

**FOUNDATION INVESTIGATION AND DESIGN REPORT
W-N/S RAMP LESLIE STREET/CNR OVERHEAD
AND PEDESTRIAN OVERPASS
HIGHWAY 401 AND LESLIE STREET INTERCHANGE
CITY OF TORONTO
W.P. 2061-13-00, Site 37-206/5**

GEOCRES NO.: 30M14-440

Report to

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Appendix B	Pedestrian Overpass and Retaining/Noise Barrier Combination Wall Boreholes PB-01 and PB-02; Boreholes W-01 and W-02
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Appendices A and B include:

- Record of Borehole Sheets
- Laboratory Test Results
- Drawings titled “Borehole Locations and Soil Strata”

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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 the proposed replacement of the Highway 401 W-N/S Ramp, Leslie Street/CNR Overhead, and relocation of the pedestrian bridge over the CNR tracks located at the Oriole GO Station in the City of Toronto, Ontario. Both structures are part of the proposed reconstruction and rehabilitation of the Highway 401 and Leslie Street interchange. There is also a proposed retaining/noise barrier combination wall to be located along the south side of the future EBL Collector adjacent to the west approach to the new W-N/S Ramp bridge. This report covers the proposed W-N/S Ramp structure and its approaches, the retaining/noise barrier wall and the pedestrian bridge.

The purpose of this investigation was to explore the subsurface conditions along both structures, and based on the data obtained, to provide borehole location and soil strata drawings with stratigraphic profiles and cross sections (where applicable), records of boreholes, laboratory test results and a written description of the subsurface conditions. A model of the subsurface conditions was developed from the data obtained during the course of the present investigation and selected data from a preliminary investigation carried out previously by others.

Thurber was retained by MMM Group Limited (MMM) to carry out the foundation investigation at this site on behalf of the Ministry of Transportation Ontario (MTO) under Purchase Order No. 2013-E-0032.

During the preparation of this report and in addition to the boreholes drilled, reference has been made to subsurface information contained in previous foundation reports for the general vicinity of the interchange. The titles of these reports are listed as follows:

- Coffey Geotechnics Inc. report titled "Preliminary Foundation Investigation and Design Report, CN Rail Overpass Structure, Highway 401 Rehabilitation from Leslie Street to

Warden Avenue, MTO Central Region”, G.W.P. 2130-01-00, GEOCREs No. 30M14-330, Delcan Corporation, Project TRANETOB01245AA-AB, dated September 30, 2011 (Reference 1).

- Coffey Geotechnics Inc. report titled “Preliminary Foundation Investigation and Design Report, Oriole GO Parking Overpass Structure, Highway 401 Rehabilitation from Leslie Street to Warden Avenue, MTO Central Region”, G.W.P. 2130-01-00, GEOCREs No. 30M14-333, Delcan Corporation, Project TRANETOB01245AA-AC, dated September 30, 2011 (Reference 2).
- Coffey Geotechnics Inc. report titled “Foundation Engineering Assessment Report, Highway 401 and Leslie Street Interchange, Toronto, Ontario”, G.W.P. 2130-01-00, GEOCREs No. 30M14-328, Delcan Corporation, Project TRANETOB01245AA-AA, dated August 10, 2011 (Reference 3).

2 SITE AND PROJECT DESCRIPTION

The site is located at the southwest quadrant of the existing Highway 401 and Leslie Street interchange in the City of Toronto, Ontario. The existing CNR overhead and pedestrian bridges are located at the Oriole GO Station, which is on the west side of Leslie Street and south side of Highway 401. The proposed ramp is to replace the existing ramp which connects the Highway 401 Eastbound Collectors with northbound and southbound traffic on Leslie Street.

It is understood that the proposed W-N/S ramp includes a new three-span bridge with approach embankments, and the pedestrian overpass is to be relocated to the south with a new foundation.

The site lies within an area of industrial and commercial lands and the terrain is generally flat. Overall, this physiographic region is slightly undulating and decreases in elevation in a southerly direction toward Lake Ontario.

Photographs in Appendix C show the general layout of the site and the existing structures at the time of the investigation.

According to the Physiography of Southern Ontario by L.J. Chapman and D.F. Putnam, 1984, the project site is located within the physiographic region known as the South Slope. The South Slope is a smooth and drumlinized till plain that has formed as a result of glacial action and deposition of till materials just south of the Oak Ridges Moraine. The South Slope contains a variety of soils that have developed over till. The depth of the overburden in the general area can be expected to be more than 50 m. Within and adjacent to the Don River valley, the site area is underlain by glacio-lacustrine sands, silts, silty clay and glacial till deposits.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project was carried out from March 17 to April 16, 2015. A total of ten boreholes were drilled for the proposed structures at the site. Six boreholes (numbered R-03 to R-05, R-07 to R-09) were drilled and sampled along the proposed alignment of the new structure and approaches to depths ranging from 23.3 to 30.8 m (Elevations 114.1 to 117.5m). Two boreholes (numbered PB-01 and PB-02) were drilled near each end of the proposed, relocated pedestrian bridge to 23.2 and 26.3 m depths (Elevations 117.5 and 115.8 m), respectively. Boreholes R-01 and R-02 were located close to the westerly limit of the proposed retaining wall alignment.

Boreholes R-04, R-05, R-07, PB-02 and R-09 were originally positioned for a “short 3-span” bridge configuration based on information provided by MMM. Subsequent to completion of these boreholes, information from MMM indicated that a “long 3-span” configuration was instead adopted by Metrolinx. As a result of the significance increase in length of the mid-span, Borehole R-07 is now approximately 15 m away from the proposed location of Pier 2.

Reference has been made to Boreholes W3, 11 and 14, drilled during the previous investigations conducted in 1964 and 2009 (References 1, 2 and 3) by others. These boreholes are included in Appendix A.

The site investigation and field testing for the retaining/noise barrier combination wall was carried out on November 30, 2015 when two boreholes, numbered W-01 and W-02, were drilled and sampled near the proposed wall alignment. The boreholes were advanced to depths of 9.6 m to 9.8, or Elevations 143.2 m to 145.0 m.

The approximate locations of the boreholes drilled for the CNR overhead and the retaining wall during the current investigation are shown on a Borehole Locations and Soil Strata Drawing in Appendix A. The approximate locations of the boreholes drilled for the pedestrian bridge are presented on a Borehole Locations and Soil Strata Drawing in Appendix B. The coordinates and elevations of the boreholes are given on the drawings and on the individual Record of Borehole Sheets in Appendices A and B.

The borehole locations were initially established in the field by Thurber relative to existing site features. Utility clearance was obtained at all borehole locations prior to drilling.

During the current investigation, a track mounted D54 drill rig was used in conjunction with hollow-stem augers to advance the boreholes. Soil samples were obtained at selected intervals using a 50 mm nominal diameter split spoon sampler in conjunction with the Standard Penetration Test (SPT).

In addition to the SPT samples, relatively undisturbed samples of cohesive soils were collected from twelve thin wall Shelby tubes advanced at selected depths in most boreholes except in

Boreholes R-05 and R-07. The in situ shear strength of the firm cohesive soils was also assessed using an MTO 'N' size shear vane.

Borehole R-03 was supplemented by a dynamic cone penetration testing (DCPT) conducted from the base of the sampled borehole until practical refusal to dynamic cone advance.

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 five selected boreholes to permit longer term groundwater level monitoring. The completion details of the piezometers and boreholes are summarized in Table 3.1.

Table 3.1 – Piezometer and Borehole Completion Details

Borehole Number	Piezometer Tip Depth / Elevation (m)	Completion Details
R-01	None installed	Backfilled with bentonite holeplug and auger cuttings to surface.
R-02	23.2/122.5	Backfilled with filter sand from 24.8 to 19.2 m, bentonite holeplug from 19.2 to 15.2 m, bentonite holeplug and auger cuttings from 15.2m to ground surface.
R-03	None installed	Backfilled with bentonite holeplug and auger cuttings to surface.
R-04	15.5/127.7	Borehole caved from 27.6 to 15.5 m. Backfilled with filter sand from 15.5 to 10.4 m, bentonite holeplug from 10.4 to 7.9 m, bentonite holeplug and auger cuttings from 7.9 m to ground surface.
R-05	None installed	Backfilled with bentonite holeplug and auger cuttings to surface.
R-07	None installed	Backfilled with bentonite holeplug and auger cuttings to surface.
R-08	26.1/115.9	Backfilled with filter sand from 26.1 to 22.3 m, bentonite holeplug from 22.3 to 19.9 m, bentonite holeplug and auger cuttings from 19.9 m to ground surface.
R-09	27.7/116.2	Borehole caved in from 29.2 to 27.7 m. Backfilled with filter sand from 27.7 to 24.1 m, bentonite holeplug from 24.1 to 21.9 m, bentonite holeplug and auger cuttings from 21.9 to 1.5 m, then bentonite holeplug from 1.5 m to ground surface.
PB-01	22.9/117.8	Backfilled with filter sand from 23.4 to 18.9 m, bentonite holeplug from 18.9 to 15.2 m, bentonite

Borehole Number	Piezometer Tip Depth / Elevation (m)	Completion Details
		holeplug and auger cuttings from 15.2 to 2.1 m, then bentonite holeplug from 2.1 m to ground surface.
PB-02	None installed	Backfilled with bentonite holeplug and auger cuttings to surface.
W-01	None installed	Backfilled with bentonite holeplug and auger cuttings to 0.2 m, then asphalt to ground surface.
W-02	9.2/143.8	Backfilled with filter sand from 9.8 to 5.2 m, bentonite holeplug from 5.2 to 4.6 m, bentonite holeplug and auger cuttings from 4.6 to 0.3 m, sand from 0.3 to 0.1 m, cement from 0.1 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 to determine the plasticity characteristics. The results of the laboratory testing are summarized on the Record of Borehole sheets and figures included in Appendices A and B.

Three specimens were prepared from thin wall Shelby tube samples obtained in Boreholes R-04, R-08 and R-09 for one-dimensional oedometer (consolidation) tests. The detailed results are shown in Appendix A.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets in Appendices A and B. Details of the encountered soil stratigraphy are presented in these records and on the “Borehole Locations and Soil Strata” drawings in Appendices A and B. General description of the stratigraphy is given in the following paragraphs. The factual information established at the borehole locations governs any interpretation of the site conditions.

5.1 CNR Overhead and Approaches (Boreholes R-01 to R-05, R07 to R-09, PB-02)

Boreholes R-03 to R-05, R-07 to R-09 and PB-02 were drilled near the proposed CNR overhead foundation elements and approaches. Boreholes R-01 and R-02 were drilled along the proposed retaining/noise barrier wall alignment. Records of boreholes, laboratory testing results and stratigraphic drawings are contained in Appendix A.

In general, the stratigraphy along the alignment of the new bridge and its approaches typically consists of surficial topsoil, and sand and silt/silty clay fill overlying native, compact sands and silts. Occasional layers of organics and clayey silt mixed with organics

were noted below the fill. A deposit of very soft to firm silty clay was contacted below the sandy silt to silty sand layer. The silty clay is underlain by very dense sandy silt to silty sand till. The groundwater level is typically at or within 2 m depth of the top surface of native soils.

5.1.1 Topsoil

Topsoil was encountered surficially in all of the boreholes except in Borehole R-05. The thickness of the topsoil ranged from 50 to 200 mm. The topsoil thickness may vary between and beyond the borehole locations, and the data is not intended for the purpose of estimating quantities.

5.1.2 Sand and Silt Fill

A layer of brown to grey sand and silt to silty sand fill was contacted below the topsoil in Boreholes R-03, R-04, R-07, R-08, R-09, PB-02 and surficially in Borehole R-05. The cohesionless fill typically contains trace gravel, trace to some clay and occasional rootlets. Occasional cobbles were inferred within the sand and silt fill near 2.9 m depth in Borehole R-09. The thickness of this fill ranges from 1.4 to 4.2 m. The depth to the base of the sand and silt fill varied from 1.6 to 4.3 m (Elevations 137.7 to 143.3 m).

The SPT 'N' values measured in the sand and silt fill typically ranged from 13 to 41 blows per 0.3 m of penetration indicating a compact to dense condition. SPT 'N' values of 3 to 8 blows per 0.3 m of penetration indicate the presence of very loose to loose zones in Boreholes R-05, R-08 and PB-02. In Borehole R-09, SPT 'N' values of 55 to 60 blows per 0.3 m of penetration were measured indicating very dense conditions. The moisture content in the sand and silt fill ranged from 9% to 23%.

Samples of sand and silt fill were subjected to laboratory gradation analysis. Grain size distribution curves for the sand and silt fill samples are presented on the Record of Borehole sheets included in Appendix A and on Figure A1 of Appendix A. The results of the laboratory tests are summarized as follows:

Soil Particles	Percentage (%)
Gravel	0 to 11
Sand	53 to 59
Silt	26 to 38
Clay	4 to 10

5.1.3 Silty Clay Fill

A layer of silty clay fill containing some to with sand, trace gravel and occasional rootlets was contacted below the topsoil in Boreholes R-01 and R-02 and below the sand and silt fill in Borehole R-03, at 1.6 m depth. The silty clay fill was brown to grey in colour. The thickness of the silty clay fill ranged from 1.9 to 2.5 m. The depth to the base of the silty clay fill was 2.1 m (Elevations 146.0 and 143.6 m) in Boreholes R-01 and R-02, respectively, and 4.1 m (Elevation 140.8 m) in Borehole R-03.

SPT 'N' values in the silty clay fill varied from 9 to 31 blows per 0.3 m of penetration, indicating a stiff to hard consistency. The moisture contents ranged from 11% to 21%.

Two samples of silty clay fill were subjected to laboratory gradation analysis. Grain size distribution curves for the silty clay fill samples are presented on the Record of Borehole sheets included in Appendix A and on Figure A2 of Appendix A. The results of the laboratory tests are summarized as follows:

Soil Particles	Percentage (%)
Gravel	0
Sand	31 to 32
Silt	34 to 39
Clay	30 to 34

5.1.4 Organics and Clayey Silt mixed with Organics

Layers of organics and clayey silt mixed with organics and occasional rootlets were encountered below the fill in Boreholes R-04 and R-09, at 2.0 and 4.1 m depths, respectively. These layers were brown to dark brown in colour, and had thicknesses of 300 mm in Borehole R-04 and 1.5 m in Borehole R-09.

An SPT 'N' value of 19 blows per 0.3 m of penetration, indicating a very stiff consistency, was measured in Borehole R-09. The moisture contents of samples from these layers were 15% and 17%.

5.1.5 Sands and Silts

Native brown to grey sand and silt, sandy silt and silty sand to sand were contacted below the fill and organics at depths ranging from 2.1 to 5.6 m in Boreholes R-02 to R-05, R-07 to R-09, and PB-02. These cohesionless soils contain trace to some gravel and clay. Occasional cobbles were inferred in Boreholes R-01 and R-03. The thickness of the sand and silt layers varied from 1.3 to 5.2 m. The depth to the base of the sands and silts ranged from 4.1 to 10.8 m (Elevations 133.1 to 141.6 m).

Layers of sand with trace to some silt and clay, trace gravel were contacted immediately below the fill in Boreholes R-06 and R-07 at 4.1 and 2.3 m depths, respectively. In Borehole R-03, a 4.6 m thick layer of sand was encountered at 22.4 m depth immediately above the glacial till.

Sand and silt interlayers were encountered within the silty clay deposit at 5.6 m depth in Borehole R-01, at 15.2 m depth in Borehole R-03, and at 17.1 m depth in Borehole R-06. The thickness of these interlayers within the silty clay deposit ranged between 1.1 and 1.6m.

SPT 'N' values of the sands and silts ranged from 6 to 53 blows per 0.3 m of penetration, with most values lying between 10 and 30 blows, indicating a typically compact state. Lower 'N' values of 6 to 8 blows and higher 'N' values of 31 to 53 blows indicate the presence of loose zones and dense to very dense zones, respectively. The moisture contents varied from 7% to 25%.

Samples of the sands and silts were subjected to laboratory gradation analysis. Results of the tests are presented on the Record of Borehole sheets included in Appendix A and on Figures A3, A4 and A5 of Appendix A. The results of the laboratory test are summarized as follows:

Soil Particles	Sandy Silt/Silty Sand Percentage (%)	Sand Percentage (%)	Silt Percentage (%)
Gravel	0 to 11	0	0
Sand	39 to 61	83 to 86	6 to 11
Silt	21 to 57	10 to 14	73 to 90
Clay	4 to 12	3 to 4	4 to 16

5.1.6 Silty Clay

An extensive deposit of brown to grey silty clay was encountered below the fill at 2.1 m depth in Borehole R-01 and below the sand and silt layers in the remaining boreholes at depths ranging from 2.1 to 10.8 m. The silty clay contains some sand to with sand and trace gravel with occasional sand seams. The thickness of the silty clay ranged from 10.1 to 16.3 m. The depths to the base of the silty clay ranged from 12.2 to 24.4 m (Elevations 119.5 to 135.9 m).

SPT 'N' values of 9 to 25 blows per 0.3 m of penetration, indicating a stiff to very stiff consistency, were measured within the upper 2 to 3 m of the silty clay in Boreholes R-01, R-02 and R-08. The SPT 'N' values measured in the silty clay ranged from 0 to 10 blows per 0.3 m of penetration with most values lying between 1 and 8 blows. In situ vane

testing indicated that the undrained shear strength ranges from 30 to 50 kPa. This data indicated that the silty clay has a typically soft to firm consistency with occasional stiff zones. Locally in Boreholes R-02, R-04, R-08 and R-09 below about 15 to 20 m depths, the situ vane testing indicated that the undrained shear strength ranges from 55 to 80 kPa corresponding to a stiff consistency. The moisture contents of the silty clay ranged from 10% to 43%.

Samples of silty clay were subjected to gradation analysis and Atterberg Limits testing. Grain size distribution results are presented on the Record of Borehole sheets of Appendix A and in Figures A6 to A8 of Appendix A. Atterberg Limits test results are shown on the Records of Boreholes and also presented on Figures A11 to A13 of Appendix A. The results of the laboratory test are summarized as follows:

Soil Particles	Percentage (%)
Gravel	0 to 5
Sand	0 to 38
Silt	27 to 51
Clay	21 to 62

Soil Particles	Percentage (%)
Liquid Limit	18 to 44
Plasticity Index	7 to 25

The results indicate that the silty clay typically has low to medium plasticity (CL to CI). One sample from Borehole R-03 taken at 17.1 m depth (Elevation 127.6) was in the group CL-ML.

The results of oedometer (consolidation) testing conducted on two samples of the silty clay obtained from Borehole R-04 and R-09 are included in Appendix A 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
R-04	10.7 - 11.3	Silty clay (CL)	37.3	18.4	1.01	150	145	0.97	0.37	0.085
R-08	12.2 - 12.8	Silty clay (CI)	52.4	17.1	1.40	183	180	0.98	0.76	0.043
R-09	15.2- 15.8	Silty clay (CI)	39.1	18.1	1.1	195	200	1.02	0.45	0.043

Comparison of the existing and preconsolidation pressures (p_o' and p_c') derived from the test results indicate that the silty clay is typically normally consolidated. The coefficient of consolidation, c_v , recorded during the test was generally in the order of 4.5×10^{-4} to 6.5×10^{-3} 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.

5.1.7 Sandy Silty to Silty Sand Till

Grey sandy silt to silty sand till was encountered below the silty clay in all the boreholes, at depths ranging from 12.2 to 27.0 m. The till contains trace gravel, trace to some clay and occasional inferred cobbles. Occasional sand seams were also noted within the till.

A 1.6 m thick layer of sand was found embedded within the till in Borehole R-06 at 26.7 m depth. A 1.3 m thick layer of silt was contacted within the till at 16.3 m depth in Borehole R-01.

All the boreholes were terminated within the sandy silt to silty sand till at depths ranging from 20.2 to 30.8 m (Elevations 114.1 to 127.9 m).

A dynamic cone penetration test was conducted in Borehole R-03 below 30.6 m depth (Elevation 114.3 m) and terminated upon refusal at 30.8 m depth (Elevation 114.1 m).

Most SPT 'N' values ranged from 78 blows per 0.3 m of penetration to greater than 100 blows for less than 0.3 m of penetration indicating a typically very dense state. SPT 'N' values ranging from 14 to 48 blows per 0.3 m of penetration, indicating compact to dense zones, were noted in Boreholes R-01, R-02, R-04 and R-05 at various elevations. In Borehole R-03, an SPT 'N' value of 8 blows per 0.3 m of penetration indicates a loose zone. The moisture content ranged from 7% to 24%.

Samples of the sandy silt to silty sand till were subjected to laboratory gradation analysis. Test results are presented on the Record of Borehole sheets included in Appendix A and in Figures A9 and A10 of Appendix A. The results of the laboratory test are summarized as follows:

Soil Particles	Percentage (%)
Gravel	0 to 9
Sand	26 to 64
Silt	31 to 55
Clay	5 to 24

Glacial till deposits contain cobbles and boulders which may account for the high SPT 'N' values and resistance to augering.

5.1.8 Groundwater Level

Water levels were observed in the open boreholes upon completion of drilling operations. A total of four standpipe piezometers were installed to monitor water levels after completion of drilling. The water levels measured in the piezometers are summarized in Table 5.2 which also includes water levels observed in the open boreholes upon completion of drilling.

Table 5.2 – Water Level Measurements

Borehole Number	Date	Water Levels		Comment
		Depth (m)	Elevation (m)	
R-01	March 30, 2015	5.5	142.6	Open borehole
R-02	March 31, 2015	5.4	140.3	Piezometer
	April 22, 2015	5.3	140.4	
	June 3, 2015	5.2	140.5	
	June 17, 2015	5.3	140.4	
R-03	March 25, 2015	4.3	140.6	Open borehole
R-04	March 31, 2015	4.5	138.7	Piezometer
	April 22, 2015	4.7	138.5	
	June 3, 2015	4.6	138.6	
	June 17, 2015	4.6	138.6	
R-05	March 19, 2015	5.2	135.6	Open borehole
R-07	March 23, 2015	5.0	135.7	Open borehole
R-08	April 22, 2015	5.9	136.1	Piezometer
	June 3, 2015	3.7	138.3	
	June 17, 2015	5.5	136.5	
R-09	April 22, 2015	5.9	138.0	Piezometer
	June 3, 2015	5.6	138.3	
	June 17, 2015	3.7	140.2	
PB-02	April 15, 2015	7.0	135.1	Open borehole

The groundwater readings presented above are short term observations. Seasonal fluctuations of the groundwater level are to be expected. In particular, the groundwater level may be at higher elevations after the spring snowmelt or after periods of heavy rainfall.

5.2 Pedestrian Overpass

Two boreholes, numbered PB-01 and PB-02, were drilled near each abutment of the proposed pedestrian bridge. Records of boreholes, laboratory testing results and stratigraphic drawings are contained in Appendix B.

In general, the stratigraphy at the pedestrian bridge consists of surficial topsoil overlying a layer of cohesionless fill, which is underlain by native sands and silts. A layer of organics

was contacted below the fill in Borehole PB-01. The sands and silts are underlain by an extensive deposit of soft to firm and occasionally stiff silty clay. Below the silty clay, hard silty clay till was contacted in Borehole PB-01 and very dense sand and silt till was encountered in Borehole PB-02.

5.2.1 Topsoil

Topsoil was encountered surficially in both boreholes. The thickness of the topsoil was 200 and 75 mm, respectively, in Boreholes PB-01 and PB-02.

The topsoil thickness may vary between and beyond the borehole locations, and the data is not intended for the purpose of estimating quantities.

5.2.2 Sand and Silt Fill

A layer of sand and silt to silty sand fill containing trace gravel and some clay was encountered below the topsoil in both boreholes. The thickness of this fill was 2.0 and 4.0m in Boreholes PB-01 and PB-02, respectively. The depth to the base of the fill was 2.2 and 4.1 m (Elevations 138.5 and 138.0 m) in Boreholes PB-01 and PB-02, respectively.

SPT 'N' values in the fill ranged from 7 to 31 blows per 0.3 m of penetration, indicating a loose to dense state. The moisture content of the fill ranged from 10% to 21%.

A sample of the sand and silt fill was subjected to laboratory gradation analysis. Grain size distribution curve for the fill sample is presented on the Record of Borehole sheets included in Appendix B and Figure B1 of Appendix B. The results of the laboratory test are summarized as follows:

Soil Particles	Percentage (%)
Gravel	6
Sand	54
Silt	30
Clay	10

5.2.3 Organics

A 300 mm thick layer of organics was contacted below the fill at 2.2 m depth in Borehole PB-01. The layer of organics contains roots and rootlets, and was dark brown in colour. The depth to the base of the organics was at 2.5 m (Elevation 138.2 m).

The moisture content in the organics was measured at 26%.

5.2.4 Sands and Silts

Native silty sand to sand and silt was contacted below the layer of organics at 2.5 m depth in Borehole PB-01 and below the fill at 4.1 m depth in Borehole PB-02. These deposits contain trace clay and trace gravel. The thickness of the sands and silts ranged from 3.1 to 4.6 m. The depth to the base of the sands and silts was 5.6 and 8.7 m (Elevations 135.1 and 133.4 m) in Boreholes PB-01 and PB-02, respectively.

SPT 'N' values of the sands and silts typically ranged from 12 to 27 blows per 0.3 m of penetration indicating a compact state. An SPT 'N' value of 6 blows per 0.3 m of penetration, indicating a loose zone, was measured in Borehole PB-02. The moisture content varied from 4% to 27%.

Two samples of the sands and silts were subjected to laboratory gradation analysis. Results of the tests are presented on the Record of Borehole sheets included in Appendix B and on Figure B2 of Appendix B. The results of the laboratory test are summarized as follows:

Soil Particles	Percentage (%)
Gravel	0
Sand	40 to 60
Silt	33 to 58
Clay	2 to 7

5.2.5 Silty Clay

Brown to grey silty clay was encountered below the sands and silts in both boreholes at 5.6 and 8.7 m depths. The silty clay generally contains some to with sand with occasional sand seams. Locally in Borehole PB-01, a 0.9 m thick layer of sandy silt was contacted within the silty clay at 13.3 m depth. The overall thickness of the silty clay deposit was 13.2 to 13.6 m. The depth to the base of the silty clay was 19.2 and 21.9 m (Elevations 121.5 and 120.2 m) in Boreholes PB-01 and PB-02, respectively.

SPT 'N' values ranged between 0 and 14 blows per 0.3 m of penetration with most values lying between 0 and 3 blows. In situ vane testing indicated that the undrained shear strength ranges from 28 to 72 kPa with most values lying between 28 and 47 kPa. This data indicates that the silty clay is typically soft to firm with occasional stiff zones. The moisture content in the silty clay ranged from 10% to 26%.

Samples of the silty clay were subjected to gradation analysis and Atterberg Limits testing. Grain size distribution results are presented on the Record of Borehole sheets in Appendix B and on Figure B3 in Appendix B. Atterberg Limits test results are shown on the Records

of Boreholes and also presented on Figure B5 of Appendix B. The results of the laboratory test are summarized as follows:

Soil Particles	Percentage (%)
Gravel	0 to 3
Sand	24 to 37
Silt	27 to 47
Clay	21 to 46

Soil Particles	Percentage (%)
Liquid Limit	20 to 38
Plasticity Index	9 to 21

The results indicate that the silty clay typically has low (CL) to occasional medium (CI) plasticity.

5.2.6 Silty Clay Till

Grey silty clay till containing trace of sand was contacted below the silty clay at 19.2 m depth in Borehole PB-01 which was terminated within the till at 23.2 m depth (Elevation 117.5 m).

The SPT 'N' values recorded in the silty clay till were greater than 100 blows for less than 0.3 m of penetration indicating a hard consistency. The moisture content of the silty clay till varied from 5% to 21%.

A sample of the silty clay till was subjected to gradation analysis. Grain size distribution results are presented on the Record of Borehole sheets in Appendix B and on Figure B4 in Appendix B. The results of the laboratory test are summarized as follows:

Soil Particles	Percentage (%)
Gravel	0
Sand	5
Silt	65
Clay	30

Glacial till deposits contain cobbles and boulders which may account for the high SPT 'N' values.

5.2.7 Sand and Silt Till

Grey sand and silt till containing some clay, trace gravel and occasional sand layers was contacted below the silty clay at 21.9 m in Borehole PB-02. A 700 mm thick layer of

cobbles and gravel was contacted within the till at 24.4 m depth. Borehole PB-02 was terminated within this till at 26.3 m depth (Elevation 115.8 m).

SPT 'N' values were greater than 100 blows for less than 0.3 m of penetration indicating a very dense state. The moisture content varied from 3% to 9%.

Glacial till deposits contain cobbles and boulders which may account for the high SPT 'N' values.

5.2.8 Groundwater Level

Water levels were observed in the open boreholes upon completion of drilling operations. One standpipe piezometer was installed to monitor water levels after completion of drilling. The water levels measured in the piezometer are summarized in Table 5.3 which includes the water level observed in the open borehole upon completion of drilling.

Table 5.3 – Water Level Measurements

Borehole Number	Date	Water Levels		Comment
		Depth (m)	Elevation (m)	
PB-01	April 22, 2015	5.4	135.3	Piezometer
	June 3, 2015	2.8	137.9	Piezometer
PB-02	April 15, 2015	7.0	135.1	Open borehole

The groundwater readings at this site are short term observations. Seasonal fluctuations of the groundwater level are to be expected. In particular, the groundwater level may be at higher elevations after the spring snowmelt or after periods of heavy rainfall.

5.3 Retaining/Noise Barrier Combination Wall

Two boreholes, numbered W-01 and W-02, were drilled near the alignment of the proposed combination wall which will serve the dual purpose of retaining a slope within its lower portion and as a noise barrier wall within its upper portion. Records of boreholes, laboratory testing results and stratigraphic drawings are provided in Appendix B.

In general, the stratigraphy at the boreholes consists of the pavement structure and surficial cohesionless fill or very stiff to hard native silty clay overlying compact to very dense native sands and silts, which are underlain by hard silty clay. The groundwater level was measured at about 6.5 m depth below existing ground surface.

5.3.1 Pavement Structure and Fill

The pavement structure consists of 125 mm to 150 mm of asphalt overlying 0.6 m to 0.8 m of sand, some gravel. Below the pavement, sand and silt fill containing some clay and

trace gravel was encountered in Borehole W-02. The depth to the base of the pavement structure and fill was 0.9 m (Elevation 153.7 m) and 3.0 m (Elevation 150.0 m) in Boreholes W-01 and W-02, respectively.

In Borehole W-02, the sand fill was in a compact state as indicated by an SPT 'N' value of 19 blows for 0.3 m penetration. 'N' values of 75 and 46 blows per 0.3 m of penetration measured in the sand and silt fill indicated very dense to dense conditions. Measured moisture contents of the fill ranged between 10 and 11 percent.

A sample of the sand and silt fill was subjected to laboratory gradation analysis. Grain size distribution curve for the fill sample is presented on the Record of Borehole sheets included on Figure B6 of Appendix B. The results of the laboratory test are summarized as follows:

Soil Particles	Percentage (%)
Gravel	2
Sand	43
Silt	42
Clay	13

5.3.2 Sand and Silt

A deposit of native sand and silt, trace to some clay and trace gravel was encountered in Borehole W-01 below an upper silty clay layer and in Borehole W-02 below the fill. The thickness of the sand and silt was 4.4 m and 2.6 m in Boreholes W-01 and W-02, respectively. The depth to the base of the sand and silt is between 7.2 m and 5.6 m (Elevation 147.4 m).

SPT 'N' values of the sand and silt ranged from 24 blows per 0.3 m of penetration to greater than 100 blows for less than 0.3 m penetration, indicating a compact to very dense condition. Measured moisture contents varied from 2 to 15 percent.

Two samples of the sand and silt were subjected to laboratory gradation analysis. Results of the tests are presented on the Record of Borehole sheets included in Appendix B and on Figure B7 of Appendix B. The results of the laboratory test are summarized as follows:

Soil Particles	Percentage (%)
Gravel	2 to 3
Sand	35 to 43
Silt	42 to 50
Clay	4 to 21

5.3.3 Silty Clay

The upper silty clay layer encountered in Borehole W-01 was 1.9 m thick with a base at 2.8m depth (Elevation 151.8 m). A lower, grey silty clay deposit containing trace to some sand was encountered below the sand and silt in both boreholes, which were terminated within this deposit at depths of 9.6 m to 9.8 m (Elevations 145.0 and 143.2 m) in Boreholes W-01 and W-02, respectively.

In the upper silty clay, SPT 'N' values ranged from 21 blows for 0.3 m penetration to greater than 100 blows for less than 0.3 m penetration indicating a very stiff to hard consistency. 'N' values, ranging between 58 and 88 blows per 0.3 m of penetration, indicated that the lower silty clay has a hard consistency throughout. Measured moisture contents in the silty clay ranged between 10 and 20 percent.

Samples of the silty clay were subjected to gradation analysis and Atterberg Limits testing. Grain size distribution results are presented on the Record of Borehole sheets in Appendix B and on Figure B8 in Appendix B. Atterberg Limits test results are shown on the Records of Boreholes and also presented on Figure B9 of Appendix B. The results of the laboratory test are summarized as follows:

Soil Particles	Percentage (%)
Gravel	0
Sand	10 to 13
Silt	58 to 63
Clay	24 to 32

Soil Particles	Percentage (%)
Liquid Limit	25
Plasticity Index	8

The results indicate that the silty clay has low (CL) plasticity.

5.3.4 Groundwater Level

Water levels were observed in the open boreholes upon completion of drilling operations. One standpipe piezometer was installed in Borehole W-02 to monitor water levels after completion of drilling. The water level data is summarized in Table 5.4 which includes the water level observed in the open boreholes upon completion of drilling.

Table 5.4 – Water Level Measurements

Borehole Number	Date	Water Levels		Comment
		Depth (m)	Elevation (m)	
W-01	November 30, 2015	dry	-	Open borehole
W-02	November 30, 2015 March 29, 2016	Dry 6.5	- 146.5	Open borehole Piezometer

The groundwater readings at this site are short term observations. Seasonal fluctuations of the groundwater level are to be expected. In particular, the groundwater level may be at higher elevations after the spring snowmelt or after periods of heavy rainfall.

6 MISCELLANEOUS

The borehole locations on site were initially established by Thurber. Underground utility clearances were obtained for the borehole locations prior to drilling. The northing and easting coordinates and elevation at each as-drilled borehole location were provided by MMM surveyors.

The drilling and sampling equipment was supplied and operated by Walker Drilling Ltd. of Utopia, Ontario. The field work was supervised on a full time basis by Ms. Eckie Siu of Thurber.

Laboratory testing was carried out at Thurber's Toronto area, MTO approved, high complexity laboratory.

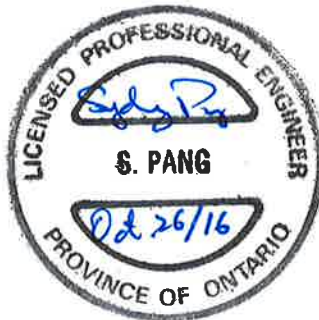
Overall supervision of the field program was conducted by Mr. Stephane Loranger, C.E.T. of Thurber. Compilation of data and preparation of the report were carried out by Ms. R. Palomeque Reyna, P.Eng. and Dr. Sydney Pang, P.Eng.

Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects, reviewed the report.

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**FOUNDATION INVESTIGATION AND DESIGN REPORT
W-N/S RAMP LESLIE STREET/CNR OVERHEAD
AND PEDESTRIAN OVERPASS
HIGHWAY 401 AND LESLIE STREET INTERCHANGE
CITY OF TORONTO
W.P. 2061-13-00, Site 37-206/5**

GEOCRETS NO.: 30M14-440

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 in selecting and designing suitable foundation systems for the W-N/S Ramp Leslie Street/CNR Overhead structure and the relocated pedestrian overpass at the Highway 401 and Leslie Street interchange area in Toronto.

A preliminary General Arrangement (GA) drawing dated July 2015 prepared by MMM indicates that it is proposed to replace the existing W-N/S Ramp with a new triple span bridge and associated approaches. The proposed ramp structure has a curved alignment in plan and consists of steel box girders. It has two 20.0 m long approach spans and a 30.0 m long centre span supported by two piers and two abutments. The proposed bridge approaches will be in the order of 250 to 300 m in length. Based on preliminary cross sections provided by MMM, the fill height along the approaches is up to the order of 8 m above existing ground surface at the abutments. The approach embankments are proposed to be vertically sided and extend to some distances away from the bridge abutments, beyond which the fill would transition to typical sideslopes until the new ramp merges with the Highway 401 eastbound lanes at the westerly limit and intersects with Leslie Street at the easterly limit. The vertical sides would imply the use of RSS walls, SSM retained by conventional walls, or lightweight fill such as EPS blocks. The width of the new ramp deck would be in the order of 12 m.

In addition, MMM also provided profiles and cross-sections of the proposed retaining/noise barrier combination wall. The proposed wall is approximately 120 m in length (Stations 25+000 to 25+120). The lower, retaining portion of the wall will be up the order of 2.5 m in height and will replace an existing concrete toe wall. The surface of the retained fill varies from a horizontal configuration to a maximum inclination of 2H : 1V. Retaining wall panels will be used within the

retained height (lower portion) of the wall. Above the retained height, noise barrier panels will be used.

The discussions and recommendations presented in this report are based on the factual data obtained during the course of this investigation. Reference has also been made to selected data from previous preliminary investigations by others. The plans and sections used for preparation of this report have been provided by MMM.

8 RAMP EMBANKMENTS, BRIDGE AND RETAINING WALL FOUNDATIONS

The subsurface conditions at this site generally consists of surficial sand and silt/silty clay fill overlying native, compact sands and silts. Occasional inclusions of organics and clayey silt mixed with organics were noted below the fill. An extensive deposit of very soft to firm and compressible silty clay was contacted below the sands and silts. The silty clay is underlain by very dense sandy silt to silty sand till. The groundwater level is typically at 4 to 5 m depths below existing ground surface.

Based on the July 2015 preliminary GA drawing, the piers and abutments will be supported on concrete caissons.

8.1 Embankments

The design and performance of new embankments will need to satisfy the requirements stipulated in the MTO document titled “Embankment Settlement Criteria for Design” dated July 2010. The ramp alignment is underlain by compressible foundation soils; in particular the silty clay that will undergo time-dependent (consolidation) settlement under additional loading. It is also noted that there are records of at least one embankment failure during construction of the existing Highway 401 west approach to Leslie Street and Don River, which prompted the construction of a stabilizing berm. Detailed foundation settlement and stability analysis results are presented in Section 10.

Information from MMM indicates that up to 12 months of “waiting time” can be made available during construction after the fill is placed. If a 6 to 8 m high RSS wall is built and allowed to sit for the full duration of this time, the estimated post construction foundation settlement would still be in excess of the maximum permissible values stipulated in the MTO embankment settlement criteria. This settlement would also induce downdrag forces on the abutment caissons.

An overview of the various types of lightweight fill and ground improvement technique has been provided in Coffey Geotechnics Inc. report titled “Preliminary Foundation Investigation Report, Existing Viaduct West of Highway 401 and Leslie Street, Highway 401 Rehabilitation from Leslie Street to Warden Avenue, MTO Central Region”, G.W.P. 2130-01-00, GEOCREC No. 30M14-335, Project TRANETOB01245AA-AE, dated

December 19, 2011 (Reference 4). The lightweight materials considered were extruded polystyrene (XPS), expanded polystyrene (EPS), blast furnace slag, tire derived aggregate (TDA) and expanded clay. Thurber has also considered the potential use of cellular concrete. XPS, TDA and expanded clay have not been approved for use to date by MTO due to environmental and other issues. Both blast furnace slag (unit weight up to 12 to 13 kN/m³) and cellular concrete (unit weight between 4 and 6 kN/m³) are not sufficiently light to be of practical use to minimize settlement at this site. On the other hand, there is time and space constraints associated with this site that prohibit the effective implementation of ground improvement techniques such as preloading/surcharging/wick drains and electro-osmotic treatment. The effectiveness of deep soil mixing will also be limited by the time constraint at this site. In addition, this method requires specialist contractors and is unlikely to be cost-effective for this project.

From a foundation engineering perspective, it is considered that Expanded Polystyrene (EPS) is the preferred material for constructing the higher sections of the embankments. The EPS is a super lightweight fill (unit weight of not greater than 1 kN/m³) that would minimize post construction settlement, minimize downdrag forces on deep foundations, enhance stability of the retained embankment and reduce lateral pressures on the retaining structures. Conventional earth fills may be used elsewhere for lower fill heights along the new ramp where it merges with the existing highway and street lanes.

For embankment design and construction, Section 10 focuses on the foundation engineering aspects of using EPS. The selection of EPS grade must be appropriate for satisfying pavement and traffic loading conditions. The design of EPS embankments should be carried out in accordance with Transportation Research Board (TRB) NCHRP Report 529 entitled “Guideline and Recommended Standard for Geofoam Applications in Highway Embankments”.

The selected ramp design involving the use of EPS will extend from the back of the abutment walls to locations along the ramp alignment where fills of 3 m or lower will be used until the ramp merges with the Highway 401 EB lanes on the west side and Leslie Street SB lanes on the south side.

8.2 Foundation Alternatives

8.2.1 Ramp Bridge

Consideration has been given to the following foundation types for the new ramp bridge:

- Augered caissons (drilled shafts)
- Driven steel H-piles
- Driven steel pipe piles

- Spread footings.

The preliminary GA drawing indicates that concrete caissons are proposed for foundation support to the new bridge at the abutments and piers. Advantages and disadvantages of feasible foundation alternatives are presented in the table in Appendix D.

From a foundation engineering perspective, spread footings are not considered a feasible foundation option for the bridge due to low geotechnical capacity, anticipated time-dependent settlements as a result of consolidation of the underlying compressible silty clay, and the need for relatively deep excavations to found the footings on native soils below the fill. This option is, therefore, not recommended and foundation recommendations are not developed.

Augered caissons (drilled shafts) and driven steel H-piles are technically feasible options for the abutments provided the approach fills immediately behind the abutments consist of EPS (with pavement structure, earth cover and concrete slab, where used) that would minimize downdrag forces on the deep foundation elements. It is our understanding that due to the close proximity of residential developments and in view of the city noise and vibration by-law requirements, pile driving may not be permitted at this site. Driven pipe piles are expected to cause the same level of, or more severe, vibration than H-piles during installation and, therefore, not recommended for this site. Pile caps may be avoided for augered caissons which are considered as the only feasible option for the piers, since there is insufficient space at track level within the CNR/GO property to drive piles and construct pile caps.

From a foundations technical, constructability and cost-effectiveness perspective, the recommended foundations at the piers and abutments for this bridge are augered caissons founded within the very dense sandy silt to silty sand till. Alternatively, steel H-piles driven to achieve resistance in the dense till is a feasible option for the abutments, provided that the noise and vibration requirements can be satisfied.

8.2.2 Retaining Wall for Embankment Fill

For retaining the EPS blocks, shallow strip footings to carry the weight of the cladding wall and the PL-3 barrier at ramp grade may be used in conjunction with an engineered fill pad resting on the subgrade.

Where conventional earth fill such as SSM is used for ramp construction and sloping sides are not feasible due to space restrictions, conventional concrete retaining walls may have to be used. RSS blocks are required to be constrained by proprietary vertical panel walls.

Alternatively, driven H-piles and augered caissons (drilled shafts) are feasible, where required, to provide lateral and vertical geotechnical resistances for the walls.

Based on settlement and stability considerations discussed elsewhere in this report, EPS has been selected for use at this site. The required foundation for the retaining system is strip footings founded on an engineered fill pad to support the cladding wall.

8.3 Augered Caissons (Drilled Shafts) – Ramp Bridge

8.3.1 Axial Resistance

The preliminary GA drawing indicates that the proposed piers and abutments are to be supported on caissons.

Based on results of boreholes advanced during this investigation, augered caissons founded within the underlying very dense sandy silt to silty sand till are feasible to support the piers and abutments. It is recommended that the caissons be drilled to a minimum of 2 m into the underlying “100-blow” sandy silt to silty sand till. Tables 8.1 and 8.2 present the recommended founding depths and elevations for the caissons at the piers and abutments, as well as geotechnical resistances recommended for typical caisson diameters.

Table 8.1 – Founding Levels and Geotechnical Resistances for Augered Caissons at the Piers

Foundation Element	Approx. Caisson Length* (m)	Founding Elevation (m)	Axial Geotechnical Resistance			
			Factored ULS _r (kN)	SLS (kN)	Factored ULS _r (kN)	SLS (kN)
			1.8 m Diameter		2.1 m Diameter	
Pier 1	24	117	11,000	7,000	15,000	8,000
Pier 2	24	117				

* Caisson lengths in the above table are measured from the existing ground surface at approximate Elevations 140 to 141 m.

Table 8.2 – Founding Levels and Geotechnical Resistances for Augered Caissons at the Abutments

Foundation Element	Approx. Caisson Length* (m)	Founding Elevation (m)	Axial Geotechnical Resistance					
			Factored ULS _r (kN)	SLS (kN)	Factored ULS _r (kN)	SLS (kN)	Factored ULS _r (kN)	SLS (kN)
			0.9 m Diameter		1.2 m Diameter		1.5 m Diameter	
East Abutment	24	116	3,000	2,500	4,800	4,000	7,200	6,000
West Abutment	24	117						

- * Caisson lengths in the above table are measured from the proposed underside of cap at approximate Elevation 140 m at the east abutment and at approximate Elevation 141 m at the west abutment.

It is noted, for larger diameter caissons, that it takes greater vertical movement at the base to fully mobilize the available geotechnical resistance. As such, the increase in SLS values (for up to 25 mm settlement) is not necessarily proportional to the increase in the corresponding ULS values.

The use of EPS behind the abutments will practically eliminate the downdrag load on the deep foundations.

8.3.2 Lateral Resistance

Lateral bridge loadings can be geotechnically resisted by the caissons through passive pressure developed along the embedded portion of the shaft.

For lateral resistance design, soil-caisson interaction analyses may be carried out using the coefficient of horizontal subgrade reaction values provided in Table 8.3 below.

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

Fill, Sands and Silts

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

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

where p_{ult} = ultimate lateral resistance mobilized by a caisson, kPa

z = depth of embedment of caisson, m

D = caisson diameter, m

n_h = coefficient related to soil density, kN/m^3

γ' = submerged unit weight of soil, kN/m^3

K_p = passive earth pressure coefficient

Silty Clay

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

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

where C_u = undrained shear strength of cohesive soils, kPa

γ = total unit weight of soil, kN/m^3

D = caisson diameter, m

Table 8.3 – Recommended Geotechnical Parameters for Lateral Resistance Design

Foundation Unit	Borehole Number	Approx. Elevation (m)	Undrained Shear Strength C_u (kPa)	n_h (kN/m ³)	K_p	Unit Weight γ (kN/m ³)	Soil Conditions
West Abutment	R-04	143 to 141	-	3,000	3.0	20	Fill compact
		141 to 136	-	2,500	3.0	10*	Silty Sand compact to dense
		136 to 123	40	-	-	19	Silty Clay firm
		below 123	-	10,000	4.0	12*	Sand and Silt Till very dense
Pier 1	R-05 R-07	141 to 138	-	3,000	3.0	20	Sand fill, compact to dense
		138 to 134	-	2,500	3.0	10*	Sandy Silt to Sand compact
		134 to 120	25	-	-	19	Silty Clay, very soft to firm
		below 120	-	10,000	4.0	12*	Sand and Silt Till very dense
Pier 2	R-07 R-09	141 to 138	-	3,000	3.0	20	Sand fill, loose to compact
		138 to 135	-	2,500	3.0	10*	Sandy Silt to Sand compact
		135 to 120	25	-	-	19	Silty Clay very soft to firm
		below 120	-	10,000	4.0	12*	Sand and Silt Till very dense
East Abutment	R-09	144 to 140	-	3,000	3.0	21	Sand fill, compact to very dense
		140 to 133	-	2,500	3.0	10*	Sandy Silt to Silt

Foundation Unit	Borehole Number	Approx. Elevation (m)	Undrained Shear Strength C_u (kPa)	n_h (kN/m ³)	K_p	Unit Weight γ (kN/m ³)	Soil Conditions
							compact
		133 to 120	40	-	-	19	Silty Clay firm to stiff
		below 120	-	10,000	4.0	12*	Sand and Silt Till very dense

* Buoyant unit weight for cohesionless soil below the water table

The spring constant, K , for analysis may be obtained by the expression, $K = k_s \times d_z \times D$ (kN/m), where k_s is the coefficient of horizontal subgrade reaction (kN/m³), D is the caisson diameter (m), d_z is the length (m) of the caisson segment or element used in the analysis. The ultimate lateral resistance on any one segment of caisson, P_{ult} , may be obtained from the expression, $P_{ult} = p_{ult} \times d_z \times D$. This represents the ultimate load at the contact between the soil and the caisson above which additional load cannot be supported at greater displacements.

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

Where a caisson 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:

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

where D is the diameter of the caisson, and spacing is measured centre to centre.

Where a caisson 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:

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

Intermediate values may be obtained by interpolation.

8.3.3 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 the very dense sandy silt to silty sand till. An NSSP addressing this issue must be included in the contract documents to alert the bidders. Suggested wording for such an NSSP is provided in Appendix E.

The resistance values provided in Tables 8.1 and 8.2 above are based on end bearing and shaft friction, assuming that the walls and base of each caisson are cleaned of loose, soft or otherwise disturbed material prior to placement of concrete. Based on piezometric measurements to date, the groundwater level was just below the interface of the fill and native soil. Soil sloughing, squeezing of the soft clay and water seepage will occur in unsupported holes. Construction of caissons will require the use of temporary steel liners to support the caisson sidewalls and to provide seepage cut-off where required. Concrete should be placed with a minimum delay after each caisson is drilled, cleaned and inspected. Consideration should be given to using the tremie technique to place concrete inside the caisson hole. Suggested wording for an NSSP addressing caisson construction is provided in Appendix E.

8.4 Driven H-Piles – Ramp Bridge

8.4.1 Axial Resistance

The abutments may also be supported on steel H-piles driven to practical refusal within the underlying very dense sandy silt to silty sand till. 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 boulders. The pile tips should, therefore, be reinforced to enhance driving (see Section 8.4.3).

The elevations at which the H-piles are anticipated to develop the required resistance, and the vertical, factored geotechnical resistances at Ultimate Limit States (ULS) and geotechnical resistances at Serviceability Limit States (SLS) for the two pile sections driven into the glacial till are presented in Table 8.4 below.

Table 8.4 – Estimated Pile Tip Elevations and Axial Resistances of H-Piles

Foundation Unit	Borehole Number	Estimated Pile Tip Elevation	HP 310 x 110		HP 360 x 132	
			Factored ULS (kN)	SLS (kN)	Factored ULS (kN)	SLS (kN)
West Abutment	R-04	117	1,400	1,200	1,600	1,400
East Abutment	R-09	116				

The SLS values correspond to a pile settlement up to 25 mm.

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

The pile tip elevations shown in Table 8.4 should be used for estimating purposes only. The actual pile tip elevations will be controlled during pile driving as described in Section 8.4.3 Pile Installation.

8.4.2 Lateral resistance

For pile lateral resistance design, soil-pile interaction analyses may be carried out using the coefficient of horizontal subgrade reaction values provided in Table 8.3.

The methodology outlined in Section 8.3.2 above may be used to estimate the lateral geotechnical resistance of a pile by substituting the caisson diameter, D with the pile width, B.

8.4.3 Pile installation

All piles shall be installed in accordance with OPSS 903.

Pile driving must be controlled in accordance with Standard SS 103-11 (Hiley Formula) and an ultimate pile resistance must be specified by the designer. The Hiley Formula does not need to be used until the pile tip is within 2 m above the design tip elevation. The appropriate pile driving note to be shown on the contract drawing is “Piles to be driven in accordance with Standard SS103-11 using an ultimate geotechnical resistance of “R” kN per pile where “R” must have a minimum value of twice the design load at ULS, but must be driven below Elevation 120 m”.

Glacially derived soils inherently contain cobbles and boulders. In order to minimize the risk of pile damage while driving through boulders, cobbles and harder/denser zones in the glacial tills to achieve the required tip elevations and soil resistance, it is recommended that the pile tips be reinforced with protection devices such as the Titus Steel Standard Points for H Piles or approved equivalent.

8.5 Frost penetration

The design frost penetration depth is 1.2 m for this site.

8.6 Impact of Foundation Construction on Existing Bridge

Construction of new foundations must be carried out using procedures that minimize the potential for undermining or disturbing the existing foundations. It is understood that the existing bridge is supported on 12 BP 53 (HP 310 x 79 equivalent) piles based on limited documentation from Geocres information.

Monitoring of the existing bridge foundations during new foundation installation is recommended. As a minimum, a monitoring program requiring the contractor to establish and monitor movement of reference points over each foundation unit of the existing bridge, as well as monitoring of vibration, should be implemented during installation of the new foundations. Pre-construction condition survey should be carried out for the existing structure. Suggested wording for an NSSP to this effect is included in Appendix E.

8.7 Retaining Structures

The selected ramp embankment design involves the use of EPS to be retained by cladding walls, between the abutments and the locations where the embankments are greater than 3m in height. The following Section 8.7.2 provides foundation recommendations for the cladding wall footings. Beyond the EPS zones, the embankments will consist of granular or approved earth fill formed at inclinations of 2H : 1V or flatter (see Section 10). Section 11 provides foundation recommendations for the section of retaining/noise barrier combination wall. If conventional concrete retaining walls are used, the following Section 8.7.1 provides foundation recommendations for footing design.

8.7.1 Conventional Concrete Wall Footings

Boreholes R-01 and R-02 indicate that the thickness of existing surficial fill is about 2.1 m and is comprised of silty clay of stiff to very stiff consistency. It is important to note that fill has an inherently random nature and composition. It is, therefore, recommended that retaining wall footings, where used to retain conventional earth fill, be extended below this fill and founded on the underlying native, stiff silty clay or compact sandy silt. The recommended founding depths and elevations for spread footings are given in Table 8.5.

Table 8.5 – Recommended Highest Founding Depths and Elevations

Borehole	Footing on Native Undisturbed Soil		
	Depth below existing ground surface (m)	Founding Elevation (m)	Native Soil
R-01	2.3	145.8	Stiff Silty Clay
R-02	2.3	143.4	Compact Sandy Silt

Roadway protection (temporary shoring) may be required at some locations during new footing construction.

Provided a minimum footing width of 1.5 m is maintained, retaining wall footings founded on the stiff silty clay or compact sandy silt at the levels shown in Table 8.5 may be designed for the following values:

- Factored geotechnical resistance at Ultimate Limit States (ULS) of 250 kPa
- Geotechnical resistance at Serviceability Limit States (SLS) of 175 kPa.

The geotechnical resistances quoted above are for concentric, vertical loads only. In the case of eccentric or inclined loading, the geotechnical resistance must be calculated as illustrated in the CHBDC 2014 Clauses 6.10.3 and 6.10.4.

The geotechnical SLS resistance values given above are based on an estimated total settlement not exceeding 25 mm. This settlement is expected to be substantially complete by the end of construction.

The sliding resistance of mass concrete placed on the native, stiff silty clay or compact sandy silt may be computed on the basis of an ultimate coefficient of friction at the interface of 0.4. This is an “ultimate” value and requires a degree of sliding movement to occur to fully mobilize the resistance.

All footings should be provided with a minimum of 1.2 m of earth cover, or its thermal equivalent, over the footing base (founding elevation) as protection against frost action.

8.7.2 Cladding Wall Footings

For sections of the ramp where EPS constitutes the embankment fill, the existing surficial fill consists of compact sand and silt and/or very stiff to hard silty clay. The footings for carrying the weight of the cladding wall and PL-3 barrier at ramp grade may be founded at 1.2 m depth, or below, on a minimum 500 mm thick engineered fill pad resting on the compact sand and silt fill. The engineered fill pad must consist of OPSS.PROV 1010 Granular A compacted to 100% of its Standard Proctor Maximum Dry Density (SPMDD)

at a moisture content within 2% of optimum. The engineered pad must laterally extend at least 500 mm beyond the limits of the footing.

Provided a minimum footing width of 1 m is maintained, these cladding wall footings founded on a fill pad resting on compact sand and silt fill subgrade as outlined above may be designed for the following values:

- Factored geotechnical resistance at Ultimate Limit States (ULS) of 225 kPa
- Geotechnical resistance at Serviceability Limit States (SLS) of 150 kPa.

The geotechnical resistances quoted above are for concentric, vertical loads only. In the case of eccentric or inclined loading, the geotechnical resistance must be calculated as illustrated in the CHBDC Clause 6.7.3 and Clause 6.7.4.

The sliding resistance of precast concrete placed on the compacted Granular A pad may be computed on the basis of an ultimate coefficient of friction at the interface of 0.4.

8.7.3 Footing Construction

The bases of the footing excavations should be inspected by a geotechnical engineer to confirm that the founding subgrade is the native, undisturbed silty clay or sandy silt for the conventional footings, and the sand and silt fill for the EPS cladding footings. The subgrade should conform to the design requirements and be adequately prepared to receive concrete. Concrete or mud slab should be placed within 24 hours following completion of excavation to prevent deterioration of the approved subgrade. The mud slab should be at least 100 mm thick and formed with the same class of concrete as that of the footings. Where sub-excavation is required to remove unsuitable material from below the design founding level, the founding surface should be re-established using engineered fill or mass concrete of the same class as the footing.

All footing construction procedures should follow the guidelines provided in OPSS 902. Results from the boreholes indicate that the groundwater level is below the design footing elevations. It is important to note that water seepage from perched water within the fill as well as accumulation of precipitation and surface runoff should be expected. Dewatering prior to and during footing excavation will be required to construct the footings in the dry and to prevent sloughing of the sides or disturbance of the base of the excavation due to the inflow of groundwater. Suggested wording for an NSSP to this effect is presented in Appendix E.

9 PEDESTRIAN BRIDGE FOUNDATIONS

9.1 Foundation Alternatives

The proposed pedestrian bridge is similar to the existing steel truss superstructure, which is understood to be supported on augered caissons.

The subsurface conditions at this site generally consists of surficial compact to loose sand and silt to silty sand fill overlying a layer of native, compact silty sand to sand and silt. Underlying the surficial soils is a deposit of typically soft to firm silty clay which is underlain by very dense sand and silt till or hard silty clay till. The groundwater level was measured at just below the top surface of the surficial sands and silts.

From a foundations technical, constructability and cost-effectiveness perspective, the recommended foundation type for the relocated pedestrian bridge is augered caissons (drilled shafts) founded within the very dense sand and silt till or hard silty clay till.

Driven piles are technically feasible but require relatively large excavations for pile cap construction. The space and noise/vibration constraints at the site (vicinity of existing GO Station) render this option impractical and foundation recommendation has not been developed for this option. Space restriction, deep excavation and the need to employ roadway protection during construction preclude the use of spread footings at this site.

9.2 Augered Caissons – Pedestrian Bridge

9.2.1 Axial Resistance

Based on the results of Boreholes PB-01 and PB-02, augered caissons founded within the surficial compact sands and silts into the underlying very dense glacial tills are feasible to support the pedestrian bridge. The caissons should be designed based on sidewall friction. Table 8.6 presents the recommended founding depths and elevations for the caissons as well as geotechnical resistances for typical caisson diameters.

Table 8.6 – Founding Levels and Geotechnical Resistances for Augered Caissons for Pedestrian Bridge

Borehole #	Approx. Caisson Length* (m)	Founding Elevation (m)	Axial Geotechnical Resistance			
			Factored ULS _r (kN)	SLS (kN)	Factored ULS _r (kN)	SLS (kN)
			0.9 m Diameter		1.2 m Diameter	
PB-01 and PB-02	10	131	400	320	540	430
	12	129	480	400	700	560

* Caisson lengths in the above table are measured from the existing ground surface at approximate Elevation 141 m.

9.2.2 Lateral Resistance

Lateral bridge loadings can be geotechnically resisted by the caissons through passive pressure developed along the embedded portion of the shaft.

For lateral resistance design, soil-caisson interaction analyses may be carried out using the coefficient of horizontal subgrade reaction values provided in Table 8.7 below in conjunction with the equations and methodology outlined in Section 8.3.2 for the ramp bridge.

Table 8.7 – Recommended Geotechnical Parameters for Lateral Resistance Design

Foundation Unit	Borehole Number	Approx. Elevation (m)	Undrained Shear Strength C_u (kPa)	n_h (kN/m ³)	K_p	Unit Weight γ (kN/m ³)	Soil Conditions
Pedestrian Bridge	PB-01 PB-02	141 to 138	-	2,500	3.0	20	Sand and silt fill compact to loose
		138 to 134	-	3,000	3.2	10*	Sands and Silts compact
		134 to 121	35 to 40	-	-	19	Silty Clay soft to firm

9.2.3 Caisson Installation

Requirements for caisson installation are similar to those outlined in Section 8.3.3 for the ramp bridge.

10 RAMP EMBANKMENT DESIGN AND CONSTRUCTION

10.1 Embankment Stability

Based on cross-sections provided by MMM, limit equilibrium global stability analyses have been carried out for various fill configurations. The stability analysis was carried out using the commercially available slope stability program GEO-SLOPE by employing the Morgenstern-Price method. The emphasis of the analysis is on the use of RSS blocks and/or Select Subgrade Material (SSM) in various combinations that are governed by geometric configurations provided in MMM's cross-sections available to us. It is

understood that the need to use vertical walls at some locations are largely due to space restrictions on both sides of the proposed embankment.

Selected results of the stability analysis are summarized in Table 8.8 below.

Table 8.8 Stability of Retained Embankment

Approx. Station & Figure #	Approximate Location Along Ramp	Fill Height (m)	Wall Type, Fill Type Slope and Ground Configurations	Approximate Factor of Safety (F.S.)
~10+030 Figures F1, F2	West & East Approaches (immediately behind bridge abutments)	8 (on flat ground)	RSS wall with two vertical sides	1.65
			SSM fill with conventional retaining walls on spread footings or deep foundations	1.50
~10+110 Figures F3, F4	East Approach	7 (on flat ground)	RSS wall with two vertical sides	1.80
			SSM fill with conventional retaining walls on spread footings or deep foundations	1.35
~10+000 Figures F5, F6	West Approach	5 (near crest of slope)	RSS wall with two vertical sides	1.45
			SSM fill with conventional retaining walls on spread footings or deep foundations	1.35
~9+960 to 9+980 Figures F7, F8	West Approach	6 (adjacent to Hwy.401 slope)	SSM fill retained by RSS wall near fill slope crest	1.30
			SSM fill with conventional retaining walls on spread footings or deep foundations	1.25
~9+920 Figures F9, F10	West Approach	7 (on 9 m high Hwy.401 slope)	RSS wall perched on slope	1.30
			SSM fill with conventional retaining walls on spread footings or deep foundations perched on slope	1.25
~9+740 Figure 11	West Approach	3 (adjacent to 3 m high Hwy.401 slope)	SSM fill with 2H : 1V side slope	1.35

Graphical output of selected stability computation is presented on Figures F1 to F11 of Appendix F. Should EPS blocks with an assumed unit weight of 1 kN/m^3 be used in some combinations in conjunction with 1.5 m of cover including the pavement structure and concrete slab with an assumed combined unit weight of 23 kN/m^3 , the resulting Factors of Safety for each of the cases will be higher than those presented in Table 8.8 above.

For long term geotechnical design conditions, a retaining structure located adjacent to, or perched on, embankment slopes should be designed for a Factor of Safety (F.S.) in the order of 1.5 against global instability. Fill slopes with an inclination not steeper than 2H : 1V should have F.S. of at least 1.3.

Results of the analyses indicate that retaining structures using SSM do not satisfy the minimum F.S. requirement in most cases, except for the sections immediately behind the abutments where retaining walls supported on footings or piles are used. RSS walls generally satisfy the minimum F.S. requirement along sections where the fills are vertical on both faces. SSM fill up to 3 m in height with sideslope inclination of 2H : 1V will be stable.

As discussed below, however, the embankment design will be governed by settlement considerations.

10.2 Embankment Settlement

According to the preliminary GA drawing, up to about 8 m of new fill will be required to construct the new ramp. The following discusses the magnitude and nature of foundation settlements that can be induced by conventional earth fill and EPS blocks.

The currently applicable MTO embankment settlement criteria for design (July 2010) stipulates that the acceptable post-construction settlement limits of freeway embankment widening are 50 mm total settlement and 200 : 1 differential settlement. For approach embankments behind the ramp bridge abutments, the post-construction settlement limits are 25 mm for the first 20 m behind the abutments and 50 mm from 20 to 50 m distances.

Placement of new fill will induce immediate settlement of the surficial cohesionless fill and native sands and silts, and time-dependent compression of the typically soft to firm silty clay deposit. Primary consolidation settlement will be induced at locations where the fill height results in increase in vertical stress in excess of the pre-consolidation pressure of the silty clay. A summary of the settlement analysis on a station by station basis is presented in Table 1 following the text of this report.

If only conventional earth fill (SSM, granular) is used for embankment construction, fill heights of 5 to 8 m, which encompass the immediate bridge approaches and beyond, would induce post construction foundation settlements of between 60 to 100 mm.

Vertical stress due to the self-weight of the EPS blocks is practically negligible. At locations where EPS is used and for estimation of foundation settlement, it has been assumed that the EPS blocks have a bulk unit weight of 1 kN/m³. For the cover materials (earth cover, concrete slab, pavement structure) with an assumed overall thickness of 1.5m, an assumed combined bulk unit weight of 23 kN/m³ has been used.

Based on these criteria and our settlement calculations above, we recommend the following options:

Option 1 – For both the widening and new construction, embankment fill may be constructed entirely with EPS blocks below the 1.5 m cover for embankment sections greater than 3.0 m in height. A longitudinal transition zone should be provided between the embankment section higher than 3.0 m and the embankment section lower than 3.0 m. The EPS blocks should be held in place by vertical cladding walls founded at shallow depths on both sides or on one side, depending on the locations along the ramp.

Option 2 – Alternatively, below the 1.5 m thick cover materials, EPS fill may be used from immediately behind the abutments to Station 9+985 (25+485) at the west approach and to Station 10+155 at the east approach. Beyond these locations are the transition zones where a combination of EPS, granular or SSM fill may be used until the ramp merges with Highway 401 at the westerly limit and with Leslie Street at the easterly limit. An appropriate wall should be designed to contain the EPS, granular or fills. Beyond the point where the EPS fill ends, granular or SSM fill should be placed on the existing highway embankment fill at a sideslope inclination of 2H : 1V.

For both options, an approximately 5H : 1V longitudinal transition zone, with EPS fill thickness decreasing from a full EPS section towards a full granular/SSM fill section, should be provided to minimize differential settlements along the ramp embankment.

10.3 Settlement of EPS Embankment

Deformation behaviour of EPS blocks varies between different products. In general, the product literature indicates that long term design loads should not exceed the linear elastic range, and that combined live and dead load stresses should not exceed the compressive resistance at 1% strain. For example, EPS 29 has an elastic modulus of 7.5 MPa and a compressive resistance of 75 kPa at 1% deformation.

Creep behaviour of EPS blocks under loadings has been a design issue. The proprietary supplier of EPS blocks should be able to provide data regarding creep deformation. For preliminary estimates, information from the technical literature indicates that post construction settlement of EPS that may be attributed to creep at 50% of compressive strength loading over 50 years would be less than 1%. Creep movement will likely be reduced if higher strength EPS is used.

10.4 Embankment Construction

Embankment subgrade preparation and construction should be carried out in accordance with OPSS.PROV 206 and OPSS.PROV 501. The conventional fill material should consist of OPSS.PROV 1010 Select Subgrade Material (SSM) or Granular A or B Type II

materials. Benching of existing fill slopes should be carried out as per OPSD 208.010 prior to placing new fills. Benched fills placed and compacted as recommended, with a slope inclination not steeper than 2H : 1V and fill heights not exceeding those discussed in Section 10.1 above, are expected to satisfy global stability requirements and remain stable.

It is recommended that all exposed slope surfaces be vegetated and seeded in accordance with current MTO practice and with reference to OPSS.PROV 804.

11 RETAINING/NOISE BARRIER COMBINATION WALL FOUNDATIONS

The design of this combination wall requires the lower portion of the wall for earth and slope retention, whereas the upper portion of the wall serves as conventional noise barrier. It is recommended that this wall be designed as a conventional retaining wall and then checked for resistance against wind loading as a noise barrier wall. Section 12 of this report provides recommendations for lateral earth pressures considerations. In view of the close proximity of the property line, this type of wall is feasible to cope with the space restrictions since it does not require excavation behind the wall. It also serves the dual purpose of a temporary shoring and a permanent wall. All construction work will be carried out from the highway right-of-way.

11.1 Foundation Alternatives

Based on preliminary drawings provided by MMM, it is envisaged that the proposed wall will first be constructed as a soldier pile and lagging roadway protection (temporary shoring) wall behind the existing concrete toe wall. The H-piles will be encased inside concrete filled augered holes each of 0.76 m in diameter. The existing wall will then be removed and the lagging completed as the excavation reaches the subgrade level, after which concrete retaining wall panels will be installed. Installation of noise barrier panels and reconfiguration of the retained slope will follow where required.

Although driven H-piles is a feasible option from a foundations perspective, it is not recommended for use for this wall due to likely issues associated with the noise and vibration during installation close to the residential developments, and the lower lateral resistance which is the major design criterion for this wall. The option of H-piles encased in concrete filled augered holes will be used.

11.2 Augered Caissons

11.2.1 Axial Resistance

Augered caissons embedded within the very dense sand and silt and/or hard silty clay are feasible to support the retaining wall. Table 11.1 presents possible founding depths and

elevations for the caissons as well as geotechnical resistances recommended for some typical caisson diameters.

**Table 11.1 – Founding Levels and Axial Geotechnical Resistances
for Augered Caissons to support the Retaining Wall**

Caisson Length below final road grade (m)	Reference Borehole	Approx. Founding Elevation (m)	Axial Geotechnical Resistance			
			Factored ULSf (kN)	SLS (25 mm settle -ment) (kN)	Factored ULSf (kN)	SLS (25 mm settle -ment) (kN)
			0.76 m Diameter		0.9 m Diameter	
8	W-01	146.5	900	750	1,200	1,000
	W-02	145.0				
10	W-01	144.5	1,200	950	1,600	1,300
	W-02	143.0				

11.2.2 Lateral Resistance

The design of this combination wall is expected to be governed by lateral loading conditions. For caisson lateral resistance design, soil-caisson interaction analyses may be carried out using the geotechnical parameters provided in Table 11.2.

Table 11.2 – Recommended Geotechnical Parameters for Lateral Resistance Design

Borehole Number	Approx. Elevation (m)	Undrained Shear Strength C_u (kPa)	n_h (kN/m ³)	K_p	Unit Weight γ (kN/m ³)	Soil Conditions
W-01	154 to 153	-	2,000	3.0	20	Sand Fill compact
	153 to 151	100	-	-	20	Silty Clay very stiff
	151 to 147	-	7,000	3.5	11*	Sand & Silt very dense
	147 to 145	200	-	-	21	Silty Clay hard
W-02	153 to 150	-	4,000	3.0	20	Sand & Silt Fill compact to very dense

Borehole Number	Approx. Elevation (m)	Undrained Shear Strength C_u (kPa)	n_h (kN/m ³)	K_p	Unit Weight γ (kN/m ³)	Soil Conditions
	150 to 147	-	8,000	3.4	10*	Silt compact
	147 to 143	200	-	-	21	Silty Clay hard

The methodology outlined in Section 8.3.2 above may be used to estimate the lateral geotechnical resistance of the caisson.

11.2.3 Caisson Installation

Caisson installation may be carried out as discussed in Section 8.3.3.

11.3 Global Wall Stability

Based on preliminary information provided by MMM, Thurber carried out limit equilibrium stability analyses for selected cross-sections of the proposed wall for long and short term cases. The stability analysis was carried out using the commercially available slope stability program GEO-SLOPE by employing the Morgenstern-Price method.

The computed Factor of Safety (F.S.) for the retaining wall configuration under drained and undrained conditions at this site is greater than 1.5. Selected slope stability computation output is shown on Figures F12 to F16 of Appendix F.

As per typical MTO requirements, a F.S. of 1.3 is acceptable for short term or total stress (undrained) conditions. A F.S. of 1.5 is acceptable for long term (drained) conditions. The results indicate that these acceptance criteria are generally satisfied for the cases analysed.

12 LATERAL PRESSURES ON RETAINING WALLS

12.1 EPS Sections

Lateral pressures acting on the back side of the retaining walls along EPS sections depend on surcharge including traffic loading and the combined dead weight of the pavement structure, earth cover and concrete slab for the EPS blocks. Horizontal stress from the EPS blocks should be negligible.

For a fully drained condition, the lateral pressures can generally be given by the expression:

$$p_h = q$$

where: p_h = horizontal pressure on the wall at depth h (kPa)
 q = value of any surcharge including traffic loading (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 the pavement granular / earth cover and decreasing to 0 at the bottom of the layer with an approximate depth of up to 1.5 m. Compaction equipment to be used adjacent to retaining structures should be restricted in accordance with OPSS.PROV 501.

It is recommended that drainage measures be provided, where applicable, to enhance positive drainage from the EPS blocks behind the retaining walls.

12.2 Conventional Fill Sections

The backfill to abutment walls and retaining walls along the conventional fill sections should be in accordance with OPSS 902. Granular or other fill materials meeting the requirements of OPSS.PROV 1010 should be used.

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.

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

$p_h = K (\gamma h + q)$
where: p_h = horizontal pressure on the wall at depth h (kPa)
 K = earth pressure coefficient (see Table 12.1)
 γ = unit weight of retained soil (see Table 12.1)
 h = depth below top of fill where pressure is computed (m)
 q = value of any surcharge including traffic loading (kPa).

For non-draining or partially drained backfill conditions such as the case for the cast-in-place walls, it is recommended that full hydrostatic pressure be included in the design.

In accordance with Clause 6.9.3 of the CHBDC, a compaction surcharge should be added. The magnitude should be 12 kPa at the top of fill and decreasing to 0 kPa at a depth of 1.7 m for Granular A or Granular B Type II. Compaction equipment to be used adjacent to retaining structures should be restricted in accordance with OPSS.PROV 501.

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

Table 12.1 – Earth Pressure Coefficients

Wall Condition	Earth Pressure Coefficient (K)					
	OPSS Granular A and Granular B Type II $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$		Select Subgrade Material or Existing Embankment Fill $\phi = 30^\circ, \gamma = 20.0 \text{ kN/m}^3$	
	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 CHBDC.

It is recommended that perforated sub-drains and weep holes be installed, where applicable, to provide positive drainage of the granular backfill behind the abutment walls. Reference should be made to OPSD 3102.100.

13 EPS SUBGRADE PREPARATION AND INSTALLATION

It is anticipated that the new embankment fill will have a founding subgrade of compact to dense sand and silt fill in the vicinity of the ramp bridge, transitioning to a stiff to hard silty clay fill westerly away from the bridge. Embankment subgrade preparation must be carried out in accordance with OPSS.PROV 206. Prior to placing the EPS blocks, it is recommended that a levelling pad be constructed on the existing fill subgrade by placing, levelling and compacting a 150 mm thick layer of OPSS.PROV 1010 Granular A or Granular B Type II material in accordance with OPSS.PROV 501. Compaction of earth cover above the EPS blocks must also be carried out in accordance with OPSS.PROV 501.

Sample wordings for an NSSP on installation of EPS blocks are provided in Appendix E. It is noted that the technical requirements will depend on the adopted design for this site as well as requirements of the proprietary supplier, and are therefore subject to change.

14 EXCAVATION AND GROUNDWATER CONTROL

Temporary excavations will be required during bridge and embankment construction at this site. All temporary excavations must be carried out in accordance with the Occupational Health and Safety Act (OHSA).

Currently available preliminary GA drawings indicate that excavation for foundation construction will extend predominantly through the surficial fill. There may be some locations where the underlying native sands and silts would be exposed. For the purpose of OHSA, the existing fills and native sands and silts above the groundwater level are classified as Type 3 soils. If exposed below the groundwater level, the sands and silts are classified as a Type 4 soils. Based on existing information, the excavations would likely not extend below the groundwater level.

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

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.

It is anticipated that the amount of seepage from perched water within the fills would be limited. For temporary excavations, 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 etc. Filtered sumps must be designed properly so that construction drainage water containing eroded soil and fines do not flow onto existing roadways and the GO property.

The design of 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.

15 ROADWAY PROTECTION

Roadway protection will be required during construction of the ramp and associated works adjacent to the existing Highway 401 lanes and the W-N/S ramp. An item titled “Protection System” as per OPSS.PROV 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 protection system 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 walls. It is anticipated that the soldier piles will need to be installed within the native sands and silts underlying the fill in order to develop the required toe resistance. It is anticipated that the shoring system may be stiffened by cross bracings, where applicable.

A 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 (existing fills)
	=	0.33 (native sands and silts)
K_p	=	3.0 (native sands and silts)

Alternatively, sheetpiles may be used as roadway protection provided that the sheetpiles are driven to the desired depths rather than installed by vibratory methods, which carries the risk of causing embankment and pavement distress.

The designer of the roadway protection system should check whether the depth of pile is sufficient to provide lateral resistance and 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.

16 SEISMIC CONSIDERATIONS

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 0.08g (g is gravitational acceleration)

The soil profile type has been classified as Type I. Therefore, according to Table 4.4.6.1 of the CHBDC, a Site Coefficients “S” (ground motion amplification factor) of 1.0 should be used in seismic design.

The foundation soils at the site are assessed as not being prone to liquefaction.

17 EXISTING UTILITIES, EMBANKMENTS AND ADJACENT STRUCTURES

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 must not be damaged during construction of the new bridge, retaining structures and associated approach and embankment fills. If necessary, relocation of, and/or special protective measures for affected utilities may be required. An existing 600 mm diameter watermain is to be replaced due to conflicts with the proposed works. A separate foundation investigation has been proposed for installation of the new watermain.

In addition, placement of new fill adjacent to the existing Highway 401 eastbound collector lanes will induce foundation settlement that could result in pavement distress on the travelled lanes of the highway. Should this occur, remedial measures including temporarily re-paving the affected areas would likely be required.

In order to confirm that the ramp construction works will not result in adverse effects on the existing bridge and buried utilities in the vicinity, it is recommended that the following be carried out prior to the commencement of construction:

- Carry out pre-construction condition survey including documentation of any existing distress associated with the existing bridge structure and utilities. Any distress should be reported to and discussed with the structure/utility owner.
- Implement an instrumentation and monitoring program to include settlement monitoring during installation of roadway protection (shoring), excavation and new foundation construction (caissons and footings). Establish review and alert level criteria for allowable settlement and lateral movement following discussions with the owner of the structure/utility. Establish review and alert level criteria for vibration levels (in terms of peak particle velocity, ppv) during pile driving and caisson augering (at the west abutment). Establish and agree on remedial action, if required, prior to start of construction.
- Establish reference points over each abutment and piers of the existing structure and to monitor movement of these points relative to known, fixed reference points on a regular basis during driving of piles.
- Carry out post-construction condition survey of the existing structures/utilities.

During ramp construction, potential impact of fill placement on the existing pavement surface of Highway 401 should be closely monitored.

- Daily visual inspection of the pavement surface must be carried out in the vicinity of the fill placement. If cracks form in the pavement or settlement is observed to occur, these matters must immediately be brought to the attention of the Contract Administrator for determining as to whether remedial action is required. Such action may include temporarily re-paving the affected areas.

Suggested wording for monitoring is provided in Appendix E.

18 CONSTRUCTION CONCERNS

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

1. Foundation construction in close proximity to existing utilities and bridge foundations.

Settlement and vibration monitoring of the existing utilities and bridge foundations should be conducted before and during construction. Settlement monitoring should continue after construction.

2. Ramp construction adjacent to the Highway 401 eastbound lanes.

Monitoring of potential settlement and pavement distress on the adjacent Highway 401 eastbound lanes is recommended. Remedial measures to temporarily re-pave the affected areas may need to be carried out.

3. Presence of cobbles and boulders within the glacial tills.

Glacial deposits inherently contain cobbles and boulders, which may affect installation of the caissons. The Contractor shall be prepared to remove, drill through and/or penetrate these obstructions and extend the caissons to competent foundation level.

4. Caisson installation

The cohesionless sands and silts at depth could 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 liners must not be installed by vibratory means to avoid adverse effects on the GO rail tracks, GO station platform and other existing facilities.

5. Excavations

Care must be exercised during excavation to avoid disturbing the founding subgrade. The exposed subgrade soils should be expeditiously inspected, approved and protected from disturbance.

19 CLOSURE

Numerical analysis for embankment design was carried out by Mr. Keli Shi, P.Eng. Foundation assessment and report preparation was carried out by Dr. Sydney Pang, P.Eng.

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

THURBER ENGINEERING LTD.



Keli Shi, P.Eng.
Foundation Engineer



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



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

TABLE 1 – SUMMARY OF FOUNDATION SETTLEMENT ESTIMATES

Embankment Widening

Station		Embankment Height (m)	Granular Fill Thickness (m)	EPS Thickness (m)	Foundation Settlement							
From	To				Maximum		Hwy 401 Shoulder		5 m from Hwy 401 Shoulder		10 m from Hwy 401 Shoulder	
					Settle. During Constr. ⁽¹⁾	Post Constr. Settle. ⁽²⁾	Settle. During Constr. ⁽¹⁾	Post Constr. Settle. ⁽²⁾	Settle. During Constr. ⁽¹⁾	Post Constr. Settle. ⁽²⁾	Settle. During Constr. ⁽¹⁾	Post Constr. Settle. ⁽²⁾
25+200	25+240	3.0	3.0	-	40	25 ⁽³⁾	20	25	10	20	5	10
25+240	25+320	6.0	6.0	-	100	55	45	45	15	35	5	30
			4.0	2.0	60	45 ⁽³⁾	30	35	10	30	5	25
			1.5	4.5	25	20 ⁽³⁾	15	20	10	20	5	15
25+320	25+380	6.5	6.5	-	125	70	30	45	10	35	5	25
			4.0	2.5	75	45 ⁽³⁾	20	30	10	25	5	20
			1.5	5.0	30	20 ⁽³⁾	10	15	5	15	5	15
25+380	25+460	6.0	6.0	-	140	95	30	55	5	45	5	30
			3.0	3.0	60	50 ⁽³⁾	20	35	5	30	5	25
			1.5	4.5	30	30 ⁽³⁾	10	25	5	20	5	20
25+460	25+500	5.5	5.5	-	130	135	35	60	10	45	0	35
			3.0	2.5	50	50 ⁽³⁾	15	35	5	30	0	20
			1.5	4.0	25	20 ⁽³⁾	5	20	5	20	0	15

(1) 12-month construction period

(2) 20 years after construction

(3) Meets MTO Post-Construction Settlement Criteria for Embankment Widening on Freeways (50 mm)

TABLE 1 – SUMMARY OF FOUNDATION SETTLEMENT ESTIMATES

Approach Embankment on Either Side of the Bridge

Station		Embankment Height (m)	Granular Fill Thickness (m)	EPS Thickness (m)	Foundation Settlement							
From	To				Maximum		Hwy 401 Shoulder		5 m from Hwy 401 Shoulder		10 m from Hwy 401 Shoulder	
					Settle. During Constr. ⁽¹⁾	Post Constr. Settle. ⁽²⁾	Settle. During Constr. ⁽¹⁾	Post Constr. Settle. ⁽²⁾	Settle. During Constr. ⁽¹⁾	Post Constr. Settle. ⁽²⁾	Settle. During Constr. ⁽¹⁾	Post Constr. Settle. ⁽²⁾
10+000	10+015	6.0	6.0	-	65	60	-	-	-	-	-	-
			4.0	2.0	45	35 ⁽⁴⁾	-	-	-	-	-	-
			1.5	4.5	15	5 ⁽⁴⁾	-	-	-	-	-	-
10+015	10+035	7.5	7.5	-	85	85	-	-	-	-	-	-
			3.0	4.5	35	20 ⁽³⁾	-	-	-	-	-	-
			1.5	6.0	15	5 ⁽³⁾	-	-	-	-	-	-
10+105	10+125	8.0	8.0	-	90	95	-	-	-	-	-	-
			2.5	5.5	30	15 ⁽³⁾	-	-	-	-	-	-
			1.5	6.5	15	5 ⁽³⁾	-	-	-	-	-	-
10+125	10+155	7.0	7.0	-	85	80	-	-	-	-	-	-
			5.0	2.0	60	50 ⁽⁴⁾	-	-	-	-	-	-
			1.5	5.5	15	5 ⁽⁴⁾	-	-	-	-	-	-
10+155	10+180	5.0	5.0	-	60	60 ⁽⁵⁾	-	-	-	-	-	-
			1.5	3.5	15	10 ⁽⁵⁾	-	-	-	-	-	-
10+180	10+350	4.0	4.0	-	50	50 ⁽⁶⁾	-	-	-	-	-	-
			1.5	2.5	20	10 ⁽⁶⁾	-	-	-	-	-	-

(1) 12-month construction period

(2) 20 years after construction

(3) Meets MTO Post-Construction Settlement Criteria for Transitions (0-20 m behind abutments) on Freeways (25 mm)

(4) Meets MTO Post-Construction Settlement Criteria for Transitions (20-50 m behind abutments) on Freeways (50 mm)

(5) Meets MTO Post-Construction Settlement Criteria for Transitions (50-75 m behind abutments) on Freeways (75 mm)

(6) Meets MTO Post-Construction Settlement Criteria for Transitions (≥ 75 m behind abutments) on Freeways (100 mm)

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

DISCONTINUITY SPACING		STRENGTH CLASSIFICATION			
Bedding	Bedding Plane Spacing	Rock Strength	Approximate Uniaxial Compressive Strength		Field Estimation of Hardness*
			(MPa)	(psi)	
Very thickly bedded	Greater than 2m	Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Thickly bedded	0.6 to 2m				
Medium bedded	0.2 to 0.6m	Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Thinly bedded	60mm to 0.2m				
Very thinly bedded	20 to 60mm	Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Laminated	6 to 20mm				
Thinly Laminated	Less than 6mm	Medium Strong	25.0 to 50.0	3,500 to 7,500	Breaks under single blow of geological hammer.
		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

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.

Appendix A

West N/S Ramp CNR Overpass at Leslie Street Boreholes R-01 to R-05, R-07 to R-09, PB-02 and Boreholes from previous investigation

- Record of Borehole Sheets
- Laboratory Test Results
- Drawing titled “Borehole Locations and Soil Strata

RECORD OF BOREHOLE No PB-02

1 OF 3

METRIC

W.P. 2061-13-00 LOCATION Pedestrian Overpass N 4 847 255.7 E 315 747.1 ORIGINATED BY ES
HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
DATUM Geodetic DATE 2015.04.14 - 2015.04.15 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						WATER CONTENT (%)			
								20 40 60 80 100						20 40 60			
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			
142.1	GROUND SURFACE																
0.0 0.1	TOPSOIL: (75mm) Silty SAND , some clay, trace gravel Dense Brown to Dark Brown Moist (FILL)		1	SS	31		142										
140.7							141										
1.4	Compact to Loose		2	SS	14		140							6 54 30 10			
			3	SS	7												
			4	SS	8		139										
138.0							138										
4.1	SAND and SILT , trace clay, trace gravel Compact Brown Moist		5	SS	23		137										
	Wet		6	SS	12		136							0 40 58 2			
	Loose Grey		7	SS	6		135										
							134										
133.4																	
8.7	Silty CLAY , some to with sand Soft to Firm Grey Wet		8	SS	3		133										

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No PB-02

2 OF 3

METRIC

W.P. 2061-13-00 LOCATION Pedestrian Overpass N 4 847 255.7 E 315 747.1 ORIGINATED BY ES
 HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2015.04.14 - 2015.04.15 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT		NATURAL MOISTURE CONTENT		LIQUID LIMIT		UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)		W _p	W	W _L	GR		SA	SI	CL		
Continued From Previous Page								○ UNCONFINED + FIELD VANE	● QUICK TRIAXIAL × LAB VANE												
	Silty CLAY , some to with sand, trace gravel Firm Grey Wet							20	40	60	80	100	20	40	60						
			1	TW	PH																
			9	SS	1																
			10	SS	0																
	Sand seams		11	SS	1																
125.8																					
16.3	Stiff																				
			12	SS	1																
			13	SS	14																
122.7																					
19.4																					

Continued Next Page

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

RECORD OF BOREHOLE No PB-02

3 OF 3

METRIC

W.P. 2061-13-00 LOCATION Pedestrian Overpass N 4 847 255.7 E 315 747.1 ORIGINATED BY ES
HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
DATUM Geodetic DATE 2015.04.14 - 2015.04.15 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE															
	Continued From Previous Page		14	SS	2		20	40	60	80	100	PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	
120.2	Silty CLAY , some to with sand, trace gravel Soft Grey Wet														
21.9	SAND and SILT , some clay, trace gravel Very Dense Grey Moist (TILL)		15	SS	101/ 0.250										
117.7															
24.4	Some gravel and cobbles Very Dense		16	SS	100/ 0.075										
117.0															
25.1	Occasional sand layers														
115.8			17	SS	108/ 0.250										
26.3	END OF BOREHOLE AT 26.3m. WATER LEVEL AT 7.0m UPON COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND AUGER CUTTINGS TO SURFACE.														

ONTMT4S 1205.GPJ 2015TEMPLATE(MTO).GDT 11/3/15

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

RECORD OF BOREHOLE No R-01

1 OF 3

METRIC

W.P. 2061-13-00 LOCATION W-N/S Ramp Leslie St. / CNR Overhead N 4 847 294.4 E 315 382.5 ORIGINATED BY ES
HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
DATUM Geodetic DATE 2015.03.27 - 2015.03.30 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)			
								○ UNCONFINED + FIELD VANE	● QUICK TRIAXIAL × LAB VANE	W _P W W _L			
148.1	GROUND SURFACE						20 40 60 80 100						GR SA SI CL
0.0	TOPSOIL: (175mm)												
0.2	Silty CLAY , some to with sand, trace gravel, occasional rootlets Stiff Brown to Dark Brown (FILL)						148						
			1	SS	10		147						
			2	SS	11		146						
146.0							145						
2.1	Silty CLAY , trace sand, trace gravel Stiff Brown Moist		3	SS	13		144						
	Occasional sand seams		4	SS	15		143						
							142						
	Very Stiff Grey		5	SS	21		141						
142.5							140						
5.6	SAND and SILT , some clay Compact Grey Wet		6	SS	11		139						
140.9													
7.2	Stiff Wet		7	SS	11								
	Firm		8	SS	6								

Continued Next Page

+³, ×³: Numbers refer to Sensitivity
20
15 10 5 0 (%) STRAIN AT FAILURE

METRIC

[illegible]

+³, ×³: Numbers refer to Sensitivity


ONTMT4S 1205.GPJ 2015TEMPLATE(MTO).GDT 11/3/15

RECORD OF BOREHOLE No R-01

3 OF 3

METRIC

W.P. 2061-13-00 LOCATION W-N/S Ramp Leslie St. / CNR Overhead N 4 847 294.4 E 315 382.5 ORIGINATED BY ES
 HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2015.03.27 - 2015.03.30 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE								
	Continued From Previous Page		14	SS	102/ 0.250		128	20	40	60	80	100	20	40	60	
127.9	SAND and SILT (TILL) END OF BOREHOLE AT 20.2m. WATER LEVEL AT 5.5m UPON COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND AUGER CUTTINGS TO SURFACE.															
20.2																

RECORD OF BOREHOLE No R-02

1 OF 3

METRIC

W.P. 2061-13-00 LOCATION W-N/S Ramp Leslie St. / CNR Overhead N 4 847 294.8 E 315 475.5 ORIGINATED BY ES
 HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2015.03.26 - 2015.03.26 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				WATER CONTENT (%)				
								○ UNCONFINED + FIELD VANE		● QUICK TRIAXIAL × LAB VANE		W _P	W	W _L		
145.7	GROUND SURFACE					20	40	60	80	100	20	40	60			
0.0	TOPSOIL: (100mm)															
0.1	Silty CLAY , some to with sand, trace gravel Very Stiff to Stiff Brown to Grey Moist (FILL)		1	SS	16							○				
			2	SS	9							○		0 31 39 30		
143.6																
2.1	Sandy SILT , trace clay, trace gravel Compact Grey Moist		3	SS	14							○				
	Dark Brown to Brown		4	SS	17							○				
141.6																
4.1	Silty CLAY , some to with sand, trace gravel Very Stiff Brown Moist		5	SS	25							○				
			6	SS	15							○				
138.5																
7.2	Firm		7	SS	6							○		3 38 28 31		
			1	TW	PH											
					</											

Continued Next Page

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

METRIC

[illegible]

+³, ×³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No R-02

3 OF 3

METRIC

W.P. 2061-13-00 LOCATION W-N/S Ramp Leslie St. / CNR Overhead N 4 847 294.8 E 315 475.5 ORIGINATED BY ES
 HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2015.03.26 - 2015.03.26 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa 20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE									
	Continued From Previous Page		14	SS	78												
	SAND and SILT , trace gravel, trace clay, occasional inferred cobbles Very Dense Grey Wet (TILL)																
			15	SS	133/ 0.300												
			26	SS	113/ 0.300												
			27	SS	100/ 0.250												
120.9																	
24.8	END OF BOREHOLE AT 24.8m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 3.04m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) Mar 31/2015 5.4 140.3 Apr 22/2015 5.3 140.4 Jun 03/2015 5.2 140.5 Jun 17/2015 5.3 140.4																

ONTMT4S 1205.GPJ 2015TEMPLATE(MTO).GDT 11/3/15

RECORD OF BOREHOLE No R-03

1 OF 4

METRIC

W.P. 2061-13-00 LOCATION W-N/S Ramp Leslie St. / CNR Overhead N 4 847 296.9 E 315 615.8 ORIGINATED BY ES
 HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2015.03.24 - 2015.03.25 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				WATER CONTENT (%)					
144.9	GROUND SURFACE							20	40	60	80	100					
0.0	TOPSOIL: (175mm)							20	40	60	80	100					
0.2	SAND and SILT, trace gravel Compact Brown Moist (FILL)		1	SS	25		144							○			
143.3																	
1.6	Silty CLAY, some to with sand, trace gravel Very Stiff to Hard Grey to Brown Moist (FILL)		2	SS	17		143							○			
			3	SS	31		142							○			
			4	SS	29		141							○			
140.8																	
4.1	Silty SAND, some gravel, trace to some clay, occasional inferred cobbles Very Dense to Dense Brown Moist		5	SS	53		140							○			
							139										
			6	SS	30		138							○	○		
	Moist to Wet																
			7	SS	44		137							○			
							136										
135.7																	
9.2	Silty CLAY, some sand Soft Grey Wet		8	SS	3		135							○	○		

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No R-03

2 OF 4

METRIC

W.P. 2061-13-00 LOCATION W-N/S Ramp Leslie St. / CNR Overhead N 4 847 296.9 E 315 615.8 ORIGINATED BY ES
 HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2015.03.24 - 2015.03.25 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				WATER CONTENT (%)									
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE				W _P W W _L									
	Continued From Previous Page							20	40	60	80	100	20	40	60						
	Silty CLAY , some sand Firm Grey Wet								3.0 +												
			1	TW	PH		134														
									3.0 +												
			9	SS	3									○							
							132														
									3.0 +												
	Some sand		10	SS	1		131							○							
									3.0 +												
129.7							130														
15.2	Sandy SILT , trace gravel Very Loose Grey Wet		11	SS	3		129							○							
128.6																					
16.3	some to with sand, trace gravel Firm						128							⊞				3	37	32	28
			12	SS	3																
									3.0 +												
							127														
			13	SS	8									○							
							126														
							125														

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

RECORD OF BOREHOLE No R-03

3 OF 4

METRIC

W.P. 2061-13-00 LOCATION W-N/S Ramp Leslie St. / CNR Overhead N 4 847 296.9 E 315 615.8 ORIGINATED BY ES
 HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2015.03.24 - 2015.03.25 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				WATER CONTENT (%)				GR	SA	SI	CL			
								20 40 60 80 100	W _p W W _L													
	Continued From Previous Page							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE														
123.9	Silty CLAY , some to with sand, occasional silt seams Firm Grey Wet		14	SS	8		124						○									
21.0	Sandy SILT , trace gravel, trace clay Very Dense Grey Moist		15	SS	107/ 0.250		123						○									
122.5																						
22.4	SAND , some silt, trace clay Very Dense Grey Wet		16	SS	78		122						○					0	86	10	4	
							121															
			17	SS	100/ 0.250		120						○									
	Loose		18	SS	8		119						○									
117.9							118															
27.0	Silty SAND , trace clay, trace gravel Very Dense Grey Moist (TILL)		19	SS	113/ 0.225		117						○									
	Compact Wet		20	SS	16		116						○						0	64	31	5
							115															

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No R-03

4 OF 4

METRIC

W.P. 2061-13-00 LOCATION W-N/S Ramp Leslie St. / CNR Overhead N 4 847 296.9 E 315 615.8 ORIGINATED BY ES
 HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2015.03.24 - 2015.03.25 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
	Continued From Previous Page																
114.1	Silty SAND , trace gravel, trace clay Very Dense Grey Wet (TILL)		21	SS	114/												
30.8	SAMPLED BOREHOLE TO 30.6m. START DCPT AT 30.6m. END OF BOREHOLE AT 30.8m UPON DCPT REFUSAL. WATER LEVEL AT 4.3m UPON COMPLETION. BOREHOLE BACKFILED WITH BENTONITE HOLEPLUG AND AUGER CUTTINGS TO SURFACE.				0.250												

RECORD OF BOREHOLE No R-04

1 OF 3

METRIC

W.P. 2061-13-00 LOCATION W-N/S Ramp Leslie St. / CNR Overhead N 4 847 301.7 E 315 659.3 ORIGINATED BY ES
 HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2015.03.17 - 2015.03.17 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)		
								○ UNCONFINED		+ FIELD VANE									
143.2	GROUND SURFACE							20	40	60	80	100							
0.0	TOPSOIL: (50mm)						143												
	SAND and SILT, trace gravel Compact Brown to Grey Moist (FILL)		1	SS	26		142						○						
			2	SS	26								○						
141.2							141						○						
2.0	TOPSOIL, occasional rootlets																		
140.9	Compact Dark Brown (300mm)		3	SS	13								○						
2.3	Silty SAND, trace clay, trace gravel Compact to Dense Dark Brown to Grey Moist		4	SS	10		140						○			0 61 32 7			
	Brown		5	SS	31		139												
													○						
							138												
	Wet		6	SS	26		137						○						
136.0							136												
7.2	Silty CLAY, some to with sand, trace gravel Firm Grey Moist		7	SS	4								○			0 32 35 33			
							135												
										2.0 +									
			8	SS	3		134						○						

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

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15
10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No R-04

2 OF 3

METRIC

W.P. 2061-13-00 LOCATION W-N/S Ramp Leslie St. / CNR Overhead N 4 847 301.7 E 315 659.3 ORIGINATED BY ES
 HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2015.03.17 - 2015.03.17 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						
	Continued From Previous Page						20 40 60 80 100	PLASTIC LIMIT W _P NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L						
	Silty CLAY , trace to some sand, trace gravel Firm Grey Wet						2.0 +							
			1	TW	PH									
							1.0 +							
			9	SS	2									
							2.0 +							
	Becoming some to with sand		10	SS	1									
							2.0 +							
128.0														
15.2	SILT , some sand, trace gravel Very Loose Wet		11	SS	1									
126.9														
16.3														
			12	SS	6									
							2.5 +							
	Stiff													
			13	SS	3									

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

RECORD OF BOREHOLE No R-04

3 OF 3

METRIC

W.P. 2061-13-00 LOCATION W-N/S Ramp Leslie St. / CNR Overhead N 4 847 301.7 E 315 659.3 ORIGINATED BY ES
 HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2015.03.17 - 2015.03.17 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)			GR	SA	SI	CL
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× LAB VANE					20	40	60				
Continued From Previous Page																						
123.1			14	SS	8		123							○			3	38	48	11		
20.1	SAND and SILT , some clay, trace gravel Very Dense Grey Moist (TILL)						122															
			15	SS	108		121							○								
							120															
120.6	Wet						119															
22.6	Dense		16	SS	48		118							○								
							117															
							116															
119.3							115															
23.9	Some clay to clayey, trace gravel Moist		17	SS	100/ 0.250		114							○			7	26	43	24		
							113															
							112															
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							0															

ONTMT4S 1205.GPJ 2015TEMPLATE(MTO) GDT 11/3/15

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

RECORD OF BOREHOLE No R-05

1 OF 3

METRIC

W.P. 2061-13-00 LOCATION W-N/S Ramp Leslie St. / CNR Overhead N 4 847 304.4 E 315 700.8 ORIGINATED BY ES
 HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2015.03.18 - 2015.03.19 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				WATER CONTENT (%)				GR	SA	SI	CL
								○ UNCONFINED + FIELD VANE	● QUICK TRIAXIAL × LAB VANE										
140.8	GROUND SURFACE																		
0.0	SAND and SILT , trace gravel Loose to Compact Brown Moist (FILL)		1	SS	5														
			2	SS	17														
	Trace clay		3	SS	19											0	58 38 4		
137.9																			
2.9	Sandy SILT , trace gravel, trace clay Compact Brown Moist to Wet		4	SS	22														
			5	SS	8														
	Loose																		
135.2																			
5.6	Silty CLAY , some to trace sand Very Soft to Soft Grey Wet		6	SS	1														
			7	SS	1											0	18 32 50		
			8	SS	3														

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No R-05

2 OF 3

METRIC

W.P. 2061-13-00 LOCATION W-N/S Ramp Leslie St. / CNR Overhead N 4 847 304.4 E 315 700.8 ORIGINATED BY ES
 HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2015.03.18 - 2015.03.19 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE					W _p	W	W _L		
	Continued From Previous Page							20	40	60	80	100					
	Silty CLAY , trace to some sand, trace gravel Very Soft to Firm Grey Wet		9	SS	2		130										
							129										
			10	SS	3		128										
							127										
	Becoming some to with sand, occasional sand seams		11	SS	2		126										
							125										
			12	SS	4		124										
							123										
			13	SS	1		122										
							121										
122.2 18.6	SAND and SILT , trace gravel, occasional sand seams Compact Grey Moist (TILL)		14	SS	14												

Continued Next Page

+³, ×³: Numbers refer to Sensitivity 20 15 10 5 0 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No R-05

3 OF 3

METRIC

W.P. 2061-13-00 LOCATION W-N/S Ramp Leslie St. / CNR Overhead N 4 847 304.4 E 315 700.8 ORIGINATED BY ES
 HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2015.03.18 - 2015.03.19 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa 20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE					WATER CONTENT (%) 20 40 60 W _p W W _L				
	Continued From Previous Page																
	SAND and SILT, trace gravel, occasional sand seams Very Dense Grey Moist (TILL)	0	15	SS	108												
		4															
		8															
		12															
		16	16	SS	100/ 0.250											6 36 49 9	
		20															
		24															
		28															
		32															
117.5		36	17	SS	106/ 0.250												
23.3	END OF BOREHOLE AT 23.3m. WATER LEVEL AT 5.2m UPON COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND AUGER CUTTINGS TO SURFACE.																

RECORD OF BOREHOLE No R-07

1 OF 3

METRIC

W.P. 2061-13-00 LOCATION W-N/S Ramp Leslie St. / CNR Overhead N 4 847 296.2 E 315 705.8 ORIGINATED BY ES
 HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2015.03.20 - 2015.03.23 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE LIQUID CONTENT LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				WATER CONTENT (%)				
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE								
								20 40 60 80 100	20 40 60							
140.7	GROUND SURFACE															
0.0 0.1	TOPSOIL: (75mm)															
	Silty SAND , trace gravel, trace clay, occasional rootlets Dense Grey Moist (FILL)		1	SS	38		140									
			2	SS	41		139									
138.4																
2.3	SAND , some silt, trace clay, trace gravel Compact Brown Moist to Wet		3	SS	19		138									
			4	SS	19										0 83 14 3	
							137									
136.5																
4.2	Sandy SILT , trace clay, trace gravel Compact Brown to Grey Wet		5	SS	10		136									
							135									
	Loose		6	SS	7											
							134									
133.5																
7.2	Silty CLAY , some to with sand, trace gravel Soft to Very Soft Grey Wet		7	SS	2		133									
							132									
			8	SS	1											
							131									

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15 10 5 10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No R-07

2 OF 3

METRIC

W.P. 2061-13-00 LOCATION W-N/S Ramp Leslie St. / CNR Overhead N 4 847 296.2 E 315 705.8 ORIGINATED BY ES
 HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2015.03.20 - 2015.03.23 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
	Continued From Previous Page													
	Silty CLAY , some to with sand, trace gravel Soft to Very Soft Grey Wet		9	SS	2		130							0 27 40 33
							129							
			10	SS	1		128							
							127							
			11	SS	1		126							
125.9							125							
14.8			12	SS	5		124							
	Firm						123							
			13	SS	4		122							0 21 36 43
122.9							121							
17.8			14	SS	1									

Continued Next Page

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

RECORD OF BOREHOLE No R-07

3 OF 3

METRIC

W.P. 2061-13-00 LOCATION W-N/S Ramp Leslie St. / CNR Overhead N 4 847 296.2 E 315 705.8 ORIGINATED BY ES
 HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2015.03.20 - 2015.03.23 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
								20 40 60 80 100					W _P	W	W _L		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE									
	Continued From Previous Page		15	SS	15												
120.0	Silty CLAY , some to with sand, trace gravel Stiff to Very Stiff Grey Moist						120										
20.7	SAND and SILT , trace to some clay, trace gravel Very Dense Grey Moist (TILL)		16	SS	100/ 0.100		119										4 41 40 15
							118										
			17	SS	104												
							117										
116.1			18	SS	109/ 0.250												
24.6	END OF BOREHOLE AT 24.6m. WATER LEVEL AT 5.0m UPON COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND AUGER CUTTINGS TO SURFACE.																

+³, ×³: Numbers refer to Sensitivity 20 15 10 5 0 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No R-08

1 OF 3

METRIC

W.P. 2061-13-00 LOCATION W-N/S Ramp Leslie St. / CNR Overhead N 4 847 243.4 E 315 754.1 ORIGINATED BY ES
 HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2015.04.15 - 2015.04.16 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE LIQUID CONTENT LIMIT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE				WATER CONTENT (%) w _P w w _L					GR	SA	SI	CL
142.0	GROUND SURFACE							20	40	60	80	100								
0.0	TOPSOIL: (63mm)							20	40	60	80	100								
0.1	SAND and SILT , some clay, trace gravel Compact Brown to Dark Brown Moist (FILL)		1	SS	29		141							○						
	Occasional rootlets		2	SS	15		140							○						
			3	SS	13		139							○					0	53 37 10
	Very Loose		4	SS	3		138							○						
137.7																				
4.3	SAND and SILT Compact Brown Moist		5	SS	22		137							○						
136.4																				
5.6	Silty CLAY , trace sand occasional oxide staining Very Stiff to Stiff Brown Wet		6	SS	17		136							○					0	0 51 49
			7	SS	9		135							○						
133.3																				
8.7							133							○						
	Occasional sand seams Very Soft Grey		8	SS	1															

Continued Next Page

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

RECORD OF BOREHOLE No R-08

2 OF 3

METRIC

W.P. 2061-13-00 LOCATION W-N/S Ramp Leslie St. / CNR Overhead N 4 847 243.4 E 315 754.1 ORIGINATED BY ES
 HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2015.04.15 - 2015.04.16 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				W _p	W	W _L			GR
	Continued From Previous Page							20	40	60	80	100					
	Silty CLAY , trace sand Firm Grey Wet		9	SS	0		131	3.2 +									
							130	2.7 +									
			1	TW	PH		129										Oedometer Test 0 0 38 62
			10	SS	2		128	2.9 +									0 8 35 57
							127	2.7 +									
			11	SS	0		126										
	Firm to Stiff						125	4.3 +									
			12	SS	1		124										
							123	3.4 +									
			13	SS	1												
								4.3 +									

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No R-08

3 OF 3

METRIC

W.P. 2061-13-00 LOCATION W-N/S Ramp Leslie St. / CNR Overhead N 4 847 243.4 E 315 754.1 ORIGINATED BY ES
HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
DATUM Geodetic DATE 2015.04.15 - 2015.04.16 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE									
	Continued From Previous Page																
120.1	Silty CLAY , trace sand, occasional sand seams Firm to Stiff Grey Wet		14	SS	4												
21.9	SAND and SILT , trace gravel, trace clay Very Dense Grey Moist (TILL)																
			15	SS	100/ 0.250												
	Occasional sand layer		16	SS	103/ 0.225												
115.7			17	SS	100/ 0.275												
26.3	END OF BOREHOLE AT 26.3m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 3.04m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) Apr 22/2015 5.9 136.1 Jun 03/2015 3.7 138.3 Jun 17/2015 5.5 136.5																









ONTMT4S 1205.GPJ 2015TEMPLATE(MTO).GDT 11/3/15

RECORD OF BOREHOLE No R-09

1 OF 4

METRIC

W.P. 2061-13-00 LOCATION W-N/S Ramp Leslie St. / CNR Overhead N 4 847 279.3 E 315 738.4 ORIGINATED BY ES
HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
DATUM Geodetic DATE 2015.04.13 - 2015.04.13 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
								20 40 60 80 100						
143.9	GROUND SURFACE													
0.0	TOPSOIL: (200mm)													
0.2	SAND and SILT, some clay, trace gravel Compact to Very Dense Brown Moist (FILL)		1	SS	26									
			2	SS	18									
			3	SS	60									
	Occasional rootlets													
			4	SS	55									
	Occasional inferred cobbles													
139.8														
4.1	Clayey SILT, mixed with organics, occasional roots Very Stiff Brown Moist		5	SS	19									
138.3	Sandy SILT, trace clay, trace gravel Compact Brown Moist													
			6	SS	16									
														
			7	SS	20									
														
135.1														
8.8	SILT, trace sand, trace clay Compact Brown Moist		8	SS	20									

Continued Next Page

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

METRIC

[illegible]

+³, ×³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No R-09

3 OF 4

METRIC

W.P. 2061-13-00 LOCATION W-N/S Ramp Leslie St. / CNR Overhead N 4 847 279.3 E 315 738.4 ORIGINATED BY ES
 HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2015.04.13 - 2015.04.13 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE					WATER CONTENT (%) W _p W W _L				
	Continued From Previous Page		13	SS	7												
	Silty CLAY , some to with sand, trace gravel Firm Grey Wet																
	Stiff Moist		14	SS	10												
119.5																	
24.4	Sandy SILT , some clay, trace gravel Very Dense Grey Moist (TILL)																
			15	SS	100/ 0.250											9 34 41 16	
			16	SS	104/ 0.300												
114.7			17	SS	108/ 0.225												
29.2	END OF BOREHOLE AT 29.1m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 3.04m slotted screen. BOREHOLE CAVED FROM 29.2m																

Continued Next Page

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

RECORD OF BOREHOLE No R-09

4 OF 4

METRIC

W.P. 2061-13-00 LOCATION W-N/S Ramp Leslie St. / CNR Overhead N 4 847 279.3 E 315 738.4 ORIGINATED BY ES
 HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2015.04.13 - 2015.04.13 CHECKED BY RPR

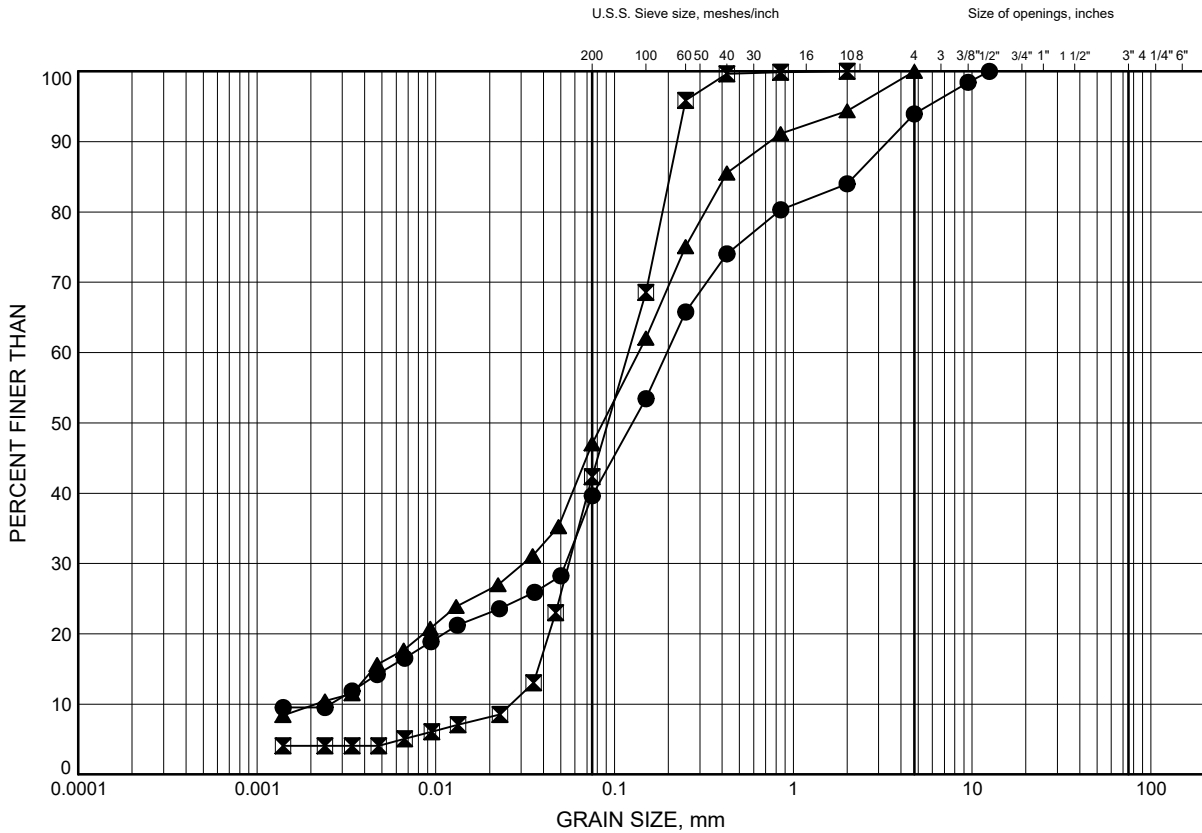
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)			
								20	40	60	80	100		20	40	60		GR	SA	SI	CL
	Continued From Previous Page																				
	TO 27.7m DEPTHS UPON COMPLETION.																				
	WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) Apr 22/2015 5.9 138.0 Jun 03/2015 5.6 138.3 Jun 17/2015 3.7 140.2																				

Hwy 401 Leslie Street 2013-E-0032

GRAIN SIZE DISTRIBUTION

FIGURE A1

SAND & SILT FILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	PB-02	1.83	140.27
⊠	R-05	2.59	138.21
▲	R-08	2.59	139.41

Date November 2015
W.P. 2061-13-00



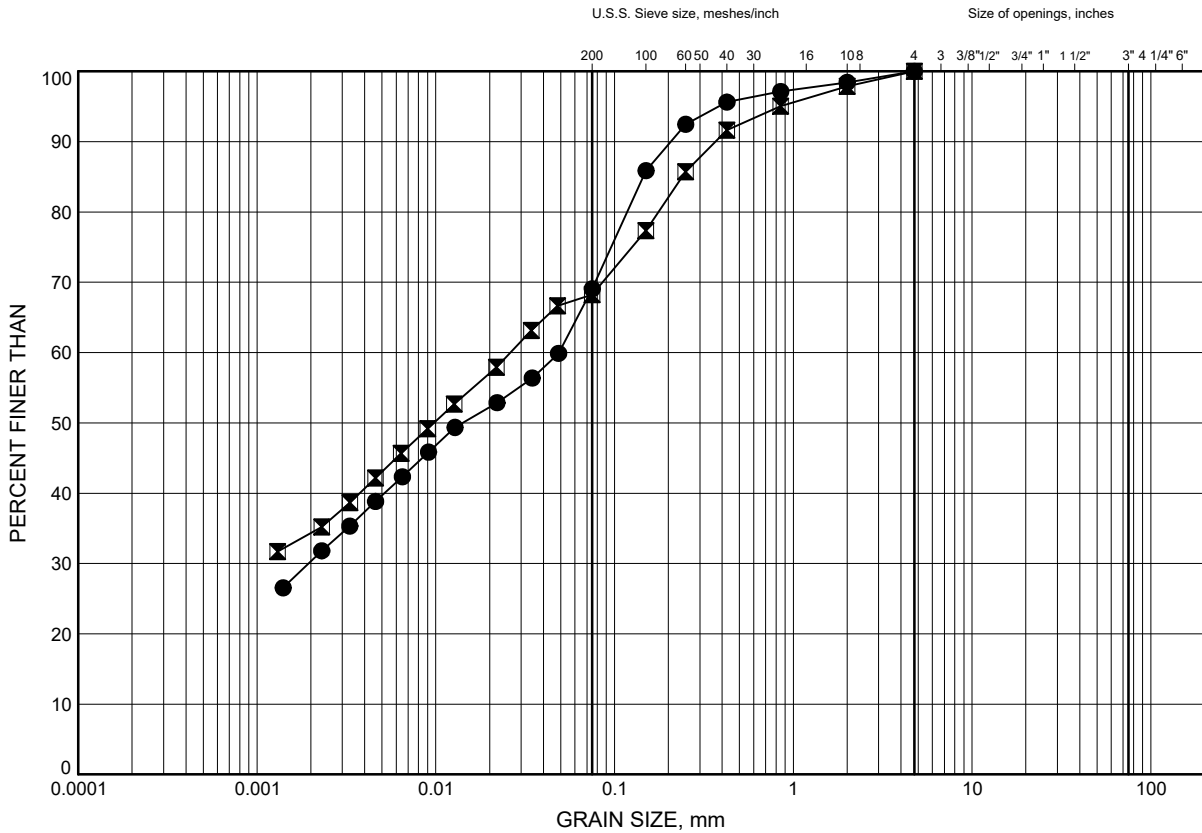
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Hwy 401 Leslie Street 2013-E-0032

GRAIN SIZE DISTRIBUTION

FIGURE A2

SILTY CLAY FILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	R-02	1.83	143.87
⊠	R-03	2.59	142.31

Date November 2015
W.P. 2061-13-00

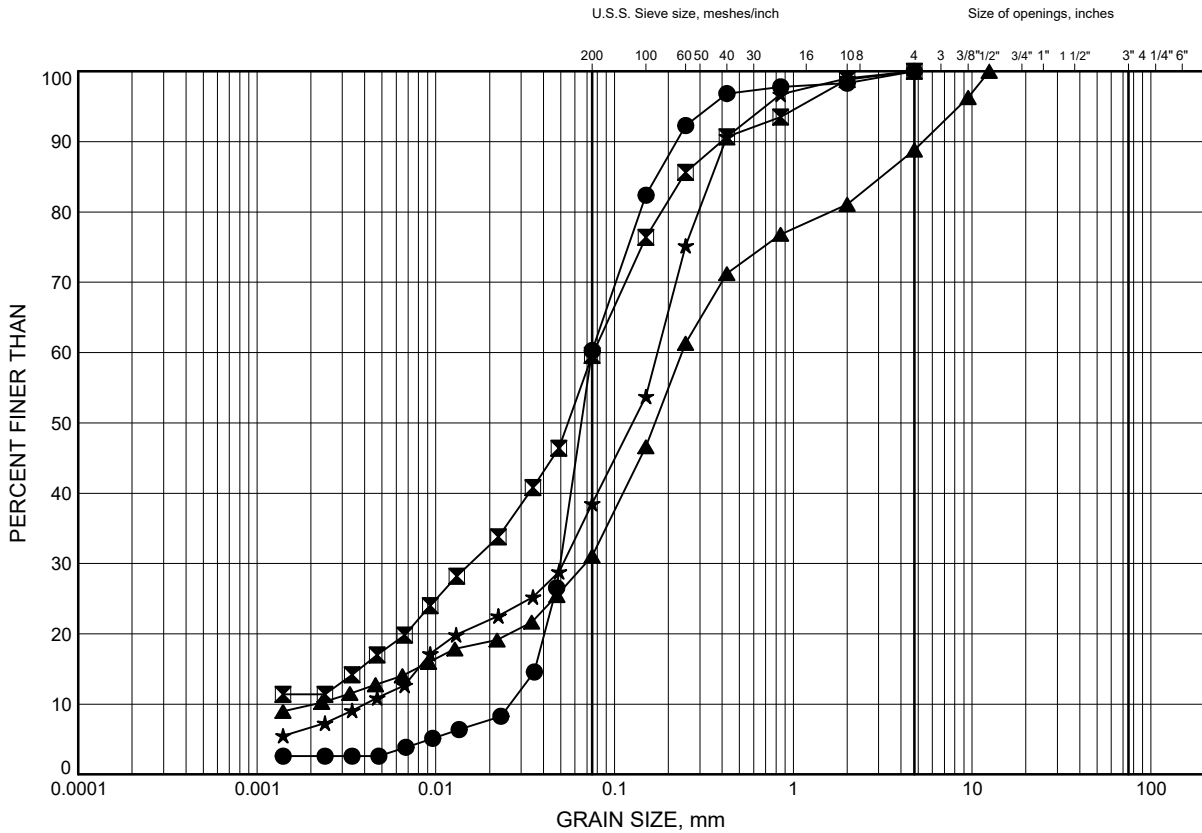


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Hwy 401 Leslie Street 2013-E-0032
GRAIN SIZE DISTRIBUTION

FIGURE A3

SAND & SILT to SILTY SAND



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	PB-02	6.40	135.70
⊠	R-01	6.40	141.70
▲	R-03	4.88	140.02
★	R-04	3.35	139.85

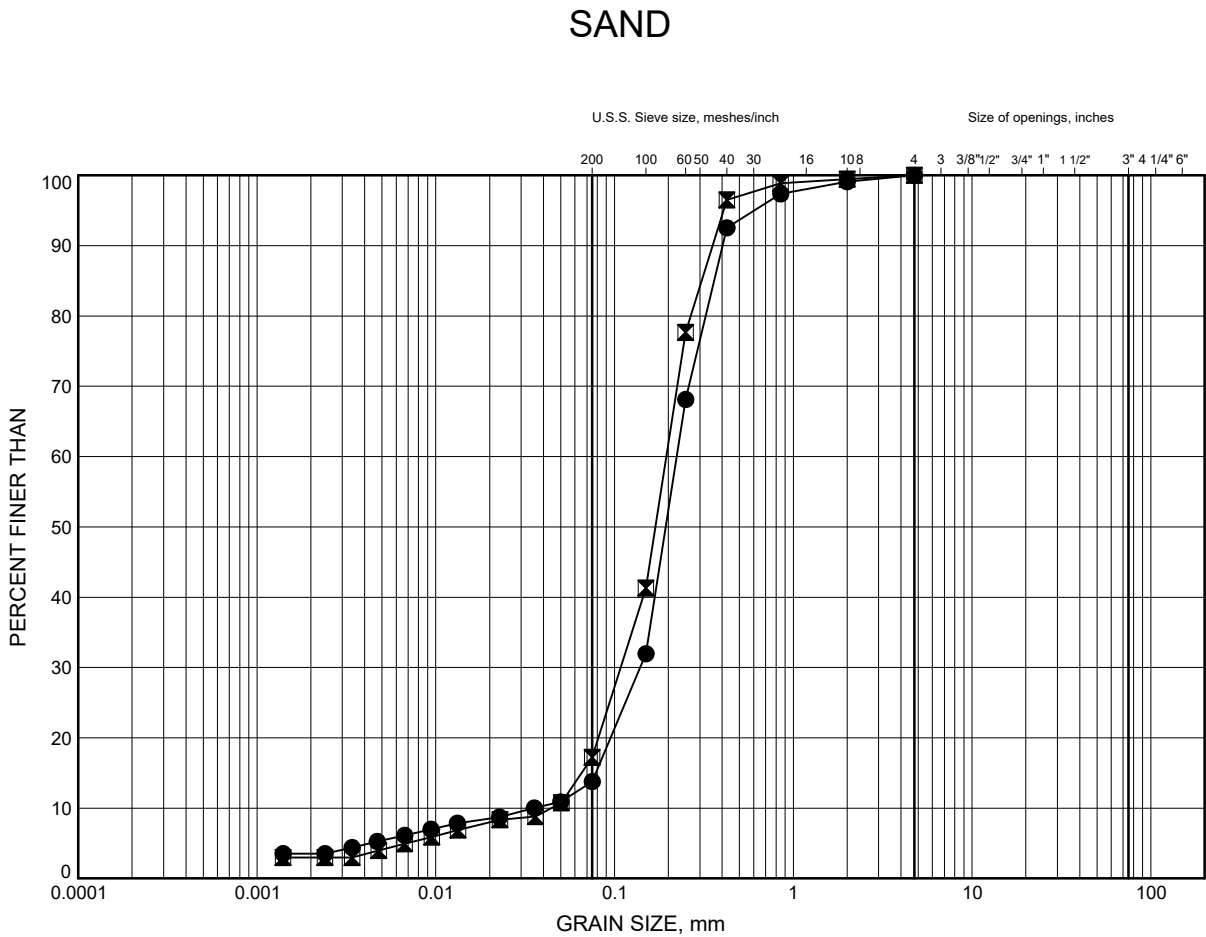
Date November 2015
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Hwy 401 Leslie Street 2013-E-0032
GRAIN SIZE DISTRIBUTION

FIGURE A4



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	R-03	23.16	121.74
⊠	R-07	3.35	137.35

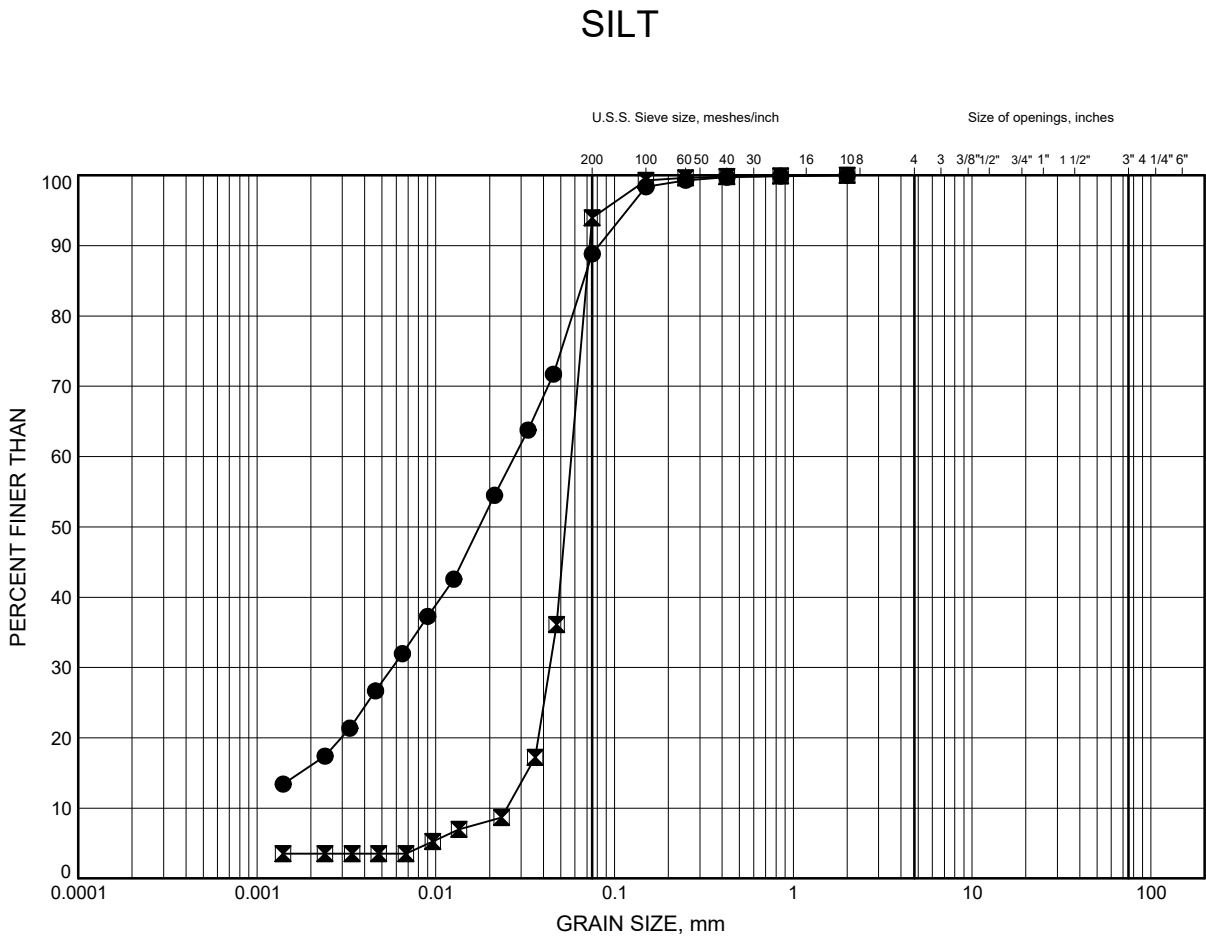
Date November 2015
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GRAIN SIZE DISTRIBUTION

FIGURE A5



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	R-01	17.07	131.03
⊠	R-09	9.45	134.45

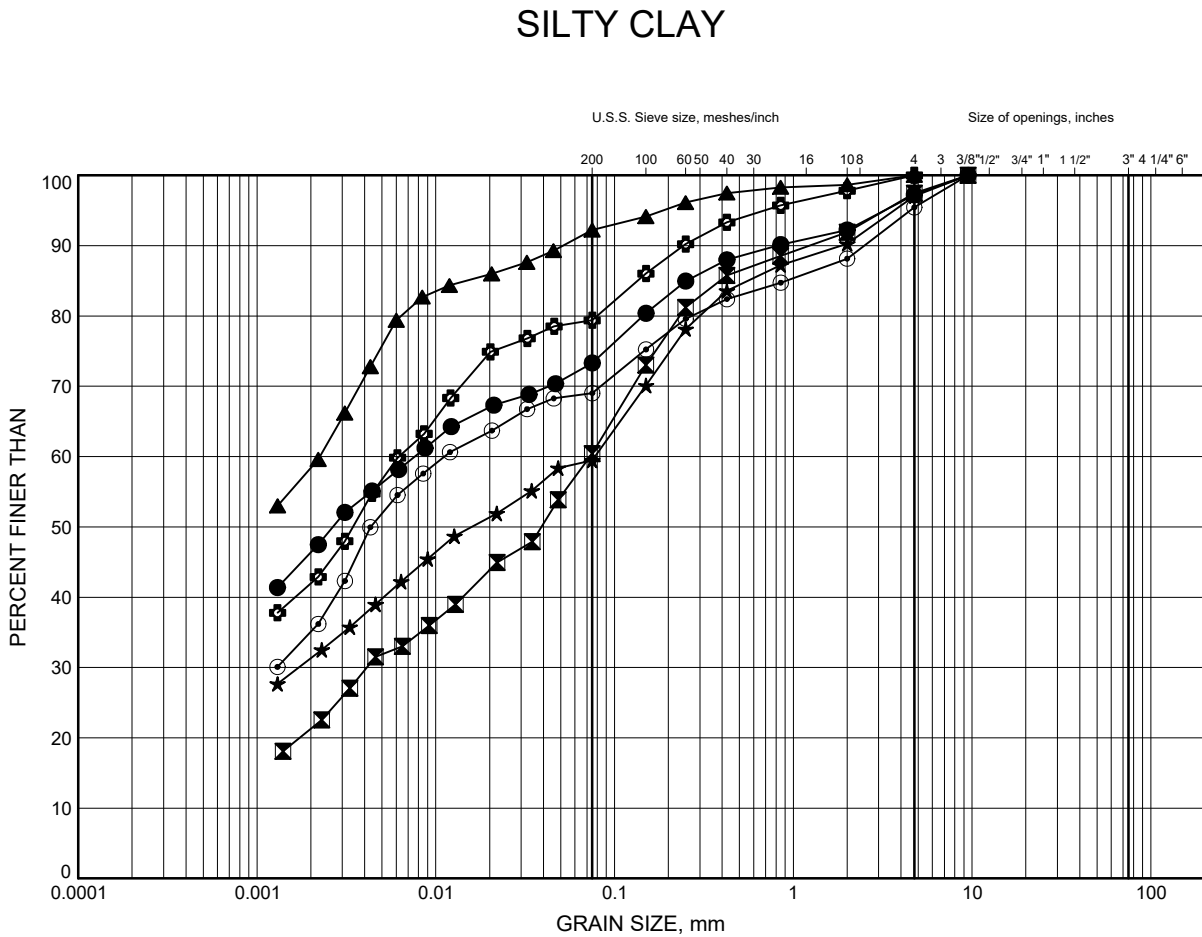
Date November 2015
W.P. 2061-13-00



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Hwy 401 Leslie Street 2013-E-0032
GRAIN SIZE DISTRIBUTION

FIGURE A6



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	PB-02	14.02	128.08
⊠	PB-02	17.07	125.03
▲	R-01	2.59	145.51
★	R-02	7.92	137.78
⊙	R-02	14.02	131.68
⊕	R-03	9.45	135.45

Date November 2015
W.P. 2061-13-00

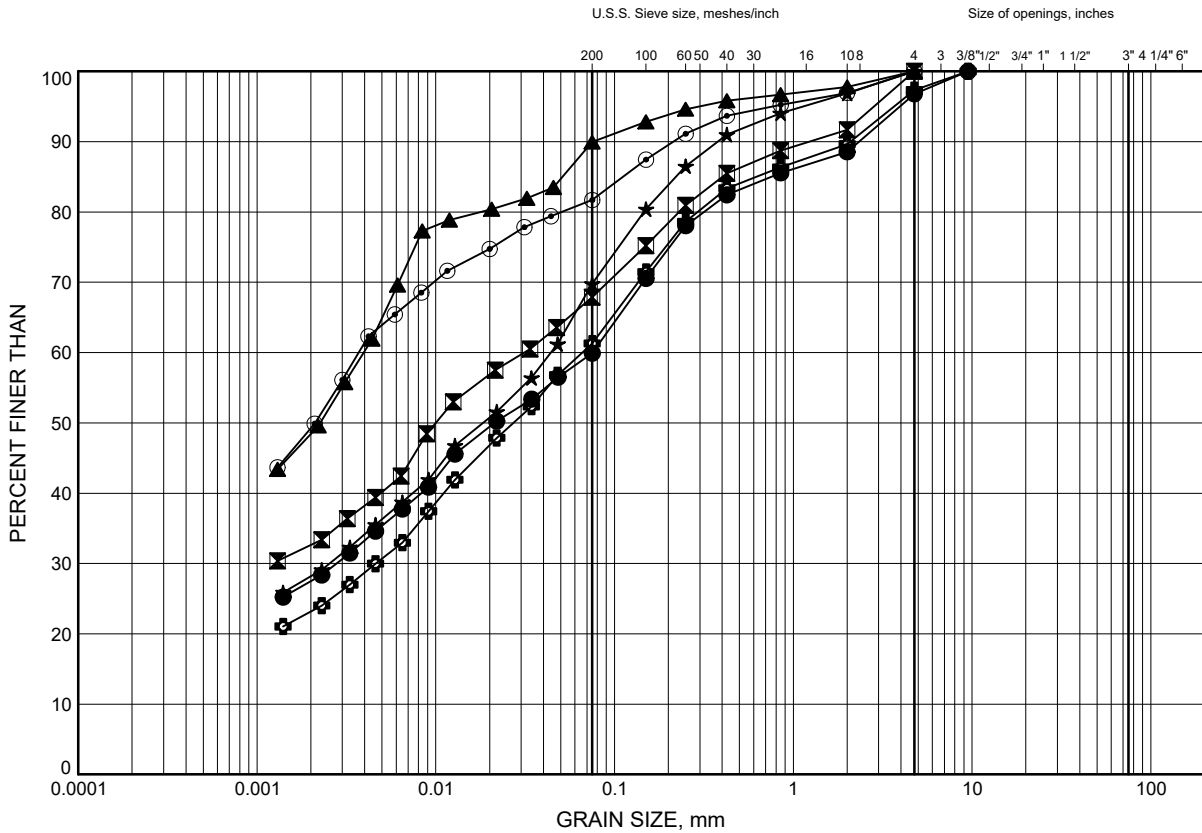


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Hwy 401 Leslie Street 2013-E-0032
GRAIN SIZE DISTRIBUTION

FIGURE A7

SILTY CLAY



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	R-03	17.07	127.83
⊠	R-04	7.92	135.28
▲	R-04	10.97	132.23
★	R-04	14.02	129.18
⊙	R-05	7.92	132.88
⊕	R-05	15.54	125.26

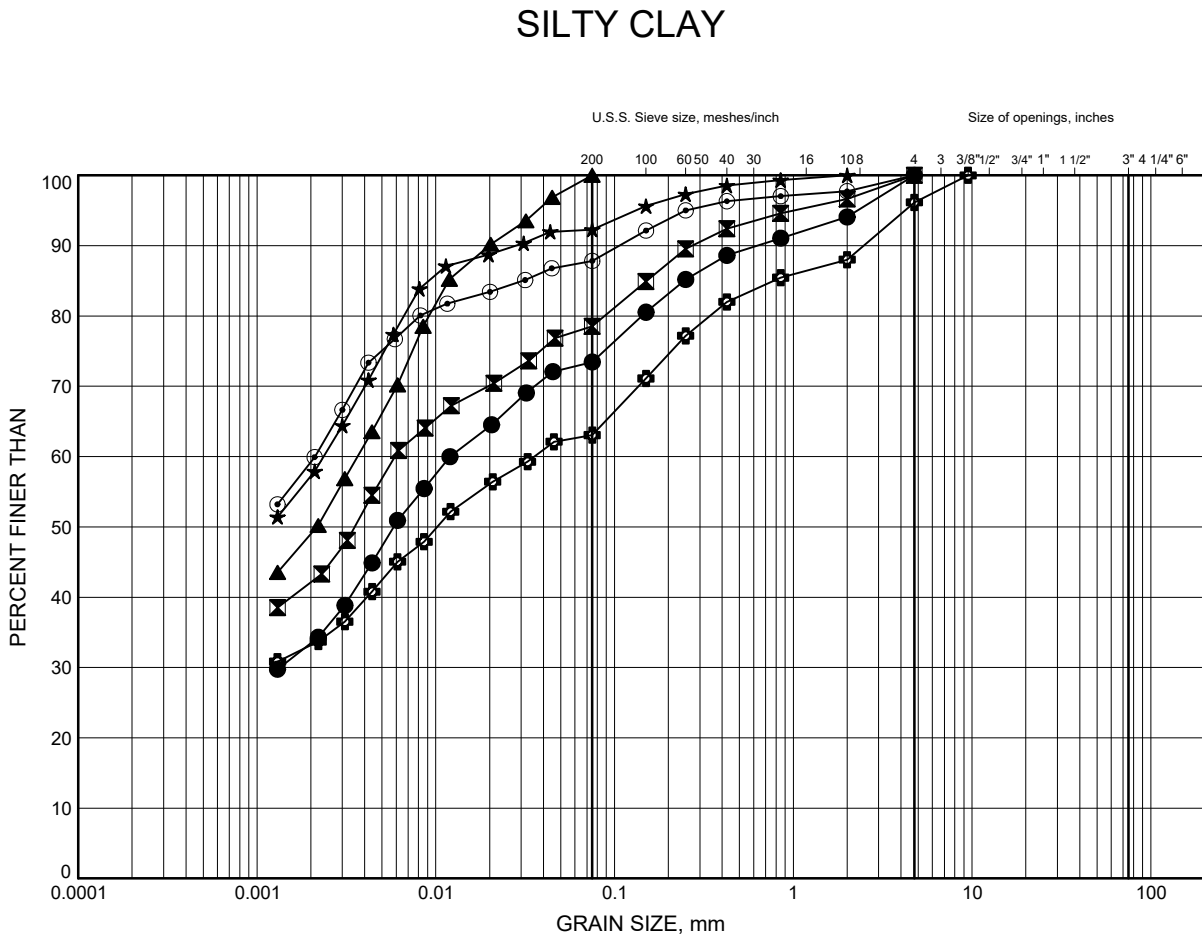
Date November 2015
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Hwy 401 Leslie Street 2013-E-0032
GRAIN SIZE DISTRIBUTION

FIGURE A8



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	R-07	10.97	129.73
⊠	R-07	17.07	123.63
▲	R-08	6.40	135.60
★	R-08	14.02	127.98
⊙	R-09	15.54	128.36
⊕	R-09	18.59	125.31

Date November 2015
W.P. 2061-13-00

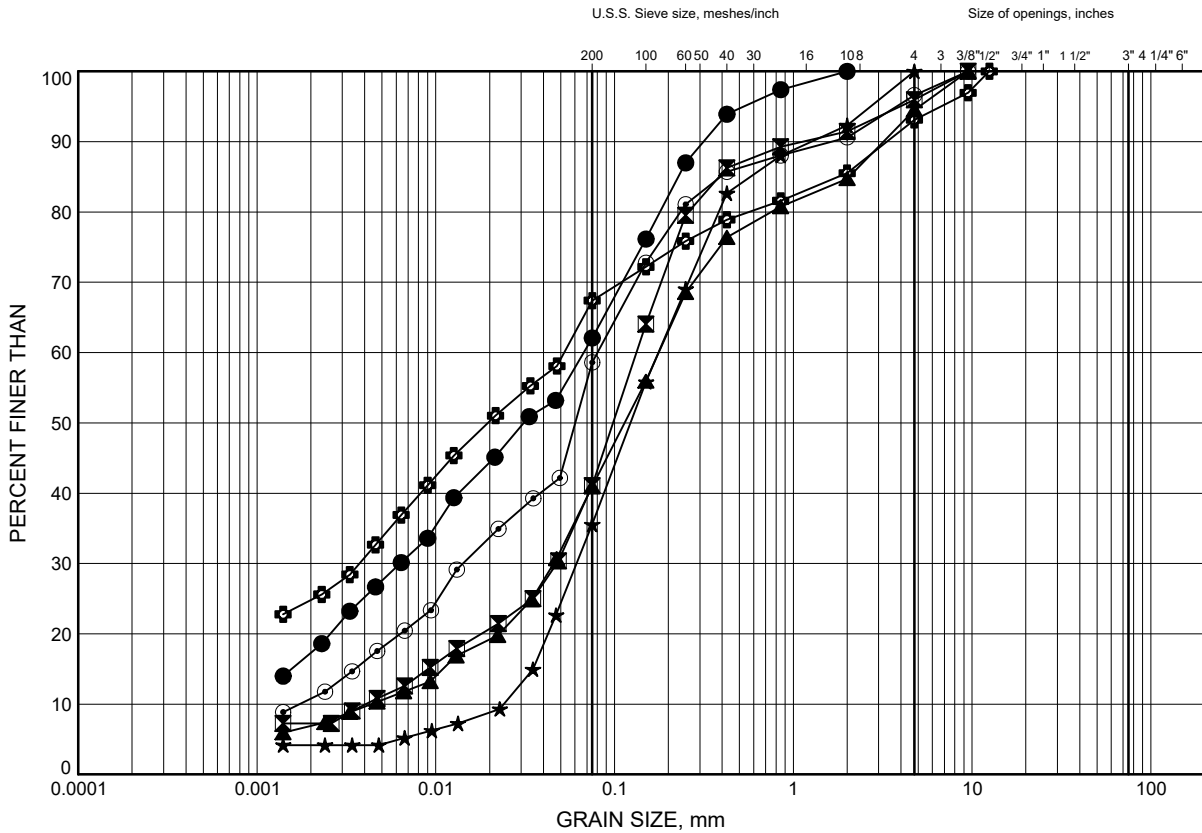


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Hwy 401 Leslie Street 2013-E-0032
GRAIN SIZE DISTRIBUTION

FIGURE A9

SANDY SILT to SILTY SAND TILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	R-01	12.50	135.60
⊠	R-02	18.59	127.11
▲	R-02	23.16	122.54
★	R-03	29.26	115.64
⊙	R-04	20.12	123.08
⊕	R-04	24.69	118.51

Date November 2015
W.P. 2061-13-00

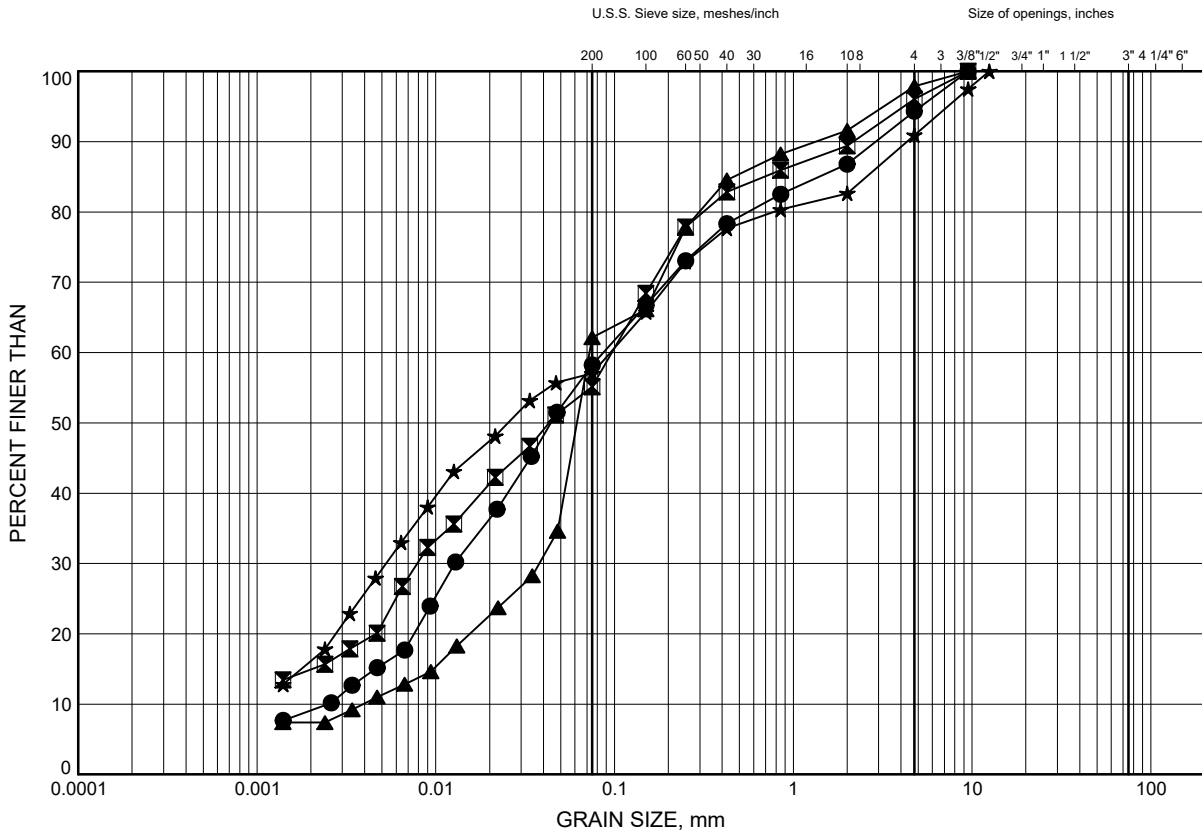


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Hwy 401 Leslie Street 2013-E-0032
GRAIN SIZE DISTRIBUTION

FIGURE A10

SANDY SILT to SILTY SAND TILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	R-05	21.64	119.16
⊠	R-07	21.64	119.06
▲	R-08	23.16	118.84
★	R-09	26.21	117.69

Date November 2015
W.P. 2061-13-00



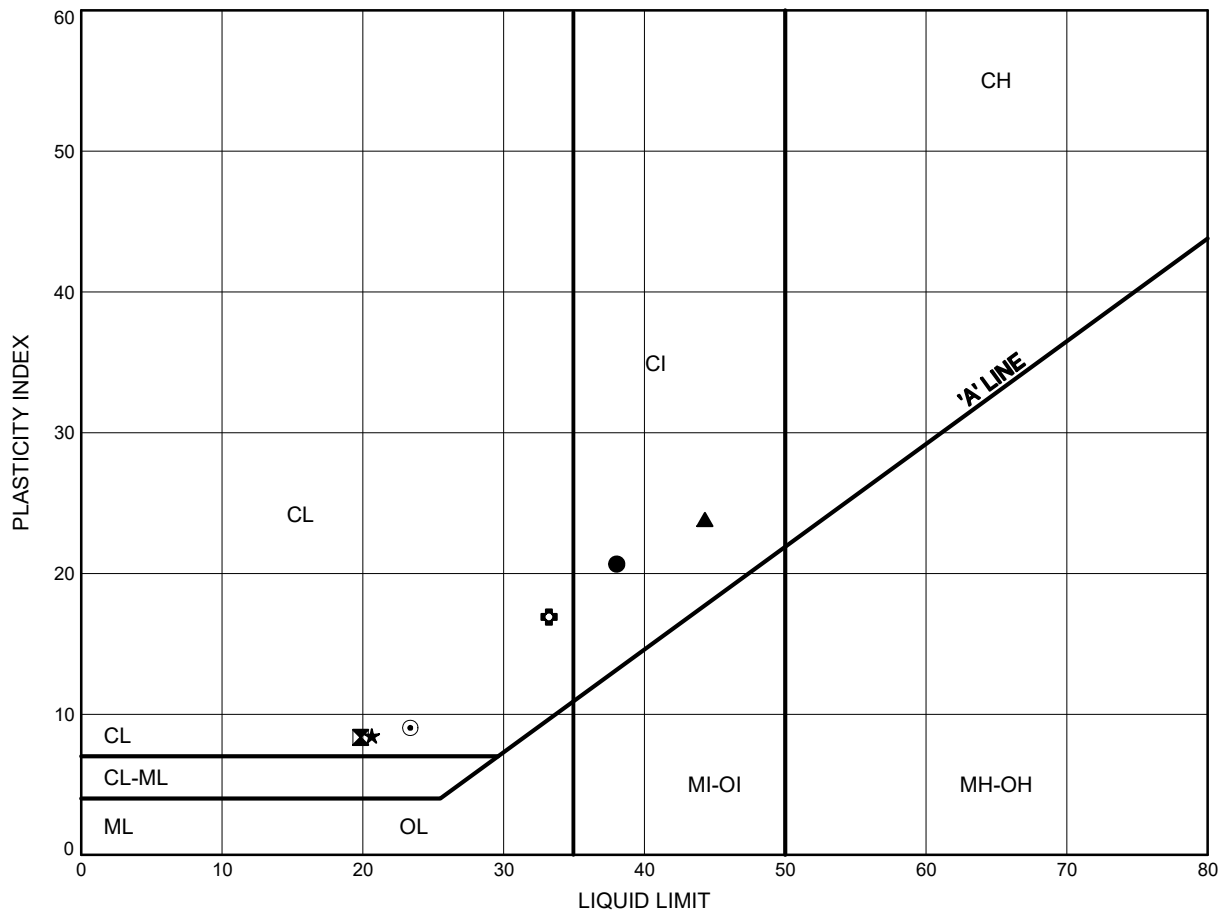
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Hwy 401 Leslie Street 2013-E-0032

ATTERBERG LIMITS TEST RESULTS

FIGURE A11

SILTY CLAY



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	PB-02	14.02	128.08
⊠	PB-02	17.07	125.03
▲	R-01	2.59	145.51
★	R-02	7.92	137.78
⊙	R-02	14.02	131.68
⊕	R-03	9.45	135.45

Date November 2015
W.P. 2061-13-00



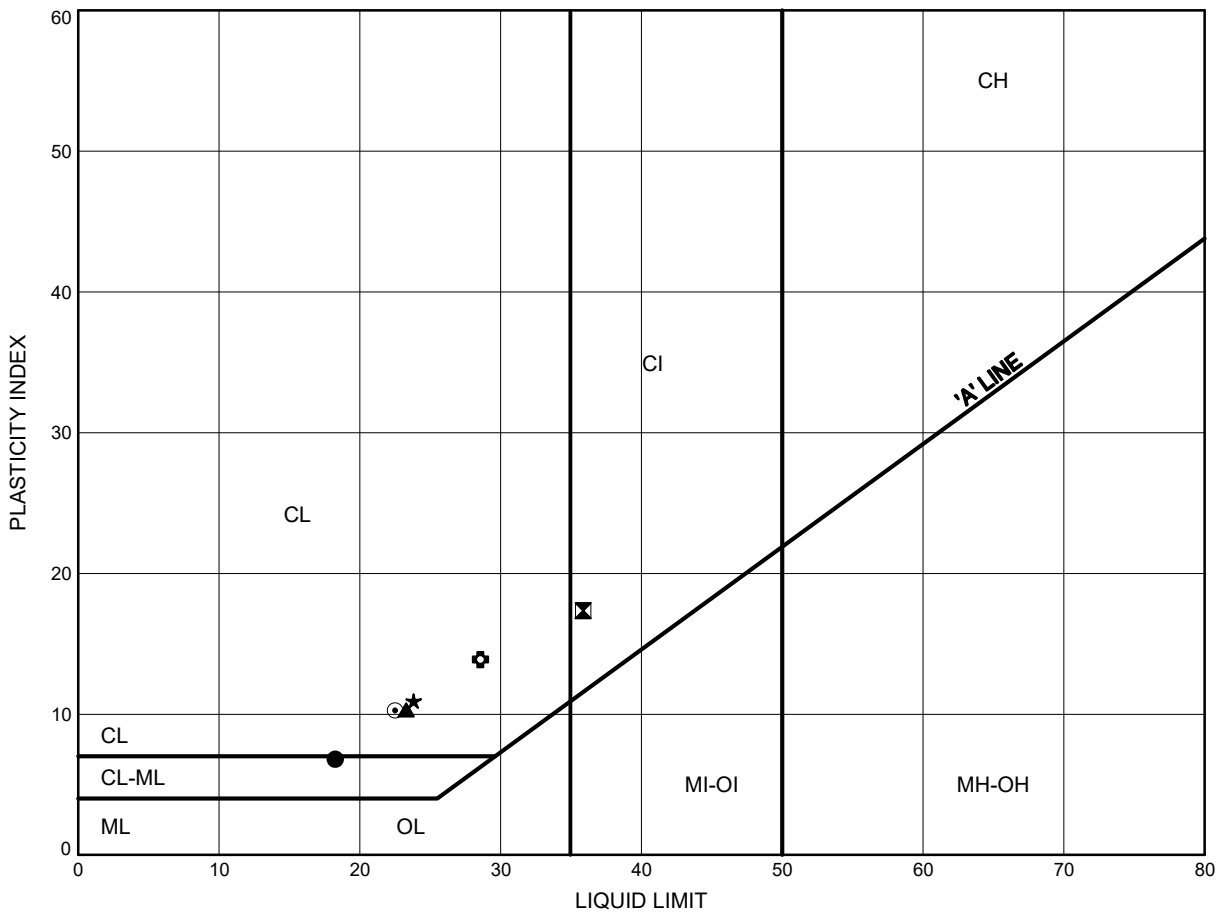
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Hwy 401 Leslie Street 2013-E-0032

ATTERBERG LIMITS TEST RESULTS

FIGURE A12

SILTY CLAY



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	R-03	17.07	127.83
⊠	R-04	10.97	132.23
▲	R-04	14.02	129.18
★	R-05	7.92	132.88
⊙	R-05	15.54	125.26
⊕	R-07	10.97	129.73

Date November 2015
W.P. 2061-13-00



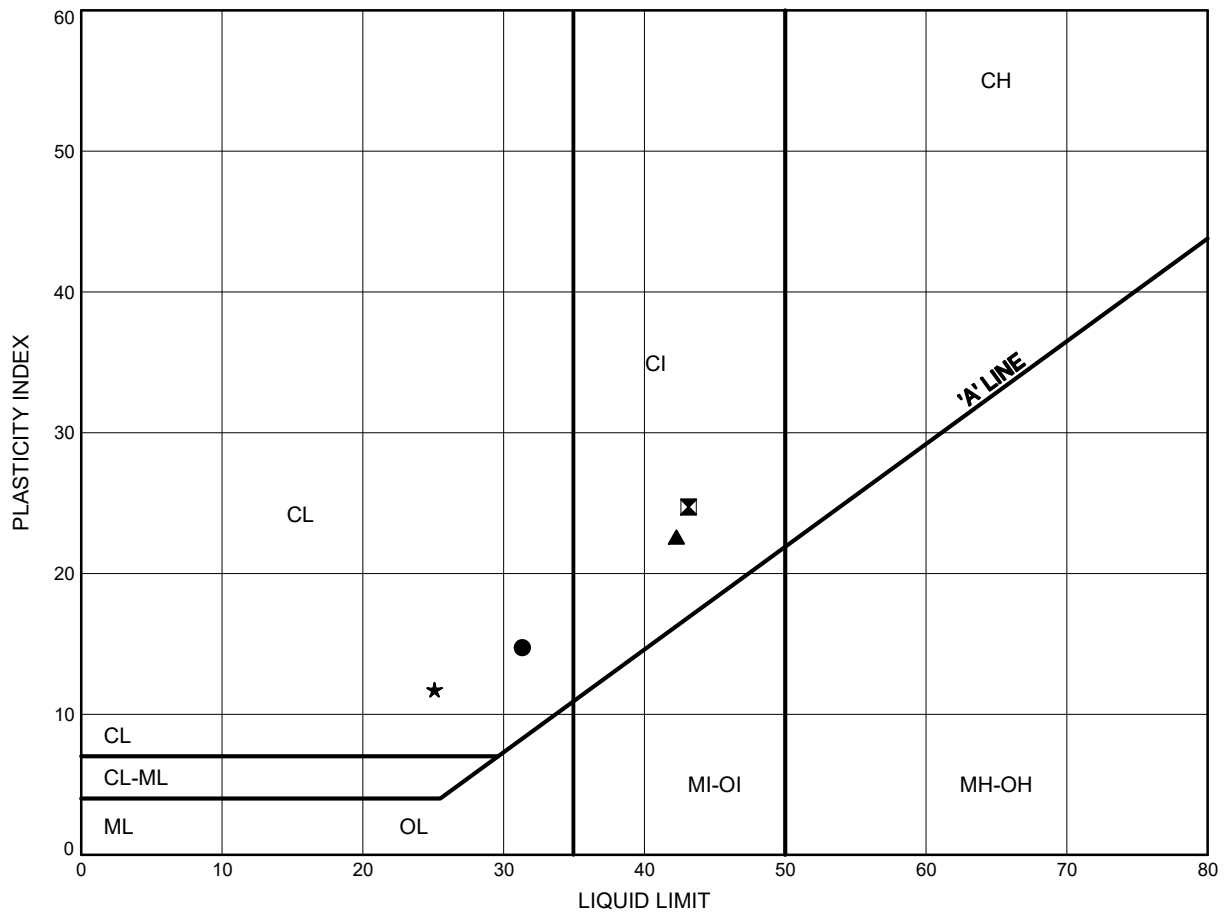
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Hwy 401 Leslie Street 2013-E-0032

ATTERBERG LIMITS TEST RESULTS

FIGURE A13

SILTY CLAY



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	R-07	17.07	123.63
⊠	R-08	14.02	127.98
▲	R-09	15.54	128.36
★	R-09	18.59	125.31

Date November 2015
W.P. 2061-13-00



Prep'd AN
Chkd. RPR

Consolidation Test Report

CLIENT: **MMM Group Limited**

FILE NUMBER: **19-5161-205**

PROJECT: **Highway 401 and Leslie Foundations**

REPORT DATE: **9-Apr-2015**

TEST DATES: **March 24, 2015 - April 06, 2015**

SAMPLE: **BH R-04 TW1 (35' - 37')**
Silty Clay, grey, 10% Sand, 42% Silt and 48% Clay, LL=35.9%, PL=18.5%.

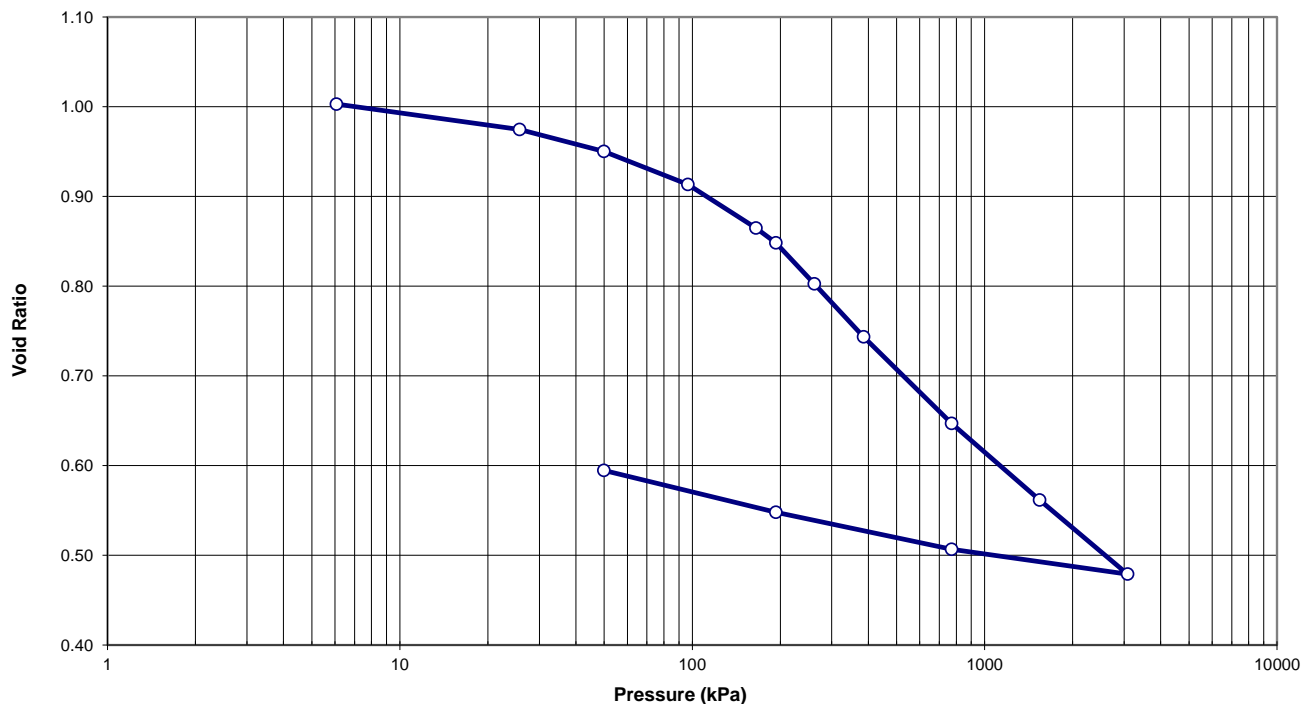
PROCEDURE: Test carried out in accordance with Standard Test Method for One-Dimensional Consolidation Properties of Soils, ASTM D 2435-04, method A

	<u>Start of Test</u>	<u>End of Test</u>
Wet Dens. (kg/m^3)	1876.4	2139.0
Dry Dens. (kg/m^3)	1366.6	1726.0
Moisture Cont. (%)	37.3	23.9
Void Ratio	1.014	0.595

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

Void Ratio vs. Pressure

Project #: 19-5161-205
 Client: MMM Group Limited
 Project Name: Highway 401 and Leslie Foundations
 Sample: BH-R-04-TW1 (35' - 37')



Consolidation Test Report

Highway 401 and Leslie Foundations

19-5161-205

BH R-04 TW1 (35' - 37')

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

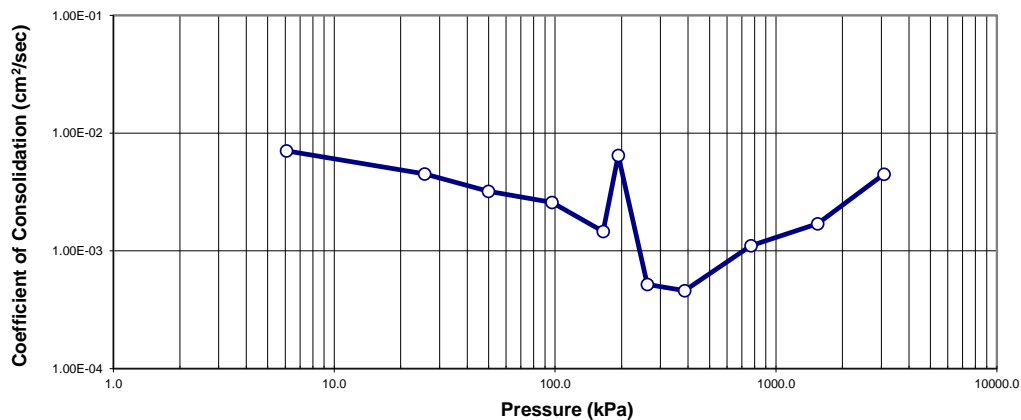
LOADING: A seating load of 6.1 kPa was applied and the consolidometer was flooded with distilled water. Sample was monitored to ensure no swelling effect occurred before the start of the test. Subsequent loads were applied after a constant load increment duration of 24 hours.

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	25.400					1.014		
6.1	25.261	25.331	-0.069	3.20	7.08E-03	1.003	9.01E-04	6.26E-07
25.7	24.903	25.082	-0.180	4.93	4.51E-03	0.975	7.23E-04	3.20E-07
49.9	24.597	24.750	-0.133	6.76	3.20E-03	0.950	5.08E-04	1.59E-07
96.6	24.132	24.365	-0.230	8.12	2.58E-03	0.913	4.04E-04	1.02E-07
164.9	23.519	23.826	-0.288	13.69	1.47E-03	0.865	3.72E-04	5.35E-08
193.2	23.310	23.415	-0.032	2.99	6.47E-03	0.848	3.14E-04	1.99E-07
261.5	22.733	23.022	-0.260	36.00	5.20E-04	0.803	3.63E-04	1.85E-08
385.7	21.989	22.361	-0.450	38.44	4.60E-04	0.744	2.63E-04	1.19E-08
770.7	20.773	21.381	-0.785	14.59	1.11E-03	0.647	1.44E-04	1.56E-08
1540.7	19.695	20.234	-0.650	8.53	1.70E-03	0.562	6.74E-05	1.12E-08
3081.4	18.652	19.174	-0.460	2.89	4.49E-03	0.479	3.44E-05	1.51E-08
770.7	19.002	18.827				0.507		
193.2	19.521	19.262				0.548		
49.9	20.111	19.816				0.595		

Coefficient of Consolidation vs. Pressure

Project #: 19-5161-205
Client: MMM Group Limited
Project Name: Highway 401 and Leslie Foundations
Sample: BH-R-04-TW1 (35' - 37')



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

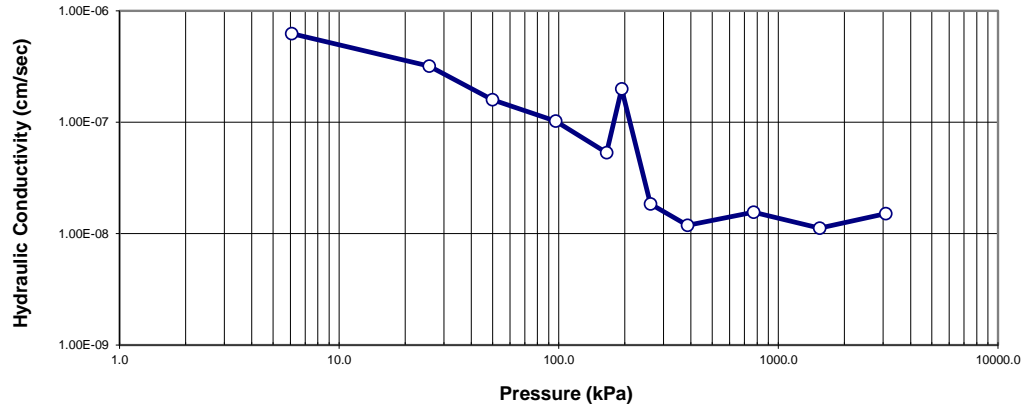
Consolidation Test Report

Highway 401 and Leslie Foundations
19-5161-205

BH R-04 TW1 (35' - 37')

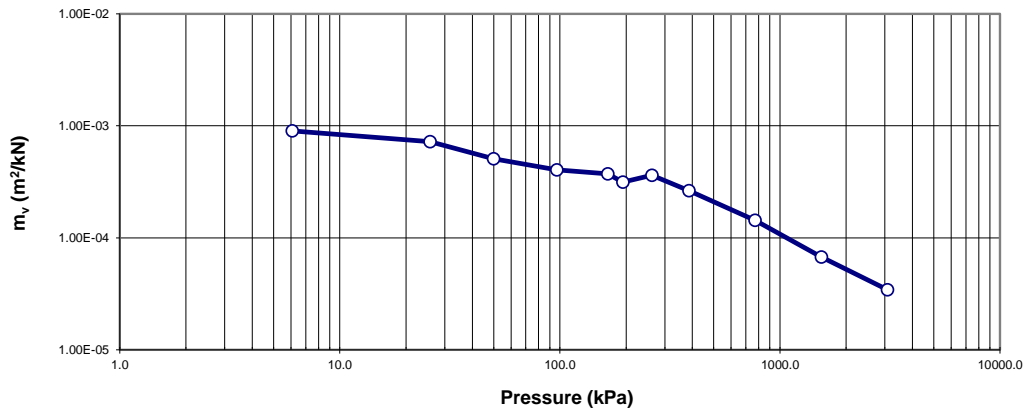
Hydraulic Conductivity vs. Pressure

Project #: 19-5161-205
Client: MMM Group Limited
Project Name: Highway 401 and Leslie Foundations
Sample: BH-R-04-TW1 (35' - 37')



m_v vs. Pressure

Project #: 19-5161-205
Client: MMM Group Limited
Project Name: Highway 401 and Leslie Foundations
Sample: BH-R-04-TW1 (35' - 37')



Consolidation Test Report

CLIENT: **MMM Group Limited**

FILE NUMBER: **19-5161-205**

PROJECT: **Highway 401 & Leslie Foundations**

REPORT DATE: **25-Aug-2015**

TEST DATES: **August 05, 2015 - August 17, 2015**

SAMPLE: **BH R8-TW1 (40' - 42')**
Silty Clay, grey, 38% Silt and 62% Clay, PL=23%, LL=47%

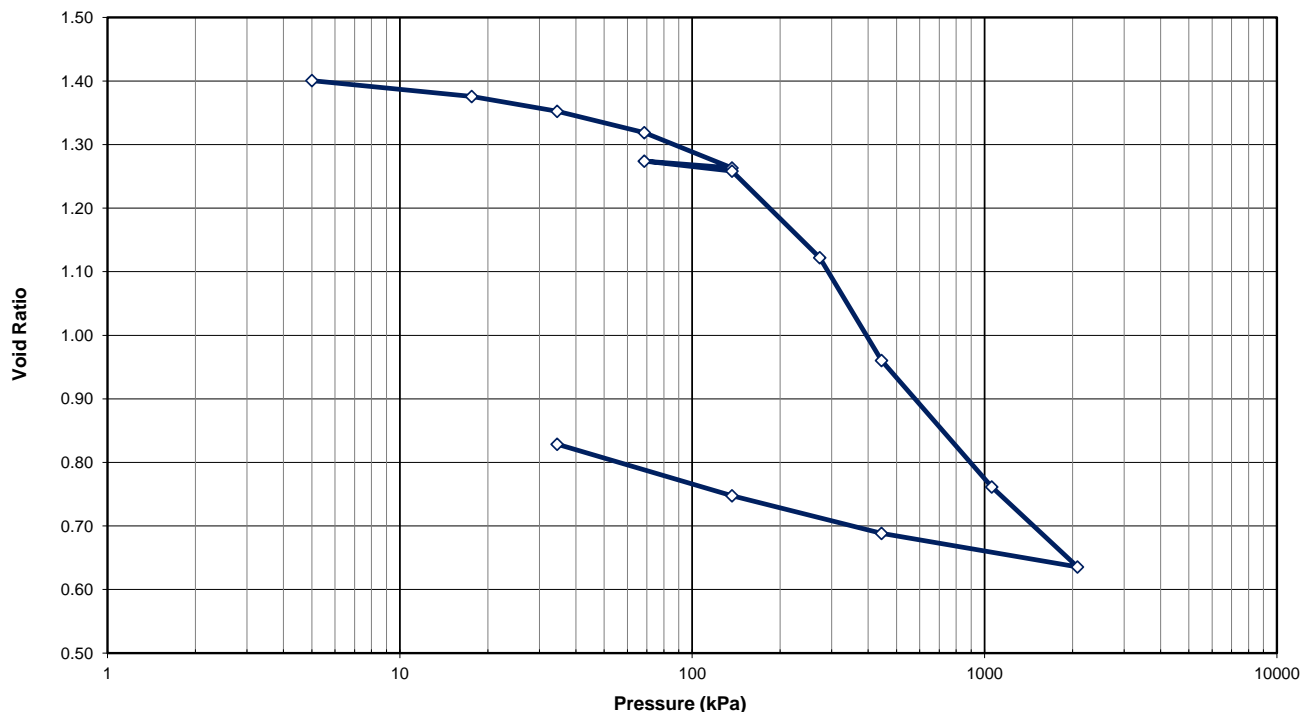
PROCEDURE: Tested in accordance with Standard Test Method for One-Dimensional Consolidation Properties of Soils, ASTM D 2435-04, method B

	<u>Start of Test</u>	<u>End of Test</u>
Wet Dens. (kg/m ³)	1741.9	2011.4
Dry Dens. (kg/m ³)	1143.1	1505.5
Moisture Cont. (%)	52.4	33.6
Void Ratio	1.408	0.829

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

Void Ratio vs Pressure

Project #: 19-5161-205
 Client: MMM Group Limited
 Project Name: Highway 401 & Leslie Foundations
 Sample: BH R8-TW1 (40' - 42')



Consolidation Test Report

Highway 401 & Leslie Foundations
19-5161-205

BH R8-TW1 (40' - 42')

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

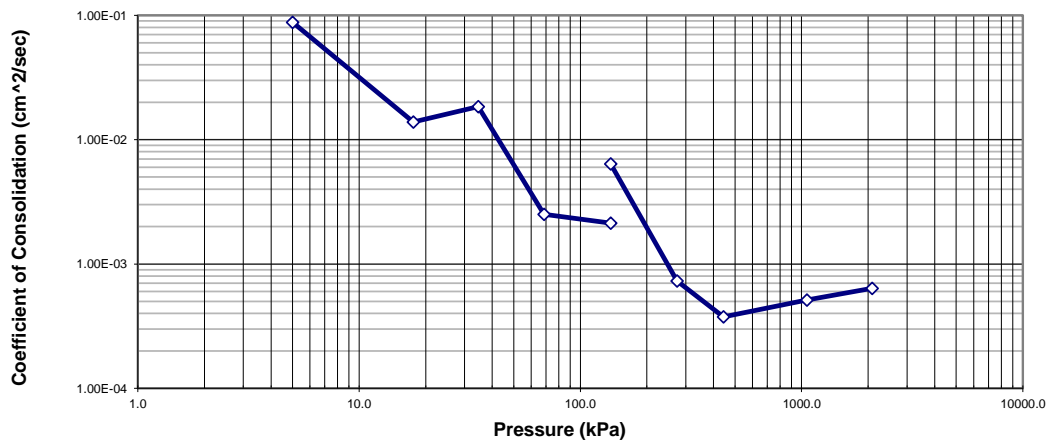
LOADING: A seating load of 5.0 kPa was applied and the consolidometer was flooded with distilled water. Sample was monitored to ensure no swelling effect occurred before the start of the test. Subsequent loads were applied after 100% primary consolidation was reached.

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

Pressure (kPa)	Corr. Hgt (mm)	Avg. Hgt. (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					1.408		
5.0	19.935	19.968	-0.024	0.16	8.80E-02	1.401	6.50E-04	5.61E-06
17.6	19.730	19.833	-0.072	1.00	1.39E-02	1.376	8.16E-04	1.11E-06
34.5	19.536	19.633	-0.048	0.74	1.84E-02	1.353	5.82E-04	1.05E-06
68.5	19.256	19.396	-0.145	5.29	2.51E-03	1.319	4.22E-04	1.04E-07
136.9	18.793	19.025	-0.257	6.00	2.13E-03	1.263	3.52E-04	7.34E-08
68.5	18.883	18.838				1.274		
136.9	18.755	18.819	-0.067	1.96	6.38E-03	1.259	9.91E-05	6.20E-08
273.2	17.621	18.188	-0.535	16.00	7.31E-04	1.122	4.44E-04	3.18E-08
443.8	16.279	16.950	-0.960	27.04	3.75E-04	0.960	4.46E-04	1.64E-08
1057.7	14.629	15.454	-1.220	16.40	5.14E-04	0.762	1.65E-04	8.33E-09
2080.1	13.582	14.106	-0.696	11.02	6.38E-04	0.636	7.00E-05	4.38E-09
443.8	14.020	13.801				0.688		
136.9	14.511	14.266				0.747		
34.5	15.185	14.848				0.829		

Coefficient of Consolidation vs Pressure

Project #: 19-5161-205
Client: MMM Group Limited
Project Name: Highway 401 & Leslie Foundations
Sample: BH R8-TW1 (40' - 42')



Notes: C_v and k calculated using t₉₀ values

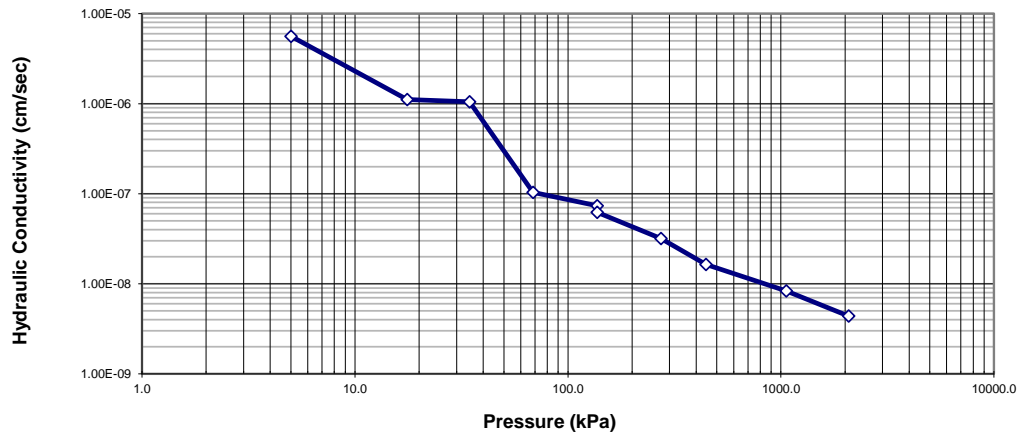
Consolidation Test Report

Highway 401 & Leslie Foundations
19-5161-205

BH R8-TW1 (40' - 42')

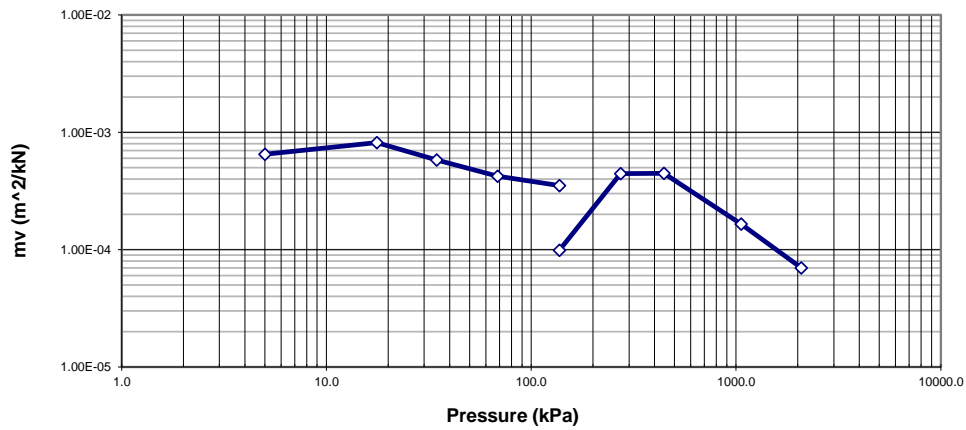
Hydraulic Conductivity vs Pressure

Project #: 19-5161-205
Client: MMM Group Limited
Project Name: Highway 401 & Leslie Foundations
Sample: BH R8-TW1 (40' - 42')



mv vs Pressure

Project #: 19-5161-205
Client: MMM Group Limited
Project Name: Highway 401 & Leslie Foundations
Sample: BH R8-TW1 (40' - 42')



Consolidation Test Report

CLIENT: **MMM Group Limited**

FILE NUMBER: **19-5161-205**

PROJECT: **Highway 401 and Leslie Foundations**

REPORT DATE: **14-May-2015**

TEST DATES: **April 20, 2015 - May 01, 2015**

SAMPLE: **BH R-09 TW2 (50' - 52')**
Silty Clay, grey, 12% Sand, 29% Silt and 59% Clay, LL=42.3%, PL=19.7%.

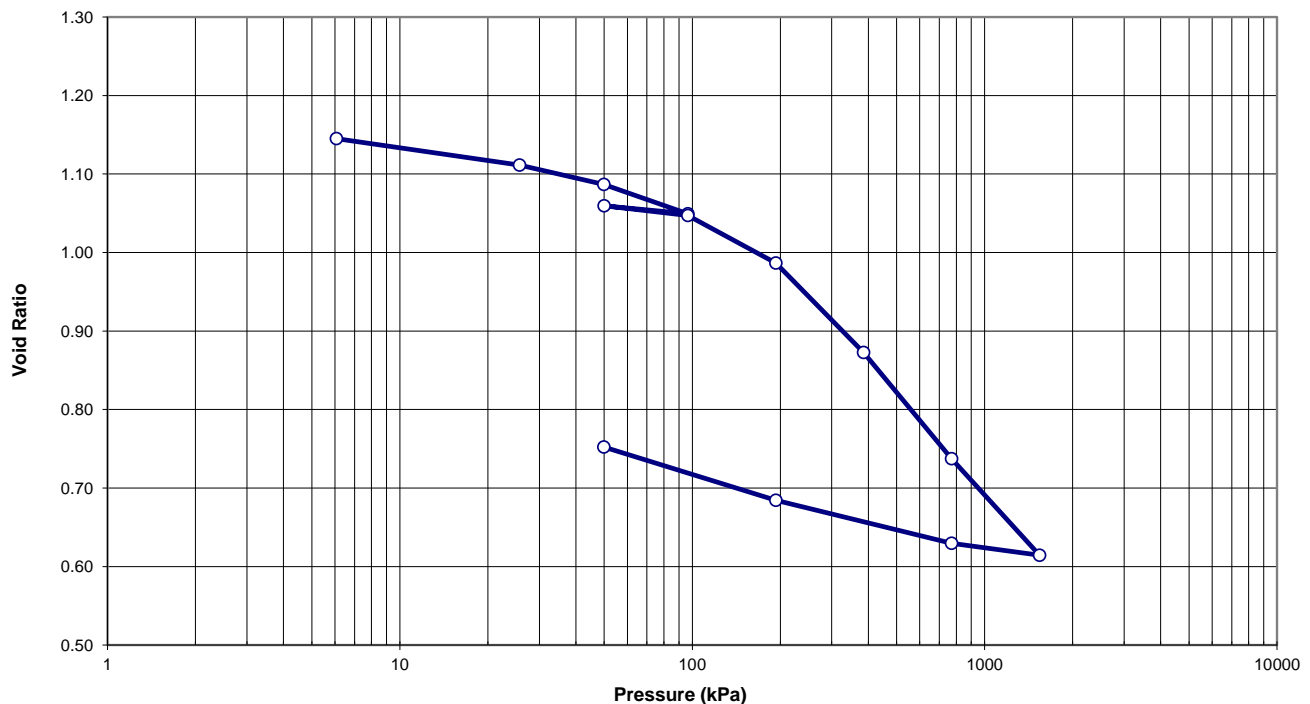
PROCEDURE: Test carried out in accordance with Standard Test Method for One-Dimensional Consolidation Properties of Soils, ASTM D 2435-04, method A

	<u>Start of Test</u>	<u>End of Test</u>
Wet Dens. (kg/m ³)	1843.0	2070.0
Dry Dens. (kg/m ³)	1281.1	1570.4
Moisture Cont. (%)	39.1	31.8
Void Ratio	1.148	0.752

Note: A Specific Gravity (Gs) of 2.75 was assumed for the void ratio and saturation calculations.

Void Ratio vs. Pressure

Project #: 19-5161-205
 Client: MMM Group Limited
 Project Name: Highway 401 and Leslie Foundations
 Sample: BH R-09 TW2 (50' - 52')



Consolidation Test Report

Highway 401 and Leslie Foundations

19-5161-205

BH R-09 TW2 (50' - 52')

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

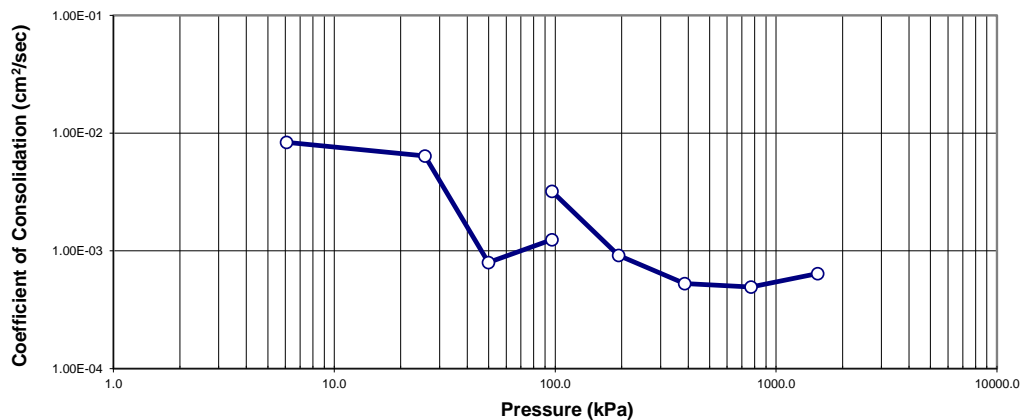
LOADING: A seating load of 6.1 kPa was applied and the consolidometer was flooded with distilled water. Sample was monitored to ensure no swelling effect occurred before the start of the test. Subsequent loads were applied after a constant load increment duration of 24 hours.

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	25.400					1.148		
6.1	25.365	25.383	-0.044	2.72	8.36E-03	1.145	2.27E-04	1.86E-07
25.7	24.968	25.167	-0.214	3.50	6.40E-03	1.111	7.99E-04	5.01E-07
49.9	24.675	24.822	-0.200	27.35	7.96E-04	1.087	4.85E-04	3.79E-08
96.6	24.234	24.455	-0.265	16.97	1.24E-03	1.049	3.82E-04	4.66E-08
49.9	24.353	24.294				1.059		
96.6	24.208	24.281	-0.089	6.50	3.20E-03	1.047	1.28E-04	4.01E-08
193.2	23.492	23.850	-0.434	21.90	9.18E-04	0.987	3.06E-04	2.75E-08
385.7	22.143	22.818	-0.900	34.81	5.28E-04	0.873	2.98E-04	1.55E-08
770.7	20.544	21.344	-1.140	32.60	4.94E-04	0.737	1.88E-04	9.08E-09
1540.7	19.091	19.818	-1.000	21.62	6.42E-04	0.614	9.19E-05	5.78E-09
770.7	19.269	19.180				0.629		
193.2	19.919	19.594				0.684		
49.9	20.720	20.320				0.752		

Coefficient of Consolidation vs. Pressure

Project #: 19-5161-205
Client: MMM Group Limited
Project Name: Highway 401 and Leslie Foundations
Sample: BH R-09 TW2 (50' - 52')



Notes: C_v and k calculated using t₉₀ values

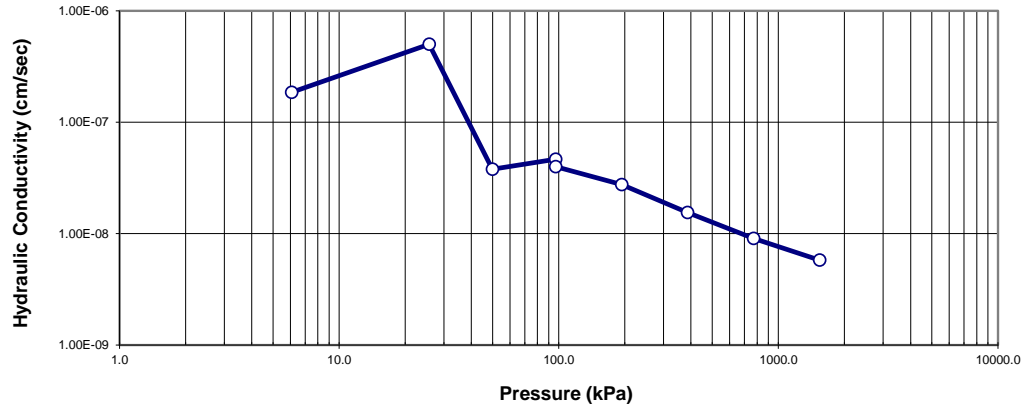
Consolidation Test Report

Highway 401 and Leslie Foundations
19-5161-205

BH R-09 TW2 (50' - 52')

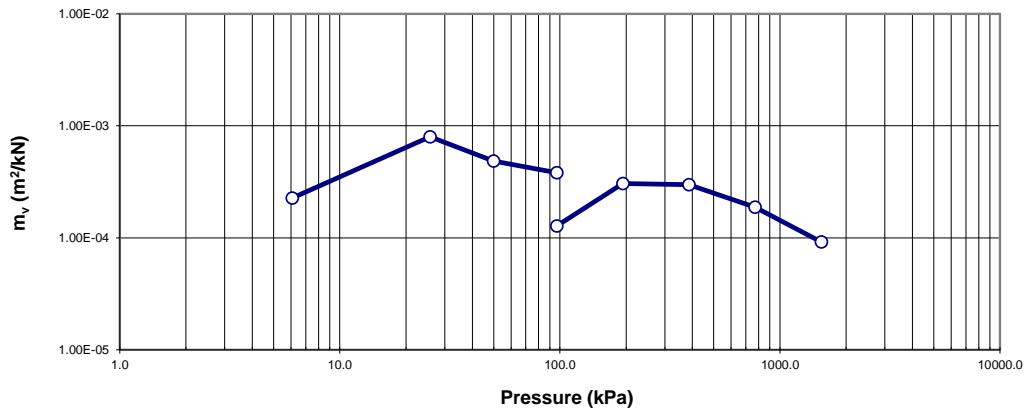
Hydraulic Conductivity vs. Pressure

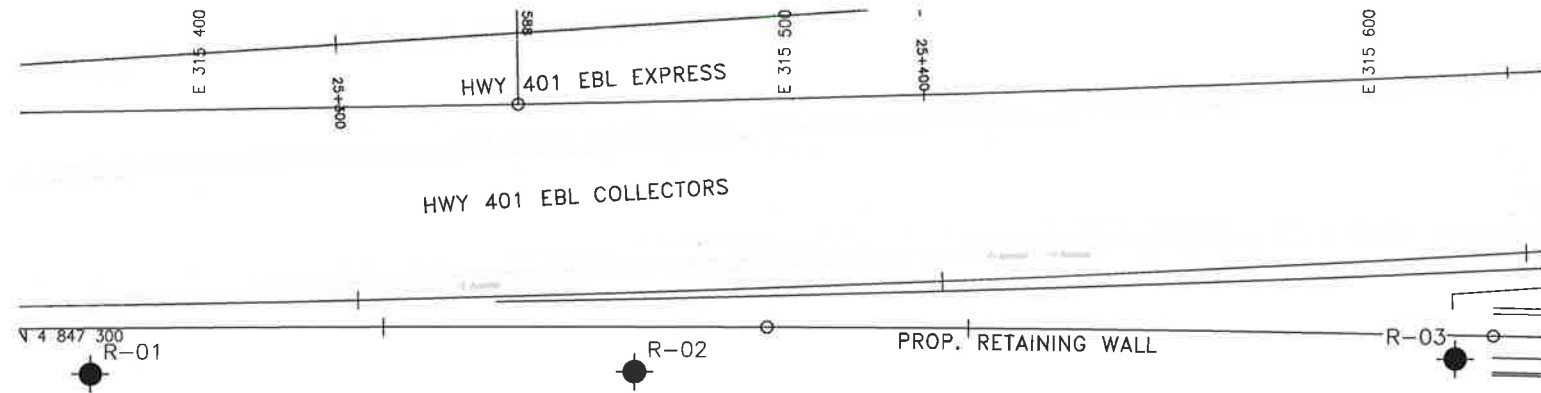
Project #: 19-5161-205
Client: MMM Group Limited
Project Name: Highway 401 and Leslie Foundations
Sample: BH R-09 TW2 (50' - 52')



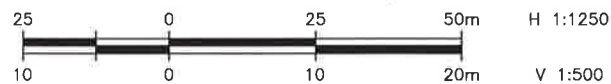
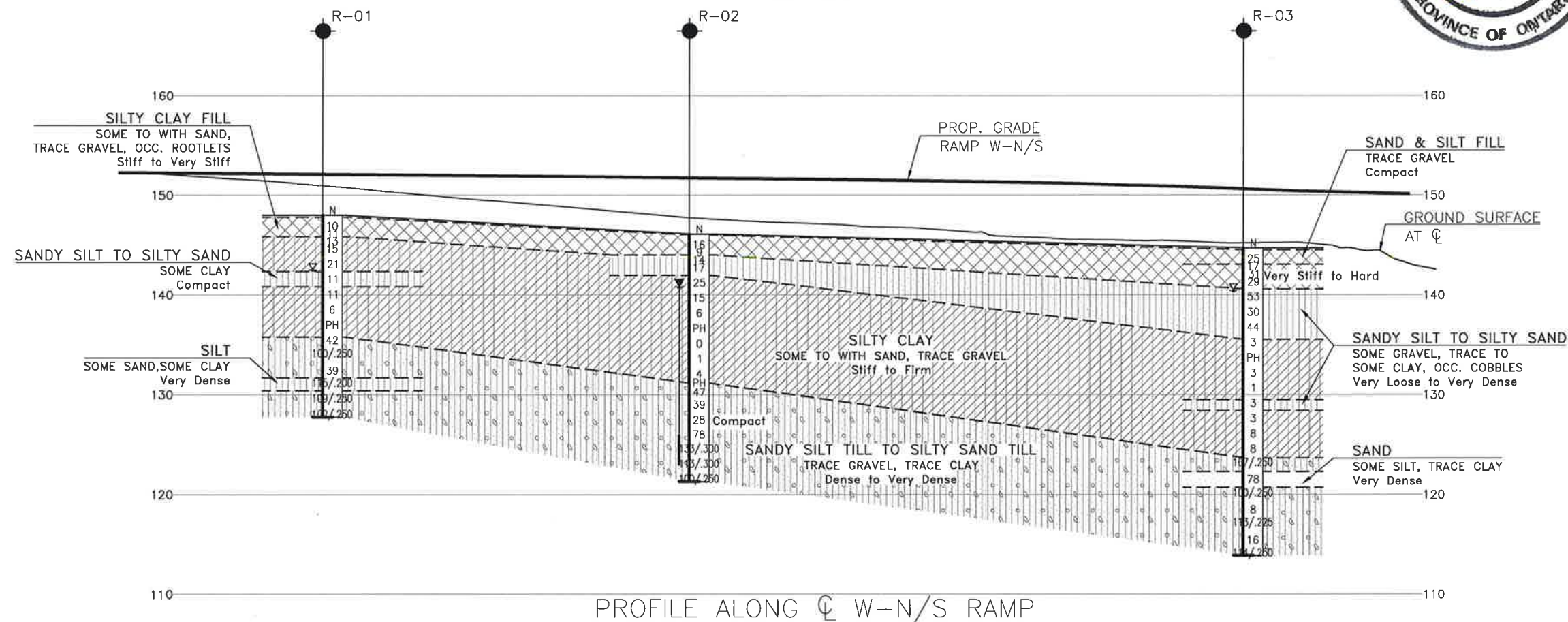
m_v vs. Pressure

Project #: 19-5161-205
Client: MMM Group Limited
Project Name: Highway 401 and Leslie Foundations
Sample: BH R-09 TW2 (50' - 52')





WOODSWORTH RD.



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

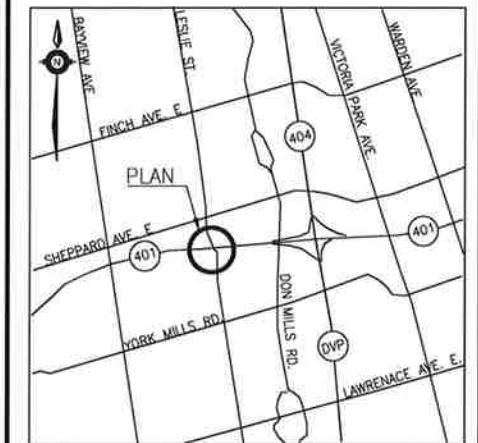


CONT No
WP No 2061-13-00

HIGHWAY 401
W-N/S RAMP WEST APPROACH
LESLIE ST./CNR OVERHEAD
BOREHOLE LOCATIONS AND SOIL STRATA



SHEET



KEYPLAN

LEGEND

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

NO	ELEVATION	NORTHING	EASTING
PB-01	140.7	4 847 229.6	315 712.1
PB-02	142.1	4 847 255.7	315 747.1
R-01	148.1	4 847 294.4	315 382.5
R-02	145.7	4 847 294.8	315 475.5
R-03	144.9	4 847 296.9	315 615.8
R-04	143.2	4 847 301.7	315 659.3
R-05	140.8	4 847 304.4	315 700.8
R-06	143.6	4 847 318.2	315 729.5
R-07	140.7	4 847 296.2	315 705.8
R-08	142.0	4 847 243.4	315 754.1
R-09	143.9	4 847 279.3	315 738.4

-NOTES-

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

GEOCRES No. 40M14-440

REVISIONS	DATE	BY	DESCRIPTION
DESIGN	RPR	CHK SKP	CODE
DRAWN	AN	CHK RPR	SITE
LOAD	DATE	OCT 2016	DWG 1

Appendix B

Pedestrian Overpass and Retaining/Noise Barrier Combination Wall Boreholes PB-01 and PB-02; Boreholes W-01 and W-02

- Record of Borehole Sheets
- Laboratory Test Results
- Drawing titled “Borehole Locations and Soil Strata

RECORD OF BOREHOLE No PB-01

1 OF 3

METRIC

W.P. 2061-13-00 LOCATION Pedestrian Overpass N 4 847 229.6 E 315 712.2 ORIGINATED BY ES
HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
DATUM Geodetic DATE 2015.03.30 - 2015.03.31 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE									WATER CONTENT (%)
140.7	GROUND SURFACE							20	40	60	80	100					
0.0	TOPSOIL: (200mm)							20	40	60	80	100					
0.2	SAND and SILT, trace gravel Compact Dark Brown to Brown Moist (FILL)						140										
			1	SS	13												
			2	SS	11		139										
138.5																	
2.2	ORGANICS clayey, roots and rootlets																
138.2	Compact Dark Brown Moist (300mm)																
2.5	Silty SAND, trace clay, trace gravel Compact Brown Moist		3	SS	20		138										
			4	SS	20												
							137										
			5	SS	27		136										0 60 33 7
135.1							135										
5.6	Silty CLAY, some to with sand Soft to Firm Grey Moist																
			6	SS	3												
							134										
			7	SS	1		133										
							132	2.7									
			1	TW	PH												
							131										

Continued Next Page

+³, ×³: Numbers refer to Sensitivity
20
15 10 5 0
(%) STRAIN AT FAILURE

METRIC

[illegible]

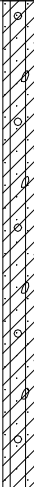
+³, ×³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No PB-01

3 OF 3

METRIC

W.P. 2061-13-00 LOCATION Pedestrian Overpass N 4 847 229.6 E 315 712.2 ORIGINATED BY ES
 HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2015.03.30 - 2015.03.31 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
							20	40	60	80	100	20	40	60			
	Continued From Previous Page																
	Silty CLAY , trace sand Hard Grey Moist (TILL)				0.175												
			15	SS	102/ 0.250												
117.5			16	SS	100/ 0.225												
23.2	END OF BOREHOLE AT 23.2m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 3.0m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) Apr 22/2015 5.4 135.3 Jun 03/2015 2.8 137.9																

RECORD OF BOREHOLE No PB-02

1 OF 3

METRIC

W.P. 2061-13-00 LOCATION Pedestrian Overpass N 4 847 255.7 E 315 747.1 ORIGINATED BY ES
HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
DATUM Geodetic DATE 2015.04.14 - 2015.04.15 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				WATER CONTENT (%)				
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE				W _p	W	W _L		
142.1	GROUND SURFACE						20	40	60	80	100	20	40	60		GR SA SI CL
0.0 0.1	TOPSOIL: (75mm) Silty SAND , some clay, trace gravel Dense Brown to Dark Brown Moist (FILL)		1	SS	31								○			6 54 30 10
140.7													○			
1.4	Compact to Loose		2	SS	14								○			
			3	SS	7								○			
			4	SS	8								○			
138.0																
4.1	SAND and SILT , trace clay, trace gravel Compact Brown Moist		5	SS	23								○			
	Wet		6	SS	12								○			
	Loose Grey		7	SS	6								○			
133.4																
8.7	Silty CLAY , some to with sand Soft to Firm Grey Wet		8	SS	3								○			

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No PB-02

2 OF 3

METRIC

W.P. 2061-13-00 LOCATION Pedestrian Overpass N 4 847 255.7 E 315 747.1 ORIGINATED BY ES
 HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2015.04.14 - 2015.04.15 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT		NATURAL MOISTURE CONTENT		LIQUID LIMIT		UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)										
								○ UNCONFINED + FIELD VANE	● QUICK TRIAXIAL × LAB VANE	W _p	W	W _L								
	Continued From Previous Page							20 40 60 80 100	20 40 60 80 100	20 40 60										
	Silty CLAY , some to with sand, trace gravel Firm Grey Wet						132		3.3 +											
			1	TW	PH		131					○								
								2.3 +												
			9	SS	1		130					○								
							129		3.1 +											
			10	SS	0		128						○				3 24 27 46			
							127		3.1 +											
	Sand seams		11	SS	1		126													
125.8 16.3	Stiff								4.3 +											
			12	SS	1		125										3 37 39 21			
							124		4.0 +											
			13	SS	14		123					○								
122.7 19.4																				

Continued Next Page

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

RECORD OF BOREHOLE No PB-02

3 OF 3

METRIC

W.P. 2061-13-00 LOCATION Pedestrian Overpass N 4 847 255.7 E 315 747.1 ORIGINATED BY ES
 HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2015.04.14 - 2015.04.15 CHECKED BY RPR

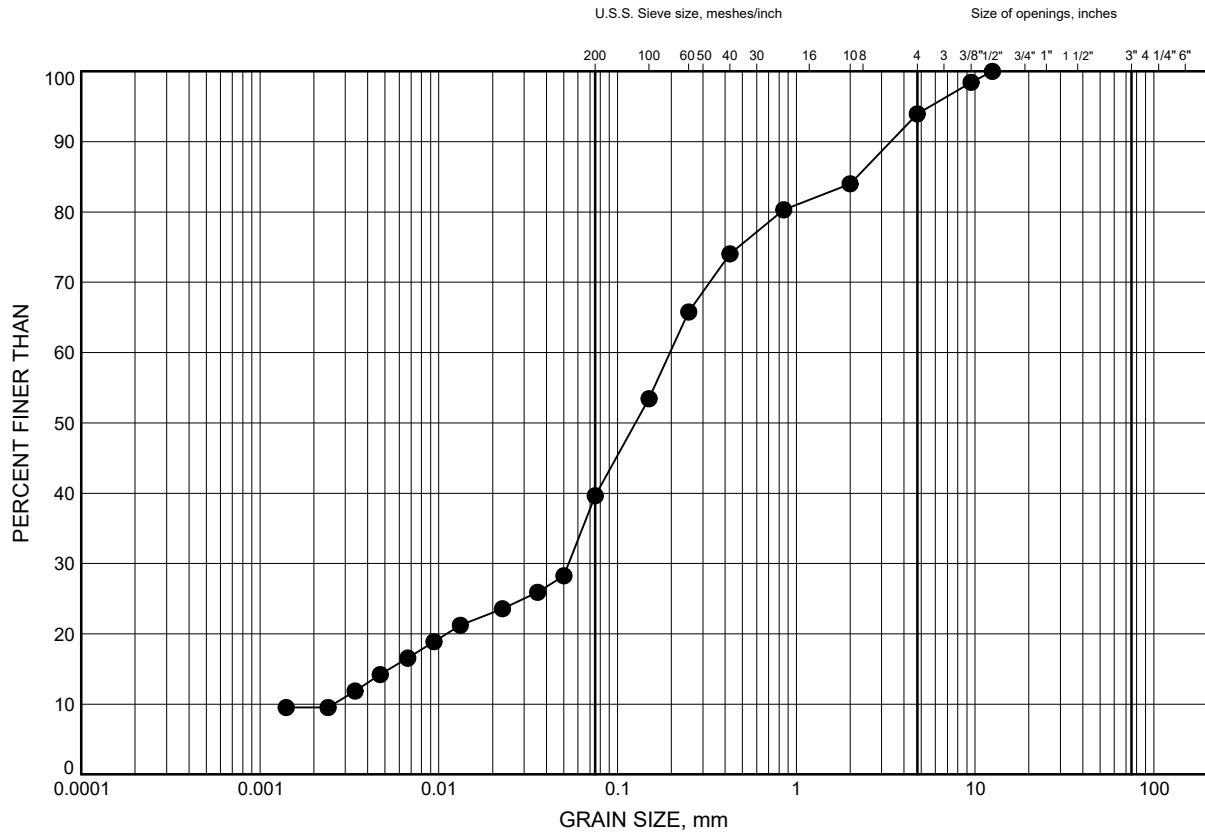
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
<div><div>20406080100</div><div>○ UNCONFINED + FIELD VANE</div><div>● QUICK TRIAXIAL × LAB VANE</div><div>20406080100</div></div>														
	Continued From Previous Page		14	SS	2		122							
120.2	Silty CLAY , some to with sand, trace gravel Soft Grey Wet						121							
21.9			SAND and SILT , some clay, trace gravel Very Dense Grey Moist (TILL)					120						
				15	SS	101/ 0.250		119						
								118						
117.7	Some gravel and cobbles Very Dense		16	SS	100/ 0.075		117							
24.4							116							
117.0	Occasional sand layers													
25.1														
115.8	END OF BOREHOLE AT 26.3m. WATER LEVEL AT 7.0m UPON COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND AUGER CUTTINGS TO SURFACE.		17	SS	108/ 0.250									
26.3														

ONTMT4S 1205.GPJ 2015TEMPLATE(MTO).GDT 11/3/15

Hwy 401 Leslie Street 2013-E-0032
GRAIN SIZE DISTRIBUTION

FIGURE B1

SILTY SAND TILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	PB-02	1.83	140.27

Date November 2015
W.P. 2061-13-00

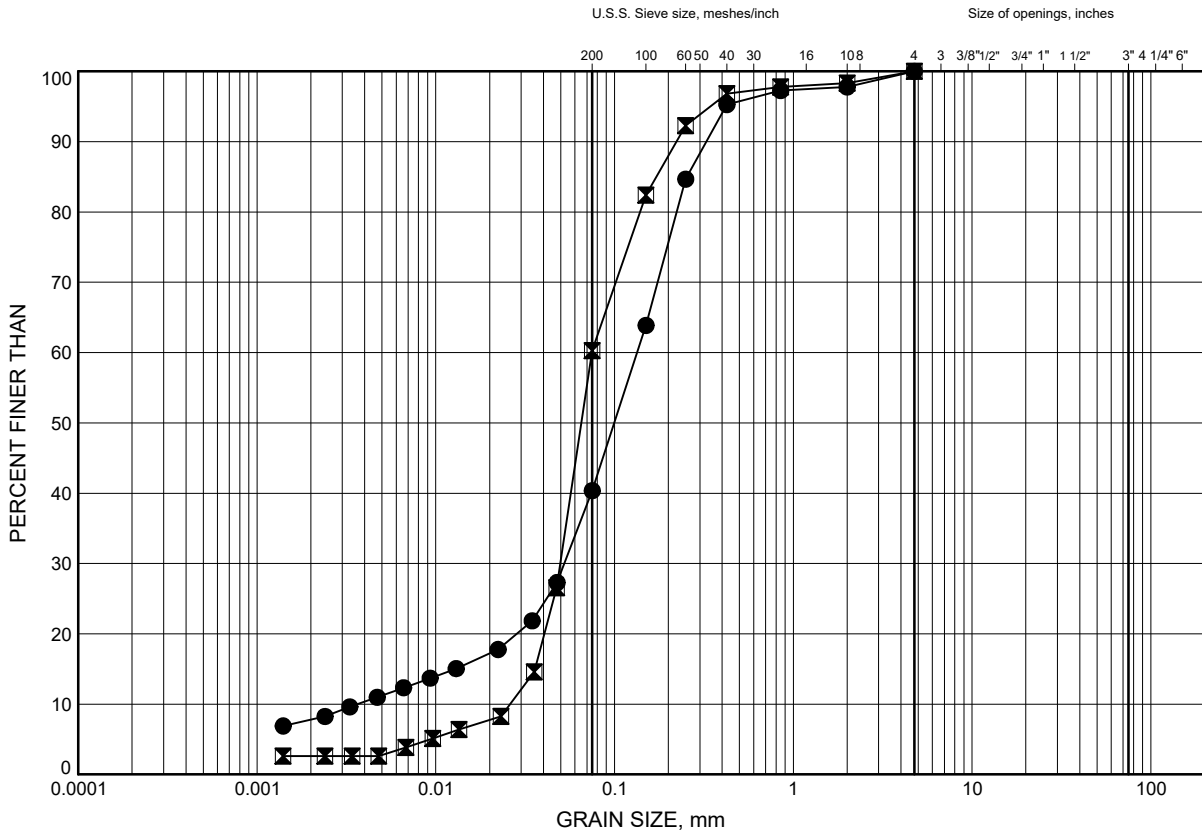


Prep'd AN
Chkd. RPR

Hwy 401 Leslie Street 2013-E-0032
GRAIN SIZE DISTRIBUTION

FIGURE B2

SILTY SAND to SAND & SILT



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	PB-01	4.88	135.82
⊠	PB-02	6.40	135.70

Date November 2015
W.P. 2061-13-00

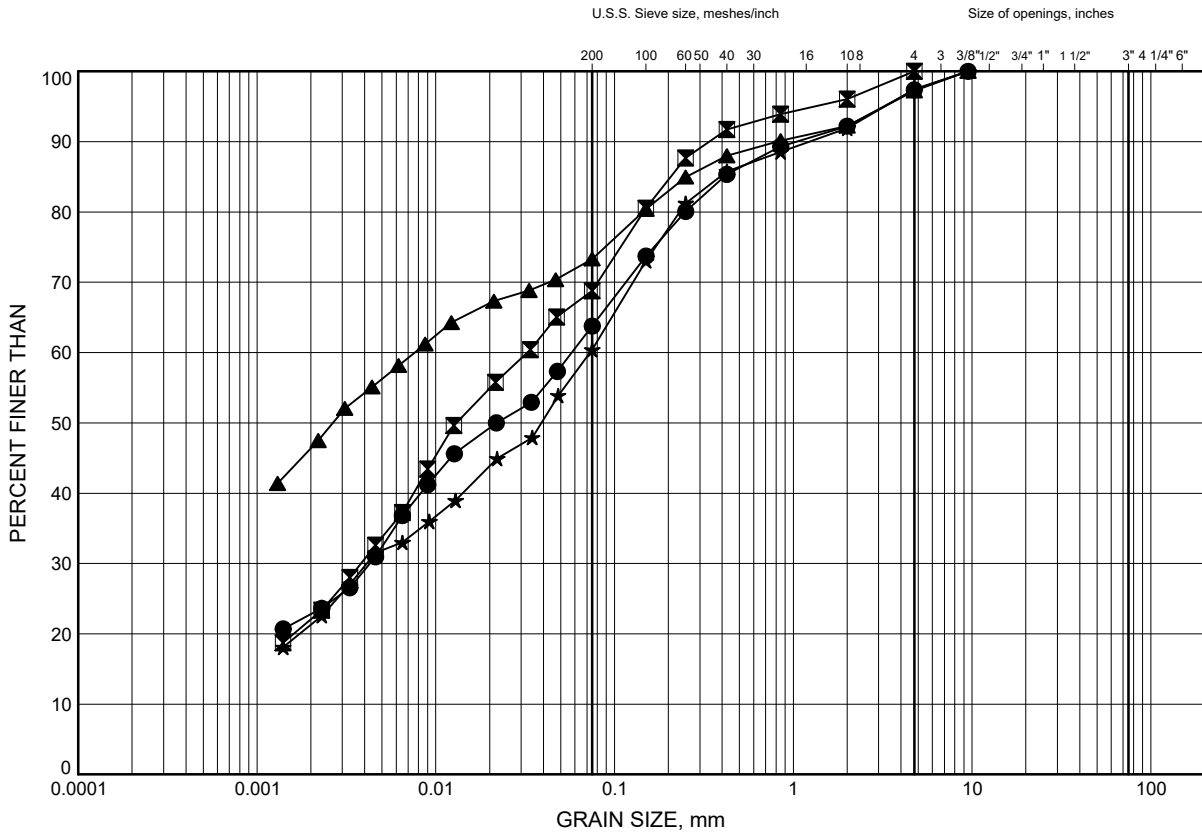


Prep'd AN
Chkd. RPR

Hwy 401 Leslie Street 2013-E-0032
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)
●	PB-01	12.50	128.20
◻	PB-01	18.59	122.11
▲	PB-02	14.02	128.08
★	PB-02	17.07	125.03

Date November 2015
W.P. 2061-13-00

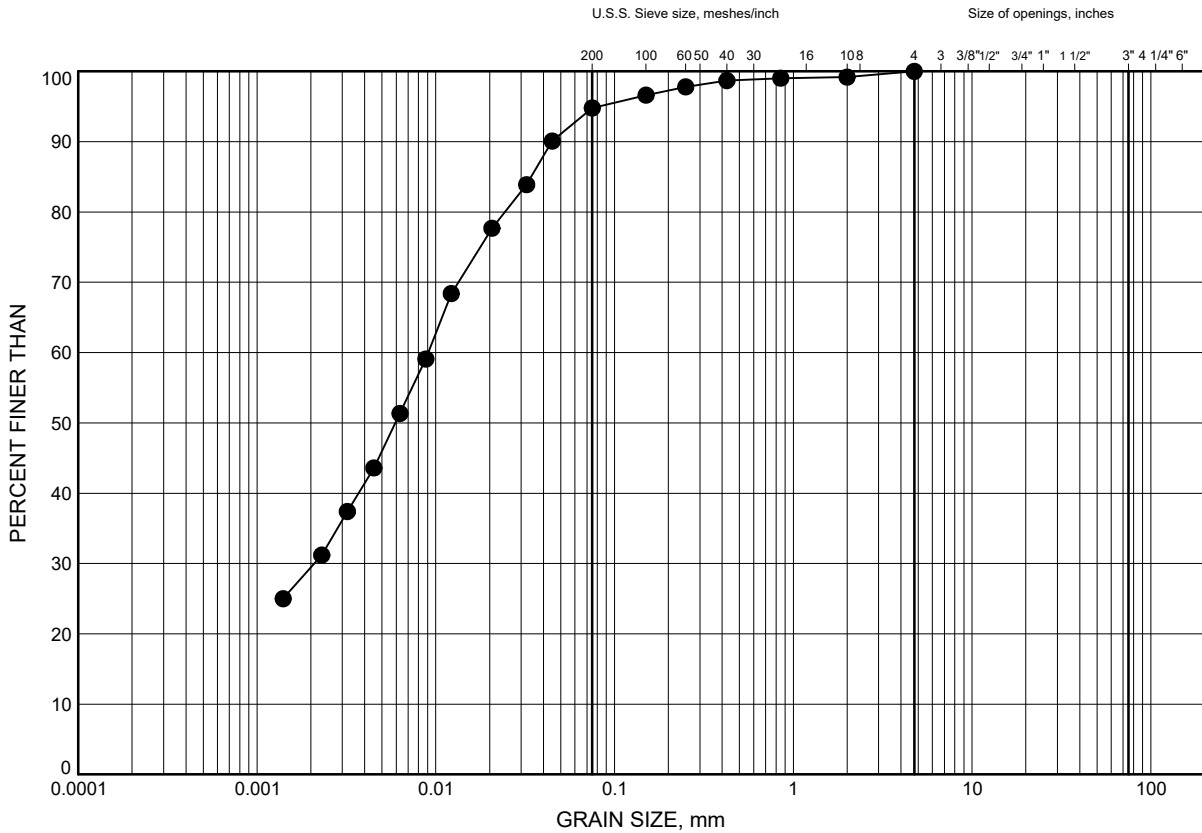


Prep'd AN
Chkd. RPR

Hwy 401 Leslie Street 2013-E-0032
GRAIN SIZE DISTRIBUTION

FIGURE B4

SILTY CLAY TILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	PB-01	21.64	119.06

Date November 2015
W.P. 2061-13-00



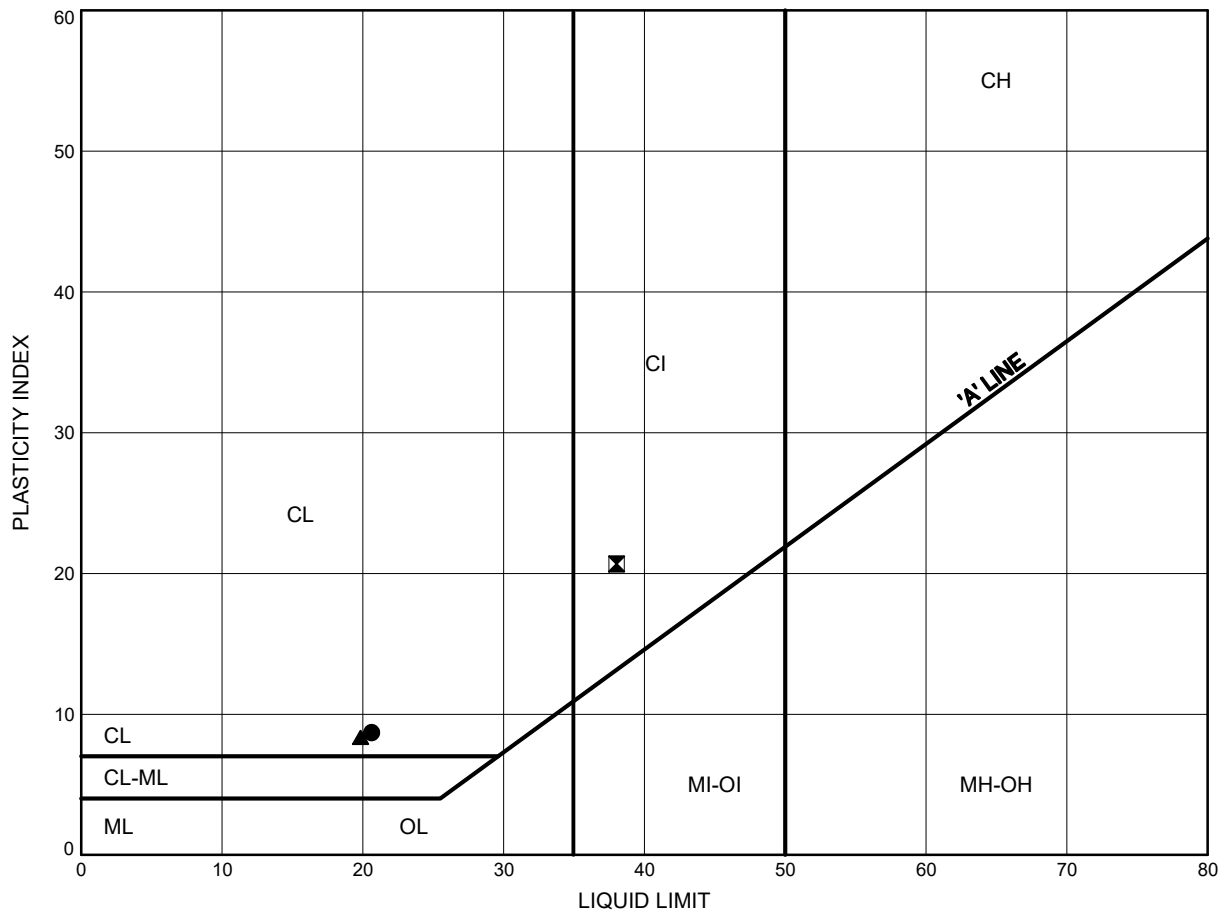
Prep'd AN
Chkd. RPR

Hwy 401 Leslie Street 2013-E-0032

ATTERBERG LIMITS TEST RESULTS

FIGURE B5

SILTY CLAY



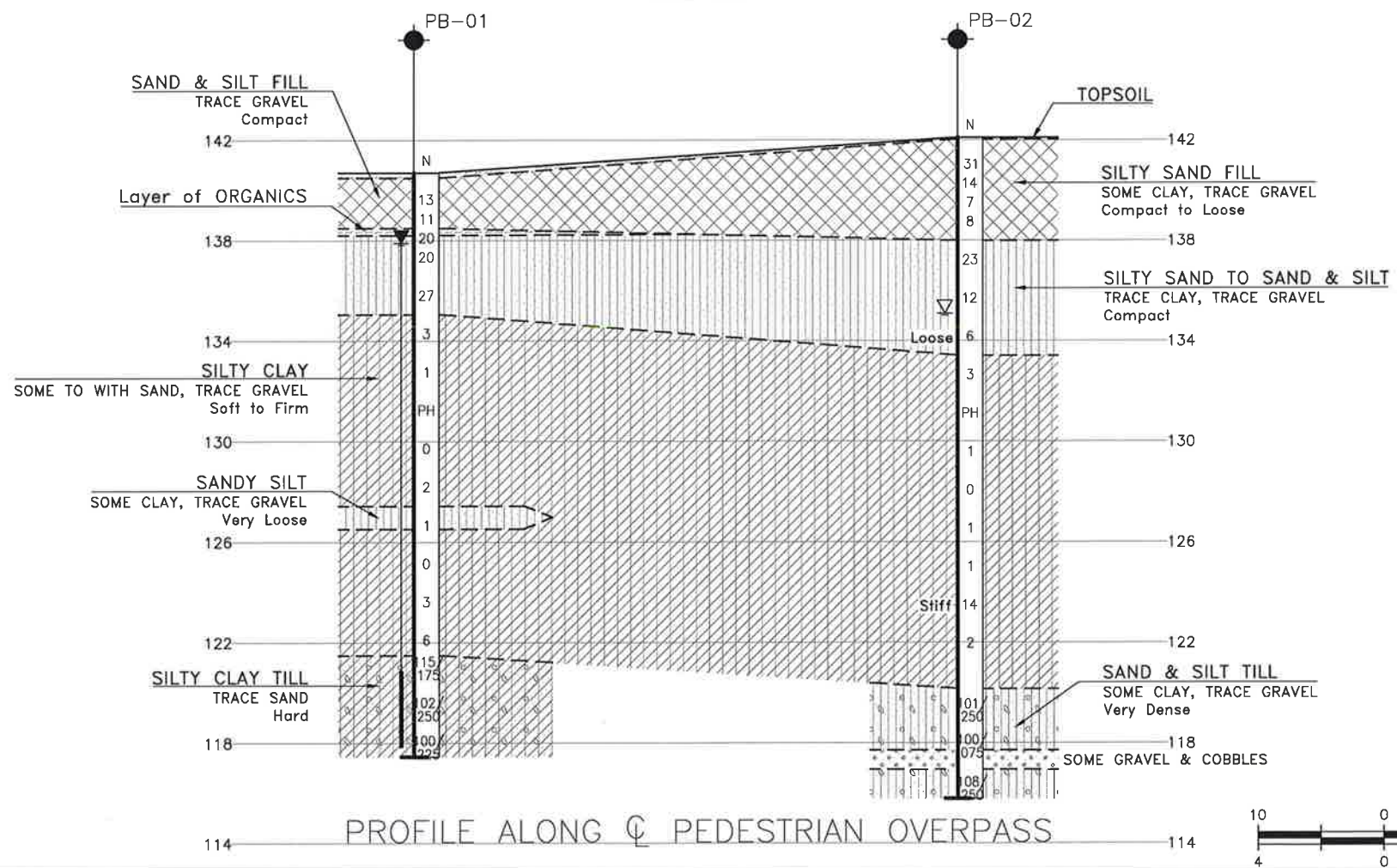
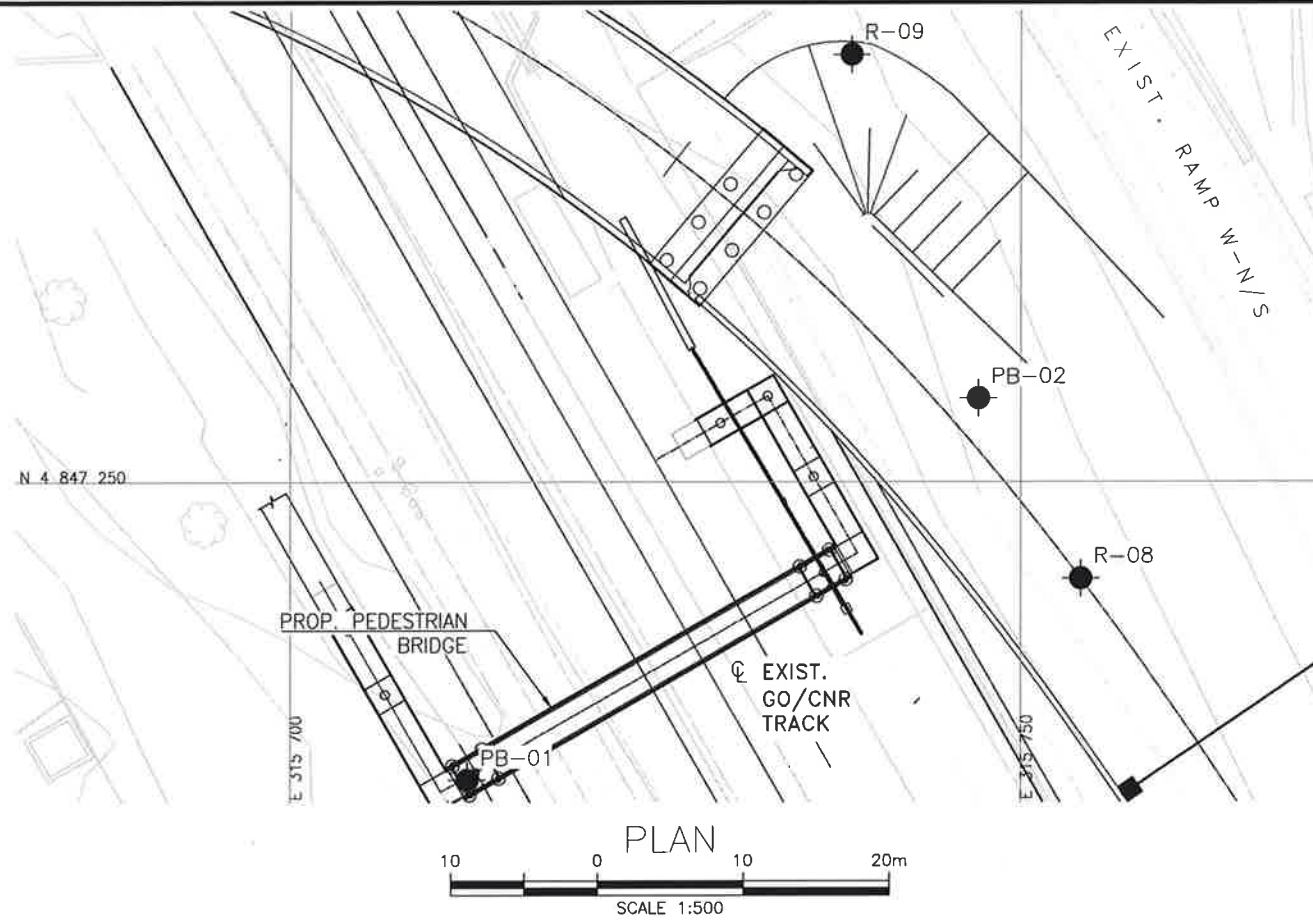
LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	PB-01	12.50	128.20
⊠	PB-02	14.02	128.08
▲	PB-02	17.07	125.03

Date November 2015
W.P. 2061-13-00



Prep'd AN
Chkd. RPR



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

CONT No
WP No 2061-13-00

HIGHWAY 401
PEDESTRIAN OVERPASS
LESLIE ST./CNR OVERHEAD
BOREHOLE LOCATIONS AND SOIL STRATA








SHEET



KEYPLAN

L E G E N D

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

NO	ELEVATION	NORTHING	EASTING
PB-01	140.7	4 847 229.6	315 712.1
PB-02	142.1	4 847 255.7	315 747.1
R-01	148.1	4 847 294.4	315 382.5
R-02	145.7	4 847 294.8	315 475.5
R-03	144.9	4 847 296.9	315 615.8
R-04	143.2	4 847 301.7	315 659.3
R-05	140.8	4 847 304.4	315 700.8
R-06	143.6	4 847 318.2	315 729.5
R-07	140.7	4 847 296.2	315 705.8
R-08	142.0	4 847 243.4	315 754.1
R-09	143.9	4 847 279.3	315 738.4

-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. 30M14-440

[illegible]

RECORD OF BOREHOLE No W-01

1 OF 2

METRIC

W.P. 2061-13-00 LOCATION Combination Wall N 4 847 294.6 E 315 151.6 ORIGINATED BY ES
HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
DATUM Geodetic DATE 2015.11.30 - 2015.11.30 CHECKED BY SKP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)							
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				WATER CONTENT (%)				GR	SA	SI	CL				
154.6	GROUND SURFACE							20	40	60	80	100											
0.0	ASPHALT:(125mm)																						
0.1	SAND, some gravel Brown Moist (FILL)		1	GS			154																
153.7																							
0.9	Silty CLAY, some sand, trace gravel Very Stiff to Hard Brown Moist		1	SS	21		153											0	11	63	26		
			2	SS	85/ 0.250																		
			3	SS	100/ 0.175		152																
151.8																							
2.8	SAND and SILT, trace clay, trace gravel Very Dense Brown to Grey Moist		4	SS	79		151																
			5	SS	50/ 0.125		150												3	43	50	4	
			6	SS	71		149																
							148																
147.4																							
7.2	Silty CLAY, trace to some sand Hard Grey Moist		7	SS	88		147													0	10	58	32
			8	SS	82		146																
145.0																							
9.6	END OF BOREHOLE AT 9.6m. BOREHOLE OPEN AND DRY UPON						145																

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No W-01

2 OF 2

METRIC

W.P. 2061-13-00 LOCATION Combination Wall N 4 847 294.6 E 315 151.6 ORIGINATED BY ES
 HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2015.11.30 - 2015.11.30 CHECKED BY SKP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
	Continued From Previous Page																
	COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND AUGER CUTTINGS TO 0.2m, THEN ASPHALT TO SURFACE.																

RECORD OF BOREHOLE No W-02

1 OF 2

METRIC

W.P. 2061-13-00 LOCATION Combination Wall N 4 847 300.2 E 315 227.7 ORIGINATED BY ES
 HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2015.11.30 - 2015.11.30 CHECKED BY SKP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										
153.0	GROUND SURFACE							20	40	60	80	100						
0.0	ASPHALT:(150mm)							20	40	60	80	100						
0.2	SAND, some gravel Brown Moist (FILL)		1	GS														
152.2																		
0.8	SAND, some silt, trace gravel Compact Brown Moist (FILL)		1	SS	19		152											
151.5																		
1.5	SAND and SILT, some clay, trace gravel Very Dense to Dense Brown Moist (FILL)		2	SS	75		151											
			3	SS	46													2 43 42 13
150.0							150											
3.0	SAND and SILT, some clay, trace gravel Compact to Very Dense Grey and Brown Moist		4	SS	24		149											2 35 42 21
			5	SS	103/ 0.275		148											
147.4																		
5.6	Silty CLAY, trace to some sand Hard Grey Moist		6	SS	58		147											
							146											
			7	SS	74		145											
							144											
143.2			8	SS	65													
9.8	END OF BOREHOLE AT 9.8m.																	

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No W-02

2 OF 2

METRIC

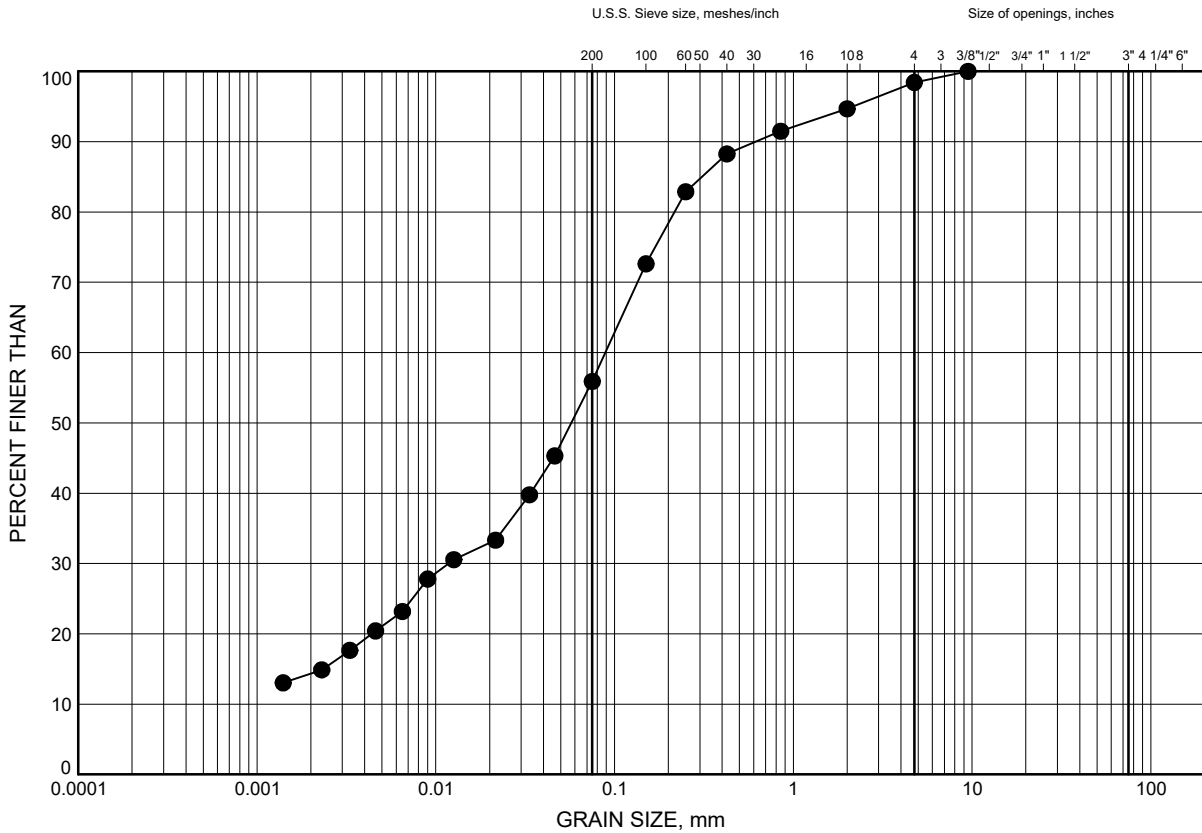
W.P. 2061-13-00 LOCATION Combination Wall N 4 847 300.2 E 315 227.7 ORIGINATED BY ES
 HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2015.11.30 - 2015.11.30 CHECKED BY SKP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
	Continued From Previous Page BOREHOLE OPEN AND DRY UPON COMPLETION. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 3.0m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) 2016.03.29 6.5 146.5																

Combination Wall GRAIN SIZE DISTRIBUTION

FIGURE B6

SAND & SILT FILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	W-02	2.59	150.41

Date March 2016
W.P. 2061-13-00

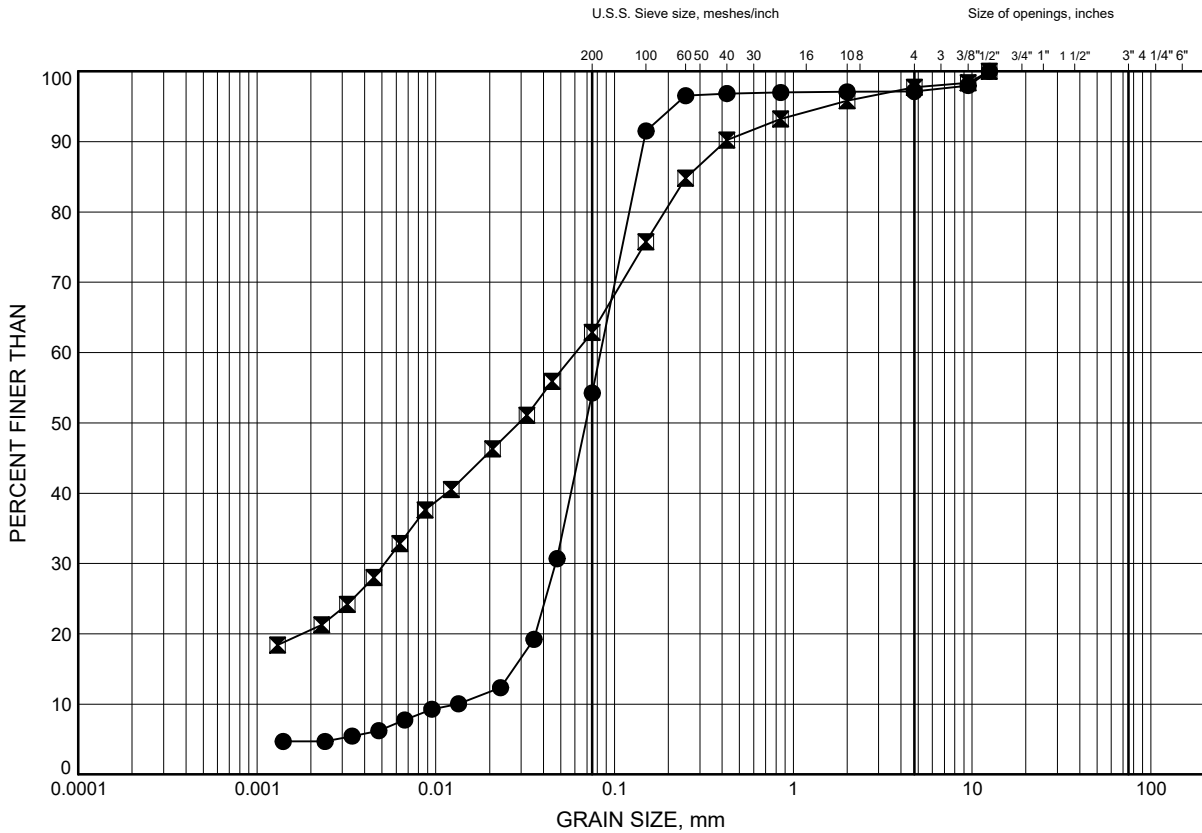


Prep'd AN
Chkd. SKP

Combination Wall GRAIN SIZE DISTRIBUTION

FIGURE B7

SAND & SILT



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	W-01	4.71	149.89
⊠	W-02	3.35	149.65

Date March 2016
W.P. 2061-13-00

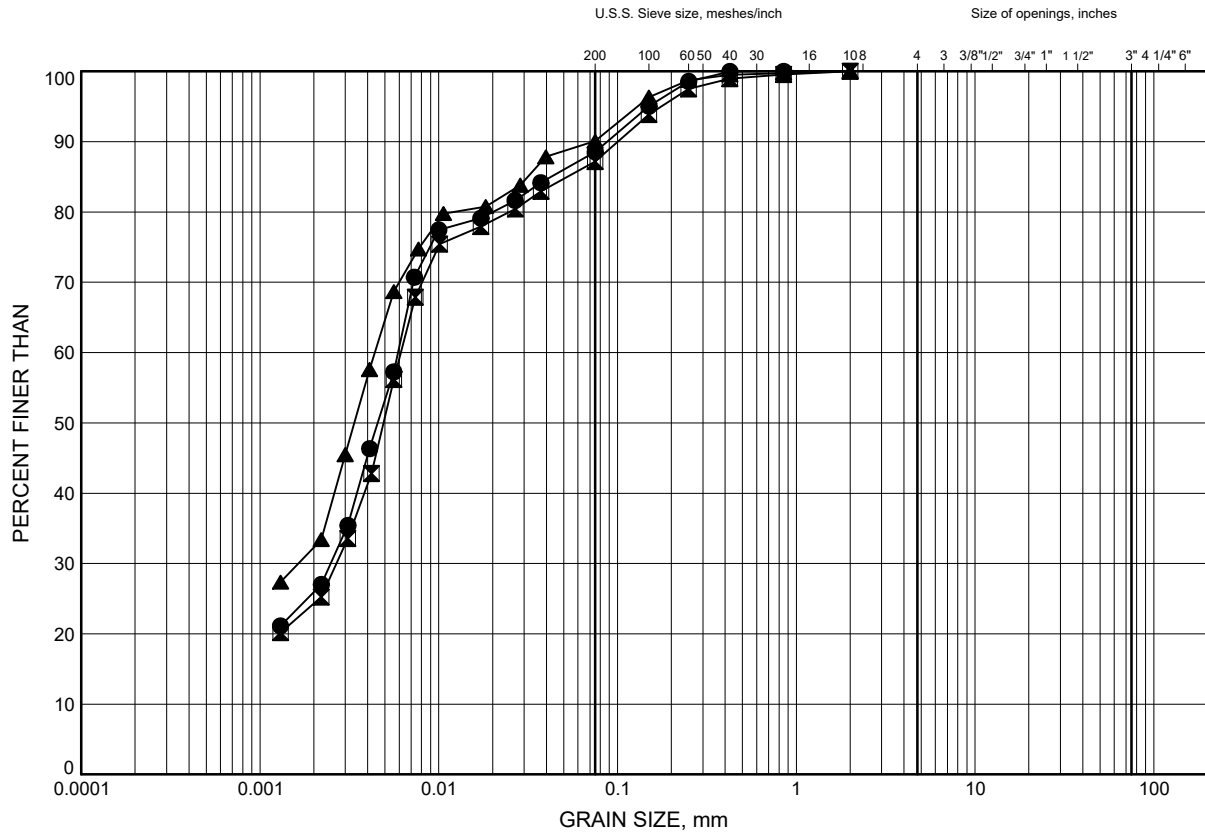


Prep'd AN
Chkd. SKP

Combination Wall 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)
●	W-01	1.07	153.53
⊠	W-01	1.73	152.87
▲	W-01	7.85	146.75

Date March 2016
W.P. 2061-13-00

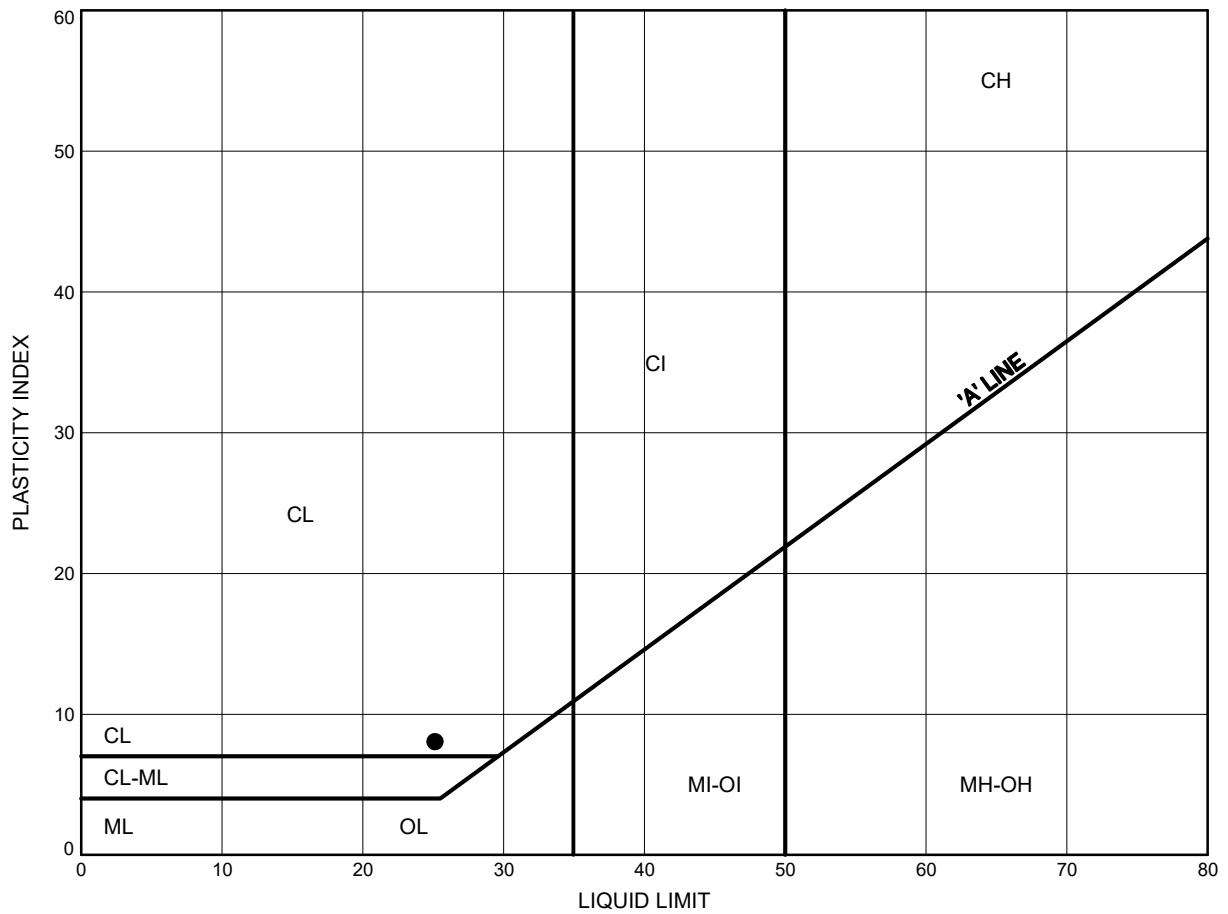


Prep'd AN
Chkd. SKP

Combination Wall
ATTERBERG LIMITS TEST RESULTS

FIGURE B9

Silty CLAY



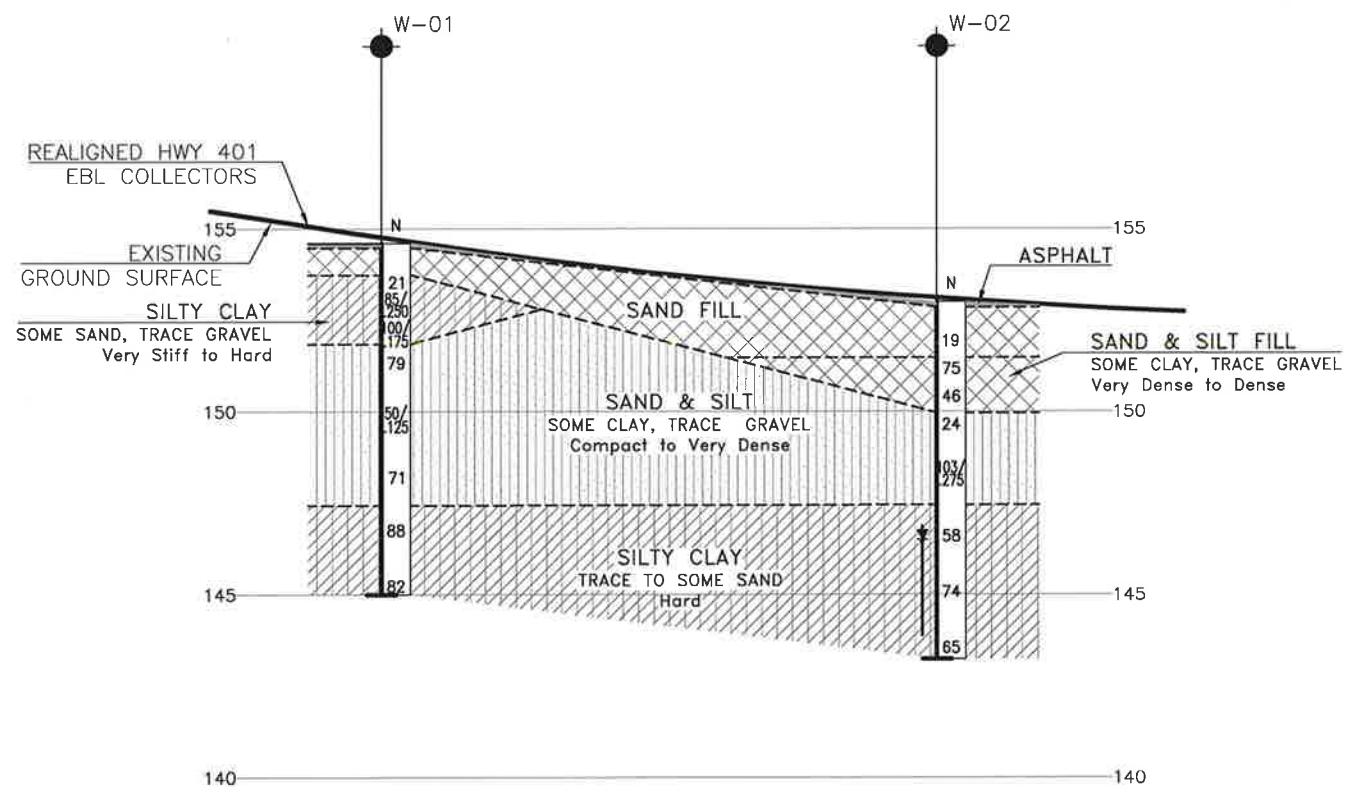
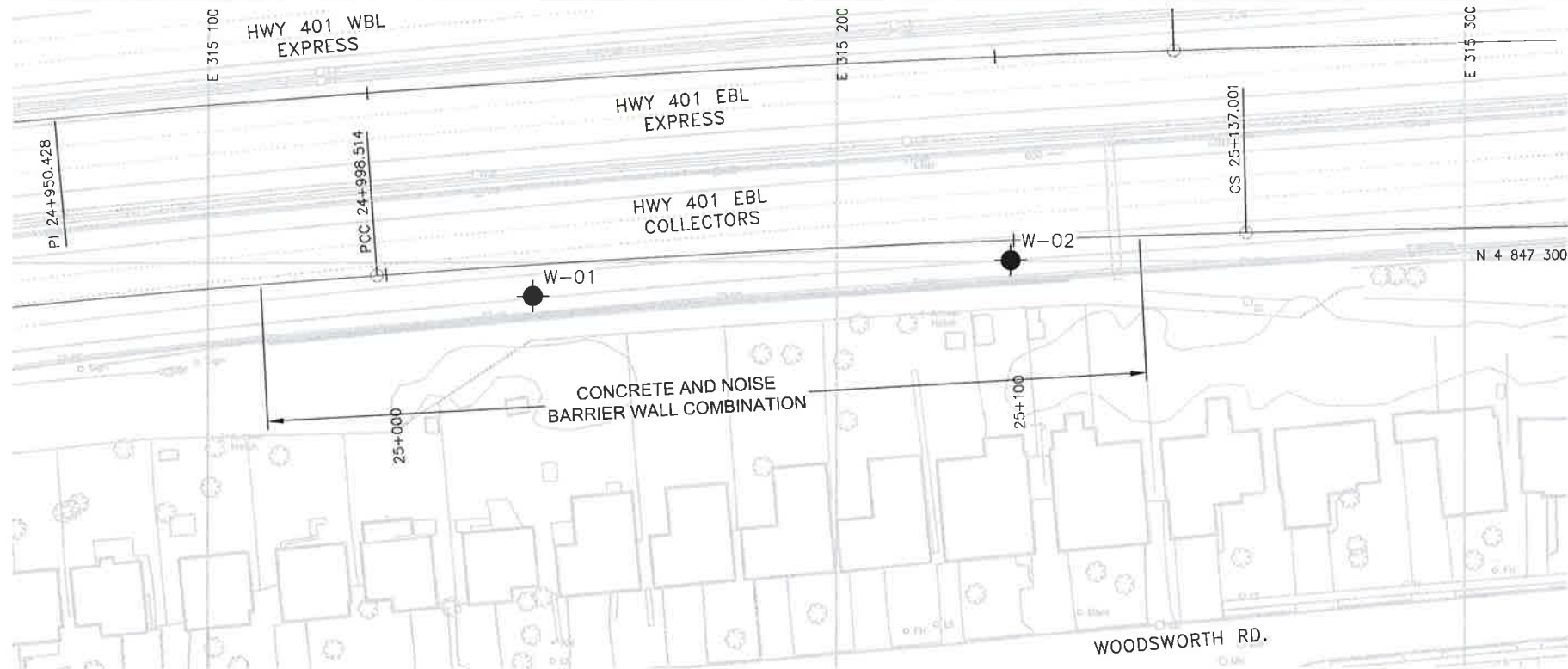
LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	W-01	1.07	153.53

Date March 2016
W.P. 2061-13-00



Prep'd AN
Chkd. SKP



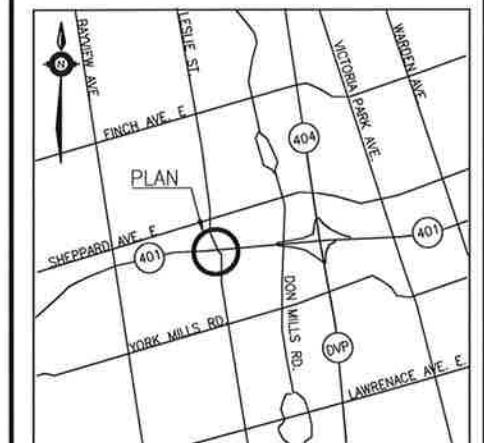
PROFILE ALONG ϕ REALIGNED
HWY 401 EBL COLLECTORS



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

CONT No
WP No 2061-13-00

HIGHWAY 401
EASTBOUND LANES COLLECTORS
COMBINATION WALL
BOREHOLE LOCATIONS AND SOIL STRATA



KEYPLAN

LEGEND

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

NO	ELEVATION	NORTHING	EASTING
W-01	154.6	4 847 294.6	315 151.6
W-02	153.0	4 847 300.2	315 227.7

-NOTES-

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

GEOCREs No. 40M14-440



REVISIONS	DATE	BY	DESCRIPTION
DESIGN	SKP	CHK	SKP
DRAWN	AN	CHK	SITE
LOAD	DATE	OCT 2016	DWG 1

Appendix C

Site Photographs

Replacement of CNR Overpass and Pedestrian Bridge
Highway 401 & Leslie St., Toronto, Ontario



Photo 1.- Existing CNR Overpass at Highway 401 & Leslie St.

Replacement of CNR Overpass and Pedestrian Bridge
Highway 401 & Leslie St., Toronto, Ontario

Pedestrian bridge over tracks

CNR Overpass



Photo 2.- Existing CNR Overpass and Pedestrian Bridge at Oriole GO Station

Replacement of CNR Overpass and Pedestrian Bridge
Highway 401 & Leslie St., Toronto, Ontario



Photos 3 and 4.- Existing CNR Overpass and Pedestrian Bridge



Photo 5.- Existing CNR Overpass /Hwy 401

Appendix D
Comparison of Foundation Alternatives

19-5161-205

COMPARISON OF FOUNDATION ALTERNATIVES

Foundation Unit	Spread Footings on Native Soils	Driven Steel H-Piles into Native Glacial Till	Augered Caissons (Drilled Shafts) into Native Glacial Tills
	<p>Advantages:</p> <ul style="list-style-type: none"> i. Ease of construction. ii. Lower cost than deep foundations. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Low geotechnical resistance. ii. Time dependent consolidation settlement iii. Relatively deep excavation required to found footings on native soils iv. Dewatering may be required, depending on depth of excavation and groundwater level at time of construction. v. Roadway protection will be required. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. Relatively high axial geotechnical resistance, provided EPS is used behind abutments to practically eliminate downdrag. ii. Required for integral abutments. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher unit cost than footings. ii. Require roadway protection for constructing pile caps where space permits. iii. Vibration and noise due to pile driving could have adverse effects on existing bridge, sewer line, and have to satisfy city by-laws on vibration and noise. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. Higher lateral resistance is available due to larger diameter. ii. Less number of caissons is required for each foundation element than if steel piles were used. iii. Caisson caps may be avoided resulting in lesser excavation requirements. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Steel liners will be required to install caissons to minimize sidewall sloughing and water seepage. ii. Tremie concrete may need to be used. iii. Potential basal instability if water-bearing soils are exposed at the base.
Abutments	NOT FEASIBLE	TECHNICALLY FEASIBLE IF NO ISSUES ON VIBRATION AND NOISE	RECOMMENDED
Piers		TECHNICALLY FEASIBLE IF NO ISSUES ON VIBRATION AND NOISE, AND NO SPACE RESTRICTION	

Appendix E

List of OPSS Documents and NSSP Wording

19-5161-205

1. List of OPSS Documents Referenced in this Report

- OPSS 903
- OPSS 206
- OPSS.PROV 804
- OPSS 501
- OPSS.PROV 539
- OPSS 902
- OPSS.PROV 1010
- OPSD 3000.100
- OPSD 3100.150
- OPSD 3102.100
- OPSD 208.010

2. Suggested Text for NSSP on “Caisson Installation at Abutments”

All caissons shall be installed in accordance with OPSS 903. Caissons will extend through existing fill, native sands and silts, silty clay and into the underlying sand and silt till. The caisson installation equipment shall be capable of dislodging and removing any obstructions such as cobbles, boulders and to penetrate very dense/hard layers within the glacial till.

Construction of caissons will require the use of temporary steel liners to support the caisson sidewalls and to provide seepage cut-off where required. Concrete shall be placed with a minimum delay after each caisson is drilled, cleaned and inspected. Consideration should be given to using tremie techniques to place concrete inside the caisson hole.

3. Suggested Text for NSSP on “Monitoring of Existing Bridge”

Monitoring of the existing bridge abutments and piers is required during construction of the new bridge foundations and embankment fills. As a minimum, two reference points must be established on each abutment and pier of the existing structure, and the vertical and lateral positions of these points must be surveyed relative to known, fixed reference datum points on a regular basis.

The suggested monitoring frequency is as follows:

- Three readings on separate days prior to construction to establish a baseline
- Twice daily while any foundation construction or other subsurface construction is in progress
- Daily for one week after completion of foundation construction
- Twice weekly for the following week.

The vertical and horizontal precision of readings should be ± 2 mm. All readings must be reported to the Contract Administrator within 24 hours and immediately if any movement exceeds limits set by the structural designers.

The Contract Administrator must be advised of the importance of monitoring and be required to advise the Ministry immediately if the vertical and horizontal movements exceed the specified limits.

4. Suggested Text for NSSP on “Monitoring of Highway 401 Eastbound Lanes”

During ramp construction, potential impact of fill placement on the existing pavement surface of the Highway 401 eastbound lanes should be closely monitored.

Daily visual inspection of the pavement surface shall be carried out in the vicinity of the fill placement. If cracks form in the pavement or settlement is observed to occur, the observations must be immediately reported to the Contract Administrator for determining whether remedial action is required. Such action may include temporarily re-paving the affected areas.

5. Suggested Text for NSSP on “Footing Construction”

All footing construction procedures shall follow the guidelines provided in OPSS 902.

The bases of the foundation excavations should be inspected by a geotechnical engineer to confirm that the footing subgrade is in the native, undisturbed, stiff silty clay or compact sandy silt conforming to the design requirements and has been adequately prepared to receive concrete. Concrete or mud slab should be placed within 24 hours following completion of excavation to prevent deterioration of the approved subgrade. The mud slab should be at least 100 mm thick and formed with the same class of concrete as that of the footings. Where sub-excavation is required to remove unsuitable material from below the design founding level, the founding surface should be re-established using engineered fill or mass concrete of

the same class as the footing. Where an engineered fill pad is used below the footing, the subgrade should first be inspected and approved prior to placing and compacting approved fill materials.

Water seepage from perched water within the fill as well as accumulation of precipitation and surface runoff should be expected. Dewatering prior to and during footing excavation will be required to construct the footings in the dry and to prevent sloughing of the sides or disturbance of the base of the excavation due to the inflow of groundwater.

6. Sample Text for NSSP on “Expanded Polystyrene Embankment”

EXPANDED POLYSTYRENE EMBANKMENT – Item No.

Special Provision

REQUIREMENTS FOR EXPANDED POLYSTYRENE EMBANKMENT FILL

1.0 SCOPE

This special provision covers the requirements for the supply and construction of the rigid expanded polystyrene embankment fill and associated works as shown on the contract drawings.

2.0 REFERENCES

This special provision refers to the following standards, specifications or publications.

2.1 National Standards of Canada

CAN/CGSB - 51.20 M87

2.2 ASTM

ASTM D1621 Test Method for Compressive Properties of Rigid Cellular Plastics

ASTM C203 Test Method for Breaking Load and Flexural Properties of Block Type Thermal Insulation

ASTM C177 Test Method for Steady State Heat Flux Measurements and Thermal Transmission Properties by Means of the Heat Flow Apparatus

ASTM D2842 Test Method for Water Absorption by Rigid Cellular Plastics

ASTM D2863 Test Method for Measuring the Minimum Oxygen Content

ASTM D2126 Test Method for Response of Rigid Cellular Plastics to Thermal and Humid Aging

2.3 OPSS - Ontario Provincial Standard Specification

OPSS 212 Borrow

OPSS.PROV 501 Compaction

OPSS 517 Dewatering

OPSS.PROV 1010 Aggregates – Granular A, B, M, and Selected Subgrade Material

OPSS 1605 Expanded Extruded Polystyrene Pavement Insulation

OPSS 1860 Geotextiles

3.0 SUBSURFACE CONDITIONS

The subsurface conditions at the site are described in the Foundation Investigation Report for this Contract.

4.0 DEFINITIONS

For the purpose of this special provision, the following definitions apply:

Rigid Expanded Polystyrene: Moulded rigid blocks produced by a process of pre-expansion, aging and forming of petroleum based raw material.

Rigid Extruded Expanded Polystyrene: Rigid boards made by extrusion of expanded polystyrene beads.

6 **Production Lot:** The quantity of rigid polystyrene blocks produced in a continuous period of manufacturing the same grade and thickness of product within the same production day.

Quality Verification Engineer: Quality Verification Engineer means an Engineer with a minimum of five (5) years experience related to the design and/or construction of expanded polystyrene systems of similar scope to that in the Contract, or alternatively has demonstrated expertise by providing satisfactory quality verification services for the work at a minimum of two (2) projects of similar scope to the Contract. The Quality Verification Engineer shall be retained by the Contractor to ensure conformance with the contract documents and issue of certificate(s) of conformance.

5.0 QUALIFICATION

The Contractor shall have on site at the commencement of the work, a representative of the supplier of the rigid expanded polystyrene to advise on recommended construction procedure.

The Contractor shall maintain liaison with the supplier throughout the construction of the embankment for advice and guidance as required. Periodic site visits by the supplier should be coordinated as required.

6.0 SUBMISSION AND DESIGN REQUIREMENTS

6.1 Submission of Shop Drawings

At least three weeks before the commencement of work, the Contractor shall submit to the Contract Administrator six copies of the shop drawings and method statement signed and sealed by the Quality Verification Engineer that provides full details of materials and construction procedure.

6.2 Delivery, Storage, Handling, and Protection

The Contractor shall submit the method of delivery, storage, handling and protection from damage by weather, traffic, construction staging and other causes as per the rigid expanded polystyrene manufacturer's requirement.

6.3 Construction

The contractor shall submit full details of the following.

- a) The method of foundation excavation and preparation.
- b) Construction of levelling pad.
- c) The method of placement of expanded polystyrene blocks including temporary ballasting and protection of blocks during installation. The shop drawings shall indicate laying pattern and block dimensions on a layer-by-layer basis.
- d) The method and limits of placement of polyethylene sheeting.
- e) The method of placement of 125 mm reinforced concrete base pad (or equivalent).
- f) The method of placement of subbase material.
- g) The method of placement of side slope cover.

6.4 Quality Verification Engineer

- (1) The Contractor shall submit details of the sequence and method of installation to the Quality Verification Engineer for review. The submittals shall satisfy the specifications and at a minimum include a detailed description of proposed installation procedures. The details shall be submitted at least three weeks prior to the installation of the rigid expanded polystyrene embankments the Contractor shall also submit to the Contract Administrator, for information purposes, details of the sequence and method of installation. The submittals shall satisfy the specifications and at a minimum contain the above information as provided to the Contractor's Quality Verification Engineer.
- (2) The Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer a minimum of one week prior to commencement of work under this item. The Certificate shall state that the installation procedures are in conformance with the requirements and specifications of the contract documents. Quality test certificates for each production lot supplied, showing compliance with all requirements of this special provision shall be obtained by the Contractor and submitted to the Contract Administrator prior to installation. Upon completion of the Expanded Polystyrene Embankment, the Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer stating that the Expanded Polystyrene Embankment has been constructed in conformance with the installation procedures and specifications of the contract documents.

7.0 MATERIALS

7.1 Granular Levelling Pad

The levelling pad shall consist of a Granular "A" material with gradation and physical requirements as specified in OPSS.PROV 1010.

7.2 Rigid Expanded Polystyrene

7.2.1 General

7.2.1.1 The Contractor shall submit:

1. A general statement as to the type, composition, and method of production of the material.
2. The manufacturer's name, address, phone number, identification of a contact person and description of experience background in the manufacturing of the rigid expanded polystyrene.
3. Certification of compliance of physical and mechanical properties.

4. An identification of a laboratory accredited by the Standards Council of Canada to conduct the testing of the physical and mechanical properties of the rigid expanded polystyrene.
5. The physical and mechanical properties of the rigid expanded polystyrene including:
 1. Geometry
 2. Nominal Density
 3. Compressive Strength
 4. Flexural Strength
 5. Thermal Resistance
 6. Dimensional Stability
 7. Flammability
 8. Water Absorption
6. Aging and durability characteristics of the polystyrene including the chemical, biological and ultra-violet degradation resistance of the rigid polystyrene.
7. A sample of the expanded polystyrene material to the Quality Verification Engineer for review.
8. To the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer a minimum of one week prior to commencement of work under this item. The Certificate shall state that the expanded polystyrene material is in conformance with the requirements and specifications of the contract documents.

7.2.1.2 Production Lots

Each block of the same production lot shall be stamped with the same production code showing plant identification, type and date of production. The polystyrene shall be free from defects affecting serviceability.

7.2.2 Detail Requirements

Requirements shall be as shown in Table 1 and as described below.

7 Table 1 – Material Properties

PROPERTY	UNIT	REQUIREMENTS	TEST PROCEDURE
Geometry - Linear - Flatness - Squareness - Thickness	mm	1200 x 600 x 300 with tolerances $\pm 1\%$ 10 mm in 3 m $\pm 0.5\%$ -3, +5	

Replacement of CNR Overpass and Pedestrian Bridge
Highway 401 & Leslie St., Toronto, Ontario

Compressive Strength	kPa (min.)	110	ASTM D1621 (Procedure A)
Flexural Strength	kPa (min.)	240	ASTM C203
Dimensional Stability	% linear change (max.)	1.5	ASTM D2126
Thermal Resistance	m ² .°C/W (min. for 25mm thickness)	0.7	ASTM C177 or C518
Flammability	Limiting Oxygen Index (min.)	24	ASTM D2863
Water Absorption	% by Volume (max.)	4	ASTM D2842

7.2.2.1 Geometry

The expanded polystyrene shall be supplied in the form of rectangular parallel blocks of minimum acceptable dimensions of 1200 mm x 600 mm x 300 mm.

The maximum deviation from the specified linear dimensions shall be $\pm 1\%$. The flatness of the block faces shall be within ± 10 mm of a line formed by a 3 m straight edge.

The maximum difference in corner-to-corner dimensions (squareness) shall be 0.5%. The thickness shall be within -3 to $+5$ mm.

7.2.2.2 Compressive Strength

The minimum compressive strength, measured in accordance with ASTM D1621, Procedure A, shall be 110 kPa at a strain of not more than 5%. The maximum permissible permanent stress level should not exceed 30% of the compressive strength of the material at 5% strain.

7.2.2.3 Flexural Strength

The minimum flexural strength of the polystyrene shall be 240 kPa. The flexural strength shall be determined in accordance to ASTM C203, method 1, Procedure B.2.7.4 Dimensional Stability.

7.2.2.4 Dimensional Stability

Dimensional Stability shall be determined in accordance with ASTM D2126, Procedure G. A tolerance of 1.5% shall be satisfied.

7.2.2.5 Thermal Resistance

The thermal resistance shall be 0.7 m².°C/W for a 25 mm thickness using the following equation and using the average value from three specimens:

$$R_{25\text{mm}} = \frac{R_{\text{measured}}}{\text{thickness (mm)}} \times 25$$

The thermal resistance shall be measured in accordance with ASTM C177 or C518.

7.2.2.6 Flammability

The expanded polystyrene shall be classified as to surface burning characteristics in accordance with CAN/ULC - 51022 having a flame spread rating less than 500. The expanded polystyrene shall have a minimum limiting oxygen index measured in accordance with ASTM D2863.

7.2.2.7 Water Absorption

The water absorption as measured by ASTM D2842 shall be limited to 4% by volume.

7.2.2.8 Chemical Resistance

The expanded polystyrene shall be resistant to common inorganic acids and alkalies. A table identifying the chemical resistance as either resistant limited or not resistant shall be submitted.

7.2.2.9 Biological Resistance

The expanded polystyrene shall be resistant to biological degradation caused by organisms or enzymes.

7.2.2.10 Environmental

The expanded polystyrene shall be inert, non-nutritive and highly stable and shall not produce undesirable gases or leachate.

8.0 DELIVERY, STORAGE AND HANDLING

The product shall be suitably marked to identify its type, number and the manufacturer's name or trademark.

The Contractor shall protect the expanded polystyrene from exposure to sunlight to avoid ultraviolet degradation as per manufacturer's recommendation.

Protection of materials and works from damage by weather, traffic, construction staging, fire or vandalism and other causes shall be the responsibility of the Contractor.

9.0 CONSTRUCTION

9.1 Foundation Excavation

Foundation excavation shall be carried out to the design elevations shown on the drawings. Any softened, loosened or deleterious materials at the foundation footing elevation shall be subexcavated and replaced with Granular 'A' or Granular 'B' material.

9.2 Leveling Pad

Place, level and compact a layer of Granular 'A' or Granular 'B' material in accordance with OPSS.PROV 501 to within ± 30 mm of the design elevation. The leveling pad shall not deviate by more than 10 mm at any location on a 3 m straight edge over the limits of the bottom course of blocks. The leveling pad shall not be placed on frozen ground.

9.3 Installation of Blocks

- (1) The individually marked blocks shall be placed on the prepared leveling pad. The top surface of the first layer of blocks is to be set plane and level. Local trimming of the blocks may be necessary.
- (2) Subsequent successive layers shall be oriented with the long axis of blocks positioned at 90° to the previous layer in order to avoid continuous joints. Block joints shall be offset and staggered between layers.
- (3) A continuous check shall be kept to ensure the evenness of the blocks is satisfactory in each layer. Blocks shall be laid with joints with maximum opening of 10 mm between blocks. Differences in heights between adjacent blocks in the same layer should not exceed 5 mm.
- (4) Sloping end adjustments at the abutments shall be accomplished by leveling terraces in the subsoil in accordance with the block thickness.
- (5) Temporary ballast shall be provided as necessary to prevent movement of expanded polystyrene both in storage and as placed due to windy conditions. Timber fasteners or equivalent shall be used as necessary.
- (6) The expanded polystyrene embankment shall be protected from accidental ignition due to welding, smoking, grinding or cutting tools, etc. The Contractor shall take all necessary precautions to prevent ignition of the expanded polystyrene.
- (7) The expanded polystyrene shall be protected from organic solvents and other aggressive, harmful chemicals during construction. The proposed method of protection during construction shall be submitted to the Contractor's Quality

Verification Engineer for review and to the Contract Administrator for information purposes.

- (8) Exposed blocks shall be covered immediately to avoid possible burrowing by animals.
- (9) Individually marked blocks shall be fabricated and placed to ensure the top surface matches the elevation and crossfall shown on the drawings.
- (10) The top surface and side surfaces of the expanded polystyrene shall be covered with 0.6 mil polyethylene sheeting extending onto adjacent work at the longitudinal ends of the embankment. All joints shall be lapped a minimum of 300 mm to provide a fully sealed enclosure.
- (11) The contractor shall install the concrete base pad as detailed elsewhere in the contract.

10.0 EQUIPMENT

All cutting of polystyrene materials shall be done by electric equipment or by hand.

Heavy equipment shall be limited in weight and size and restricted in operation to avoid damaging the expanded polystyrene as per the manufacturer's requirement.

11.0 QUALITY ASSURANCE

General

The Contract Administrator may undertake an independent testing program of the expanded polystyrene. Sampling and testing will be carried out in conformance with the relevant test procedure. The physical and thermal property testing identified in Table 1 will be conducted. A recognized testing laboratory accredited by the Standards Council of Canada shall conduct the testing.

Sampling Frequency

Sufficient sample material shall be obtained from blocks randomly selected by the Contract Administrator from each production lot as soon as the material arrives on site. As a minimum, three blocks shall be tested.

Acceptance/Rejection

Failure of any one of the sample blocks to comply with any requirements of this special provision shall be cause for rejection of the production lot from which it was taken. Replacement of the blocks shall be at the Contractor's expense.

12.0 MEASUREMENT FOR PAYMENT

Actual Measurement

Measurement will be by volume in cubic metres measured in its original position and based on cross-sections.

13.0 PAYMENT

Basis of Payment

The Concrete Base pad and granular leveling pad shall be paid for with the appropriate tender items as detailed elsewhere in the contract.

Payment at the contract price for the above tender item shall be full compensation for all labour, materials and equipment to do the work as described above and no extra payments will be made.

Appendix F
Selected Slope Stability Analysis Results

19-5161-205

Name: Existing Fill Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 30 ° Piezometric Line: 1
 Name: Sand/Silt Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 30 ° Piezometric Line: 1
 Name: Upper Clay Unit Weight: 18.5 kN/m³ Piezometric Line: 1 C-Datum: 40 kPa C-Rate of Change: -1.5 kPa/m Limiting C: 30 kPa Elevation: 137 m
 Name: Lower Clay Unit Weight: 19 kN/m³ Piezometric Line: 1 C-Datum: 30 kPa C-Rate of Change: 5 kPa/m Limiting C: 70 kPa Elevation: 130 m
 Name: Sand/Silt Till Unit Weight: 22 kN/m³ Cohesion: 0 kPa Phi: 34 ° Piezometric Line: 1
 Name: RSS Unit Weight: 22 kN/m³ Cohesion: 200 kPa Phi: 34 ° Piezometric Line: 1

8.0 m RSS WALL
 (Approx. Sta. 10+030)

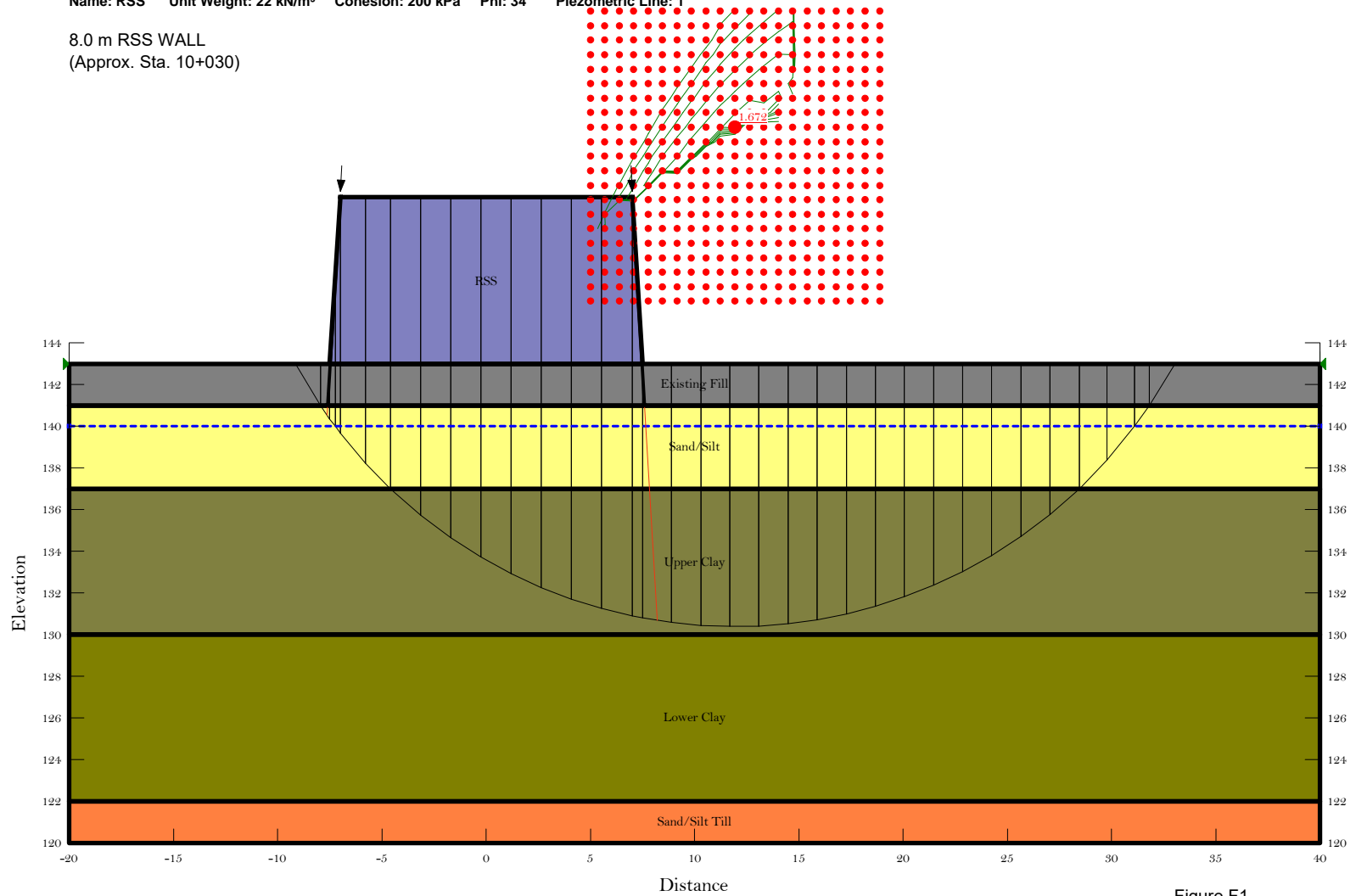
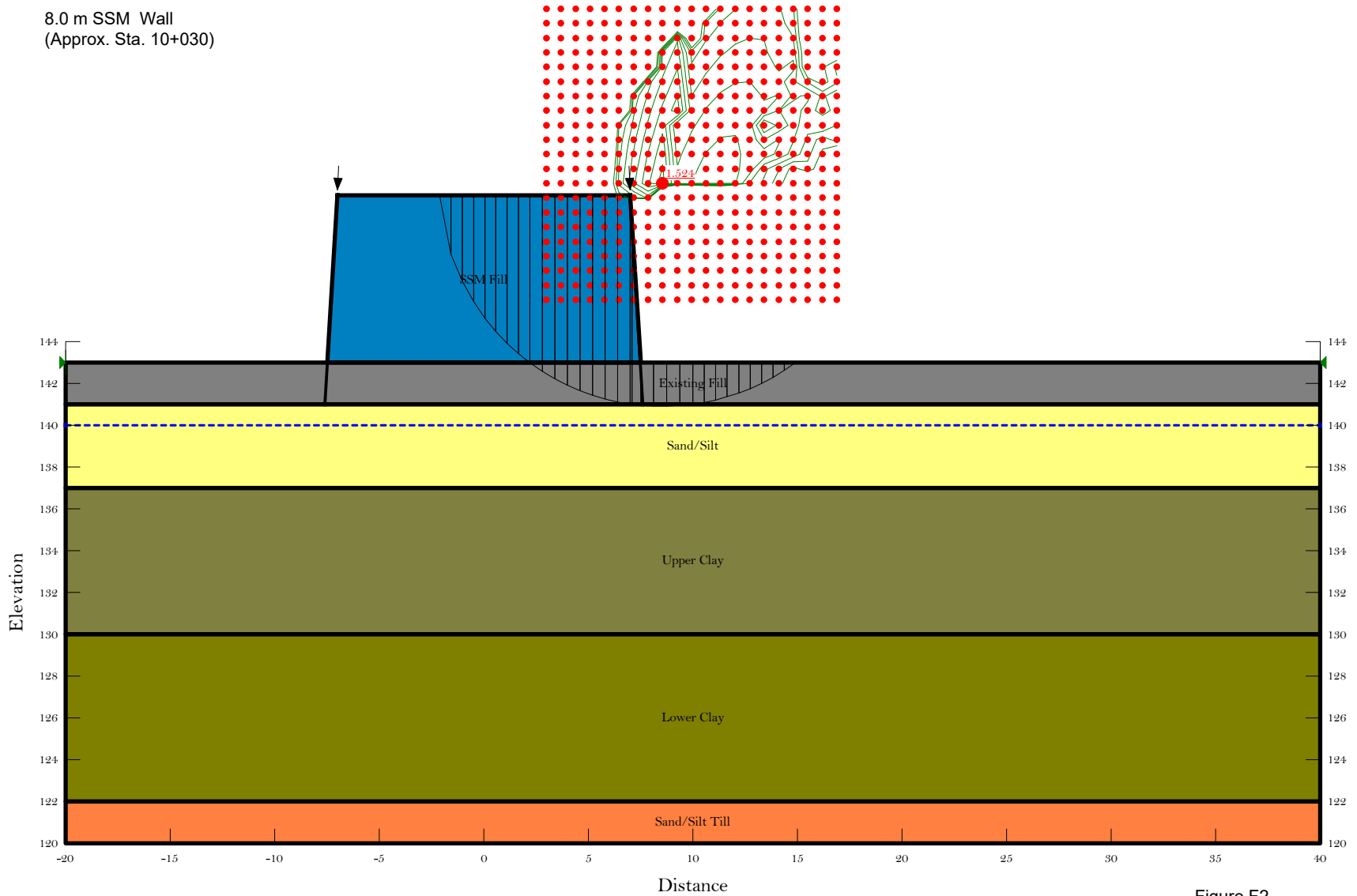


Figure F1

Directory: H:\19\5161\205 Hwy 401 & Leslie Foundations\Analysis\Stability\Nov 5 2005\,File Name: 8m_new RSS vertical.gsz

Name: Existing Fill Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 30 ° Piezometric Line: 1
 Name: Sand/Silt Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 30 ° Piezometric Line: 1
 Name: Upper Clay Unit Weight: 18.5 kN/m³ Piezometric Line: 1 C-Datum: 40 kPa C-Rate of Change: -1.5 kPa/m Limiting C: 30 kPa Elevation: 137 m
 Name: Lower Clay Unit Weight: 19 kN/m³ Piezometric Line: 1 C-Datum: 30 kPa C-Rate of Change: 5 kPa/m Limiting C: 70 kPa Elevation: 130 m
 Name: Sand/Silt Till Unit Weight: 22 kN/m³ Cohesion: 0 kPa Phi: 34 ° Piezometric Line: 1
 Name: SSM Fill Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 32 ° Piezometric Line: 1

8.0 m SSM Wall
 (Approx. Sta. 10+030)



Directory: H:\19\5161\205 Hwy 401 & Leslie Foundations\Analysis\Stability\Nov 5 2005\File Name: 8m_new SSM vertical.gsz

Figure F2

Name: Existing Fill Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 30 ° Piezometric Line: 1
 Name: Sand/Silt Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 30 ° Piezometric Line: 1
 Name: Upper Clay Unit Weight: 18.5 kN/m³ Piezometric Line: 1 C-Datum: 40 kPa C-Rate of Change: -1.5 kPa/m Limiting C: 30 kPa Elevation: 137 m
 Name: Lower Clay Unit Weight: 19 kN/m³ Piezometric Line: 1 C-Datum: 30 kPa C-Rate of Change: 5 kPa/m Limiting C: 70 kPa Elevation: 130 m
 Name: Sand/Silt Till Unit Weight: 22 kN/m³ Cohesion: 200 kPa Phi: 34 ° Piezometric Line: 1
 Name: RSS Unit Weight: 22 kN/m³ Cohesion: 200 kPa Phi: 34 ° Piezometric Line: 1

7m RSS WALL
 (Approx. Sta. 10+110)

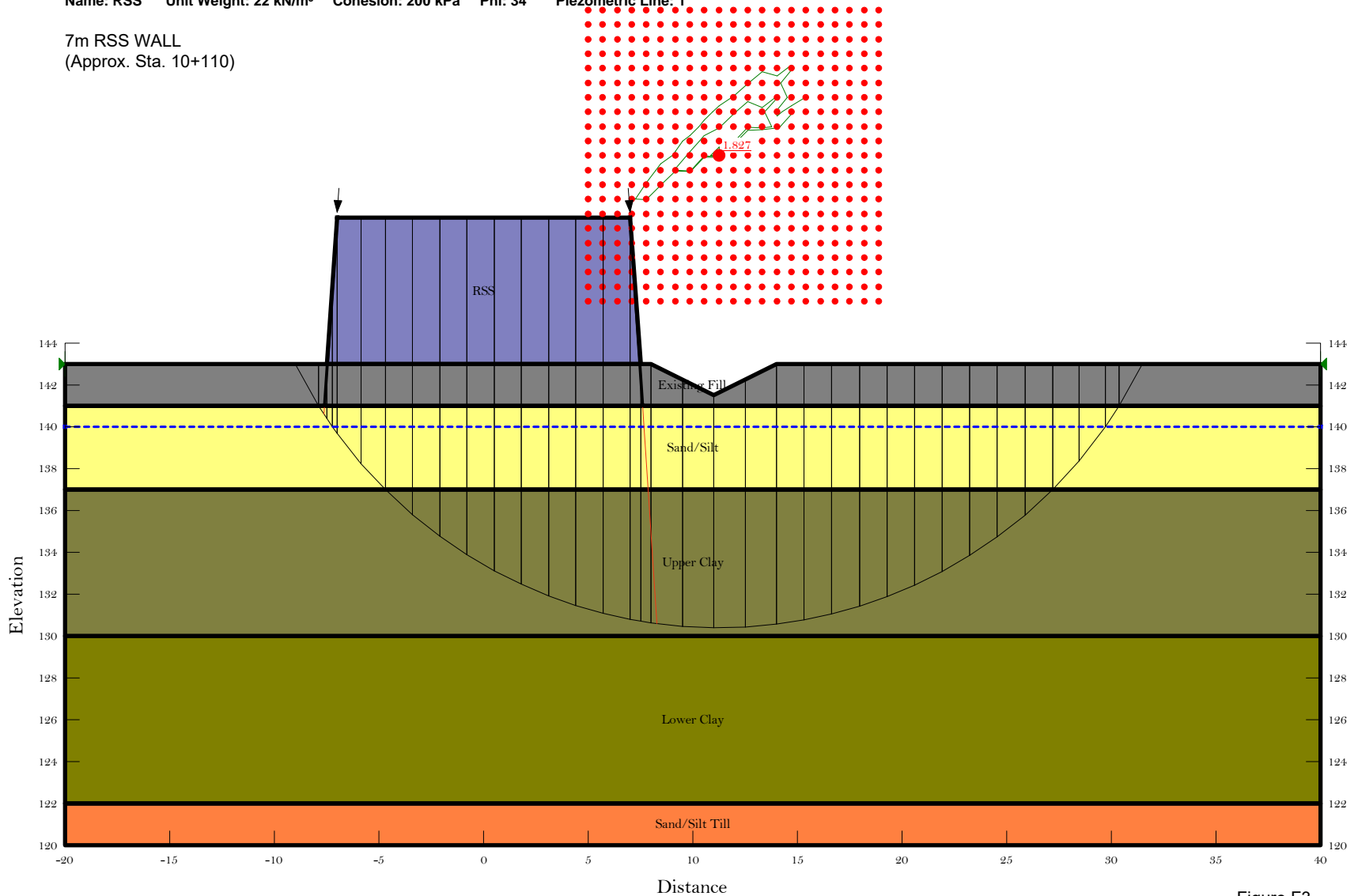


Figure F3

Directory: H:\19\5161\205 Hwy 401 & Leslie Foundations\Analysis\Stability\Nov 5 2005\File Name: 7m_new RSS vertical.gsz

Name: Existing Fill Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 30 ° Piezometric Line: 1
 Name: Sand/Silt Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 30 ° Piezometric Line: 1
 Name: Upper Clay Unit Weight: 18.5 kN/m³ Piezometric Line: 1 C-Datum: 40 kPa C-Rate of Change: -1.5 kPa/m Limiting C: 30 kPa Elevation: 137 m
 Name: Lower Clay Unit Weight: 19 kN/m³ Piezometric Line: 1 C-Datum: 30 kPa C-Rate of Change: 5 kPa/m Limiting C: 70 kPa Elevation: 130 m
 Name: Sand/Silt Till Unit Weight: 22 kN/m³ Cohesion: 0 kPa Phi: 34 ° Piezometric Line: 1
 Name: SSM Fill Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 32 ° Piezometric Line: 1

7m SSM With Retaining Wall
 (Approx. Sta. 10+110)

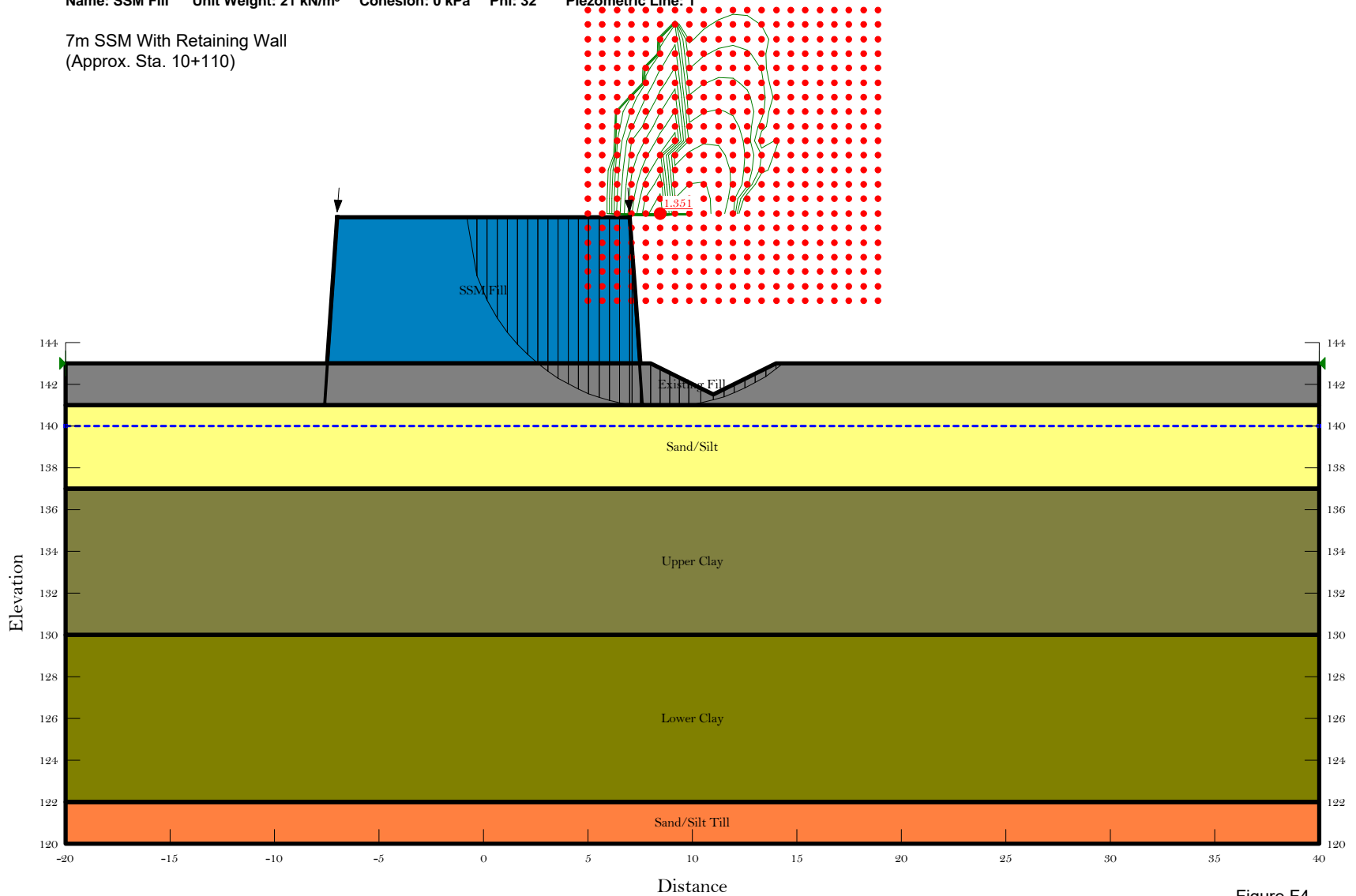


Figure F4

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Name: Existing Fill Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 30 ° Piezometric Line: 1
 Name: Sand/Silt Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 30 ° Piezometric Line: 1
 Name: Upper Clay Unit Weight: 18.5 kN/m³ Piezometric Line: 1 C-Datum: 40 kPa C-Rate of Change: -1.5 kPa/m Limiting C: 30 kPa Elevation: 137 m
 Name: Lower Clay Unit Weight: 19 kN/m³ Piezometric Line: 1 C-Datum: 30 kPa C-Rate of Change: 5 kPa/m Limiting C: 70 kPa Elevation: 130 m
 Name: Sand/Silt Till Unit Weight: 22 kN/m³ Cohesion: 0 kPa Phi: 34 ° Piezometric Line: 1
 Name: RSS Unit Weight: 22 kN/m³ Cohesion: 200 kPa Phi: 34 ° Piezometric Line: 1

5 m RSS Wall
 (Approx. Sta. 10+000)

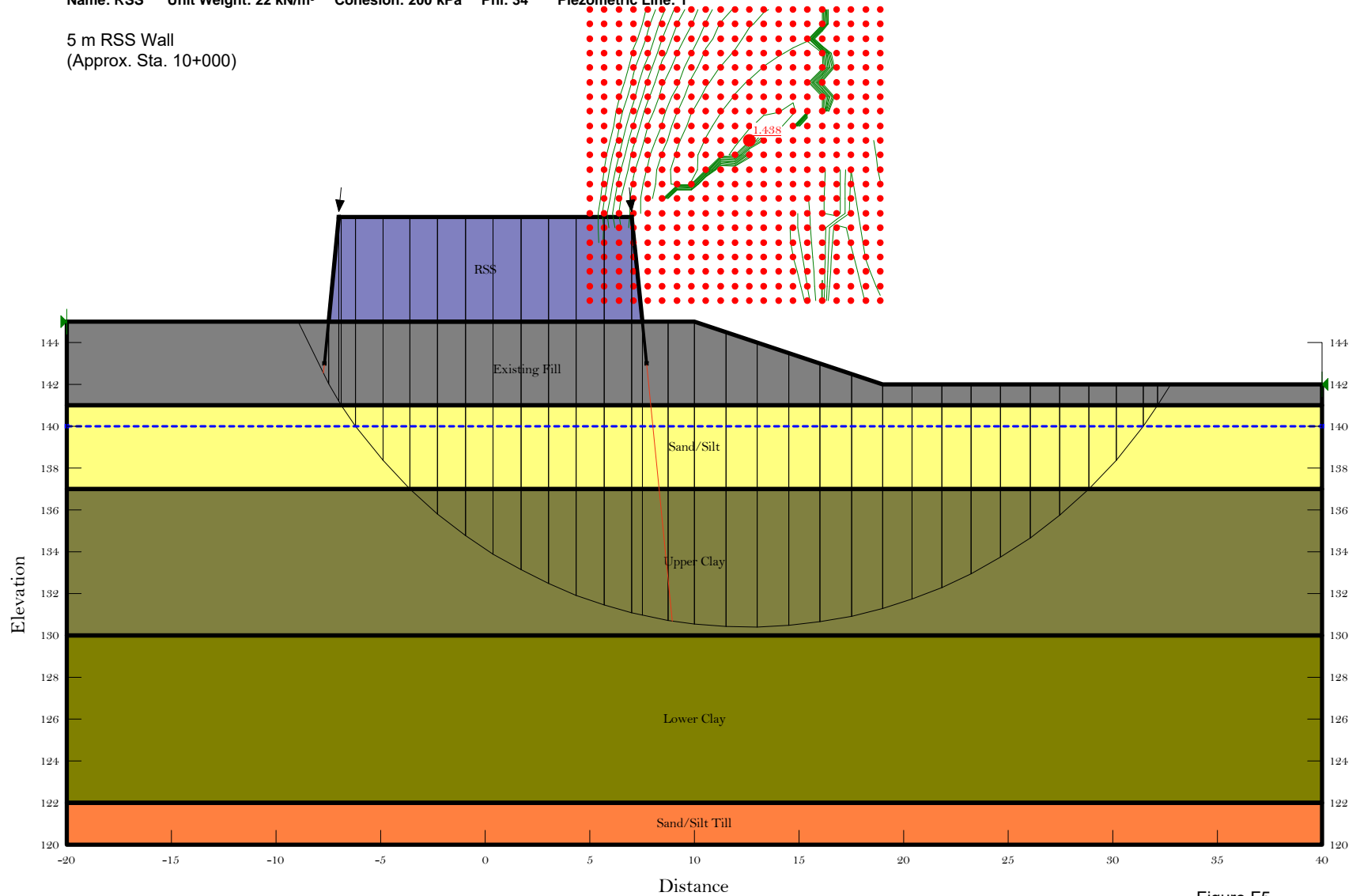


Figure F5

Directory: H:\19\5161\205 Hwy 401 & Leslie Foundations\Analysis\Stability\Nov 5 2005\File Name: 5m_new RSS vertical.gsz

Name: Existing Fill Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 30 ° Piezometric Line: 1
 Name: Sand/Silt Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 30 ° Piezometric Line: 1
 Name: Upper Clay Unit Weight: 18.5 kN/m³ Piezometric Line: 1 C-Datum: 40 kPa C-Rate of Change: -1.5 kPa/m Limiting C: 30 kPa Elevation: 137 m
 Name: Lower Clay Unit Weight: 19 kN/m³ Piezometric Line: 1 C-Datum: 30 kPa C-Rate of Change: 5 kPa/m Limiting C: 70 kPa Elevation: 130 m
 Name: Sand/Silt Till Unit Weight: 22 kN/m³ Cohesion: 0 kPa Phi: 34 ° Piezometric Line: 1
 Name: SSM Fill Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 32 ° Piezometric Line: 1

5 m SMM with Retaining Wall
 (Approx. Sta. 10+000)

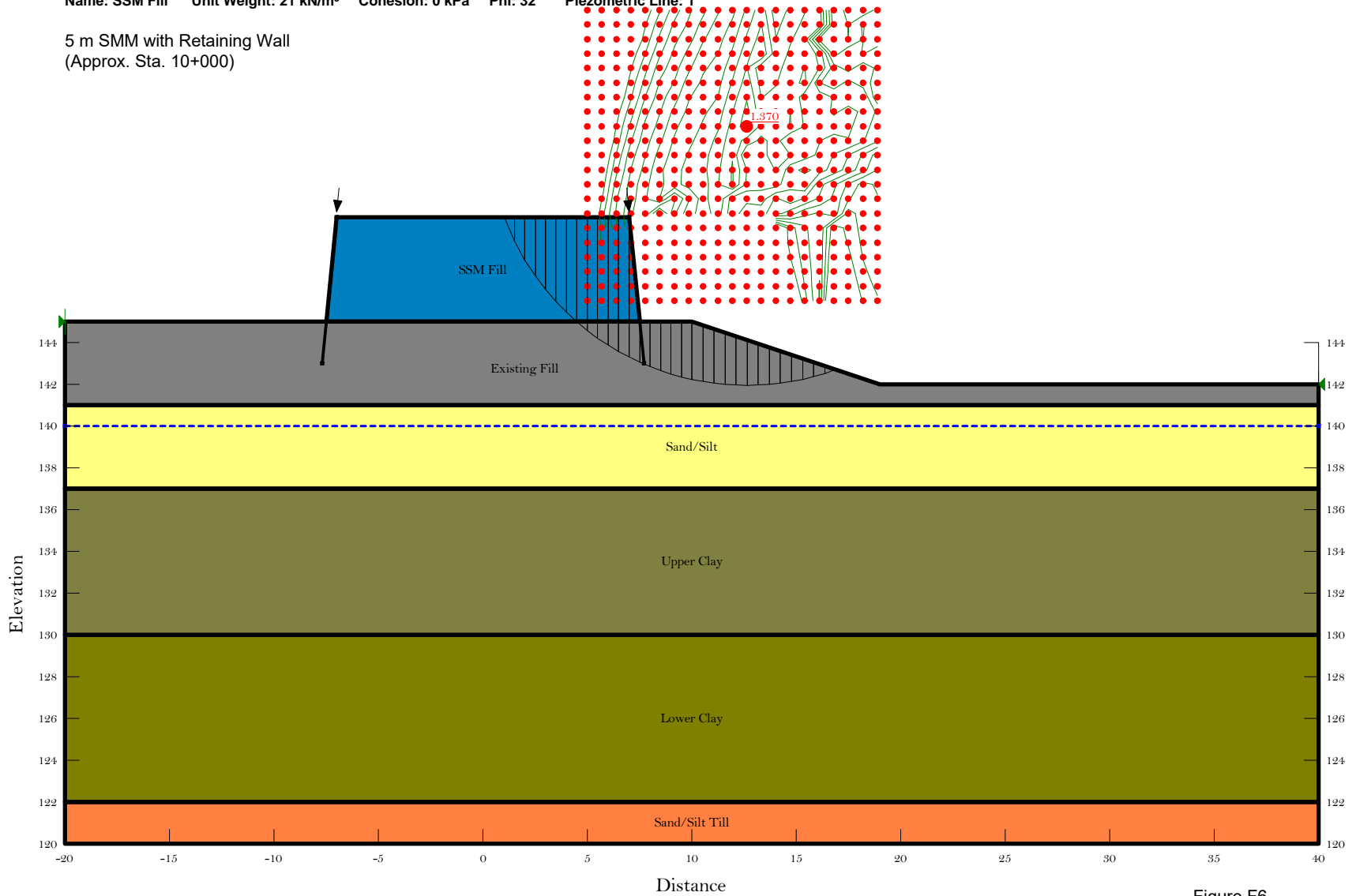
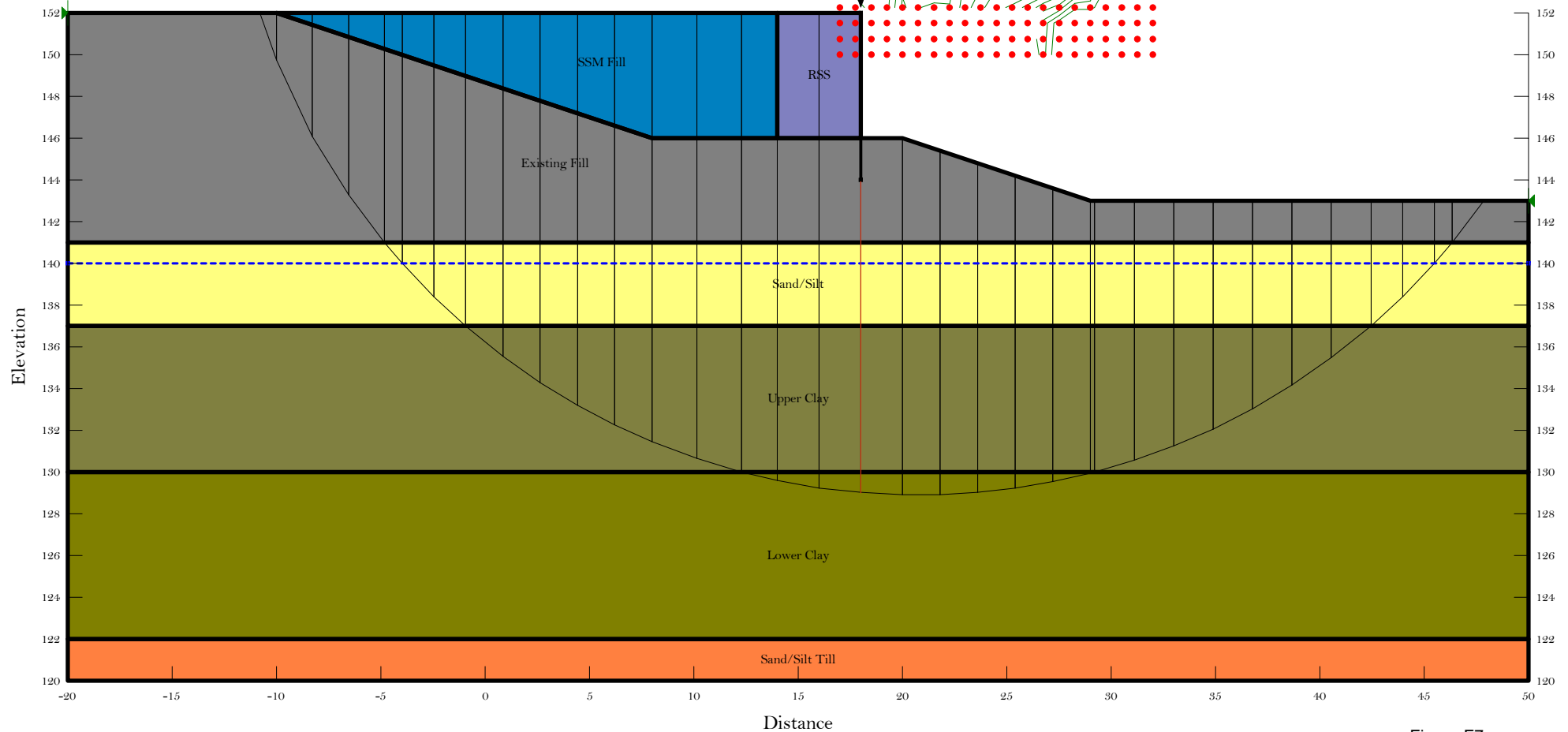


Figure F6

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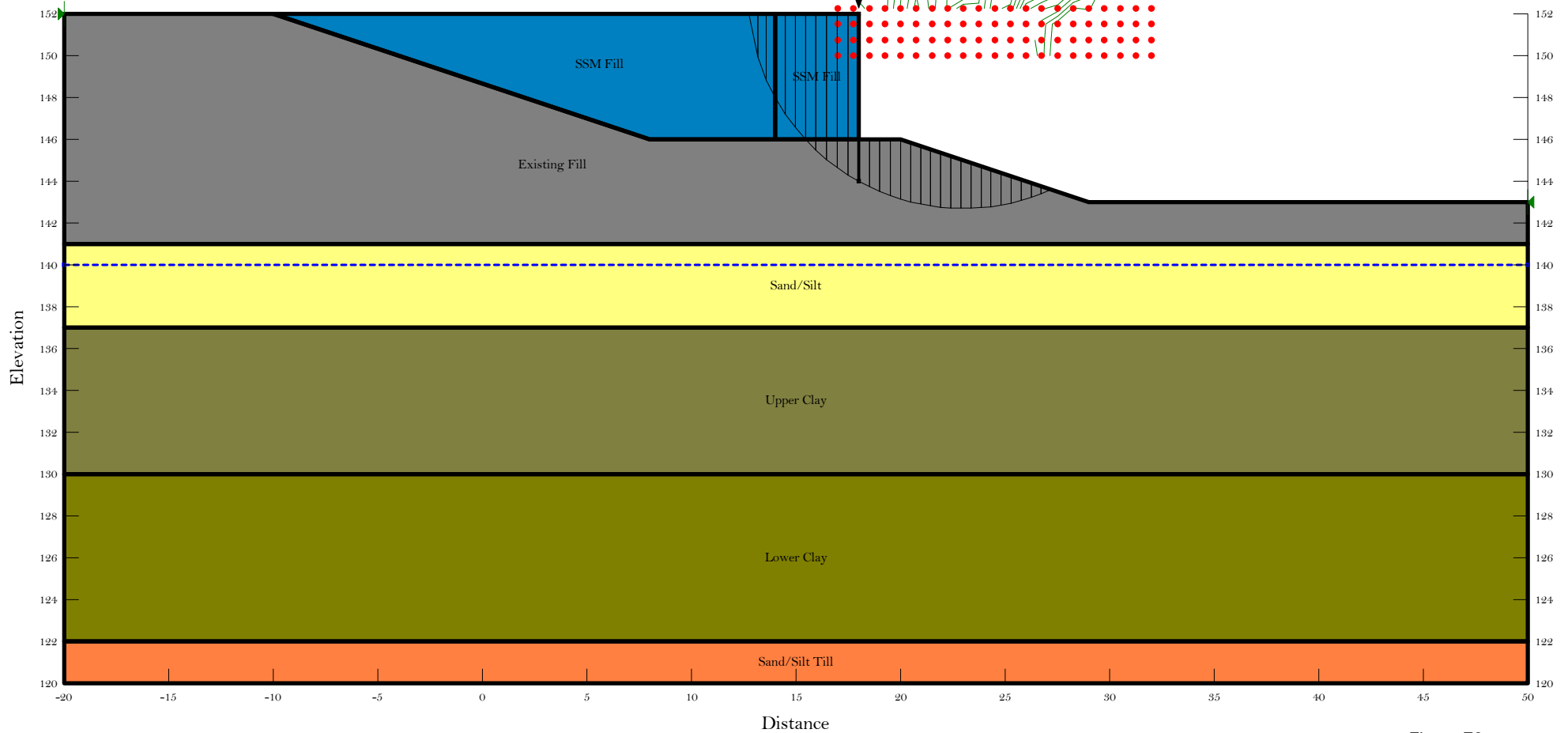
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 Name: Sand/Silt Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 30 ° Piezometric Line: 1
 Name: Upper Clay Unit Weight: 18.5 kN/m³ Piezometric Line: 1 C-Datum: 40 kPa C-Rate of Change: -1.5 kPa/m Limiting C: 30 kPa Elevation: 137 m
 Name: Lower Clay Unit Weight: 19 kN/m³ Piezometric Line: 1 C-Datum: 30 kPa C-Rate of Change: 5 kPa/m Limiting C: 70 kPa Elevation: 130 m
 Name: Sand/Silt Till Unit Weight: 22 kN/m³ Cohesion: 0 kPa Phi: 34 ° Piezometric Line: 1
 Name: RSS Unit Weight: 22 kN/m³ Cohesion: 200 kPa Phi: 32 ° Piezometric Line: 1
 Name: SSM Fill Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 32 ° Piezometric Line: 1

6 m SSM Retained by RSS Against Slope
 (Approx. Sta. 9+960 to 9+980)



Name: Existing Fill Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 30 ° Piezometric Line: 1
 Name: Sand/Silt Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 30 ° Piezometric Line: 1
 Name: Upper Clay Unit Weight: 18.5 kN/m³ Piezometric Line: 1 C-Datum: 40 kPa C-Rate of Change: -1.5 kPa/m Limiting C: 30 kPa Elevation: 137 m
 Name: Lower Clay Unit Weight: 19 kN/m³ Piezometric Line: 1 C-Datum: 30 kPa C-Rate of Change: 5 kPa/m Limiting C: 70 kPa Elevation: 130 m
 Name: Sand/Silt Till Unit Weight: 22 kN/m³ Cohesion: 0 kPa Phi: 34 ° Piezometric Line: 1
 Name: SSM Fill Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 32 ° Piezometric Line: 1

6 m SMM with Retaining Wall Against Slope
 (Approx. Sta. 9+960 to 9+980)



Name: Existing Fill Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 30 ° Piezometric Line: 1
 Name: Sand/Silt Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 30 ° Piezometric Line: 1
 Name: Upper Clay Unit Weight: 18.5 kN/m³ Piezometric Line: 1 C-Datum: 40 kPa C-Rate of Change: -1.5 kPa/m Limiting C: 30 kPa Elevation: 137 m
 Name: Lower Clay Unit Weight: 19 kN/m³ Piezometric Line: 1 C-Datum: 30 kPa C-Rate of Change: 5 kPa/m Limiting C: 70 kPa Elevation: 130 m
 Name: Sand/Silt Till Unit Weight: 22 kN/m³ Cohesion: 0 kPa Phi: 34 ° Piezometric Line: 1
 Name: RSS Unit Weight: 22 kN/m³ Cohesion: 200 kPa Phi: 34 ° Piezometric Line: 1

7 m RSS Wall perched on 9 m slope
 (Approx. Sta. 9+920)

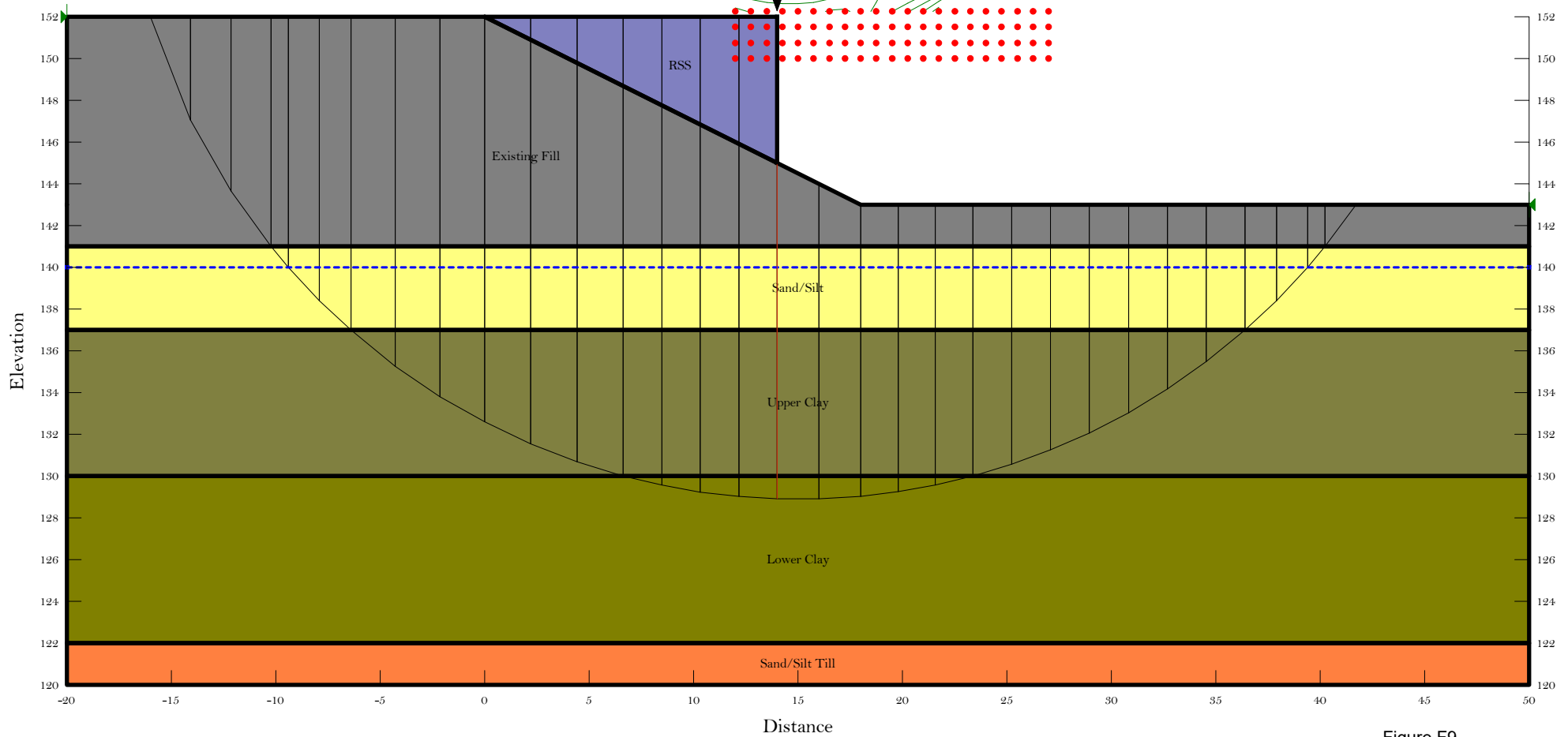


Figure F9

Name: Existing Fill Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 30 ° Piezometric Line: 1
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 Name: Upper Clay Unit Weight: 18.5 kN/m³ Piezometric Line: 1 C-Datum: 40 kPa C-Rate of Change: -1.5 kPa/m Limiting C: 30 kPa Elevation: 137 m
 Name: Lower Clay Unit Weight: 19 kN/m³ Piezometric Line: 1 C-Datum: 30 kPa C-Rate of Change: -5 kPa/m Limiting C: 70 kPa Elevation: 130 m
 Name: Sand/Silt Till Unit Weight: 22 kN/m³ Cohesion: 0 kPa Phi: 34 ° Piezometric Line: 1
 Name: SSM Fill Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 32 ° Piezometric Line: 1

7 m SSM with Retaining Wall perched on 9 m slope
 (Approx. Sta. 9+920)

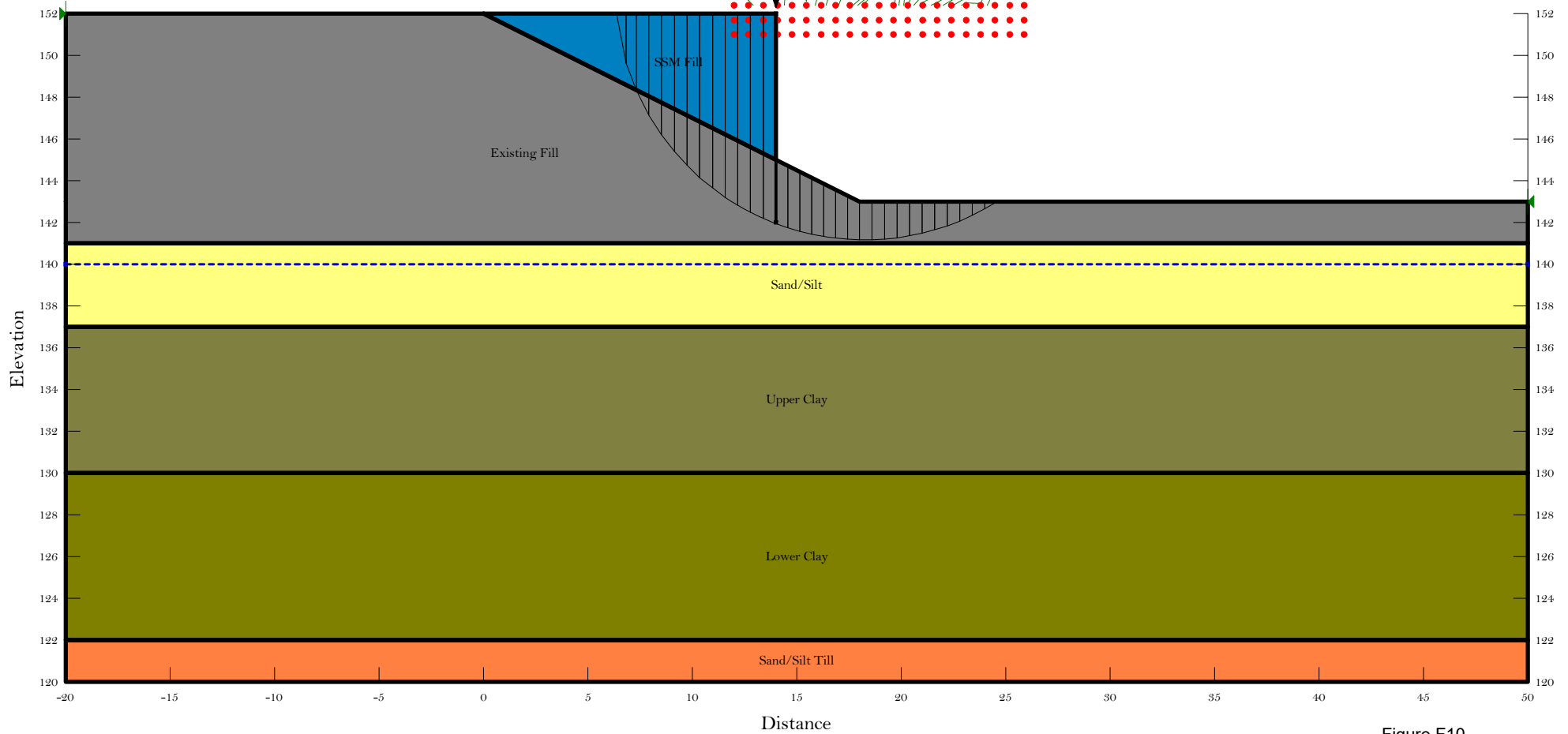


Figure F10

Directory: H:\19\5161\205 Hwy 401 & Leslie Foundations\Analysis\Stability\Nov 5 2005\, File Name: 7m_vertical on 9m slope SSM (1).gsz

Name: Existing Fill Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 30 ° Piezometric Line: 1
 Name: Sand/Silt Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 30 ° Piezometric Line: 1
 Name: Upper Clay Unit Weight: 18.5 kN/m³ Piezometric Line: 1 C-Datum: 40 kPa C-Rate of Change: -1.5 kPa/m Limiting C: 35 kPa Elevation: 139 m
 Name: Lower Clay Unit Weight: 19 kN/m³ Piezometric Line: 1 C-Datum: 40 kPa C-Rate of Change: 5 kPa/m Limiting C: 70 kPa Elevation: 135 m
 Name: Sand/Silt Till Unit Weight: 22 kN/m³ Cohesion: 0 kPa Phi: 34 ° Piezometric Line: 1
 Name: SSM Fill Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 32 ° Piezometric Line: 1

3 m SSM on 3 m slope
 (Approx. Sta. 9+740)

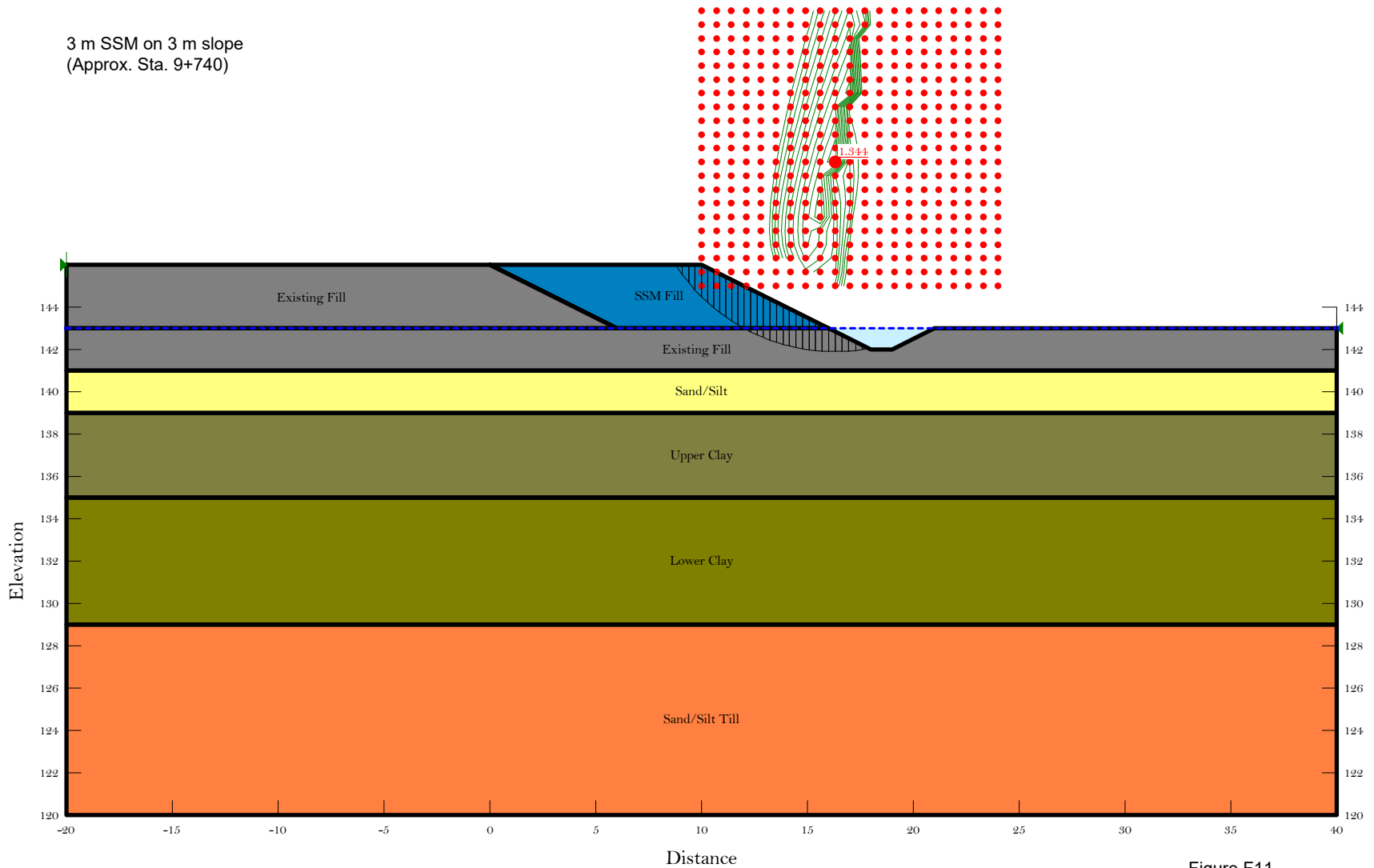
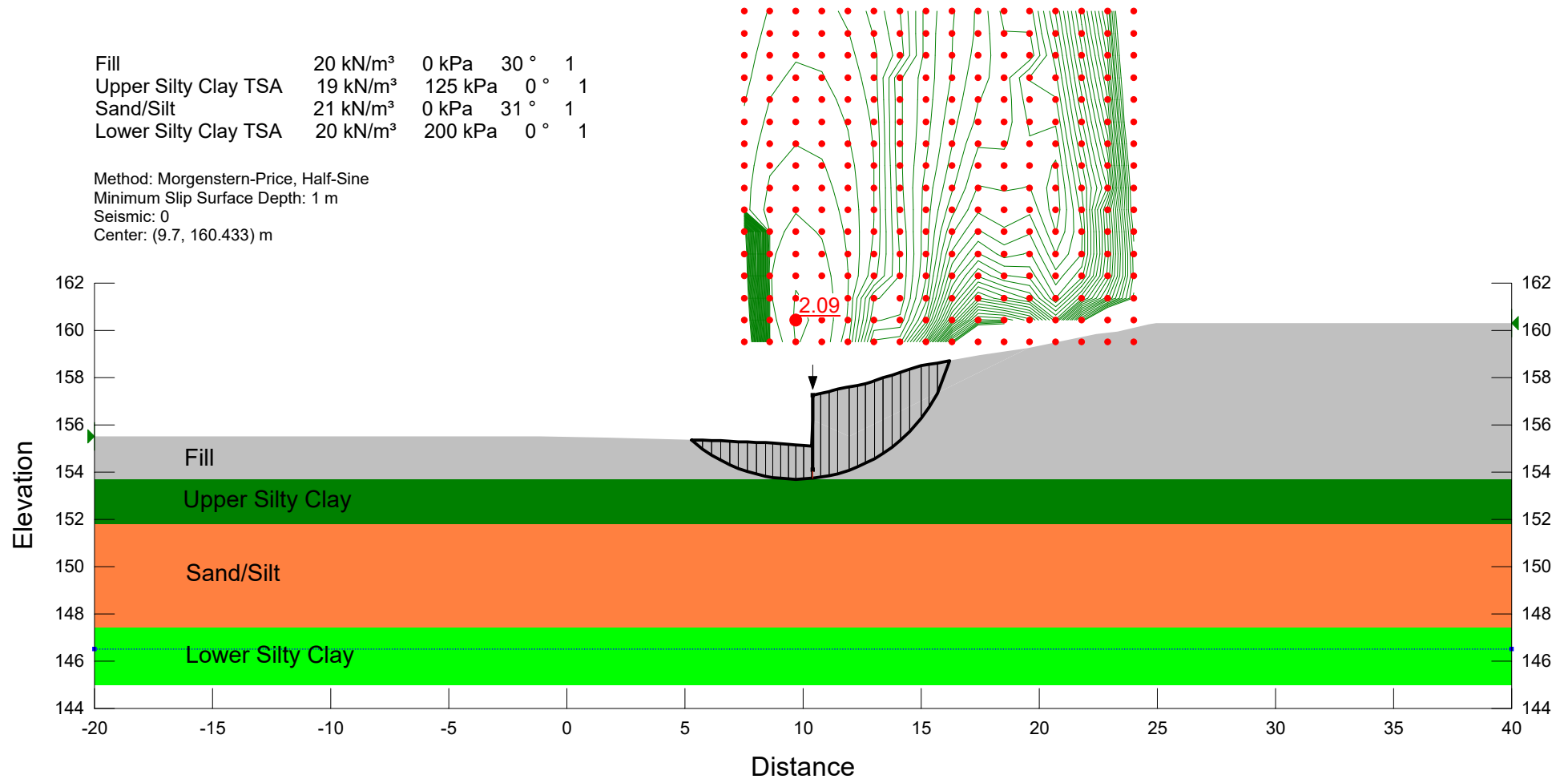


Figure F11

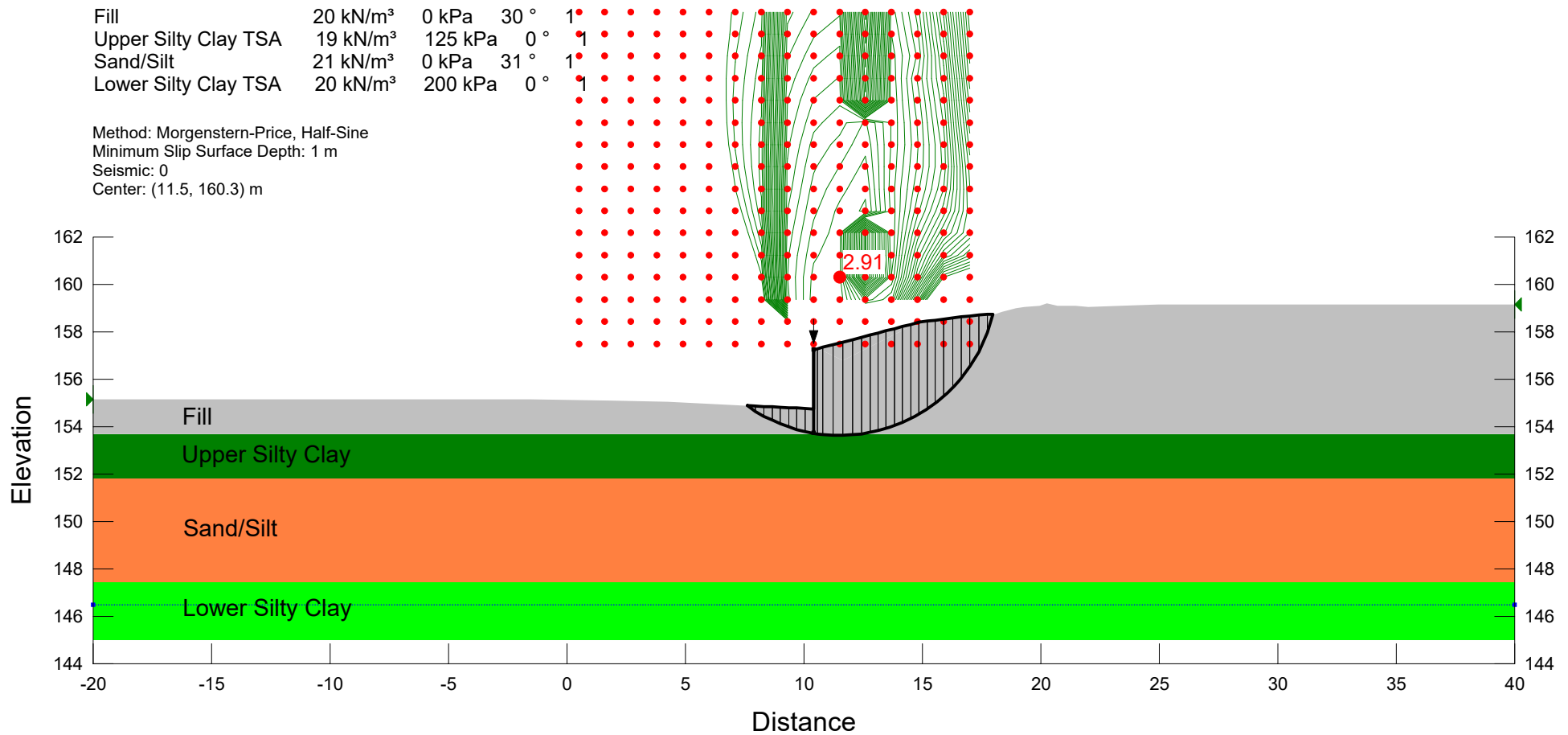
HIGHWAY 401 & LESLIE STREET COMBINATION WALL STA. 25+000 (UNDRAINED)

FIGURE F12



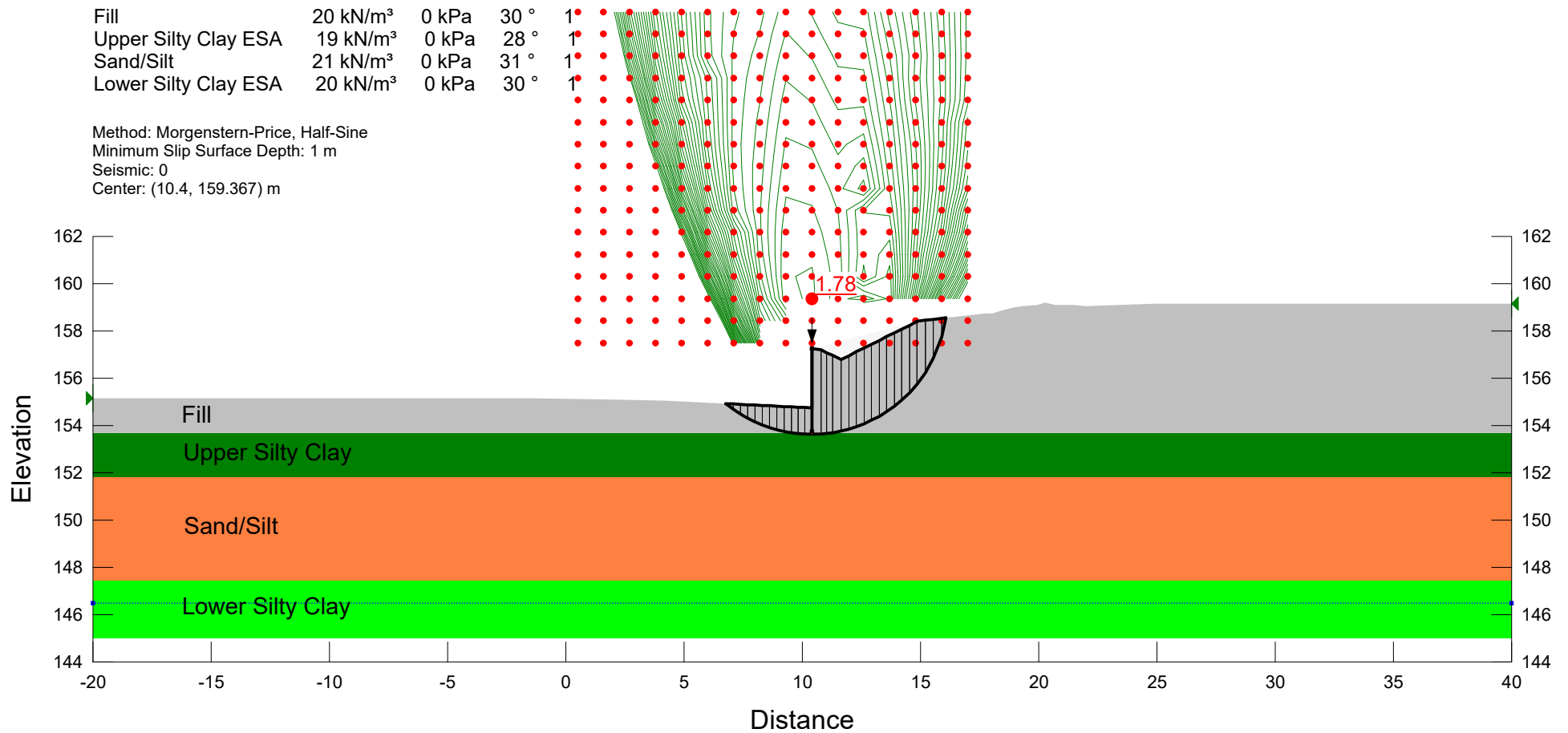
HIGHWAY 401 & LESLIE STREET COMBINATION WALL STA. 25+010 (UNDRAINED)

FIGURE F13



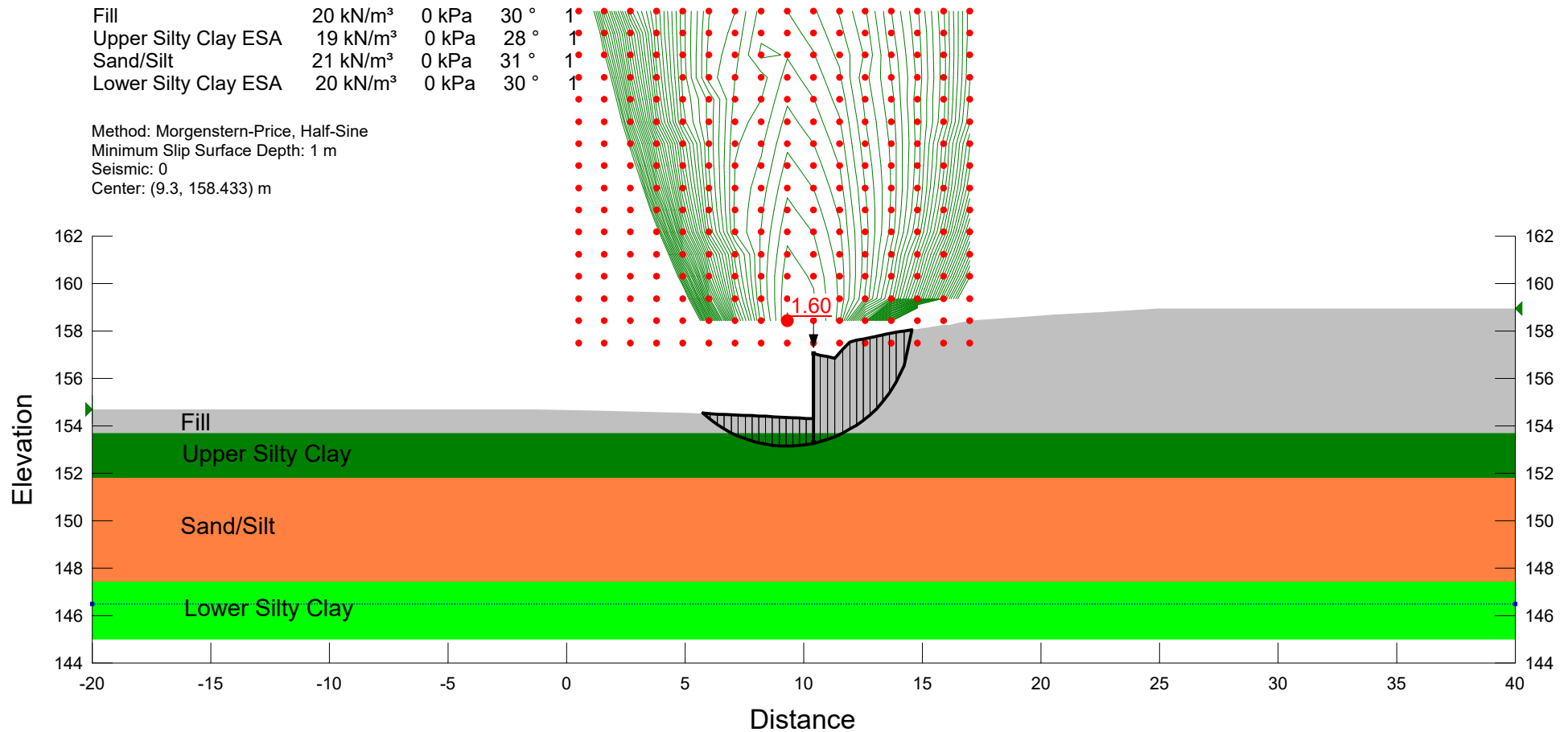
HIGHWAY 401 & LESLIE STREET COMBINATION WALL STA. 25+010 (DRAINED)

FIGURE F14



HIGHWAY 401 & LESLIE STREET COMBINATION WALL STA. 25+025 (DRAINED)

FIGURE F15



HIGHWAY 401 & LESLIE STREET COMBINATION WALL STA. 25+025 (DRAINED)

FIGURE F16

