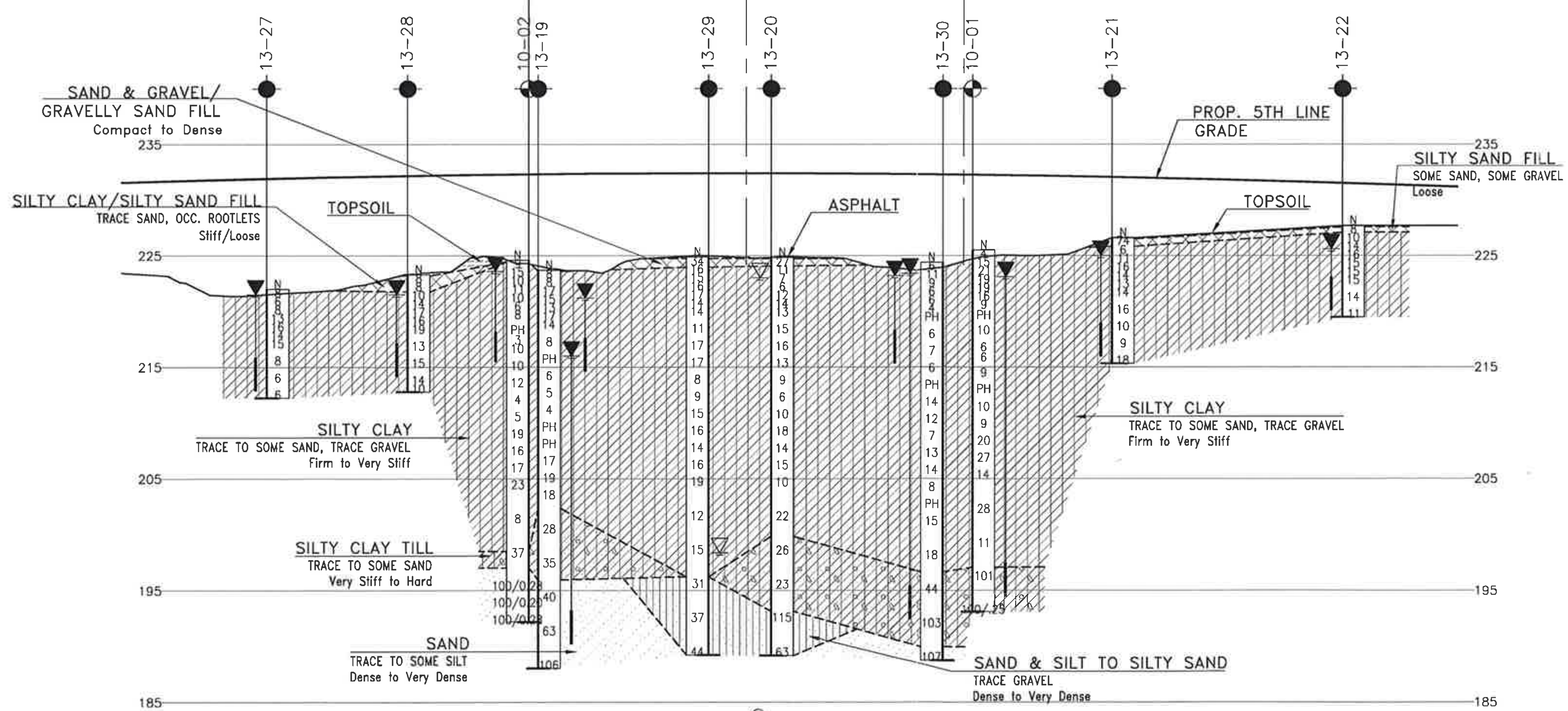


PLAN

20 0 20 40m



KEYPLAN



PROFILE ALONG ℄ 5TH LINE

20 0 20 40m
10 0 10 20m

LEGEND

⊕	Borehole (Current Investigation)
⊕	Borehole (Previous Investigations)
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
▽	Water Level (Open Borehole)
▼	Head Artesian Water
	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
10-01	225.5	4 881 710.7	294 841.6
10-02	224.6	4 881 654.2	294 774.3
13-19	223.7	4 881 691.4	294 765.7
13-20	224.9	4 881 705.9	294 805.2
13-21	226.6	4 881 708.6	294 868.2
13-22	227.7	4 881 720.5	294 907.9
13-27	222.0	4 881 653.3	294 725.5
13-28	223.3	4 881 664.6	294 748.7
13-29	224.9	4 881 655.2	294 807.7
13-30	224.4	4 881 675.0	294 846.0
B2-1	224.7	4 881 670.2	294 831.5

-NOTES-

1) The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
2) This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

GEOCRES No. 31D-591

REVISIONS	DATE	BY	DESCRIPTION		
DESIGN SKP	CHK KMY	CODE	LOAD	DATE MAR 2015	
DRAWN AN	CHK SKP	SITE	STRUCT	DWG 1	

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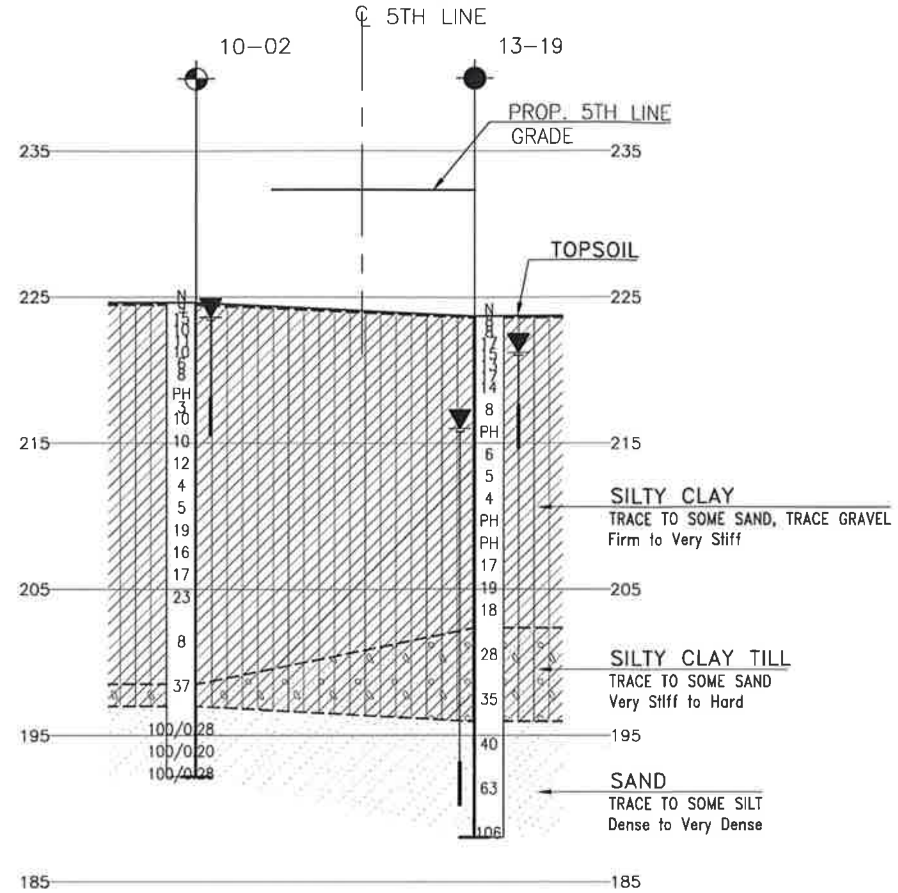
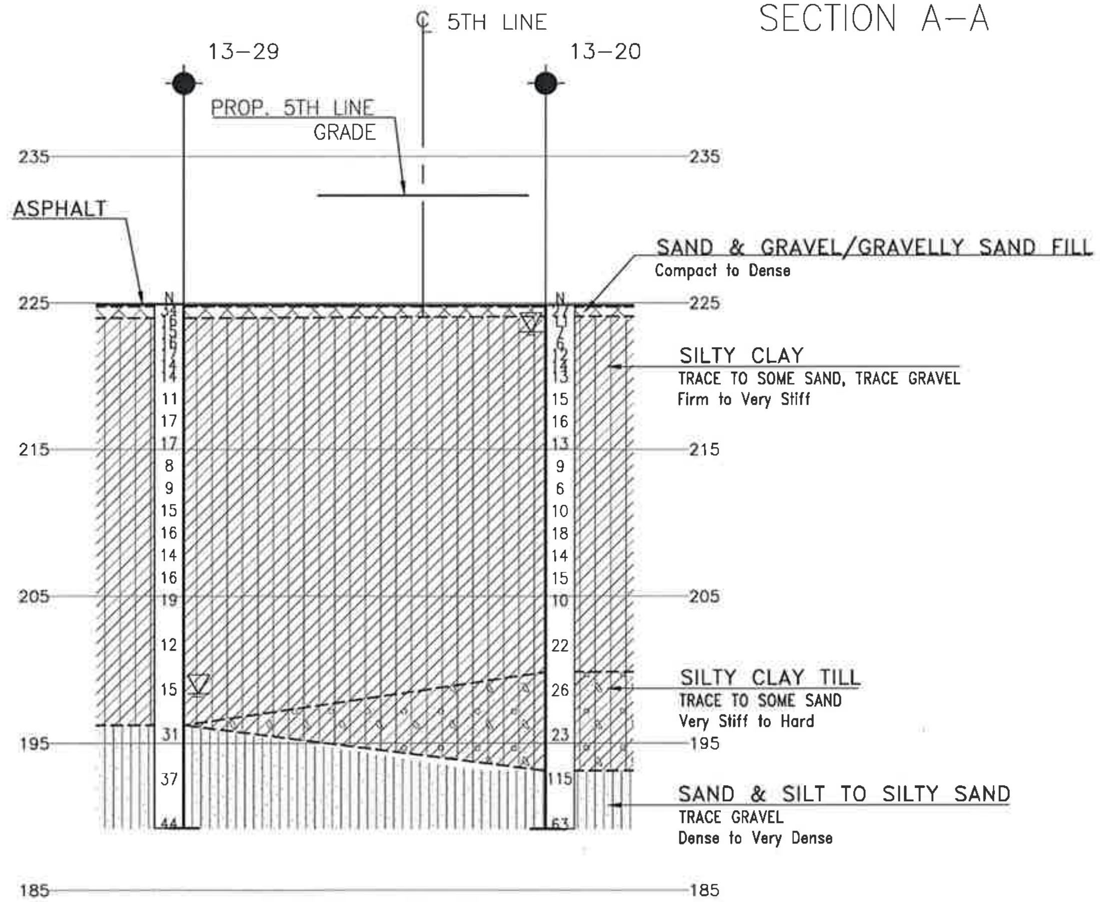
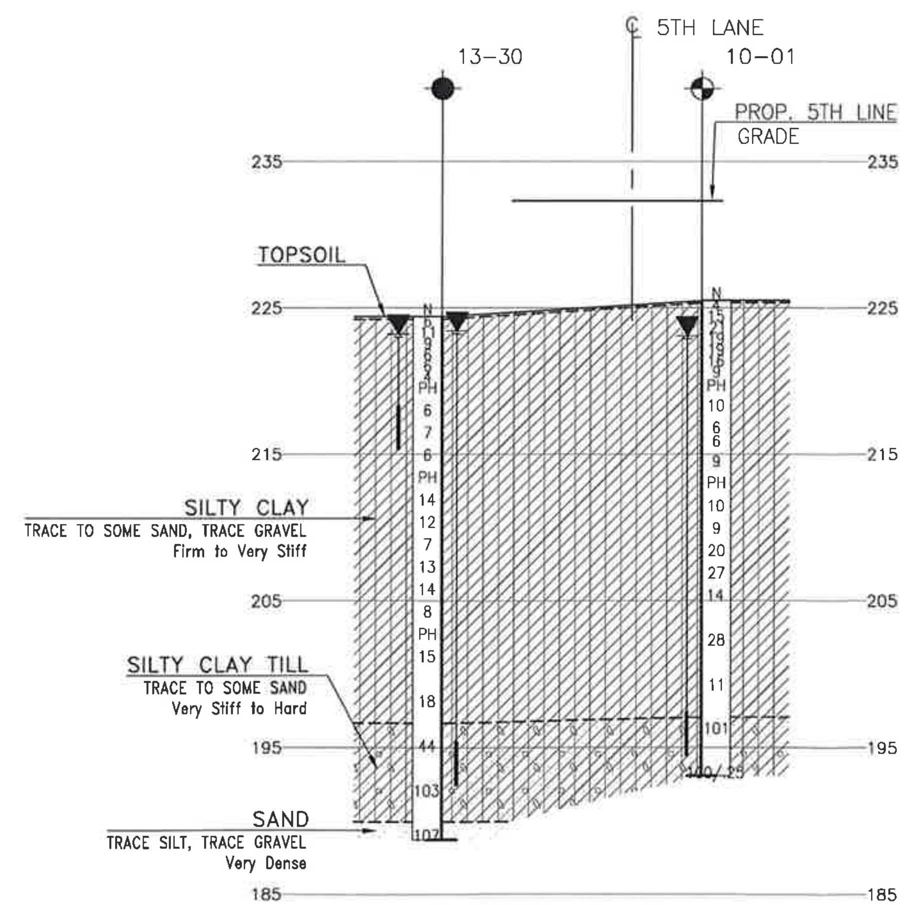
MINISTRY OF TRANSPORTATION, ONTARIO



SECTION A–A



SECTION B–B



SECTION C–C

20 0 20 40m
10 0 10 20m

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

LICENSED PROFESSIONAL ENGINEER
S. PANG
Mar 20/15
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
Mar 20/15
PROVINCE OF ONTARIO

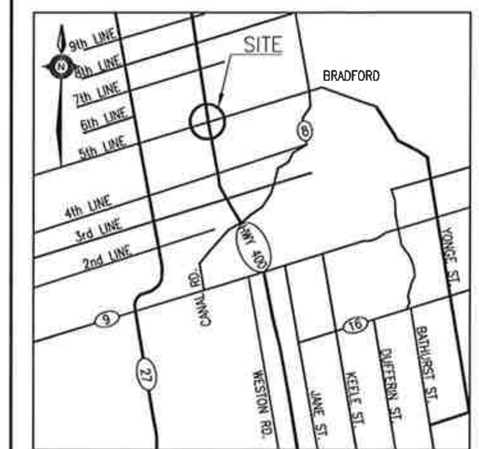
CONT No
WP No P-13-13

HIGHWAY 400
AT LINE 5
UNDERPASS BRIDGE
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

AECOM

THURBER ENGINEERING LTD.



KEYPLAN

LEGEND

	Borehole (Current Investigation)
	Borehole (Previous Investigations)
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
	Water Level (Open Borehole)
	Head Artesian Water
	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
10–01	225.5	4 881 710.7	294 841.6
10–02	224.6	4 881 654.2	294 774.3
13–19	223.7	4 881 691.4	294 765.7
13–20	224.9	4 881 708.1	294 813.1
13–21	226.6	4 881 708.6	294 868.2
13–22	227.7	4 881 720.5	294 907.9
13–27	222.0	4 881 653.3	294 725.5
13–28	223.3	4 881 664.6	294 748.7
13–29	224.9	4 881 656.1	294 810.8
13–30	224.4	4 881 675.0	294 846.0
B2–1	224.7	4 881 670.2	294 831.5

-NOTES-

1) The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
2) This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

GEOCRES No. 31D-591

REVISIONS	DATE	BY	DESCRIPTION
DESIGN SKP	CHK KMY	CODE	LOAD	DATE MAR 2015
DRAWN AN	CHK SKP	SITE	STRUCT	DWG 2

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MINISTRY OF TRANSPORTATION, ONTARIO

Appendix D

Site Photographs

Photo 3.- West abutment, existing bridge

Photo 2.- East abutment, existing bridge

Photo 1.- Highway 400 and 5th Line , north side

Photo 2.- Highway 400 and 5th Line , south side

Appendix E

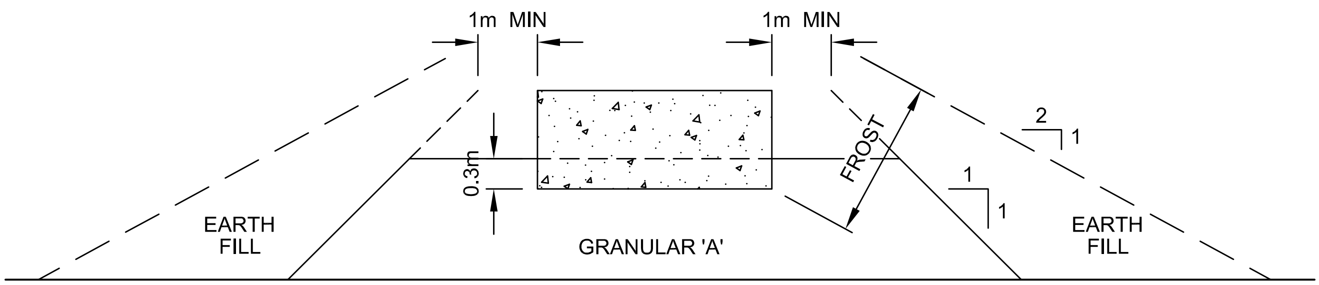
Comparison of Foundation Alternatives for Underpass Bridge

COMPARISON OF FOUNDATION ALTERNATIVES

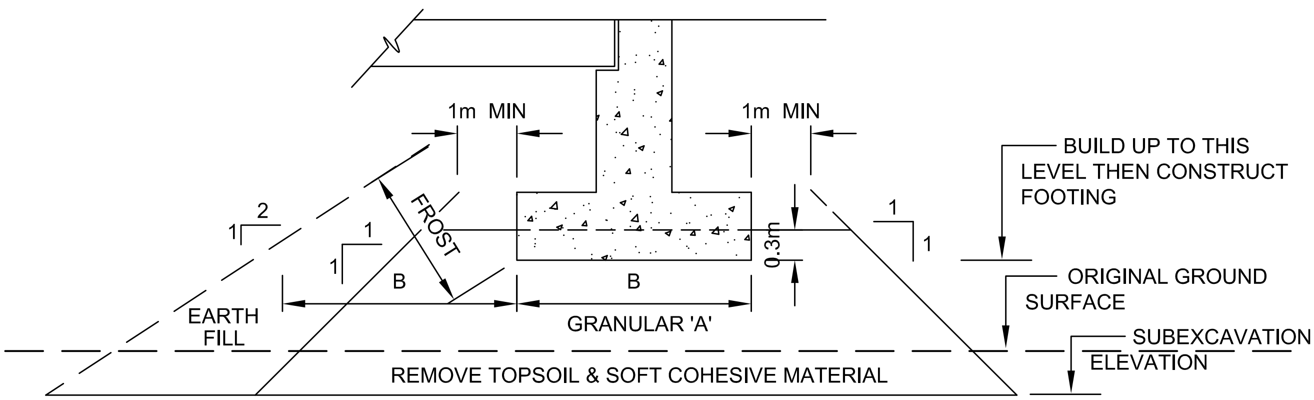
Foundation Unit	Spread Footings on Native Soils	Driven Steel H-Pile into Native Glacial Till/Sand and Silt	Augered Steel H-Pile into Silty Clay	Augered Caissons (Drilled Shafts) into Native Silty Clay
	Advantages: i. Ease of construction. ii. Lower cost than deep foundations. ***Disadvantages:*** i. Relatively low geotechnical resistance is available. ii. Potential for long-term settlement of foundation soils due to consolidation under approach fill loads. iii. Dewatering may be required, depending on depth of excavation and groundwater level at time of construction.	***Advantages:*** i. Ease of construction. ii. Required for integral abutments. iii. Dewatering not required for pile installation. ***Disadvantages:*** i. Subject to downdrag force at abutments unless preloading / surcharging is implemented. ii. At pier, no downdrag force but no distinct end-bearing stratum within typical maximum pile length. iii. Relatively lower lateral resistance is available given the pile dimension. iv. Potential obstruction to pile penetration at elevations higher than the design tip elevations. v. Larger number of piles will likely be required to resist foundation loads.	***Advantages:*** i. Higher axial resistance than driven H-pile. ii. Higher lateral resistance can be provided by permanently grouting in temporary liner and filling annulus with concrete. iii. Much lower risk of pile obstruction during installation at locations higher than the design tip elevations ***Disadvantages:*** i. Not suitable for integral abutments ii. Steel casings required to pre-drill holes. iii. Tremie concrete may need to be used. iv. Potential basal instability if water-bearing soils are exposed at the base. v. Difficulty in cleaning base of sockets before lowering H-piles.	***Advantages:*** i. Higher axial resistance than H-pile. ii. Higher lateral resistance is available due to larger diameter. iii. Less number of caissons is required for each foundation element than if steel piles were used. ***Disadvantages:*** i. Not suitable for integral abutments ii. Higher unit cost than steel driven piles. iii. Steel liners will be required to install caissons to minimize sidewall sloughing and water seepage. iv. Tremie concrete may need to be used. v. Potential basal instability if water-bearing soils are exposed at the base. vi. Difficulty in cleaning and inspecting bases.
Abutments	**NOT RECOMMENDED**	**RECOMMENDED**	**NOT RECOMMENDED**	**FEASIBLE (if non-integral abutments)**
Pier	**NOT RECOMMENDED**	**RECOMMENDED**	**FEASIBLE**	**FEASIBLE (if restricted space)**

Appendix G

Figure 1



CROSS-SECTION



LONGITUDINAL SECTION

NOTES:

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

ABUTMENT ON COMPACTED FILL
SHOWING GRANULAR 'A' CORE

THURBER ENGINEERING LTD.

ENGINEER :	DRAWN :	APPROVED :
LPG	MFA	AEG
DATE : OCTOBER 2013	SCALE : N.T.S.	DRAWING No. FIGURE 1

Appendix F

List of OPSS Documents and NSSP Wording

1. List of OPSS Documents Referenced in this Report

- OPSS 903
- OPSS 206
- OPSS 804
- OPSS 805
- OPSS 501
- OPSS 539
- OPSS 902
- OPSS 10103
- OPSD 3000.100

2. Suggested Text for NSSP on "Drilling of Caisson Sockets"

The native soil deposits generally increase in strength with depth and contain hard zones throughout. Caisson installation through glacially derived soil deposits may encounter cobbles and/or boulders, and the installation equipment should be capable of dislodging and removing such obstructions. Augering and excavation through the obstructions and hard zones may be difficult.

Appendix H

Selected Embankment Stability Analyses Results

Title: Highway 400, 5th Line
Comments: Embankment Stability Analysis
Name: Analysis WA-ES (1.5H:1V, 11m)

Surcharge	21 kN/m³	0 kPa	35 °	1	0	Yes
Silty Clay I (ES)	20 kN/m³	0 kPa	30 °	1	0.3	No
Silty Clay II (ES)	19 kN/m³	0 kPa	30 °	1	0.6	No
Silty Clay III (ES)	21 kN/m³	0 kPa	32 °	1	0.3	No
Embankment Fill (West)	21 kN/m³	0 kPa	35 °	1		No
Embankment Fill (East)	21 kN/m³	0 kPa	30 °	1		No

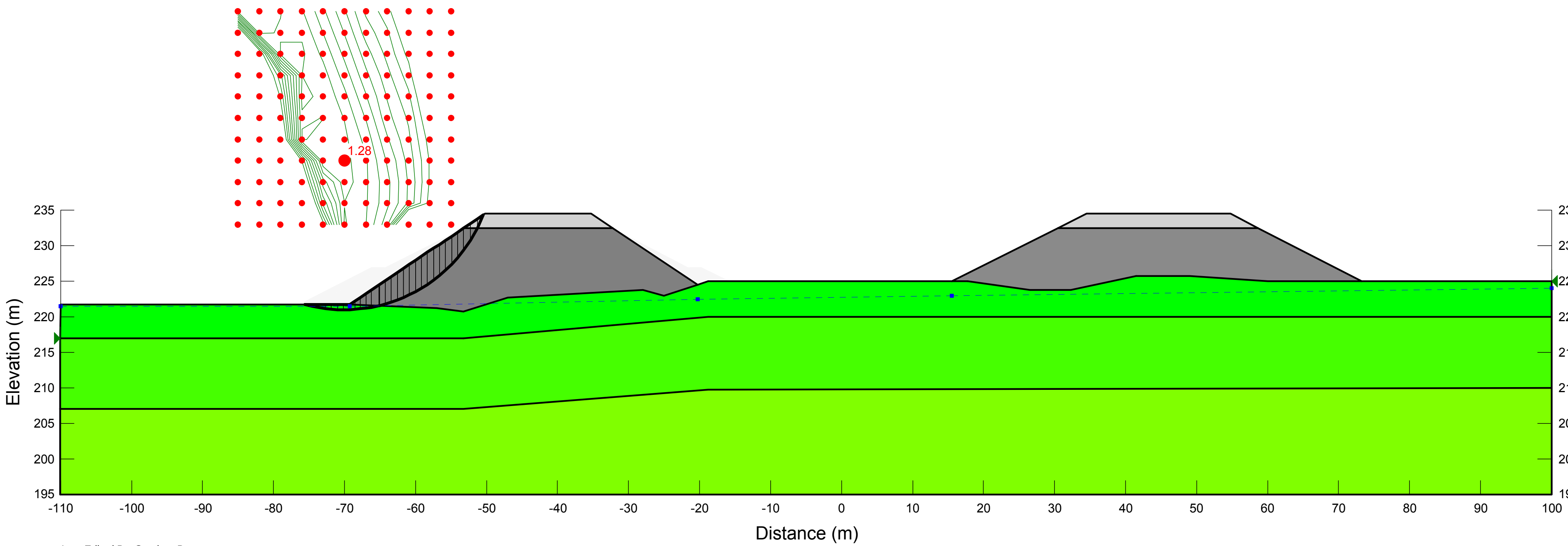


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Figure 1

Title: Highway 400, 5th Line
Comments: Embankment Stability Analysis
Name: Analysis WA-TS (1.5H:1V, 11m)

Surcharge 21 kN/m³ 0 kPa 35 ° 1 0 Yes
Silty Clay I (TS) 20 kN/m³ 45 kPa 5 kPa/m 70 kPa 1 0 No
Silty Clay II (TS) 19 kN/m³ 70 kPa -1.5 kPa/m 55 kPa 1 0 No
Silty Clay III (TS) 21 kN/m³ 55 kPa 5 kPa/m 125 kPa 1 0 No
Embankment Fill (West) 21 kN/m³ 0 kPa 35 ° 1 No
Embankment Fill (East) 21 kN/m³ 0 kPa 30 ° 1 No

Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1 m
Horz Seismic Coef.: 0
Center: (-61, 236) m, Radius: 29 m

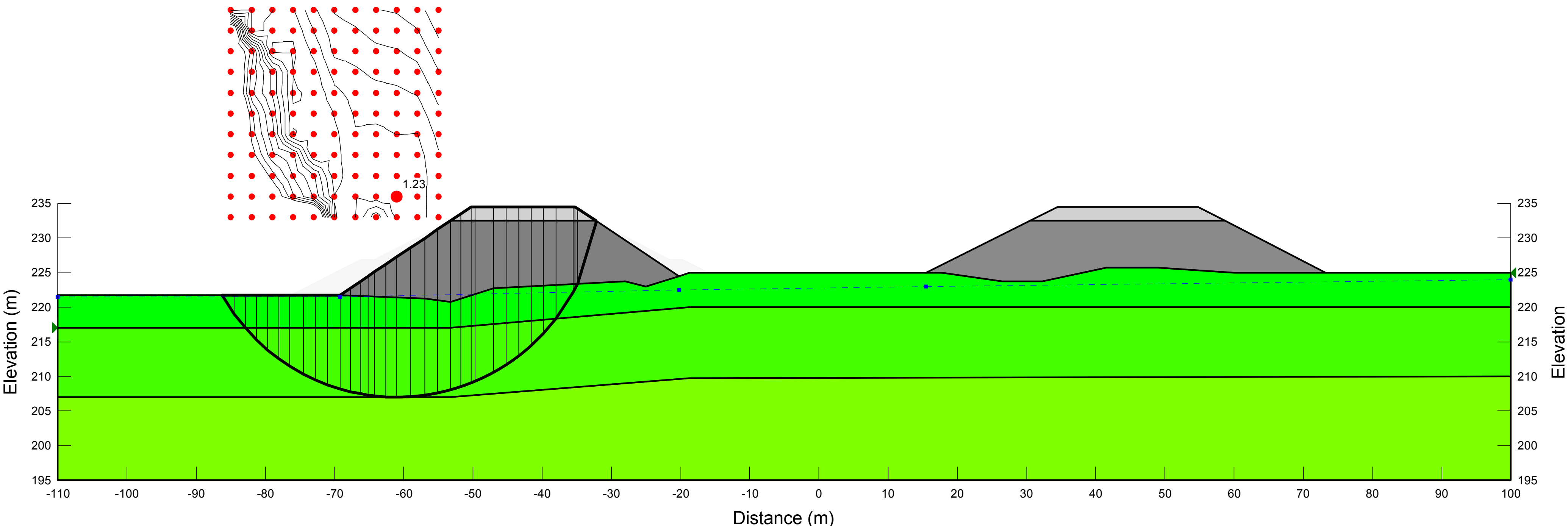

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Figure 2

Title: Highway 400, 5th Line
Comments: Embankment Stability Analysis
Name: Analysis WA-ES (2H:1V, 11m)

Surcharge	21 kN/m³	0 kPa	35 °	1	0	Yes
Silty Clay I (ES)	20 kN/m³	0 kPa	30 °	1	0.3	No
Silty Clay II (ES)	19 kN/m³	0 kPa	30 °	1	0.6	No
Silty Clay III (ES)	21 kN/m³	0 kPa	32 °	1	0.3	No
Embankment Fill (West)	21 kN/m³	0 kPa	35 °	1		No
Embankment Fill (East)	21 kN/m³	0 kPa	30 °	1		No

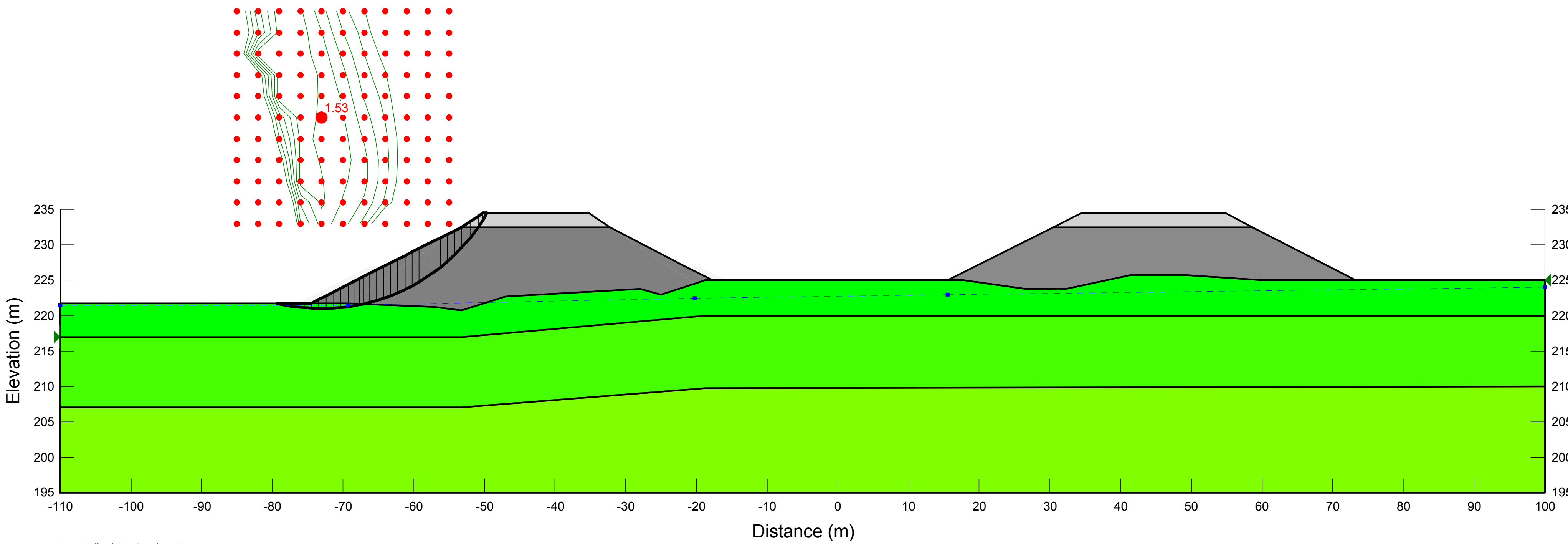

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Figure 3

Title: Highway 400, 5th Line
Comments: Embankment Stability Analysis
Name: Analysis WA-ES (2H:1V, 9m)

Silty Clay I (ES)	20 kN/m³	0 kPa	30 °	1
Silty Clay II (ES)	19 kN/m³	0 kPa	30 °	1
Silty Clay III (ES)	21 kN/m³	0 kPa	32 °	1
Embankment Fill (West)	21 kN/m³	0 kPa	35 °	1
Embankment Fill (East)	21 kN/m³	0 kPa	30 °	1

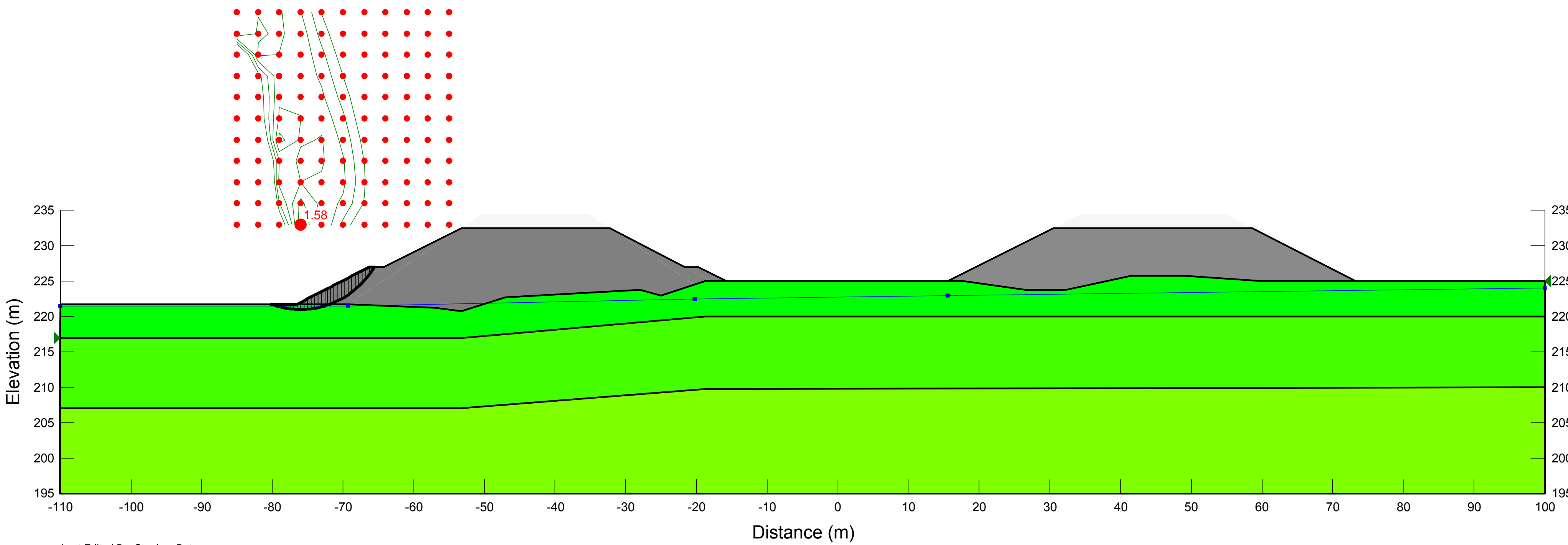

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Figure 4

Title: Highway 400, 5th Line
Comments: Embankment Stability Analysis
Name: Analysis WA-ES (2H:1V, 9m)_ssm

Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1 m
Horz Seismic Coef.: 0
Center: (-76, 233) m, Radius: 12 m

Silty Clay I (ES)	20 kN/m³	0 kPa	30 °	1
Silty Clay II (ES)	19 kN/m³	0 kPa	30 °	1
Silty Clay III (ES)	21 kN/m³	0 kPa	32 °	1
Embankment Fill (East)	21 kN/m³	0 kPa	30 °	1

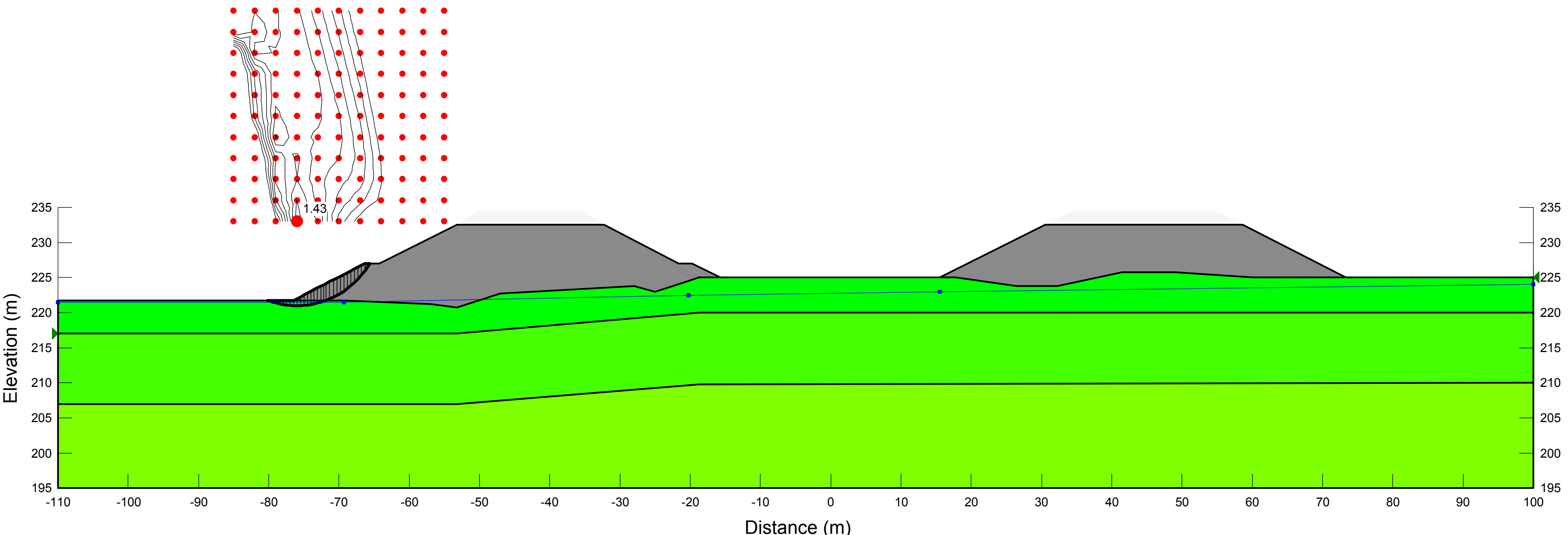

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Figure 5

Title: Highway 400, 5th Line
Comments: Embankment Stability Analysis
Name: Analysis WA-TS (2H:1V, 9m)

Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1 m
Horz Seismic Coef.: 0
Center: (-64, 242) m, Radius: 35 m

Silty Clay I (TS)	20 kN/m³	45 kPa	5 kPa/m	70 kPa	1
Silty Clay II (TS)	19 kN/m³	70 kPa	-1.5 kPa/m	55 kPa	1
Silty Clay III (TS)	21 kN/m³	55 kPa	5 kPa/m	125 kPa	1
Embankment Fill (West)	21 kN/m³	0 kPa	35 °	1	
Embankment Fill (East)	21 kN/m³	0 kPa	30 °	1	

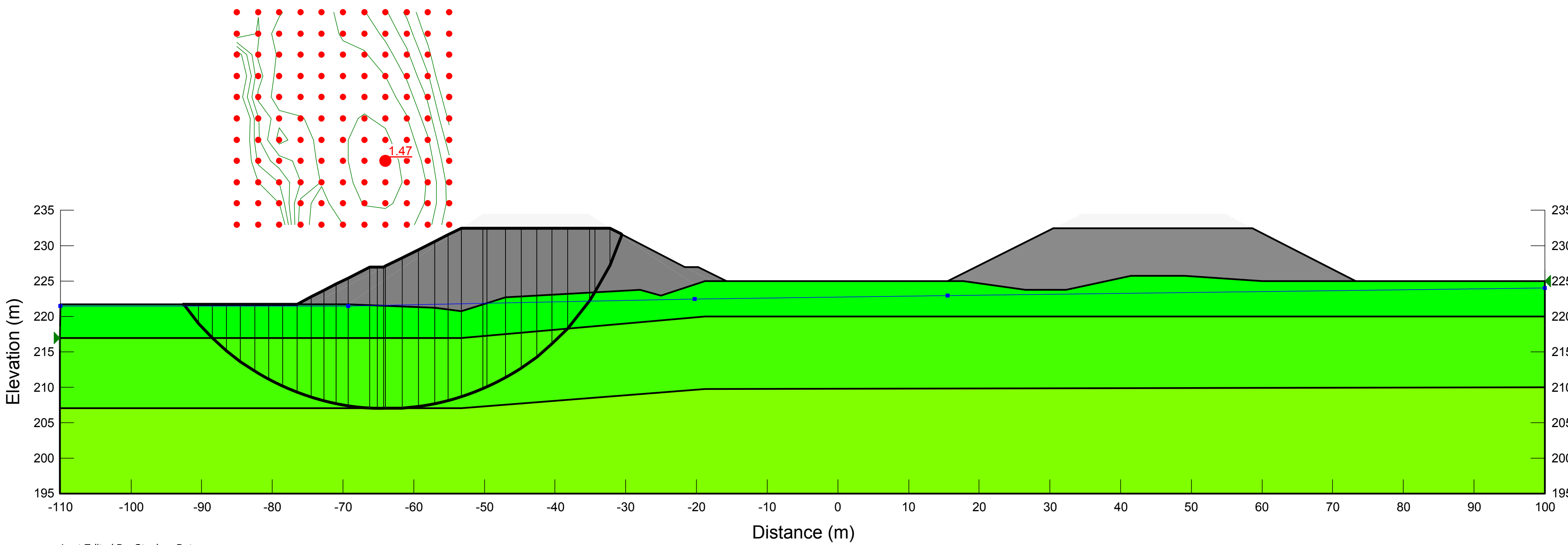

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Figure 6

Title: Highway 400, 5th Line
Comments: Embankment Stability Analysis
Name: Analysis WA-TS (2H:1V, 9m)_ssm

Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1 m
Horz Seismic Coef.: 0
Center: (-64, 242) m, Radius: 35 m

Silty Clay I (TS)	20 kN/m³	45 kPa	5 kPa/m	70 kPa	1
Silty Clay II (TS)	19 kN/m³	70 kPa	-1.5 kPa/m	55 kPa	1
Silty Clay III (TS)	21 kN/m³	55 kPa	5 kPa/m	125 kPa	1
Embankment Fill (East)	21 kN/m³	0 kPa	30 °	1	

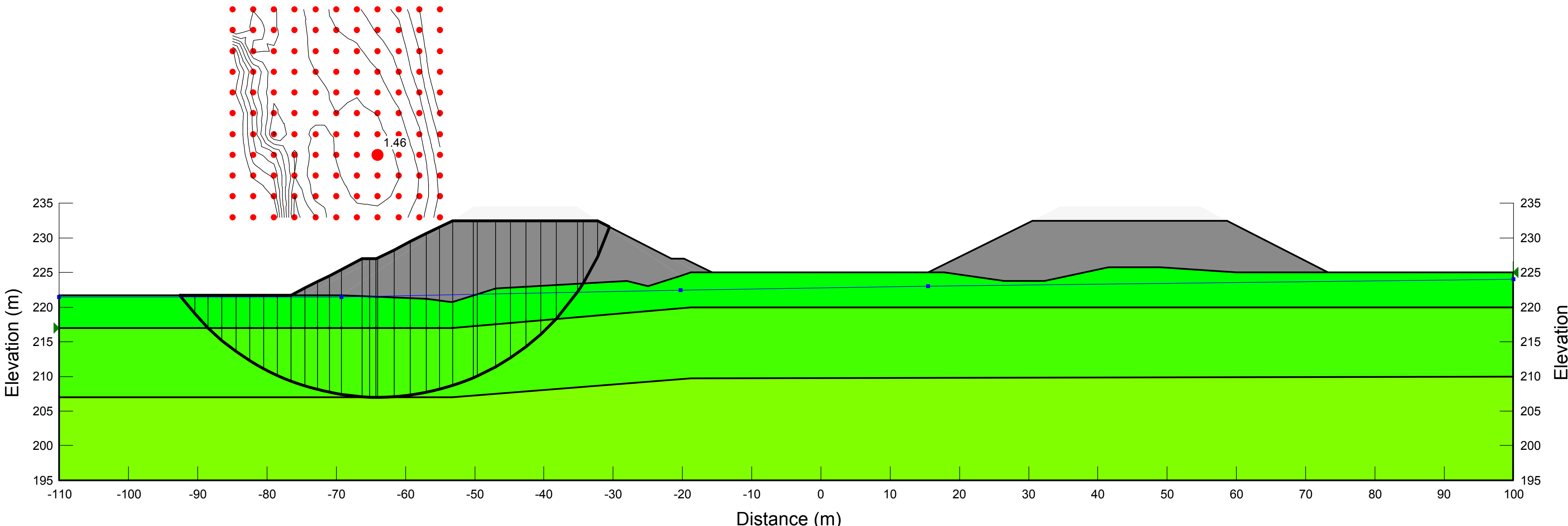

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Figure 7

Title: Highway 400, 5th Line
Comments: Embankment Stability Analysis
Name: Analysis EA-ES (2H:1V, 10m)

Surcharge	21 kN/m³	0 kPa	35 °	1	0	Yes
Silty Clay I (ES)	20 kN/m³	0 kPa	30 °	1	0.3	No
Silty Clay II (ES)	19 kN/m³	0 kPa	30 °	1	0.6	No
Silty Clay III (ES)	21 kN/m³	0 kPa	32 °	1	0.3	No
Embankment Fill (West)	21 kN/m³	0 kPa	35 °	1		No
Embankment Fill (East)	21 kN/m³	0 kPa	30 °	1		No

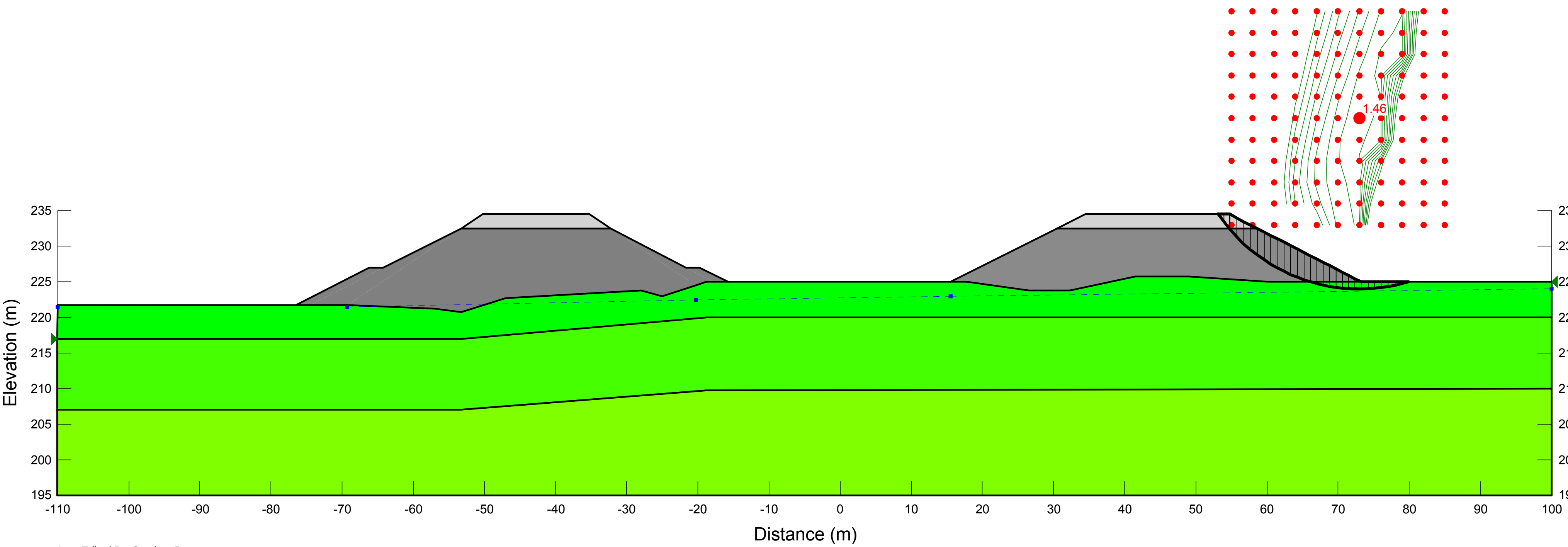

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Figure 8

Title: Highway 400, 5th Line
Comments: Embankment Stability Analysis
Name: Analysis EA-ES (2H:1V, 8m)

Silty Clay I (ES)	20 kN/m³	0 kPa	30 °	1
Silty Clay II (ES)	19 kN/m³	0 kPa	30 °	1
Silty Clay III (ES)	21 kN/m³	0 kPa	32 °	1
Embankment Fill (West)	21 kN/m³	0 kPa	35 °	1
Embankment Fill (East)	21 kN/m³	0 kPa	30 °	1

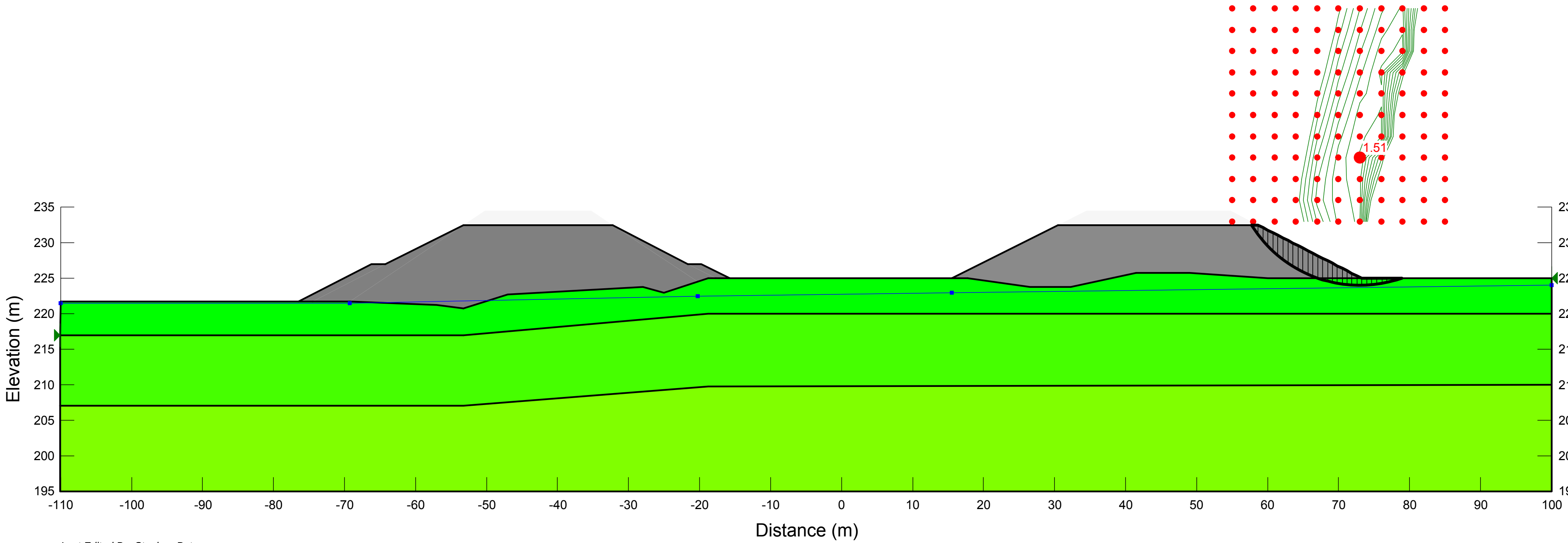

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Figure 9

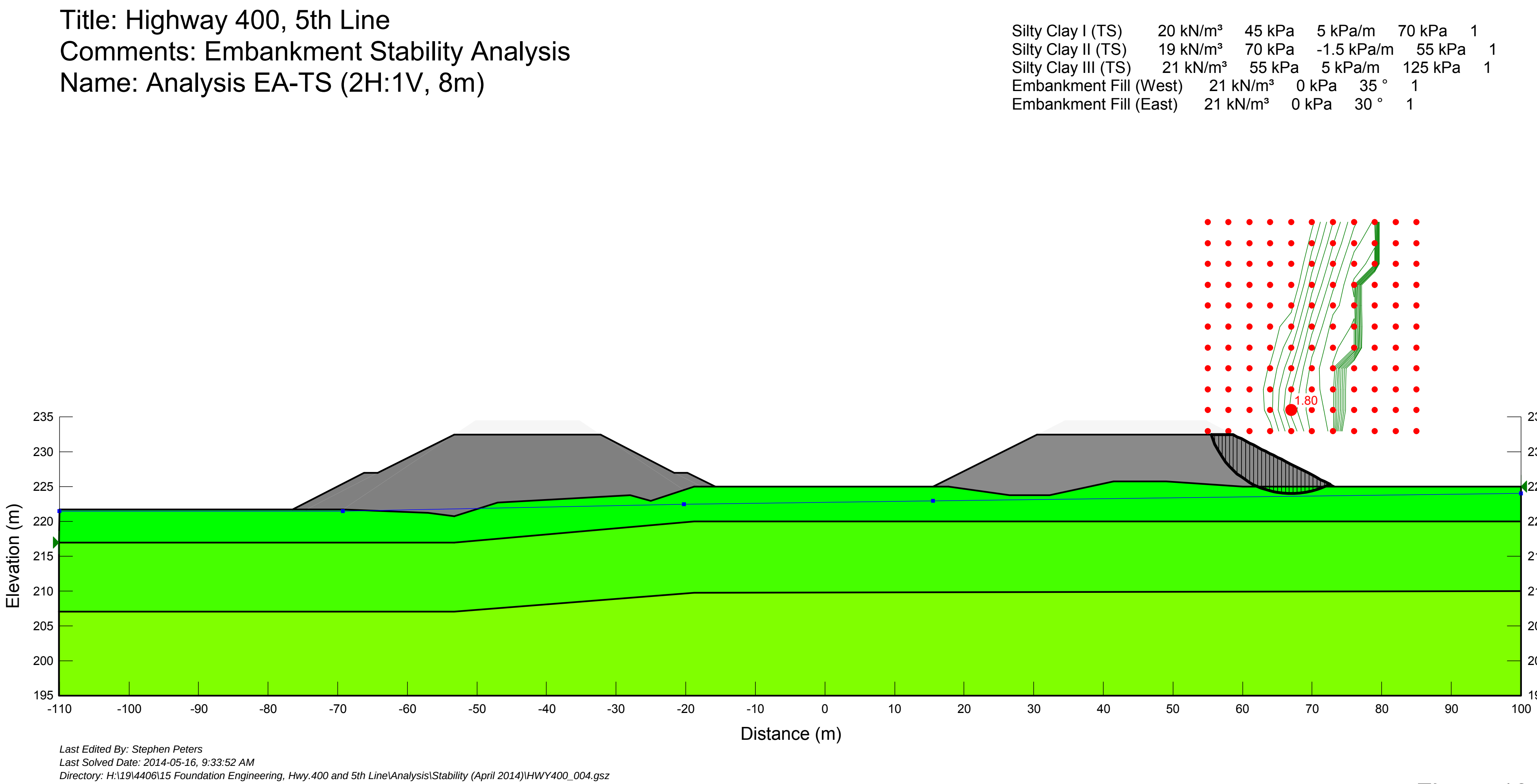
Title: Highway 400, 5th Line
Comments: Embankment Stability Analysis
Name: Analysis EA-TS (2H:1V, 8m)
Silty Clay I (TS) 20 kN/m³ 45 kPa 5 kPa/m 70 kPa 1
Silty Clay II (TS) 19 kN/m³ 70 kPa -1.5 kPa/m 55 kPa 1
Silty Clay III (TS) 21 kN/m³ 55 kPa 5 kPa/m 125 kPa 1
Embankment Fill (West) 21 kN/m³ 0 kPa 35 ° 1
Embankment Fill (East) 21 kN/m³ 0 kPa 30 ° 1
1.80
Elevation (m)
Distance (m)
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Figure 10

Appendix I

Instrumentation and Monitoring Program (Advance Contract)

SUPPLY AND INSTALLATION OF MONITORING EQUIPMENT - Item No.

Special Provision

1.0 GENERAL

1.0.1 Scope

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

- Settlement Rods (SR)
- Pavement Markers (PM)
- Structure Points (SP)

1.0.2 Purpose

The purpose of these instruments is to monitor settlements of the base of the preload fills for the proposed Highway 400 - Line 5 Underpass Bridge, the pavement of Highway 400 roadway in the vicinity of preloading/surcharging areas, and the existing 5th Line Structure and the immediate approaches, during the preloading and surcharging of the approach fills for the proposed Highway 400 - Line 5 Underpass Bridge.

1.0.3 Personnel

The Contractor shall retain a Geotechnical Consultant with MTO classification of Geotechnical (Structures and Embankments) – **Medium Complexity**, to carry out the supply and installation of geotechnical instruments.

1.0.4 Notification

The Contract Administrator (CA) shall be notified a minimum of 7 working days in advance of commencing the installation of instruments.

1.0.5 Submission Requirements

The Contractor shall submit details of proposed installation methods, including data acquisition method, survey benchmarks, and installation schedule to the Contract Administrator, a minimum of 7 days before the start of instrument installation.

1.0.6 Drawings

Reference shall be made to the following drawings:

- Monitoring Instrument Location Plan – Sheet 1;
- Monitoring Sections – Sheet 2;
- Instrument Details – Sheet 3.

1.0.7 Subsurface Conditions

The subsurface conditions at the site are described in the following technical memorandum:

Preliminary Foundation Recommendations, Bridge Foundations and Immediate Approaches, Proposed Highway 400 5th Line Underpass Bridge, West of Bradford, Ontario, Project # P-13-03, dated March 17, 2014, prepared by Thurber Engineering Ltd.

1.0.8 Equipment Operation and Weather Conditions

All installation and monitoring equipment and associated materials shall be capable of withstanding the range of temperatures possible for their locations within the ground or on the surface year-round.

The Contractor shall repair or replace any non-functioning monitoring instruments caused by the Contractor's work as required at no cost to the Town of Bradford - West Gwillimbury.

1.1 INSTALLATION

Table 1 - Instrument Quantities and Locations

Structure	Location	Settlement Rod (SR)	Pavement Marker (PM)		Structure Point (SP)
		Quantity	Quantity	Location	Quantity
Preload/Surcharge Fills for Proposed Line 5 Underpass Bridge	West Abutment	4	3	Hwy 400 West Shoulder	-
	East Abutment	4	3	Hwy 400 East Shoulder	-
Existing 5th Line Structure	West Approach	-	2	North & South Shoulders	-
	West Abutment	-	-	-	2
	East Abutment	-	-	-	2
	East Approach	-	2	North & South Shoulders	-
	Total	8	10	-	4

1.1.1 Instrument Locations

Prior to the installation of instruments, the Contractor shall accurately survey and stake the location of each instrument and obtain a ground elevation at each instrument location. Approximate instrument locations are provided in the drawing Sheet 1. The actual locations shall be decided in the field in consultation with the CA and the Foundation Monitoring Consultant.

1.1.2 Survey Benchmarks

The Contractor shall provide a minimum of two non-yielding survey benchmarks at the site.

The number and locations of benchmarks shall be such that direct sighting is possible from all instruments to at least one benchmark. At least one benchmark shall be provided on each side of Highway 400. Benchmarks shall be located in areas not affected by construction activities.

1.1.3 Accuracy of Surveying for Elevations

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

1.1.4 Materials and Equipment

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

1.1.5 Marking and Labelling

The location of any above ground monitoring fixture shall be made clearly visible to nearby traffic including construction equipment. Marking shall be of sufficient size to be visible from a reversing vehicle.

1.1.6 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 the Contractor's cost.

2.0 SETTLEMENT ROD (SR) - SUPPLY & INSTALLATION

2.1 General

2.1.1 Scope

This section contains the requirements for the supply and installation of settlement rods.

The purpose of the settlement rods is to monitor settlement of the base of the preload fills for the proposed Highway 400 - Line 5 Underpass Bridge during preloading and surcharging.

2.1.2 General Procedure

The settlement rod shall be attached to a plate placed on the prepared, stripped subgrade. As embankment construction proceeds, the rod 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.

2.1.3 Locations

The locations of the settlement rods are shown on the drawing Sheet 1 and are given in Table 2.

Table 2 - Installation Locations for Settlement Rods (SR)

<table>
<tr><th>Instrument Type</th><th colspan="2">Location</th><th>Station / Offset *</th></tr>
<tr><td rowspan="8">Settlement Rod (SR)</td><td rowspan="8">Preload/Surcharge Fills for Proposed Line 5 Underpass Bridge</td><td rowspan="4">West Abutment</td><td>9+957 o/s 15m S</td></tr>
<tr><td>9+960 CL</td></tr>
<tr><td>9+963 o/s 15m N</td></tr>
<tr><td>9+967 CL</td></tr>
<tr><td rowspan="4">East Abutment</td><td>10+037 CL</td></tr>
<tr><td>10+040 o/s 15m S</td></tr>
<tr><td>10+043 CL</td></tr>
<tr><td>10+046 o/s 15m N</td></tr>
</table>

Note: * Offset from centreline (CL) of the proposed Line 5 Underpass Bridge.

2.2 Materials

2.2.1 General

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

2.2.2 Plate

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

2.2.3 Rod

The Contractor shall supply a steel pipe Schedule 40 with an outside diameter not less than 25mm (1"), supplied in lengths as required to complete the installation.

The top end of each length of rod shall be threaded to receive a cap. The top of the rod should be angled such that a single survey point can be clearly identified and returned to.

2.2.4 Friction Reducing Sleeve

The Contractor shall supply a friction reducing sleeve consisting of Schedule 40 - 50mm (2") O.D. PVC pipe cut perpendicular to the axis of the pipe.

2.2.5 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 with the following gradation:

MTO Sieve Designation	Percentage Passing
4.5 mm #4	100%
2 mm #10	80% -100%
850 µm #20	20% - 100%
425 µm #40	5% - 40%
150 µm #100	0% - 5%

2.3 Installation

2.3.1 General

The Contractor shall install settlement rods as per the instrument details drawing prior to any fill placement.

2.3.2 Settlement Plate

The settlement plate shall be installed horizontally on the prepared, stripped subgrade.

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

2.3.3 Rod

The rod shall be fixed to the centre of the plate and 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.

2.3.4 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 top of the settlement rod shall extend 25 mm above the top of the friction reducing sleeve at all times.

2.3.5 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 1.5 m above the surrounding fill. This exercise shall be coordinated with the Contract Administrator.

2.3.6 Protective Surround

The CSP, friction-reducing sleeve and sand protective surround shall be extended with the rods.

The settlement rod shall be in the centre 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.

2.3.7 Installation Details

The elevation, easting and northing of the centre 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 plate to the top of rod shall be measured to an accuracy of ± 2mm or better.

2.4 Documentation

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

- Easting and northing;
- Elevation of the base of plate and the top of rod;
- Distance between the base of plate and the top of rod;
- Dates of installation;
- Installation notes / sketches;
- Description of settlement rods, sleeve and plate.

2.5 Coordination with Monitoring

2.5.1 Baseline Readings

The Contractor shall obtain three daily sets of baseline readings on three consecutive days. Elevations shall be surveyed to an accuracy of ± 2 mm or better.

The baseline readings shall be obtained at least 7 days prior to start of placement of preload fills.

2.5.2 Monitoring

Monitoring shall be conducted during construction of the preload fills and during the waiting period after the fill has reached the top of surcharge elevation. The Contractor shall provide safe access to the settlement rods for monitoring, including but not limited to providing traffic control as required and snow clearing in the winter.

Adjustments in the length of any settlement rod shall be coordinated with the Contract Administrator to allow surveying 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.

3.0 PAVEMENT MARKER (PM) - SUPPLY & INSTALLATION

3.1 General

3.1.1 Scope

This Section contains the requirements for the supply and installation of pavement markers.
The purpose of the pavement markers is to monitor settlement of asphalt paved surface of the

Highway 400 roadway in the vicinity of the preloading/surcharging areas, and asphalt paved surface of the approach embankments to the existing 5th Line Underpass Structure.

3.1.2 General Procedure

Pavement markers shall be rigidly affixed so as not to move relative to the asphalt pavement surface to which they are attached.

3.1.3 Locations

The locations of pavement markers are shown on the drawing Sheet 1 and are given in Table 1.

3.2 Materials

3.2.1 General

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

3.2.2 Steel Markers

The Contractor shall supply hardened steel markers with an exposed convex head, similar to surveyor's PK nails, treated or coated to resist corrosion. The steel markers shall have a minimum diameter of 12mm and have sufficient length for anchoring in the pavement and to withstand the weather conditions and effects of traffic.

The exposed nail head shall be equipped with reflective paint or reflective tape to allow for measurements with level survey equipment.

3.3 Installation

3.3.1 General

Traffic shall be managed by the Contractor using short term lane closures in accordance with the Ontario Traffic Manual (OTM), Book 7.

3.4 Documentation

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

- Easting, northing and elevation;
- Dates of installation;
- Installation notes / sketches.

3.5 Coordination with Monitoring

3.5.1 Baseline Readings

The Contractor shall obtain three daily sets of baseline readings on three consecutive days. Elevations shall be surveyed to an accuracy of ± 2 mm or better.

The baseline readings shall be obtained at least 7 days prior to start of placement of preload fills.

3.5.2 Monitoring

Monitoring shall be conducted during construction of the preload fills and during the waiting period after the fill has reached the top of surcharge elevation. The Contractor shall provide safe access to the pavement markers for monitoring, including but not limited to providing traffic control as required and snow clearing in the winter.

4.0 STRUCTURE POINT (SP) - SUPPLY & INSTALLATION

4.1 General

4.1.1 Scope

This section contains the requirements for the supply and installation of structure points.

The purpose of the structure points is to monitor settlement of the abutments of the existing 5th Line Underpass Structure.

4.1.2 General Procedure

The structure points shall be installed in the structure concrete of the abutments of the existing 5th Line Underpass Structure prior to any preload fill placement.

4.1.3 Locations

The locations of structure points are shown on the drawing Sheet 1 and are given in Table 1.

4.2 Materials

4.2.1 Anchor Bolt

The Contractor shall supply 70 mm long by 12 mm diameter stainless steel Hex expansion anchor or equivalent for the installation of structure point in pre-drilled hole in the structure concrete.

4.3 Installation

4.3.1 General

The structure point shall be installed as per the attached instrument details drawing.

Traffic control, if required, shall be managed by the Contractor using short term lane closures in accordance with the Ontario Traffic Manual (OTM), Book 7.

4.3.2 Pre-drilled Holes

The anchor bolt shall be installed in a hole pre-drilled on the outside walls of the abutment structure. The hole will be drilled either horizontally or vertically and shall be clear of any drill powder or loose overbreak materials prior to installation of the anchor bolt.

4.3.3 Anchor Bolt

The anchor bolts shall be inserted into the pre-drilled holes while maintaining contact with the inside wall. The bolts shall be completely fixed in the material after tightening.

4.4 Documentation

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

- Date of installation;
- Location on abutment, northing and easting;
- Elevation of the top of structure point;
- Installation notes / sketches / photographs.

4.5 Coordination with Monitoring

4.5.1 Baseline Readings

The Contractor shall obtain three daily sets of baseline readings on three consecutive days. Elevations shall be surveyed to an accuracy of ± 2 mm or better.

The baseline readings shall be obtained at least 7 days prior to start of placement of preload fills.

4.5.2 Monitoring

Monitoring shall be conducted during construction of the approach fills and during the waiting period after the fill has reached the top of surcharge elevation. The Contractor shall provide safe access to the structure points for monitoring, including but not limited to providing traffic control as required and snow clearing in the winter.

5.0 DECOMMISSIONING OF INSTRUMENTS

5.1 General

The Contractor shall decommission all the settlement rods, pavement markers and structure points at the end of the waiting period unless advised otherwise.

6.0 PAYMENT

6.1 Basis of Payment

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

<u>MONITORING PROGRAM</u> - Item No.

Special Provision

1.0 GENERAL

Foundation Monitoring Consultant; Services, Deliverables and Records; and the Foundation Monitoring Plan apply to all the Instrumentation Monitoring. Instrumentation monitoring is required for the following items:

- Settlement Rods (SR)
- Pavement Markers (PM)
- Structure Points (SP)

The instrumentation monitoring services include:

- Requirements for data collection, data reduction and reporting;
- Adherence to criteria used to assess the foundation performance based on the monitoring data collected from the instrumentation installed by others.

1.1 Foundation Monitoring Consultant

It is understood that the Town of Bradford - West Gwillimbury (TBWG) will retain a Foundation Monitoring Consultant to carry out the monitoring and data interpretation services for the project.

1.2 Services, Deliverables and Records

The Foundation Monitoring Consultant shall:

- Review reports with instrumentation installation details submitted by the Contractor;
- The Contract Administrator or the TBWG to retain an experienced registered land surveyor to monitor the settlement rods (SR), pavement markers (PM) and structure points (SP);
- Reduce settlement data supplied by the surveyor, and prepare reports;
- Transmittal of instrumentation readings and reports to the Contract Administrator;
- Interpret instrumentation readings as needed for the purposes of determining when the settlement below the preload/surcharge fills in the abutment areas is essentially complete;
- Notify the Contract Administrator if critical instrument readings, as specified herein, for any instrumentation are reached. Discuss as soon as possible (within 24 hours) with the Contract Administrator response action(s), and submit a plan of action, to prevent the critical instrument readings from being exceeded.

A monthly progress report shall be provided to the Contract Administrator, TBWG and MTO. The progress report shall discuss the Contractor's operations including installation of instrumentation and progress of fill construction, and a summary of the monitoring that was completed for the month.

2.0 PURPOSE

The purpose of this Monitoring Program is to monitor settlements of the base of the preload/surcharge fills for the proposed Highway 400 - Line 5 Underpass Bridge, the pavement of Highway 400 roadway in the vicinity of preloading/surcharging areas, and the existing 5th Line Structure and the immediate approaches, during the preloading and surcharging of the abutment areas for the proposed Highway 400 - Line 5 Underpass Bridge.

The timing for surcharge/preload removal and installation of the abutment piles shall be controlled by the instrumentation readings.

The instrumentation shall not be decommissioned unless instructed by the TBWG or the CA.

3.0 DRAWINGS

Reference shall be made to the following drawings included in the Contract Document.

- Monitoring Instrument Location Plan (Sheet 1)
- Monitoring Sections (Sheet 2)
- Instrument Details (Sheet 3)

4.0 SUBSURFACE CONDITIONS

The subsurface conditions at the site are described in the following technical memorandum:

- Preliminary Foundation Recommendations, Bridge Foundations and Immediate Approaches, Proposed Highway 400 5th Line Underpass Bridge, West of Bradford, Ontario, Project # P-13-03, dated March 17, 2014, prepared by Thurber Engineering Ltd.

5.0 EQUIPMENT OPERATION

Monitoring shall be conducted during construction of the preload/surcharge fills and during the waiting period after the fill has reached top of surcharge elevation. All monitoring equipment shall be maintained and rendered operational throughout the monitoring period.

Any equipment malfunction shall be investigated and attempts shall be made to remedy the malfunction. Notification of any equipment malfunction and equipment that cannot be repaired shall be given to the Contract Administrator (CA). Documentation of the possible causes and suggested remedial measures shall be forwarded to the Contract Administrator (CA).

6.0 READING SCHEDULE AND FREQUENCY

6.1 The Foundation Monitoring Consultant shall save and archive survey data in electronic and hard copy format.

6.2 Monitoring shall commence immediately after the installation of an instrument. Monitoring is to continue until surcharge/preload removal. The actual length of the monitoring period depends on the construction schedule amongst other factors, and is estimated to be at least 6 months following the completion of fill placement.

6.3 The minimum monitoring frequencies for each instrument along with the anticipated number of readings are given in Table 1. The monitoring frequency is the same for each individual instrument in the table. Instruments shall be read more or less frequently if judged to be required by the Contract Administrator (CA).

Table 1 - Minimum Monitoring Frequency

Stage	Frequencies	Anticipated Number of Readings Per Instrument (**)
Baseline Readings (*)	Three readings on three consecutive days following completion of installation at least 7 days before start of preload fill placement.	3
Just prior to embankment construction	One reading just prior to fill placement.	1
During construction of approach fill	One reading for every 1.5 m fill lift or two readings per week, whichever is greater.	10 to 12
After approach fill has reached top of surcharge to surcharge/preload removal	Weekly for the first month; Biweekly for the second month; Monthly from the third month.	Variable

(*) Baseline Readings: Value of instrumentation readings taken prior to construction to provide a baseline against which all subsequent readings are compared to assess the settlement.

(**) Number of readings may vary.

7.0 INSTRUMENTATION SPECIFIC REQUIREMENTS

7.1 Surveying

The elevations of all instruments shall be surveyed to an accuracy of ± two (2) mm or better and shall be reported to the nearest millimetre.

Surveying for settlement monitoring shall be conducted by a registered surveyor, to be retained by the CA or the TBWG, with appropriate equipment and experience.

7.2 Reporting

An updated processed copy of monitoring data accompanied by a brief interpretation shall be provided to the Contract Administrator within five (5) working days after each set of readings is obtained, unless the trend of the readings is considered unusual (such as accelerated rate of settlement) by the Foundation Monitoring Consultant in which case the subject readings should be reported immediately. The data shall be presented in tabular and graphical form.

As a minimum, the following shall be reported to the Contract Administrator within five (5) days of obtaining a set of readings from all instruments:

- A plot of settlement/heave versus time for each instrument;
- Preload fill height versus time at both abutments;
- Plan view, cross section and profile sketches showing the top of fill location of the embankment, while the settlement readings were being obtained.

7.3 Settlement of Preload/Surcharge Fills

The total settlements are estimated at the base of the preload/surcharge fills including immediate settlement, primary consolidation settlement and secondary compression settlement. Table 2 summarizes the estimated settlements at the base of preload/surcharge fills, against which the monitored settlement at the settlement rods will be compared.

Table 2 – Estimated Settlements at Settlement Rods (SR)

<table>
<tr><th>Instrument Type</th><th colspan="2">Location</th><th>Station / Offset *</th><th>Estimated Settlement (mm)</th></tr>
<tr><td rowspan="8">Settlement Rod (SR)</td><td rowspan="8">Approach Fills for Proposed Line 5 Underpass Bridge</td><td rowspan="4">West Abutment</td><td>9+957 o/s 15m S</td><td rowspan="4">190</td></tr>
<tr><td>9+960 CL</td></tr>
<tr><td>9+963 o/s 15m N</td></tr>
<tr><td>9+967 CL</td></tr>
<tr><td rowspan="4">East Abutment</td><td>10+037 CL</td><td rowspan="4">170</td></tr>
<tr><td>10+040 o/s 15m S</td></tr>
<tr><td>10+043 CL</td></tr>
<tr><td>10+046 o/s 15m N</td></tr>
</table>

Note: * Offset from centreline (CL) of the proposed Line 5 Underpass Bridge.

7.4 Review and Alert Levels

Review and alert levels have been specified for pavement markers and structure points. If the settlement measured exceeds the review levels in Table 3, the Foundation Monitoring Consultant shall immediately inform the Contract Administrator and MTO, and discuss response action(s). The Contractor shall submit a plan of action(s) to prevent the alert level from being reached. All construction work may be continued such that alert levels are not reached unless advised otherwise.

If the settlement measured reaches or exceeds the alert levels in Table 3, the Foundation Monitoring Consultant shall immediately inform the Contract Administrator and MTO, and the Contract Administrator shall instruct the Contractor to stop all construction activities on and within the embankment. No further preload/surcharge embankment construction shall take place on the affected embankment until all of the following conditions are satisfied:

- The cause of the exceedance has been identified and analyzed by the Foundation

Monitoring Consultant;

- Any corrective action(s) deemed necessary by the Foundation Monitoring Consultant has been implemented;
- The Contract Administrator deems it is safe to proceed.

Table 3 – Review and Alert Levels

Instrument Type	Location		Response Levels (mm)	
			Review	**Alert**
Pavement Marker (PM)	Highway 400	West Shoulder	20	-
		East Shoulder		
	Existing 5th Line Underpass Structure	West Approach	20	-
		East Approach		
Structure Point (SP)	Existing 5th Line Underpass Structure	West Abutment	10	20
		East Abutment		

8.0 CONTROL MONITORING LEVELS

8.1 General

The monitoring program will provide input for the timing for removing the preload fills for abutment pile installation.

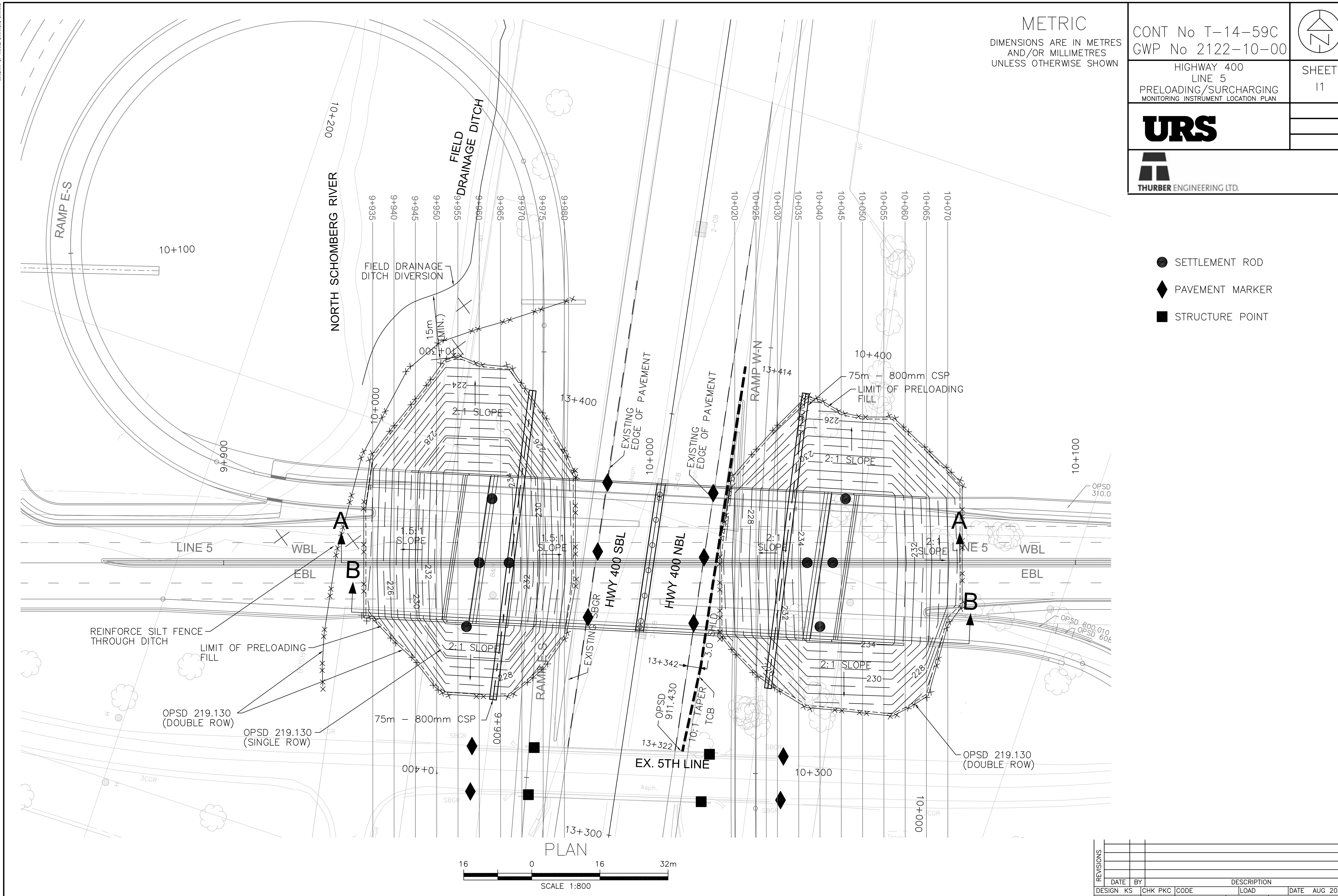
8.2 Stabilization of Settlements due to Primary Consolidation

Settlement data obtained at the settlement rods (SR) allow an approximate assessment of the total settlement due to immediate settlement and primary consolidation, and the approximate time required for settlements due to primary consolidation to stabilize.

The anticipated magnitude of total settlement and the required time for settlements due to primary consolidation to stabilize shall be assessed from settlement rod readings.

9.0 FINAL REPORT

Upon completion of the monitoring program, a final monitoring report shall be issued to the Contract Administrator and MTO. The monitoring results shall be presented in tabular and graphical form as described above for each instrument type. Interpretation of the monitoring readings shall be included in the report.



METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN
CONT No T-14-59C
GWP No 2122-10-00
HIGHWAY 400
LINE 5
PRELOADING/SURCHARGING
MONITORING INSTRUMENT LOCATION PLAN
SHEET
I1
URS
THURBER ENGINEERING LTD.
SETTLEMENT ROD
PAVEMENT MARKER
STRUCTURE POINT
RAMP E-S
NORTH SCHOMBERG RIVER
FIELD DRAINAGE DITCH
FIELD DRAINAGE DITCH DIVERSION
15m (MIN.)
EXISTING EDGE OF PAVEMENT
RAMP W-N
75m – 800mm CSP
LIMIT OF PRELOADING FILL
2:1 SLOPE
1.5:1 SLOPE
LINE 5
WBL
EBL
HWY 400 SBL
HWY 400 NBL
A
B
REINFORCE SILT FENCE THROUGH DITCH
OPSD 219.130 (DOUBLE ROW)
OPSD 219.130 (SINGLE ROW)
OPSD 911.430
10:1 TAPER
TCB
3.0 SHLD
EX. 5TH LINE
OPSD 310.0
OPSD 600.010
PLAN
SCALE 1:800
REVISIONS
DATE
BY
DESCRIPTION
DESIGN KS
CHK PKC
CODE
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DATE AUG 2014
DRAWN MFA
CHK KS
SITE
STRUCT
DWG 1
MINISTRY OF TRANSPORTATION, ONTARIO
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MINISTRY OF TRANSPORTATION, ONTARIO

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

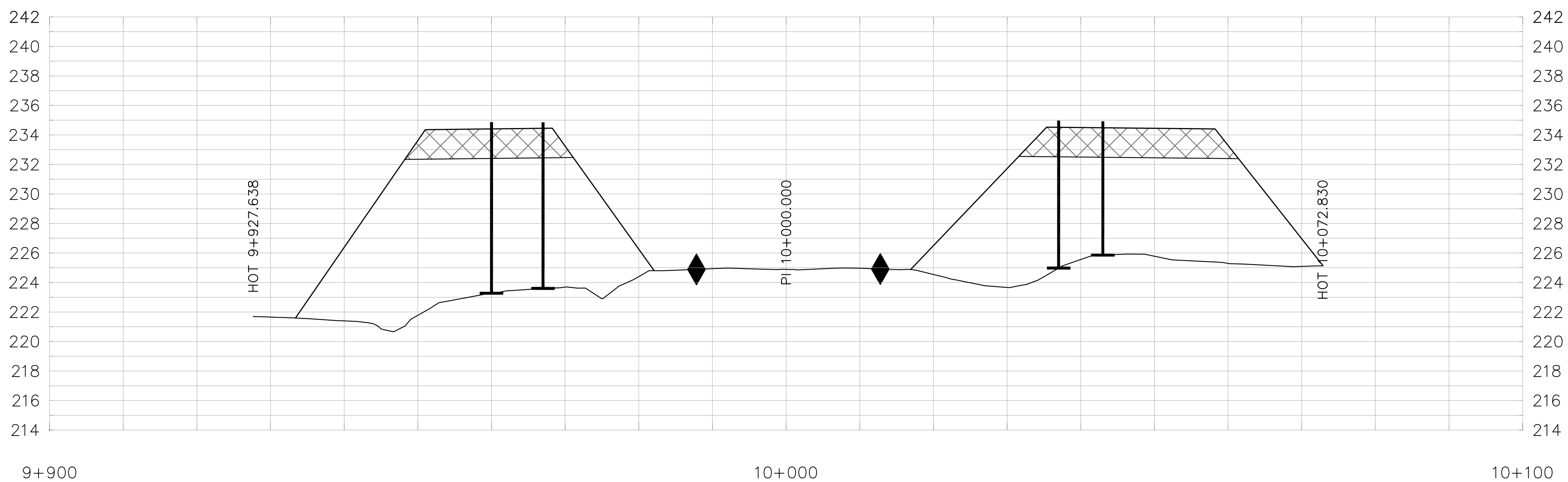
CONT No T-14-59C
GWP No 2122-10-00

HIGHWAY 400
LINE 5
PRELOADING/SURCHARGING
MONITORING SECTIONS

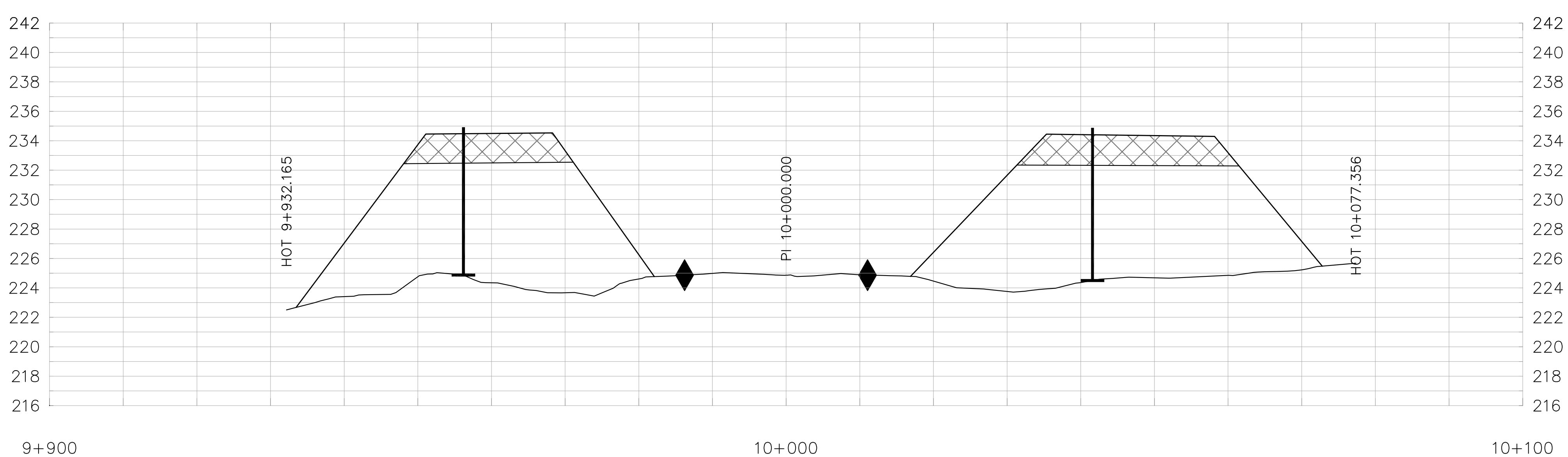
SHEET
12

URS

THURBER ENGINEERING LTD.



SECTION A–A



SECTION B–B

SETTLEMENT ROD

PAVEMENT MARKER

SURCHARGE (2m)

16 0 16 32m H 1:800

8 0 8 16m V 1:400

REVISIONS	DATE	BY	DESCRIPTION

DESIGN KS	CHK PKC	CODE	LOAD	DATE AUG 2014
DRAWN MFA	CHK KS	SITE	STRUCT	DWG 2

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MINISTRY OF TRANSPORTATION, ONTARIO

METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

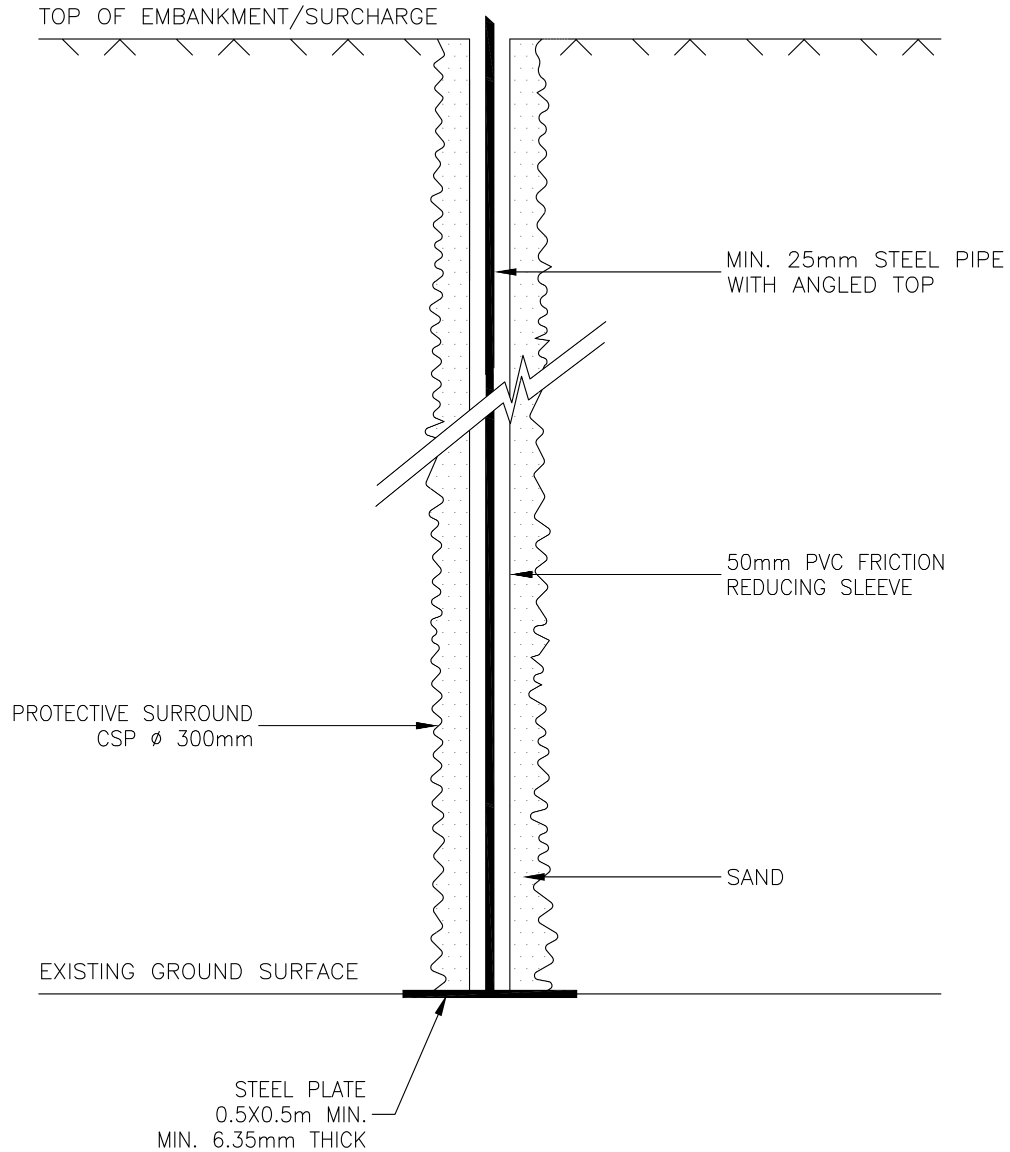
CONT No T-14-59C
GWP No 2122-10-00

HIGHWAY 400
LINE 5
PRELOADING/SURCHARGING
INSTRUMENT DETAILS

SHEET
13

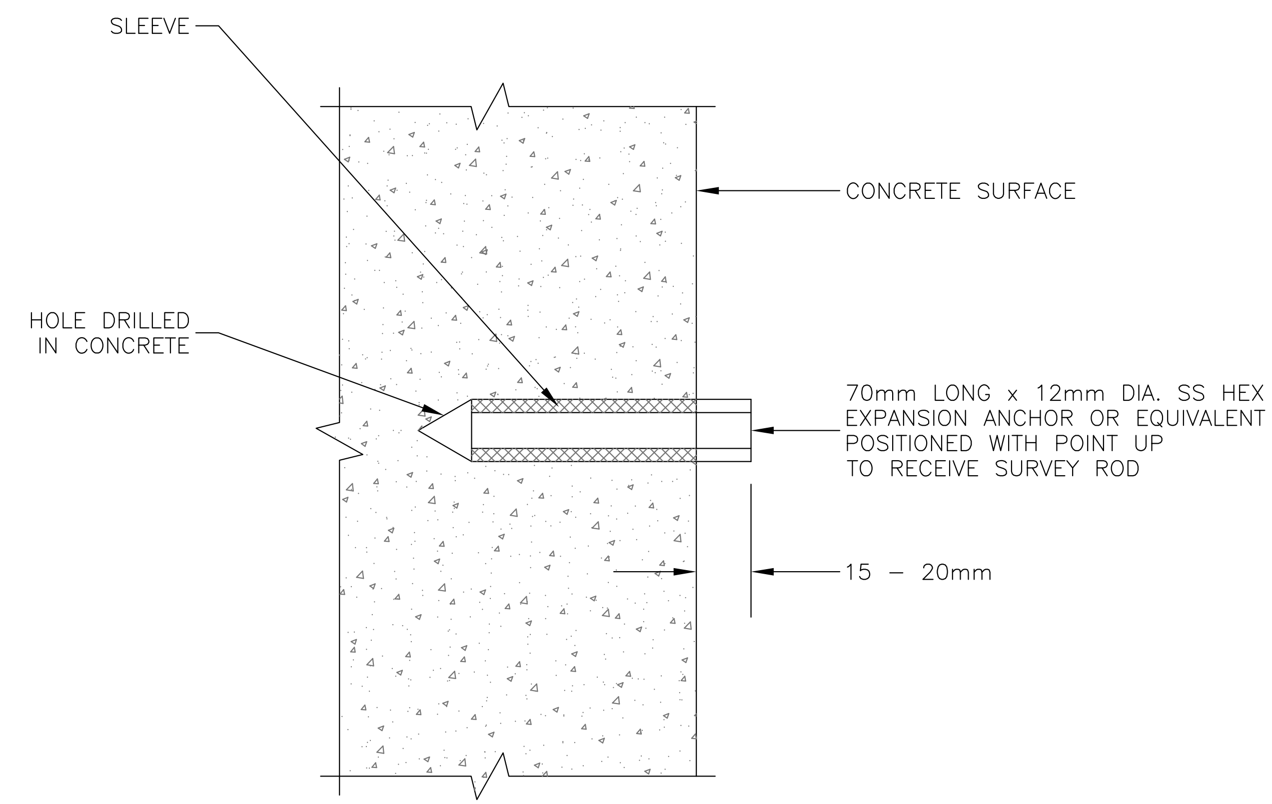
URS

THURBER ENGINEERING LTD.



SETTLEMENT ROD

N.T.S.



STRUCTURE POINT

N.T.S.

REVISIONS	DATE	BY	DESCRIPTION			
DESIGN KS	CHK PKC	CODE	LOAD	DATE AUG 2014		
DRAWN MFA	CHK KS	SITE	STRUCT	DWG 3		

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